

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 30M5-159

DIST. 4 REGION

W.P. No. 380-85-01

CONT. No. 90-79

W. O. No.

STR. SITE No.

HWY. No. 403

LOCATION Hwy 403 & Hwy 6
Retaining Wall Rehabilitation

No of PAGES -



OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

G.I.-30 SEPT. 1976

METRIC

DIMENSIONS ARE IN METRES
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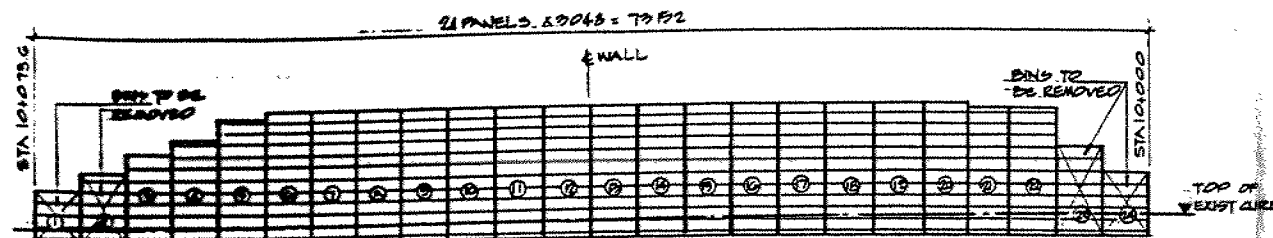
CONT No 90-79
WP No 380-85-01

HWY. 403 RETAINING WALL
SCOPE OF WORK

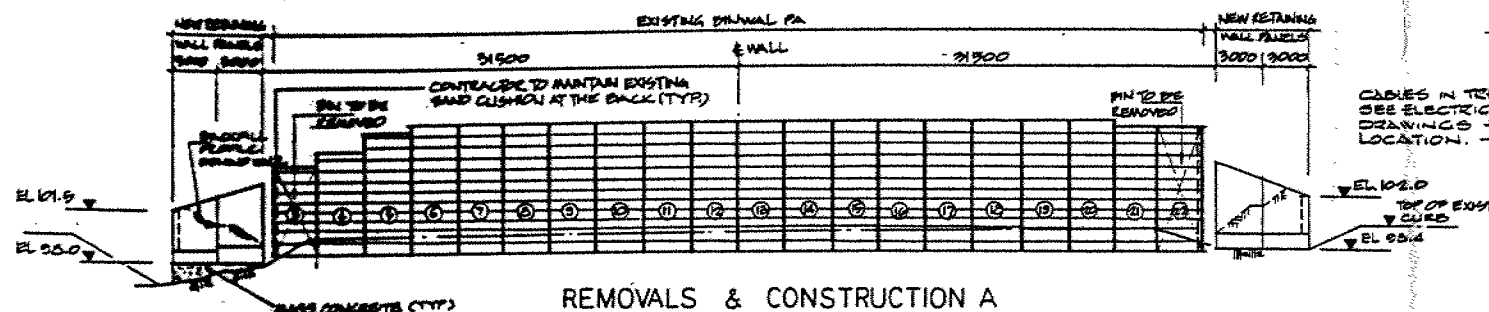


SHEET
49

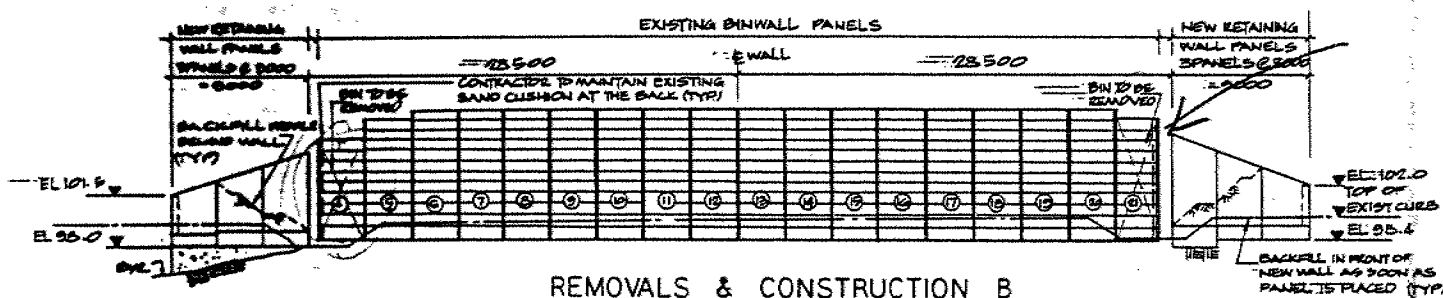
DELCAN ENGINEERS
PLANNERS
ARCHITECTS



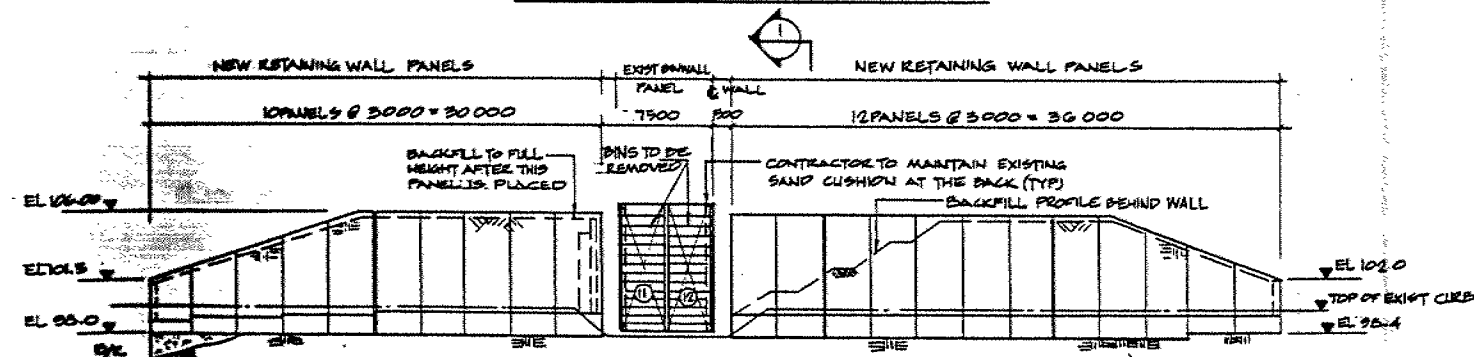
EXISTING BINWALL-ELEVATION



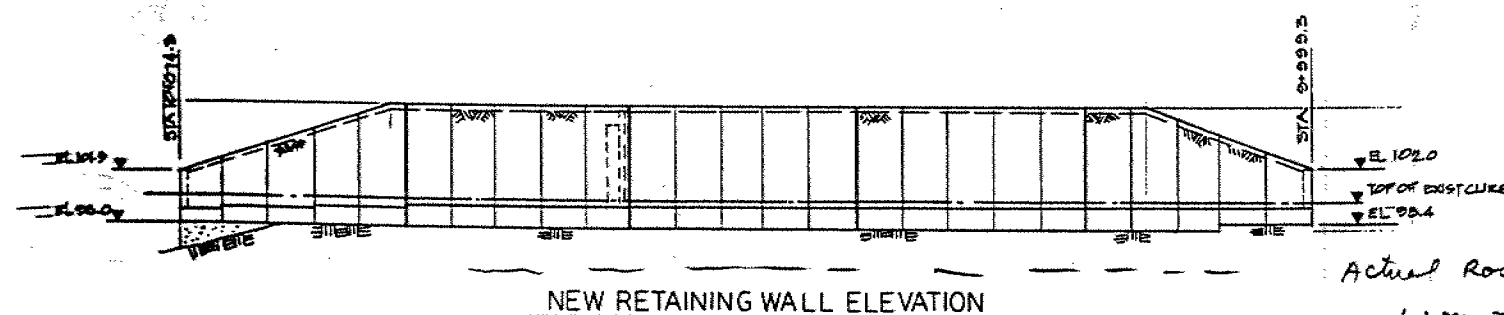
REMOVALS & CONSTRUCTION A



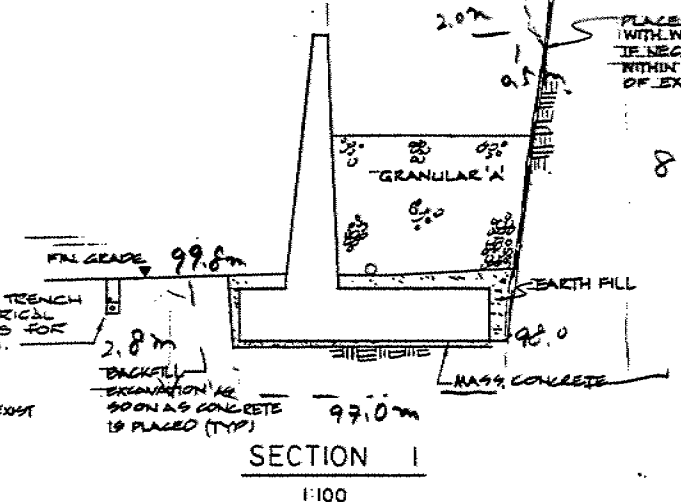
REMOVALS & CONSTRUCTION B



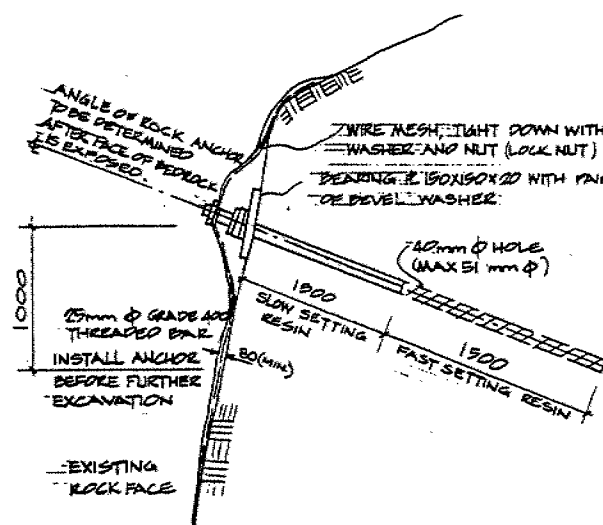
REMOVALS & CONSTRUCTION C



NEW RETAINING WALL ELEVATION



SECTION 1



TYPICAL RESIN ROCK ANCHOR

CONSTRUCTION STAGING

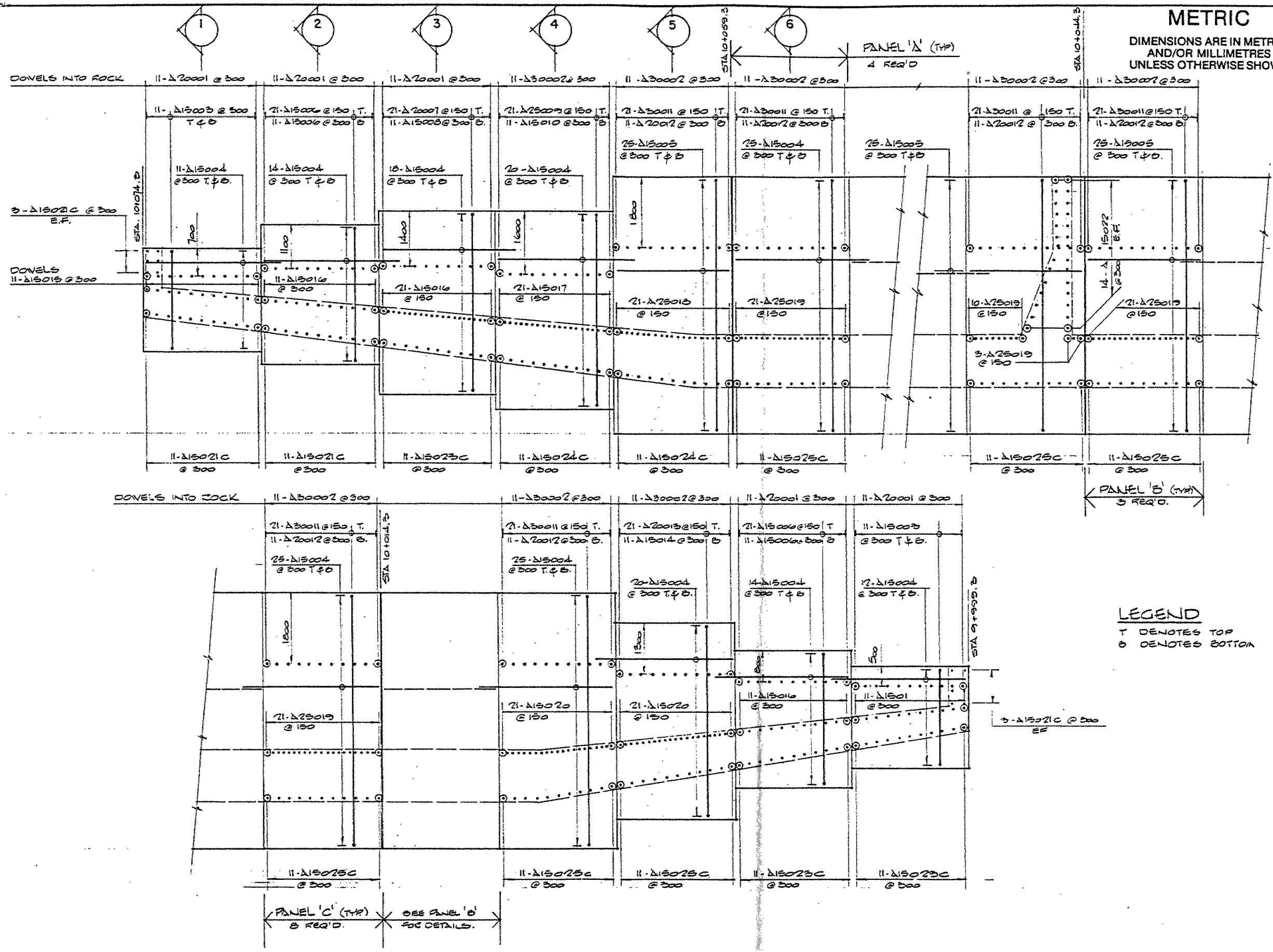
- Place temporary concrete barriers in accordance with the layout shown on the grading drawings.
- Complete removals in accordance with the grading drawings. Contractor is to take measures to ensure protection of the new high voltage cable installation in front of the wall.
- Removal of the existing bins shall proceed from each end. Contractor is to maintain the existing sand cushion between the back of the existing bins and the rock face by means of temporary lagging or other means suitable to the Engineer.
- Contractor is to ensure the stability of the existing bin wall at each stage of the progressive excavation and removal. A detailed proposal for removing and stabilizing the existing wall is to be submitted to the Engineer prior to commencing the work.
- As the existing rock face is exposed at each panel of the binwall, the Contractor shall place shotcrete and mesh in accordance with the contract drawings. The rock face shall be secured in this manner within 24 hours of being exposed.
- The Engineer will inspect the rock face as it is progressively exposed and will advise the contractor to install rock anchors if necessary.
- Complete excavation for structure foundations. At all locations, the Contractor will protect the exposed bedrock at the founding elevation with 150 mm of mass concrete within eight hours of exposing the bedrock surface. At all locations where the elevation of the bedrock varies from the theoretical founding elevation, the theoretical foundation elevation will be made up using mass concrete.
- Construct foundations and retaining walls.
- Install subdrain.
- Place and compact granular backfill to the structures.
- Repeat the above steps for the remaining binwall panels. Only one panel is to be removed at each end of the wall unless directed otherwise by the Engineer, and as noted below.
- When the new panels with wingwalls have been constructed, backfill is to be placed to the full height of the wall. The remaining panels may then be removed and the new wall constructed.
- Complete roadworks in accordance with the grading drawings.



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION	DATE
DESIGN R.T.	CHK K.P.	CODE	LOAD	DATE MAR 90
DRAWN R.A.	CHK R.T.	SITE 36-27	STRUCT	SCHEME
				DWG 3

PLAN-1118 87-03



METRIC

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WP No 380-85-01



HWY. 403 RETAINING WALL
FOOTING REINFORCEMENT

SHEET
50

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NOTES:

• FOR SECTION DETAILS SEE
DWG. 6

LEGEND

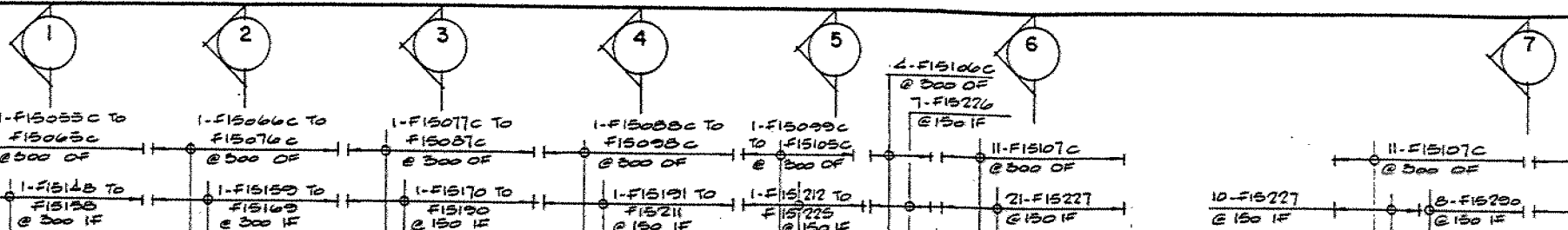
T DENOTES TOP
B DENOTES BOTTOM

FOOTING PLAN
1:50

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REVISIONS								
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DESIGN	27	CHK	10	CODE	LOAD	DATE	11-27-87	
DRAWN	10	CHK	27	SITE	30-27	STRUCT	SCHEME	DWG. 4



METRIC

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WP No 380-85-01

HWY. 403 RETAINING WALL
WALL REINFORCEMENT

SHEET
51

DELCAN

**ENGINEERS
PLANNERS
ARCHITECTS**

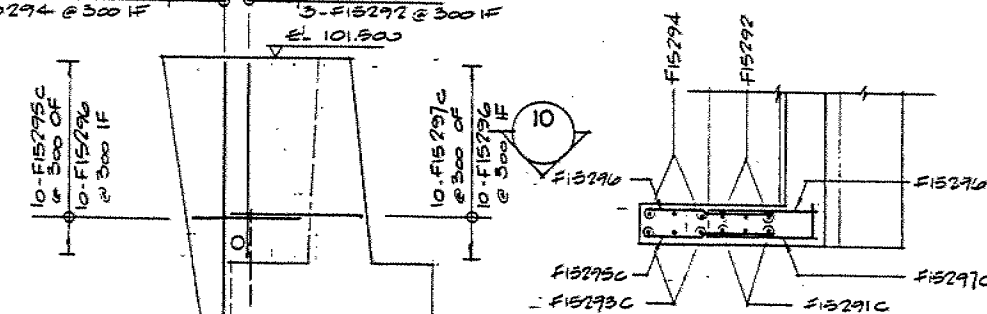
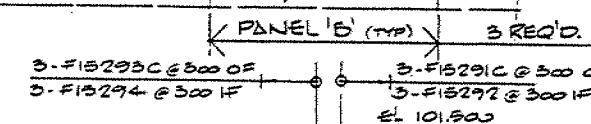
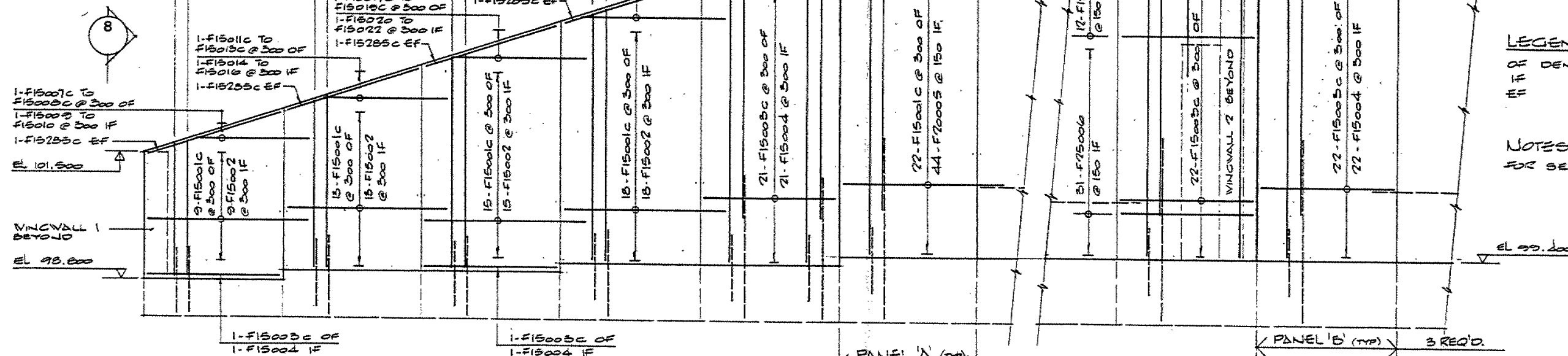


LEGEND

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NOTES :

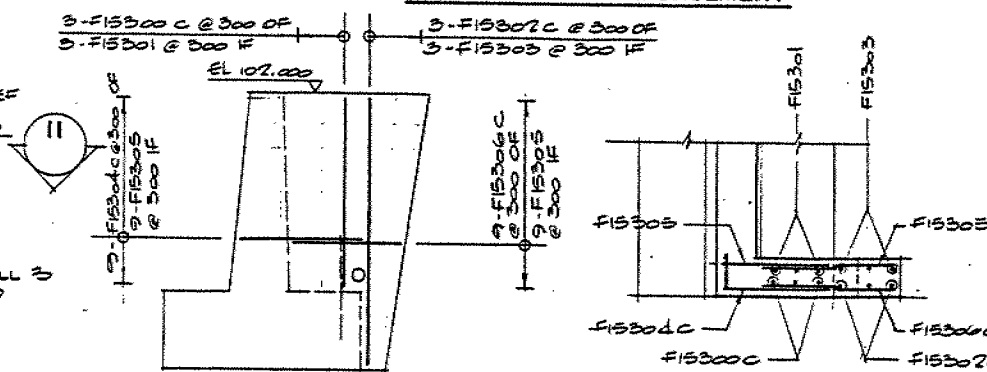
FOR SECTION DETAILS SEE DWG 5



ELEVATION 8
1:50

SECTION 10

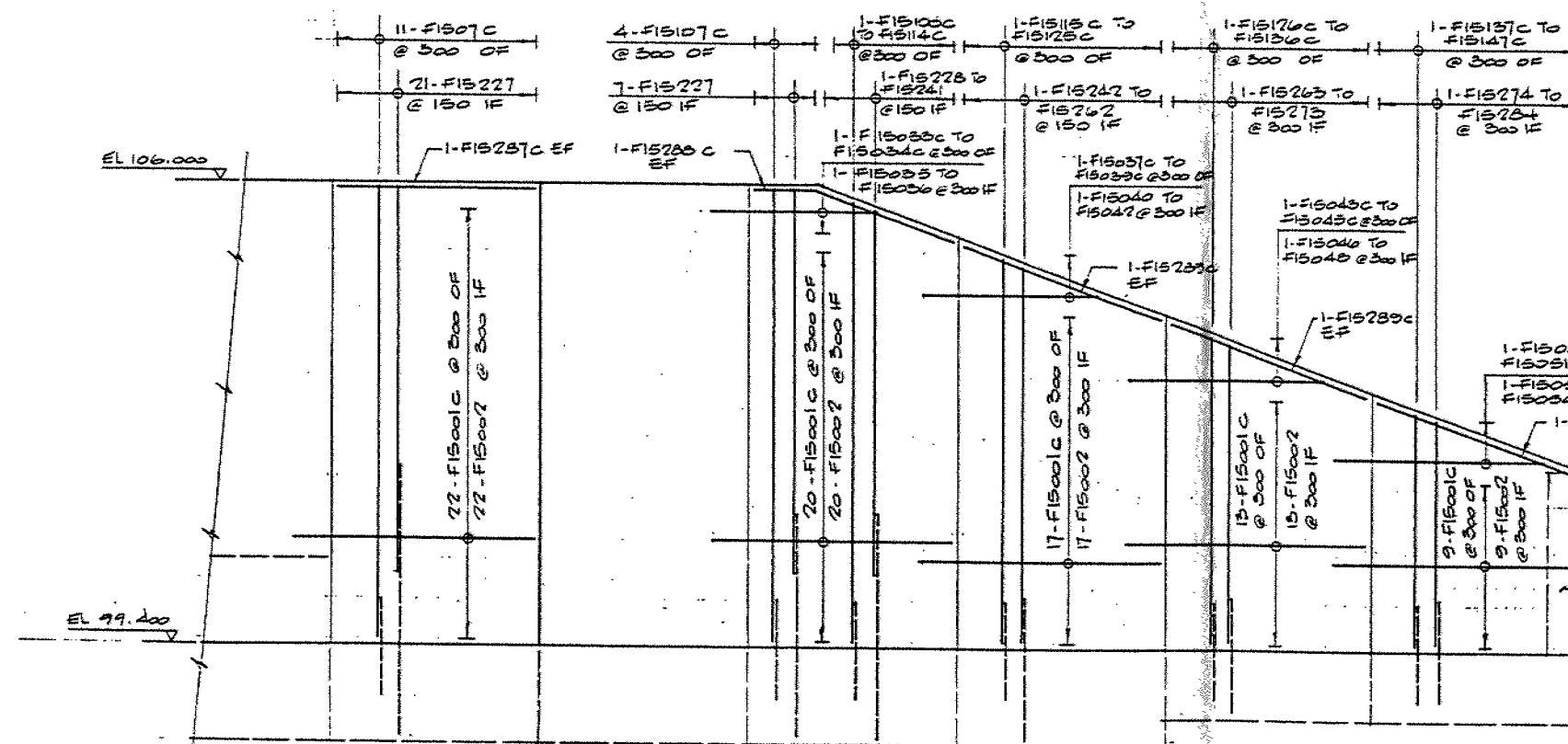
WINGWALL | REINFORCEMENT



ELEVATION 9

SECTION 11

WINGWALL 3 REINFORCEMENT



ELEVATION
1:50

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100 mm ON ORIGINAL DRAWING

REVISIONS					
	DATE	BY	DESCRIPTION		
	DESIGN ZT	CHK KP	CODE	LOAD	DATE / /



METRIC
DIMENSIONS ARE IN METRES
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CONT No 90-79
WP No 380-85-01

HWY. 403 RETAINING WALL
SECTIONS

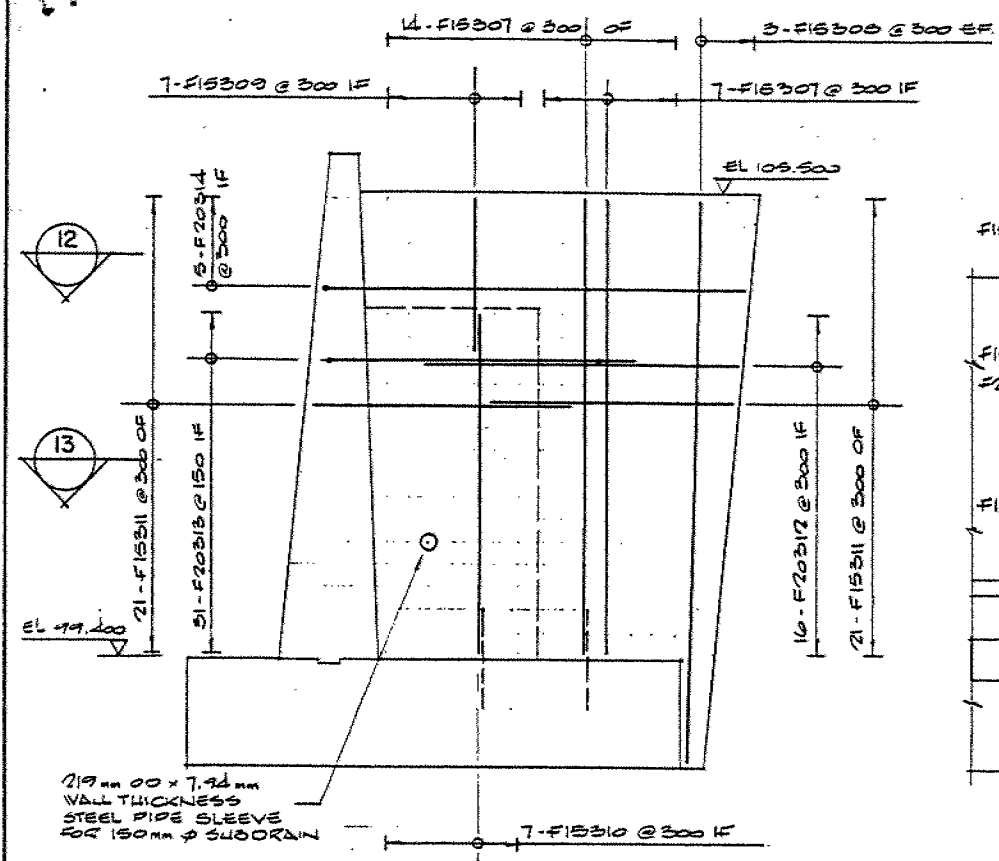
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DELCAN

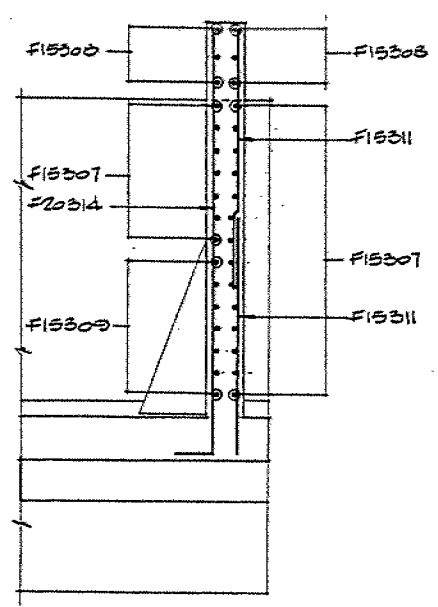
ENGINEERS
PLANNERS
ARCHITECTS

LEGEND

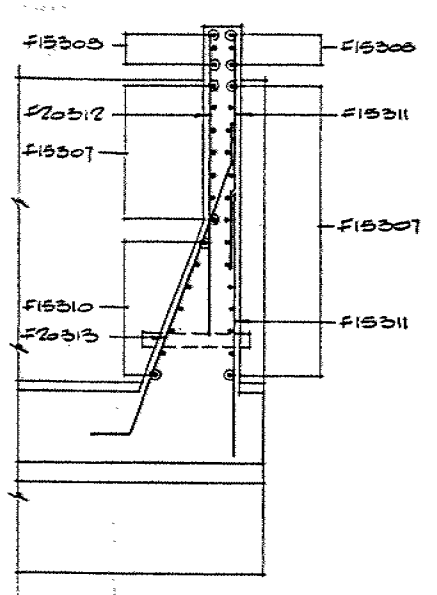
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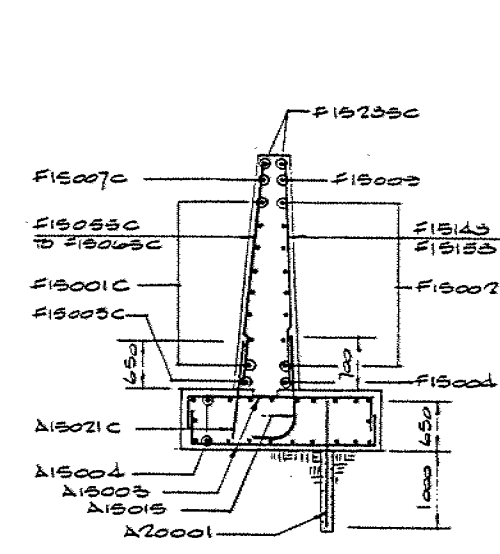
ELEVATION 7
1:50



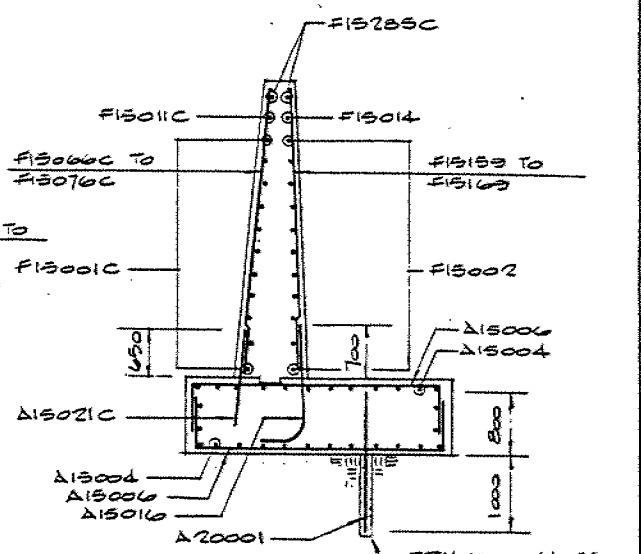
SECTION 12
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SECTION 13
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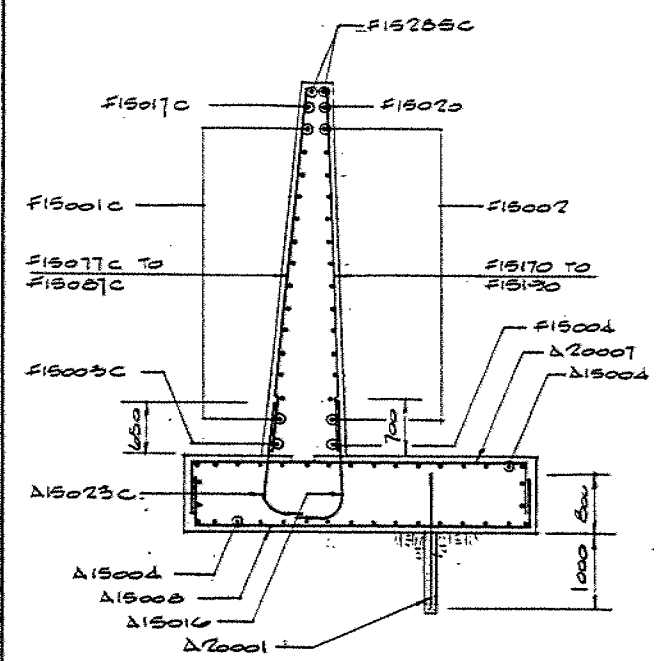


SECTION 1
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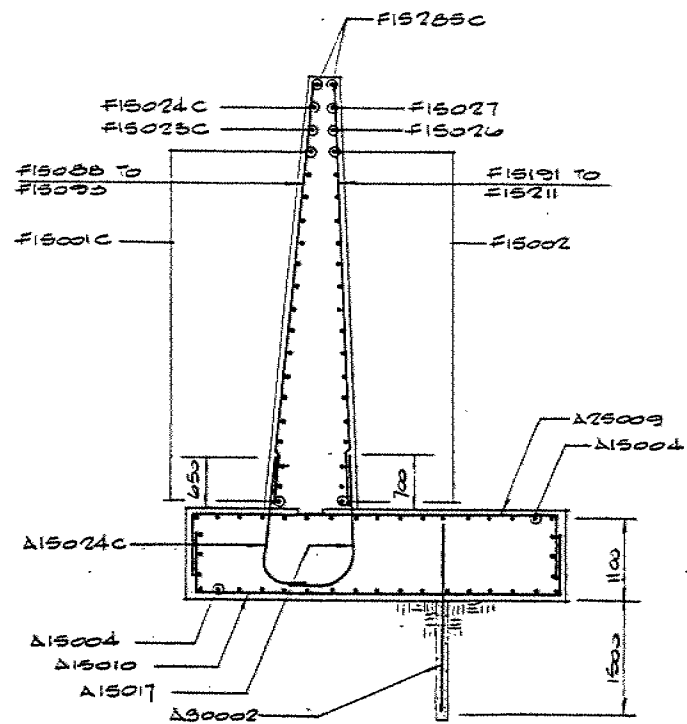


SECTION 2
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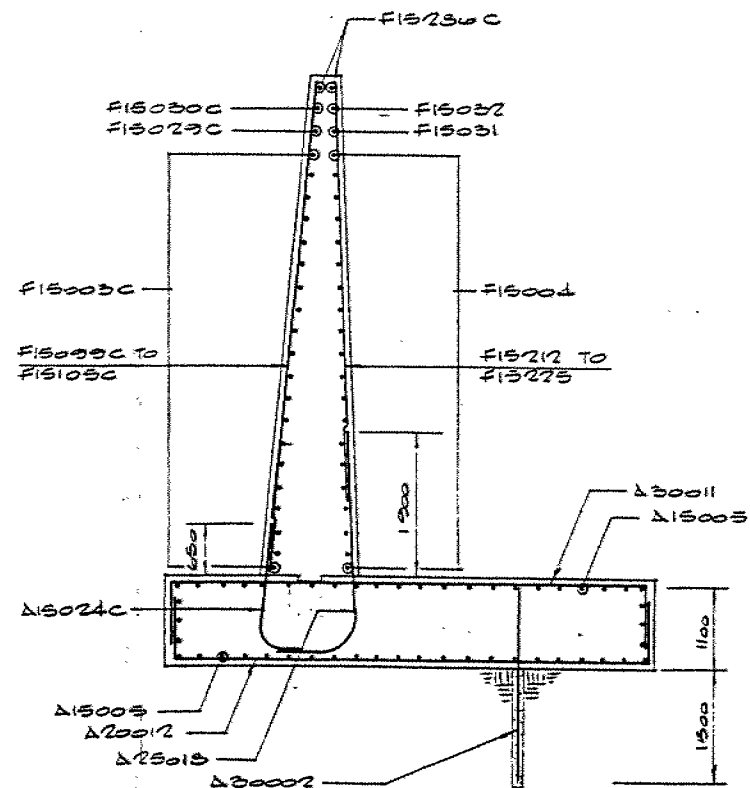
WINGWALL 2 REINFORCEMENT



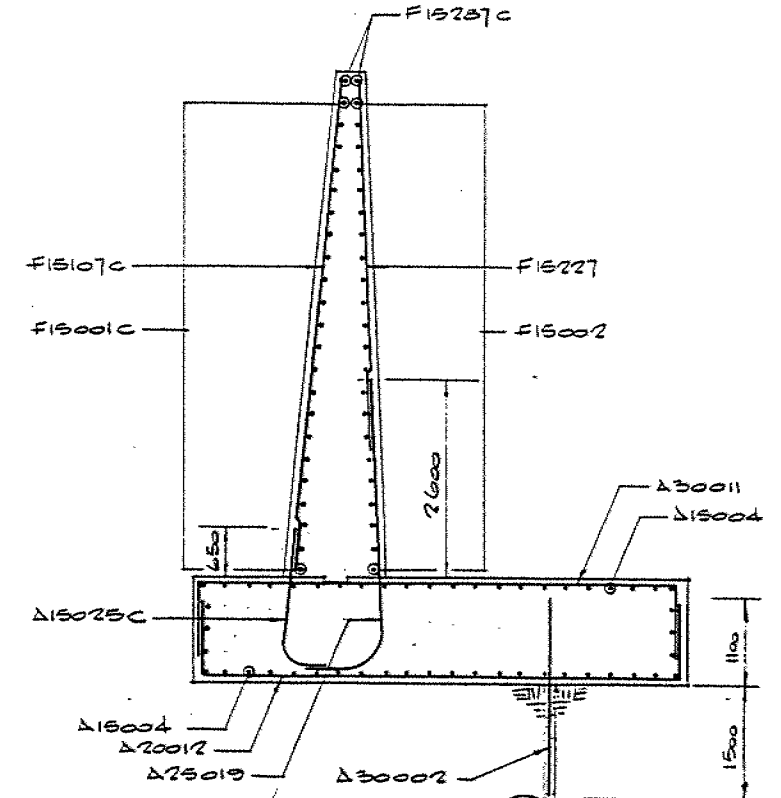
SECTION 3
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SECTION 4
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SECTION 5
1:50



SECTION 6
1:50

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION	DATE	BY
DESIGN RT	CHK	K.P.	CODE	LOAD	DATE MAR 90
DRAWN K.H.	CHK	RT	SITE 36-27	STRUCT	SCHEME DWG 6

CONT 90-79
ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION

WP 380-85-01

DIST 4

HWY 403

STR SITE

Steel Bin Retaining Wall Rehabilitation
Hwy. 403 Ramp NW Hamilton

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FOUNDATION INVESTIGATION REPORT

For

Steel Bin Retaining Wall Rehabilitation

Hwy. 403 Ramp NW Hamilton

W.P. 380-85-01

District 4, Burlington

INTRODUCTION

This report contains the results of a foundation investigation at the above mentioned site carried out during the period 88 06 28 to 88 06 30. This investigation was conducted at the request of Central Region Structural Office to determine subsoil conditions along the face of an existing steel bin wall retaining structure. The fieldwork consisted of 3 sampled boreholes and 3 dynamic cone penetration tests. The borings were advanced by NW casing and by NQ core barrels using an auger mounted on a trailer. Sampling was performed to a maximum depth of 6.3 m and the dynamic cone tests to a maximum depth of 2.8 m.

SITE DESCRIPTION

The site is located on Ramp NW and Hwy. 403 at the junction of Hwy. 403 and Hwy. 6. An existing steel bin wall supports a $1\frac{1}{2}:1$ backslope. The steel bin wall has rusted panels and some distortions of the wall stretchers and vertical posts. The backfill in the bins is shale which has weathered into clayey silt. The boreholes were drilled 3 m in front of the existing bin wall on the shoulder of the ramp.

SUBSURFACE CONDITIONS

The subsoil consists of up to 0.8 m of sand and gravel fill overlying 0.2 m to 2.0 m of sand and gravel and clayey silt (weathered shale). Shale of the Queenston formation underlies the weathered shale. All the sampled overburden was dry.

The boundaries of the different strata together with the field test results are shown on the Record of Borehole sheets contained in the Appendix of

this report. Dwg. No. 3808501-A shows the layout of the boreholes and includes the borehole drawings.

Sand and Gravel

The NW ramp's road bed consists of granular base to a depth of 0.8 m. The granular base is dry and dense.

Sand, Gravel & Clayey Silt (Weathered Shale)

Reddish brown weathered shale forms the subgrade of Ramp NW. This layer ranges from 0.2 to 2.0 m thick with the thickness increasing from East to West. This layer is dry and very dense and consists of a mixture of clayey silt and sand and gravel. The weathered shale acted as a non-cohesive soil but it does gain a consistency when water is added.

Shale Bedrock

Bedrock was encountered below the overburden material at the following elevations.

BH #1	2.7 m	below ground level
BH #2	1.3 m	below ground level
BH #3	1.0 m	below ground level

The shale bedrock is easy to auger and it deteriorates rapidly when exposed. The rock core samples were examined by Mr. S.A. Senior, Geological Engineer, and his description is included in the Appendix of this report.

DISCUSSION AND RECOMMENDATIONS

The existing steel bin wall has a slight distortion in its metal posts and the wall stretchers and panels are corroded. It is believed that the bin wall is directly in front of a shale bedrock face previously excavated for the cut and that the bin has been placed directly on bedrock about 1 metre below the shoulder. The bins are filled with shale which has weathered to clayey silt thereby trapping water in the bin wall and probably causing frost damage.

It is proposed to place a new retaining wall in front of the existing wall. The new retaining wall will have precast concrete panels inserted between pilasters. The pilasters will be supported on caissons and tied back with rock anchors.

In view of the encountered subsurface conditions the following foundation recommendations are being made:

Caissons

The retaining wall may be supported on caissons augered into the shale. The caissons can be placed on shale 4.0 m below the top of ground. At this depth it is estimated that a design load of 2000 kN may be used for a 1.2 m diameter caisson.

For the purposes of the O.H.B.D.C. the following values are recommended:

Factored Bearing Capacity at U.L.S. : 2000 kN

Bearing Capacity at S.L.S. Type II : Will not govern design since the loads required to produce detrimental settlement of the structures will be much larger than the recommended values for factored bearing capacity at U.L.S.

Backfill

Backfill to the retaining wall should consist of granular materials in accordance with MTO SP 109F03. Earth pressures should be computed based on a braced excavation with backfill pressure set at 0.65 Ka. The soil parameters to be used are:

	<u>Granular 'A'</u>	<u>Granular 'B'</u>	<u>Shale</u>
Angle of Internal Friction ϕ	35°	30°	25°
Unit Weight (kN/m ³) γ	22.8	21.2	21.2

The backfill pressure should be adjusted to reflect the surcharge created by the backslope.

Lateral Resistance

Lateral Resistance can be provided by rock anchors and by the caissons. The anchors should be strand type with double corrosion protection. The anchors may be placed at an angle of 45° from the horizontal. The free stressing zone can be assumed to extend from the wall to 1.0 m below the shoulder (probable top of rock). The bond zone therefore can be assumed to start at 1.0 m below the shoulder.

A bond stress of 300 kPa may be used for design. For the purposes of the O.H.B.D.C. the following values can be used:

Factored Capacity at U.L.S. : 600 kPa
Capacity at S.L.S. Type II : 300 kPa

However since the existing retaining wall will provide some lateral resistance a higher bond stress of 500 kPa can be used for design.

The minimum allowable bond zone length is 3 m. Rock anchors should be placed so that they will not intersect the steel posts of the existing bin wall. The anchors should be at least 300 mm from the steel posts.

Passive resistance can be provided by the caissons below the depth of frost penetration. Frost penetrates to a depth of 1.2 m in this area.

Passive resistance can be calculated from the equation:

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

(only horizontal surface case)

If the caissons are spaced no more than $2\frac{1}{2}$ D apart a solid wall of pressure resistance can be assumed between the caissons.

Anchor Testing

Proof Testing

The rock anchors should all be proof tested to 130% of the design load. Some adjustments may be required to the anchor lengths if rock is not encountered at 1 m below the shoulder as predicted. The proof tests should be conducted at the face of the existing bin wall using jacks braced against a whaler. The whaler will transmit the load evenly across the bin wall to prevent deformation of individual posts of the bin wall.

150%
1.5T

Anchor Installation

The contract drawings should indicate the gauge of steel that the Contractor will have to penetrate. The rear wall of the bin wall may be more difficult to cut through than the front wall stretchers as the Contractor will have to drill through the rear wall.

Frost Protection

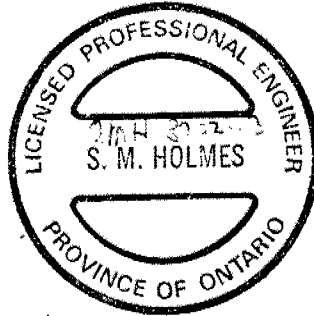
The retaining wall should be protected from frost damage by providing 1.2 m of granular material between the new wall and the existing steel bin wall. If there is not sufficient space for this quantity of granular materials then equivalent styrofoam insulation should be provided.

Drainage

Drainage is required between the existing steel bin wall and the new retaining wall. A subdrain should be installed at the base of the granular material to provide drainage.

MISCELLANEOUS

The fieldwork for this project was supervised by Mr. S. Holmes, Foundation Engineer. The equipment used was owned and operated by Longyear Canada Ltd. This report was prepared by Mr. S. Holmes and reviewed by Mr. M. Devata.



S. Holmes
S. Holmes, P.Eng.
Foundation Engineer

M. Devata
M. Devata, P.Eng.
Chief Foundation Engineer

APPENDIX

RECORD OF BOREHOLE No 1

METRIC

W P 380-85-01 LOCATION Ramp N-W West Corner Retaining Wall ORIGINATED BY SMH
 DIST 4 HWY 403 BOREHOLE TYPE NW Casing NQ Rock Core and Cone Test COMPILED BY SMH
 DATUM Assumed DATE 88 06 28 - 29 CHECKED BY SMH

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	SIRAT PLOT	NUMBER	TYPE			'N' VALUES	20					
99.9	Shoulder Surface												
99.8	Asphalt												
0.1	Sand and Gravel (Fill)		1	CS									
99.2													
0.7	Sand and Gravel Clayey Silt (Weathered Shale)		3	RC NQ	REC 15%								RQD = 0
			4	SS	19								
97.2			5	SS	100								
2.7	Shale Bedrock		6	RC NQ	REC 93%								RQD = 63%
			7	RC NQ	REC 100%								RQD = 74%
95.2	End of Borehole												
4.7	* Borehole Dry												

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 2

METRIC

W P 380-85-01 LOCATION Ramp N-W 38 m East of West Corner Retaining Wall ORIGINATED BY SMH
 DIST 4 HWY 403 BOREHOLE TYPE NW Casing NQ Rock Core, Cone Test COMPILED BY SMH
 DATUM Assumed DATE 88 06 29 CHECKED BY SMH

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100			
99.8	Shoulder Surface													
99.7	Asphalt													
0.1	Sand and Gravel (Fill)					*								
99.0	Sand and Gravel and Clayey Silt (Weathered Shale)		1	SS	71	28 cm								
98.5	Very Dense		2	AS	*									
1.3			3	RC NQ	REC 100%									RQD = 38%
			4	RC NQ	REC 100%									RQD = 69%
			5	RC NQ	REC 100%									RQD = 95%
	Shale													
	Bedrock		6	RC NQ	REC 98%									RQD = 95%
			7	RC NQ	REC 100%									RQD = 100%
93.5	End of Borehole													
6.3	* Borehole Dry * AS - Auger Sample													

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 3

METRIC

W P 380-85-01 LOCATION Ramp N-W 73 m East of West Corner Retaining Wall
 DIST 4 HWY 403 BOREHOLE TYPE NW Casing, NQ Rock Core & Cone Test ORIGINATED BY SMH
 DATUM Assumed DATE 88 06 30 COMPILED BY SMH
 CHECKED BY SMH

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 Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
99.6	Shoulder Surface																
98.4	Asphalt																
0.1	Sand and Gravel (Fill)																
98.8	Sand and Gravel and Clayey																
0.8	Silt (Weathered Shale)																
98.5																	
1.0																	
	Shale		1	RC	REC												RQD = 13%
	Bedrock			NQ	97%												
			2	RC	REC												RQD = 93%
				NQ	100%												
96.3	End of Borehole																
3.3	* Borehole Dry																

OFFICE REPORT ON SOIL EXPLORATION

DESCRIPTION OF ROCK CORE - WP 380-85-01

CORE RECOVERY				CORE DESCRIPTION	
HOLE #	DEPTH (m)	%CR*	%RQD*	DEPTH	DESCRIPTION
1	2.49- 3.18	93	63	2.49- 2.59	Weathered shale, gravel.
	3.18- 4.70	100	74	2.59- 4.70	SHALE , dark reddish brown interbedded with 22% light bluish grey SILTSTONE (up to 10 cm beds); very fine grained; medium strong rock; unweathered to slightly weathered; close spaced fractures: irregular, planar, horizontal, closed, unaltered; extremely close spaced fractures from 3.78-3.89 m.
2	1.30- 1.60	100	38	1.30- 6.32	SHALE , dark reddish brown interbedded with 19% light bluish grey SILTSTONE (up to 10 cm beds); very fine grained; medium strong rock; unweathered to slightly weathered; close spaced fractures: irregular to smooth, planar, horizontal, closed to slightly open, minor gypsum mineralization (1mm).
	1.60- 2.21	100	69		
	2.21- 3.18	100	95		
	3.18- 4.80	98	95		
	4.80- 6.32	100	100		
3	0.97- 1.78	97	13	0.97- 3.30	SHALE , dark reddish brown interbedded with 27% light bluish grey SILTSTONE (up to 10 cm beds); very fine grained; medium strong rock; unweathered to slightly weathered, clay seam from 1.40-1.42 m; close spaced fractures: irregular to smooth, planar, horizontal, closed; extremely close spaced fractures from 1.19-1.40 m; very close spaced fractures from 1.42-1.88 m.
	1.78- 3.30	100	93		

*CR = CORE RECOVERY

*RQD = ROCK QUALITY DESIGNATION

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

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 (R Q D), FOR MODIFIED RECOVERY, IS:

R Q D (%)	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

MECHANICAL PROPERTIES OF SOIL

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

STRESS AND STRAIN

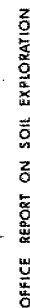
u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa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

PHYSICAL PROPERTIES OF SOIL

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

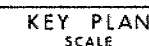
OFFICE REPORT ON SOIL EXPLORATION





RECORD OF BOREHOLE No 2							METRIC		
W P 380-85-01		LOCATION Ramp N-W 38 m East of West Corner Retaining Wall 3.0 m South of Retaining Wall Face		ORIGINATED BY SMH					
DIST 4 HWY 403		BOREHOLE TYPE NW Casing NQ Rock Core, Cone Test		COMPILED BY SMH					
DATUM Assumed		DATE 88 06 29		CHECKED BY		10			
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ■ QUICK TRIAXIAL x LAB VANE	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA Si Cl
ELEV DEPTH	DESCRIPTION	STRAT. PLT	NUMBER						
99.8	Shoulder Surface								
98.4	Asphalt								
0.1	Sand and Gravel (Fill)			*					
99.0									
0.8	Sand and Gravel and Clayey Silt (Weathered Shale) Very Dense		1	SS	71	28 cm			
98.5			2	AS	*				
1.3			3	RC NQ	REC 100%				RQD = 38%
			4	RC NQ	REC 100%				RQD = 69%
			5	RC NQ	REC 100%				RQD = 95%
	Shale Bedrock		6	RC NQ	REC 98%				RQD = 95%
			7	RC NQ	REC 100%				RQD = 100%
93.5	End of Borehole								
6.3	* Borehole Dry * AS - Auger Sample								



RECORD OF BOREHOLE No 3					METRIC						
W P 380-85-01		LOCATION Ramp N-W 73 m East of West Corner Retaining Wall (E East Corner) 3.0 m South of Retaining Wall Face			ORIGINATED BY SHM						
DIST 4 HWY 403		BOREHOLE TYPE NW Casing, NQ Rock Core & Cone Test			COMPILED BY SHM						
DATUM Assumed		DATE 88 06 30			CHECKED BY _____						
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 VALUES								
99.6	Shoulder Surface										
99.4	Asphalt										
0.1	Sand and Gravel (Fill)										
98.8	Sand and Gravel and Clayey Silt (Weathered Shale)										
0.8											
98.6				*							
1.0											
			1	RC NQ REC 97%							RQD = 13%
	Shale										
	Bedrock		2	RC NQ REC 100%							RQD = 93%
96.3											
3.3	End of Borehole										
	* Borehole Dry										

SHEET



- | | |
|---|---------------------------------------|
|  | Bore Hole |
|  | Dynamic Cone Penetration Test {Cone} |
|  | Bore Hole & 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 |

REV	DATE	BY	DESCRIPTION
Geocross No 30M5-159			
HWY No 403		DIST 4	
SUBMD 5H	CHECKED	DATE 1989 02 08	SITE
DRAWN 50	CHECKED	APPROVED	DWG 3808501-A

FOUNDATION INVESTIGATION REPORT

CONTRACT NO 90-79



Ministry of
Transportation

INDEX

<u>Page No:</u>	<u>DESCRIPTION</u>
1	Index
2	Abbreviations & Symbols
3 - 13	Foundation Investigation Report for: Steel Bin Retaining Wall Rehabilitation W.P. 380-85-01, Site - Hwy. 403, District 4, Burlington

Note: For purposes of the contract, this report supercedes all other Foundation Reports prepared by, or for the Ministry in connection with the above mentioned project.

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

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{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_t	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

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

FOUNDATION INVESTIGATION REPORT
For
Steel Bin Retaining Wall Rehabilitation
Hwy. 403 Ramp NW-Hamilton
W.P. 380-85-01
District 4, Burlington

INTRODUCTION

This report contains the results of a foundation investigation at the above mentioned site carried out during the period 88 06 28 to 88 06 30 and 89 11 15 to 89 11 17. These investigations are conducted at the request of Central Region Structural Office to determine subsoil conditions along the face of an existing steel bin wall retaining structure. The fieldwork consisted of 6 sampled boreholes and 3 dynamic cone penetration tests. The borings were advanced by NW casing and by NQ or BXL core barrels using an auger mounted on a trailer. Sampling was performed to a maximum depth of 6.3 m and the dynamic cone penetration tests to a maximum depth of 2.8 m.

SITE DESCRIPTION

The site is located on Ramp NW and Hwy. 403 at the junction of Hwy. 403 and Hwy. 6. An existing steel bin wall supports a $1\frac{1}{2}$:1 backslope. The steel bin wall has rusted panels and some distortions of the wall stretchers and vertical posts. The backfill in the bins is shale which has weathered into clayey silt. The boreholes were drilled 3 m in front of the existing bin wall on the shoulder of the ramp.

SUBSURFACE CONDITIONS

The subsoil consists of up to 0.8 m of sand and gravel fill overlying 0.2 m to 2.0 m of sand and gravel and clayey silt (weathered shale). Shale of the Queenston formation underlies the weathered shale. All the sampled overburden was dry.

The boundaries of the different strata together with the field test results are shown on the Record of Borehole sheets contained in the Appendix of this report. Dwg. No. 3808501-A shows the layout of the boreholes and includes the borehole drawings.

Sand and Gravel

The NW ramp's road bed consists of granular base to a depth of 0.8 m. the granular base is dry and dense.

Sand, Gravel & Clayey Silt (Weathered Shale)

Reddish brown weathered shale forms the subgrade of Ramp NW. This layer ranges from 0.2 to 2.0 m thick with the thickness increasing from East to West. This layer is dry very dense and consists of a mixture of clayey silt and sand and gravel. The weathered shale acted as a non-cohesive soil but it does gain a consistency when water is added.

Shale Bedrock

Bedrock was encountered below the overburden material at the following depths:

BH #1	2.7 m	below ground level
BH #2	1.3 m	below ground level
BH #3	1.0 m	below ground level
BH #4	1.5 m	below ground level
BH #5	2.1 m	below ground level
BH #6	1.6 m	below ground level

The shale bedrock is easy to auger and it deteriorates rapidly when exposed. The rock core samples were examined by Mr. S.A. Senior Geological Engineer, and his description is included in the Appendix of this report.

MISCELLANEOUS

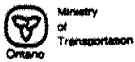
The fieldwork for this project was supervised by Mr. S. Holmes, Foundation Engineer. The equipment used was owned and operated by Longyear Canada Ltd. This report was prepared by Mr. S. Holmes and reviewed by Mr. M. Devata.



Taecheul Kim
T.C. Kim, P.Eng.
Foundation Engineer

M. Devata
M. Devata, P.Eng.
Chief Foundation Engineer

APPENDIX

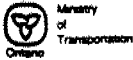


RECORD OF BOREHOLE No 1										METRIC			
W P 380-85-01		LOCATION Ramp N-W West Corner Retaining Wall		ORIGINATED BY SMH									
DIST 4 HWY 403		BOREHOLE TYPE NW Casing NO Rock Core and Cone Test		COMPILED BY SMH									
DATUM Assumed		DATE 88 06 28 - 29		CHECKED BY									
SOIL PROFILE		SAMPLES		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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER			TYPE	'N' VALUES	20					
99.9	Shoulder Surface												
99.8	Asphalt												
0.1	Sand and Gravel (Fill)		1	CS									
99.2													
0.7	Sand and Gravel		3	RC	REC								RQD = 0
	Clayey Silt (Weathered Shale)			NQ	15%								
			4	SS	19								
97.2			5	SS	100	13 cm							RQD = 63%
2.7	Shale		6	RC	REC								
	Bedrock			NQ	93%								
			7	RC	REC								RQD = 74%
				NQ	100%								
95.2	End of Borehole												
4.7	* Borehole Dry												

OFFICE REPORT ON SOIL EXPLORATION

+3, x⁵: Numbers refer to Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE



RECORD OF BOREHOLE No 2

METRIC

W P 380-85-01 LOCATION Ramp N-W 38 m East of West Corner Retaining Wall ORIGINATED BY SMR
 DIST 4 HWY 403 BOREHOLE TYPE NW Casing NO Rock Core, Cone Test COMPILED BY SMR
 DATUM Assumed DATE 88 06 29 CHECKED BY 12

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
99.8	Shoulder Surface										
99.7	Asphalt										
0.1	Sand and Gravel (Fill)										
99.0	Sand and Gravel and Clayey Silt (Weathered Shale)		1	SS	71	28 cm					
98.5	Very Dense		2	AS	*						
1.3			3	RC NQ	REC 100%						RQD = 38%
			4	RC NQ	REC 100%						RQD = 69%
			5	RC NQ	REC 100%						RQD = 95%
	Shale										
	Bedrock		6	RC NQ	REC 98%						RQD = 95%
			7	RC NQ	REC 100%						RQD = 100%
93.5	End of Borehole										
93.3	* Borehole Dry * AS - Auger Sample										

OFFICE REPORT ON SOIL EXPLORATION



RECORD OF BOREHOLE No 3

METRIC

W P 380-85-01 LOCATION Ramp N-W 73 m East of West Corner Retaining Wall ORIGINATED BY SMH
 DIST 4 HWY 403 BOREHOLE TYPE NW Casing, NQ Rock Core & Cone Test COMPILED BY SMH
 DATUM Assumed DATE 88 06 30 CHECKED BY SMH

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20 40 60 80 100	Wp	W	W _L		
99.6	Shoulder Surface												
98.4	Asphalt												
0.1	Sand and Gravel (Fill)												
98.8	Sand and Gravel and Clayey Silt (Weathered Shale)												
0.8													
98.6													
1.0													
	Shale Bedrock		1	RC NQ	REC 97%								RQD = 13%
			2	RC NQ	REC 100%								RQD = 93%
96.3	End of Borehole												
3.3	* Borehole Dry												

OFFICE REPORT ON SOIL EXPLORATION

+3, x5: Numbers refer to Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE



RECORD OF BOREHOLE No 4										METRIC					
W P 380-85-01		LOCATION Ramp N-W 12m East of West Corner Retaining Wall 3.0m South of Retaining Wall						ORIGINATED BY PM							
DIST 4 HWY 403		BOREHOLE TYPE NW Casing, BXL Rock Core						COMPILED BY PM							
DATUM Assumed		DATE 89 11 15 to 89 11 16						CHECKED BY							
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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE						
99.9	Shoulder Surface														
0.0	Asphalt														
0.1															
99.2	Sand and Gravel (Fill)		1	CS											
0.7	Sand & Gravel & Clayey Silt (Weathered Shale) Transition Zone		5	SS	71/	5cm									
98.4	Very Dense		7	RC BXL											
1.5			8	RC BXL											
	Shale Bedrock		9	RC BXL											
			10	RC BXL											
96.1															
3.8	End of Borehole														
	* Water Level not Established														

OFFICE REPORT ON SOIL EXPLORATION

<div style="display: flex; justify-content: space-between;"> <div> RECORD OF BOREHOLE No 5 Ramp N-W 1m East of West Corner Retaining Wall 3.0m South of Retaining Wall Face </div> <div> METRIC </div> </div>														
W P 380-85-01		LOCATION 3.0m South of Retaining Wall Face				ORIGINATED BY PM								
DIST 4 HWY 403		BOREHOLE TYPE NW Casing, BXL Rock Core				COMPILED BY PM								
DATUM Assumed		DATE 89 11 17				CHECKED BY								
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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20 40 60 80 100	SHEAR STRENGTH kPa					
99.9	Shoulder Surface													
0.0	Asphalt													
0.1	Sand and Gravel (Fill)													
99.1														
0.8	Sand and Gravel and Clayey Silt (Weathered Shale) Compact to Very Dense		1	SS	19	*	99							
			2	SS	81									
97.8			3	SS	120/	13cm	98							
2.1			4	SS	40/	1cm								
	Shale Bedrock		5	RC BXL			97							
96.7														
3.2	End of Borehole													
	* Water Level not Established													

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 6										METRIC					
W P 380-85-01		LOCATION Ramp N-W 6.5m East of West Corner Retaining Wall				ORIGINATED BY PM									
DIST 4 HWY 403		BOREHOLE TYPE NW Casing, BXL Rock Core				COMPILED BY PM									
DATUM Assumed		DATE 89 11 17				CHECKED BY									
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 Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60					
99.9	Shoulder Level														
99.8	Asphalt														
0.1	Sand and Gravel (Fill)														
99.2															
0.7	Sand & Gravel, Clayey Silt, Weathered Shale		1	SS	69/23cm										
	Transition Zone		2	SS	100/9cm										
98.3	Very Dense														
1.6	Sound Shale		3	RC											
97.9	Bedrock			BXL											
2.0	End of Borehole														
	* Water Level not Established														

OFFICE REPORT ON SOIL EXPLORATION

*3, *5: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE

DESCRIPTION OF ROCK CORE - WP 380-85-01

CORE RECOVERY				CORE DESCRIPTION	
HOLE #	DEPTH (m)	%CR*	%RQD*	DEPTH	DESCRIPTION
1	2.49- 3.18	93	63	2.49- 2.59	Weathered shale, gravel.
	3.18- 4.70	100	74	2.59- 4.70	SHALE , dark reddish brown interbedded with 22% light bluish grey SILTSTONE (up to 10 cm beds); very fine grained; medium strong rock; unweathered to slightly weathered; close spaced fractures: irregular, planar, horizontal, closed, unaltered; extremely close spaced fractures from 3.78-3.89 m.
2	1.30- 1.60	100	38	1.30- 6.32	SHALE , dark reddish brown interbedded with 19% light bluish grey SILTSTONE (up to 10 cm beds); very fine grained; medium strong rock; unweathered to slightly weathered; close spaced fractures: irregular to smooth, planar, horizontal, closed to slightly open, minor gypsum mineralization (1mm).
	1.60- 2.21	100	69		
	2.21- 3.18	100	95		
	3.18- 4.80	98	95		
	4.80- 6.32	100	100		
3	0.97- 1.78	97	13	0.97- 3.30	SHALE , dark reddish brown interbedded with 27% light bluish grey SILTSTONE (up to 10 cm beds); very fine grained; medium strong rock; unweathered to slightly weathered, clay seam from 1.40-1.42 m; close spaced fractures: irregular to smooth, planar, horizontal, closed; extremely close spaced fractures from 1.19-1.40 m; very close spaced fractures from 1.42-1.88 m.
	1.78- 3.30	100	93		

*CR = CORE RECOVERY

*RQD = ROCK QUALITY DESIGNATION

ROCK CORE DESCRIPTION **WP 380-85-01**

Page 1 of 1.

CORE RECOVERY					CORE DESCRIPTION	
BH#	RC#	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
4	6	1.22-1.45	0	0	1.22-3.80	SHALE , dark reddish brown interbedded with greyish green SILTSTONE (5%); very fine grained; weak to very weak rock; slightly weathered to unweathered; very close to close spaced fractures: horizontal laminar joints; smooth, planar.
	7	1.42-1.74	60	0		
	8	1.80-2.41	75	0		
	9	2.41-3.35	79	55		
	10	3.35-3.80	100	54		
5	5	2.29-3.20	56	49	2.29-3.20	SHALE , dark reddish brown interbedded with greyish green SILTSTONE (18%); very fine grained; weak to very weak rock; slightly weathered to unweathered; very close to close spaced fractures: horizontal laminar joints; smooth, planar.
6	3	1.40-2.01	61	40	1.40-2.01	SHALE , dark reddish brown interbedded with greyish green SILTSTONE (11%); very fine grained; weak to very weak rock; slightly weathered to unweathered; very close to close spaced fractures: horizontal laminar joints; smooth, planar.

*CR = CORE RECOVERY

*RQD = ROCK QUALITY DESIGNATION

(NOTE: Depths are approximated in zones of poor core recovery)

Logged by: SAS, Soils and Aggregates Section.

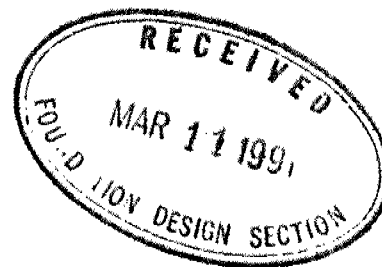


Golder Associates Ltd.
CONSULTING ENGINEERS

March 4, 1991

Our ref: 911-1317

MINISTRY OF TRANSPORTATION ONTARIO
Central Building
1201 Wilson Avenue
Room 315
Foundation Design Section
Downsview, Ontario
M3M 1J8



ATTENTION: Mr. M. S. Devata, P.Eng.

**RE: BIN WALL REMOVAL
HIGHWAY 403 NEAR PLAINS ROAD**

Dear Sirs:

Golder Associates was retained by the Ministry of Transportation Ontario (MTO) to carry out a review of a proposed alternative to a concrete retaining wall structure to be built on Highway 403 westbound, just west of Highway 6. The terms of reference included the review of documentation prepared by the contractor's consultants Mountainview Geotechnical Ltd., an MTO Foundation Investigation Report on Contract No. 90-79, the contract documents prepared by Delcan for Plains Road over Highway 403 Bridge Replacement and Retaining Wall Reconstruction, Contract drawings for Contract No. 90-79 and a statement by Underground Services 1983 Ltd. on the proposed alternative stabilization methodology. In addition, a site visit was carried out on February 26, 1991 by Dr. J.M. Boyd, Golder Associates and Mr. T. Kim, MTO.

SITE CONDITIONS

As part of the proposed construction contract, an existing steel bin wall is to be removed, cleared back to the existing rock surface and replaced by a concrete retaining wall, backfilled with granular backfill for the purpose of drainage and capped with earth backfill to permit the growth of vegetation.

The existing bin wall is to be excavated and replaced in sections to ensure the ongoing stability of the slope. The existing wall is approximately 8.5 metres in height, decreasing at both ends to approximately 3.5 metres in height. Above the wall, existing backfill meets the rockface which rises at 1.5:1 to a crest line in excess of 20 metres above the top of the existing wall. Along the crest there is a fence about 2 metres back from the edge enclosing level property with the building set back an additional 50 to 70 metres.

The slope is covered with existing vegetation on topsoil and weathered rock. Wire mesh fencing has been placed on the face of the slope to impede soil creep and to generally protect the soil slope face against erosion. At the time of the visit, there was also a light covering of snow over the face. There appears to be no existing signs of any instability on this slope, although some surficial movement was evident on an adjacent slope closer to Plains Road. At each end of the retaining wall, a drainage catchment had been provided with a collector manhole at the top of the slope at each end of the retaining wall section. The existing bin wall was corroded in a number of places allowing the backfill to be seen through the face of the wall.

During the site visit, a small test pit was dug by hand near the base of the rock section of the wall just above the backfill. The test pit revealed about 200 mm of topsoil and weathered bedrock followed by red and mottled greyish-green mudstones of the Queenston formation.

Although an attempt was made to dig a small test pit at the crest of the slope, this was unsuccessful due to the wire mesh laid on the slope face.

Based on discussions with the MTO, the original slope excavation was ripped and remained stable during the construction of the bin wall even though the exposed rock appeared quite rubbly and weathered.

DISCUSSION OF CONTRACTOR'S PROPOSAL

Because of concerns over the ongoing stability of the rock wall during the replacement of the existing bin wall structure, the specifications require the contractor to remove the wall in sections and replace it with the concrete retaining wall and the backfill before proceeding to the removal of the rest of the retaining wall. Traffic flow will have to be maintained during this operation and therefore the contractor will be working at the base of the hill in a relatively narrow construction zone to carry out the work. He will undoubtedly have difficulty in terms of limited access and restricted sequencing of his activities. A relatively slow progress in construction can therefore be anticipated.

The contractor has proposed the excavation of the bin wall in horizontal slices from the top of the wall with the concurrent placement of a stabilization mat dowelled into the rock and a geosynthetic drainage fabric over the face of the rock. This would in turn be covered with 75 to 150 mm of shotcrete. Thereafter, the bin wall and backfill would be removed in 1.8 m vertical lifts with rockbolts being installed at the base of the first lift. For each lift, an initial layer of 75 to 150 mm of shotcrete would be placed on the rock, drainage fabric would be installed over the shotcrete and 75 to 150 mm of shotcrete would be used to cover it. This operation would continue to the base of the existing bin wall. Once excavation was complete, a toe pocket equipped with vertical dowels would be excavated and a rebar mat placed over the entire face. Shotcrete would be used to enclose the first rebar mat.

Thereafter, a second rebar mat would be placed and further shotcrete used to cover this to a minimum depth of 50 mm.

EVALUATION OF CONTRACTOR'S PROPOSAL

Drainage

The contractor proposes to apply shotcrete to the face of the rock as it is exposed in the excavation sequence. This will protect the rock against further ravelling and deterioration during construction and is a sensible precaution in terms of protecting the rock from further deterioration and protecting the contractor's work force from small pieces of rock falling off the face. Unfortunately, this initial layer of shotcrete will also quite effectively prevent surface drainage of the broken rock mass and will essentially ensure that very little seepage will take place from the rock mass into the covering drainage fabric. The Queenston shale at the site is quite impervious in its intact condition, but there is expected to be a zone of several metres thickness which is fractured due to the original construction and due to weathering effects which could allow the percolation of water. Some form of hydraulic connection, therefore, is necessary between this fractured surface rock zone and the drainage fabric to ensure that it works satisfactorily.

The contractor has not provided any details of the proposed termination of the drainage fabric in a drainage pipe. Indeed, on Page 5 of the Mountainview Geotechnical report there is an indication that the fabric terminates some distance above the collector drain which appears to have been shotcreted in place at the excavated toe of the proposed shotcrete wall. Ideally, the drainage fabric should be brought to the toe of the face and wrapped around the drainage pipe at this location.

Because the drainage fabric will only be covered by approximately a maximum 400 mm thickness of shotcrete wall, it is likely that during protracted cold weather the drainage fabric

will freeze and its benefit will be lost. Indeed, it might even act to prevent the drainage of water protected from freezing at greater depth within the slope.

Temporary Slope Stability

In view of the apparent rock condition in the test pit, which was dug by the author in the existing slope, it is considered likely that the contractor could, in fact, take the bin wall down and maintain stability of the rock slope by means of a shotcrete application. To be certain of the success of this process, however, allowance should be made for spot rock bolting as required and for the placement of wire mesh on the face as the excavation proceeds. It is possible, however, that construction activities could result in the surficial movement of overlying topsoil and weathered bedrock along the face of the slope above the steep portion of the cut. Before allowing the contractor to adopt this construction approach, it would be prudent to excavate several hand dug test pits at the base of the rock slope and at the crest of the rock slope to verify the uniformity and thickness of the overlying soil material. If this slippage occurred during construction, additional steps involving the replacement of the soil and remediation of the fencing wire and overlying vegetation might also be necessary.

A regular process of examination of the exposed rock slope by a qualified specialist should also be required to ensure that any unfavourable conditions in the exposed rock were recognized and appropriate remedial action taken before proceeding with the bin wall removal.

Shotcrete Quality

The existing shotcrete specification is intended for material to be applied as a temporary support in relatively thin layers to the rock face and, as such, is not completely appropriate for the contractor's proposed purpose.

As a temporary support, shotcrete is frequently mixed with an accelerating additive which is intended to provide high early strength, an additive which is inappropriate for long term requirements since it can result in early deterioration of exposed shotcrete. In order to achieve the thicknesses of shotcrete placement which are proposed by the contractor, without the use of such a deteriorating additive, it would be necessary to use a silica fume shotcrete which permits the build-up of greater thicknesses without resorting to a flash set additive. Great care would be required to ensure that the shotcrete was placed without sagging or without the development of gaps and holes behind the rebar which could also result in earlier deterioration of the finished wall.

The existing specification allows the contractor to use a hand held applicator nozzle which is the way the majority of temporary support shotcrete is placed. For an application of the proposed quality, however, more elaborate equipment such as remote-controlled applicator booms and the like would be appropriate to allow the operator the necessary fine control over the placement of the shotcrete. Hand placement under the circumstances would not be recommended because of the associated difficulty of placing the material accurately and to a sufficient standard to ensure that the final wall would remain intact and visually acceptable over a protracted period. It would also be appropriate to consider the addition of fibres to the shotcrete for this purpose.

Adequacy of Design

There is expected to be a very limited load imposed upon the retaining wall by the rock. In general, soft or broken rock would be removed during construction, and given the level of protection provided by the shotcrete cover layer, it is very unlikely that much in the way of loading would actually occur on the shotcrete retaining wall throughout its lifetime.

The uncertainty of in situ conditions, however, suggest that the contractor should be prepared with some form of contingency for the possibly very irregular rockface that he

might actually expose during construction. Backfill lends itself ideally to this sort of circumstance. Placement of shotcrete and a shotcrete retaining wall on the face does not, except to suggest that allowances should be made for considerable extra volumes of shotcrete during construction.

CONCLUSIONS AND RECOMMENDATIONS

1. A permanent shotcrete wall of the type proposed by the contractor could be constructed.
 - (a) Provision should be made for the use of wire mesh and rock bolts as supplementary support to the original coating of shotcrete during the excavation of the face.
 - (b) Additional investigation in the form of shallow test pits should be carried out to ensure that the loose and surficial layer overlying the rock slope remains thin across the site.
 - (c) The contractor would need to submit much more detail on the proposed shotcrete mix and shotcrete application methods and equipment in order for the MTO to be completely certain of the appropriateness of the proposal.
 - (d) It would be a good idea for the contractor to show to the MTO an example of a shotcrete wall constructed in this manner which has stood the test of time.
 - (e) More details are required on the proposed revised drainage scheme. In particular, the contractor should suggest methods whereby the anticipated problem with freezing could be overcome.

2. Consideration should be given to removing the construction sequence restrictions imposed by the current contract.
 - (a) Additional hand dug test pits at the crest and base of the rock slope to confirm the thickness of the overlying loose surficial material should be carried out as part of this consideration.
 - (b) In the course of the excavation process, the site should be examined on a regular basis to ensure that the exposed rock conditions encountered during the removal of the existing bin wall correspond to the stability conditions anticipated by the current records.
 - (c) It is quite possible that some form of surficial slip will occur during this phase of the construction and provision should be made to remove the material from the slip and to reconstitute the face of the slope at the end of construction.
3. The replacement retaining wall as designed would produce a better final product than the proposed shotcrete wall.
 - (a) There will be a better finishline on the surface on a poured concrete wall than could be achieved with even the best technique and equipment in shotcrete.
 - (b) The proposed construction method whereby the bin wall is removed in segments and replaced in segments provides a much more defined opportunity for examining in-situ conditions in a safe manner than the excavation method proposed by the contractor.
 - (c) The drainage provisions are more positive than can be arranged with a shotcrete wall.

- (d) The design itself is much more conservative than that proposed by the contractor.
- (e) A final concrete wall provides a known long term performance capability which is not matched by the shotcrete wall.
- (f) There will be less waste to be disposed of in the specified method of construction than in that proposed by the contractor. This is particularly the case since large amounts of shotcrete would rebound during construction and would have to be loaded and taken off site for disposal.

Yours truly,

GOLDER ASSOCIATES LTD.

A handwritten signature in dark ink, appearing to read "J. M. Boyd". The signature is fluid and cursive, with a horizontal line extending from the end.

Dr. J. M. Boyd, P.Eng.

JMB/cg

memorandum



To: V.F. Boehnke
Head, Structural Section
4th Floor, Atrium Tower

Date: 1990 02 26

Atten: R. Barsalou, Sr. Structural Eng.

From: Foundation Design Section
Room 315, Central Building

Re: Hwy. 403 Retaining Wall
W.P. 380-85-01
District 4, Burlington

Further to a meeting at your office on January 31, 1990 and subsequent telephone conversation between your Mr. K. Wong and our T. Kim on February 19, 1990, this section has reviewed the slope stability of the temporary rock slope at the aforementioned structure and provides the following comments for the technical details and construction methodology for the retaining wall reconstruction.

Stability Conditions of Temporary Rock Slope

Stability analyses were carried out to evaluate the effect of the excavation cut to the backslope for the retaining wall foundation at the above structure.

An effective stress analysis was applied for calculation of slope stabilities of the excavation cut. Since the proposed cut is only temporary during the construction of the retaining wall, a minimum factor of safety of 1.2 was incorporated for the analyses. Based on the analyses, the following conclusions have been derived:

For the overall stability of the unsupported rock surface prior to backfilling, it is noted that as expected, a shallow surficial failure is most likely to develop at or above the founding elevation of footing rather than a deep seated failure. It is therefore our opinion that mass concrete shotcrete should be placed over the exposed rock surface with a steel wire mesh as soon as the shale bedrock is excavated to the design surface in order to prevent the shaly bedrock from further deterioration. It should be noted that if further support of rock surface is required, rock bolts should be installed along the exposed rock slope with an approximate length of 3 metres as shown on Figure 1.

.../2

The spacing of rock bolts should be determined by Contractors based on the exposed rock conditions.

A current draft non-standard special provision for the rock bolts, steel mesh and shotcrete procedures at the rock surface are included in this memo for your design purpose.

Review of the Construction Notes

The construction notes on the drawings have been reviewed by this section. Based on our review, it should be noted that the detail procedures for the rock bolts, steel mesh and shotcrete be included in the construction notes. We have no further comments on the drawings.

We believe that this memorandum meets with your present requirements. If you have any further questions, please contact this office.

Tae C. Kim

Tae C. Kim, P. Eng.
Foundation Design Engineer

for

P. Payer, P. Eng.
Sr. Foundation Engineer

PP/TCK/jb

cc: K. Wong - Central Regional Structural Section
K. Price - Delcan

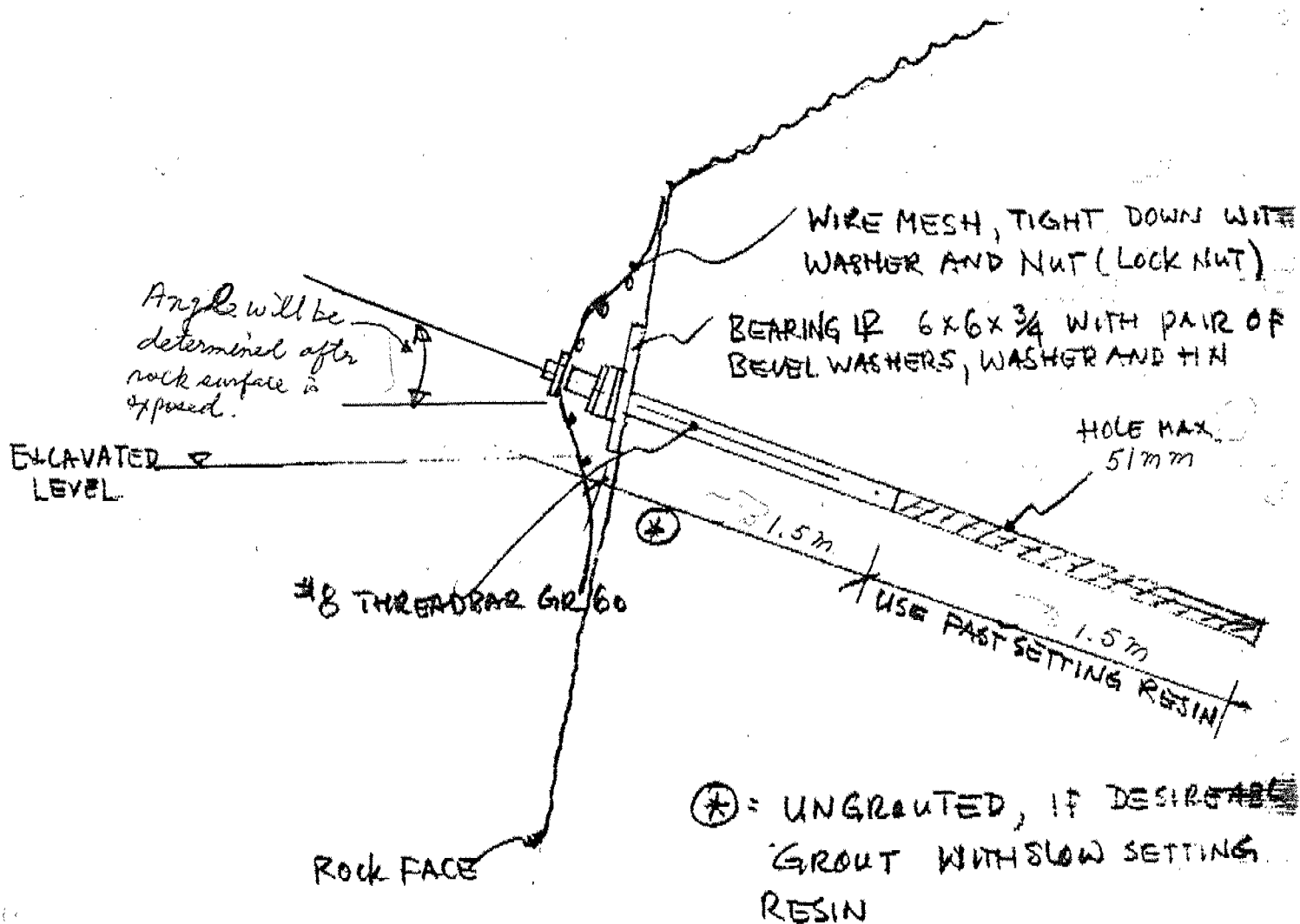


FIGURE 1. TYPICAL RESIN ANCHOR BOLT

DELSCAN

ENGINEERS
PLANNERS
ARCHITECTS

February 6, 1990

Our Ref: 01-2800-A01

Mr. Murty Devata
Head: Foundation Design Section
Minsitry of Transportation, Ontario
1201 Wilson Avenue
Central Building, Room 315
Downsview, Ontario
M3M 1J8

Attention: Mr. P. Payer, P.Eng.
Senior Foundation Engineer

Dear Sirs:

Re: Highway 403 Retaining Wall
WP.171-87-01

We include a number of retaining wall cross sections as discussed at our earlier meeting, for your review with regard to overall stability of the rock mass and the potential for slip circle failure.

We appreciate your taking the time to address this matter.

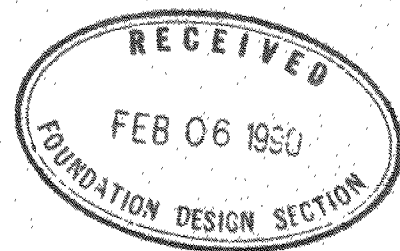
Yours very truly,



Ken D. Price, P.Eng.
Section Head: Bridges

KDP:do
Encls:

c.c. R. Barsalou - MTO Structures
W. V. Anderson
R. Tam



DELCAN CORPORATION

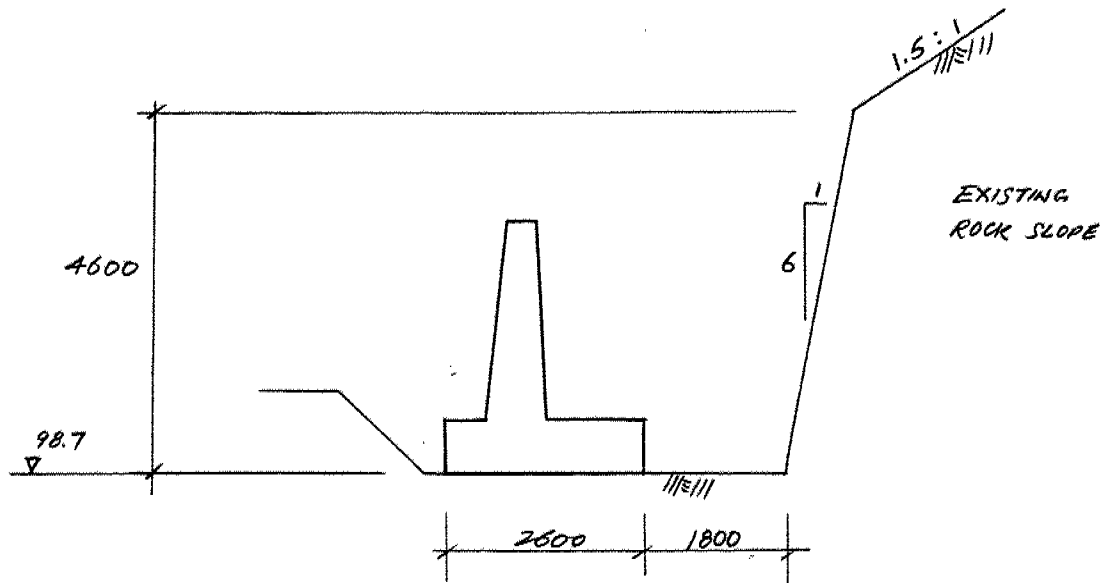
133 WYNFORD DRIVE, NORTH YORK, METROPOLITAN TORONTO CANADA, M3C 1K1 • (416) 441-4111
TELEX 06-8686-89 • FAX: (416) 441-4131

ST. JOHN'S, MONTREAL, OTTAWA, HAMILTON, NIAGARA FALLS, LONDON, THUNDER BAY, WINNIPEG, REGINA, SASKATOON,
CALGARY, VICTORIA, VANCOUVER.

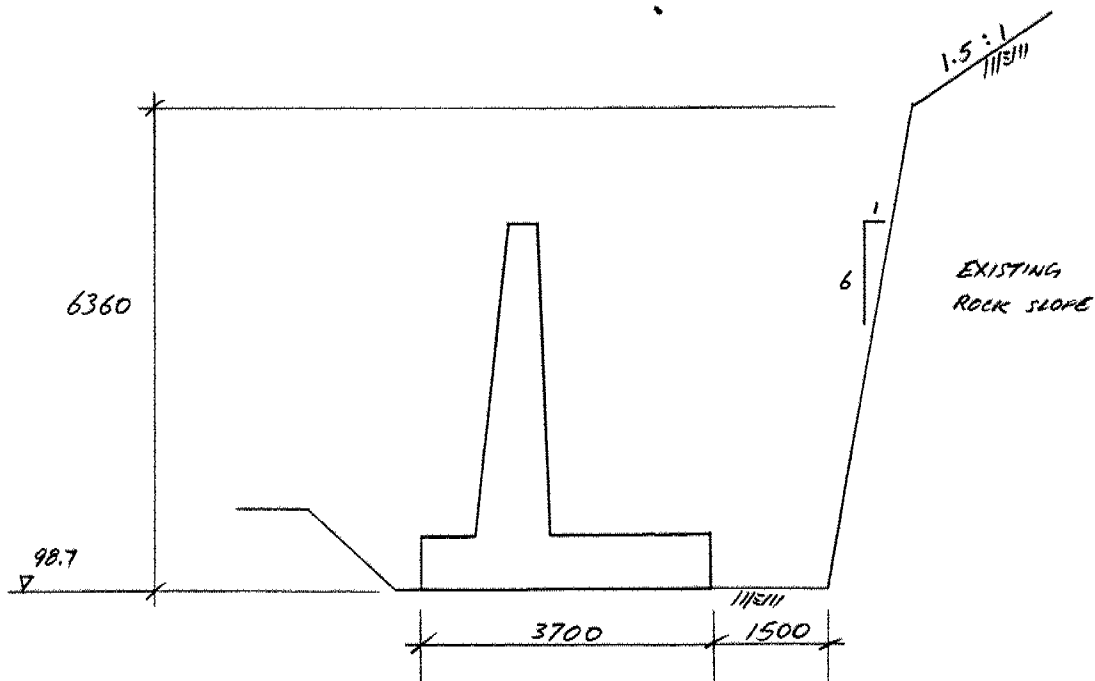
DELCAN

SUBJECT HWY 403 / HWY 6 JOB NO. _____
RETAINING WALL SHEET NO. _____ OF _____
MADE BY R.T. DATE _____ CHECKED BY _____ DATE _____

TYPICAL SECTIONS FOR SLOPE STABILITY.



TYPE 1 : WIDTH OF OPENED SLOPE = 6m.

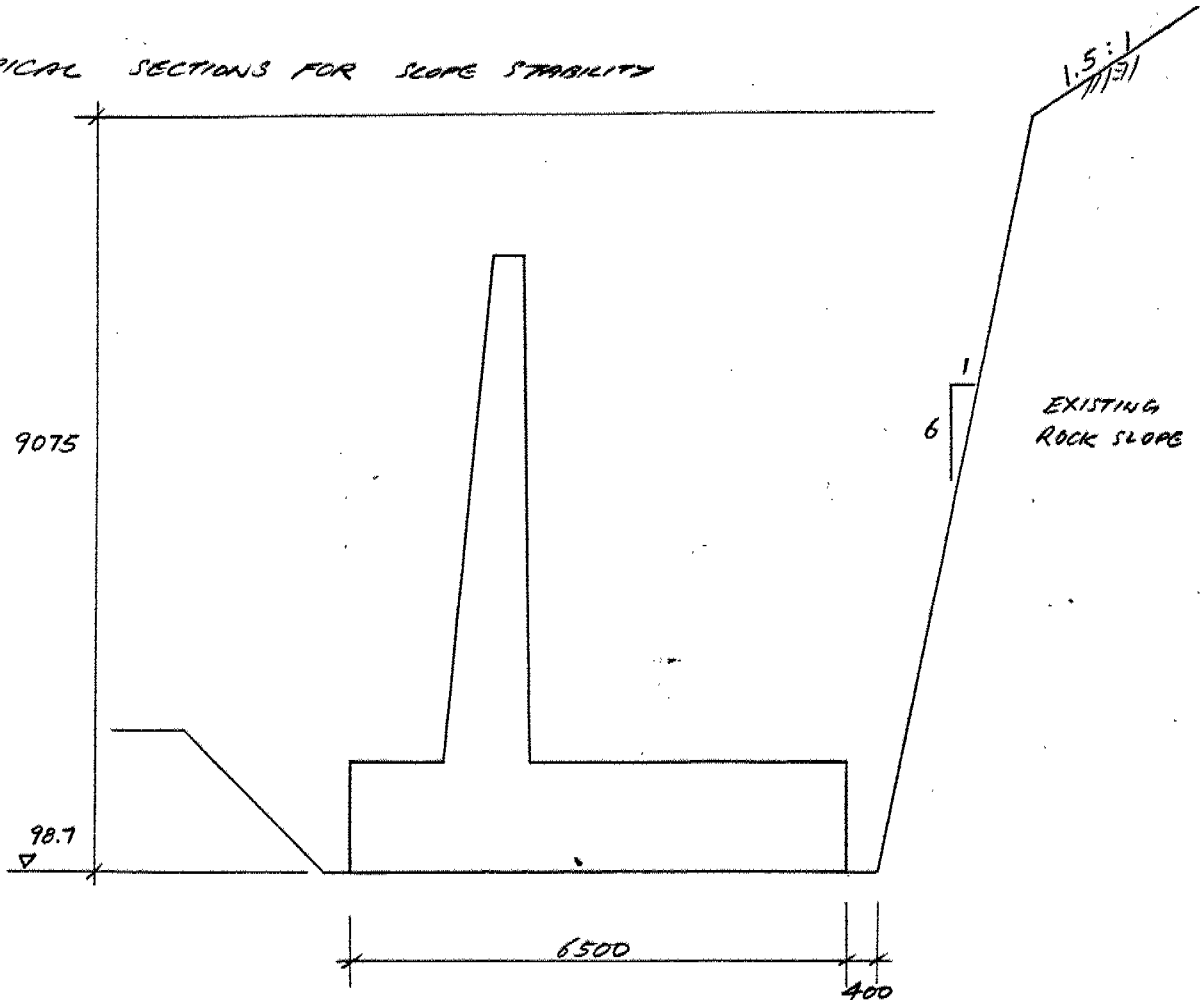


TYPE 2 : WIDTH OF OPENED SLOPE = 6m.

DELCAN

SUBJECT HWY 403 / HWY 6 JOB NO. _____
RETAINING WALL SHEET NO. _____ OF _____
MADE BY R.T. DATE _____ CHECKED BY _____ DATE _____

TYPICAL SECTIONS FOR SLOPE STABILITY



TYPE 3 : WIDTH OF OPENED SLOPE = 6m

memorandum



To: Distribution

Date: 1989 12 01

From: Foundation Design Section
Room 315, Central Building

RE: W.P. 380-85-01
Steel Bin Retaining Wall Rehabilitation
Highway 403 Ramp NW Hamilton

As per your request, some additional fieldwork has been carried out at the above location to determine the bedrock elevation.

The three additional boreholes and their relative bedrock elevations are listed below.

<u>Borehole Number</u>	<u>Sound Bedrock Elevation (m)</u>
4	1.5
5	2.1
6	1.6

Refer to attached figure 1 for their locations, relative to the boreholes completed earlier. The additional boreholes are also attached.

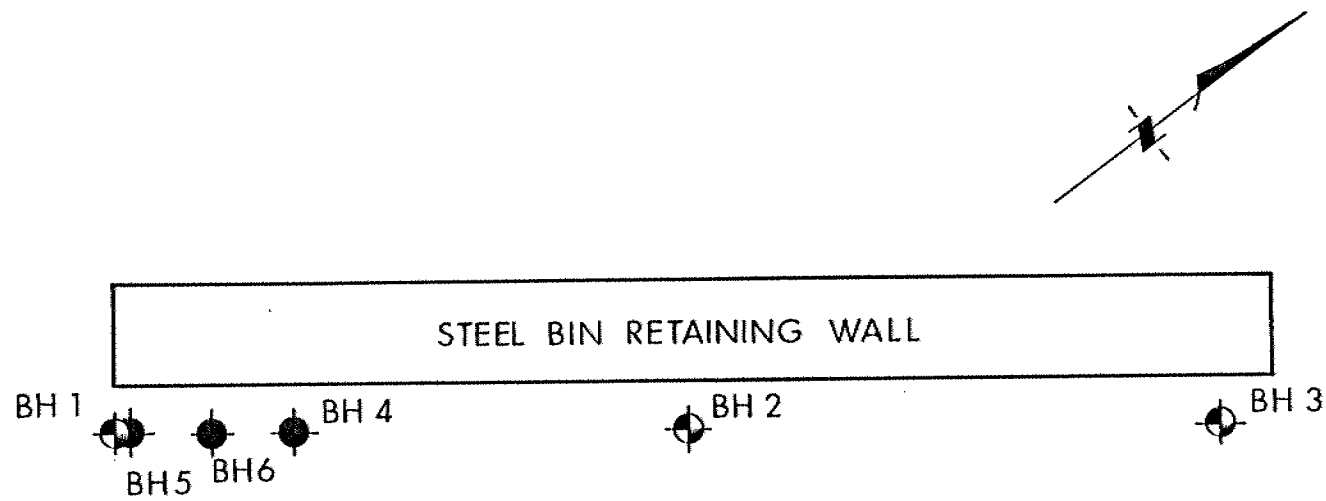
Please add this letter and attachments to our original report.

A handwritten signature in cursive script, appearing to read "P. Marks".

P. Marks, P. Eng.
Foundation Engineer

PM/jb
Attach.

cc: G.C.E. Burkhardt (3)
G. Cautillo
J. Smrcka (2)
A. Wittenberg
K. Bassi
S.J. Dunham
G. Szekreny
File ✓



PLAN
NTS

Geocres No 30M5 -159

WP 380-85-01

Dist 4

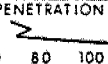




Figure No 1

RECORD OF BOREHOLE No 4

METRIC

W P 380-85-01 LOCATION Ramp N-W 12m East of West Corner Retaining Wall ORIGINATED BY PM
DIST 4 HWY 403 BOREHOLE TYPE NW Casing, BXL Rock Core COMPILED BY PM
DATUM Assumed DATE 89 11 15 to 89 11 16 CHECKED BY _____

OFFICE REPORT ON SOIL EXPLORATION

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
99.9	Shoulder Surface																
0.0	Asphalt																
0.1																	
99.2	Sand and Gravel (Fill)		1	CS													
0.7	Sand & Gravel & Clayey Silt (Weathered Shale) Transition Zone		5	SS	71/	5cm	99										
98.4	Very Dense		7	RC BXL													
1.5			8	RC BXL			98										
	Shale Bedrock		9	RC BXL			97										
96.1			10	RC BXL													
3.8	End of Borehole																
	* Water Level not Established																

+³, x⁵: Numbers refer to
Sensitivity

20
15 \diamond 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 5

METRIC

W P 380-85-01 LOCATION Ramp N-W 1m East of West Corner Retaining Wall
 DIST 4 HWY 403 BOREHOLE TYPE NW Casing, BXL Rock Core
 DATUM Assumed DATE 89 11 17

ORIGINATED BY PM
 COMPILED BY PM
 CHECKED BY

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
99.9	Shoulder Surface																
0.0	Asphalt																
0.1	Sand and Gravel																
99.1	(Fill)																
0.8	Sand and Gravel and Clayey Silt (Weathered Shale) Compact to Very Dense		1	SS	19	*	99										
97.8			2	SS	81		98										
2.1			3	SS	120/												
			4	SS	60/												
	Shale Bedrock		5	RC BXL			97										
96.7																	
3.2	End of Borehole																
	* Water Level not Established																

OFFICE REPORT ON SOIL EXPLORATION

+3, x5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10



RECORD OF BOREHOLE No 6

METRIC

W P 380-85-01 LOCATION Ramp N-W 6.5m East of West Corner Retaining Wall
DIST 4 HWY 403 BOREHOLE TYPE NW Casing, BXL Rock Core
DATUM Assumed DATE 89 11 17
ORIGINATED BY PM
COMPILED BY PM
CHECKED BY

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100					
99.9	Shoulder Level															
99.8	Asphalt															
0.1	Sand and Gravel (Fill)					*										
99.2																
0.7	Sand & Gravel, Clayey Silt, Weathered Shale		1	SS	69/23cm	99										
	Transition Zone		2	SS	100/9cm											
98.3	Very Dense															
1.6	Sound Shale Bedrock		3	RC BXL		98										
97.9																
2.0	End of Borehole															
	* Water Level not Established															

OFFICE REPORT ON SOIL EXPLORATION

S-HOLMES

Structural Section
Central Region
Atrium Tower
1201 Wilson Ave.
Downsview, Ontario
M3M 1J8

Telephone: 235-5511

1989-10-19

Mr. K.D. Price
DELCAN
133 Wynford Drive
North York, Ontario
M3C 1K1

Dear Mr. Price:

Re: Addendum to Structural Design Report
Rehabilitation of Various Structures
W.P. 380-85-01, Hwy 403/Hwy 6 Retaining Wall
W.P. 171-81-01, Hwy 403 - Plains Road Underpass
District 4, Burlington

Further to our telephone conversation of 89-09-19, we wish to initiate a change in the design in the above captioned retaining wall.


Representatives of both the Structural and Foundation Design Sections met on 89-09-18 to discuss the design as developed by DelCan consisting of tied back pilasters with in-fill panels. As you are aware, several concerns about this design had been expressed by the Foundation Design Section with respect to anchorage capacity, proof and production testing of tie-backs, and drilling through existing steel bin wall components. Refined cost estimates of the project, developed from the submitted D4, were also evaluated and it was observed that the estimated costs of this design had escalated well above the original estimates. With these issues under consideration, the use of an alternate design was discussed. The Foundation Design Section reiterated their earlier recommendations that staged removal of the existing steel bin retaining wall would be possible, permitting the reconstruction of the wall with a more conventional system. Cost estimates for conventional wall systems were reviewed, and the logistics of construction staging discussed. It was then concluded in the meeting that the existing design would be shelved and the consultant requested to initiate a redesign with a standard reinforced concrete cantilever retaining wall.

We would therefore request that DelCan review this and the attached letter from the Foundation Design Section, and submit a cost estimate for the redesign of the wall with a cantilever concrete retaining wall. This estimate should include all charges to carry out the redesign, re-initiate the review process including Technical Review, and costs to assemble and coordinate the complete contract package. Costs for any associated redesign of the grading component of the work shall not be included, but should be addressed directly from your design section to the Ministry's Planning and Design representative.



We trust you will find this satisfactory. Please do not hesitate to contact the undersigned for addition information or clarification on the above.

Yours truly,



Robert R. Barsalou
Senior Structural Engineer
For:
G.C.E. Burkhardt
Head, Structural Section

RRB

cc

K. Pilgrim
D. Gray
S. Holmes
G. Greene
K. Bassi
R. Reel

memorandum



To: G.C.E. Burkhardt
Head
4th Floor, Atrium Tower

Date: 1989 10 11

From: Foundation Design Section
Room 315, Central Building

RE: W.P. 38-85-01
Hwy. 403/Hwy. 6
Retaining Wall
District 4, Burlington

In response to our meeting of 89 09 18, we are making the following recommendations for the design of a standard reinforced concrete cantilevered retaining wall.

It is believed that the existing bin wall is directly in front of a shale bedrock face previously excavated for the cut and that the portion above the bin wall is fill placed with a 1- $\frac{1}{2}$:1 slope after filling the bin. Also it is believed that the bin has been placed directly on the bedrock which is located about one metre below the shoulder of the road. Therefore behind the binwall there should be a competent shale bedrock face. It should then be possible to remove sections of the existing bin wall to construct a new cantilevered concrete wall. We are providing at this time recommendations for the design of the concrete cantilevered retaining wall and copies of the Borehole Logs.

Bearing Capacity

The retaining wall may be supported on unweathered shale bedrock providing there is adequate frost protection. At station 10 + 000 the unweathered rock elevation is 99.0 m. At station 10 + 035.6 the unweathered rock elevation is 98.5 m. At Station 10 + 073.6 the unweathered rock elevation is 96.9 m. The rock elevation between stations 10 + 035.6 to 10 + 073.6 will be investigated at a later date. To provide an even base for the footing mass concrete can be placed directly on top of the shale bedrock where required.

For purposes of the O.H.B.D.C. the following values are recommended:

Factored Bearing Capacity at U.L.S.: 1500 kPa

Bearing Capacity at S.L.S. Type II: Will not govern design since the loads required to produce detrimental settlement of the structures will be much larger than the recommended values for factored bearing capacity at U.L.S.

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Traffic Protection

If excavation for the footing is deep enough to require traffic protection this office can provide bearing capacities for placing the footing on weathered shale. Placing the footing on weathered shale could reduce the required depth of excavation.

Backfill

Backfill to the retaining wall should consist of granular A in accordance with MTO SP 109F03. The backfill should be placed as steep as possible to fill the annular space between the exposed shale face and the retaining wall.

The coefficient of lateral pressure to be used for design is 0.5.

The lateral pressure should be adjusted to reflect the surcharge created by the backslope.

Sliding Resistance

The horizontal thrust on the retaining wall footing can be resisted by friction along the footing base. A friction angle of 25° may be assumed for resistance against sliding between the concrete footing and the shale bedrock. The excavation within the bedrock for the footing should be carried out by techniques other than blasting because it is important that the footing be placed on sound shale. Therefore construction techniques which disturb the shale can not be used. In addition passive resistance can be provided by dowels. The dowels should extend a minimum of 1.0 m into the shale. The shale should provide a resisting pressure of 0.5 MPa against lateral forces for a 1.0 m dowel. If the dowel extends more than 1.0 m into the shale the shale should provide a resisting pressure of 1.0 MPa for that portion of the dowel which is longer than 1.0 m.

The exposed rock surface at the footing level should be covered with a 150 mm thick mass concrete pad within 8 hours of exposure.

Frost Protection

Frost protection is not normally required for footings on bedrock. However, shale is susceptible to frost action, therefore a minimum earth cover of 1.2 m is recommended.

Removal of the Existing Bin Wall

The existing bin wall must be removed in sections to keep the unsupported open face length to a minimum. When a section is removed it is important that the bin adjacent to the bin being removed remains wholly intact. Therefore the sidewall stretcher of the next bin must be left in place. The exposed shale bedrock face should be immediately sprayed with 25 mm of gunite. The new cantilevered concrete retaining wall should then be erected. When the concrete has achieved sufficient strength the backfill should be placed behind the wall at as steep an angle as possible and the adjacent bin can then be removed. In order to maximize productivity two removal schemes may be considered.

Removal from West to East

At the west end of the wall between Sta. 10 + 074 to 10 + 062 the bin wall can be removed two bins at a time. From Sta. 10 + 062 to Sta. 10 + 006 the bins must be removed one at a time. From Sta. 10 + 006 to Sta. 10 + 000 the last two bins can be removed together.

Simultaneous Removal from East & West

To increase productivity, the existing bins can be removed from both ends of the wall. At the west end of the wall between Sta. 10 + 074 to 10 + 062 the retaining wall can be removed 2 bins at a time. At the east end of the wall the two bins between Sta. 10 + 000 and 10 + 006 can be removed together. Working simultaneously from both ends of the wall the remaining bins can be removed one bin at a time. When there are only 3 sections of bin wall remaining, a wing wall must be constructed flush to the shale face such that it can retain fill for the full height of the new retaining wall. The wing wall should be erected only on one side of the remaining bins (i.e. at bin 11 or bin 15). If the wing wall is constructed at bin 11 (Sta. 10 + 040), then west of the wing wall, the newly constructed cantilevered concrete retaining wall should be completely backfilled. The remaining 3 bins could then be removed one bin at a time starting with bin 14.

It should be noted that when the shale face is exposed during construction it may be possible to increase the number of bins removed simultaneously. This could only be determined during construction and would depend on the condition of the exposed shale face and the ability to retain the overburden on the backslope above the excavation.

Protection from falling debris should be provided by a scheme such as a wire mesh fixed at the top of the slope with rock bolts and fixed at the toe of the slope. A method of catching falling debris from the backslope should be provided such as wire mesh. A copy of the protection scheme with an Engineer's stamp should be provided by the Contractor and shown to this office and to the Structural Office at least 10 days prior to commencing removal of the bin wall.

Steph Holmes

S. Holmes, P. Eng.
Foundation Engineer
for

P. Payer, P. Eng.
Sr. Foundation Engineer

SH/PP/mmj

Attach.

memorandum

Tel: 3731



To: G.C.E. Burkhardt
Head
Structural Section
4th Floor, Atrium Tower

Date: 1989 08 30

From: Foundation Design Section
Room 315, Central Building

RE: W.P. 380-85-01
Hwy. 403/Hwy. 6
Retaining Wall
District 4, Burlington

This office is responding to your request for further information concerning the design of the retaining wall. As discussed at our June 27, 1989 meeting the present design has not followed several of the foundation recommendations. The key recommendation which was not followed was to incline the anchors at 45° from the horizontal. The anchors were instead designed at 30° due to structural considerations. In view of this change in anchor design and following site visits with representatives from Dywidag anchors and Birmingham Construction we are making the following revised recommendations.

Proof Testing

To check that the design bond stress of 500 kPa can be achieved at a 30° angle, two test anchors are required prior to the installation of the production anchors.

The anchors can be tested against the overburden immediately east and west of the existing retaining wall. It is estimated that the tests will cost \$10,000 each. These anchors will not be used for the new wall. The anchors should be loaded to 2T where T is the working load of the anchors.

The test anchors can be included in the Contract package or they can be done as a separate Contract. If the testing is done on a separate Contract staging and traffic protection will be a concern. If the test anchors are part of the main Contract there must be two weeks between the completion of the test anchors and the installation of the production anchors. This will allow time for the anchor bond lengths to be adjusted if required. Attached to this memo is a revised NSSP for soil and rock anchor installation and testing.

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Anchor Installation

Anchor installation will be difficult for the following reasons:

- 1) The upper row of anchors will require a platform to raise most drills to the required height.
- 2) The upper row of anchors must be drilled through the rear wall stretcher of the existing bin wall. This will be a two stage operation. The steel wall stretcher will need to be cored with a diamond bit then the bit will be withdrawn and normal casing will be used to continue the anchor installation.
- 3) The equipment required to install the anchors will probably be a small rotary percussion drill with a fixed boom length. A small drill will require 7.0 m perpendicular to the wall to operate. The staging should reflect this.

To reduce unanticipated costs the drawings should detail the bin type retaining wall showing the locations of bolts, posts, thickness of the steel plate (2.5 mm) and the depth of corrugations. The drilling costs will probably reflect the difficulties and unknowns caused by drilling through the rear wall stretcher.

To facilitate anchor installation there are several possible alternatives which could be considered:

- 1) Raise the top row of anchors above the top of the existing bin wall such that the rear wall stretcher is not drilled through the stretcher or is drilled at a shallow depth and the panel could be exposed by excavation.
- 2) Space the anchors to miss the existing bolts and posts.
- 3) Create a separate item for drilling through the bin wall and expect a higher than average drilling cost.

The rock line behind the existing bin wall is uncertain but is thought that the existing bin wall was founded on rock 1.0 m below the top of shoulder. The free stressing zone should extend to the top of rock and the elevation should be shown as a reference. The free stressing zone should be filled with bentonite grout.

The estimated cost to install the 600 metres of anchors under the present design could be as high as \$4,000 per anchor due to the difficult conditions, in particular drilling through the bin wall.

Acceptance Tests (Production Anchors)

The production anchors should be tested against the existing bin wall and then extended to the new wall. Couplings are available for Dywidage Threadbar anchors but the bars can only carry 743 kN. The anchor load should be distributed against the existing wall. The anchors should be loaded to 130% of their design load (1.3T).

Backfill and Anchor Stressing

Under the current design backfill to the existing bin wall will be difficult for the following reasons:

- 1) The corrugations of the front wall stretcher will be difficult to fill and compact.
- 2) The existing bin wall structure contains loosely compacted fill with voids visible behind the front wall stretcher.
- 3) Overcompaction may deform the new wall.

To facilitate proper compaction it is recommended that the front wall stretcher be removed in sections as the new wall is erected. The panels should be removed starting with the bottom wall stretchers. As the backfill is placed and compacted the anchors should be continually checked and snugged to prevent excess lateral deflection of the new wall. The new wall should be as flexible as possible to allow for deformations. The wall will require constant monitoring to prevent excess deflection as the anchors are stressed in conjunction with placing the backfill.

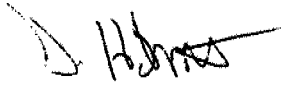
The stress on Dywidag threadbar anchors can be more readily adjusted than multistrand anchors because the bars are threaded. The Threadbar Anchors can only carry 743 kN so to allow for easier adjustments the anchor loads would have to be reduced to 743 kN.

Drainage

Run off from the top of the slope must be prevented from eroding the exposed face when the front panels of the bin wall are removed.

Other Alternatives

It is believed that the existing bin wall and overlying fill could be removed back to the rockface which prevailed at the time of the original construction of the wall and that the resulting configuration would remain stable until completion of a new wall provided this is done without delay.


S. Holmes, P. Eng.
Foundation Engineer

for

P. Payer, P. Eng.
Sr. Foundation Engineer

SH/PP/mmj

c.c. - K.G. Bassi

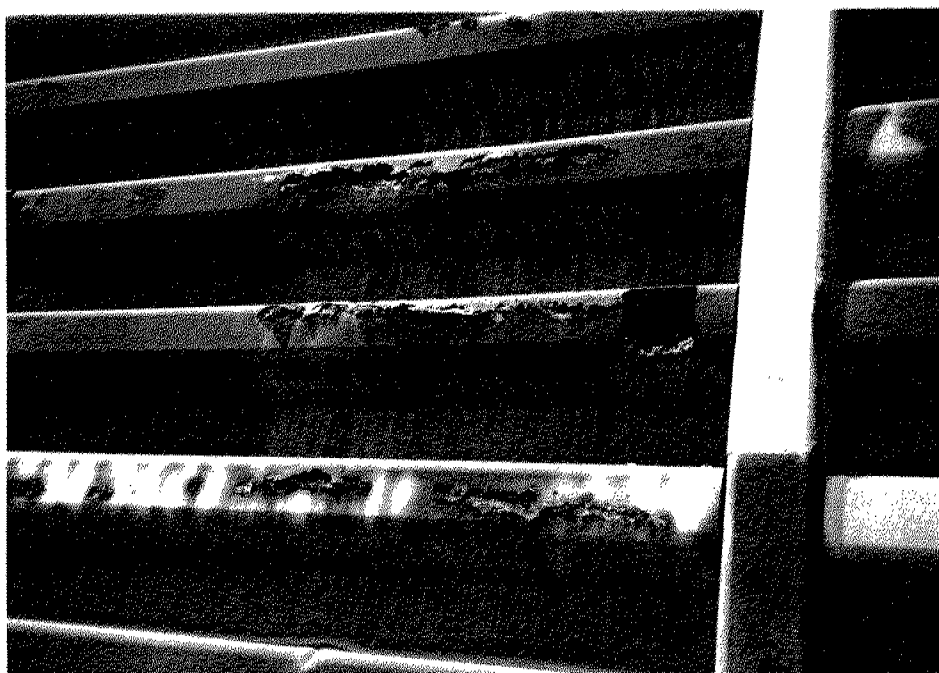
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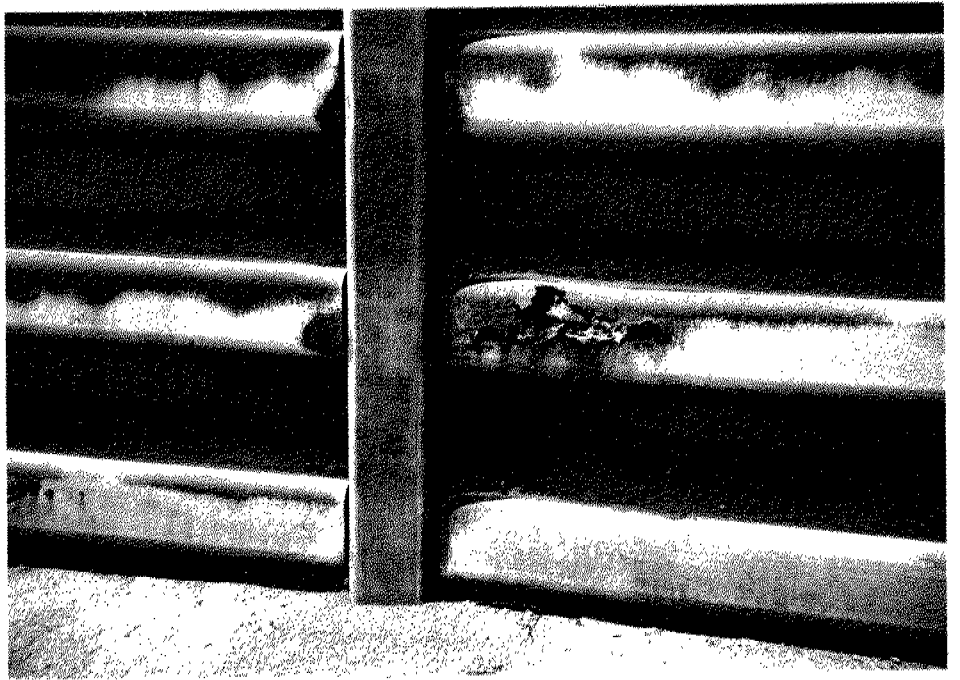
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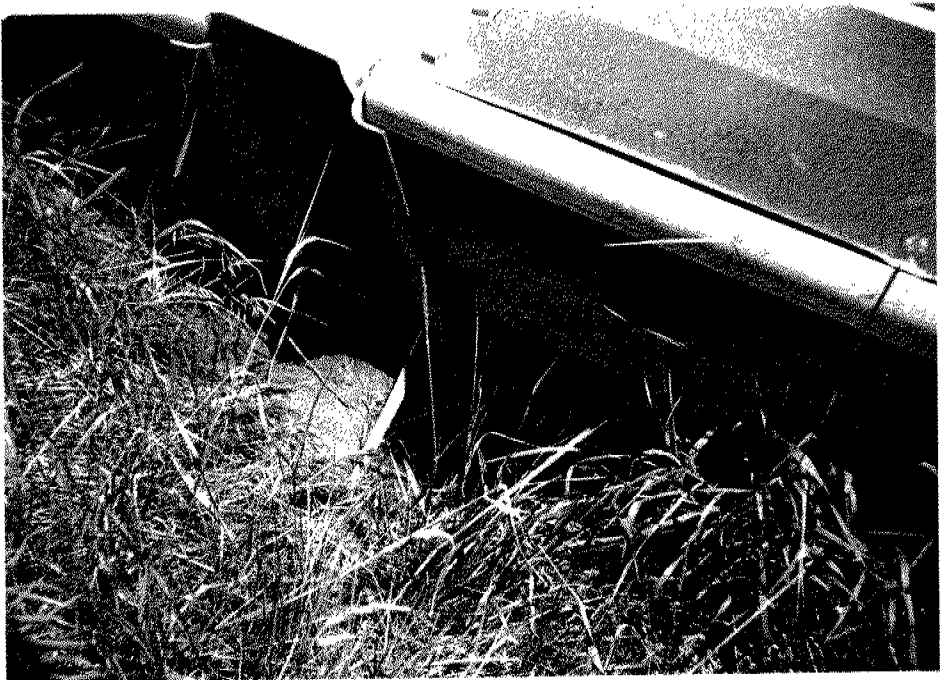
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HWY 403 / HWY 6

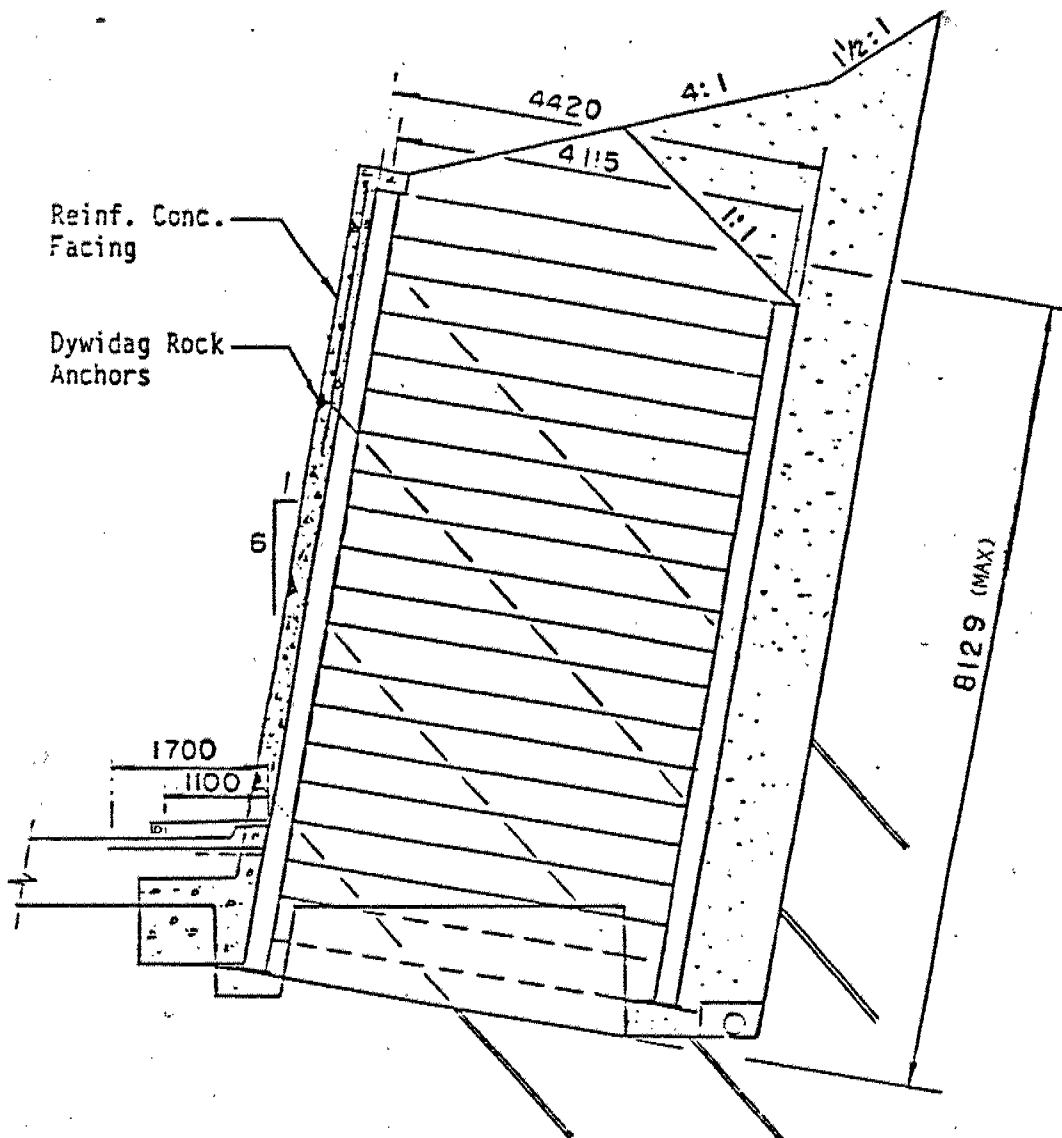
BIN TYPE RETAINING WALL







HWY 6 RETAINING WALL



STEEL BIN TYPE RETAINING WALL

memorandum

235-3731 EXT. 7096



To: Mr. Alfred Ho
Senior Structural Engineer
Central Region

Date: 1987 04 28

From: Foundation Design Section
Room 315, Central Building
Downsview

Re: Hwy. 403/Hwy. 6 Retaining Wall Rehabilitation
District #4, Burlington

This is in reply to your memo of 87 01 20 regarding your proposed rehabilitation of the bin retaining wall on Hwy. 403 near the intersection with Hwy. 6. We have reviewed Drawings 1 and 2 of the bin wall provided by you and we have inspected the site in the field but unfortunately we have not been able to obtain any contract drawings or cross sections showing the original conditions which prevailed at the start of construction. However we have had discussions with G. Disher of C.C. Parker and Associates (consultants who designed the wall) and from these discussions and from our own observations it seems likely that the binwall is directly in front of a shale bedrock face previously excavated for the cut and that the portion above the binwall is fill placed with a 1-1/2:1 slope after filling the bin. Also it is believed that the bin has been placed directly on the bedrock which is located about one metre below the shoulder of the road. The condition of the binwall in the opinion of the writer is not really as bad as it appears. The slight distortions in the metal posts are probably due to the fact that a suitable compressible layer was not placed under the base plates of the posts and the corroding of the wall stretchers is the only real structural problem. However we would suggest that these wall stretchers could easily be replaced by new ones, one at a time. If you consider this to be inadequate or impractical we would suggest the following alternatives:

- 1) Rebuild the binwall completely using either galvanized steel or precast concrete units.
- 2) Build a new conventional reinforced concrete wall.

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A handwritten signature in dark ink, appearing to be "ACB", written over a horizontal line.

- 3) Face the existing binwall with a reinforced concrete slab anchored into the rock face behind the binwall. Based on pulling tests previously carried out in the Queenston Shale in this area anchors inclined at 45 degrees to the vertical may be designed assuming an ultimate bond capacity of 600 kPa. The recommended minimum safety factor is 2.00.

It is believed that the existing wall and overlying fill could be removed back to the rockface which prevailed at the time of the original construction of the wall and that the resulting configuration would remain stable until completion of a new wall provided this is done without delay.

Please consider the various options we have described and advise us whether we can be of further help in this matter.



K. G. Selby, P. Eng.
Chief Foundation Engineer (West)

KGS/nd