

DOCUMENT MICROFILMING IDENTIFICATION

G.I.-30 SEPT. 1976

GEOCRES No. 42A-47

DIST. 53 REGION

W.P. No. 316-85-03

CONT. No.

W. O. No.

STR. SITE No. 39E-85

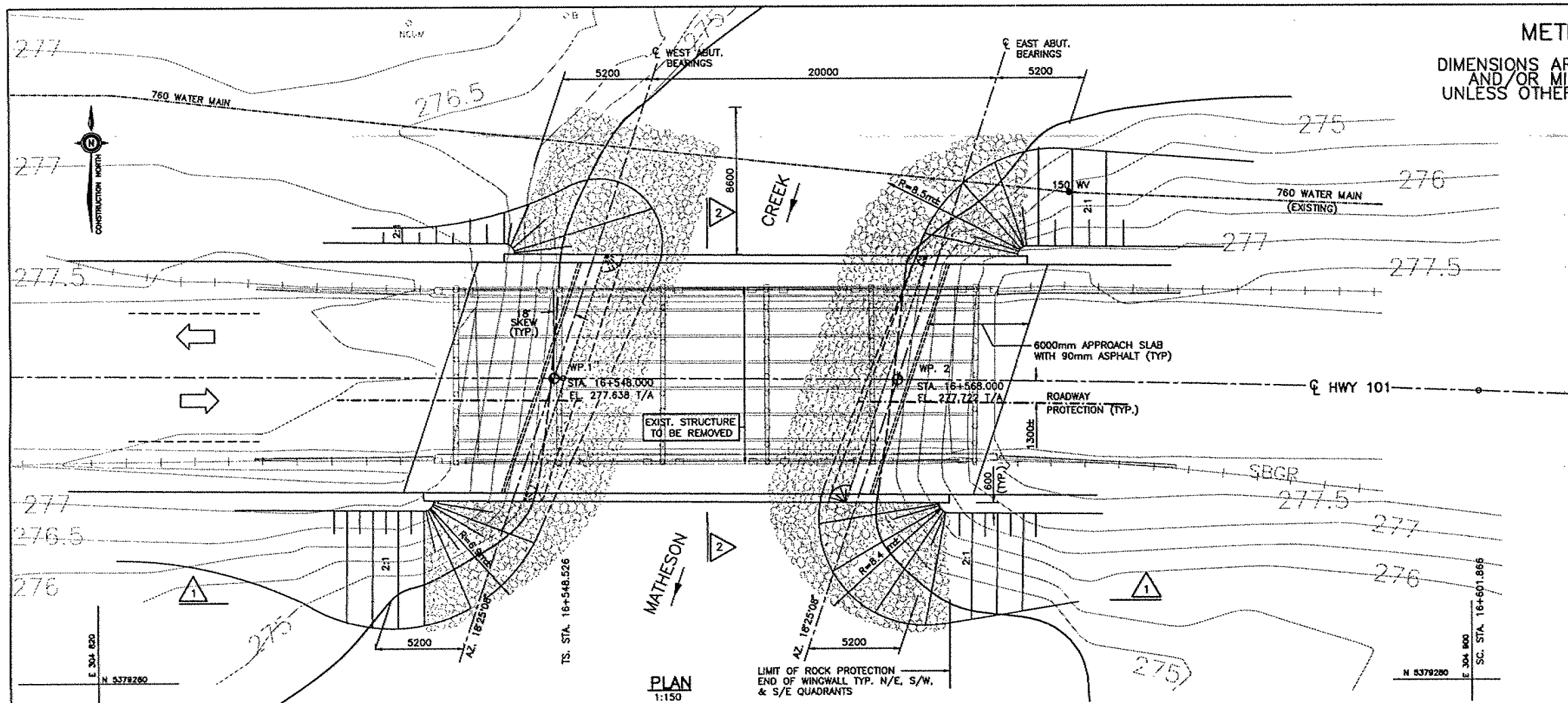
HWY. No. 101

LOCATION HWY 101 & MATHESON CREEK

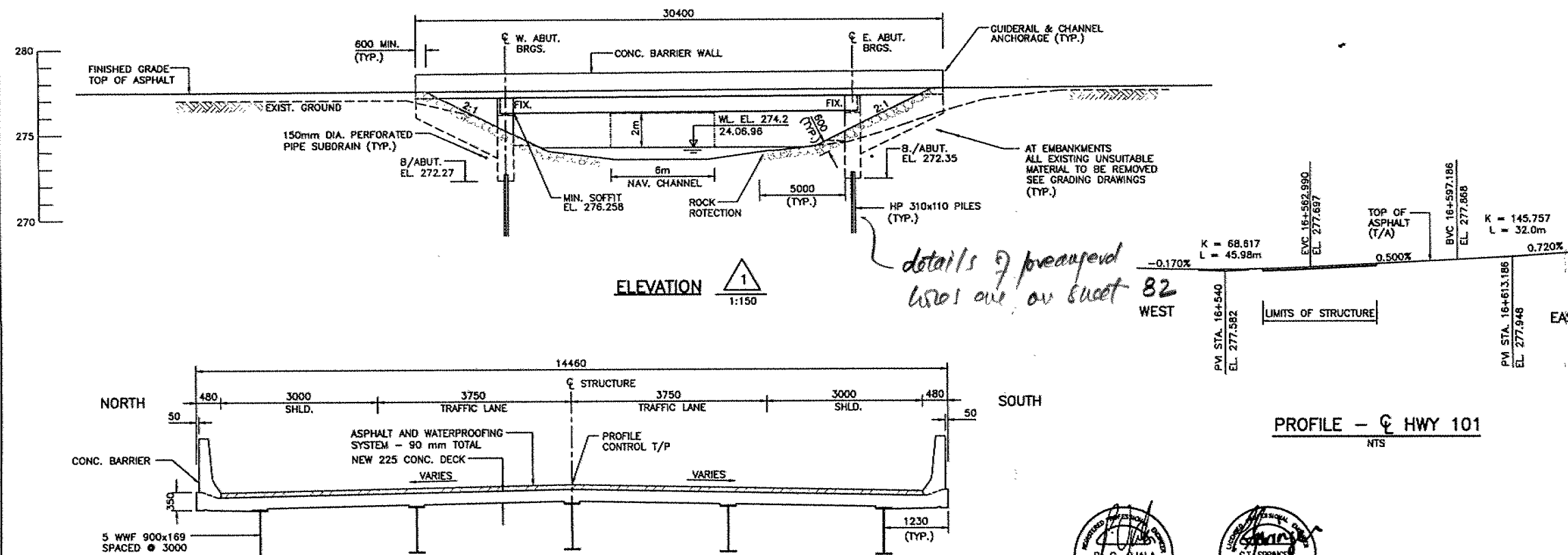
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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:



PLAN  
1:150



ELEVATION  
1:150

PROFILE -  $\bar{C}$  HWY 101  
NTS

BM 303  
EL. 279.707  
HCP 0760293  
CONCRETE MONUMENT WITH BRASS CAP  
LOCATED ON N. ROW, 170m  $\pm$  EAST OF  
THE EASTERN EXIST. BRIDGE ABUTMENT

2  
1:50

DRAWING NOT TO BE SCALED  
100 mm ON ORIGINAL DRAWING

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

DIST No 53  
CONT No  
WP No 316-85-03  
HWY 101-BRIDGE REPLACEMENT  
MATHESON CREEK BRIDGE  
GENERAL ARRANGEMENT

SHEET  
77

DS-Lea Associates Ltd.  
Consulting Engineers & Planners  
Toronto • Winnipeg • St. John's • Vancouver • London • Ottawa

GENERAL NOTES:

CLASS OF CONCRETE ..... 30 MPa

CLEAR COVER TO REINFORCING

DECK TOP ..... 70  $\pm$  20  
BOT. .... 40  $\pm$  10

REMAINDER (UNLESS NOTED OTHERWISE) ..... 70  $\pm$  20

REINFORCING STEEL

REINFORCING STEEL SHALL BE GRADE 400 UNLESS NOTED OTHERWISE  
BAR MARKS WITH SUFFIX "C" DENOTE COATED BARS

UNLESS SHOWN OTHERWISE, TENSION LAP LENGTHS NOT INDICATED ON THE CONTRACT DRAWINGS SHALL BE CLASS B.

BAR HOOKS SHALL BE MINIMUM LENGTH AND STIRRUPS SHALL HAVE MINIMUM HOOKS, UNLESS INDICATED OTHERWISE.

CONSTRUCTION NOTES

THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESSES FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT.

BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH CONCRETE ABUTMENTS KEEPING THE HEIGHT OF THE BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN ELEVATION BE GREATER THAN 500mm.

LEGEND:

- T/A - DENOTES TOP OF ASPHALT
- WP. - DENOTES WORKING POINT
- B./ABUT. - DENOTES BOTTOM OF ABUTMENT

LIST OF DRAWINGS:

- GENERAL ARRANGEMENT
- BOREHOLE LOCATION AND SOIL STRATA
- STAGING DETAILS
- ROADWAY PROTECTION I
- ROADWAY PROTECTION II
- FOUNDATION LAYOUT
- ABUTMENTS - I
- ABUTMENTS - II
- WINGWALLS
- STRUCTURAL STEEL - I
- STRUCTURAL STEEL - II
- DECK DETAILS
- DECK REINFORCING
- BARRIER WALL W/O RAILING
- 6000 mm APPROACH SLAB
- STANDARD DETAILS
- QUANTITIES - STRUCTURE I
- QUANTITIES - STRUCTURE II

APPLICABLE STANDARD DRAWINGS:

- OPSD - 4010.00 GUIDE RAIL AND CHANNEL ANCHORAGE
- OPSD - 4601.000 LOCATION OF SITE NUMBER & DATE FIGURES

DATE	BY	DESCRIPTION
DESIGN CTS	CHK PO	CODE 0480C 91
DRAWN ADG	CHK CTS	SITE 39E-85
		STRUCT
		SCHEME
		DWG 01

*Received in April 1998 with the Technical Review package*





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

DIST No 53  
CONT No  
WP No 316-85-03  
HWY 101-BRIDGE REPLACEMENT  
MATHESON CREEK BRIDGE  
FOUNDATION LAYOUT



DS-Lea Associates Ltd.  
Consulting Engineers - Planners  
Vancouver - Winnipeg - Toronto - Ottawa - London - Montreal

NOTES:

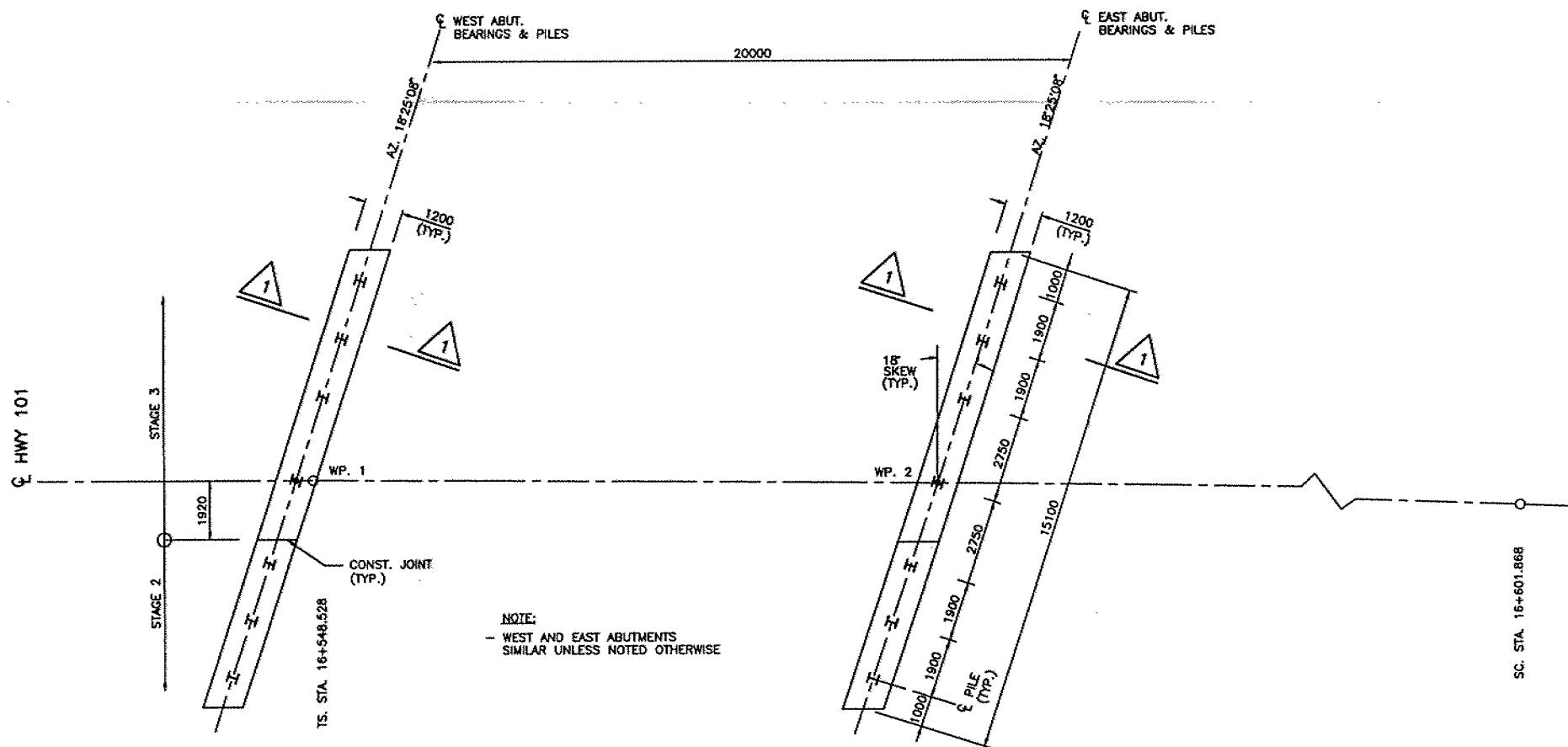
- 600mm \* C.S.P. DENOTES GALVANIZED CORRUGATED STEEL PIPE WITH SQUARE ENDS 1.3mm METAL THICKNESS, AS LISTED IN D.S.M. 4.60.80 TYPE II HELICAL.
- ALL PILES ARE HP 310x110 STEEL 'H' PILES WITH DRIVING SHOES.
- SPACING OF PILES TO BE MEASURED AT UNDERSIDE OF FOOTING
- PILE LENGTH SHOWN IS THEORETICAL LENGTH BELOW CUT-OFF ELEVATION.
- ALL PILES TO BE DRIVEN TO IN ACCORDANCE WITH STANDARD SS103-11 USING ULTIMATE CAPACITY OF 3450 kN/PILE.
- PILES SHOULD BE DRIVEN WITH HAMMER CAPABLE OF DELIVERING AN ENERGY OF AT LEAST 52000 JOULES/BLOW.

PILE DESIGN DATA:

MAX. COMBINED FACTORED LOADS:  
SLS = - N/A  
ULS = 1450 kN/PILE

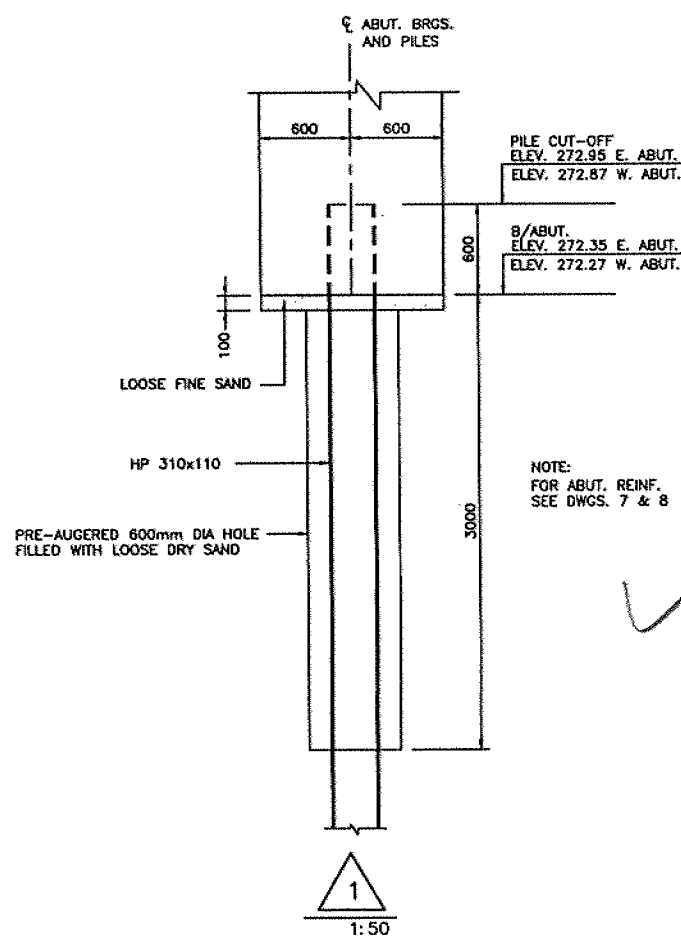
W. P. COORDINATES				
W.P. No.	LOCATION	STATION	COORDINATES	
1	W. ABUT.	STA. 16+548.000	N 5379297.892	E 304848.620
2	E. ABUT.	STA. 16+568.000	N 5379297.746	E 304886.620

LIST OF PILES			
LOCATION	No.	APPROXIMATE LENGTH (mm)	BATTER
W. ABUT.	7	16	-
E. ABUT.	7	15	-



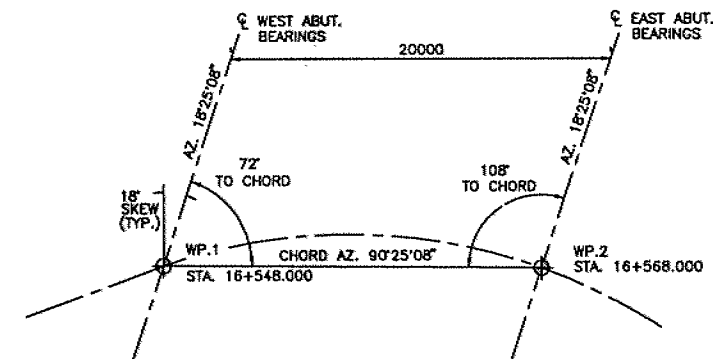
NOTE:  
- WEST AND EAST ABUTMENTS  
SIMILAR UNLESS NOTED OTHERWISE

PLAN  
1:100



NOTE:  
FOR ABUT. REINF.  
SEE DWGS. 7 & 8

1  
1:50



LAYOUT PLAN  
MTS



DRAWING NOT TO BE SCALED  
100 mm ON ORIGINAL DRAWING

APPLICABLE STANDARD DRAWINGS:

- OPSD - 3301.00 SPLICE AND DRIVING SHOE DETAILS FOR STEEL 'H' PILES
- OPSD - 3922.00 SUPPORTS FOR BOTTOM REINFORCING

REVISIONS		DATE	BY	DESCRIPTION
DESIGN	CTS	CHK	PO	CODE CHBDC 91
DRAWN	ADG	CHK	CTS	SITE 39E-85
		LOAD CLASS 'A'		DATE APRIL 1998
		STRUCT		SCHEME
		DWG		06



Ministry  
of  
Transportation

*FILE  
Copy*

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## **FOUNDATION DESIGN SECTION**

# **foundation investigation and design report**

**ENGINEERING MATERIALS OFFICE**  
**FOUNDATION DESIGN SECTION**

WP 316-85-03 DIST 53  
HWY 101 STR SITE 39E-85

Matheson Creek Bridge

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**GEOCRES 42A-47**

**DATE APR 03 1997**

**FOUNDATION INVESTIGATION REPORT**  
**For**  
**Matheson Creek Bridge**  
**W.P. 316-85-03, Site 39E-85**  
**Highway 101, District 53, New Liskeard**

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**INTRODUCTION**

This report summarizes the results of a field investigation that was carried out for the construction of a new bridge structure over Matheson Creek, on Hwy 101. The new bridge will replace the existing bridge.

The investigation was carried out at the request of Northern Region Structural Section. This report applies to proposed bridge structure and its approaches within 20m of the structure (Sta. 16+526 to 16+590).

**SITE DESCRIPTION**

The site is located at the Matheson Creek on Hwy 101, about 19 km east of Timmins and about 33 km west of west junction of Hwy 11 and Hwy 101. The site is located in MTO District 53, New Liskeard, Ontario.

The surrounding area at the site is covered with shrubs and small trees. On the south side of the bridge, the east bank of the creek is swampy. The creek flows from north to south. The creek channel under the bridge is about 10m wide. At the time of the investigation, the water level in the river was at elevation 273.0m. The water depth in the creek was 0.6m.

The existing bridge is a 5-span structure over wooden piers. The piers and abutments are made of eleven 0.3m diameter timber pile bents. The abutments are made of timber pile bents and gabion baskets. The bridge deck is supported on steel I-beams that rest on timber piers.

## **INVESTIGATION PROCEDURES**

The field investigation for this project was conducted between 1996 11 19 and 1996 11 22. The field work for the Foundation Investigation consisted of drilling two boreholes (BH 1 and BH 2) and one cone test (BH3). The boreholes were put down at the northwest and southeast quadrant of the existing creek crossing. The northeast and southwest sides were not accessible with a track mounted drilling machine. Due to a swamp and difficult access to the southeast side of the bridge, BH 2 was drilled at a distance of about 12m from the bridge. However, a dynamic cone test (BH 3) was carried out close to the bridge to confirm that the soil condition at the proposed east abutment was similar to the soil condition encountered in BH 2. Borehole BH 1 was terminated in bedrock. The bedrock was proven by obtaining a 1.3m core. Borehole BH2 was terminated in a very dense sand and gravel deposit. The sand and gravel deposit was not encountered in BH1.

The boreholes were drilled in the ground using a track-mounted auger machine equipped with 82mm ID hollow stem augers and BX size coring equipment. These boreholes were advanced to depths 20.3m and 19.9m respectively.

Soil samples were recovered by means of a 50mm OD split spoon sampler driven into the soil according to the specifications of the Standard Penetration Test (ASTM D 1586). Samples were retrieved at intervals ranging from 0.75m to 1.5m. In cohesive soil, undisturbed samples were also obtained at strategic locations by pushing 54mm ID Shelby tubes. Frequent field vane shear tests were carried out within the cohesive layer in order to determine the undrained and remoulded shear strength of the materials. Once practical refusal to auguring was encountered, BX-size bedrock cores was obtained from Borehole 1 at a depth of 19m. Groundwater was monitored during drilling and after completion of the boreholes.

The Laboratory testing program for representative soil samples consisted of:

- Grain Size Analyses
- Natural Moisture Content
- Atterberg Limit, and
- Consolidation Test

The results of the laboratory tests are plotted on the borehole logs. The bedrock core was logged by D.A. Williams, Petrographer of the Soils and Aggregates Section of MTO.

The boreholes were staked out by the Pavements and Foundations Section. Ground surface elevations and locations at the borehole locations were provided by the Survey and Plan Section of the MTO, Northern Region.

## **SUBSURFACE CONDITIONS**

The Record of Borehole Sheets in the Appendix illustrate the subsurface conditions at the borehole locations. The locations and elevations of the boreholes and cone test, along with stratigraphical profiles based on the borehole data are shown on Drawing No. 3168503-A.

The predominant surficial deposit at the site is a silty clay to clayey silt. The thickness of this deposit ranged from 9.4m to 10m. The silty clay deposit was underlain by a non cohesive 6.4m to 6.8m thick silty sand to sandy silt deposit. In Borehole 1 the silty sand to sandy silt deposit was directly overlying the bedrock. In Borehole 2 the silty sand deposit was underlain by a sand and gravel deposit. Following are the detailed descriptions of the soil strata encountered.

### **Fill**

The fill was only encountered in Borehole 1 (near the northwest corner of the bridge) as the surficial deposit. The fill consisted of cohesive silty clay material. The thickness of the fill was 2.6m. The N-values within this layer was recorded as 7 and 9 blows/0.3m that suggested that the fill is stiff.

### **Silty Clay to Clayey Silt**

The predominant surficial deposit at the site was a silty clay to clayey silt. The top elevation of this deposit ranged from 274.4m (BH 1) to 275.2m (BH 2). The thickness of this deposit ranged from 9.4m (BH 2) to 10m (BH 1). The MTO field vane was used to determine the undrained shear strength of the soil. The undrained shear strength ranged from 20 kPa to 68 kPa that suggested that the deposit is in soft to stiff state.

### **Silty Sand to Sandy Silt**

This non cohesive silty sand to sandy silt deposit was underlying the silty clay deposit. The top elevation of this deposit ranged from 264.4m (BH 1) to 265.8m (BH 2). The thickness ranged from 6.4m to 6.8m. The N-values ranged from 6 blows to 123 blows/20 cm. The low N-values were occasional and thought to be disturbed and not representative. The deposit in general, is very dense.



### Sand and Gravel

This non cohesive deposit was only encountered in Borehole 2. The silty sand deposit was underlying the sand and gravel deposit. The sand and gravel deposit contained occasional boulders (needed coring to go through this deposit). The N-values in the sand and gravel deposit exceeded 100 blows suggesting this deposit to be very dense. Drilling in the sand and gravel deposit was very difficult.

### Bedrock

In Borehole 1 the silty sand to sandy silt deposit was directly overlying the bedrock. The bedrock was encountered only in Borehole 1. The bedrock was encountered at elevation 258.0m at a depth of 19m below the ground surface. The bedrock is classified as Meta-Greywacke Sandstone of the Superior Province. The length of the bedrock core was 1.3m. The result of the core analyses is plotted on the Borehole log.

### Groundwater Condition

Groundwater was monitored in open boreholes. Groundwater level in the boreholes was slightly above the water level in the creek. The groundwater table stabilized at a depth of 1.9m (BH 2) and 3.5m (BH 1) below the ground surface. The groundwater table in the boreholes ranged from 273.3m (BH 2) to 273.5m (BH 1). It should be noted that the groundwater is subject to seasonal fluctuation.

## **DISCUSSION AND RECOMMENDATIONS**

It is proposed to replace the existing bridge with a new bridge. The new bridge will be a single span ( 25.4 m). It is required to construct the bridge on integral abutments. The existing highway, at the bridge is about 9.2m wide. The highway will be widened and the new width will be about 12.6m wide. The widening will take place on both north and south sides of the highway, symmetrical to the existing centre line. The grade of Hwy 101 will be raised by 0.6m (final elevation 278.2m).

### **Structure Foundations**

The soft to stiff clayey silt deposit is not suitable for spread footings. The bridge structure can be supported on steel H-piles driven to bedrock or very dense sand and gravel deposit. It is expected that the tip of the piles will reach to elevation 257m at the east abutment and 258m at the west abutment. It is possible that the pile would not reach to the bedrock at the west abutment and stop within the very dense silty sand to sandy silt deposit.

After the piles are advanced to elevation 259.0m at the east abutment and elevation 261.5m at the west abutment, the pile driving should be controlled by the Hiley Formula as per MTO Standards SS 103-10 or SS 103-11, assuming ultimate capacities as shown below.

The recommended bearing capacities of H-piles founded on bedrock or very dense sand and gravel are as follow:

	<u>HP 310X110</u>	<u>HP 310X79</u>
Factored Axial Resistance at ULS	1600 k/pile	1150 k/pile
Factored Horizontal Resistance at ULS	80 k/pile	60 k/pile
Horizontal Resistance at SIS.	60 k/pile	40 k/pile
Ultimate Pile Resistance for Hiley Formula	3450 k/pile	2475 k/pile

The serviceability Limit State (SIS) would not apply for piles founded on bedrock or very dense sand and gravel.

Due to a grade raise of about 0.6m and the new fill beside the existing approach embankment, there will be some settlement in the embankments. This will cause downdrag forces on the piles. It is recommended that the pile capacities for the abutments should be reduced. The reduced pile capacities will be as follows:

	<u>HP 310X110</u>	<u>HP 310X79</u>
Factored Axial Resistance at ULS	1450 k/pile	1000 k/pile
Factored Lateral Resistance at ULS	80 k/pile	60 k/pile
Lateral Capacity at SIS.	60 k/pile	40 k/pile
Ultimate Pile Resistance for Hiley Formula	3450 k/pile	2475 k/pile

If integral abutments are not considered, then the lateral capacities of the piles may be supplemented by the horizontal component of the battered piles.

To facilitate pile driving, particle sizes of any fill placed beneath the pile locations should be restricted to 75mm.

#### Integral Abutment

This site is feasible for integral abutments. Therefore, integral abutments may be considered for this site. However, please check if the skew requirement of the integral abutment is met.

The integral abutment will be constructed on steel piles driven to bedrock or sand and gravel layer. The piles shall be driven through 0.5m diameter and 3m deep pre-augured holes filled with uniformly graded sand. Following is the NSSP for the special sand used for integral abutment.

#### NSSP - Backfill to Integral Abutment-Augured Hole

The annular space between the preaugured oversize hole and the pile shall be backfilled with uniformly graded sand. The gradation for the uniformly graded sand shall be as follows:

<b>MTO SIEVE DESIGNATION</b>	<b>PERCENTAGE PASSING BY MASS</b>
<b>2 mm (#10)</b>	<b>100</b>
<b>600 mm (#30)</b>	<b>80 - 100</b>
<b>425mm (#40)</b>	<b>40 - 80</b>
<b>250 mm (#60)</b>	<b>5 - 25</b>
<b>150 mm (#100)</b>	<b>0 -6</b>

Alternatively, commercially available materials which meets the above gradation may be considered.

#### Embankment Stability

The height of the existing embankment is about 3.6m. The side slopes of the existing embankments are at 2H:1V and flatter. It is proposed to raise the height of the embankment by 0.6m (final elevation 278.2m). In addition, the embankment will be widened by 3.4m symmetrical to the existing centreline. The width of the highway will therefore, change from existing 9.2m to 12.6m.

The side slopes of the embankment should be constructed at 2H:1V or flatter. However, it appears that if the embankments are constructed at 2H:1V, in some areas, the toe of the embankment will encroach into the creek. This matter should be reviewed during planning and design. This may require constructing retaining walls.

Before placement of the new fill, all surficial topsoil or any organic material should be removed within the plan limits of the embankments.

Slope stability analyses have been carried out for the proposed height of embankment 0.6m (final elevation 278.2m). Based on the analyses, no stability problems are anticipated for the proposed height of the embankments if the embankments are constructed as recommended.

#### Embankment Settlement

Due to grade raise of 0.6m, some settlement will take place in the embankment. The settlement will be in the order of 150mm to 200 mm. It is expected that 75 % of the settlement will take place within one year.

### Lateral Earth Pressure

Free draining granular material such as Granular 'A' or 'B', or rockfill is recommended as appropriate backfill to abutment walls to prevent hydrostatic pressure buildup.

If rockfill is used for approaches, special care will be required to avoid damaging to the abutment. It would be preferable to place a 0.3m cushion of Granular 'A' or smaller rockfill (with diameter of less than 300mm), between the structure and the main mass of rock fill. Granular material may also be used at the approaches.

For design purposes, the following properties for backfill are recommended:

Granular 'A'	$\gamma = 22.8 \text{ kN/m}^3$	$\phi = 35^\circ$
Granular 'B'	$\gamma = 21.2 \text{ kN/m}^3$	$\phi = 30^\circ$
Rockfill	$\gamma = 18.0 \text{ kN/m}^3$	$\phi = 35^\circ$

At rest condition ( $K_0$ ) may be assumed to apply for unyielding structures. If the structure is designed to be yielding then active condition ( $K_a$ ) should be used.

### Frost Protection

A soil cover of 2.4m or equivalent will be required for frost cover for pile caps. The pile caps should be constructed below the frost depth or the scour depth whichever is greater.

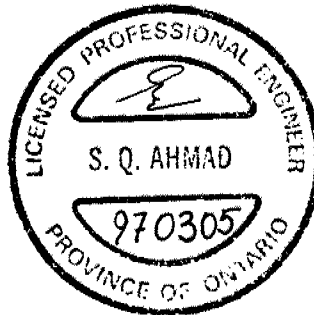
### Excavation and Dewatering

Excavation for the pile caps will take place within cohesive silty clay material. Excavation for the pile caps can be carried out within the confines of continuous steel sheeting driven into the clay. For base stability, the sheeting should extend 0.5 B below the base of the excavation, where B is the width of the excavation. For design of the sheeting, an earth pressure coefficient of  $K=1.0$  should be used. No major dewatering will be required. Minor seepage will be controlled by sump pumps.

MISCELLANEOUS

The field work for this project was carried out under the supervision of K.S.Q. Ahmad, Foundation Engineer. The equipment used was owned and operated by Master Soil Investigation Ltd..

The report was written by K.S.Q. Ahmad, Foundation Engineer, reviewed and approved by T.C. Kim, Senior Foundation Engineer.



A handwritten signature in cursive script, reading 'S.Q. (Ken) Ahmad'.

S.Q. (Ken) Ahmad, P. Eng.  
Foundation Engineer



A handwritten signature in cursive script, reading 'Tae chul Kim'.

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

## **APPENDIX**

# RECORD OF BOREHOLE No 1

1 OF 1

METRIC

W.P. 316-85-03 LOCATION Station 16+540.9; Offset 9.1m Lt C HWY 101 ORIGINATED BY KA  
 DIST 53 HWY 101 BOREHOLE TYPE HS Auger, B-Casing, BX Core COMPILED BY KA  
 DATUM Geodetic DATE 1996 11 21.22 CHECKED BY TK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100				
277.0	Ground Surface															
0.0	Silty Clay, Brown to Grey Moist, Stiff (Probable Fill)		1	SS	9		276									
			2	SS	7											
274.4																
2.6	Silty Clay to Clayey Silt Brown to Grey Moist to Wet Soft to Firm		3	ss	0		274									
			4	TW	PM											
			5	SS	0											
			6	TW	PM		272									0 0 8 92
			7	SS	0											
			8	SS	5		270									0 0 55 45
			9	SS	5		268									
			10	SS	52		266									
264.4	Silty Sand to Sandy Silt Occasional Layers of Coarse Sand Some Gravel, Grey, Wet Very Dense		11	SS	94		264									26 17 51 6
12.6			12	SS	102	/15cm	262									1 90 (9)
			13	SS	123	/20cm	260									7 54 33 6
258.0			14	RC	REC	95%	258									R0D 80%
19.0	Bedrock, Sandstone															
256.7																
20.3	End of Borehole															



# RECORD OF BOREHOLE No 2

1 OF 1

METRIC

W.P. 316-85-03 LOCATION Station 16+580.2; Offset 14.9m Rt. HWY 101 ORIGINATED BY KA  
 DIST 53 HWY 101 BOREHOLE TYPE HS Auger, B-Casing, BX Core COMPILED BY KA  
 DATUM Geodetic DATE 1996 11 19-21 CHECKED BY TK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100				
275.2	Ground Surface															
0.0			1	SS	2		274									
			2	SS	1											
	Silty Clay to Clayey Silt Brown to Grey Moist to Wet Soft to Stiff		3	SS	6		272									
			4	TW	PH											
			5	SS	4		270									0 0 41 59
			6	SS	11		268									
265.8							266									
9.4			7	SS	31		264									9 55 27 9
			8	SS	105											
	Silty Sand, Some Gravel Occasional Layers of Coarse Sand Grey, Wet Loose to Very Dense		9	SS	40		262									
			10	SS	6		260									10 56 31 3
259.0			11	SS	61											
16.2			12	SS	101	/15cm	258									38 41 (21)
	Sand and Gravel Occasional Boulders Very Dense		13	SS	120	/5cm										
255.3			14	SS	120	/8cm	256									54 32 (14)
19.9	End of Borehole															

# RECORD OF BOREHOLE No 3

1 OF 1

METRIC

W.P. 316-85-03 LOCATION Station 16+564.2; Offset 8.6m Rt. G HWY 101 ORIGINATED BY KA  
 DIST 53 HWY 101 BOREHOLE TYPE Cone Test COMPILED BY KA  
 DATUM Geodetic DATE 1996 11 21 CHECKED BY TK

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 × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W <sub>p</sub> NATURAL MOISTURE CONTENT W LIQUID LIMIT W <sub>L</sub> WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
274.4	Ground Surface										
0.0	Probable Silty Clay to Clayey Silt Soft to Stiff						274				
							272				
							270				
							268				
266.0							266				
8.4	Probable Silty Sand, Some Gravel Loose to Very Dense						264				
263.1											
11.3	End of Cone Test										

## 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
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### MECHANICAL PROPERTIES OF SOIL

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

### PHYSICAL PROPERTIES OF SOIL

$\rho_s$	kg/m <sup>3</sup>	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	$e_{min}$	1, %	VOID RATIO IN DENSEST STATE
$\gamma_s$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$\rho_w$	kg/m <sup>3</sup>	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
$\gamma_w$	kN/m <sup>3</sup>	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	n PERCENT - DIAMETER
$\rho$	kg/m <sup>3</sup>	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	kg/m <sup>3</sup>	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	q	m <sup>3</sup> /s	RATE OF DISCHARGE
$\gamma_d$	kN/m <sup>3</sup>	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
$\rho_{sat}$	kg/m <sup>3</sup>	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
$\gamma_{sat}$	kN/m <sup>3</sup>	UNIT WEIGHT OF SATURATED SOIL	$I_C$	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	kg/m <sup>3</sup>	DENSITY OF SUBMERGED SOIL	$e_{max}$	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m <sup>2</sup>	SEEPAGE FORCE
$\gamma'$	kN/m <sup>3</sup>	UNIT WEIGHT OF SUBMERGED SOIL						

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

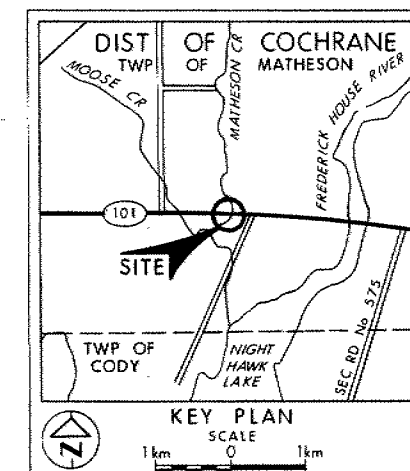
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MATHESON CREEK

SHEET

BORE HOLE LOCATIONS & SOIL STRATA



LEGEND

- 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)
- W.L. at time of investigation 1996 11

No	ELEVATION	STATION	OFFSET
1	277.0	16+540.9	9.1 m LT
2	275.2	16+580.2	14.9 m RT
3	274.4	16+564.2	8.6 m RT

NOTE

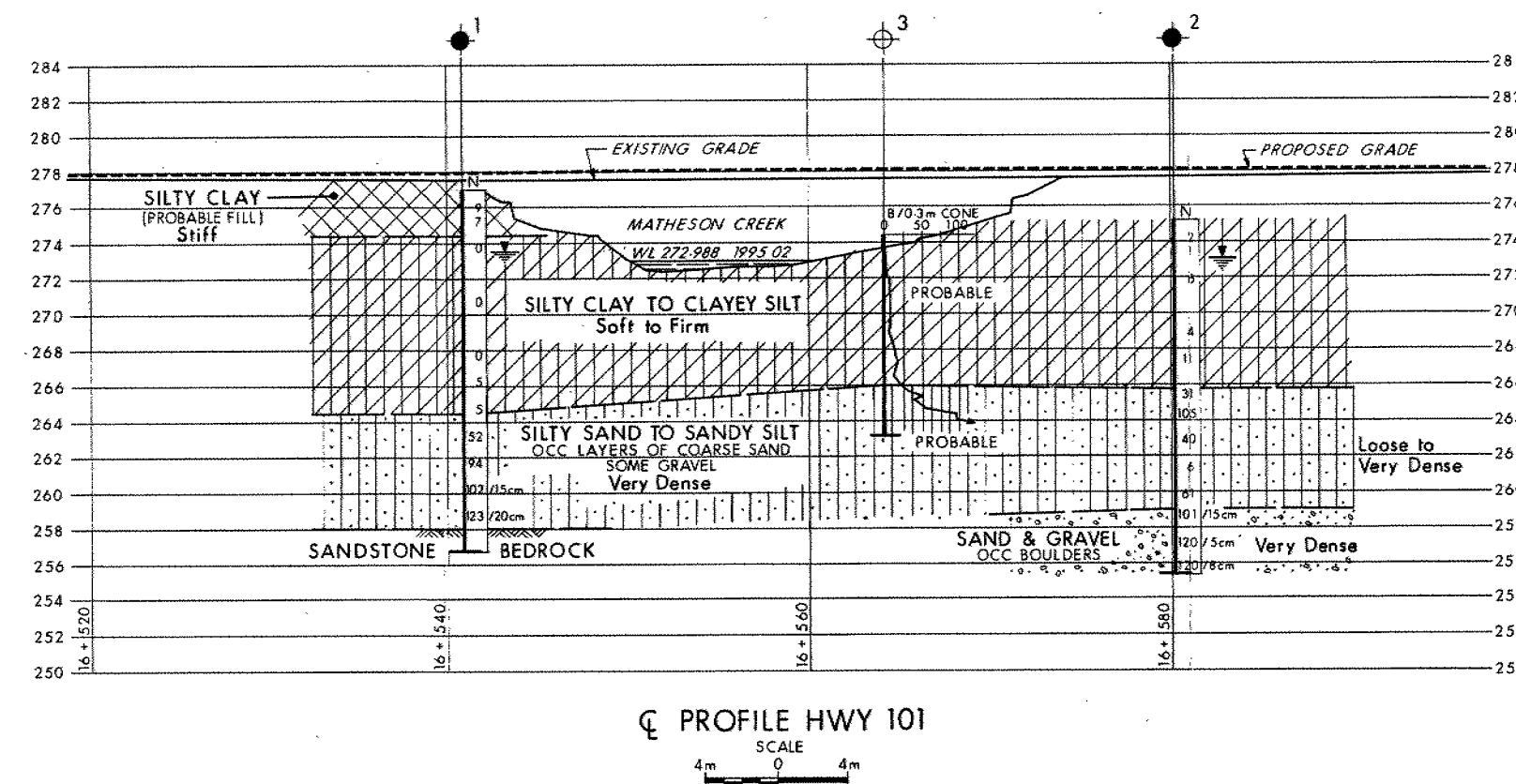
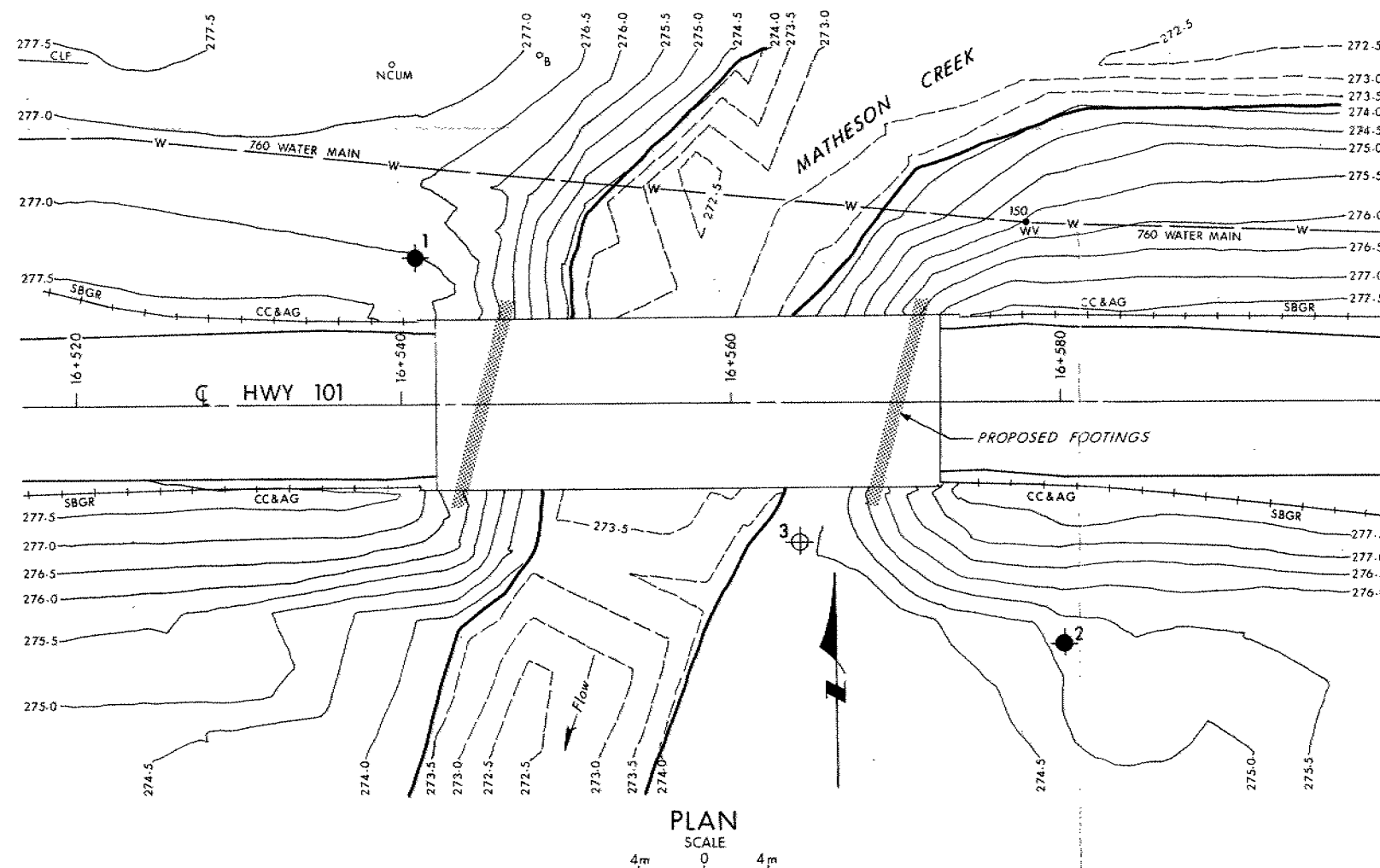
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section GC 2.01 of CPS Gen Cond

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