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DIST. 9 REGION

W.P. No. 11-81-01 (A)

CONT. No. 93-62

W. O. No.

STR. SITE No.

HWY. No. 17

LOCATION Hwy 17 & Nawan Rd.
(Reg. Rd. 57)

No of PAGES -

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

G.I.-30 SEPT. 1976

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Ministry
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FOUNDATION DESIGN SECTION

**foundation
investigation and
design report**

ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION

CONT 93-62

WP 11-81-01 (A)

DIST 9

HWY 17

STR SITE

Hwy. 17 and Navan Rd. Interchange

(Reg. Road 57)

District 9, Ottawa

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PRELIMINARY
FOUNDATION INVESTIGATION REPORT
For
W.P. 11-81-01 (A)
Hwy 17 and Navan Rd Interchange
(Reg. Rd 57)
District 9, Ottawa

INTRODUCTION

This report summarizes the information obtained from a preliminary foundation investigation carried out at the above-noted site, and provides preliminary recommendations pertaining to the structure foundations and the stability and settlement of the approaches.

The fieldwork was carried out between 86 05 21 and 86 05 28 and consisted of 3 sampled boreholes (BH 1, BH 2, and BH 3) advanced by a continuous flight auger machine equipped with hollow stem augers. A dynamic cone penetration test accompanied BH 1 and BH 3. The boreholes were advanced to depths of 41.5, 21.9, and 36.5 m for BH 1, 2, and 3 respectively.

SITE DESCRIPTION

The site, as shown on DWG 1, is located at the proposed interchange of Reg. Rd 57 and Hwy 17. The site is situated in the Township of Cumberland, Regional Municipality of Ottawa-Carlton, approximately 3.3 km east of Champlain Road.

Land in the immediate area is primarily used for agricultural purposes. The topography is generally flat to gently rolling.

Physiographically, the site is situated in the region known as the "Ottawa Valley Clay Plains". This area consists of clay plains interrupted by ridges of rock or sand (Ref: Chapman & Putnam).

SUBSURFACE CONDITIONS

General

The Record of Borehole Sheets in the Appendix illustrate the subsurface conditions at the borehole locations.

Under the surficial topsoil or roadway bedding, a layer 2-3 m thick consisting of stiff to very stiff desiccated clay of high plasticity is encountered. Beneath this layer is a similar clay of high plasticity ranging in consistency from firm to stiff. The thickness of this underlying weaker stratum can be considered to be approximately 35 m. Underlying the clay is a hard glacial till deposit consisting of a heterogeneous mixture of silty clay, sand and gravel. Underlying the till is limestone bedrock.

The following is a description of the conditions encountered at the site.

Clay of High Plasticity

A stiff to very stiff layer of desiccated brown clay of high plasticity 2 to 3 m thick was encountered at the borehole locations. Immediately underlying this upper "crust" is the predominant stratum across the site; a grey clay of high plasticity. The total thickness of the clay deposit was found to be 38.4 and 36.3 m in BH 1 and 3 respectively, and extending down to Elev. 16.3 and 15.7.

The physical properties of this clay as determined from field and laboratory tests are summarized as follows:

		<u>Range</u>	<u>Average</u>	<u>No. of Tests</u>
Unit Weight	(γ)	15.4-18.9 kN/m ³	16.3	15
Natural Moisture Content	(W)	30.5-69%	56.4	21
Liquid Limit	(W _L)	48.5-67%	55.1	21
Plastic Limit	(W _p)	21-29.5%	25.2	21
Plasticity Index	(I _p)	24-36.5%	30	21
Shear Strength (Field)	(C _u)	19->100 kPa	-	28
Sensitivity (Field)		2-11	6.5	28
Unconfined Strength (Lab)	(C _u)	22.5-75.1 kPa	48.5	13

The results of the 21 Atterberg Limits tests carried out on samples of this material are shown in Figure 1 in the Appendix. The results indicate that this clay is inorganic and of high plasticity (CH Group).

Figure 2 in the Appendix summarizes the results of the Atterberg Limits tests in relation to depth. With the exception of the clay which forms the upper "crust", the moisture content is found to generally exceed the liquid limit. However, with depth the moisture content becomes closer to the liquid limit, and eventually below the liquid limit value.

The consistency of the clay deposit, as determined by unconfined, undrained shear and field vane tests becomes stiffer with depth. Figure 3 in the Appendix illustrates the relationship between the in-situ shear strength and depth. It is to be noted that the shear strength of the upper "crust" is appreciably greater than that of the underlying clay.

Sensitivity within this highly plastic clay was measured in 28 in-situ tests to range between 2 and 11, with an average of 6.5. Sensitivity is defined as the ratio of the undisturbed to the remoulded shear strength. Clays with sensitivities between 4 and 8 are referred to as 'sensitive'. Those with

sensitivities between 8 and 16 are referred to as extra-sensitive. (Ref: Soil Mechanics, R.F. Craig, 1978). Therefore, this material can be considered to be sensitive to extra-sensitive.

Consolidation tests on 10 samples of this material were carried out. The results are summarized on the Record of Borehole Sheets.

Organic inclusions in the form of thin, and randomly spaced seams are evident in the clay deposit at depths greater than 20 m. An organic content test was carried out on one sample of the clay stratum (BH 1, Sample #17). The results indicated that 1% organic inclusions are present. This represents a negligible amount.

Glacial Till

Beneath the cohesive stratum previously described there exists a deposit of hard glacial till. The till deposit was found to be 1.5 and 0.5 m in thick in BH 1 and BH 3 respectively.

An Atterberg Limits test was carried out on a sample of the cohesive material. The results are shown on Figure 1 in the Appendix. The results indicate that the fines of this material can be considered to be a silty clay of low plasticity (CL Group).

A grain size distribution test was carried out on a sample of this glacial deposit. The results of the one test are as follows: 47% gravel; 16% sand; 22% silt; 15% clay. The deposit can be described as a heterogeneous mixture of silty clay, sand, gravel. It should also be noted, however, that boulders and cobbles may be randomly encountered within the till.

Bedrock

Bedrock was proven in BH 1 by obtaining a 1.6 m core sample. In BH 3, bedrock was assumed to occur at the elevation where the dynamic cone test was refused, and where the split spoon was found to "bounce".

Bedrock at this location consists of limestone of the Leray beds of the Trenton and Black River formation (Palaeozoic Period).

Groundwater Conditions

Observations of water levels in open boreholes were made during the course of the field investigation. The groundwater level was found to occur at Elev. 48.0 in BH 1 and 2. In BH 3, the groundwater level was found to be at Elev. 44.6. This indicates that the hydraulic gradient slopes gently down in a northerly direction towards the Ottawa River.

DISCUSSION AND RECOMMENDATIONS

General

It is proposed to construct a structure to carry Regional Rd 57 (Navan Rd) over Hwy 17. This structure is to accomodate the proposed widening of Hwy 17 from an existing 2-lane highway to a 4-lane divided highway. The height of the approaches could be as high as 10 m above the existing ground surface.

Results of this preliminary foundation investigation indicate that underlying between 2 and 3 m of generally stiff to very stiff desiccated clay of high plasticity is the predominant stratum across the site - a clay of high plasticity ranging in consistency from firm to stiff with depth. This clay stratum is up to 38 m in thickness. The clay is underlain by up to 1.5 m of till composed of a heterogeneous mixture of silty clay, sand and gravel. The till is in turn underlain by limestone bedrock.

The presence of the extensive deposit of highly compressible clay requires that special attention be given to the stability of the embankments, as well as to the settlement which will be induced by the approach fills. The following are our preliminary recommendations for the design and construction of the structure and the associated approach fills with emphasis on stability and settlement.

It should be noted that the recommendations presented in this report are preliminary in nature and should not be used for final design purposes. When the interchange and structure geometry is finalized, a detailed subsurface investigation will be required.

APPROACHES

Stability

The critical condition for stability of an embankment on normally or slightly over consolidated clay, as is the case with this clay deposit, generally occurs during or immediately after construction. This being the case, a total stress analysis ($\phi = 0$) provides a suitable means of assessing the stability of the embankment. The total stress analysis takes into consideration the undrained shear strength properties of the foundation and embankment soils.

In this preliminary analysis of the proposed embankment, the following assumptions were made:

- Fill Material: non-cohesive

Bulky Density $\gamma = 21.0 \text{ kN/m}^3$

Angle of Shearing Resistance $\phi = 28^\circ$

- Foundation Subsoils: clay of high plasticity

<u>Depth below ground surface</u>	<u>Cu (kPa)</u>	<u>Bulk Density (kN/m³)</u>
0 - 3 m	60	17
3 - 8 m	40	16
8 - 13 m	52	16
13 - 18 m	63	16
18 - 23 m	75	16
23 - 25 m	83	16
> 25 m	100	16

- . Maximum height of approaches is restricted to 10 m.
- . A minimum factor of safety of 1.3 is considered acceptable.
- . Berms, where required, are constructed at mid-height.
- . All slopes are constructed at 2H:1V.
- . All surficial organic, soft, or loose material is removed prior to placing fill.
- . Berms are to be provided both in the longitudinal and transverse direction at the approaches.

A typical section analyzed is shown on Figure A (Pg. 9). A 10 m high fill, constructed with 2H:1V front and side slopes would be marginally stable as indicated by the geometry of the failure circle and the associated factor of safety of 1.1. Figure B (Pg. 9) shows that for this height of fill, a 10 m counterbalancing mid-height berm would be required in order to ensure a minimum factor of safety of 1.3.

The results of the stability analysis have been summarized in terms of fill height (H) vs. required length of mid-height berm (L). This relationship is shown on Figure C (Pg. 10). As indicated by this curve no berm is required for fills less than 8 m in height. For fill heights between 8 and 10 m, the associated mid-height berm length can be obtained from the curve. This Section should be consulted if fills greater than 10 m are required.

It may be possible to reduce the width or totally neglect berms if the embankments are constructed of light-weight material. Berms could be completely eliminated if light-weight slag (max. unit weight 14 kN/m^3) was to be

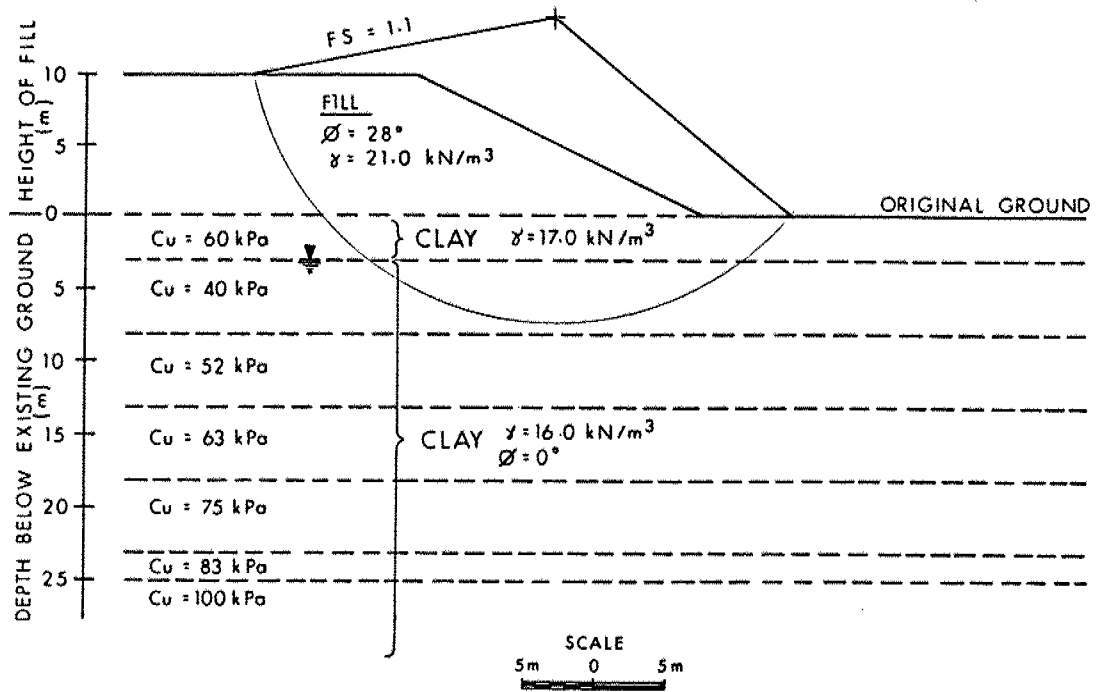


Fig A

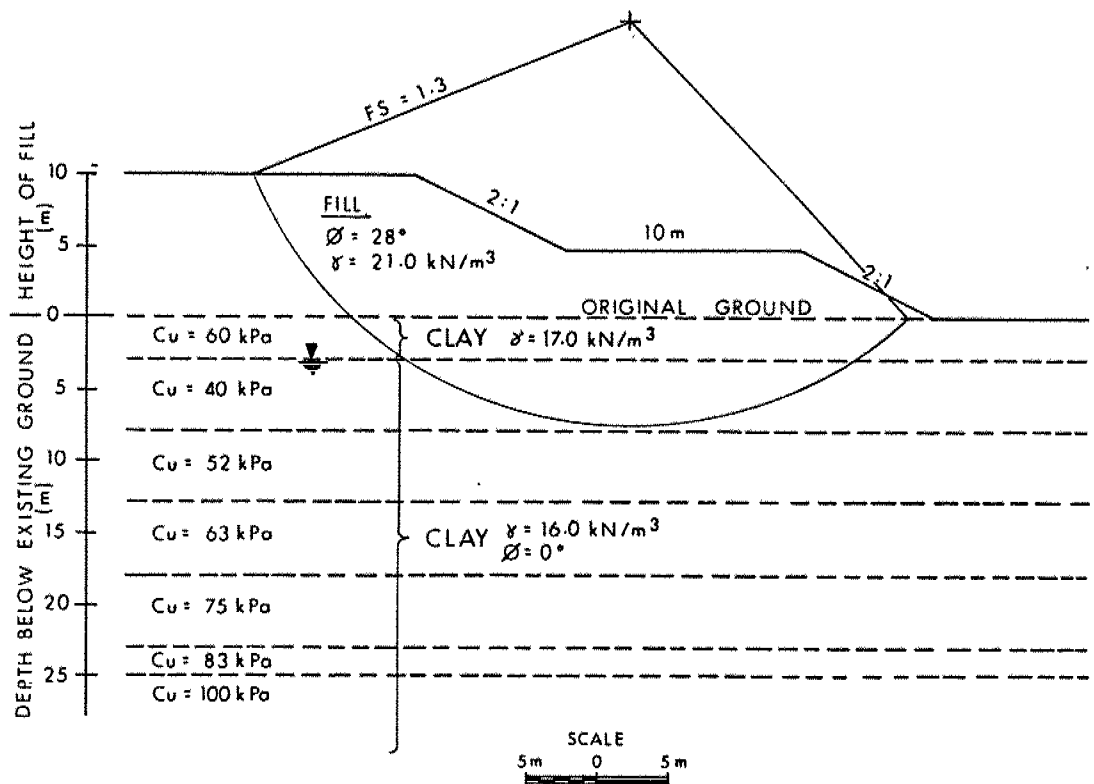


Fig B

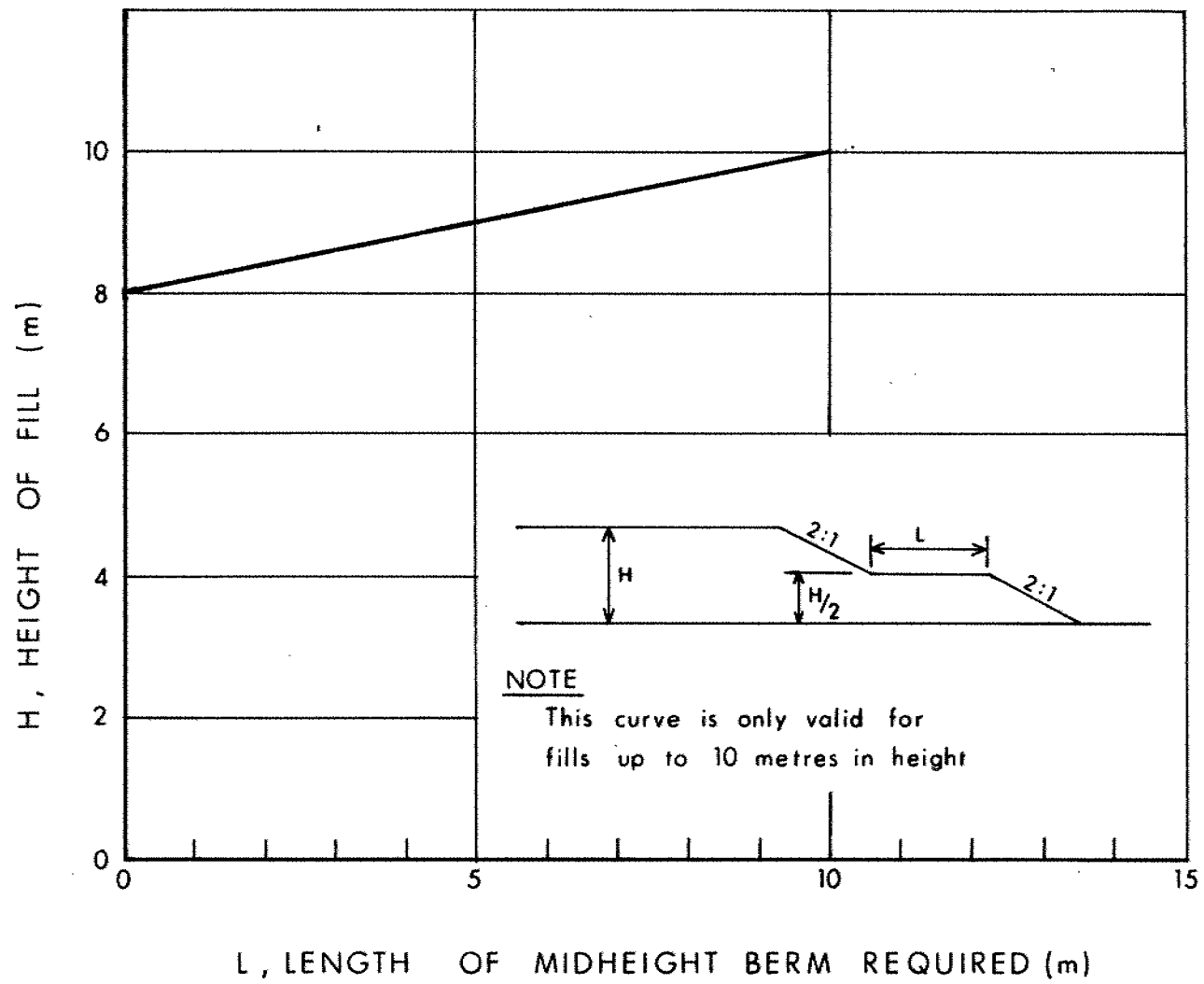


Fig C

WP 11-81-01(A)

used for the embankment construction. Additional details if required could be obtained from this Section.

It should be noted that the relationship in Figure C should only be used for preliminary design purposes. When the design is finalized, a detailed stability analysis of all fills will be required.

Settlement

The underlying compressible clay stratum will experience appreciable settlements due to consolidation under the additional stresses imposed by the proposed embankment. Settlement calculations were carried out and the results are summarized on Figure D (Pg. 12). In the settlement analysis, it was assumed that the weight of the fill material used would not exceed 21.0 kN/m^3 , and that berms would be incorporated as per Figure C for fill heights greater than 8 m. In addition, it was assumed that the compressible zone extends to a depth of 27 m below the existing ground surface.

Figure D shows the relationship between fill height and expected settlement of the native foundation material along the centreline of the embankment. As illustrated on this figure, fills greater than 6.5 m in height will cause non-proportionally greater settlements than fills less than 6.5 m. Fills exceeding 6.5 m in height will induce stresses of a magnitude greater than the soils' preconsolidation pressure. As a result, settlements will be appreciably larger for fills greater than 6.5 m.

In addition to the settlements shown in Figure D, the settlement of the fill material itself should also be taken into consideration. It is estimated that cohesive fill material will settle approximately 0.5% of its height in the long

SETTLEMENT vs EMBANKMENT HEIGHT

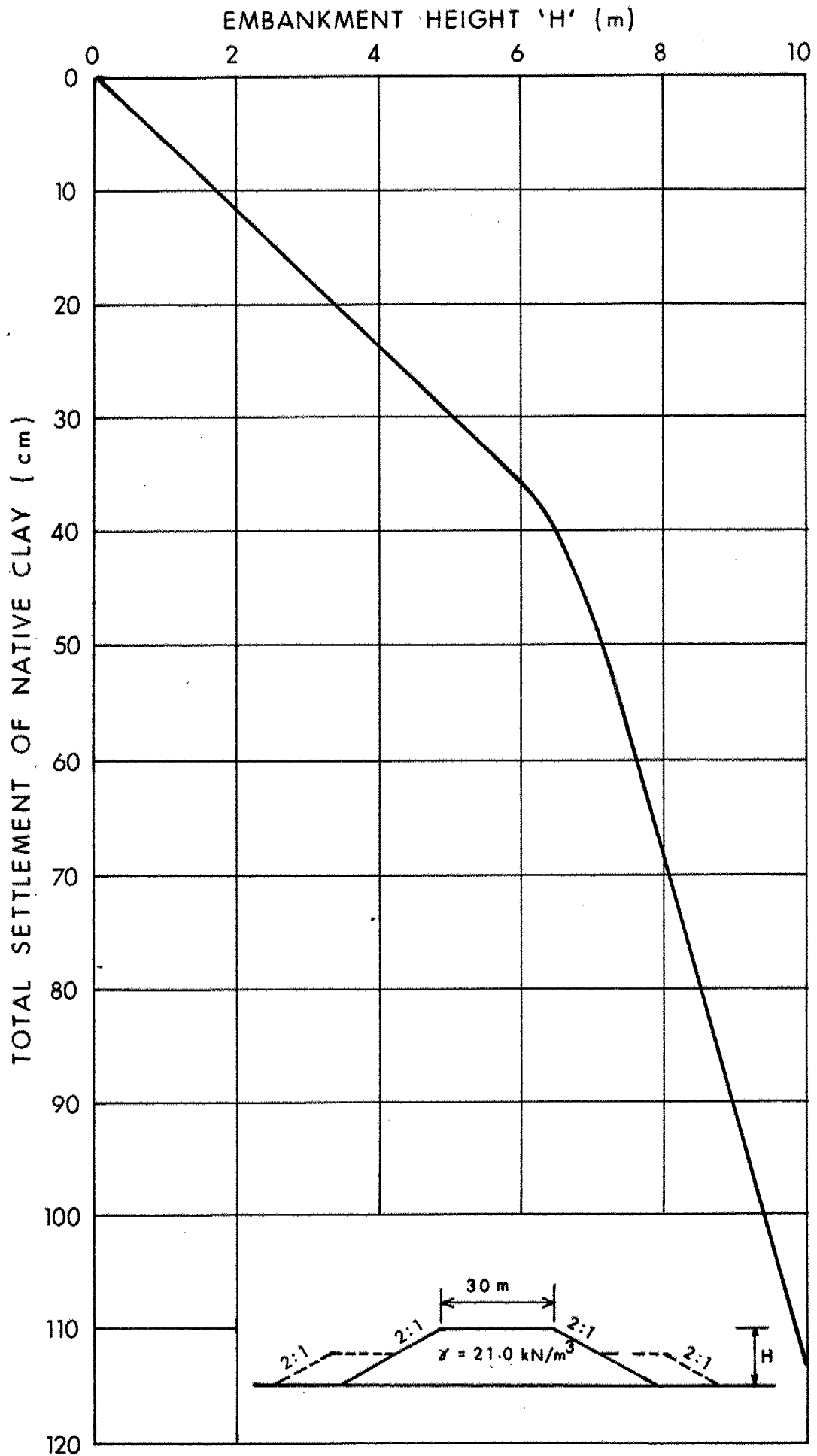


Fig D

term due to it's self weight. Non-cohesive fill material will also experience settlement, however, it can be anticipated to occur during construction.

Settlements in clays generally continue to occur long after the embankment has been constructed. It is estimated that the total settlement described in Figure D will occur over a 10-12 year period. However, it is expected that the initial 40-50% of the settlement will occur within the first 2 years.

In order to accelerate post-construction settlements, it may be possible to pre-load the embankments with an additional 2 m surcharge for a minimum period of 18 months. It should be noted that if the surcharge is incorporated for the 10 m fills, mid-height berms of up to 20 m will be required during the pre-load period. This option obviously has considerable implications on property requirements. This section should be contacted for additional details of pre-loading.

Another option which could be considered involves the use of light-weight slag fill. It is estimated that a 10 m high embankment constructed of slag material having a maximum unit weight of 14 kN/m^3 , would experience about 40 cm total settlement at the centre of the embankment. If this option is selected this Section should be contacted regarding additional details. This alternative would reduce property requirements since fills of 9 to 10 m in height constructed of this material would not require extensive stabilizing berms.

In order to reduce or accelerate the anticipated settlements other options involving expanded polystyrene styrofoam fill or drain wicks could be considered. Additional details could be provided by this Section if these alternatives are given consideration.

STRUCTURE FOUNDATIONS

The proposed structure may be supported on steel H-piles, equipped with reinforced tips (to facilitate pile driving through the glacial till) and driven to bedrock. For estimating purposes the following bedrock elevations may be used:

South abutment: Elev. 14.8
North abutment: Elev. 15.4
Piers : interpolation between the above 2 elevations.

The following design values are recommended for the piles at the piers, assuming that no fill material is required at the pier locations. If at the pier locations fill is required, these loadings may have to be reduced because of the effects of negative skin friction resulting from the consolidation of the underlying clay deposit.

(Pier) <u>Pile Type</u>	<u>Factored Capacity</u> <u>at ULS</u>	<u>Capacity at</u> <u>SLS Type II</u>
310 HP 110	1600 kN per pile	1150 kN per pile
310 HP 79	1150 kN per pile	850 kN per pile

Negative skin friction will be imposed on the piles supporting the abutments due to settlement of the highly plastic clay underlying the approach embankment. Furthermore, these forces, combined with lateral movement of the subsoil due to the strain imposed by the embankment loading, will tend to displace the piles laterally. In order to minimize rotation of the abutments, the wingwalls should also be supported on steel H-piles driven to bedrock. Piles should be battered in both directions in order to resist the lateral forces.

The following design values are recommended for the abutments and wing walls:

(Abutment) <u>Pile Type</u>	<u>Factored Capacity</u> at ULS	<u>Capacity at</u> SLS Type II
310 HP 110	1280 kN per pile	920 kN per pile
310 HP 79	920 kN per pile	660 kN per pile

If desired, the abutment footings (supported on steel H-piles) may be perched within the embankment fill. To facilitate pile driving, particle size in the fill immediately underneath the pile locations should not exceed 75 mm.

EARTH PRESSURE CALCULATIONS

Backfill to structures should consist of granular material in accordance with MTC Standard Special Provision #121 (Oct. 1983). Computation of earth pressures should be carried out in accordance with Section 6.6.1.2 of 1983 O.H.B.D.C.

For design purposes, the physical properties of the backfill are as follows:

<u>Material</u>	<u>ϕ</u>	<u>γ</u>
Granular 'A'	35°	22.0 kN/m ³
Granular 'B'	30°	21.2 kN/m ³

Geotechnically, the foundation is considered to be non-yielding as the piles will be driven to bedrock and the at-rest condition (K_0) applies for lateral earth pressures. Structurally, the active (K_a) condition may control the earth pressure design due to the length of the piles.

GENERAL RECOMMENDATIONS

- . All topsoil and surficial organic material within the plan limits of the embankment should be removed.
- . All fill material placed in the area where the piles will penetrate should be restricted to a maximum particle size of 75 mm.
- . No dewatering problems are anticipated for footing excavations. Any localized seepage may be controlled by pumping from sumps.
- . An earth cover of 1.8 m (or equivalent) should be provided to all underside of footings to protect against frost action.

MISCELLANEOUS

The preliminary recommendations outlined in this report are based on a limited number of boreholes located within a relatively small area of the proposed interchange. When the final conceptual design is available for the interchange it will be necessary to carry out a detailed subsurface investigation. Therefore, the recommendations given in this report are subject to revision.

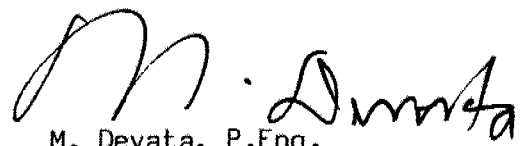
The fieldwork for this project was carried out under the supervision of Mr. L. Politano, Project Foundations Engineer, and B. Schuknecht, Student Engineer.

This report was prepared by L. Politano and was reviewed by Mr. M. Devata, Chief Foundations Engineer (East).

The drilling equipment used was owned and operated by Downing Drilling Co. Ltd. of Quebec. Downing was sub-contracted by Marathon Soil Drilling Inc., which was in contract with MTC.



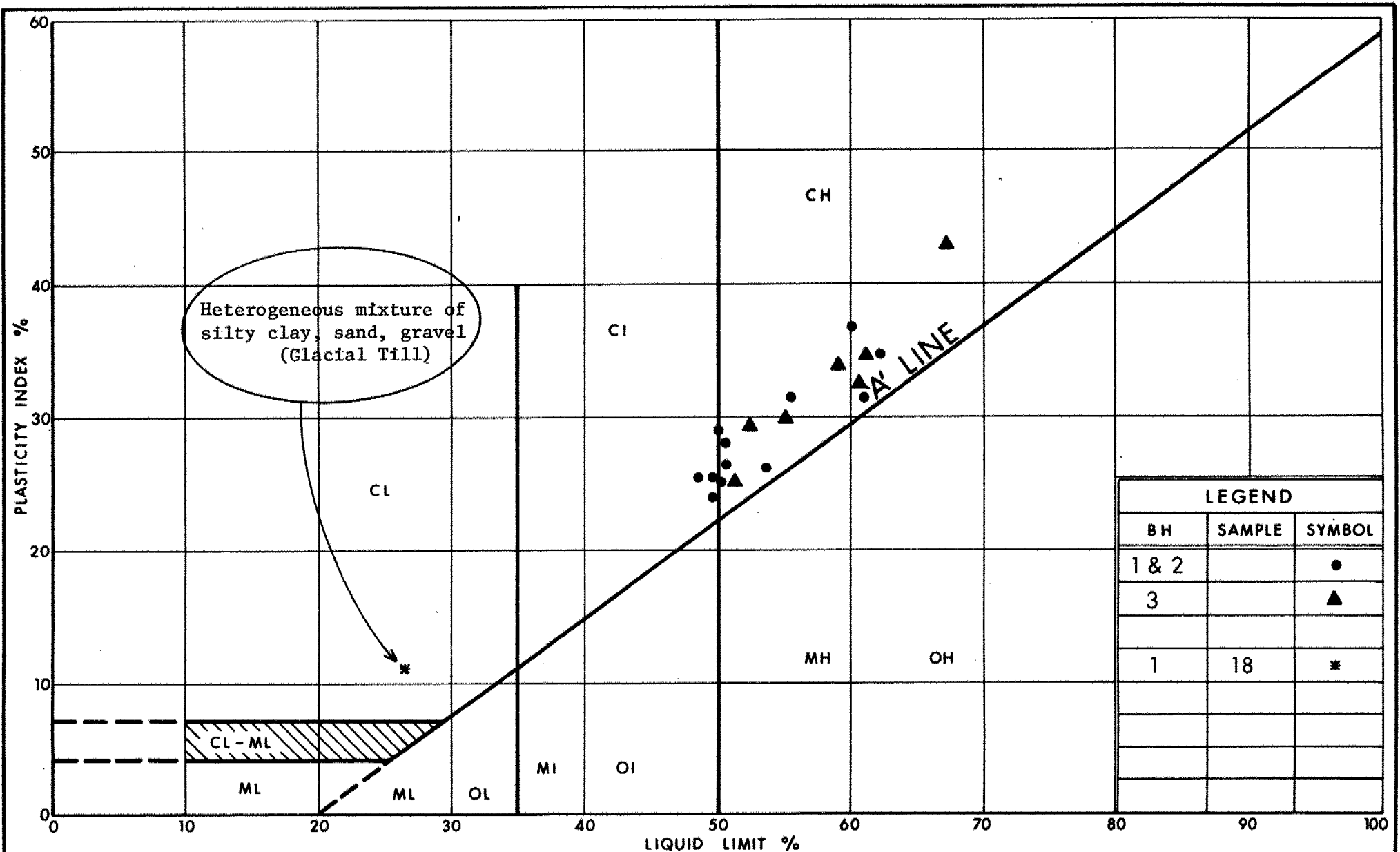
L. Politano, P.Eng.
Project Foundations Engineer



M. Devata, P.Eng.
Chief Foundations Engineer
(East)

September 1986

APPENDIX



Ministry of
Transportation and
Communications

PLASTICITY CHART CLAY OF HIGH PLASTICITY

FIG No 1

W P 11-81-01 (A)

ATTERBERG LIMITS vs DEPTH

WATER CONTENT (%)

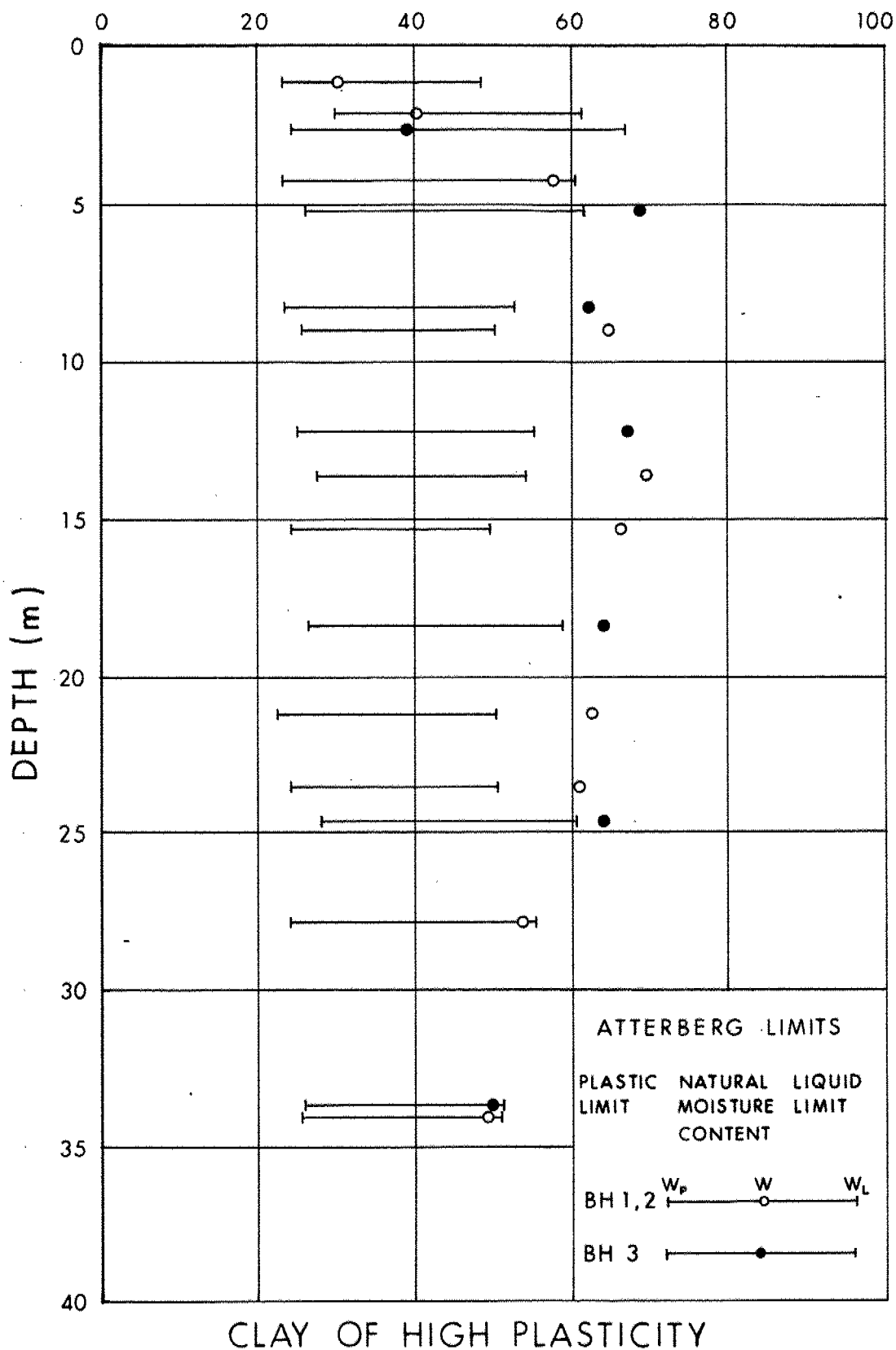


Fig 2

SHEAR STRENGTH vs DEPTH

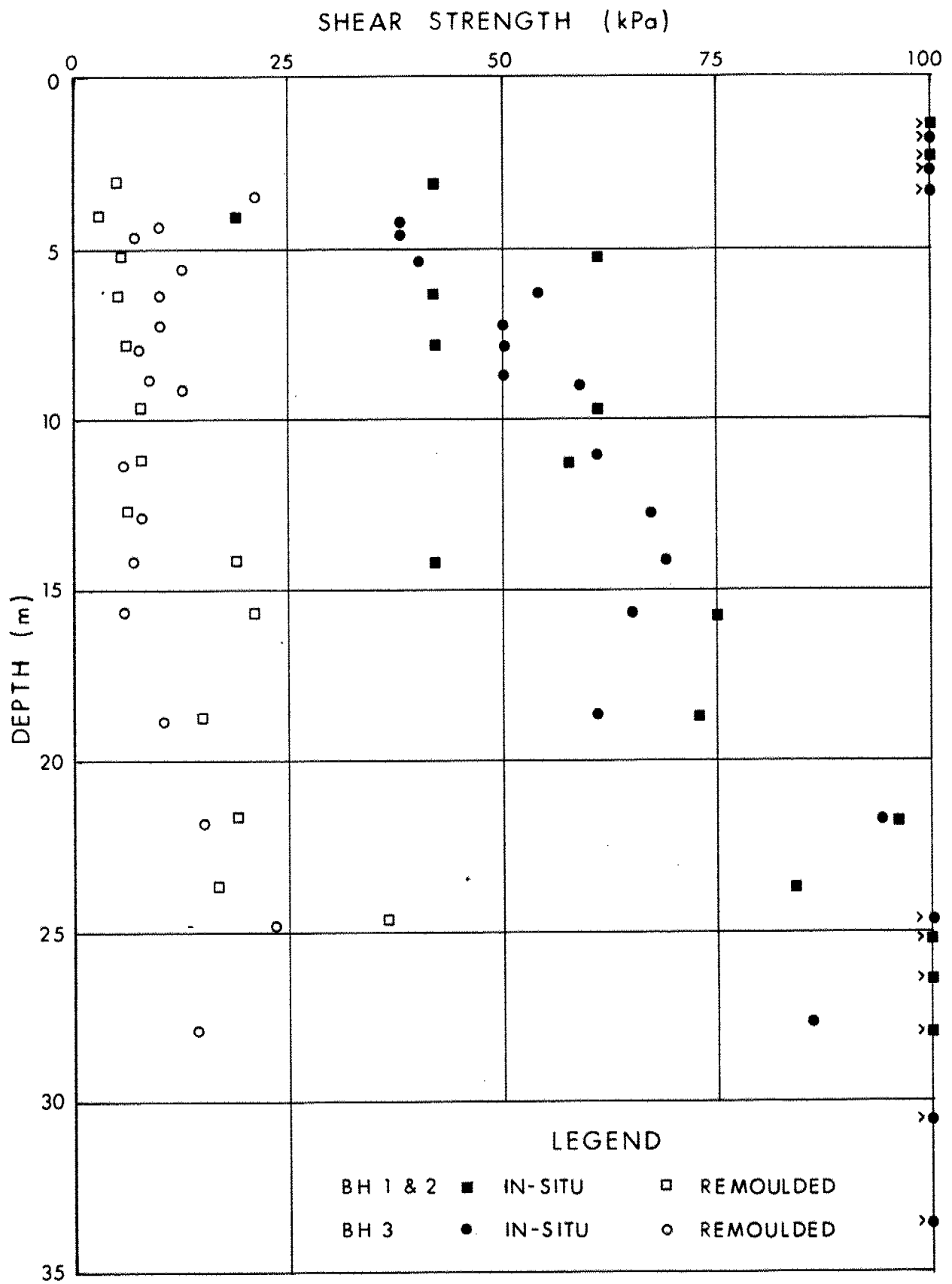


Fig No 3

WP 11-81-01(A)



RECORD OF BOREHOLE No 1

METRIC

W P 11-81-01(A) LOCATION STA. 13+272.4; %s 43.2 m Rt Q Hwy. 17 ORIGINATED BY B.S.
DIST 9 HWY 17 BOREHOLE TYPE Hollow stem auger, BQ rock core COMPILED BY L.P.
DATUM Geodetic DATE 86 05 21, 22, 23 CHECKED BY

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 20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES								
54.7	Shoulder Surface												
0.0	Sand and Gravel (fill)		1	SS	6		54						
	Desiccated		2	SS	6								
			3	SS	11								
			4	SS	5		52						
	Clay of High Plasticity		5	SS	2								
			6	SS	2								
			7	SS	1		50						
			8	SS	1								
			9	SS	1		48						
			10	SS	2								
	Firm to Stiff		11	SS	2		46						
			12	TW	PH								
			13	TW	PH		44						
			14	TW	PH								
			15	TW	PH		42						
							40						
							38						
							36						
							34						
							32						
							30						
							28						
							26						

Continued

+3, x5: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE

Continued

consolidation
on TW 14
e₀ = 1.73
P_c = 710 kPa
C_c = 1.49



RECORD OF BOREHOLE No 1 Continued METRIC

W P 11-81-01(A) LOCATION STA. 13+272.4; 9/8 43.2 m Rt Q Hwy. 17 ORIGINATED BY B.S.
DIST 9 HWY 17 BOREHOLE TYPE Hollow stem auger, BQ rock core COMPILED BY L.P.
DATUM Geodetic DATE 86 05 21, 22, 23 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	WATER CONTENT (%)					
								SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE						
24.2	Continued		16	TW	PH		24							
30.5	Clay of High Plasticity						22						ORG=12%	
			17	SS	9		20							
							18							
16.3	Heterogeneous Mixture of silty clay, sand, gravel (Glacial Till)		18	SS	103		16						47 16 22 15	
38.4							14							
14.8	Limestone Bedrock		19	SS	-									
39.9			1	RC										
			2	RC										
			3	RC										
13.2			4	RC										
41.5	End of Borehole													

+³, x⁵: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 2

METRIC

W P 11-81-01(A) LOCATION STA. 13+272.2; 9s 46.9 m Rt G Hwy. 17 ORIGINATED BY B.S.
 DIST 9 HWY 17 BOREHOLE TYPE Hollow stem auger COMPILED BY L.P.
 DATUM Geodetic DATE 86 05 24 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	'N' VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100					
54.7	Shoulder Surface															GR SA SI CL
0.0	Sand and Gravel (Fill)		1	TW	PH		54		Q						18.9	
	Desiccated		2	TW	PH					>100						
			3	TW	PH		52			>100						
			4	TW	PM			7	9						17.0	consolidation on TW 5 e _o = 1.61 P _c = 154 kPa C _c = 0.78
	Clay of High Plasticity		5	TW	PM		50		4	11					15.7	
			6	TW	PH				d						15.4	
							48		8							
			7	TW	PH				7							
							46								15.6	consolidation on TW 8 e _o = 1.97 P _c = 188 kPa C _c = 1.44
			8	TW	PM					8						
							44			8						
	Firm to Stiff		10	TW	PM		42			8						
			11	TW	PH		40		2							
			12	TW	PH		38			4					15.7	consolidation on TW 12 e _o = 1.82 P _c = 200 kPa C _c = 1.16
			13	TW	PH		36			5						consolidation on TW 13 e _o = 1.79 P _c = 430 kPa C _c = 1.37
							34									
32.8			14	TW	PH					5					16.0	
21.9	End of Borehole															

+3, x5: Numbers refer to
Sensitivity

20
15
10

5 (% STRAIN AT FAILURE

METRIC

W P 11-81-01(A) LOCATION STA. 13+305.4; 0/s 57.6 m Lt. C Hwy. 17 ORIGINATED BY B.S.
DIST 9 HWY 17 BOREHOLE TYPE Hollow stem auger COMPILED BY L.P.
DATUM Geodetic DATE 86 05 26, 27, 28 CHECKED BY _____

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ KN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20 40 60 80 100	W _p W W _L	20 40 60			
								SHEAR STRENGTH kPa		WATER CONTENT (%)			
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE							

51.9	Ground Surface												
0.0	topsoil												
	Desiccated		1	TW	PH								
			2	TW	PH								
			3	TW	PH								
			4	TW	PH								
	Clay of High Plasticity												
			5	TW	PH								
			6	TW	PM								
			7	TW	PM								
	Firm to Stiff		8	TW	PM								
			9	TW	PH								
			10	TW	PH								
			11	TW	PH								
			12	TW	PH								
		13	TW	PH									
		14	TW	PH									

50

48

46

44

42

40

38

36

34

32

30

28

26

24

22

17.8

15.5

16.2

16.0

15.9

15.9

consolidation on TW 4:
 $e_o = 1.85$
 $P_c = 215$ kPa
 $C_c = 2.78$

consolidation on TW 6:
 $e_o = 1.77$
 $P_c = 163$ kPa
 $C_c = 1.00$

consolidation on TW 8:
 $e_o = 1.82$
 $P_c = 292$ kPa
 $C_c = 1.38$

consolidation on TW 12:
 $e_o = 1.81$
 $P_c = 545$ kPa
 $C_c = 1.50$

consolidation on TW 14:
 $e_o = 1.53$
 $P_c = 770$ kPa
 $C_c = 1.26$

Continued



RECORD OF BOREHOLE No 3 Continued METRIC

W P 11-81-01(A) LOCATION STA. 13+305.4; 9/8 57.6 m Lt G Hwy. 17 ORIGINATED BY B.S.
DIST 9 HWY 17 BOREHOLE TYPE Hollow stem auger COMPILED BY L.P.
DATUM Geodetic DATE 86 05 26, 27, 28 CHECKED BY _____

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 20 40 60 80 100	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES							
21.4	Continued		15	TW	PH							
30.5				16	SS	6						
15.4				17	SS	-	*					
36.5	End of Borehole											
	* Spoon Bouncing Probable Bedrock											

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

Copy for the information of

545-4719

October 29, 1986

J.W. Reid, Head
Planning and Design Section
Kingston, Ontario

Att'n K.S. Shepherd
Sr. Project Manager

From: Structural Section
Eastern Region
Kingston, Ontario

Re: W.P. 11-81-01, Interchange Proposal
Highway 17 and Navan Road Interchange Structure
District 9 - Ottawa



I refer to the Preliminary Foundation Investigation Report dated September 19, 1986, for the above noted interchange.

Of interest is the statement on Page 8 that "for fills less than 8 m in height, no mid-height berms are required".

The obvious savings, if height of fills is kept below 2 m is illustrated on the attached two sketches:

- SK.1 shows a four span bridge with a total span of 90 m and 10 m wide mid-height berms.
- SK.2 shows a two span bridge with a total span of 70 m.

The difference in cost between the two structures would be approximately \$250,000.

Maintaining fills to below 8 metres, even if it puts a strain on other design components, appears to be prudent.

For your consideration.

A. Van Dalen / 10

A. Van Dalen
Structural Officer
for:
E.C. lane
Head, Structural Section

ECL:AVD:bd

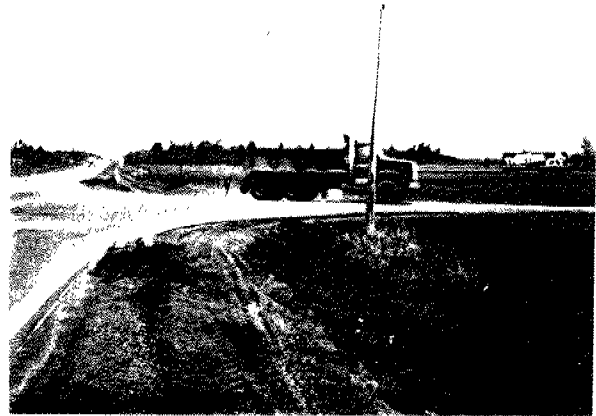
c.c. M. Devata

REVISIONS				
	DATE	BY	DESCRIPTION	
	DESIGN	CHECK	LOADING	DATE <i>0CT/86</i>
	<i>DRAWING TWIN</i>	CHECK <i>AVD</i>	SITE No	DWG. <i>SE 1</i>

REVISIONS	DATE		BY	DESCRIPTION	DATE
	DESIGN	CHECK		LOADING	
DRAWING TWIN	CHECK	RVD	SITE No		DWG SK 2



NAVAN ROAD & HWY. 17
LOOKING WEST ALONG HWY. 17



NAVAN ROAD & HWY. 17
LOOKING WEST ALONG HWY. 17



NAVAN ROAD & HWY. 17
LOOKING NORTH
(MTC PATROL YARD RIGHT)



NAVAN ROAD & HWY. 17
LOOKING SOUTH

NAVAN ROAD/HWY. 17 INT.
PROPOSAL

MAY 8, 1986