

MEMORANDUM

23-63-116

To: Mr. H. Greenland,
District Engineer,
Hamilton, Ontario.

FROM: Foundation Section,
Materials & Testing Div.,
Room 107, Lab. Bldg.

DATE: July 20, 1964

OUR FILE REF.

IN REPLY TO

SUBJECT:

FOUNDATION INVESTIGATION REPORT

For

Slope Failure - Hwy. 403, Culvert
Sta's. 105+10 - 106+70, District 4.

W.J. 64-F-39 - W.P. ~~231-58-3~~

Cont. 63-116 140-57-1


Attached, we are forwarding to you, our detailed foundation investigation report on the subsoil conditions existing at the above site.

We believe that you will find the factual data and conclusions, as well as recommendations pertaining to remedial measures contained therein, adequate for your future needs. Should additional information be required, please do not hesitate to contact our Office.

KYL/MceF

Attach.

cc: Messrs. H. A. Tregaskes
G. K. Hunter(2)
C. C. Parker & Assoc. Ltd.
T. J. Kovich
D. M. Hopper


A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

Foundations Office
Gen. Files ✓

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FOUNDATION INVESTIGATION REPORT

For

Slope Failure - Hwy. 403, Culvert
Sta's. 105+10 - 106+70, District 4.
W.J. 64-F-39 - W.P. 231-58-3
Cont. 63-116

1. INTRODUCTION:

A request dated May 6, 1964, for a foundation investigation to determine the cause of a slope failure at the above-noted site was received from Mr. R. Schonfeld, Project Soils Engineer, Toronto Region.

2. DESCRIPTION OF SITE:

At this location, a drainage channel approx. 30 ft. wide and 10 ft. deep, was excavated along the toe of the North bank of the valley wall. In this area, the valley wall is approx. 70 ft. high with private homes located at the top. Slopes of $2\frac{1}{2} : 1$ are found along the bottom half of the wall increasing to $1\frac{1}{2} : 1$ for the upper half.

A 36-inch Ø sanitary sewer running parallel to and approx. 40 ft. to the North of the channel is located on the lower slopes of the valley wall.

cont'd. /2 ...

3. HISTORY OF FAILURE:

The first construction in the area of the failure was for the 36-inch Ø sewer which ran along the lower slopes of the North valley wall. The excavation was carried out on or about October 15, 1963. During construction the ground appeared to be relatively dry, and no sloughing of the banks was observed.

The excavation for the channel was carried out on or about February 28, 1964. The ground appeared to be reasonably dry also, and no failing of the banks occurred during the construction period.

The slope failure which was approx. 150 ft. long (Sta. 105+10 - 106+70) and 35 ft. wide at its widest point, occurred sometime during early April 1964. The top edge of the failure was found to be just South of the 36-inch Ø sewer, while at the bottom, the failure caused the bank to move into the channel excavation.

At the time of failure, the channel was in operation although still unlined, and water from the spring run-off was running through the channel.

After the failure, the channel was dammed upstream (to the West) and the water diverted into a parallel channel to the South.

4. FIELD AND LABORATORY WORK:

Three sampled boreholes supplemented by five holes employing continuous field vane testing, were performed at the site.

cont'd. /3 ...

4. FIELD AND LABORATORY WORK: (cont'd.) ...

Boreholes 1 and 2 were placed in the failure area, while B.H. #3 was placed on the bank above the failure area.

Samples were recovered at required depths using a 2" I.D. thin-walled sampler in cohesive soils, while in non-cohesive soils, a 2" O.D. split-spoon sampler was employed. The dimensions of the split-spoon sampler and the energy used in driving it, conform to the requirements of the Standard Penetration Test.

Soil samples were visually examined and classified in the field before transportation to the laboratory where a further visual classification was performed.

In addition, Atterberg limits, moisture content and unit weight determinations, grain size analyses and unconfined compression tests were conducted where applicable, on representative samples in the laboratory.

A series of consolidated undrained tests were conducted on undisturbed samples from B.H. #2 in order to determine the effective stress parameters C' and ϕ' .

The locations and elevations of all boreholes are shown on Dwg. 64-F-39A which accompanies this report.

5. SUBSOIL CONDITIONS:

5.1) General:

The subsoil at the failure site consists of a deposit of clayey silt overlying a layer of silt which is assumed to overlie shale bedrock.

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

As noted previously, the canal excavation in this area runs along the floor of the valley at the base of the North wall of the valley.

The subsoil in the valley and on the lower portion of the valley wall was found to be a clayey silt. This clayey silt extends to a depth of 20 \pm ft. below the valley floor.

A thin layer of sandy silt was found to cover the clayey-silt in B.H.'s #2 and #3, with a thin layer of silty clay covering the sandy silt in B.H. #2.

A deposit of silt was found under the clayey silt in B.H.'s #1 and #2. In B.H. #3, sampling was discontinued before reaching the silt.

The silt deposit was proved to a depth of 20 \pm in B.H. #2 where refusal was encountered. The refusal occurred at el. 242.1 (approx. 36 ft. below the valley floor), and this was assumed to be the bedrock elevation.

Field examination of bedrock outcrops near the site showed the bedrock to be red shale with seams of green calcareous shale.

5.2) Silty Clay:

This material was found only at B.H. #2 and extended from the surface to a depth of 2 $\frac{1}{2}$ ft. where a deposit of sandy silt was encountered.

One set of Atterberg Limits was conducted and the clay was found to have a Liquid Limit of 34.0%; plastic limit of 20.6%, and moisture content of 26.6%.

This clay is assumed to be spoil from the nearby sewer excavation.

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

5.3) Sandy Silt:

This material was found under the silty clay in B.H. #2 and had a thickness of $1\frac{1}{2}$ ft., while in B.H. #3 it extended from the surface to a depth of 5 ft. A grain size distribution test on a sample from B.H. #2 gave the following constituents:

Gravel	1%
Sand	27%
Silt	56%
Clay	16%

The one split-spoon test taken gave an 'N' value of 5 blows per foot, indicating the material to be in a loose state.

It would appear that this material was washed down from the upper part of the valley wall.

5.4) Clayey Silt:

This is the predominant material at the site and the strata in which the failure occurred.

B.H.'s #1 and #2 which were put in the failure zone, showed a thickness of $13\frac{1}{2}$ ft. in B.H. #1 and 24 ft. in B.H. #2. In B.H. #3 which was placed to the North and above the failure area, the clayey silt was proved to a depth of 37 ft. (el. 253.7) at which point the borehole was terminated.

Zones in the clayey silt containing thin seams of brown silty clay were encountered in all three boreholes and varied in thickness between 5.2 ft. in B.H. #1 and 1.5 ft. in B.H. #3.

cont'd. /6 ...

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

5.4) Clayey Silt: (cont'd.) ...

This layered zone was found to dip 6° N. This tilting of an originally horizontal strata indicates the possibility that a slope failure had previously occurred in this area.

Shear strengths of the clayey silt in B.H.'s #1 and #2 ranged from a low of 160 p.s.f. at the top of the deposit to a maximum of 930 p.s.f. near the bottom.

Due to the stiffness of the material in the upper part of the stratum in B.H. #3, it was necessary to take split-spoon samples. A maximum 'N' value of 23 blows per foot was recorded indicating the material to be very stiff. The shear strength tended to decrease with depth with a minimum strength of 400 p.s.f. in the lower portion of the borehole.

In all boreholes, the field vane shear strength values were found to be larger than the unconfined compression test values.

A series of consolidated undrained tests on samples from B.H. #2 gave values of C'_{cu} and ϕ'_{cu} of 145 p.s.f. and 22° , respectively for C_a ($\sigma_1 - \sigma_3$) maximum failure criterion.

The following were found to be the Atterberg Limits for the clayey silt:

	<u>Max.</u>	<u>Min.</u>	<u>Representative Value</u>
Liquid Limit	35%	24%	33%
Plastic Limit	21%	14%	17%
Moisture Content	35%	21%	31%
Unit Weight	128	107	121

cont'd. /7 ...

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

5.4) Clayey Silt: (cont'd.) ...

It should be noted that in nearly all tests the moisture content was at or near the liquid limit.

In the thin layers of brown silty clay, it was possible to obtain moisture contents only. Values ranged from 35% to 52% with a value of 44% being considered representative. Visual examination showed this material to have a blocky type structure.

5.5) Silt:

The silt was encountered in B.H.'s #1 and #2, but was not encountered to the depth sampled in B.H. #3. 'N' values ranged from 10 to 43 blows per foot, indicating the material to be in a compact to dense state.

A grain size distribution test on a sample from B.H. #2 gave the following constituents:

Gravel	3%
Sand	19%
Silt	66%
Clay	12%

One set of Atterberg Limits was taken and gave the following values: Liquid limit = 26.9; Plastic limit = 14.9; Moisture content = 17.6%. However, these values are not felt to be truly representative of the material as a whole.

cont'd. /8 ...

6. GROUND WATER CONDITIONS:

Examination of the site at the time of the field investigation disclosed several areas of wet ground in the failure area. Further investigation showed the source of this water to be from one or more outcropping water-bearing sand and gravel seams approx. 9 inches thick, which are exposed on the upper slopes of the valley wall. A rough measurement of the flow was 16 ozs. ⁺ in 3 min.

To determine the water conditions more accurately, Norwegian Piezometers were installed beside B.H.'s #1, #2, and #3.

The locations and depths of these piezometers are shown on Dwg. 64-F-39A.

In the same figure the observed water levels are also plotted. It is seen that the water levels recorded by the piezometers within the failure area decrease slightly towards the toe of the slope.

It can be seen that an artesian condition exists in the area of B.H. #1 with respect to the ground surface before failure occurred.

Measurements on May 27 showed the water level to be at the surface in B.H.'s #1 and #3 and 2 ft. below the surface in B.H. #2. This indicates that in effect, the water table was at the surface throughout the whole slope.

Measurements on June 30 showed the water table to be at - 1.5 ft. in B.H. #1, - 2.5 ft. in B.H. #2 and - 3.5 ft. in B.H. #3.

This decrease in the water table is to be expected since the weather has been dry and little rainfall has occurred between May 27 and June 30.

cont'd. /9 ...

6. GROUND WATER CONDITIONS: (cont'd.) ...

Since the failure occurred in the spring (early April) where there is generally the highest water table due to spring run-off combined with traditionally wet weather, it can be safely assumed that at the time of failure, the water table was at or above the ground surface.

7. DISCUSSION AND RECOMMENDATIONS:

Analyses of the slope failure indicates that the failure was probably of a rotational type. The geometry of the slip area tends to confirm this.

The cause of the failure can be attributed to an extremely high water table during the spring plus the saturation and probable softening of the slopes due to water flowing from seepage zones outcropping on the upper banks.

Analysis of the slip on a total stress basis, assuming an average shear strength of 400 p.s.f., gave a factor of safety of unity which indicates the bank was on the verge of failure from the time of construction. Consequently, the saturation of the slopes during the spring probably increased the unit weight of the material slightly, but still enough to cause failure.

Analysis of the slip on an effective stress basis, assuming the water table at the surface and using a C'_{cu} and ϕ'_{cu} of 145 p.s.f. and 22° , respectively, gave a F.S. of the order of 0.85.

cont'd. /10 ...

7. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

The recommendations for reconstruction are as follows:

7.1) In the vicinity of the failure area a new channel should be re-excavated with the existing centre line moved approx. 10 ft. to the South.

Since the failure slopes are now stable, this move will eliminate any instability caused by excavating the toe of the slide where it has partly filled in the existing channel.

The new channel should be constructed as per the design cross-section; however, the upper banks should be cut back to $2\frac{1}{2} : 1$ slopes during construction. (See Dwg. 64-F-39B.)

During reconstruction of the channel, all soft material should be removed from the slopes and the area should be backfilled to the top of the channel lining and then brought back horizontally to intersect the existing ground as shown in the drawing.

The backfilling should be done as soon as possible after excavation in order to increase the bank stability.

7.2) If it is not desired to shift the centre line, the open channel should be replaced by a box culvert in the vicinity of the failure area. The culvert should be placed on spread footings with a contact pressure not to exceed 0.5 t.s.f.

For construction purposes, the existing bank should be cut back on $2\frac{1}{2} : 1$ slopes extending back to approx. 10 ft. from the South edge of the sewer excavation and then slope up to meet existing ground (See Dwg. 64-F-39B.) The area should be backfilled to the top of the culvert and then brought back horizontally to intersect the existing ground.

Backfilling should be done as soon as possible after excavation in order to increase bank stability.

cont'd. /11 ...

7. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

7.3) The spoil piled on the bank above the channel in the area of Sta's. 103+50 ⁺ to 105+10 is endangering the stability of the channel bank in this area. It is recommended that the spoil be removed as soon as possible and the bank be graded to the design cross-section.

8. SUMMARY:

1. The slope failure was caused by a Spring high water table plus saturation of the slope caused by seepage zones located on the upper slopes.
2. The channel should be reconstructed with the centre line shifted 10 ft. to the South through the failure zone. If it is desired to keep the present centre line, a box culvert should replace the open channel through the failure zone.
3. The spoil piled on the bank above the channel between Sta's. 103+50 ⁺ to 105+10 should be removed.

9. MISCELLANEOUS:

The field work performed during the period from May 12 - 26, 1963, was done by Messrs. G. G. Cherrington and P. Payer, Project Foundation Engineers. The preparation of this report was undertaken by Mr. G. G. Cherrington.

The investigation was carried out under the general supervision of Mr. K. G. Selby, Senior Foundation Engineer, who also reviewed this report.

July 1964

APPENDIX I.

ABBREVIATIONS USED IN THIS REPORT

PENETRATION RESISTANCE

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

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

DESCRIPTION OF SOIL

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

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

TYPE OF SAMPLE

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

SOIL TESTS

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

ABBREVIATIONS USED IN THIS REPORT

SOIL PROPERTIES

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

GENERAL

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

STRESS AND STRAIN

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

EARTH PRESSURE

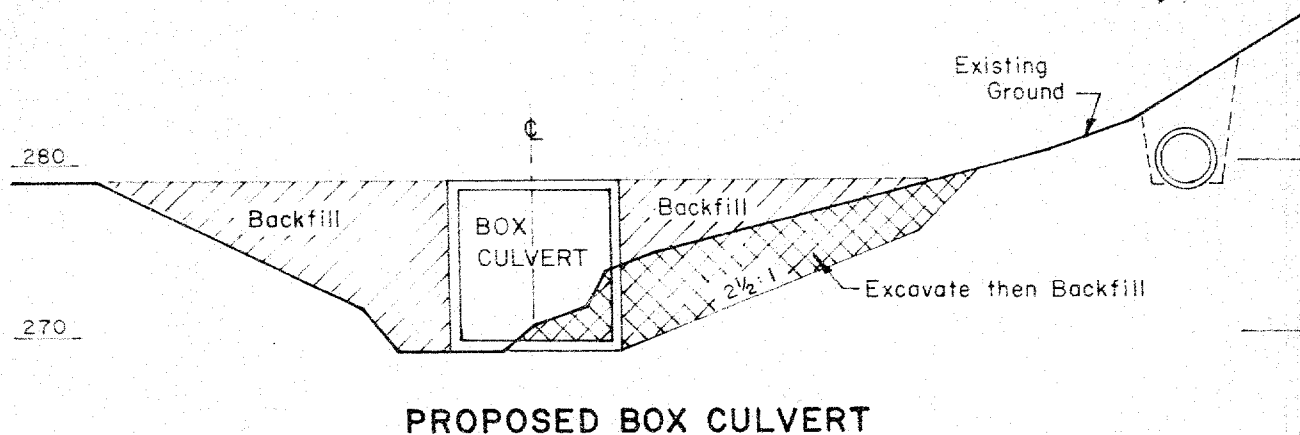
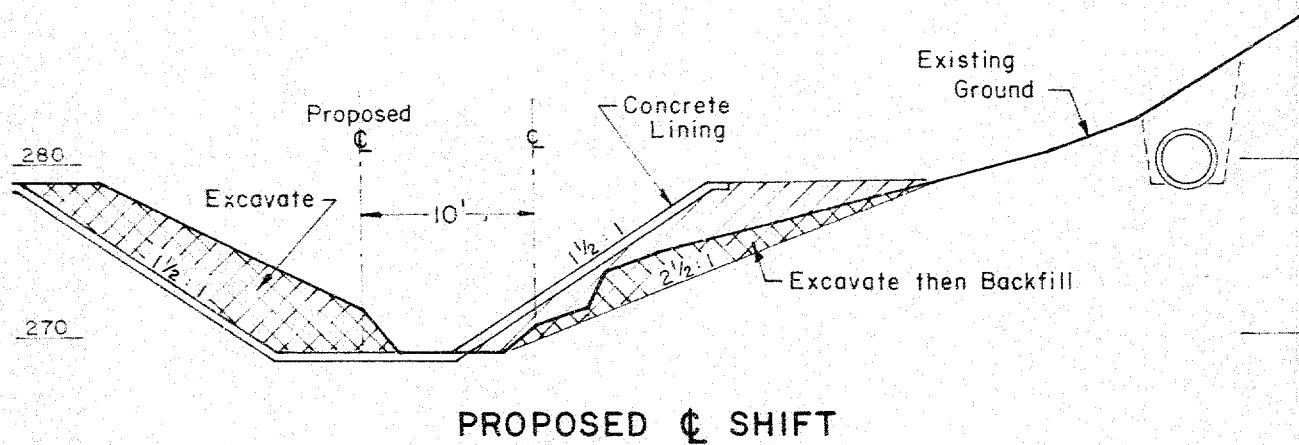
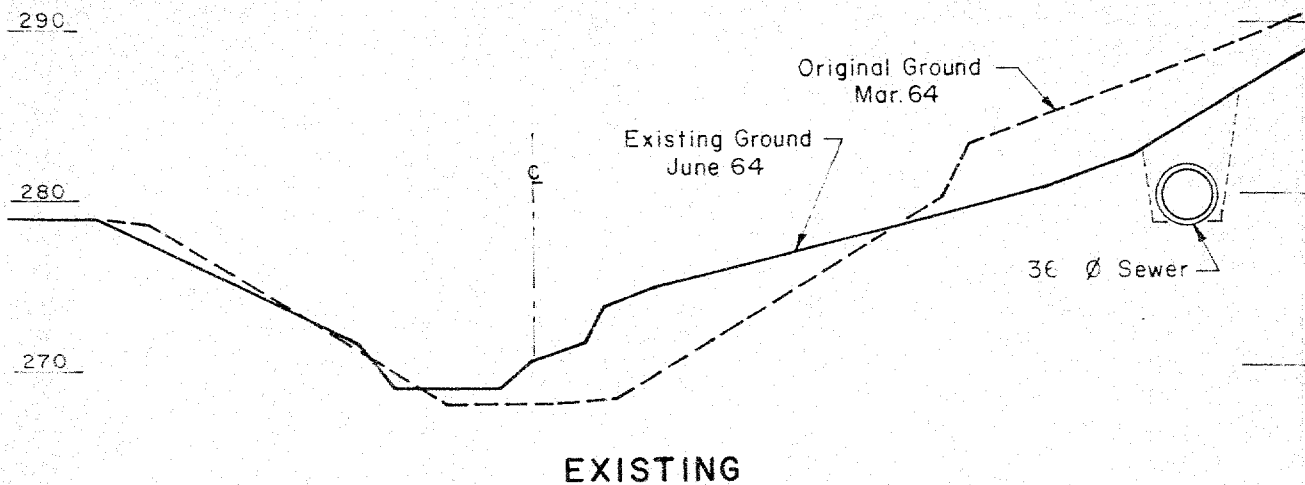
d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_0	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

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

SLOPES

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



TYPICAL
CHANNEL CROSS - SECTIONS
STA. 105+00± TO STA. 106+70±
SCALE: 1" = 10'

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB 64-P-39 LOCATION Hwy. 403 Sta. 105+93.10' Rt. ORIGINATED BY G.C.
W.P. 231-58-3 BORING DATE May 12-13, 1964. COMPILED BY G.C.
DATUM Geodetic 274.55 BOREHOLE TYPE Washboring & Vane Hole CHECKED BY K.S.

SOIL PROFILE			SAMPLES		DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					FLUID LIMIT - PL PLASTIC LIMIT - PI WATER CONTENT - W			REMARKS	
ELEV DEPTH	DESCRIPTION	STRAT. PLCT	NUMBER	TYPE	BLWS / FOOT	SHEAR STRENGTH P.S.F. o - Unconfined Comp. + - Field Vane					WATER CONTENT % 20 40 60			
274.55 0.0	<u>Clayey silt, firm to very soft.</u>		1	TW	PM	3.7	3.7	3.4						123
	Gray,		2	TW	PM	270	4.4	8.3						118
	with layers of brown silty clay between El. 269.5 and El. 264.3		3	TW	PM		4.4	5.6						116
			4	TW	PM		4.0	5.0						125
			5	TW	PM		9.3	8.5						121
			6	TW	PM		5.8	6.0						118
			7	TW	PM		1.6	1.5						
261.05			8	SS	5									
13.5	<u>Silt, dense</u>													
258.05	Gray		9	SS	36									
16.5	End of borehole.													

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

JOB 64-F-39 LOCATION Hwy. 403 Sta. 105+93 32' Rt. ORIGINATED BY G.C.
W.P. 231-58-3 BOREHOLE DATE May 14 - 21, 1964. COMPILED BY G.C.
DATUM Geodetic 281.17 BOREHOLE TYPE Washboring & Vane Hole CHECKED BY K.S.

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	STRAT. PLCT	SAMPLES			ELEV. SOIL	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	SHEAR STRENGTH P.S.F. o - Unconfined Comp. + - Field Vane	LIQUID LIMIT PLASTIC LIMIT WATER CONTENT	BULK DENSITY	REMARKS
			NUMBER	TYPE	BLOWS / FOOT						
281.17	Groundlevel										
	<u>Silty clay.</u>					280					
278.7			1	TW	PM		5.2 8.5				
277.7	<u>Sandy silt.</u>		2	TW	PM		3.1 2.6				
4.0			3	TW	PM		4.0 3.2				
	<u>Clayey Silt,</u> <u>Very soft to med.</u> <u>stiff, gray with</u> <u>occasional pockets</u> <u>and seams of gray</u> <u>silt.</u>		4	TW	PM		3.3 2.7				
			5	TW	PM		2.4 2.8				
			6	TW	PM		2.9 3.4				
			7	TW	PM	270	5.3				
			8	TW	PM		4.5 3.7				
			9	TW	PM		3.1 3.1				
	<u>With layers of</u> <u>brown silty clay</u> <u>between El. 267.2</u> <u>& El. 262.7</u>		10	TW	PM		10.0 10.7				
			11	TW	PM		8.0 9.4				
			12	TW	PM		5.8 5.3				
			13	TW	PM	260	5.4 6.0				
			14	TW	PM		3.3 5.7				
257.17			15	TW	PM						
24			16	SS	43						
	<u>Silt, dense to</u> <u>compact, gray.</u>		17	SS	40						
			18	SS	39	250					
			19	SS	29						
			20	SS	10						
242.07			21	SS	40/7"						
39.1	<u>Shale Bedrock</u> <u>(Assumed)</u> <u>End of borehole.</u>					240					

Gr 3%
Sa 19%
Si 66%
Cl 12%

DEPARTMENT OF HIGHWAYS - TESTING
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 3

FOUNDATION SECTION

JOB 64-F-39 LOCATION Hwy. 403 Sta. 105+93 61' Rt. ORIGINATED BY G.C.
 W P 231-58-3 BORING DATE May 21-26, 1964. COMPILED BY G.C.
 DATUM Geodetic 295.94 BOREHOLE TYPE Washboring & Vane Hole CHECKED BY K.S.

SOIL PROFILE		SAMPLES		ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT _____ PLASTIC LIMIT _____ WATER CONTENT _____		BULK DENSITY	REMARKS	
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE		SHEAR STRENGTH P.S.F. o - Unconfined Comp. + - Field Vane		WATER CONTENT %				
295.94	Groundlevel			295	250	500	750	1000	1250		▼295.94 0.0
0.0	Sandy silt, loose, brown.										
290.94		1	SS	5							
5.0	Clayey silt very stiff to soft, gray, with occasional pockets and seams of gray silt.	2	SS	23							
		3	SS	14							
		4	SS	12							
		5	SS	4							
		6	TW	PM							126
		7	TW	PM							120
		8	TW	PM							121
		9	TW	PM							121.5
		10	TW	PM							123.5
		11	TW	PM							121.5
		12	TW	PM							121.5
		13	TW	PM							122
		14	TW	PM							123
		15	TW	PM							124
261.44		16	TW	PM							128.5
34.5	With layers of brown silty clay.	17	TW	PM							123
259.94		18	TW	PM							119
36.0	Clayey silt, soft to stiff, gray.	19	TW	PM							120
		20	TW	PM							120
		21	TW	PM							115
253.69		22	TW	PM							130
42.25	End of borehole.										

15 0 5 % Strain at Failure
10

MEMORANDUM

To: Mr. T. J. Kovich,
Regional Materials Engr.,
Toronto.

FROM: Foundation Section,
Materials & Research Div.,
Room 107, Lab. Bldg.

Attn: Mr. R. Schonfeld

DATE: June 12, 1964

OUR FILE REF.

IN REPLY TO

SUBJECT:

Stability Problems - Culvert
Sta. 105+10 to Sta. 106+70,
Cont. 63-116 - W.J. 64-F-39
District No. 4

At the request of Mr. R. Schonfeld of the Soils Section, this Section has recently completed an investigation to determine the cause of a slope failure at the above site.

This is a preliminary report outlining recommended reconstruction procedures and will be followed in due time by a detailed report.

The failure took place along the side of a drainage channel which was in a cut section through soft to medium stiff clayey silt. Preliminary analysis indicates the stability of the bank prior to failure was approx. unity.

Our recommendations for reconstruction are as follows:

- (1) In the vicinity of the failure area a new channel should be re-excavated with the existing centre line moved approx. 10 ft. to the south.

Since the failure slopes are now stable, this move will eliminate any instability caused by excavating the toe of the slide where it has partly filled in the existing channel.

The new channel should be constructed as per the design cross-section; however, the upper banks should be cut back to 2½:1 slopes, during construction. (See Drawing)

After reconstruction of the channel, all soft material should be removed from the slopes and the area should be backfilled to the top of the channel lining and then brought back horizontally to intersect the existing ground, as shown in the drawing.

The backfilling should be done as soon as possible after excavation in order to increase the bank stability.

cont'd. /2 ...

June 12, 1964

- (2) If it is not desired to shift the centre line, the open channel should be replaced by a box culvert in the vicinity of the failure area. The culvert should be placed on spread footings with a contact pressure not to exceed 0.5 T.S.F.

For construction purposes, the existing bank should be cut back on 2½:1 slopes extending back to approx. 10 ft. from the south edge of the sewer excavation and then sloped up to meet existing ground (See Drawing).

The area should be backfilled to the top of the culvert and then brought back horizontally to intersect the existing ground.

Backfilling should be done as soon as possible after excavation in order to increase bank stability.

- (3) The spoil piled on the bank above the channel in the area of stations 103+50 \pm to 105+10 is endangering the stability of the channel bank in this area. It is recommended that the spoil be removed as soon as possible and the bank be graded to the design cross-section.

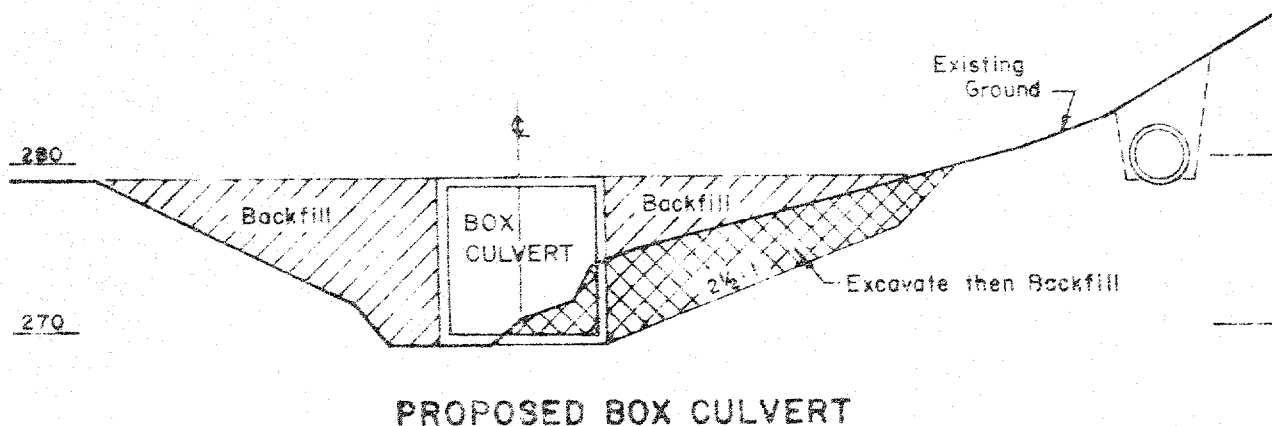
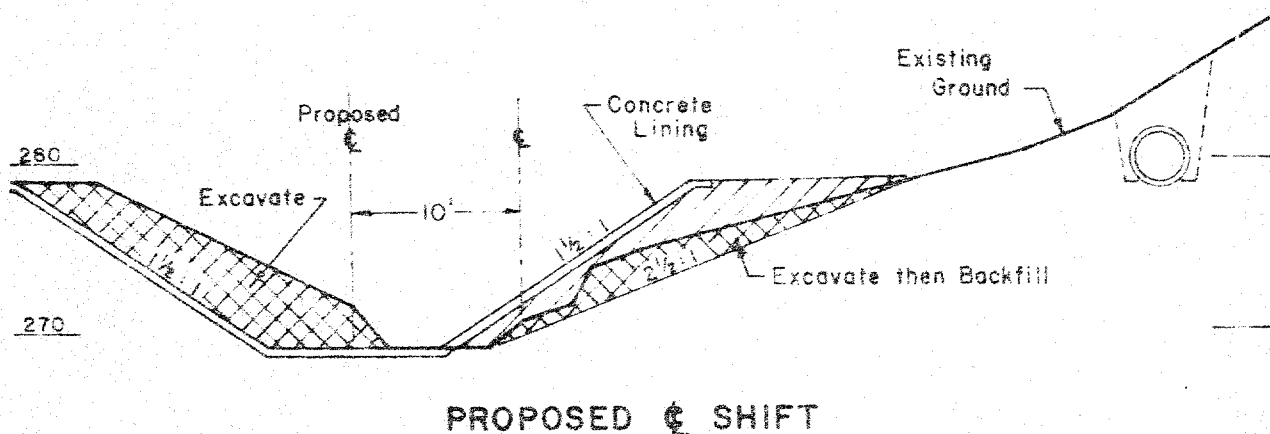
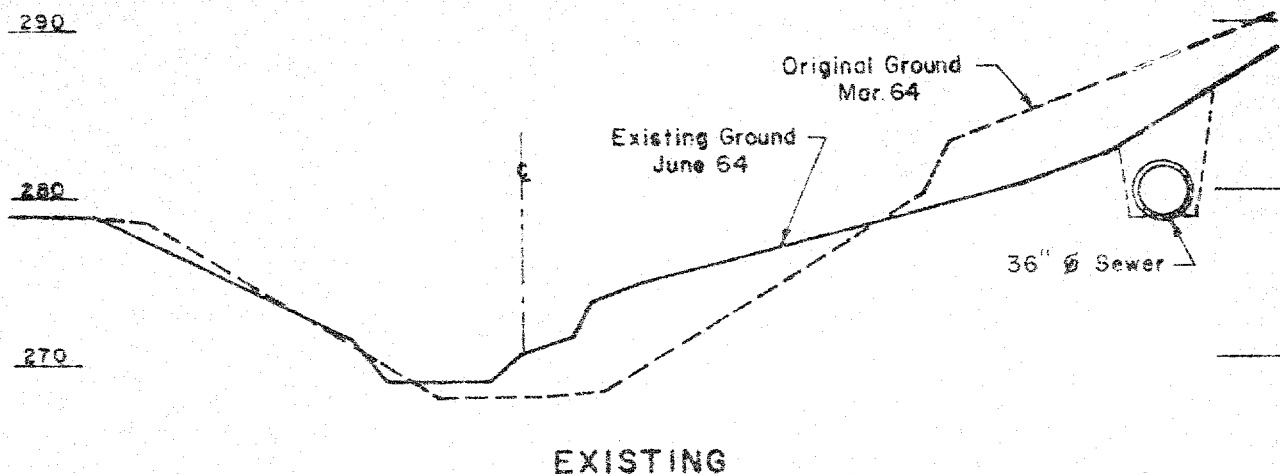
If any questions arise concerning this report, please feel free to contact this Section.

GGC/MdeF
Attach.

cc: Messrs. H. A. Tregaskes
H. D. McMillan
G. K. Hunter (2)
H. Greenland
D. M. Hopper

Foundations Office
Gen. Files

G. G. Cherrington
G. G. Cherrington,
PROJECT FOUNDATION ENGINEER
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER



260

TYPICAL
CHANNEL CROSS - SECTIONS
STA. 105+00 \pm TO STA. 106+70 \pm
SCALE: 1" = 10'

Mr. T. J. Kovich,
Regional Soils Engr.,
Room 134-A, Lab. Bldg.

Attn: Mr. R. Schonfeld

Mr. A. G. Stermac,
Principal Foundation Engr.,
Foundation Section,
Room 107, Lab. Bldg.

January 13, 1964

Proposed Channel Cut - Sta. 453+00 to 460+00,
Hwy. #403, Hamilton, Ontario. - W.P. 140-57-1.

64-F-1

A meeting was held on January 6, 1964, in connection with the above project. Present were Messrs. A. G. Stermac, T. J. Kovich, K. Y. Lo, R. Schonfeld and K. G. Selby.

Following is a summary of the conclusions reached at the meeting, together with a review of the soil conditions determined to date during our present field investigation:

(1) Soil Conditions:

Six borings have been carried out covering the full length of the proposed retaining wall and located some 10 to 15 feet in front of it. The borings revealed that the upper 10 ft. consists of layers of silt, clayey silt, sand and gravel, with the predominant material being silt. The relative density of the material was found to vary from loose to compact. Below 10 ft., the soil generally consists of denser glacial deposits. The excavation for the proposed channel will be to a depth of about 10 ft. below existing ground level.

(2) Ground Water Conditions:

At the borehole locations, the ground water level (which stabilized rapidly), was found to vary from 1.5 to about 2.5 feet below ground level. Seepage zones could be observed in the face of the existing cut which has been carried out behind the proposed wall, generally 2 - 3 ft. above ground level and in isolated cases, higher.

(3) Stability of Cut Section & Retaining Wall:

The retaining wall, as presently proposed, is located on the lower half of what would be otherwise a continuous $1\frac{1}{2} : 1$ slope. It therefore contributes only to the instability of the section. Since the

cont'd. /2 ...

January 13, 1964

(3) Stability of Cut Section & Retaining Wall: (cont'd.) ...

predominant subsoil material is silt in a loose to compact state, a continuous $1\frac{1}{2} : 1$ slope would be expected to have a safety factor slightly greater than 1.0, provided that the ground water can be controlled so as not to seep out of the surface of the finished slope. Even if these conditions are realized during the normal process of construction, we do not consider this to be an entirely satisfactory engineering solution. Since it seems that additional property cannot be obtained in order to construct a more favourable slope, we can only suggest steps to be taken to improve the present design. These are as follows:

(a) The proposed retaining wall should be dispensed with and replaced with a cut-off trench filled with a suitable filter material. The trench should be located along the centreline of the proposed wall alignment and excavated to a depth of about 2 ft. above the proposed channel base. The width should be not less than 3.0 ft. Drainage can be effected into the channel by means of lateral drains. The finished $1\frac{1}{2} : 1$ slopes should be covered with a suitable filter material not less than 1.0 ft. thick.

(b) Whether or not the wall is removed, as large a berm as is possible should be constructed between the channel top and the remaining slope. The presently proposed distance of 1 ft. between the channel and the retaining wall face is considered to be most unsatisfactory.

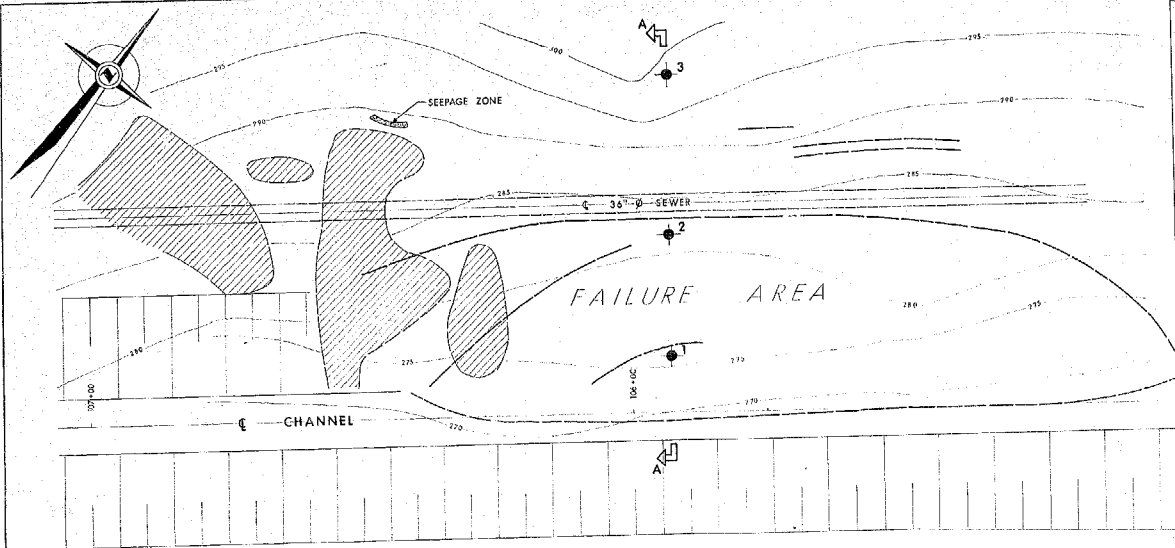
Since the stability of the section is dependent on a number of uncertainties, the most important being the behavior of the ground water, we believe that the possibility of a failure must be presumed to exist. Without the retaining wall, such a failure would be less 'catastrophic' and could probably be more easily remedied. This would seem to us to be a further argument in favour of dispensing with the wall.

Please keep in touch with us regarding further developments and if you have any further queries, do not hesitate to contact this Office.

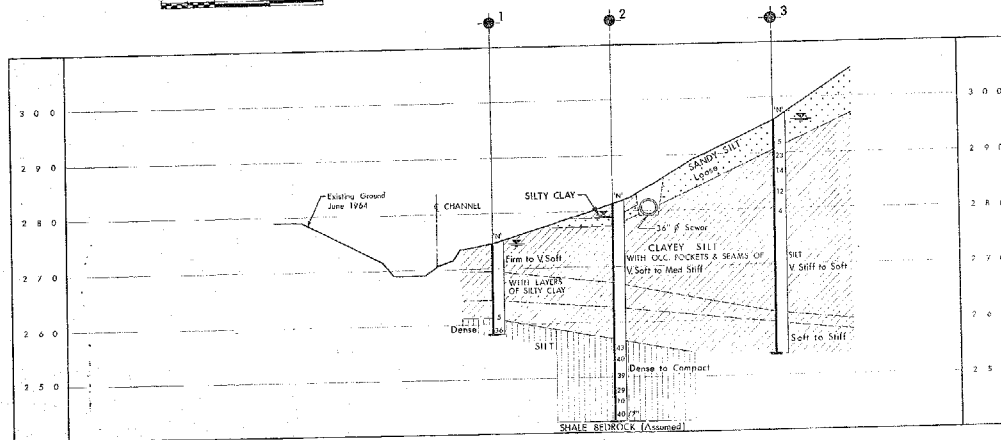
KGS/MdeF

cc: Foundations Office
Gen. Files

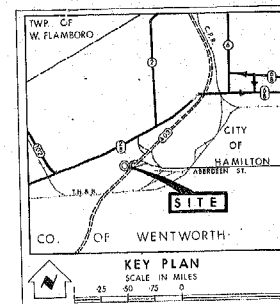

K. G. Selby,
SENIOR FOUNDATION ENGINEER



PLAN
SCALE
10 5 0 10 20 FT



SECTION A-A
SCALE
10 5 0 10 20 FT



E 288° 00'
N 99° 12' 00"
2:17

LEGEND

- Bore Hole
- ⊙ Cone Penetration Hole
- ⊙ Bore & Cone Penetration Hole
- Water levels established at time of field investigation (May 27, 1964)
- Wet Area
- Tension Cracks

NO.	ELEVATION	STATION	OFFSET
1	274.55	105 + 93	10' RT.
2	281.17	105 + 93	32' RT.
3	295.94	105 + 93	61' RT.

NOTE

THE INFORMATION CONTAINED ON THIS DRAWING WAS OBTAINED FROM THE FIELD INVESTIGATION AND IS NOT TO BE USED FOR ANY OTHER PURPOSES WITHOUT THE WRITTEN PERMISSION OF THE ENGINEER. THE ENGINEER ASSUMES NO LIABILITY FOR ANY DAMAGE OR INJURY TO PERSONS OR PROPERTY ARISING FROM THE USE OF THIS DRAWING.

DEPARTMENT OF HIGHWAYS - ONTARIO
HAMILTON & W. FLAMBORO TOWNSHIP

CHANNEL BANK FAILURE CHEDoke EXPRESSWAY

SHOWING POSITIONS & IDENTIFICATION OF BOREHOLE

AREA 403	STATION 4	COUNTY WENTWORTH
TOWNSHIP	CITY OF HAMILTON	LOT
CONTRACT NO.	W. 238-28-3	DATE 15 JULY 1964
DESIGNED BY	ENGINEER	DATE 15 JULY 1964
CHECKED BY	DATE 15 JULY 1964	DATE 15 JULY 1964
APPROVED BY	DATE 15 JULY 1964	DATE 15 JULY 1964

K. J. L. 464-F-39 A
1/1/70-52-1