

GEOCRES No. 40J16-49DIST. 1 REGION W.P. No. CONT. No. W. O. No. 73-11082STR. SITE No. HWY. No. 21LOCATION BEAR CREEK FAILURENo of PAGES -OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.REMARKS:

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

TO: Mr. J. G. Forster, (2)
Senior Soils Engineer,
Southwestern Region,
London, Ontario.

FROM: Foundations Office,
Design Services Branch,
West Bldg., Downsview.

ATTENTION:

DATE: December 31, 1973.

OUR FILE REF.

IN REPLY TO JAN - 4 1974

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For
Bear Creek Bank Failure
Petrolia Bypass (Hwy. #21)
Twp. of Enniskillen, Co. of Lambton
District #1 (Chatham)
W.O. 73-11082

Attached we are forwarding to you our foundation investigation report on the subsoil conditions existing at the above-mentioned site, the cause of the bank failure and the remedial measures to stabilize the slope.

We believe that the factual data and recommendations contained herein will prove adequate for your requirements. Should additional information be required, please do not hesitate to contact our Office.

AP/ao
Attch.
c.c. W. Katarynczuk
A. P. Watt
A. Rutka
A. Wittenberg
J. R. Roy


A. G. Stermac,
PRINCIPAL FOUNDATIONS ENGINEER.

Foundations Files
Documents

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FOUNDATION INVESTIGATION REPORT
For
Bear Creek Bank Failure
Petrolia Bypass (Hwy #21)
Twp. of Enniskillen, Co. of Lambton
District #1 (Chatham)
W.O. 73-11082

1. INTRODUCTION:

In response to a request from Mr. J. G. Forster, Senior Soils Engineer, Southwestern Region, a foundation investigation has been carried out by the Foundations Office at the above mentioned failure location. The purpose of this investigation is to determine the subsoil conditions existing at the site, the causes of the bank failure and to suggest remedial measures.

2. DESCRIPTION OF THE SITE:

The site of the failure is located at the north limits of Petrolia. At this place, the Bear Creek has been diverted parallel to and approximately 120 ft. east of Hwy #21. East bank of the creek is more than 30 ft. high and rises at about 3 horizontal to 1 vertical slope. Some signs of instability of the slope were noticed some two years ago and recommendations were given by this Office to flatten the east bank of the river, back to the fence line, to 3 horizontal to 1 vertical by placing sufficient rock fill in order to achieve this. Presumably these recommendations were followed but this Office was never actually notified. The present failure took place in fall of 1973 at a place where the east

bank is the steepest. The failed area is about 135 ft. wide near the bottom of the slope and extends to a vertical height of approximately 25 ft.

3. FIELD AND LABORATORY INVESTIGATION:

A total of two sampled boreholes were carried out near the crest of the failed area. Disturbed samples were obtained in Borehole #1 by means of a standard 2 inch O.D. split spoon sampler; the energy in driving it conformed to the requirements of the Standard Penetration Test. Undisturbed samples were recovered in Borehole #2 using 2 inch I.D. Shelby tubes which were pushed into the soil hydraulically. Field vane tests were attempted 10 times from ground surface to 45 ft. depth, but it was not possible to turn the vane even once.

All boreholes and cross-sections were surveyed in the field by personnel from Chatham District Construction Surveys Section. The locations and elevations of the borings are shown on Drawing No. 73-11082A which accompanies this report.

All borehole samples were subjected to a careful visual examination and classification in the field and subsequently in the laboratory. Following this inspection, laboratory tests were carried out on selected samples to determine the following physical properties:

- Atterberg Limits
- Moisture Content
- Grain Size Distribution
- Bulk Density
- Unconfined Undrained Shear Strength
- Effective Stress Parameters

The results of the field and laboratory tests are summarized on the Record of Borehole Sheets contained in the Appendix of this report.

4. SUBSOIL CONDITIONS:

Except for a 2 ft. thick layer of silt with sand and clay at the surface, the subsoil consists of silty clay with traces of sand and gravel. The upper 11 ft. of this stratum is brown in colour and the rest is grey.

Tests for Natural Moisture Content and Atterberg Limits indicate the following properties:

Liquid Limit	38 - 47%
Plastic Limit	19 - 24%
Natural Moisture Content	16 - 28%

It was not possible to turn the field vane, indicating that the field vane shear strength everywhere is more than 2,000 psf. Unconfined compression tests carried out in the laboratory gave erratic results indicating undrained shear strength between 1165 and 3000 psf.

Three consolidated undrained triaxial tests with pore pressure measurements were carried out to determine the effective stress parameters. The results of the tests indicate the following values for the parameters.

$$\begin{aligned}C' &= 0 \\ \phi' &= 27^\circ - 28^\circ \\ \gamma &= 124 - 127 \text{ p.c.f.}\end{aligned}$$

No water was observed in either borehole, because of the short duration of field work and relatively low permeability of the subsoil.

5. DISCUSSION AND RECOMMENDATIONS:

Stability analyses have been carried out in terms of effective stresses to determine the factor of safety against long term failure. The following parameters have been used for Bishop and Morgenstern method of analysis.

$$\begin{aligned}C' &= 0 \\ \phi' &= 27.5^\circ\end{aligned}$$

If the slope was perfectly dry (i.e. $r_u = 0$) it would have a factor of safety of 1.56. However, the bank has failed indicating that a factor of safety of 1.0 was reached. This is obtained by using an r_u value of 0.32 in the Bishop and Morgenstern method. This points out that the instability of the slope was caused by a build up of pore pressures in the soil mass as a result of ground water seepage.

Another contributing factor to the instability is the erosion of the toe. This removes the support at the bottom and produces locally steep slopes, which are inherently unstable. This results in progressive failure of the slope. The most critical period for these failures is the spring thaw of every year and also during the period of heavy precipitation.

In order to stabilize the present failure and to prevent further failures the following remedial measures are suggested:

That portion of the slope which lies below the fence line should be built up to a 3 horizontal to 1 vertical slope using rock fill. The rock fill should extend into the creek as shown on Drawing 73-11082A. This procedure will reinforce the toe of the slope and prevent its erosion. This treatment should extend between stations 513 + 80 and 515 + 60 and should be tapered at both ends for a distance of about 25 ft.

The portion of the slope above the fence should be covered with 1 ft. thick granular blanket. The upper limit of the blanket should be 5 - 10 ft. beyond the top of the failed area. This will help carry surface run off into the creek and prevent its seepage into the underlying soil mass.

As a further precaution a cut-off drain about 8 ft. deep and 2 ft. wide should be installed on top of the slope about 20 ft. beyond the crown or crest of the present failed area. This drain should be backfilled with free-draining granular material and should discharge into the creek. This

will reduce the inflow of subsurface water into the failed area.

As mentioned above, the time of spring thaw is a critical period for failures, therefore, no remedial measures should be carried out before that. It is suggested that this Office should inspect the slope before repairs are undertaken. At that time we may modify or confirm recommendations contained in this report, dependent upon the conditions then prevailing.

6. MISCELLANEOUS:

The field investigation was carried out from September 28th to October 1st, 1973 under the direction of Mr. P. Korgemagi, Project Foundation Engineer. The entire project was under the supervision of Mr. Anand Prakash, Senior Foundation Engineer, who also prepared this report. This report was reviewed by Mr. K. G. Selby, Supervising Foundation Engineer.

The equipment used was owned and operated by Master Soil Investigation Ltd.

A. Prakash

A. Prakash, P. Eng.,



K. G. Selby

K. G. Selby, P. Eng.,

AP/ji
Dec. 28, 1973.

APPENDIX I

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 1

JOB 73-11082 LOCATION Sta. 514 + 69 228' Rt. 6 ORIGINATED BY PK
 W.P. BORING DATE September 28th to October 1st, 1973 COMPILED BY PK
 DATUM Geodetic BOREHOLE TYPE Auger CHECKED BY W.J.

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT				LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w w_p — w — w_L WATER CONTENT %			BULK DENSITY γ P.C.F.	REMARKS GR. SA. SI. CL.
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE								
651.9	Ground Level												20 40 60		
0.0 649.9 2.0	Silt with sand&clay					650									0 3 43 54
	Brown Grey Silty clay, Traces of sand and gravel. Very stiff		1	SS	21	Dry Hole							○		
			2	SS	17								○	—	
			3	SS	17										
			4	SS	18								○		
			5	SS	18	640							○	—	
			6	SS	10										
			7	SS	11								○		2 9 53 36
			8	SS	15										
			9	SS	12										
			10	SS	14	630							○	—	
			11	SS	20										
			12	SS	15								○	—	0 4 43 53
			13	SS	15										
			14	SS	14								○		
			15	SS	16	620									
			16	SS	26								○	—	0 10 60 30
			17	SS	21										
			18	SS	17								○		
			19	SS	19										
							610								
			20	SS	25								○	—	
						600									
596.4			21	SS	27								○		
55.5	End of Borehole					590									WL not established

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE N^o 2

JOB 73-11082 LOCATION Sta. 514 + 92 227' Rt. C ORIGINATED BY PK
W.P. BORING DATE October 1st, 1973 COMPILED BY PK
DATUM Geodetic BOREHOLE TYPE Auger CHECKED BY D.F.

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	LIQUID LIMIT ——— w_L PLASTIC LIMIT ——— w_p WATER CONTENT ——— w	BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	WATER CONTENT % $w_p \quad w \quad w_L$		
							1000 2000	20 40 60		
651.0	Ground Level									
0.0	Silt with sand & clay					650				
649.0						Dry Hole				
2.0							> 2000			
						640	> 2000			
	Brown Grey		1	TW	PH		> 2000			
			2	TW	PH	630	> 2000			
	Silty clay, Traces of sand and gravel Very stiff		3	TW	PH		> 2000			
			4	TW	PH	620	> 2000			
			5	TW	PH		> 2000			
			6	TW	PH		> 2000			
			7	TW	PH	610	> 2000			
							> 2000			
599.5			8	SS	37	600				
51.5	End of Borehole									
						590				

OFFICE REPORT ON SOIL EXPLORATION

ABBREVIATIONS & SYMBOLS USED IN THIS REPORTPENETRATION RESISTANCE

'N'=STANDARD PENETRATION RESISTANCE : - 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>c LB./SQ.FT.</u>	<u>DENSENESS</u>	<u>'N' BLOWS / FT.</u>
VERY SOFT	0 - 250	VERY LOOSE	0 - 4
SOFT	250 - 500	LOOSE	4 - 10
FIRM	500 - 1000	COMPACT	10 - 30
STIFF	1000 - 2000	DENSE	30 - 50
VERY STIFF	2000 - 4000	VERY DENSE	> 50
HARD	> 4000		

TERMS TO BE USED IN DESCRIBING SOILS:-

TRACE < 10% , SOME 10-25% , WITH 25-40% , > 40% SILTY, SANDY, GRAVELLY, CLAYEY ETC.

TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.T.	SLOTTED TUBE SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE

P.H. SAMPLE ADVANCED HYDRAULICALLY

P.M. SAMPLE ADVANCED MANUALLY

SOIL TESTS

U	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
UU	UNCONSOLIDATED UNDRAINED TRIAXIAL	F.V.	FIELD VANE
CIU	CONSOLIDATED ISOTROPIC UNDRAINED TRIAXIAL	C	CONSOLIDATION
CID	" " DRAINED "	S	SENSITIVITY
CAU	" ANISOTROPIC UNDRAINED "		
CAD	" " DRAINED "		

ABBREVIATIONS & SYMBOLS USED IN THIS REPORTSOIL 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
w_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
σ'	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
ϵ	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO (μ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

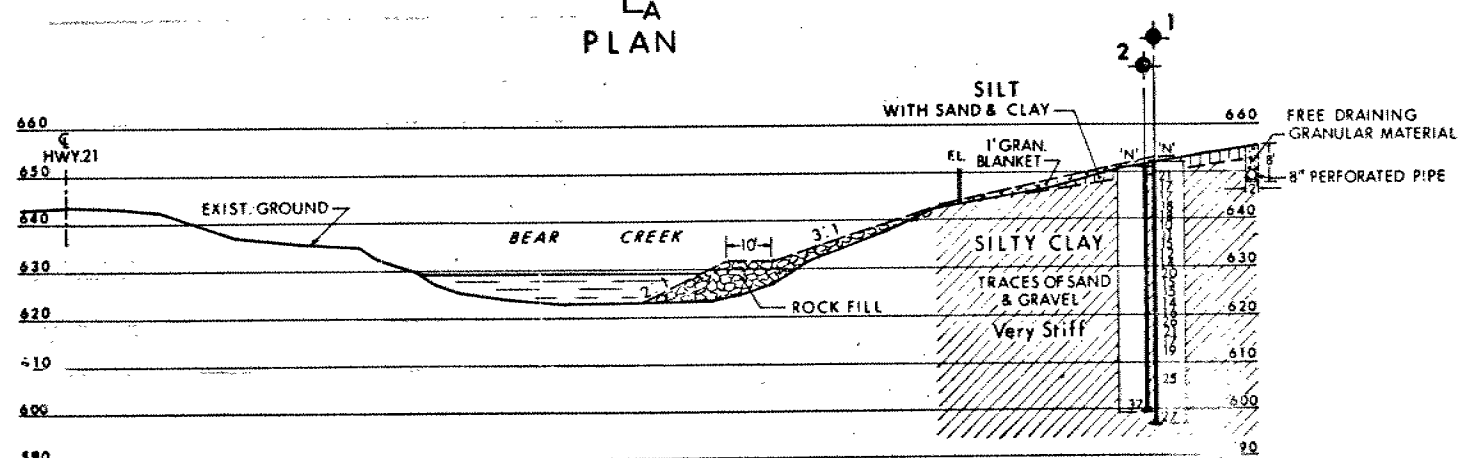
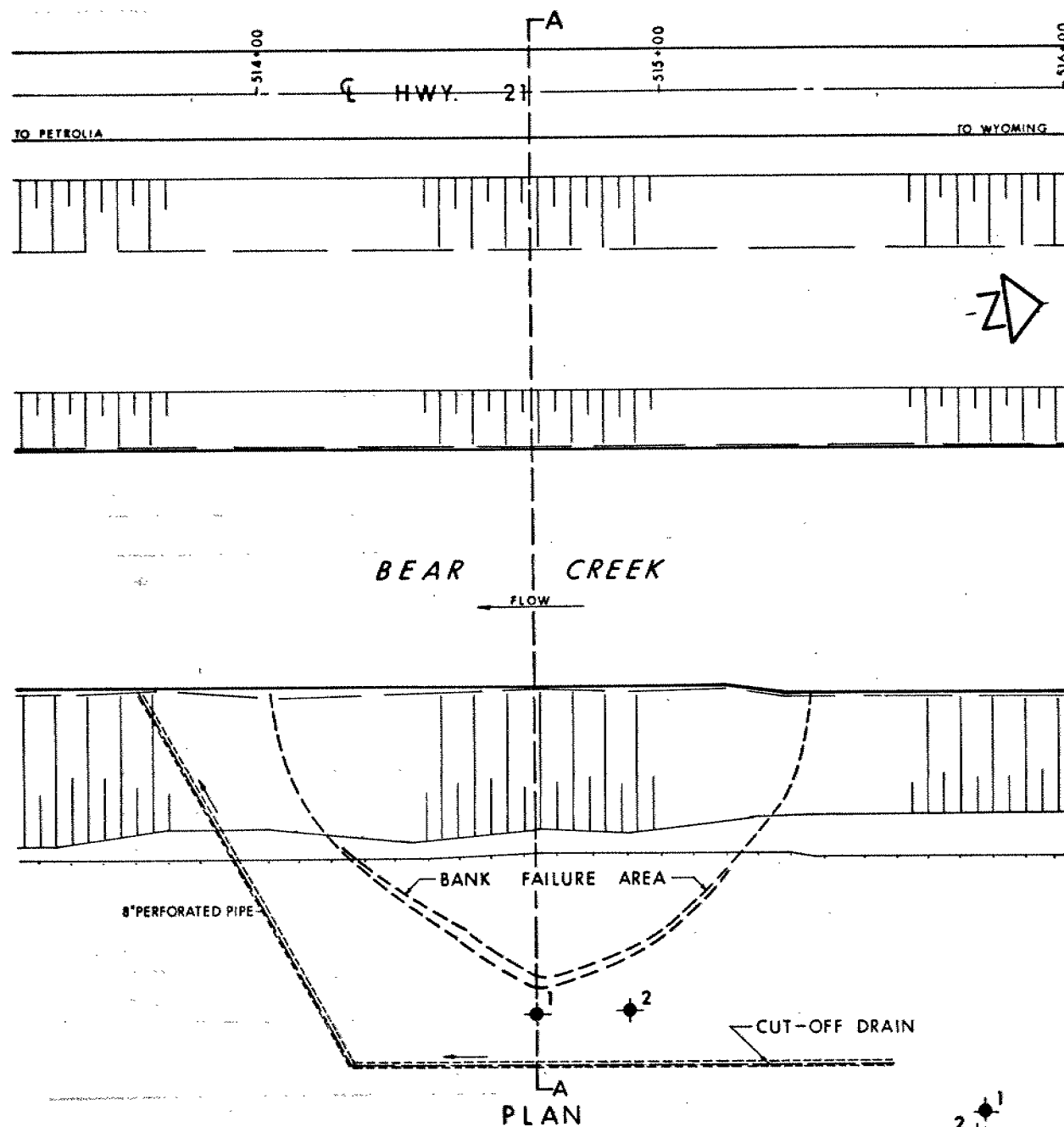
d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_0	COEFFICIENT OF EARTH PRESSURE AT REST

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

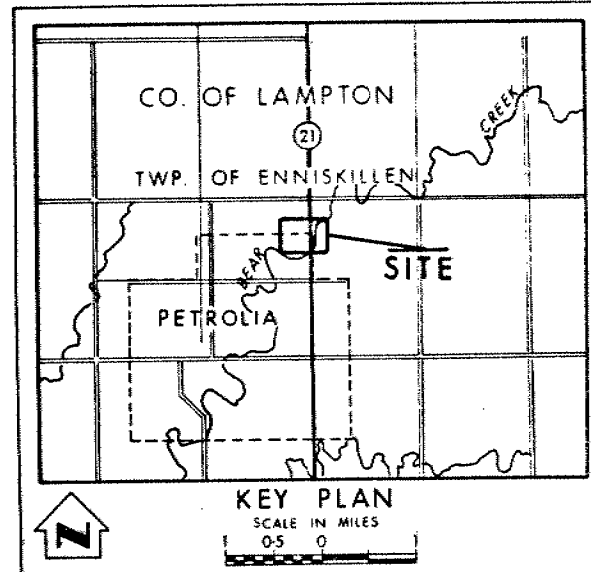
SLOPES

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



SECTION A-A

20 10 0 20 FT.



LEGEND

- Bore Hole
- Cone Penetration Test
- Bore Hole & Cone Test
- Water Levels established at time of field investigation.
- Water Levels Not established

NO.	ELEVATION	STATION	OFFSET
1	651.9	514+69	228 RT.
2	651.0	514+92	227 RT.

NOTE:
The complete foundation investigation report for this structure may be examined at the Structural Office and Foundations Office, Downsview, and at the CHATHAM District Office.

— NOTE —

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

REVISIONS	DATE	BY	DESCRIPTION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO
DESIGN SERVICES BRANCH—FOUNDATIONS OFFICE

BEAR CREEK BANK FAILURE

HIGHWAY NO. 21 DIST. NO. 1
CO. OF LAMPTON
TWP. OF ENNISKILLEN LOT 15 & 16 CON. XII

BORE HOLE LOCATIONS & SOIL STRATA

SUBMIT A P	CHECKED	WFO NO	DRAWING NO
DRAWN OL J	CHECKED	WO NO 73-11082	73-11082A
DATE 28 DEC 1973	SITE NO	BRIDGE DRAWING NO	
APPROVED	CONT NO		





VIEW OF THE SLIDE