

#67-F-282M

SITE INVESTIGATION

BEAR CREEK BRIDGE

HARLEY-DYMOND TWP.

54 2552  
Site 47-212

D O M I N I O N   S O I L   I N V E S T I G A T I O N   L I M I T E D

77 CROCKFORD BOULEVARD - SCARBOROUGH ONTARIO CANADA - TELEPHONE 751-6565

BRANCH  
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TELEPHONE GE 8-3851



FOUNDATION ENGINEERS

ASSOCIATED COMPANY  
SOIL TESTING AND ENGINEERING LTD.  
34 BRENTFORD ROAD,  
KINGSTON 5, JAMAICA, WEST INDIES  
TELEPHONE: 66596

Our Ref. No: 6-12-3

9th January 1967.

Sutcliffe Company,  
Consulting Engineers,  
P.O. Box 430,  
New Liskeard, Ontario.

Attention: Mr. R.W. Brotherhood, P.Eng.

Re: Site Investigation. Proposed Bear Creek  
Bridge, Harley-Dymond Township Line.

Dear Sirs,

We have the pleasure of enclosing our detailed report on the soil investigation carried out at the above site.

The investigation has indicated that the site is underlain by deep varved clay deposits typical of those encountered in the vicinity of New Liskeard. The deposit shows a distinct zoning with regard to shear strength and exceptionally low shear strength values were recorded in the top 10 feet or so of the subsoil.

We believe that the recent failure of the east bank can be directly attributed to ground water seepage and the low shear strength of the top zone in which the slide has occurred. Calculations indicate that a sufficient factor of safety can only be secured if the slopes are flattened to 1 vertical in 6 horizontal and the construction of approach fills is avoided. This, however, would make the construction of a considerably longer structure necessary.

The new structure could be supported on friction piles, embedded into the clay for some 40 to 50 feet in order to achieve a safe working capacity of 20 tons.

# DOMINION SOIL INVESTIGATION LIMITED

Our Ref. No: 6-12-3

2.

We trust that you will find the following report sufficiently detailed and to contain all the required information. However, should you have any queries we would be glad to give you further assistance. It has been a pleasure to have been associated with you on this project.

Yours very truly,

DOMINION SOIL INVESTIGATION LIMITED



I. P. Lieszkowsky, P.Eng.,  
Chief Engineer.

IPL/me

DATE	6-12-3	PROJECT NO.	6-12-3
CLIENT	DOMINION SOIL INVESTIGATION LIMITED	ENGINEER	I. P. LIESZKOWSKY
LOCATION		DATE OF REPORT	6-12-3
DESCRIPTION		SCALE	
REMARKS		BY	I. P. LIESZKOWSKY
		CHECKED BY	
		DATE	

THE SUTCLIFFE COMPANY  
CONSULTING ENGINEERS  
NEW LISKEARD, ONTARIO

REPORT  
ON  
SITE INVESTIGATION  
FOR  
PROPOSED BRIDGE  
OVER  
BEAR CREEK  
HARLEY-DYMOND TOWNSHIP LINE, ONTARIO

SUBMITTED BY  
DOMINION SOIL INVESTIGATION LIMITED  
77 CROCKFORD BOULEVARD  
SCARBOROUGH, ONTARIO.

OUR REFERENCE NO: 6-12-3

JANUARY, 1967.

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### INTRODUCTION

At the request of the Sutcliffe Company, Consulting Engineers, an investigation was carried out at the site of the proposed new Bear Creek Bridge on the Harley-Dymond Township Line. An old timber structure which served as a crossing at this point collapsed recently due to the gradual failure of the east bank of the creek. The purpose of the investigation was to analyse the stability of the approaches and to give recommendations for the foundation design of a new bridge.

To determine the significant soil properties relevant to the design of the proposed crossing, four boreholes were put down to a maximum depth of 80 ft. in the locations shown on the Site Plan (Enclosure No. 2). The information on this Plan was taken from the Consulting Engineers Drawing No. 4827-A9, on which elevations were referred to a temporary bench mark assigned a value of 200 ft. Both disturbed and undisturbed soil samples were recovered from the boreholes and in-situ vane tests were performed, where applicable, to measure the shear strength of the subsoil. The results of the borings are shown on the Geotechnical Data Sheets (Enclosures No. 3 to 6 inclusive) and details of the procedure are described in Appendix I.

### THE SLIDE

The Harley-Dymond Townships Line Road crosses Bear Creek a few miles north-west of New Liskeard which lies in a wide flat valley underlain by deep glacial clay.

Bear Creek has cut an approximately 30 foot deep channel into the floor of the valley. The banks of the creek, although showing occasional signs of instability or past landslides, are generally flat - of the order of 10 to 15 degrees.

The collapse of the old timber bridge was caused by the gradual progressive slide of the east bank displacing the piled foundation of the structure. The approaches to the old bridge were formed by earth embankments at the abutments grading into shallow cut near the top of the slope. Judging from the embankment on the west side, which has not failed, it seems that the height of fill was probably no more than 10 ft.

### SOIL CONDITIONS

The sequence of natural strata discovered in borehole No. 1 consists of about 9 ft. of brown silty clay and silt overlying a deep bed of grey varved clay. This result agrees with the findings of previous investigations in the area in which the properties of the varved clay have been closely studied (References: 1 to 3).

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the natural soil outside the slide area are brown clays with varying amounts of sand or silt and consistencies ranging from firm to stiff. The slide material is, of course, mixed and very loose soil. The predominant soil is, therefore, the varved clay which is described in detail below.

#### Varved Clay

In the field, the varved clay appears as a soft grey silty clay with no obvious laminations. Closer examination of vertical sections in the laboratory, particularly after drying, shows an alternation of dark-coloured clay layers with light-coloured clayey silt layers up to one-inch in thickness. The plastic properties of any bulk sample obviously depend on the relative proportions of these two soils which, separately, are known to have the following approximate values:

	<u>Liquid Limit</u>	<u>Plasticity Index</u>	<u>Liquidity Index</u>
Dark layers	70%	45	above .9
Light layers	30%	15	0.6 to 1.0

The samples tested lie within this range except that the natural moisture contents of samples near the top of the clay are generally above the liquid limit, which gives values of liquidity index well above 1 (Enclosures No. 3 to 6).

The undrained shear strength of the varved clay as measured in the field by the vane increases with depth from about 300 lbs/ft.<sup>2</sup> or less to almost 2000 lbs/ft.<sup>2</sup> at elevation 110 ft. The most significant point is that some very low strengths were recorded just below the clay surface both in the field and laboratory e.g. 180 lbs/ft.<sup>2</sup> vane reading at 13 ft. in borehole No. 1 and 130 lbs/ft.<sup>2</sup> from an unconfined compression test at 5 ft. in borehole No. 3. The sensitivity of the clay measured by the field vane ranged from 3 to 8 which is considered high.

It is understood that water wells in the area penetrate to the bottom of the varved clay at about elevation 40 ft. where an Artesian aquifer is encountered. A subsurface profile along the alignment is included in Enclosure No. 7.

### DISCUSSION

Since the new crossing is to be on the same alignment as the old, the cause of the recent failure must be considered before new foundation proposals can be made.

### Slide Analysis

The landslide on the east side is said to have occurred some 20 years after construction of the bridge, starting in natural ground north of the alignment and affecting the structure mainly in the lower half of the slope where the axes of the slide and the bridge cross at an angle. For this reason, and because the method of construction would seem to increase stability by

cutting from the top of the slope and filling lower down, it is believed that the failure was a natural slide not directly caused by the influence of the structure.

In order to analyse the type of failure and reconstruct the events, an attempt was made to determine the depth of the failure zone. The inclination of the displaced piles give some direct field evidence because a deep slip would rotate the piles to point up-hill whereas a shallow movement would tend to force the piles to lean downhill. It appears that most of the piles which are leaning up-hill have been considerably displaced, most probably by shearing in the failure zone and that those leaning downhill are still near their original positions. Further field evidence was gained from a close study of the orientation of the varves which were expected to be tilted from their natural horizontal position above the failure zone. In all the samples examined, the varves were horizontal and no other evidence of a failure plane was detected. This fact suggests that the top of the varved clay in the slide area is either the lower surface of the failure plane or the original surface of the clay along which the overlying material slipped. Taken altogether therefore, the field evidence points more towards a shallow slide than a deep seated failure.

A theoretical analysis of the slope's stability was also made by the  $\phi = 0$  method, which is reported by previous investigators (References: 1 and 2) to give a better estimate of

the safety factor than effective stress analyses. Using the reconstructed profile along the axis of the slide before failure and assuming zones of constant unit weight and shear strength as shown on Enclosure No. 8, the factor of safety was calculated for two circular failure sections. The rather low shear strength of 200 lbs/ft.<sup>2</sup> was assumed for the upper clay zone because shear strength and moisture content measurements indicate very soft zones near the surface of the clay. The calculated factors of safety were 1.3 for the deeper circle and 1.0 for the shallow one, indicating that failure is likely to occur near the ground surface whereas there is a sufficient safety factor against a deep seated slide.

The evident conclusion, therefore, is that the slope failed by sliding along, or just below, the very soft upper zone of the varved clay. In this zone the shear strength would be adversely affected by ground water seepage and it seems probable that softening of the clay in an exceptionally wet period caused a local failure which triggered the observed slow progressive movement.

Under these conditions, the factor of safety can only be increased by flattening the slope. Theoretically, the maximum safe slope against shallow type of failure is half the effective angle of shearing resistance which, for this varved clay deposit, has been found to range from about 20° to 28° (References: 1 and 2).

Since the slope which failed was about 1 in 4 ( $14^{\circ}$ ), it is recommended that for this project the lower limit of this range should be used which would then give a slope of  $10^{\circ}$  or 1 in 6. This agrees well with the measured value of the flatter and stable slopes on the west side of the creek.

Although the varved clay beneath the fill on the west side is not well enough explored to justify a separate theoretical analysis, the properties are comparable with those already discussed and the same factors will affect stability. Since the weight of fill and the end slope are probably greater than they were on the east side, the factor of safety must be considered too low and the same remedies should be applied to both sides.

If the present deck level of 185 ft. is maintained the profile along the new bridge will be as shown in Enclosure No. 9 which, since the addition of fill should be avoided, indicates a deck extension of about 105 ft. and a new total length of 220 ft. It is also apparent that this length can be considerably reduced by lowering the deck, provided other design considerations permit it.

Since a similar slide in natural slopes beside the road must be prevented, it will be necessary to grade these slopes to the recommended 1 in 6 for some distance on each side. This distance cannot be calculated but one-half the greatest width of the present slide, i.e. 50 ft. each side of the road, should be adequate. All graded slopes should be sodded or seeded to prevent surface erosion.

FOUNDATIONS

Because the shear strength of the varved clay is too low for an economical spread footing design, the bridge will have to be supported on piles. Also, since an adequate bearing layer for end-bearing piles is expected to be at about elevation 40 ft., shorter piles will be supported by side friction in the varved clay and their bearing capacity will increase with pile length. It has been found that the adhesion between the varved clay and driven piles reaches the full shear strength as estimated from in-situ vane tests after a period of 60-90 days (Reference: 1). On this basis the following safe bearing capacities for 12 inch diameter piles have been estimated using a safety factor of 3:-

TABLE I

Borehole No.	Pile capacity for 50 ft. embedded length	Embedded length required for 20 ton capacity
1	14 tons	58 ft.
2	17 tons	54 ft.
3	24 tons	45 ft.
4	22 tons	45 ft.

Thus piles with a safe working load of 20 tons must be driven between 45 and 58 ft. into the clay depending on the location, and left for about 90 days before being fully loaded.

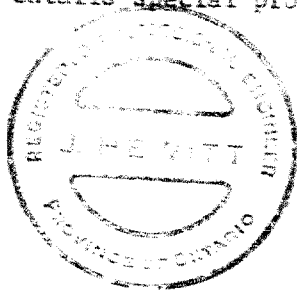
ALTERNATIVE DESIGN

It has been suggested that an embankment with culverts might prove a more economical design than a longer bridge. An economic comparison of the two designs is beyond the scope of this report but, without undertaking a detailed theoretical analysis, some general comments may be given as a guide. Experience with embankment failures in the area (References: 1 and 2) has shown that construction without berms to heights approaching 20 ft. is unsafe. A stable design for this project would probably involve two berms and a total base width of over 250 ft., which is almost equivalent to a side slope of 1 in 5 for the fill material. Difficulties in construction could be expected because of the soft surface of the clay and, furthermore, excessive long term settlements of the embankment would occur.

CONCLUSIONS

1. Beneath about 10 ft. of fill or overburden, the site is underlain by gray varved clay ranging from soft to stiff with depth.
2. The foundations of the present structure were carried away on the east bank by failure of the natural slope along a shallow plane near the top of the varved clay.
3. To avoid similar failures in the future, the slopes on both banks should be graded to 1 in 6 for about 50 ft. each side of the road and approach fills should be avoided. This will involve a longer bridge structure than the previous one.

4. The structure must be founded on piles, for which safe loads are given in the report.
5. An alternative design using an embankment and culverts entails special problems of construction and stability.



DOMINION SOIL INVESTIGATION LIMITED

*J. Hewitt*  
J. Hewitt, P.Eng.

JH/me



APPENDIX IPROCEDURE

The work in the field was carried out between December 12th and 16th inclusive 1966, using a skid-mounted diamond drill machine and standard washboring technique. Both disturbed and undisturbed samples were recovered by a 2" outside diameter split-spoon sampler or 2" inside diameter thin-walled Shelby tube samplers. The samplers were advanced either by static weight or in case of the undisturbed samples by a 140 lb. hammer falling freely 30 inches. The number of blows required for 12" of penetration are recorded as the standard penetration test results or 'N' values. In between the samples the in-situ undrained shear strength of the subsoil was measured by a 4-bladed vane test apparatus which was rotated at a slow rate until failure of the soil occurred and the applied torque at failure was recorded. From the measured torque and the dimensions of the vane the in-situ shear strength of the clay was calculated. The results of the standard penetration tests and the field vane tests are plotted on the Geotechnical Data Sheets for the boreholes (Enclosures No. 3 to 6 inclusive).

The samples were visually classified in the field and carefully sealed to prevent the loss of moisture and shipped into the laboratory of Dominion Soil Investigation Limited for further examination and testing. The laboratory testing consisted of the determination of the: natural moisture content, natural unit

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weight, the Atterberg limits and the unconfined compressive strength of the encountered soil types. The results of the laboratory tests are plotted on the individual borehole logs.

REFERENCES

1. V. Milligan, L.G. Soderman and A. Rutka:  
Experience with Canadian Varved Clays  
Journal Soil Mechanic's and Foundation Division  
Proc. A.S.C.E. Vol. 88, No. SM4, 1962.
2. K.Y. Lo and A.G. Stermac:  
Failure of An Embankment Founded on Varved Clay  
Department of Highways, Ontario  
Report No. 54, 1964
3. W.J. Eden and M. Bozozuk:  
Foundation Failure of a Silo on Varved Clay  
The Engineering Journal of Canada, Vol. 45,  
No. 9, 1962.

E n c l o s u r e s

# LIST OF SYMBOLS, ABBREVIATIONS AND NOMENCLATURE.

## SOIL COMPONENTS AND GROUND WATER CONDITIONS.

BOULDER	COBBLE	GRAVEL		SAND			SILT	CLAY	ORGANICS	BEDROCK	GROUND WATER LEVEL	DEPTH OF CAVE-IN
		COARSE	FINE	COARSE	MEDIUM	FINE						
φ	> 8"	3"	3/4"	4.75mm	2.0	0.42	0.074	0.002	>	NO SIZE LIMIT		
U.S. Standard Sieve Size		No. 4		No. 10	No. 40	No. 200						

## SAMPLE TYPES.

AS Auger sample

CS Sample from casing

ChS Chunk sample

RC Rock core

% Recovery

SS Split spoon sample

TP Piston, thin walled tube sample

TW Open, thin walled tube sample

WS Wash sample

SAMPLER ADVANCED BY static weight w  
 " pressure p  
 " tapping t

OBSERVATIONS  
 MADE WHILE  
 CORING

Steady pressure  
 No pressure  
 Intermittent pressure

Washwater returns  
 Washwater lost

## PENETRATION RESISTANCES.

DYNAMIC PENETRATION RESISTANCE : to drive a 2" dia., 60° cone attached to the end of the drilling rods into the ground, expressed in blows per foot

STANDARD PENETRATION RESISTANCE, -N- : to drive a 2" outside dia. split spoon sampler 1 foot into the ground, expressed in blows per foot

EXTRAPOLATED -N- VALUE

The energy for the penetration resistances is supplied by a 40 lb. hammer falling 50 inches

SYMBOL

322

## SOIL PROPERTIES.

W % Water content

LL % Liquid limit

PL % Plastic limit

PI % Plasticity index

LI Liquidity index

 $\gamma$ 

Natural bulk density (unit weight)

e

Void ratio

RD

Relative density

Cv

Coeff. of consolidation

mv

Coeff. of volume compressibility

k

Coeff. of permeability

C

Shear strength in terms of total stress

 $\phi$ 

Angle of internal friction in terms of effective stress

 $\phi'$ 

## UNDRAINED SHEAR STRENGTH.

- DERIVED FROM -

TRIAXIAL

COMPRESSION

UNCONFINED

TEST

LABORATORY

VANE TEST

FIELD

POCKET

PENETROMETER

TEST

- St

Strain at failure is represented by direction of stem

20%  
 15% + 5%  
 10%

St : sensitivity =  $\frac{\text{shear strength in undisturbed state}}{\text{shear strength in remoulded state}}$

## SOIL DESCRIPTION.

COHESIONLESS SOILS :

RD :

COHESIVE SOILS :

C lbs/sq ft

Very loose

Loose

Compact

Dense

Very dense

0 - 15 %

15 - 35 %

35 - 65 %

65 - 85 %

85 - 100 %

Very soft

Soft

Firm

Stiff

Very stiff

Hard

less than 250

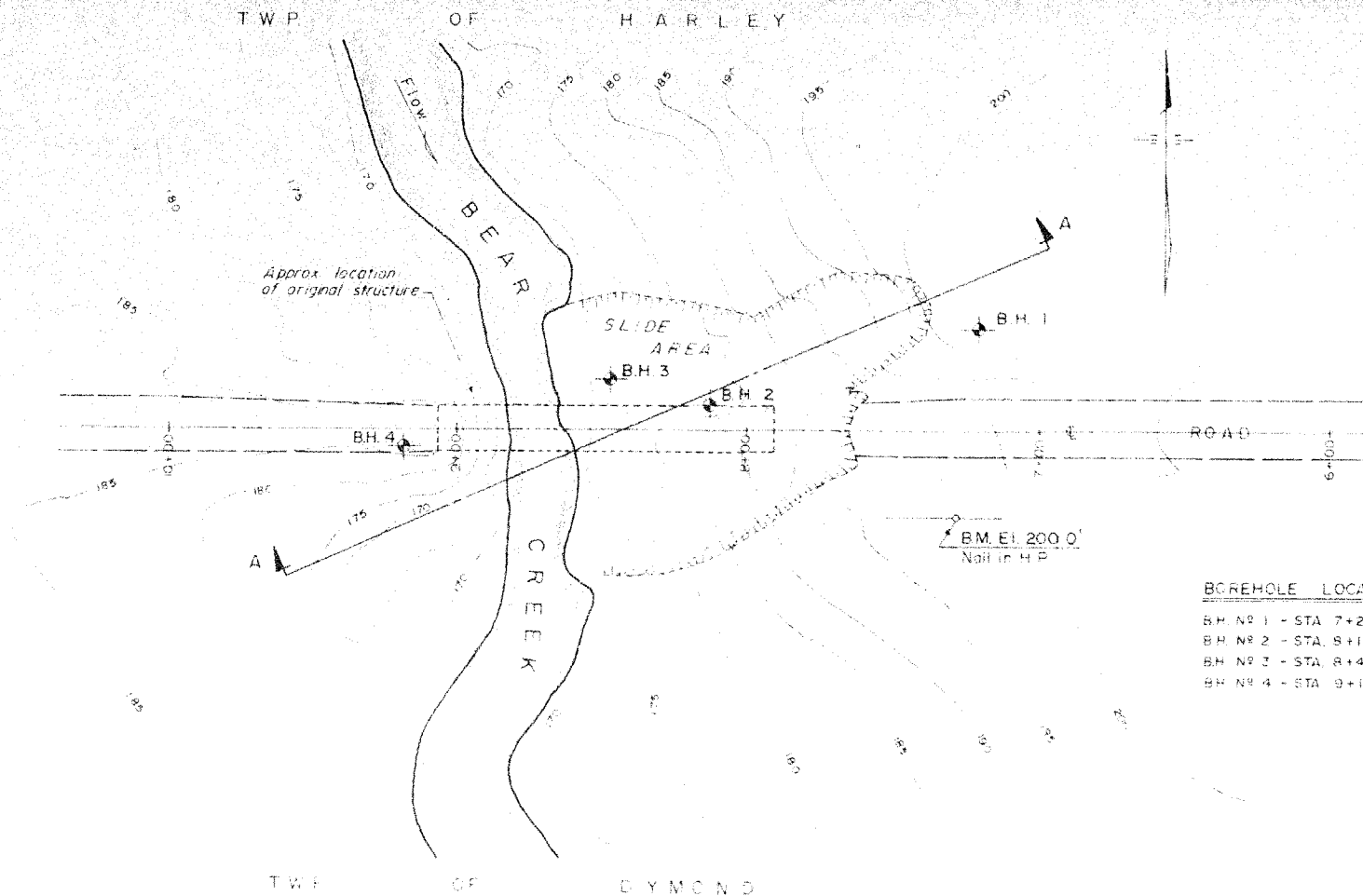
250 - 500

500 - 1000

1000 - 2000

2000 - 4000

over 4000



#### BOREHOLE LOCATIONS

- BH No 1 - STA. 7+20.0, 35' RT. of C
- BH No 2 - STA. 8+10.5, 9' RT. of C
- BH No 3 - STA. 9+16.7, 18' RT. of C
- BH No 4 - STA. 9+16.7, 6' LT. of C

### BOREHOLE LOCATION PLAN

SCALE 1" = 40' Feet

DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT

# GEOTECHNICAL DATA SHEET FOR BOREHOLE . . . .

OUR REFERENCE NO. 6 - 12 - 3

CLIENT SUTCLIFFE COMPANY  
PROJECT PROPOSED BEAR CREEK BRIDGE  
LOCATION HARLEY - DYMOND TWP., NEW LISKEARD  
DATUM ELEVATION

METHOD OF BORING WASH BORING  
DIAMETER OF BOREHOLE 2 7/8"  
DATE DEC. 12, 1966

ENCLOSURE NO. 3

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %				REMARKS	
				NUMBER	TYPE	No. Advance- ment of Sampler	2,0	4,0	6,0	8,0	10,0	PL WL LI					
							SHEAR STRENGTH 1000 lbs. sq. ft.					1	2	3	4		5
201.8	0	GROUND SURFACE															
200	1.0	TOPSOIL															
		Stiff, grey - brown Mottled CLAY		1	SS	6											
	5.0	Firm, brown SILTY CLAY		2	T.W	P											
	9.0																
190	10			3	T.W	P											
		Grey		4	T.W	P											
	20	Soft		5	T.W	P											
180		VARVED		6	T.W	P											
	30			7	T.W	P											
170		CLAY															
	40			8	T.W	P											
160		Firm															
	50			9	T.W	P											
150																	
	60																
140		END OF BOREHOLE															

W.L. E: 196.8'  
Dec. 15, 1966  
3:00 PM

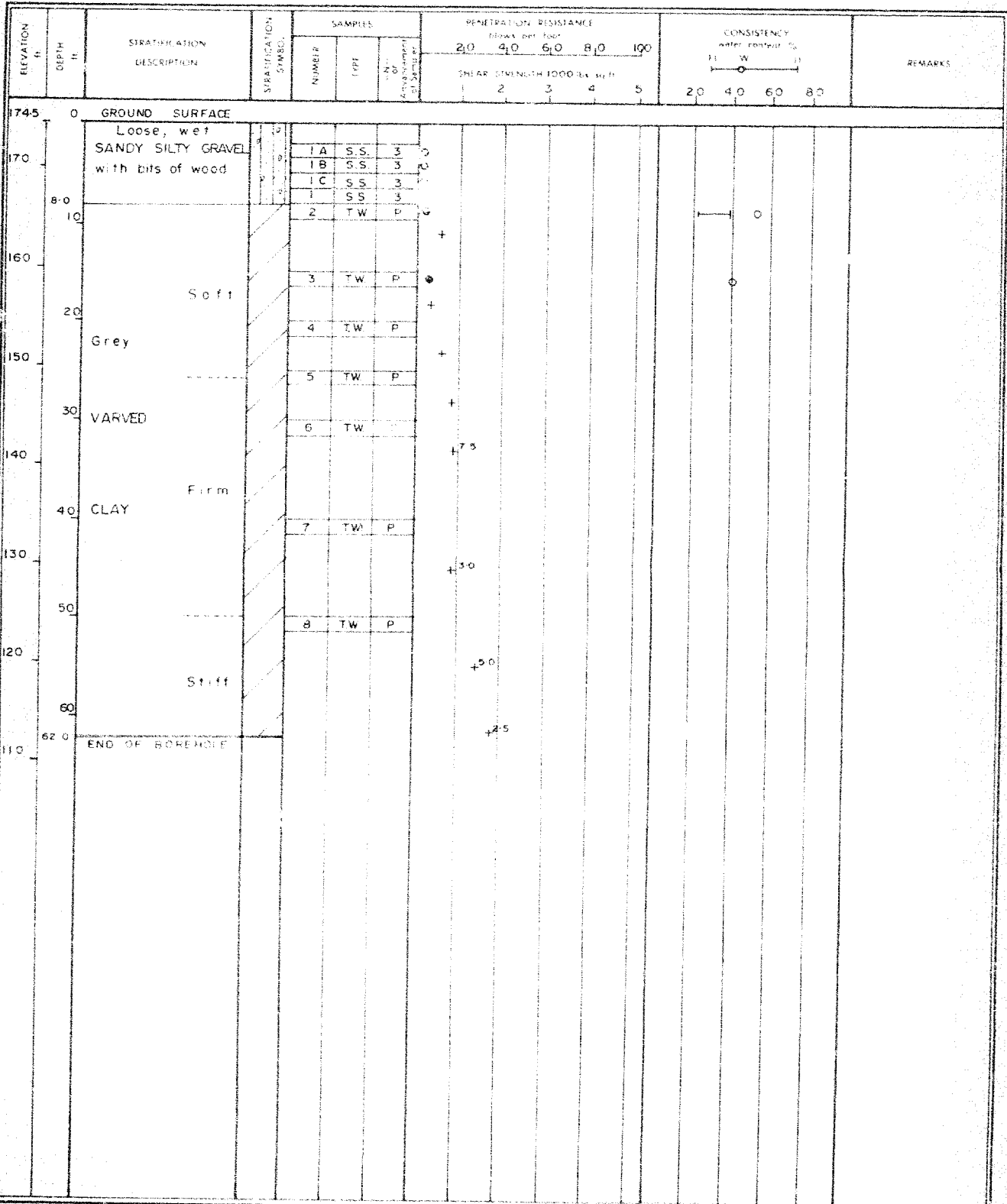
# GEOTECHNICAL DATA SHEET FOR BOREHOLE 2

OUR REFERENCE NO. 6-12-3

CLIENT: SUTCLIFFE COMPANY  
PROJECT: PROPOSED BEAR CREEK BRIDGE  
LOCATION: HARLEY-DYMOND TWP - NEW LISKEARD  
DATUM ELEVATION:

METHOD OF BORING: WASHBORING  
DIAMETER OF BOREHOLE: 2 7/8"  
DATE: DEC. 14, 1966

ENCLOSURE NO. 4





















# GEOTECHNICAL DATA SHEET FOR BOREHOLE . . . 3 . .

OUR REFERENCE NO. 6 - 12 - 3

CLIENT: SUTCLIFFE COMPANY  
PROJECT: PROPOSED BEAR CREEK BRIDGE  
LOCATION: HARLEY-DYMOND TWP., NEW LISKEARD  
DATUM ELEVATION

METHOD: WASHBORING  
DIAMETER OF BOREHOLE: 2 7/8"  
DATE: DEC. 15, 1966

ENCLOSURE NO. 5

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %				REMARKS
				NUMBER	TYPE	No. Advancement of sampler	20	40	60	80	100	PL	W	LL		
							SHEAR STRENGTH 1000 lbs. sq. ft.									
							1	2	3	4	5	20	40	60	80	
173.0	0	GROUND SURFACE														
170	3.0	TOPSOIL		1A	SS	2	0									
		1B		SS	2	0										
		1C		SS	1	0										
		1		TW	P											
	10	Soft					+									
		2		TW	P											
160							+									
		3		TW	P											
	20	Firm					+									
		4		TW	P											
150		Grey														
	30	Stiff		5	TW	P										
140		VARVED					+	7.5								
	40			6	TW	P										
130		CLAY														
	50			7	TW	P										
120																
	60															
110																
	70															
100																
	80															
90		END OF BOREHOLE		8	TW	P										
	90															

VERTICAL SCALE: 1 IN TO 10 FT

DOMINION SOIL INVESTIGATION LIMITED

MADE: D. A. M. CHD

OUR REFERENCE NO. 6-12-3

# GEOTECHNICAL DATA SHEET FOR BOREHOLE 4

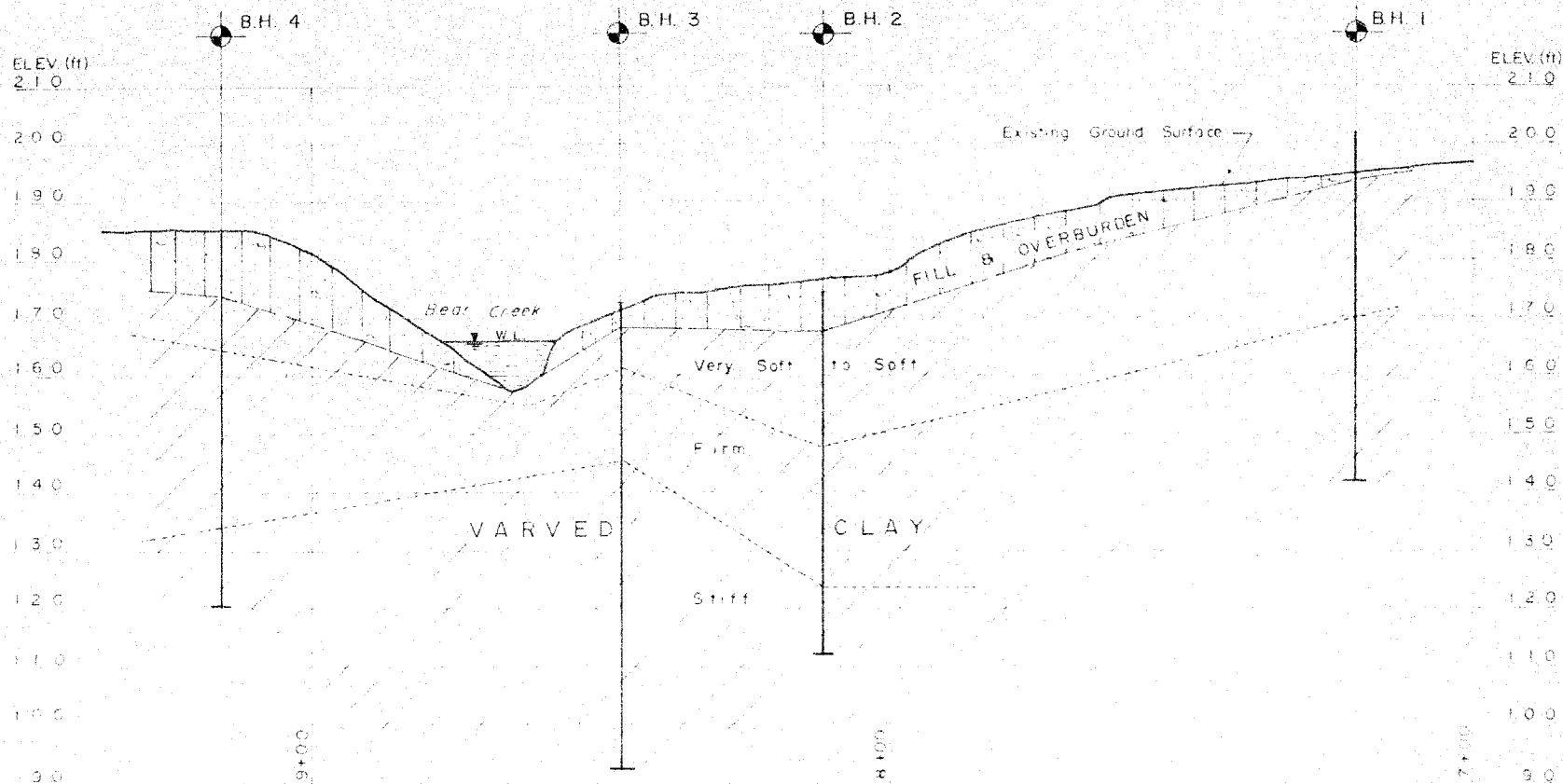
CLIENT: SUTCLIFFE COMPANY  
PROJECT: PROPOSED BEAR CREEK BRIDGE  
LOCATION: HARLEY - DYMOND TWP., NEW LISKEARD  
DATUM ELEVATION

METHOD OF BORING: WASHBORING  
DIAMETER OF BOREHOLE: 2 7/8"  
DATE: DEC 16, 1966

ENCLOSURE NO. 6

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %			REMARKS
				NUMBER	TYPE	N <sub>60</sub> Adjusted to Standard	20	40	60	80	100	PL	W	FL	
							SHEAR STRENGTH 1000 lbs. sq. ft.								
							2	3	4	5					
185.2	0	GROUND SURFACE													
180	10	Firm to Stiff Brown SANDY CLAY FILL with gravel		1	SS	9	0								
				2	SS	6	0								
170	20	Soft		3A	SS	3	0								
				3	TW	P									
160	30	Grey		4	SS	6	0								
				4A	TW	P									
150	40	Firm VARVED		4B	TW	P									
140	50	CLAY		5	TW	P									
130	60	Stiff		6	TW	P									
120	70	END OF BOREHOLE		7	TW	P									
110															

LARGE TIMBERS  
FROM 11'-6" TO  
ABOUT 15'

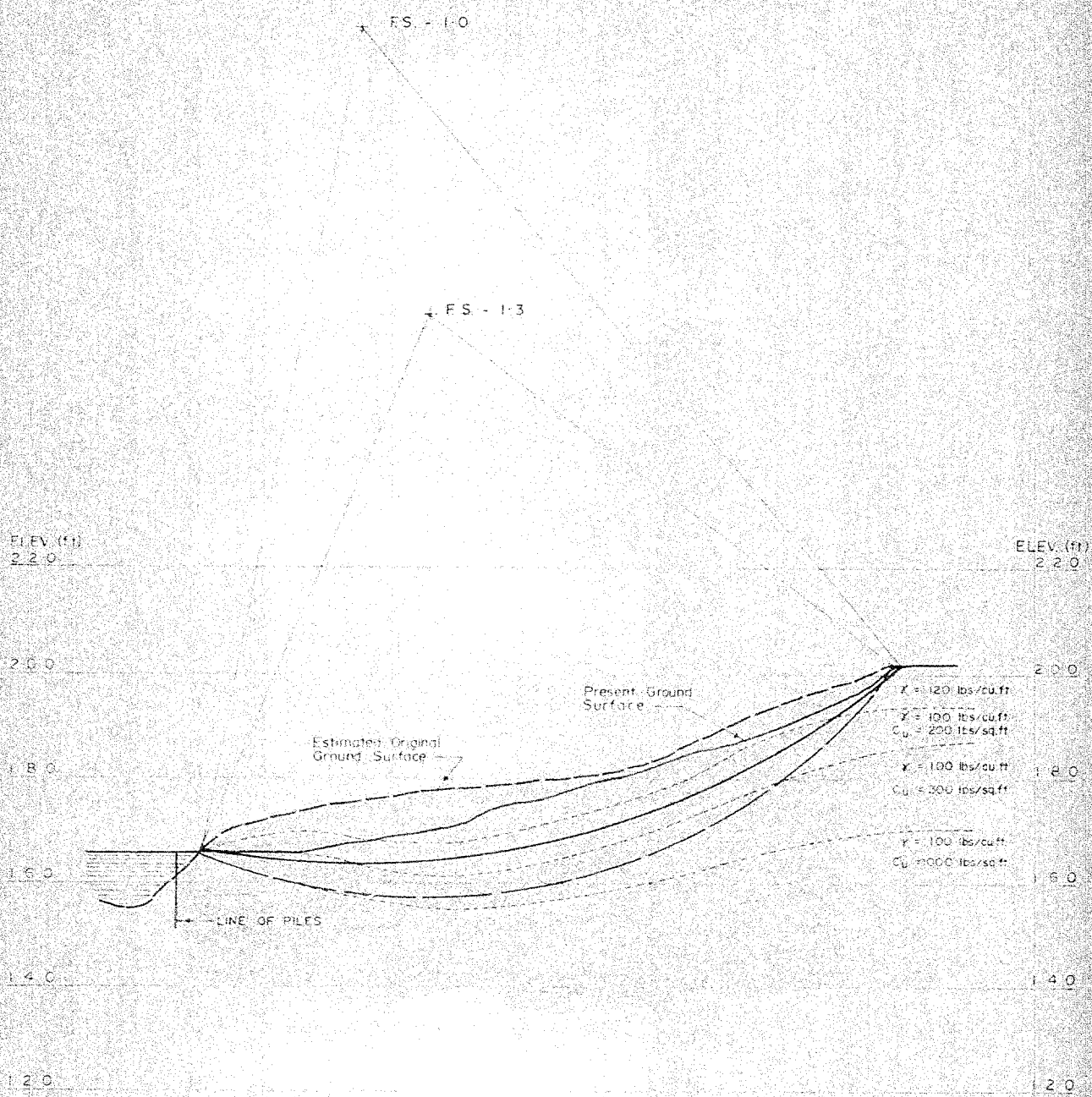


# SUBSURFACE PROFILE

(ALONG C OF ROAD)

SCALE 1" = 20' Feet

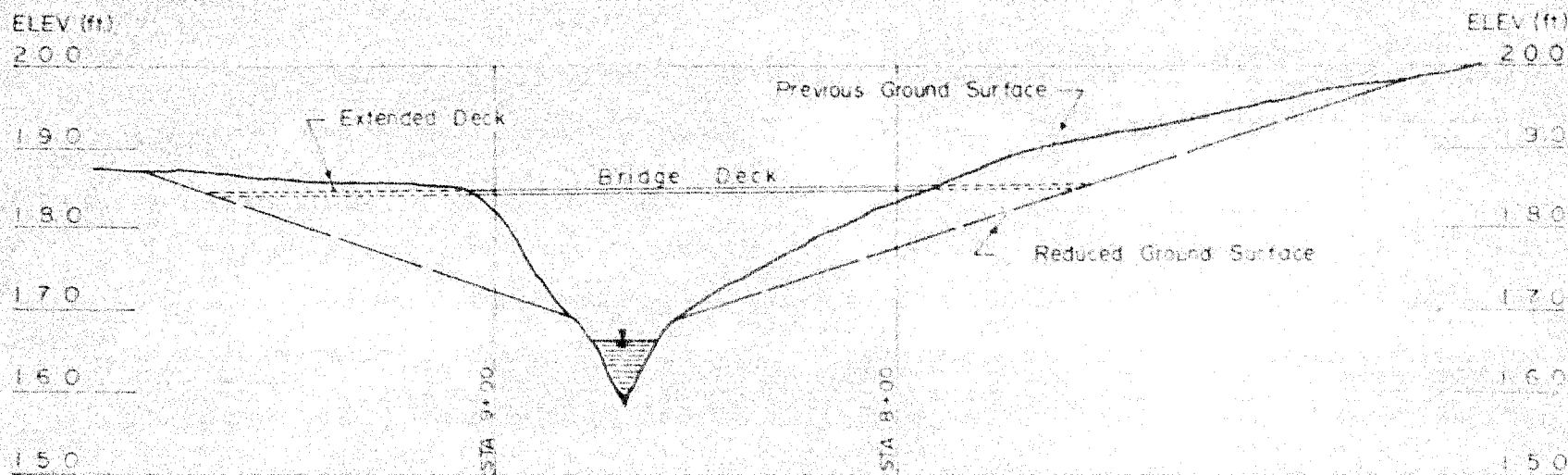
DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT



STABILITY - SECTION A-A THROUGH SLIDE

SCALE 1" = 20 Feet

DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT



## SECTION ALONG ALIGNMENT SHOWING NEW CONSTRUCTION

SCALE Horizontal - 1" = 40 Feet  
Vertical - 1" = 20 Feet

DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT