#69-F-43

W.P. 144-63-01

H.W.Y. #401

BLOOMFIELD ROAD

CHINNICK DRAIN

CRM. 110 LAB. [illegible]

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. B. P. Davis,
Bridge Engineer,
Bridge Office,
Admin. Bldg.

From: Foundation Section,
Materials & Testing Office,
Room 107, Lab. Bldg.

Attention: Mr. S. McCombie

Date: August 29, 1969

Our File Ref.

In reply to

SEP 22 1969

Subject:

FOUNDATION INVESTIGATION REPORT
For
Proposed Chinnick Drain Culvert
Station 74+83 - Bloomfield Road
West Access to Chatham from Hwy. #401
County of Kent
District No. 1 (Chatham)
W.J. 69-F-43 -- W.P. 144-63-01

Enclosed please find our foundation investigation report for abovementioned structure. We believe that the information contained within the report will be sufficient for design purposes. Should any further information be required however, or should any points require clarification please contact this Office.

A. G. Stermac
PRINCIPAL FOUNDATION ENGINEER.

AGS/ia
Attach.
cc: Messrs. B. P. Davis (2)
E. A. Tregaskes
D. W. Farren
W. Zonnenberg
P. C. Brown
A. P. Watt
J. Roy
P. A. Singh
G. Tilley, Giffells Ass. Ltd.
Foundation Files
General Files.

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FOUNDATION INVESTIGATION REPORT
For
Proposed Chinnick Drain Culvert
Station 74+83 - Bloomfield Road
West Access to Chatham from Hwy. #401
County of Kent
District No. 1 (Chatham)
W.J. 69-F-43 -- W.P. 144-63-01

1. INTRODUCTION:

A request to carry out a foundation investigation for a proposed new culvert at the above location was received from Mr. J.G. Forster, Senior Soils Engineer, London, in a memo addressed to Mr. A.G. Stermac, Principal Foundation Engineer and dated April 23, 1969.

A proposal has been made by Giffels Associates Limited, that the culvert have dimensions 20 ft. x 12 ft. and be of the rigid frame-open type.

An investigation was subsequently carried out by this section to determine the subsoil conditions existing at the site of the proposed culvert.

This report contains the results of our field and laboratory investigation together with recommendations pertaining to the foundation of the new structure.

2. DESCRIPTION OF SITE:

The site is located on Bloomfield Road in Raleigh Township, Kent County, approximately 1 mile from the southern line of the Chatham city limits.

The investigation was carried out adjacent to the site of the existing culvert.

The topography is very flat with farm land on both sides of the road.

Physiographically the site is located in the region referred to as the St.Clair Clay Plains. There is a deep cover of overburden consistency of 100 to 200 feet of fairly uniform clay resting on shale bedrock.

cont'd./2

3. FIELD WORK:

Three boreholes and two dynamic cone penetration tests were carried out. Boring was achieved by means of a conventional diamond drill adopted for soil sampling purposes. Samples were recovered by means of standard split-spoon samplers which were hammered into the soil or in 2 inch O.D. Shelby tubes which were pushed manually and occasionally driven.

The method used to advance the cone in the dynamic cone penetration tests conformed to the requirements of the standard penetration test.

Where possible field vane tests were carried out at various depths in order to determine the undrained shear strength of the cohesive material. All soil samples recovered were carefully examined and classified in the field before being transported to the laboratory.

The locations and elevations of the boreholes were surveyed by the Project Foundation Engineer and are relative to the existing culvert. These locations and elevations are shown on drawing 69-F-43A attached to this report.

4. LABORATORY TESTING:

A careful visual inspection of all samples was carried out in the laboratory prior to commencement of the testing program. Tests were then performed on a selection of samples to determine the following physical properties of the various soil types.

Liquid Limit
Plastic Limit
Natural Moisture Content
Grain Size Distribution
Undrained Shear Strength

On completion of laboratory testing the various soil samples were classified as to type and consistency generally according to the Unified Soil Classification system.

cont'd./3

5. <u>SOIL TYPES & SOIL CONDITIONS</u>:

5.1) <u>General</u>:

The investigation was carried out to a depth of some 30 feet below existing ground level . Within this depth the subsoil consists of clayey silt, beginning underneath the topsoil and extending down to the limit of the investigation. A shallow pocket of silty sand was found in B.H.#2 but this material was not found in either of the other boreholes.

5.2) <u>Clayey Silt with some Sand traces Gravel (Grey)</u>:

This deposit lies underneath the topsoil and extends down to the limit of the investigation in all boreholes. "N" values from standard penetration tests ranged from 5 to 29 blows per foot, the low valves being observed closest to the surface. The consistency is estimated to range from firm to very stiff but generally very stiff.

Laboratory tests gave the following results: -

Grain Size Distribution	Gravel % 1 - 4 (Av. 2) Silt %25 - 54 (Av.47)	Sand 2-22 (Av.18) Clay 28-46 (Av.33)
Liqrid Limit %	27 - 34	
Plastic Lmt. %	15 - 18	
Moist. Cont. %	11 - 36	Av. 21 .
Bulk Density pcf	118 - 137	

Undrained shear strength   Field Vane psf 1400 - 72000

Sensitivity 14 - 2.1

Unconfined Compression Test psf 1,100 - 2,800

Triaxial psf 1,600 - 3,800

cont'd./4

5. <u>SOIL TYPES & SOIL CONDITIONS:</u> (cont'd.) ...

5.2 <u>Clayey Silt with some Sand traces Gravel (Grey)</u> cont'd.

A plot of the variation of shear strength with depth is shown in Fig. 4; from this it is seen that the shear strength varies from a minimum of 1100 psf at elevation 574.0 to a maximum of 3800 psf at elevation 568.0.

For design purposes the material can be taken as having an average shear strength of 1,750 psf.

5.3 <u>Silty Sand with traces of Gravel and Clay:</u>

This material was found to be three feet in depth and was only found in borehole #2. The "N" value observed was 37 blows per foot indicating a very dense consistency.

Laboratory tests gave the following results: -

Grain Size Distribution	Gravel %	7	Sand %	64
	Silt %	22	Clay %	7

Natural moisture content %	11
Liquid Limit %	15.5
Plastic Limit %	19

6. <u>GROUNDWATER CONDITIONS:</u>

The water levels in the boreholes at the completion of field operations were found to be as follows: -

B H	1	El. 580.0
B H	2	El. 584.3
B H	3	El. 581.2

7. <u>DISCUSSION AND RECOMMENDATIONS:</u>

It is proposed to replace the existing culvert, a rigid frame open type of section 17.5 feet span, height 8.5 feet and length approximately 32 feet, with a similar structure of greater cross section ie. 20 feet x 12 feet and of length 100 feet.

cont'd./5

7. <u>DISCUSSION AND RECOMMENDATIONS</u>: (cont'd.) ...

Subsoil in this location consists of a firm to very stiff clayey silt extending from below the topsoil to the limit of the investigation, ie. some 30 feet below existing ground level (A shallow pocket of dense silty sand 3 feet thick was found in B.H.#2 only, this should not cause any problems).

Laboratory tests have shown that below approximate elevation 574.0 the properties of the deposit are such that a bearing capacity of 3500 psf can be assumed. It is proposed to support the new culvert on spread footings at approximately 571.0; in this case the previously mentioned value of 3500 psf can be used for design purposes.

8. <u>MISCELLANEOUS</u>:

The field work for this project was carried out during the period June 20th to June 25th, 1969.

Equipment used was owned by P.V.K. & Sons Ltd.

Supervision of the field work was carried out by Mr. G. Allen, Project Foundation Engineer.

This report was written by Mr. P. Taylor and reviewed by Mr. K. Selby, Supervising Foundation Engineer.

September, 1969.

APPENDIX I

DEPARTMENT OF HIGHWAYS ONTARIO

FORM [illegible]

ACTION SLIP

DATE [illegible] 23/69

TO Mr. Stewart Dawson

FROM J. Foster, London

- [ ] NOTE AND FILE
- [ ] NOTE AND REPORT TO ME
- [ ] RETURN WITH MORE DETAILS
- [ ] NOTE AND SEE ME
- [ ] PLEASE ANSWER
- [ ] FOR YOUR APPROVAL
- [ ] RETURN WITH YOUR COMMENTS
- [ ] PREPARE REPLY FOR MY SIGNATURE
- [ ] TAKE APPROPRIATE ACTION
- [ ] PER YOUR REQUEST
- [ ] FOR YOUR SIGNATURE
- [ ] FOR YOUR INFORMATION
- [ ] INVESTIGATE AND REPORT
- [ ] ______

REMARKS Could you include this investigation in your work program this spring? Our [illegible] report is due June 25th and if possible we would like to have this information in the report at that time.

DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT

FORM OB-MT-126 (REV 1969)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO

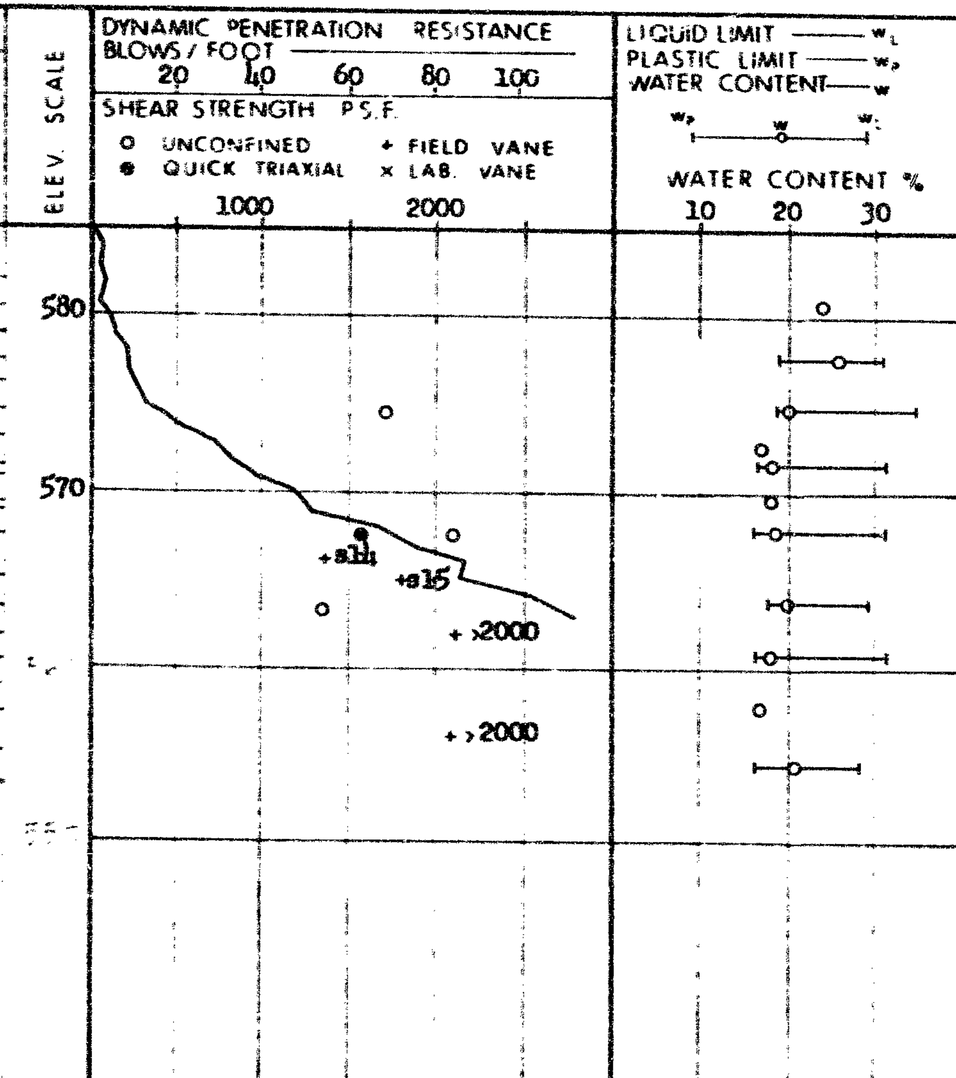
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 1

FOUNDATION SECTION

JOB	69-F-43	LOCATION	Chatham Hwy.7116 Sta. 75 + 10 o/s 41' Lt.	ORIGINATED BY	GA
W.P.	144-63-01	BORING DATE	June 20 & 23, 1969	COMPILED BY	GA
DATUM	Geodetic	BOREHOLE TYPE	Washboring & NX Casing	CHECKED BY	

SOIL PROFILE ELEV / DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	BLOWS / FOOT	BULK DENSITY γ P.C.F.	REMARKS GR. SA. SI. CL.
584.7	Ground Level					
0.0	Topsoil					
1.0	Clayey silt with some sand and traces of gravel. Firm to very stiff.					
		1	SS	5		580.0
		2	SS	6		
		3	TW	PM	118	1 17 54 28
		4	SS	23		4 12 48 36
		5	TW	PM		
		6	SS	22		2 21 46 31
		7	TW	PM	131	
		8	TW	PM	131	
		9	SS	22		
		10	SS	21		2 20 46 32
553.2		11	SS	22		
31.5	End of Borehole					



FORM OB-MT-126 (REV. 1969)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 2

FOUNDATION SECTION

JOB 69-F-43 LOCATION Chatham Hwy.7116 Sta. 75 + 28 o/s 16' Rt. ORIGINATED BY GA

W.P. 144-63-01 BORING DATE June 23 & 24, 1969 COMPILED BY GA

DATUM Geodetic BOREHOLE TYPE Washboring, NX Casing CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F.	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
586.3	Ground Level							
0.0	Topsoil							▼584.3
0.5	Clayey silt with some sand & traces of gravel.	1	SS	9				
		2	SS	8	580			
		3	SS	6				
573.0	Firm to stiff	4	TW	PH		+ > 2000	131	
17.3	Silty sand with traces of gravel & clay. Dense	5	SS	37				7 64 22 7
570.0		6	SS	17	570			
20.3	Clayey silt with some sand & traces of gravel	7	TW	15/12"		● 3820	137	
		8	TW	PH				
		9	SS	22		+ > 2000		1 20 49 30
	Very stiff	10	TW	15/3"	560		134	
554.8		11	SS	29				
31.5	End of Borehole				550			

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE — 1000 2000

LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w — WATER CONTENT % 10 20 30

FORM OS-MT-126 (REV 1969)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 3

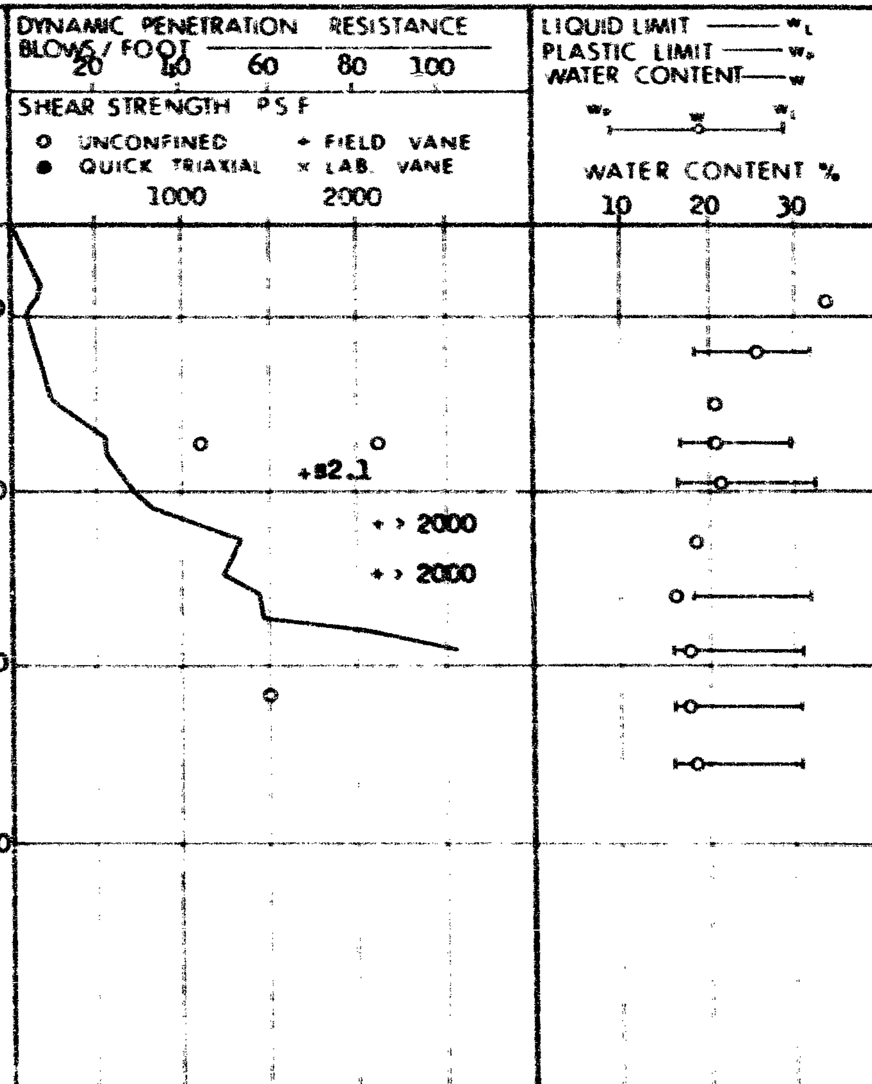
FOUNDATION SECTION

JOB 69-F-43 LOCATION Chatham Hwy.7116 Sta. 75 +12 o/s 54' Rt. ORIGINATED BY GA

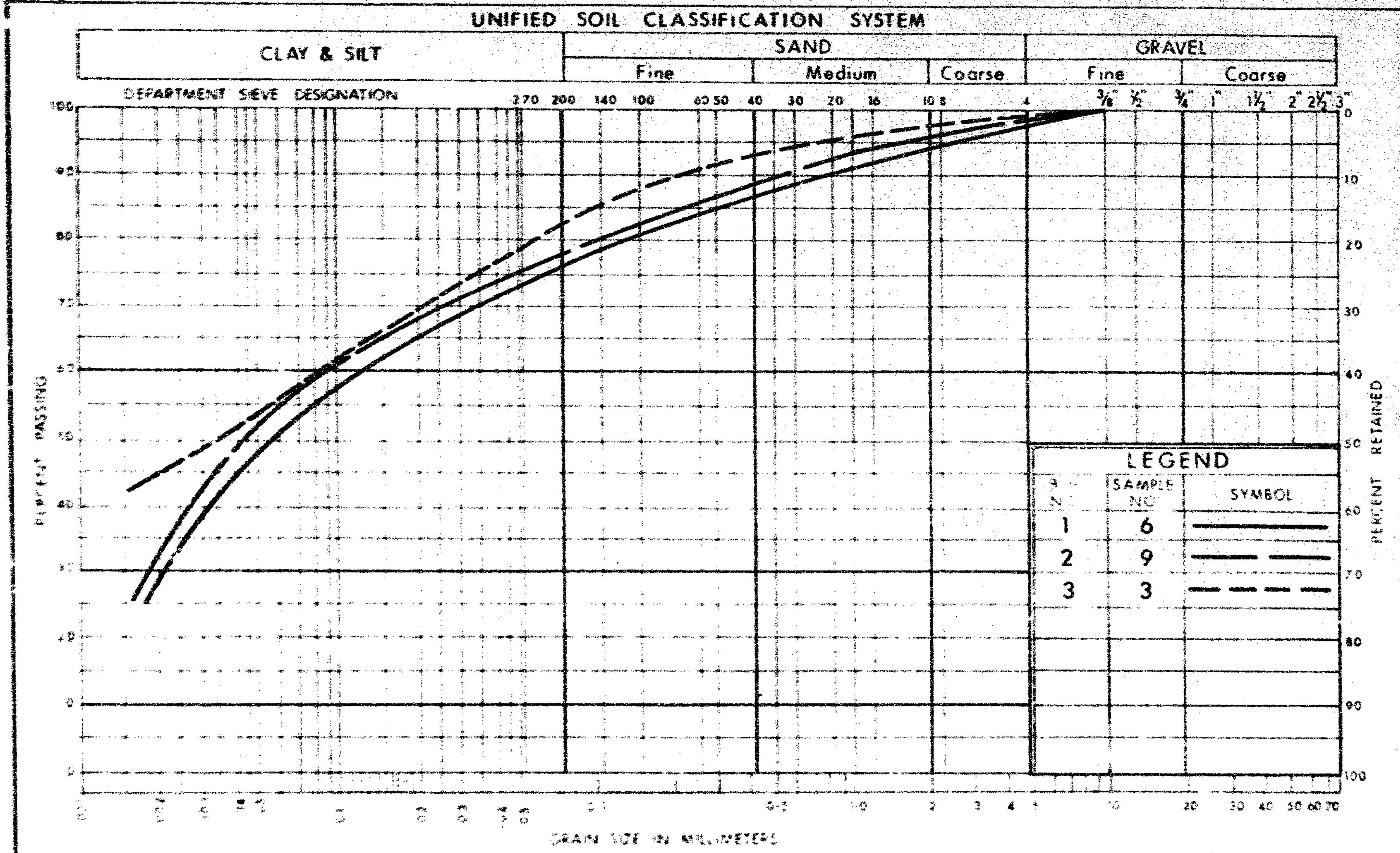
W P 144-63-01 BORING DATE June 25, 1969 COMPILED BY GA

DATUM Geodetic BOREHOLE TYPE Washboring, NX Casing CHECKED BY

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FOOT	BULK DENSITY γ P.C.F.	REMARKS GR SA SI CL
585.0	Ground Level					
	Topsoil					
1.0						▼581.2
	Clayey silt with some sand & traces of gravel. Firm to very stiff	1	SS	11		
		2	SS	6		
		3	SS	7		1 16 37 46
		4	TW	PM	129	
		5	TW	PM		
		6	SS	11		1 22 46 31
		7	SS	-		
		8	SS	22		
		9	TW	11/6"	139	
553.5		10	SS	15		
31.5	End of Borehole					



FORM DB MT -
REV APR 1965



DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION

GRAIN SIZE DISTRIBUTION

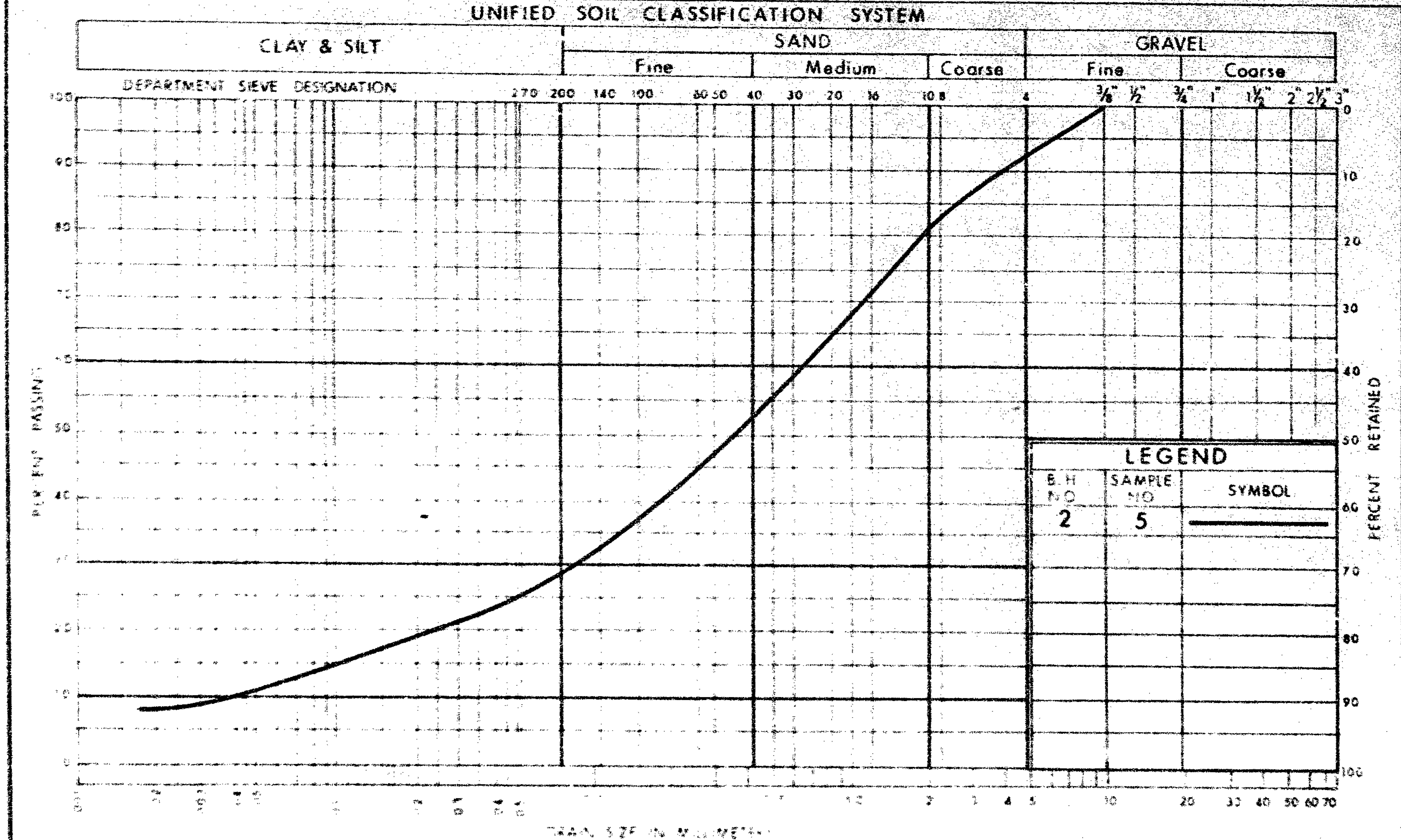
CLAYEY SILT

WP No. 144 - 63 - 01

JOB No. 69 - F - 43

FIG. NO. 1

FORM DB MT-?
REV APR 1965



DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION

GRAIN SIZE DISTRIBUTION

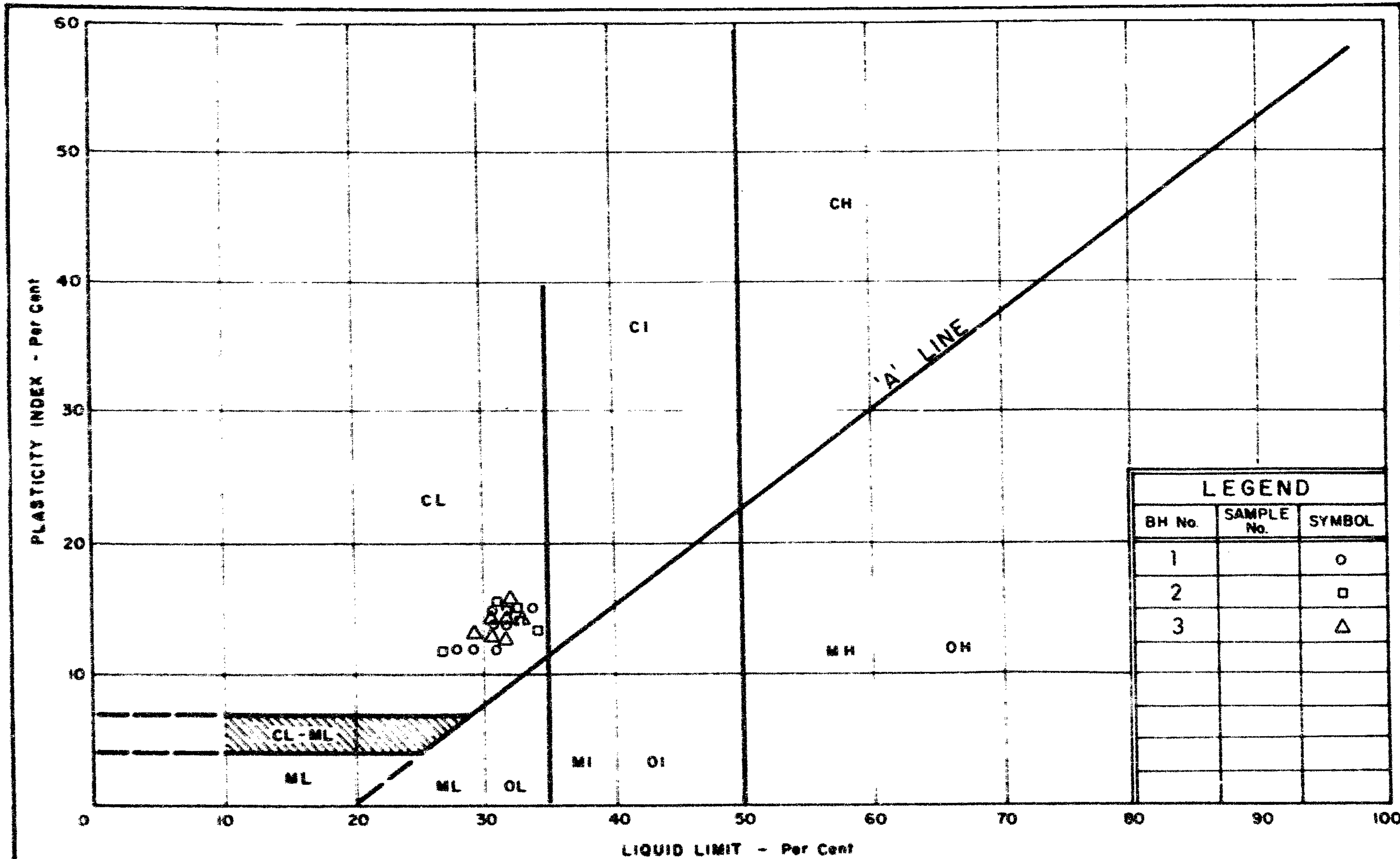
SILTY SAND

W.P. No. 144 – 63 – 01

JOB No. 69 – F – 43

FIG. NO. 2

FORM OB-MT-[illegible]2
REVISED JULY, 1966



DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION

PLASTICITY CHART
CLAYEY SILT

W.P No. 144 - 63 - 01
JOB No. 69 - F - 43
FIG. NO. 3

SHEAR STRENGTH VS. ELEVATION

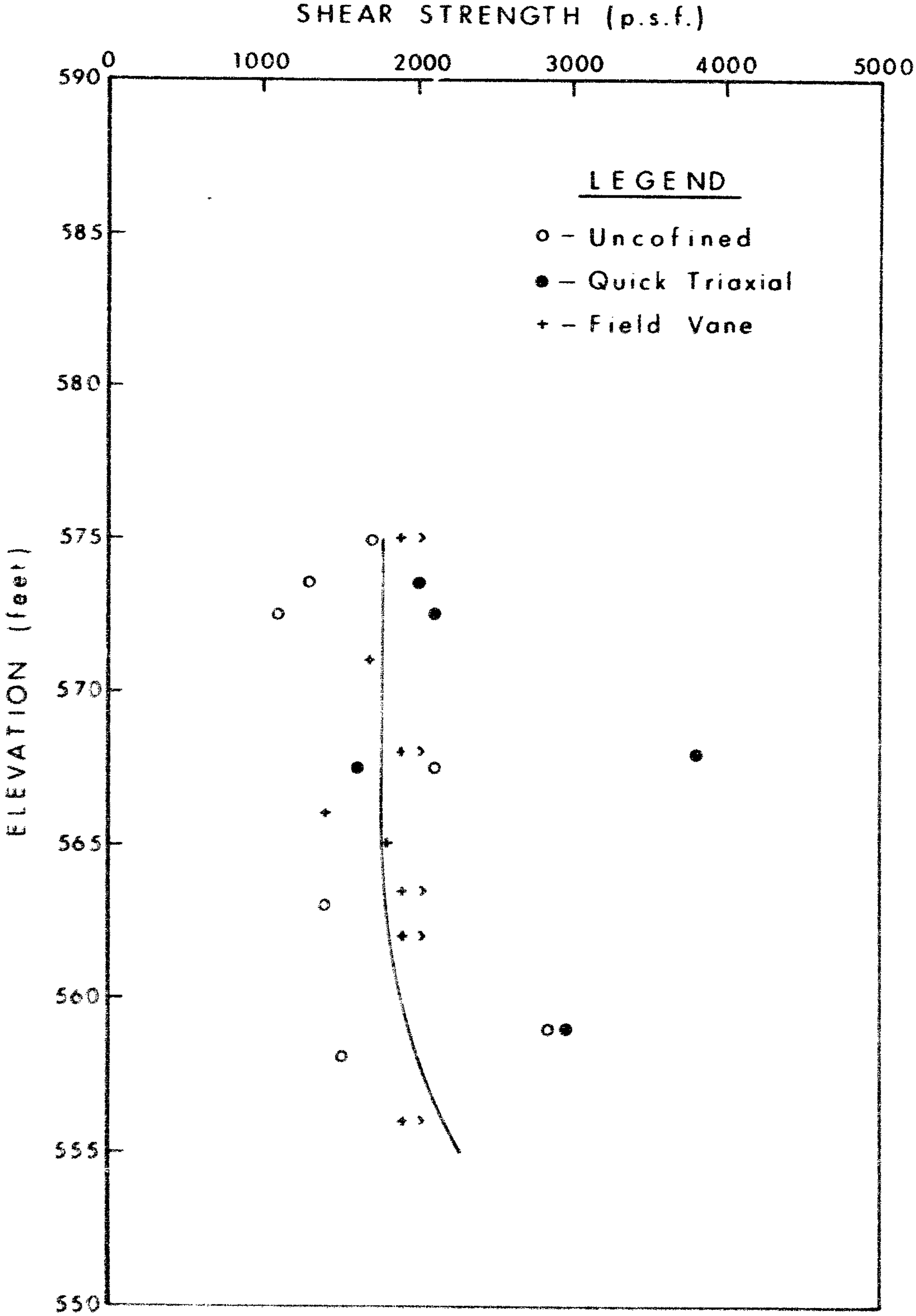


Fig. 4

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 :-

CONSISTENCY	'N' BLOWS/FT.	c LB./SQ.FT.
VERY SOFT	0 - 2	0 - 250
SOFT	2 - 4	250 - 500
FIRM	4 - 8	500 - 1000
STIFF	8 - 15	1000 - 2000
VERY STIFF	15 - 30	2000 - 4000
HARD	> 30	> 4000

DENSENESS	'N' BLOWS / FT.
VERY LOOSE	0 - 4
LOOSE	4 - 10
COMPACT	10 - 30
DENSE	30 - 50
VERY DENSE	> 50

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 INTERCEPT } IN TERMS OF EFFECTIVE STRESS $\tau_f = c' + \sigma' \tan \phi'$
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION } IN TERMS OF EFFECTIVE STRESS $\tau_f = c' + \sigma' \tan \phi'$
c_u	APPARENT COHESION } IN TERMS OF TOTAL STRESS $\tau_f = c_u + \sigma \tan \phi$
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION } IN TERMS OF TOTAL STRESS $\tau_f = c_u + \sigma \tan \phi$
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 (YOUNGS MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

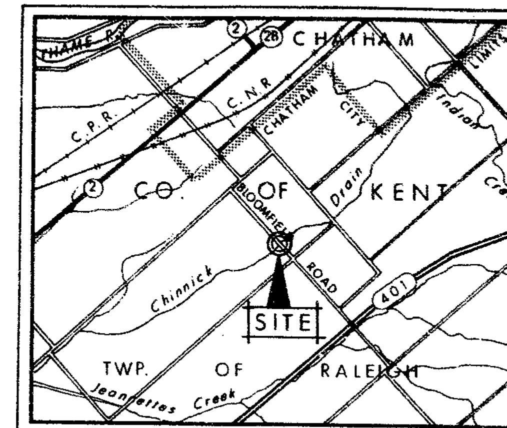
d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_o	COEFFICIENT OF EARTH PRESSURE AT REST

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

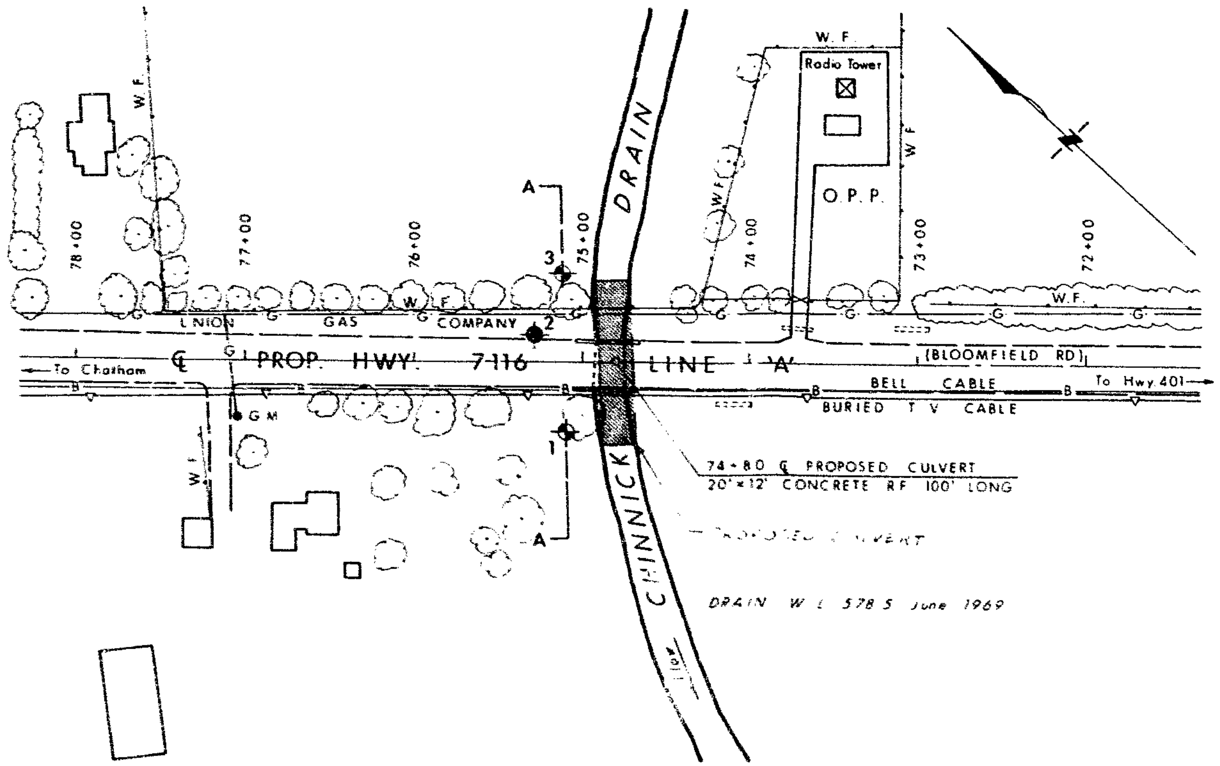
H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
β	ANGLE OF SLOPE TO HORIZONTAL



KEY PLAN

SCALE IN MILES

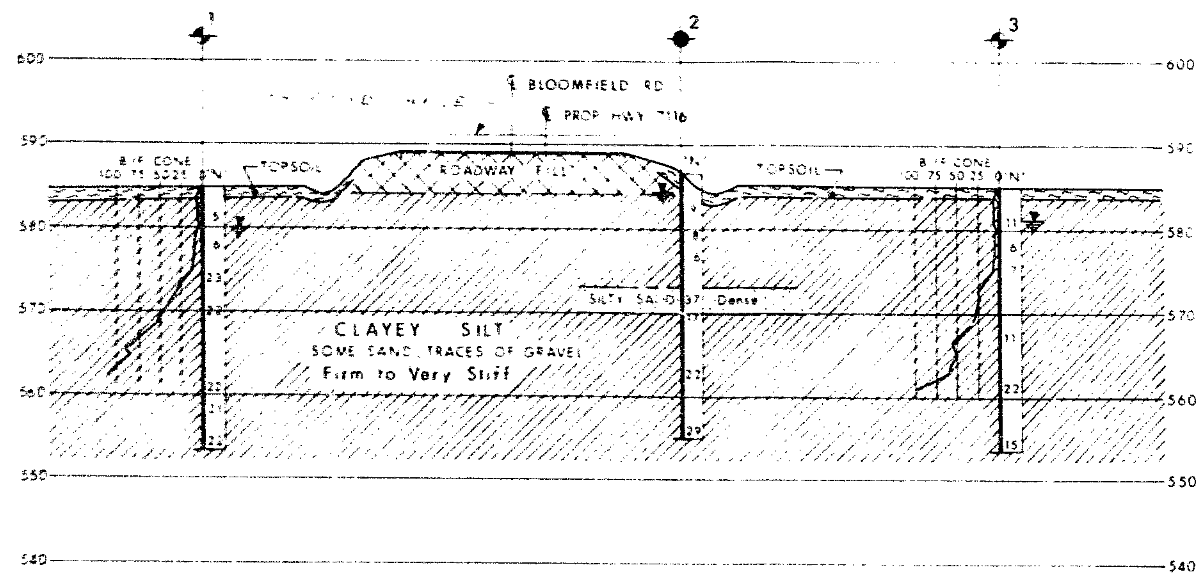
1 .5 0 1 2 Miles



PLAN

SCALE

50 25 0 50 100 FT.



SECTION A-A

SCALE

10 5 0 10 20 30 FT

LEGEND

- Bore Hole
- Cone Penetration Hole
- Bore & Cone Penetration Hole
- Water Levels established at time of field investigation. June 1969

NO.	ELEVATION	STATION	OFFSET
1	584.7	75 + 10	41' LT
2	586.3	75 + 28	16' RT.
3	585.0	75 + 12	54' RT.

– NOTE –

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS – ONTARIO

MATERIALS & TESTING OFFICE – FOUNDATION SECTION

CHINNICK DRAIN

KING'S HIGHWAY NO. PROP 7116 LINE 'A' DIST. NO. 1

CO. KENT

TWP. RALEIGH LOT 18 CON. VI

BORE HOLE LOCATIONS & SOIL STRATA

SUBM'D G. A.	CHECKED	W.P. NO. 144-63-01	M.&T. DRAWING NO. 69-F-43A
DRAWN G. P.	CHECKED	JOB NO. 69-F-43	
DATE Sept. 10, 1969		SITE NO	BRIDGE DRAWING NO.
APPROVED		CONT. NO.	

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. K. L. Kleinsteiber,
Municipal Bridge Liaison Engr.,
Bridge Office,
Admin. Bldg.

From: Foundation Section,
Materials & Testing Office,
Room 107, Lab. Bldg.

Date: June 12, 1969

Our File Ref.

In reply to

Subject: Bridge over Big Creek
Lots 18 - 19, Con. VII
Township of Windham -
District No. 2 (London)

As requested by you, we have reviewed the Consultant, William Trow's report concerning the bottom heave which occurred during excavation of the north abutment foundation of Big Creek Bridge. At the time, the prevailing river water level was some 5 ft. higher than normal at El. 91.5, the foundation level was at El. 77.0, and steel interlocking sheet piling had been driven to El. 68.0. During his second field investigation, the Consultant discovered the existence of an artesian pressure at the surface of the bedrock with a head to El. 113 ±. This artesian pressure was not discovered in his original investigation. It has not yet been determined, however, whether excess pressure exists within the silt stratum in which the footing was to be located, although this seems very likely to be the case.

If an excess pressure exists within the silt layers, then further excavation into this material will again result in bottom heave even though the river level is down to normal - i.e., El. 86.0. Extending sheeting would, in this case, be of no advantage unless it could be driven into an impermeable cohesive layer. It now appears that the most practical solution to the problem would be to support both abutments on piles driven to bedrock, and to construct the pile caps at a level at which 'boiling' would not occur - i.e., about 3 ft. higher than the original spread footing level. The sheeting should still be driven as before and should be permanently left in place to ensure the stability of the backfill material behind the abutments as well as to provide temporary protection during construction of the abutment foundations. With regard to piles, it is believed that 12-3/4-inch O.D. steel tubes would be suitable if driven to bedrock: the maximum allowable load for this type of pile may be assumed for design purposes. Since the piles will be driven closed end into an aquifer under a high artesian pressure, it is recommended that the shoes be constructed flush with the pile and not projecting as is normally the case. In this instance, it is believed that steel tubes would be superior to steel H-piles, since the latter

..... 2

Mr. K. L. Kleinsteiber,
Municipal Bridge Liaison Engr.,
Bridge Office,
Admin. Bldg.

June 12, 1969

might allow water to percolate through voids formed in the soil between the flanges.

The present excavation for the north abutment should be backfilled with granular material up to the level of the proposed pile cap - i.e., El. 80 ± prior to driving piles.

The foregoing has been discussed with the Soil Consultant and with Mr. D. Vallee, the Bridge Consultant. They are in agreement with our conclusions and recommendations.

K G Selby

KGS/MdeF

K. G. Selby,
SUPERVISING FOUNDATION ENGINEER
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

cc: Messrs. H. C. Dernier
D. Vallee

Foundations Files
Gen. Files

WM. D. COLBY, B. Sc.
CIVIL ENGINEER

MEMBER OF SOCIETY OF PROFESSIONAL
ENGINEERS OF ONTARIO

TELEPHONE 354-1280

45 GRAND AVE. WEST
CHATHAM, ONTARIO

February 14, 1969.

Giffels Associates Limited,
60 Adelaide Street, East,
TORONTO 1, Ontario.

Re;-Bloomfield Road from Highway # 401
to Park Ave. and Chinnick Drain,
Township of Raleigh.
W.P. 144-63-01 Your Job # C 4082

Gentlemen:

The Township of Raleigh asked me to write you re-the Chinnick Drain Culvert and the required size. This Drain will soon be cleaned out and improved, to provide better drainage for the area.

The approximate size of this Culvert is approximately 17.5 foot span and a height of 8.5 feet, which is 150 square feet of usable cross-section for drainage. If this Culvert is to be replaced with a new one, the usable cross section should be at least 180 square feet. I would prefer to hold the bottom of the existing deck of the Culvert at or near the same elevation, and widen the span to get the required cross-section.

As this drain is very important to this part of Raleigh Township and part of the City of Chatham, it is very important that footings be used in the design and that the top of the footing should be at least three feet below the existing bottom of the drain. This drain will be cleaned this spring and the existing bottom will be lowered approximately 1 ½ feet, so if the top of the footings is kept 3' below the existing bottom, we will still have some depth to lower the drain bottom, if it is ever required. It is imperative, that nothing be placed in the drain bed to stop the deepening of the drain, or to impede the flow of water.

Yours truly,

WM. D. COLBY 111 ENGINEERING LTD.,

Wm D. Colby

Wm. D. Colby, B. Sc.,
Consulting Civil Engineer.

Giffels ASSOCIATES LIMITED

68 ADELAIDE STREET EAST, TORONTO 1, ONTARIO TEL. 365-3781

April 21, 1969

Mr. J.R. Roy, P.Eng.
Regional Materials Engineer
Department of Highways, Ontario
335 Saskatoon St.
London, Ontario

Re: W.P. 144-63-01, Hwy.7116
West Access to Chatham from
Hwy. 401, Our Job C4082

Dear Sir :

We enclose a copy of a letter we have received from Mr. Wm. D. Colby regarding the Chinnick Drain Culvert at Sta. 74+83 on Bloomfield Road.

It appears probable that a new culvert will be needed and we expect that a 20' x 12' rigid frame-open type, as per DD-1214-B, will be the most suitable. We therefore enclose a sketch showing how the bottom of the footing will probably be at El.571, a distance of 18 ft. below the existing road surface.

Would you kindly arrange for a check on the bearing capacity at this elevation.

Yours very truly

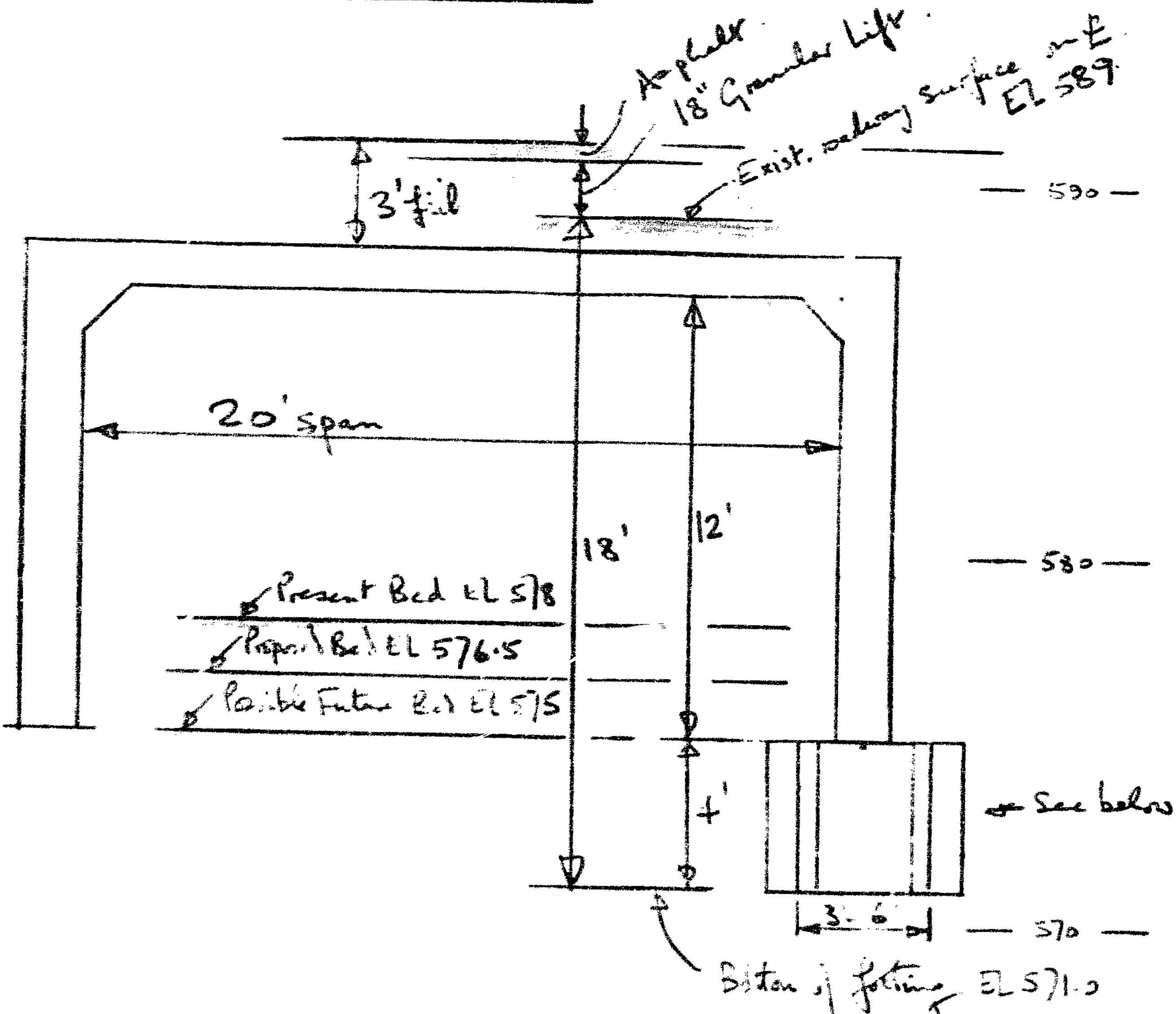
GIFFELS ASSOCIATES LIMITED

G.R. Tilly, P.Eng.
Project Manager

cc: W. Zonnenberg (Att: D.D. Murray) DHO
G.N. Farantatos

enc.

CHINNICK DRAIN CULVERT.



for 3000 psf.	footing width ≈	5'-3"
3500	≈	4'-0"
4000	≈	3'-3"
4500	≈	2'-6"

Suggest check soils for a 3'-6" wide footing. This requires D.L allowable bearing capacity of about 3700 psf.