30L-45

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

MEMORANDUM

TO: Mr. T. J. Kovich, (2)
Regional Materials Engineer,
Central Region,
3501 Dufferin St., Downsview.

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

ATTENTION:

DATE: July 17, 1972.

OUR FILE REF.

IN REPLY TO JUL 19 1972

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For
Failure of Approach Embankments
Overhead Structure at the Crossing of
Hwy. #140 and C.N.R.
Township of Humberstone, County of Welland
W.O. 72-11025 --- W.P. 60-69-02
Contract ~~72-2~~ 70-212

30L-45
GEOCRES No.

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

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

A. G. Stermac,
PRINCIPAL FOUNDATIONS ENGINEER.

AGS/ao
Attch.
cc: Messrs. F. G. Allen
D. W. Farren
A. Rutka
B. R. Davis
D. M. Hopper
P. J. Harvey
C. R. Robertson (Attn: D. Waller - 2)
G.C.E. Burkhardt
B. J. Giroux
G. A. Wrong
B. A. Singh

Foundations Files
Documents

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1. INTRODUCTION:

This Office carried out a sub-surface investigation for the then proposed structure at the crossing of Hwy. #140 and the C.N.R., in the Township of Humberstone, County of Welland, during October and November, 1968. Recommendations pertaining to the design of foundations, as well as the stability and settlement considerations associated with the approach fills were presented in Report No. W.J. 68-F-73, dated December 4, 1968.

The south and north approach fills to the structure were constructed in May, 1971. On July 5, 1971, major instability occurred along the south approach; instability also developed along the north approach during the latter part of August of the same year. Following these failures the Foundations Office was requested to carry out an investigation of sufficient scope to aid in the assessment of what remedial measures need be taken to ensure the stability of the approaches. The request was presented by Mr. T. J. Kovich, Regional Materials Engineer, Central Region.

Visual observations have been made by both personnel from the Central Region Materials Section as well as the Foundations Office. In addition, sub-surface investigations have been carried out, at two different periods, in order to assess the physical properties of the fill and the parent subsoil.

This report presents all the visual and factual data accumulated, prior to and following the failure. In addition, remedial measures are proposed which should ensure the long-term stability of the failed sections.

2. DESCRIPTION OF THE SITE AND GEOLOGY:

The site is located some 200 feet east of the intersection of Forkes Rd. and Kleinsmith Rd., approximately 2 miles east of Welland Junction. At this location the C.N.R. tracks, which run parallel to Forkes Rd., are about 100 feet to the north. The tracks are elevated about 4 feet above the surrounding ground level on a 25 feet wide embankment. Forkes Rd. is a two-lane, paved county road; the profile grade of this road is about 1 to 2 feet above the surrounding terrain. Shallow ditches run along both sides of Forkes Rd., as well as the C.N.R. embankment.

The surrounding area is generally flat-lying; the surficial drainage is very poor. The land to the north of the site is used for farming purposes, while the land to the south is, at present, abandoned.

Physiographically, the site is situated in the region known as the Haldimand Clay Plain. In this area the subsoil is composed of extensive, mainly glacial-lacustrine deposits, laid down in glacial Lake Warren, during the Wisconsinan Age. These deposits are composed of stratified silts and clays, and are generally underlain by a basal glacial till sheet, which in turn, is followed by dolomitic linestone bedrock of the Salina formation, Silurian period.

3. CONSTRUCTION DETAILS:

3.1) Structure Scheme:

The 42 feet wide overhead structure at the crossing of Hwy. #140 an the C.N.R. has three spans (40'-50'-40'). In the vicinity of the crossing the profile grades of the

C.N.R. and Hwy. #140 are at elevation 586 and 614, respectively. At these grades the maximum height of the approaches, in the longitudinal and transverse directions, are 26 and 32 feet, respectively.

The structure and related approaches have been constructed in an area where the predominant stratum is a stiff to hard clay to silty clay. The thickness of this deposit ranges from 73 to 82 feet. This cohesive stratum is underlain by a thin glacial till deposit then dolomite bedrock.

The two piers for the structure are founded on spread footings, located at elevation 575.5 - i.e., in the upper very stiff portion of the clay. It is understood that they were designed using an allowable bearing value of 2.5 t.s.f. The abutments are supported on hexagonal section (maximum dimension 16") pre-stressed concrete piles which supposedly are driven to bedrock. These piles were designed using an allowable load of 100 tons/pile.

The fill used to form the approaches was composed of a clay to silty clay, which was obtained from borrow pits located in close proximity to the site. This fill material is of similar geologic origin to that of the parent cohesive deposit across this site.

3.2) <u>Observations During Placement of Fill</u>:

Visual observations were made by Regional and District personnel during the placement of the fill to form the approaches to this structure, these are summarized in the paragraphs to follow.

Fill placement along the north approach commenced on May 4, 1971; the fill was placed to final grade. The fill placement along the south approach commenced on May 21, 1971; the height during this stage was approximately 4 to 5 feet below final grade. All the fill was placed directly on the existing terrain - i.e., the topsoil was not removed.

The surficial drainage, in the vicinity of the approaches,

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particularly the south, was, at the time of fill placement, generally poor. Numerous ponds of water existed in this area. The fill material placed in the lower portion of the embankment section, along the south approach, appeared to have a higher natural water content than the fill placed elsewhere on this site. These conditions would make it difficult to adequately compact the fill in the lower portion of the embankments, particularly along the south approach. Further, the availability of free water would tend to lead to softening of the clay fill with time.

4. DESCRIPTION OF FAILURE:

i) South Approach:

On July 5, 1971, major instability developed along the south approach, specifically between Stations 211 + 00 and 216 + 50. In this area the fill subsided about 2 feet. Longitudinal tension cracks, up to 3 feet wide, opened up within the main core. Further, bulging was noticed at the toe of the fill; the maximum extent of this bulge was 3 feet beyond the original geometry.

On September 14, 1971, the embankment was repaired. The revised sections, from Station 211 + 50 northerly to the south abutment incorporated 20 feet long mid-height berms. In addition, it was recommended that the surficial organic material, located at the toe of the original section, be sub-excavated to a minimum depth of 2 feet. The sub-excavation so formed was then to be backfilled with acceptable compacted earth material (refer to the memo written by Mr. M. Devata, Supervising Foundations Engineer, dated August 12, 1971).

A second failure occurred along this approach on October 1, 1971. The failure originally developed on the west side of the embankment, eventually enveloping the east side. The magnitude and extent of the subsidence, tension cracks and toe bulging were similar to those discussed previously.

ii) North Approach:

The north ramp initially failed on August 30, 1971, approximately 1-1/2 months after the south ramp showed signs of distress. The degree of distress was, however, less than that along the south approach. The north approach was repaired on September 22, 1971, using the scheme adopted on the south approach. A second less severe failure occurred in the first week of January, 1972. The west side of this approach showed more distress than the east. Information,provided by District personnel, has indicated that the berm constructed on the east side, following the initial failure, was longer than that on the west.

5. FIELD AND LABORATORY INVESTIGATION:

5.1) General:

Four boreholes were put down for the original foundation investigation at this site, during October and November of 1968 (No.'s 1, 2, 3 and 4). Following the initial failure of the south approach fill in July, 1971, seven boreholes were put down in strategic areas (100 series numbering). In addition, six borings were put down (200 series numbering) in February, 1972, to investigate the reasons for the second failure. Representative samples were obtained during the various investigation phases. Groundwater level observations were carried out, throughout this period, in piezometers installed in both the fill and parent subsoil. In addition, the groundwater levels in the open boreholes, at the remaining locations were recorded.

The locations and elevations of all of the boreholes, which were surveyed by District #4 personnel, are shown on Drawing No. W.O. 72-11025A. A typical stratigraphical section across the site, inferred from the boring data, is plotted on this drawing.

All the samples were subjected to a visual examination in the field and subsequently in the laboratory. Following this examination, laboratory testing was carried out on selected representative samples. This testing is summarized on the borelog

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sheets and Figures 1 and 2 contained in the Appendix of this report.

5.2) Subsoil and Bedrock Conditions:

5.2.1) Fill (Silty Clay to Clay):

A number of the borings, put down following the failures, penetrated the fill placed along the approaches; the maximum depth of fill encountered was of the order of 32 feet. The fill is composed of a clay to silty clay, with a trace of sand.

Atterberg limit testing, carried out on samples from the fill, indicate that the material has a plasticity in the intermediate to high range. The natural moisture content within the fill varied from 23 to [illegible] percent, in general, there is an increase in the lower portion of the fill immediately above its contact with original ground. The compaction characteristics of the fill material were determined by carrying out two laboratory standard Practor Compaction Tests; the results from this testing are summarized on Figure #1 in the Appendix of this report. The values obtained from this testing are summarized below.

Optimum Compaction Bulk Density - 122 to 122.5 p.c.f.
Optimum Compaction Water Content - 24%

Referring to thise values it can be seen that in many areas throughout the fill, particularly in the lower zones, the inplace moisture content is considerably higher than the optimum compaction water content. This is graphically illustrated on Figure #3 appended to this report.

The undrained shear strength properties of the fill were determined in the field as well as in the laboratory. This testing gave values which ranged from 1,200 p.s.f. to greater than 2,000 p.s.f. This would indicate that the consistency of the major portion of the fill ranges from stiff to hard. The standard penetration testing carried out gave 'N' values which corroborate the range in consistency quoted above.

A laboratory programme was carried out to determine the engineering properties of the cohesive fill in terms of effective stresses. This was done by carrying out a series of isotropical consolidated undrained triaxial compression tests, in which the excess pore water was monitored (C.I.U. test). The results of this testing, which are plotted on Figure #2, are summarized below.

Apparent Effective Cohesive Intercept (c') - 0-120 p.s.f.
Apparent Effective Angle of Internal Friction (ø') - 23°

5.2.2) <u>Clay to Silty Clay</u>:

The fill is underlain by a clayey topsoil, approximately 1 foot thick. The topsoil is followed by a 73 to 82 feet thick stratum, composed of a clay to silty clay with a trace of sand and gravel. The upper 15 to 20 feet of the deposit is brown in colour; it is considered that this zone has been desiccated. Numerous layers and seams of sand and silt, up to 3 inches thick, are present throughout the stratum.

The physical properties of the overall stratum, as determined by field and laboratory testing, are summarized on the borelog sheets; a brief resume follows.

Atterberg limit tests carried out on samples of the cohesive material indicate that it is inorganic with a plasticity in the intermediate to high range. The consistency of the overall stratum, as determined by the undrained shear strength testing, varies from hard to very stiff, in the upper 15 to 20 feet (desiccated zone), decreasing to very stiff to stiff with depth.

Effective stress testing was carried out on a sample from the parent cohesive stratum using the procedure outlined in sub-section 5.2.1). The results of this testing, which are plotted on Figure #3, are summarized below:

Apparent Effective Cohesive Intercept (c') - 280 p.s.f.
Apparent Angle of Internal Friction (ø') - 25°

The compressibility characteristics of this subsoil were determined by laboratory consolidation testing, the results of which were summarized in report W.J. 68-F-73. This testing indicated that the clay is preconsolidated by about 2 to 4 t.s.f. in excess of the existing overburden pressure. It is estimated that the upper 15 to 20 feet of the stratum (desiccated crust) is preconsolidated by something in excess of 5 t.s.f.

5.2.3) Lower Deposits:

The cohesive stratum is underlain by a basically cohesive glacial till composed of a clayey silt with sand and gravel. The thickness of the glacial till varies from 1 to 6 feet. The standard penetration resistance or 'N' values vary from 29 blows/ft. to well over 100 blows/ft., indicating that the consistency of the cohesive deposit ranges from very stiff to hard.

The glacial till is underlain by a grey dolomite bedrock. The surface of the bedrock was encountered between elevations 497 and 500; which corresponds to depths of from 79 to 85 feet below existing ground surface.

6. GROUNDWATER CONDITIONS:

Groundwater level observations have been carried out during the period of the investigation in i) sealed piezometers installed in the fill as well as in the cohesive stratum, and ii) the open holes at the remaining boring locations. These observations are recorded on the borelog sheets and summarized on Drawing No. 72-11025A. The results indicate that, prior to placement of the fill, the groundwater level in the cohesive stratum ranged from elevation 576 to 579 - i.e., some 3 to 5 feet below ground surface. The piezometric groundwater level in the glacial till, underlying the clayey silt stratum, ranged from elevation 554 to 558 - i.e., some 25 feet below ground level. These observations would indicate that there is some downward drainage from the upper cohesive stratum down into the glacial till deposit.

Following placement of fill the water level, in the parent cohesive subsoil, rose to elevations between 588 and 603.

The variation is an indication of the build-up in excess pore water pressure due to the fill loading.

Water level observations, carried out in piezometers installed in the fill have given an erratic pattern. The results would seem to indicate that the upper portion of the fill is dry. The water level in some isolated areas of the lower zone of the fill (immediately above the topsoil) rose to about elevation 598 - i.e., a level some 17 feet above the original ground surface (refer to B.H. #210). This is probably due to the fact that this zone was in communication with free water during fill placement.

7. DISCUSSION AND RECOMMENDATIONS:

7.1) Reasons for Failure:

As discussed in detail in Section 4) the south approach exhibited more distress than the north. This being the case, the discussion contained herein will pertain primarily to the former. The instability could have originated as either a deep-seated rotational type of failure within the parent cohesive foundation subsoil, or alternatively a failure confined to the new fill. These two possibilities will be discussed in detail in the following paragraphs.

i) Deep-Seated Failure in Foundation Subsoil:

Stability analyses, carried out prior to the original construction of the embankment (refer to Report W.O. 68-F-73), have indicated that fills of the order of 30 feet in height will be stable, with respect to a deep-seated failure within the foundation subsoil, provided i) standard 2:1 slopes are employed and ii) suitable earth fill is used and it is properly compacted. These computations were carried out using a total stress approach where the analyses are based on the undrained shear strength of the fill and parent cohesive subsoil, as well as the magnitude of the induced loading. The pre-failure undrained shear strength profile for the parent subsoil is plotted on Figure #1 of the aforementioned report. In addition, the stability was checked in terms of effective stresses. In this method the stability is governed by the stress-strain characteristics of the fill and parent cohesive stratum as well as the buildup and eventual dissipation of excess pore water pressure due to load application. These computations also provided an adequate factor of safety with respect to a deep-seated failure (F.S. $\geqq$ 1.3).

The borings, put down in the affected areas following failure, have indicated that the shear strength pattern throughout the parent cohesive stratum has remained basically unaltered. If the failure was deep-seated, then, in the critical areas bounded by the surface of the failure envelope, the silty clay should have been remoulded due to shear deformation; this would have led to some reduction in strength in these zones. Since this was not found to be the case, it is inferred that the instability must have been attributed to something other than a failure within the parent cohesive foundation subsoil.

ii) Failure Within New Fill:

If the failures are not of a deep-seated nature, then they must have originated within the new fill. As mentioned in Subsection 3.2) the fill, in the lower portion of the embankments, was placed and compacted in a wet environment. Further, the topsoil was not stripped. It is inferred that these factors probably led to the formation of a softened zone which encompasses the lower 3 to 4 feet of the fill as well as the natural topsoil cover. The failure surface would then tend to be located within this zone, which would have been a path of least resistance. This mode of failure will be discussed in detail in the paragraphs to follow.

The stability of a critical section along the south approach (at Station 214 +50), prior to the original failure in July 1971, was studied in detail using the effective stress approach developed by Messrs. Bishop and Morgenstern.*

*Bishop, A.W. and Morgenstern, N., "Stability Coefficients for Earth Slopes," Geotechnique, Vol. 10, No. 4, 1960.

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The following were assumed for computational purposes.

a) Fill Details (Immediately Prior to Failure):

Fill Height - 24 ft. (4 feet below proposed final grade).

Average Slope - 2-1/4:1

b) Engineering Parameters (Predicted From Laboratory Testing Results):

	Fill Lower Softened Zone	Remaining
Apparent Effective Cohesive Intercept (C')	0	120 p.s.f.
Apparent Effective Angle of Internal Friction (ϕ')	23°	23°

Average Pore Pressure Ratio (r_u) = 0.25
where $r_u = \frac{\Delta u}{\gamma H}$

Δu - excess pore water pressure (p.s.f.)

γ - bulk unit weight of fill (p.c.f.)

H - height of fill (ft.)

The results of the computations have indicated that the fill section itself was in a limiting state of equilibrium (F.S. $\leqq$ 1.0) during this critical period. As such it is believed that the failure could have originated within this lower softened zone of the fill-topsoil complex.

An extension of these computations have indicated that, in order to ensure the long-term stability of this section, when constructed to final grade (height 28 feet), the side slopes should be constructed no steeper than 3-1/2:1 overall. In these computations it was assumed that a minimum factor of safety of 1.3 should be obtained to ensure the stability of the section being investigated.

7.2) Recommended Remedial Measures:

In order to ensure the long-term stability of the approach fills at this site it will be necessary to employ flatter overall slopes; this could be accomplished by constructing counter balancing mid-height berms. In addition, it is recommended

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that a reinforced zone, composed of either rock fill or a granular type of material, be placed at the toe of the reconstructed section. The final selection of the material to be used in this toe zone is to be decided upon by the Central Regional Materials Section. This reinforced zone, which should extend a minimum of 2-1/2 feet into the fill and 2-1/2 feet into natural ground (maximum thickness 5 feet), will serve two main purposes; namely, it will,

i) provide a zone of higher shear strength and thus improve the stability of any potential failure surfaces passing through this area, and

ii) confine any softened material located beneath the core of the embankments, thus preventing the tendency for such soils to undergo large lateral strains.

Stability analyses were carried out to determine what berm lengths would be required, for various fill heights, when such a composite section is employed. These analyses were based on a method developed by N. Janbu*. Using this method, the critical surface need not be cylindrical in shape, instead it may assume any general configuration and thus maximize its length within zones of relative weakness. Based on these computations a revised geometry is recommended for sections along the south approach extending from Station 211 + 00 northerly to 217 + 00; these are shown on Drawing No. 72-11025B. Referring to this drawing, it can be seen that:

i) the maximum length of berm recommended is 35 feet (Stations 216 + 00 and 217 + 00 where the height of fill is of the order of 32 feet),

ii) all slopes are 2:1. The berm, however, should slope towards the top at 20:1.

iii) the reinforced toe is to extend from Station 212 + 00 to 217 + 00. The recommended dimensions of this toe are shown on the sections presented on Drawing 72-11025B.

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*Janbu, N. "Stability Analysis of Slopes with Dimensionless Parameters," Harvard Soil Mechanics Series, No. 46, 1954.

In order to relieve the build up of excess hydrostatic groundwater pressure positive drainage measures should be provided within the reinforced toe.

The west side of the north approach should be reconstructed using the procedures outlined for the south approach. The berm lengths and extent of the reinforced toe, should be based on the requirements specified for the various fill heights along the south approach. As discussed in Subsection 4. ii) the east side of the north approach appeared to be stable following the second failure. This is inferred to be due to the fact that the berms constructed on the east side, following the first failure, were longer than those constructed on the west. This being the case it is believed that initially the reinforced toe section need not be installed along the east side. We would recommend, however, that this area be kept under observation during the reconstruction period. If any signs of distress are noticed, they should be brought to the attention of this Office so that additional remedial measures can be initiated to ensure the overall stability of this section of the north approach.

All loosened and disturbed fill material, located in areas affected by mass slumping and major tension cracks, along both approaches, should be removed prior to placing new fill in these areas. If so desired, this excavated material could be used to flatten the outer portion of the bermed slopes.

This report should be read in conjunction with a letter, dated August 12, 1971, which was written by Mr. M. Devata, Supervising Foundations Engineer, and addressed to Mr. D. Waller, Construction Engineer, District No. 4 (Hamilton).

8. MISCELLANEOUS:

The field work, performed during the periods of July 13 to 20, 1971, and February 3 to 17, 1972, was carried out under the supervision of Mr. S. A. Ahmad, Project Foundations Engineer.

The equipment used was owned and operated by Master Soil Investigation Ltd. and Dominion Soil Investigation Ltd., both of Toronto.

This report was written by Mr. B. T. Darch, Senior Foundations Engineer and reviewed by Mr. M. Devata, Supervising Foundations Engineer.

B.T. Darch

B. T. Darch, P. Eng.

REGISTERED PROFESSIONAL ENGINEER
B. T. DARCH
PROVINCE OF ONTARIO

M. Devata

BTD/ao
M. Devata, P. Eng.

July 17, 1972.

APPENDIX I

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 1 (68-F-73)

FOUNDATION SECTION

JOB 72-11025

W.P. 60-68-02

DATUM Geodetic

LOCATION Sta. 216+50 Ø East Side Hwy. o/s 25' Rt.

BORING DATE Oct. 17 - Nov. 1, 1966

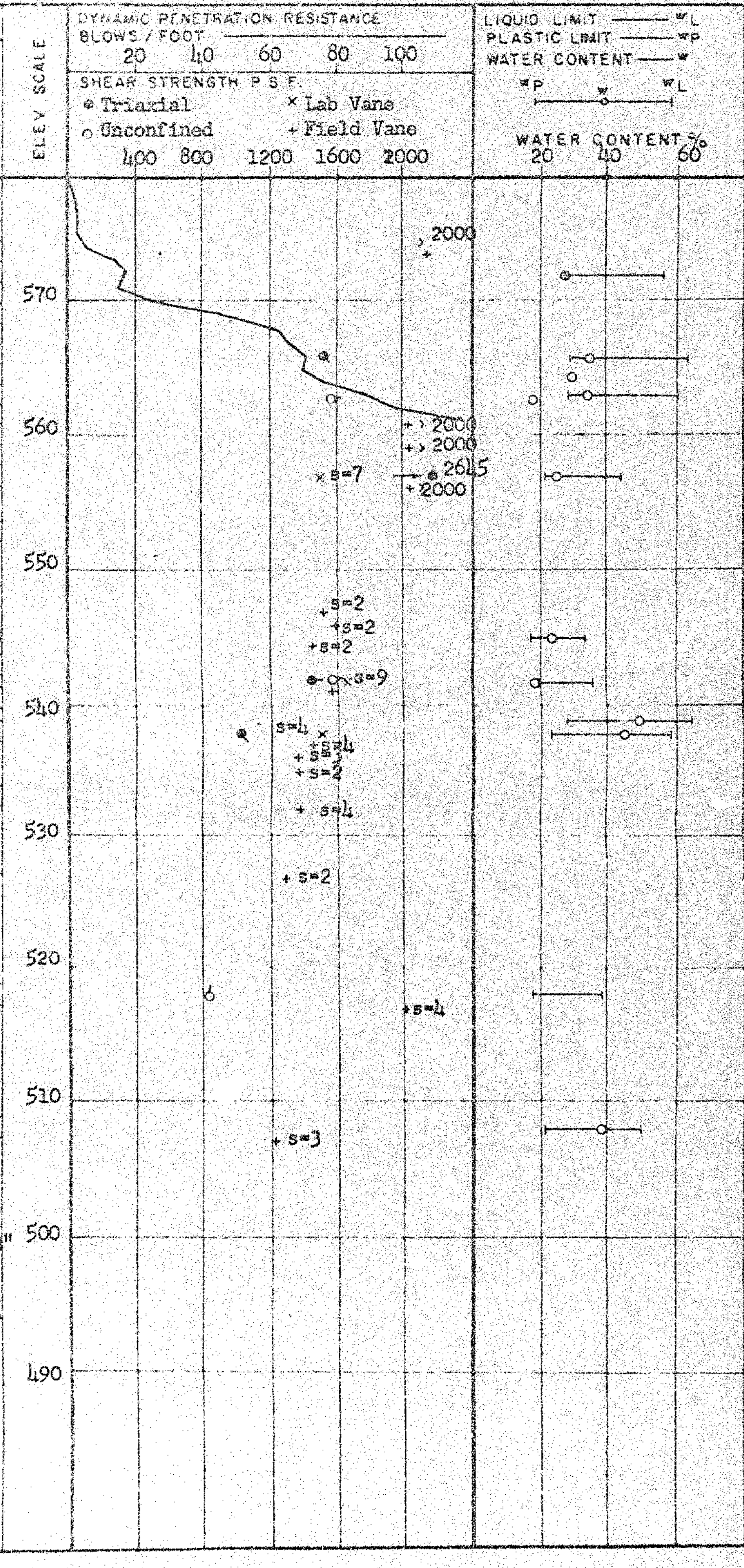
BOREHOLE TYPE Cont. Flight auger & diamond drill

ORIGINATED BY WH

COMPILED BY WH

CHECKED BY [illegible]

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT	BULK DENSITY P.C.F.	REMARKS Gr.Sa.Si.C
579.0	Ground Level					
0.0		1A	SS	17		576
		2	SS	24		0 1 29 70
	Silty clay to clay with trace of sand & gravel with occ. silt layers up to 3" thick throughout. Pocket of silty sand with gravel 3" thick at elev. 539	3	SS	24		
		4	TW	PH	116	0 1 33 66
		5	SS	17		
		6	3" TW	PH	114	Piez.# Tip.elev.56
		7	SS	21		
		8	TW	PH	127.5	554.
		9	SS	20		
		10	TW	PH		
546.0 / 33.0	Clayey silt zone	10A	TW	PH		
		11	SS	9		
540.0		11A	TW	PH	127	
39.0	Brown to reddish brown Stiff to very stiff.	12A	TW	PH		
		12	TW	PH	111	0 1 22 77
		13	SS	16		
		14	TW	PH		
		15	TW	PH	121	
		16	SS	12		
500.5 / 78.5	(Glacial Till) Very dense Sand with gravel & fragments of bedrock.	17	SS	40/4"		Piez.# Tip Elev. 449
79.3 / 494.6	Dolomite bedrock with gypsum lenses. Occ. layers of calcareous shale. Seams.	18	BXL RC	80% Rec.		
84.4	End of Borehole					



DEPARTMENT OF HIGHWAYS – ONTARIO

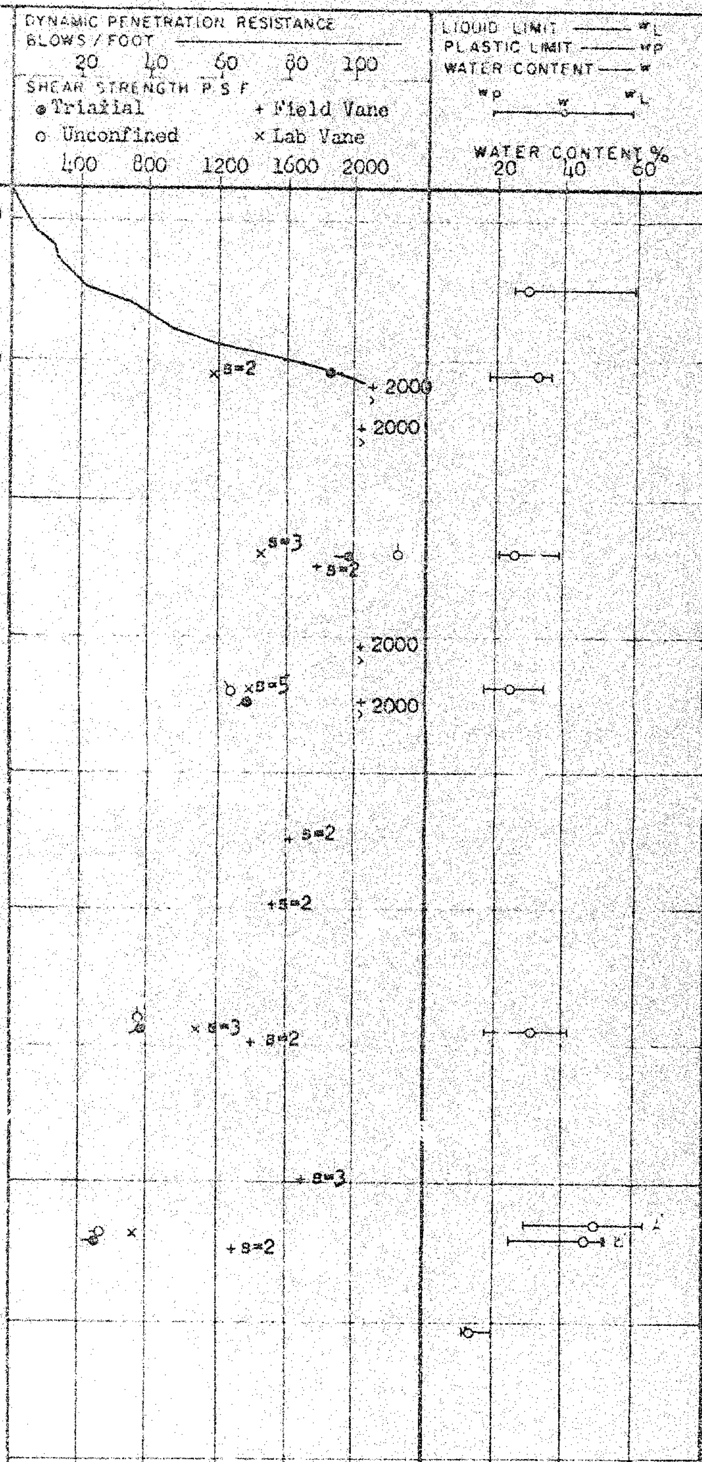
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 2 (68-F-73)

FOUNDATION SECTION

JOB 72-11025	LOCATION Sta. 219+10 Ø East Side Hwy. o/s 38' Lt.	ORIGINATED BY WH	
W.P. 60-68-02	BORING DATE Oct. 23-29, 1968	COMPILED BY WH	
DATUM Geodetic	BOREHOLE TYPE Cont. flight auger & diamond drill	CHECKED BY	

ELEV. / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS Gr.Sa.Si.Cl.
582.0	Ground Level						
0.0	Clayey silt to clay with trace of sand & gravel	1	SS	26	580		
		2	SS	21			0 1 (99)
		3	SS	21			
	Occ. very thin grey silt seams containing clear gypsum crystals above elev. 559.	4	TW	PH	570	116	
		5	SS	14			
		6	TW	PH			
		7	SS	24	560		▽558
		8	TW	PH		128	
		9A	SS	10	550		
		10	TW	PH		125	
					540		
		11	TW	PH			
		12	SS	12	530		
	Brown to reddish brown						
		13	TW	PH	520	121	
	Firm to very stiff						
510.0		14	SS	2	510		
72.0	Occ. thin sand seams up to ½" thick	15	TW	PH		112 / 108	
500.0 / 82.0	Glac. Till-clayey silt with sand, gravel and gypsum. Very stiff.	16	TW	PH	500		Piez. #3 Tip. Elev. 500
497.5 / 84.5	Dolomite bedrock with a bed of gypsum 2' thick. Sound. Grey.	17	SS	29			12 34 39 15
493.0		18	BXL RC	76% Rec			
89.0	End of Borehole				490		



DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

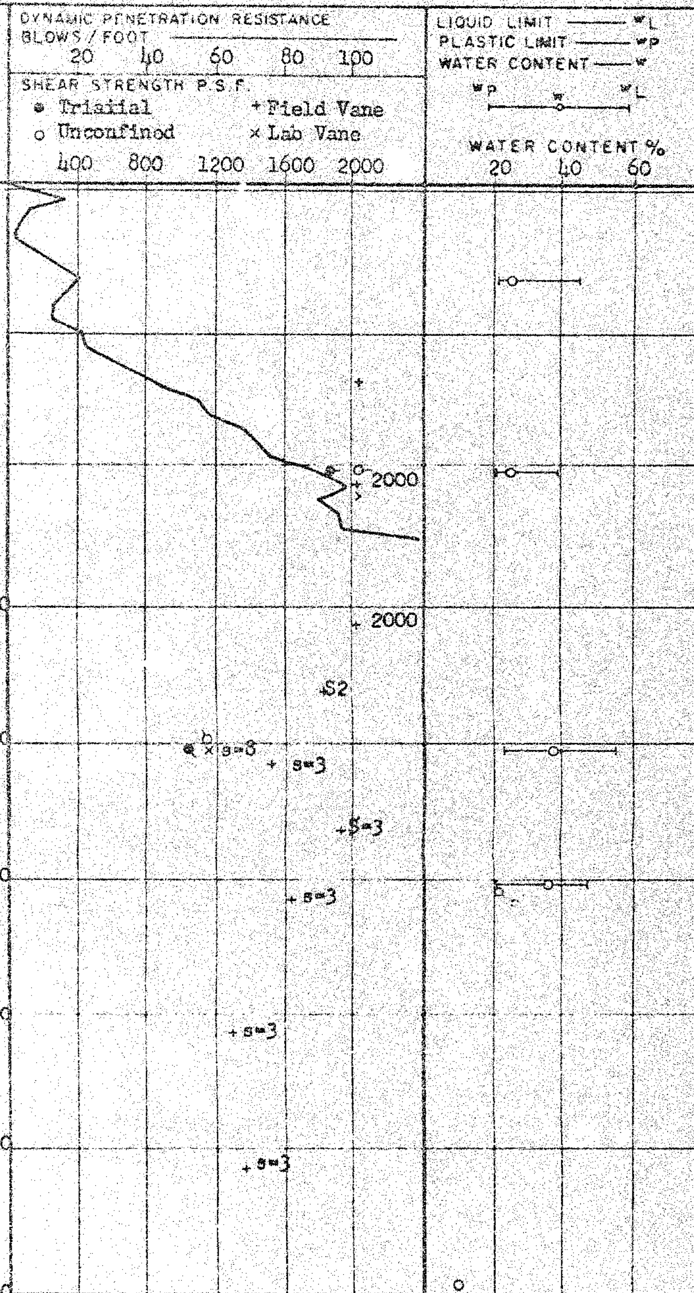
RECORD OF BOREHOLE NO. 3 (68-F-73)

FOUNDATION SECTION

JOB	72-11025	LOCATION	Sta. 217+53 ⌀ East Side Hwy. o/s 73' Rt.
W.P.	60-68-02	BORING DATE	Oct. 28-29, 1968
DATUM	Geodetic	BOREHOLE TYPE	Cont. Flight Auger

ORIGINATED BY WH
COMPILED BY WH
CHECKED BY

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS Gr.Sa.Si.Cl
580.5	Ground Level				580		
0.0	Roadway Fill Clayey silt with some sand & gravel. V.Stiff. dark grey.	1	SS	17			▼ 575.5
576.0 / 4.5	Silty clay to clay with trace of sand occ. very thin grey silt seams containing clear gypsum crystals above elev. 554.	2	SS	36			
		3	SS	27	570		
		4	TW	PH			
		5	SS	13			
		6	TW	PH	560	127	0 1 54 45
		7	SS	22			
		8	3" TW	PH	550		
		9	SS	7			
		10	3"TW	PH	540	125 / 111	
535.0 / 45.5	Occasional silt layers up to 3" thick.	11	SS	11			
		12	3"TW	PH	530		0 0 90 10 (silt seam)
525.0 / 55.5	Brown to reddich brown	13	SS	-	520		
	Firm to very stiff	14	TW	PH	510		
502.5 / 78.0	Glacial Till Clayey silt with sand & grav. Hard. Brown.	15	SS	70/6"	500		
500.0 / 80.5	End of borehole Probable ~~Mxx~~ Bedrock						



DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 4 (68-F-73)

FOUNDATION SECTION

JOB 72-11025
W P 60-68-02
DATUM Geodetic

LOCATION Sta. 221+00 ℄ East Side Hwy. o/s 5' Lt.
BORING DATE Oct. 30-31, 1968
BOREHOLE TYPE Diamond Drill - NX Casing

ORIGINATED BY WH
COMPILED BY WH
CHECKED BY

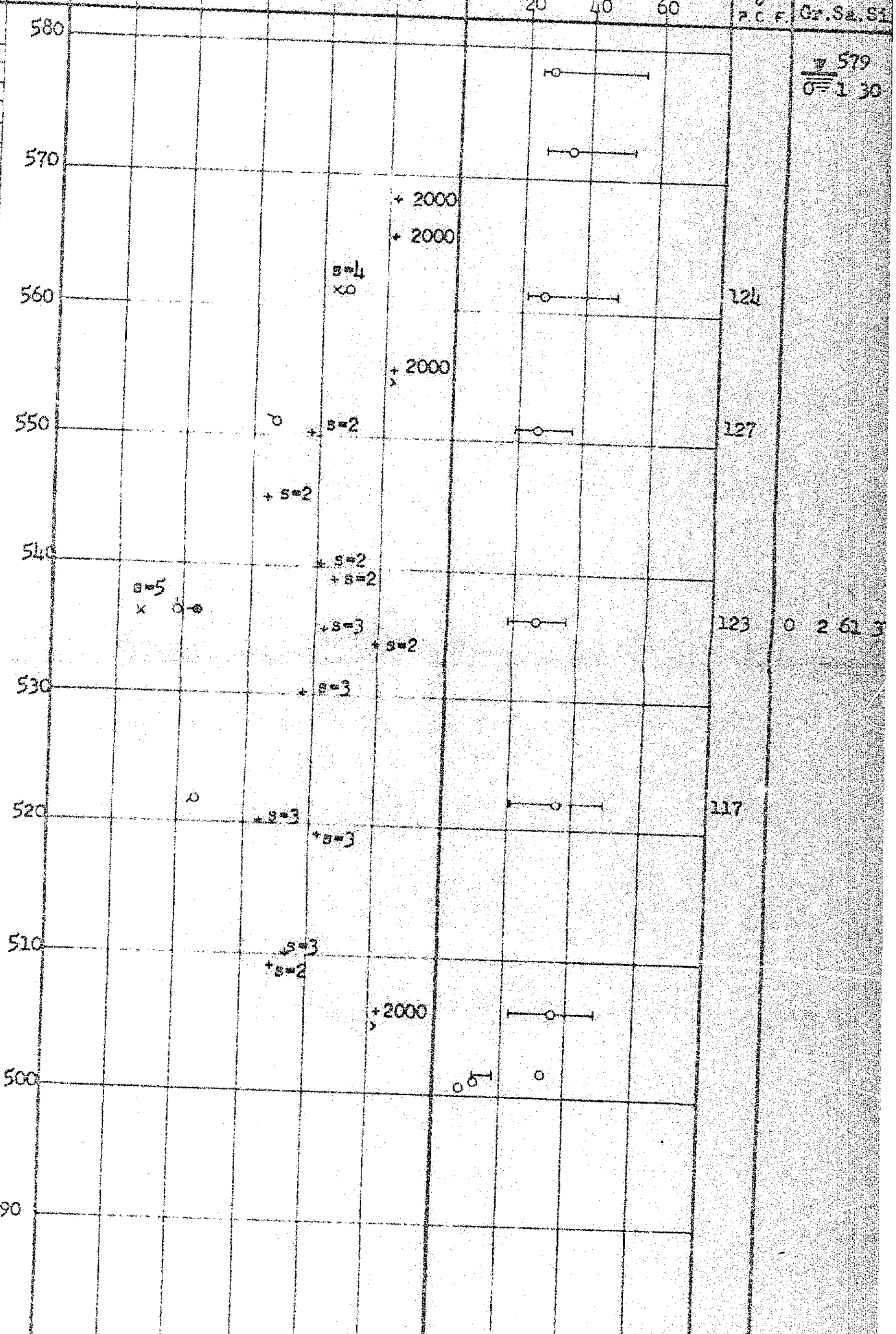
SOIL PROFILE

ELEV DEPTH	DESCRIPTION
582.0	Ground Level
0.0	Silty clay to clay with trace of sand & gravel occ. very thin grey silt seams containing clear gypsum crystals above elev. 560 Brown to reddish brown Stiff to hard
534.0 48.0	Occ. layers of silt up to 3" thick
520.0 62.0	
504.0 78.0	Glacial till-clayey silt with sand & grav. Hard. Brown to grey.
497.3 84.7	End of Borehole Probable Bedrock

SAMPLES

NUMBER	TYPE	BLOWS / FOOT
1	SS	76
2	SS	31
3	SS	19
4	TW	PM
5	SS	17
6	SS	17
6A	SS	13
7	TW	PM
8	SS	14
9	TW	PM
10	TW	PM
11	SS	19
12	TW	PM
13	SS	12
14	TW	PM
15	SS	81
16	SS	64/3"

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT
SHEAR STRENGTH P.S.F. ⊕ Triaxial + Field Vane ○ Unconfined Comp. × Lab Vane
LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w
BULK DENSITY γ P.C.F.
REMARKS Gr.Sa.Si.



FORM 08-M 26 (REV. 1971)   OFFICE REPORT ON SOIL EXPLORATION

7

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS
DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 101

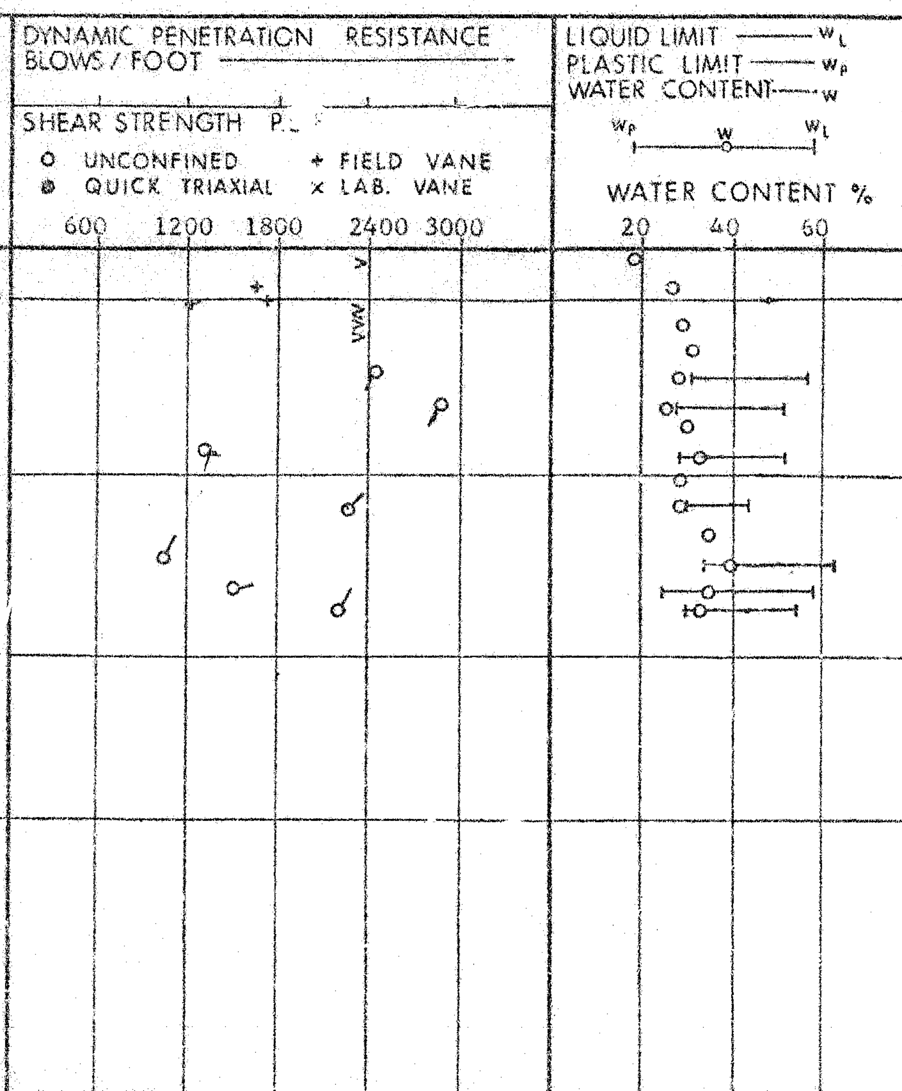
FOUNDATION SECTION

JOB 72-11025   LOCATION C.N.R. & Forkes Road, Sta. 214 + 60 32' Rt. Hwy. #140   ORIGINATED BY S.A.

W.P. 60-68-02   BORING DATE July 13, 1971   COMPILED BY W.V.U.

DATUM Geodetic   BOREHOLE TYPE NX Casing   CHECKED BY O.E.

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
582.8	(Reddish-Brown)	1	SS	9			
		2	SS	14	580		El. 579.0
	Silty clay to clay,	3	SS	14			
	trace of sand.	4	SS	7			
		5	TW	E.		125	
	(Grey-Brown)	6	TW	PM		127	
		7	SS	14			
		8	TW	PM		121	
		9	SS	13	570		
	Stiff to very stiff.	10	TW	PM		124.5	
		11	SS	13			
		12	TW	PM		114	
		13	TW	PM		118	
		14	TW	PM		120	
					560		
		15	TW	PM			
		16	TW	PM			
551.2		17	TW	PM	550		
31.6	End of borehole.						



DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS
DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 102

FOUNDATION SECTION

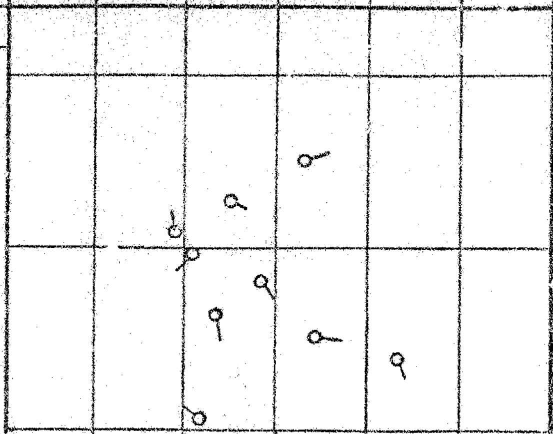
JOB 72-11025
W.P. 60-68-02
DATUM Geodetic

LOCATION Forkes Road & C.N.R., Sta. 214 + 60 32' Rt. Hwy. 14
BORING DATE July 14-15, 1971
BOREHOLE TYPE NX-AX Casing (Dry Boring)

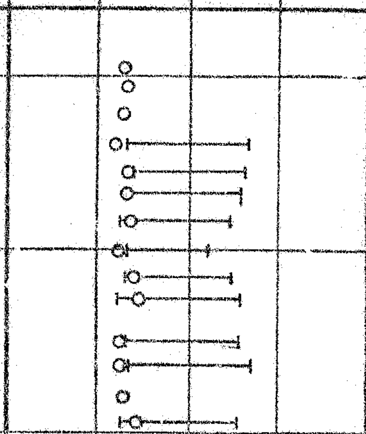
ORIGINATED BY S.A.
COMPILED BY W.V.U.
CHECKED BY

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS
603.3	Silty clay to clay, trace of sand (Fill) (Grey-Brown) Stiff to very stiff.	1	SS	10	600		Dry
		2	SS	8			
		3	SS	7			
		4	TW	PM			
		5	TW	PM		122	
		6	TW	PM		120	
		7	TW	PM		122	
		8	TW	PM	590	122	
		9	TW	PM		124.5	
		10	TW	PM		121	
		11	TW	PM		127	
		12	TW	PM		124	
		13	TW	PM			
		14	SS	---	580		
579.3		15				121	
24.0	End of borehole.						

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT
SHEAR STRENGTH P.S.F. — o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB. VANE — 600 1200 1800 2400 3000



LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w — WATER CONTENT % 20 40 60



GR. SA. SI. CL.

FORM OB-MT-126 (REV. 1971)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 102 A

FOUNDATION SECTION

JOB 72-11025

W.P. 60-68-02

DATUM Geodetic

LOCATION Forkes Road & C.N.R., Sta. 214+60 32' Rt. Hwy. #140

BORING DATE July 15, 1971

BOREHOLE TYPE NX

ORIGINATED BY S.A.

COMPILED BY W.V.U.

CHECKED BY

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE — 600 1200 1800 2400 3000

LIQUID LIMIT — w_L; PLASTIC LIMIT — w_P; WATER CONTENT — w

WATER CONTENT % — 20 40 60

BULK DENSITY γ P.C.F.

REMARKS GR.SA.SI.CL.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	REMARKS
603.3	SOIL PROFILE					
	Silty clay to clay, trace of sand - Fill. Reddish-Brown. Stiff to very stiff.				600	
		1	SS	20		
		2	SS	12		
		3	SS	4	590	
		4	SS	9		
		5	SS	10		
		6				
		7	SS	15		
578.8		8	SS	12	580	
577.8	Clayey Topsoil.	9	SS	33		
25.5	Silty clay to clay, trace of sand. Occasional silt and sand layers up to 3" thick. Stiff to very stiff.	10	SS	11		El. 577.0
		11	SS	57		
		12	SS	27	570	
		13	SS	13		
		14	SS	12		
557.8		15	SS	15	560	

FORM OB-MT-[illegible]26 (REV. 1971) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS
DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 103

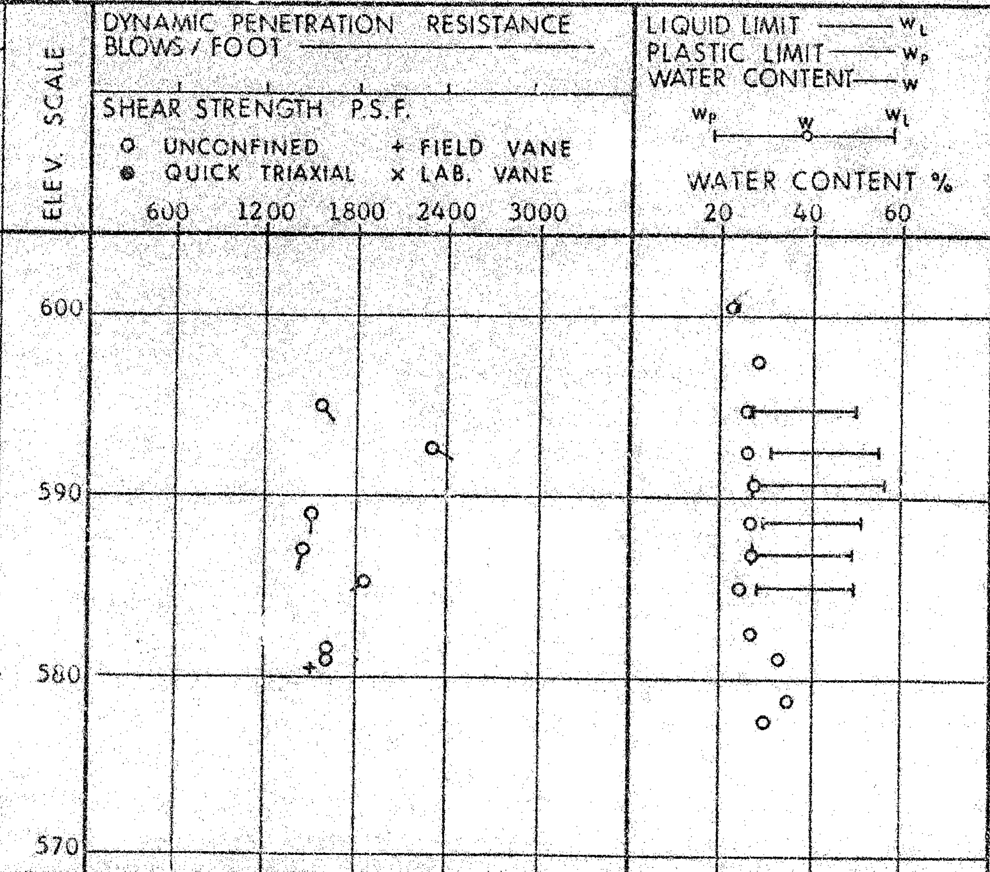
FOUNDATION SECTION

JOB 72-11025
W.P. 60-68-02
DATUM Geodetic

LOCATION Forkes Road & C.N.R., Sta. 214+50 18' Lt. Hwy. #140
BORING DATE July 15, 1971
BOREHOLE TYPE NX Casing

ORIGINATED BY S.A.
COMPILED BY W.V.U.
CHECKED BY O.E.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL
604.6	Clay to silty clay, trace of sand - fill. Reddish-Brown. Stiff to very stiff.	1	SS	18		
		2	SS	15		
		3	TW	PM	125	
		4	TW	PM	124.5	
		5	TW	PM	120	
		6	TW	PM	124.5	
		7	TW	PM	124.0	
		8	TW	PM	125	
582.1 22.5		9	TW	PM		
580.1	Clayey Topsoil.	10	TW	PM		
24.5	Clay to silty clay, trace of sand. Grey-Brown. Stiff to very stiff.	11	TW	PM		El. 579.0
		12	TW	PM		
		13	TW	PM		
		14	TW	PM		
569.1		15	TW	PM		
35.5	End of borehole.					



DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 103 A

FOUNDATION SECTION

JOB 72-11025 LOCATION Forkes Road & C.N.R., Sta. 214+50 18'Lt. Hwy. #140 ORIGINATED BY S.A.

W.P. 60-68-02 BORING DATE July 19, 1971 COMPILED BY W.V.U.

DATUM Geodetic BOREHOLE TYPE NX Casing CHECKED BY OE.

SOIL PROFILE ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB. VANE) 600 1200 1800 2400 3000	LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w; WATER CONTENT % 20 40 60	BULK DENSITY γ P.C.F.	REMARKS GR SA SI CL
604.6										
	Clay to silty clay, trace of sand - fill Reddish-brown Stiff to very stiff.		1	SS	21	600				DRY
			2	SS	12					
			3	SS	16	590				
582.1			4	SS	15					
22.5	End of borehole.					580				

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 104

FOUNDATION SECTION

JOB 72-11025 LOCATION Forkes Road and C.N.R., Sta. 214+50 86' Lt. Hwy. #140 ORIGINATED BY S.A.

W.P. 60-68-02 BORING DATE July 20, 1971 COMPILED BY W.V.U.

DATUM Geodetic BOREHOLE TYPE NX Casing CHECKED BY

SOIL PROFILE			SAMPLES				DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB. VANE) 600 1200 1800 2400 3000	LIQUID LIMIT — w_L; PLASTIC LIMIT — w_P; WATER CONTENT — w; WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE				
582.8										
	Clay to silty clay, trace of sand. Reddish-brown, Stiff to very stiff.		1	SS	8					
			2	SS	13	580				W.L. July 20/71
			3	SS	13		2000			
			4	SS	11					
			5	SS	18					
			6	SS	21					
			7	TW	PM					
			8	SS	31					
			9	SS	20	570				
567.8			10	SS	11					
15.0	End of borehole.									

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DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 105

FOUNDATION SECTION

JOB 72-11025 LOCATION Forkes Road and C.N.R., Sta. 219+50 86' Lt. Hwy. 140 ORIGINATED BY S.A.

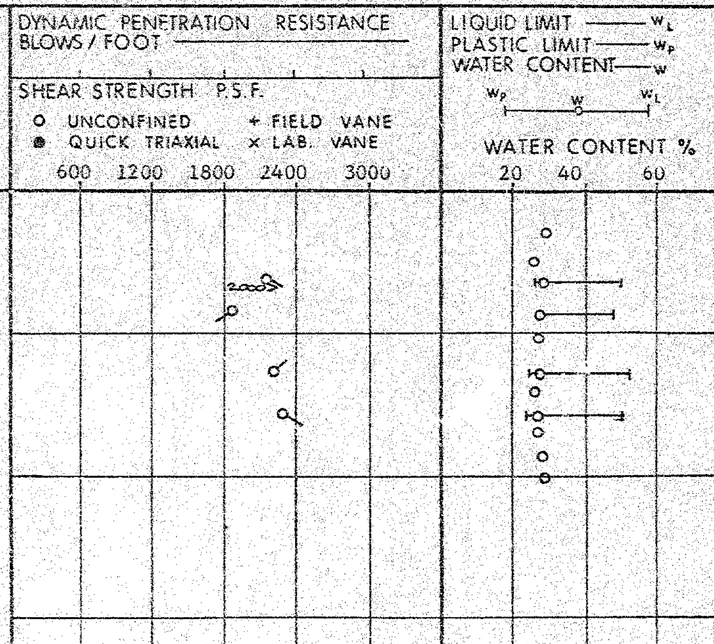
W.P. 60-68-02 BORING DATE July 20, 1971 COMPILED BY W.V.U.

DATUM Geodetic BOREHOLE TYPE NX Casing CHECKED BY OE.

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT
600.0	Clay to silty clay, trace of sand (Fill) Reddish-brown. Stiff to very stiff.	
581.0 / 19.0	Clayey Topsoil.	
579.0 / 21.0	Clay to silty clay, trace of sand. Stiff to hard.	
567.5 / 32.5	End of borehole.	

NUMBER	TYPE	BLOWS / FOOT	BULK DENSITY γ P.C.F.	REMARKS
1	SS	15		
2	SS	12		
3	TW	PM	125	
4	TW	PM	125	
5	SS	12		
6	TW	PM	125	
7	SS	17		
8	TW	PM	126	
9	SS	22		
10	SS	19		
11	SS	13		
12	SS	23		
13	TW	PM		W.L. July 20/71
14	SS	35		
15	SS	44		
16	SS	15		
17	TW	PM		
18	SS	17		

ELEV. SCALE: 590, 580, 570



DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS
DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 206

FOUNDATION SECTION

JOB 72-11025 LOCATION Hwy. #140 Sta. 221 + 40 O/S 3' Lt. ORIGINATED BY R.R.B.
W.P. 60-68-02 BORING DATE February 3 and 4, 1972 COMPILED BY R.R.B.
DATUM Geodetic BOREHOLE TYPE C.M.E. Augering. CHECKED BY [illegible]

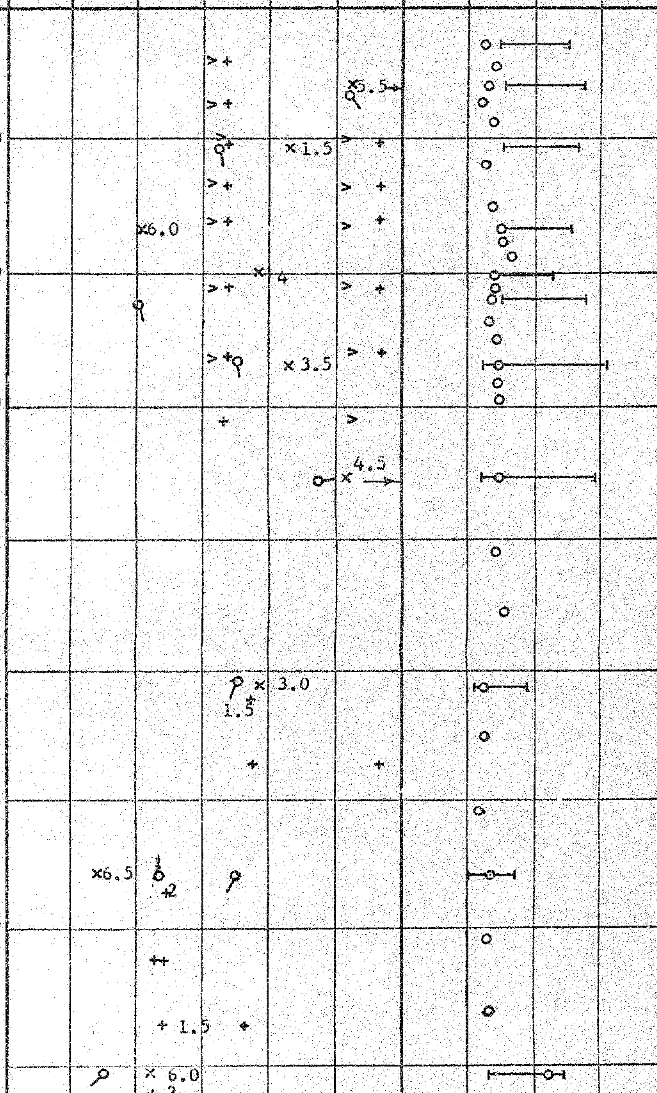
ELEV. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS
609.8 / 0.0	Silty clay to clay, trace of sand (Fill) Greyish-brown	1	SS	23			
		2	SS	18			
		3	TW	PH		128.5	
		4	SS	24			
		5	SS	14			
	Stiff to very stiff.	6	TW	PH	600	124.0	
		7	SS	35			
		8	SS	14			
		9	TW	PH			
		10	SS	12			
		11	SS	9			
		12	TW	PH			
		13	SS	10	590		
		14	SS	13			
		15	TW	PH		123.0	
		16	SS	21			
		17	SS	18			
		18	TW	PH		124.5	
580.3		19	SS	20			P. 29' DRY
	Clayey Topsoil.	20	SS	21	580		
30.5	Silty clay to clay, trace of sand and gravel.	21	TW	PH		122	
	(occasional silt and sand layers up to 3" thick)	22	SS	22	570		
		23	SS	14			
	Stiff to very stiff.	24	TW	PH	560	125	
		25	SS	11			
		26	SS	16	550		
		27	TW	PH		125	
		28	SS	8	540		
		29	SS	10			P. 77.6
526.8		30	TW	PH	530	114	
83.0	End of borehole.						

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

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

LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w

WATER CONTENT %



20
15 5 % STRAIN AT FAILURE
10

FORM 08-MT-126 (REV. 1971)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS
DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 207

FOUNDATION SECTION

JOB 72-1102.5 LOCATION Hwy. #140 Sta. 221 + 46 O/S 60' Lt. ORIGINATED BY R.R.B.

W.P. 60-68-02 BORING DATE February 9, 1972 COMPILED BY R.R.B.

DATUM Geodetic BOREHOLE TYPE C.M.E. Augering CHECKED BY

ELEV. DEPTH (596.0)	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
0.0	Silty clay to clay, trace of sand (Fill) Reddish-Brown Stiff to very stiff.		1	SS	16			
			2	SS	16			
			3	TW	PH	590	126.0	
			4	SS	17			
			5	SS	13			
			6	TW	PH		123.0	
			7	SS	14			
			8	SS	17			
			9	TW	PH		120.0	
579.5			10	SS	13	580		
578.0	Clayey Topsoil.		11	TW	PH			
18.0	Silty clay to clay, trace of sand. (Occasional seams of silt and sand up to 3" thick) Very stiff.		12	TW	PH		126.0	
			13	SS	36			
			14	SS	18	570		
			15	TW	PH		123.0	P. 33'
558.5			16	SS	11	560		
37.6	End of borehole.							

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

SHEAR STRENGTH P.S.F.
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB. VANE
600 1200 1800 2400 3000

LIQUID LIMIT — w_L
PLASTIC LIMIT — w_P
WATER CONTENT — w

WATER CONTENT %
20 40 60

2 5 5.7 3 2

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS
DESIGN SERVICES BRANCH

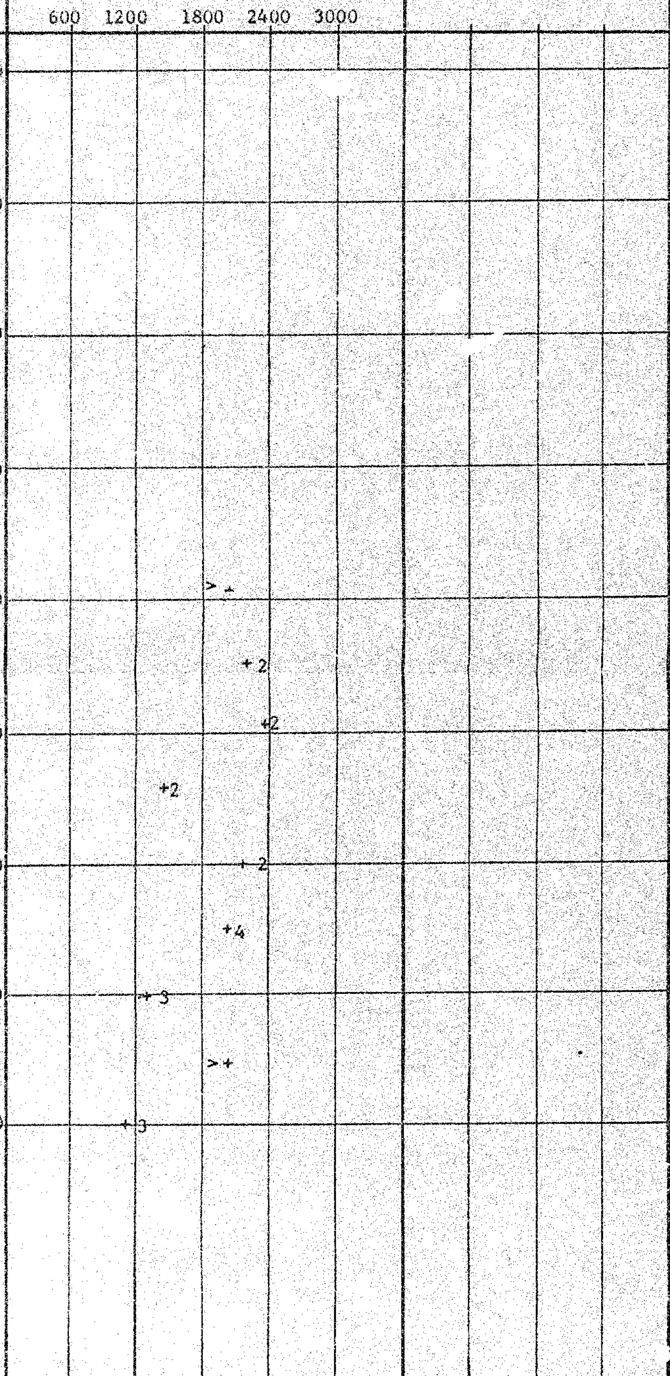
RECORD OF BOREHOLE No. 208

FOUNDATION SECTION

JOB 72-11025 — LOCATION Hwy. #140 Sta. 212 + 00 O/S 15' Lt. — ORIGINATED BY R.R.B.
W.P. 60-68-02 — BORING DATE February 9, 1972 — COMPILED BY R.R.B.
DATUM Geodetic — BOREHOLE TYPE C.M.E. Augering & NX Casing Washbore — CHECKED BY

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	REMARKS
602.7 / 0.0	Silty clay to clay, trace of sand (Fill) Reddish-Brown Stiff to very stiff.		1	SS	13	600	
			2	SS	11		
			3	SS	9		
			4	TW	PH		
			5	SS	16		
			6	SS	13		
			7	TW	PH		
			8	SS	29	590	
			9	SS	16		
			10	TW	PH		
			11	SS	16		
			12	SS	18		
			13	TW	PH		
			14	SS	22	580	
			15	SS	37		
			16	TW	PH		
575.5			17	SS	24		
574.5	Clayey Topsoil.		18	SS	36		P 31' DRY
28.2	Silty clay to clay, trace of sand (Occasional seams of silt and sand up to 3" thick throughout) (Grey-Brown) Stiff to very stiff.		19	TW	PH		
			20	SS	68	570	
			21	SS	24		
			22	TW	PH	560	
			23	SS	18		
			24	SS	12	550	
			25	TW	PH		
			26	SS	8	540	
			27	SS	13		
			28	TW	PH	530	
			29	SS	11		P 75'
519.7			30	SS	8	520	
83.0	End of borehole.						

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT
SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE
LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w
WATER CONTENT %
BULK DENSITY γ P.C.F.
REMARKS GR. SA. SI. CL.



20
15 — 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 209

FOUNDATION SECTION

JOB 72-11025 — LOCATION Hwy. #1[illegible] Sta. 212 + 00 O/S 7[illegible]' Rt. — ORIGINATED BY R.R.B.

W.P. 60-68-02 — BORING DATE February 14 & 15, 1972 — COMPILED BY R.R.B.

DATUM Geodetic — BOREHOLE TYPE C.M.E. Augered — CHECKED BY [illegible]

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	REMARKS
592.5 / 0.0	Silty clay to clay (Fill) Reddish-Brown Very stiff to hard.	1	SS	26	590	
		2	SS	47		
		3	SS	27		
		4	SS	24		
		5	TW	PH		
		6	SS	52		
		7	SS	13	580	
578.0	Softened Zone. Firm	8	TW	PH		
		9	SS	9		
577.0	Clayey Topsoil.	10	SS	32		
15.5	Silty clay to clay, trace of sand and gravel. Grey-Brown Very stiff to hard.	11	TW	PH		
		12	SS	40		
		13	SS	19	570	
		14	TW	PH		
						P [illegible]29½'
561.0		15	SS	19	560	
31.5	End of borehole.					

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

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

LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w

WATER CONTENT %

BULK DENSITY γ P.C.F.

REMARKS GR.SA.SI.CL.

FORM OB-MT- 6 (REV. 1971) OFFICE REPOR N SOIL EXPLORATION

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS
DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 210

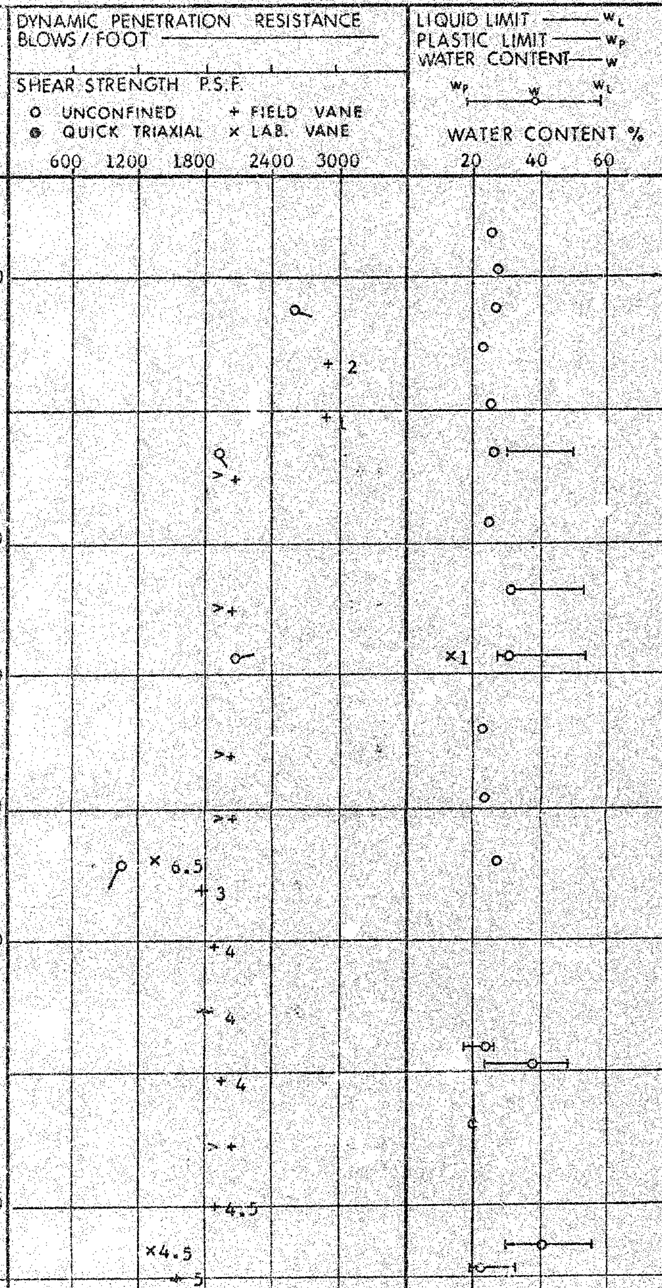
FOUNDATION SECTION

JOB 72-11025 LOCATION Hwy. #140 Sta. 215 + 00 O/S 9' Lt. ORIGINATED BY R.R.B.

W.P. 60-68-02 BORING DATE February 15 and 16, 1972 COMPILED BY R.R.B.

DATUM Geodetic BOREHOLE TYPE CHECKED BY

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS
607.4 / 0.0	Silty clay to clay, trace of sand and gravel (Fill) Reddish-Brown. Stiff to very stiff.	1	SS	15			
		2	SS	17	600		
		3	TW	PH		125.0	
		4	SS	15			
		5	SS	15	590		
		6	TW	PH		125.0	
583.9							
582.9	Clayey Topsoil.						P 24'6"
24.5	Silty clay to clay, trace of sand and gravel. (Occasional seams of silt and sand up to 3" thick) Stiff to very stiff.	7	SS	31	580		
		8	SS	16			
		9	TW	PH	570	123.0	
		10	SS	16			
		11	SS	14	560		
		12	TW	PH		127	
		13	SS	10	550		
		14	SS	7			
		15	TW	PH	540	126	
		16	SS	20			
		17	SS	8	530		
		18	TW	PH		121	
524.4 / 83.0	End of borehole.						



20
15 — 5 % STRAIN AT FAILURE
10

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 211

FOUNDATION SECTION

JOB 72-11025 LOCATION Hwy. #140 Sta. 213 + 36 O/S 100 Ft. Rt. ORIGINATED BY R.R.B.

W.P. 60-68-02 BORING DATE February 17, 1972 COMPILED BY R.R.B.

DATUM Geodetic BOREHOLE TYPE C.M.E. Auger CHECKED BY [illegible]

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

SHEAR STRENGTH P.S.F. — o UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB. VANE — 600, 1200, 1800, 2400, 3000

LIQUID LIMIT — w_L; PLASTIC LIMIT — w_P; WATER CONTENT — w

WATER CONTENT % — 20, 40, 60

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLE NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS GR. SA. SI. CL.
585.2	Ground level.							
0.0	Silty clay to clay, trace of sand & gravel (Fill)		1	SS				
			2	SS	34	580		
576.7	Very stiff.		3	SS	19			
8.5	Silty clay to clay, trace of sand and gravel.		4	TW	PH			EL. 575 o
			5	SS	15			
	Grey-brown.		6	SS	22	570		
	Stiff to very stiff.		7	TW	PH		119	
			8	SS	17	560		
553.6			9	SS	16			
31.5	End of borehole.					550		

STANDARD PROCTOR COMPACTION TEST RESULTS

72 - 11025

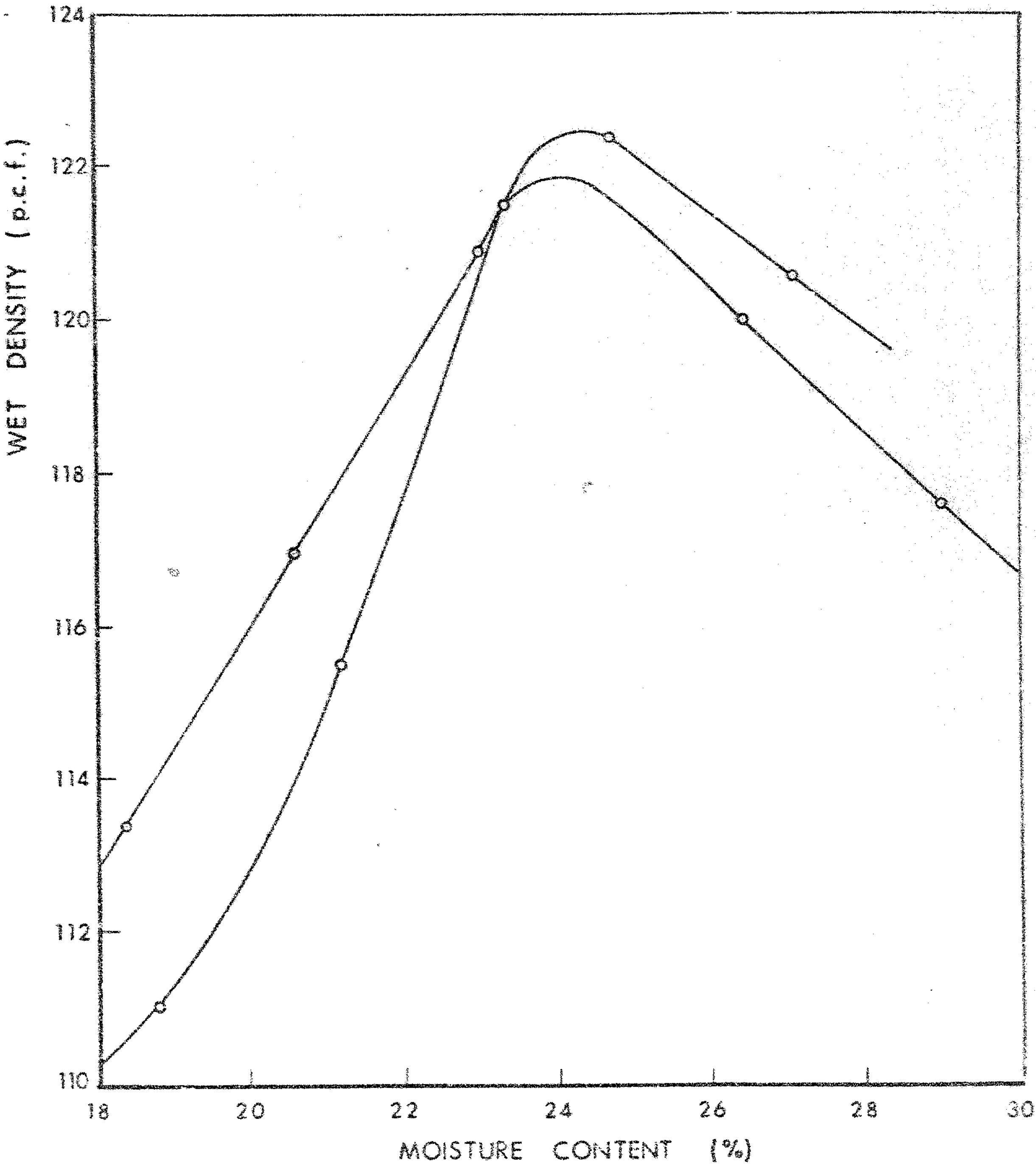
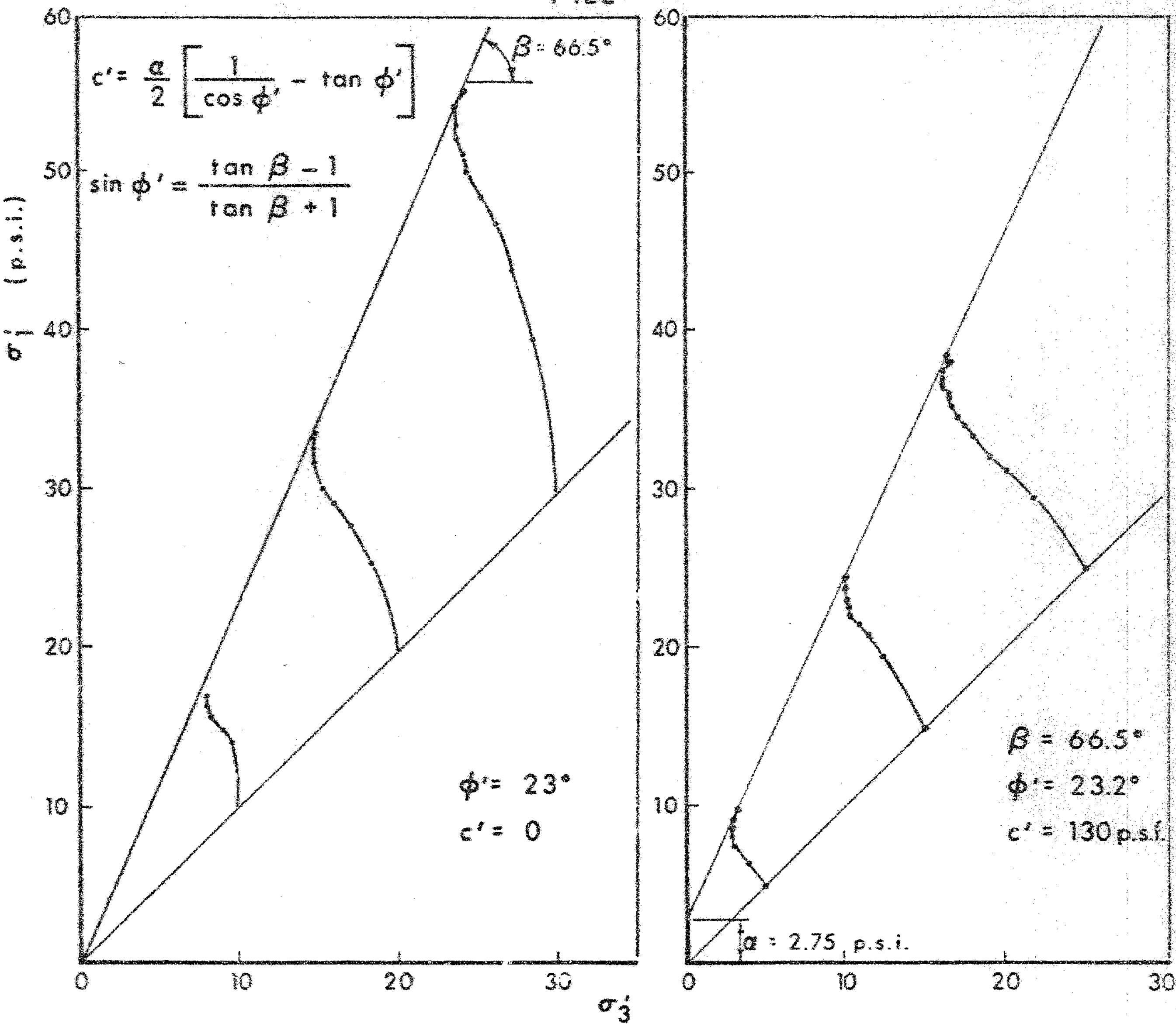


FIG. 1

EFFECTIVE STRESS LABORATORY TESTS

72-11025

FILL



— Isotropically Consolidated Undrained Tests with Pore Pressure Measurements —

PARENT MATERIAL

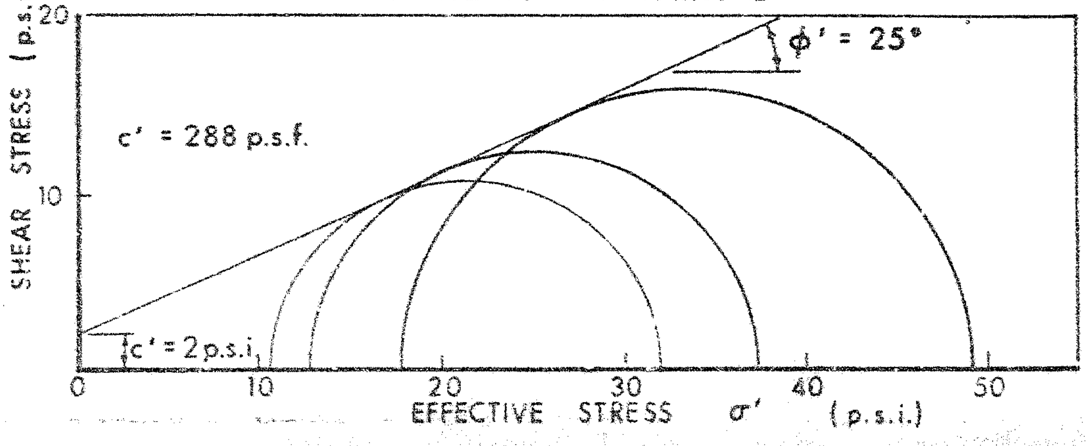


FIG. 2

MOISTURE CONTENT OF FILL MATERIAL

72-11025

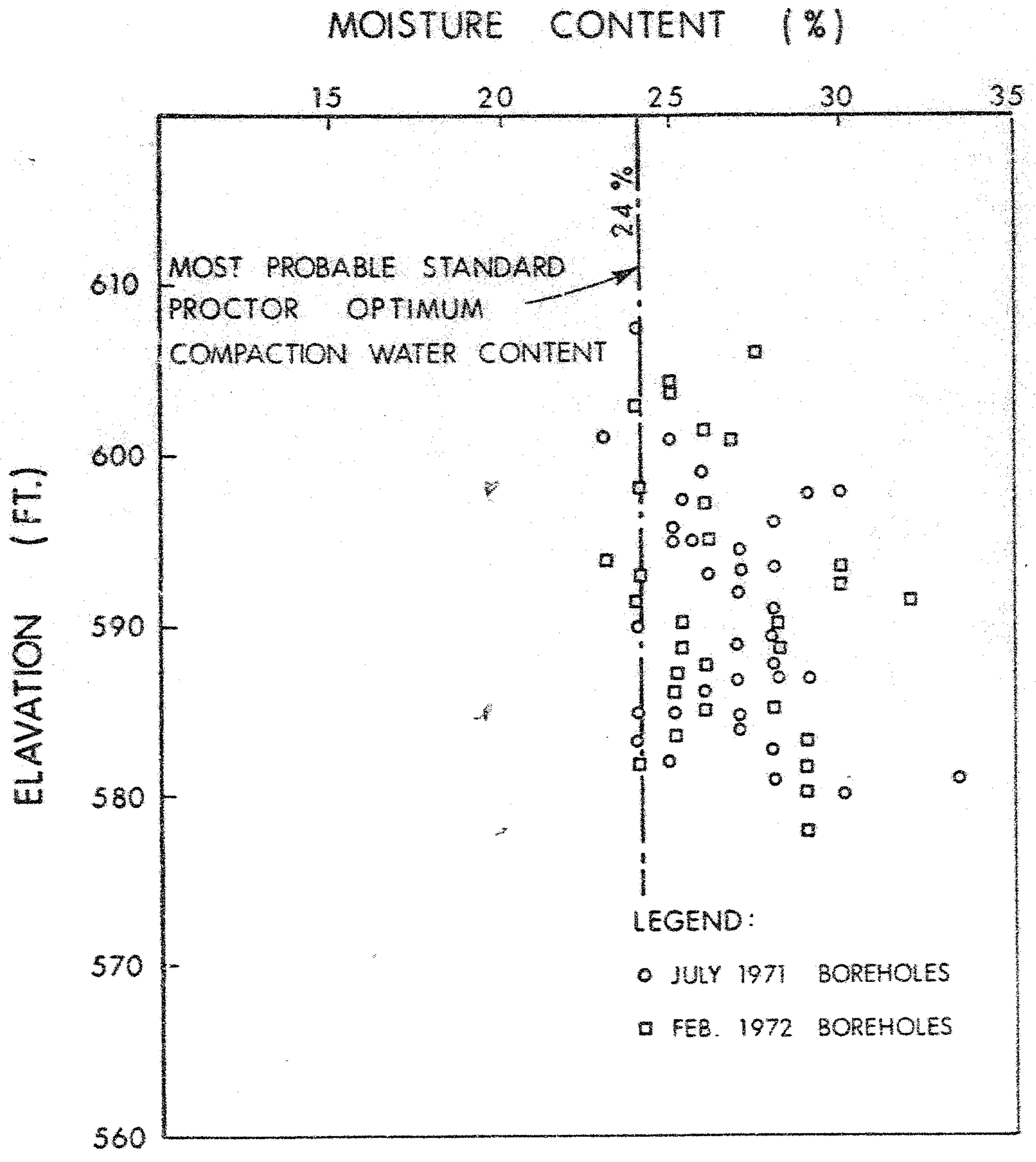
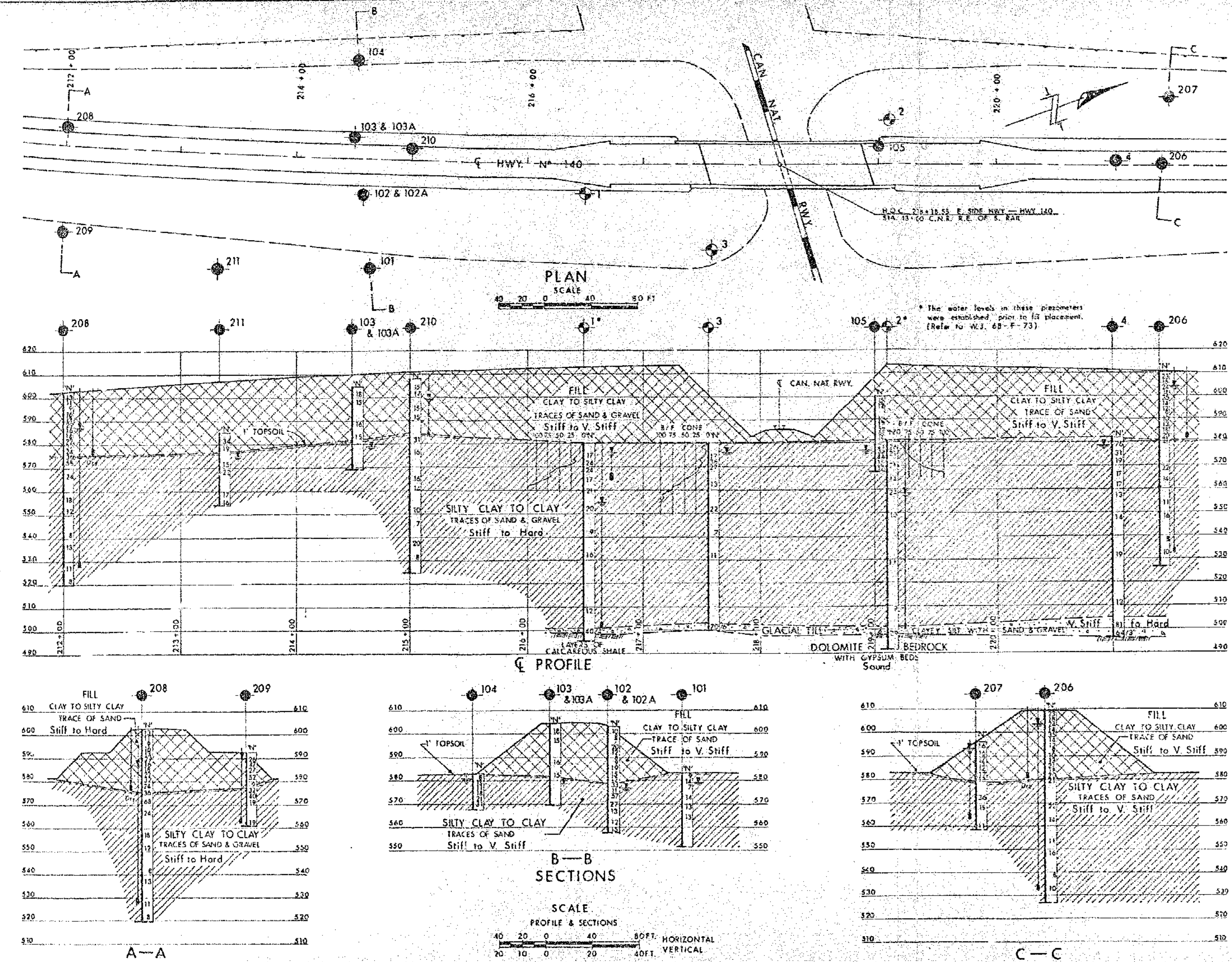
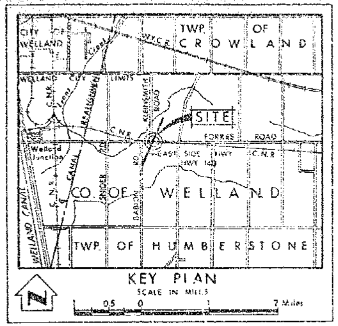


FIG. 3





LEGEND

- Bore Hole
- Cone Penetration Test
- Bore Hole & Cone Test
- Water Levels established at time of field investigation
- Piezometer

NO	ELEVATION	STATION	OFFSET	
1	579.0	216 + 50	25' RT.	OCT 1968
2	582.0	219 + 10	38' LT	
3	580.5	217 + 53	73' RT	
4	582.0	221 + 00	5' LT.	
101	582.8	214 + 67	94' RT	JULY 1971
102&102A	600.3	214 + 60	32' RT.	
103&103A	604.6	214 + 50	18' LT.	
104	582.8	214 + 50	86' LT.	
105	600.0	219 + 50	85' LT.	
206	609.8	221 + 40	2' LT.	FEB. 1972
207	596.0	221 + 46	60' LT	
208	602.7	212 + 00	15' LT.	
209	592.5	212 + 00	75' RT	
210	607.4	215 + 00	9' LT	
211	585.2	213 + 36	100' 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

MINISTRY OF TRANSPORTATION & COMMUNICATIONS
DESIGN SERVICES BRANCH —— FOUNDATIONS OFFICE

CANADIAN NATIONAL RAILWAYS
NEAR FORKES ROAD

HIGHWAY NO 140 (EAST SIDE HWY.) DIST. NO 4
CO WELLAND
TWP. HUMBERSTONE LOT 16 & 20 CON. V & IV

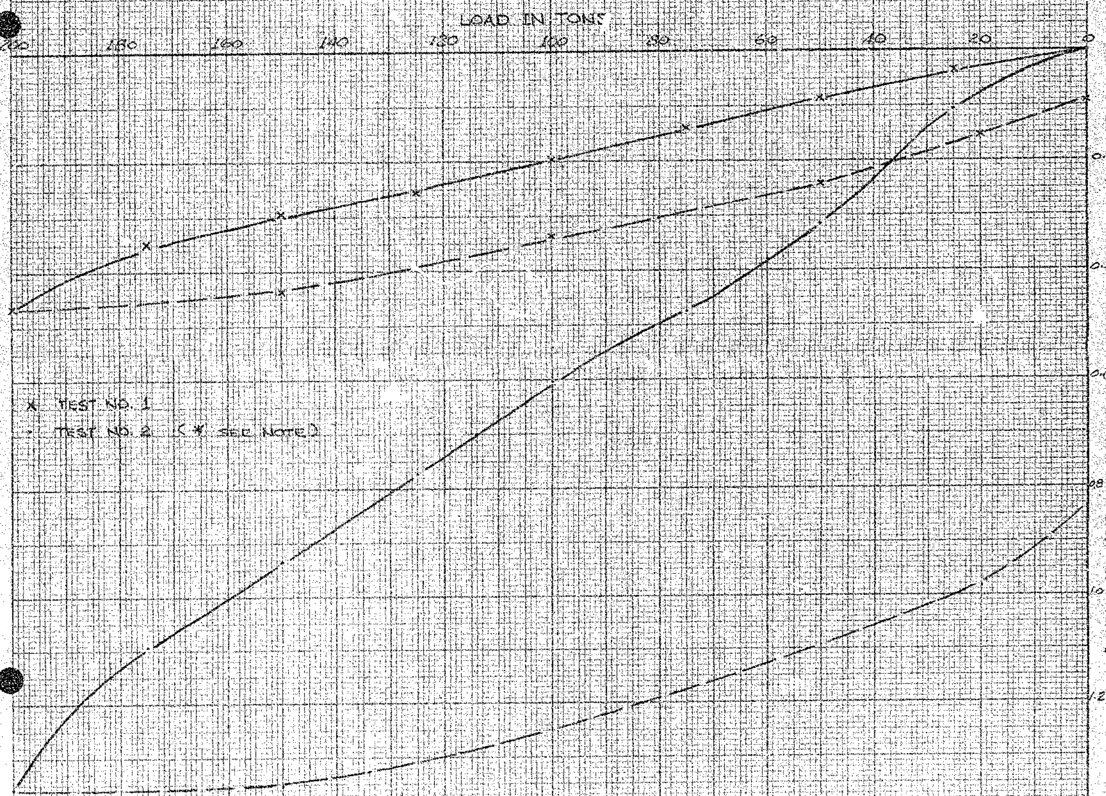
BORE HOLE LOCATIONS & SOIL STRATA

SUBM'D. B.T.D.	CHECKED	W.P. NO. 60-68-02	DRAWING NO 72-11025A
DRAWN O.E.	CHECKED	JOB NO 72-11025	
DATE JUNE 23, 1972.		SITE NO	BRIDGE DRAWING NO
APPROVED PRINCIPAL FOUNDATION ENGINEER		CONT. NO.	

72-11035

DESCRIPTION OF TEST PILES : "HERKULES" PRECAST CONCRETE PILE TYPE 800
(12" flat to flat Hexagonal)

TEST NO	1	2
BEARING STRATUM	driven to bedrock	driven to 10 feet into very dense glacial till
FINAL SET	25 blows/ ½ inch	25 blows/ ½ inch
DRIVING ENERGY	10,500 ft-lbs	13,200 ft-lbs
LOCATION & REFERENCE	Netherby Rd overpass (Cont 1039 - S.L.S.A)	Wilhelm Rd. Overpass (Cont. 1114 - S.L.S.A)



NOTE: DUE TO THE EXCESSIVE SETTLEMENT OF THE TEST PILE AT DESIGN LOAD, THE ST. LAWRENCE SEAWAY AUTHORITY PERSONNEL CONSIDERED THIS TEST UNSATISFACTORY. AFTER RE-DRIVING THE TEST PILE WITH VARIOUS DRIVING ENERGIES, THE FOLLOWING CRITERIA FOR REFUSAL WERE ESTABLISHED.

i) 42 blows per inch for a minimum of 2 inches penetration using a driving energy of 19,800 ft-lbs (3 feet drop height)

or ii) 62 blows per inch for a minimum of 2 inches penetration using a driving energy of 13,200 ft-lbs (2 feet drop height)

Ontario
Department of Transportation and Communications
XXXXXXXXXXXXXXXXXXXXXX

MEMORANDUM

W.O. 72-11025 - KMB

TO: Mr. C. R. Robertson,
District Engineer,
District #4 - Hamilton.

FROM: Foundations Office,
Design Services Branch,
Downsview, Ont.

ATTENTION: Mr. D. Waller,
Construction Engr.

DATE: August 12, 1971

OUR FILE REF.

IN REPLY TO

SUBJECT:

Embankment Instability
South Approach Sta. 211 + 50 to 216 + 50
Hwy. #140 from 0.95 miles North of
Hwy. #3 at Port Colborne Northerly
3.32 mi. to Town Line Rd.
W.O. 68-F-73 --- Cont. 70-212
District #4 (Hamilton)

In response to a request from your Project Supervisor, Mr. J. Castellan, the above-mentioned site was visited by the writer on July 9, 1971. During the site visit, it was observed that longitudinal tension cracks were visible between Stations 211 + 50 and 216 + 50. These cracks are approximately 3 feet in maximum width and extend from 5 to 30 feet on either side of the centre line of Hwy. #140.

In this portion, subsidence up to 2 feet was noticed between the affected area and the remainder of the embankment surface. Significant bulging of the toe of the embankment in the order of 2 to 3 feet was observed in certain locations. Tension cracks of any major magnitude were not noticeable at the north approach.

Open excavations in the affected area by means of back-hoe have been carried out on July 8, 1971 under the supervision of Messrs. T. J. Kovich, Regional Materials Engineer and P. Penev, Project Soils Engineer. Observations made on July 8, 1971 by the Regional Materials Section, as well as by the writer on July 9, 1971, are summarized as follows:

i) Presence of a soft thin layer (9 to 12 inches) of cohesive organic material at the contact of the fill material and natural subsoil.

ii) Tension cracks observed on the surface of roadway embankment appear to extend down to the organic layer located at the original ground surface.

iii) The open pit trench excavated on the side slope of the embankment closest to Sta. 214 + 00 revealed seepage of water into the excavation.

..... 2

Re: Embankment Instability -- W.O. 68-F-73 -- Cont. 70-212

Information obtained from District personnel with regard to the construction of the approaches, is as follows:

1) The south and north approach fills were constructed during the early and late part of May, 1971, respectively.

2) The surficial drainage at the south approach location at the time of construction, was generally poor and consequently wet conditions prevailed over the general area during the placement of the embankment fill material.

3) The fill material for the north and south approach embankments were obtained from two different borrow pits, located north and south of the existing C.N.R. tracks. Visual observations of the borrow pits showed that the material used for the construction of the south approach fill from the south pit appeared to be of a higher moisture content than the optimum.

In order to define the properties of the material and underlying subsoil, a foundation investigation was initiated by this Section. The borings revealed that the in-situ moisture content of the fill material within the affected area ranges randomly from 23 to 35 percent, with an average of 28 percent. The values are consistently higher than the optimum compacted moisture content of this material, which from laboratory testing, was found to be about 22 percent. In the north approach fill area, however, moisture content of the embankment fill material was approximately 1 to 2 percent above the optimum. It should be noted that the strength of the cohesive soils compacted well in excess of the optimum moisture, will have significant reduction in strength and often constitutes failures of the embankments.

Based on the observations carried out at the site, together with the information obtained from our recent field investigation, it is our opinion that the instability of the south approach fill was not a deep-seated failure caused by the overstressing of the natural subsoil, but was mainly due to a failure of the fill material at the contact of original ground surface and the embankment material. The instability of the embankment has been attributed to the high moisture content of the fill material and the presence of cohesive organic layer at the original ground surface.

..... 3

Mr. C. R. Robertson,
District Engineer,
District #4 - Hamilton.
Attn: Mr. D. Waller, Const. Engr.

August 12, 1971

Re: Embankment Instability -- W.O. 68-F-73 -- Cont. 70-212

RECOMMENDED REMEDIAL MEASURES (Ref. 68-F-73B) -

1) All the failed material in the affected area between Sta. 211 + 50 and Sta. 216 + 50 should be removed. This material may be used for the berm construction.

2) Half-height berms having a width of 20 feet should be constructed, as shown on the drawing, on either side of the south approach embankment of Hwy. #140, starting from Sta. 211 + 50 to the limits of south abutment location in the transverse direction. These berms should have a smooth transition towards the longitudinal direction with standard 2:1 slope.

3) Prior to placing the berms, sub-excavation of the organic material for a minimum depth of 2 feet should be carried out and backfilled with acceptable compacted earth material. The transverse extent of the sub-excavation should be as per Drawing No. 68-F-73B.

4) After completion of sub-excavation and berm construction, the remainder of the embankment can be completed to the profile grade with material using proper compaction, at around optimum moisture content.

5) Any minor tension cracks at the north approach embankment should be excavated down to its full depth and replaced, with well compacted fill material prior to placing any new fills in this area.

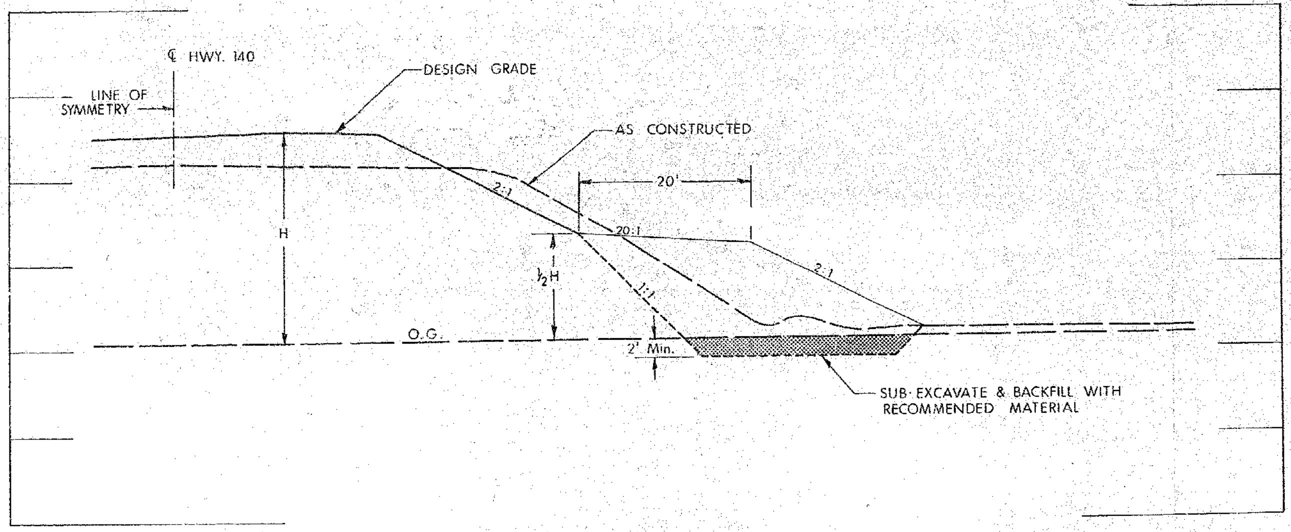
The remedial measures recommended above, have been discussed with Mr. T. J. Kovich, Regional Materials Engineer, who is in complete agreement with our comments. These recommendations were further discussed with Mr. Doug Waller, Construction Engineer, Hamilton District Office, by the writer and Mr. T. J. Kovich.

We believe that the aforementioned information is sufficient for your present construction requirements; however, if we can be of further assistance, please contact our Office.

M. Devata

M. Devata,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

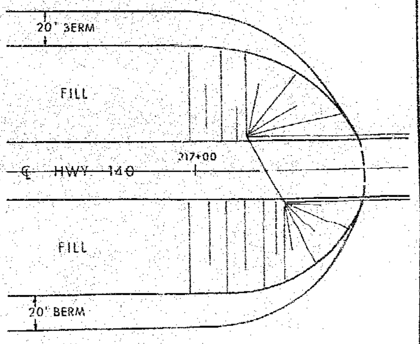
MD/MdeF
cc: Messrs. F. G. Allen
B. R. Davis
T. J. Kovich
J. MacDougall
D. M. Hopper
B. J. Giroux
Foundations Files
Documents



TYPICAL SECTION FOR SOUTH APPROACH

SCALE 1" = 10'

SOUTH APPROACH



BERM TRANSITION PLAN

N.T.S.

DEPARTMENT OF HIGHWAYS ONTARIO MATERIALS and TESTING OFFICE	HWY. 140 & C.N.R. CROSSING SUGGESTED REMEDIAL MEASURES (SOUTH APPROACH) DIST. 4 CONT. 70-212
DATE 10 AUG. 1971	W.P. NO. 60-68-02 / DRAWING NO. 68-F-73B

72-11025

Mtls file

Mr. B. H. Newington,
Municipal Engineer,
Hamilton District.

Materials & Testing Office,
Central Region.

May 16, 1972.

Your memo dated May 1, 1972.

Regional Niagara
Penn Central Crossing Overpass
Regional Road 25, Netherby Road
Hamilton District

By coincidence, Mr. M. Devata reviewed the above-noted site on May 11, 1972 as part of an observation tour of various newly-constructed fills required by S.L.S.A. work on the Welland Canal.

His observation was that there is no deep-seated failure involved which would cause a complete collapse. The distortion is typical of a fill constructed under unfavourable weather conditions. Normally, there is an initial major re-adjustment of the fill mass and then, over the years, minor distortions take place. As soon as these distortions become intolerable, maintenance procedures are required.

In summary, there would appear to be no cause for concern that a serious failure would occur. However, we are faced with a continuing and irritating maintenance demand for an unpredictable period of time.

Original Signed By
T. J. KOVICH

T. J. Kovich,
Regional Materials Engineer.

TJK/js.

cc: M. Devata

72-11025

Department of Highways Ontario
Copy for the information of
T. STERMAC

Mr. A. McKim
Assistant Construction Engineer
Central Building

C.S. Grebski
Structural Design Engineer
Structural Office - West Bldg.

June 15, 1972

Re C.N.R. Overhead (Forkes Road)

Attached herewith is a copy of the memo from Mr. Gluppe regarding the condition of the Forkes Road Bridge.

I am in agreement with Mr. Gluppe's recommendations and believe they should be carried out as soon as possible.

It may be that our Foundation Section will make further recommendations as I have asked them to have a look at this structure.

CSG/hvh
Encl.

C.S. Grebski
Structural Design Engineer

cc T. Stermac
W. Lin
D. Gluppe
B. Davis

72-11025

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To:	Mr. C.S. Grebski Structural Design Engineer	From:	D.R. Gluppe Structural Design Office West Building
Attention:		Date:	June 14, 1972
Our File Ref.		In reply to	

Subject: Re C.N.R. Overhead (Forkes Road)
W.P. 60-68-02 Site 34-234
Highway 140 District 4

On June 13, 1972, a visit was made to the above site. Members of the party were: P. McWatt, R. Kan, D. Gluppe. The following points were noted:

The slope in front of the abutments was approximately 1 to 1 and very uneven. The abutments were only partially backfilled. The top of the pile caps and the wing walls were completely exposed.

Due to the influence of improper backfilling and sloping, the structure is becoming unsafe. The North abutment wing wall has dropped ±4" at the end, and the expansion joint at the North abutment has opened up.

To improve the existing conditions, both abutments should be properly backfilled completely to their full height, and the slope in front of the abutments should be made at 2 to 1 as detailed on our construction drawings. As a further safety measure, the ends of the wing walls should be supported at the North abutment.

D.R. Gluppe

DRG/hvh

D.R. Gluppe
Structural Project Engineer

7_-110 25

Mr. C.R. Robertson, P.Eng.,
District Engineer, Hamilton.

Mr. D. Waller, P.Eng.

Construction Branch,
Operations Division,
3rd floor, Central Bldg.

June 19, 1972.

WP: 60-68-02,
Site 34-234,
Contract 70-212,
CNR Overhead at Forbes Rd.,
Hwy. District 4.

I am enclosing a copy of a memo from D. Cluppe to C. Grebski dated 14th June and a copy of a memo from C. Grebski to A.R. McKim dated 15th June concerning the above structure.

While I agree that the existing conditions should be improved I am suggesting that the situation should be investigated further. Monty Devata of the Foundations Section is going to supervise this over the next month or so to find out just why the abutments are moving.

It is likely some piles and other remedial work will be required at all wing walls although only the North abutment has move significantly at the moment. When the Foundation Section give us their answer we will be in touch with you about remedial work to the abutments and to the expansion joints.

F. McWatt, P.Eng.,
Bridge Construction Engineer.

FMcW/ml
encl.
cc: Mr. C.Grebski, P.Eng.,
Mr. M. Devata, P.Eng.,
Foundations.

RECEIVED
FOUNDATIONS OFFICE
JUN 20 1972
Ministry of Transportation & Communications

Mr. C. S. Grebski,
Structural Design Engineer,
West Bldg., Downsview.

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

August 3, 1972.

Tilting of Abutments
C.N.R. Overhead (Forkes Road)
Hwy. #140, Twp. of Humberstone,
Co. of Welland, District #4 (Hamilton)
W.O. 72-11025 -- W.P. 60-68-02

Embankment instability occurred shortly after the completion of the approach fills of the above-mentioned structure. A detailed sub-surface investigation revealed that the instability of the approach fills was not a deep-seated failure caused by the overstressing of the natural subsoil, but was mainly due to a failure of the fill material at the contact of original ground surface and the embankment material. Suggested remedial measures were submitted in a memo to Mr. C. R. Robertson, District Engineer, Hamilton, dated August 12, 1971, and subsequently in a detailed report (W.O. 72-11025), dated July 17, 1972.

The Foundations Office was advised by the District #4 personnel, during June, 1972, that both the north and south abutments showed signs of distress. According to available information, movement of the abutments was first noticed during February, 1972. However, no periodic measurements for the relative movement between the abutments and the deck were carried out by the field personnel of the District. The Foundations Office initiated a detailed program for monitoring the relative movements between the deck and the abutments, during June 1972, and these were carried out regularly. The results indicate that the north and south abutment moved away from the deck by an additional distance of 0.2 and 0.09 inches respectively, in a period of one month since June 1972. If this trend continues, in our opinion, this may cause further distress to the structural integrity of the abutments. The observations indicated that the movement of the abutments is of a rotational type, and that the back end of the north abutment wing wall moved downwards as much as 4 inches.

.....2

The evidence we have at the present time will not allow us to give an exact reason for such movements but to speculate. In view of this, it is assumed that the rotational movements of the abutments can be attributed to the following reason.

> It is possible that the piles of the front row may have been driven to bedrock, whereas those of the back row were terminated within the glacial till stratum. If such is the case, the back row piles may settle further into the till stratum due to the imposed negative skin friction loadings.

The present observations indicate that the rotational movement of the abutments is still in progress. In view of this, it is essential that some remedial measures should be incorporated. In our opinion, the most suitable method to prevent further tilting of the abutments is to support the extreme end of the abutment wing walls by steel H-piles driven to bedrock. Such remedial measures are warranted immediately without delay, especially in the case of the north abutment where the distress is more severe. However, it may be economical to incorporate such remedial measures for the south abutment at the same time.

Should you have further query, please do not hesitate to contact our Office.

M. Devata

M. Devata,
SUPERVISING FOUNDATIONS ENGINEER.

MD/ao

cc: A. E. Argue
B. R. Davis
D. Waller
A. P. McKim

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JUL 21 1972
Ministry of Transportation & Communications

File P. ...
M. Devata

ONTARIO

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS

MINISTER: HONOURABLE GORDON CARTON, Q.C.   DEPUTY MINISTER: A.T.C. McNAB

Box 279, Burlington
July 14, 1972

MINUTES OF MEETING
HELD JULY 11, 1972
IN THE DISTRICT OFFICE

SUBJECT: CONTRACT NO. 70-212 WP-60-68-01 802
EMBANKMENT FAILURES 72-11-025

Attending:

Bot Construction Limited
J. Sloski

Ministry of Transportation and Communications
M. Devata
F. Kovich
D. A. Waller
J. Castellan
G. Ross

Granular Pad at toe of embankment slope

Drainage to be provided by fine grading subgrade and placing Granular "A" French Drain. Granular "A" to be used for the pad.

C.N.R. Overhead north abutment is showing continued movement in June and July/72. Observation of this movement will be continued for 3-4 weeks. If the bridge stabilizes no further action will be taken.

District will supply pile driving records to M. Devata, Foundations Section, West Building, Downsview.

Berm Construction

Construction of the berms was proposed as follows:-

1. Construct all berms from embankment fill on south side as first operation. ?
2. Double heavy surplus material on south embankment fill. ?
3. Place north embankment on south embankment.? what does this mean ?
4. Construct north embankment.

continued/2

Waste material from construction of Granular Pads to be placed in borrow pit if sufficient water has been removed from the borrow pad at time of construction.

No Granular Pad to be placed at northeast side.

Cross-sections of existing conditions on the northeast side to be supplied by District to Foundations Section.

<u>Limits of remedial construction at structure</u>

No excavation to be carried out within a line 50 feet from end of wing-walls and sloping at 2:1. Prior to any earth excavation on embankment granular backfill at abutments to be completed.

Approval of Bridge Office is to be obtained before loaded scrapers are allowed to cross the bridge.

G. Ross
District Construction Supervisor

GR:cdk

c.c. All Attending

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

MEMORANDUM

TO: Mr. A. E. Argue,
Director,
Design Services Branch,
West Bldg., Downsview.

ATTENTION:

OUR FILE REF.

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

DATE: August 11, 1972

IN REPLY TO

SUBJECT: Contract 70-212
Overhead Structure at Crossing of
Hwy. #140 and C.N.R.

At the meeting on August 3 in your Office, concerning the problem of abutment tilting at the above-mentioned structure, I made the statement that some corrections in the Field Notebook were made in my Office.

The attached memorandum is self-explanatory.

A. G. Stermac,
PRINCIPAL FOUNDATIONS ENGINEER.

AGS/ht
Attch.

c.c. B. R. Davis
C. Grebski
W. L. Lin
A. E. McKim

Foundations Files
Documents

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

MEMORANDUM

TO: Mr. M. Devata,
Supervising Foundations Engineer,
West Bldg., Downsview.

FROM: A. Tieman,
Student Technician,
Foundations Office.

ATTENTION:

DATE: August 9, 1972

OUR FILE REF.

IN REPLY TO

SUBJECT: Measurement of Abutment Movement
Overhead Structure at Crossing of
Hwy. #140 and C.N.R.
Twp. of Humberstone, Co. of Welland
W.O. 72-11025 - W.P. 60-68-02
Contract 70-212

At the request of Mr. M. Devata, Supervising Foundations Engineer, this memorandum has been written to clarify my position with regard to the confusion surrounding the pile driving records for the above-mentioned project.

In early June, 1972, I became involved with this project in initiating an instrumentation program to monitor the movement of the bridge abutments of the structure. At that time, considerable confusion was aroused when I reviewed the pile driving records of the aforementioned structure with regard to both the depth of the piles and the direction of their batter. A similar situation was encountered by Mr. Devata and Mr. B.T. Darch, Senior Foundations Engineer, in a site visit previously.

In an attempt to clarify the situation I conferred with personnel from Hamilton District and was informed that the data was not accurate since the pile driving record had been kept by a student during the previous summer. I was given data kept by the contractor which verified the information in the pile record fieldbook with respect to the length of the piles. I left the pile record fieldbook with District personnel but picked it up again on a subsequent trip to the site, feeling it contained information pertinent to my project.

It has been drawn to my attention that during a recent meeting in our Office concerning this project a conclusion was reached that certain changes had been made in the pile driving record fieldbook. These changes disperse much of the confusion about the batter of the piles. The changes in the field book involve the redirection of North arrows and relabelling of the abutment diagrams in such a way that the direction of the batter of the piles now corresponds to that which is indicated on the design drawings, which was not the case previously.

I would like to state that the above-mentioned changes in the pile driving record fieldbook were not initiated or carried out by myself. I recognize the legality of such a document and consequently, would never change the information contained in a field book in such a manner. I hope that this letter meets with your approwal.

Andre Tieman.

AT/ht

A. Tieman,
Student Technician.

L 72-11025

00018

00018

1973 JAN 29 AM 9:31

DOWN HAMN I JANUARY 29/73

MR M DEVATA SUPVR FOUNDATION SECTION

SUBJECT CONTRACT 70-212

FORKES ROAD APROACH FILLS HIGHWAY 140

BOT CONSTRUCTION

THIS IS TO INFORM YOU THAT CRACKING AT THE CENTRE LINE OF THE PAVEMENT AT THREE DIFFERENT LOCATIONS ON THE SOUTH APPROACH FILL HAS DEVELOPED SINCE FIRDAY TO AN OPENING OF APPROXIMATELY 5/8 OF AN INCH IN ADDITION SEPARATION OF THE BACK SLOPE HAS DEVELOPED ABOUT TWO FEET BEHIND THE GUIDE POSTS

WE ARE NOT SURE IF WE WILL HAVE TO CLOSE THIS SECTION OF HWY 140 AND DETOUR TRAFFIC FOR THIS REMEDIAL WORK TO BE CARRIED OUT

WE UNDERSTAND THAT YOU HAVE BEEN INVOLVED IN THIS CONTINOUS PROBLEM OF THESE APPROACHES FILLS SINCE THE EARLY CONSTRUCTION STAGES

IN VIEW OF THIS WE WOULD APPRECIATE YOUR INSPECTION AND RECOMMENDATIONS FOR REPAIR AT YOUR EARLIEST OPPORTUNITY

H G POTTS MAINTENANCE ENGINEER

LAM

RECEIVED
FOUNDATIONS OFFICE
JAN 29 1973
Ministry of Transportation & Communications

TELETYPE TELETYPE TELETYPE TELETYPE

72-11025

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

TO: Mr. A. Stermac,
Pr. Foundation Engineer.

FROM: W. R. Bennett

ATTENTION: M. Devata,
Sup. Foundation Engineer.

DATE: April 5, 1973

OUR FILE REF.

IN REPLY TO

SUBJECT: Contract 70-212; Highway 140 from Highway 3 to Townline Road

Some time ago you asked for comments regarding a longitudinal crack in the H.L.1 pavement on the approaches to the C.N.R. Overpass in the vicinity of Forkes Road.

Attached is an observation from Mr. C. Beales, Central Region Supervising Asphalt Inspector, dated March 28, 1973. You will note that Mr. Beales' opinion is that this longitudinal crack may be due to earth fill settlement since the crack was wider on March 28 than it had been about a month earlier. You will also note that there is some movement in the curb and gutter which might indicate some degree of settlement.

I have asked Mr. Beales to observe if this crack progresses when making periodic visits to the work in future. You will be advised of his observations.

W.R. Bennett

W. R. Bennett,
Principal Materials Engineer.

WRB:mv
Encl.

c.c. C. Mirza

58-1429

MATERIALS AND TESTING DIVISION – D.H.O.

CONTRACT INSPECTION REPORT

CONTRACT NO. 70-212 REPORT NO. 1 (Fer 1973) DATE OF INSPECTION March 28 1973

INSPECTION ASPECT: (check ✓): BITUMINOUS ☑, CONCRETE ☐, SOILS ☐.

CONTRACT LOCATION From 0.95 mi. N. of Hwy 3 to Town Line Road 3.34 mi.

DISTRICT 4 HWY. NO. 140 CONTRACTOR Port Colborne ... Canada Ltd.

OBSERVATIONS AND RECOMMENDATIONS Sub-Contractor for H.M. Mixing & Paving Warren Bitulithic Co. St Catharines Branch

To Messrs. W. Bloom, J.T. Corkill & W.R. Bennett.

A visual inspection of the approaches to the C.N.R. overpass N.C. of Forkes Road on the above contract was made on the above date

On the S. side of the structure about 150 lin'l ft. from the bridge deck for approx 60 lin'l ft the ℄ joint has opened (probably due to the earth fill settling) about ½ inch in width & may further open after spring thaw.

Also about 20 ft from bridge deck the curb & gutter has moved causing an opening about ½ inch wide for about 20 lin'l ft.

The N. side approach appears reasonably satisfactory with no visible cracking or openings. However it appears that a single paver was used, probably the ℄ joint was not raked resulting in the S.B. lane being a trifle higher than the adjacent N.B. lane at or near the ℄ joint

In view of the probable further joint opening on the S. side of the structure no remedial action is therefore recommended at this time.

It is respectfully suggested that after spring thaw a further inspection with a view to remedial action be taken

GENERAL IMPRESSION OF INSPECTION PROCEDURES: EXCELLENT ☐, GOOD ☐, FAIR ☐, POOR ☐.

CONSTRUCTION PROCEDURES: EXCELLENT ☐, GOOD ☐, FAIR ☐, POOR ☐.

SIGNED [signature]

Signature

72-11025

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

TO: Mr. M. Devata
Supvr. Foundation Section
Downsview, Ontario

FROM: H. G. Potts
Maintenance Engineer
District #4, Hamilton

ATTENTION:

DATE: April 11, 1973

OUR FILE REF.

IN REPLY TO

SUBJECT: Contract 72-212 - Forkes Road Approach Fills - Highway #140
Sot Construction

We attach for your information, a copy of a teletype which we submitted to you January 24, 1973 re: some cracking appearing on the southerly approach fill since the preceding Friday, January 19, 1973.

As we recall, you carried out an investigation immediately, taking measurements at the control points which we had established on each of the cracks.

We also believe that you did not consider the problem tooserious at the time, however, suggested that we continue measuring the width of the cracks at these control points and submit our findings to you.

We attach a report from our Patrolman, Mr. J. C. Wagner, dated April 10, 1973 for your information and analysis.

H. G. Potts
Maintenance Engineer

HGP:lam

c.c. — H. G. Potts
M. T. Scrimshaw

P.S. - I understand you will be visiting the above area in question during the week of April 16, 1973, and have been in conversation with Mr. M. Scrimshaw, Mtce. Supvr., about slope failures on the section of road.
Would you please investigate and report.

DOWN HAMN 1 JANUARY 2

MR M DEVATA SUPVR FOUNDATION SECTION

SUBJECT CONTRACT 70-212

FORKES ROAD APROACH FILLS HIGHWAY 140

BOT CONSTRUCTION

THIS IS TO INFORM YOU THAT CRACKING AT THE CENTRE LINE OF THE PAVEMENT AT THREE DIFFERENT LOCATIONS ON THE SOUTH APPROACH FILL HAS DEVELOPED . NCE FIRDAY TO AN OPENING OF APPROXIMATELY 5/8 OF AN INCH IN ADDITION SEPARATION OF THE BACK SLOPE HAS DEVELOPED ABOUT TWO FEET BEHIND THE GUIDE POS.

WE ARE NOT SURE IF WE WILL HAVE TO CLOSE THIS SECTION OF HWY 140 AND DETOUR TRAFFIC FOR THIS REMEDIAL WORK TO BE CARRIED OUT

WE UNDERSTAND THAT YOU HAVE BEEN INVOLVED IN THIS CONTINOUS PROBLEM OF THESE APPROACHES FILLS SINCE THE EARLY CONSTRUCTION STAGES

IN VIEW OF THIS WE WOULD APPRECIATE YOUR INSPECTION AND RECOMMENDATIONS FOR REPAIR AT YOUR EARLIEST OPPORTUNITY

H G POTTS MAINTENANCE ENGINEER

LAM

PAT 18

JAN. 24/73 CRACKS HWY 140 FORKS RD. [illegible]

PD-18 J. WAGNER

	APRIL 10/73.	
LENGTH #I CRACK. 40' 5"	43'	
WIDTH #I CRACK 6" SOUTH END.	6 ½"	
#I 5 ¾" NORTH END.	6"	
LENGTH #2 CRACK 34'	36'	
WIDTH #2 SOUTH END. 6"	6"	
WIDTH #2 NORTH END. 6"	6½	

J Wagner

MEMORANDUM

TO: Mr. C. R. Robertson, (3)
District Engineer,
District #4,
Hamilton, Ontario.

ATTENTION: Mr. H. Potts,
Maintenance Engineer.

OUR FILE REF.

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

DATE: June 15, 1973.

IN REPLY TO

SUBJECT: *Instability of Approach Embankments,*
Hwy. #140 and C.N.R. Overhead
Twp. of Humberstone, County of Welland
District #4 (Hamilton), Cont. 70-212
W.O. 72-11025 - W.P. 60-68-02

In response to verbal requests from the District Maintenance Personnel, site visits have been made by the members of the Foundations Office during January 1973 and April 1973, at the above-mentioned site. This memo will summarize all our observations made during the site visits and subsequent recommendations pertaining to the remedial measures which were given to the District Maintenance Personnel.

Observations:

The following visual observations were made during January 1973 by Messrs. M. Devata and B. T. Darch.

1) Cracks, running parallel to the roadway, were noticed on the upper slopes. These cracks generally are present at the contact between the select granular base course and the remaining silty clay fill. The District Construction personnel have indicated that these cracks were originally minor (1/8"); however, they were as wide as 5/8" during the time of this site visit.

2) Localized surficial sloughing was noted on that portion of the slope (2:1) above the berm, particularly the east and west side of the north approach fill. It would appear that this condition is confined to an area not more than 3 to 4 feet beneath the surface of the slope.

   The seeding and mulching on these slopes was placed in the late fall of 1972 and it did not hold properly due to lack of root development.

....2

It would appear that surface water seeped into these surficial zones thus softening the cohesive fill which, in turn, led to the surficial sloughing.

3) A longitudinal crack was noticed at the centre-line of the pavement along both approaches. It is our belief that this cracking has nothing to do with the instability of the slopes. Instead it is probably due to an improperly formed paving joint from one lane to the other.

4) The 2:1 slope, below the berm appears to be reasonably stable.

5) In addition to the Hwy. #140 site visit a number of St. Lawrence Seaway Authority structure sites were visited where the silty clay has been used as fill. It was interesting to note that where 3:1 slopes were employed the embankments appeared reasonably stable. However, where 2:1 slopes are present the same type of surficial instability has been noticed.

Subsequent site visit made during April 1973 by Messrs. M. Devata and P. Payer further confirmed the aforementioned comments. The additional observations are as follows:

1) The subdrains located within the granular backfill behind the abutments, with the exception of that at the north-east corner, were not functioning. Water was seeping out underneath these drains.

It would appear that the cohesive fill material in these areas had been softened by the water seeping out underneath these drains, which in turn, led to the surficial instability. Soil samples were taken from these surficial wet zones to determine the in situ moisture contents.

2) As mentioned previously, localized surficial sloughings were evident on the upper portion of the side slopes. In order to substantiate this, several hand-dug test pits up to 3 feet deep were put down on the side slopes above the berm elevation. Soil samples were also obtained at these test pits to determine in situ moisture contents.

Discussions and Recommended Remedial Measures:

As mentioned previously, soil samples were taken randomly in the distressed areas to determine the in situ moisture contents. The results obtained indicate that the in situ moisture content of the silty clay fill material ranges from 51% immediately below the surface to 30% some 2 to 3 ft. beneath the slope surface. The optimum moisture content for this type of fill material is only 25%. These results, together with our visual observations at the site, indicate that the slope instability of the approach embankments of C.N.R. overhead structure at Hwy. 140 appears to be of a surficial nature. The primary cause of these surficial sloughings (failures) is due to the action of water seeping through the slopes which resulted in softening of the silty clay fill material. In order to stabilize the slopes, it is necessary to provide an adequate drainage system to prevent softening of the fill material.

It is, therefore, recommended that the following remedial measures be carried out to control the seepage.

1) All the surficial softened material in the affected area should be removed, and replaced with suitable earth material.

2) A granular blanket, consisting of a free draining material, such as granular 'A', should be placed on the side slopes above the berm elevation in the distressed area, as shown on Figure #1. The blanket should have a minimum thickness of 18 inches. Positive drainage measures should be adopted to relieve the water from the blanket. This can be achieved by placing a subdrain (8" diameter perforated C.I.P.) at the toe of the upper slope. A typical section is shown on Figure No. 1. This system should drain into the existing ditches at the toes of the approach fills.

3) The corner areas of the approach embankments can be remedied either by placing a granular blanket as shown on Figure #2 or by constructing counterfort drains as shown on Figure #3. The details of the placement of the granular blanket will be similar to those discussed under Item 2. In both cases, all softened wet fill material should be removed from the affected areas and replaced with suitable earth material.

   If the scheme consisting of counterfort drains is chosen, these drains should have a minimum base width as well as depth of 3 feet extending radially from the ends of the wing wall as shown on Figure #3. Any water collected by these drains should be intercepted by the cutoff drain with 8" diameter perforated C.I.P. mentioned in Item 2.

4) In all cases, the slopes, after the completion of the granular blanket, should be protected with topsoil, either sodded or seeded as per current M.T.C. requirements.

The remedial measures recommended above, have been discussed with Mr. M. T. Scrimshaw. In addition, a complete set of drawings showing the recommended remedial measures was also provided the District on May 23, 1973.

We would appreciate it if you would keep us advised as to the future course of events in this matter. If you have any further queries or if any of the foregoing requires clarification, please feel free to call us.

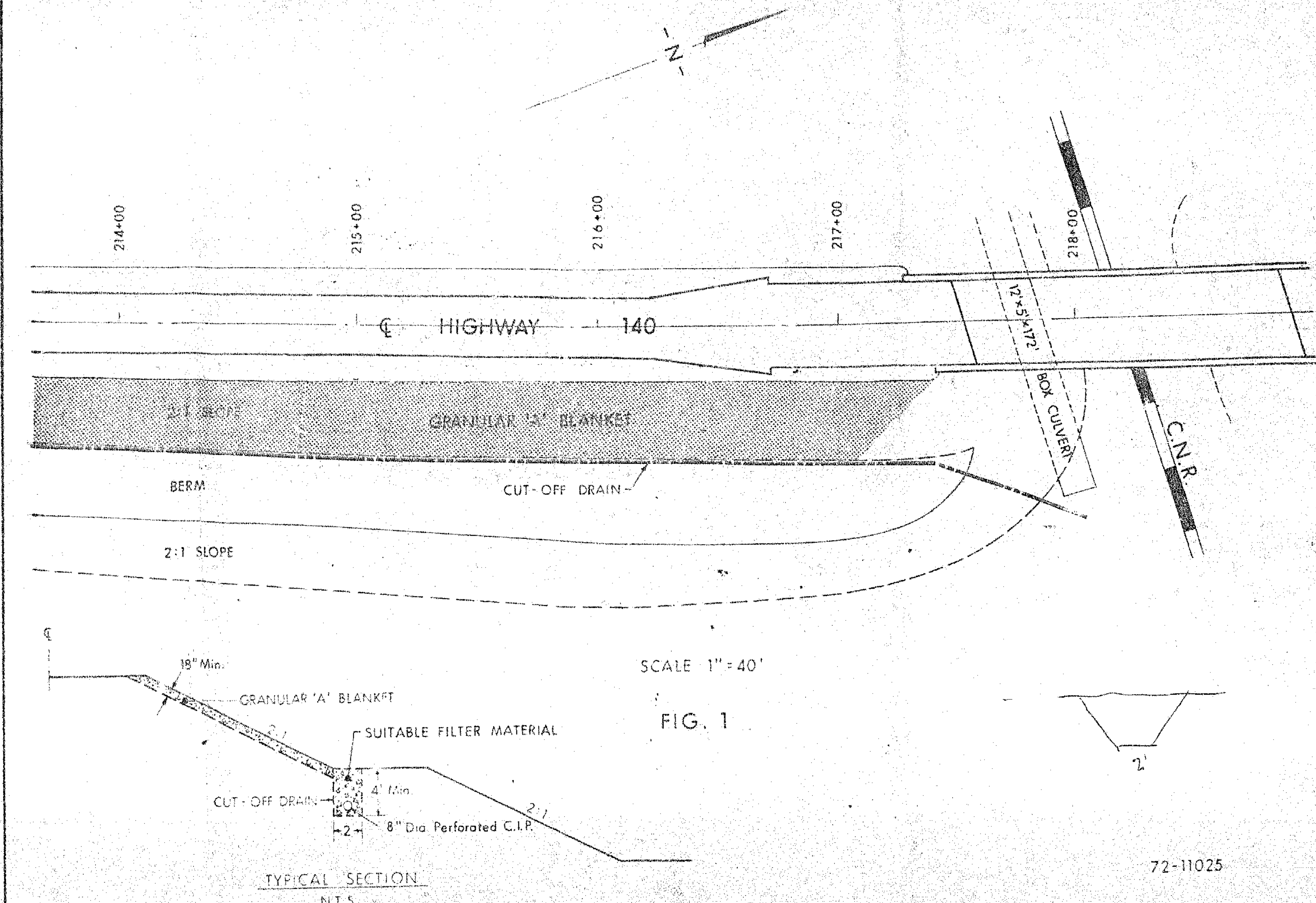
M. Devata

MD/ao

M. Devata,
SUPERVISING FOUNDATIONS ENGINEER.

c.c. A. E. Argue
A. Rutka
R. S. Pillar
W. F. Birch
B. J. Giroux
J. M. Crannie
C. Mirza

Foundations Files
Documents



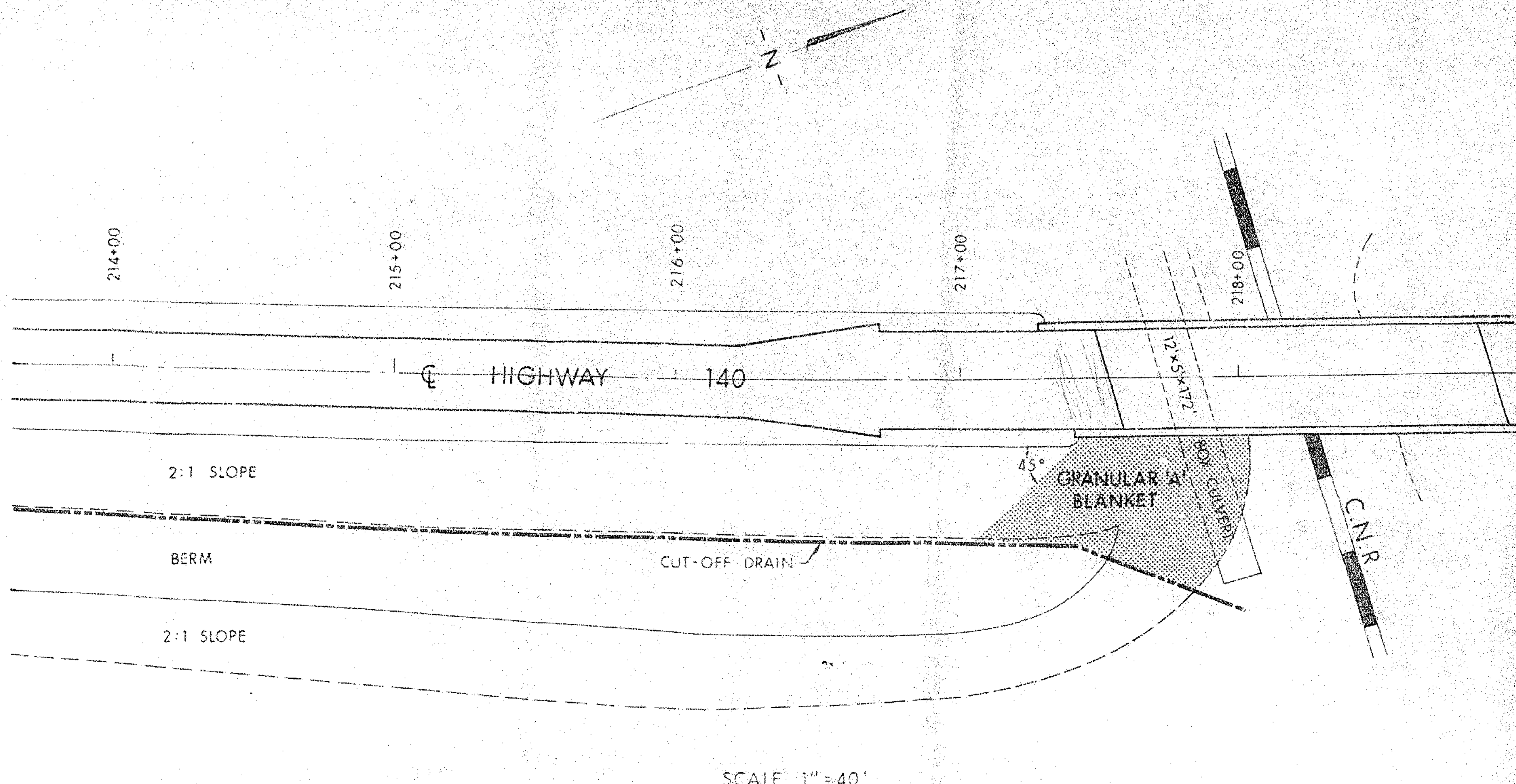
SCALE 1" = 40'

FIG. 1

TYPICAL SECTION

N.T.S.

72-11025



NOTE :
TYPICAL SECTION SAME AS FIG. 1

FIG. 2

72-11025

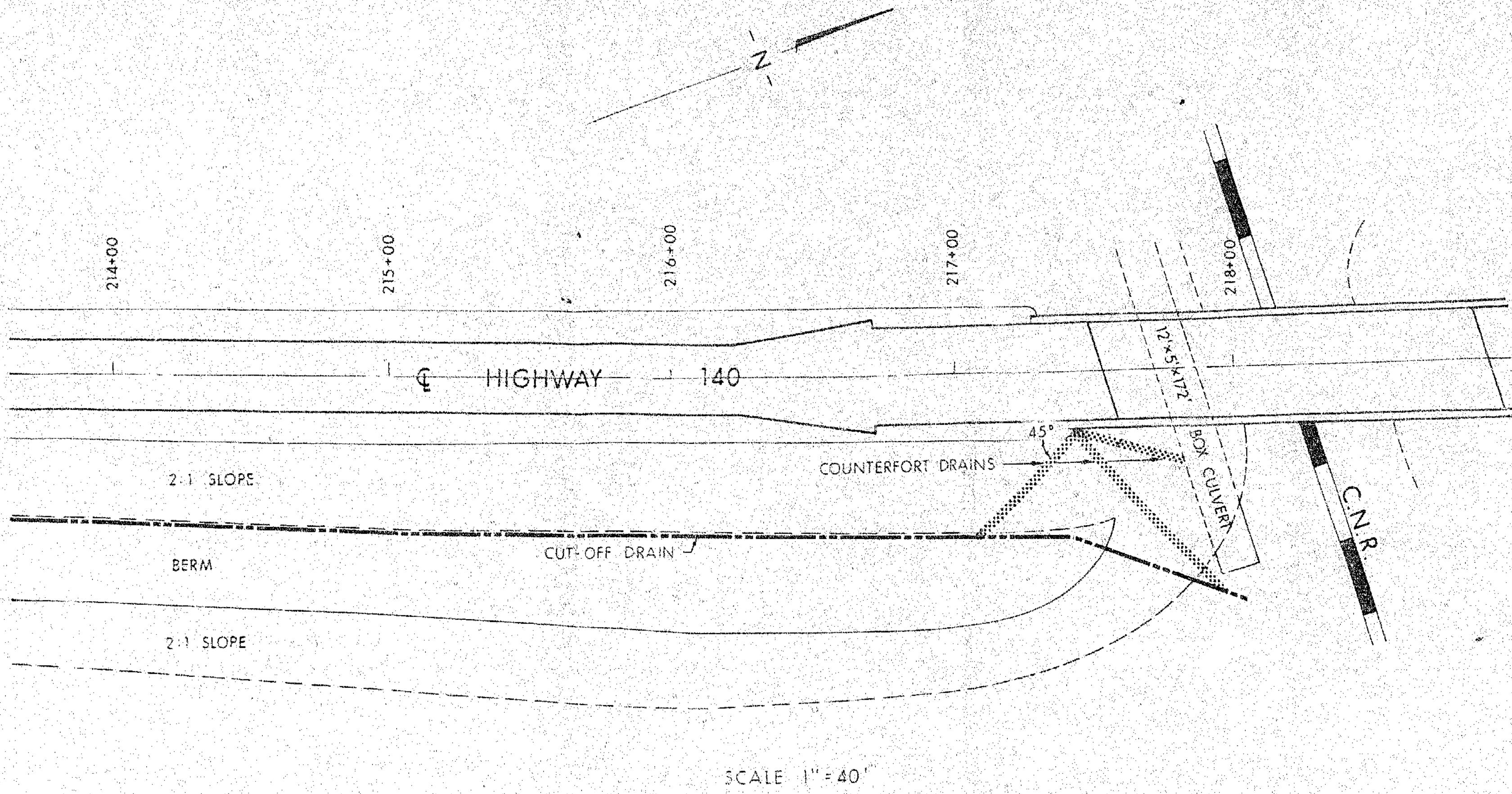


FIG. 3

72-11025

70-212

To Foundation files From M. Devata
Sept 11th 1973

Sub: Forkes Road Approach Fills, Hwy #140.
Instability of Approaches.

A meeting was held at Mr G Metcalf's office to discuss the remidial measures suggested by this office of memo June 15th 1973. The meeting was attend by the following.

Mr G A. Metcalf
Mr S. Cant
Mr M. Devata.

Mr G Metcalf was of the opinion that the remidial measures suggested in our memo (June 15th 1973) are not necessary and no measures should be incorporated. The writer indicated ~~a~~ that the recommendications were submitted to the District and ~~the~~ incorporating the remedial measures will be the responsibility of the District and Mr Metcalf.

M. Devata
Supervising Foundation Engineer.

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

TO: Mr. C. Mirza
Head, Soil Mechanics Section
Geotechnical Office

FROM: P.P. I. Section
Geotechnical Section

ATTENTION: M. Devata

DATE: June 27, 1974

OUR FILE REF.

IN REPLY TO

SUBJECT: FIELD VISIT AT THE CROSSING OF Hwy. 140 & CNR (Forkes Road)

The P.P.I. Section took readings of the slope indicator at the above site on the 25th of June 1974. On the same day the writer visited the site, and made the following observations:

1. The west side of the north abutment foreward slope was badly eroded, a fairly deep irregular gully developed. The cause of the distress is surface water run-off, which gradually washes the soil away from the edge of the abutment. (Photo #1)

2. Tilting of the north abutment away from the bridge. (Photo #2 & 5)

3. The perforated drain in the north abutment granular backfill is likely plugged, water seeps below drain, washing material out and forming bad erosion. (Photo #3)

4. 12" diametre CIP in north approach fill, connected to manhole , ends at the toe of the upper slope. Water from the CIP runs along the berm and the lower slope. Since there is no protection against washout, a deep gully developed along the berm and along the lower slope. (Photo #4)

5. The settlement of the north approach fill is best noticeable at the end of the approach slab. This was resurfaced recently. The fill material appears to be squeezed laterally, pushing the curbs outward. (Photo #6)

6. Due to the settlement of the fill, the top of the slope indicator now is above grade and hence badly damaged. Reading of the indicator, however, was possible.

   Free water lever in the indicator was found to be at 14 ft. (4.3m) below grade, roughly at the elevation of the top of berm.

No imminent danger is foreseen due to above distresses, however, remedial measures should be taken to prevent larger deteriorations.

A. K. Barsvary

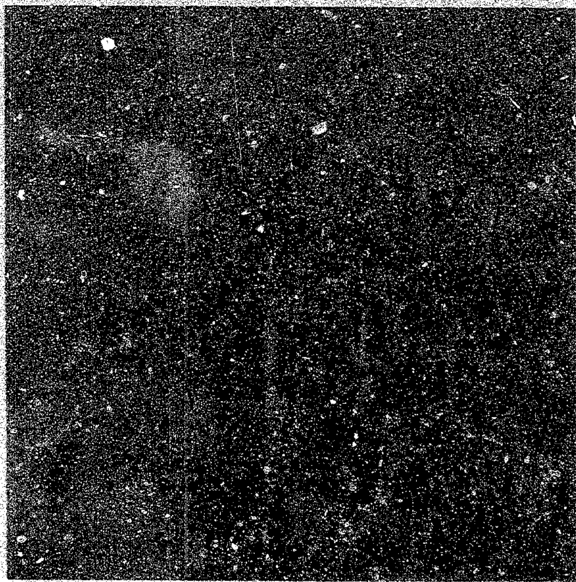
AKB:mt

RECEIVED
GEOTECHNICAL OFFICE
JUN 27 1974
SOIL MECHANICS
MINISTRY OF TRANSPORTATION & COMMUNICATIONS

A. K. Barsvary
Head, P.P.I. Section

HWY. 140 & CNR AT FORKES ROAD

June 1974



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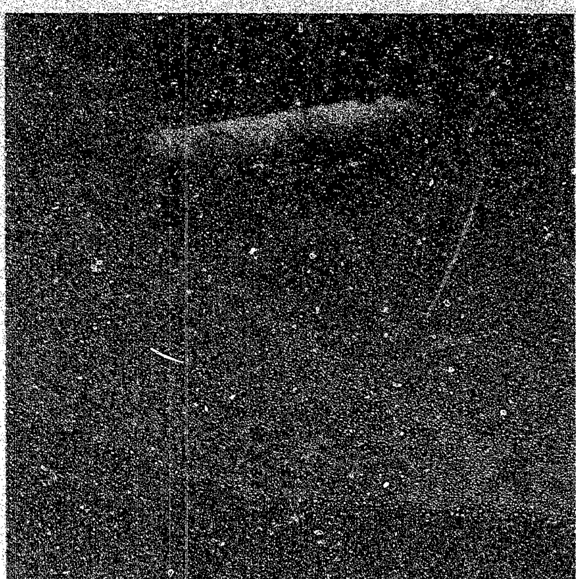
2



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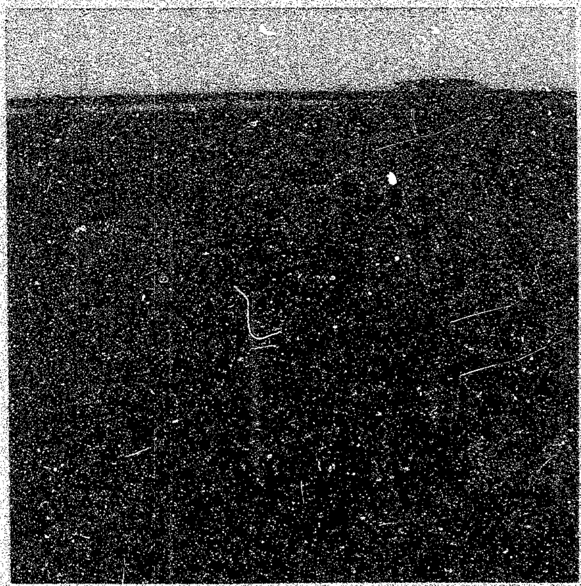
HWY. 140 & CNR AT FORKES ROAD

June 1974



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5

6

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

TO: Mr. C. Mirza
Head, Soil Mechanics Section
Geotechnical Office

FROM: P.P.I. Section
Geotechnical Section

ATTENTION: M. Devata

DATE: June 28, 1974

OUR FILE REF.

IN REPLY TO

SUBJECT:

HWY. 140 & CNR AT FORKES ROAD
W.O. 72-11025, INSTRUMENTATION

1. Slope Indicator readings and plots show no movement. Water level at 14½ feet below top of indicator is same as previous readings.

2. Structure shows no movement between pin locations still being monitored, and in fact a[illegible] readings are approximately 0.010" less than last reading in Novemb[illegible] 3.

3. Gap measuremen[illegible] at the abutment and curb locations show some movement. Curbs at the abutment locations have moved outward from ℄ at both abutment approaches. This lateral movement ranges from 0.5" to 1.6" since January 26, 1973. Refer to Alex's memo, item number 5, dated June 27, 1974.

4. Protective cap and slope indicator casing cap destroyed and missing. (See Photo) This was discussed with George Green of the M.T.C. district forces and a new slope indicator casing cap was left with him; with the understanding that it could be used if you decide to have it repaired by the district and the readings continued in the future.

H. D. Reed
H. D. Reed
P.P.I. Technician

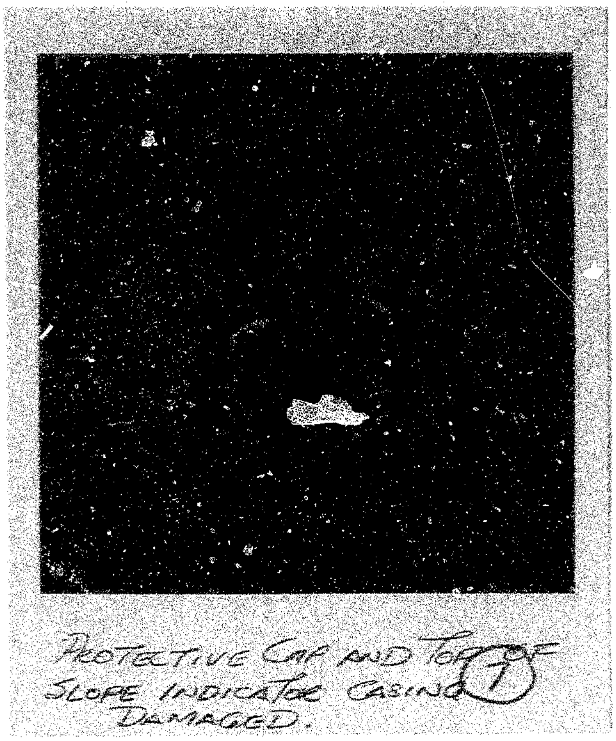
HDR/gs

Cont. 70-212
WP 60-68-02

RECEIVED
GEOTECHNICAL OFFICE
JUN 28 1974
SOIL MECHANICS
MINISTRY OF TRANSPORTATION & COMMUNICATIONS

HWY. 140 & CNR AT FORKES ROAD

June 1974



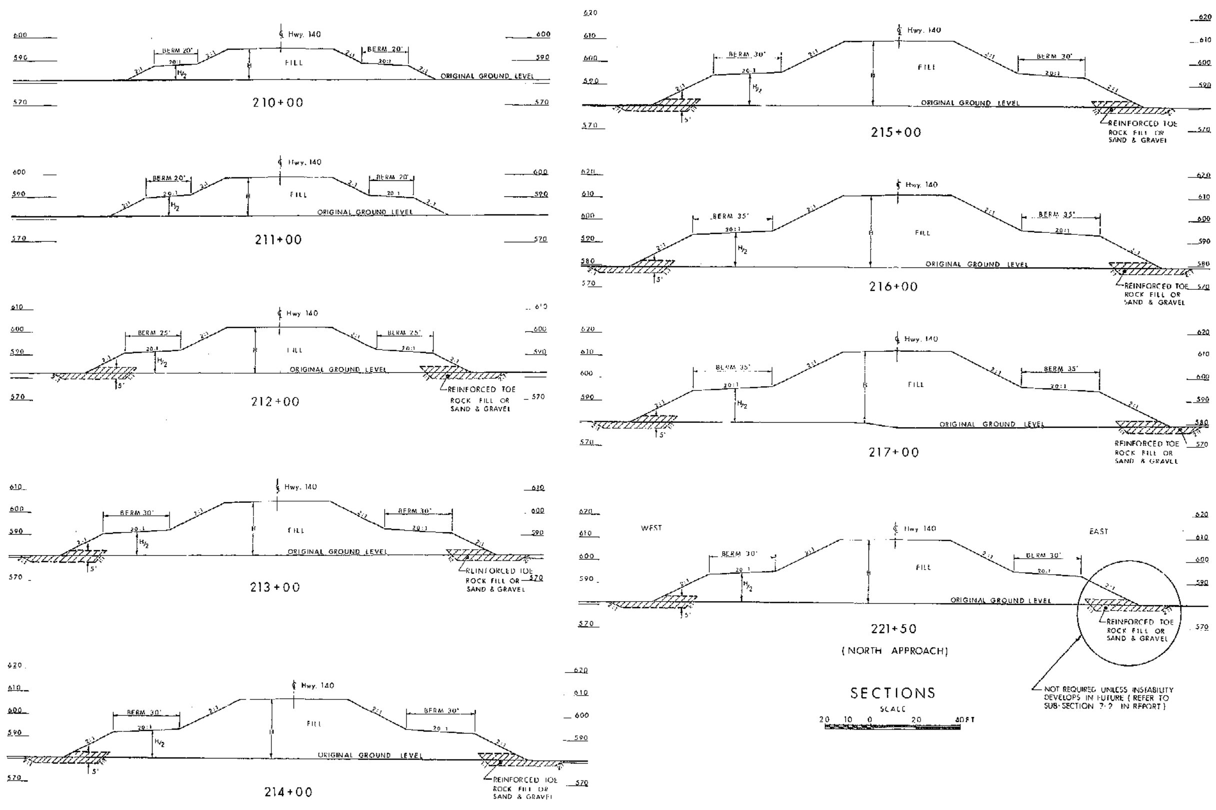
CONT. 70-212

HWY. 140 +

C.N.R. EMBANK.

FAILURE

30L-45



KEY PLAN
SCALE IN MILES

LEGEND

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

NO.	ELEVATION	STATION	OFFSET

— 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

PRINT RECORD		
NO.	FOR	DATE

MINISTRY OF TRANSPORTATION & COMMUNICATIONS
DESIGN SERVICES BRANCH — FOUNDATIONS OFFICE

APPROACH FILL STABILITY
(SOUTH APPROACH)

HIGHWAY NO. 140 (EAST SIDE HWY.) DIST. NO. 4
CO. WELLAND
TWP. HUMBERSTONE LOT ____ CON. ____

RECOMMENDED REMEDIAL MEASURES

SUBM'D. B.T.D.	CHECKED	W.P. NO. 60-68-02	DRAWING NO. 72-11025B
DRAWN	CHECKED	JOB NO. 72-11025	
DATE April 23, 1972		SITE NO.	BRIDGE DRAWING NO.
APPROVED PRINCIPAL FOUNDATION ENGINEER		CONT. NO.	

GEOCRES Nº 30L-45

SOME DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENTS