(30L-54)

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

30L14-23
GEOCRES No.

MEMORANDUM

TO: Mr. G.C.E. Burkhardt, (3)
Reg. Structural Planning Eng.,
Central Region, Toronto.

FROM: Soil Mechanics Section,
Geotechnical Office,
West Building, Downsview.

ATTENTION:

DATE: March 7th, 1974.

OUR FILE REF.

IN REPLY TO

MAR 13 1974

SUBJECT: FOUNDATION INVESTIGATION REPORT
For
Embankment Settlements at Hwy. #58 and
Forkes Road
District #4, Hamilton

W.P. 78-73-01 W.O. 73-11097

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.

K.G. Selby

K.G. Selby,
Supervising Engineer.

KGS/mj
c.c. E.J. Orr
B.R. Davis
A. Rutka
R.S. Pillar
~~R.G. Gascoyne~~ C.R. Robertson
B.J. Giroux
C. Mirza
G.A. Wrong
B.A. Singh

Files
Documents

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Foundation Investigation Report

for

Embankment Settlements at Hwy. #58 and Forkes Rd.

District #4 - (Hamilton)

W.P. 78-73-01 - W.O. 73-11097

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1. *INTRODUCTION:*

It has been proposed to replace the concrete deck at the existing Forkes Road overpass on Hwy. #58, south of Welland; sometime during 1974. Visual observations have disclosed what appears to be rather large settlements at the approaches to this structure. In view of this, Mr. C.G.E. Burkhardt, Regional Structural Planning Engineer, Central Region, requested this Office to carry out a subsoil investigation at the site in an effort to determine the causes of the settlements observed to date and the likely magnitude of future settlement if any, so that remedial treatment may be incorporated into the design of the deck replacement. The request was contained in a memo dated December 3, 1973.

This report contains the results of the subsequent foundation investigation, together with our comments and recommendations pertaining to the observed settlements and overall embankment stability and remedial action to allow for future settlement.

2. *DESCRIPTION OF THE SITE AND GEOLOGY:*

The site is located at the intersection of Hwy. 58 and Forkes Road near the southern boundary of the City of Welland, Regional Municipality of Niagara. The structure at the site carries Hwy. 58 over Forkes Road in a north-south direction. The structure consists of three (27' x 46' x 22') simply supported steel girder spans supporting a concrete deck and two piers between the abutments. Piers and abutments are founded on spread type footings placed within the original ground.

Topographically the site lies in a flat, mainly agricultural area. In general, the area is poorly drained. Some light residential development exists to the east and west of the site along Forkes Road.

Physiographically the site lies within the region termed the Haldimand Clay Plain which lies between Lake Erie and the Niagara Escarpment.

Geologically, the subsoil is composed of glacial and glacial lacustrine deposits of Wisconsin age, which overlie bedrock of the Pleistocene epoch.

The Wisconsin deposits are divided into a number of relatively distinct zones. The upper zone is composed of predominantly clay sized particles with small amounts of sand and gravel, and with well defined but discontinuous layers or pockets of stratified material which is also predominantly composed of clay sized particles. This zone may have been deposited beneath a readvancing glacier that over-rode and reworked an underlying deposit of layered or varved clay. Hence, much of the 'till' is essentially compos d of layered or varved clay sediments.

The next zone is more easily defined as a glacial till best described as a silty clay with some sand, a trace of gravel.

Beneath this zone is a zone of layered clay. This material was probably deposited in a short term lake which was formed in front of a glacier that occupied the Lake Ontario Basin. At least once the ice readvanced over this clay deposit creating a zone composed predominatly of the same material as the layered (varved) zones lying above and below, but of a mottled appearance.

A fourth zone was identified at this site, and was composed of thick irregular layers of material of both silt and clay sized particles. This material was probably deposited during an early ice retreat when the ice front had moved some distance away.

The clay zones are underlain by a generally thin glacial drift or in some areas by fluvial deposits.

Bedrock is part of the Salina formation of the

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Silurian period, Paleozoic epoch. The Salina formation is mainly dolomite with shale interbeds and containing numerous gypsum inclusions.

3. *FIELD AND LABORATORY WORK:*

Two sampled boreholes were put down with one dynamic cone penetration test adjacent to each borehole. Each borehole was advanced through the embankment fill and into the original ground by means of a continuous hollow stem Bombardier mounted C.M.E. 55 augering machine. Disturbed samples were obtained using 2-inch O.D. split-spoon samplers driven according to the specifications for the Standard Penetration Test. Undisturbed samples were obtained using 2-inch I.D. Shelby tubes which were pushed hydraulically into the soil. With few exeptions, field vane tests were performed 18 inches (0.457 m) below the level at which undisturbed samples were taken. Cone tests were driven with a driving energy of 350 ft. lb (475 J) per blow.

In addition to the above described conventional sampling and testing, one hole was augered through the roadway pavement some 10 feet (3.05 m) from the south abutment and the depth of pavement was measured. Adverse weather and traffic conditions prevented futher measurement of this type.

All samples were examined visually in the field, when possible, and again in detail in the laboratory. A comprehensive laboratory test program was carried out on selected undisturbed samples, in the course of which Atterberg limits, natural moisture content, bul density, shear strength (total and effective stress parameters), consolidation characteristics and permeability characteristics were determined.

The results of the field and laboratory tests are summarized on the Record of Borehole Sheets and figures 1-5 which are contained in the Appendix of this report, and will be discussed in some detail in a subsequent section.

The locations and elevations of the boreholes and cone tests together with the estimated stratigraphical profile are given on Drawing No. 75-11097A, which is also contained in the Appendix. The stratigraphical profile is based on the information contained on the Record of Borehole Sheets and geological evidence.

Borehole locations and elevations were surveyed in the field by personnel from Central Region Engineering Surveys Section, Toronto.

4. *SUBSOIL CONDITIONS:*

4.1) General:

Beneath the approach fills at the site is a deep deposit of silty clay to clay which can be subdivided into four zones by physical appearance and soil property variations.

The cohesive deposit is underlain by fluvial fine sand followed probably by bedrock.

A description of the subsoil follows. The shear strength varied considerably with depth and will be discussed in a subsequent section, as will be the consolidation characteristics of the subsoil.

4.2) Approach Embankments (Fill):

One borehole was advanced through each approach fill; Borehole #1 through the south embankment and Borehole #2 through the north embankment. With the exception of some 4.0 feet (1.2 m) of road bed gravel, the approaches consist of silty clay and clay fills containing traces of sand and gravel. No field vanes could be turned in the fill materials indicating undrained shear strength to be greater than 2000 PSF (95.8 kN/m^2) everywhere. Some 18 ft. (5.5 m) of fill was intersected at Borehole #1 (south approach) and 21 ft. (6.4 m) at Borehole #2 (north approach).

4.3) Silty Clay to Clay (Zone 1):

Immediately beneath the approach fills at each boring location a 5-8 in. (0.127-0.203 m) layer of black organic topsoil was discovered and this material was underlain by a 15-16 ft. (4.6-4.9 m) thick layer of silty clay to clay with traces of sand and gravel. This layer is characterized as a mottled material with well defined but discontinuous layers or pockets of layered material. Oxidation and desiccation were apparent throughout the zone and occasional thin vertical gypsum seams were noted.

The consistency of the deposit may be described as very stiff. Shear strength was with one exeption, greater than 2000 PSF (95.8 kN/m^2) everywhere.

The layered and mottled material had liquid limits ranging from 62-70%, plastic limits ranging from 28-31% and natural moisture content ranging from 27-37%. In general, the moisture content was below the plastic limit indicating the material to be overconsolidated; due probably to desiccation The layered material was slightly more plastic than the mottled material. Figure 1 is a detailed plasticity chart for the entire cohesive deposit.

The bulk density of the material (layered and mottled) varied between 115 and 128 P.C.F. (1.8-2.1 T/m^3) with the average being 122 P.C.F. (2.0 T/m^3).

4.4) Silty Clay to Clay (Zone 2):

This zone might best be described as a glacial till. The layer varies between 8-12 ft. (2.4-3.7 m) in thickness and was intersected at each boring location. The zone, contains small amounts of sand and occasional gravel. In addition, occasional silt seams and pockets were discovered within the main deposit. In general, the zone is grey in colour, but some evidence of mottling was noted.

The consistency of the deposit may be described as firm. Undrained shear strengths as determined by laboratory and field testing ranged between 270 and 920 PSF (12.9-44.0 kN/m^2).

Physical properties of the deposit as determined from laboratory tests are as follows and are plotted on figure 1:

Liquid Limit	%	58-63
Plastic Limit	%	28-30
Natural Moisture Content	%	33-47

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The bulk density of this material was determined from laboratory tests, to range from 103-119 P.C.F. (1.7-1.9 T/m^3) with an average value of 111 PCF (1.8 T/m^3).

4.5) Silty Clay to Clay: (Zones 3 and 4):

These two zones, although different in physical appearance, are composed of basically the same materials. Taken together these zones varied in thickness from 31.5 - 56 ft. (9.6 - 17.1 m)

Zone 3 is generally a layered material, red, brown and grey in colours. Between layered sections, the material is brown to red-brown in colour. The material varies irregularly between silty clay and clay with occasional silt seams and pockets. Borehole #2 was terminated within this layer at elevation 520 ft. (80.53 m) and at this location, the layered zones could not be easily identified, At the location of Borehole #1, the borehole was advanced through Zone 3 into Zone 4 at elevation 510 ft. (85.2 m).

The consistency of Zone 3 may be described as being firm to stiff as indicated by undrained shear strength values of 630-1120 PSF (30.2 - 53.6 kN/m^2).

Physical properties of Zone 3 material as determined from laboratory tests are as follows and are plotted on figure 1:

Liquid Limit	%	56-59
Plastic Limit	%	25-27
Natural Moisture Content	%	32-45

The bulk density of Zone 3 material varies from 112-118 PCF (1.8 - 1.9 T/m^3).

Zone 4 was a heavily stratified material consisting of irregular layers of brown silt, brown clay, grey silty clay and brown clayey silt, with occasional gravel. Its consistency may be described as firm to stiff. The bulk density of this zone as determined from a single laboratory test is 121 PCF (1.9 T/m^3).

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4.6) *Fine Sand:*

Beneath Zone 4 a minimum 2 ft. (0.6 m) layer of fine grey sand was discovered. Borehole #1 was terminated within this layer.

The relative density of this deposit may be described as dense, based on a standard Penetration Test 'N' value of 82 blows/ft (0.3 m)

Although no borings were advanced below the sand stratum, based on previous subsoil investigation done by this office in the same general vicinity the sand stratum is underlain by dolomite bedrock with shale interbeds.

5. *GROUNDWATER CONDITIONS:*

During the course of the field work, attempts were made to measure groundwater levels in the open boreholes. However, due to the relatively impermeable nature of the subsoil and short duration of the field work groundwater levels could not be established in the open boreholes.

Subsequently, we were able to establish groundwater levels from information supplied by the St. Lawrence Seaway Authority.

As a result of the Welland Canal Channel relocation to the east of the site area, the Seaway Authority has been pumping groundwater from deep wells some 1.5 to 3 miles (2.41-4.83 km) from this site in order to dewater the various roadway tunnels and other excavations involved in the channel relocation. In order to monitor groundwater drawdown, the Seaway Authority has installed and monitored a network of piezometers throughout the Welland area including one piezometer in the immediate vicinity of the Forkes Road - Hwy. 58 site. From their records, we have been able to establish with reasonable accuracy the pre-channel relocation groundwater levels of the site as well as the water level variation at the site since pumping commenced in September 1968, until the present time. Water level drawdown reached a steady state condition in mid 1971 and little variation has been noted since that time. The latest reading on the piezometer near the site was made by the Seaway Authority on December 30, 1973.

Pre-channel relocation groundwater levels at Forkes Road and Hwy. 58 (ie. prior to 1969) were at elevation 575 $\pm$ ft. (175 m). By mid 1971 the steady state drawdown conditions had lowered the water levels some 35 ft. (10.7 m) to elevation 540 $\pm$ ft (164.5 m). The piezometer reading in December 1973 indicated groundwater levels to be at elevation 545 $\pm$ ft. (166.1 m). See figure 6 for a plot of water level variations versus time.

6. *DISCUSSION OF SHEAR STRENGTH AND CONSOLIDATION CHARACTERISTICS:*

6.1) Unconsolidated Undrained Tests:

Undrained shear strength values within the cohesive fill and subsoil were determined using a standard MTC vane in the field. In addition three laboratory unconfined compression tests were performed on selected samples. The results of the U.C. tests indicated strength values of approximately 30-55% of those obtained at comparable elevations within the subsoil using the field vane. It is well known that with increasing depth, it is increasingly difficult to obtain undisturbed samples and in the limiting case sample disturbance is such that in fact, laboratory U.C. tests measure the remoulded strength. The ratio of undisturbed shear strength to remoulded strength (sensitivity) obtained from field vane tests ranged from 2-3.4. The same range in ratios between laboratory U.C. and field vane strengths indicates to some extent the disturbed nature of samples, but also tends to confirm the validity of strength values as determined from the field vane tests.

Within the fill both field vane and laboratory U.C. tests gave undrained shear strength values in excess of 2000 PSF (95.8 kN/m^3). From the original ground level elev. 580 $\pm$ ft. (176.8 m) to elev. 560 $\pm$ ft. (170.7 m), the undrained strength based on field vane tests varied between 640 and 1120 PSF (30.6-53.6 kN/m^2). Below elev. 560 ft. (170.7 m) the undrained shear strength based on field vane tests varied between 630 and 1120 PSF (50.2 and 53.6 kN/m^2).

The following table summarizes the laboratory and field strength values obtained and indicates the average values used for analytical purposes.

Elevation	Laboratory U.C. Tests	Field Vane	Average Shear Strength Assumed
Embankment Fill	2900 psf ($138.5\ kN/m^2$)	>2000 psf (>$95.8\ kN/m^2$)	2500 psf ($119.7\ kN/m^2$)
580 - 560 ft (176.8 - 170.7 m)	270 psf ($12.93\ kN/m^2$)	1020->2000 psf (48.8->$95.8\ kN/m^2$)	2000 psf ($95.8\ kN/m^2$)
560 - 498 ft (170.7 - 151.8 m)	630 psf ($30.3\ kN/m^2$)	640-1120 psf (30.6 - $48.8\ kN/m^2$)	800 psf ($38.3\ kN/m^2$)

6.2) Consolidated Undrained Triaxial Tests with Pore Pressure Measurement:

The detailed test results for consolidated undrained tests (stage tests) with pore pressure measurements are presented in figures 3, 4 and 5.

The effective stress parameters are quite consistent throughout the cohesive deposit with C' = 0 psf and Ø' = 21-23°. For effective stress stability analysis, values of C'=0 and Ø' = 22° have been used for the subsoil below original ground level (elevation 580 ± ft. (176.8 m)) and C' = 150 psf (4.79 kN/m^2) and Ø' = 23° for the cohesive approach fills.

6.3) Consolidation Characteristics:

The results of the consolidation tests conducted are presented in graphical form in figure 2. Interpretation of these results to obtain preconsolidation pressures was made using the Casagrande and Schmertmann construction if possible.

Based on the consolidation test results, it appears that the deposit is lightly overconsolidated below elevation 565 ± ft. (172.2 m). Above this elevation, the material is more heavily overconsolidated due probably to desiccation. Within this desiccated crust very little if any consolidation settlement can be expected.

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7. *DISCUSSION AND RECOMMENDATIONS:*

7.1) General:

It is proposed to replace the concrete deck on the structure carrying Hwy. 58 over Forkes Road in the City of Welland, Regional Municipality of Niagara. The deck replacement has become necessary due to the severe deterioration of concret in the decks - ie. spalling, cracking, exposed reinforcing steel etc. In addition to the deterioration of the decks, large settlements have been observed between the approach embankments and adjacent structure abutments. For example, the ends of the concrete curbs have settled differentially about 6-10 inches (0.152-0.254 m) from the rest of the structure. Accordingly, it is further proposed to incorporate remedial measures to accomodate future settlements, into the redesign of the deck.

A subsoil investigation has been completed by this office in order to assess the existing subsoil and groundwater conditions and, based on this information, a thorough analysis of overall structure and embankment stability has been done. Also, settlement analyses have been done and compared with measured values of settlements to determine the causes of these observed settlements and to allow a prediction of the magnitude of future settlements to be made.

Subsoil at the site consists generally of a deep deposit of silty clay to clay underlain by a fine sand stratum and then bedrock.

7.2) Embankment Stability:

During the course of the field work, visual observations disclosed no signs of incipient slope failure or potential instability. Side slopes are built to 1 3/4 : 1 and are well sodded. No tension cracks, sod slippage or bulging were noticed. Forward slopes are retained by 6-8 ft. (1.8-2.4 m) high concrete retaining walls. Behind the walls, the slopes are built to 2:1 and are riprapped and paved. No cracking or tilting of the retaining walls was noted suggesting the forward slopes to be stable in situ.

Based on the total and efective stress parameters discussed previously, both 'total' and 'effective' slope stability analyses have been performed. The results of these analyses indicate that the forward and side slopes have adequate factors of safety with respect to deep rotational failure. The analyses are based on the assumption that no drastic changes in loading will occur due to the new deck. It should be emphasized that if the new deck does weigh considerably more than the existing deck, further stability analyses will be required at a future time to assess the impact of the increased loading on the stability of the forward embankment slopes.

7.2) Settlements:

During the course of the field investigation the structure and approach fills were examined visually for tell-tale signs of differential settlements. As previously noted, the concrete curbs along the top inside portion of the abutment wing walls had broken away from the walls, due probably to the general concrete deterioration, and these curbs resting on the fills had tilted downwards away from the deck suggesting the fills had settled differentially as much as 6-12 inches (0.1524-0.305 m) relative to the rest of the structure. From discussions with District maintenance personnel, we have established that patching in the area of fills adjacent to the abutments has been required many times since the structure was built in 1955. At the time of the field work, the depth of pavement in the north bound lane on the south approach was measured as being 12-14 inches (0.305-0.356 m) suggesting the fill at that location had settled differentially at least 9-11 inches (0.229-0.279 m) to the south abutment. It is then reasonable to expect a similar amount of settlement and possibly much more in the fill at the north abutment. However, besides these observations there are no other obvious signs such as tilting of any parts of the structure which would indicate significant differential settlement across the structure.

As previously discussed under the section on groundwater conditions there has been a lowering of the general groundwater levels in this area by some 30 $\pm$ ft. (9.14 m); this drawdown being the result of pumping water from the aquifer below the cohesive deposits in the area, by the St Lawrence Seaway Authority, in connection with the relocation of the Welland

Canal Channel to the east of the site. As a result of this groundwater lowering, areal settlements of varying amounts, depending on the depth of overburden and variation in consolidation characteristics of the subsoil, are to be expected.

Based on visual observations and a knowledge of the groundwater drawdown, several preliminary conclusions may be drawn:

(1) Settlements of the subsoil under the fills and structures has occured due to drawdown of the groundwater and due to imposed loading.

(2) Settlements within the fills have also occured for various reasons, one probably being the downward percolation through the fills of water from the granular road bed and surface of the fills.

In January 1974, a precise survey of the structure was made by personnel of the Central Region Engineering Surveys Section, Toronto. As a result, settlements of 9 and 23 inches (0.229 and 0.584 m) were measured at the south and north abutments respectively and 11.8 and 17.3 inches (0.300 and 0.439 m) at the south and north piers respectively.

A detailed settlement analysis, based on well established consolidation characteristics of the cohesive subsoil as determined by laboratory tests was performed. Total settlements for consolidation due to imposed fill and structure loading and consolidation due to groundwater drawdown have been calculated. Time-settlement computations lead to a prediction that 90% of the settlement due to imposed loading under the structure and fills should have occured within 20 to 25 years after construction. Hence - it is reasonable to assume that consolidation of the subsoil beneath the structure and fills due to their imposed loads, is essentially complete at this point in time. A similar time period has been predicted for 90% of the drawdown settlement to occur. Since pumping and drawdown has been underway for approximately 5 years, we estimate that up to 40% of the drawdown consolidation has already occured. From the foregoing considerations we have arrived at a predicted value of settlement

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still to take place of the order of 4-5 inches (0.1016 - 0.1270 m) over the next 10 - 15 years. Refer to Table I for a summary of predicted and measured settlements.

It will be noted that except for the case with the south abutment, there is very poor agreement between predicted and measured settlements.

There are several probable reasons for these discrepancies. The measured settlement figures were arrived at by taking the difference between the recent surveyed elevations and the corresponding elevations shown on the original design drawings. Since the predicted settlements do appear to be reasonable and proportionate relative to the points for which they were determined, and since no proof can be found that the original design elevations were in fact adhered to when the structure was built, we believe the measured settlements to be highly suspect. Further evidence to cast doubt on the original design elevations was obtained in discussions with District maintenance personnel. The bridge maintenance foreman has advised us that a number of the bridge girders are of a different size than those reflected by the design drawings. Finally, with the exception of the general concrete deterioration of the deck and pier caps, there has been no structural damage of the type associated with large differential settlements. There is no obvious tilting of structural parts and the expansion joints are operating satisfactorily.

It should be emphasized at this point that although the measured settlement figures are suspect, and the predicted values reasonable based on the information available to us, it would be very desirable from our point of view to obtain at least two more sets of elevations on the piers and abutments, these two sets of readings should be taken at 6 month intervals. With such information, the accuracy of the present predicted settlement values could be ascertained or corrected and a much more precise prediction of future time-settlements made.

Based on the predicted settlement values we have several recommendations to offer pertaining to remedial measures that can be incorporated into the redesign of the

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deck. The new deck should be designed such that its weight is comparable or less than that of the existing deck so that no loads in excess of those presently carried by abutments and piers are transmitted to the subsoil. Twenty foot concrete approach slabs should be used to minimize any further differential settlements between fills and abutments that may still occur. Thirdly, the new deck should be designed to accomodate up to 1 1/2 inches (0.038 m) of differential settlement that occur between piers and abutments. Fourthly, those sections of the fills which have been adversely affected by large amounts of differential settlements, particularly the sections within 30-40 ft. (9.14-12.19 m) of the structure abutments should be repaired in the following manner. The clay surface of the fill beneath the granular road bed should be shaped and cambered. Then the granular fill subgrade should be replaced and should extend over the full top width of the embankments. With these measures, it should be possible to prevent ponding of water beneath the road surface and provide good drainage for surface water out to the embankment side slopes, and minimize further settlements within the fills themselves.

Finally, we reserve the right to review and revise our predictions and recommendations in one year's time, based on the results of two more sets of measured settlements taken at 6 month intervals.

8. *MISCELLANEOUS:*

The field work was carried out during the period of December 10-19, 1973 under the supervision of Mr. L.J. Hodge, Project Foundations Engineer. Equipment used was owned and operated by P.V.K. and Sons Drilling Company.

This report was prepared by Mr. L.J. Hodge and reviewed by Mr K. G. Selby, Supervising Foundation Engineer.

L. J. Hodge

L. J. Hodge,
Project Foundations Engineer.

K. G. Selby

K.G. Selby,
Supervising Foundation Engineer.

LJH/sh

February 26, 1974

TABLE 1: SUMMARY OF PREDICTED AND MEASURED SETTLEMENTS AT HWY. 58 - FORKES ROAD O'PASS

W.O. 73-11097

Breakdown of Consolidation Settlements	North Abutment 20' Ht.		North Pier		South Pier		South Abutment 17' Ht.	
	Predicted	Actual	Predicted	Actual	Predicted	Actual	Predicted	Actual
Total S: Imposed Loading & Drawdown	15.1" (0.38 m)	22.2" (0.56 m)	9.9' (0.25 m)	17.3" (0.44 m)	8.6" (0.22 m)	11.8" (0.30 m)	13.8" (0.35 m)	9" (0.23 m)
Total S: Drawdown Only	7.7" (0.20 m)		7.4" (0.19 m)		6.4" (0.16 m)		7.8" (0.20 m)	
Est. Drawdown Settlement to Start of 1974	3.1" (0.08 m)		3.0" (0.08 m)		2.6" (0.07 m)		3.1" (0.08 m)	
Predicted Settlement still to occur	4.6" (0.12 m)		4.4" (0.11 m)		3.8" (0.01 m)		4.7" (0.12 m)	

ASSUMPTIONS: (1) Due to fill - S is 90% complete.

(2) Due to Drawdown - S is 40% complete.

APPENDIX I

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH — FOUNDATIONS OFFICE

RECORD OF BOREHOLE № 1

JOB 73-11097 — LOCATION Hwy. 58, Sta. 179 + 43, o/s: 18' RT ℄ — ORIGINATED BY LJH

W.P. 78-73-01 — BORING DATE December 10 - 11, 1973 — COMPILED BY LJH

DATUM Geodetic — BOREHOLE TYPE Hollow Stem Auger Bombardier Mounted — CHECKED BY

(m)	ELEV. / DEPTH ft.	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	BLOWS/FOOT (0.3 m)	ELEV. SCALE ft./(m.)	SHEAR STRENGTH P.S.F. (kN/m²)	WATER CONTENT %	BULK DENSITY γ P.C.F. (T/m³)	REMARKS GR.SA.SI.CL
181.5 / 0.0	595.4 / 0.0	Ground Level								
180.3 / 1.2	591.4 / 4.0	Gravel								
		FILL Silty clay to clay, traces sand and gravel. Brown Very stiff to hard.	1	SS	11	590 (179.8)				
			2	TW	PH		+>2000			
			3	TW	PH		+>2000			
			4	TW	PH	580 (176.8)	+>2000			
176.0 / 5.5	577.4 / 18.0	Organic top soil Black. Silty clay to clay, num. silt seams/pockets, traces of sand and gr. Grey-brown. Stiff (Zone 1)	5	TW	PH		+>2000			
			6	TW	PH	570 (173.7)	+>2000		128 (2.1)	
		Layered, red brown and grey	7	TW	PH		+	o	115 (1.8)	
170.4 / 10.1	562.4 / 33.0	Silty clay, occ. silt seams/pockets; occ. gravel Grey to mottled. Firm (Zone 2)	8	TW	PH	560 (170.7)	+	o	119 (1.9)	
169.0 / 12.5	554.4 / 41.0	Layered red, brown, grey mottled	9	TW	PH		o +	o	103 (1.7)	
		Layered red, brown, grey Silty clay to clay, occ. silt seams and pockets Brown to red-brown Firm to Stiff. (Zone 3)	10	TW	PH	550 (167.6)	+		117 (1.9)	w.l. el.545 established Dec. 30/73 from info. supplied by St.Lawrence Seaway Authority
			11	TW	PH					
			12	TW	PH	540 (164.6)	+	o	116 (1.9)	
		Layered, grey, brown	13	TW	PH	530 (161.5)	+	o	118 (1.9)	
		Layered, grey, brown	14	TW	PH	520 (158.5)	o +	o	112 (1.8)	
155.5 / 26.0	510.2 / 85.2	Layered red, brown, grey Irregular layers of Brown silt. Brown clay (Zone 4) Grey silty clay Brown clayey silt Occ. gravel Firm to stiff	15	TW	PH	510 (155.4)			121 (1.9)	
151.9 / 29.6	498.4 / 97.0					500 (152.4)				
151.3 / 30.2	496.4 / 99.0	Sand, fine, grey, dense End of Borehole	16	SS	82					

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT (0.3 m) 20 40 60 80 100

SHEAR STRENGTH P.S.F. (kN/m²) o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 500 1000 1500 2000 2500

50 100 (kN/m²)

LIQUID LIMIT — W_L PLASTIC LIMIT — W_P WATER CONTENT — W

WATER CONTENT % 20 40 60

20 / 15 5 / 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO

DESIGN SERVICES BRANCH

RECORD OF BOREHOLE Nº 2

FOUNDATIONS OFFICE

JOB 73-11097
W.P. 78-73-01
DATUM Geodetic

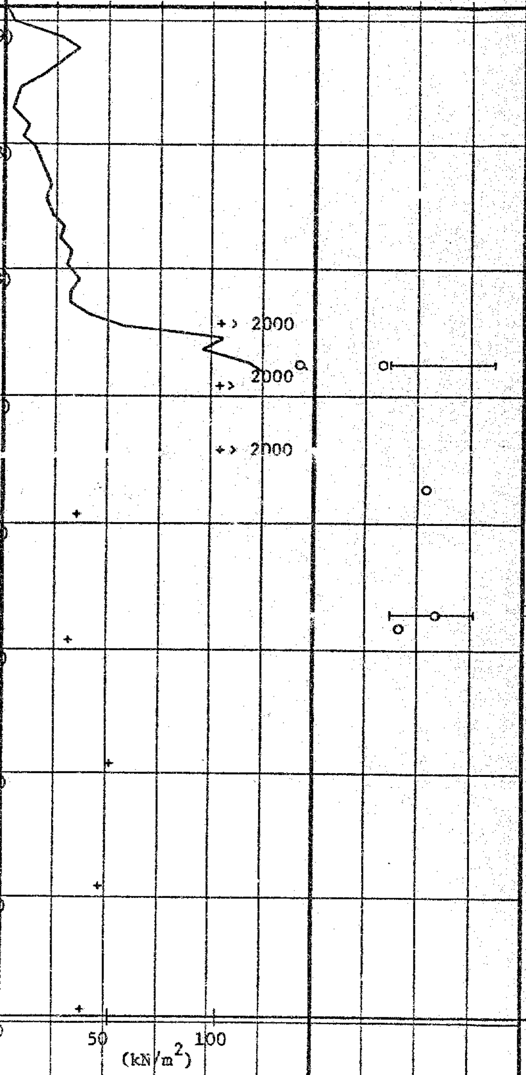
LOCATION Hwy. 58, Sta. 180 + 63, o/s: 18' LT ℄
BORING DATE December 19, 1973
BOREHOLE TYPE Hollow Stem Auger Bombardier Mounted

ORIGINATED BY LJH
COMPILED BY LJH
CHECKED BY

(m)	ELEV. DEPTH ft.	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT (0.3m)	ELEV. SCALE ft./(m.)	WATER CONTENT %	BULK DENSITY γ P.C.F. (T/m³)	REMARKS GR.SA.SI.CL.
183.1	600.8	Ground Level				600 (182.9)			
0.0	0.0	Gravel							
181.9 / 1.2	596.8 / 4.0	FILL Silty clay to clay traces to some sand, traces gravel. Brown. Very Stiff to hard.				590 (179.8)			
176.7 / 6.4	579.8 / 21.0	~~Organic topsoil-Black~~ Silty Clay to clay, numerous silt seams occ. gravel Grey-brown, some mottled areas. Stiff to very stiff. Layered red, brown, grey (Zone 1)	1	TW	PH	580 (176.8)			
			2	TW	PH	570 (173.7)		122 (2.0)	
			3	TW	PH				
171.8 / 11.3	563.8 / 37.0	Silty clay occ. silt seams/pockets. some irregular grey, and brown layering. Grey Firm. (Zone 2)	4	TW	PH	560 (170.7)			
168.2 / 14.9	551.8 / 49.0	Silty clay to clay occ. silt seams/pcts. Brown to red brown. Layered zones not defined. firm to stiff (Zone 3)	5	TW	PH	550 (167.6)		110 (1.8)	w.l. el.545 est.30/12/73 from info. supplied by St.Lawrence Seaway Authority.
			6	TW	PH	540 (164.6)			
			7	TW	PH	530 (161.5)			
158.6 / 24.5	520.3 / 80.5	End of Borehole	8	TW	PH	520 (158.5)			
						510 (155.4)			

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT (0.3 m) 20 40 60 80 100
SHEAR STRENGTH P.S.F. (kN/m²) ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 1000 2000

LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W
WATER CONTENT % 20 40 60



20 15 5 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

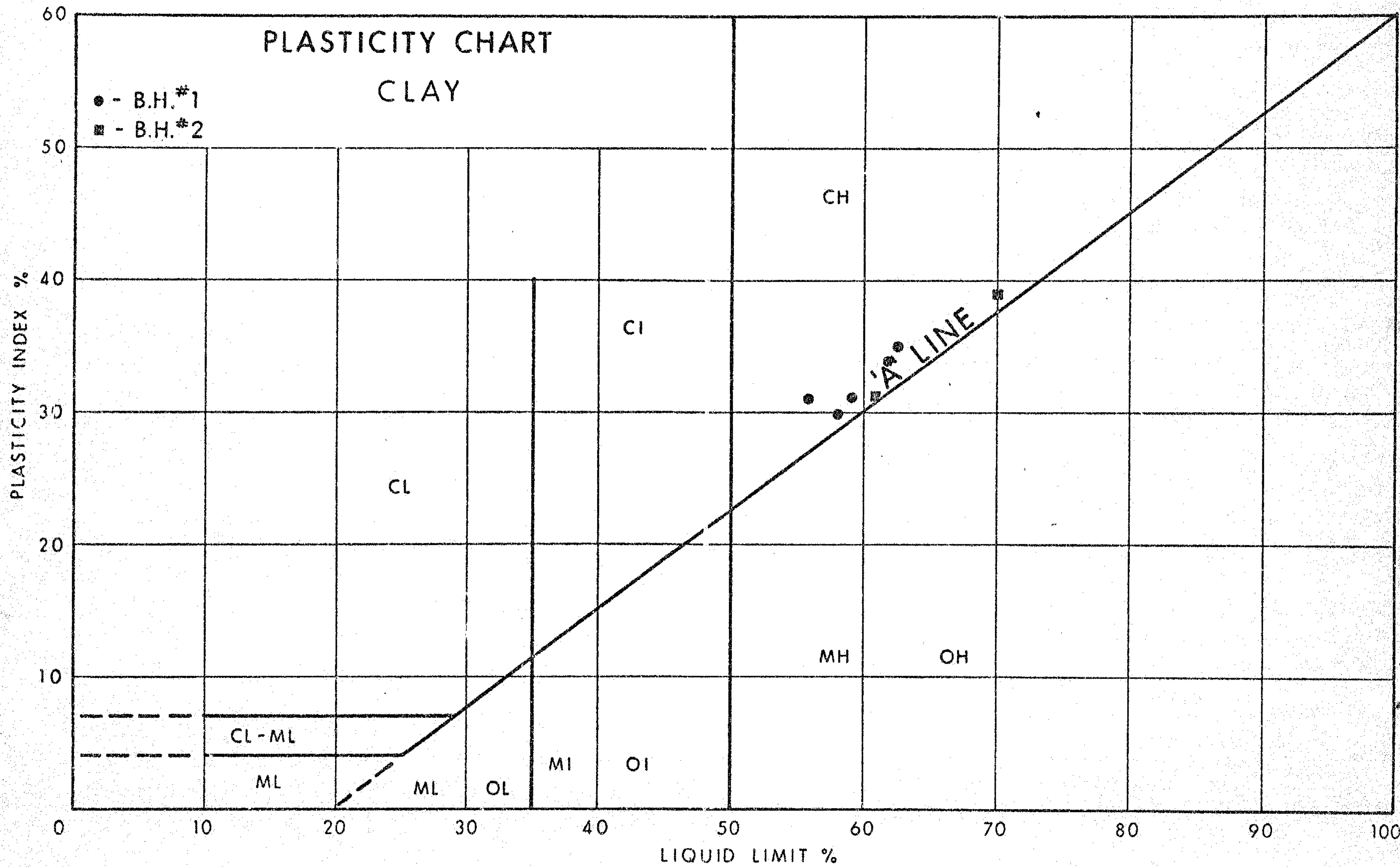

PLASTICITY CHART
CLAY
● - B.H.#1
■ - B.H.#2
CH
CI
'A' LINE
CL
MH
OH
CL-ML
ML
ML
OL
MI
OI
PLASTICITY INDEX %
LIQUID LIMIT %
0
10
20
30
40
50
60
70
80
90
100

FIG. 1

VOID RATIO – PRESSURE CURVES

JOB NO. 73-11097

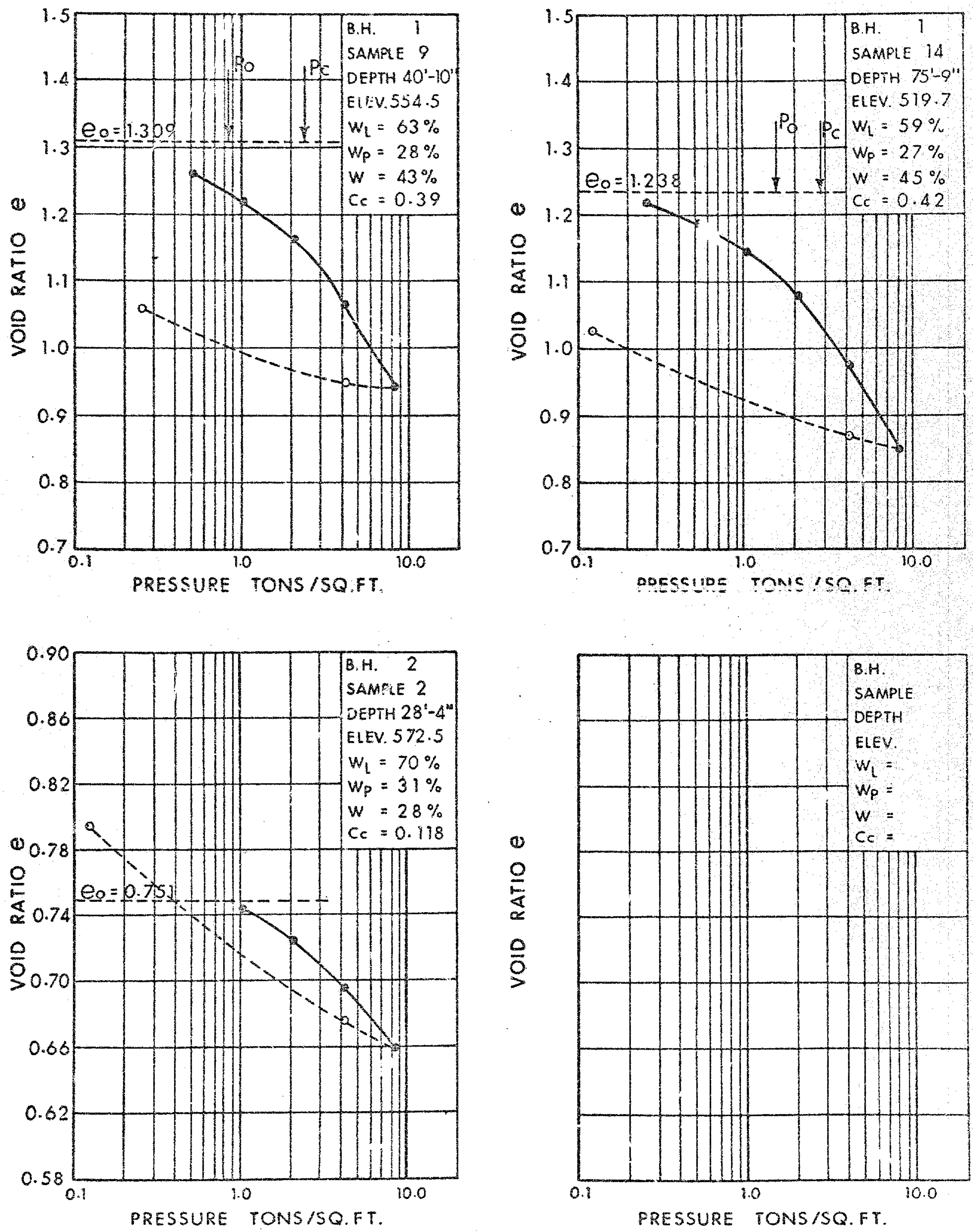
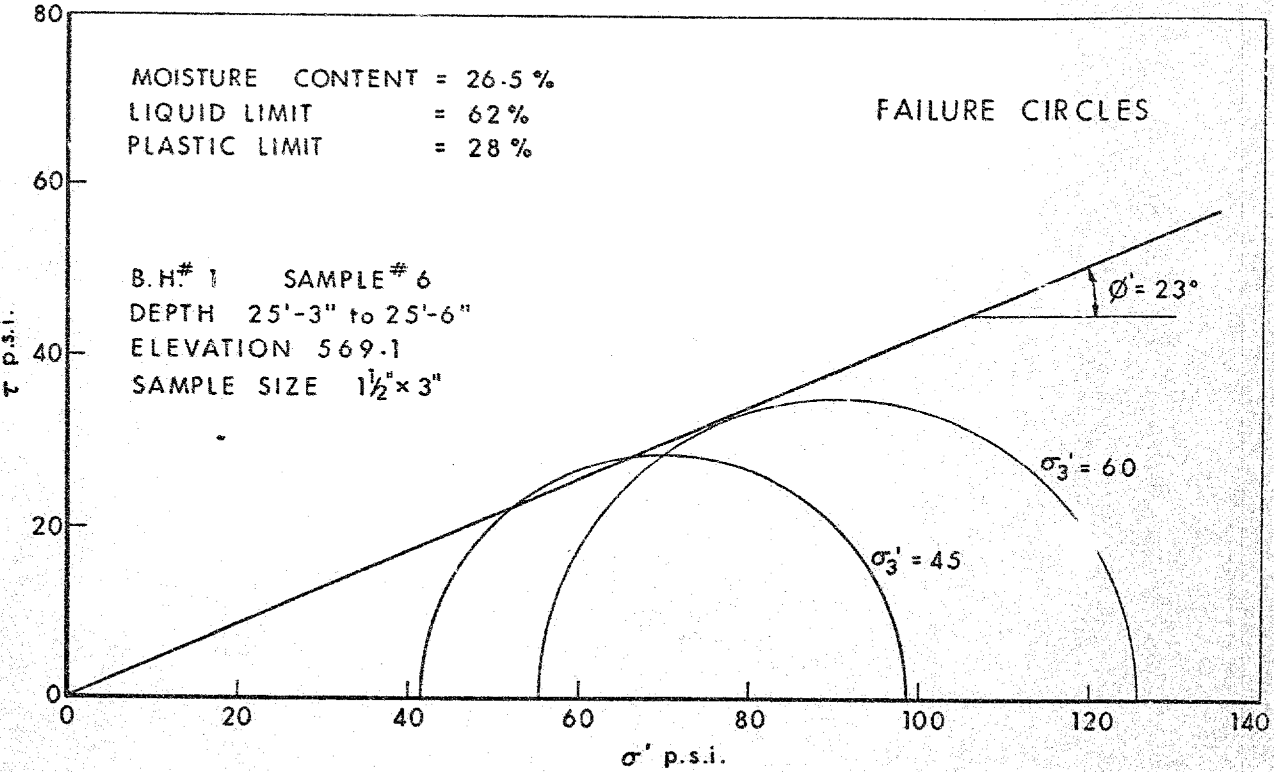


FIG. 2



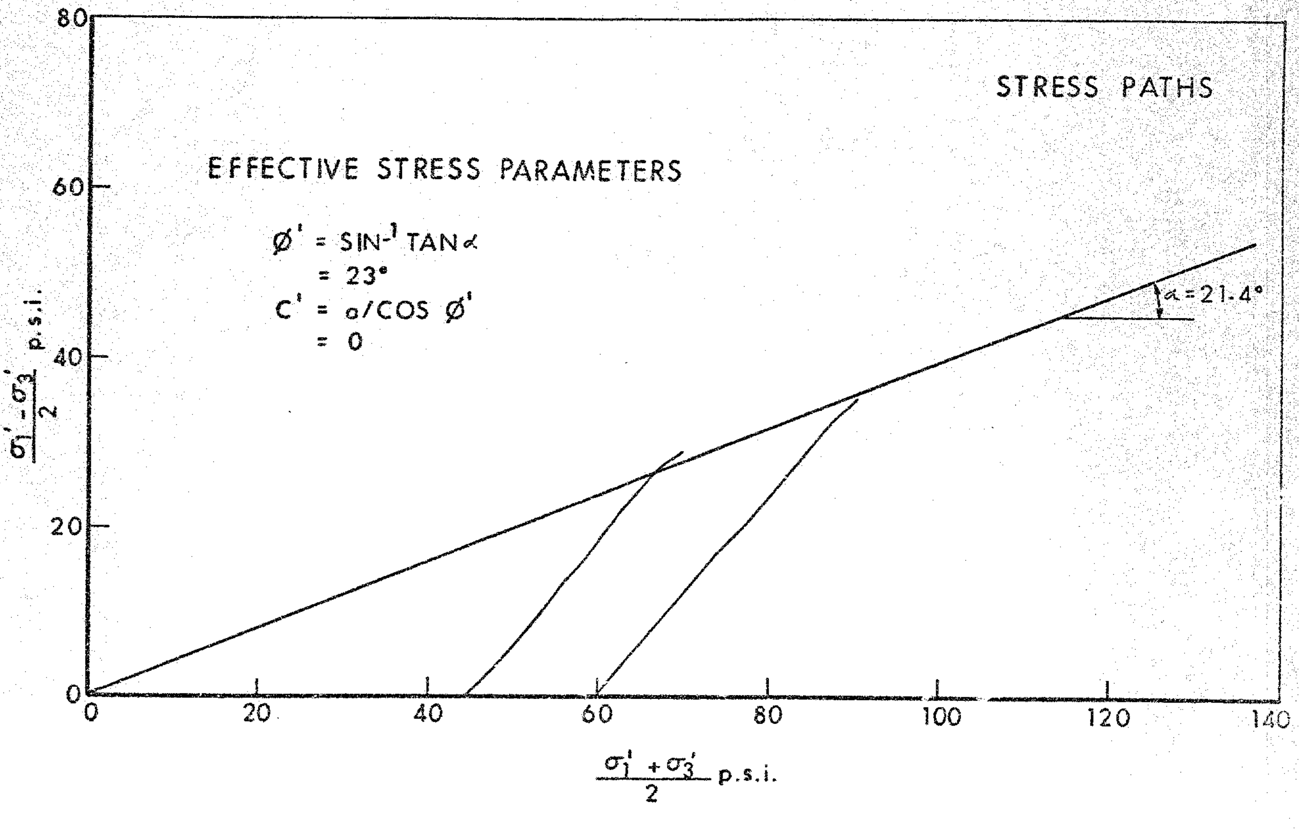
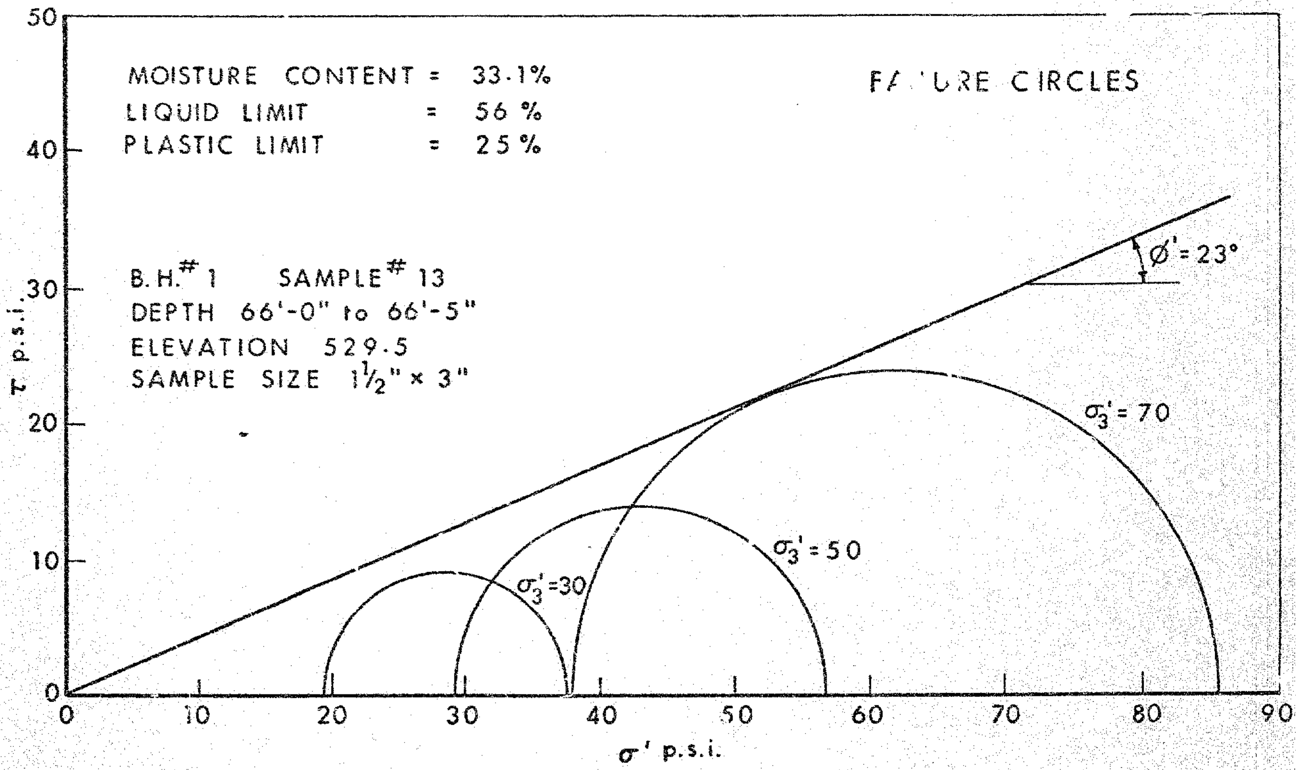


FIG. 3

W.O. 73-11097



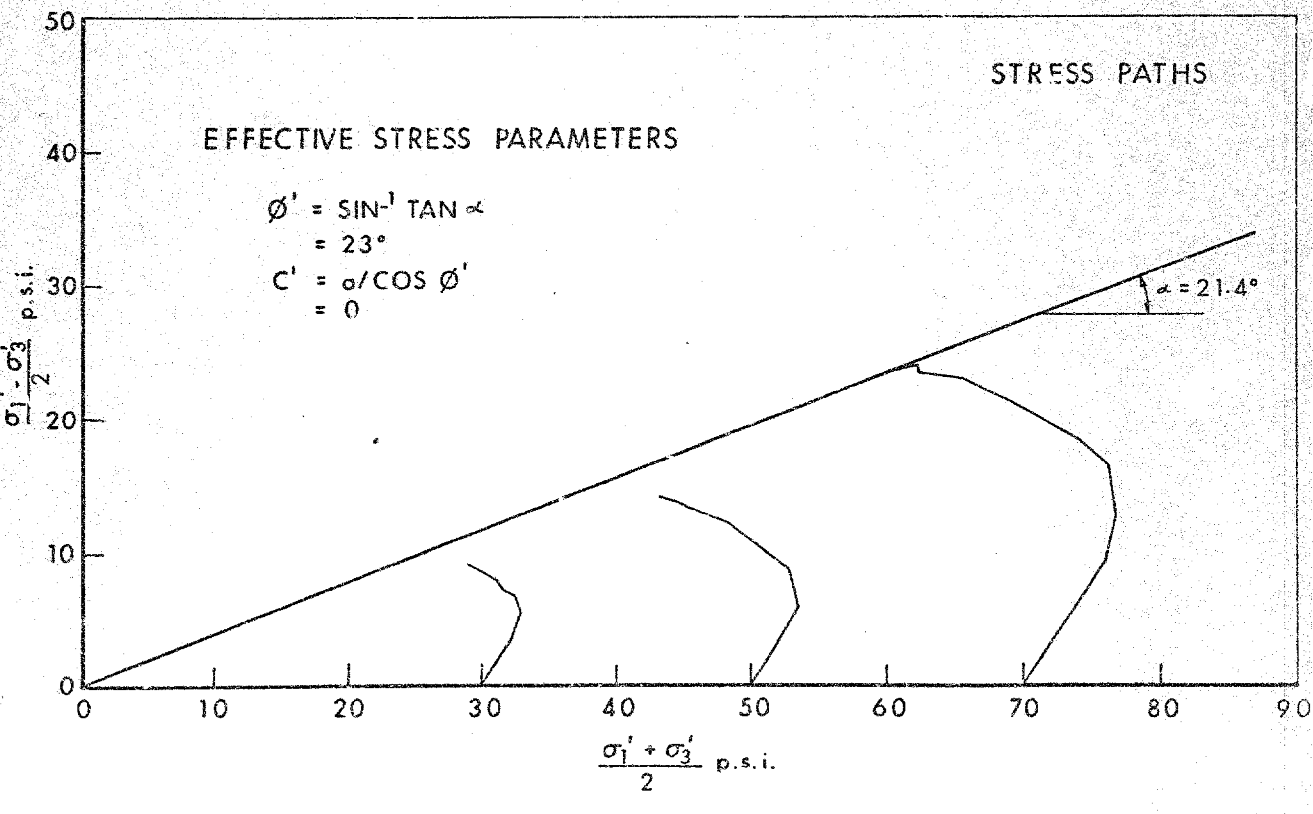
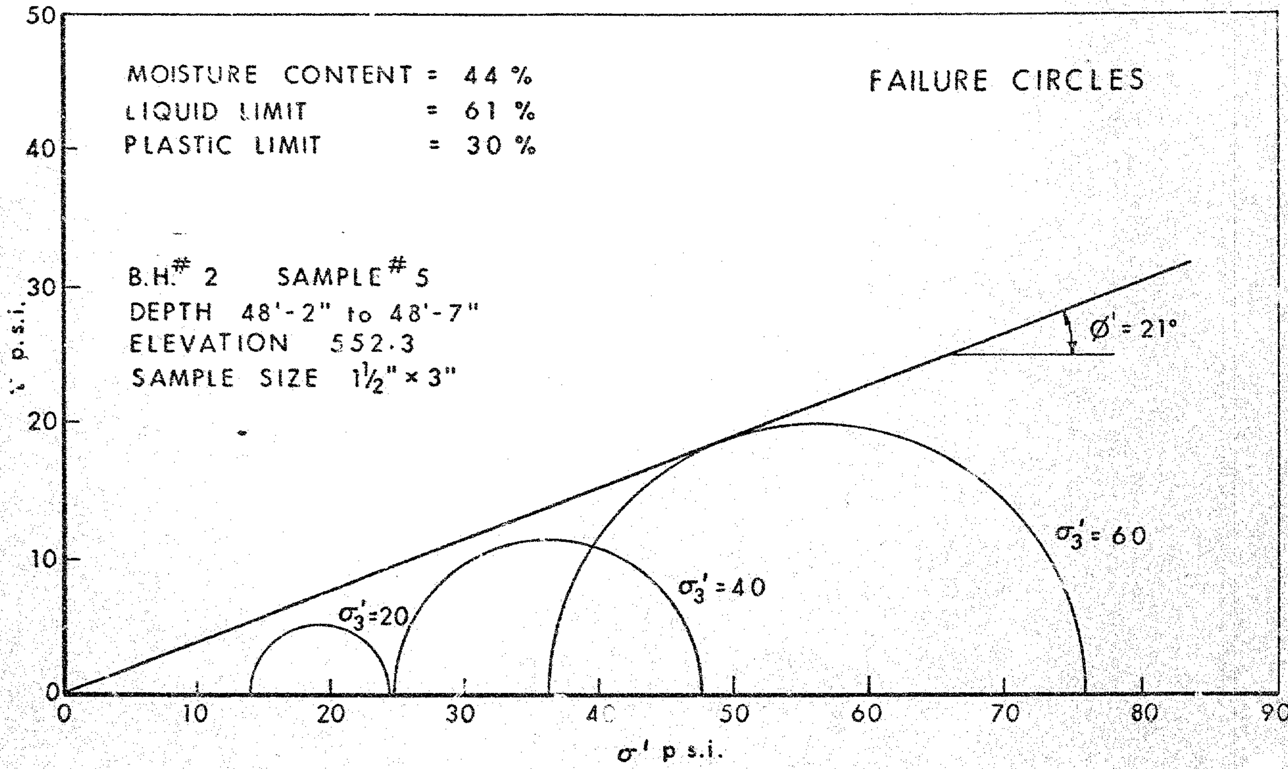


FIG. 4

W.O. 73-11097



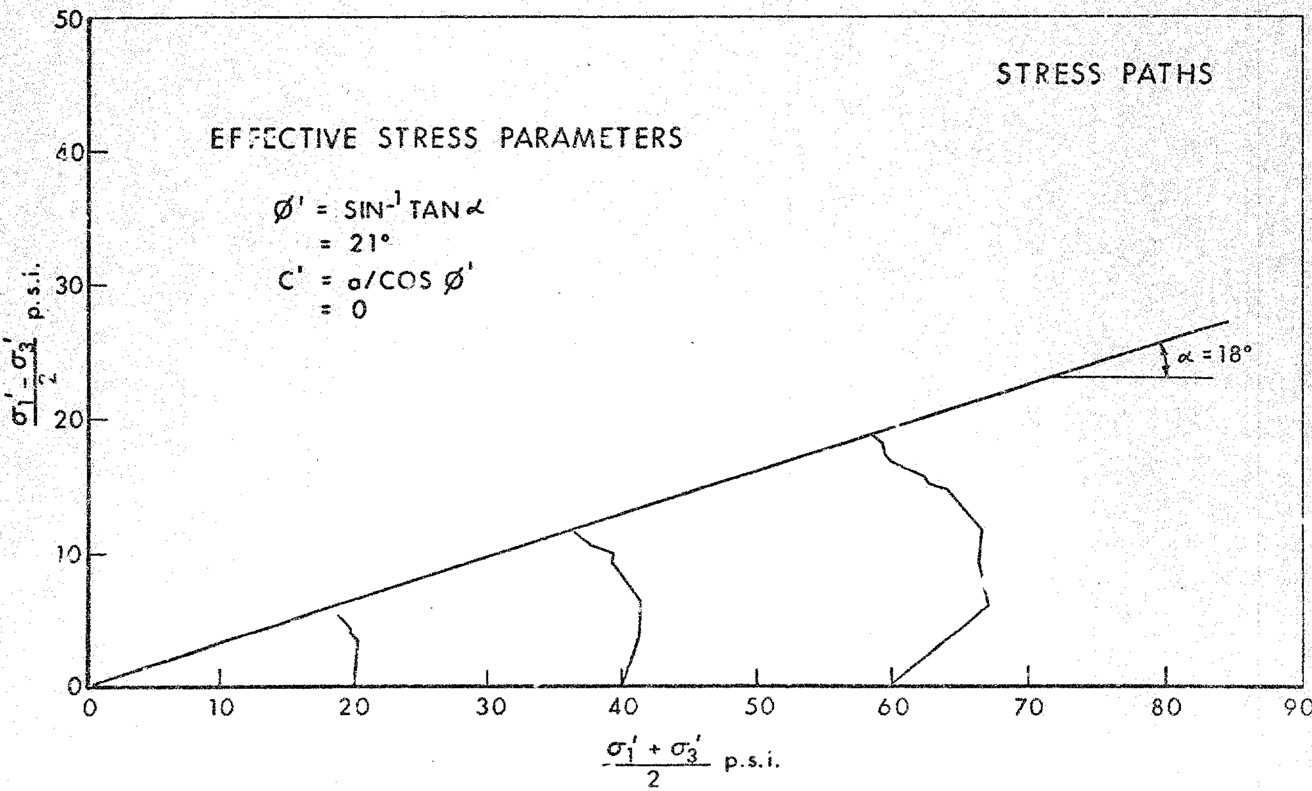
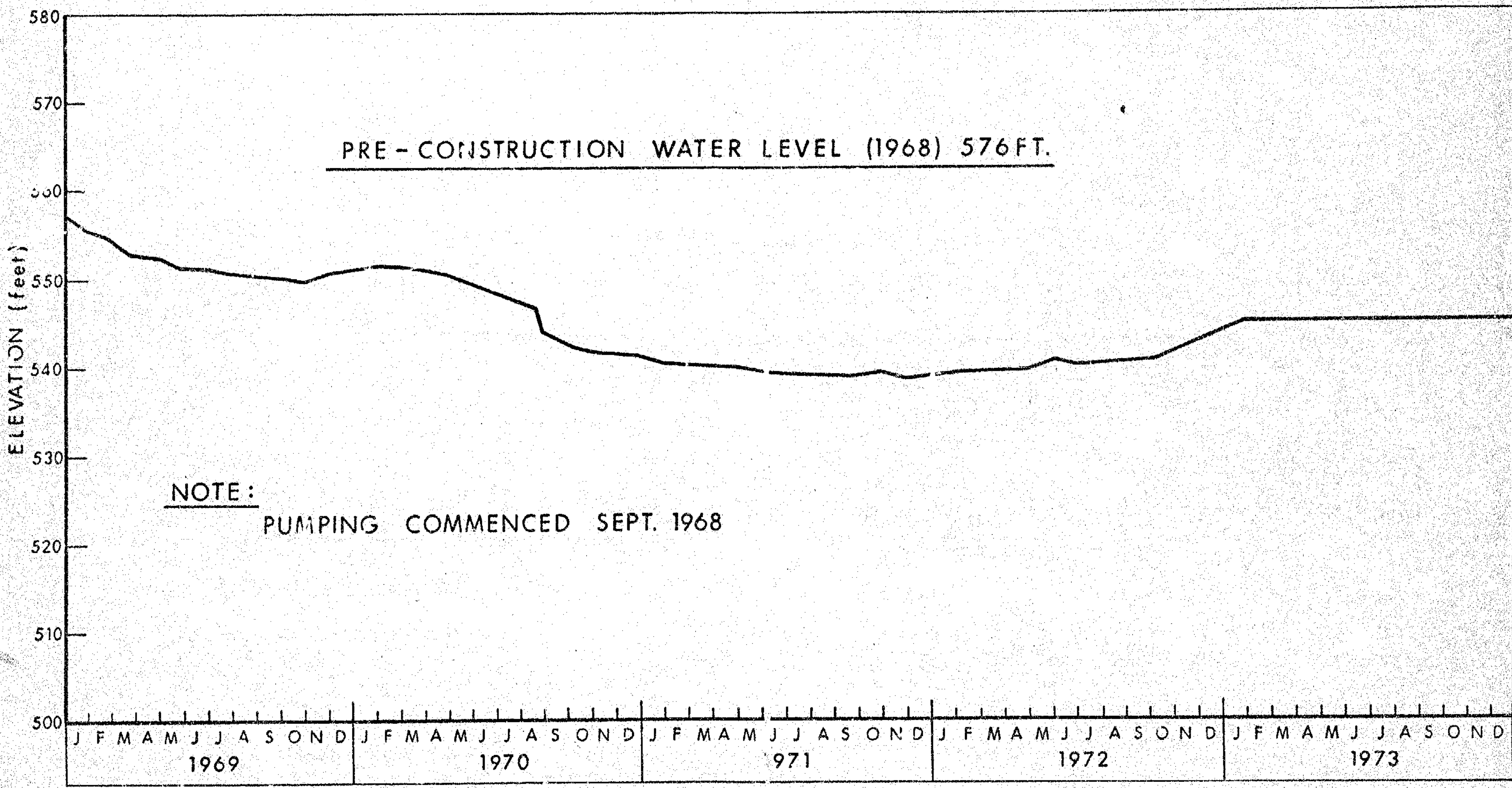


FIG. 5

W.O. 73-11097



VARIATION IN PIEZOMETRIC LEVEL IN ROCK AQUIFER, VICINITY OF FORKES ROAD AND HWY. 58 DUE TO GROUND WATER DRAWDOWN BY: ST. LAWRENCE SEAWAY AUTHORITY PUMPING OPERATIONS.

FIG. 6

W.O. 73-11097

ABBREVIATIONS & SYMBOLS USED IN THIS REPORT

PENETRATION RESISTANCE

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

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

DESCRIPTION OF SOIL

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

CONSISTENCY	c LB./SQ.FT.
VERY SOFT	0 - 250
SOFT	250 - 500
FIRM	500 - 1000
STIFF	1000 - 2000
VERY STIFF	2000 - 4000
HARD	> 4000

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

TERMS TO BE USED IN DESCRIBING SOILS:-

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

TYPE OF SAMPLE

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

P.H. SAMPLE ADVANCED HYDRAULICALLY
P.M. SAMPLE ADVANCED MANUALLY

SOIL TESTS

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

FD-9b (Rev. Jan. 73)

ABBREVIATIONS & SYMBOLS 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
w_S SHRINKAGE LIMIT
I_L LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
e_{max} VOID RATIO IN LOOSEST STATE
e_{min} VOID RATIO IN DENSEST STATE
I_D DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
RELATIVE DENSITY D_r IS ALSO USED
h HYDRAULIC HEAD OR POTENTIAL
q RATE OF DISCHARGE
v VELOCITY OF FLOW
i HYDRAULIC GRADIENT
k COEFFICIENT OF PERMEABILITY
j SEEPAGE FORCE PER UNIT VOLUME
m_v COEFFICIENT OF VOLUME CHANGE $= \frac{-\Delta e}{(1+e)\Delta\sigma'}$
c_v COEFFICIENT OF CONSOLIDATION
C_c COMPRESSION INDEX $= \frac{\Delta e}{\Delta \log_{10} \sigma'}$
T_v TIME FACTOR $= \frac{c_v t}{d^2}$ (d, DRAINAGE PATH)
U DEGREE OF CONSOLIDATION
τ_f SHEAR STRENGTH
c' EFFECTIVE COHESION INTERCEPT
ϕ' EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
IN TERMS OF EFFECTIVE STRESS $\tau_f = c' + \sigma' \tan \phi'$
c_u APPARENT COHESION
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

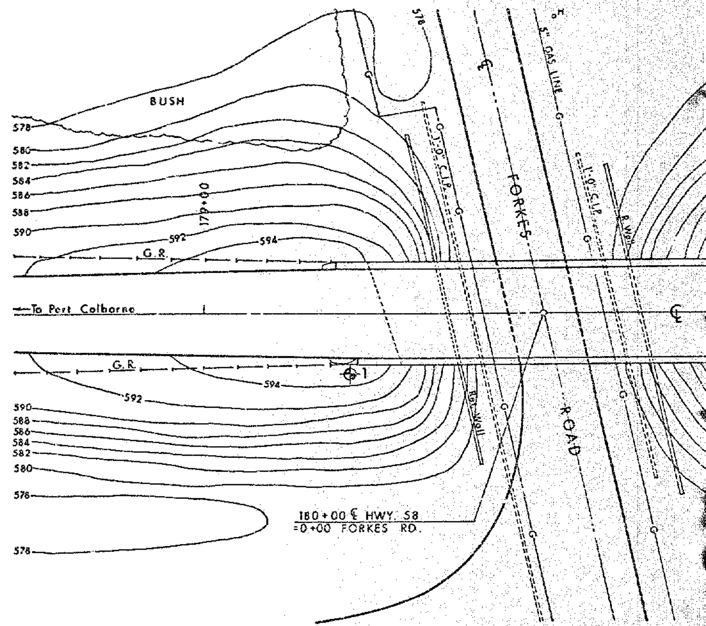
d DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ ANGLE OF WALL FRICTION
K DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_o COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

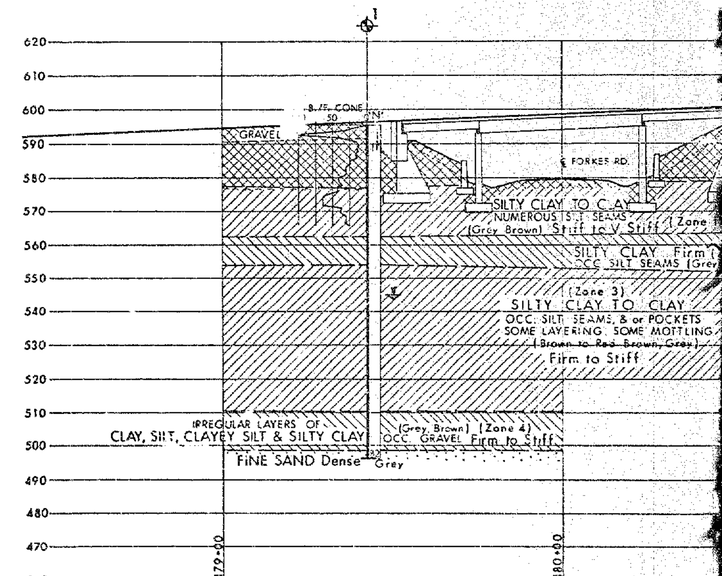
B BREADTH OF FOUNDATION
L LENGTH OF FOUNDATION
D DEPTH OF FOUNDATION BENEATH GROUND
N DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
k_s MODULUS OF SUBGRADE REACTION

SLOPES

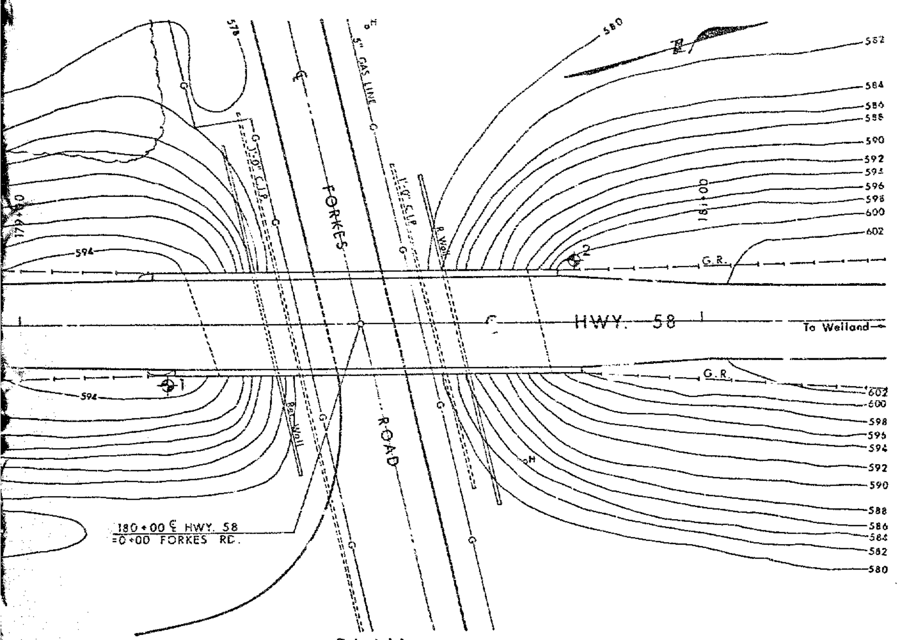
H VERTICAL HEIGHT OF SLOPE
D DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
β ANGLE OF SLOPE TO HORIZONTAL



PLAN
SCALE
20 10 0 20 40FT.

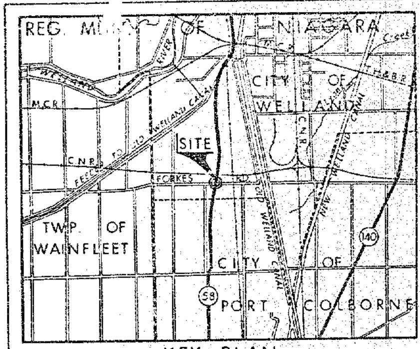


℄ PROFILE
SCALE
20 10 0 20 40FT.



PLAN

SCALE 20 10 0 20 40FT.



KEY PLAN

SCALE IN MILES

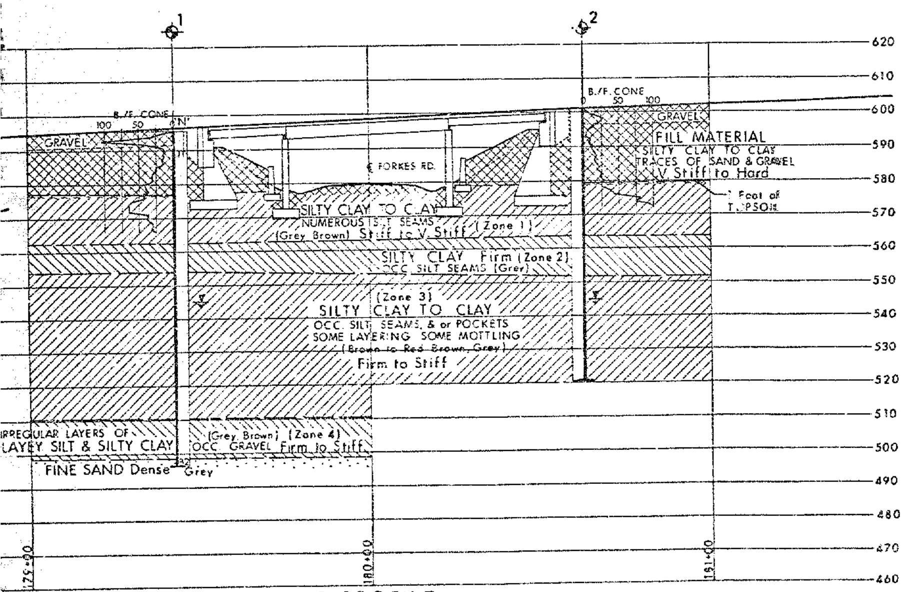
LEGEND

- Bore Hole
- Cone Penetration Test
- Bore Hole & Cone Test
- Water Levels established at time of field investigation.
- Water Levels estimated Dec. 1973 from information Supplied by ST. LAWRENCE SEAWAY AUTHORITY

NO.	ELEVATION	STATION	OFFSET
1	595.4	179+43	18' RT.
2	600.8	180+63	18' LT.

NOTE: FOR CONTRACT DOCUMENT
The complete foundation investigation report for this structure may be examined at the Strucural Office and Foundations Office, Downsview, and at the HAMILTON District Office.

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



℄ PROFILE

SCALE 20 10 0 20 40FT

REF. NO. 2B-739
REF NO 8-190-7

REVISIONS	DATE	BY	DESCRIPTION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO
ENGINEERING SERVICES BRANCH - GEOTECHNICAL OFFICE

FORKES ROAD

HIGHWAY NO. 58 DIST. NO. 4
REG. MUNICIPALITY OF NIAGARA
City of WELLAND LOTS 24,25,30&31 CON IV & V

BORE HOLE LOCATIONS & SOIL STRATA

SUBM'D L.J.H.	CHECKED	W.P. NO 78-73-01	DRAWING NO 73-11097A
DRAWN	CHECKED	W.O. NO 73-11097	
DATE Feb. 7, 1974		SITE NO.	BRIDGE DRAWING NO
APPROVED		CONT. NO.	

73-11097

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

TO: Mr. A. G. Stermac,
Principal Foundation Engineer,
West Building.

FROM: G. C. E. Burkhardt,
Structural Planning Office,
3501 Dufferin Street.

ATTENTION: K. Selby

DATE: December 3,1973.

OUR FILE REF.

IN REPLY TO

SUBJECT: Deck Replacements for Forkes Road Overpass
and C.N.R. Structure,
Hwy. 58 south of Welland,
W.P. 78-73-01 and -02, District 4.

Further to our conversation, attached is a portion of a map showing the location of the above structures and photographs taken in the vicinity.

It is proposed to replace the concrete decks on these structures during 1974.

As there appears to be a great deal of settlement at the approaches to these structures please arrange for an investigation to be carried out so that any remedial treatment may be incorporated into the design of the deck replacements.

We would appreciate receiving your report and recommendations by mid February or earlier.

W. M. Killin

WMK:lm
Attach.

W. M. Killin,
for:
G. C. E. Burkhardt,
REG. STRUCTURAL PLANNING ENG.

c.c. J. D. Barclay
J. Cullen
J. Anderson
R. Fitzgibbon

FEB. 13/

FIELD WORK START	DEC. 10, 73
FIELD WORK END	DEC 13, 73
REVIEW DATE	JAN 30, 74
MAILING	FEB 6, 74

M.D.D. FEB. 13/74

Design Services Branch,
1201 Wilson Avenue,
Downsview, Ontario.
M3M 1J8

December 12, 1973.

P.V.K. & Sons,
R.R. #4,
Brantford, Ontario.
N3T 5L7

Dear Sirs:

This letter confirms our request of December 6, 1973, for the supply of a C.M.E. 55 W.H.S.A. together with all necessary equipment, as specified under the terms of our Contract Agreement, at Welland, Ontario, on December 10, 1973.

Mobilization will be from Burford, Ont.

Our Project Numbers are W.O. 73-11097 and W.O. 73-11098.

Yours truly,

ORIGINAL SIGNED BY
A. G. STERMAC

KGS/so

A. G. Stermac,
PRINCIPAL FOUNDATIONS ENGINEER.

c.c. W. W. Fry
(Attn: Mrs. M. Porter)

Foundations Files
Documents

Copy for the information of

Mr. K. Selby

WO 73-11097
* WO 73-11098

Mr. C. S. Grebski,
Structural Design Engineer,
West Building.

G. C. E. Burkhardt,
Structural Planning Office,
3501 Dufferin Street.

April 5, 1974.

C.N.R. Overhead,
Site 34-111, W.P. 78-73-02,
and
Forks Road Underpass,
Site 34-112, W.P. 78-73-01,
Highway 58, District 4.

At this time, we had intended to provide Structural Planning Reports outlining specific requirements for the design of new decks for the above structures.

However, on receipt of the Foundation Investigation Reports for these structures it became apparent that design review of the existing structures is now necessary.

The Soils Mechanics Section has carried out a detailed study on these structures and included in their Foundation Investigation Report are the following observations, predictions and recommendations:

- Settlements of 18 inches were measured at the abutments and 14.6 and 13 inches at the south and north piers respectively of the C.N.R. Overhead.

- Settlements of 9 and 23 inches were measured at the south and north abutments respectively and 11.8 and 17.3 inches at the south and north piers respectively on Forks Road Underpass.

- Settlements of up to 7 inches may be expected on the C.N.R. Overhead within 10 years.

- Settlements of up to 4 to 5 inches may be expected on Forks Road Underpass within 10 to 15 years.

- The recommendation that the new decks should be designed such that its weight is comparable or less than that of the existing deck so that no loads in excess of those presently carried by the abutments and piers are transmitted to the subsoil.

- That they reserve the right to review and revise their predictions and recommendations in one years time, based on the results of two more sets of measured settlements taken at 6 month intervals.

In view of this information and the recommendations contained in the Foundation Investigation Report such a design review of the existing structure is necessary before we can ascertain the feasibility of improving the cross-section on the structures.

We therefore request that such a review be carried out on the existing structures and that cost estimates be prepared, based on the proposed cross-sections "A", "B", and "C" as detailed on the attached data sheet.

The following are attached for your information.

C.N.R. OVERHEAD:

Drawing 2B-700	Deck & Handrail Details
2B-701	Bent & Abutment Details
2B-714	Plan & Elevations
D-4641	Retaining Walls - Plan & Elevation
D-5518	Deck Reinforcing
D-5648	Repairs
Foundation Investigation Report	W.O. 73-11098

FORKS ROAD UNDERPASS:

Drawing 2B-739	Plan & Elevation
2B-740	Deck Details
2B-741	Bent & Abutment Details
D-4641-1	Retaining Walls - Elevation & Details
D-4641-2	Retaining Walls - Plan & Steel Table
D-5518	See C.N.R. Overhead
D-5648	See C.N.R. Overhead
Foundation Investigation Report	W.O. 73-11097

The attached prints of the structural drawing are those marked by the personnel of Engineering Surveys as a result of the precise survey of the structures.

A copy of the field notes are also attached for your information.

Structural Planning Reports will be issued following receipt of your design review and cost estimates and we, therefore, would appreciate these projects receiving prompt attention.

WMK:lm
Attach.

W. M. Killin,
STRUCTURAL PLANNING SUPERVISOR,
for:
G. C. E. Burkhardt,
REG. STRUCTURAL PLANNING ENG.

c.c. R. G. Gascoyne
G. K. Hunter
R. S. Pillar
R. Fitzgibbon
W. Lin
W. McFarlane
J. Anderson
C. R. Robertson
K. Selby

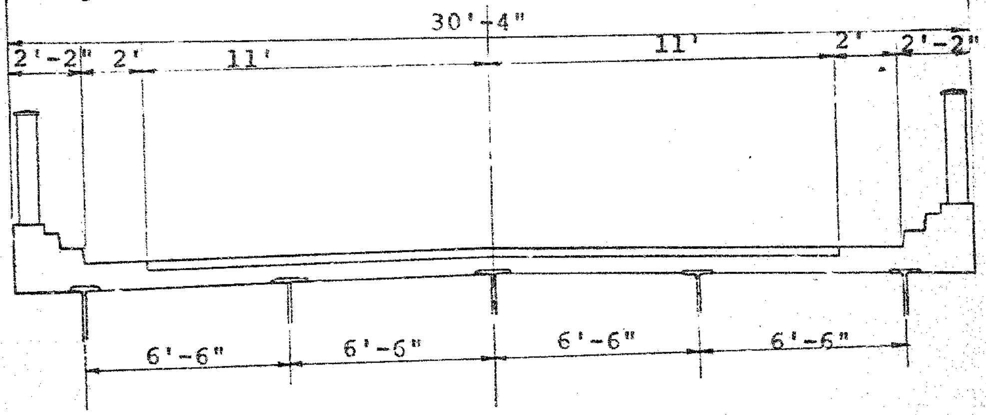
RECEIVED
GEOTECHNICAL OFFICE
APR - 9 1974
SOIL MECHANICS
MINISTRY OF TRANSPORTATION & COMMUNICATIONS

C.N.R. OVERHEAD

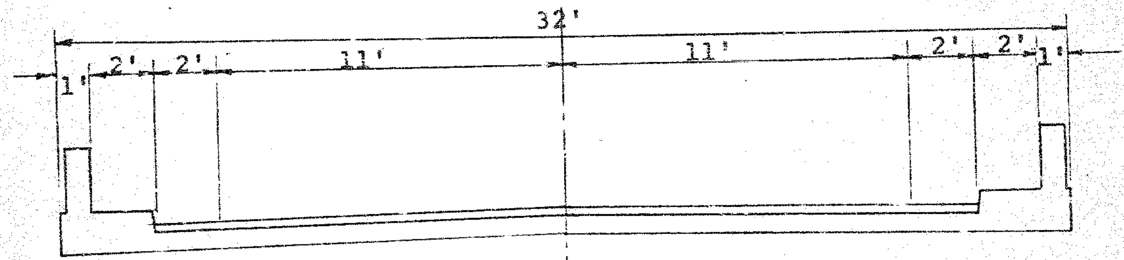
SITE 34-111,
W.P. 78-73-02.
Highway 58, District 4.

FORKS ROAD UNDERPASS

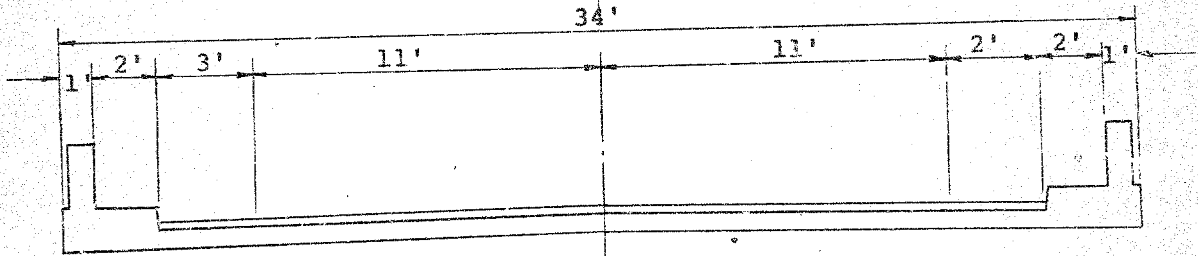
SITE 34-112
W.P. 78-73-01
Highway 58, District 4.



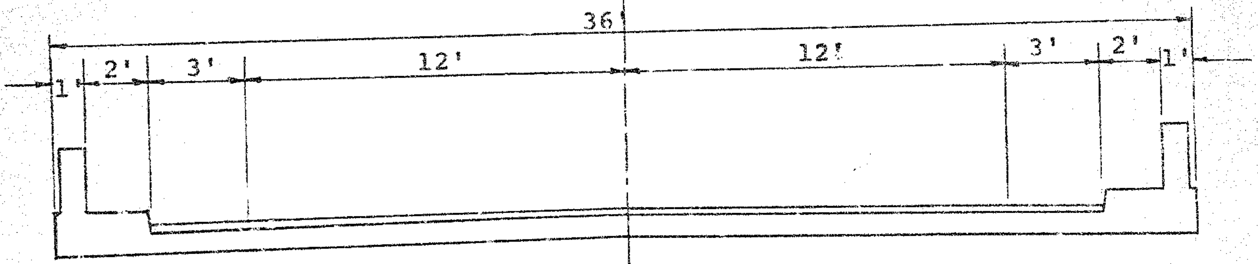
EXISTING CROSS-SECTION



PROPOSED CROSS-SECTION "A"



PROPOSED CROSS-SECTION "B"



PROPOSED CROSS-SECTION "C"

Ontario

Ministry of
Transportation and
Communications

Memorandum

To: Mr. C. Mirza,
Head, Soils Mechanics Office,
West Building.

From: Structural Office,
West Building, Downsview.

Attention:

Date: March 12, 1975.

Our File Ref.

In Reply to

Subject: Deck Replacement for Forks Road Underpass
W. P. 78-73-01, Site 34-112,
Highway 58, District 4.

Attached herewith we are submitting the final bridge drawings which show the foundation design for this structure.

Kindly give us your comments at your earliest convenience.

C. S. Grebski,
Structural Design Engineer.

CSG/cf
Attch.

finalized
MARCH 24/75

RECEIVED
GEOTECHNICAL OFFICE
MAR 1 8 1975
SOIL MECHANICS
MINISTRY OF TRANSPORTATION & COMMUNICATIONS

Mar 18

73-1109

Mr. C. Grabski
Structural Design Engineer
Structural Design Office
West Building, Downsview

Soil Mechanics Section
Geotechnical Office
West Building, Downsview

April 14, 1975

W.P. 78-73-01

DECK REPLACEMENT FOR FORKS ROAD UNDERPASS

W.P. 78-73-01, Site 34-112
Highway 58, District 4

We have reviewed the final design drawings (34-112, sheet 1) of the above structure.

According to our Foundation Report (W.P. 78-73-01), the following fill treatments are required:

(a) the clay surface of the fill beneath the road bed, for a section within 30 - 40 ft. of the abutment, should be reshaped and cambered, and

(b) the granular fill subgrade should be replaced and should extend over the full top width of the embankments.

We assume that these recommendations will be taken care of by Systems Design, and will be incorporated in the grading drawings.

B. Ly

B. LY
Project Engineer

for: K. G. SELBY
Supervising Engineer

c.c. J. Cullen
W. Killin
D. Gunter

Files
Record Services

Ministry of Transportation and Communications
Ontario

LJS

Memorandum

To: Mr. M. R. Ernesaks,
Regional Manager,
Regional Planning and Design,
Central Region, 3501 Dufferin St.

Attention:

From: Structural Office,
West Building, Downsview.

Date: July 9, 1976.

Our File Ref.

In Reply to

Subject: Deck Replacement,
Forks Road Underpass,
W. P. 78-73-01, Site 34-112,
Highway 58, District 4.

As this project has been stockpiled for some time, we wish to submit an updated D4 and Special Provisions for the Structure which will supersede all the contract documents previously sent to you.

Enclosed is two copies of the D4 and Special Provisions for your use.

One copy of the D4 and Special Provisions is also being forwarded to the following:

District
Systems Design Project Review
Structural Material Section
Structural Design
Estimating Office
Assistant Construction Engineer (Structures)
Regional Structural Planning Engineer.

M Stoyanoff

MS/cf
Attch.

M. Stoyanoff,
Structural Contract Engineer.

c.c. J. Wear
C. Robertson
K. Howe
B. Giroux
A. McKim
G. Burkhardt
E. Van Beilen
✓C. Mirza
R. Fitzgibbon
J. Anderson

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GEOTECHNICAL OFFICE
JUL 12 1976
SOIL MECHANICS
MINISTRY OF TRANSPORTATION & COMMUNICATIONS

KGS

Ministry of Transportation and Communications

Ontario

1-48

Memorandum

To: Mr. E. Willis,
Supervisor Contract Documentation,
Systems Design Branch,
East Building, Downsview.

From: Structural Office,
West Building, Downsview.

Attention:

Date: November 18, 1976.

Our File Ref.

In Reply to

Subject: W.P. 78-73-01, Site 34-112,
Deck Replacement for Fork Road Underpass,
Highway 58, District 4.

As a result of the Structural Review Committee meeting, dated October 27, 1976 the contract documents should be changed for the above as follows:

(1) Earth excavation required for placing approach slabs and to carry out work on the structure should be completed by Regional Planning and Design Office.

(2) Special SP for Field Clean and Paint Structural Steel should be revised to read

"At the contract price for the above tender item the Contractor shall clean and paint the structural steel, including new shoe and bed plates, in accordance with MTC Form 912 part C and as described herein. The Contractor may apply the prime coat prior to placing the new deck. The top coat shall only be applied after the new deck has been placed. Before the application of top coat paint further cleaning and touch-up prime shall be carried out where necessary."

(3) Add to the list of Material supplied by MTC:

Paints
Prime MTC Code No. 16-5-3-1 (6 gal.)
Top Coat Grey MTC Code No. 16-2-25-2 (13 gal.)

(4) Delete Protection Board from the list of Material supplied by MTC.

N. Zoltay

NZ/im

N. Zoltay,
Structural Contract
Specifications Engineer.

c.c. W. McFarlane
M. R. Ernesaks
C. R. Robertson
J. Kuprevicius
B. Giroux
A. E. McKim
C. Farrell
E. Van Beilen
C. Mirza ✓

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MINISTRY OF TRANSPORTATION & COMMUNICATIONS
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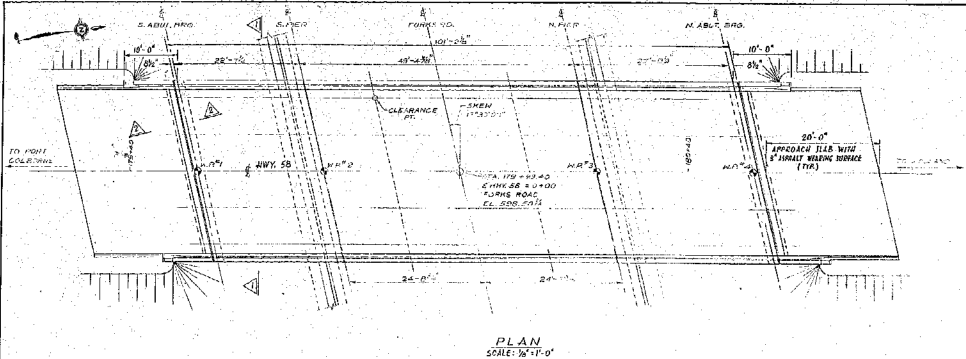
DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 30L14-23
DIST. 4 REGION CENTRAL
W.P. No. 78-73-01
CONT. No. 76-40
W.O. No. 73-11097
STR. SITE No.
HWY. No. 58
LOCATION EMBANKMENT SETTLEMENTS
AT HWY 58 & FORKES ROAD

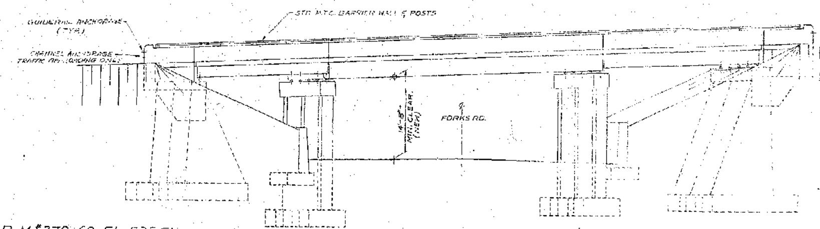
OVERSIZE DRAWINGS TO SE INCLUDED WITH THIS REPORT 3

REMARKS: documents to be unfolded before microfilmed

GJ-30 SEPT. 1976

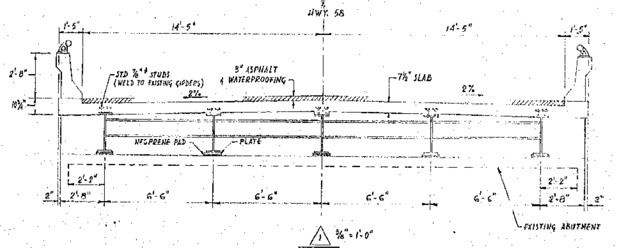


PLAN
SCALE: 1/8" = 1'-0"

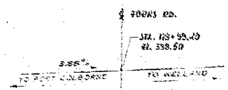


ELEVATION
SCALE: 1/8" = 1'-0"

B.M. #379-69 EL. 535.81
ONE STOREY RED BRICK HOUSE ON THE SOUTH-WEST CORNER OF FIRST AVE. AND THIRD AVE. IN THE CITY OF PORT COLBORNE 307 FT. SOUTH OF CENTERLINE OF HWY. 3. TABLET IS SET HORIZONTALLY IN NORTH FACE OF CONCRETE FOUNDATION BEING 8 FT. WEST OF NORTH-EAST CORNER AND 2.4 FT. BELOW BRICKWORK.



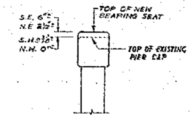
1 — 3/16" = 1'-0"



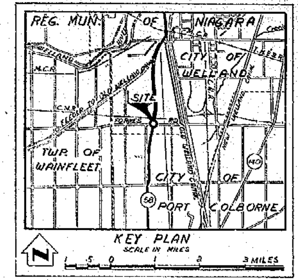
PROFILE OF HWY. 58
N.T.S.

SKEW DATA - 13° 30' 00"

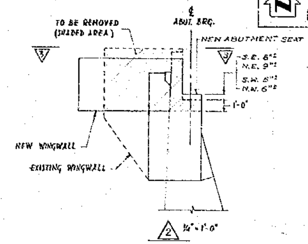
SIN	0.2334454
COS	0.9723699
TAN	0.2400788
SEC	1.0284152



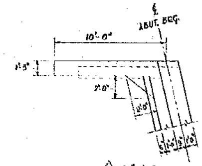
SECTION THROUGH PIER CAP
SCALE: 3/4" = 1'-0"



KEY PLAN
SCALE IN MILES



2 — 1/4" = 1'-0"



3 — 1/4" = 1'-0"

NOTES:

CLASS OF CONCRETE

APPROACH SLABS — 3000 PSI
REMAINDER — 4000 PSI

CLEAR COVER ON REINFORCING STEEL

WINGWALLS 3", BARRIER WALLS 1½"
DECK 2" TOP, 1" BOTT.

CONSTRUCTION NOTES

THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF ± 1/8".
NO CONCRETE SHALL BE PLACED ABOVE THE ABUTMENT BEARING SEATS, UNTIL THE CONCRETE IN THE DECK HAS BEEN PLACED.

CONCRETE QUANTITIES

CONCRETE IN PIERS, ABUTMENTS AND WINGWALLS — 30 C.Y.
CONCRETE IN DECK — 82 C.Y.
CONCRETE IN BARRIER WALLS — 21 C.Y.
CONCRETE IN APPROACH SLABS — 37 C.Y.

30L14-23
GEOCRES No.

LIST OF DRAWINGS

34-112-1 GENERAL PLAN
-2 BORE HOLE LOCATIONS & SOIL STRATA
-3 ABUTMENTS & PIERS - DIMENSIONS & REINF.
-4 DECK & GIRDER DETAILS
-5 CONCRETE BARRIER WALL (2'-8" HIGH)
-6 STEEL PARAPET RAILING (SINGLE TUBE)
-7 20 FT. APPROACH SLAB (FOR BARRIER WALL)
-8 STANDARD DETAILS
-9 AS CONSTRUCTED ELEV. & DIM.

PRINT RECORD

No.	FOR	DATE

REVISIONS

DATE	BY	DESCRIPTION

REGISTERED PROFESSIONAL ENGINEER — CIVIL — JOSEPH SCHOEG — PROVINCE OF ONTARIO

FOR REDUCED PLAN
USE SCALE BELOW
0 1 2 3
3 INCHES ON ORIGINAL PLAN

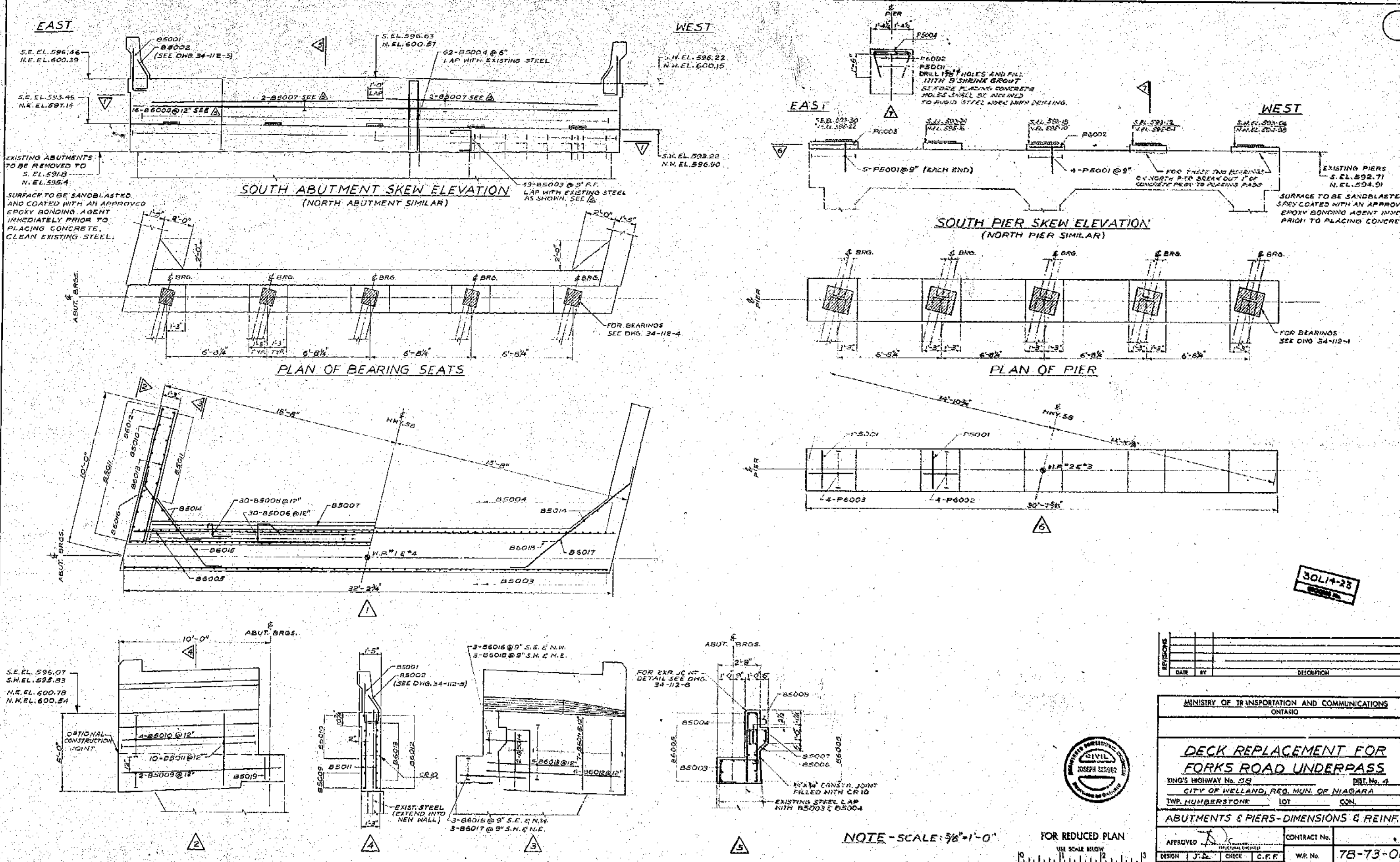
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS
ONTARIO

DECK REPLACEMENT FOR FORKS ROAD UNDERPASS

KING'S HIGHWAY No. 58 — DIST. No. 4
CITY OF WELLAND, REG. MUN. OF NIAGARA
TWP. HUMBERSTONE — LOT — CON.

GENERAL PLAN

APPROVED	STRUCTURAL ENGINEER			CONTRACT No.	
DESIGN	J.S.	CHECK	C.F.F.	W.P. No.	78-73-01
DRAWING	P.K.	CHECK	J.S.	SITE No.	34-112 SHEET 1
DATE	NOV. 1974	LOADING	HS20-44		



REVISIONS		
DATE	BY	DESCRIPTION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS
ONTARIO

DECK REPLACEMENT FOR FORKS ROAD UNDERPASS

KING'S HIGHWAY No. 58 DIST. No. 4
CITY OF WELLAND, REG. MUN. OF NIAGARA
TWP. HUMBERSTONE LOT CON.

ABUTMENTS & PIERS - DIMENSIONS & REINF.

APPROVED	STRUCTURAL ENGINEER			CONTRACT No.	
DESIGN	J.Sz.	CHECK	C.F.F.	W.P. No.	78-73-01
DRAWING	A.N.	CHECK	J.S.	SITE No. 34-112	SHEET 3
DATE	JAN. 75	LOADING	HS20-44		