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MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

30L14-24
GEOCRES No.

MEMORANDUM

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

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

ATTENTION:

DATE: March 4th, 1974.

OUR FILE REF.

IN REPLY TO

MAR 11 1974

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For
Embankment Settlements at Hwy. #58 &
C.N.R. Overpass

District #4, Hamilton

W.P. 78-73-02 W.O. 73-11098

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
Attach.

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

Files
Documents

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FOUNDATION INVESTIGATION REPORT

For

Embankment Settlements at Hwy. #58 and C.N.R. O'Pass

District #4, Hamilton

W.P. 78-73-02 W.O. 73-11098

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1. INTRODUCTION:

It has been proposed to replace the concrete deck at the existing C.N.R. overpass on Hwy. #58, south of Welland some time 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 3rd, 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 the C.N.R. line near the southern boundary of the City of Welland, Regional Municipality of Niagara. The structure at the site carries Hwy. 58 over the C.N.R. tracks in a north-south direction. The structure consists of three (29'5½" x 38'3-5/8" x 29'5½") 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.

Physiographically the site lies within the region termed the Haldiman 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 overrode and reworked an underlying deposit of layered or varved clay. Hence, much of the 'till' is essentially composed 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 predominantly of the same material as the layered (varved) zones lying above and below, but of a mottled appearance.

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

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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 exceptions, 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.

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, bulk 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 1-4 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. 73-11098A, 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.

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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 three zones by physical appearance and soil property variations.

The cohesive deposit is probably underlain by fluvial fine sand or glacial till followed 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 north embankment and Borehole #2 through the south 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 29 ft. (8.8 m) of fill was intersected at Borehole #1 (north approach) and 28 ft. (8.5 m) at Borehole #2 (south approach).

4.3) Silty Clay to Clay (Zone 1):

Immediately beneath the approach fills at each boring location a 5-8 inch (0.127-0.203 m) layer of black organic topsoil was discovered and this material was underlain by a 14-15 ft. (4.3-4.6 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 being firm to very stiff. Shear strength was with one exception, greater than 2000 PSF (95.8 kN/m^2) everywhere.

The layered and mottled material had liquid limits ranging from 58-69%, plastic limits ranging from 26-31% and natural moisture content ranging from 25-40%. 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 121 and 128 P.C.F. (1.9-2.1 T/m^3) with the average being 124 P.C.F. (2.0 T/m^3).

4.4) <u>Silty Clay to Clay (Zone 2):</u>

This zone might best be described as a glacial till. The layer varied between 6-10 ft. (1.8-3.0 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 field testing ranged between 640 and 720 PSF (30.6-34.5 kN/m^2).

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

Liquid Limit	%	46
Plastic Limit	%	24
Natural Moisture Content	%	24-34

The bulk density of this material was determined from laboratory tests, to range from 112-118 P.C.F. (1.8-1.9 T/m^3) with an average value of 115 P.C.F. (1.8 T/m^3).

4.5) Silty Clay to Clay:(Zone 3):

This zone is composed of basically the same materials as the above zones. The minimum thickness of the zone varied between 28.5-42.5 ft. (8.7-13.0 m).

Zone 3 is generally a layered material, red, brown and grey in colour. 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. Both Boreholes terminated within this layer at elevations 523.6 and 514.3 ft. (159.6 and 156.8 m).

The consistency of Zone 3 may be described as being firm to stiff as indicated by undrained shear strength values of 640-1120 PSF (30.6-53.6 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	% 46-47
Plastic Limit	% 21-24
Natural Moisture Content	% 22-34

The bulk density of Zone 3 material varies from 114-128 PCF (1.8-2.1 T/m^3).

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 and Hwy. 58, some 800 ft (244 m) south of the 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 30th, 1973.

Pre-channel relocation groundwater levels at the C.N.R. line and Hwy. 58 (i.e. 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 5 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 vane in the field. In addition laboratory unconfined compression tests were performed on selected samples. The results of the U.C. tests indicated strength values of approximately 50-100% of those obtained at comparable elevations within the subsoil using the field vane. It is well known that with imcreasing 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 tends to confirm the validity of strength values as determined from the field vane tests.

Within the fill, the field vane tests gave undrained shear strength values in excess of 2000 P.S.F. (95.8 kN/m^3). From the original ground level elev. 580$\pm$ ft. (176.8 m) to elevation 560$\pm$ ft. (170.7 m), the undrained strength based on field vane and laboratory U.C. tests varied between 1120 and greater than 2000 P.S.F. (53.6 - 95.8$^+$ kN/m^2). Below elevation 560$\pm$ ft. (170.7 m) the undrained shear strength based on field vane test varied between 630 and 1120 P.S.F. (30.2-53.6 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	>2000 psf (>95.8 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)	2800 psf (134.0 kN/m^2)	1120->2000 psf (53.6->95.8 kN/m^2)	2000 psf (95.8 kN/m^2)
560 - 498 ft. (170.7-151.8 m)	510 psf (24.4 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 <u>Pore Pressure Measurement:</u>

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

The effective stress parameters are quite consistent throughout the cohesive deposit with C'= 360 psf and Ø'=21° above elevation 565$\pm$ ft. (172.2$\pm$ m) and C'=0 psf, Ø'= 22° below elevation 565$\pm$ ft. (172.2$\pm$ m). For effective stress stability analysis, values of $C' = 0$ and $\phi' = 22°$ have been used for the subsoil below elevation 565 ft. (172.2 m) and $C' = 150$ psf (4.79 kN/m^2) and $\phi' = 23°$ for the cohesive approach fills and original ground material above elevation 565 ft. (172.2 m) (i.e. 'crust zone').

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 Shmertmann 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 dessication. Within this dessicated 'crust' very little if any consolidation settlement can be expected.

7. DISCUSSION AND RECOMMENDATIONS:

7.1) General:

It is proposed to replace the concrete deck on the structure carrying Hwy. 58 over the C.N.R. Line in the City of Welland, Regional Municipality of Niagara. The deck replacement has become necessary due to the severe deterioration of concrete in the decks - i.e. 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 accommodate future settlements, into the redesign of the decks.

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 cone. 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 probably by a fine sand or glacial till 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 well rip-rapped and paved. No cracking or tilting of the retaining walls was noted suggesting the forward slopes to be stable in-situ.

Based on total and effective stress parameters discussed previously, both 'total' and 'effective' stress 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.

7.3) Settlements:

During the course of the field investigation the structure and approach fills were examined visually for telltale 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

the abutments has been required many times since the structure was built in 1955, confirming settlements within the fills themselves. 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 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, area settlements of varying amounts, depending of 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) Settlement of the subsoil under the fills and structures has occurred due to drawdown of the groundwater and due to imposed loading.

(2) Settlements within the fills have also occurred 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 18 inches (0.457 m) were measured at the abutments and 14.6 and 13.0 inches (0.371 and 0.330 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 from 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 occurred 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 occurred. From the foregoing considerations we have arrived at a predicted value of settlement still to take place of the order of 6-7.5 inches (0.152 - 0.191 m) over the next 10-15 years. Refer to Table I for a summary of predicted and measured settlements.

It will be noted that there is poor agreement between predicted and measured settlements at the pier locations, however, predicted and measured settlements at the abutments show good agreement.

Based on the measured and predicted settlements reflected in Table I, several different interpretations concerning completed and future settlements can be drawn. As noted, it is reasonable to assume that consolidation settlements due to imposed fill and structure loading are 90% or more complete at this point in time, and future settlements will be the result of drawdown consolidation only. In arriving at the predicted settlement values shown in Table I, similar consolidation characteristics, subsoil and drainage paths were assumed to exist beneath fills and piers. If in-situ subsoil conditions and drainage paths are considerably different from those assumed, the predicted values could be highly suspect. In fact, no borings were done near the pier locations and subsoil and consolidation characteristics beneath them could be very different than those at the abutment locations. If so, the predicted settlements at the piers may be too low or drawdown time-settlement predictions too long. In either case, the measured pier settlements may be correct, and could even be essentially complete at this point in time.

If the predicted and measured values are accepted as correct, then future settlements of 6-7.5 inches (0.152-0.191 m) with differential settlements of 1-1½ inches (0.025-0.038 m) between piers and abutments may still be expected to occur within the next 10 years. If, on the other hand, predicted and measured settlements at the abutments are assumed correct, and if more than 90% of the drawdown consolidation has already occurred beneath the piers, then slightly more than 7 inches (0.178 m) of settlement can still be expected at the abutments in the next 10 years with little or no future settlement at the piers. In this latter case, differential settlements between piers and abutments of up to 7 inches (0.178 m) can be expected.

Other interpretations may also be made, and the only acceptable solution to this problem is for this Office to acquire more information on the time-settlement characteristics at the site. Hence, it is essential that at least two more sets of measured elevations be taken at pier and abutment locations at 6-month intervals.

For the present time, 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 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 accommodate up to 1-½ inches (0.038 m) of differential settlement that could 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 roadbed should be reshaped 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 further settlements within the fills themselves.

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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 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 Engineer.

L.J. Hodge

L.J. Hodge
Project Engineer.

K.G. Selby

K.G. Selby,
Supervising Engineer.

_ I/mj
March, 1974.

APPENDIX I

TABLE 1: SUMMARY OF PREDICTED AND MEASURED SETTLEMENTS AT HWY. 58 - C.N.R. O'PASS

Breakdown of Consolidation Settlements	North Abutment 30' (9.14 m) Ht.		North Pier		South Pier		South Abutment 30' (9.14 m) Ht.	
	Predicted	Actual	Predicted	Actual	Predicted	Actual	Predicted	Actual
Total S: Imposed Loading and Drawdown	24.1" (0.61 m)	18" (0.46 m)	14" (0.36 m)	13" (0.33 m)	14.0" (0.36 m)	14.6" (0.37 m)	24.1" (0.61 m)	18" (0.46 m)
Total S: Drawdown Only	12.3" (0.31 m)		10.4" (0.26 m)		10.4" (0.26 m)		12.3" (0.31 m)	
Est. Drawdown Settlement to start of 1974	5.0" (0.13 m)		4.0" (0.10 m)		4.0" (0.10 m)		5.0" (0.13 m)	
Predicted Settlement still to occur	7.3" (0.19 m)		6.4" (0.16 m)		6.4" (0.16 m)		7.3" (0.19 m)	

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

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

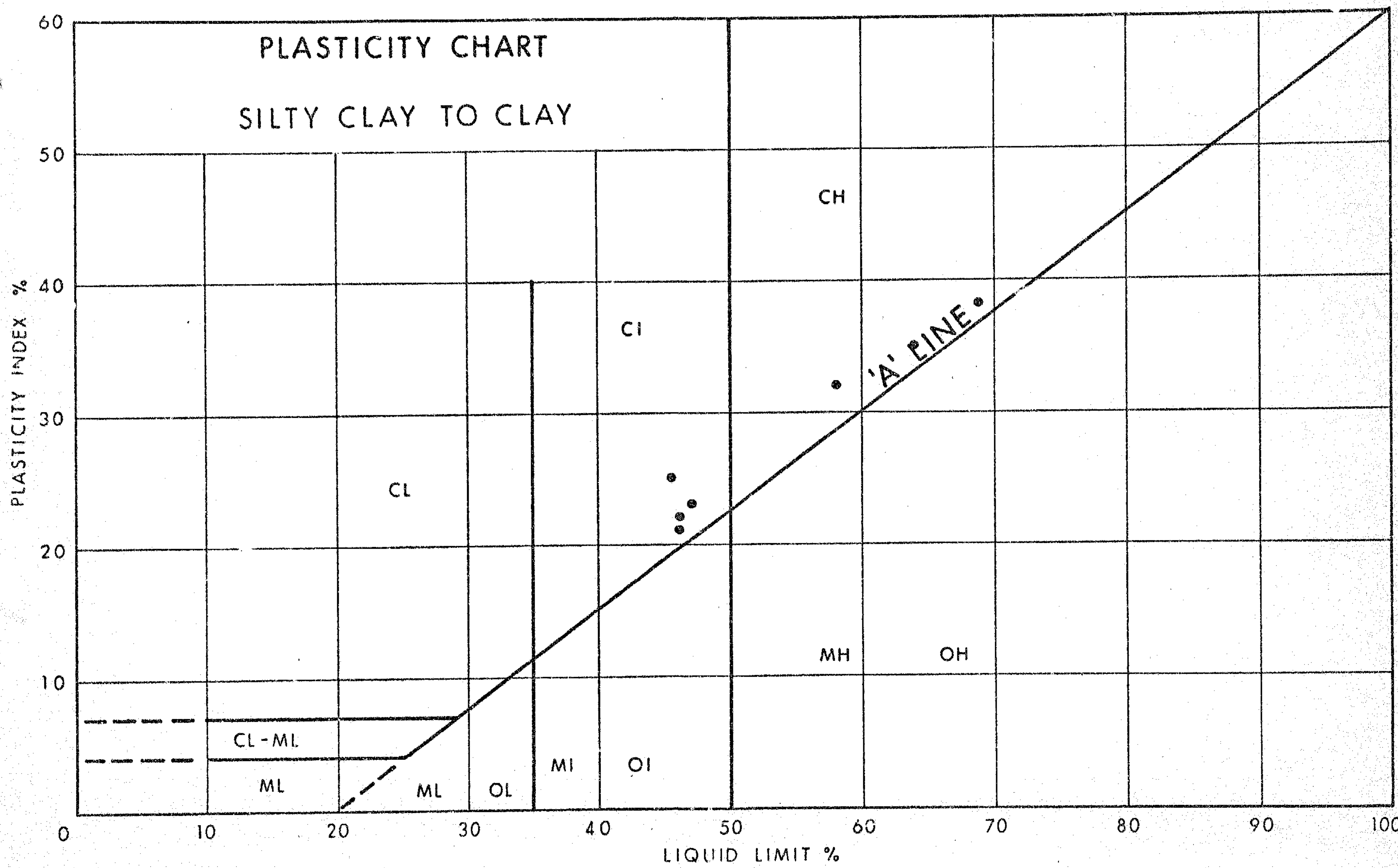


FIG. 1

W.O. 73-11098

VOID RATIO–PRESSURE CURVES

JOB NO. 73–11098

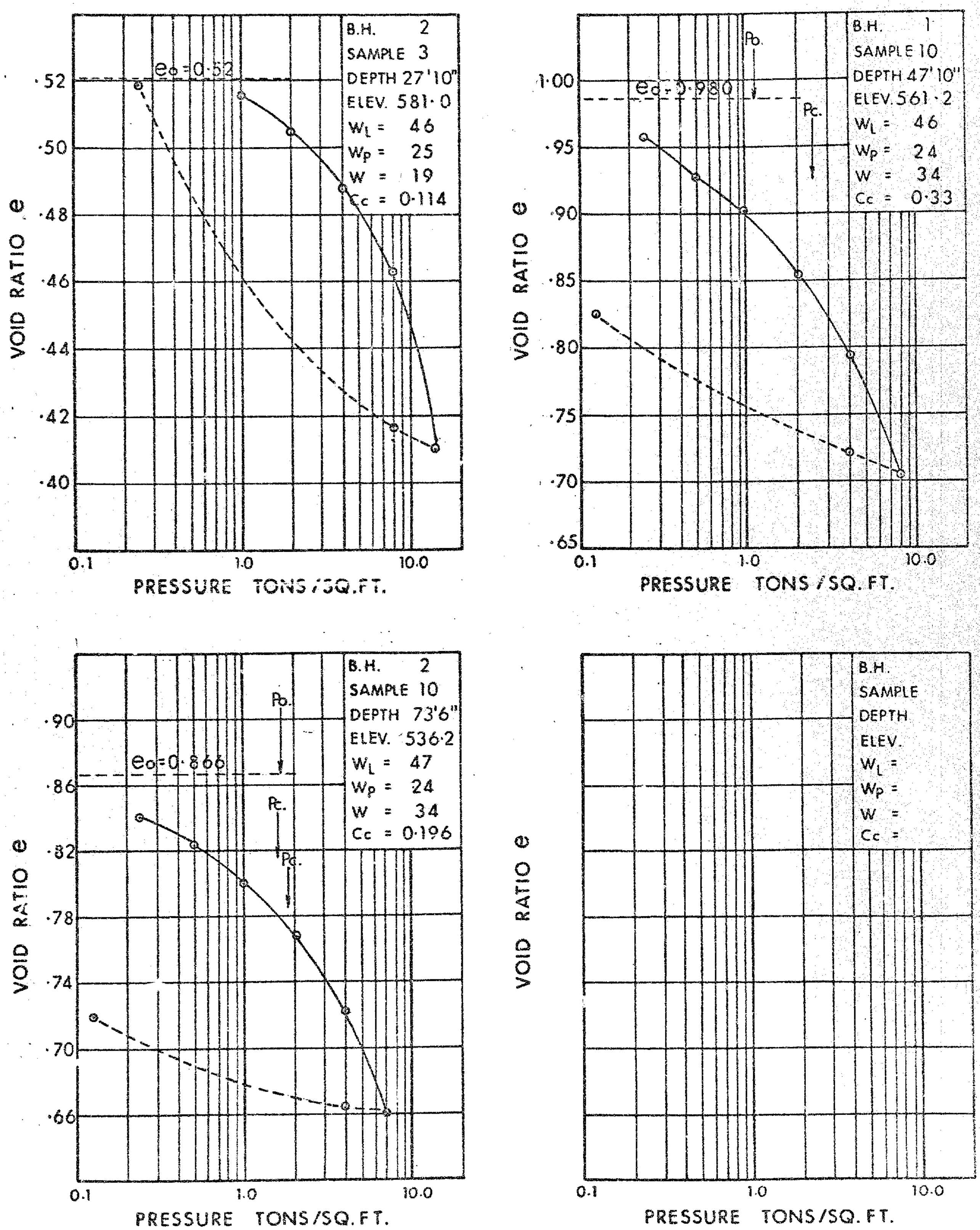
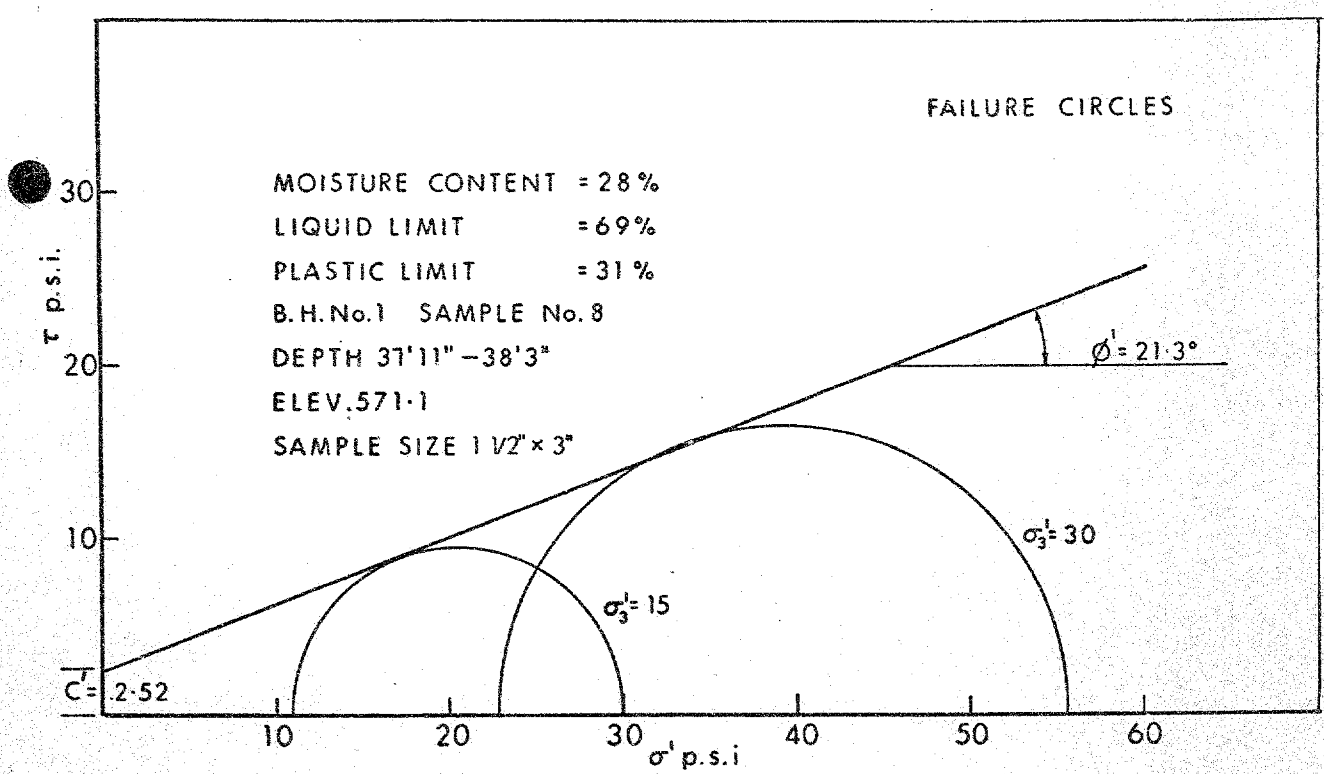


FIG. 2



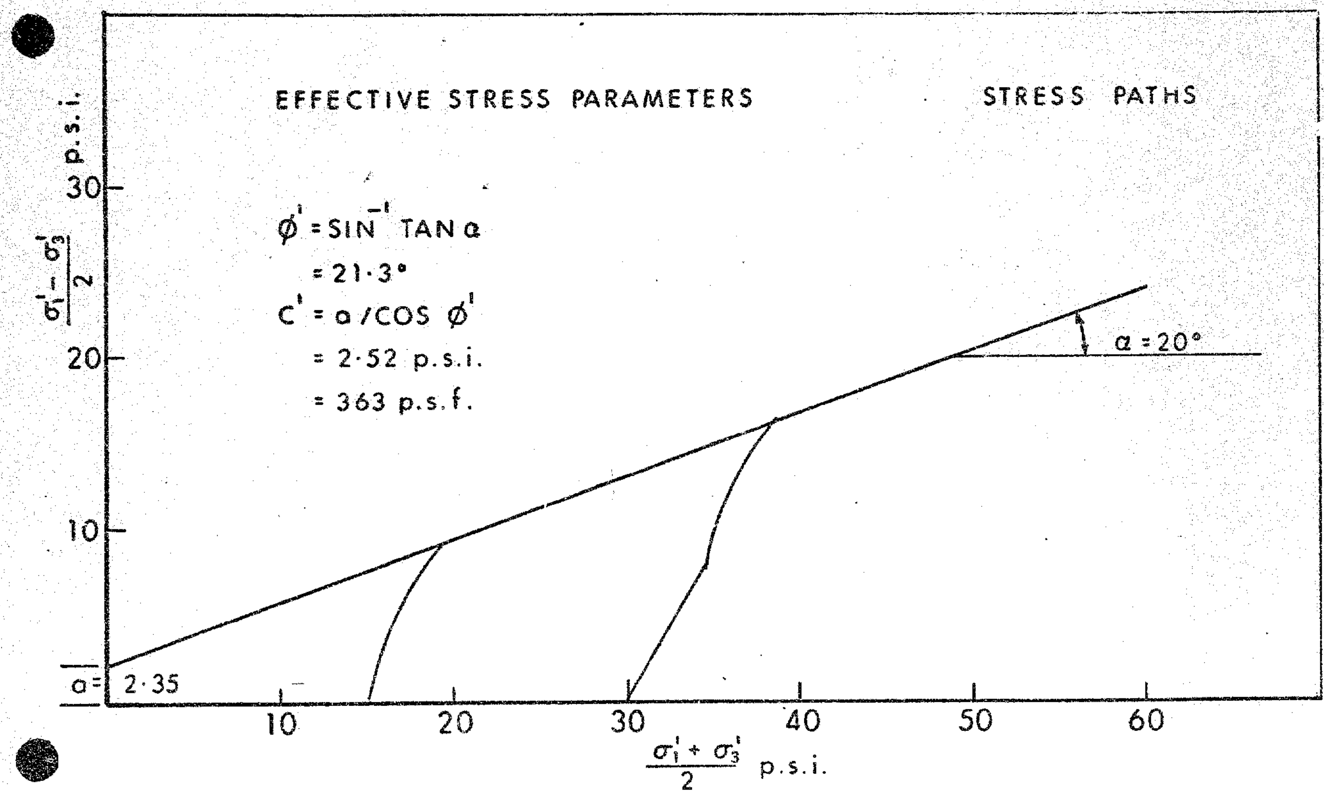
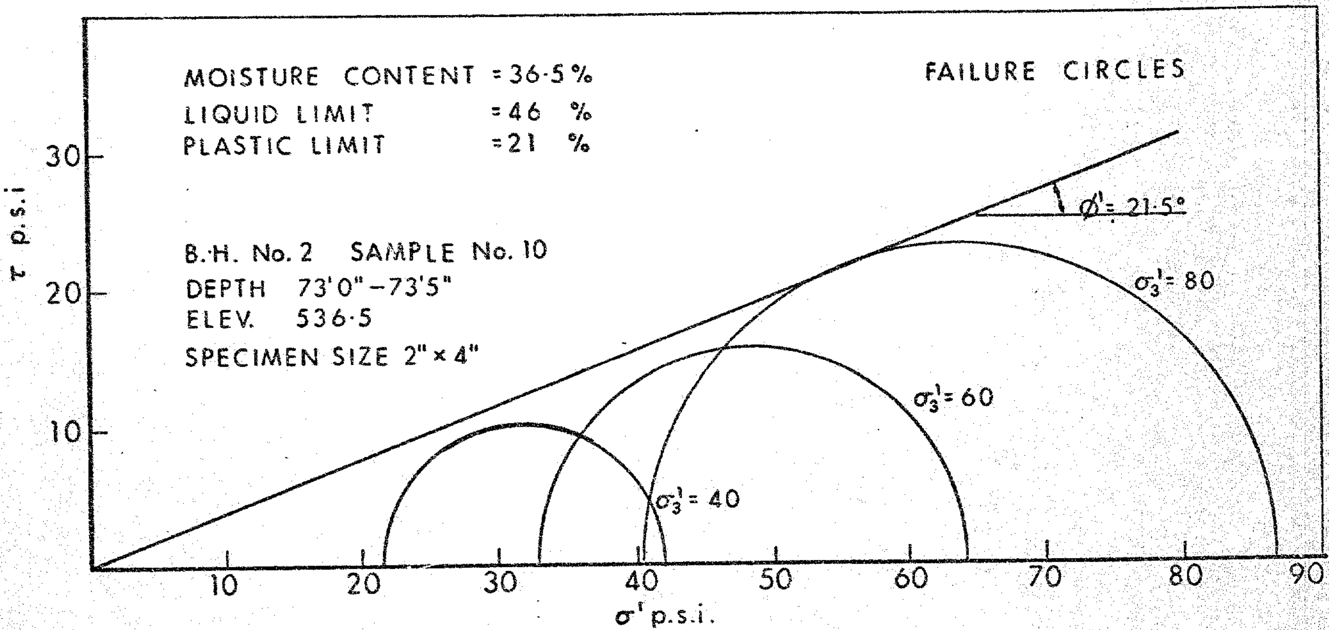
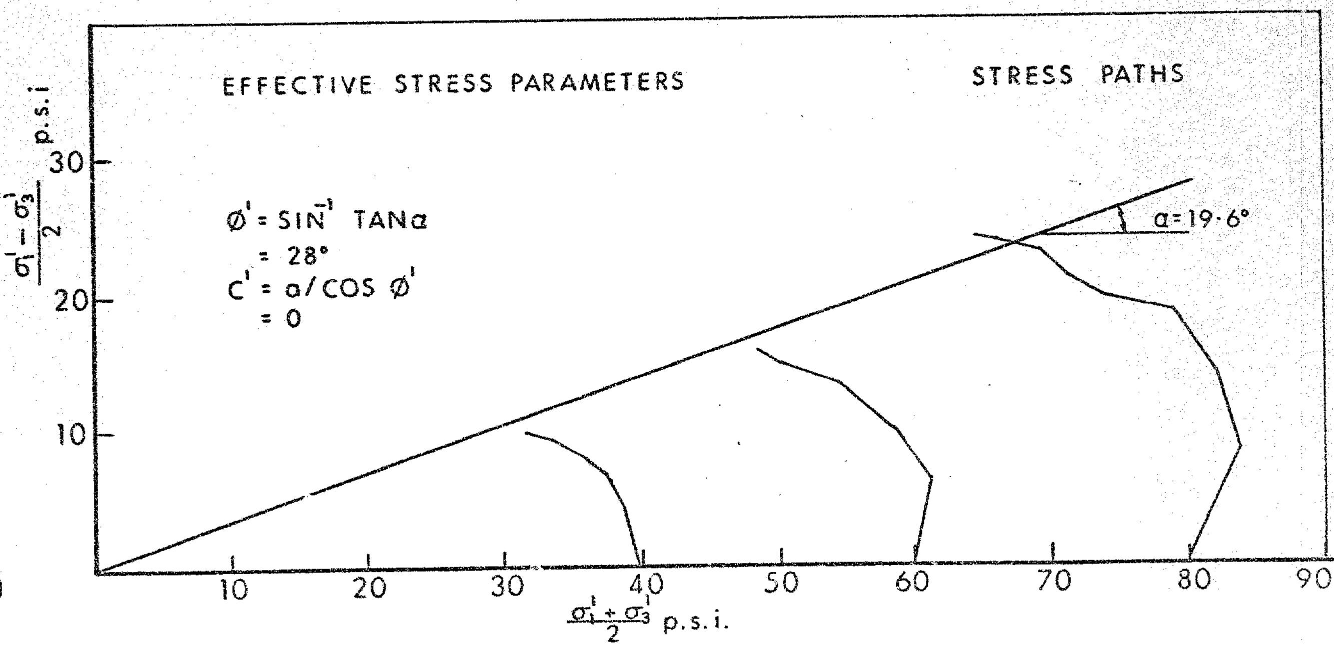


FIG. 3

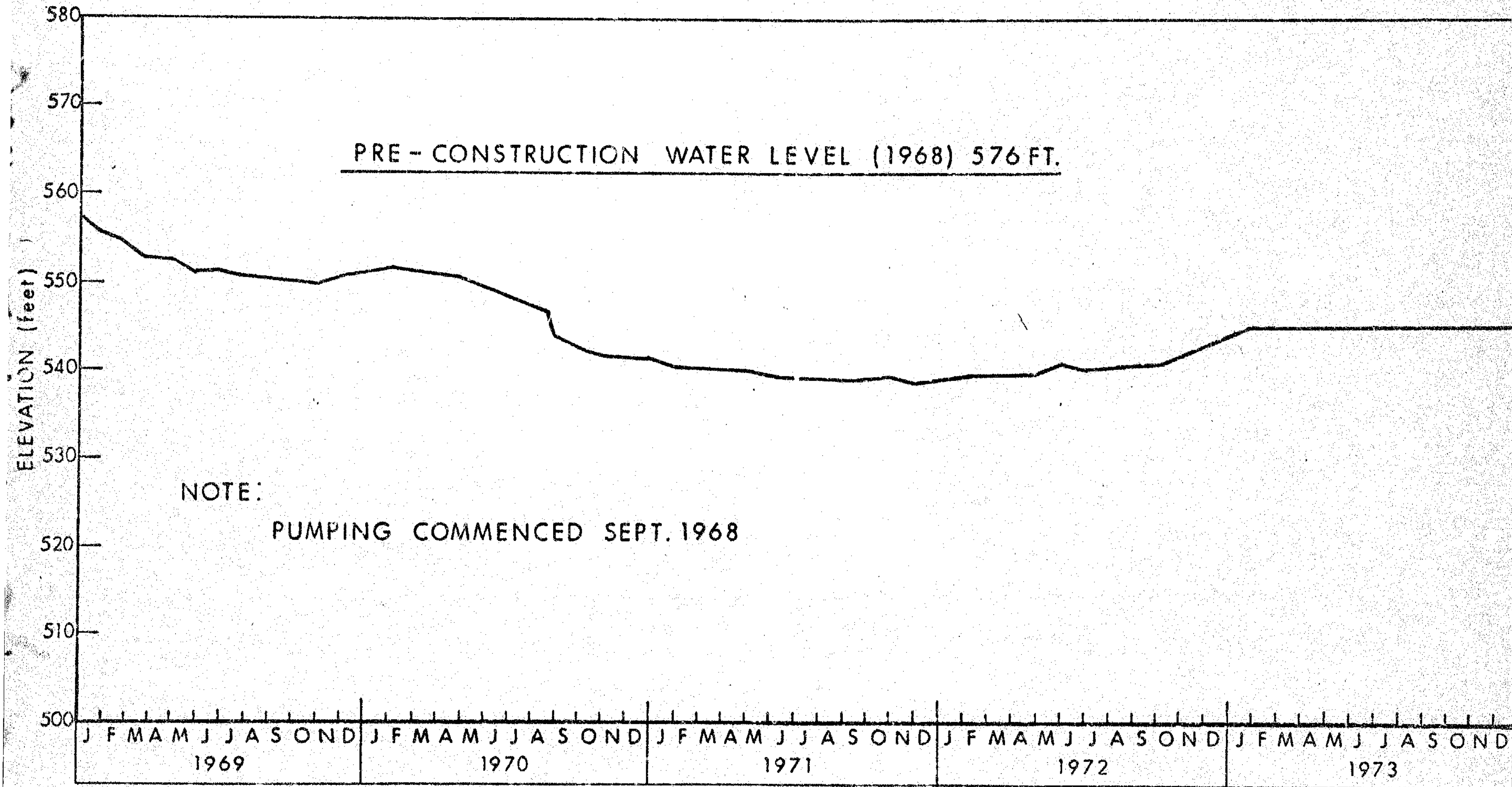
W.O.73-11098





W.O.73-11098

FIG. 4



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

FIG. 5

W.O. 73-11098

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS - ONTARIO

DESIGN SERVICES BRANCH — FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 1

JOB 73-11098
W.P. 78-73-02
DATUM Geodetic

LOCATION Hwy. 58, Sta. 187 + 26, o/s: 20' LT &
BORING DATE December 12, 13, 1973
BOREHOLE TYPE Hollow Stem Auger Bombardier Mounted

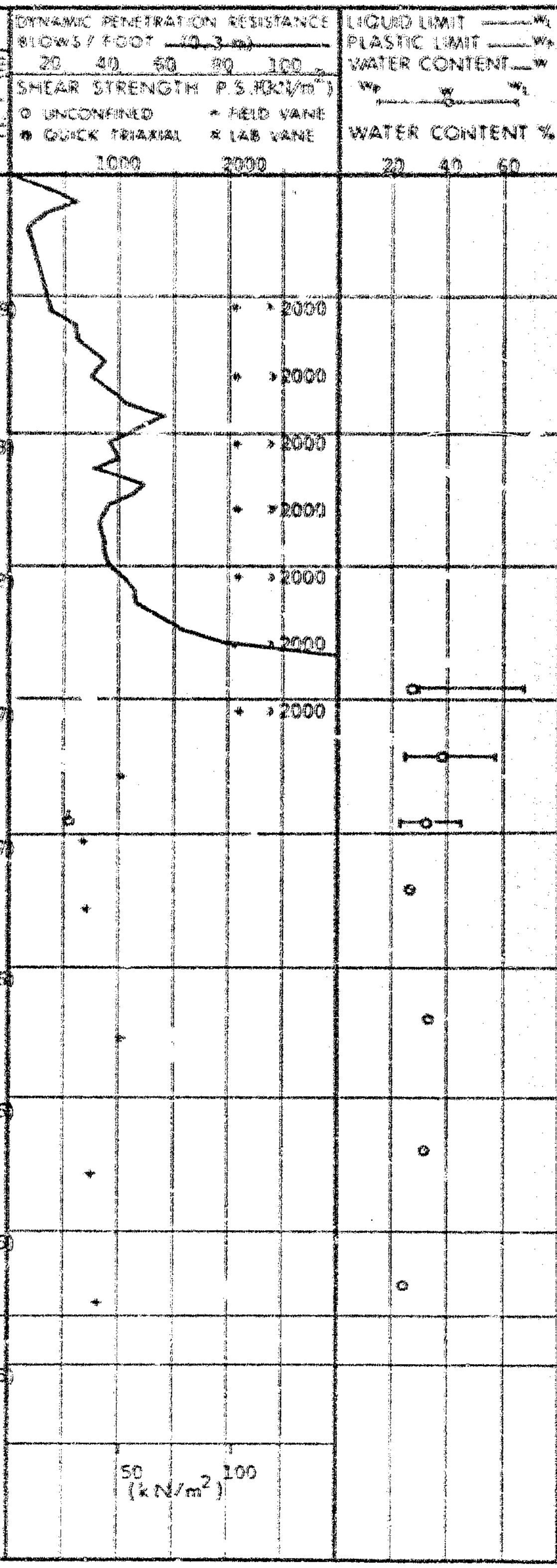
ORIGINATED BY LJH
COMPILED BY LJH
CHECKED BY

ELEV. m / DEPTH m	ELEV. ft / DEPTH ft.	SOIL PROFILE DESCRIPTION
185.6	609.1	Ground Level
0.0	0.0	
184.4 / 1.2	605.1 / 4.0	Gravel FILL
		Silty Clay to clay some sand and traces of gravel Brown to red-brown Very Stiff to Hard.
175.6 / 10.1	576.1 / 33.0	(Zone 1) Silty clay to clay numerous silt seams/pockets, occ. gravel, pockets sandy Grey-brown to mottled layered red, brown gr. Firm to V. Stiff
171.3 / (14.3)	562.1 / 47.0	(Zone 2) Silty clay, occ. silt seams/pockets Grey Firm.
168.3 / 17.4	552.1 / 57.0	(Zone 3) Silty clay to clay occasional silt pockets, occ. gravel Red-brown. Firm to Stiff mottled red, brown, grey
159.6 / 26.1	523.6 / 85.5	End of Borehole

SAMPLES

NUMBER	TYPE	BLOWS/FOOT (0.3 m)	BULK DENSITY γ P.C.F. (T/m³)
1	SS	11	
2	TW	PH	
3	TW	PH	
4	TW	PH	
5	TW	PH	
6	TW	PH	
7	TW	PH	
8	TW	PH	128 (2.1)
9	TW	PH	
10	TW	PH	112 (1.8)
11	TW	PH	118 (1.9)
12	TW	PH	117 (1.9)
13	TW	PH	120 (1.9)
14	TW	PH	128 (2.1)

ELEV. SCALE ft. / (m): 600 (182.9), 590 (179.8), 580 (176.8), 570 (173.7), 560 (170.7), 550 (167.6), 540 (164.6), 530 (161.5), 520 (158.5)



REMARKS: GR SA SI CL

w.l. el.545 est. 30/12/7

From info. supplied by St. Lawrence Seaway Authority

20 / 15 ◇ 5 / 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH — FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 2

JOB 73-11098 — LOCATION Hwy. 58, Sta. 186 + 04, o/s: 20' RT ℄ — ORIGINATED BY J.H.

W.P. 73-73-02 — BORING DATE December 18, 1973 — COMPILED BY J.H.

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

m	ELEV DEPTH ft.	DESCRIPTION	NUMBER	TYPE	BLOWS/FOOT (0.3 m)	ELEV. SCALE ft. (m)	SHEAR STRENGTH P.S.F.	WATER CONTENT %	BULK DENSITY γ P.C.F. (T/m³)	REMARKS
185	609.8	Ground Level								
0.0	0.0									
184.6 / 1.2	605.8 / 4.0	Gravel								
		FILL Silty clay to clay, some sand and traces of gravel Brown Very Stiff to Hard.				600 (182.9)				
			1	TW	PH		2000			
						590 (179.8)				
			2	TW	PH		2000			
			3	TW	PH	580 (176.8)	2000		133 (2.1)	
176.1 / 9.8	577.8 / 32.0	Silty clay to clay numerous silt/sand pockets and seams, occ. gravel. Brown, grey-brown to grey some mottling some irregular layering Very Stiff (Zone 1)	4	TW	PH		2000			
			5	TW	PH	570 (173.7)	2000		123 (2.0)	
			6	TW	PH		2000		121 (1.9)	
171.5 / 14.3	562.8 / 47.0	Silty clay occ. silt layers, tr. sand and gravel, grey Stiff. (Zone 2)	7	TW	PH	560 (170.7)				
169.7 / 16.2	556.8 / 53.0	Layered, red, brown, grey	8	TW	PH				115 (1.8)	
		Silty clay to clay occ. silt/sand seams and pockets. occ. gravel. Red brown Firm. (Zone 3)				550 (167.6)				
			9	TW	PH				117 (1.9)	
		Some mottling.				540 (164.6)				W.L. el.545 est.30/12/73 from info. supplied by the St. Lawrence Seaway Authority.
			10	TW	PH				115 (1.8)	
						530 (161.5)				
						520 (158.5)				
156.8 / 29.1	514.3 / 95.5	Layered, red brown, grey. End of Borehole.	11	TW	PH	510 (155.4)			114 (1.8)	

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 — 50 100 (kN m²)

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

REMARKS GR SA SI CL

20 15 5 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

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 — IN TERMS OF EFFECTIVE STRESS $\tau_f = c' + \sigma' \tan \phi'$
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION — IN TERMS OF EFFECTIVE STRESS $\tau_f = c' + \sigma' \tan \phi'$
c_u	APPARENT COHESION — IN TERMS OF TOTAL STRESS $\tau_f = c_u + \sigma \tan \phi$
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION — IN TERMS OF TOTAL STRESS $\tau_f = c_u + \sigma \tan \phi$
μ	COEFFICIENT OF FRICTION
S_t	SENSITIVITY

GENERAL

π	$= 3 \cdot 1416$
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

FD-9a (Rev. Jan.73)

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.	DENSENESS	'N' BLOWS / FT.
VERY SOFT	0 - 250	VERY LOOSE	0 - 4
SOFT	250 - 500	LOOSE	4 - 10
FIRM	500 - 1000	COMPACT	10 - 30
STIFF	1000 - 2000	DENSE	30 - 50
VERY STIFF	2000 - 4000	VERY DENSE	> 50
HARD	> 4000		

TERMS TO BE USED IN DESCRIBING SOILS:-

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

TYPE OF SAMPLE

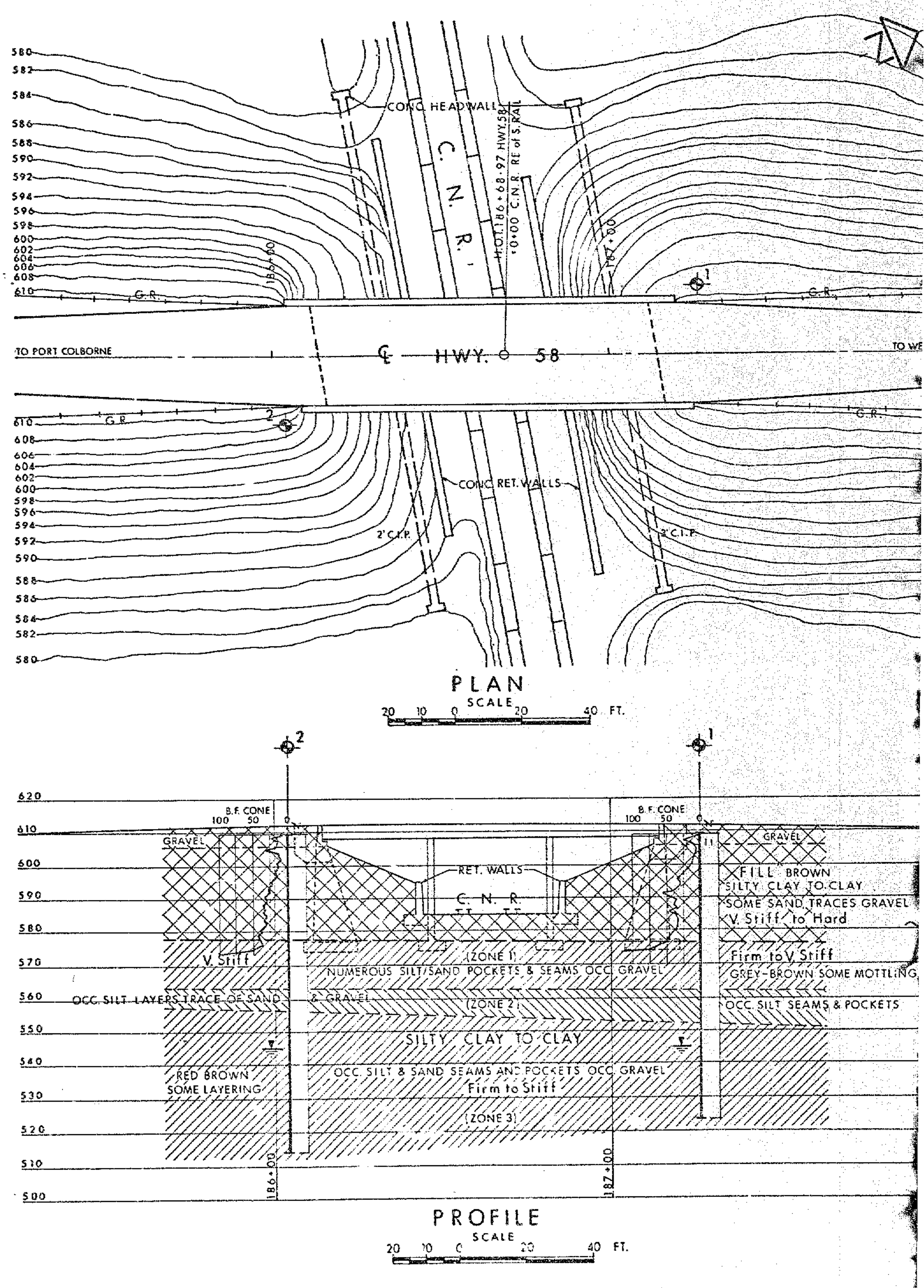
S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.T.	SLOTTED TUBE SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE

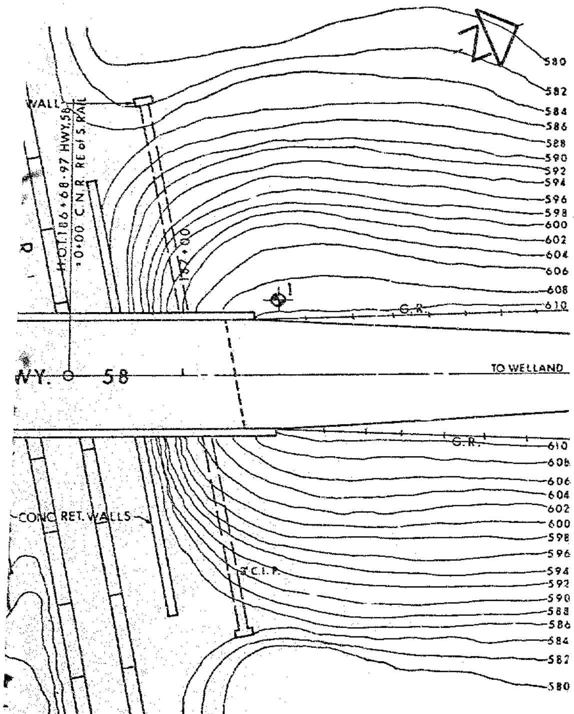
P.H. SAMPLE ADVANCED HYDRAULICALLY

P.M. SAMPLE ADVANCED MANUALLY

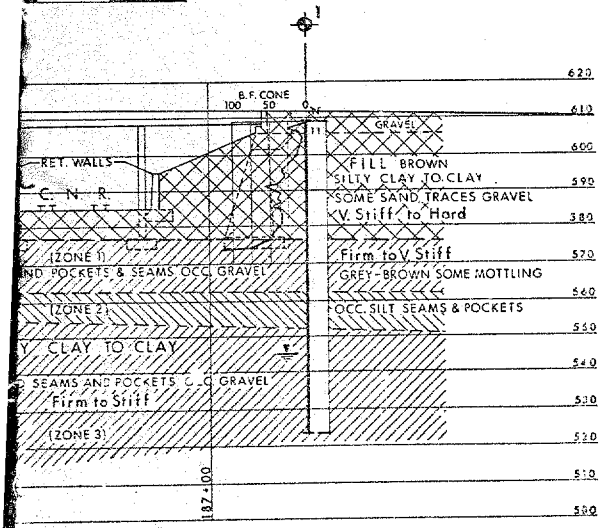
SOIL TESTS

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


CONC HEADWALL
C. N. R.
H.O.T.186+68.97 HWY.58
0+00 C.N.R. RE of S. RAIL
186+00
187+00
G.R.
TO PORT COLBORNE
℄ HWY. 58
TO WE
CONC RET. WALLS
2' C.I.P.
PLAN
SCALE
20 10 0 20 40 FT.
B.F. CONE
GRAVEL
RET. WALLS
C. N. R.
V. Stiff
(ZONE 1)
NUMEROUS SILT/SAND POCKETS & SEAMS OCC. GRAVEL
OCC. SILT LAYERS TRACE OF SAND & GRAVEL
(ZONE 2)
SILTY CLAY TO CLAY
RED BROWN
SOME LAYERING
OCC. SILT & SAND SEAMS AND POCKETS OCC. GRAVEL
Firm to Stiff
(ZONE 3)
FILL BROWN
SILTY CLAY TO CLAY
SOME SAND TRACES GRAVEL
V. Stiff to Hard
Firm to V. Stiff
GREY-BROWN SOME MOTTLING
OCC. SILT SEAMS & POCKETS
PROFILE
SCALE
20 10 0 20 40 FT.

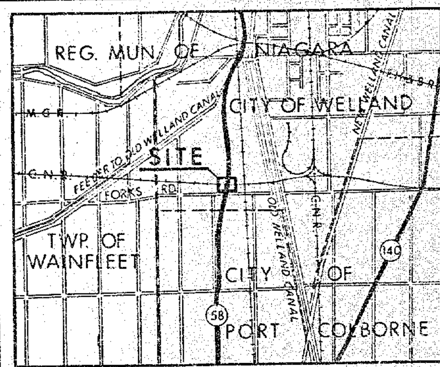


PLAN
SCALE 0 20 40 FT.



PROFILE
SCALE 0 20 40 FT.

REF. B-190-7
D-190-1-6



KEY PLAN
SCALE IN MILES
0.5 0 1

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		
1	609·1	187+26	20' LT.
2	609·8	186+04	20' RT.

NOTE:
The complete foundation investigation report for this structure may be examined at the Structural 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.

REVISIONS	DATE	BY	DESCRIPTION

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

C.N.R. & HWY.58

HIGHWAY NO. 58 DIST. NO. 4
REGIONAL MUNICIPALITY OF NIAGARA
TWP. OF WAINFLEET LOT 25 CON. V

BORE HOLE LOCATIONS & SOIL STRATA

SUBM'D J. H.	CHECKED	W.P. NO 78-73-02	DRAWING NO. 73-11098A
DRAWN O.L.J.	CHECKED	W.O. NO 73-11098	BRIDGE DRAWING NO.
DATE 5 FEB 1974		SITE NO.	
		CONT. NO.	

S-11098

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

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/ao

A. G. Stermac,
PRINCIPAL FOUNDATIONS ENGINEER.

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

Foundations Files
Documents

File No.: 58-2[illegible]-1

THE ST. LAWRENCE SEAWAY AUTHORITY

ADMINISTRATION DE LA VOIE MARITIME DU SAINT-LAURENT

Construction Branch,
P. O. Box 592,
St. Catharines,
Ontario L2R 6W8,
Feb. 7, 1974.

Mr. James Hodge,
Soil Mechanics Section,
Geotechnical Office,
Engineering Services Branch,
Ministry of Transportation & Communication,
1201 Wilson Avenue,
Downsview, Ontario.
M3N 1J8.

Dear Mr. Hodge:

Re: <u>Consolidation Data - Highway 58 and Forkes Road, Welland</u>

As discussed earlier today, you will find attached consolidation test data (e - log p curves) which have been determined in our laboratories for a site located at the north-east corner of the intersection of the abandoned Welland Canal and the new west approach to the Townline Road/Rail tunnel. Six graphs are included, as well as a graphic log for the boring involved.

The site of the boring (our No. W-1295-1) is probably the closest to the problem which you are studying. I hope that the information will prove useful.

Yours truly,

C. J. Christensen,
Materials & Testing Engineer.

CJC/cg

Encl.

Ministry of Transportation and Communications
Ontario

Memorandum

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

From: Structural Office,
West Building, Downsview.

Attention:

Date: March 12, 1975.

Our File Ref.

In Reply to

Subject: Deck Replacement for C.N.R. Overhead,
W. P. 78-73-02, Site 34-111,
Highway 58, District 4.

RECEIVED
GEOTECHNICAL OFFICE
MAR 13 1975
SOIL MECHANICS
MINISTRY OF TRANSPORTATION & COMMUNICATIONS

So
mar 18

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.

CSG/cf
Attch.

C. S. Grebski,
Structural Design Engineer.

W.O. 73-1109P

Finalized
24 Mar. 1975
A. Jansson

Foundation Report recommended:

(I) Structure be designed to tolerate differential settlement of the order of 1½" (Simply supported Girders) o.k.

(II) Fill treatment involving

(a) resloping & ambering the clay surface within 30-40 ft of the abutments;

(b) (free-draining) granular fill subgrade to full width of the embankment top to provide drainage

Apr. 25, 75.
B. J.

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

Soil Mechanics Section
Geotechnical Office
West Building, Downsview

April 10, 1975

W.P. 78-73-02

DECK REPLACEMENT FOR CNR OVERHEAD
W.P. 78-73-02, Site 34-111
Hwy. 58, District 4

We have reviewed the final design drawings (34-111, sheet 1) of the above structure.

We note that our recommendations concerning fill treatments are not shown in the drawings. In our Foundation Report, it was recommended that:

(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.

After discussion with Mr. J. Cullen, we trust that these recommendations will be incorporated in the grading drawings.

B. L. Ly

B. LY
Project Engineer

for: K. G. SELBY
Supervising Engineer.

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

File
Record Services

Ministry of Transportation and Communications
Ontario

1573

Memorandum

To: Mr. M. R. Ernesaks
Regional Manager
Reg. Planning & Design
Central Region

Attention:

From: Structural Office
West Building
Downsview, Ontario

Date: July 12, 1976

Our File Ref

In Reply to

Subject: W.P. 78-73-02 : 34-112
Deck Replacement for C.N.R. Overhead
Hwy. 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.
Structural Maintenance Engineer
Soil Mechanics Section

N. Zoltay
N. Zoltay,
Structural Contract
Specifications Engineer

NZ/jl
Encl.

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

RECEIVED
GEOTECHNICAL OFFICE
JUL 13 1976
SOIL MECHANICS
MINISTRY OF TRANSPORTATION & COMMUNICATIONS

KGS

Ministry of Transportation and Communications
Ontario

K28

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. 78-73-02, Site 34-111,
Deck Replacement for C.N.R. Overhead,
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 (7 gal.)
Top Coat Grey MTC Code No. 16-2-25-2 (16 gal.)

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

N. Zoltay,
Structural Contract
Specifications Engineer.

NZ/im

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

DOCUMENT MICROFILMING IDENTIFICATION

G.I-30 SEPT. 1976

GEOCRES No. 30L14-24

DIST. 4 REGION CENTRAL

W.P. No. 78-73-02

CONT. No. 76-40

W.O. No. 73-11098

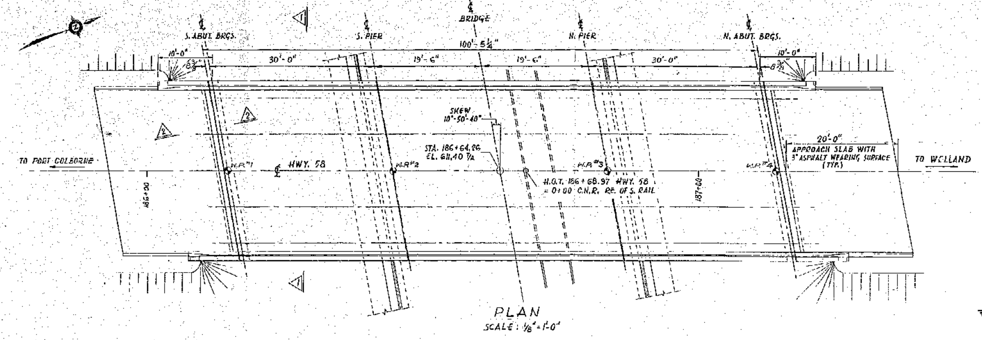
STR. SITE No:

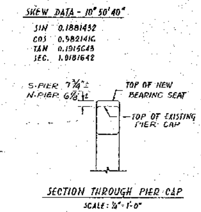
HWY. No. 58

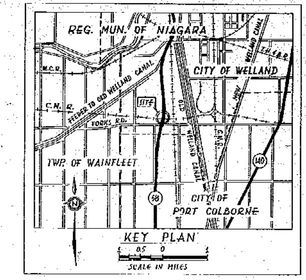
LOCATION EMBANKMENT SETTLEMENTS /HWY58/ CNR OVERPASS

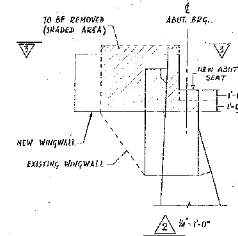
OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. 2

REMARKS: 2 documents to be unfolded before microfilming



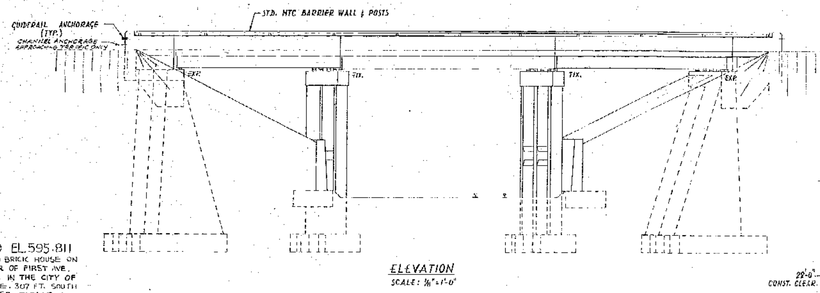






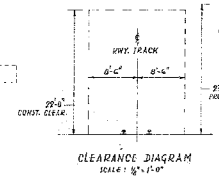
LIST OF DRAWINGS

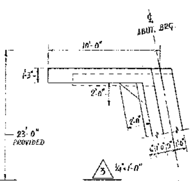
34-111 - 1 - GENERAL PLAN
2 - BORE HOLL LOCATIONS & SOIL STRATA
3 - ABUTMENTS & PIERS - DIMENSIONS & REINF.
4 - DECK & GIRDER DETAILS
5 - CONCRETE BARRIER WALL (2'-3" HIGH)
6 - STEEL PARAPET RAILING (SINGLE TUBE)
7 20 FT. APPROACH SLAB (FOR BARRIER WALL)
8 - STANDARD DETAILS
9 - AS CONSTRUCTED ELEV. & DIM.



B.M. #379-69 EL. 595.811
ONE STOREY RED BRICK HOUSE ON THE S.W. CORNER OF FIRST AVE. AND THIRD AVE. IN THE CITY OF PORT COLBORNE. 307 FT. SOUTH OF ℄ OF HWY #3. TABLET IS SET HORIZONTALLY IN NORTH FACE OF CONCRETE FOUNDATION, BEING 9 FEET WEST OF N-E. CORNER AND 2.4 FEET BELOW BRICKWORK.

PRINT RECORD		
No.	FOR	DATE





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.

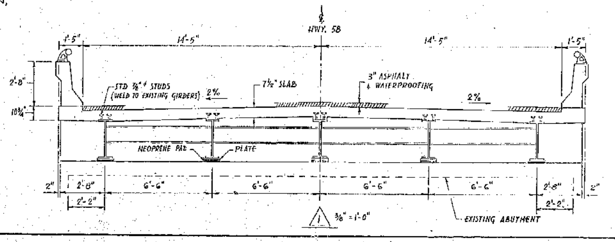
CONSTRUCTION NOTES

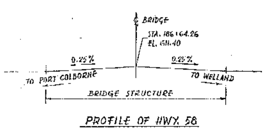
THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF ± ⅛".
NO CONCRETE SHALL BE PLACED ABOVE THE ABUTMENT BEARING SEATS, UNTIL THE CONCRETE IN THE DECK HAS BEEN PLACED.

CONCRETE QUANTITIES

CONCRETE IN PIERS, ABUTMENTS AND WINGWALLS — 34 C.Y.
CONCRETE IN DECK — 51 C.Y.
CONCRETE IN BARRIER WALLS — 21 C.Y.
CONCRETE IN APPROACH SLABS — 37 C.Y.

30L1424
GEOCRES No.





FOR REDUCED PLAN
USE SCALE BELOW
3 INCHES ON ORIGINAL PLAN

REVISIONS		
DATE	BY	DESCRIPTION

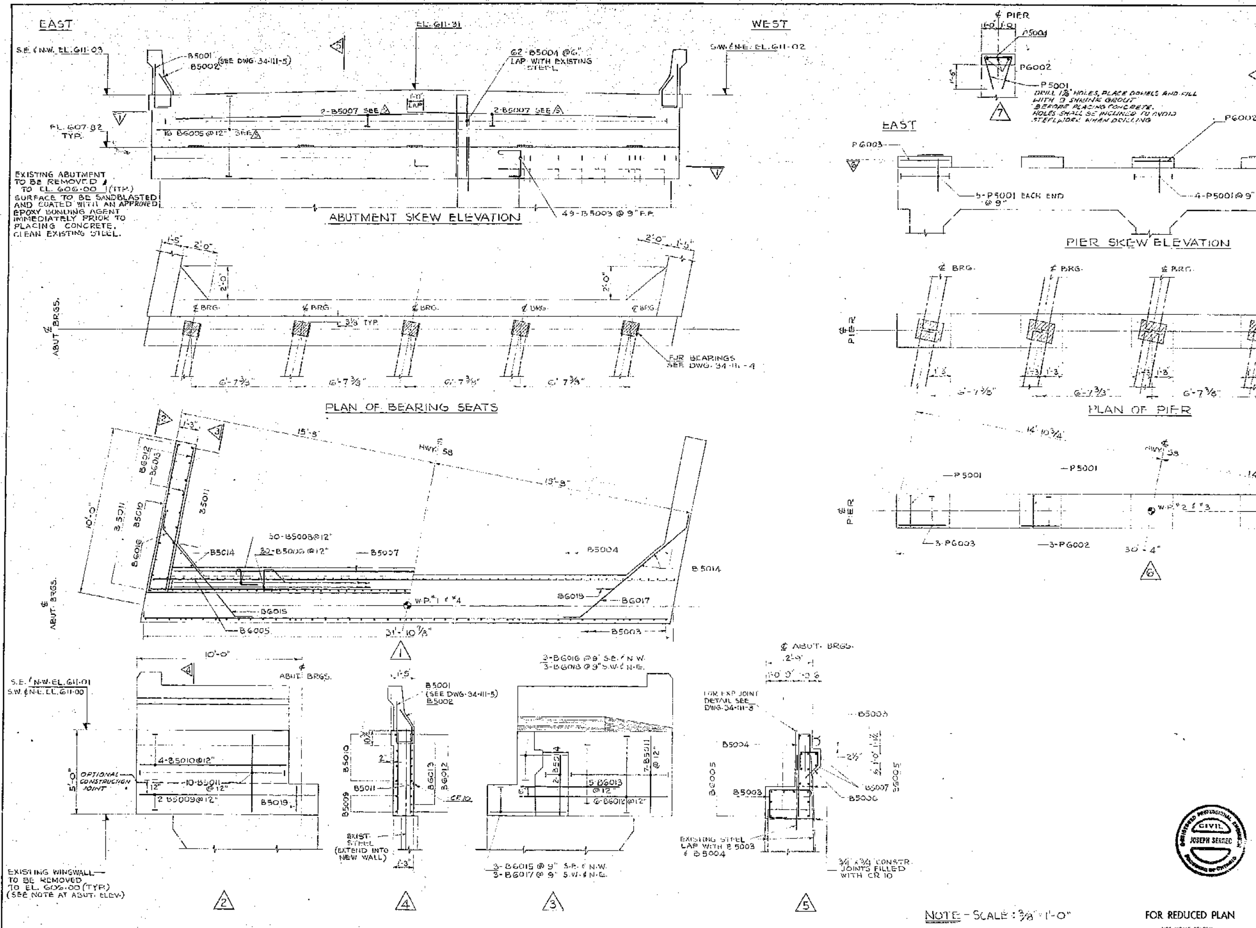
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS
ONTARIO

DECK REPLACEMENT FOR
C.N.R. OVERHEAD

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

GENERAL PLAN

APPROVED	STRUCTURAL ENGINEER			CONTRACT No.	
DESIGN	J.	CHECK	C.F.F.	W.P. No.	78-73-02
DRAWING	R.K.	CHECK	J.	SITE No. 34-111	SHEET 1
DATE	NOV. 1974	LOADING	HS 20-44		



REVISIONS		
DATE	BY	DESCRIPTION

FOR REDUCED PLAN
USE SCALE BELOW
3 INCHES ON ORIGINAL PLAN

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS
ONTARIO

DECK REPLACEMENT FOR
C.N.R. OVERHEAD

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

ABUTMENTS & PIERS - DIMENSIONS & REINF.

APPROVED				CONTRACT No.	
DESIGN	J. L.	CHECK	C. F. R.	W.P. No.	78-73-02
DRAWING	D. G.	CHECK		SITE No. 34-111	SHEET 3
DATE	JAN-1975	LOADING	HS 20-44		

30L14-24