DOCUMENT MICROFILMING IDENTIFICATION

G.I.-30 SEPT. 1976

GEOCRES No. 31F-82

DIST. 9 REGION

W.P. No. 2-67-04

CONT. No. 80-12

W. O. No.

STR. SITE No. 29-162

HWY. No. 17N

LOCATION C.N.R. Overhead
4.5 mi W of Hwy 41

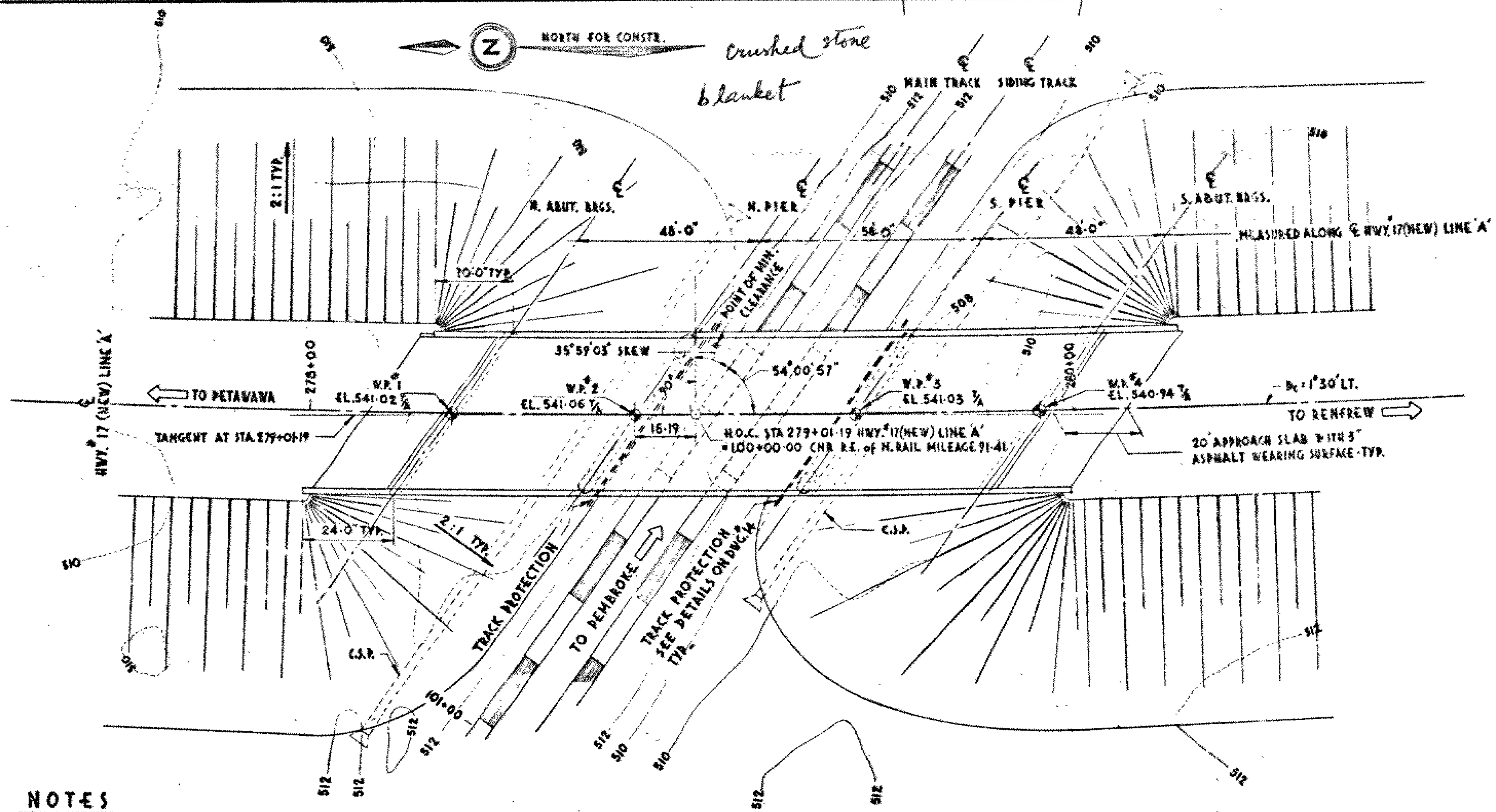
No of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

preload + Roadway protection. NB Check possible settlements of the tracks

DIST. No 9	
CONT No	
WP No 2-67-04	
CNR OVERHEAD Approx. 4.5 Miles West of Hwy 41 GENERAL PLAN	SHEET



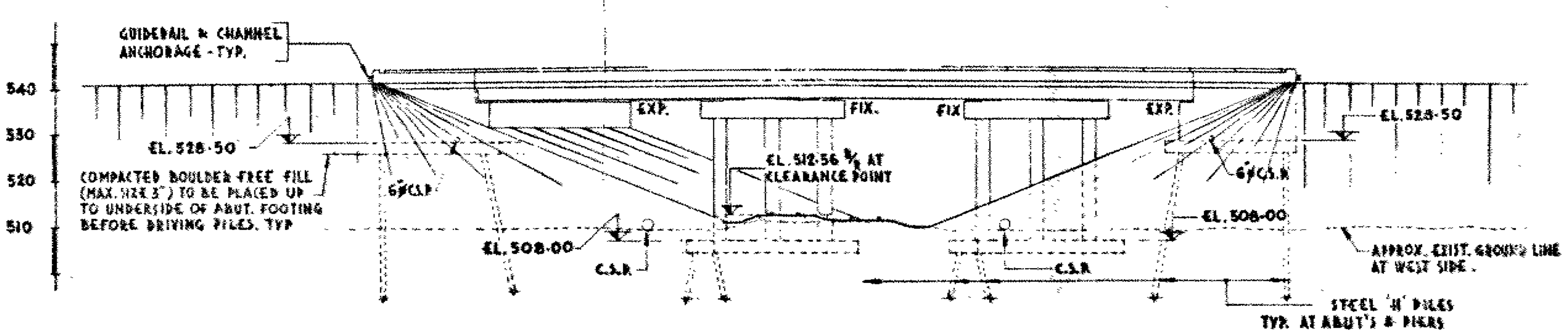
NOTES

- W.P. DENOTES WORKING POINT.
- T/A DENOTES TOP OF ASPHALT WEARING SURFACE.
- B/R DENOTES BASE OF RAIL.

P L A N

SCALE: 1" = 20'

APPROACH SLABS, ASPHALT AND WATERPROOFING ARE NOT PART OF THIS CONTRACT.



E L E V A T I O N

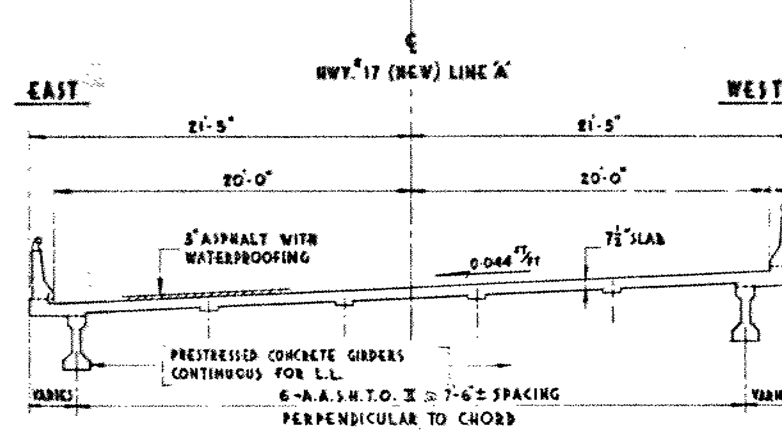
SCALE: 1" = 20'

TO BE USED FOR ESTIMATING PURPOSES ONLY
DATE FEB 21 1978

CONCRETE QUANTITIES

CONCRETE QUANTITIES ARE LISTED BELOW FOR THE APPROPRIATE CONC. LUMP SUM TENDER ITEMS.

1- CONCRETE IN PIERS, ABUTMENTS AND WING WALLS	C.Y. 3000 P.S.I. C.Y. 4000 P.S.I.	
2- CONCRETE IN DECK AND DIAPHRAGMS	C.Y.	
3- CONCRETE IN BARRIER WALLS	C.Y.	
4- CONCRETE IN APPROACH SLABS	C.Y.	



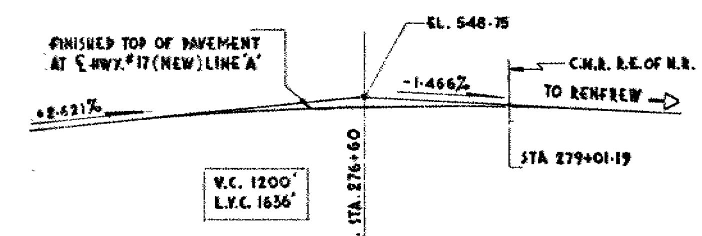
TYP. DECK SECTION

SCALE: 3/16" = 1'-0"

B.M. 511.42

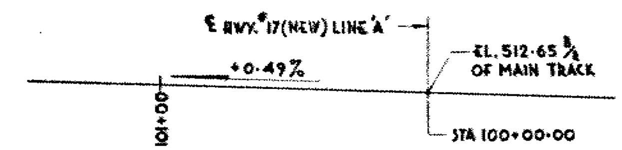
GEODETIC DATUM

N & W IN MIX ROOT OF 1.5 PINE 103' RT. OF 275+59



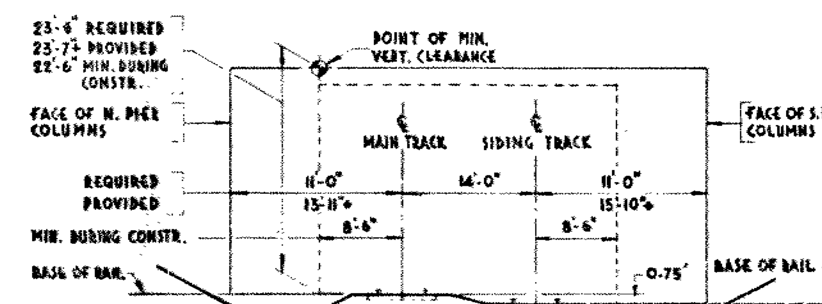
PROFILE OF HWY. 17 (NEW) LINE 'A'

N.T.S.



PROFILE OF C.N.R. R.E. OF N.R.

N.T.S.



RAILWAY CLEARANCE DIAGRAM

CLEARANCES SHOWN ARE PERPENDICULAR TO ℄ TRACKS (N.T.S.)

NOTES

- **CLASS OF CONCRETE**

DECK, BARRIER WALLS & PIERS . . . . . 4000 P.S.I.
PRESTRESSED GIRDERS . . . . . 5000 P.S.I.
REMAINDER . . . . . 3000 P.S.I.

- **REINFORCING STEEL GRADE**

GRADE 60 OR AS NOTED ON DRAWINGS.

- **CLEAR COVER TO REINF. STEEL**

FOOTINGS AND ABUTMENTS . . . . . 3"
PIER COLUMNS . . 2½", PIER CAPS . . . . 2"
DECK TOP . . . . 2", DECK BOTTOM . . 1½"
DIAPHRAGMS AND BARRIER WALLS . . . 1½"
APPROACH SLABS . . . . . 2"

- **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.

TO ACHIEVE THE MIN. CLEAR COVER OF 2" SPECIFIED AT TOP OF DECK, THE TOP LAYER OF REINFORCEMENT SHALL BE PLACED, PRIOR TO CONCRETING, WITH A CLEAR COVER OF 2½" ± ½" TOLERANCE.

LIST OF DRAWINGS

29-162-1 GENERAL PLAN
2 BOREHOLE LOCATIONS & SOIL STRATA
3 FOUNDATION LAYOUT & REINF.
4 N. ABUTMENT
5 S. ABUTMENT
6 PIERS
7 PRESTRESSED GIRDERS & BEARINGS
8 DECK
9 BARRIER WALL
10 STEEL PARAPET RAIL (SINGLE TUBE)
11 20 FT APPROACH SLAB
12 AS CONSTRUCTED ELEV. & DIM.
13 STANDARD DETAILS
29-162-14 STANDARD DETAILS

RECEIVED FEB 22 1978 MINISTRY OF TRANSPORTATION & COMMUNICATIONS SOIL MECHANICS

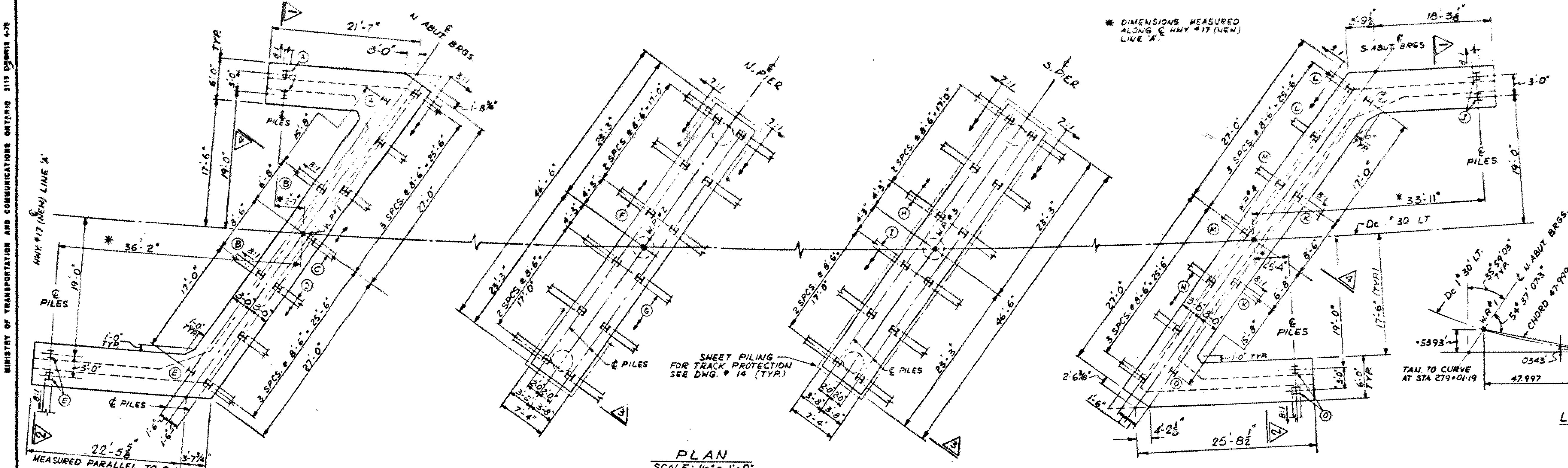
FOR REDUCED PLAN USE SCALE BELOW
0 1 2 3
3 INCHES ON ORIGINAL PLAN

REVISIONS | DATE | BY
DESIGN R.K. | CHECK | DATE FEB 78
DRAWING G.C. | CHECK | SITE No 29-162 | DWG 1

CONT No	
WP No	2-57-04

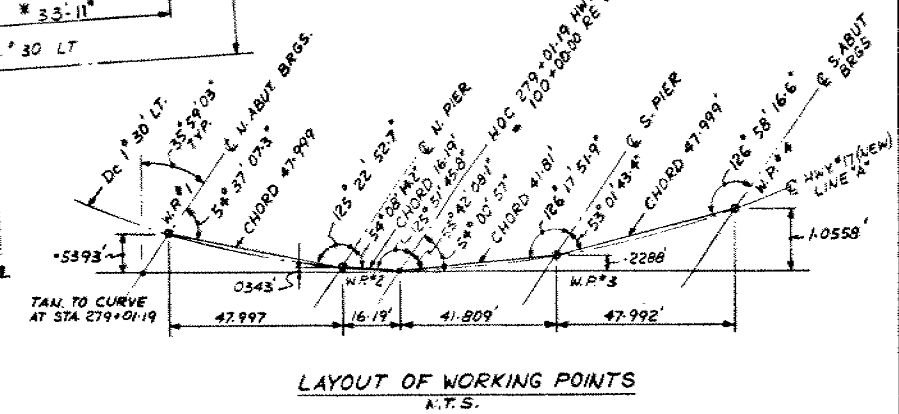
C.N.R. OVERHEAD
Approx 4.5 Miles West of Hwy. 41
FOUNDATION LAYOUT & REINF.

SHEET



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

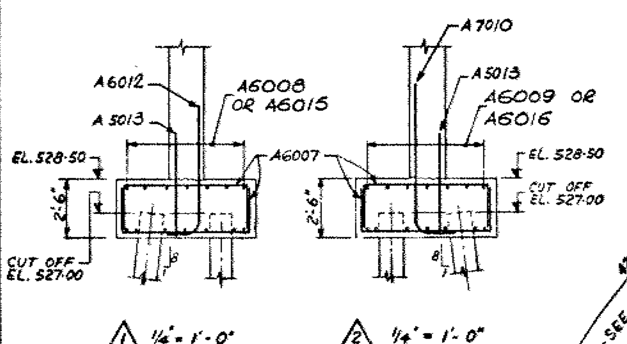
NOTE - LETTERS IN CIRCLES DENOTE GROUP OF PILES.



LAYOUT OF WORKING POINTS
N.T.S.

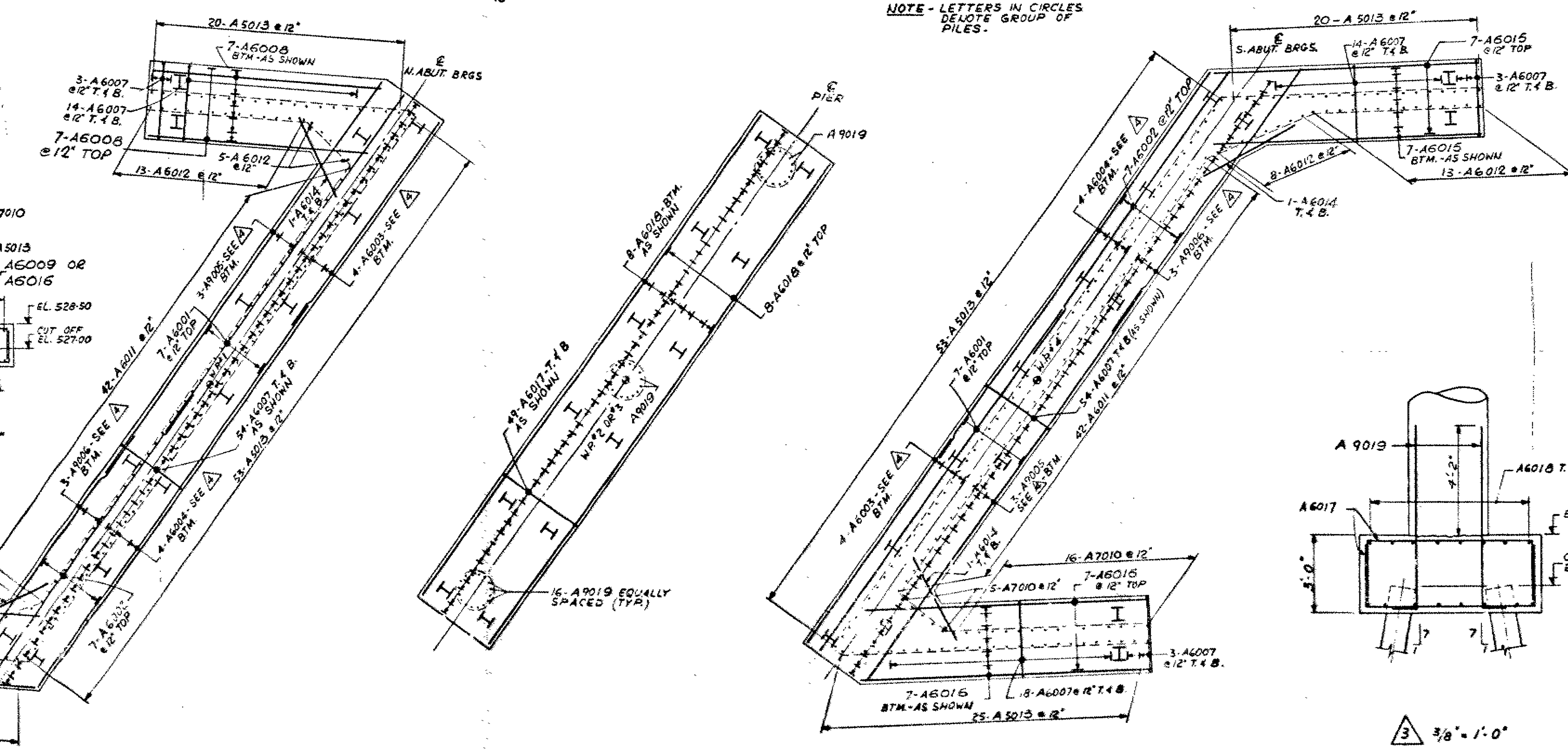
NOTES -

- ALL PILES TO BE HP 12 x 53
- PILES TO BE DRIVEN TO BEDROCK
- T & B DENOTES TOP & BOTTOM
- PILE SPACING TO BE MEASURED AT UNDERSIDE OF FOOTING
- REINF. FOR N. PIER & S. PIER FTGS. SIMILAR
- EXCAVATION FOR PIER FTGS. TO BE CARRIED OUT WITHIN INTERLOCKING BRACED STEEL SHEET PILES DRIVEN TO EL. 498.5 FOR N. PIER & EL. 502.5 FOR S. PIER - AS SHOWN ON DWG. #14.

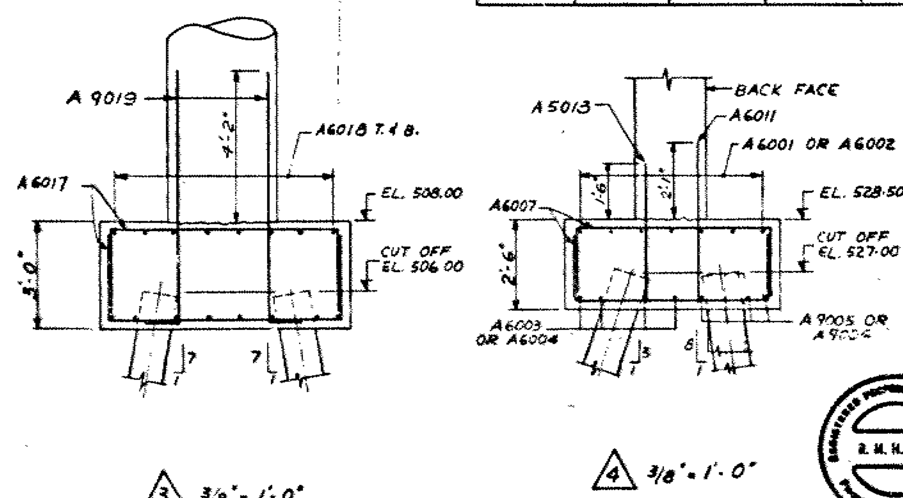


PILES

LOCATION	GROUP	BATTER	No REQ'D	LENGTH
NORTH ABUTMENT	A	8 : 1	1	104'
		VERT.	2	
	B	8 : 1	2	108'
	C	3 : 1	4	112'
	D	3 : 1	3	114'
	E	8 : 1	1	110'
		VERT.	2	
N. PIER	F	7 : 1	6	88'
	G	7 : 1	6	90'
S. PIER	H	7 : 1	6	90'
	I	7 : 1	6	92'
SOUTH ABUTMENT	J	8 : 1	1	108'
		VERT.	2	
	K	8 : 1	2	114'
	L	3 : 1	2	116'
	M	3 : 1	2	118'
	N	3 : 1	3	122'
	O	8 : 1	1	116'
		VERT.	2	



FOOTING REINF.
SCALE: 3/16" = 1'-0"



R. H. KAN

FOR REDUCED PLAN
USE SCALE BELOW
3 INCHES ON ORIGINAL PLAN

REVISIONS	DATE	BY	DESCRIPTION

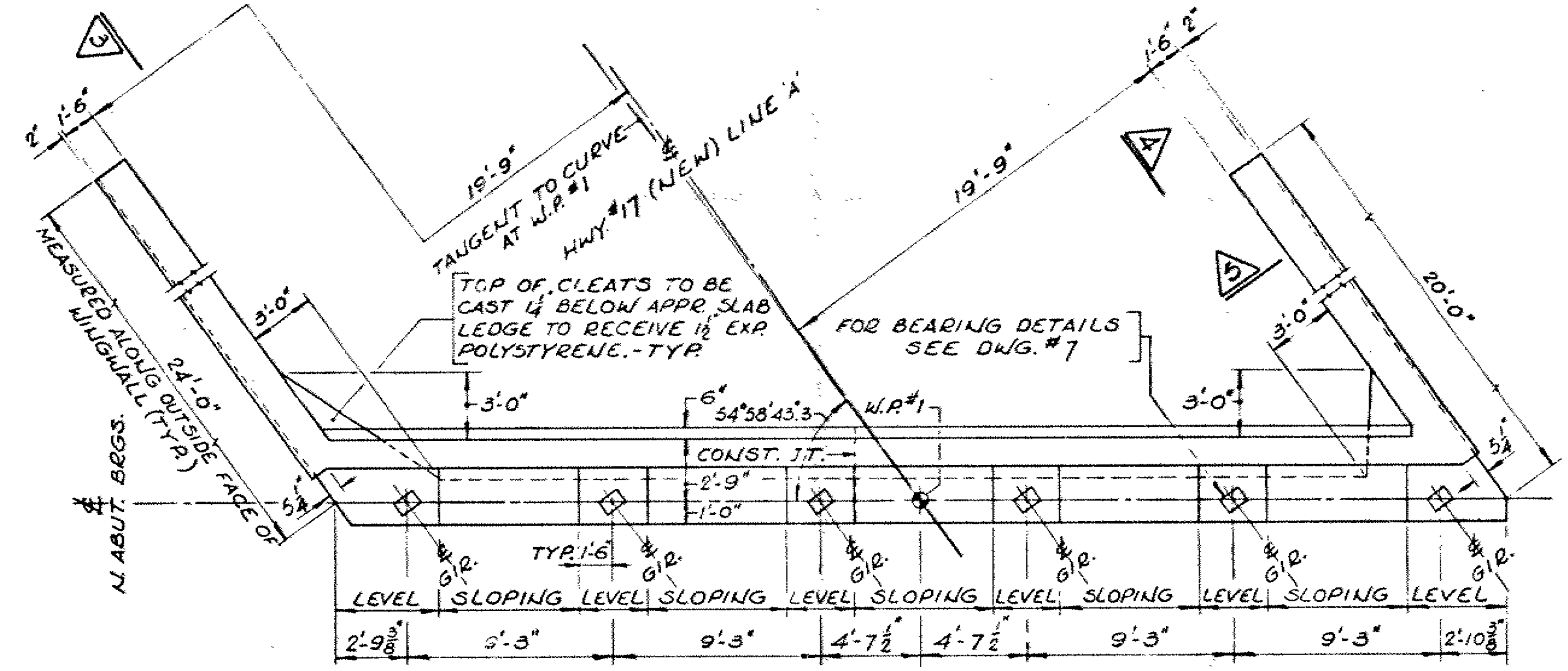
DESIGN	CHECK	LOADING HS 20-44	DATE
DRAWING	CHECK	SITE No 29-162	DWG 3

CONT No	
WP No 2-67-04	
C.N.R. OVERHEAD Approx. 4.5 Miles West of Hwy.41 NORTH ABUTMENT	SHEET

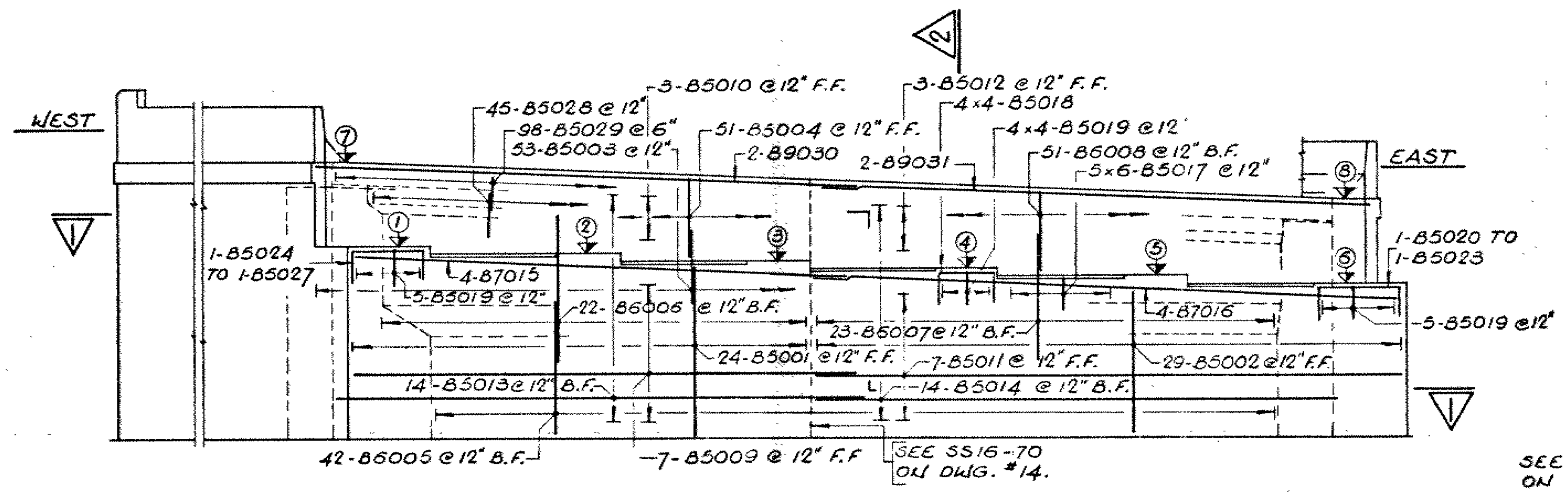
NOTES:

- F.F. DENOTES FRONT FACE.
- B.F. " BACK FACE.
- O.F " OUTSIDE FACE.
- I.F. " INSIDE FACE.
- SCALE: 1/4" = 1'-0" UNLESS OTHERWISE NOTED
- WING WALLS TO BE SET PARALLEL TO ℄ HWY. #17 (NEW) LINE 'A'.

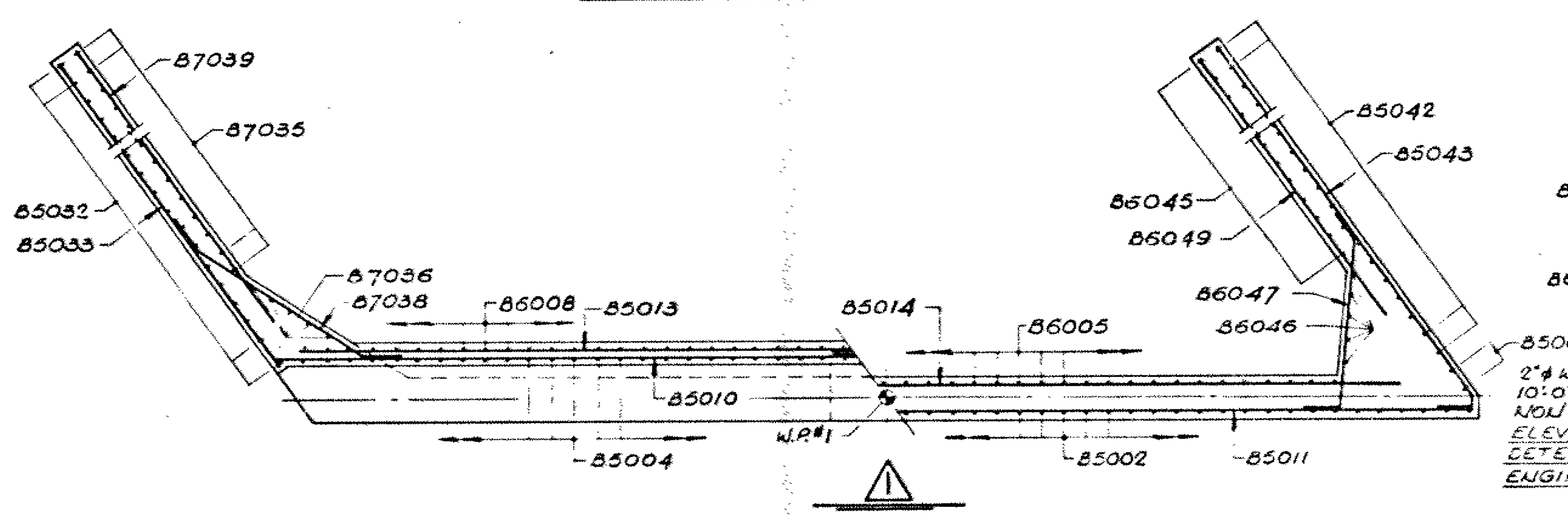
POINT	ELEV.	
1	537.68	AT ℄ ABUT. BRGS.
2	537.35	
3	537.03	
4	536.70	
5	536.37	
6	536.05	
7	541.89	AT F.F. OF BALL. WALL
8	540.17	

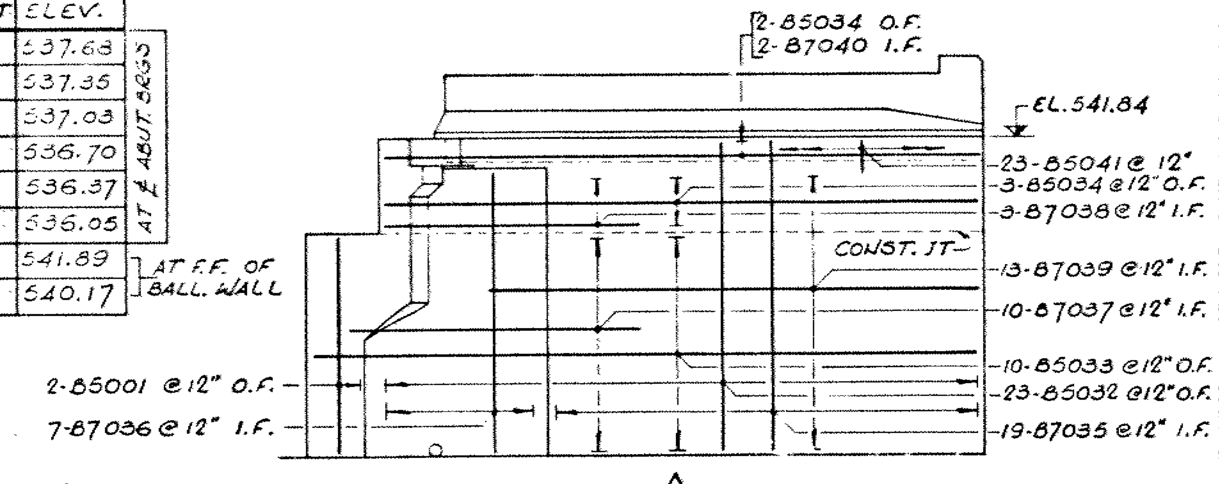


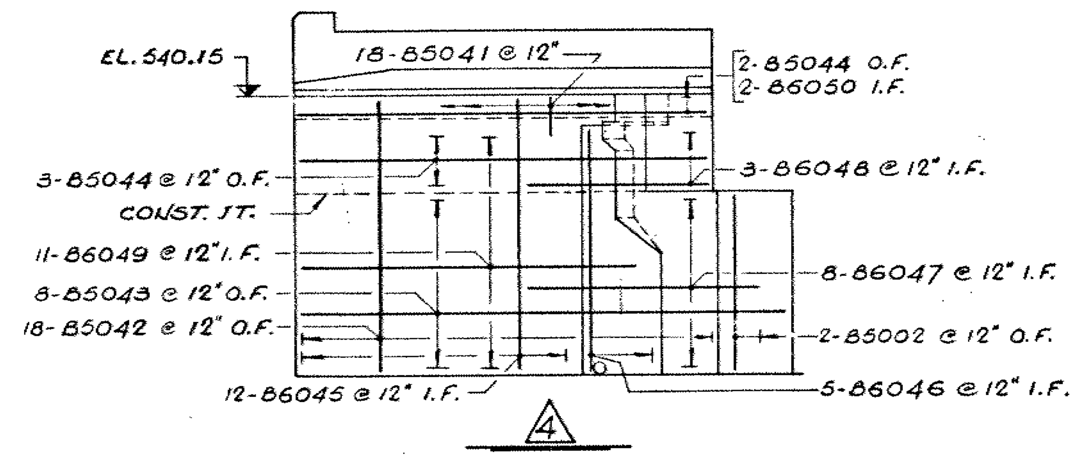
PLAN

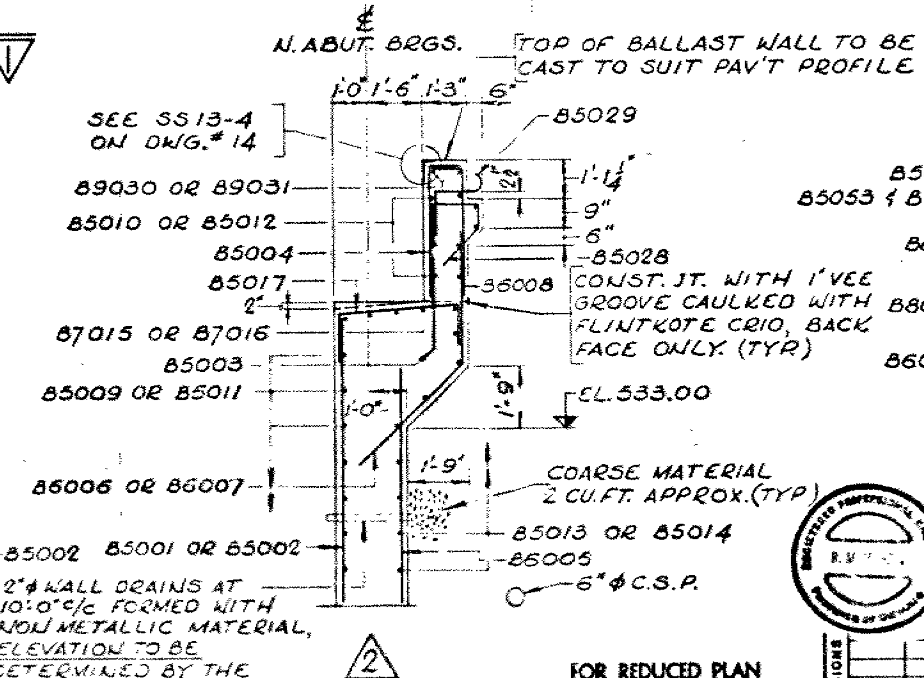


ELEVATION

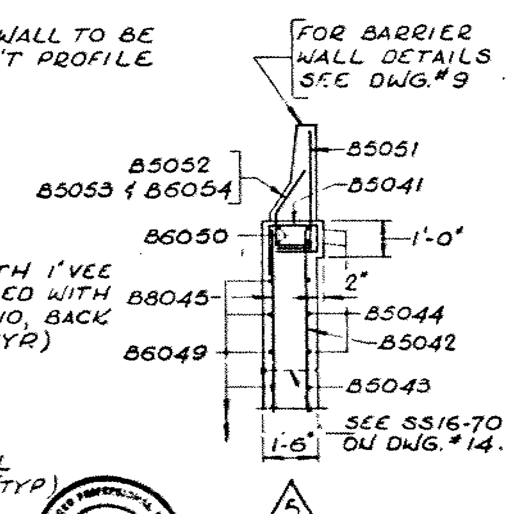








SCALE: 3/8" = 1'-0"



SCALE: 3/8" = 1'-0"

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

REVISIONS DATE	BY	DESCRIPTION	
DESIGN	CHECK	LOADING	DATE
DRAWING	CHECK	SITE No.	DWG

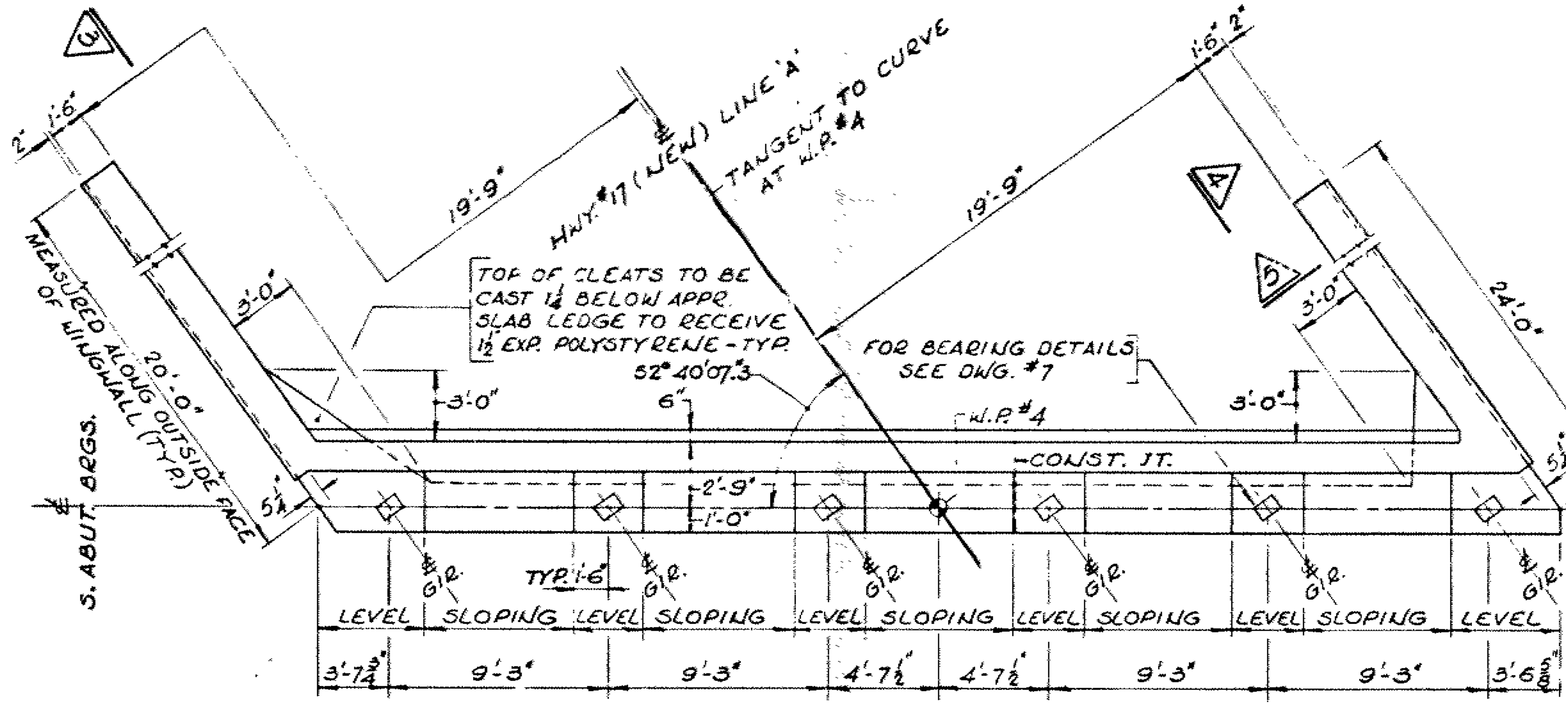
CONT No	
WP No 2-67-04	
C.N.R. OVERHEAD Approx. 4.5 Miles West of Hwy. 41 SOUTH ABUTMENT	SHEET

NOTES:

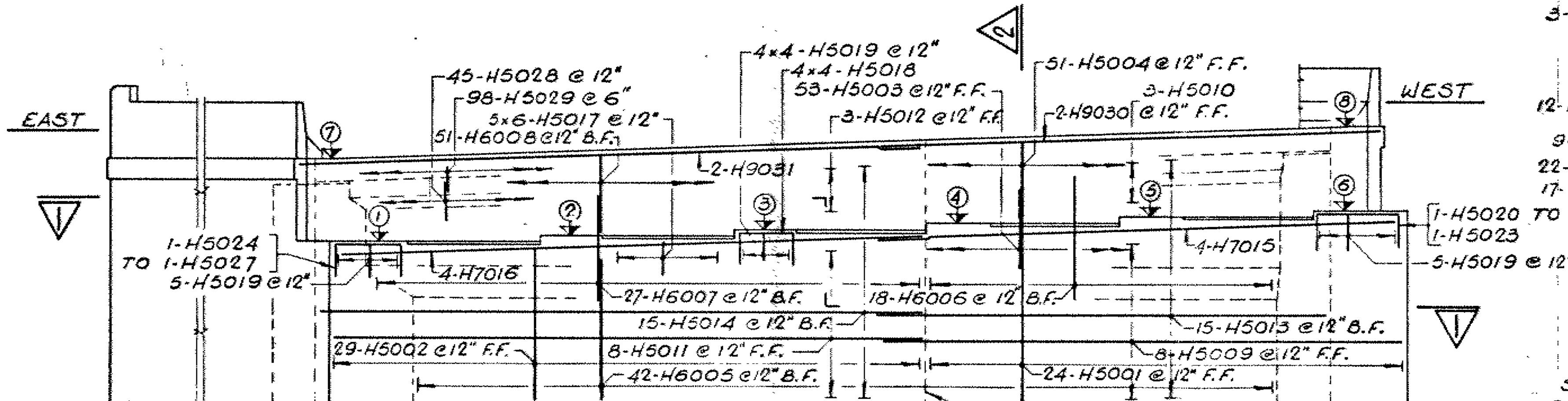
- F.F. DENOTES FRONT FACE.
- B.F. " BACK FACE.
- O.F. " OUTSIDE FACE.
- I.F. " INSIDE FACE.
- SCALE: 1/4" = 1'-0" UNLESS OTHEWISE NOTED.
- WING WALLS TO BE SET PARALLEL TO ℄ HWY. #17 (NEW) LINE 'A'.

POINT	ELEV.
1	535.93
2	536.27
3	536.60
4	536.94
5	537.28
6	537.62
7	540.02
8	541.85

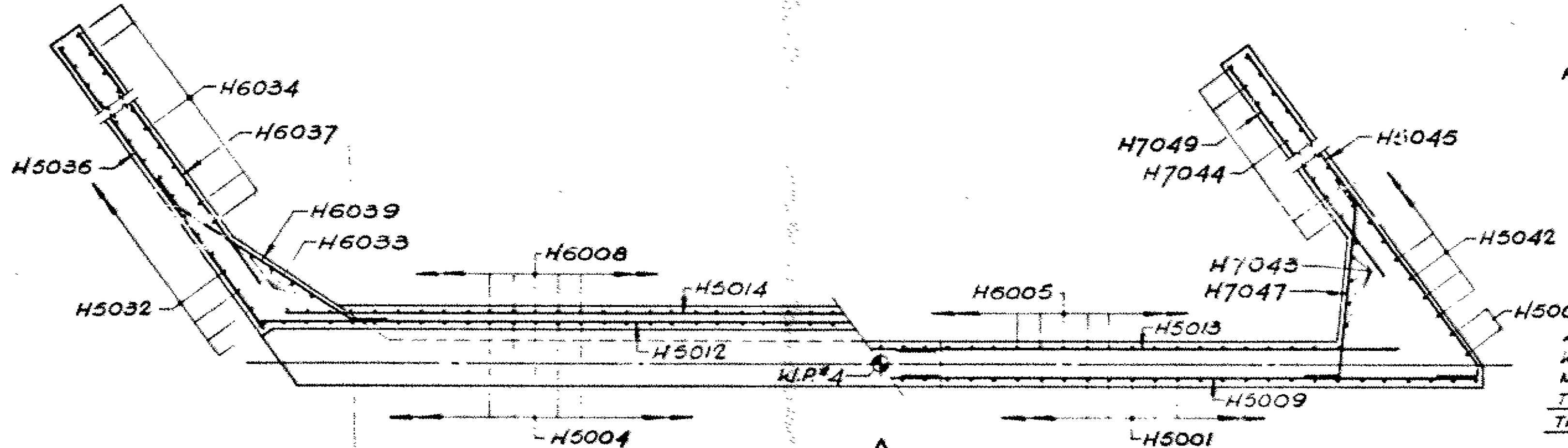
AT F.F. OF BALL. WALL AT ℄ ABUT. BRGS.

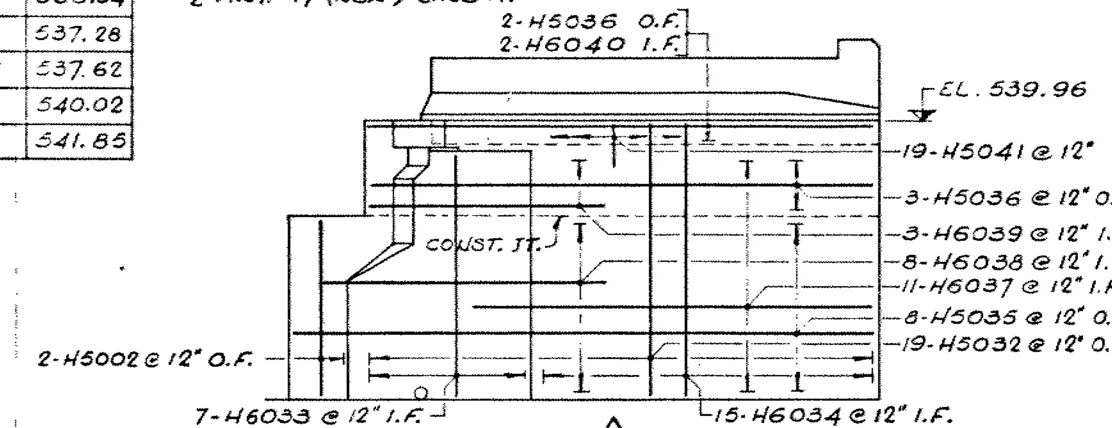


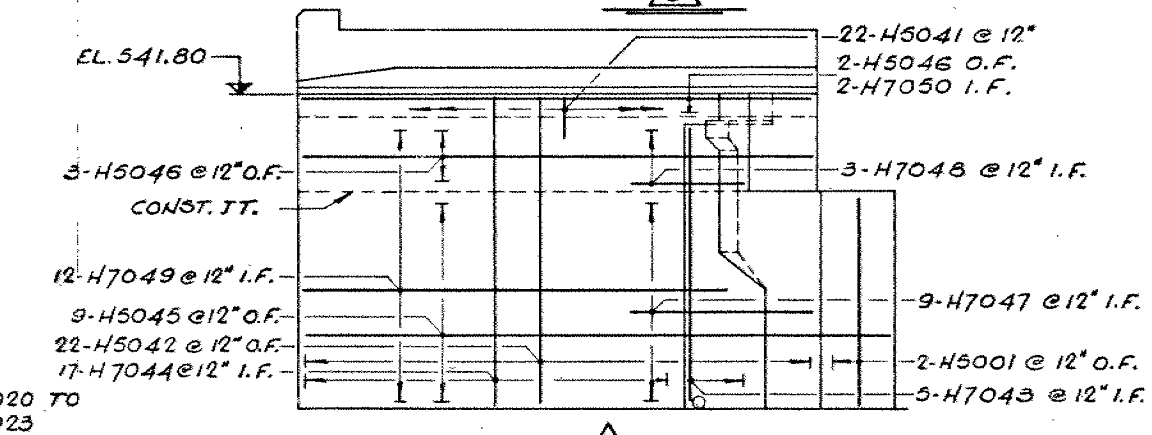
PLAN

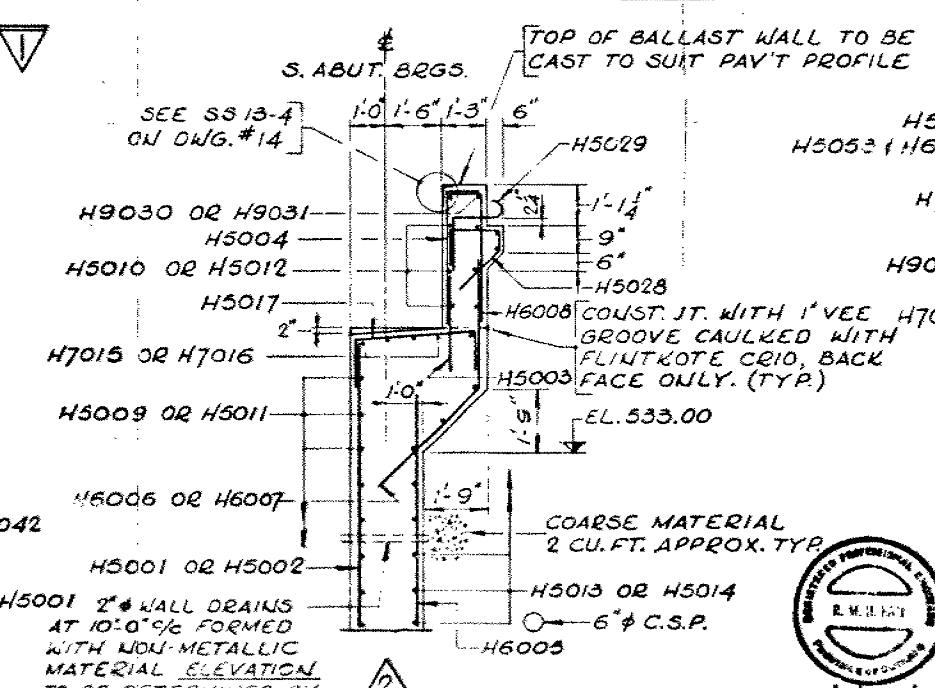


ELEVATION

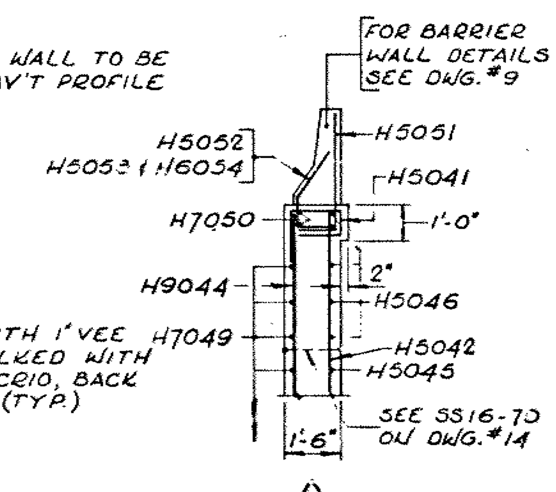








SCALE: 3/8" = 1'-0"



SCALE: 3/8" = 1'-0"

FOR REDUCED PLAN USE SCALE BELOW

3 INCHES ON ORIGINAL PLAN

REVISIONS		
DATE	BY	DESCRIPTION

DESIGN	CHECK	LOADING HS20-44	DATE FEB 78
DRAWING	CHECK	SITE No 29-152	DWG 5

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

31F-82
GEOCRES No.

MEMORANDUM

TO: T.C. Kingsland (2)
Regional Structural Planning Engineer
Eastern Region
Kingston

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

ATTENTION:

DATE: January 8, 1976

JAN 14 1976

OUR FILE REF.

IN REPLY TO

SUBJECT: FOUNDATION INVESTIGATION REPORT
for
W.P. 2-67-04
Hwy. 17N District #9
CNR Overhead
4.5 Miles West of Hwy. 41

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 requirements. Should additional information be required, please do not hesitate to contact our Office.

M. Devata

M. Devata
Supervising Engineer

cc: E.J. Orr
B.R. Davis
B.J. Giroux
G.A. Wrong
A.J. Percy
E.R. Saint
J.M. Childs

R. Hore
J. Anderson)
R. Forrest) memo only
G. Sloan)
Files ✓
Record Services

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3. FIELD AND LABORATORY WORK

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7. MISCELLANEOUS

FOUNDATION INVESTIGATION REPORT

for

W.P. 2-67-04
Hwy. 17N District #9
CNR Overhead
4.5 Miles West of Hwy. 41

1. INTRODUCTION

A request to carry out the foundation investigation at the site of the above mentioned structure was received by this Section in a memorandum dated September 15, 1975, from Mr. T. Kingsland, Regional Structural Planning Engineer, Eastern Region. An investigation was subsequently carried out by this Section to determine the subsoil and groundwater conditions at the site.

This report presents the results of the investigation and our recommendations pertaining to the design of the proposed structure foundations, as well as the stability and settlement considerations associated with the approach fills.

2. DESCRIPTION OF SITE AND GEOLOGY

The site is located about 3½ miles west of the limits for the Town of Pembroke. The area lies just east of the intersection of Biesenthal Road and the CNR tracks in the Twp. of Alice, County of Renfrew.

In the immediate vicinity of the site, the area is relatively flat, lying between elevations 510 and 514. The area is well drained and no natural watercourses are discernible. A shallow ditch of about a 2 ft. depth runs parallel to the CNR tracks on the south side. The CNR tracks are constructed on an embankment of about 2 to 2.5 ft. high. The area is heavily wooded and covered with both evergreen and deciduous trees.

In general, the ground slopes northwards. No rock outcrops are visible, except 2000 ft. northeast of the site where a rocky knoll, which is partially exposed, rises to about 50 ft. above the surrounding terrain. Sand and gravel pits of up to about 10 ft. deep can be observed just south of the proposed site.

Physiographically, this region is known as the Ottawa Valley Clay Plains. The granitic gneiss bedrock of the Precambrian age has undergone glacial change, with the resulting deposition of glacial till. The Champlain Sea then inundated the area leaving deposits of marine clay surrounding higher islands of glacial material. Final emergence of the land upon the retreat of the salt water sea left the topography free to be weathered by local streams and atmospheric conditions.

3. FIELD AND LABORATORY WORK

A total of eight boreholes, two of which were accompanied by dynamic cone penetration tests, were carried out during the field investigation. Boring was achieved by means of a bombardier mounted hollow-stem auger machine adapted for soil sampling purposes.

During the fieldwork, disturbed samples were obtained by standard split-spoon samplers driven into the soil with an energy of 350 ft.-lb. per blow according to the specifications of the Standard Penetration Test. Where cohesive deposits were encountered, the split-spoon sampling was supplemented by taking 2 inch I.D. Shelby tubes, which were pushed manually or hydraulically into the soil. In addition, field vane tests were carried out, where possible, to obtain the undrained shear strength of the cohesive stratum. Bedrock was proven in three of the borings by obtaining BXL size rock core samples. Groundwater level observations were carried out in the open boreholes during the period of the investigation. In addition, two Peaker type piezometers were installed at two of the boring locations.

The soil, bedrock and groundwater conditions encountered at the boring locations are presented on the Record of Borehole Sheets, appended to this report. The locations and elevations of the boreholes were provided by the personnel from the Engineering Surveys Office, Eastern Region. The elevations in this report are referenced to a Geodetic Datum. Boring locations and elevations are shown on Dwg. No. 26704-A, together with an estimated stratigraphical profile along the proposed centreline.

All samples were subjected to a careful visual examination in the field and subsequently in the laboratory. Following the examination, laboratory tests were carried out on representative samples to determine the following engineering properties of the overburden:

Natural Moisture Contents
Atterberg Limits
Grain Size Distributions
Bulk Unit Weights
Undrained Shear Strength Measurements
Shear Strength Measurements in terms of
Effective Stress (C.I.U. Tests with Pore Pressure Measurements)
Consolidation Characteristics

The results of the laboratory testing were plotted on the individual Record of Borehole Sheets and also summarized on Figures No. 1 to 7, inclusive, all of which are appended to this report.

4. SUBSOIL AND BEDROCK CONDITIONS

(4.1) General

Beneath a thin mantle of topsoil (6 to 8 inches), the surficial deposit consists of fine to very fine grained sand with a trace of silt and its thickness varies from 3 to 10 feet. The relative density of this material ranges from very loose to compact. The sand is underlain by a stiff to very stiff stratum of clayey silt to silty clay with varying amounts of sand ranging in thickness from 20 to 36 feet. Underlying this cohesive deposit is the predominant stratum of very loose to compact silty sand to sandy silt deposit with trace to some clay, having a total thickness ranging from 50 to 63 feet. The fine to very fine granular material is then followed by a more coarser deposit of 3 to 12 feet in thickness, and consisting of dense to very dense sand and gravel, with some silt. The overburden is underlain by gneiss bedrock. The surface of the bedrock ranges from elevations 412.9 to 425.0, which indicates that the depth to bedrock varies from 84 to 97 feet below ground surface.

The boundaries between the various deposits, as determined at the boring locations, are shown on the accompanying Record of Borehole Sheets. An estimated stratigraphical profile across the site, inferred from the boring data, is plotted on Drawing No. 26704-A. The subsoil and bedrock types encountered, from ground surface downward, are presented in the subsections to follow.

(4.2) Surficial Deposit

A surficial deposit of fine to very fine grained sand with a trace of sand is encountered beneath a thin topsoil cover (6 to 8 inches). Standard penetration testing was carried out within this material, and the results are plotted on the Record of Borehole Sheets. This testing gave 'N' values which generally range from 7 to 22 blows per foot, indicating that the relative density varies from loose to compact. However, at Borehole No. 2 which was put down in the ditch, the relative density of the surficial deposit was very loose. Grain-size distribution testing was carried out on samples obtained from this deposit and the results are plotted in an envelope form on Figure 2.

(4.3) Clayey Silt to Silty Clay

The surficial deposit is underlain by a stratum of grey, brittle clayey silt to silty clay and in general with traces of sand. Higher sand contents were observed in some of the samples. Occasional to some pockets and seams of silt are present throughout the stratum. Within this deposit few random layers of silty sand to sandy silt, which range from ¼ inch to up to 1 foot in thickness, are present in Borehole's No. 4 and No. 6. The thickness of the cohesive zone varies from 20 feet (B.H. 3) to 36 feet (B.H.6).

The engineering properties of the cohesive subsoil, as determined by the field and laboratory testing, are plotted on the Record of Borehole Sheets and summarized in tabular form below.

Index Properties	Range	(Average)
Natural Moisture Contents W (%)	30-53	(41)
Liquid Limits W_L (%)	27-40	(34)
Plastic Limits W_p (%)	14-24	(20)
Liquidity Index I_L	0.79 - 3.00	(1.58)
Bulk Unit Weight γ (p.c.f.)	106 - 118	(112)

Compressibility Characteristics	Range
Initial Void Ratio (e_o)	1.02 - 1.46
Compression Index (C_c)	0.37 - 0.81
Degree of Preconsolidation (P_c - P_o') p.s.f.	3,700 - 7,240
Undrained Shear Strength (C_u) p.s.f.	
In Situ Vane Tests	1080 ->2,240
Unconfined Compression Tests	930 - 2035
Quick Triaxial Test (one test)	1242
Sensitivity (By In Situ Vanes)	3 - 9

The Atterberg Limit Test results, given in the Table, are also summarized on the Plasticity Chart, Figure 1. The testing indicates that the cohesive stratum is inorganic and of low to intermediate plasticity. The natural moisture content is at or above the liquid limit as indicated by liquidity indices which are generally greater than unit. This is usually typical of the sensitive marine clay. Grain-size distribution testing carried out on samples obtained from this stratum are plotted in an envelope form on Figure 3.

Referring to Figures No. 5 and 6, and the undrained shear strength values presented in the Table, it is estimated that the consistency of the stratum varies from stiff to very stiff. The sensitivity, defined as the ratio of the undrained shear strength of the soil in an undisturbed state to that of the soil in a remoulded condition, as determined in the field in situ vane tests ranges from 3 to 9 with an average of 5.5. This would indicate that the clayey silt to silty clay stratum is generally sensitive.

The consolidation characteristics of the cohesive stratum were determined by carrying out four laboratory oedometer tests, the results of which are shown as a Void Ratio vs. Pressure Plots on Figure 7. Three of the curves obtained from the laboratory testing were reconstructed by utilizing the Schmertmann* technique. The results indicate that the cohesive stratum is preconsolidated by

*Schmertmann, J.H., 1953. The undisturbed consolidation behavior of Clay. Am. Soc. of Civil Engrs., Paper No. 2775 pp. 1201-1233.

about 3,700 to 7,240 p.s.f. in excess of the existing overburden pressure.

In addition to the more routinely employed tests previously described, the engineering properties of the cohesive stratum in terms of effective stresses were also determined. This was performed by carrying out an isotropically consolidated undrained triaxial compression test in which the excess pore water pressure buildup and eventual dissipation, due to the applied load, was monitored throughout (CIU test). The results of this testing are summarized below.

C' (Apparent Effective Cohesive Intercept) - 0
ø' (Apparent Effective Angle of Friction) - 31°

(4.4) <u>Silty Sand to Sandy Silt</u>

The cohesive stratum is underlain by the predominant deposit of fine to very fine silty sand to sandy silt and in general with trace to some amounts of clay. Random layers of clayey silt to silt of up to about 18 inches in thickness are also present within this deposit. Borehole No's 3 and 6 were terminated within this stratum. However, elsewhere, the thickness of this deposit varies from 50 feet (B.H. 5) to 63 feet (B.H. 1). Grain-size distribution testing was carried out on samples obtained from this material. The results are plotted in an envelope form on Figure 4.

Standard penetration testing was performed within this deposit and the results are plotted on the Record of Borehole Sheets, This testing gave 'N' values which range from 1 blow/18" to 22 blows/foot. Based on these results, it is estimated that the relative density of this material varies from very loose in the upper portion and changing to compact in the lower portion of the deposit.

(4.5) <u>Sand and Gravel, Some Silt</u>

The fine to very fine granular material is underlain by more coarser deposit of sand and gravel with some silt. This material which ranges in thickness from 2.7 feet to 11.5 feet is found immediately above bedrock. Standard Penetration Tests carried out within this deposit gave 'N' values ranging from 56 to 140 blows/foot.

Based on these tests and the nature of the augering operation, it is estimated that the relative density of this material is dense to very dense.

(4.6) Gneiss Bedrock

Bedrock was proven in B.H.'s 1, 2 and 5, by obtaining 4.5 to 5 feet of BXL size rock core samples, and refusal to augering (probable bedrock surface) was reached in B.H.'s 4 and 7. The surface of the bedrock varies from elevations 412.9 and 425.0, which indicates that the depth to bedrock ranges from 84 to 97 feet below ground surface.

The bedrock can be identified as hard gneiss and generally sound, as evidenced by the quality and the percentage recovery of the rock cores. Detailed descriptions of the rock cores as described by Mr. B. Glassford, Geologist for M.T.C., are presented on the Diamond Drill Record Sheet included in the Appendix.

5. GROUNDWATER CONDITIONS

Groundwater level observations were carried out during the course of the field investigation by recording the water level in the open boreholes. The observations are plotted on the Record of Borehole Sheets. The groundwater levels obtained by these observations, vary from elevations 507.8 to 512.0, and their corresponding depths from ground surface range from 0 to 1.5 feet.

Artesian water conditions were encountered during drilling operations when the boreholes (1, 2, 5, 7, 8) were extended into the lower granular deposit. Artesian water was not observed in B.H.'s 3, 4 and 6 since the augers were removed immediately once the boreholes reached the required depth and it is believed that there was not adequate time to activate the artesian flow at the ground surface.

The flow of the artesian water observed from the 3¼" I.D. hollow stem augers when extended to the bedrock surface was estimated to be 2.5 to 3.5 gpm near the ground surface. The augers were then extended above ground level in order to determine the stabilized artesian heads and found to be 4.5 feet, 5 feet and 6 feet above ground surface at B.H.'s No. 1, 2 and 5 respectively.

In addition, two piezometer of the Peaker type namely P7 (B.H.7) and P8 (B.H.8) were installed. The tip elevations of the piezometers and the observed stabilized artesian heads are shown on the Record of Borehole sheets, as well as on Dwg. No. 26704-A.

The drilling operations at B.H.'s 1 and 5 revealed that the artesian flow increased considerably once the borings extended into the bedrock surface. From this, together with the water level observations carried out for this investigation, it appears that the source of the artesian water is primarily confined within the upper portion of the bedrock.

6. DISCUSSION AND RECOMMENDATIONS

(6.1) General

It is proposed to construct a new highway designated as Hwy. 17N which will bypass the Town of Pembroke. At present, the proposed highway will be a two lane roadway. It is understood at a later stage when Hwy. 17N becomes a part of Hwy. 417, the roadway will consist of two lanes in each direction, separated by a wide median.

The proposed bypass will require several structures, one of them being at the crossing of Hwy. 17N (Line 'A') and the C.N.R. tracks, approximately 0.2 miles east of the intersection of Biesenthal Road and C.N.R. tracks, in the Twp. of Alice, County of Renfrew. The new structure will be a 3 span (60'-50'-60') having a width of 44 ft. The profile grade of Hwy. 17N in the vicinity of the crossing will be at elevation 542, whereas the ground elevation is about 510. The C.N.R. tracks, are constructed on a 2 to 2.5 ft. high embankment at about elevation 512.5. To realize the proposed grade, an embankment of 32 ft. will be required.

The subsoil, bedrock, and groundwater conditions encountered in this area, have been discussed previously in this Report in Sections No. 4 and 5. An inferred stratigraphical profile, along the proposed centreline, is shown on Drawing No. 26704-A.

Recommendations for the structure foundations and the approach embankments are given in the following subsections.

(6.2) Stability of Embankments

Analyses, in terms of total stresses have been carried out, both in the longitudinal and transverse directions, to determine the stability of the fills immediately after construction. In this method of analysis, stability is governed by the applied loads and the stress-strain and undrained shear strength properties of the foundation and fill material, and the effect of pore pressure changes are not considered. The following data (B.H.6), and assumptions were used:

Fill Material (Granular Type)

Bulk Density	γ = 135 p.c.f.
Angle of Shearing Resistance	ϕ = 30°
Embankment Slopes	2:1

Foundation Subsoil

Soil Type	Elev. (Ft.)	γ(p.c.f.)	γ(p.c.f.)	ϕ	Cu(p.s.f.)
1. Sand	510-503	130	67.6	30°	0
2. Clayey silt to silty clay	503-467	112	49.6	0	1,200

Water level elevation - 510

The stability computations carried out indicate that the proposed embankment height of 32 ft. will be stable, if constructed with 2:1 slopes (2 horizontal to 1 vertical).

The long term stability of the fills was also studied in terms of effective stresses. In this method the stability is governed by the stress-strain characteristics of the subsoil as well as the excess pore water pressure within the subsoil. The following values were used for computational purposes:

Fill Material (Granular Type - same as used in Total Stress Analysis)

Foundation Subsoil

Soil Type	Elev. (Ft.)	γ(p.c.f.)	γ(p.c.f.)	ϕ'	C'(p.s.f.)
1. Sand	510-505	130	67.6	30°	0
2. Clayey silt to silty clay	505-478	112	49.6	30°	0

Elevation	Excess Pore Water Pressure
510.0 (Ground Level)	At ground surface
476.0	3.0 ft. above ground surface

The stability analyses in terms of effective stresses indicate that the long term stability of the proposed embankment height of 32 ft. will be satisfactory provided they are constructed with 2:1 slopes.

(6.3) Settlement of Embankments

The underlying compressible clay stratum will undergo settlements due to consolidation, over a period of time, under the weight of the embankment. Settlement computations were, therefore, carried out. The induced stress increase within the foundation subsoil due to a 32 ft. surcharge loading, was computed by Purdue* method, and is less than the preconsolidation pressure of the stratum. The consolidation of the cohesive subsoil will therefore be primarily of a recompression nature. Settlements under the centerline of the embankment due to the induced stress increases were computed using both the oedometer curve, and the reconstructed curve obtained from the Schmertmann technique (Fig. 7). For a 32 ft. high embankment, it is estimated that the magnitude of the settlement will be of the order of 6 to 8 in. and should occur in a period of about 18 months. Settlements in the surficial sand deposit will be elastic and can be completed during the construction of the embankment. It is also estimated that 50% of the settlement will be completed in a period of 3 months.

If scheduling permits approach fills should be constructed and left in place at least for a period of 3 months prior to construction of the structure foundations. This will minimize post construction maintenance problems.

* Perloff, W.H., Baladi,G.Y. and Harr, M.F. 1967. Stress Distribution Within and Under Long Elastic Embankments. Embankments and their Foundations, Highway Research Record, No. 181, pp. 12-40.

(6.4) Structure Foundations

It is considered that the most practical type of foundations would be end-bearing piles, which will reduce the settlement of the structure components to a negligible amount and, therefore, a continuous structure could be employed. The following types of end-bearing piles could be utilized:

1) End Bearing Steel Piles

The piers and abutments may be supported on end-bearing steel piles driven to bedrock. For estimating purposes, it can be assumed that the pile tips will be located at the elevations given in the following table:

Location	Approx. Station	Estimated Tip Elevation	Reference B.H.'s
South Abut.	279+99	413 to 421	1,7
South Pier	279+39	416 to 419	2,4,1
North Pier	278+89	419 to 422	4,2,5
North Abut.	278+29	419	5

The piles can be designed using the ultimate capacity of the pile section chosen. For instance 12BP74 steel H-piles can be designed for 95 tons/pile.

Settlement of the foundation subsoil will occur due to the embankment surcharge loading as discussed previously. This settlement may induce some negative skin frictional loads on the end-bearing piles supporting the abutments. In addition to the negative skin frictional forces, movement of subsoil due to strain imposed by the embankment loading, will generally tend to displace the long slender piles laterally and can cause rotation of the abutments. In view of this, we recommend that consideration be given to supporting the extreme ends of the wing walls on end-bearing steel piles founded as aforementioned. It is considered that this will improve the stability of the abutment in the longitudinal direction.

The construction of the pile caps at the pier locations will require a dewatering scheme to enable construction to be carried out in a relatively dry condition. Otherwise, the fine to very fine sandy type of soil is likely to 'boil' under conditions of unbalanced hydrostatic head. One possible de-watering scheme that could be employed is driving interlocking steel sheeting at least 2 ft. into the relatively impermeable clayey silt to silty clay stratum. Any minor inflow that might enter the excavation area could be controlled by conventional means such as pumping from sumps. The sheeting will prevent possible boiling of the foundation soil and sloughing in of the side walls, and thus protect the stability of the adjacent rail-way embankment.

2) <u>Concrete Caissons Founded on Bedrock</u>

As an alternative, the piers and abutments may be founded on bored piles or caissons extended to bedrock. The shaft will have to be cased to avoid caving of the overburden. Due to the presence of artesian water at bedrock surface, the casing or liner should extend at least 2 ft. above the artesian head and thus stabilize the flow. Then for the lower 45 ft. approximately,the concrete could be placed by the tremie method. Once the artesian pressure is balanced by the weight of the tremie concrete, the remaining portion of the caisson could be constructed in a relatively dry condition. In view of the pre-vailing artesian conditions, the liner should be left in place and hence provide an adequate seal between the caisson and the surrounding soil.

An allowable load of 50 tons per square foot for the end-bearing area could be utilized. As an example a 36 in. diameter end-bearing caisson founded on bedrock will provide a design load of 350 tons/pile. For resisting any lateral thrust the caissons may be socketed into the rock, this aspect can be discussed further during design process.

The caissons may be extended as pier columns to the under-side of the deck. This would avoid any excavations close to the railroad tracks, and thus eliminate any track protection requirements.

3) <u>Expanded Base Franki Type Concrete Caissons</u>

Alternatively, end-bearing expanded base caisson piles (Franki type) may be considered. For example, a 16 in. Ø rammed shaft could develop about 50 tons per pile provided it is extended some 10 ft. into the lower granular deposit. For estimating purposes, a cost of about $25/ft. could be used for this type. The detailed information of this type of caissons could be investigated further if needed.

The construction of the pile caps at the pier locations will require a dewatering scheme. Comments made on this matter in alternative No. 1 is also applicable here.

No bouldery or rock fill should be placed in areas where piles are to be constructed. In addition to the three types of end-bearing piles discussed, a combination of these could also be considered.

7. MISCELLANEOUS

The field work for this investigation was carried out during the period of October 28, 1975 to November 20, 1975, under the supervision of Mr. H. Shah, Project Engineer.

The equipment used for subsoil sampling was owned and operated by Atcost Drilling Company.

This report was written by Mr. H. Shah and was reviewed by Mr. M. Devata, Supervising Engineer.

H. Shah

H. Shah, P. Eng.
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
H. M. SHAH
PROVINCE OF ONTARIO

M. Devata

M. Devata, P. Eng.
Supervising Engineer

APPENDIX

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO

ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

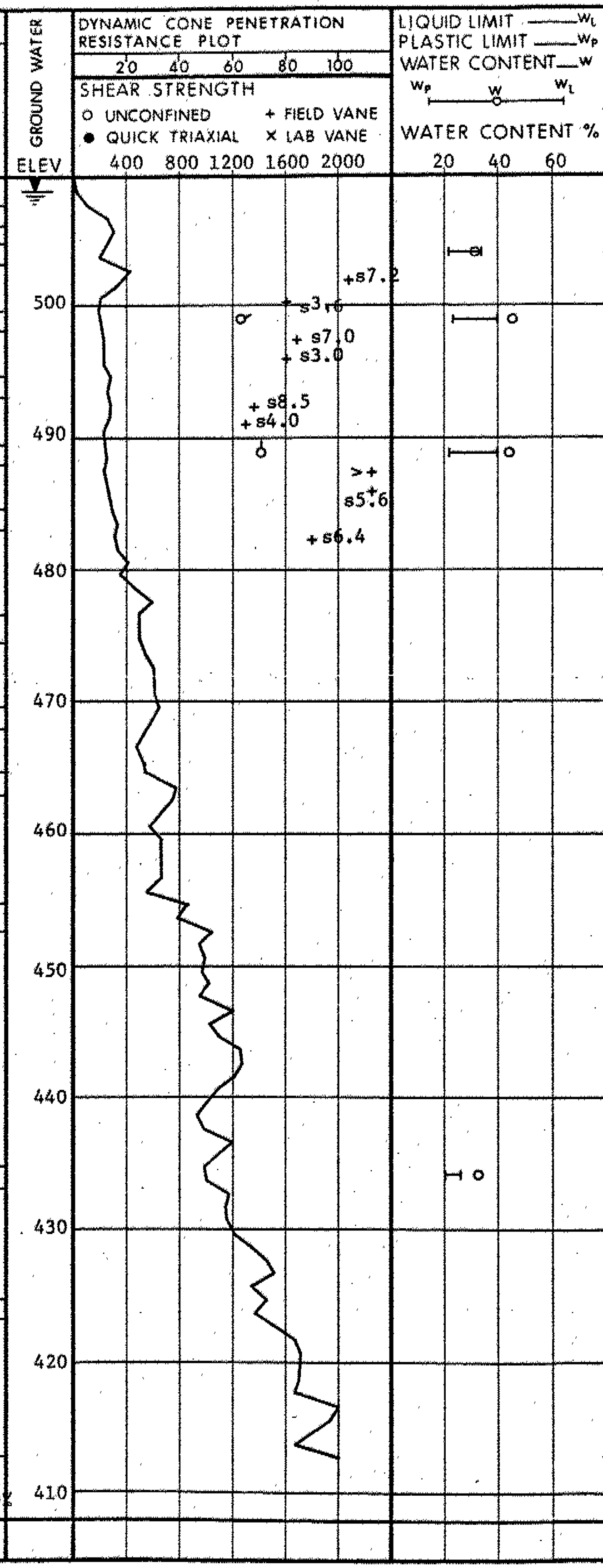
RECORD OF BOREHOLE Nº 1

WP 2-67-04 LOCATION Sta. 279 + 80 17' Rt. ORIGINATED BY HS

DIST 9 HWY 17N BORING DATE Oct. 28 to Oct. 31, 1975 COMPILED BY HS

DATUM Geodetic BOREHOLE TYPE H.S.Augers, BW Casing, BXL Core & Cone Test CHECKED BY [signature]

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	UNIT WEIGHT γ PCF	REMARKS
509.7	Ground Level					Head 514.7; Head 510.7; % GR SA SI CL
0.0	Topsoil					
	Sand, (fine to very fine), trace of silt. Compact	1	SS	15		0 93 (7)
504.7 / 5.0	Clayey silt to silty clay, traces of sand-brittle, sensitive grey occasional pockets of silt. Stiff to Very Stiff	2	SS	10		
		3	TW	PH	114	
		4	SS	2		
		5	TW	PH	110	e_o=1.16 C_c=0.49
		6	SS	4		Art. Water 479.7
480.9 / 28.8	Silty sand to sandy silt. Loose to Compact	7	SS	5		0 58 41 1
		8	SS	4		
		9	SS	6		0 75 (25)
	with random layers of clayey silt to silt.	10	SS	7		
		11	SS	11		
		12	SS	11		
417.7 / 92.0	Sand & gravel, some silt. Dense to Very Dense					Art. Water 412.9
412.9 / 96.8	Gneiss Bedrock Sound	13	RC BXL	Rec 100%		Flow=3 gpm
408.4 / 101.3	End of Borehole					



20
15 ⌖ 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

ENGINEERING SERVICES BRANCH–GEOTECHNICAL OFFICE–SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 2

WP 2-67-04 LOCATION Sta. 279 + 60 18' Lt. (Ditch) ORIGINATED BY HS

DIST 9 HWY 17N BORING DATE Nov. 3 to Nov. 5, 1975 COMPILED BY HS

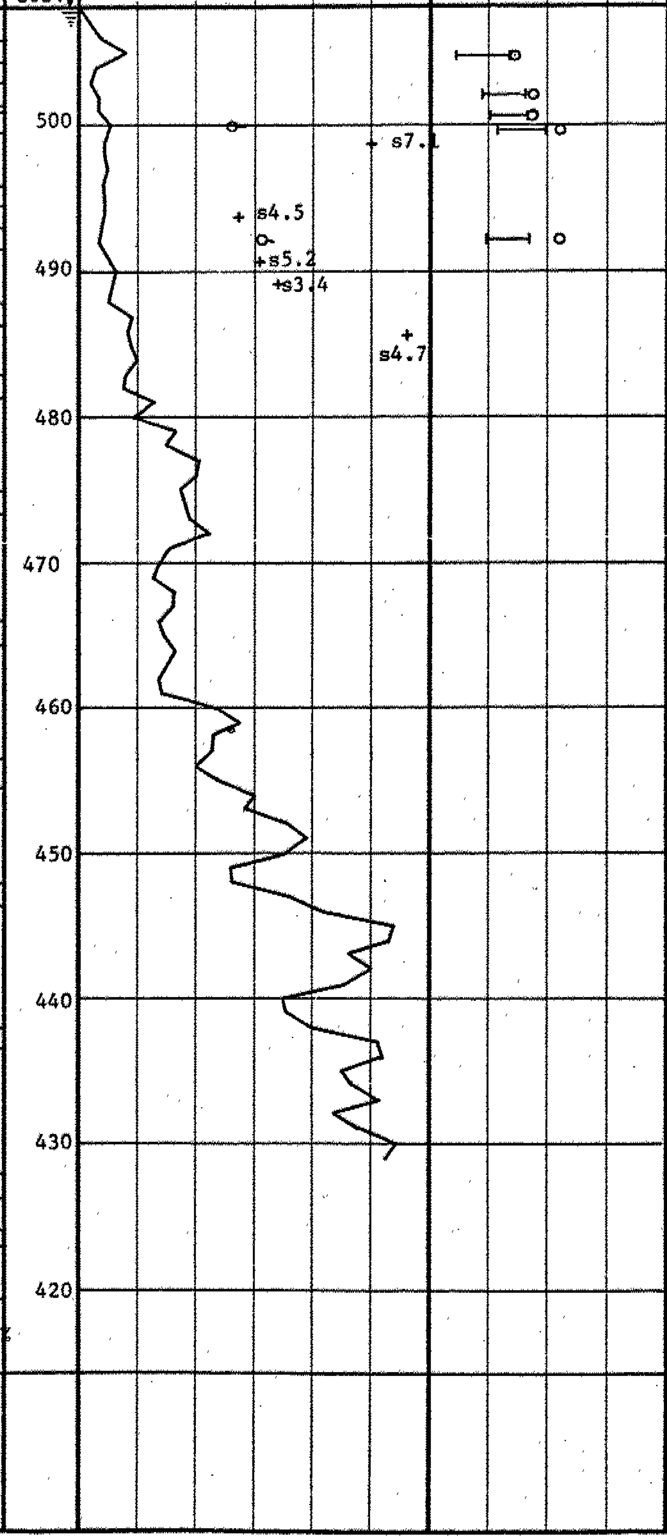
DATUM Geodetic BOREHOLE TYPE H.S.Augers, BW Casing, BXL Core, and Cone Test CHECKED BY

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 400 800 1200 1600 2000

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

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	UNIT WEIGHT γ PCF	REMARKS (Head 514. ∇; Head 508.5 ∇) % GR SA SI CL
508.0	Ground Level					
0.0 / 505.0	Sand (fine), trace of silt. Very Loose	1	SS	3		0 93 (7
3.0	Clayey silt to silty clay, traces of sand brittle, sensitive, grey, some pockets and seams of silt. Stiff	2	SS	6		0 5 66 2
		3	TW	PH	110	
		4	SS	3		
		5	TW	PH	116	
		6	SS	3		
482.3		7	SS	2		0 21 74 5
25.7	Silty sand to sandy silt, trace of clay. Loose to Very Loose, with random layers of clayey silt to silt with some sand.	8	SS	4		
		9	SS	8		A= 0 88 (12) B= 0 33 59 8
		10	SS	7		
		11	SS	6		0 82 (18)
						Art. Water 438.0 ∇
431.0		12	SS	10		
77.0	Sand & gravel, some silt. Very Dense	13	SS	68		43 45 (12)
419.5		14	SS	140		Art. Water 419.5 ∇
88.5 / 414.5	Gneiss Bedrock Sound	15	RC BXL	Rec 100%		Flow=2.5gpm
93.5	End of Borehole					



20 / 15 ○ 5 / 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

ENGINEERING SERVICES BRANCH–GEOTECHNICAL OFFICE–SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 3

WP 2-67-04 LOCATION Sta. 281 + 92 ¢ ORIGINATED BY HS

DIST 9 HWY 17N BORING DATE November 6, 1975 COMPILED BY HS

DATUM Geodetic BOREHOLE TYPE H.S. Augers CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	SHEAR STRENGTH	WATER CONTENT %	UNIT WEIGHT γ PCF	REMARKS % GR SA SI CL
513.6	Ground Level									
0.0	Sand (very fine), trace to some silt.		1	SS	11	510				0 89 (11)
	Compact		2	SS	12					0 97 (3)
503.6			3	SS	22					
10.0	Clayey silt, traces of sand - brittle, sensitive, grey, occasional pockets and seams of silt, sand.		4	SS	4		s4.0			
			5	TW	PH	500			109	e_o=1.25 c_c=0.45
			6	TW	PH		s4.2			0 4 64 32
	Stiff to Very Stiff		7	SS	9	490				
483.6			8	TW	PH				118	
30.0	Silty sand to sandy silt, trace to some clay.		9	SS	6	480				A= 0 18 69 13 B= 0 69 (31)
	Loose to Very Loose		10	SS	3					
472.1			11	SS	4					
41.5	End of Borehole									

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH: o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 400 800 1200 1600 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

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

ENGINEERING SERVICES BRANCH–GEOTECHNICAL OFFICE–SOIL MECHANICS SECTION

RECORD OF BOREHOLE № 4

WP 2-67-04	LOCATION Sta. 278 + 70 20' Rt.	ORIGINATED BY HS
DIST 9 HWY 17N	BORING DATE November 6 to November 11, 1975	COMPILED BY HS
DATUM Geodetic	BOREHOLE TYPE H.S. Augers	CHECKED BY [signature]

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER / ELEV	UNIT WEIGHT γ PCF	REMARKS % GR SA SI CL
510.4	Ground Level						
0.0	topsoil				510		
	Sand (fine to very fine), trace of silt.	1	SS	20			
	Compact	2	SS	13			0 93 (7)
500.9	Loose	3	SS	7			0 97 (3)
9.5	Clayey silt, trace to some sand - brittle, sensitive, grey	4	SS	5	500	112	
		5	TW	PH			
		6	TW	PH			
	silty sand	7	SS	6			
	sand	8	TW	PH	490	114	
	occasional pockets and seams of silt, sand.	9	SS	11			0 26 58 16
477.4	Stiff to Very Stiff	10	SS	4	480		
33.0	Silty sand to sandy silt, trace of clay	11	SS	2			0 23 68 9
	with random layers of clayey silt to silt with some sand	12	SS	5	470		
	Loose to Very Loose	13	SS	7	460		0 78 (22)
	Compact	14	SS	14	450		
		15	SS	20	440		0 80 (20)
421.9		16	SS	22	430		
88.5 / 419.2	Sand & gravel, some silt. Very Dense	17	SS	120/3" bouncing	420		
91.2	End of Borehole Probable Bedrock						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 400 800 1200 1600 2000

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

+ 83.0

>+

Artesian water not observed, but augers were pulled out soon after reaching the probable surface of the bedrock.

20
15 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 5

WP 2-67-04 | LOCATION Sta. 278 + 45 22' Lt. | ORIGINATED BY HS
DIST 9 HWY 17N | BORING DATE November 11 to November 13, 1975 | COMPILED BY HS
DATUM Geodetic | BOREHOLE TYPE H.S. Augers, BW Casing, BXL Core | CHECKED BY

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
509.0	Ground Level
0.0	topsoil
	Sand (fine), trace of silt. Compact
500.7 / 8.3	Clayey silt to silty clay with varying amounts of sand - brittle, sensitive, grey, some pockets and seams of silt. Stiff
478.5 / 30.5	Silty sand to sandy silt, trace to some clay with random layers of clayey silt to silt with some sand. Loose
428.2 / 80.8	Sand & gravel, some silt. Very Dense
425.0 / 84.0	Gneiss Bedrock Sound
420.0 / 89.0	End of Borehole

SAMPLES

NUMBER	TYPE	'N' VALUES	SHEAR STRENGTH	UNIT WEIGHT γ PCF	REMARKS GR SA SI CL
1	SS	11			
2	SS	14			0 95 (5)
3	SS	8			
4	SS	6	+s6.3		0 31 61 8
5	TW	PH	+s4.4, +s3.6	106	0 15 59 26; e_o=1.46; C_c=0.81
6	TW	PH	+s7.3, >+		
7	TW	PH		112	e_o=1.13; C_c=0.70
8	SS	7			
9	SS	1/18"			0 81 (19)
10	SS	7			0 41 45 14
11	SS	10			0 38 61 1
12	SS	56			0 21 58 21
13	RC BXL	Rec 100%			

GROUND WATER: at ground surface.

REMARKS: ∇ 513.5; ∇ 512.0; Art Water ∇ 425.0; Art.Water ∇ 420.0; Flow=3.5gpm

ELEV scale: 500, 490, 480, 470, 460, 450, 440, 430, 420

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 400 800 1200 1600 2000

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

15 ◇ 5 (20) % STRAIN AT FAILURE

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

ENGINEERING SERVICES BRANCH–GEOTECHNICAL OFFICE–SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 6

WP 2-67-04 LOCATION Sta. 276 + 71 ℄ ORIGINATED BY HS

DIST 9 HWY 17N BORING DATE November 13-14, 1975 COMPILED BY HS

DATUM Geodetic BOREHOLE TYPE H.S. Augers CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	SHEAR STRENGTH	UNIT WEIGHT γ PCF	REMARKS % GR SA SI CL
510.5	Ground Level				510			
0.0	topsoil							
	Sand (fine to very fine) trace of silt.	1	SS	21				0 97 (3)
503.5	Loose to Compact	2	SS	9				0 95 (5)
7.0	Clayey silt to silty clay, trace to some sand – brittle, sensitive, grey	3	SS	3	500	s3.5		
	silt	4	TW	PM		s9.0		
	some pockets & seams of silt. Occasional thin layers of sandy silt.	5	TW	PH	490	s6.7, s6.7	110	e_0=1.02 c_c=0.37
		6	TW	PH		s6.0		
		7	SS	5	480	s8.7, s4.7		
	Stiff to Very Stiff	8	TW	PH		s5.5	113	
467.5		9	SS	4	470			0 16 67 17
43.0	Silty sand to sandy silt, trace to some clay. Loose							A: 0 78 (22)
464.0		10	SS	6				B: 0 31 56 13
46.5	End of Borehole							

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 400 800 1200 1600 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

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

ENGINEERING SERVICES BRANCH–GEOTECHNICAL OFFICE–SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 7

WP 2-67-04 LOCATION Sta. 280 + 16 22' Lt. ORIGINATED BY HS

DIST 9 HWY 17N BORING DATE November 17 to November 19, 1975 COMPILED BY HS

DATUM Geodetic BOREHOLE TYPE H.S. Augers, BW Casing CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT —W_L PLASTIC LIMIT —W_P WATER CONTENT —W / WATER CONTENT %	UNIT WEIGHT γ	REMARKS GR SA SI CL %
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	ELEV				
510.2	Ground Level					510				Head 514.2
	Straight Augering (No Sampling) 0-85' Easy to fairly easy augering					500, 490, 480, 470, 460, 450, 440, 430				Piezometer 425.2 P7
425.2 / 85.0	Augering-Very difficult going-probable sand & gravel. Very Dense									
421.2 / 89.0	End of Borehole Probable Bedrock					420				

20 15 -○- 5 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

ENGINEERING SERVICES BRANCH - GEOTECHNICAL OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 8

WP 2-67-04 LOCATION Sta. 279 + 90 18' Rt. ORIGINATED BY HS

DIST 9 HWY 17N BORING DATE November 19 to November 20, 1975 COMPILED BY HS

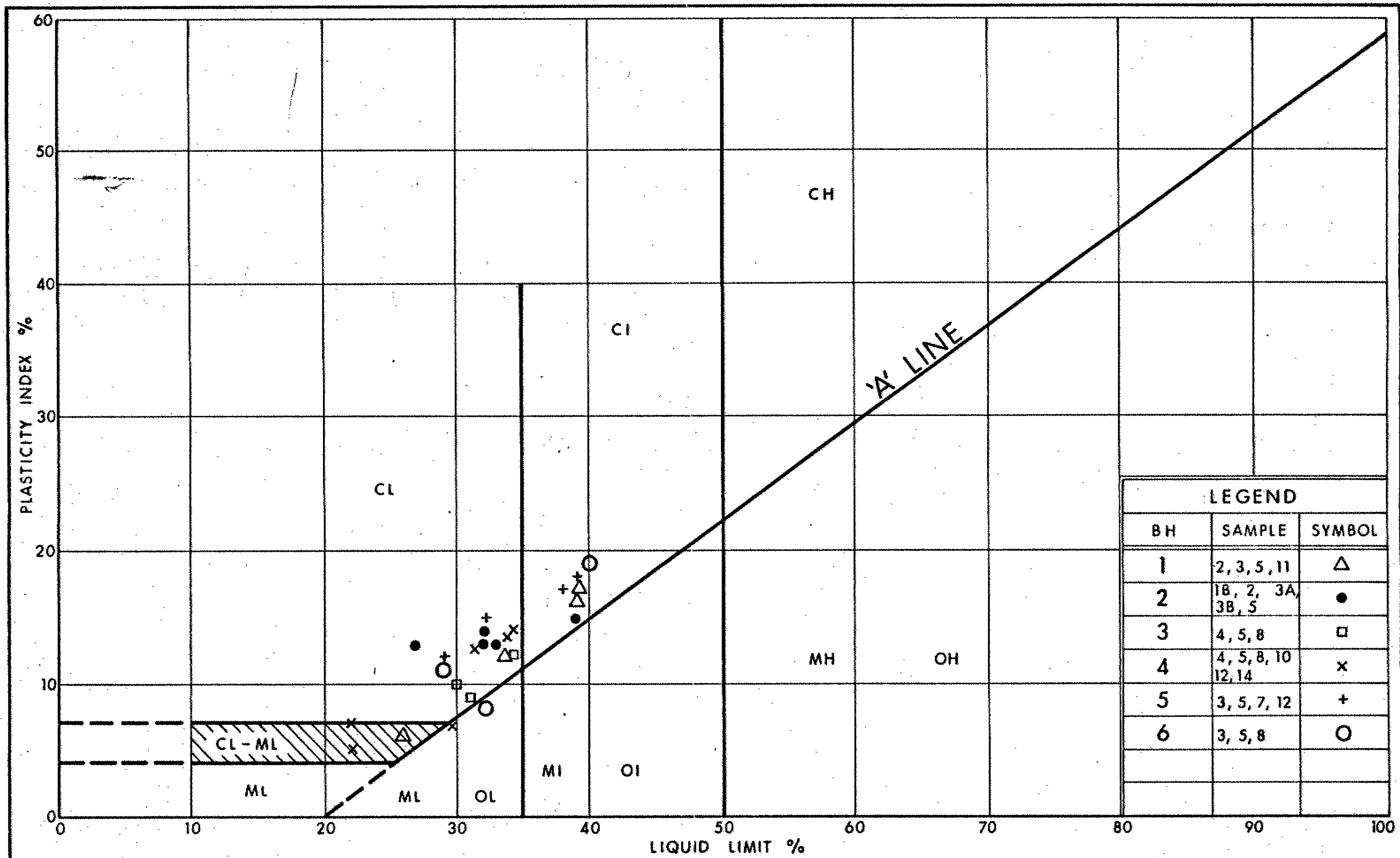
DATUM Geodetic BOREHOLE TYPE H.S. Augers, BW Casing CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT W_L; PLASTIC LIMIT W_P; WATER CONTENT W; WATER CONTENT %	UNIT WEIGHT γ	REMARKS Head 513.0 % GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	ELEV				
510.0	Ground Level									
0.0	Straight Augering (No Sampling)					500				
						490				
						480				Piezometer 475.0 P8
475.0 / 35.0	End of Borehole									

15 20 5 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Oct 75, FF-S-21

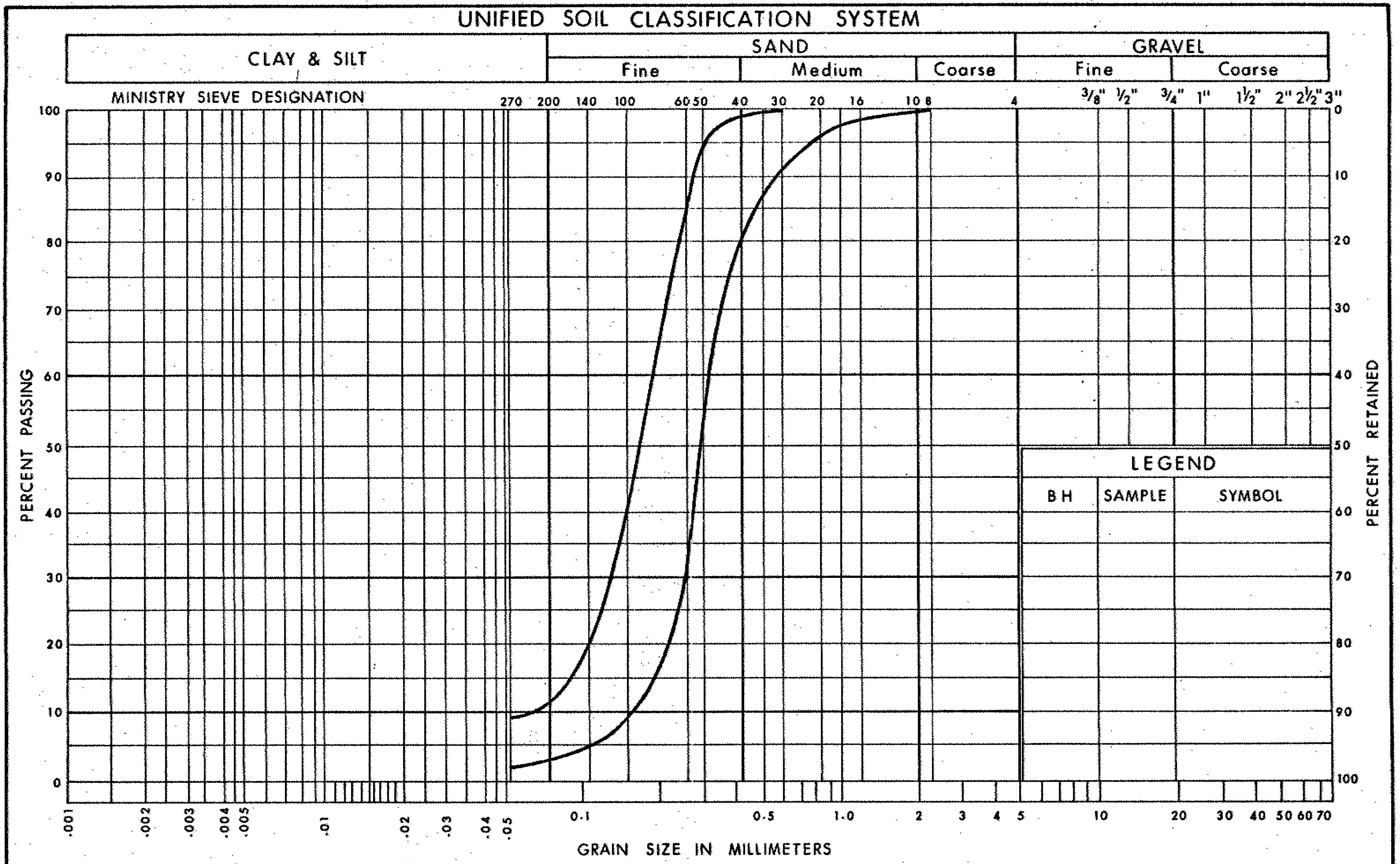


BH	SAMPLE	SYMBOL
1	2, 3, 5, 11	△
2	1B, 2, 3A, 3B, 5	●
3	4, 5, 8	□
4	4, 5, 8, 10 12, 14	×
5	3, 5, 7, 12	+
6	3, 5, 8	O

Ministry of Transportation and Communications
Ontario
ENGINEERING SERVICES BRANCH

PLASTICITY CHART
CLAYEY SILT TO SILTY CLAY
TRACE TO SOME SAND

FIG No 1
W P 2 - 67 - 04

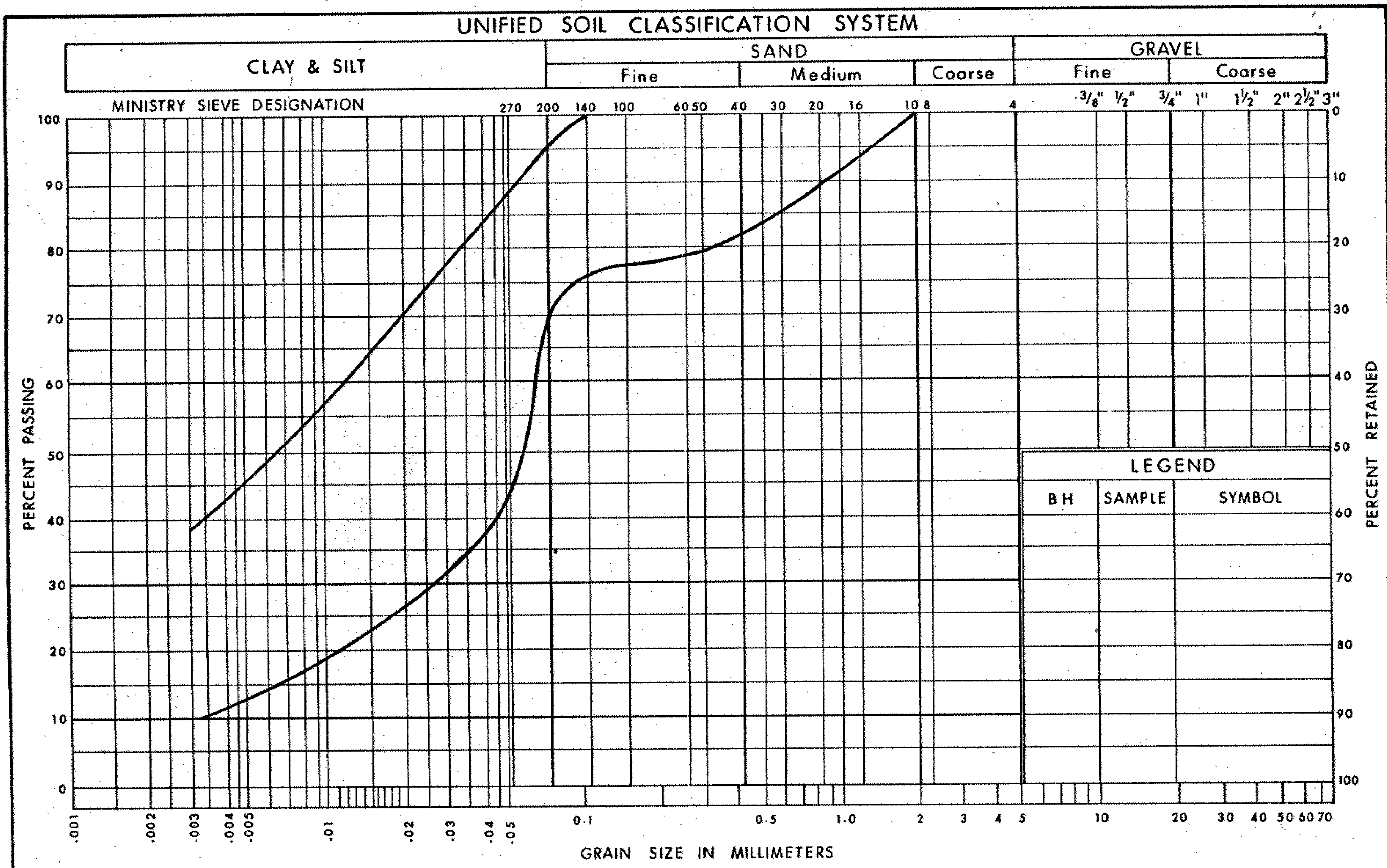


Ministry of Transportation and Communications
Ontario
ENGINEERING SERVICES BRANCH

GRAIN SIZE DISTRIBUTION
SAND (Fine to V. Fine)
TRACE OF SILT

FIG No 2

W P 2 -67 - 04



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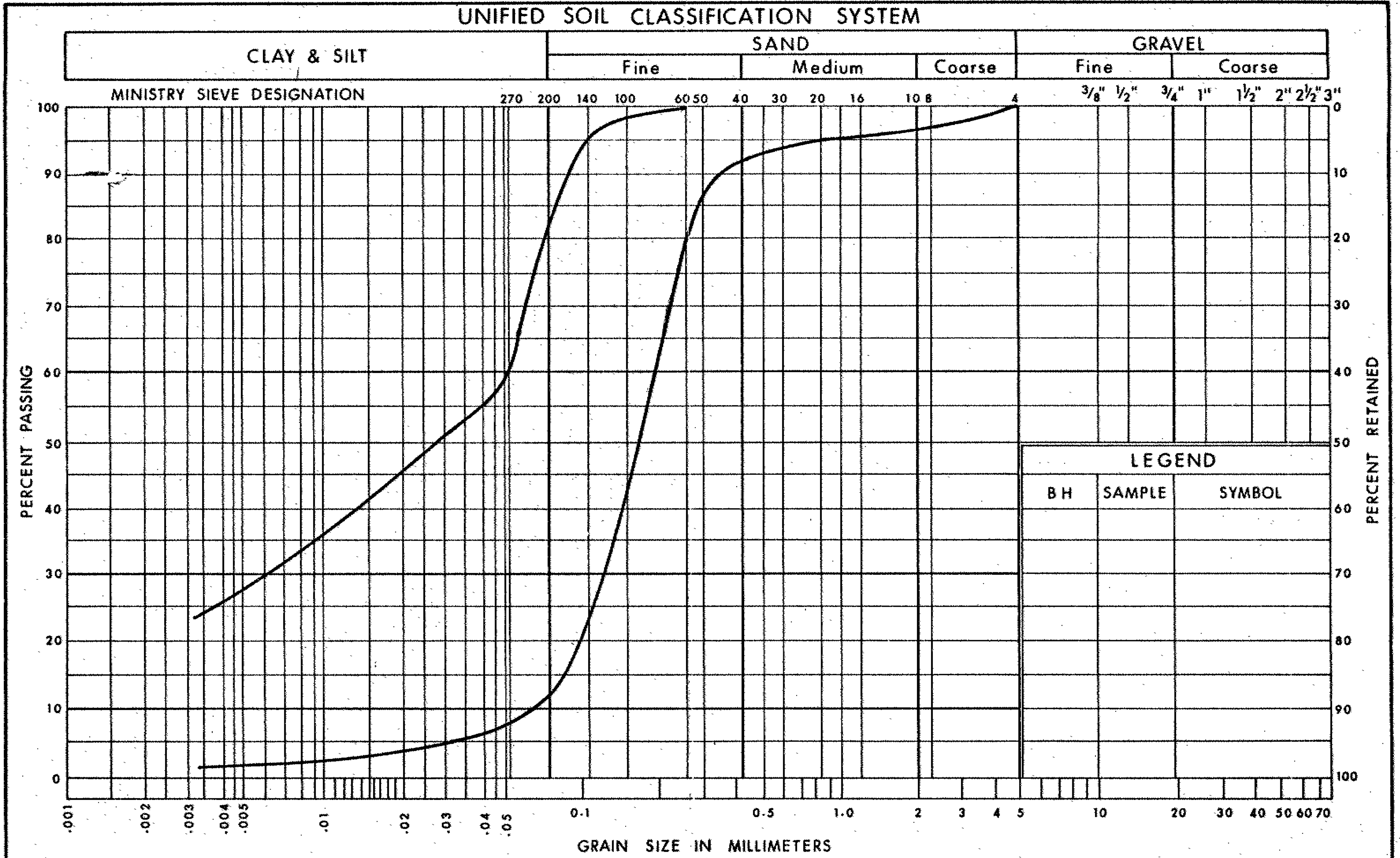
GRAIN SIZE DISTRIBUTION
CLAYEY SILT TO SILTY CLAY
TRACE TO SOME SAND

FIG No 3

W P 2 - 67 - 04



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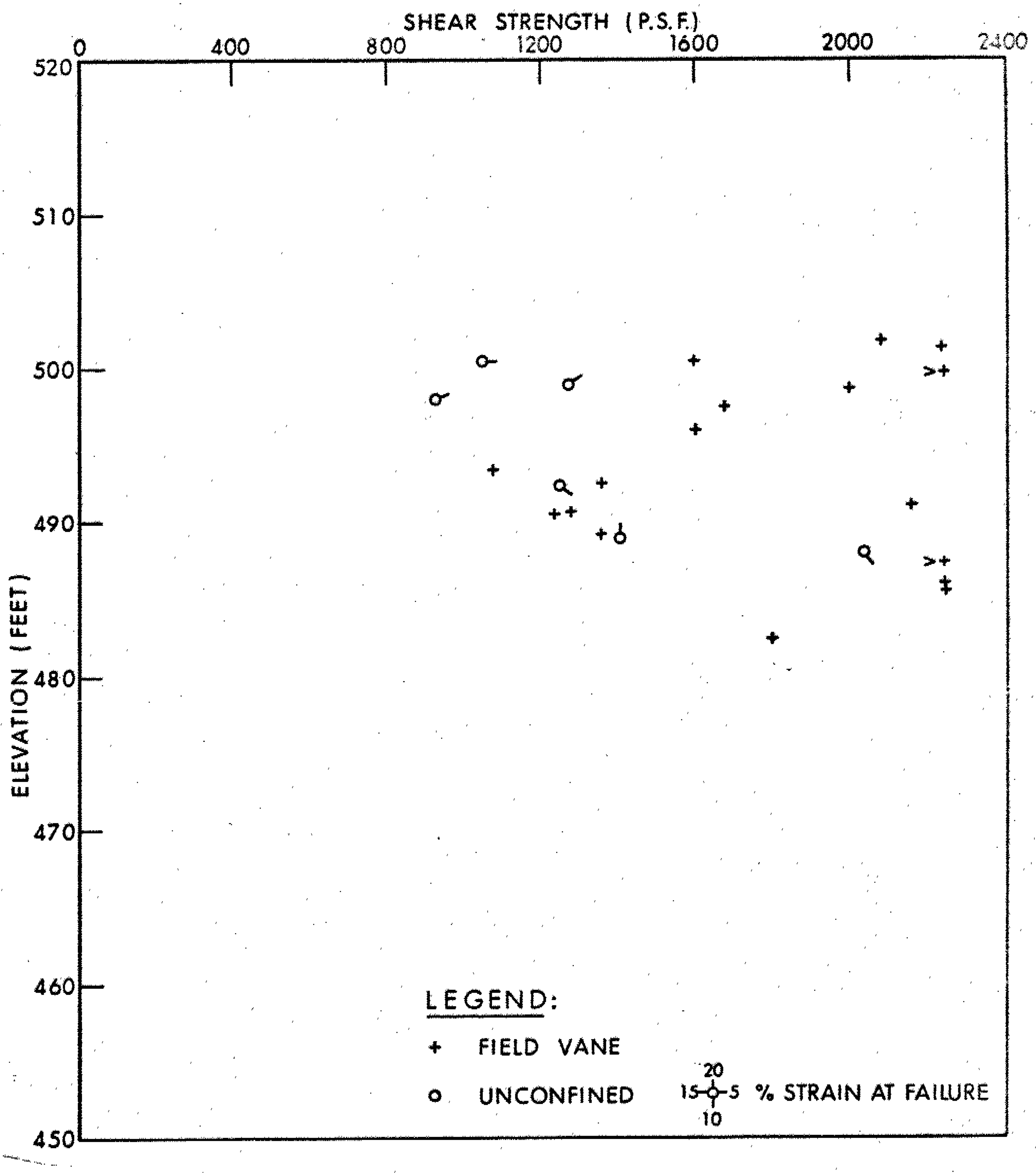
GRAIN SIZE DISTRIBUTION
SILTY SAND TO SANDY SILT
TRACE TO SOME CLAY

FIG No 4

W P 2 - 67 - 04

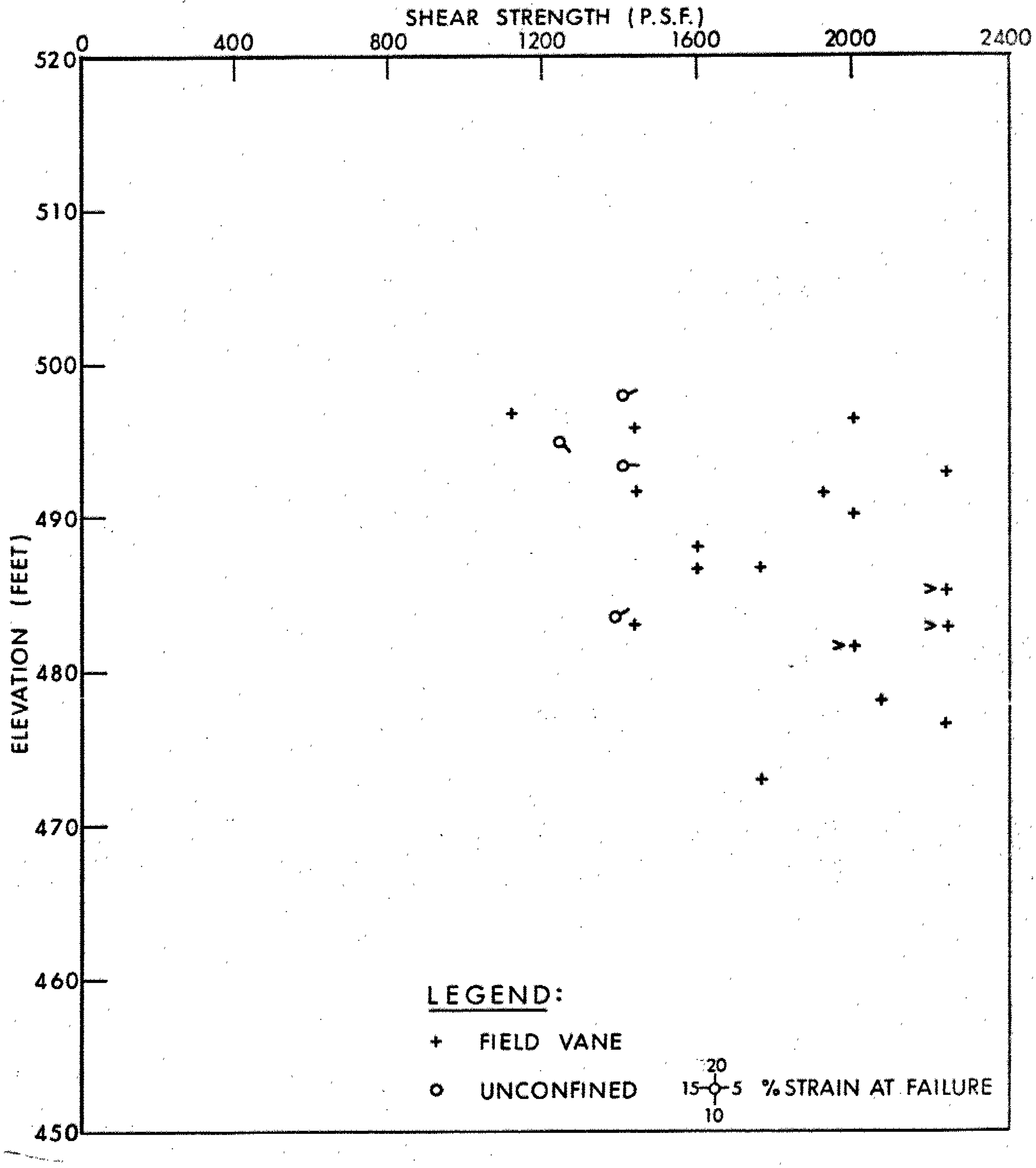


PLOT OF SHEAR STRENGTH Vs. ELEVATION

South Approach Area (B.H.'S. 1, 2 & 3)

Fig. 5

W.P. 2-67-04



PLOT OF SHEAR STRENGTH Vs. ELEVATION

North Approach Area (B.H.'S 4, 5 & 6)

Fig. 6

W.P. 2-67-04

FF - 71

VOID RATIO – PRESSURE CURVES

W.P. NO. 2 - 67 - 04

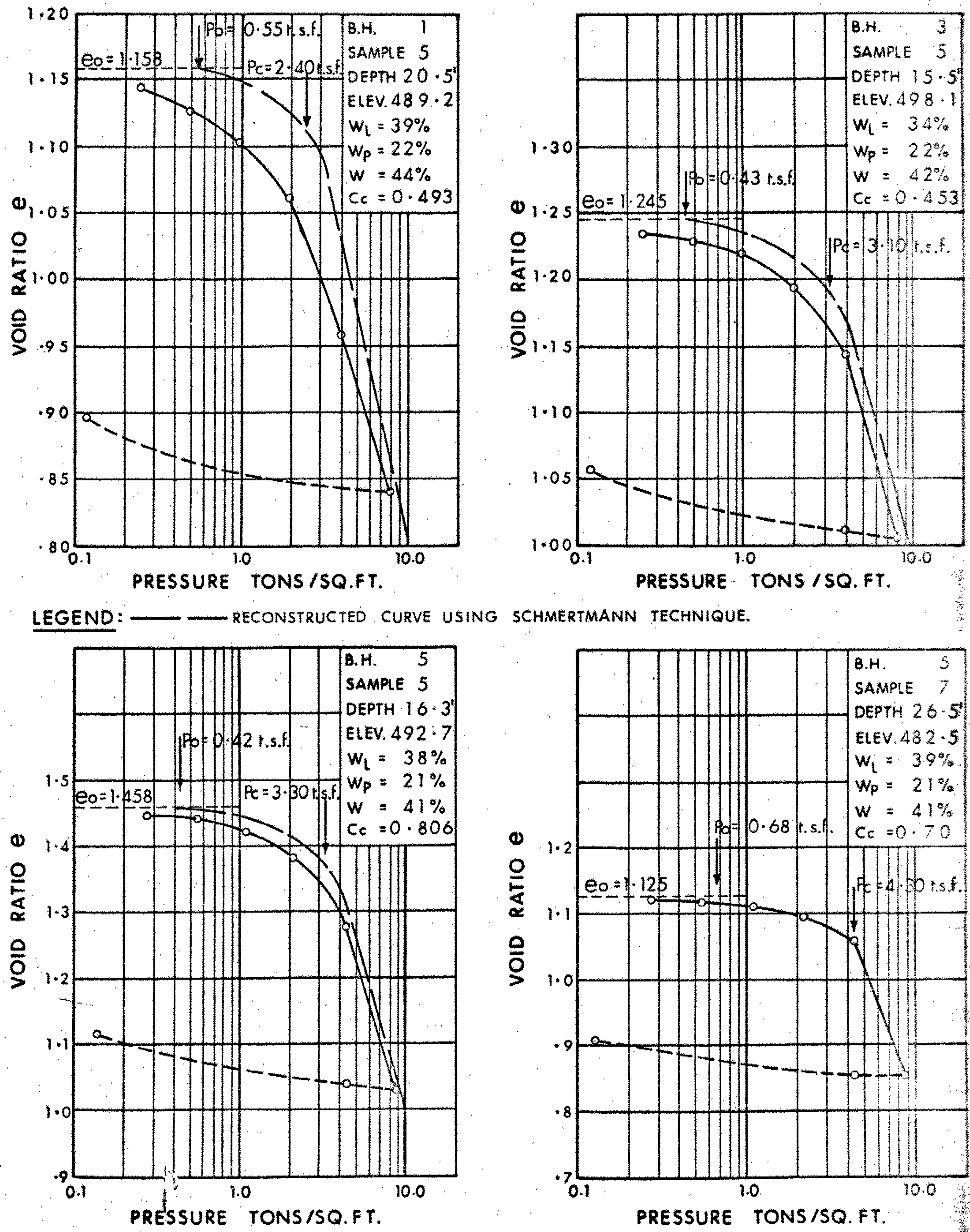


FIG. 7

FORM OB-MT-113
JANUARY 1970

DEPARTMENT OF HIGHWAYS ONTARIO

DIAMOND DRILL RECORD

HOLE NO. ______ SHEET NO. ______

PROPERTY W.P. 2-67-04
LOCATION CNR & Hwy. 17 Crossing - Pembroke

LATITUDE
DEPARTURE
BEARING

DIP
90°

TOTAL FOOTAGE

ELEV. COLLAR
DATUM
DATE STARTED
DATE COMPLETED
DRILLED BY
LOGGED BY

FOOTAGE FROM	FOOTAGE TO	FORMATION	SAMPLE NUMBER			REMARKS
		HOLE #2				
88'6"	93'6"	Gneiss, hard, pink to grey colour, fine to med. texture,				lineation at 30°
		vertical joint throughout entire core section.				
		HOLE #5				
84'	89'	Gneiss, hard, pink to grey colour, fine to med. texture,				lineation at 30°
		6" pegmatitic gneiss at 85'6"				6" broken core at 88'
		HOLE #1				
96'10"	98'10"	Gneiss, hard, black colour, med to coarse texture, high %				lineation at 30°
		of biotite mica				inclined joint fracture 80°
98'10"	101'4"	Gneiss, hard, pink colour, coarse texture				lineation at 30°
						12" broken core at 100'

DATE OF EXAMINATION November 26, 1975

B.K. Glassford

FF-A-24(a) (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.
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 "		

FF-A-24(b) (Rev. Jcn.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

  (c', ϕ':) IN TERMS OF EFFECTIVE STRESS $\tau_f = c' + \sigma' \tan \phi'$
- c_u APPARENT COHESION
- ϕ_u APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION

  (c_u, ϕ_u:) 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 $2{\cdot}7183$
- $\log_e a$ 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

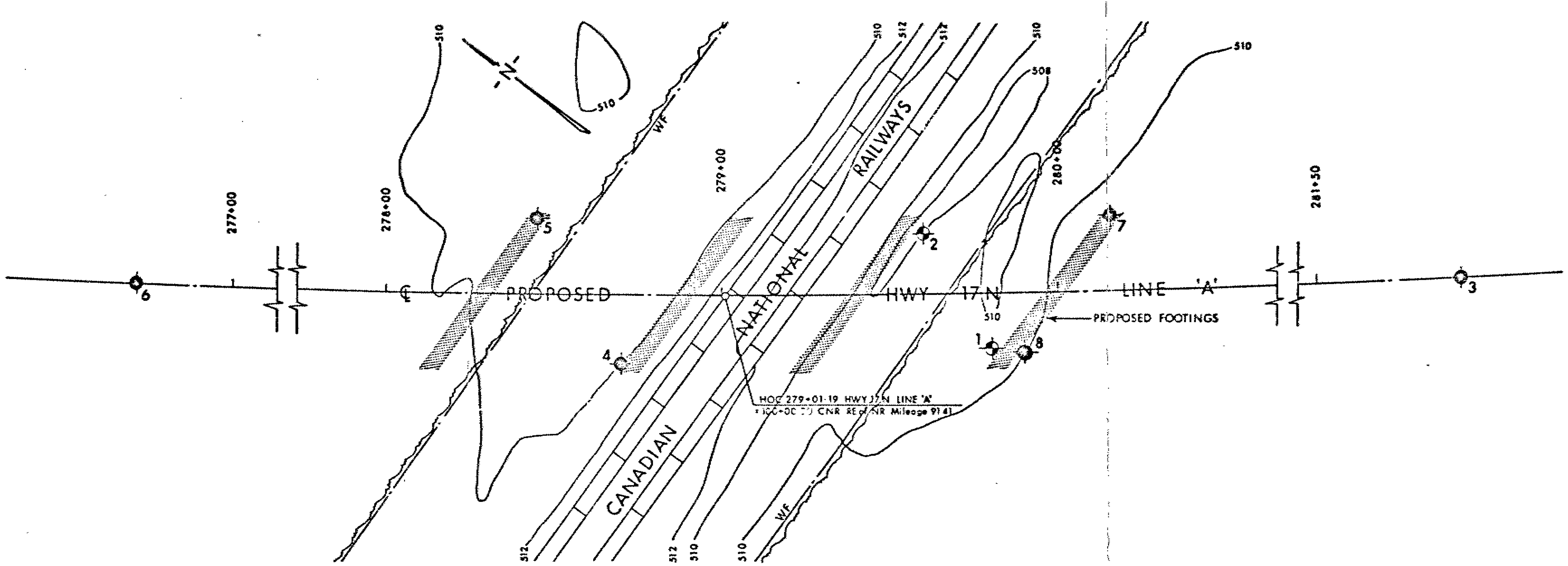
- d DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
- δ ANGLE OF WALL FRICTION
- K *DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
- K_o COEFFICIENT OF EARTH PRESSURE AT REST

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

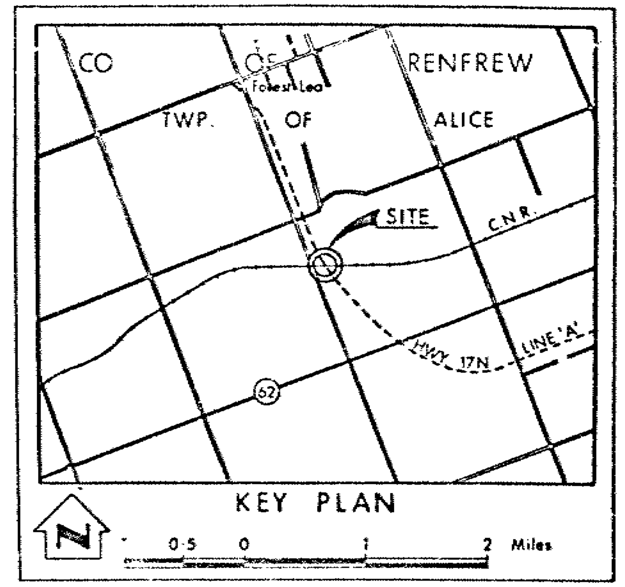
SLOPES

- 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 40 FT.



LEGEND

Bore Hole

Dynamic Cone Penetration Resistance Test
B/F CONE - Blows/Ft. Cone Test (350 ft lbs. energy/blow)

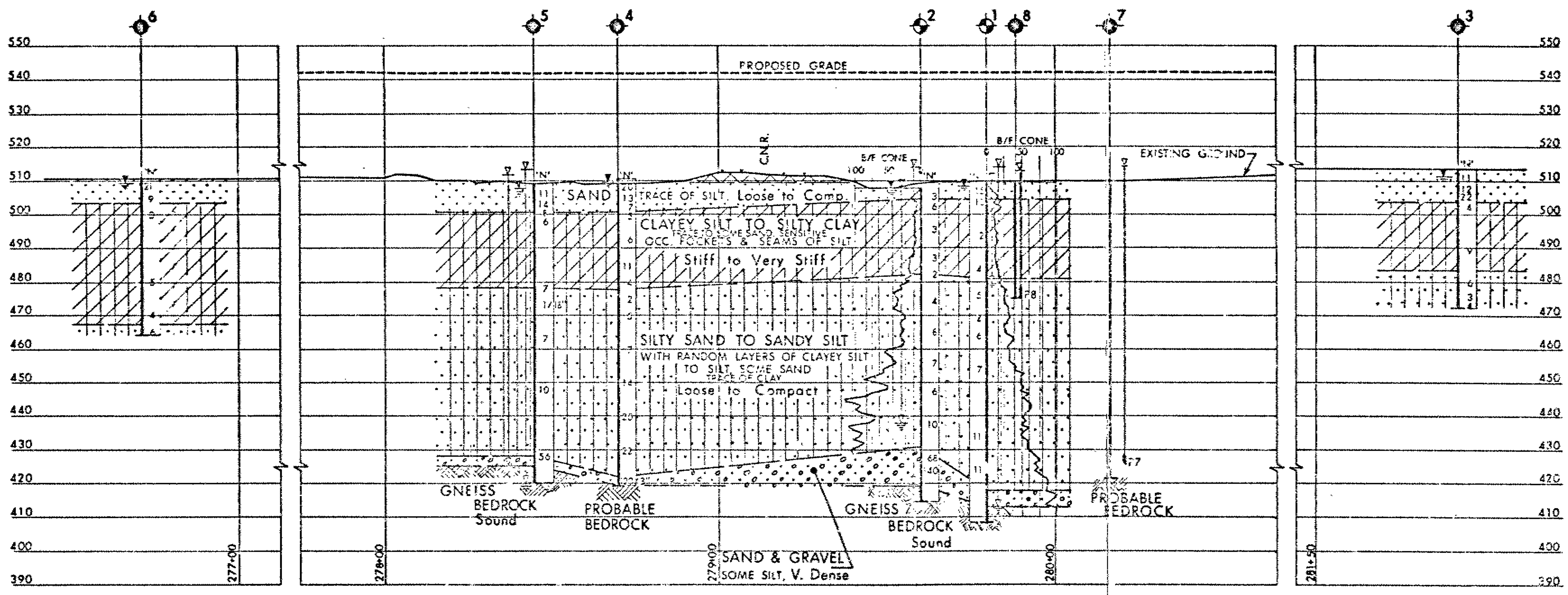
Bore Hole & Cone Test

Water Levels established at time of field investigation, OCT. & NOV. 75

Head / Encountered — ARTESIAN WATER

Piezometer

NO.	ELEVATION	STATION	OFFSET
1	509 · 7	279 + 80	17' RT.
2	508 · 0	279 + 60	18' LT.
3	513 · 6	281 + 92	℄
4	510 · 4	278 + 70	20' RT.
5	509 · 0	278 + 45	22' LT.
6	510 · 5	276 + 71	℄
7	510 · 2	280 + 16	22' LT.
8	510 · 0	279 + 90	18' RT.



℄ PROFILE - LINE 'A'

SCALE 20 10 0 20 40 FT.

REF. NO. E-5265-1 Aug/75

— NOTE —

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

NOTE FOR CONTRACT DOCUMENT

The complete foundation investigation report for this structure may be examined at the Structural Office and Foundations Office, Downsview, and at the OTTAWA District Office.

REVISIONS		
DATE	BY	DESCRIPTION

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

CANADIAN NATIONAL RAILWAYS

HIGHWAY NO 17 N LINE 'A' DIST NO 9
CO RENFREW
TWP ALICE LOT 26 CON 12

BORE HOLE LOCATIONS & SOIL STRATA

SUBMD H.S. CHECKED H.S.	WP NO 2-67-04	DRAWING NO. 26704-A
DRAWN Sect CHECKED	WO NO	
DATE 30 DECEMBER 1975	SITE NO 29-162	BRIDGE DRAWING NO.
APPROVED	CONT NO	