

G.I.-30 SEPT. 1976

GEOCRES No. 40J7-6DIST. 1 REGION W.P. No. 257-66-03CONT. No. W. O. No. STR. SITE No. HWY. No. EC Row ExpwyLOCATION E.C. Row ExpwyWINDSORNo. of PAGES - OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. REMARKS:

## MEMORANDUM

*Extra*  
GEOCREES No -  
4057-6

TO: Mr. A. P. Watt (2)  
Regional Bridge Planning Engr.  
Southwestern Region  
London, Ontario

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

ATTENTION:

DATE: December 14, 1971

OUR FILE REF.

IN REPLY TO DEC 22 1971

SUBJECT:

FOUNDATION INVESTIGATION REPORT  
For

The Proposed E.C.Row Expressway  
and Chesapeake and Ohio Railway  
Crossing, Lot 97 - Con. 2  
City of Windsor - County Essex  
District #1 - (Chatham)

W.P. 71-11114 - W.P. 257-66-05.

Attached, we are forwarding to you our detailed foundation investigation report on the subsoil conditions existing at the above structure 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.

AGS:mt  
Attach.

ls/

*K. L. Suter*  
A. G. Stermac  
Principal Foundation Engineer

cc: Messrs: D. W. Farren  
B. R. Davis  
A. Rutka  
W. A. Zonnenberg  
F. C. Brown  
B. J. Giroux  
J. R. Roy  
G. A. Wrong  
B. A. Singh

Foundation Files  
Documents

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# FOUNDATION INVESTIGATION REPORT

For

The Proposed E.C.Row Expressway  
and Chesapeake and Ohio Railway  
Crossing - Lot 97 - Con. 2  
City of Windsor - County Essex  
District #1 - (Chatham)  
W.O. 71-11114 - W.P. 257-66-05

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## 1. INTRODUCTION:

A request for a foundation investigation at the crossing of the Proposed E. C. Row Expressway and C & O Railway was received from Mr. A. P. Watt, Regional Bridge Planning Engineer, in a memorandum dated October 12, 1971

A preliminary foundation investigation covering this area. was carried out in April 1968 (68-11015-3). A more detailed field investigation was subsequently carried out to determine the subsoil conditions existing at the site.

This report contains the results of both investigations and our recommendations pertaining to the design of the proposed structure foundations and approach embankments.

## 2. DESCRIPTION OF THE SITE:

The site of the proposed overpass structure is situated in the eastern part of the City of Windsor, approx. 0.2 miles east of the intersection of the existing E. C. Row Blvd. and Walker Rd.

The surrounding terrain, with the exceptions of the approx. 2 ft. high railway embankment and the approx. 3 ft. deep drainage ditches, is flat and cultivated farmland.

Physiographically the site is located in the region referred to as the St. Clair Clay Plain.

### 3. FIELD AND LABORATORY INVESTIGATION PROCEDURES:

A total of 7 samples boreholes and 12 dynamic cone penetration tests was carried out during the course of the field work. Boring was achieved by means of bombardier mounted continuous flight auger machines, and conventional diamond drilling equipment adapted for soil sampling purposes. During the field work, disturbed samples were obtained by means of a standard split-spoon sampler; the energy used in driving it conformed to the requirements of the Standard Penetration Test. 'Undisturbed' samples were recovered using 2 inch I.D. shelly tubes which were pushed into the soil hydraulically or by hand. Where possible, field vane tests were carried out at elevations generally 12 inches below sample depths.

Dynamic cone penetration tests were carried out adjacent to each borehole with the exception of B.H.#67, and also at 6 other locations. Driving energy to advance the cone was 350 ft.-lbs. per blow.

The bedrock was proved at two borehole locations using AXT Rock Coring equipment.

All boreholes were surveyed in the field by personnel from London Region Engineering Surveys Section. The locations and elevations of the borings are shown on Drawing No. 71-11114A which accompanies this report.

All samples were visually examined and classified at the site as well as in the laboratory. Following this inspection laboratory tests were carried out on selected samples to determine the following physical properties:

- Atterberg Limits
- Moisture Content
- Grain-Size Distribution
- Undrained Shear Strength
- Bulk Density

The test results are summarized on the record of borehole sheets contained in the Appendix of this report.

#### 4. SOIL TYPES AND SOIL CONDITIONS:

##### 4.1) General:

Generally uniform subsoil conditions were found to prevail over the site area. The subsoil consists of a deep deposit of cohesive soil, followed by limestone bedrock. The boundaries between different deposits are shown on the record of borehole sheets attached to the Appendix. The estimated stratigraphical profile of Drawing 71-11114A is based upon this information.

From ground level downward, the various strata are described in some detail with regard to soil types and soil properties, as follows:

##### 4.2) Clayey silt with sand and traces of gravel:

This deposit was intersected in all borings and extends from immediately below the ground surface, down to the bedrock surface for a minimum depth of 130 feet. The material in the deposit consists of clayey silt with sand and traces of gravel. A plot of Plasticity Index versus Liquid Limit (Fig.1) shows the points to fall within the CL zone. In some boreholes relatively thin layers of granular soils were found to occur within the main deposit.

A highly overconsolidated zone, due to desiccation and/or weathering, with a thickness ranging from 7 to 10 feet, was found to extend from the upper surface of the stratum. This zone is brown in color due to oxidation and apart from the upper 3 to 6 feet (frost affected zone) has a very stiff to hard consistency: 'N' values ranged from 21 to 83 blows per foot. Based on the Standard Penetration Test results only, the undrained shear strength of this desiccated zone is estimated to be in the order of 2,500 PSF to 10,000 PSF. Below the desiccated layers the color of the soil is grey and the consistency ranges somewhat randomly from stiff to hard. For design purposes the following undrained shear strength values are suggested:

Ground Level - El. 611	2,000 PSF
El. 611 - El. 603	5,000 PSF
603 - 590	2,500 PSF
590 - 570	1,750 PSF
570 - 540	1,250 PSF
540 - 487	1,500 PSF

Physical properties of the overall deposit, as determined from field and laboratory tests, are as follows:

Natural Moisture Content: (%)	9.9 to 18.4
Liquid Limit: (%)	19.6 to 30.4
Plastic Limit: (%)	11.7 to 16.7
Bulk Density: (PCF)	131.0 to 137.5
Unconfined Shear Strength: (PSF)	988 to 2620
Field Vane Test: (PSF)	960 to 2000 +
Sensitivity:	1.2 to 1.9
'N' Value: (Blows/Ft.)	10 to 83

Typical grain-size distribution curves are included in the Appendix of this report (Fig.2).

#### 4.3) Limestone Bedrock:

Bedrock at this site was found to consist of generally ~~solid~~ <sup>Sound</sup> limestone at el. 487 (B.H.'s 68 & 70).

#### 5.) GROUNDWATER CONDITIONS:

The following groundwater levels were observed during the field investigation:

B.H.#67	El: Not Established
68	Not Established
70	Not Established
73	Not Established
77	Not Established
139	El: 602.8
144	595.9

It is pointed out that the foregoing quoted figures may not represent the true groundwater levels, due to the relatively impermeable nature of the subsoil and the short duration of the field work.

## 6.) DISCUSSION AND RECOMMENDATIONS:

### 6.1) General:

It is proposed to build a three-span (44'-53'-44') twin-structure overpass at the crossing of the E. C. Row Expressway and the C. & O. Railway. The proposed profile grade of the E. C. Row Expressway will be approximately 27 ft. above the existing C. & O. Railway grade of elevation 620±.

As described in the previous paragraphs of this report, the subsoil at the site consists of a deep deposit of clayey silt with sand and traces of gravel, underlain by limestone bedrock. The upper 14 to 17 feet of the deposit is a very stiff to hard desiccated surface crust. Below this depth the undrained shear strength of the material decreases. The desiccated surface crust appears to be suitable for spread footing type foundations.

Because of the compressible nature of the subsoil, it is inevitable that consolidation settlements will occur over a longterm period due to the imposed loads of structure and embankment. Past experience, however, indicates that these settlements will be of a minor nature.

### 6.2) Foundations:

#### a) Spread Footings in Original Ground:

The entire structure may be supported on spread footings placed within the very stiff to hard desiccated zone of the subsoil between El. 611 and El. 603. A safe net pressure of 3.5 TSF may be assumed for design purposes.

The desiccated zone is susceptible to softening on contact with water, therefore, it is recommended that the base of the footing excavations be protected by concrete working slab, immediately on exposure.



All foundations should be protected against frost action by at least 4 feet of earth cover. No dewatering problems are anticipated.

The estimated maximum settlement will be in the order of 1.0 to 1.5 inches under the pier footings.

b) Spread Footings on Compacted Fill:

As an alternative, the abutments may be supported on spread footings placed on well compacted, suitable granular material within the approach fills. A safe design load of 2.0 TSF may be assumed. The granular material should consist of G.B.C. Class 'A' and should be fully compacted according to the current Standards. A detailed construction scheme is outlined on Figure 3 of the Appendix.

c) Perched Abutments on Short Piles:

As a second alternative, the abutments may be constructed within the approach fills and supported on short piles driven through the fill and some 10.0 feet into the original ground. In the case of 12-3/4" O.D. and 1/4" thick wall steel tube piles, a safe design load of 25 tons per pile may be used.

It should be pointed out, that this latter proposal is based on experience with similar structures and similar subsoil conditions in the general area.

Regardless of which method is adopted (a, b or c) the structure should be built to accommodate the 3.0 to 3.5 inches differential settlement between the abutments and piers.

d) End-Bearing Piles:

As another alternative, the abutments and piers may be supported on steel H-piles driven to bedrock. The maximum allowable load for the particular steel section may be assumed for design purposes.

### 6.3) Approach Embankments:

The shear strength of the subsoil is such that it will be able to safely support the 31 ft. high approach embankments constructed with 2:1 side slopes. The fill should consist of well compacted acceptable material. Care should be taken to ensure that no bouldery fill is placed within the approaches through which piles have to be driven, and it is recommended that this portion of the fill contain no larger grain sizes than 3 inches.

Based on performance of structures and embankments built in the same general area and under somewhat similar subsoil conditions, it is estimated that a maximum settlement of 4 to 5 inches will take place over a long period of time under the fill at the abutment location. To minimize the effect of differential settlements between the abutments and pier footings, it is recommended that the approach embankments be built in advance of the structure for as long a period as possible. The topsoil and the soft surficial material should be removed in accordance with the pertinent Standards within the construction area.

### 7. MISCELLANEOUS:

The field investigation was carried out during the period April 1 to 4, 1968, and November 17 to 19, 1971, under the supervision of Mr. A. Prakash and Mr. P. Payer, Project Foundation Engineers.

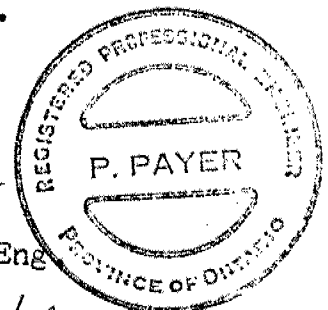
Equipment was owned and operated by Dominion Soil Investigation Ltd. and Master Soil Investigation Ltd.

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

December 10, 1971

*P. Payer*  
P. Payer, P. Eng.

*K. G. Selby*  
K. G. Selby, P. Eng.



"APPENDIX"

DEPARTMENT OF HIGHWAYS- ONTARIO  
MATERIALS & TESTING OFFICE

## RECORD OF BOREHOLE No. 67 (68-F-15-3) FOUNDATION SECTION

JOB 71-11111 LOCATION Co-ords. 100, 006N; 77, 491E ORIGINATED BY A.P.  
 W.P. 257-66-05 BORING DATE April 1 & 2, 1968 COMPILED BY AMS  
 DATUM Geodetic BOREHOLE TYPE Cont. flight auger CHECKED BY K.K.

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT — $w_L$			BULK DENSITY	REMARKS		
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT					PLASTIC LIMIT — $w_p$						
						SHEAR STRENGTH P.S.F.					WATER CONTENT — $w$							
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE					10 20 30							
						500	1000	1500	2000	2500								
616.7	Ground level																	
0.0	Clayey silt with sand traces of gravel.  Hard to Very Stiff Stiff		1	SS	12	610									134	2 29 40 29		
			2	SS	38													
			3	SS														
			4	SS		600												
			5	SS														
			6	SS														
			7	TW	PH	590												
			8	TW	PH													
			9	TW	PH													
			10	TW	PH	580												
			11	SS	14													
			12	TW	PH													
563.7	End of Borehole					560									134			
53.0																		

# OVERSIZE DRAWING

DEPARTMENT OF HIGHWAYS- ONTARIO  
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 73 ( 68-F-15-3) FOUNDATION SECTION

JOB 71-11114 LOCATION Co-ords. 100, 111N; 77, 660E. ORIGINATED BY GEH  
W.P. 257-66-05 BORING DATE April 3, 1968 COMPILED BY AMS  
DATUM Geodetic BOREHOLE TYPE Continuous Flight Auger CHECKED BY KK

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT ——— $w_L$ PLASTIC LIMIT ——— $w_P$ WATER CONTENT ——— $w$			BULK DENSITY $\gamma$ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		20	40	60	80	100	$w_P$	$w$	$w_L$		
616.3	Ground level															
0.0																
			1	SS	59	610										
			2	SS	83											
			3	SS	34	600										
			4	SS	22											
			5	SS	20	590										
			6	SS	19											
			7	TW	PH	580										
			8	TW	PH											
			9	TW	PH	570										
563.3																
53.0	End of Borehole					560										

FOUNDATION SECTION

JOB 71-11134 LOCATION Co-ords. 100, 064N; 77, 550E

ORIGINATED BY AP

W.P. 257-66-05 BORING DATE April 4, 1968

COMPILED BY AMS

DATUM Geodetic BOREHOLE TYPE Cont. Flight Auger

CHECKED BY NK

[illegible]

FOUNDATION SECTION

CHECKED BY K. K.

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION	RESISTANCE	LIQUID LIMIT	PLASTIC LIMIT	WATER CONTENT	BULK DENSITY $\gamma$	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT	20 40 60 80 100	W <sub>p</sub> W <sub>L</sub>	W <sub>p</sub> W <sub>L</sub>	W		
618.8	Ground level												
0.0	Probable clayey silt with sand and trace of gravel					610							
604.8	End of Conehole					600							



DEPARTMENT OF HIGHWAYS- ONTARIO  
MATERIALS & TESTING OFFICE

## RECORD OF BOREHOLE No. 135

FOUNDATION SECTION

JOB 71-11114 LOCATION Co-ords. 100, 008N; 77, 556E. ORIGINATED BY P.P.  
 W.P. 257-66-05 BORING DATE Nov. 17, 1971 COMPILED BY P.P.  
 DATUM Geodetic BOREHOLE TYPE Cone Test Only CHECKED BY K/K

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT — $w_L$ PLASTIC LIMIT — $w_p$ WATER CONTENT — $w$		BULK DENSITY $\gamma$ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		BLOWS/FOOT	SHEAR STRENGTH P.S.F.	WATER CONTENT %			
618.7	Ground level											
0.0	Probable clayey silt with sand and trace of gravel											
603.7												
15.0	End of Conehole											

(Roadway)

610

600





DEPARTMENT OF HIGHWAYS- ONTARIO MATERIALS & TESTING OFFICE		<b>RECORD OF BOREHOLE No. 140</b>		FOUNDATION SECTION	
JOB <u>71-11114</u>		LOCATION <u>Co-ords. 100, 124N; 77, 487E</u>		ORIGINATED BY <u>P.P.</u>	
W.P. <u>257-66-05</u>		BORING DATE <u>Nov. 18, 1971</u>		COMPILED BY <u>P.P.</u>	
DATUM <u>Geodetic</u>		BOREHOLE TYPE <u>Cone Test Only</u>		CHECKED BY <u>R.K.</u>	

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT ——— $w_L$				BULK DENSITY $\gamma$ P.C.F.	REMARKS		
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT					PLASTIC LIMIT ——— $w_p$							
							20	40	60	80	100	WATER CONTENT ——— $w$							
							SHEAR STRENGTH P.S.F.					WATER CONTENT %							
							<div><div></div> UNCONFINED</div> <div><div></div> QUICK TRIAXIAL</div>					<div><div></div> FIELD VANE</div> <div><div></div> LAB. VANE</div>				<div><div><div><math>w_p</math></div><div><math>w</math></div><div><math>w_L</math></div></div></div>			
616.4	Ground level																		
0.0	Probable clayey silt with sand and trace of gravel	<div></div>				610													
604.4																			
12.0	End of Cone Test					600													

FOUNDATION SECTION

JOB 71-11114 LOCATION Co-ords. 100, 006N; 77, 653E ORIGINATED BY P.P.  
W.P. 257-66-05 BORING DATE Nov. 18, 1971 COMPILED BY P.P.  
DATUM Geodetic BOREHOLE TYPE Cone Test Only CHECKED BY N.K.

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT — $w_L$		BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT	SHEAR STRENGTH P.S.F.	PLASTIC LIMIT — $w_p$	WATER CONTENT — $w$		
616.7	Ground level											
0.0	Probable clayey silt with sand and trace of gravel					610						
601.7												
15.0	End of Cone Test					600						

DEPARTMENT OF HIGHWAYS- ONTARIO  
MATERIALS & TESTING OFFICE

## RECORD OF BOREHOLE No. 144

FOUNDATION SECTION

JOB 71-11114 LOCATION Co-ords. 100, 063N; 77, 640E ORIGINATED BY P.P.  
 W.P. 257-66-05 BORING DATE Nov. 18, 1971 COMPILED BY P.P.  
 DATUM Geodetic BOREHOLE TYPE Cont. flight auger CHECKED BY N.K.

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT — $w_L$		BULK DENSITY $\gamma$ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT	RESISTANCE	PLASTIC LIMIT — $w_p$	WATER CONTENT — $w$		
616.4	Ground level											
0.0	Clayey silt with sand and trace of gravel Stiff to Hard		1	SS	21							
			2	SS	72							
			3	SS	44							
			4	SS	46							
			5	SS	70							
			6	SS	39							
			7	SS	21							
			8	SS	18							
			9	SS	19							
			10	SS	15							
594.9												
21.5	End of Borehole					590						

W.L. 595.9'

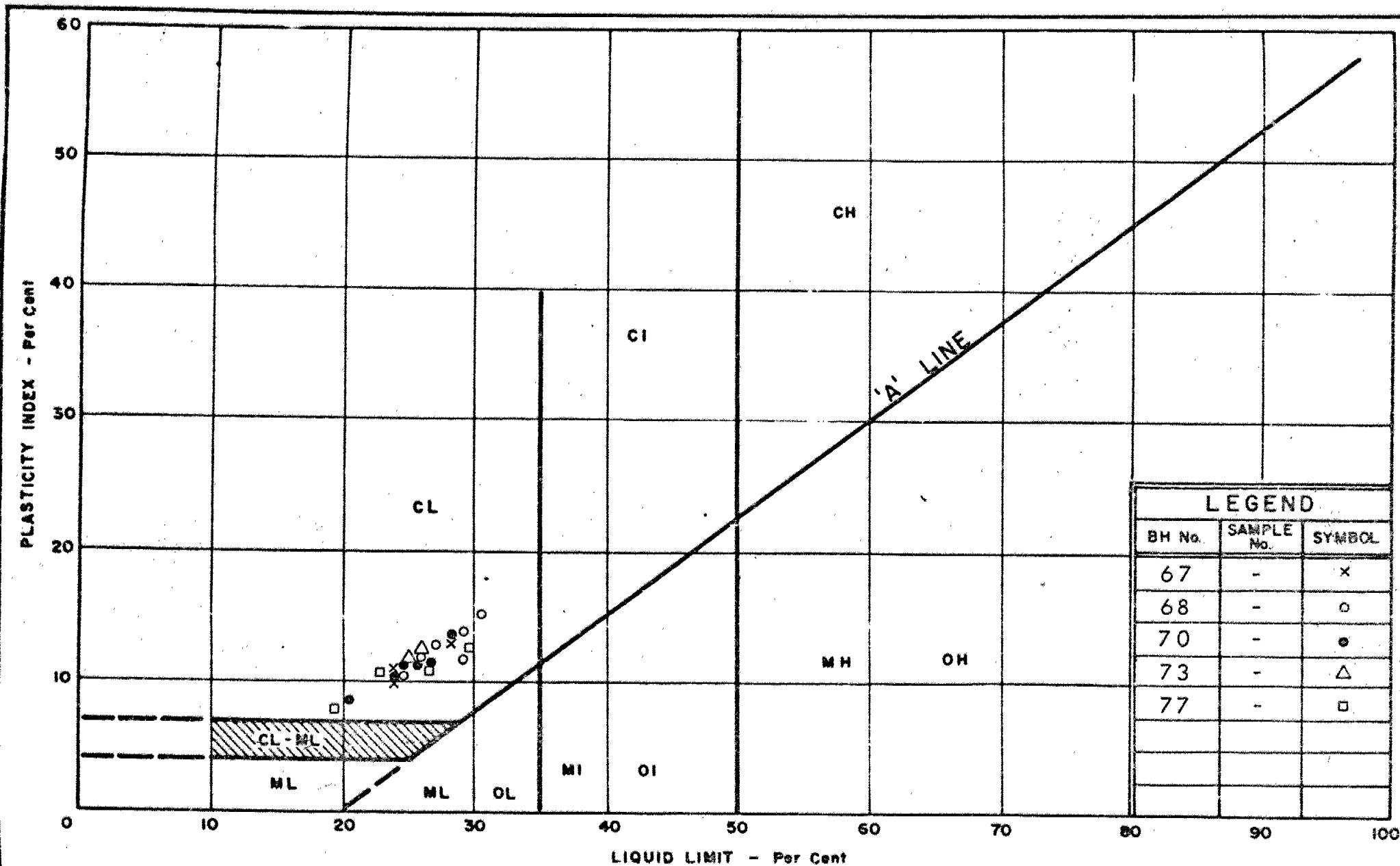
DEPARTMENT OF HIGHWAYS- ONTARIO  
MATERIALS & TESTING OFFICE

## RECORD OF BOREHOLE No. 145

FOUNDATION SECTION

JOB 71-11114 LOCATION Co-ords. 100, 102N; 77, 632E ORIGINATED BY P.P.  
W.P. 257-66-05 BORING DATE Nov. 19, 1971 COMPILED BY P.P.  
DATUM Geodetic BOREHOLE TYPE Cone Test Only CHECKED BY NK

[illegible]



DEPARTMENT OF HIGHWAYS  
MATERIALS and  
TESTING  
DIVISION

# PLASTICITY CHART CLAYEY SILT

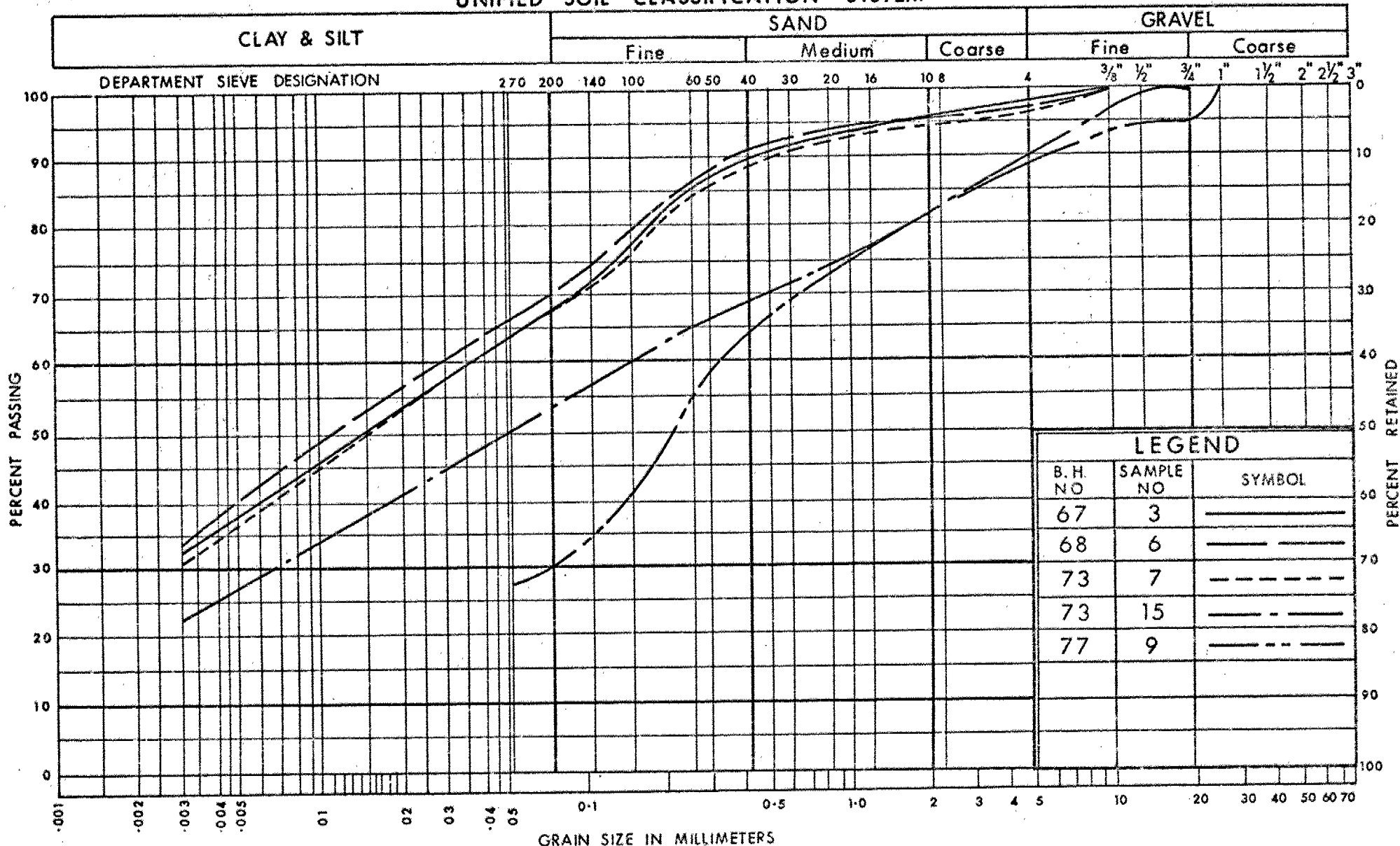
WP. No. 257 - 66 - 05

JOB No. 71 - 11114

FIG. 1



# UNIFIED SOIL CLASSIFICATION SYSTEM



## GRAIN SIZE DISTRIBUTION CLAYEY SILT

DEPARTMENT  
OF  
TRANSPORTATION AND COMMUNICATIONS

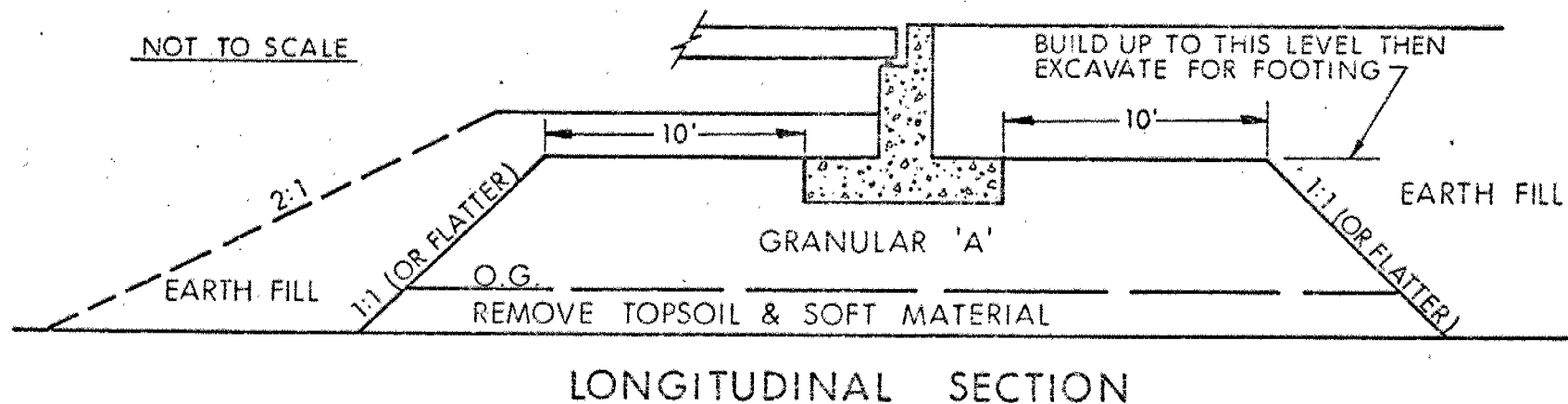
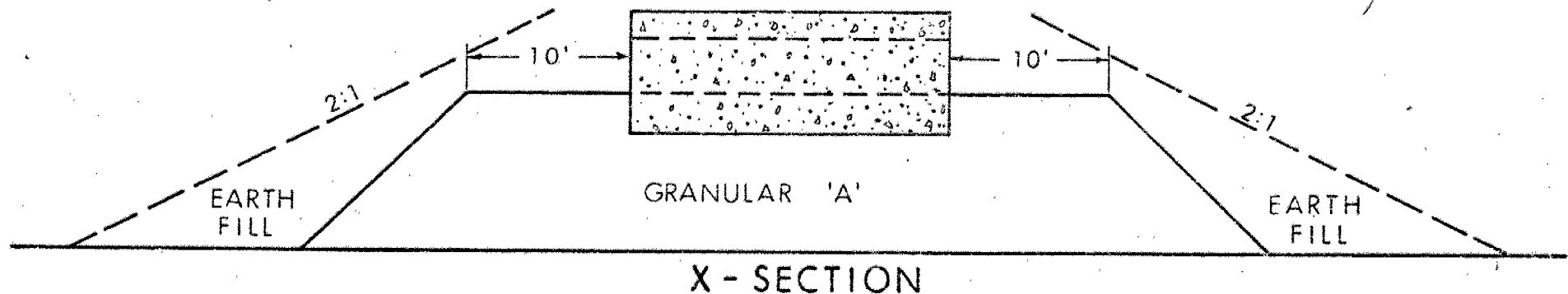
DESIGN SERVICES  
BRANCH

W.P. No. 257-66-05

JOB No. 71-11114

FIG. 2

## ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE



### NOTES

- 1 - REMOVE TOPSOIL &/OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A'.
- 2 - PLACE GRANULAR 'A' TO TOP OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT D. T. C. STANDARDS.
- 3 - EXCAVATE COMPACTED GRANULAR 'A' MATERIAL FOR FOOTING.

## ABBREVIATIONS USED IN THIS REPORT

### PENETRATION RESISTANCE

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

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

### DESCRIPTION OF SOIL

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

<u>CONSISTENCY</u>	<u>'N' BLOWS / FT.</u>	<u>c LB. / SQ. FT.</u>	<u>DENSENESS</u>	<u>'N' BLOWS / FT.</u>
VERY SOFT	0 - 2	0 - 250	VERY LOOSE	0 - 4
SOFT	2 - 4	250 - 500	LOOSE	4 - 10
FIRM	4 - 8	500 - 1000	COMPACT	10 - 30
STIFF	8 - 15	1000 - 2000	DENSE	30 - 50
VERY STIFF	15 - 30	2000 - 4000	VERY DENSE	> 50
HARD	> 30	> 4000		

### TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.B.	SCRAPER BUCKET SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE
S.T.	SLOTTED TUBE SAMPLE		
	P.H. SAMPLE ADVANCED HYDRAULICALLY		
	P.M. SAMPLE ADVANCED MANUALLY		

### SOIL TESTS

Q <sub>u</sub>	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V.	FIELD VANE
Q <sub>cu</sub>	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Q <sub>d</sub>	DRAINED TRIAXIAL	S	SENSITIVITY

## ABBREVIATIONS USED IN THIS REPORT

### SOIL PROPERTIES

$\gamma$	UNIT WEIGHT OF SOIL (BULK DENSITY)
$\gamma_s$	UNIT WEIGHT OF SOLID PARTICLES
$\gamma_w$	UNIT WEIGHT OF WATER
$\gamma_d$	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
$\gamma'$	UNIT WEIGHT OF SUBMERGED SOIL
G	SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$
e	VOID RATIO
n	POROSITY
w	WATER CONTENT
$S_r$	DEGREE OF SATURATION
$w_L$	LIQUID LIMIT
$w_p$	PLASTIC LIMIT
$I_p$	PLASTICITY INDEX
s	SHRINKAGE LIMIT
$I_L$	LIQUIDITY INDEX $= \frac{w - w_p}{I_p}$
$I_C$	CONSISTENCY INDEX $= \frac{w_L - w}{I_p}$
$e_{max}$	VOID RATIO IN LOOSEST STATE
$e_{min}$	VOID RATIO IN DENSEST STATE
$I_D$	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
	RELATIVE DENSITY $D_r$ IS ALSO USED
h	HYDRAULIC HEAD OR POTENTIAL
q	RATE OF DISCHARGE
v	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
k	COEFFICIENT OF PERMEABILITY
j	SEEPAGE FORCE PER UNIT VOLUME
$m_v$	COEFFICIENT OF VOLUME CHANGE $= \frac{-\Delta e}{(1+e)\Delta\sigma}$
$c_v$	COEFFICIENT OF CONSOLIDATION
$C_c$	COMPRESSION INDEX $= \frac{\Delta e}{\Delta \log_{10} \sigma}$
$T_v$	TIME FACTOR $= \frac{c_v t}{d^2}$ (d, DRAINAGE PATH)
U	DEGREE OF CONSOLIDATION
$\tau_f$	SHEAR STRENGTH
$c'$	EFFECTIVE COHESION INTERCEPT
$\phi'$	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
$c_u$	APPARENT COHESION
$\phi_u$	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
$\mu$	COEFFICIENT OF FRICTION
$S_t$	SENSITIVITY

### GENERAL

$\pi$	= 3.1416
e	BASE OF NATURAL LOGARITHMS 2.7183
$\log_e a$ OR $\ln a$	NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$	LOGARITHM OF a TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

### STRESS AND STRAIN

u	PORE PRESSURE
$\sigma$	NORMAL STRESS
$\bar{\sigma}$	NORMAL EFFECTIVE STRESS ( $\bar{\sigma}$ IS ALSO USED)
$\tau$	SHEAR STRESS
$\epsilon$	LINEAR STRAIN
$\gamma$	SHEAR STRAIN
$\nu$	POISSON'S RATIO ( $\mu$ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
$\eta$	COEFFICIENT OF VISCOSITY

### EARTH PRESSURE

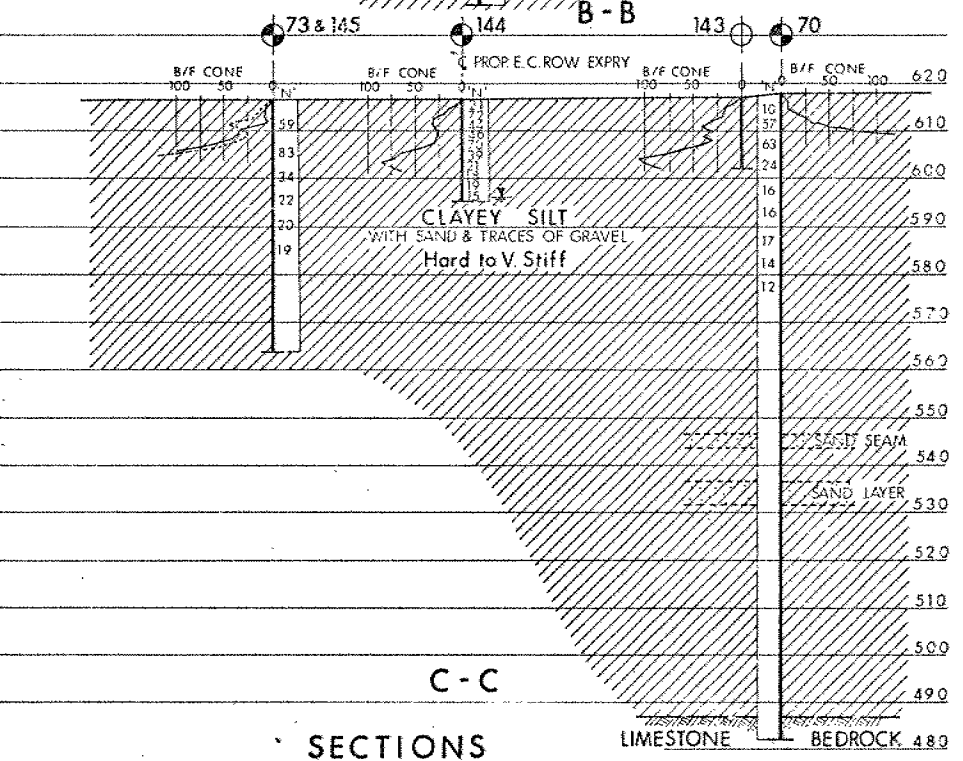
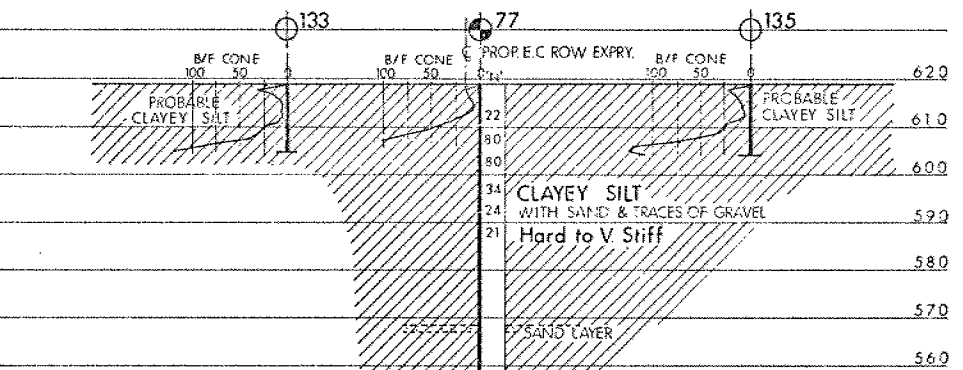
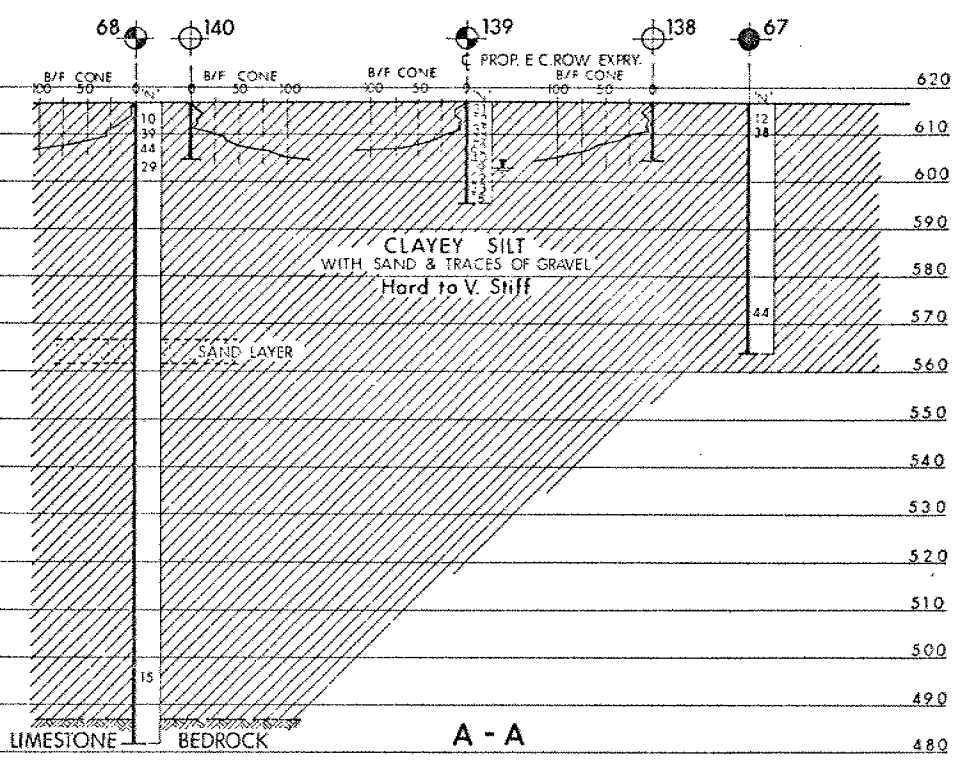
d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
$\delta$	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
$K_0$	COEFFICIENT OF EARTH PRESSURE AT REST

### 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

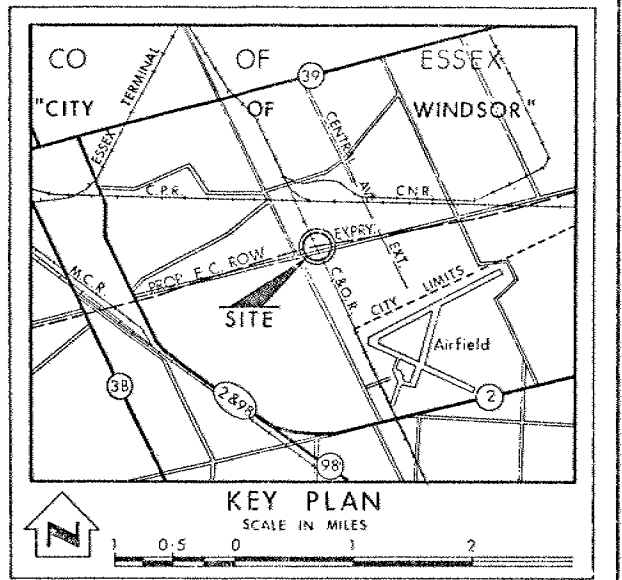
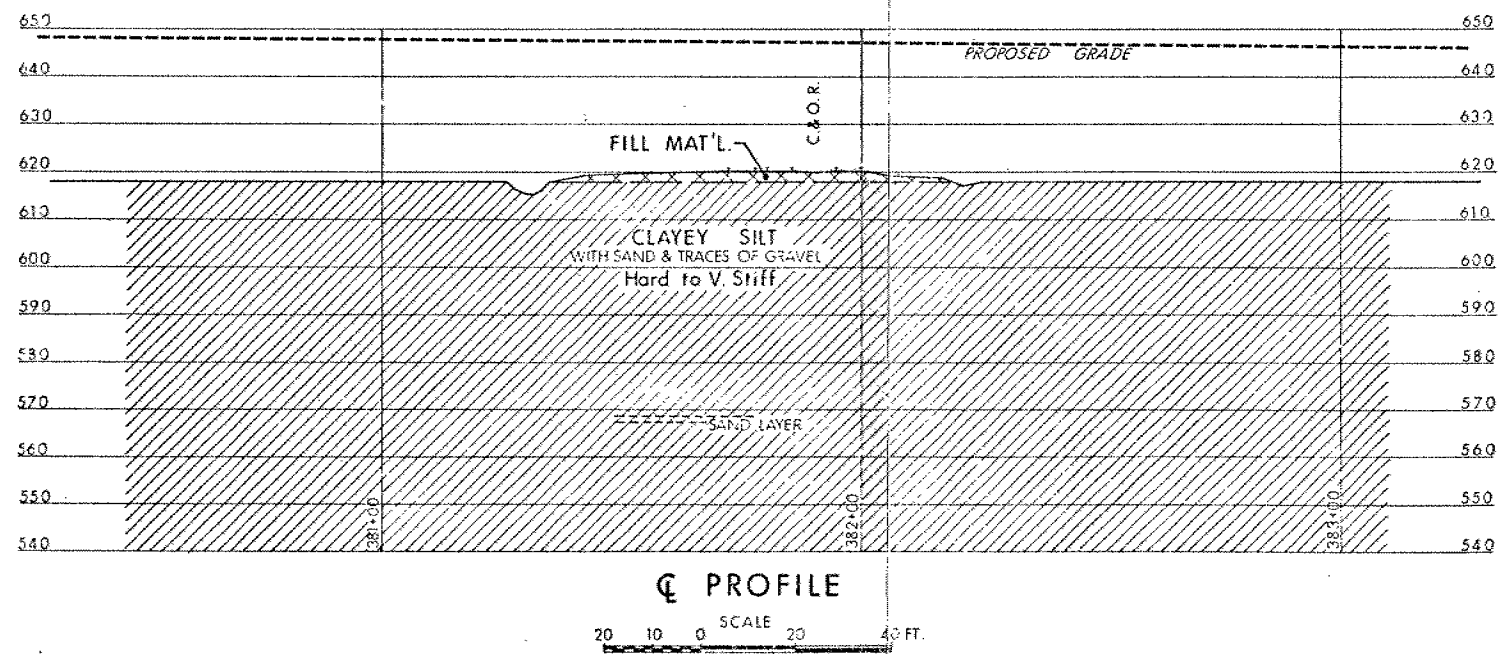
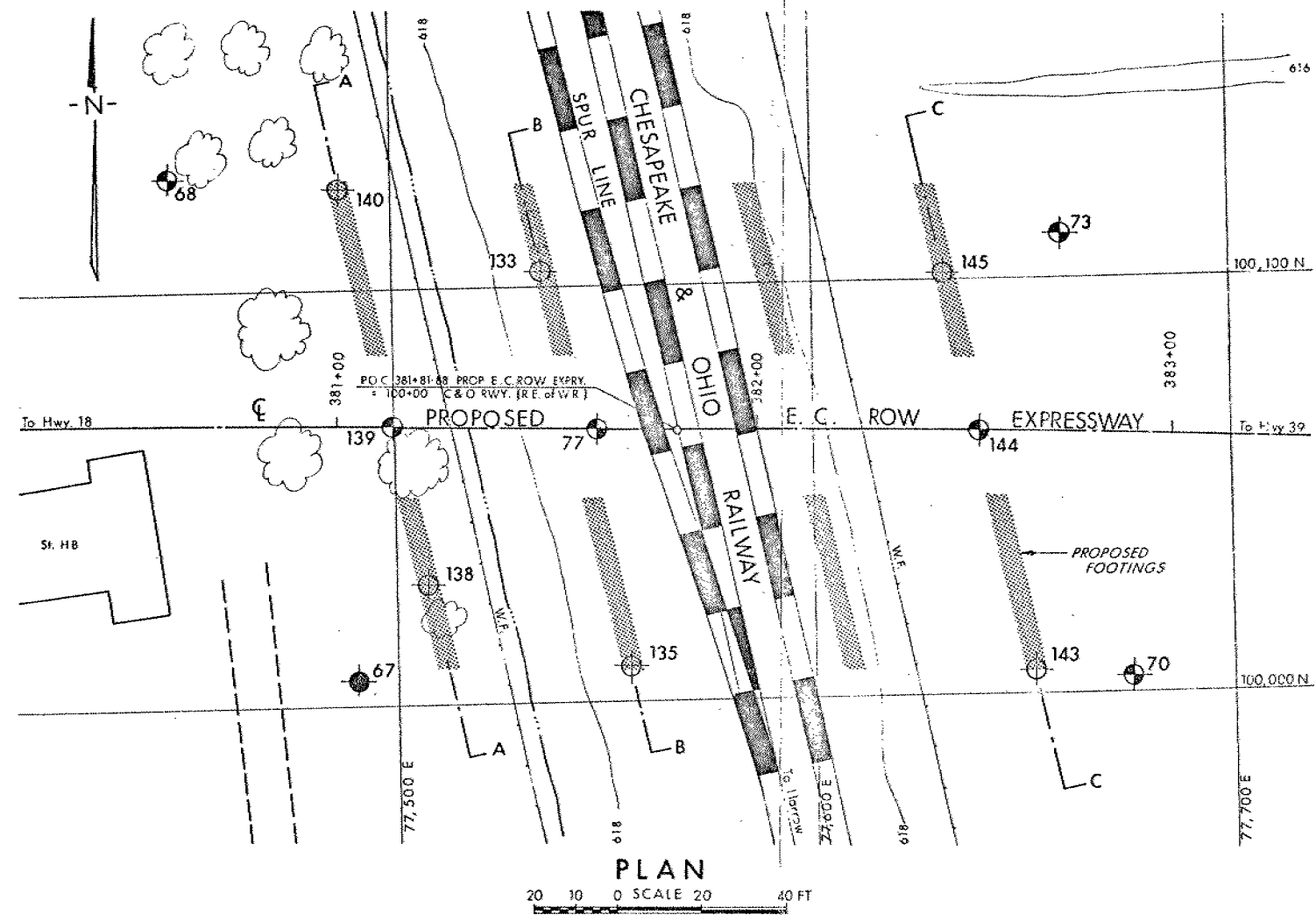
### SLOPES

H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
$\beta$	ANGLE OF SLOPE TO HORIZONTAL



SECTIONS

20 10 0 SCALE 20 40 FT.



## LEGEND

Bore Hole

Cone Penetration Test

Bore Hole & Cone Test

Water Levels established at time  
of field investigation. NOV. 1971

WATER LEVELS DURING FIELD INVESTIGATION  
ESTABLISHED IN BORE HOLES 139 & 144  
ONLY (NOV. 1971)

NO.	ELEVATION	CO - ORDINATES	
		NORTH	EAST
67	616.7	100,006	77,491
68	616.7	100,128	77,447
70	617.5	100,004	77,676
73	616.3	100,111	77,660
77	618.9	100,064	77,550
133	618.8	100,104	77,536
135	618.7	100,008	77,556
138	616.3	100,029	77,508
139	616.4	100,067	77,500
140	616.4	100,124	77,487
143	616.7	100,006	77,653
144	616.4	100,063	77,640
145	616.4	100,102	77,632

**NOTE**

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

DATE	BY	DESCRIPTION

DEPARTMENT OF TRANSPORTATION & COMMUNICATIONS  
DESIGN SERVICES BRANCH - FOUNDATION OFFICE

**CHESAPEAKE & OHIO RAILWAY**

HIGHWAY NO. PROP. E.C. ROW EXPR. DIST NO. 1  
CO. ESSEX CITY OF WINDSOR  
TWP. LOT CON.

**BORE HOLE LOCATIONS & SOIL STRATA**

SUBMD. P.F.	CHECKED	W.P. NO. 257-66-05	DRAWING NO.
DRAWN S.O.	CHECKED	JOB NO. 71-11114	<b>71-11114 A</b>
DATE 16 DEC 1971	SITE NO.	BRIDGE DRAWING NO.	
APPROVED	CONT. NO.		

GEORES No  
4057-6

PILE LOADING TEST  
E. C. ROW EXPRESSWAY  
AT  
CHESAPEAKE & OHIO RAILWAYS  
CITY OF WINDSOR, DIST. #1 (CHATHAM)  
W.O. 74-11025 W.P. 257-66-05

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1. INTRODUCTION
2. DESCRIPTION OF THE SITE & SUBSOIL CONDITIONS
3. PILE DETAILS, DRIVING DATA & TEST ARRANGEMENT
4. PILE LOADING TESTS
5. EVALUATION OF TEST RESULTS
6. MISCELLANEOUS

Pile Loading Test  
At the proposed Crossing of  
Chesapeake & Ohio Railways  
and  
E. C. Row Expressway  
City of Windsor District # 1  
(Chatham)  
W.P. 257-66-05    W.O. 74-11025

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1. INTRODUCTION

Pile loading tests were carried out at the proposed overpass structure, situated at the Crossing of Chesapeake & Ohio Railways and E. C. Row Expressway, in the City of Windsor. On account of the temporary shortage of steel "H" piles it was decided that tubular steel piles should be used. The purpose of the test was to determine :

- (a) Whether or not 12-3/4 inch (324 mm) O.D. x 0.25 inch (6 mm) concrete filled steel tube piles could be economically driven through deep deposits of cohesive soils (up to 140 ft. (42.7 m) in the Windsor area to bedrock.
- (b) Whether or not preaugering the hole, to facilitate pile driving is necessary.
- (c) The most economical method and most suitable technique to drive the piles.
- (d) The maximum allowable bearing capacity of the piles when driven to bedrock.

The project was carried out under Work Order No. 74-31501 by Bermingham Construction Limited under the supervision of M.T.C. Soil Mechanics Section with assistance by the P.P.I. Section. Work in the field commenced on July 16th, 1974, and was completed on July 31st, 1974.

2. DESCRIPTION OF THE SITE & SUBSOIL CONDITIONS

The site is located in the eastern part of the City of Windsor, approximately 0.2 miles (0.3 km) east of the intersection of the



existing E. C. Row Blvd. and Walker Rd.

Physiographically the site belongs to the St. Clair Plain Region. Detailed description of the subsoil conditions of the proposed structure area, is given in the original Foundation Report # W.O. 71-11114.

In order to confirm the subsoil stratigraphy an additional boring was carried out 5 ft. (1.5 m) north of the mid-point of the test piles (B.H. # P-1). The boring revealed a deep deposit of clayey silt with sand and trace of gravel, followed by limestone bedrock. The log of this boring is included in the Appendix of this report. Fig. # 1 shows soil stratigraphy, sketch of piles, standard penetration resistance and pile driving curves.

### 3. PILE DETAILS, DRIVING DATA AND TEST ARRANGEMENT

A total of three test piles was driven between the 16th and 17th of July 1974. The three piles were driven in a straight line at 5.5 ft. (1.7 m) centres. The exact locations and records pertaining to the pile driving are included in the Appendix of this report. The particulars of the test piles are as follows :

PILE # 1 : 12-3/4" (324 mm) O.D. x 0.25" (6 mm) wall thickness A.S.T.M A252 Grade II Straight Weld steel tube was driven on July 17th, 1974. The pile was driven to bedrock El. 487.75 (148.8 m) in the following manner : Down to El. 499.25 (150.3 m) B - 400 hammer was used with a rated energy of 46,000 ft./lb. (62.368 kJ) per blow. From that point, down to El. 487.75 (148.8 m) a smaller hammer of B - 225 was used, with a rated energy of 29,000 ft./lb. (39.319 kJ) per blow. The hammer was changed in order to prevent structural damage to the pile when it contacts the bedrock. Final length of the pile driven in the ground

was 131.5 ft. (40.1 m) and the time of continuous driving was 95 minutes.

PILE # 2 : 12-3/4" (324 mm) x 0.25" (6 mm) wall thickness A.S.T. A252 Grade II Spiral Weld steel tube was driven to bedrock on July 17th, 1974. The same driving technique as for pile # 1 was used. Final length driven in the ground was 130.7 ft. (39.8 m) and total time of continuous driving was 64 minutes.

PILE # 3 : 12-3/4" (324 mm) O.D. x 0.25" (6 mm) wall thickness A.S.T.M A252 Grade II Straight Weld steel tube was driven by means of a Birmingham B-400 hammer, with a rated energy of 46,000 ft./lb (62.368 kJ) per blow. Due to the fact that too high an energy per blow was used during the entire time of the pile driving, it was very difficult to establish the point of contact with the bedrock. Consequently, in spite of very careful observation, the pile has been overdriven, buckling the bottom part of the pile for 13.8 ft. (4.2 m). No pile loading test was carried out on this pile.

All three steel tube piles were fitted with circular flat plate shoes, with the same diameter as that of the piles. Upon driving, the piles were carefully checked inside for buckling, and then, Piles # 1 and 2 were filled with concrete having a 28-day compressive strength of 4000 p.s.i (27.58 MPa).

Pile test location, test piles and borehole location, and pile test arrangement, are shown on Figures 2, 3 and 4 respectively.

#### 4. PILE LOADING TESTS

Two 12-3/4" (324 mm) O.D. x 0.25" (6 mm) wall thickness

steel tube piles numbered 1 and 2 were tested. As was mentioned earlier, both piles had closed ends, were driven to bedrock and were filled with concrete. The tests were performed in accordance with the National Building Code of Canada. Loads were applied to a particular pile under test by means of two 200 tons (1.78 MN) hydraulic jacks, acting between the pile top and a steel reaction beam attached to a box structure, containing slightly more than 400 tons (3.56 MN) of sand. Vertical deflections of the pile under test, were measured by means of four gauges, mounted in four corners of a 2" thick (5.1 mm) steel plate, welded to the head of the pile. These gauges measured the pile movement relative to two independently supported reference beams. The average of the four gauges was taken to be the pile deflection. The two reference beams were 16 ft. (4.9 m) long, and were supported at the ends by spikes driven 5 ft. (1.5 m) below ground level. Some 8 ft. (2.4 m) distance was maintained between test pile and reference beam supports, so that no ground movements, caused by the loading, could have any influence on the reference beams. As an additional precaution, elevations were taken on the reference beams during the tests.

Application of loads to the piles was carried out as follows : The loads were applied in increments ranging from 10 to 30 tons (89 to 267 kN). After each increment was added, the prevailing load was maintained for a period of two hours, or until the rate of settlement fell below 0.01 in/h (0.3 mm/h), whichever was shorter. When the maximum load of 240 tons (2.14 MN) (twice the tentative design load) was reached, this load then was maintained for a period of 24 hours, then removed to zero by decreasing the load to 180, 140, 60 and 20 tons (1.6, 1.25, 0.535 and 0.178 MN).

After a short interval, the piles were again loaded rapidly up to 240 tons (2.14 MN) and then in increments of 20 tons (0.178 MN) up to 400 tons (3.56 MN) for pile # 1 and up to 380 tons (3.38 MN) for pile # 2. The load was then removed by decreasing the load by 50%, 25%, 10% and 0 of the maximum load applied. For each test, curves of load versus time, load versus deflection and deflection versus time have been plotted and are included in the Appendix of this report on Figures 5 and 6. It may be seen on Figures 5 and 6 that under 240 ton loads, both piles experienced total deflection of approximately 1.2" (30.5 mm). By increasing the load on pile # 1 up to 400 tons (3.56 MN) the total deflection was about 2.2" (55.9 mm), some 1.5" (38.1 mm) of which was elastic deformation. The corresponding figures under the 380 tons load on pile # 2 were 2.1" (53.3 mm) and 1.2" (30.5 mm). It is anticipated that under the design load of 120 tons less than 0.5" (12.7 mm) of settlement would occur.

5. EVALUATION OF TEST RESULTS:

In a memorandum dated August 9th, 1974, the Soil Mechanics Section, based on the pile loading and pile driving tests, made the following recommendations :

- (1) Steel tube piles of the types tested can be driven to a depth of 140 ft. (42.7 m) without preaugering.
- (2) The capacity of the driving hammer will have to be restricted to less than 30,000 ft./lb. (40.678 kJ) per blow during the last 3 ft. (0.9 m) of driving in order to prevent structural damage to the pile when it contacts the bedrock.
- (3) Safe loads up to 120 tons (1.07 MN) per pile may be assumed for design purposes.

- (4) Vertical piles may be fitted with standard shoe plates without reinforcing.
- (5) It would be advantageous for battered piles to be fitted with standard shoe plates reinforced with 2-1/2" deep (63 mm) x 3/4" thick (19 mm) cross plates.
- (6) The foregoing recommendations are applicable to other sites where similar subsoil conditions prevail. These include Walker Road & E. C. Row, Central Ave. & E. C. Row, and Central Ave. & C.N. & C.P.R.

6. MISCELLANEOUS

The pile loading tests were carried out under supervision of Messrs H. Szymanski and H. Reed of the Go-P.P.I. Section.

This report was written by Mr. H. Szymanski. The entire project was under direction of Mr. K. G. Selby. Mr. A. K. Barsvary, Head of the P.P.I. Section, reviewed the report.

*H. Szymanski*  
H. Szymanski  
P.P.I. Technician

*for*

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

HS/jw

" APPENDIX "

RECORD OF BOREHOLE NO P1

W.P. 257-66-05 LOCATION 5' North of Mid-Point of Test Piles  
 DIST. 1 (Chatham) E.C. Row Expy BORING DATE 31 July & 1 Aug. 1974  
 DATUM Geodetic BOREHOLE TYPE C.M.E. 750, Hollow Stem  
 ORIGINATED BY H.D.R.  
 COMPILED BY H.D.R.  
 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT — $w_L$ PLASTIC LIMIT — $w_p$ WATER CONTENT — $w$			UNIT WEIGHT $\gamma$	REMARKS
ELEV. DEPTH ft.	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES (0.3 in.)		20 40 60 80 100					$w_p$ — $w$ — $w_L$				
							SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE					WATER CONTENT % 10 20 30				
m						ft/m									GR. SA. SI. CL.	
188.8	619.4	Ground Level														
0.0	0.0	Fill													W.L. Not Obtained	
187.9	616.4														4 31(65)	
0.9	3.0															
		Stiff to Hard														
		Brown														
		Grey														

Continued

20  
15  $\diamond$  5 % STRAIN AT FAILURE  
10

W.P. 257-66-05 LOCATION 5' North of Mid-Point of Test Piles ORIGINATED BY H.D.R.  
DIST. 1(Chatham) HWY. E.C.Row Expy BORING DATE July 31 & Aug. 1, 1974 COMPILED BY H.D.R.  
DATUM Geodetic BOREHOLE TYPE C.M.E. 750, Hollow Stem CHECKED BY \_\_\_\_\_

m	SOIL PROFILE		STRAT PLOT	SAMPLES		GROUND WATER ELEV. ft./m	DYNAMIC CONE PENETRATION RESISTANCE PLOT		LIQUID LIMIT ——— $w_L$ PLASTIC LIMIT ——— $w_p$ WATER CONTENT ——— $w$		UNIT WEIGHT  $\gamma$	REMARKS		
	ELEV. DEPTH ft.	DESCRIPTION		NUMBER	TYPE		'N' VALUES	20 40 60 80 100		$w_p$ ——— $w$ ——— $w_L$			WATER CONTENT % 10 20 30	
								SHEAR STRENGTH						
							$\circ$ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL      x LAB VANE							

15  $\pm$  5 % STRAIN AT FAILURE



# BRIDGE CONSTRUCTION - PILE DRIVING RECORD

DISTRICT NO. 1 CONTRACT NO. \_\_\_\_\_ STRUCTURE W.P. NO. 257-66-05  
CONTRACTOR BERMINGHAM DESIGN LOAD OF PILE TO BE DETERMINED  
HAMMER DETAILS: TYPE B-400 WEIGHT \_\_\_\_\_ HEIGHT OF FALL OR ENERGY 46000ft/lb  
TYPE OF ANVIL OR CAP ANVIL WEIGHT OF ANVIL OR CAP 1400 lb  
PILE DETAILS STEEL TUBULAR 12-3/4" O.D. x 0.250" STRAIGHT WELD BATTER: VERTICAL  
PILE NO. 1 LOCATION E.C.ROW & CHESAPEAKE & OHIO Rwy IN WINDSOR DATE DRIVEN JULY 17/74

TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.
27'	1	1		26	18		51	26		76	34
1:33	2	2		27	17		52	27		77	30
	3	2		28	16		53	25		78	32
	4	1		29	17		54	27		79	33
	5	1		30	15		55	27		80	36
	6	3		31	17		56	26		81	35
	7	2		32	19	99'	57	3+21		82	34
	8	2		33	19	Splicing	58	26		83	30
	9	7		34	18	2:30	59	26		84	34
	10	9		35	19	2:52	60	27		85	31
	11	10		36	19		61	38		86	31
	12	12		37	20		62	29		87	30
	13	14		38	19		63	27		88	31
	14	14		39	20		64	27		89	30
	15	15		40	19		65	27		90	32
	16	14		41	21		66	27		91	31
	17	14		42	21		67	26		92	29
	18	15		43	22		68	27		93	28
	19	15		44	22		69	27		94	28
	20	15		45	22		70	30		95	27
	21	14		46	23		71	29	129'	96	23+8
	22	16		47	23		72	28	Splicing	97	31
	23	16		48	25		73	28	3:15	98	32
60'	24	10+6		49	25		74	29	3:45	99	36
Splicing	25	18		50	23		75	29		100	25

DETAILS FOR FINAL SIX INCHES OF PENETRATION	1	2	3	4	5	6
BLOWS PER INCH						
MEASURED REBOUND IN INCHES						
FINAL LENGTH OF PILE	FINAL CUT OFF ELEVATION					

REPORT TO BE SENT TO:-

GEOTECHNICAL OFFICE  
ATTENTION: PRODUCT & PROCESS IMPROVEMENT SECTION,  
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS,  
DOWNSVIEW, ONTARIO

SIGNED \_\_\_\_\_

NAME (PRINT) \_\_\_\_\_

DATE \_\_\_\_\_

ATTACH SKETCH OF PILE NUMBERING SYSTEM

NOTES:

In general this form should be completed for every tenth pile in a group, but at least one is required for every pier and abutment.

Piles driven vertically should be selected where possible.

Pile Details must include type, dimensions and weight per foot, details of shoe, and slope of batter: e.g. 12 3/4" O.D. steel tube x 0.251" @ 33 lbs. per foot vertical. 12 3/4" x 1/2" steel plate shoe.

Details for the final six inches of penetration must be completed for all piles except in the case of an end bearing pile driven to bedrock. Final length of pile, and final cut off elevation must always be given.

The total length being driven is the full length of the pile and remains unchanged until a length is cut off or spliced on.

The penetration in blows per foot must be recorded for every foot of penetration of the pile.

Measured rebounds recorded on this form must be the average for each individual inch for the final six inches of penetration.

# BRIDGE CONSTRUCTION - PILE DRIVING RECORD

DISTRICT NO. 1 CONTRACT NO. \_\_\_\_\_ STRUCTURE W.P. NO. 257-66-05  
 CONTRACTOR BERMINGTON DESIGN LOAD OF PILE TO BE DETERMINED  
 HAMMER DETAILS: TYPE B-400 WEIGHT \_\_\_\_\_ HEIGHT OF FALL OR ENERGY 160000 ft./lb  
 TYPE OF ANVIL OR CAP ANVIL WEIGHT OF ANVIL OR CAP 1400 lb  
 PILE DETAILS STEEL TUBULAR 12-3/4" O.D. x 0.250" STRAIGHT WELD BATTER: VERTICAL  
 PILE NO. 1 LOCATION E.C. ROW & CHESAPEAKE & OHIO Rwy IN WINDSOR DATE DRIVEN JULY 17/74

TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.
	10 <sup>1</sup>	37	134'	126	59		51			76	
	10 <sup>2</sup>	36	Splicing	127			52			77	
	10 <sup>3</sup>	37	& Changing	128			53			78	
	10 <sup>4</sup>	39	Hammer	129			54			79	
	10 <sup>5</sup>	38	Tb B-225	30			55			80	
	10 <sup>6</sup>	39	4:12	31			56			81	
	10 <sup>7</sup>	37	4:43	32			57			82	
	10 <sup>8</sup>	40		33			58			83	
	10 <sup>9</sup>	39		34			59			84	
	110	40		35			60			85	
	111	41		36			61			86	
	112	42		37			62			87	
	113	42		38			63			88	
	114	46		39			64			89	
	115	49		40			65			90	
	116	48		41			66			91	
	117	49		42			67			92	
	118	47		43			68			93	
	119	48		44			69			94	
	120	50		45			70			95	
	121	50		46			71			96	
	122	52		47			72			97	
	123	55		48			73			98	
	124	56		49			74			99	
	125	57		50			75			100	

DETAILS FOR FINAL SIX INCHES OF PENETRATION	1	2	3	4	5	6
BLOWS PER INCH						
MEASURED REBOUND IN INCHES						
FINAL LENGTH OF PILE	FINAL CUT OFF ELEVATION					

REPORT TO BE SENT TO:-

GEOTECHNICAL OFFICE  
 ATTENTION: PRODUCT & PROCESS IMPROVEMENT SECTION,  
 MINISTRY OF TRANSPORTATION AND COMMUNICATIONS,  
 DOWNSVIEW, ONTARIO

SIGNED \_\_\_\_\_

NAME (PRINT) \_\_\_\_\_

DATE \_\_\_\_\_

ATTACH SKETCH OF PILE NUMBERING SYSTEM

NOTES:

In general this form should be completed for every tenth pile in a group, but at least one is required for every pier and abutment.

Piles driven vertically should be selected where possible.

Pile Details must include type, dimensions and weight per foot, details of shoe, and slope of batter: e.g. 12 3/4" O.D. steel tube x 0.251" @ 33 lbs. per foot vertical. 12 3/4" x 1/2" steel plate shoe.

Details for the final six inches of penetration must be completed for all piles except in the case of an end bearing pile driven to bedrock. Final length of pile, and final cut off elevation must always be given.

The total length being driven is the full length of the pile and remains unchanged until a length is cut off or spliced on.

The penetration in blows per foot must be recorded for every foot of penetration of the pile.

Measured rebounds recorded on this form must be the average for each individual inch for the final six inches of penetration.

# BRIDGE CONSTRUCTION - PILE DRIVING RECORD

DISTRICT NO. 1 CONTRACT NO. \_\_\_\_\_ STRUCTURE W.P. NO. 257-66-05  
CONTRACTOR BERMINGHAM DESIGN LOAD OF PILE TO BE DETERMINED  
HAMMER DETAILS: TYPE B-225 WEIGHT \_\_\_\_\_ HEIGHT OF FALL OR ENERGY 29000ft/lb  
TYPE OF ANVIL OR CAP ANVIL WEIGHT OF ANVIL OR CAP 1100 lb  
PILE DETAILS STEEL TUBULAR 12-3/4" x 0.250" STRAIGHT WELD BATTER: VERTICAL  
PILE NO. 1 LOCATION E.C. ROW & CHESAPEAKE & OHIO RAILWAY IN WINDSOR DATE DRIVEN JULY 17/74

TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT. Per Inch	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT. Per Inch	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT. Per Inch	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.
	1	13		26	13		51	15		76	
	2	12		27	11		52	17		77	
	3	11		28	11		53	18		78	
	4	12		29	13		54	19		79	
	5	11		30	10		55	19		80	
	6	14		31	10		56	19		81	
	7	12		32	11		57	18		82	
	8	12		33	12		58	18		83	
	9	12		34	10		59	15		84	
	10	12		35	10	131'	60	16		85	
	11	11	129'	36	18		61	18		86	
127'	12	12		37	14	7/4	62	16		87	
	13	14		38	12	Rocess	63	12		88	
	14	12		39	10	5:05	64	18		89	
	15	13		40	10	5:10	65	42		90	
	16	14		41	11	5:12	66	70	PILE BOUNCING	91	
	17	11		42	10		67		@ 131'6"	92	
	18	13		43	6+4		68			93	
	19	12		44	8		69			94	
	20	10		45	11		70			95	
	21	12		46	17		71			96	
	22	13		47	12		72			97	
	23	10	130	48	10		73			98	
128'	24	11		49	7		74			99	
	25	11		50	12		75			100	

DETAILS FOR FINAL SIX INCHES OF PENETRATION	1	2	3	4	5	6
BLOWS PER INCH	18	16	12	18	42	70
MEASURED REBOUND IN INCHES						0.75
FINAL LENGTH OF PILE	132.3'					FINAL CUT OFF ELEVATION 620.07

REPORT TO BE SENT TO: -

SIGNED \_\_\_\_\_

GEOTECHNICAL OFFICE  
ATTENTION: PRODUCT & PROCESS IMPROVEMENT SECTION,  
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS,  
DOWNSVIEW, ONTARIO

NAME (PRINT) H. SZYMANSKI

DATE JULY 17/74

ATTACH SKETCH OF PILE NUMBERING SYSTEM

4-74

4-74

08-MT-285

NOTES:

In general this form should be completed for every tenth pile in a group, but at least one is required for every pier and abutment.

Piles driven vertically should be selected where possible.

Pile Details must include type, dimensions and weight per foot, details of shoe, and slope of batter: e.g. 12 3/4" O.D. steel tube x 0.251" @ 33 lbs. per foot vertical. 12 3/4" x 1/2" steel plate shoe.

Details for the final six inches of penetration must be completed for all piles except in the case of an end bearing pile driven to bedrock. Final length of pile, and final cut off elevation must always be given.

The total length being driven is the full length of the pile and remains unchanged until a length is cut off or spliced on.

The penetration in blows per foot must be recorded for every foot of penetration of the pile.

Measured rebounds recorded on this form must be the average for each individual inch for the final six inches of penetration.

# BRIDGE CONSTRUCTION - PILE DRIVING RECORD

DISTRICT NO. 1 CONTRACT NO. \_\_\_\_\_ STRUCTURE W.P. NO. 257-66-05  
 CONTRACTOR BERMINGHAM DESIGN LOAD OF PILE TO BE DETERMINED  
 HAMMER DETAILS: TYPE B-400 WEIGHT \_\_\_\_\_ HEIGHT OF FALL OR ENERGY 16000ft/11  
 TYPE OF ANVIL OR CAP ANVIL WEIGHT OF ANVIL OR CAP 1400 lb.  
 PILE DETAILS STEEL TUBULAR 12-3/4" O.D. x 0.250" SPIRAL WELD BATTER: VERTICAL  
 PILE NO. 2 LOCATION E.C.R. & CHESAPEAKE & OHIO Rwy IN WINDSOR DATE DRIVEN JULY 17/74

TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.
60'	1	1		26	12+9		51	19		76	28
8:08 am	2	2		27	21		52	21		77	29
	3	1		28	20		53	20		78	29
	4	2		29	20		54	22		79	28
	5	2		30	20		55	24		80	30
	6	2		31	20		56	23		81	30
	7	5		32	22		57	22		82	29
	8	3		33	21	120'	58	16+5		83	28
	9	4		34	20	Splicing	59	21		84	29
	10	9		35	21	8:36 am	60	20		85	28
	11	10		36	20	8:58 am	61	21		86	27
	12	12		37	20		62	20		87	27
	13	16		38	20		63	23		88	27
	14	16		39	20		64	23		89	27
	15	18		40	21		65	24		90	24
	16	17		41	19		66	24		91	22
	17	16		42	19		67	24		92	22
	18	18		43	20		68	24		93	22
	19	18		44	19		69	25		94	23
	20	17		45	19		70	25		95	23
	21	18		46	19		71	25		96	22
	22	19		47	20		72	26		97	23
	23	19		48	20		73	25		98	23
	24	19		49	20		74	26		99	24
	25	20		50	21		75	28		100	24

DETAILS FOR FINAL SIX INCHES OF PENETRATION	1	2	3	4	5	6
BLOWS PER INCH						
MEASURED REBOUND IN INCHES						
FINAL LENGTH OF PILE	FINAL CUT OFF ELEVATION					

REPORT TO BE SENT TO:-

GEOTECHNICAL OFFICE  
 ATTENTION: PRODUCT & PROCESS IMPROVEMENT SECTION,  
 MINISTRY OF TRANSPORTATION AND COMMUNICATIONS,  
 DOWNSVIEW, ONTARIO

SIGNED \_\_\_\_\_

NAME (PRINT) \_\_\_\_\_

DATE \_\_\_\_\_

ATTACH SKETCH OF PILE NUMBERING SYSTEM

NOTES:

In general this form should be completed for every tenth pile in a group, but at least one is required for every pier and abutment.

Piles driven vertically should be selected where possible.

Pile Details must include type, dimensions and weight per foot, details of shoe, and slope of batter: e.g. 12 3/4" O.D. steel tube x 0.251" @ 33 lbs. per foot vertical. 12 3/4" x 1/2" steel plate shoe.

Details for the final six inches of penetration must be completed for all piles except in the case of an end bearing pile driven to bedrock. Final length of pile, and final cut off elevation must always be given.

The total length being driven is the full length of the pile and remains unchanged until a length is cut off or spliced on.

The penetration in blows per foot must be recorded for every foot of penetration of the pile.

Measured rebounds recorded on this form must be the average for each individual inch for the final six inches of penetration.



# BRIDGE CONSTRUCTION - PILE DRIVING RECORD

DISTRICT NO. 1 CONTRACT NO. \_\_\_\_\_ STRUCTURE W.P. NO. 257-66-05  
CONTRACTOR BERMINGHAM DESIGN LOAD OF PILE TO BE DETERMINED  
HAMMER DETAILS: TYPE B-400 WEIGHT \_\_\_\_\_ HEIGHT OF FALL OR ENERGY 46000ft/lb  
TYPE OF ANVIL OR CAP ANVIL WEIGHT OF ANVIL OR CAP 1400 lb.  
PILE DETAILS STEEL TUBULAR 12-3/4" O.D. x 0.250" SPIRAL WELD BATTER: VERTICAL  
PILE NO. 2 LOCATION E.C.R. & CHESAPEAKE & ONTO HWY IN WINDSOR DATE DRIVEN JULY 17, 1974

TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.
120'	10 <sup>1</sup>	22		26			51			76	
	10 <sup>2</sup>	25		27			52			77	
	10 <sup>3</sup>	25		28			53			78	
	10 <sup>4</sup>	25		29			54			79	
	10 <sup>5</sup>	25		30			55			80	
	10 <sup>6</sup>	26		31			56			81	
	10 <sup>7</sup>	29		32			57			82	
	10 <sup>8</sup>	29		33			58			83	
	10 <sup>9</sup>	29		34			59			84	
	11 <sup>0</sup>	30		35			60			85	
	11 <sup>1</sup>	31		36			61			86	
	11 <sup>2</sup>	33		37			62			87	
	11 <sup>3</sup>	35		38			63			88	
	11 <sup>4</sup>	36		39			64			89	
	11 <sup>5</sup>	35		40			65			90	
	11 <sup>6</sup>	38		41			66			91	
140'	11 <sup>7</sup>	1+37		42			67			92	
Splicing	11 <sup>8</sup>	45		43			68			93	
9:23	11 <sup>9</sup>	44		44			69			94	
10:02	12 <sup>0</sup>	44		45			70			95	
	12 <sup>1</sup>	44		46			71			96	
	12 <sup>2</sup>	42		47			72			97	
Recess	12 <sup>3</sup>	46		48			73			98	
10:06	12 <sup>4</sup>	17+22		49			74			99	
10:28	25			50			75			100	

DETAILS FOR FINAL SIX INCHES OF PENETRATION	1	2	3	4	5	6
BLOWS PER INCH						
MEASURED REBOUND IN INCHES						
FINAL LENGTH OF PILE	FINAL CUT OFF ELEVATION					

REPORT TO BE SENT TO:-

GEOTECHNICAL OFFICE  
ATTENTION: PRODUCT & PROCESS IMPROVEMENT SECTION,  
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS,  
DOWNSVIEW, ONTARIO

SIGNED \_\_\_\_\_

NAME (PRINT) \_\_\_\_\_

DATE \_\_\_\_\_

ATTACH SKETCH OF PILE NUMBERING SYSTEM

61-9 4-74

4-74

08-MT-285

NOTES:

In general this form should be completed for every tenth pile in a group, but at least one is required for every pier and abutment.

Piles driven vertically should be selected where possible.

File Details must include type, dimensions and weight per foot, details of shoe, and slope of batter: e.g. 12 3/4" O.D. steel tube x 0.251" @ 33 lbs. per foot vertical. 12 3/4" x 1/2" steel plate shoe.

Details for the final six inches of penetration must be completed for all piles except in the case of an end bearing pile driven to bedrock. Final length of pile, and final cut off elevation must always be given.

The total length being driven is the full length of the pile and remains unchanged until a length is cut off or spliced on.

The penetration in blows per foot must be recorded for every foot of penetration of the pile.

Measured rebounds recorded on this form must be the average for each individual inch for the final six inches of penetration.

# BRIDGE CONSTRUCTION - PILE DRIVING RECORD

DISTRICT NO. 1 CONTRACT NO. \_\_\_\_\_ STRUCTURE W.P. NO. 257-66-05  
 CONTRACTOR BERMINGHAM DESIGN LOAD OF PILE TO BE DETERMINED  
 HAMMER DETAILS: TYPE B=400 5-225 FROM 128' DOWN WEIGHT \_\_\_\_\_ HEIGHT OF FALL OR ENERGY 450000 ft-lb  
 TYPE OF ANVIL OR CAP ANVIL WEIGHT OF ANVIL OR CAP 1100 lb  
 PILE DETAILS STEEL TUBULAR 12-3/4" O.D. x 0.25" SPIRAL WELD BATTER: VERTICAL  
 PILE NO. 2 LOCATION E.C.R. & CHESAPEAKE & OHIO Rwy IN WINDSOR DATE DRIVEN JULY 17/74

TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / ft. Per Inch	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / ft. Per Inch	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / ft. Per Inch	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / ft. Per Inch
	1	2		26	3		51	8		76	12
	2	2		27	4		52	8		77	11
	3	2		28	3		53	7		78	9
	4	3		29	4		54	8		79	10
	5	2		30	4		55	9	11:35	80	33
	6	2		31	3		56	8		81	
	7	3		32	4		57	8		82	
	8	4		33	4		58	8		83	
	9	3		34	4		59	10		84	
	10	3		35	4	129'	60	8		85	
	11	3	127'	36	4		61	9		86	
125'	12	4		37	3		62	9		87	
	13	3		38	3		63	12		88	
	14	3		39	4		64	9		89	
	15	3		40	4		65	9		90	
	16	5		41	4		66	10		91	
	17	3		42	4		67	12		92	
	18	4		43	3		68	10		93	
	19	3		44	4		69	11		94	
	20	3		45	4		70	12		95	
	21	4		46	4		71	12		96	
	22	3		47	4	130'	72	12		97	
	23	3	128'	48	4		73	8		98	
126'	24	4	10:30	49	7+7		74	10		99	
	25	3	11:30	50	8		75	10		100	

DETAILS FOR FINAL SIX INCHES OF PENETRATION	1	2	3	4	5	6
BLOWS PER INCH	10	12	11	9	10	33
MEASURED REBOUND IN INCHES						
FINAL LENGTH OF PILE	131.5'					
FINAL CUT OFF ELEVATION	620.05					

REPORT TO BE SENT TO:-

GEOTECHNICAL OFFICE  
ATTENTION: PRODUCT & PROCESS IMPROVEMENT SECTION,  
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS,  
DOWNSVIEW, ONTARIO

SIGNED \_\_\_\_\_  
NAME (PRINT) H. SZYMANSKI  
DATE JULY 17/74  
ATTACH SKETCH OF PILE NUMBERING SYSTEM

GI-9 4-74

OB-MT-285 4-74

NOTES:

In general this form should be completed for every tenth pile in a group, but at least one is required for every pier and abutment.

Piles driven vertically should be selected where possible.

Pile Details must include type, dimensions and weight per foot, details of shoe, and slope of batter: e.g. 12 3/4" O.D. steel tube x 0.251" @ 33 lbs. per foot vertical. 12 3/4" x 1/2" steel plate shoe.

Details for the final six inches of penetration must be completed for all piles except in the case of an end bearing pile driven to bedrock. Final length of pile, and final cut off elevation must always be given.

The total length being driven is the full length of the pile and remains unchanged until a length is cut off or spliced on.

The penetration in blows per foot must be recorded for every foot of penetration of the pile.

Measured rebounds recorded on this form must be the average for each individual inch for the final six inches of penetration.

# BRIDGE CONSTRUCTION - PILE DRIVING RECORD

DISTRICT NO. 1 CONTRACT NO. \_\_\_\_\_ STRUCTURE W.P. NO. 257-66-05  
 CONTRACTOR BERMINGHAM DESIGN LOAD OF PILE TO BE DETERMINED  
 HAMMER DETAILS: TYPE B-400 (BERMINGHAM) WEIGHT \_\_\_\_\_ HEIGHT OF FALL OR ENERGY 46000 ft/lb  
 TYPE OF ANVIL OR CAP ANVIL WEIGHT OF ANVIL OR CAP 1400 lb.  
 PILE DETAILS STEEL TUBULAR 12-3/4" O.D. x 0.250" (STRAIGHT WELD) BATTER: VERTICAL  
 PILE NO. 3 LOCATION E.C.R. & CHESAPEAKE & OHIO RAILWAY IN DATE DRIVEN 16 JULY 1974

TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.
60'	1	2		26	21		51	32		76	30
11:59	2	2		27	21		52	31		77	33
	3	2		28	20		53	29		78	32
	4	1		29	20		54	30		79	32
	5	2		30	21		55	30		80	33
	6	4		31	21		56	30		81	33
	7			32	21		57	30		82	34
	8			33	20	120'	58	24+6		83	33
	9	8		34	21	Lunch &	59	37		84	36
	10	9		35	22	Splicing	60	37		85	34
	11	14		36	23	12:30	61	36		86	36
	12	15		37	23	1:45	62	35		87	36
	13	19		38	23		63	35		88	34
	14	19		39	23		64	36		89	32
	15	18		40	24		65	33		90	32
	16	14		41	26		66	33		91	31
	17	19		42	24		67	39		92	32
	18	22		43	26		68	33		93	32
	19	20		44	27		69	36		94	31
	20	20		45	27		70	34		95	31
	21	21		46	28		71	33		96	32
	22	21		47	26		72	38		97	33
	23	20		48	30		73	32		98	31
	24	20		49	29		74	34		99	32
	25	21		50	29		75	33		100	31

DETAILS FOR FINAL SIX INCHES OF PENETRATION	1	2	3	4	5	6
BLOWS PER INCH						
MEASURED REBOUND IN INCHES						
FINAL LENGTH OF PILE	FINAL CUT OFF ELEVATION					

REPORT TO BE SENT TO:-

GEOTECHNICAL OFFICE  
 ATTENTION: PRODUCT & PROCESS IMPROVEMENT SECTION,  
 MINISTRY OF TRANSPORTATION AND COMMUNICATIONS,  
 DOWNSVIEW, ONTARIO

SIGNED \_\_\_\_\_  
 NAME (PRINT) \_\_\_\_\_  
 DATE \_\_\_\_\_  
 ATTACH SKETCH OF PILE NUMBERING SYSTEM

4-74

4-74

08-MT-285

NOTES:

In general this form should be completed for every tenth pile in a group, but at least one is required for every pier and abutment.

Piles driven vertically should be selected where possible.

Pile Details must include type, dimensions and weight per foot, details of shoe, and slope of batter: e.g. 12 3/4" O.D. steel tube x 0.251" @ 33 lbs. per foot vertical. 12 3/4" x 1/2" steel plate shoe.

Details for the final six inches of penetration must be completed for all piles except in the case of an end bearing pile driven to bedrock. Final length of pile, and final cut off elevation must always be given.

The total length being driven is the full length of the pile and remains unchanged until a length is cut off or spliced on.

The penetration in blows per foot must be recorded for every foot of penetration of the pile.

Measured rebounds recorded on this form must be the average for each individual inch for the final six inches of penetration.

# BRIDGE CONSTRUCTION - PILE DRIVING RECORD

DISTRICT NO. 1 CONTRACT NO. \_\_\_\_\_ STRUCTURE W.P. NO. 257-66-05  
 CONTRACTOR BERMINGHAM DESIGN LOAD OF PILE TO BE DETERMINED  
 HAMMER DETAILS: TYPE B-400 (BERMINGHAM) WEIGHT \_\_\_\_\_ HEIGHT OF FALL OR ENERGY 46000ft/l  
 TYPE OF ANVIL OR CAP ANVIL WEIGHT OF ANVIL OR CAP 1400 lb.  
 PILE DETAILS STEEL TUBULAR 12-3/4" O.D. x 0.250" (STRAIGHT WELD) BATTER: VERTICAL  
 PILE NO. 3 LOCATION E.C.R. & CHESAPEAKE & OHIO Rwy IN WINDSOR DATE DRIVEN 16 JULY 1974

TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS / FT.
	10 <sup>1</sup>	32		12 <sup>6</sup>	61		51			76	
	102	33		12 <sup>7</sup>	60		52			77	
	103	31		12 <sup>8</sup>	58		53			78	
	104	34		12 <sup>9</sup>	61		54			79	
	105	34		13 <sup>0</sup>	64		55			80	
	106	36		13 <sup>1</sup>	91		56			81	
	107	37		13 <sup>2</sup>	68		57			82	
	108	37		13 <sup>3</sup>	44		58			83	
	109	35		13 <sup>4</sup>	52		59			84	
	110	45		13 <sup>5</sup>	52		60			85	
	111	46		13 <sup>6</sup>	56		61			86	
	112	48		13 <sup>7</sup>	55		62			87	
	113	47		13 <sup>8</sup>	58		63			88	
	114	44		13 <sup>9</sup>	55		64			89	
	115	59		14 <sup>0</sup>	54		65			90	
150'	116	59		14 <sup>1</sup>	54		66			91	
Splicing	117	8+46		14 <sup>2</sup>	54		67			92	
2:30	118	53		14 <sup>3</sup>	59		68			93	
3:00	119	53		14 <sup>4</sup>	58		69			94	
	120	56		14 <sup>5</sup>	59		70			95	
	121	56	3:40	14 <sup>6</sup>	81		71			96	
	122	56		4 <sup>7</sup>			72			97	
	123	58		4 <sup>8</sup>			73			98	
	124	58		4 <sup>9</sup>			74			99	
	125	58		5 <sup>0</sup>			75			100	

DETAILS FOR FINAL SIX INCHES OF PENETRATION	1	2	3	4	5	6
BLOWS PER INCH						
MEASURED REBOUND IN INCHES						
FINAL LENGTH OF PILE <u>150'</u>	FINAL CUT OFF ELEVATION <u>624.36</u>					

REPORT TO BE SENT TO: \*TIP ELEV. 488.1  
PILE BUCKLED 13.8  
 GEOTECHNICAL OFFICE  
 ATTENTION: PRODUCT & PROCESS IMPROVEMENT SECTION,  
 MINISTRY OF TRANSPORTATION AND COMMUNICATIONS,  
 DOWNSVIEW, ONTARIO

SIGNED \_\_\_\_\_  
 NAME (PRINT) H. SZYMANSKI  
 DATE 16 JULY 1974  
 ATTACH SKETCH OF PILE NUMBERING SYSTEM

GI-9 4-74

4-74

08-WT-285

NOTES:

In general this form should be completed for every tenth pile in a group, but at least one is required for every pier and abutment.

Piles driven vertically should be selected where possible.

Pile Details must include type, dimensions and weight per foot, details of shoe, and slope of batter: e.g. 12 3/4" O.D. steel tube x 0.251" @ 33 lbs. per foot vertical. 12 3/4" x 1/2" steel plate shoe.

Details for the final six inches of penetration must be completed for all piles except in the case of an end bearing pile driven to bedrock. Final length of pile, and final cut off elevation must always be given.

The total length being driven is the full length of the pile and remains unchanged until a length is cut off or spliced on.

The penetration in blows per foot must be recorded for every foot of penetration of the pile.

Measured rebounds recorded on this form must be the average for each individual inch for the final six inches of penetration.



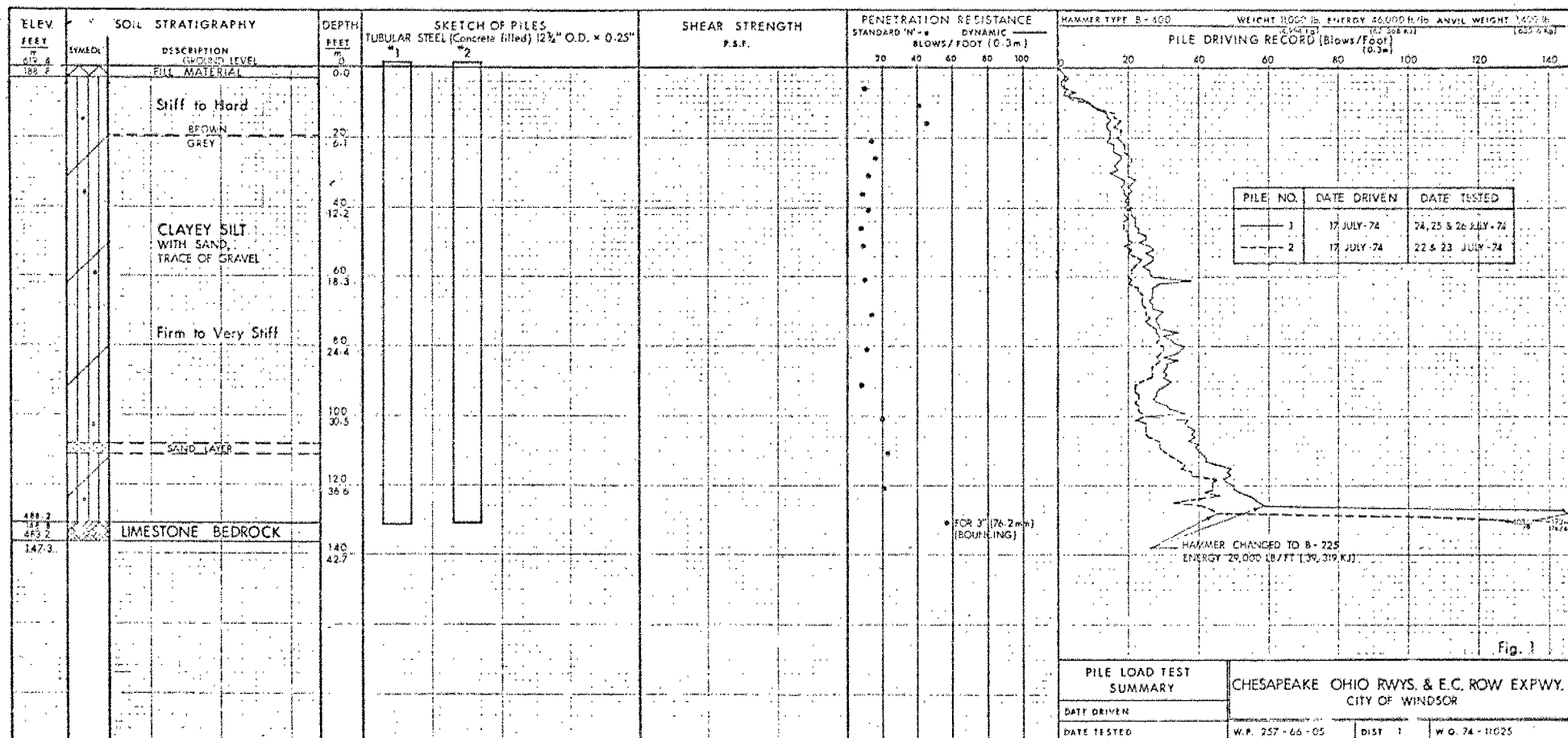
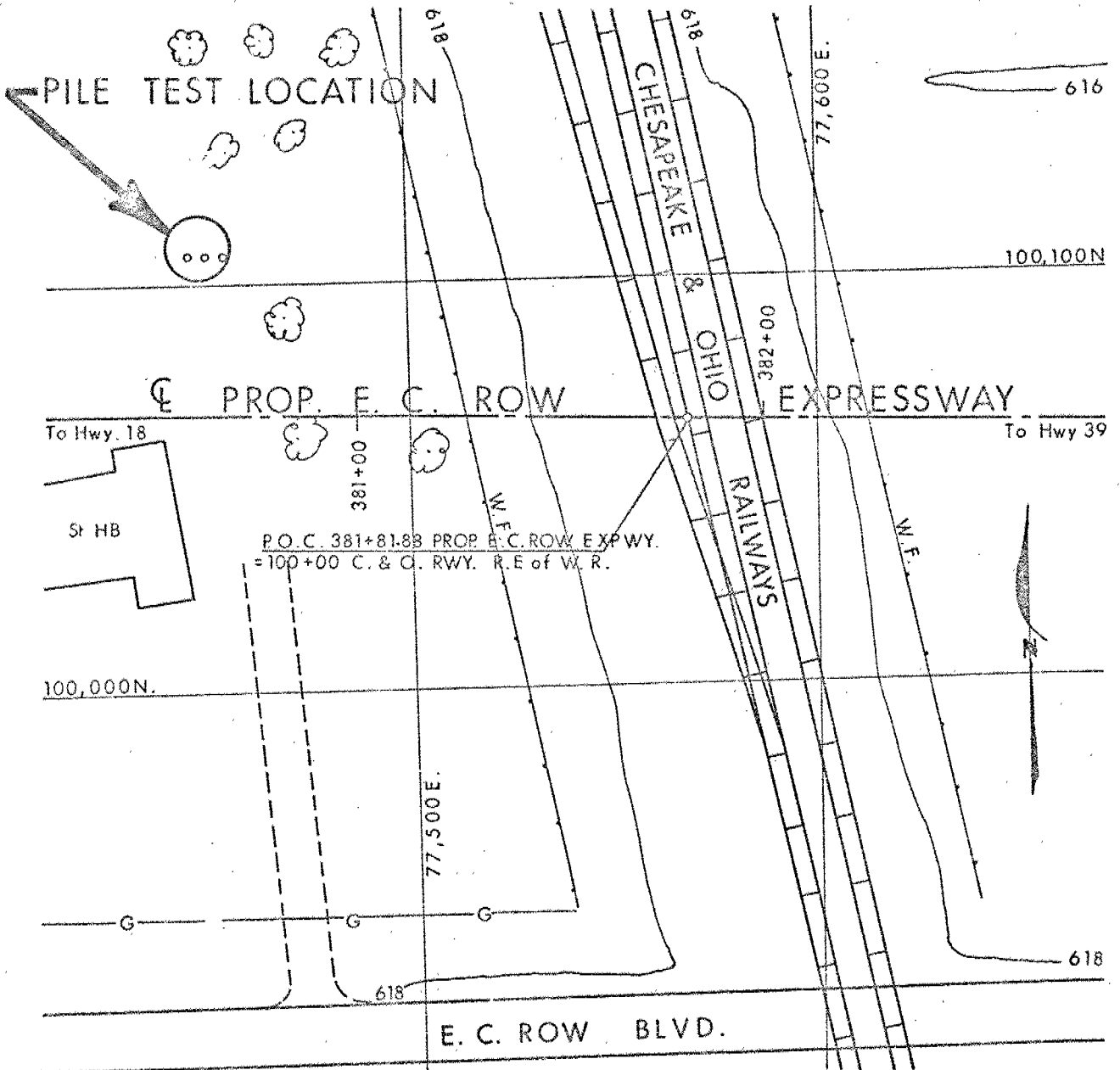


Fig. 1



-PLAN-  
SCALE: 1" = 40'

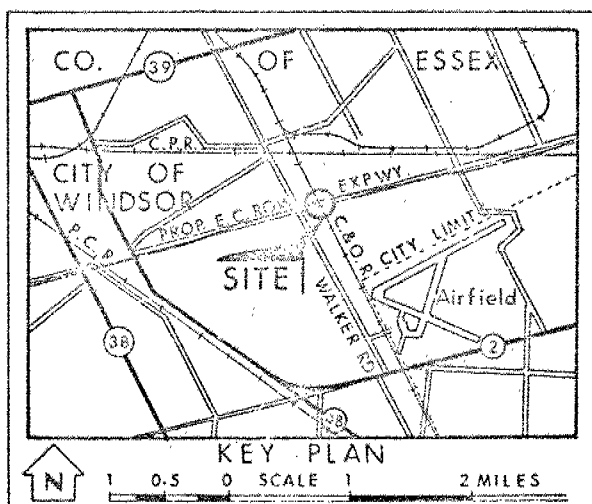


Fig. 2

REVISED: 6 JAN., 1975

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

CHESAPEAKE OHIO RAILWAYS & E.C. ROW EXPWY.

PILE TEST LOCATION

W.P. NO. 257-66-05

DIST. NO. 1

DATE June 3, 1974

W.O. NO. 74-11025

DRAWING NO 74-11025A

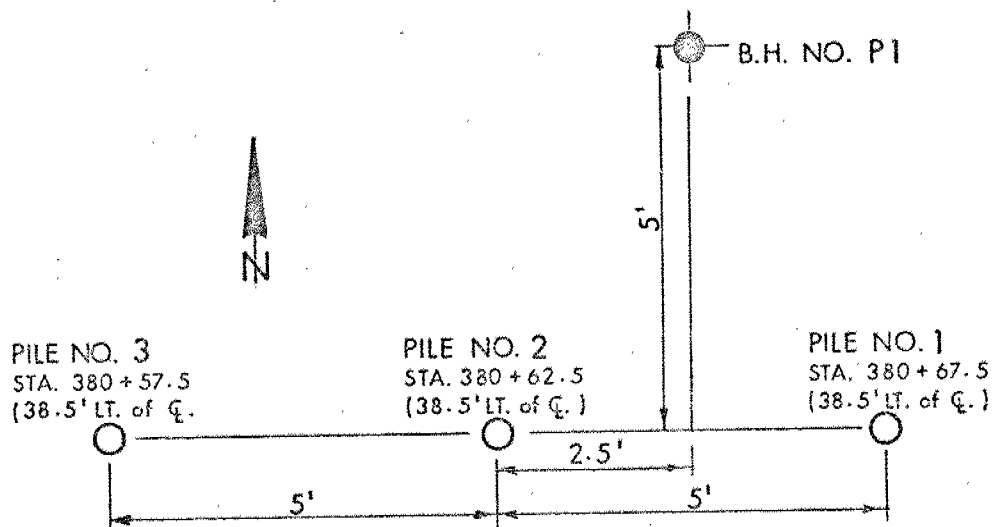
## PILE DETAILS

ALL PILES 12  $\frac{3}{4}$ " O.D.  $\times$  0.25" TUBULAR STEEL

PILE NO. 1 - STRAIGHT WELD (Concrete filled)

PILE NO. 2 - SPIRAL WELD (Concrete filled)

PILE NO. 3 - STRAIGHT WELD



## SEQUENCE OF PILE DRIVING

- 1) DRIVE PILE NO. 3 TO BEDROCK  
PILE BUCKLED DUE TO OVER DRIVING
- 2) DRIVE PILE NO. 2 TO BEDROCK
- 3) DRIVE PILE NO. 1 TO BEDROCK
- 4) FILL PILES 1 & 2 WITH 4,000 P.S.I. (27.58 MPa) CONCRETE

Fig. 3

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

CHESAPEAKE OHIO RAILWAYS & E. C. ROW EXPWY.

### TEST PILES & B.H. LOCATION

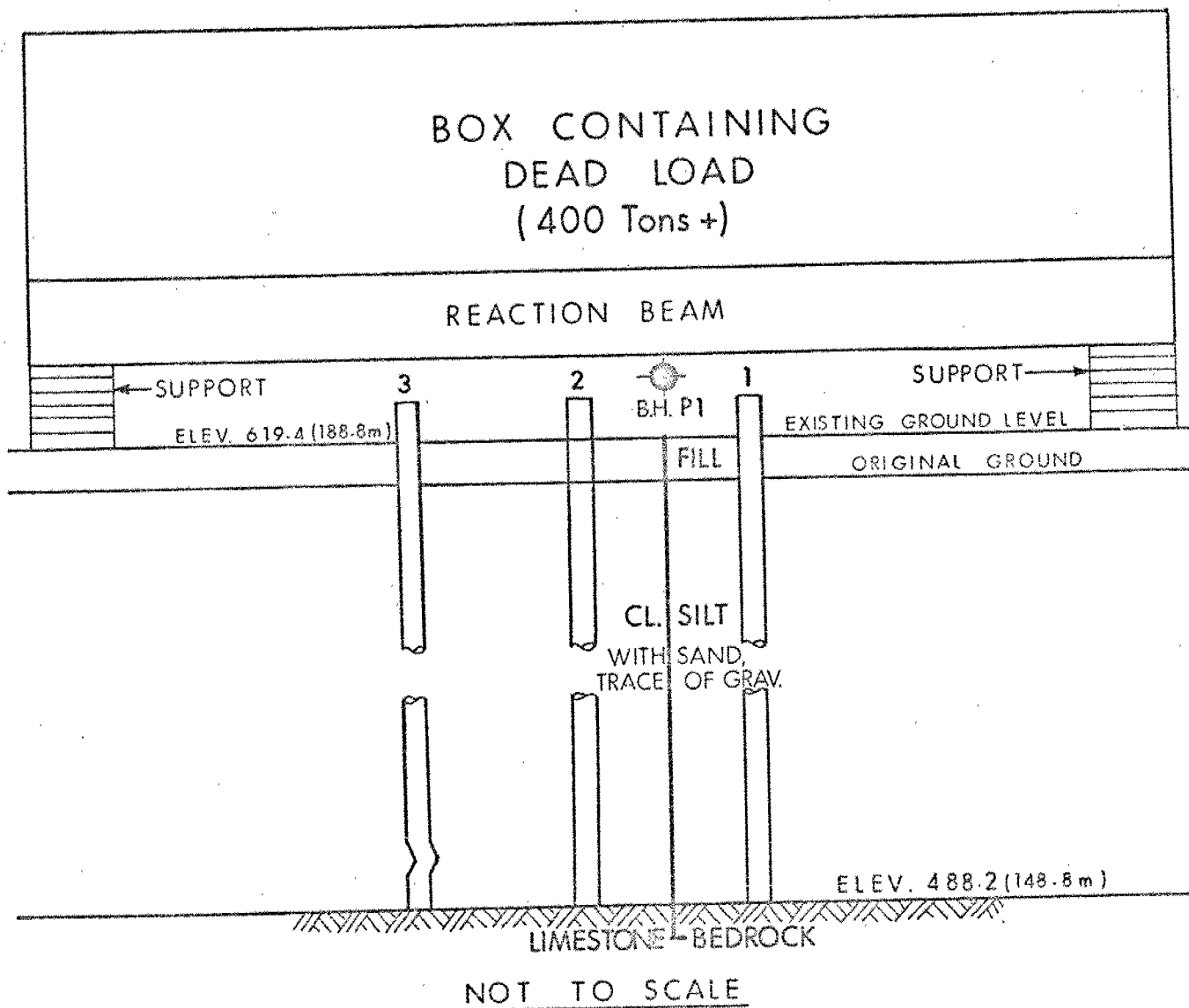
W.P. NO. 257 - 66 - 05

DIST. NO. 1

DATE: 7 JAN., 1975

W.O. NO. 74 - 11025

DRAWING NO. 74 - 11025 B



### SEQUENCE OF TESTING

- 1) PILE NO. 2
- 2) PILE NO. 1
- 3) PILE NO. 3 NOT TESTED

Fig. 4

REVISED: 7 JAN. 1975

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

CHESAPEAKE OHIO RAILWAYS & E.C. ROW EXPWY.  
PILE TEST ARRANGEMENT - ELEVATION

W.P. NO. 257-66-05

DIST. NO. 1

DATE June 3, 1974

W.O. NO. 74-11025

DRAWING NO 74-11025C

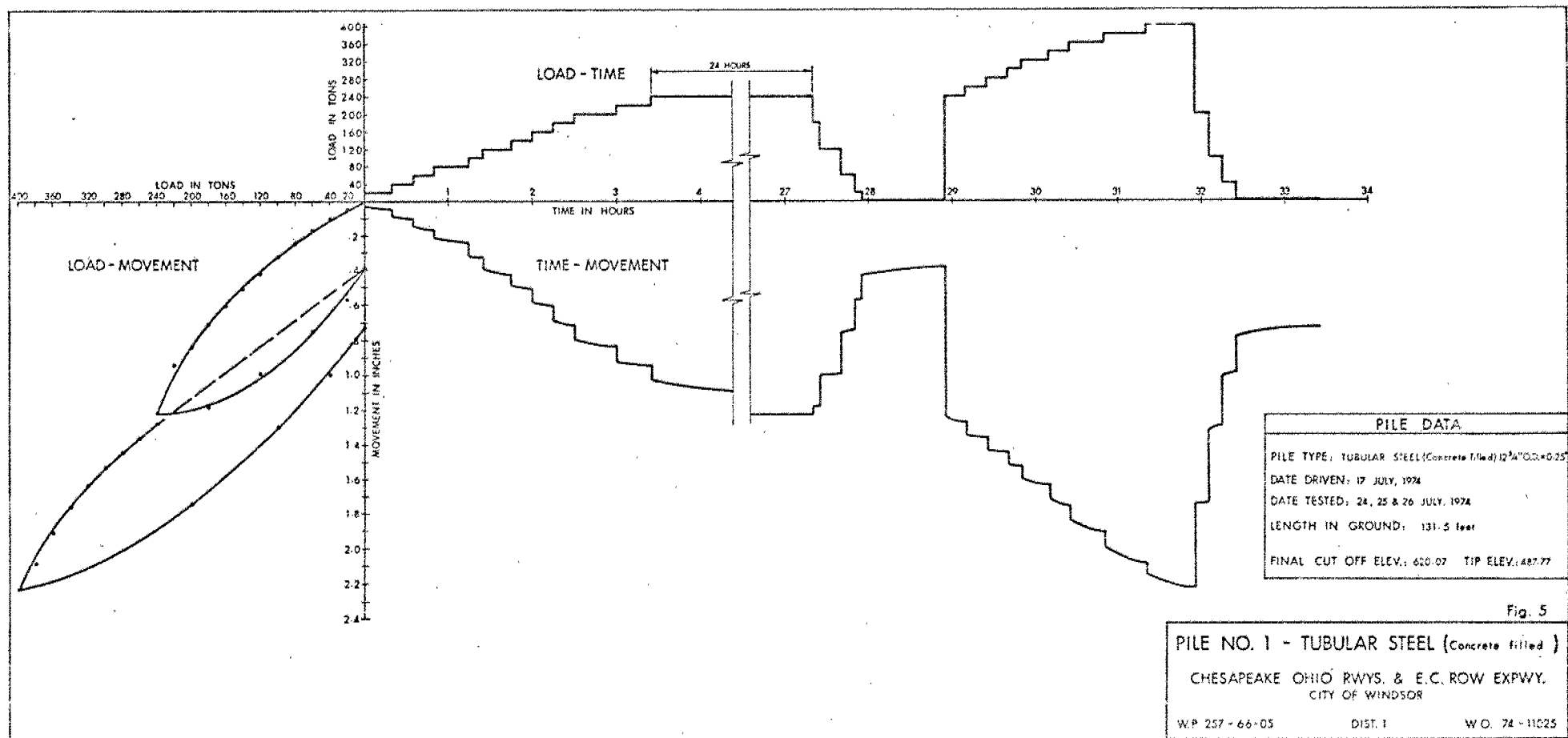


Fig. 5

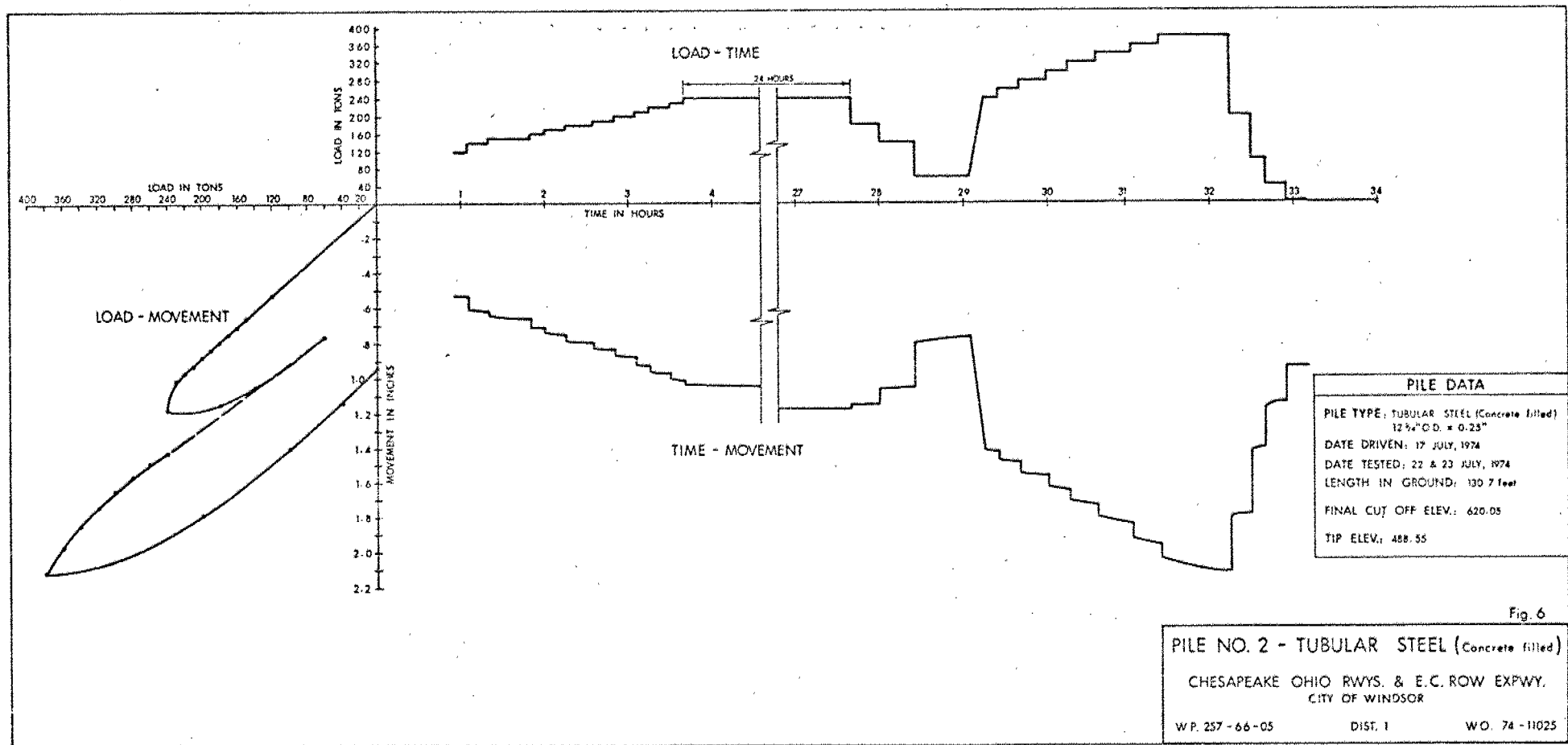


Fig. 6

ABBREVIATIONS & SYMBOLS USED IN THIS REPORTPENETRATION 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 :-

<u>CONSISTENCY</u>	<u>c LB./SQ.FT.</u>	<u>DENSENESS</u>	<u>'N' BLOWS / FT.</u>
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 "		

ABBREVIATIONS & SYMBOLS USED IN THIS REPORTSOIL PROPERTIES

$\gamma$	UNIT WEIGHT OF SOIL (BULK DENSITY)
$\gamma_s$	UNIT WEIGHT OF SOLID PARTICLES
$\gamma_w$	UNIT WEIGHT OF WATER
$\gamma_d$	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
$\gamma'$	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
$\tau_f$	SHEAR STRENGTH
$c'$	EFFECTIVE COHESION INTERCEPT
$\phi'$	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
$c_u$	APPARENT COHESION
$\phi_u$	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
$\mu$	COEFFICIENT OF FRICTION
$S_t$	SENSITIVITY

IN TERMS OF  
EFFECTIVE STRESS  
 $\tau_f = c' + \sigma' \tan \phi'$

IN TERMS OF  
TOTAL STRESS  
 $\tau_f = c_u + \sigma \tan \phi$

GENERAL

$\pi$	= 3.1416
e	BASE OF NATURAL LOGARITHMS 2.7183
$\log_e \sigma$ OR $\ln \sigma$	NATURAL LOGARITHM OF $\sigma$
$\log_{10} \sigma$ OR $\log \sigma$	LOGARITHM OF $\sigma$ 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
$\sigma$	NORMAL STRESS
$\bar{\sigma}$	NORMAL EFFECTIVE STRESS ( $\bar{\sigma}$ IS ALSO USED)
$\tau$	SHEAR STRESS
$\epsilon$	LINEAR STRAIN
$\gamma$	SHEAR STRAIN
$\nu$	POISSON'S RATIO ( $\mu$ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
$\eta$	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
$\delta$	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
$K_0$	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
$\beta$	ANGLE OF SLOPE TO HORIZONTAL



DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS & TESTING OFFICE

## RECORD OF BOREHOLE No. 68-(68-F-15-3) FOUNDATION SECTION

JOB 71-11111 LOCATION Co-ords. 100, 1287; 77, 4478. ORIGINATED BY A.P.  
 W.P. 257-66-05 BORING DATE April 1 & 2, 1968 COMPILED BY ANS  
 DATUM Geodetic BOREHOLE TYPE Cont. flight auger CHECKED BY LS

SOIL PROFILE		SAMPLES		ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT ——— PLASTIC LIMIT ——— WATER CONTENT ———		BULK DENSITY Y P.C.F.	REMARKS				
ELEV. DEPTH	DESCRIPTION	STRAT. NO.	NUMBER TYPE		SHEAR STRENGTH P.S.F.		WATER CONTENT %							
616.7	Ground level				500	1000	1500	2000	2500	10	20	30	GR. SA. SI. CL.	
0.0														
	Clayey silt with sand traces of gravel		1 SS 10	610										
			2 SS 39											
			3 SS 14											
			4 SS 29											
			5 SS											
	Hard to Very Stiff Silty		6 TW PH	600									137	2 28 47 23
			7 TW PH	590										
			8 TW PH							+1.7				
			9 TW PH	580						+1.6				
			10 TW PH							+1.0				135
	Sand layer			570										
566.7			11 TW PH	560										
50.0														
561.7														
55.0														
			12 TW PH	550									133	
			13 TW PH	540									131	
			14 TW PH	530										
			15 TW PH	520										

DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS & TESTING OFFICE

## RECORD OF BOREHOLE No. 70 (68-F015-3) FOUNDATION SECTION

JOB 70-11111 LOCATION Co-ords. 100, 001M, 77, 6768 ORIGINATED BY GRH  
 W.P. 257-66-05 BORING DATE April 2, 3, & 4, 1968 COMPILED BY AKS  
 DATUM Geodetic BOREHOLE TYPE Open Drill & Core Drill CHECKED BY X

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT ——— W <sub>L</sub> PLASTIC LIMIT ——— W <sub>P</sub> WATER CONTENT ——— W <sub>c</sub>			BULK DENSITY Y	REMARKS		
ELEV. DEPTH	DESCRIPTION	STRAIT. ROT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F.					WATER CONTENT %					
								20 40 60 80 100					10 20 30				
								500 1000 1500 2000 2500									
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE									
617.5	Ground level																
0.0																	
			1	SS	10												
			2	SS	57	610											
			3	SS	63												
			4	SS	24	600											
			5	SS	16												
			6	SS	16	590											
			7	SS	17												
			8	SS	14	580											
			9	SS	12												
						570											
			10	TW	PM												
						560											
			11	TW	PM												
						550											
546.5			12	TW	PM												
541.0	Sand seams					540											
540.5																	
536.5			13	TW	PM												
531.5	Sand layer					530											
531.5			14	TW	PM												
526.0						520											
			15	TW	PM												
						510											
						500											
			16	SS		490											
486.8																	
480.7	Limestone																
481.8	Bedrock		17	AXT	Roc												
455.7	End of Borehole			RC	98%	480											

DOCUMENT MICROFILMED IDENTIFICATION

GEOCRES No. 40 J 7-6.

DIST. 1 REGION

W.P. No. 257-66-05

CONT. No. 77-43.

W. O. No.

STR. SITE No.

HWY. No.

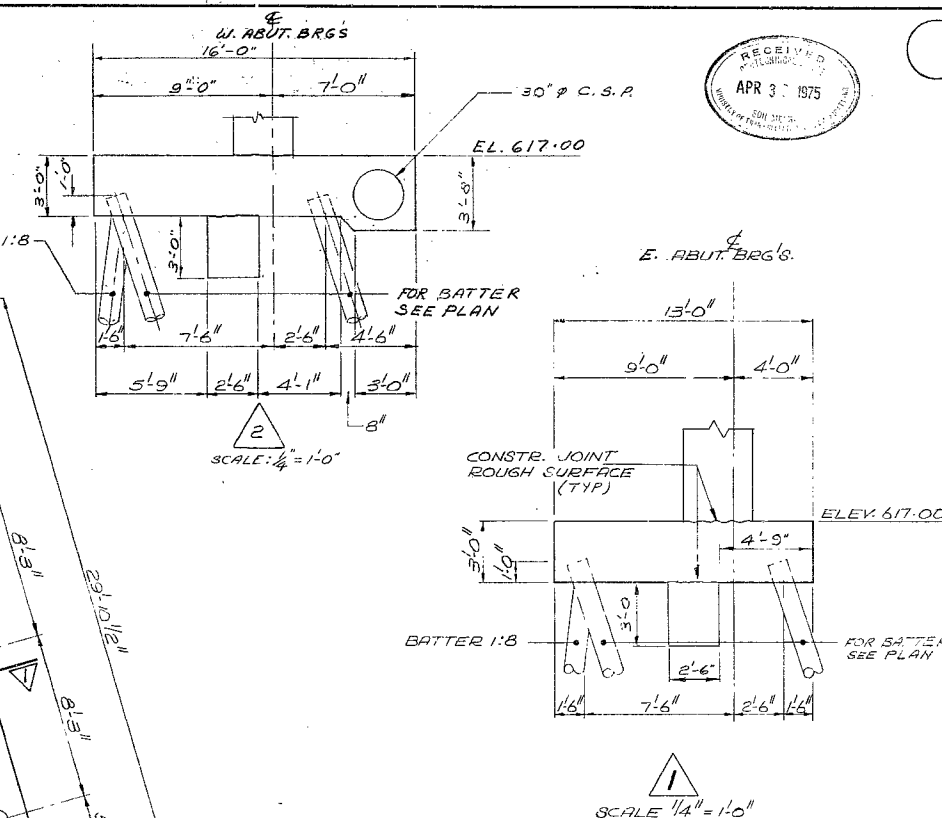
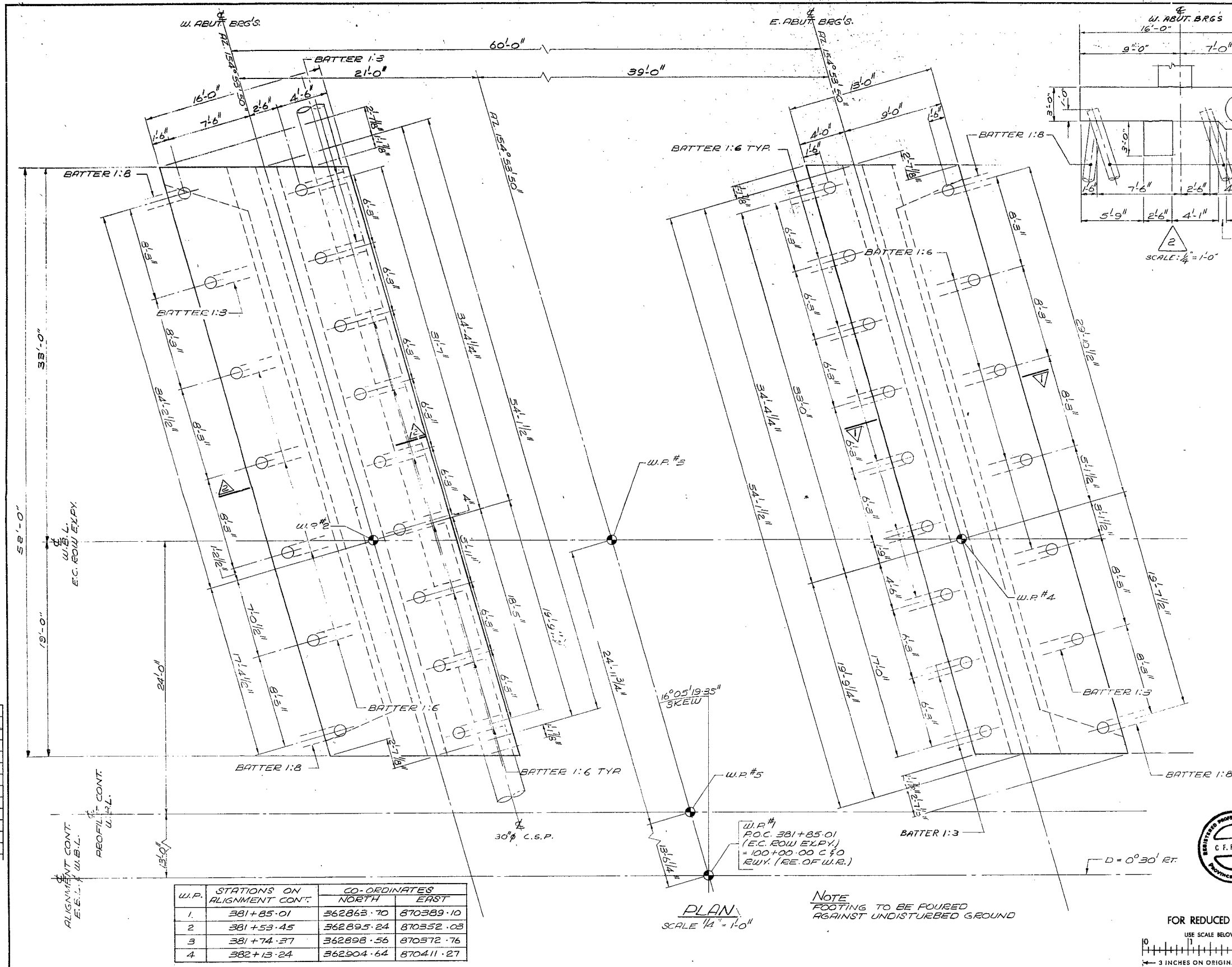
LOCATION CHESAPEAKE + OHIO

RAILWAY. X-ING OVERHEAD + PILE

LOADING TEST

OVERSEE DRAWINGS TO BE INCLUDED WITH THIS REPORT. 2

REMARKS:



NOTE: - DRIVING ENERGY MUST NOT EXCEED 30,000  
FT. LBS./BLOW WHEN PILES ARE PENETRATING  
THE ZONE BELOW EL. 495.00.

LIST OF 12 3/4" O.D. STEEL TUBE PILES			
LOCATION	Nº PILES	LENGTH	DESIGN LOAD
W. ABUT.	16	138'-0"	120 T. / PILE
E. ABUT.	16	138'-0"	120 T. / PILE

(WALL THICKNESS 1/4")

[illegible]

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS  
ONTARIO

C & C RAILWAY O'HEADS  
WESTBOUND LANES

KING'S HIGHWAY No. E.C. ROW EXPR. DIST. No. 1



CO. ESSEX

~~THE~~ CITY OF WINDSOR LOT 97 & 98 CON. 2

FOOTING / POINT

FOOTING LAYOUT

	CONTRACT No
---	-------------

APPROVED  CONTRACT NO. 

STRUCTURAL ENGINEER

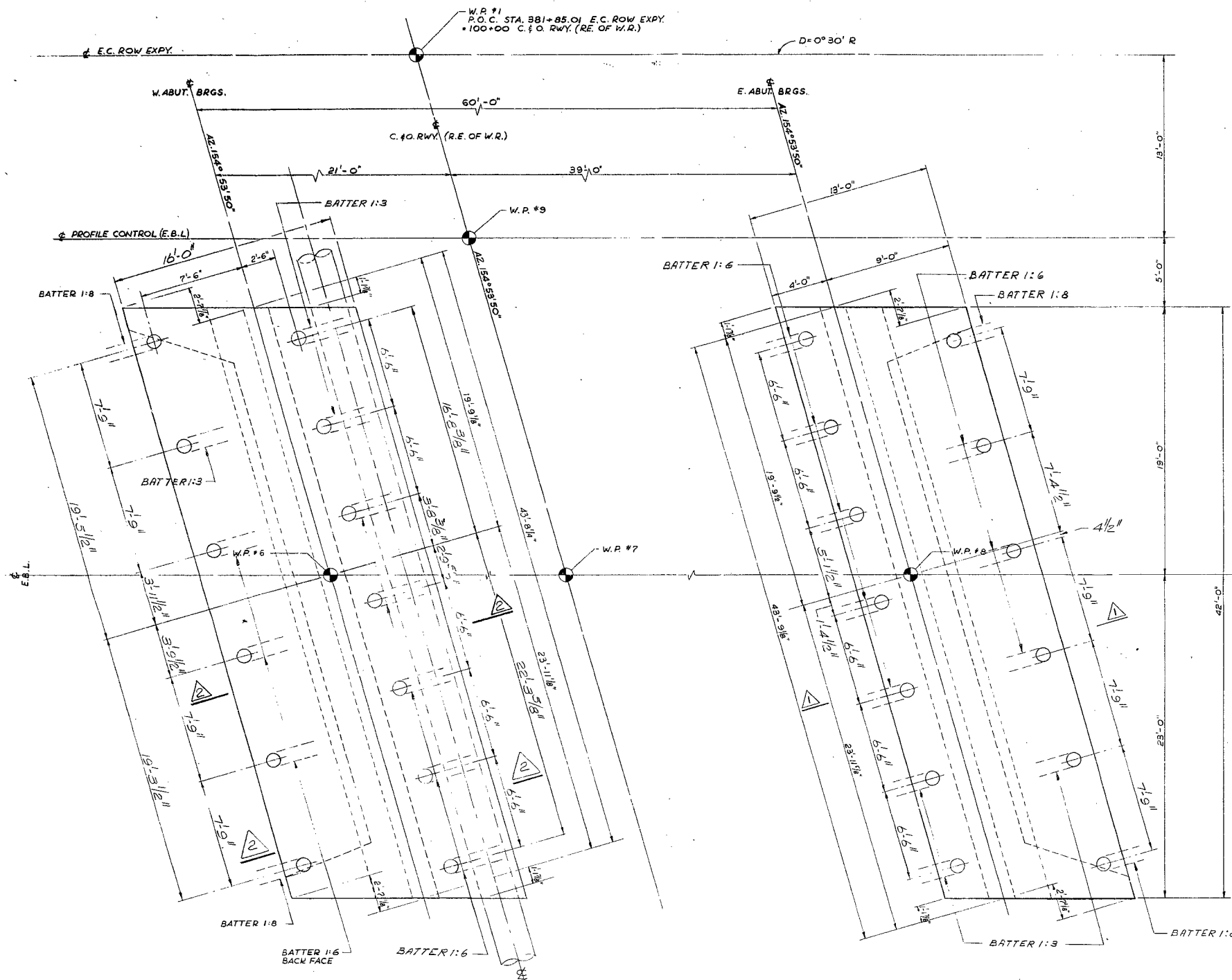
3	DESIGN	C.F.F.	CHECK	Sluc	W.P. No.	257-66-C
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DRAWING	E.O.N.	CHECK	C.F.F.	SHEET NO. 4-202-B	SHEET
---------	--------	-------	--------	-------------------	-------

DATE	AUG. 73	LOADING	HS-20-44	SITE NO.	0-292-8	SHEET
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4017-6

WBC



PLAN  
SCALE 1/4\"/>

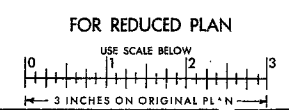
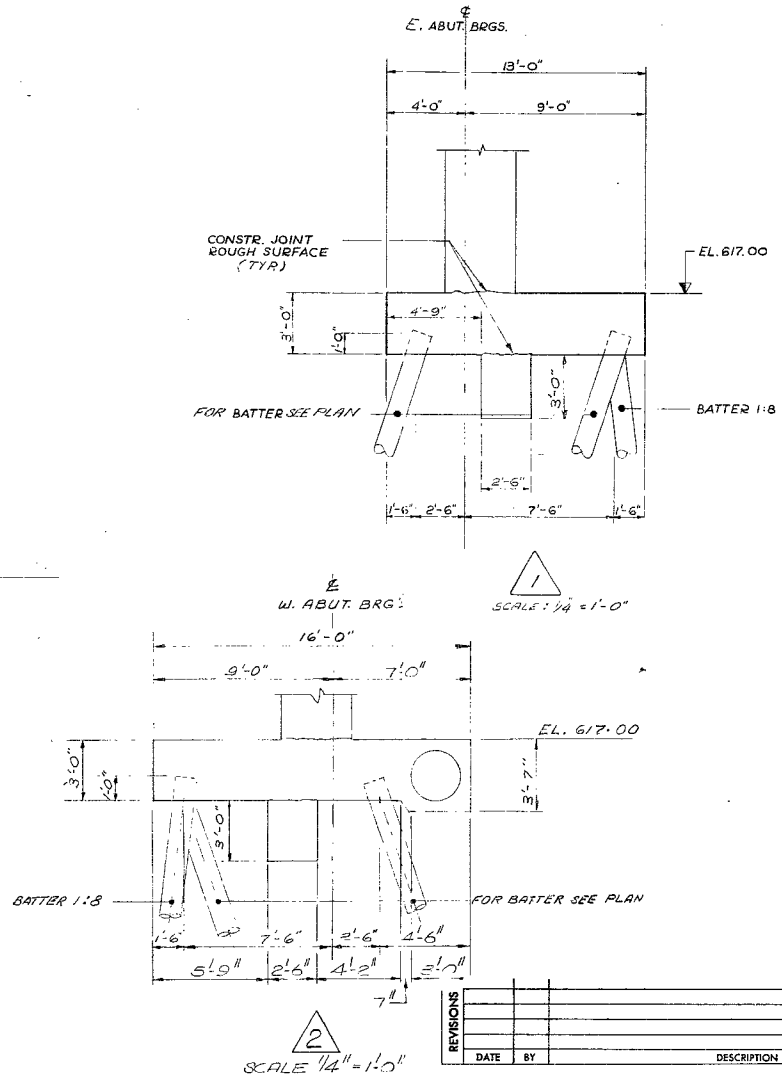
LIST OF 12 3/4\"/>

LOCATION	Nº PILES	LENGTH	DESIGN LOAD
W. ABUT.	12	138'-0"	120 T. / PILE
E. ABUT.	12	138'-0"	120 T. / PILE

NOTE: - DRIVING ENERGY MUST NOT EXCEED 30,000 FT. LBS/BLOW WHEN PILES ARE PENETRATING THE ZONE BELOW EL. 495.00

W.P.	STATIONS ON ALIGNMENT CONT.	CO-ORDINATES	
		NORTH	EAST
1	381+85.01	3628 63.70	8703 89.10
6	381+74.64	3628 25.53	8703 84.69
7	381+95.72	3628 28.82	8704 05.44
8	382+34.85	3628 34.83	8704 43.98

NOTE  
FOOTING TO BE POURED AGAINST UNDISTURBED GROUND.



REVISIONS		
DATE	BY	DESCRIPTION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS ONTARIO	
C&O RAILWAY O'HEADS EAST BOUND LANES	
KING'S HIGHWAY No. E.C. ROW EXPY.	DIST. No. 1
CO. ESSEX	
TOWN. CITY OF WINDSOR	LOT 97 & 98 CON. 2
FOOTING LAYOUT	
APPROVED <i>[Signature]</i>	CONTRACT No.
DESIGN C.F.F. CHECK C.F.F.	W.P. No. 257-66-05
DRAWING A.K. CHECK C.F.F.	SITE No. 6-292A SHEET 3
DATE AUG. 73	LOADING HS 20-44

4057-6

EPCL

DOCUMENT MICROFILMING IDENTIFICATION

G.I.-30 SEP 1976

GEOCRES No. 40I7-6

DIST. 1 REGION

W.P. No. 257-66-05

CONT. No. 77-43

W. O. No.

STR. SITE No.

HWY. No.

LOCATION Chesapeake + Ohio  
Railway X-ing Overhead +  
Pile Loading Test

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. 2

REMARKS: documents to be  
unfolded before microfilming

# FOUNDATION INVESTIGATION REPORT

For

The Proposed E.C.Row Expressway  
and Chesapeake and Ohio Railway  
Crossing - Lot 97 - Con. 2  
City of Windsor - County Essex  
District #1 - (Chatham)  
W.O. 71-11114 - W.P. 257-66-05

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## 1. INTRODUCTION:

A request for a foundation investigation at the crossing of the Proposed E. C. Row Expressway and C & O Railway was received from Mr. A. P. Watt, Regional Bridge Planning Engineer, in a memorandum dated October 12, 1971

A preliminary foundation investigation covering this area. was carried out in April 1968 (68-11015-3). A more detailed field investigation was subsequently carried out to determine the subsoil conditions existing at the site.

This report contains the results of both investigations and our recommendations pertaining to the design of the proposed structure foundations and approach embankments.

## 2. DESCRIPTION OF THE SITE:

The site of the proposed overpass structure is situated in the eastern part of the City of Windsor, approx. 0.2 miles east of the intersection of the existing E. C. Row Blvd. and Walker Rd.

The surrounding terrain, with the exceptions of the approx. 2 ft. high railway embankment and the approx. 3 ft. deep drainage ditches, is flat and cultivated farmland.

Physiographically the site is located in the region referred to as the St. Clair Clay Plain.

.....2

### 3. FIELD AND LABORATORY INVESTIGATION PROCEDURES:

A total of 7 samples boreholes and 12 dynamic cone penetration tests was carried out during the course of the field work. Boring was achieved by means of bombardier mounted continuous flight auger machines, and conventional diamond drilling equipment adapted for soil sampling purposes. During the field work, disturbed samples were obtained by means of a standard split-spoon sampler; the energy used in driving it conformed to the requirements of the Standard Penetration Test. 'Undisturbed' samples were recovered using 2 inch I.D. shelby tubes which were pushed into the soil hydraulically or by hand. Where possible, field vane tests were carried out at elevations generally 12 inches below sample depths.

Dynamic cone penetration tests were carried out adjacent to each borehole with the exception of B.H.#67, and also at 6 other locations. Driving energy to advance the cone was 350 ft.-lbs. per blow.

The bedrock was proved at two borehole locations using AXT Rock Coring equipment.

All boreholes were surveyed in the field by personnel from London Region Engineering Surveys Section. The locations and elevations of the borings are shown on Drawing No. 71-11114A which accompanies this report.

All samples were visually examined and classified at the site as well as in the laboratory. Following this inspection laboratory tests were carried out on selected samples to determine the following physical properties:

- Atterberg Limits
- Moisture Content
- Grain-Size Distribution
- Undrained Shear Strength
- Bulk Density

The test results are summarized on the record of borehole sheets contained in the Appendix of this report.



#### 4. SOIL TYPES AND SOIL CONDITIONS:

##### 4.1) General:

Generally uniform subsoil conditions were found to prevail over the site area. The subsoil consists of a deep deposit of cohesive soil, followed by limestone bedrock. The boundaries between different deposits are shown on the record of borehole sheets attached to the Appendix. The estimated stratigraphical profile of Drawing 71-11114A is based upon this information.

From ground level downward, the various strata are described in some detail with regard to soil types and soil properties, as follows:

##### 4.2) Clayey silt with sand and traces of gravel:

This deposit was intersected in all borings and extends from immediately below the ground surface, down to the bedrock surface for a minimum depth of 130 feet. The material in the deposit consists of clayey silt with sand and traces of gravel. A plot of Plasticity Index versus Liquid Limit (Fig.1) shows the points to fall within the CL zone. In some boreholes relatively thin layers of granular soils were found to occur within the main deposit.

A highly overconsolidated zone, due to desiccation and/or weathering, with a thickness ranging from 7 to 10 feet, was found to extend from the upper surface of the stratum. This zone is brown in color due to oxidation and apart from the upper 3 to 6 feet (frost affected zone) has a very stiff to hard consistency: 'N' values ranged from 21 to 83 blows per foot. Based on the Standard Penetration Test results only, the undrained shear strength of this desiccated zone is estimated to be in the order of 2,500 PSF to 10,000 PSF. Below the desiccated layers the color of the soil is grey and the consistency ranges somewhat randomly from stiff to hard. For design purposes the following undrained shear strength values are suggested:

Ground Level - El. 611 →	2,000 PSF
El. 611 - El. 603	5,000 PSF
603 - 590	2,500 PSF
590 - 570	1,750 PSF
570 - 540	1,250 PSF
540 - 487	1,500 PSF

Physical properties of the overall deposit, as determined from field and laboratory tests, are as follows:

Natural Moisture Content: (%)	9.9 to 18.4
Liquid Limit: (%)	19.6 to 30.4
Plastic Limit: (%)	11.7 to 16.7
Bulk Density: (PCF)	131.0 to 137.5
Unconfined Shear Strength: (PSF)	988 to 2620
Field Vane Test: (PSF)	960 to 2000 +
Sensitivity:	1.2 to 1.9
'N' Value: (Blows/Ft.)	10 to 83

Typical grain-size distribution curves are included in the Appendix of this report (Fig.2).

#### 4.3) Limestone Bedrock:

Bedrock at this site was found to consist of generally sould limestone at el. 487 (B.H.'s 68 & 70).

#### 5.) GROUNDWATER CONDITIONS:

The following groundwater levels were observed during the field investigation:

B.H.#67	El: Not Established
68	Not Established
70	Not Established
73	Not Established
77	Not Established
139	El: 602.8
144	595.9

It is pointed out that the foregoing quoted figures may not represent the true groundwater levels, due to the relatively impermeable nature of the subsoil and the short duration of the field work.

## 6.) DISCUSSION AND RECOMMENDATIONS:

### 6.1) General:

It is proposed to build a three-span (44'-53'-44') twin-structure overpass at the crossing of the E. C. Row Expressway and the C. & O. Railway. The proposed profile grade of the E. C. Row Expressway will be approximately 27 ft. above the existing C. & O. Railway grade of elevation 620±.

As described in the previous paragraphs of this report, the subsoil at the site consists of a deep deposit of clayey silt with sand and traces of gravel, underlain by limestone bedrock. The upper 14 to 17 feet of the deposit is a very stiff to hard desiccated surface crust. Below this depth the undrained shear strength of the material decreases. The desiccated surface crust appears to be suitable for spread footing type foundations.

Because of the compressible nature of the subsoil, it is inevitable that consolidation settlements will occur over a longterm period due to the imposed loads of structure and embankment. Past experience, however, indicates that these settlements will be of a minor nature.

### 6.2) Foundations:

#### a) Spread Footings in Original Ground:

The entire structure may be supported on spread footings placed within the very stiff to hard desiccated zone of the subsoil between El. 611 and El. 603. A safe net pressure of 3.5 TSF may be assumed for design purposes.

The desiccated zone is susceptible to softening on contact with water, therefore, it is recommended that the base of the footing excavations be protected by concrete working slab, immediately on exposure.

All foundations should be protected against frost action by at least 4 feet of earth cover. No dewatering problems are anticipated.

The estimated maximum settlement will be in the order of 1.0 to 1.5 inches under the pier footings.

b) Spread Footings on Compacted Fill:

As an alternative, the abutments may be supported on spread footings placed on well compacted, suitable granular material within the approach fills. A safe design load of 2.0 TSF may be assumed. The granular material should consist of G.B.C. Class 'A' and should be fully compacted according to the current Standards. A detailed construction scheme is outlined on Figure 3 of the Appendix.

c) Perched Abutments on Short Piles:

As a second alternative, the abutments may be constructed within the approach fills and supported on short piles driven through the fill and some 10.0 feet into the ground. In the case of 12-3/4" O.D. and 1/4" thick wall piles, a safe design load of 25 tons per pile may be assumed.

It should be pointed out, that this proposal is based on experience with similar structures and similar subsoil conditions in the general area.

Regardless of which method is adopted (a, b or c) the structure should be built to accommodate the 3.0 to 3.5 inches differential settlement between the abutments and piers.

d) End-Bearing Piles:

As another alternative, the abutments and piers may be supported on steel H-piles driven to bedrock. The maximum allowable load for the particular steel section may be assumed for design purposes.

6.3) Approach Embankments:

The shear strength of the subsoil is such that it will be able to safely support the 31 ft. high approach embankments constructed with 2:1 side slopes. The fill should consist of well compacted acceptable material. Care should be taken to ensure that no bouldery fill is placed within the approaches through which piles have to be driven, and it is recommended that this portion of the fill contain no larger grain sizes than 3 inches.

Based on performance of structures and embankments built in the same general area and under somewhat similar subsoil conditions, it is estimated that a maximum settlement of 4 to 5 inches will take place over a long period of time under the fill at the abutment location. To minimize the effect of differential settlements between the abutments and pier footings, it is recommended that the approach embankments be built in advance of the structure for as long a period as possible. The topsoil and the soft surficial material should be removed in accordance with the pertinent Standards within the construction area.

7. MISCELLANEOUS:

The field investigation was carried out during the period April 1 to 4, 1968, and November 17 to 19, 1971, under the supervision of Mr. A. Prakash and Mr. P. Payer, Project Foundation Engineers.

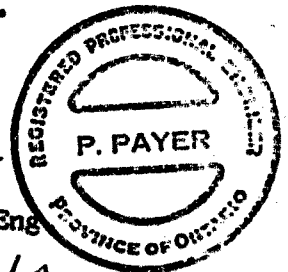
Equipment was owned and operated by Dominion Soil Investigation Ltd. and Master Soil Investigation Ltd.

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

December 10, 1971

*P. Payer*  
P. Payer, P. Eng.

*K. G. Selby*  
K. G. Selby, P. Eng.



DEPARTMENT OF HIGHWAYS- ONTARIO  
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 67 (68-F-15-3) FOUNDATION SECTION

JOB 71-11111 LOCATION Co-ords. 100, 006N; 77, 491E ORIGINATED BY A.P.  
W.P. 257-66-05 BORING DATE April 1 & 2, 1968 COMPILED BY AMS  
DATUM Geodetic BOREHOLE TYPE Cont. flight auger CHECKED BY K.K.

SOIL PROFILE		SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — $w_L$ PLASTIC LIMIT — $w_p$ WATER CONTENT — $w$			BULK DENSITY $\gamma$	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		20	40	60	80	100	$w_p$	$w$	$w_L$		
616.7	Ground level					SHEAR STRENGTH P.S.F.					WATER CONTENT %			P.C.F.	GR. SA. SI. CL.
0.0	Clayey silt with sand traces of gravel.  Hard to Very Stiff Stiff		1	SS	12										2 29 40 29
			2	SS	38										
			3	SS											
			4	SS											
			5	SS											
			6	SS											
			7	TW	PH										
			8	TW	PH			6		+1.4				134	
			9	TW	PH										
			10	TW	PH										
			11	SS	14										
			12	TW	PH									134	
563.7	End of Borehole														
53.0															

DEPARTMENT OF HIGHWAYS- ONTARIO  
MATERIALS & TESTING OFFICE

## RECORD OF BOREHOLE No. 68-(68-F-15-3)

FOUNDATION SECTION

JOB 71-11114

LOCATION Co-ords. 100, 128N; 77, 447E.

ORIGINATED BY A.P.

W.P. 857-66-05

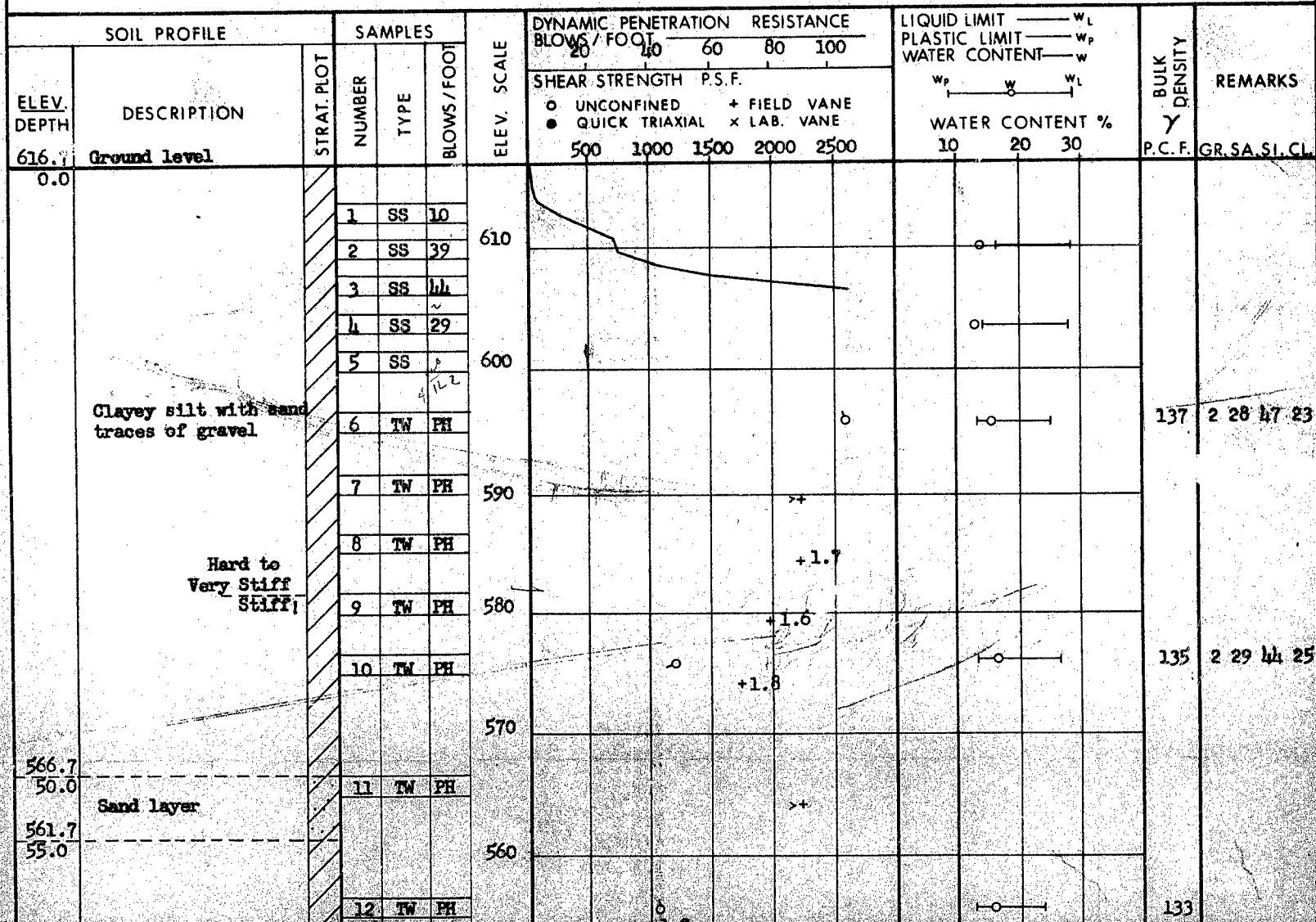
BORING DATE April 1 &amp; 2, 1968

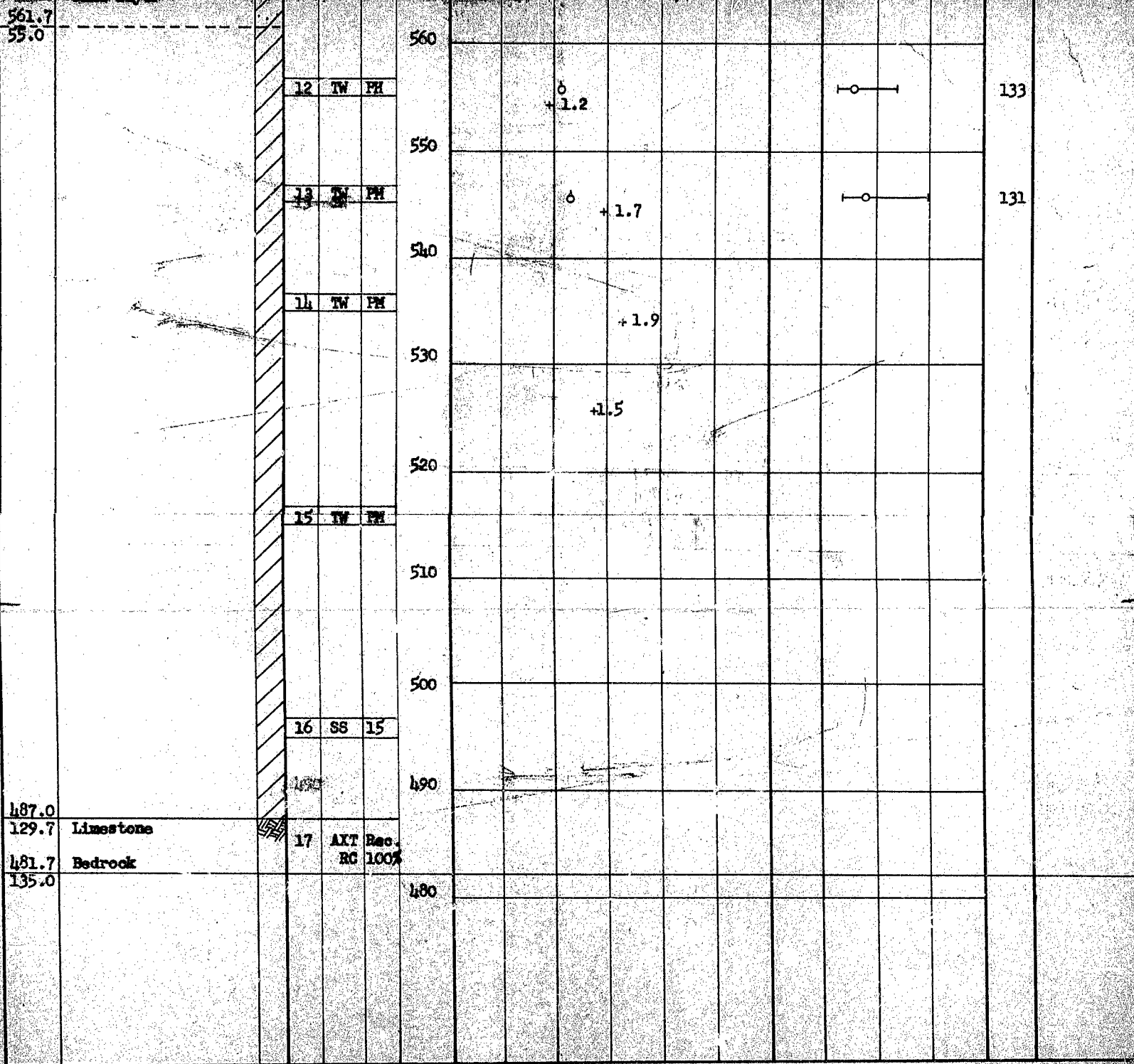
COMPILED BY AMS

DATUM Geodetic

BOREHOLE TYPE Cont. flight auger

CHECKED BY







DEPARTMENT OF HIGHWAYS- ONTARIO  
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 70 (68-F015-3)

FOUNDATION SECTION

JOB 71-11114  
W.P. 68-66-05  
DATUM Geostic

LOCATION Co-ords. 108, 004M; 77, 676E.  
BORING DATE April 2, 3, & 4, 1968  
BOREHOLE TYPE Penn Drill & Core Drill

ORIGINATED BY GSH  
COMPILED BY AMS  
CHECKED BY X

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT ——— $w_L$ PLASTIC LIMIT ——— $w_p$ WATER CONTENT ——— $w$			BULK DENSITY $\gamma$ P.C.F.	REMARKS					
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		BLOWS / FOOT					SHEAR STRENGTH P.S.F.					WATER CONTENT %				
							20 40 60 80 100					O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE					$w_p$ ——— $w$ ——— $w_L$				
												500 1000 1500 2000 2500					10 20 30				
617.5	Ground level																				
0.0																					
			1	SS	10																
			2	SS	57	610															
			3	SS	63																
			4	SS	24	600															
			5	SS	16																
			6	SS	16	590															
			7	SS	17																
			8	SS	14	580															
			9	SS	12																
						570															
			10	TV	PM																
						560															
			11	TV	PM																

Clayey silt with  
sand, traces of  
gravel  
Hard to  
Very Stiff  
Stiff

2 28 42 28

2 29 42 27



DEPARTMENT OF HIGHWAYS- ONTARIO  
MATERIALS & TESTING OFFICE

# RECORD OF BOREHOLE No. 73 ( 68-F-15-3) FOUNDATION SECTION

JOB 71-11114 LOCATION Co-ords. 100, 111N; 77, 660E. ORIGINATED BY GEH  
W.P. 257-66-05 BORING DATE April 3, 1968 COMPILED BY AMS  
DATUM Geodetic BOREHOLE TYPE Continuous Flight Auger CHECKED BY KV

SOIL PROFILE		SAMPLES		ELEV. SCALE	DYNAMIC PENETRATION BLOWS / FOOT 20 40 60 80 100	RESISTANCE 500 1000 1500 2000 2500	LIQUID LIMIT — $w_L$ PLASTIC LIMIT — $w_p$ WATER CONTENT — $w$ $w_p$ — $w$ — $w_L$ WATER CONTENT % 10 20 30	BULK DENSITY $\gamma$ P.C.F. GR. SA. SI. CL.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER						
616.3	Ground level								
0.0									
	Clayey silt with sand Traces of gravel  Hard to Very Stiff Stiff		1	SS	59	610			
			2	SS	83				
			3	SS	34	600			
			4	SS	22				
			5	SS	20	590			
			6	SS	19				
			7	TW	PH	580			
			8	TW	PH				
			9	TW	PH				
563.3									
53.0	End of Borehole					560			

RECORD OF BOREHOLE No. 77 (68-F-15-3)

FOUNDATION SECTION

JOB 71-11114 LOCATION Co-ords. 100, 064N; 77, 550E ORIGINATED BY AP  
W.P. 257-66-05 BORING DATE April 4, 1968 COMPILED BY AMS  
DATUM Geodetic BOREHOLE TYPE Cont. Flight Auger CHECKED BY NK

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — $w_L$ PLASTIC LIMIT — $w_p$ WATER CONTENT — $w$			BULK DENSITY $\gamma$ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		20	40	60	80	100	$w_p$	$w$	$w_L$		
618.9	Ground level															
0.0																
			1	SS	22											
			2	SS	80	610										
			3	SS	80											3 27 42 28
			4	SS	34	600										
	Clayey silt with sand, traces of gravel		5	SS	24											
			6	SS	21	590										
	Hard to Very Stiff		7	TW	PH											
	Stiff		8	TW	PM	580									137	4 30 27 29
			9	TW	PM	570										
568.1	Sand layer															
567.1																
			10	TW	PM	560										
555.9																
63.0	End of Borehole					550										

FOUNDATION SECTION

CHECKED BY *K. K.*

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION BLOWS / FOOT 20 40 60 80 100 SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE	LIQUID LIMIT ——— $w_L$ PLASTIC LIMIT ——— $w_p$ WATER CONTENT ——— $w$ $w_p$ — $w$ — $w_L$ WATER CONTENT %	BULK DENSITY $\gamma$ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT					
618.8	Ground level									
0.0	Probable clayey silt with sand and trace of gravel					610				
604.8										
14.0	End of Conehole					600				

### FOUNDATION SECTION

JOB 71-11114 LOCATION Co-ords. 100, 008N; 77, 556E. ORIGINATED BY P.P.  
W.P. 257-66-05 BORING DATE Nov. 17, 1971 COMPILED BY P.P.  
DATUM Geodetic BOREHOLE TYPE Cone Test Only CHECKED BY N.L.

SOIL PROFILE			SAMPLES			ELEV. SCALE ELEV. / FEET	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	LIQUID LIMIT ——— W <sub>L</sub>	PLASTIC LIMIT ——— W <sub>P</sub>	WATER CONTENT ——— W	BULK DENSITY $\gamma$ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		SHEAR STRENGTH P.S.F.	WATER CONTENT %				
618.7	Ground level						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE	W <sub>p</sub> ——— W ——— W <sub>L</sub>				
0.0	Probable clayey silt with sand and trace of gravel					610	(Roadway)					
603.7												
15.0	End of Conehole					600						

## FOUNDATION SECTION

CHECKED BY: N.K.

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION BLOWS / FOOT 20 40 60 80 100	RESISTANCE	LIQUID LIMIT — $w_L$ PLASTIC LIMIT — $w_p$ WATER CONTENT — $w$	BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT						
616.3	Ground level										
0.0	Probable clayey silt with sand and trace of gravel					610					
604.3	End of Conehole					600					

## FOUNDATION SECTION

CHECKED BY N.K.

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT ——— $w_L$ PLASTIC LIMIT ——— $w_p$ WATER CONTENT ——— $w$			BULK DENSITY $\gamma$ P.C.F.	REMARKS			
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT 20 40 60 80 100							WATER CONTENT % $w_p$ ——— $w$ ——— $w_L$ 10 20 30		
							SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE									
616.4	Ground level															
0.0	Clayey silt with sand and trace of gravel V. Stiff To Hard		1	SS	21	610										
2			SS	12												
3			SS	35												
4			SS	19												
5			SS	62												
6			SS	40												
7			SS	29												
8			SS	29												
9			SS	23												
592.9					10									SS	18	600
21.5	End of Borehole					590										



DEPARTMENT OF HIGHWAYS- ONTARIO  
MATERIALS & TESTING OFFICE

## RECORD OF BOREHOLE No. 140


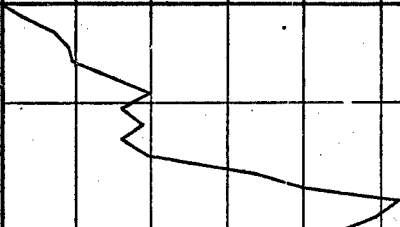
FOUNDATION SECTION

JOB 71-11114LOCATION Co-ords. 100, 124N; 77, 487EORIGINATED BY P.P.W.P. 257-66-05BORING DATE Nov. 18, 1971COMPILED BY P.P.DATUM GeodeticBOREHOLE TYPE Cone Test OnlyCHECKED BY K.K.

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT — $w_L$		BULK DENSITY $\gamma$	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		BLOWS/FOOT	20 40 60 80 100	PLASTIC LIMIT — $w_p$	WATER CONTENT — $w$		
616.1	Ground level											
0.0	Probable clayey silt with sand and trace of gravel					610						
604.4												
12.0	End of Cone Test					600						

### FOUNDATION SECTION

CHECKED BY *N.V.*

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION . RESISTANCE	LIQUID LIMIT ——— $w_L$	BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT	PLASTIC LIMIT ——— $w_p$		
616.7	Ground level						SHEAR STRENGTH P.S.F.	$w_p$ ——— $w$ ——— $w_L$		
0.0	Probable clayey silt with sand and trace of gravel					610				
601.7						600				
15.0	End of Cone Test									

DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS & TESTING OFFICE

## RECORD OF BOREHOLE No. 144

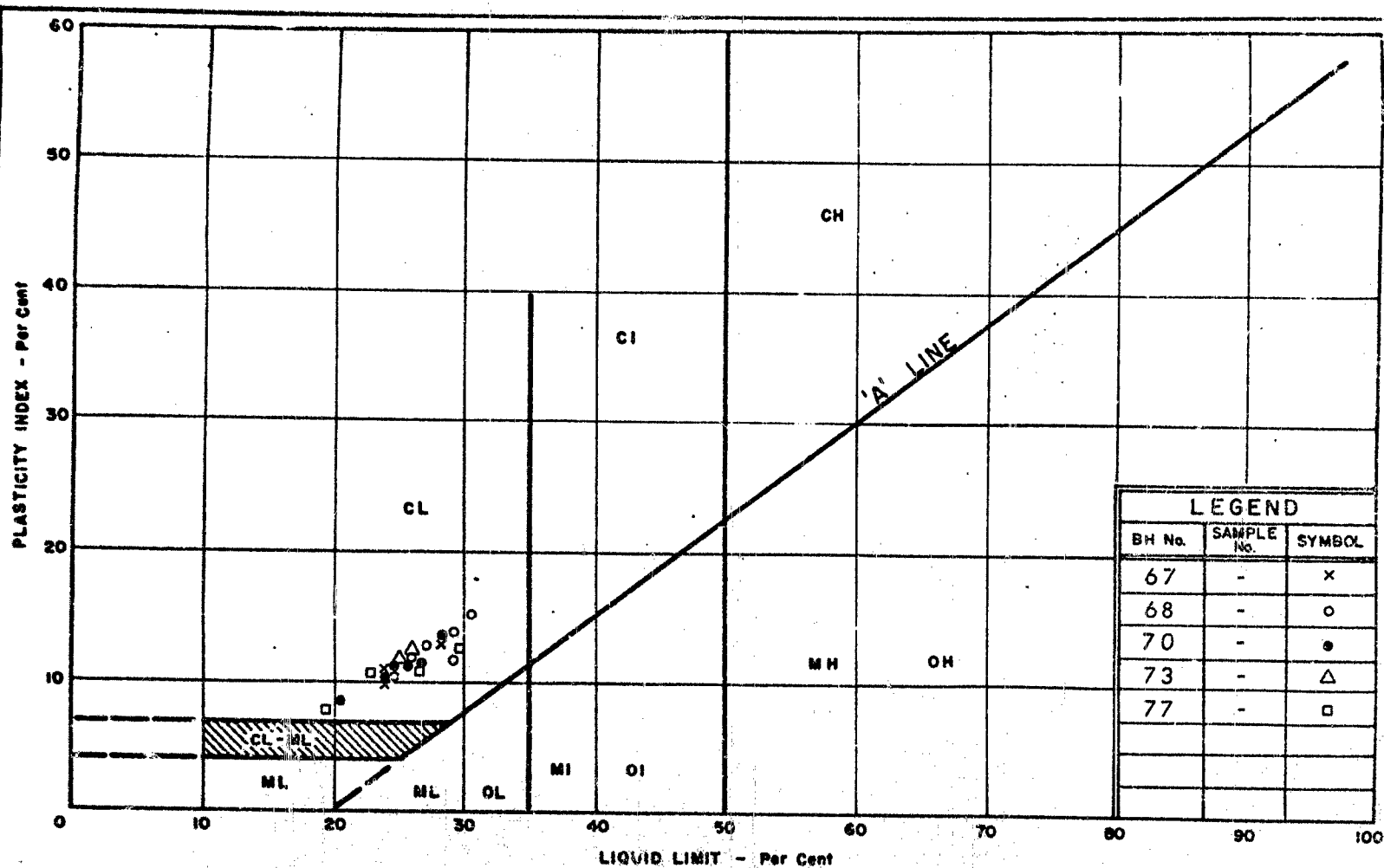
FOUNDATION SECTION

JOB 71-1111h LOCATION Co-ords. 100, 063N; 77, 640E ORIGINATED BY P.P.  
 W.P. 257-66-05 BORING DATE Nov. 18, 1971 COMPILED BY P.P.  
 DATUM Geodetic BOREHOLE TYPE Cont. flight auger CHECKED BY R. J.

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT — $w_L$ PLASTIC LIMIT — $w_p$ WATER CONTENT — $w$		BULK DENSITY $\gamma$ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		20	40	60	80		
616.4	Ground level											
0.0	Clayey silt with sand and trace of gravel Stiff to Hard		1	SS	21							
			2	SS	72							
			3	SS	44							
			4	SS	46							
			5	SS	70							
			6	SS	39							
			7	SS	21							
			8	SS	18							
			9	SS	19							
594.9			10	SS	15							
21.5	End of Borehole											

V.L.  
595.9'





DEPARTMENT OF HIGHWAY  
MATERIALS and  
TESTING  
DIVISION

# PLASTICITY CHART CLAYEY SILT

WP No. 257-66-05

JOB No. 71-11114

FIG. 1

# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND

GRAVEL

Fine

Medium

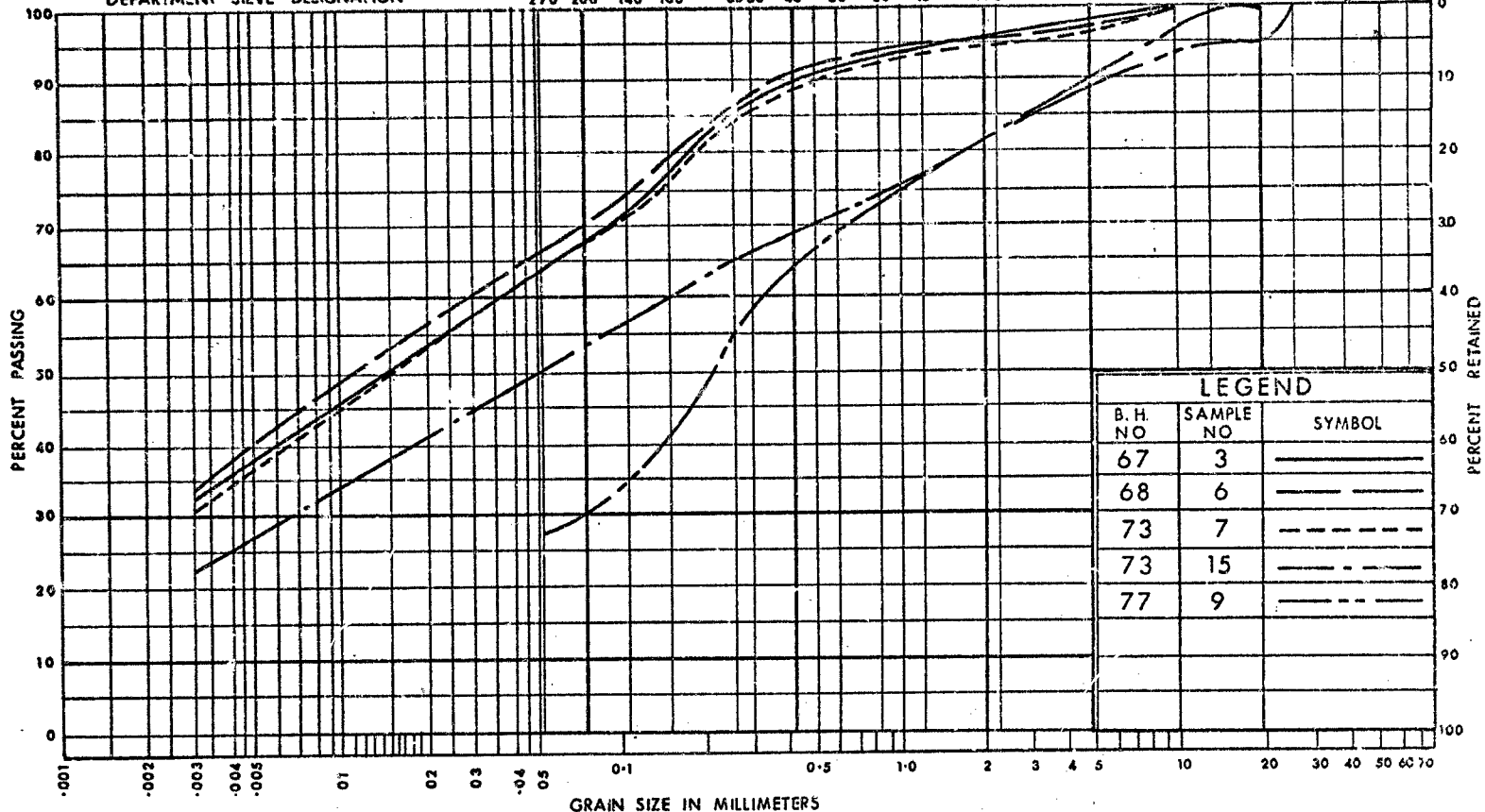
Coarse

Fine

Coarse

DEPARTMENT SIEVE DESIGNATION

270 200 140 100 60 50 40 30 20 16 10 8 4 3/4" 1/2" 3/8" 1" 1 1/2" 2" 2 1/2" 3"



## LEGEND

B. H. NO	SAMPLE NO	SYMBOL
67	3	—————
68	6	—————
73	7	-----
73	15	-----
77	9	-----

DEPARTMENT  
OF  
TRANSPORTATION AND COMMUNICATIONS



DESIGN SERVICES  
BRANCH

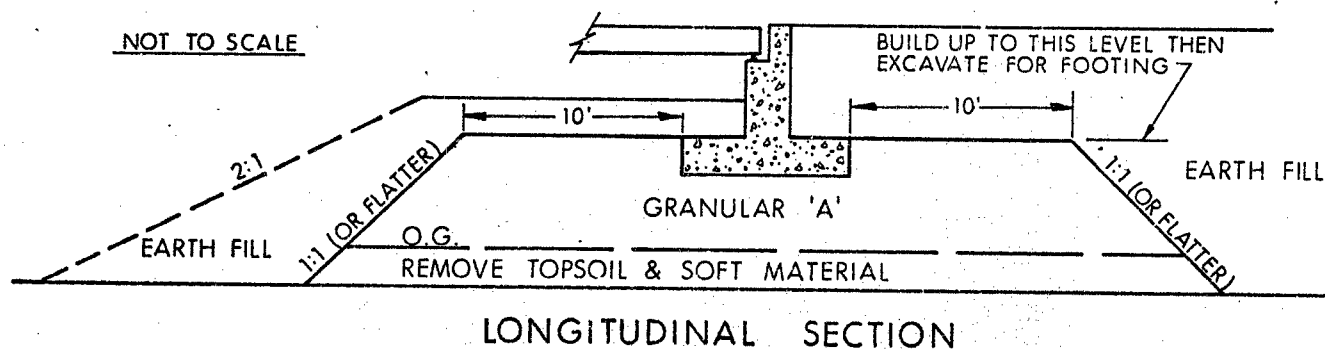
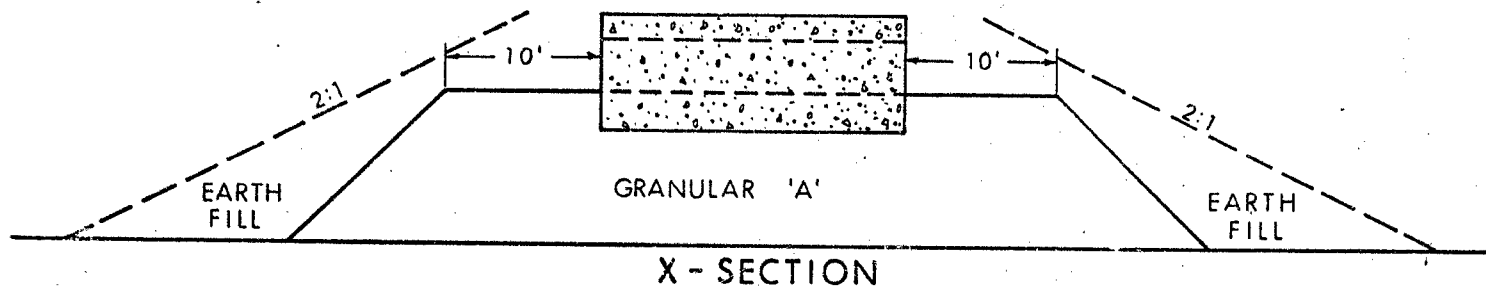
GRAIN SIZE DISTRIBUTION  
CLAYEY SILT

W.P. No. 257-66-05

JOB No. 71-11114

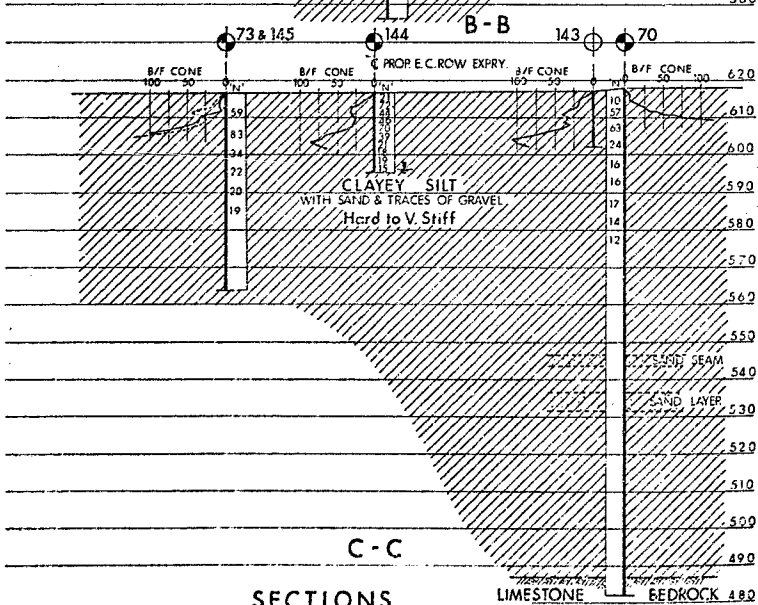
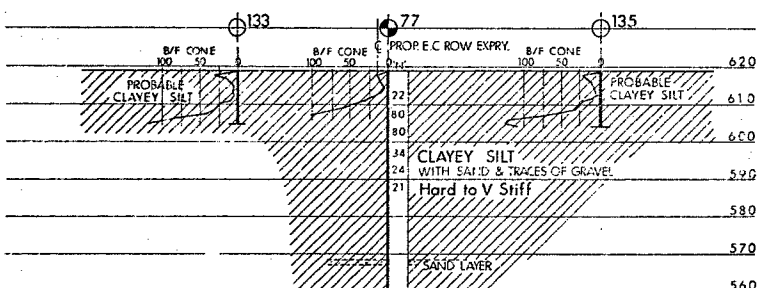
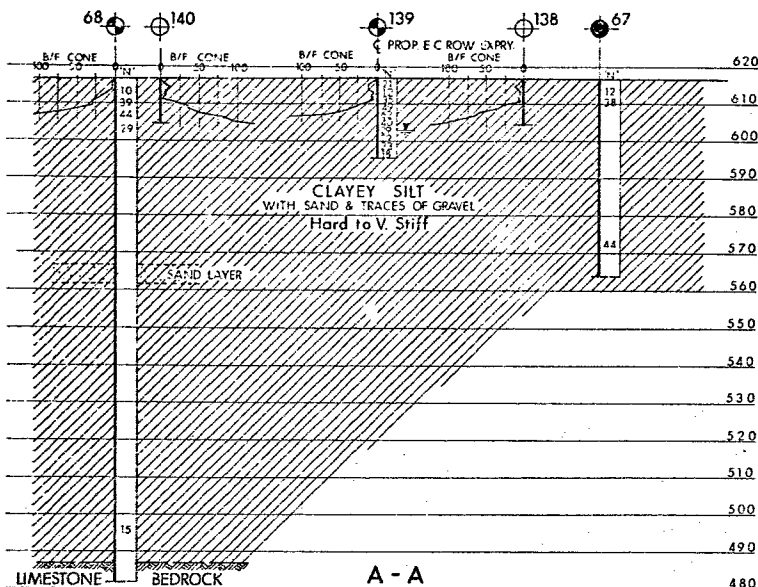
FIG. 2

## ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE



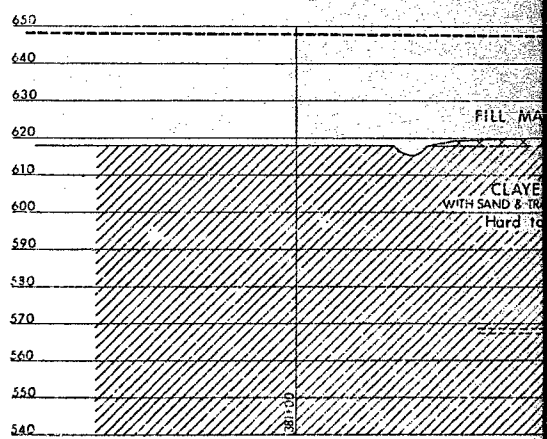
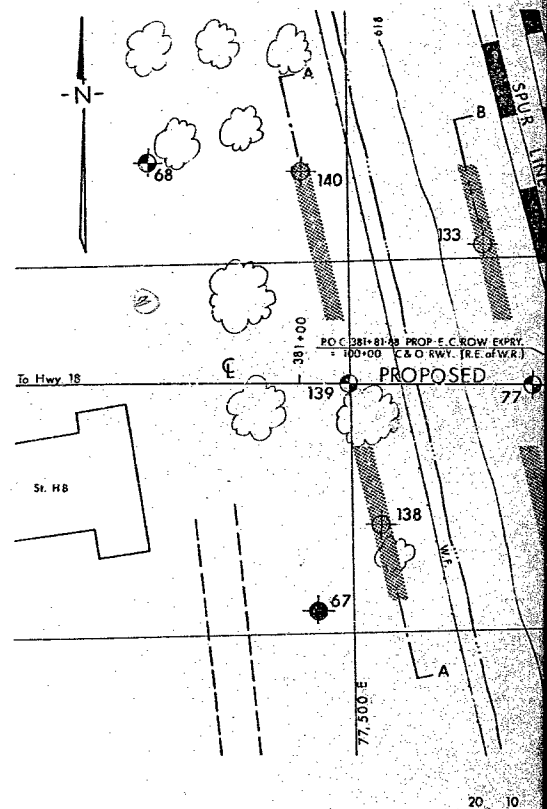
### NOTES

- 1 - REMOVE TOPSOIL & /OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A'.
- 2 - PLACE GRANULAR 'A' TO TOP OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT D. T. C. STANDARDS.
- 3 - EXCAVATE COMPACTED GRANULAR 'A' MATERIAL FOR FOOTING.

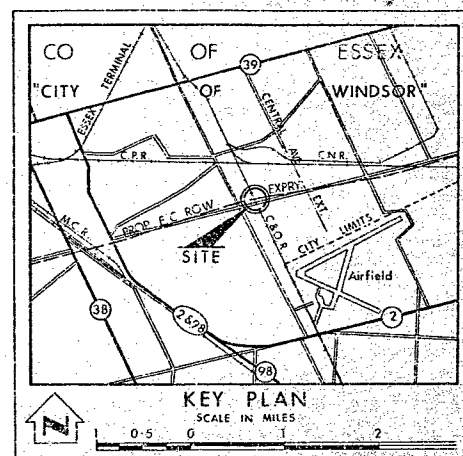
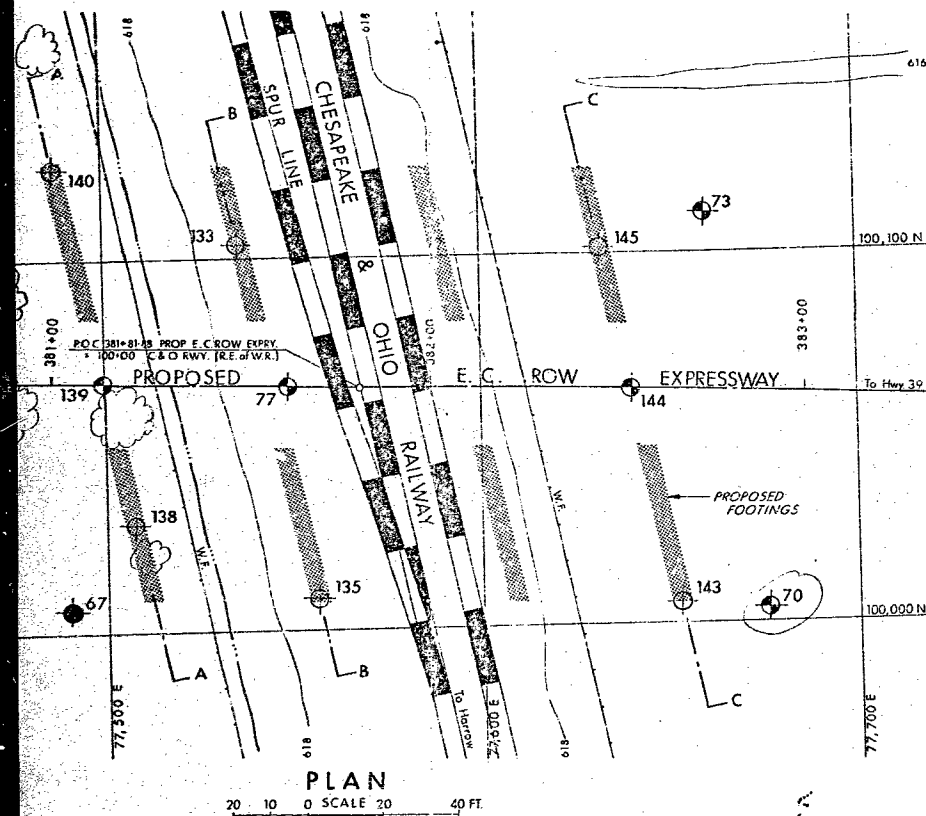


# SECTIONS

20 10 0 SCALE 20 40 FT







# LEGEND

Bore Hole

Cone Penetration Test

Bore Hole & Cone Test

Water Levels established at time of field investigation, NOV. 1971

WATER LEVELS DURING FIELD INVESTIGATION ESTABLISHED IN BORE HOLES 139 & 144 ONLY (NOV. 1971)

NO.	ELEVATION	CO - ORDINATES	
		NORTH	EAST
67	616.7	100,006	77,491
68	616.7	100,123	77,447
70	617.5	100,004	77,676
73	616.3	100,111	77,660
77	618.9	100,064	77,550
133	618.8	100,104	77,536
135	618.7	100,008	77,556
138	616.3	100,029	77,508
139	616.4	100,067	77,500
140	616.4	100,124	77,487
143	616.7	100,006	77,653
144	616.4	100,063	77,649
145	616.4	100,102	77,632

**NOTE**

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

REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF TRANSPORTATION & COMMUNICATIONS  
DESIGN SERVICES BRANCH - FOUNDATION OFFICE

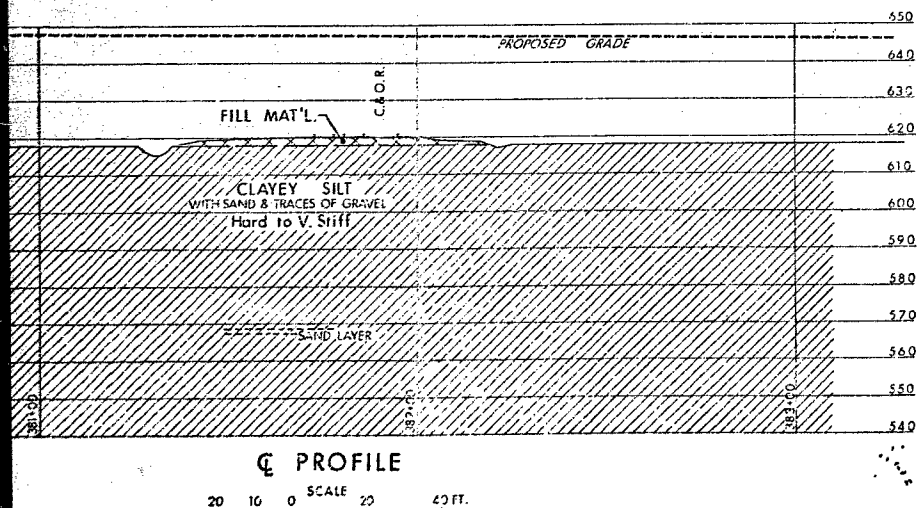
### CHESAPEAKE & OHIO RAILWAY

HIGHWAY NO. PROP. E.C. ROW EXPRY. DIST. NO. 1  
CC. ESSEX CITY OF WINDSOR  
TWP.   LOT   CON.  

### BORE HOLE LOCATIONS & SOIL STRATA

SUBWD. FP CHECKED   WP NO. 257-66-03 DRAWING NO. 71-1114 A  
DRAWN SO CHECKED   JOB NO. 71-1114  
DATE 16 DEC 1971 SITE NO.   BRIDGE DRAWING NO.    
APPROVED   CONT. NO.    
PRINCIPAL DESIGNER   ENGINEER  

REF. NO. E-5307-1



Mr. W. Katarynczuk,  
District Construction Engineer,  
Chatham.

Construction office,  
Third Floor, Central Bldg.,

February 13, 1975.

Walker Road Overpass W.P. 257-66-04, Site 6-285,  
C. & D. Rlwy. Overheads W.P. 257-66-05, Site 6-292 A & B,  
" " " " -06, Site 6-291,  
E.C. Row Expwy, District 1 - -07, Site 6-293.

This will confirm discussions and points agreed upon during our recent meeting in the Structural Design office.

Walker Road Overpass - There will be no need to pre-auger back row of piles in abutment footings as it was decided to re-locate all public utilities that are within 12 ft. of these piles.

Since the auger holes for the front row of piles are required to be larger than the diameter of the piles, the voids around the piles should be filled in the following sequence:

1. Auger the hole for the pile to a depth shown on the plan.
2. Drop pile in the hole.
3. Fill void with dry sand and heap the sand around the pile at the ground level.
4. Drive piles, placing more sand around the pile as the sand fills the void during driving.

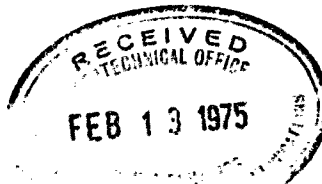
C. & O. Railway Overheads - No appreciable heave at the railway tracks is anticipated during driving of piles.

The 21" storm sewer in the vicinity of east abutment is to be replaced with reinforced sewer and therefore no problems are foreseen during driving as piles are more than 12 ft. away.

The 30" C.S.P. might be relocated to avoid excavation in proximity of the railway tracks. If this cannot be done, the plans will be revised to show the track protection for the pipe excavation and the construction sequence so that the pipe will be placed after the piles are driven.

Z. Luczka,  
Bridge Construction Engineer.

KL/JC



Mr. J. Keen,  
Reg. Structural Design Engineer,  
Structural Office,  
West Building, Downsview.

Soil Mechanics Section,  
Geotechnical Office,  
West Building, Downsview.  
October 21st, 1974.

RE: E.C. Rowe Expressway, Windsor,  
W.P. 257-66-05, 06 & 07.

We have reviewed the stability of the binwall type retaining structures employed in the E.C. Rowe Expressway project (W.P. 257-66-05, 06 & 07). For your design purposes, we are forwarding to you the following information.

The retaining structures should be designed to be stable with respect to overturning as well as sliding. Factor of safety against overturning should be no less than 2.0, while that against sliding should be greater than 1.5. Sliding failure may occur either along the interface between the granular fill and the clay base, or slightly within the clay base. In the former circumstance, shearing resistance is contributed by the friction of the granular material, whereas in the latter case by the adhesion of the clay.

A simple analysis, together with recommended values of various parameters, is attached to this letter.

B. Ly,  
Project Engineer,  
For:  
K.G. Selby,  
Supervising Engineer.

BL/mj  
c.c. Files  
Documents

74-11-825



MINUTES OF MEETING

To Discuss the type of piling to be used for the E.C. Row Expressway Structures, Walker Road, Site 6-285, Central Avenue Extension, CN-CP O'head, Site 6-287 C&O O'head Structures, Sites 6-291, 292A&B, 6-293

Time and Place: Oct. 21/74 - Mr. Grebski's Office

Present: C. Grebski, M. Stoyanoff, K. Howe, J. Kuprevicius, J. Keen (Full Time)  
W. McFarlane and C. Farrell (Part Time)

Summary

Mr. Howe reported that a quote for the supply of 12 3/4 inch. O.D. x 0.25 inch wall steel tube piles has been received and works out to be approximately \$9.00/LF. Also, the tube piles can be available to suit the contract award date of March 26, 1975.

The availability of steel H piles in the quantity required remains uncertain. H piles would cost about \$230/Ton, but could go higher considering current price escalations. If imported H piles were used, they would cost \$330/Ton or more.

Also to be considered is the fact that if tube piles are used for the E.C. Row, some of the H piles that would have been assigned to the E.C. Row could be used for other projects and to build up a depleted stockpile of H piles.

Cost Comparison

H Piles

supply H piles at \$230/Ton

cost of 12 HP 74 = \$8.50/L.F.

cost of 12 HP 53 = \$6.10/L.F.

E.C. Row Expressway Requirements -

12 HP 74# = 32,013 LF x \$8.50/LF = \$272,110.50

12 HP 53# = 18,587 LF x \$6.10/LF = \$113,307.50

Supply H piles = \$385,418.00

If H piles imported, supply =  $\frac{330}{230} \times 385,418 = \$553,000.00$

Tube Piles (filled with 3000 psi concrete)

Supply =  $37,233 \text{ LF} \times \$9.00/\text{LF} = \$335,097$

Place concrete in

piles  $\times \$1.64/\text{LF} = \$61,000$

Supply tube piles and concrete =

fill \$396,000 (approx.)

### Conclusions and Recommendations

Steel H piles would cost \$385,000.00 if bought through usual supply sources. The availability of obtaining the large quantity of H piles required for the E.C. Row structures appears uncertain. If imported, the H piles would cost \$553,000 or more. Comparatively, tube piles can be obtained in the quantity and at the time required for a firm price of \$9.00/LF. A comparative cost to supply the tube piles and fill with concrete would be \$396,000.00. The use of tube piles would allow the H piles to be used for other projects and restoring the depleted stockpile. Also, a pile load test has been carried out to prove that the tubes can be driven satisfactorily and safely support a design load of 120 tons per pile.

In view of the above considerations, all present agreed to use tube piles for the E.C. Row structures.

Design and drawing revisions to change from H piles to tube piles is to commence at the earliest possible date and should be completed by December 12/74, including all associated work, ie. D4 and specifications.



JK/CSG/ac

for:

J. Keen (secretary for meeting)  
C.S. Grebski,  
Structural Design Engineer.

cc: All Present

C. Mirza

K. Selby

A. Watt

A. Hickey

H. Chyc

J. Blevins

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

J. Keen

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

August 27th, 1974.

W.P.'s 257-66-03,04,05,06,07,09,21  
E.C. Row Expy., Windsor,  
District #1 (Chatham)

---

Following is a summary of the main points of our discussion on August 22, 1974 regarding piled foundations for the abovementioned projects.

1. For the C. & O. Rwy. structures a cost estimate of spread footings versus piled foundations indicates a much smaller saving in favour of spread footings than previously anticipated. This is partly due to the fact that as a result of the recent pile tests we are able to reduce the number of piles required by about 25%. In view of this and other (mainly settlement) considerations it was decided to adopt the piled foundation design.
2. A restriction on the use of pile driving hammers delivering more than 30,000 ft.lbs. per blow when the pile tips are within 3 ft. of bedrock, to be incorporated in the contract, requires that the bedrock surface be defined accurately at locations where piles are to be driven. To achieve this it will be necessary for the Soil Mechanics Section to carry out additional borings at all of the structure sites. In order to meet the present design schedule this drilling work should be completed and reported on by the end of October 1974.

*K.G. Selby*

K.G. Selby  
Supervising Engineer

KGS/rgb

c.c. A. Watt  
J. Anderson

Files  
Documents

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

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

August 9th, 1974.

Mr. J. Keen.

RE: File Tests - C & O Rwy. Overhead,  
E.C. Row Expressway, Dist. #1, Chatham,  
W.P. 257-66-05.

---

Following are the main conclusions to be drawn from the pile driving and loading tests carried out recently at the abovementioned site, on 12-3/4" x 1/4" wall steel tubes, spiral and helical weld.

1. Steel tube piles of the types tested can be driven to depths of 140 ft. without pre-augering.
2. The capacity of the driving hammer will have to be restricted to less than 30,000 ft.lbs./blow during the last 3 ft. of driving in order to prevent structural damage to the pile when it contacts the bedrock.
3. Safe loads up to 120 tons/pile may be assumed for design purposes.
4. Vertical piles may be fitted with standard shoe plates without reinforcing.
5. We believe that it would be advantageous for battered piles to be fitted with standard shoe plates reinforced with 2-1/2" deep x 3/4" thick cross plates, however we would like to discuss this with you further.
6. The foregoing recommendations are applicable to other sites where similar subsoil conditions prevail. These include Walker Road/E.C. Row, Central Ave./E.C. Row, and Central Ave./C.N. & C.P.R..

We will be forwarding to you a complete report in the near future.

KGS/p1

K.G. Selby,  
Supervising Engineer.





Mr. A. P. Watt,  
Regional Structural Planning Eng.,  
Southwestern Region,  
London, Ontario.

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

Mr. B. J. McKenna.

November 17, 1972.

*E. C. Row Expressway  
Walker Road Overpass  
C. & O. Railway Overheads  
Foundation Investigation Reports Review  
District #1, Chatham*

We have reviewed the proposed alignment and grade revisions  
(dated October 1972) at the following structure locations:

Walker Road Overpass

W.P. 257-66-04  
Bridge Site 6-285  
Site Plan: E-4882-1  
Foundation Investigation Report: 71-11113

C. & O. Railway Overhead on North Frontage Road

W.P. 257-66-06  
Bridge Site 6-291  
Site Plan: E-5308-1  
Foundation Investigation Report: 71-11115

C. & O. Railway Overhead on E.C. Row Expressway

W.P. 257-66-05 ✓  
Bridge Site 6-292  
Site Plan: E-5307-1  
Foundation Investigation Report: 72-11114

C. & O. Railway Overhead on South Frontage Road

W.P. 257-66-06  
Bridge Site 6-293  
Site Plan: E-5309-1  
Foundation Investigation Report: 72-11116

November 17, 1972.

It is concluded that the proposed revisions will not warrant additional field investigations and, consequently, the recommendations pertaining to the structure foundations and to the stability of the approach embankments contained in the foundation reports will not be altered.

Therefore, the recommendations of the foundation reports should be followed.

The name changes, from north and south service roads to north and south frontage roads, together with the introduction of the new Ontario co-ordinate system in place of the previous grid system are noted. It is our opinion, that changing the grid system, indicated on the foundation report drawings already in circulation is not warranted. However, the final contract drawing will be updated and the necessary amendment will be made.

PP/ao

cc: J. Anderson  
J. L. Keen  
A. Crowley

Foundations Files ✓  
Documents

For:

*P. Payer*  
P. Payer,  
Project Foundations Engineer,  
K. G. Selby,  
Supervising Foundations Engineer.

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

To: Mr. A. G. Stermac  
Principal Foundation Engineer  
Foundation Office  
West Bldg., DOWNSVIEW

FROM: Structural Planning  
Southwestern Region

ATTENTION:

DATE: October 26, 1972

OUR FILE REF.

IN REPLY TO

SUBJECT: E. C. Row Expressway  
Walker Road Overpass  
C. & O. Railway Overheads  
Foundation Investigation Reports Review  
District 1, Chatham

Enclosed please find revised E plans with alignment and grade revisions (Oct. 1972) for the following structures on the E. C. Row Expressway and North and South Frontage Roads in Windsor.

(a) Walker Road Overpass, W.P. 257-66-04, Bridge Site 6-285  
Site Plan E-4882-1, Rev. Oct. 1972

The original alignment and proposed foundation locations shown in red were the ones sent to you Oct. 12, 1971 and reported on in your Foundation Report W.O. 71-11113 dated December 9, 1971. The location of the Bore Hole and Cone Penetration Tests from your foundation report have been plotted on the enclosed E plans. The proposed new foundation locations are shown in blue on the two marked up E plans enclosed.

(b) C. & O. Railway Overhead on North Frontage Road, W.P. 257-66-06,  
Bridge Site 6-291, Site Plan E-5308-1, Rev. Oct. 1972

The original alignment and proposed foundation locations shown in red were the ones sent to you on Oct. 12, 1971 and reported on in your Foundation Report W.O. 71-11115 dated Dec., 1971. The location of the Bore Hole and Cone Penetration Tests from your foundation report have been plotted on the enclosed E plans. The proposed new foundation locations on the revised alignment are shown in blue on the two marked up E plans enclosed.

(c) C. & O. Railway Overhead on E. C. Row Expressway  
W.P. 257-66-05, Bridge Site 6-292, Site Plan E-5307-1, Rev. Oct. '72

The original alignment and proposed foundation locations shown in red were sent to you on Oct. 12, 1971 and reported on in your Foundation Report W.O. 71-11114 dated Dec. 10, 1971. The location of the Bore Hole and Cone Penetration Tests from your foundation report have been plotted on the enclosed E plans. The proposed new foundation locations on the revised alignment are shown in blue on the two marked up E plans enclosed.

(d) C. & O. Railway Overhead on South Frontage Road,  
W.P. 257-66-07, Bridge Site 6-293, Site Plan E-5309-1, Rev. Oct. '72

The original alignment and proposed foundation locations shown in red were sent to you on Oct. 12, 1971 and reported on in your Foundation Report W.O. 71-11116 dated Dec. 1971. The location of the Bore Hole and Cone Penetration Tests from your foundation report have been plotted on the enclosed E plans. The proposed new foundation locations on the revised alignment are shown in blue on the two marked up E plans enclosed.

Currently survey personnel from Southwestern Region are laying out the new alignment of the Central Avenue Extension. This alignment is approximately 700' west of the alignment previously sent you for the CN/CP Railway Overhead on Central Avenue Ext., W.P. 257-66-09, Bridge Site 6-287 and reported on by you in Foundation Report 71-11118 dated Dec. 1971. As soon as the new E plan for this bridge site has been prepared, we will be forwarding this to you with a request for a new foundation investigation. This is scheduled to be completed before November 15, 1972.

In view of the very tight schedule assigned to the Walker Rd., C. & O. and Central Avenue Extension structures, we are forwarding you now the information on the Walker Rd. and the C. & O. structures so that you may access what additional foundation investigation you may consider desirable in advance of the Central Ave. Ext. Railway Overhead foundation request in order that we may receive your foundation recommendations at the earliest possible date.

The E plans enclosed are all on the Ontario co-ordinate system rather than the project co-ordinate grid previously used. This involves both a horizontal translation and angular rotation to the former grid system.

We are enclosing ICES COGO output sheets giving the new co-ordinates for all the Bore Holes and Cone Tests previously co-ordinated by you in the reports for Walker Rd. and the C. & O. Railway Overheads (4 reports). In this printout the original point has been identified by its No. in the report. The old co-ordinates appear in the print out with this No. The point is then redefined for the purpose of recalculating its new co-ordinates by adding 10, i.e.; BH 68 becomes BH 78 and the new co-ordinates established. Some notes in red have been added to the computer output to clarify this point. Since all the E. C. Row geometry is now being calculated on the Ontario grid (3<sup>rd</sup> Modified Traverse Mercator Grid System), would you please have the bore hole co-ordinates in your Foundation Reports W.O. 71-11113, 4, 5, and 6 shown on the drawing at the end of each report revised to the Ontario Grid also.

Mr. A. G. Stermac  
Page 3  
October 26, 1972

It should also be noted that the title of North and South Service Roads shown on the E plans has been revised to North and South Frontage Roads.

Enclosed please find photographs of the Walker Road Overpass and C. & O. Railway Overhead Bridge Sites.

In order to avoid any delays when replying to this memorandum would you please send a copy direct to Mr. J. L. Keen, Regional Structural Design Engineer.

*B. J. McKenna*

B. J. McKenna  
Structural Location Engineer

BJMcK:sz  
Enc.

cc C. Grebski  
A. McConnell  
A. Crowley  
J. Anderson



## Memorandum

To: Mr. A. Wittenberg,  
Regional Manager,  
Southwestern Region,  
LONDON, Ontario.

From: Structural Office,  
West Building, Downsview

Attention:

Date: January 14, 1977

Our File Ref.

In Reply to

Subject:

W.P. 257-66-05 - Site 6-292A and B  
C & O Railway O'Head  
W.P. 257-66-06 - Site 6-291  
C & O Railway O'Head N. Frontage Rd.  
W.P. 257-66-07 - Site 6-293  
C & O Railway O'Head - S. Frontage Rd.  
E.C. Row Expressway - District 1

We reviewed the contract documents for the above bridges and completed the following changes.

Please adjust your copy of documents accordingly.

- (1) Add the following special SP, and the notation of "SP" to the applicable tender item, to all of the above bridges.

Drive Steel Tube Piles

As part of the work to be performed at the contract price for the above tender item, the Contractor shall supply all labour, material and equipment for predrilling holes for the piles as shown on the contract drawings.

Granular "A" material shall be used to fill the cavity around the piles and additional granular "A" material shall be added as required during driving.

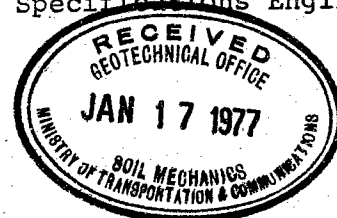
- (2) For all of the above bridges in the second and last paragraph of SP No. 904, Granular "C" Backfill should be changed to read Granular "A" Backfill.
- (3) For WP 257-66-05 and  
WP 257-66-06  
change tender item Granular "C" Backfill to Bridge to read  
"Granular "A" Backfill to Bridge."
- (4) For WP 257-66-05, change tender item Bin-Type Retaining Walls to read "Bin-Type Retaining Walls (E.B.L. Bridge and W.B.L. Bridge)"

NZ:js

CC: W. McFarlane  
J. Wear  
A.G. Kelly  
B. Giroux  
A.E. McKim

A.P. Watt  
E. Van Beilen  
C. Mirza ✓

*L. J. Hay*  
N. Zoltay  
Contract Specifications Engineer



74-11025

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

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

Mr. J. Keen.

*in memo* August 9th, 1974.

RE: Pile Tests - C & O Rwy. Overhead,  
E.C. Row Expressway, Dist. #1, Chatham,  
W.P. 257-66-05.

Following are the main conclusions to be drawn from the pile driving and loading tests carried out recently at the abovementioned site, on 12-3/4" x 1/4" wall steel tubes, spiral and helical weld.

1. Steel tube piles of the types tested can be driven to depths of 140 ft. without pre-augering.
2. The capacity of the driving hammer will have to be restricted to less than 30,000 ft.lbs./blow during the last 3 ft. of driving in order to prevent structural damage to the pile when it contacts the bedrock.
3. Safe loads up to 120 tons/pile may be assumed for design purposes.
4. Vertical piles may be fitted with standard shoe plates without reinforcing.
5. We believe that it would be advantageous for battered piles to be fitted with standard shoe plates reinforced with 2-1/2" deep x 3/4" thick cross plates, however we would like to discuss this with you further.
6. The foregoing recommendations are applicable to other sites where similar subsoil conditions prevail. These include Walker Road/E.C. Row, Central Ave./E.C. Row, and Central Ave./C.N. & C.P.R..

We will be forwarding to you a complete report in the near future.

KGS/mj

K.G. Selby,  
Supervising Engineer.

Item 1

- a) Supply 12 3/4" O.D. x 0.250 pipe, ASTM 252 Grade 2  
2 Pcs. @ 140 ft. (Straight Weld)  
1 Pc. @ 140 ft. (Spiral Weld)
- b) Supply 3 only crossed end plates
- c) Drive 2 piles to bedrock and fill with 4,000 p.s.i. concrete
- d) Load test 2 piles to a maximum test load of 400 Tons per pile
- e) On completion of tests, cut off piles 2 ft. below ground level and clear the site.  
.....Lump Sum of \$22,181.00

Item 2

- f) Supply as per items 1(a) and 1(b) above
- g) Drive 1 pile to practical refusal
- h) Pre-auger 2 holes to bedrock, place 1 pile in each hole, drive to bedrock, and fill with 4,000 p.s.i. concrete
- i) Load test 2 piles to a maximum test load of 400 Tons per pile
- j) On completion of tests, cut-off piles 2 ft. below ground level and clear the site  
.....Lump Sum of \$10,635.00

Item 3

Additional load test providing this may be done without dismantling the load box and setting it up again.  
.....Lump Sum of \$2,300.00



Telephone: (416) 248-3282.

Soil Mechanics Section,  
Geotechnical Office,  
1201 Wilson Avenue,  
DOWNSVIEW, Ontario. M3M 1J8

June 4th, 1974.

Dean Construction Co. Ltd.,  
P.O. Box 3215,  
TECUMSEH, Ontario.

ATTN: Mr. D. Sbrocca.

Dear Sir:

Please supply us with a firm lump sum quotation for carrying out the work as described on the attached sheets, together with a time schedule showing the earliest date when you would be in a position to start the work. A sketch outlining your proposals for a suitable reaction beam and anchorage system should also be included.

Please submit your quotation as follows:

- (1) Lump sum assuming that Pile #1 and Pile #2 are driven to bedrock without pre-augering and that one load test is carried out on each pile.
- (2) Additional lump sum if Pile #1 is not driven to bedrock and Pile #2 and Pile #3 are driven to bedrock in 13-inch to 14-inch diameter pre-augered holes and that load tests are carried out on Pile #2 and Pile #3 only.
- (3) Additional lump sum per test if more load tests than 2 are required.

It is to be understood that the work will be performed under the technical supervision of the M.T.C. Soil Mechanics Section and must conform to the provisions contained on the attached sheets and to the relevant M.T.C. Standards and Specifications. Your quotation must be submitted to this Office on June 12th, 1974.

Yours very truly,

K.G. Selby,  
Supervising Engineer.

KGS/mj  
Attach\*

c.c.1) Franki of Canada Ltd., 2) Birmingham Construction Ltd., ✓  
105 Nantucket Blvd., Fort Wellington N.,  
SCARBOROUGH, Ontario. HAMILTON, Ontario.

3) Western Caissons Ltd.,  
46 Creditstone Rd.,  
MAPLE, Ontario.

PILE LOAD TESTS  
Chesapeake and Ohio Railways  
Crossing at E.C. Row Expressway,  
City of Windsor, District #1, Chatham.  
W.P. 257-66-05.                      W.O. 74-11025.

1. GENERAL DESCRIPTION OF WORK:

The work to be performed by the Contractor consists of driving two or possibly three test piles, depending on driving resistance encountered, and subsequently load testing two of these piles to a maximum of 400 tons on each pile. After driving has commenced on the first pile, decisions will be made as follows, by the M.T.C. Soil Mechanics Section:-

- (a) Continue driving the first pile to bedrock and drive a second pile in the same manner to bedrock.
- or (b) Abandon the first pile and drive two other piles to bedrock in pre-augered 13-inch to 14-inch diameter holes.

All materials, equipment, and personnel necessary to carry out the work must be provided by the Contractor. The work will be performed as directed by the M.T.C. Soil Mechanics Section.

2. TEST PILES:

Test piles will be supplied by the Contractor and shall consist of the following:

- 12-3/4" O.D. x 0.25" wall thickness A.S.T.M.  
A 252 Grade II Straightweld, length 140'
- 12-3/4" O.D. x 0.25" wall thickness A.S.T.M.  
A 252 Grade II Straightweld, length 140"
- 12-3/4" O.D. x 0.25" wall thickness A.S.T.M.  
A 252 Grade II Spiral Weld, length 140'

Total length to be supplied      =      420'

All test piles will be fitted with Driving Shoes Type I as per M.T.C. Standard BD 82-2 (Dec. 1973) and filled with concrete having a 28-day compressive strength of 4000 p.s.i.

Any undriven lengths of piling remaining after completion of the work will become the property of the M.T.C.

3. DEAD LOAD AND REACTION BEAM:

The Contractor will supply and install a reaction beam and a suitable fabricated box containing sufficient material to supply the necessary reaction load. The Contractor will be fully responsible for the adequacy of these items and must make such remedial measures as are necessary at his own expense, should the system prove to be inadequate during the pile tests. The system must be adequate for a maximum of 400 tons test load.

4. LOAD TESTS.

The load tests will be carried out in general according to the National Building Code of Canada, and specifically as directed by the M.T.C. Soil Mechanics Section.

Each load test will be carried out to failure of the pile or to 400 ton test load, whichever occurs first, however, prior to reaching the maximum test load a specific load to be decided at the time by the M.T.C. Soil Mechanics Section, will be maintained for a period of 24 hours, after which time the test will be continued on to failure or to 400 tons test load.

The Contractor must provide all necessary personnel, equipment and material to make adjustments during the tests, and must have at least one skilled workman present for the complete duration of each test.

5. WORKING AREA:

The Contractor will place sufficient fill material to ensure a level and dry working area at the locations of the test piles and will erect a suitable enclosure to afford complete protection from adverse weather conditions during the load tests.

6. DRAWINGS:

The following drawings are provided:

- 74-11025    A    Pile test site location  
             B    Pile test arrangement - Plan  
             C    Pile test arrangement - Elevation  
             D    Subsoil conditions

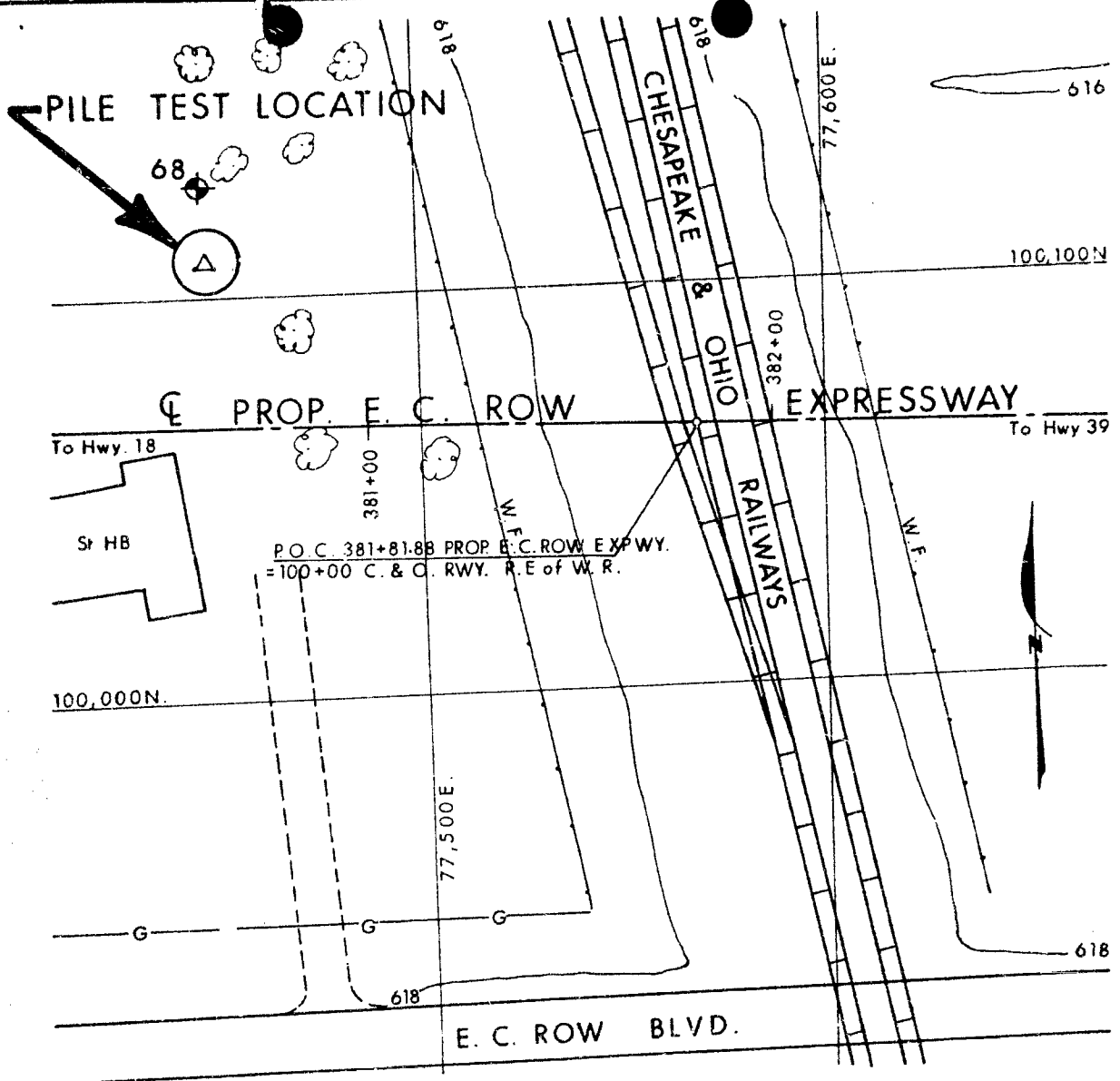
On award of the Contract, and prior to commencement of the work, the Contractor must submit a complete set of working drawings to the Ministry, together with a time schedule.

7. ACCESS TO SITE:

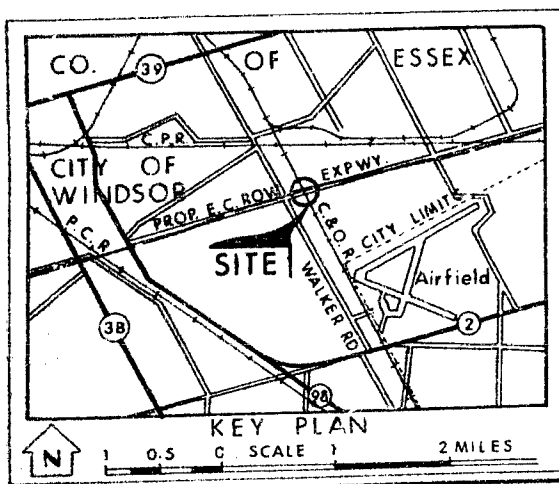
The Contractor must at his own expense, carry out all temporary work necessary to obtain access to the site for his materials, equipment and personnel.

8. CLEARING THE SITE:

On completion of the tests, the Contractor must clear the site to the satisfaction of the M.T.C. and restore it to the original condition. All piles must be cut off 2 ft. below the ground level and the resulting voids backfilled with suitable fill material.



-PLAN-  
SCALE: 1"=40'



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

CHESAPEAKE OHIO RAILWAYS & E.C. ROW EXPWY.  
PILE TEST LOCATION

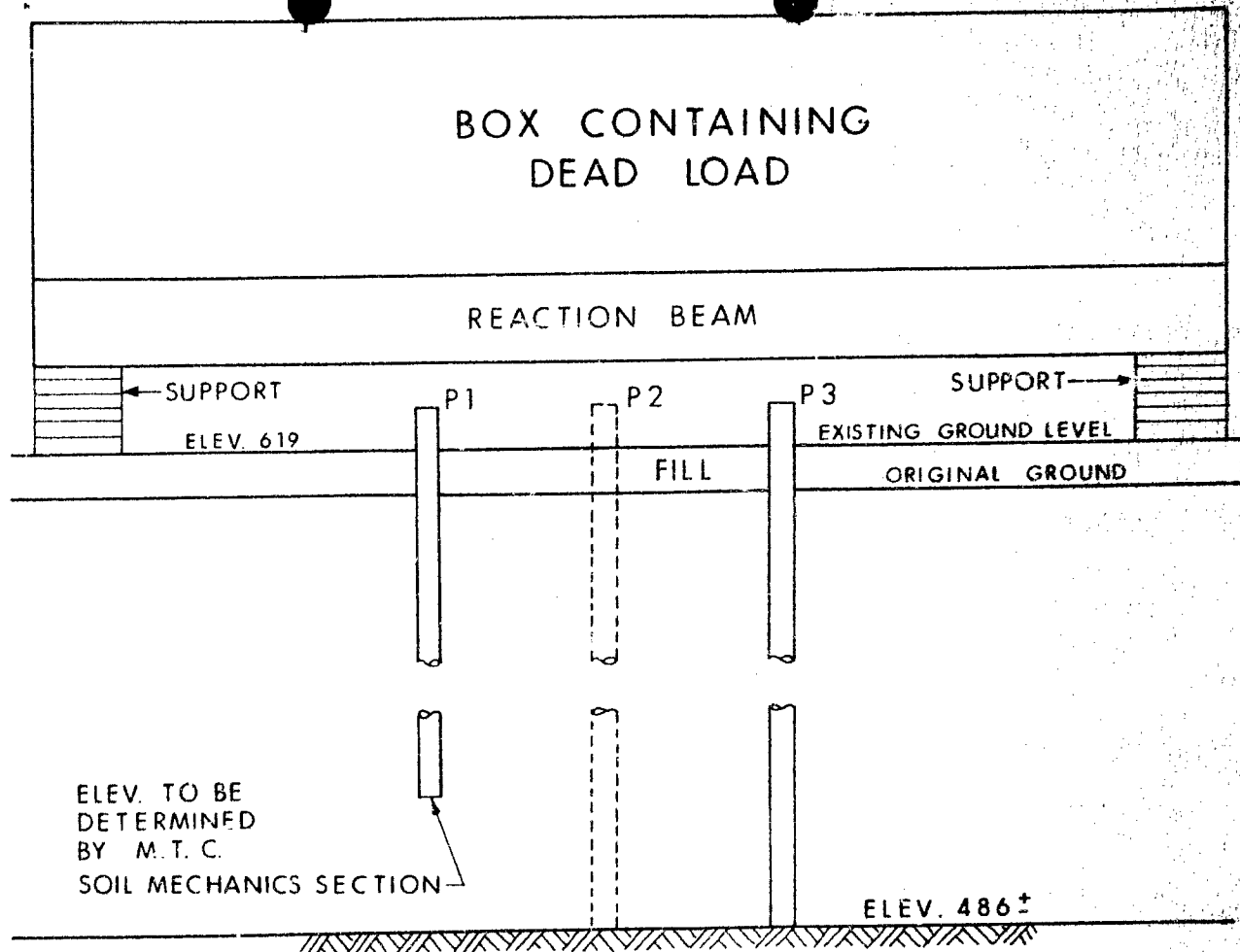
W.P. NO. 257-66-05

DIST. NO. 1

DATE June 3, 1974

W.O. NO. 74-11025

DRAWING NO 74-11025A



NOT TO SCALE

SEQUENCE OF TESTING

- 1) IF PILE # 1 & PILE # 2 ARE DRIVEN TO BEDROCK WITHOUT PRE-AUGERING. LOAD TEST ORDER WILL BE PILE # 1, THEN PILE # 2
- 2) IF PILE # 2 & PILE # 3 ARE DRIVEN TO BEDROCK IN PRE-AUGERED HOLES, LOAD TEST ORDER WILL BE PILE # 2 THEN PILE # 3 THEN (IF REQUIRED) PILE # 1

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

CHESAPEAKE OHIO RAILWAYS & E.C. ROW EXPWY.  
PILE TEST ARRANGEMENT - ELEVATION

W.P. NO. 257-66-05

DIST. NO. 1

DATE June 3, 1974

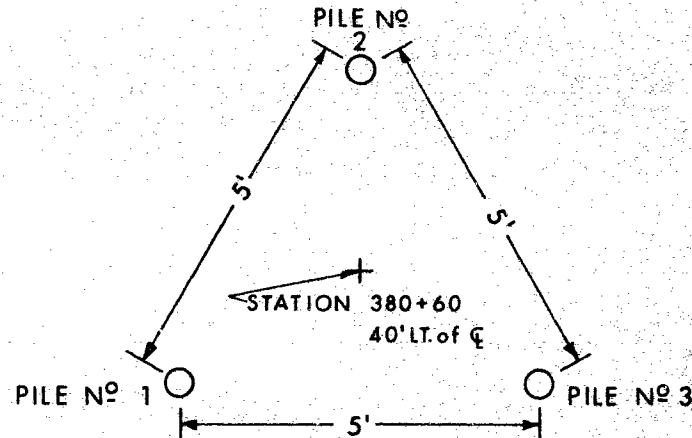
W.O. NO. 74-11025

DRAWING NO 74-11025C

ALL PILES  $12\frac{3}{4}" \times \frac{1}{4}"$   
Concrete filled STEEL TUBES

IF 2 PILES ARE DRIVEN :- { PILE # 1-STRAIGHT WELD  
PILE # 2-SPIRAL WELD

IF 3 PILES ARE DRIVEN:- { PILE # 1 & 2 - STRAIGHT WELD  
PILE # 3- SPIRAL WELD



NOT TO SCALE

SEQUENCE OF PILE DRIVING

- 1) DRIVE PILE #1 TO ELEVATION DETERMINED BY:  
M.T.C. SOIL MECHANICS SECTION.
- 2) IF PILE #1 HAS BEEN DRIVEN TO BEDROCK  
PILE #2 WILL BE DRIVEN IN SAME WAY.
- 3) IF PILE #1 HAS NOT BEEN DRIVEN TO BEDROCK  
THE CONTRACTOR WILL PRE-AUGER TWO, 13" to 14"  
DIAMETER HOLES FOR PILES 2 & 3 AND DRIVE THEM  
TO BEDROCK.
- 4) ALL DRIVEN PILES WILL BE FILLED WITH  
4,000 p.s.i. CONCRETE.

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

CHESAPEAKE OHIO RAILWAYS & E.C. ROW EXPWY.  
PILE TEST ARRANGEMENT

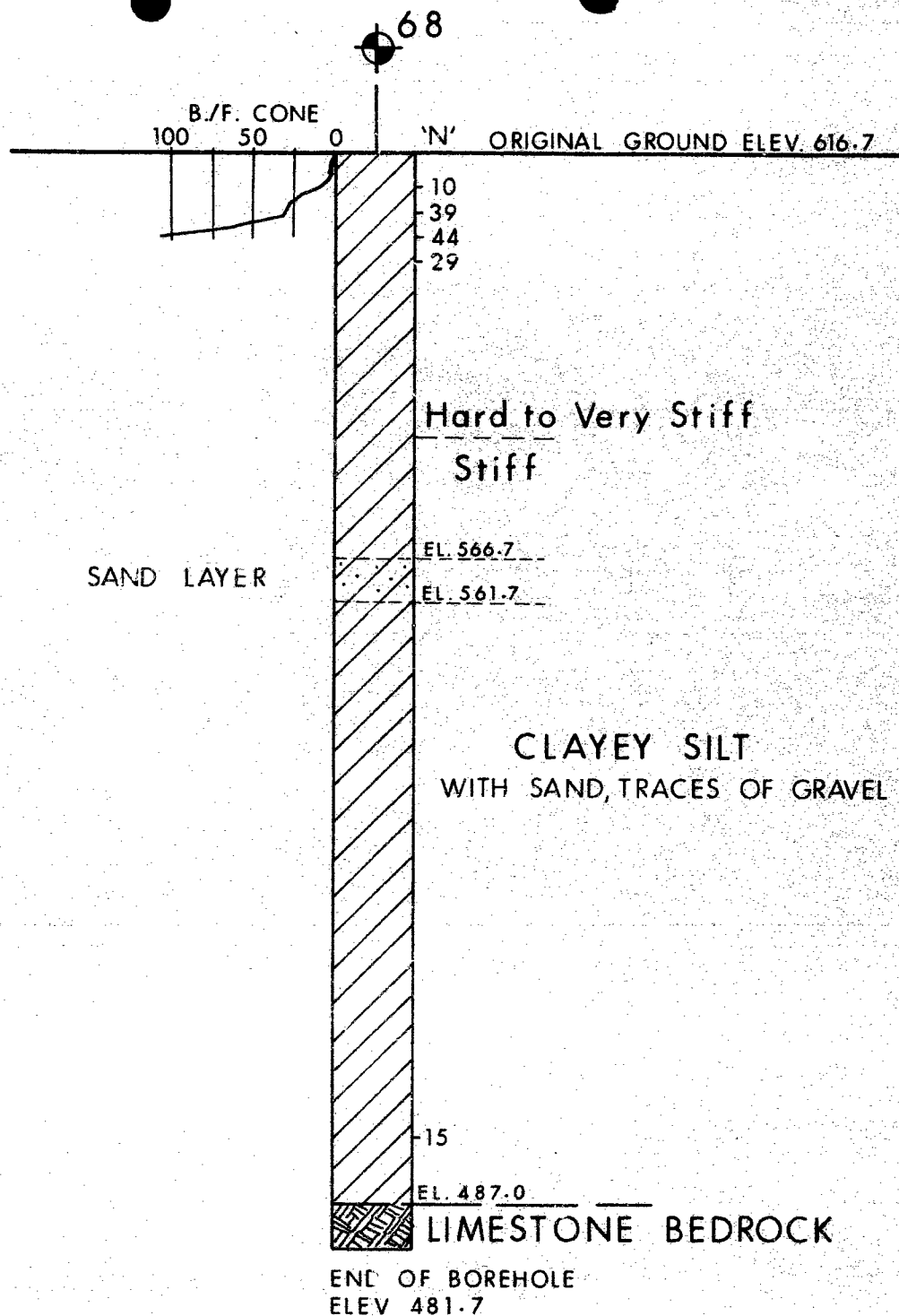
W.P. NO 257-66-05

DIST. NO. 1

DATE June 3, 1974

W.O. NO. 74-11025

DRAWING NO 74-11025B



SCALE: 1" = 20'

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

CHESAPEAKE OHIO RAILWAYS & E.C. ROW EXPWY.  
BOREHOLE DETAILS

W. P. NO. 257-66-05

DIST. NO. 1

DATE June 3, 1974

W.O. NO. 74-11025

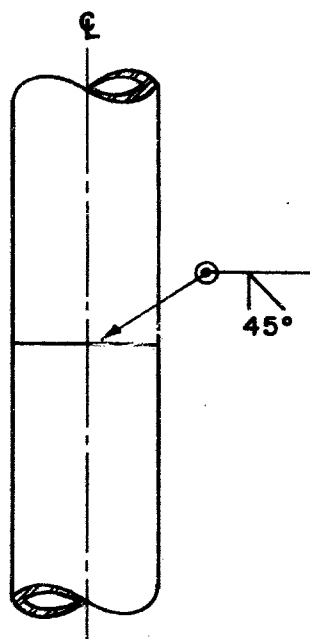
DRAWING NO. 74-11025D



# SPLICE & DRIVING SHOE DETAILS FOR STEEL TUBE PILES

BD 82-2

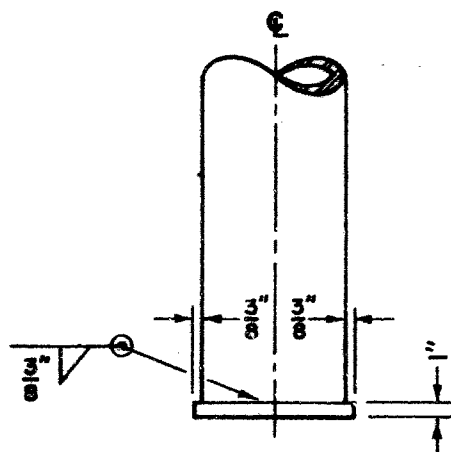
DEC. 1973



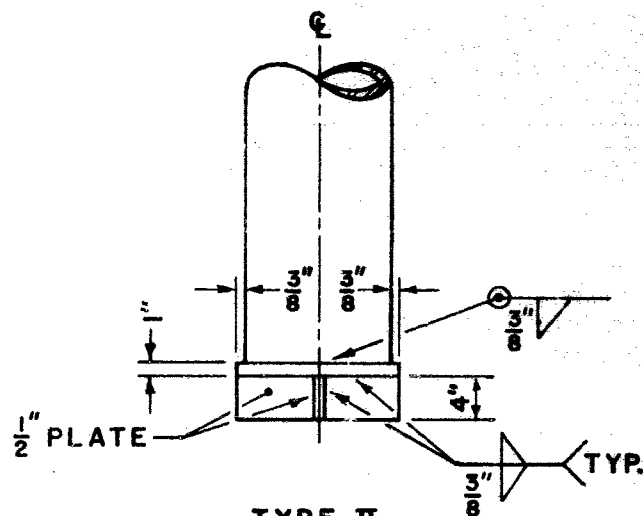
## MAXIMUM NUMBER OF SPLICES ALLOWED

LENGTH OF PILES	MAX. NUMBER OF SPLICES
20'-6" AND LESS	0
20'-7" TO 38'-6"	1
38'-7" TO 76'-6"	2
76'-7" TO 115'-0"	3

## SPLICE DETAILS



TYPE I



TYPE II

## SHOE DETAILS

### NOTES :

- SPLICE WELD SHALL BE PERPENDICULAR TO CL OF PILE.
- WELDING SHALL CONFORM TO THE LATEST ISSUE OF CSA SPECIFICATION W59 AND SHALL BE DONE BY A WELDER QUALIFIED UNDER CSA SPECIFICATION W47.

Welling Street Marine Terminal  
Hamilton, 21, Ontario

Telephone Hamilton 416-528-7924  
Toronto 416-366-6779

PILE  
FOUNDATIONS  
SINCE  
1897

# BERMINGHAM CONSTRUCTION LIMITED

July 12, 1974

Ministry of Transportation and Communications,  
Soil Mechanics Section,  
Geotechnical Office,  
1201 Wilson Avenue,  
DOWNSVIEW, Ontario  
M3M 1J8

Attention: Mr. K. G. Selby

Dear Sirs:

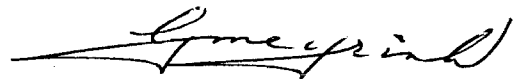
RE: WINDSOR LOAD TEST

Further to our telephone conversation of to-day, we enclose our suggested procedure for the pile load tests.

Instead of the one reaction beam shown on the sketch, we will have two reaction beams to take care of the 400 Ton maximum test load.

Yours very truly,

BERMINGHAM CONSTRUCTION LIMITED



G. Meyrink,  
Chief Engineer.

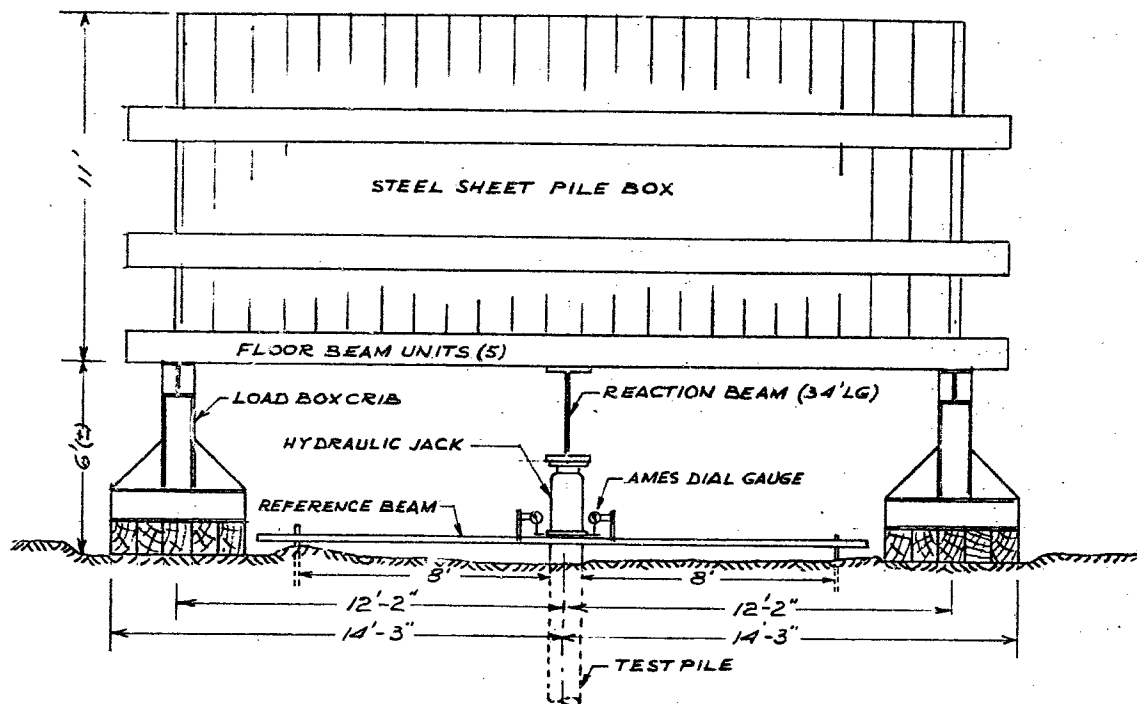
GM/r

Encls.



PILE LOAD TEST PROCEDURE

A.S.T.M. D 1143



1. Drive test pile and take pile driving record.
2. Cut off top of pile, and grind and/or trowel to produce a horizontal plane bearing surface.
3. Place load box cribs 12'-2" from center of pile on levelled ground surface so that tops of cribs are level and at the same elevation.
4. Place reaction beam on separate cribs, so that top of reaction beam is level and at same elevation as tops of load box cribs.
5. Check distance between pile cut-off and bottom of reaction beam. It should be enough to provide room for hydraulic jack and bearing plates (26").

# TESTON GAUGE LIMITED

INSTRUMENTS FOR CONTROL OF PRESSURE & TEMPERATURE

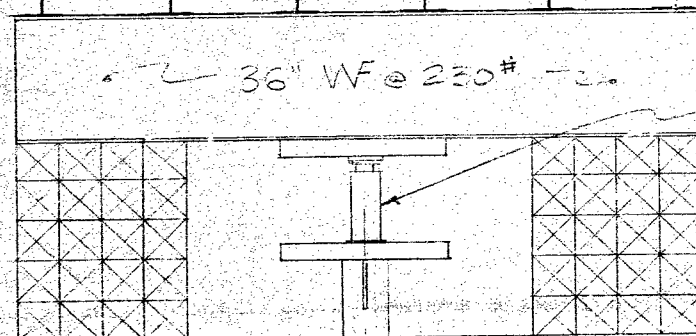
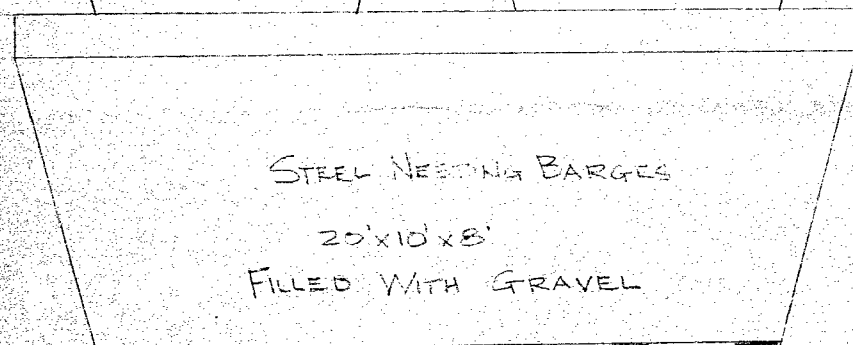
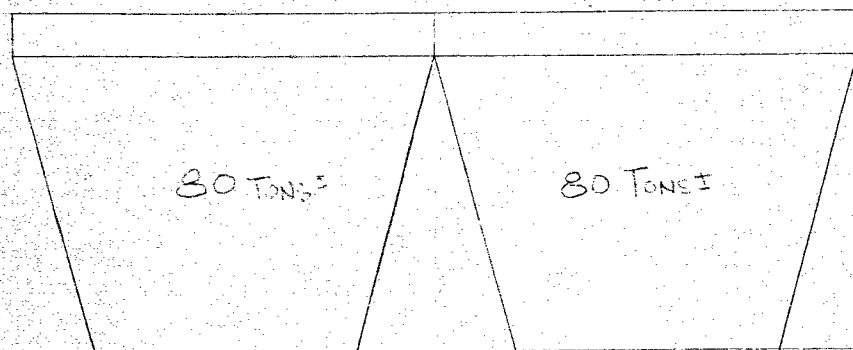
36 BRYDON DRIVE, REXDALE, ONTARIO • PHONE: 743-9166

CUSTOMER'S ORDER NO. Verbal-R. MacArthur	FED. LIC. NO.	PROV. LIC. NO.	INVOICE DATE July 16/74	INVOICE No. 4167
SHIPPED VIA Picked-up				

SOLD TO <input type="checkbox"/>	SHIPPED TO <input type="checkbox"/>
Bermingham Construction Limited, Wellington Street, Marine Terminal Hamilton, Ontario.	
TERMS NET 30 DAYS F.O.B. REXDALE, ONT.	

QUANTITY	DESCRIPTION	PRICE	TOTAL
1	Pressure Gauge 6" Dial 0-10,000 P.S.I. Calibrated and tested		\$ 15.00
P.O. #	REQ'D BY: <i>We certify that the above instrument is accurate to within 1% over the entire range.</i>		
GOODS REC'D: <i>G/L</i>	UNIT PRICE: <i>15.00</i>		
F.S.T.	P.S.T.		
EXTENSION: <i>04</i>	Dated: Rexdale, Ont. July 16, 1974.		
APP'D FOR PAYT: <i>DM</i> OR <i>DR</i>	Signed:		
ACC'T/JOB NO: <i>383</i>			
NAME:			
V.R. #: <i>7-93</i>			

PLANT BOOK ENTRY: *N/A*



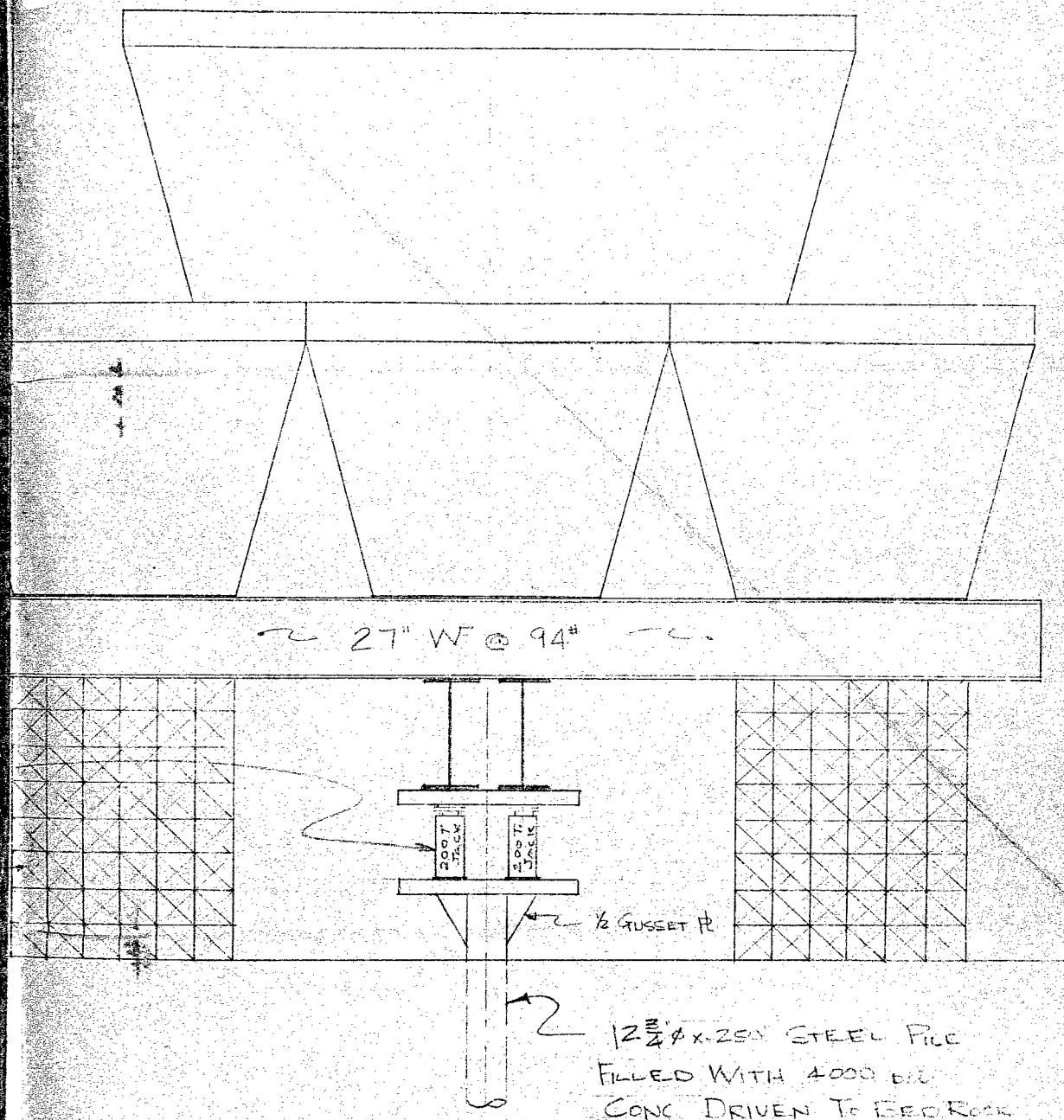
2-200 TON HYDRAULIC

TIMBER BLOCK

PROPOSED 400 TON TEST PILE

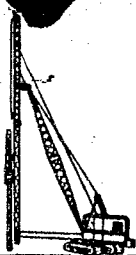
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS

SCALE 1/4" = 1'-0"



PILE  
COMMUNICATION

DEAN CONSTRUCTION CO. LTD.  
P.O. Box 3216 Phone 735-2134  
Tecumseh i Ontario



**BERMINGHAM**  
CONSTRUCTION LIMITED  
WELLINGTON STREET MARINE TERMINAL  
HAMILTON 21, ONTARIO



Established 1897

**TENDER FOR PILING**

JOB M.T.C. LOAD TEST  
ROW EXPRESSWAY/WALKER ROAD  
WINDSOR, ONTARIO

Date JUNE 12, 1974

We take pleasure in submitting the following prices, in accordance with the plans and specifications:

ITEM 1 We quote a lump sum price of \$22,181.00 to do the following work:

- a) Supply 12 3/4" O.D. x 0.250 pipe, ASTM 252 Grade 2  
2 Pcs. @ 140 ft. (Straight Weld)  
1 Pc. @ 140 ft. (Spiral Weld)
- b) Supply 3 only crossed end plates,
- c) Drive 2 piles to bedrock and fill with 4,000 p.s.i. concrete,
- d) Load test 2 piles to a maximum test load of 400 Tons per pile,
- e) On completion of tests, cut off piles 2 ft. below ground level and clear the site.

ITEM 2 We quote an additional lump sum price of \$10,635.00 to do the following work:

- f) Supply as per items 1a and 1b above,
- g) Drive 1 pile to practical refusal,
- h) Pre-auger 2 holes to bedrock, place 1 pile in each hole, drive to bedrock, and fill with 4,000 p.s.i. concrete,
- i) Load test 2 piles to a maximum test load of 400 Tons per pile,
- j) On completion of tests, cut-off piles 2 ft. below ground level and clear the site.

ITEM 3 We quote an additional lump sum price of \$2,300.00 to carry out an additional load test providing this may be done without dismantling the load box and setting it up again.

**NOTES:**

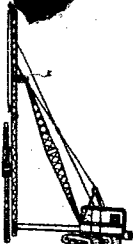
1. Our tender includes Provincial Sales Tax, but not Federal Sales Tax, which would be extra, if applicable.
2. Our tender is based on being able to move our equipment to a point approximately 80 ft. north-east of the pile test location, and on being able to walk our crane the rest of the way, and on placing a nominal amount of fill material to make a dry working area.

Continued.....

We draw your attention to our General Working Conditions which are attached hereto and are part of this Tender, and your acceptance is deemed to include same. This Tender is firm for 30 days from the date hereof, after which it is subject to our confirmation.

**BERMINGHAM CONSTRUCTION LIMITED**





# BERMINGHAM

CONSTRUCTION LIMITED

WELLINGTON STREET MARINE TERMINAL

HAMILTON 21, ONTARIO

HAMILTON  
JACKSON 8-7924  
TORONTO  
EM 6-6779

Established 1897

## TENDER FOR PILING

JOB ..... M.T.C. LOAD TEST  
..... ROW EXPRESSWAY/WALKER ROAD  
..... WINDSOR, ONT. ....

Date ..... JUNE 12, 1974

PAGE -2-

We take pleasure in submitting the following prices, in accordance with the plans and specifications:

### NOTES:

3. Item 2 above is based on M.T.C. providing free of charge to us water in sufficiently large volume to operate our wet-drill which we propose to use for pre-augering the holes.
4. We expect to be able to start early July, and estimate to require three (3) weeks to complete item 1 and four (4) weeks to complete item 2.



We draw your attention to our General Working Conditions which are attached hereto and are part of this Tender, and your acceptance is deemed to include same. This Tender is firm for 30 days from the date hereof, after which it is subject to our confirmation.

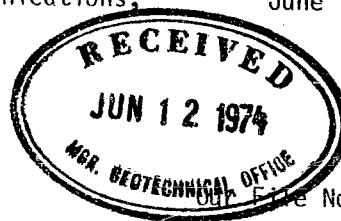
BERMINGHAM CONSTRUCTION LIMITED

*[Signature]*



Ministry of Transportation & Communications,  
Soil Mechanics Section,  
Geotechnical Office,  
1201 Wilson Avenue,  
DOWNSVIEW, Ontario.  
M3N 1J8.

June 12, 1974



Attention: Mr. K.G. Selby

File No. E-74-111

Dear Sir:

Re: Pile Load Tests, Proposed E.C. Row  
Expressway, Windsor, Ontario.

We are pleased to submit our quotations for carrying out pile load tests as requested in your letter of June 4, 1974.

- 1.) For two piles, Numbers 1 and 2 driven to bedrock and a load test performed on each  
Our Price is . . . . . \$37,900.00
- 2.) For two piles, Numbers 2 and 3, driven to bedrock in pre-augered holes one hundred and ten feet deep and 12 and/or 18 inches in diameter.  
Our Price is . . . . . \$40,900.00
- 3.) Extra to perform load test on Pile Number 1  
. . . . . \$11,000.00

The above prices are based on;

- 1.) The reaction arrangement shown on the accompanying sketch.
- 2.) Access to the site and permits required, by Others.
- 3.) Installation of testing components or piles, through obstructions and boulders on a time and material basis.
- 4.) An assumed testing duration of 72 hours.
- 5.) Location, exposure and protection of existing services, by Others.
- 6.) Acceptance of our proposal within 30 days.

If our proposal is accepted, we would expect payment to be made to us on the 25th day of each month to the extend of 85%

. . . . . 2

of the value of work performed, including materials delivered onto the site, in the preceeding month. Final payment shall be made 37 days after completion of our work.

We would be prepared to commence this work at any time, given one weeks notice.

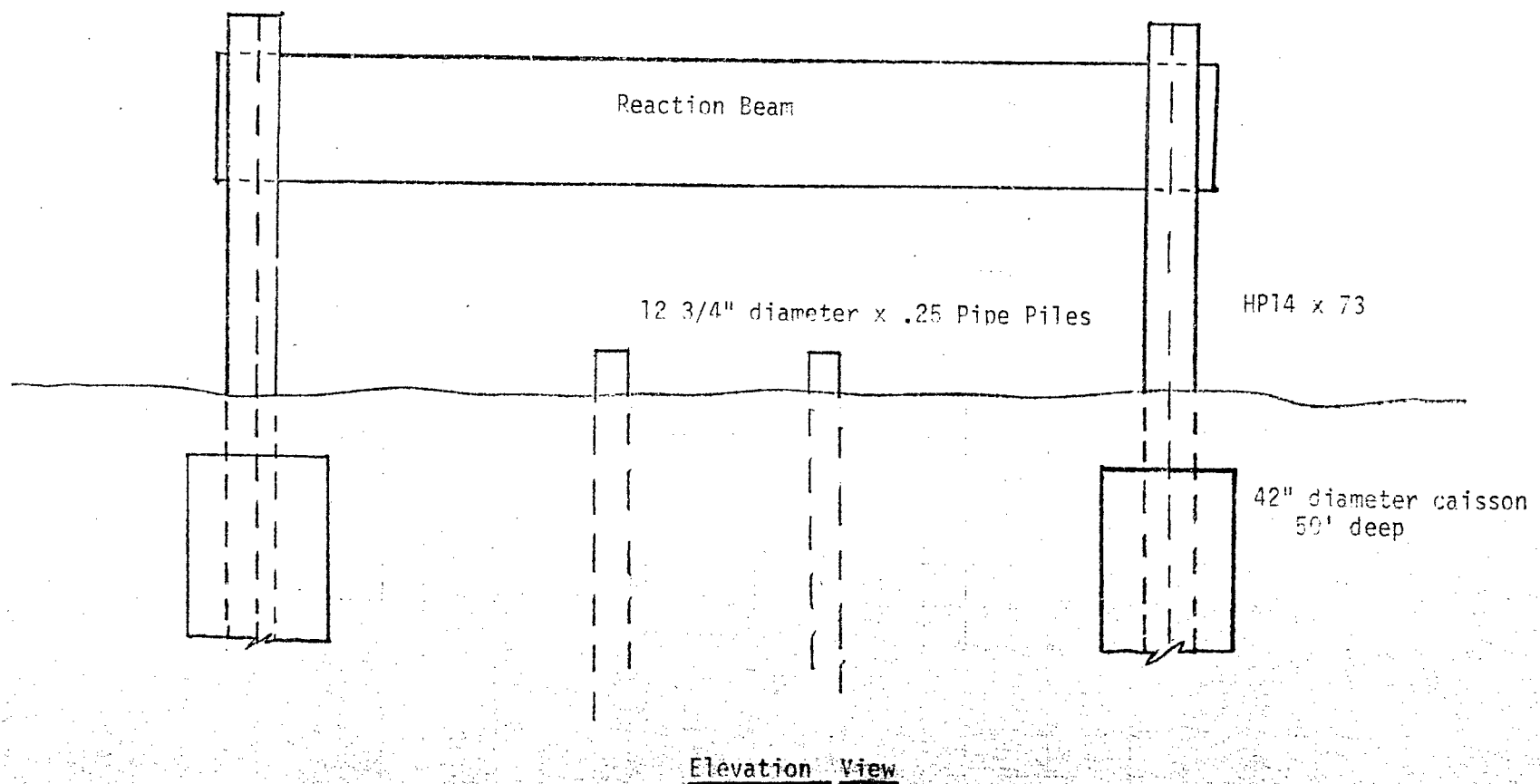
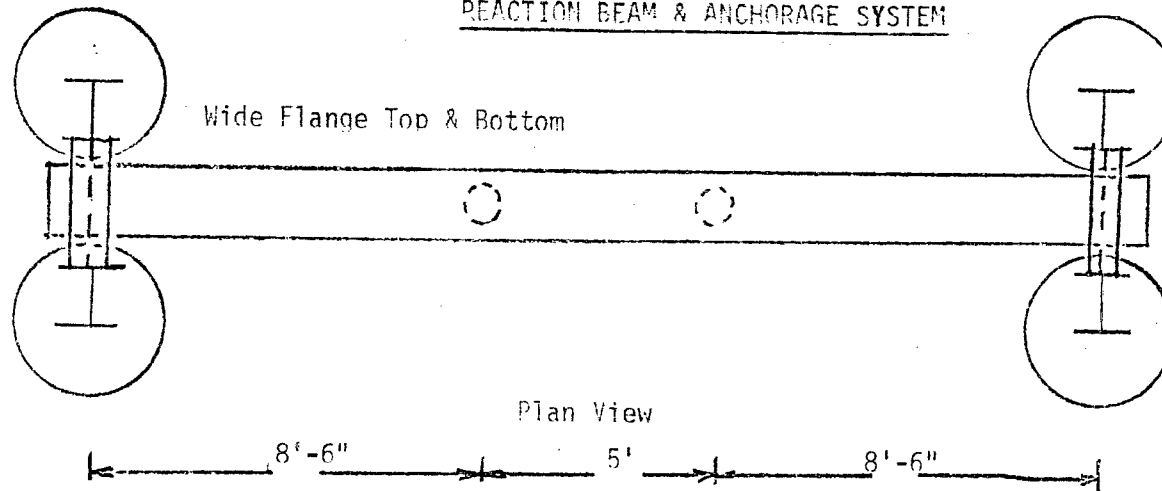
Yours very truly,

WESTERN CAISSONS (1969) LIMITED

LC:mc

L. Crane.

REACTION BEAM & ANCHORAGE SYSTEM





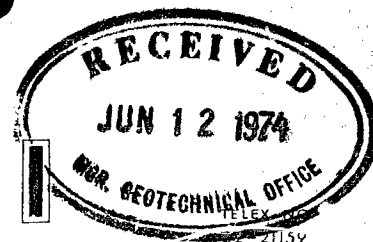
# FRANKI

CANADA LIMITED

105 NANTUCKET BLVD.

SCARBOROUGH, ONT.

MIP 2N5



CABLEGRAMS  
"FRANKITOR"  
TELEPHONE:  
751-4200

Our Reference:  
OP. 10074

June 12, 1974.

Ministry of Transportation and Communications,  
Soil Mechanics Section,  
Geotechnical Office,  
1201 Wilson Avenue,  
Downsview, Ontario.  
M3M 1J8

Re: Pile Load Test  
Chesapeake & Ohio Railways  
Crossing at E. C. Row Expressway  
City of Windsor, District No. 1, Chatham  
WP 257-66-05 WO 74-11025

Gentlemen:

We have examined tender documents for the above-noted project.

PRICES:

1. Lump sum assuming that piles #1 and #2 are driven to bedrock without pre-augering and that one load test is carried out on each pile -

TWENTY-SIX THOUSAND, EIGHT HUNDRED DOLLARS (\$26,800.00).

2. Additional lump sum if pile #1 is not driven to bedrock, and piles #2 and #3 are driven to bedrock in pre-augered holes, and that load tests are carried out on piles #2 and #3 only -

TWELVE THOUSAND, ONE HUNDRED DOLLARS (\$12,100.00).

3. Additional lump sum per test if more load tests than 2 are required -

TWO THOUSAND, EIGHT HUNDRED DOLLARS (\$2,800.00).

Cont'd...

Our Reference:  
OP. 10074

- 2 -

June 12, 1974

CONDITIONS:

1. Pre-augering for Pay Item (2) will be limited to a maximum depth of 100 feet.
2. Earliest starting date: July 22, 1974. However, actual starting date will be conditional upon previous work commitments at the time of contract award.
3. Estimated time of tests: Item 1) - 15 working days  
Item 2) - 3 working days  
Item 3) - 3 working days.
4. The M.T.C. will provide all field survey and layout.
5. It is understood that access is available to approximately 80 feet south of the test area (Dwg. No. 74-11025A).
6. Our attached General Working Conditions form part of this tender.
7. Payment will be in accordance with Item 13 of our General Working Conditions.
8. Federal Sales Tax is excluded from these prices; Provincial Sales Tax is included.
9. This tender is subject to revision if not accepted within 30 days.

Yours very truly,

FRANKI CANADA LIMITED,

*B. Clarke*

BC:AL  
Encl.

B. Clarke.



## GENERAL WORKING CONDITIONS

1. **Access to Work.**—Suitable access to and egress from the work for the transportation of our plant and materials shall be at all times provided by the purchaser, free of cost to us. If the principal has a railway siding, on or adjacent to the site, we shall be allowed use of the same, without charge, for the transportation of our plant and materials.
2. **Preparation and Protection of Site.**—(a) The purchaser shall provide us with a reasonably level working grade 6" or more above cut-off elevation. Excavations for caps, grade beams, pits, etc., shall not be made prior to the installation of our work.  
  
—(b) The purchaser shall remove in advance all surface, subsurface and overhead obstructions which might interfere with the installation of our work and the free movement and use of our equipment, and shall compensate us under the terms of Section 6 hereof for all extra work and/or delays caused us by the presence of such obstructions.  
  
—(c) The purchaser shall keep the entire area free from water.  
  
—(d) The purchaser shall shore and protect adjacent banks, structures, services and utilities and other property and shall be responsible for the continued maintenance and protection of same. We will assume no responsibility for subsurface structures, services and utilities not shown on the contract documents or the exact location of which has not been previously established, and the purchaser shall save us harmless from any liability for interfering with any such structures, services or utilities.
3. **Work in Excavated Areas.**—The excavation shall be extended at least from the center of all units. Excavations for corner units shall be made by the purchaser to allow our drivers to install same. He shall provide ramps for the descent into, and ascent from the excavation, and shall keep such excavation dry and free from water at all times. The slope of all excavations must be flat enough to avoid the danger of caving in, or such excavation shall be properly shored by the purchaser. Should we be required to drive in between shores, the purchaser shall compensate us for the extra work involved. The necessary removal and replacement of shores to allow for the movement of our machine, shall be performed by the purchaser.
4. **Permits.**—The purchaser shall secure at no cost to us all permits required by regulatory bodies for the execution of our work.
5. **Execution of the Work.**—After acceptance of the attached proposal, the work shall be commenced as soon as the equipment can be assembled on the site. We do not assume responsibility for delays caused by contingencies beyond our control such as strikes, prior commitment of equipment, transportation delays, adverse weather conditions, inundation of the site, serious accident to plant, or ground conditions substantially different from those indicated in the information furnished to us. The sequence in which the work is to proceed is to be determined by us so as to produce the best results.
6. **Delays.**—Our price is based on continuous work. If the work is delayed by others than ourselves we shall be reimbursed for such delays at the rate of \$800.00 per day or \$100.00 per hour.
7. **Field Engineering and Inspection.**—The purchaser shall provide and maintain all lines and grades and do all field engineering work, inspection and keep all records as specified or required. Stakes indicating the location of each unit shall be accurately set and maintained by the purchaser.

8. WATER.--Water necessary for our work shall be provided under pressure within hose distance of our operations. Such water shall be suitable for use in concrete.
9. LIGHTING.--In case of necessity for lighting, the purchaser shall provide, at his expense, sufficient illumination for our operations.
10. TRIMMING OF CAISSONS.--The expense of trimming the tops of caissons to grade shall be borne by the purchaser.
11. LOADING TESTS.--The purchaser shall compensate us for all load tests he requests to be made at the site. Such compensation shall be agreed upon before the tests are made.
12. CHEMICALS IN SUBSOIL.--We are not responsible for the detrimental effects to our materials caused by chemicals in the subsoil which have not been reported prior to the preparation of our quotation.
13. TERMS OF PAYMENT.--Payments shall be made to us on the 25th of each month to the extent of 85 % of the value of the work performed in the preceding month calculated according to unit schedule. Final payment shall be made 37 days after completion of our work.
14. REDUCTION IN SIZE OF CONTRACT.--Should the size of our contract be reduced, a mutually satisfactory adjustment shall be made to compensate us for our fixed charges.
15. COUNTERCHARGES.--We will not be responsible for any charges for work performed or materials furnished by others unless ordered by us in writing.
16. SUBSURFACE CONDITIONS.--In the event that during the execution of the work, subsurface conditions at the site are found to differ materially from those indicated in the contract documents and soil reports, or otherwise represented to Franki, then Franki shall promptly notify the Purchaser in writing of such conditions. The Purchaser shall promptly investigate such conditions and if he finds that they differ materially and will result in an increase or decrease in the cost of, or time required for performance of this Contract, an equitable adjustment shall be made between the parties and the Contract modified in writing accordingly.
17. ARBITRATION.--In the case of any dispute arising between the Purchaser and Franki as to their respective rights and obligations under the Contract, either party hereto shall be entitled to give to the other notice of such dispute and to request arbitration thereof; and the parties may, with respect to the particular matter then in dispute, agree to submit the same to arbitration in accordance with the applicable law of the place of building. Arbitration proceedings shall not take place until after the completion or alleged completion of the work except (a) on a question of certificate for payment, or (b) in a case where either party can show that the matter in dispute is of such nature as to require immediate consideration while evidence is available.
18. CONTRACT FORM.--Any formal contract not based specifically on this tender shall employ CCA, Document No. 1, Revised 1969.

Ken

TASK FORCE ON TUBE PILES

Minutes for Meeting #1, April 29/74 held in J. Keen's office.

Present: Chairman K. Selby, K. Howe, M. McFarlane, J. Keen

Purpose of Meeting:

To discuss various aspects concerning a pile load test in preparation for the testing of long concrete-filled steel tube piles in excess of 100 feet in length.

Purpose of Pile Load Test & History:

Due to the current shortage of steel H-piles, the Ministry is considering the use of end bearing concrete-filled steel tube piles as a substitute for end bearing steel H-piles, particularly for the E. C. Row Structures, Windsor (District #1) expected to go to contract in 1974-75. Because the driving, loading and hence, design of long steel tube piles in deep deposits of cohesive soils (up to 140 ft. in the Windsor area) can not be reliably predicted on the basis of our present experience, it is considered that a pile load test should be conducted to gain the necessary experience before changing from H to tube piles.

The Task Force was formed at the request of Mr. C. S. Grebski, Structural Design Engineer, at a preliminary meeting in his office on April 17, 1974. Mr. Grebski requested that Mr. K. Selby of the Geotechnical Section be the Chairman. The Task Force was requested to submit its recommendations by July 1, 1974.

Items Discussed:

- (a) Various sites of E. C. Row Expressway were discussed to determine the location of a test site. Structures considered were Walker Rd., Central Avenue CN and CP Overhead and the C & O Railway Overheads. It was tentatively agreed that the Central Avenue would be the best site as it has the deepest piles and probably the hardest driving conditions. Mr. K. Selby will check with the District for their advice and opinion concerning the other aspects (other than soil conditions) of carrying out the test, such as available property, access to site, conflict with existing utilities, etc.
- (b) As tube piles are available in spiral welded seam and/or straight seam, depending on the supplier; it was felt that one of each type should be tested.
- (c) Tube piles available in ASTM Grade 2,  $f_y = 35,000$  P.S.I. and Grade 3,  $f_y = 50,000$  P.S.I. (15% premium). For end bearing tube piles it was considered that Grade 3 (50,000 P.S.I.) steel would be the most economical.



TASK FORCE ON TUBE PILES

Page 2

Items Discussed (cont'd)

- (d) The cost of a pile load test was estimated by Mr. Selby to be in the order of \$15,000 upwards, depending on the extent of testing to be carried out.
- (e) Participation of industry in the test was discussed as it had been earlier suggested by Mr. Grebski that Stelco was interested in this particular test and were prepared to support it financially to the extent of \$15,000. In general, the Task Force had reservations about having anyone outside of WTC directly involved with the test. Possibly observers from the tube pile industry might be invited to view the actual pile testing. It was felt that this particular subject should be discussed at a higher level and the Task Force did not feel disposed to deal with this item at this time.
- (f) Design Loads for Test Piles  
The anticipated loads that could be applied to the test piles was discussed at length. It was considered that the size of pile to be tested would be 12 3/4 inches O.D., and most probably would have a 0.25 inch wall thickness. The pile would be installed with a flat plate end closure. For end bearing conditions it was considered that an equivalent load (applied to a 12 x 73 R-pile) could be attained, i.e. 95 Tons design load. Also, depending on the results of the test and the safety factor assured in design, the design pile load could be substantially higher than 95 Tons.
- (g) From initial calculations, the pile reaction system for loading the test pile would be tentatively designed for 400 Tons.
- (h) The calculated design load based on ultimate strength design using the formula developed at Lehigh and Illinois University tests was reviewed in connection with item (f) above. It would appear the design loads contemplated in item (f) for the 12 3/4 x 0.25 pile would be logical (i.e. 95 Tons or more). A copy of the calculation is attached to the Minutes. J. Keen will review the loads determined by the Lehigh formula with respect to other pile design codes and specifications.

Next Meeting - to be called at the discretion of the Chairman



J. L. Keen  
Secretary, Task Force Meeting  
April 23/74

JLE/ek  
Attach.

cc: All Present

Also: C. Brown C. Mirza  
B. Davis A. Watt  
C. S. Grebski