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DIST. 6 REGION

W.P. No. 48-71-15

CONT. No.

W. O. No. 73-11118

STR. SITE No.

HWY. No. 427

LOCATION STORM SEWER, HWY 427,

CARLINGVIEW

No of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

30M11-148

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

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

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

ATTENTION:

DATE: April 1st, 1974.

OUR FILE REF.

IN REPLY TO

APR 11 1974

SUBJECT: FOUNDATION INVESTIGATION REPORT
For
The Proposed Highway 427, Carlingview,
Storm Sewer System, District 6 Toronto.

W.O. 73-11118 W.P. 48-71-15#.

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

M. Devata

M. Devata,
Supervising Engineer.

MD/mj
Attach*
c.c. E.J. Orr
B.R. Davis
R.S. Pillar
~~D.P. Collins~~ H. Greenland
B.J. Giroux
D. Gunther
G.A. Wrong
B.A. Singh
McCormick & Rankin

Files
Documents

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FOUNDATION INVESTIGATION REPORT
For
The Proposed Highway 427, Carlingview,
Storm Sewer System, District 6 Toronto.

W.O. 73-11118 W.P. 48-71-150.

1. INTRODUCTION:

The Geotechnical Office was requested to carry out a subsurface investigation for the proposed Storm Sewer at Hwy. 427 and Carlingview Drive crossings in Metropolitan Toronto. The request was contained in a memo from Mr. J. Geo. Celmins, Sr. Project Design Engineer, Central Region, on February 27th, 1974. Subsequently, an investigation was carried out by this Office to determine the subsoil and groundwater conditions in this area.

This report contains the results of the investigation together with our recommendations pertaining to the design and the installation of the proposed sewer.

2. DESCRIPTION OF THE SITE AND GEOLOGY:

The site is located just south of Dixon Road between new Highway 427 and Carlingview Drive in Etobicoke. The surrounding terrain is flat to undulating and partly developed for industrial purposes.

Physiographically, the site is situated in the region known as "Peel Plain". The characteristic deposit in the vicinity of the area under investigation, is composed of a cohesive glacial till.

...../2

3. <u>FIELD AND LABORATORY WORK:</u>

Five sampled boreholes were put down during the course of field investigation. The borings were advanced by a Muskeg-Vehicle-Mounted continuous flight auger machine, (commercially known as C.M.E. 55), adapted for soil sampling purposes.

At required depths samples were taken by driving the 2" O.D. split spoon sampler into the soil according to the specifications of the Standard Penetration Test. Groundwater level observations were carried out during the period of the investigation, in the open boreholes. The subsoil and groundwater conditions encountered at the boring locations are presented in the Record of Borehole Sheets. The alignment of the proposed storm sewer as well as the borehole locations and elevations were established in the field by District 6 personnel. All elevations in this report are referred to a Geodetic datum. Boring locations and elevations, together with estimated stratigraphical sections are shown on Drawing No. 73-11118 A.

All samples were subjected to careful visual examination in the field and subsequently in the laboratory. Following this examination, laboratory tests were carried out on selected representative samples to determine the physical properties of the various soil types encountered; namely,

Natural Moisture Content
Atterberg Limits
Grain-size Distributions

The results of this testing are plotted on the Record of Borehole Sheets and summarized on Figures 1 and 2, all contained in the Appendix I of this Report.

..... /3

4. SUBSOIL CONDITIONS:

4.1) General:

The predominant stratum across the site is composed of a stiff to hard heterogeneous mixture of clayey silt, sand and gravel (glacial till). This deposit was penetrated down to a maximum depth of 24 ft. (7.3 m).

The subsoil and groundwater conditions encountered in the borings are plotted on the Record of Borehole Sheets. Stratigraphical sections have been inferred from this data and plotted on Drawing No. 73-11118 A.

4.2) Heterogeneous Mixture of Clayey Silt, Sand and Gravel (Glacial Till):

The glacial till is composed of a heterogeneous mixture of clayey silt, sand and gravel. This deposit was encountered in all borings immediately beneath the ground surface. This deposit was penetrated to a maximum depth of 24 ft. (7.3 m). Typical grain-size distribution curves for the samples of this glacial till stratum are plotted in an envelope form on Figure 1 in the Appendix I.

Results of Atterberg limit tests performed on samples recovered in this material were plotted on the Record of Borehole Sheets, as well as on the plasticity chart, Figure 2. These are tabulated below:

	Range	Average
Liquid Limit (W_L)%	17-35	26
Plastic Limit (W_P)%	12-22	17
Natural Moisture Content (W)%	7-23	15

..... /4

From these values, it may be concluded that the matrix of the glacial till is inorganic and of low plasticity.

Standard Penetration testing was carried out within this stratum and the results were plotted on the Record of Borehole Sheets. The "N" values vary from 9 to over 100 blows per foot (0.3 m). It is estimated that the consistency of this cohesive glacial till layer varies from stiff to hard, being generally hard.

5. GROUNDWATER CONDITIONS:

Groundwater level observations were carried out during the period of the investigation by recording the water level in the open Boreholes. These observations have been recorded on the Record of Borehole Sheets and summarized on Drawing No. 73-11118 A. The results of the measurements in the open boreholes indicate that the groundwater level ranges from 0.5 ft. (B.H. 4) to 20 ft. (B.H. 2), (0.2 m. to 6.1 m), below the existing ground surface.

6. DISCUSSIONS AND RECOMMENDATIONS:

6.1) General:

According to available information (from Mr. C.P. Korzeniowski, of McCormick, Rankin & Associates), this Ministry and M.O.T. will construct a storm sewer south of Dixon Road, extending from Hwy. 427 easterly to Mimico Creek, for the Borough of Etobicoke. The total length of this storm sewer is approximately 2,600 feet (800 m). The size of the sewer is to vary from 78 to 90 inches in diameter (2.0 to 2.3 m).

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The abovementioned sewer will cross the existing roadways of Hwy. 427 (Airport Expressway) and Carlingview Drive. The invert elevation of the sewer is to vary from 526 to 525 at Hwy. 427 crossing and at 504 at Carlingview Drive crossing. At these grades, the sewer will be located some 15 feet (4.5 m) and 23 feet (7.0 m) below the existing groundsurface respectively. This investigation and the comments are confined to the portion of the sewer at the two crossings mentioned previously.

It is understood that at both crossings, tunnelling operations by means of air pressure or jacking techniques are being considered. In addition, the section of the sewer at Carlingview Drive crossing may be constructed using cut and covered method. In the subsections to follow, these alternatives are discussed separately.

6.2) <u>Cut and Covered Method:</u>

This method will be considered only at the Carlingview Drive crossing. At this location the cut will be up to 23 feet (7.0 m) deep. The cut will be made in the cohesive glacial till, which is competent. No stability problems are anticipated if 1:1 temporary cut slopes are used. If steeper slopes or vertical cuts are used, properly designed bracings should be employed.

The future performance of the sewer will largely depend on the properly constructed bedding in the dry. It is therefore recommended that the sewer bedding adhere to Standards currently being used by the Ministry, specifically for Class "B-1" or "B-2" for yielding foundation (Standard No. DD-823), and be placed in a dry trench. In addition, particular attention should be paid to the compaction and shaping of the bedding material. Backfill for the sewer excavations should comply with Standard No. DD-813-B currently used by the Ministry. If the backfill is not properly placed and compacted, excessive settlement of the re-paved portion of the roadway will occur which would result with undesirable post-construction maintenance work.

The prevailing groundwater level, as recorded during the course of the field investigation is some 15 feet (4.6 m) above the invert elevation of the sewer. In view of the impervious nature of the glacial till deposit, groundwater seepage into the excavations will be negligible in quantity. It is believed that by pumping from sumps, relatively dry conditions can be maintained at the bottom of the trench.

6.3) Tunnelling Method:

As mentioned previously, the overburden consists of a heterogeneous mixture of clayey silt, sand and gravel. The gravel content ranges from 0 to 26%, being generally less than 10%, and the maximum grain-size is approximately 1½ inches (4 cm), as shown on Figure #2 appended to this report. It should be pointed out that the grain-size distribution tests were performed on samples recovered from a 2" O.D. split-spoon sampler. The insitu composition and maximum grain-size of the glacial till may vary from the values quoted above. It is believed, however, that the subsoil will not present any major problems for the tunnelling operation by means of jacking method.

If air pressure method is used, the contractors should be advised that they will be responsible for determining the prevailing groundwater level during construction period and hence the air pressure that will be used. In addition, they would be responsible for preventing leakage through the boreholes that have been drilled at the site insofar as it affects their operations. In order to ensure a safe working condition within the tunnel, it may be necessary to install a temporary bracing system.

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7. MISCELLANEOUS:

The field investigation was carried out during the period of March 7th and 8th, 1974, under the supervision of Mr. V. Korlu, Project Engineer, who also prepared this Report.

The drilling equipment was owned and operated by P.V.K. & Sons of Burford, Ontario.

The project was carried out under the general supervision of Mr. M. Devata, Supervising Engineer, who also reviewed this Report.

REGISTERED PROFESSIONAL ENGINEER
CIVIL
V. KORLU
PROVINCE OF ONTARIO

V. Korlu, P. Eng.

M. Devata, P. Eng.

VK/mj
April, 1974.

A P P E N D I X   I

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO

DESIGN SERVICES BRANCH

RECORD OF BOREHOLE Nº 1

FOUNDATIONS OFFICE

JOB 73-11118 LOCATION Sta. 1 + 30 ℄ ORIGINATED BY VK

W.P. 48-71-15 BORING DATE March 7, 1974 COMPILED BY VK

DATUM Geodetic BOREHOLE TYPE Auger and sample with C.M.E. - 55 CHECKED BY SR

m	ELEV. DEPTH ft.	SOIL PROFILE DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	BLOWS/FOOT (0.3 m)	ELEV. SCALE ft/m	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT (0.3 m); SHEAR STRENGTH P.S.F. o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT W_L, PLASTIC LIMIT W_P, WATER CONTENT W; WATER CONTENT % 10 20 30	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL
164.0	537.9	Ground Level									
0.0	0.0	Het. Mix. of		1	SS	21					W.L. 534.9 (163.0)
		Clayey Silt		2	SS	25					0 18 52 30
				3	SS	73	530 161.5				
		Brown		4	SS	150					
		Grey		5	SS	60					
		Sand and gravel		6	SS	155	520				0 28 48 24
		glacial till		7	SS	69	158.5				
157.6	516.9	Very Stiff to Hard		8	SS	119					
6.4	21.0	End of Borehole									
							510 155.4				

20
15 ⟡ 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH — FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº2

JOB 73-11118 — LOCATION Sta. 2 + 34 ℄ — ORIGINATED BY VK

W.P. 48-71-15 — BORING DATE March 7, 1974 — COMPILED BY VK

DATUM Geodetic — BOREHOLE TYPE Auger and sample with C.M.E. - 55 — CHECKED BY SR

m	ELEV. / DEPTH ft.	DESCRIPTION (SOIL PROFILE)	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT (0.3 m)	ELEV. SCALE ft/m	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT (0.3 m); SHEAR STRENGTH P.S.F. (o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	LIQUID LIMIT W_L, PLASTIC LIMIT W_P, WATER CONTENT W; WATER CONTENT % 10 20 30	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
164.4	539.5	Ground Level									
0.0	0.0	Het. Mixture of clayey silt, sand and gravel		1	SS	37					
				2	SS	54					
				3	SS	64	530				4 31 54 11
		Glacial Till		4	SS	136	161.5				
				5	SS	96					
		Brown		6	SS	76					w.l. 522.0 (159.1)
		Grey		7	SS	33	520				
				8	SS	29	158.5				
157.1	515.5	Very Stiff to Hard		9	SS	87					14 33 46 7
7.3	24.0	End of Borehole					510				
							155.4				

20
15 ⌖ 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH — FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº3

JOB 73-11118 — LOCATION Sta. 3 + 30 ℄ — ORIGINATED BY VK

W.P. 48-71-15 — BORING DATE March 7, 1974 — COMPILED BY VK

DATUM Geodetic — BOREHOLE TYPE Auger and sample with C.M.E. - 55 — CHECKED BY SR

m	ELEV. DEPTH ft.	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT (0.3 m)	ELEV. SCALE ft/m	REMARKS GR.SA.SI.CL.
163.8	537.4	Ground Level					
0.0	0.0	Het. Mixture of clayey silt, sand and gravel	1	SS	45		2 27 56 15
			2	SS	52	530	w.l. 531.0 (161.9)
			3	SS	74	161.5	
			4	SS	116		
			5	SS	86		8 27 54 11
		Brown / Grey	6	SS	118	520	
		Glacial Till	7	SS	165	158.5	
			8	SS	140		
156.5	513.4	Hard	9	SS	160		3 34 57 6
7.3	24.0	End of Borehole				510	
						155.4	

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT (0.3 m)

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

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

WATER CONTENT % 10 20 30

BULK DENSITY γ P.C.F.

20 15 5 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE №4

JOB 73-11118 LOCATION Sta. 15 + 00 ℄ ORIGINATED BY VK

W.P. 48-71-15 BORING DATE March 8, 1974 COMPILED BY VK

DATUM Geodetic BOREHOLE TYPE Auger and sample with C.M.E. - 55 CHECKED BY SR

ELEV. m / DEPTH m	ELEV. DEPTH ft.	SOIL PROFILE DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	BLOWS/FOOT (0.3 m)	ELEV. SCALE ft/m	REMARKS GR.SA.SI.CL.
157.5	516.7	Ground Level						
0.0	0.0	Het. Mixture of clayey silt, sand and gravel						w.l. 516.2 (157.3)
				1	SS	9		
				2	SS	13	510 / 155.4	4 25 53 18
				3	SS	73		
				4	SS	145		
		Brown / Grey glacial till		5	SS	99		
				6	SS	75	500 / 152.4	6 32 46 16
				7	SS	138		
				8	SS	116		
150.3 / 7.2	493.2 / 23.5	Stiff to Hard		9	SS	134		26 19 44 11
		End of Borehole					490 / 149.3	

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT (0.3 m)

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

LIQUID LIMIT w_L PLASTIC LIMIT w_P WATER CONTENT w

WATER CONTENT % 10 20 30

BULK DENSITY γ P.C.F.

OFFICE REPORT ON SOIL EXPLORATION

20 15 5 10 % STRAIN AT FAILURE

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO

DESIGN SERVICES BRANCH | FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº5

JOB 73-11118 | LOCATION Sta. 16 + 32 ℄ | ORIGINATED BY VK

W.P. 48-71-15 | BORING DATE March 8, 1974 | COMPILED BY VK

DATUM Geodetic | BOREHOLE TYPE Auger and sample with C.M.E. - 55 | CHECKED BY SR

m	ELEV. / DEPTH ft.	DESCRIPTION	NUMBER	TYPE	BLOWS/FOOT (0.3 m)	ELEV. SCALE ft/m	REMARKS GR.SA.SI.CL
159.3	522.7	Ground Level					
0.0	0.0	Het. Mixture of clayey silt, sand and gravel	1	SS	13	520 / 158.5	w.l. 519.7 (158.4)
			2	SS	20		4 30 55 11
			3	SS	41		
		Brown	4	SS	47	510	
		Grey	5	SS	21	155.4	
		Glacial till	6	SS	19		6 32 51 11
			7	SS	28		
			8	SS	33	500	
152.0	498.7	Stiff to Hard	9	SS	91	152.4	6 34 49 11
7.3	24.0	End of Borehole				490 / 149.3	

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT (0.3 m)

SHEAR STRENGTH P.S.F. — O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE

LIQUID LIMIT W_L; PLASTIC LIMIT W_P; WATER CONTENT W — WATER CONTENT % 10 20 30

BULK DENSITY γ P.C.F.

OFFICE REPORT ON SOIL EXPLORATION

20 / 15 5 / 10 % STRAIN AT FAILURE

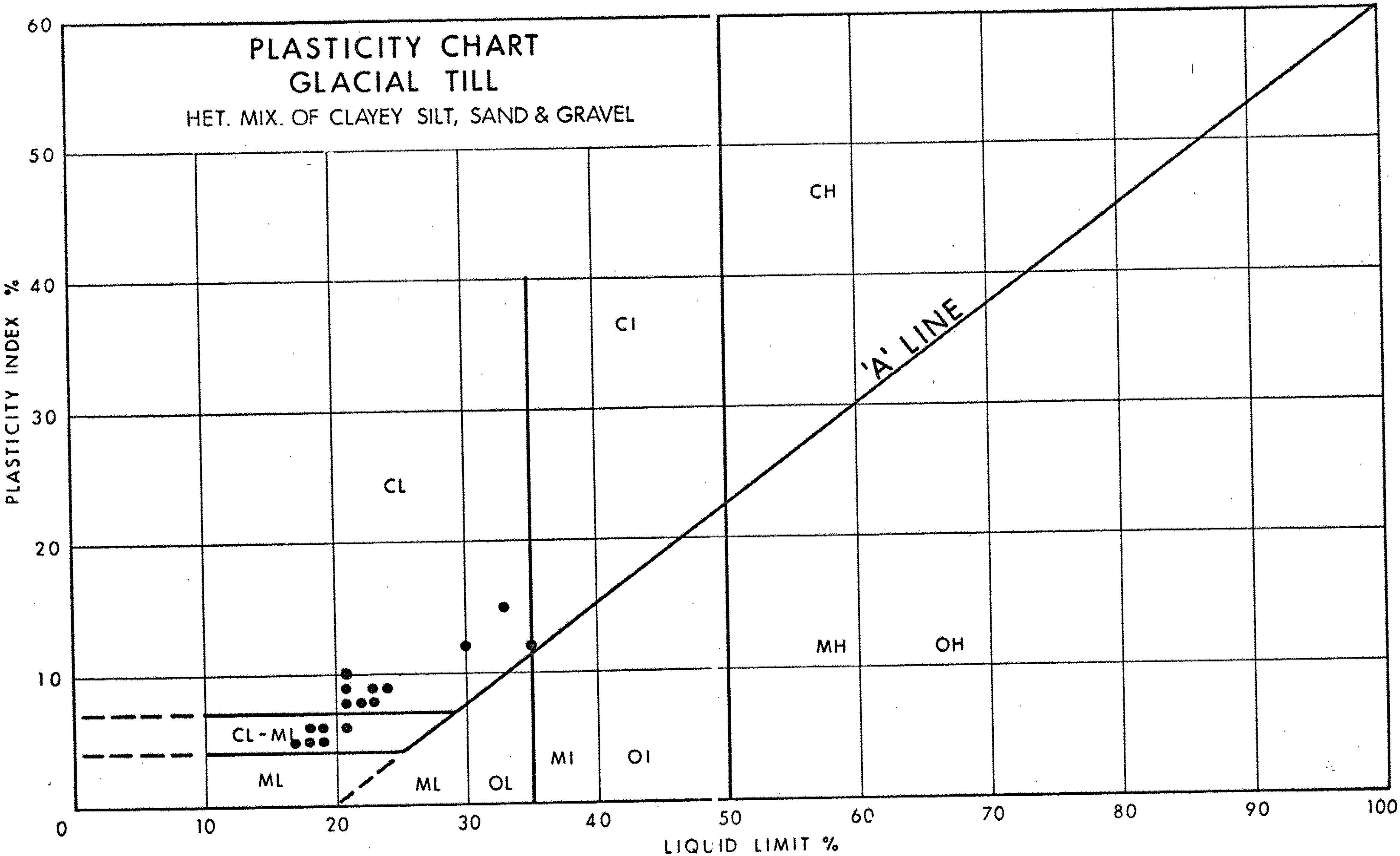


FIG.1

W.O. 73-11118

GRAIN SIZE DISTRIBUTION

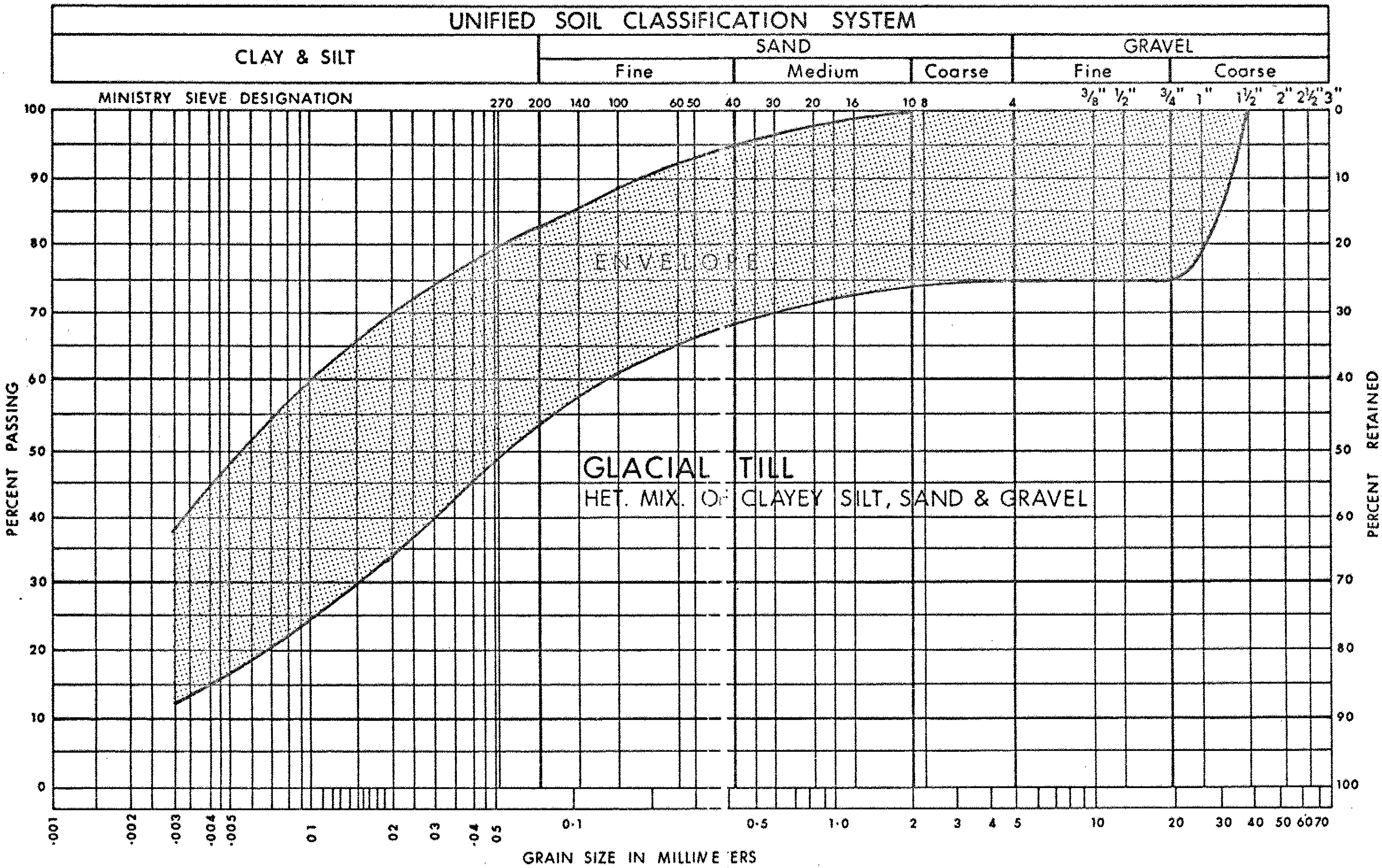


FIG. 2

W.O. 73 - 11118

ABBREVIATIONS & SYMBOLS USED IN THIS REPORT

PENETRATION RESISTANCE

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

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

DESCRIPTION OF SOIL

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

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

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

TERMS TO BE USED IN DESCRIBING SOILS:-

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

TYPE OF SAMPLE

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

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

SOIL TESTS

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

F[illegible]6b (Rev. Jan. 73)

ABBREVIATIONS & SYMBOLS USED IN THIS REPORT

SOIL PROPERTIES

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

GENERAL

π	= 3·1416
e	BASE OF NATURAL LOGARITHMS 2·7183
$\log_e a$ OR $\ln a$	NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$	LOGARITHM OF a TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

STRESS AND STRAIN

u	PORE PRESSURE
σ	NORMAL STRESS
σ'	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
ε	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO (μ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNGS MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

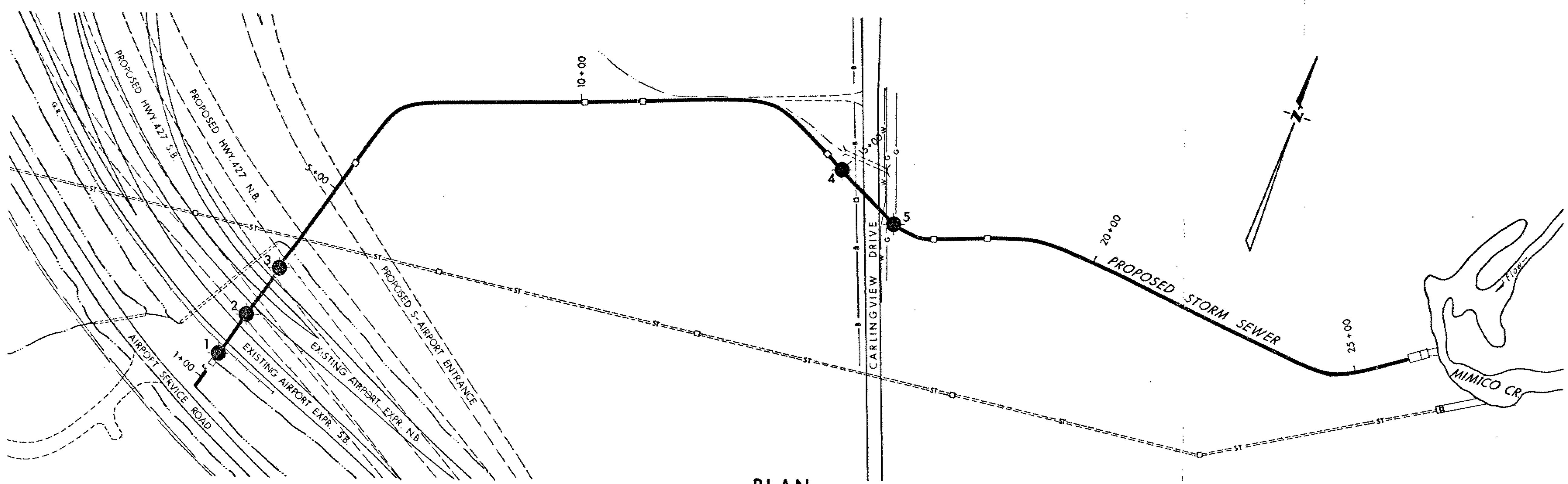
d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_o	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

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

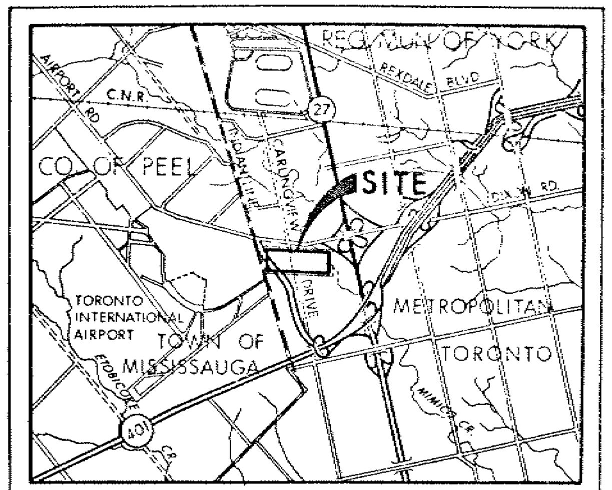
SLOPES

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



PLAN

100 50 0 SCALE 100 200 FT.



KEY PLAN

SCALE IN MILES

1 0.5 0 1 2 3

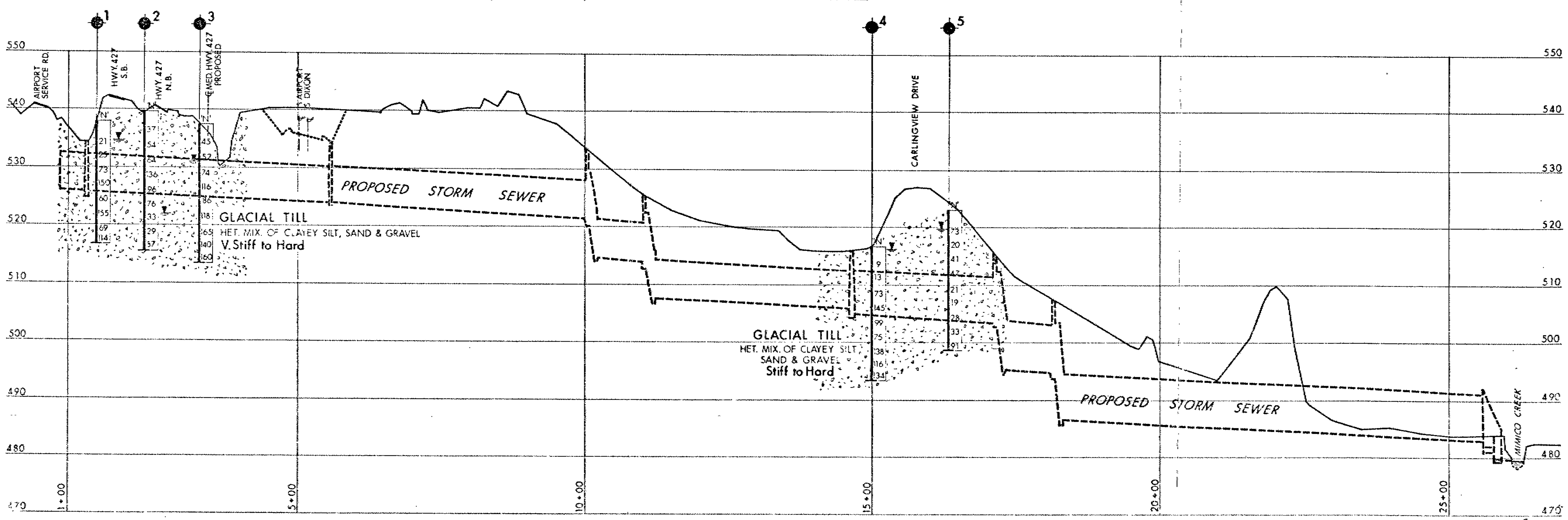
LEGEND

- Bore Hole
- Cone Penetration Test
- Bore Hole & Cone Test
- Water Levels established at time of field investigation. MARCH 1974.

NO.	ELEVATION	STATION	OFFSET
1	537·9	1 + 30	℄
2	539·5	2 + 34	℄
3	537·4	3 + 30	℄
4	516·7	15 + 00	℄
5	522·7	16 + 32	℄

— NOTE —

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



PROFILE

HORIZ. 100 50 0 SCALE 100 200 FT.

VERT. 10 5 0 10 20 FT.

NOTE FOR CONTRACT DOCUMENT

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

REVISIONS		
DATE	BY	DESCRIPTION

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

PROPOSED STORM SEWER
(CARLINGVIEW DRIVE)

HIGHWAY NO. 427 DIST. NO 6
REG. MUNICIPALITY OF YORK
METROPOLITAN TORONTO LOT CON

BORE HOLE LOCATIONS & SOIL STRATA

SUBMD V.K.	CHECKED	W.P. NO 48-71-15	DRAWING NO 73-11118 A
DRAWN S.R.	CHECKED	W.O. NO 73-11118	
DATE APRIL 4, 1974		SITE NO	BRIDGE DRAWING NO.
		CONT. NO	

W.O. 73-11118

INTRODUCTION:

The objective of this report is to present a qualitative analysis of the three alternative methods of maintaining the present storm sewer system which services the Toronto International Airport. This system will be subjected to increased earth loads from planned construction through the Highway 427 corridor. In addition, the Borough of Etobicoke, is planning a new storm sewer system (reinforced concrete) which will cross above the existing sewer in the vicinity of the proposed Highway 427 construction.

The present sewer system design is such that it cannot tolerate any further load.

Description of the Site Geology.

So stated in the Foundation Investigation Report #W.J. 69-F-59; the site is situated in the section known as the "Peel Plain" which at this site is basically a smoothed ground moraine. The glacial deposit is primarily composed of a silty clay with a trace of sand (Malton Clay Till) - in some areas the till is overlain by up to 10 feet of plastic lacustrine deposited stratified clay. The overburden is, in turn, underlain by shale and limestone bedrock some 70 to 75 feet below existing ground surface (bedrock is of the Meaford-Dundas formation, Ordovician Period).

History of the Sewer Installation (from Toronto International Airport.

The storm sewer is a 66-inch diameter C.S.P. with reinforced steel rings. The sewer was installed in the summer of 1958 to improve the drainage regime within the airport area.

The invert elevation of the storm sewer varies from 37.5 to 37 feet below the original ground level (increasing in a westerly direction in the vicinity of the highway complex).

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A cut-and-cover operation was employed for the installation of the sewer. The construction side slopes of the open excavations were maintained at a slope slightly steeper than 1:1. The sewer excavation so formed, was then backfilled with material similar in composition to the parent cohesive subsoil on site.

Based on verbal information provided, it is understood that :

1. The backfill was placed in a haphazard fashion, rather than in uniform lifts, and that it was not properly compacted,
2. The backfilled sewer trench settled up to 6 inches during the construction period alone. The observed settlement was dish-shaped, being greater in the centre of the excavation. (The backfill was placed as such, since at that time the area consisted of open fields and backroads, therefore, it was felt unnecessary at the time to compact the backfill; not looking ahead to future development).

In 1963, Indian Line Road was relocated to its present alignment. At this time the Dixon Road structure was constructed and the expressway ramps to the airport added. The south approach fill superimposes a surcharge loading on the sewer backfill, which varies from 1 ft. at the northbound lane to 10 ft. at the southbound lane.

<u>Subsoil Conditions.</u>

The predominant overburden stratum across the site is composed of Malton clay till; in general, the deposit extends down to a depth of more than 55 feet below the existing ground surface.

The backfill material is similar in composition to the parent Malton till, there are, however, localized zones throughout the area which contain a trace of organic matter.

..... /3

Based on the results of the standard penetration tests, it is estimated that the consistency of the backfill material varies from soft to firm while the parent subsoil varies from very stiff to hard. Further inspection of the samples indicates that the moisture content of the backfill material is generally considerably higher than that of the parent material (i.e. it is in a much wetter state). The groundwater level in the area is about 6 to 9 feet below the ground surface.

Based on observations from Report 69-F-59, it can be concluded that subsidence has occurred within the confines of the backfill to the sewer excavation.

The random pattern of low "N" values observed within the backfill area, implies that:

1. the fill was placed in a haphazard fashion, rather than in uniform lifts, and
2. that the fill was not properly compacted.

Under these conditions, the fill would therefore, tend to have a high percentage of open voids, which would in turn, create favourable conditions for infiltration of free water into the voids, such as would occur during periods of heavy precipitation. This, of course, would tend to soften the surrounding soil and increase the insitu moisture content (substantiated by borehole findings). This resulted in the backfill being subjected to settlement under its own weight. It is pertinent to note that the approach backfill, placed over the improperly compacted sewer backfill material, would add an additional surcharge loading, this loading would increase the magnitude of the settlement within the backfill.

Due to the face that the backfill is basically cohesive in nature, this settlement would be of a consolidation nature. This being the case, it would be time dependent (i.e. - take place over a number of years).

The existing sewer system must now be redesigned in order to accommodate the additional loads that will be imposed on the conduit due to the proposed Hwy. 427 construction. The installation of the new design must be such that it will not affect the performance of the Borough of Etobicoke's new storm sewer at their crossing points. The design must also ensure that there will

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be no differential settlement at the ground surface, such that will adversely affect the vehicular traffic flow along the proposed Hwy. 427.

Design Methods.

Before any design can be selected, the existing C.S.P. will have to be examined to determine the extent of the damage to the conduit, due to the excessive loads and settlement of the present conditions. This information will be provided by McCormick, Rankin and Associates Ltd. (Consulting Engineers).

Since the existing sewer is covered with unconsolidated material, it is expected that the loads will be increased by the necessary compaction of the backfill, as well as, by an increase in the original height of fill. The three alternative methods are:

1. to place a new 66-inch diameter C.S.P. adjacent to the existing pipe, remove the top half of the existing pipe and backfill the excavation using 1:1 slopes, with suitable compacted material.
2. To expose the existing sewer, construct a protective reinforced concrete cap over the 66-inch diameter pipe, replace the 12-inch diameter pipe, and backfill the excavation with suitable compacted earth material.
3. To replace the existing fill with properly compacted backfill incorporated with the "Imperfect Trench" method.

The estimated cost of constructing method 1. is equal to the cost for method 2. which is approximately $220,000 exclusive of engineering costs. The cost of method 3. will be approximately half of the other two methods. (I am estimating its cost in relation to the others, through the information I obtained during my research of the method). In considering the construction procedure for schemes 1. and 2., method 2. appears to have advantage, in that the concrete

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cap can be placed now over a relatively short length of 50 feet to protect the pipe under the new storm sewer, at their crossing point. Otherwise, if method 1. is employed some 310 feet of 66-inch diameter pipe will have to be placed now with two new manholes, to ensure maintenance of flow. Also, since the existing storm sewer is the main drainage system for the airport area, its vital service cannot be deleted for even a short period of time.

Although the "Imperfect Trench" method (3.) is the cheapest design of the three alternatives, it is basically a new conception in the pipeline design and its long-term effects have not been fully anticipated or recorded.

The imperfect trench was first developed by Dean Marston during his early days of research on conduit loads. He was impressed by the very high loads that may develop on projecting conduits when the conditions are such that large shearing forces add to the weight of the prism of soil directly over the conduit. He strove to devise a method of construction that would reduce or eliminate these shearing forces, or would possibly reverse their direction so that they would act benevolently as in the case of ditch conduits. With this objective, he developed the imperfect ditch method of construction, in which the soil on both sides and above the conduit for some distance is thoroughly compacted by rolling, tamping, or any other suitable method. Then a ditch is constructed in this compacted fill by removing the prism of material directly over the conduit. The ditch is refilled with very loose compressible material, after which the embankment is completed in a normal manner.

The purpose of this method of construction is accomplished by creating a condition wherein it is certain that the prism of material directly over the conduit will settle more than the adjacent prism, thereby ensuring the development of the upward shearing forces; that is, an arching action, which greatly reduces the load on the structure. The ditch in the artifically compacted material must be deep enough and the refilling material must be

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loose enough to ensure this action. Straw or other highly compressible material may be used as part of the ditch backfill to augment the settlement of the interior prism.

Favourable results were obtained by the imperfect trench method when it was employed in the construction of reinforced concrete conduits under high fills. As an example, a portion of Interstate 74 in Vermilion County, Illinois, crosses a ravine with a fill 37.5 feet above the top of a 48-inch reinforced concrete culvert. An imperfect trench excavation over the pipe was filled to about 1/3 of its depth with loose straw which was then covered with loose soil. The fill was brought up to grade, being completed in November, 1959. Periodic examinations of the site since then have found no signs of distress in the pipe. There has been no settlement in the fill and none is anticipated. Performance of the pipe was measured by observing the distortion of the pipe compared with the distortion of similar pipes installed conventionally. (Distortion is defined as the difference between the horizontal and vertical diameters of the pipe, expressed as a percentage of the theoretical pipe diameter). (Highway Research Board, Proc. 41st Annual Meeting, 1962)

Most experiments using rigid conduits and the imperfect trench method produced favourable results. However, results of the imperfect trench method used with flexible pipes were not satisfactory in reducing the stress on the pipe structure itself. The flexible pipe has been designed to deform slightly to develop the soil arching action over the pipe.

In bulletin 212 of the Corrugated Steel Pipe Institute, a subtitle states, "That the imperfect trench theory and method is very imperfect." This statement was referring to the Lethbridge Culvert in which the imperfect trench was incorporated in a 110-ft. fill over a 103-inch diameter C.S.P. The culvert was placed on a two-ft. thick granular pad placed on top of a medium plastic glacial clay. The backfill to approximately mid-height of the culvert, and for a distance of about 10 feet on either side, consisted of

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compacted sand-gravel clay mix. The remainder of the fill adjacent to and over the culvert consisted of medium plastic glacial clay. When the fill was completed to 8 feet above the culvert crown, an 8-foot wide by 6-foot deep trench was excavated over the culvert, and backfilled without compaction. This is a practice often prescribed by rigid design advocates as a procedure to effect a reduction in load on the culvert (i.e. the imperfect trench method). Culvert strain readings indicated that very little, if any, reduction in load was obtained. This agrees with previous test information for structures under high fills published by the Highway Research Board.

The vertical deflection of the culvert sections after 2 years varied from 1.5 inches to 3.8 inches, and the invert settlement after 2 years varied from about 1¼ inches at the ends and about 6½ inches at the fill centreline.

All initial experimental work with the imperfect trench method was done on rigid pipes (reinforced concrete) which usually produced satisfactory results. Recently experiments have been performed on flexible conduits with unfavourable results.

The California Department of Transportation (Division of Highways) published a paper in 1972, on the experimental results of the effects of the Standard Method of backfill (method A) and the Imperfect Trench Method (method B) on flexible culverts under high fills - the observations were recorded over a period of 6 years (from 1966 to 1972).

The experiment was carried out under 160 ft. and 68 ft. of fill for method A and under 89, 81, and 76 feet of fill for method B (with 3 to 5 feet of baled straw as a compressible material).

The conclusions of this Report are as follows:

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The nearly uniform pressure profile, linear pressure, fill height functions and small stress and moment gradients observed for method A, suggest that this method of backfill provides more favourable conditions for flexible culverts than does method B backfill. Control of deflections, a critical consideration in present culvert design, was adequate using either method, although vertical deflections under method B were smaller. Based on these results of the two research projects, the use of Method B backfill for flexible culverts is not recommended.

Also concluded in general, was that the design methods discussed in this paper did not predict the observed culvert behaviour accurately (the theoretical analyses of the report are: Marston's Theory, Spangler's Iowa Deflection Formula, the Ring Compression Theory, the Neutral Point Analysis and the Finite Element Analysis).

Also, from the observations of method B, the observed plane of equal settlement lay between 30 and 32 feet above the crown for approximately 80 feet of fill (the "plane of equal settlement", is the horizontal plane above which the settlements of the interior and exterior prisms are equal).

In a paper written by William Clarke, in 1967, on "The Loads Imposed on Conduits Laid Under Embankments or Valley Fills", he concluded that at depths imposing the "complete" condition (i.e. where the plane of equal settlement lies above the ground surface) the imperfect trench method is not suitable for use under roads, railways, industrial yards, or in other suituations where differential settlement of the surface of the fill over the pipe line would be unacceptable.

The "Imperfect Trench Method",is the least expensive of the three alternative methods, but, its long-term performance has not been fully evaluated to date. This statement is supported by the Highway Research Board, "the long-term serviceability of imperfect ditch installations presents a largely unanswered question", (from the research program report #116). Considering the uncertainty of failure and its adverse effects on the serviceability of the storm sewer due to this method, and the function of the above roadway and that of the proposed storm sewer (by the Borough of Etobicoke), it has been decided by this Section that the use of this method

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will be impractical for the new design of the existing storm sewer. It is felt that a proven method, such as method 2 (the use of a protective reinforced concrete cap) should be used for this project.

In my research I had the opportunity to speak to the Chief Engineer of the Corrugated Steel Pipe Institute (Bill Porter) about the use of the imperfect trench method with a flexible conduit. He stated that they do not recommend the use of this method for any flexible conduit installations, thus supporting our decision.

In designing the concrete cap sewer protection (designed by McCormick, Rankin & Associates), a footing soil bearing value of 2.5 tons per sq. ft. was assumed, which is a safe, conservative figure. However, our section feels that a safe soil bearing value of 4 tons per sq. ft. would be acceptable in its design, thus allowing a smaller footing width (i.e. approximately 3 ft. on each side of the pipe as compared to the previous 4.5 ft. design). Therefore, if this design criteria is used, the estimated cost of construction would be reduced from the previous figure of $220,000.

Physical Properties and Compaction Characteristics
<u>of Side Supporting Material (for Flexible Conduits).</u>

The material furnishing side support to an underground conduit has been recognized to constitute one of the most important parts of a culvert installation, especially under a high fill. This material, if properly chosen and properly compacted, will mobilize lateral pressures against the sides of the conduit that will balance the top pressures exerted by the fill overburden. Thus, even if the inherent strength of the conduit is not appreciable, the structure will be able to withstand external pressures of great amount without failing in shear or by excessive deformation.

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The Soil Mechanics Section has concluded that the most feasible method of maintaining the present storm sewer system will be to install a protective reinforced concrete cap, accompanied by properly compacted backfill material.

SUMMARY OF FULL REPORT:

- Objective: to analyse 3 methods of maintaining the present storm sewer system. It will be subjected to increased earth loads from Hwy. 427 construction. A new storm sewer will cross above the existing sewer at Hwy. 427. The sewer design must not affect these structures.
- The present sewer system cannot tolerate any further loads.
- The site geology is primarily composed of a cohesive glacial till.
- The original sewer is 66" dia. C.S.P. - installed in summer of 1958 for drainage of airport area. It is situated approximately 37 ft. below the original ground level. The backfill material was not compacted, it was placed in a haphazard fashion in the excavation of 1:1 slopes. Additional backfill was added in 1963 of 1 to 10 ft.
- The backfill material is similar in composition to the parent cohesive till, but its consistency varies from soft to firm while that of the parent material is very stiff to hard. The backfill material also has a considerably higher moisture content than that of the parent material. The ground water level is approximately 6 to 9 ft. below the groundsurface.
- Before a design can be selected the existing C.S.P. must be examined for failure.

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- The 3 designs are: i) replace the existing C.S.P.
  ii) to place a protective reinforced concrete cap over the pipe - both methods will require suitable backfill material.
  iii) to replace the existing fill with properly compacted backfill, incorporated with the "Imperfect Trench Method".
- Methods i)and ii)cost approximately $220,000, method iii) will be approximately 1/2 of this figure.
- The imperfect trench method is basically a new concept in pipeline design and its long-term effects are questionable. The imperfect trench is designed to reduce the stress on conduits (research was carried out on rigid conduits with favourable results) by reducing or eliminating the effects of the shearing forces (or having them act upwards), thus an arching action is produced.
- Unfavourable results were produced when the imperfect trench method was used with flexible conduits. This fact was published in many reports of which a few are: In bulletin 212 of the Corrugated Steel Pipe Institute, the imperfect trench method was used in a 110 ft. fill over a 103" dia. C.S.P. The culvert strain readings indicated that very little, if any, reduction in load was obtained. This agreed with previous test information published by the Highway Research Board. The California Department of Transportation (Division of Highways) published a paper in 1972, on the experimental results of the effects of the Standard Method of Backfill and the Imperfect Trench Method on flexible culverts. The observations were recorded over a period of 6 years. Based on the results of the research project, the use of the imperfect trench method for flexible culverts was not recommended. Also the design methods did not predict the observed culvert behaviour accurately. In a paper written by William Clarke, 1967, he concluded that where the plane of equal settlement was at the ground surface (or above) the imperfect trench method would be

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unacceptable due to differential settling. The Highway Research Board has stated that "the long-term serviceability of imperfect ditch installations presents a largely unanswered question". The chief engineer of the Corrugated Steel Pipe Institute (Bill Porter) stated that they do not recommend the use of the imperfect trench method with flexible conduits.

From these conclusions it was decided by the Soil Mechanics Section that the imperfect trench method was not suitable for this project.

- It was decided that the most feasible method of maintaining the existing sewer system is to install a protective reinforced concrete cap, accompanied by properly compacted backfill material.

J. A. Andersen

Mr. N.D. Smith,
Regional Systems Design Office,
Central Region, Toronto.

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

April 26th, 1974.

RE: Protection for the existing 66" Dia. Storm Sewer, Carlingview Storm Sewer System, from Toronto International Airport to Mimico Creek, Borough of Etobicoke,
W.O. 73-11118 W.P. 46-71-15.

We have reviewed the proposal prepared by McCormick, Rankin & Associates Limited (dated April 19th, 1974), and submit our comments as follows:

The subsoil in this area consists of an extensive glacial deposit of heterogeneous mixture of clayey silt, sand and gravel. The consistency of the glacial till varies from very stiff to hard (refer to B.H.#1, #2 and #3, W.O. 73-11118). The glacial till is capable of providing a safe bearing pressure of 2.5 t.s.f., which was used in designing the footings of the concrete cap sewer protection, provided that the till had not been disturbed and softened during the installation of the sewer. It is therefore, recommended that extreme care should be exercised during construction period to ensure that the reinforced concrete cap is founded on undisturbed hard, glacial till.

Other recommendations pertaining to temporary cuts, dewatering and backfilling as discussed in our Foundation Report, W.O. 73-11118 will be applicable in this project.

Should you require further information or clarification of the foregoing, please contact our Office.

C.S. Poon,
Project Engineer
For: M. Devata,
Supervising Engineer.

CSP/mj
c.c. McCormick, Rankin
(attn: Mr. C.P. Korzeniowski)

Files
Documents

73-11118

McCORMICK, RANKIN & ASSOCIATES
LIMITED

CONSULTING ENGINEERS

~~8 STAVEBANK ROAD~~
PORT CREDIT, ONTARIO
L5G 2T4

TELEPHONE 274-3477

RECEIVED
APR 19 1974
M.T.C.
REGIONAL
SYSTEMS
DESIGN

April 19, 1974

Mr. R.S. Pillar, P.Eng.
Manager, Regional Systems Design
Central Region,
Ministry of Transportation & Communications
3501 Dufferin Street,
DOWNSVIEW, Ontario.
M3K 1N6

our comments sent to N.D. Smith on April 26/74.
C.P. Poon

Attention: Mr. N.D. Smith, P.Eng.

RE: CARLINGVIEW STORM SEWER
Toronto International Airport to Mimico Creek
Borough of Etobicoke
Our File: W.O. 738-73

Dear Sir:

Further to the progress meeting for W.P. 48-71, held here on April 10, 1974, we are responding to the question of protection for the existing 66" dia. storm sewer from the airport.

At the meeting, it was agreed to investigate two alternative methods of maintaining the present sewer system, which will be subjected to increased earth loads from planned construction through the Highway 427 corridor. Since the existing sewer is covered with unconsolidated material, it is expected that the loads will be increased by the necessary compaction of the backfill, as well as, by an increase in the original height of fill. The two alternative methods are;

1) to place a new 66" dia. C.S.P. adjacent to the existing pipe, remove the top half of the existing pipe and backfill the excavation using 1:1 slopes, with suitable compacted earth material.

2) to expose the existing sewer, construct a protective reinforced concrete cap over the 66" dia. pipe, replace the 12" dia. pipe, and backfill the excavation with suitable compacted earth material (as shown on the enclosed sketch).

......

Attention: Mr. N.D. Smith, P.Eng.

The estimated cost of constructing method 1 is equal to the cost for method 2, which is approximately $220,000 exclusive of engineering costs. In considering the construction procedure for both schemes, method 2 appears to have one advantage, in that, the concrete cap can be placed now over a relatively short length of 50 feet to protect the pipe under the Borough of Etobicoke's contract for a new storm sewer, at their crossing point. Otherwise, if method 1 is employed, some 310 feet of 66" dia. pipe will have to be placed now with two new manholes, to ensure maintenance of flow. On this basis, we would like to recommend that method 2 be used to cap the existing storm sewer.

In designing the concrete cap sewer protection, we have used a footing soil bearing value of 2.5 tons per sq. ft. Without having the benefit of a foundation investigation, we feel that this value is a safe one, perhaps even conservative. However, the Foundation Section should confirm this assumption with a field investigation.

We request your decision on this matter as soon as possible, in order not to delay the Borough of Etobicoke's contract for a new storm sewer which will cross the existing sewer in question, in the area of the proposed Highway 427 construction.

We are enclosing also, two prints of a plan and profile showing the existing and proposed sewers across the planned Highway 427 development.

Yours very truly,

McCORMICK, RANKIN & ASSOCIATES LIMITED

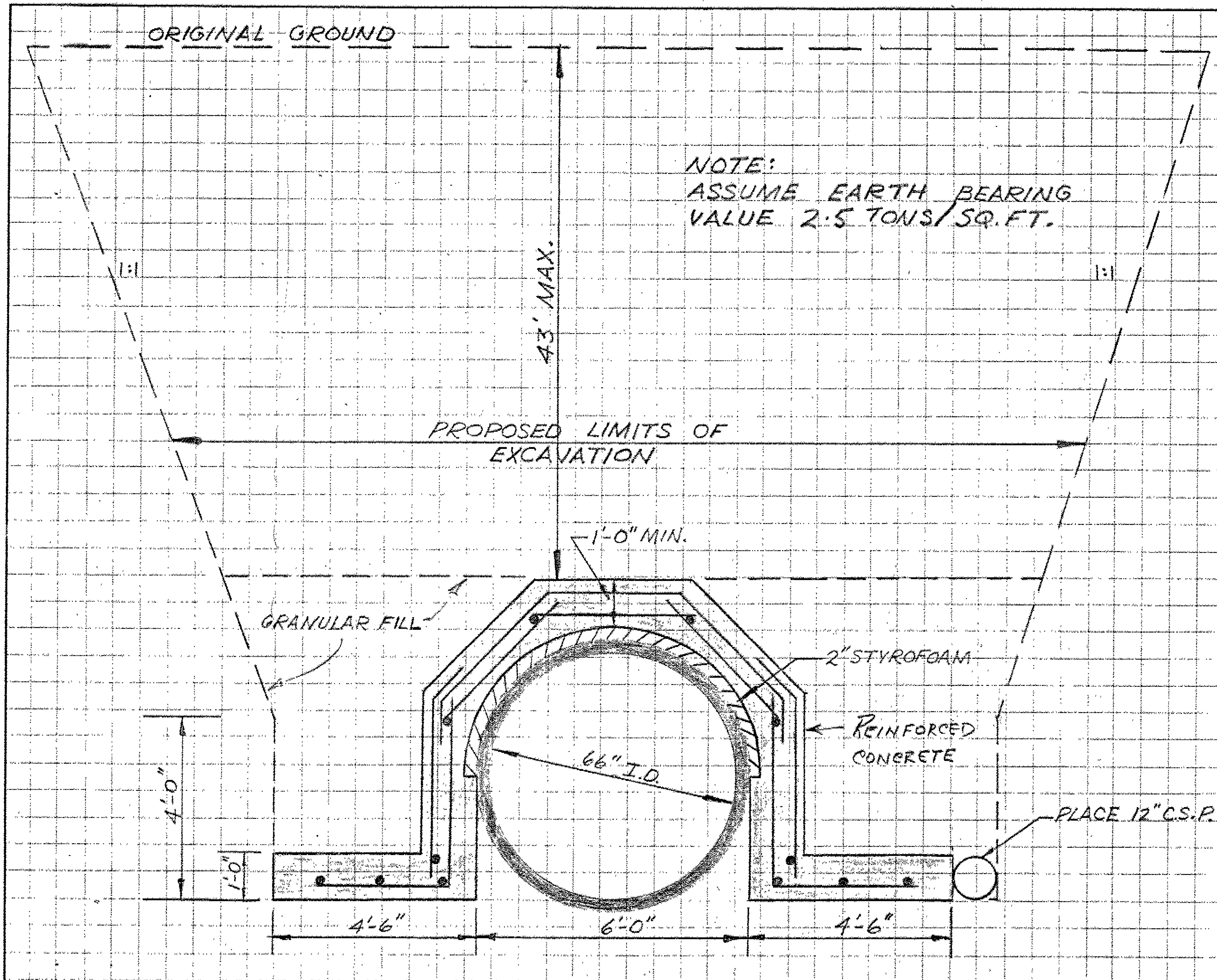
C.P. Korzeniowski

C.P. Korzeniowski, P.Eng.

CPK:db
Encl.
c.c. Mr. A.G. Taylor, P.Eng.

McCORMICK, RANKIN & ASSOCIATES
LIMITED
CONSULTING ENGINEERS

PAGE



SEWER PROTECTION
for
THE EXISTING 66" DIA. STORM SEWER

W.O. NO. 738-73 W.P. NO.

PROJECT CARLINGVIEW SEWER

DESIGNED KM DATE APRIL/74

CHECKED DATE

WORK PROJECT No.

1. Project Correspondence
2. Property
3. Utilities
4. Municipal Correspondence and Agreements
5. Traffic, Structure, Signal Illumination, Correspondence and Reports
6. Minutes of Meetings — Internal and External
7. Soils and Foundation Correspondence and Reports
8. Estimates and Contract Documents
9. Public Participation

10. ....

R. ODDS N

J.G. Celmins

R. [illegible]

D. Smith

W. Roters

D. [illegible]

[illegible]son

McCORMICK, RANKIN & ASSOCIATES
LIMITED

CONSULTING ENGINEERS

Carlingview Sewer Protection

① Existing Condition – soft silty clay with rel. high moisture
- unit weight ≈ 100 pcf.
- assumed trench width at top of pipe 9' + 2 = 11'
- maximum depth = 38'

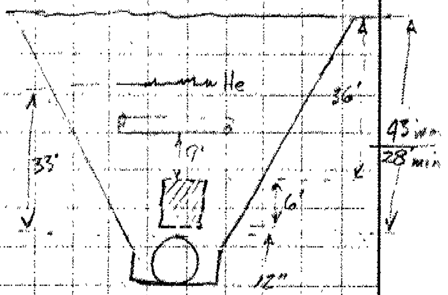
* table 28C Pipe Load = 27,580 lbs. / l.f.

2. Induced Trench Installation with properly compacted backfill.

$H = 43'$

using $\rho' = 1.0$, $r_{sd} = -0.7$

soil weight. = 125 pcf.



* figure 123 Pipe Load = 1.25 × 16,500

= 20,650 lbs / l.f. where He = 33'

3. Standard installation with properly compacted backfill

$H = 43'$, soil wt. = 125 pcf

Trench width at top of pipe = 12'

* figure 64b Pipe Load = 1.25 × 35,200

= 44,000 lbs. / l.f.

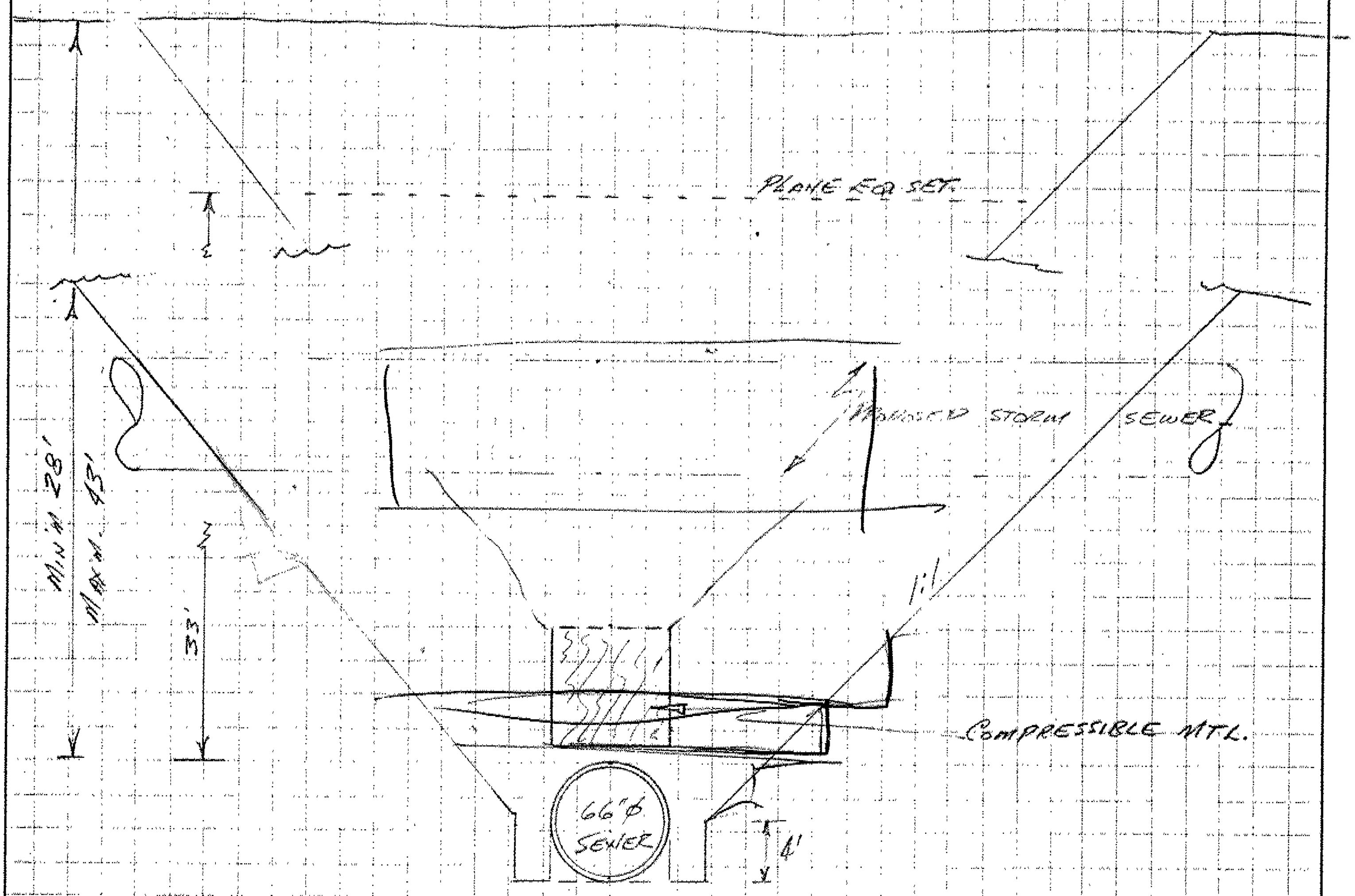
* O.C.P.A. Design Manual.

W.O. NO. 660-082 W.P. NO. ______ DESIGNED CJK. DATE 23 May 74

PROJECT 66" STORM SEWER PROTECTION CHECKED ______ DATE ______

CARLINGVIEW SEWER PROTECTION

INDUCED TRENCH TREATMENT



W.O. NO. 660-082 W.P. NO. DESIGNED GPK DATE 24 May 74
PROJECT 66" STORM SEWER PROTECTION CHECKED DATE

73-11118

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

TO: Mr. M.S. Devata,
Supervising Foundations Eng.,
Geotechnical Office,
West Bldg.

FROM: N.D. Smith,
Regional Systems Design.

ATTENTION:

DATE: February 27, 1974.

OUR FILE REF.

IN REPLY TO

SUBJECT: Re: W.P. 48-71-150, Highway 427,
Carlingview Storm Sewer System,
District #6, Toronto.

Further to our telephone conversation of Feb. 22/74, I am attaching a copy of a letter from McCormick, Rankin & Associates, outlining the foundation investigation required at the crossings of the proposed sewer line at Hwy. 427 and at Carlingview Drive, together with two copies of each of the pertinent plans.

As we agreed, would you please instigate this investigation as quickly as possible.

The cost of the investigation is to be borne by this Ministry.

N.D. Smith
Project Design Engineer
For:
J.Geo. Celmins
Sr. Project Design Engineer

NDS/GB
Attach.

c.c. McCormick, Rankin & Associates
(Attn: C.P. Korzeniowski)

March 6 /74
\$763.75 P.V.K.