

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

MEMORANDUM

TO: Mr. A. G. Stermac,
Pr. Foundations Engineer,
Foundations Office.

FROM: Materials & Testing Office,
Central Region.

ATTENTION: Mr. M. Devata.

DATE: February 7, 1972.

OUR FILE REF.

IN REPLY TO

SUBJECT: Foundation Investigation
W.P.'s 10-57-02 and -06
Stoney Creek Traffic Circle
Hamilton District

During a telephone conversation with Mr. M. Devata it was agreed that the Foundations Office would investigate the proposed storm sewers and one 6' x 4' box culvert for the Regional Materials & Testing Office at the same time they did the investigation for the Highway 20 Overpass.

Enclosed you will find a 1" = 100' scale plan showing:

- (i) the location and invert elevation for the proposed 6' x 4' concrete box culvert, station 642+50
- (ii) the location and invert elevations for the proposed storm sewer

NOTE: Across the Q.E.W., two alternate schemes are shown. It is requested that both schemes be investigated.

We would like to obtain your recommendations for the placement, bedding and backfilling for the culvert and sewers and also your recommendation regarding which storm sewer alignment across the Q.E.W. would be most suitable from a soils' point-of-view.

We are also including a plan and sections for the various proposals for the retaining wall along side the Pines Motel on Highway 20. After discussion with the Pines Motel owner, it was agreed that the high retaining wall Scheme B would be used to replace the previous superseded scheme (the location of which is shown in red on the plan). A Foundation Report was issued for this scheme on August 20, 1970 (W.O. 70-11035).

We would like you to review the new retaining wall scheme

continued:-

Mr. A. G. Stermac.

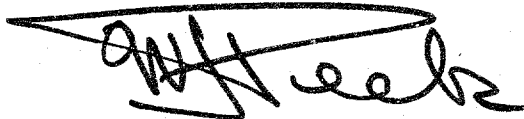
February 7, 1972.

Re: W.P.'s 10-57-02 and -06

and comment if any changes are required to your recommendations for the previous retaining wall.

If you require any field layout for your investigation, contact Mr. Jon Kozel of M. M. Dillon Limited.

If you have any question regarding the above, please feel free to contact me at any time.

A handwritten signature in black ink, appearing to read "W. J. Peck", enclosed within a hand-drawn oval.

W. J. Peck,
Project Soils Engineer.

WJP/js,

cc: M. M. Dillon Limited
(Attn: J. Kozel)
N. D. Smith
R. Fitzgibbon

72-11038
DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

MEMORANDUM

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

FROM: G. C. E. Burkhardt,
Structural Planning Office,
90 Floral Parkway.

ATTENTION: M. Devata

DATE: February 14, 1972.

OUR FILE REF.

IN REPLY TO

SUBJECT: Hwy. 20 Underpass at Stoney Ck.,
W.P. 10-57-02, Site 36-144,
Q.E.W., District 4.

Further to our discussions and correspondence of September 1971, we are attaching hereto two revised prints of the preliminary bridge drawing.

Please proceed with your investigations and the preparation of the necessary report.

We would refer you also to your drawing #69-F-70A-3 "Location of Existing Sanitary Sewer" and request that you extend this drawing to locate this sewer in the vicinity of the proposed south abutment.

JFW:lc
Attach.

J. F. Walshe

J. F. Walshe,
STRUCTURAL PLANNING SUPERVISOR,
for:
G. C. E. Burkhardt,
REG. STRUCTURAL PLANNING ENG.

c.c. C. S. Grebski
N. D. Smith
R. Fitzgibbon

Department of Highways Ontario
Copy for the information of

Mr. M. Devata

Structural Planning Office,
Central Region,
90 Floral Parkway,
Toronto 15, Ontario,
Telephone: 248-3897,
February 18, 1972.

M. M. Dillon & Co. Ltd.,
P. O. Box 219,
Station 'K',
TORONTO 12, Ontario.

Attention: J. Kozel

RE: Hwy. 20 Underpass at
Stoney Creek,
W.P. 18-57-02, Site 36-144,
O.E.W., District 4.

Dear Sir,

Herewith are two prints of the preliminary bridge drawing for this structure.

We have requested our Foundation Section to proceed with the necessary subsurface investigation. It is possible that their recommendations may result in changes being required, which may affect the location of the abutments.

The structure designers have requested that you provide the horizontal geometry necessary for the removal of the 'kink' at Sta. 0 + 00 Ramp N-E. The 'detail A' shown on the drawing should be treated as a suggestion. We would suggest that a more mathematically definable curve compound be used, regardless of the length involved, which keeping the 'excess' structure within acceptable limits. We assume that revisions to the horizontal alignment of Ramp N-E will also affect the pavement elevations in this area.

Revisions to the pavement elevations of the north collector road should await the recommendations of the Foundation Section, since the exact structural depth depends on the length of the end spans.

Your comments on the proposed structure will be appreciated.

Yours truly,

J. F. Walshe

J. F. Walshe,
STRUCTURAL PLANNING SUPERVISOR,
for:
G. C. E. Burkhardt,
REG. STRUCTURAL PLANNING ENG.

JFW:lc
encl.

c.c. C. S. Grebski
H. D. Smith
M. Devata
R. Fitzgibbon

MEMORANDUM

TO: Mr. G. C. E. Burkhardt, (2) FROM: Foundations Office,
Regional Structural Planning Engineer, Design Services Branch,
Central Region, Central Bldg., Downsview.
90 Floral Parkway,
ATTENTION: F. Walshe. DATE: April 7, 1972.
OUR FILE REF. IN REPLY TO

SUBJECT: Hwy. #20, Overpass at Stoney Creek Traffic
Circle, W.P. 10-57-02, Site 36-144, Q.E.W.
District #4 (Hamilton)

69-5-71
72-11-038
033

1. INTRODUCTION:

The original foundation investigation was carried out for a scheme incorporating an underpass structure at the crossing of the Q.E.W. and revised Hwy. #20 in an area presently occupied by the western leg of the Stoney Creek Traffic Circle. In addition, two separate structures will be required in the scheme where C.N.R. tracks cross the Q.E.W. and revised Hwy. #20, respectively. The results, together with our recommendations for the original scheme were submitted in the separate reports, W.O. Nos. 70-11035, 69-F-70 and 69-F-71. Subsequently, it was decided to abandon the railway crossings within the Stoney Creek Traffic Circle complex and construct an overpass structure at the middle of the Traffic Circle where Hwy. #20 crosses the Q.E.W. Preliminary recommendations based on the available data were submitted in a memo to the Regional Bridge Planning Section on September 20, 1971.

A request to carry out a foundation investigation at the revised location of the overpass structure at the crossing of Hwy. #20 (reconstructed) and Q.E.W. was contained in a memo from Mr. G.C.E. Burkhardt, Regional Bridge Planning Engineer, dated February 14, 1972.

Due to the urgency of this project, we have been requested to submit our written recommendations as soon as the field work has been completed. The final report will be submitted after the completion of drawings and borehole logs. A brief review of soil conditions, together with our recommendations for the structure foundations and approach fills follows.

2. SUBSOIL:

The predominant stratum across the site is composed of a grey, clayey silt to silty clay with trace of sand and gravel.

The thickness of this deposit ranges from 33 feet to 48 feet. In the northern portion of the complex the consistency of the deposit in the upper 10-12 feet ranges from soft to firm. Elsewhere the consistency of the deposit varies from firm to hard. In certain portions of the alignment, an 8 feet surficial cover of sandy silt to silt was encountered. Underlying the cohesive clayey silt to silty clay stratum is a 23 to 27 feet deposit of glacial till composed of clayey silt with sand and gravel. The glacial till is followed by sound shale bedrock with grey limestone bands.

Groundwater level measured in the open boreholes was found to vary between elevation 250 and 264; i.e. some 1 to 4 feet below existing ground surface.

3. RECOMMENDATIONS:

3.1) Approach Embankments:

It is proposed to construct approaches with the maximum fill height of 21 feet. Fills of the north approach embankment will not be stable with standard 2:1 slopes in view of the presence of soft to firm silty clay to clayey silt stratum at a relatively shallow depth beneath the original ground surface. In order to ensure the stability of the north approach the following procedures should be adopted.

- i) Fills up to 19 feet can be constructed with standard 2:1 slopes.
- ii) Fills in excess of 19 feet will require a mid-height berm - for example, a 21 feet embankment will require a mid-height berm of 15 feet in length.
- iii) A smooth transition should be affected between zero berm and full berm.

Alternatively, the stability of the north approach embankments can be ensured by excavating the upper 10-12 feet of the soft to firm compressible portion of the cohesive stratum. The excavation should extend for the full base width of the embankment and a minimum distance of 100 feet northerly from the north abutment location. The excavation so formed should be backfilled with well-compacted suitable granular type of material. If such measures are adopted the north approach embankment can also be constructed with standard 2:1 slopes without the use of any stabilizing berms. The details of excavation will be discussed in our final Foundation Report W.O. 72-11033.

Stability problems are not anticipated for the south approach embankment with standard 2:1 slopes.

3.2) Structure Foundations:

The presence of a relatively compressible zone at a shallow depth precludes the economical use of spread footings for the support of the structure elements. The new overpass structure should, therefore, be supported on end-bearing steel H-piles driven to practical refusal within the hard glacial till stratum. It is estimated that these piles can attain the maximum allowable loads at the following tip elevations:

North Abutment	205
North Pier	203
Centre Pier	205
South Pier	195
South Abutment	196 (to bedrock)

3.3) Settlement Considerations:

The maximum settlements will occur beneath the north approach fill due to the embankment loading on the compressible zone of natural subsoil. Estimated settlements in this area will be of the order of 4 inches.

We feel that the above information will provide adequate data for your design requirements. If additional information or any clarification is required, please feel free to contact this Office.

Shaher Ahmad

For: S. A. Ahmad,
Project Foundation Engineer,
M. Devata,
Supervising Foundation Engineer.

SAA/ao

cc: Messrs. A. Rutka
C. S. Grebski
A. Radkowski
C. R. Robertson
G. A. Wrong
T. J. Kovich
W. C. Friedmann
M. M. Dillon & Co. Ltd.
J. Anderson

Foundations Files
Documents



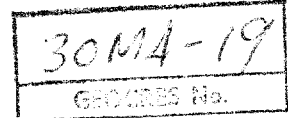
DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

MEMORANDUM

TO: Mr. G. C. E. Burkhardt, (4) FROM: Foundations Office,
Regional Structural Planning Eng., Design Services Branch,
Central Region, Central Bldg., Downsview.
90 Floral Parkway,
ATTENTION: Downsview, Ontario. DATE: May 10, 1972.
OUR FILE REF. IN REPLY TO MAY 12 1972

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For
Proposed Underpass Structure at
The Crossing of the Reconstructed
Q.E.W. & Hwy. #20, Stoney Creek
Traffic Circle, Co. of Wentworth
District No. 4 (Hamilton)
W.O. 72-11033 -- W.P. 10-57-02



Cent 74 110

Attached we are forwarding to you our detailed foundation investigation report on the subsoil conditions existing at the above-mentioned site.

We believe that the factual data and recommendations contained therein will prove adequate for your design requirements. Should additional information be required, please do not hesitate to contact our Office.

AGS/ao
Attach.

cc: Messrs. D. W. Farren

B. R. Davis

A. Rutka

P. J. Harvey

C. R. Robertson

B. J. Giroux

T. J. Kovich

G. A. Wrong

B. A. Singh

M. M. Dillon & Co. Ltd.

Foundations Files
Documents


A. G. Stermac,
PRINCIPAL FOUNDATIONS ENGINEER.

TABLE OF CONTENTS

1. INTRODUCTION.
 2. DESCRIPTION OF THE SITE AND GEOLOGY.
 3. FIELD AND LABORATORY WORK.
 4. SUBSOIL AND BEDROCK CONDITIONS.
 - 4.1) General.
 - 4.2) Silty Sand to Silt.
 - 4.3) Clayey Silt to Silty Clay.
 - 4.4) Clayey Silt with Sand and Gravel (Glacial Till).
 - 4.5) Shale Bedrock.
 5. GROUNDWATER CONDITIONS.
 6. DISCUSSION AND RECOMMENDATIONS.
 - 6.1) General.
 - 6.2) Approach Embankments.
 - 6.2.1) Stability Considerations.
 - 6.2.2) Settlement Considerations.
 - 6.3) Structure Foundations.
 - 6.3.1) Pier Foundations.
 - 6.3.2) Abutment Foundations.
 7. MISCELLANEOUS.
-

FOUNDATION INVESTIGATION REPORT
For
Proposed Underpass Structure at
The Crossing of the Reconstructed
Q.E.W. 8, Hwy. #20, Stoney Creek
Traffic Circle, Co. of Wentworth
District No. 4 (Hamilton)
W.O. 72-11033 -- W.P. 10-57-02

1. INTRODUCTION:

Major reconstruction is proposed for the Q.E.W., Hwy. #20 complex at a location about 2 miles north of Stoney Creek, Ontario. The main purpose of this reconstruction is to eliminate the existing Stoney Creek Traffic Circle.

The original foundation investigation was carried out for a scheme incorporating an underpass structure at the crossing of the Q.E.W. and revised Hwy. #20 in an area presently occupied by the western leg of the Stoney Creek Traffic Circle. Two additional structures would have been required for this scheme; namely, where the C.N.R. tracks cross the Q.E.W. and revised Hwy. #20, respectively. The results, together with our recommendations for this original scheme were previously submitted (Report Nos. W.O. 70-11035, 69-F-70 and 69-F-71).

Subsequently, it was decided to abandon the railway crossings within the Stoney Creek Traffic Circle complex and construct an underpass structure at the middle of the Traffic Circle where Hwy. #20 crosses the Q.E.W.

A request to carry out a foundation investigation at the revised location of the underpass structure at the crossing of Hwy. #20 (reconstructed) and Q.E.W. was contained in a memo from Mr. G.C.E. Burkhardt, Regional Bridge Planning Engineer, dated February 14, 1972. An investigation was subsequently carried out by this Section to determine the subsoil, bedrock

and groundwater conditions at the revised structure site.

This report contains all the factual data obtained from this investigation, together with the recommendations pertaining to the foundations for the structure, as well as the stability and settlement of the approach embankments.

2. DESCRIPTION OF THE SITE AND GEOLOGY:

The proposed structure sites are located in the immediate vicinity of the Q.E.W. - Hwy. #20, Stoney Creek Traffic Circle, which is east of Hamilton, Ontario. The Q.E.W. and Hwy. #20 have two paved lanes in both travelled directions; these lanes are separated by a median of variable width. The existing highways are in cuts which extend approximately 12 to 14 feet below the surrounding terrain; the existing side slopes are standing at approximately $2\frac{1}{2}$:1 to 3:1. The terrain, which supports light vegetation such as grass and brush cover, is gently undulating in relief between elevations 264 and 267. The structure site is located in a non-built-up area. Just west of the Traffic Circle, along Hwy. #20, however, some light industry exists.

The existing Canadian National Railway (C.N.R.) track traverses across the traffic circle in a north-south direction. It is carried on an embankment 6 to 7 feet high. Single span (51.5 feet long) rigid frame steel and concrete subway structures carry the C.N.R. over the E.B. and W.B. leg of the Q.E.W. The heights of the associated approaches, in the forward direction, are approximately 18 to 19 feet.

Physiographically the area is situated in the "Iroquois Plain," specifically in the "Niagara Fruit Belt" subsection. This area was inundated in the late Pleistocene times by a body of water known as Lake Iroquois. The overburden deposits were laid down in this lake. In the "Niagara Fruit Belt" subsection the uppermost stratum is composed of a silty clay of lacustrine origin; the thickness of this cohesive subsoil generally varies between 25 and 48 feet. The silty clay is underlain by glacial

till which, in turn, is followed by red shale bedrock of the Queenston formation, Ordovician Period.

3. FIELD AND LABORATORY WORK:

A total of 8 boreholes, 6 of which were accompanied by a dynamic cone penetration test, was put down during the period of the investigation. In addition, another cone penetration test was carried out. The borings were advanced by either a conventional diamond drill rig or a Penn drill both of which were adapted for soil sampling purposes.

Samples of the overburden were recovered at required depths, in a 2" O.D. split-spoon sampler, which was hammered into the soil in accordance with the specifications for carrying out the Standard Penetration Test. The same method was used to advance the dynamic cone penetration test. Wherever possible, these samples were supplemented by obtaining 2" I.D. Shelby tube samples, which were manually pushed into the silty clay stratum. In addition, in-situ vane tests were carried out within the softer, more compressible portions of the stratum. Bedrock was proven in three of the borings, by obtaining BX size rock core samples.

The groundwater level conditions across the site, at the time of the investigation, were determined by recording the water levels in all the open boreholes.

The location and elevation of all borings were surveyed by personnel from the Central Region, Engineering Surveys Section. The borings are shown in plan on Drawing No. 72-11033 A, together with an estimated stratigraphical profile along the proposed centre line of Hwy. #20. All elevations in this report are referenced to a geodetic datum.

All samples were subjected to a careful visual examination in the field and subsequently in the laboratory. Following this examination, laboratory testing was carried out on selected representative samples to determine the following

engineering properties of the overburden.

Bulk Density

Natural Moisture Content

Grain-Size Distribution

Atterberg Limit Tests

Undrained Shear Strength

Consolidation characteristics

The results of the testing are plotted on the Record of Borelog sheets and summarized on Figures 1 to 4, inclusive, all contained in Appendix I of this report.

4. SUBSOIL AND BEDROCK CONDITIONS:

4.1) General:

The predominant stratum across the site is composed of a brown to grey, clayey silt to silty clay with a trace of sand and gravel. The overall thickness of this deposit ranges from 34 feet to 49 feet. In certain portions of the alignment, an 8 feet surficial cover of sandy silt to silt was encountered above the cohesive stratum. The cohesive deposit is underlain by a hard cohesive glacial till which is 21 to 32 feet thick. The glacial till is followed, in turn, by shale bedrock, the surface of which was encountered at a depth of between 59 and 80 feet below the original ground surface.

The boundaries of the various deposits, as determined in the boreholes, are shown on the accompanying borelog sheets. The stratigraphical profile, plotted on Drawing No. 72-11033A has been inferred from this data.

From ground surface downward, the various subsoil and bedrock types encountered are as follows:

4.2) Silty Sand to Silt:

At some isolated locations a surficial deposit composed of compact ('N' values 17 to 25 blows/ft.) silty sand to silt

was encountered. The maximum thickness of this deposit was found to be 8 feet (B.H. #2).

4.3) Clayey Silt to Silty Clay:

Directly underlying the surficial cover of topsoil, or beneath the isolated deposit of sandy silt to silt, is the predominant overburden stratum across the site, which is composed of a grey silty clay to clayey silt with a trace of sand and gravel. The overall thickness of the deposit varied from 34 feet (B.H. #1) to 49 feet (B.H. #3). At most locations the upper 6 to 14 feet of the stratum has been desiccated; this crust zone can easily be differentiated from the underlying subsoil by its characteristic brown colour. Grain-size distribution curves for samples of clayey silt to silty clay, are plotted on Figure No. 1.

The engineering properties of the stratum, as determined by field and laboratory testing, are presented in tabular form.

<u>Identity Tests</u>	<u>Upper Desiccated Zone</u>	<u>Lower Zone</u>
Bulk Density (p.c.f.) (γ)	130 - 136	130 - 134
Liquid Limit (%) (W_L)	24 - 33	26 - 38
Plastic Limit (%) (W_P)	14 - 19	14 - 21
Natural Moisture Content (%) (W)	13 - 30	13 - 35
Liquidity Index (I_L)	0 - 1.0 (0.4)	0.1 - 0.8 (0.3)
<u>Consolidation Characteristics</u>		
Initial Void Ratio (e_o)	$\left\{ \begin{array}{l} 0.66 \\ 0.09 \\ 2,400 \end{array} \right.$	$\left\{ \begin{array}{l} 0.54 - 0.68 \\ 0.115 - 0.2 \\ 1,980 - 3,700 \end{array} \right.$
Compression Index (C_c)		
Degree of Preconsolidation ($P_c' - P_o$) (p.s.f.)		

Undrained Shear Strength (C_u)
(p.s.f.)

i) Field Vanes	700 - >2,000	500 - >2,000
ii) Lab. Tests		
Sensitivity	750 - >2,000	600 - >2,000
<u>Standard Penetration Tests ('N')</u> <u>(Blows/Ft.)</u>		
	3 - 75	5 - 23

The Atterberg Limit tests, summarized above, are also plotted on a Plasticity Chart, Figure No. 2. These results indicate that the cohesive stratum is inorganic with a plasticity in the low to intermediate range. In the overall deposit the natural moisture content ranges from a few percent below to a few percent above the plastic limit.

The undrained shear strength testing carried out gave values which vary from 500 p.s.f. at a shallow depth below ground surface, increasing to greater than 2,000 p.s.f. in the lower portion of the stratum. Based on these results, it is estimated that the consistency of the stratum ranges from soft to hard. This pattern is interrupted in the extreme southern area under investigation (refer to B.H.'s #5, 5A and 6). Here the consistency varies from firm to hard.

The consolidation characteristics of the stratum were determined by carrying out four laboratory consolidation tests, the results of which are shown as Void Ratio vs. Pressure Plots on Figure No. 3. This testing indicates that the clayey silt to silty clay is preconsolidated by anywhere from 2,000 to 3,700 p.s.f. in excess of the existing overburden pressure. The values of initial Void Ratio (e_0) and the Compression Index (C_c) are within the normal range for cohesive deposits encountered in this area.

4.4) Clayey Silt with Sand and Gravel (Glacial Till):

Underlying the cohesive stratum is a reddish-brown glacial till, composed primarily of clayey silt with sand and

gravel. The total thickness of the deposit, where fully penetrated, was found to vary from 21 feet (B.H. #3) to 32 feet (B.H. #2). Occasional random granular zones are present throughout the glacial till. Grain-size distribution curves for samples of the glacial till obtained with 2" O.D. sampling equipment, are shown on Figure No. 4.

Atterberg limit tests were carried out on samples from the deposit, the results are plotted on the Plasticity Chart, Figure #4. This testing gave values for the liquid and plastic limits which range from 19 to 30 percent and 14 to 16 percent, respectively. The corresponding moisture content varies from 8 to 17 percent.

The Standard Penetration Tests, carried out in this deposit, gave 'N' values which ranged from 47 blows/ft. to 100 blows for 1 inch. Based on these results, it is estimated that the consistency of the cohesive glacial till is hard.

4.5) Shale Bedrock:

The glacial till is directly underlain by bedrock which was proven in three of the borings, by obtaining up to 10 feet of BX size rock core samples. Over the area under investigation the surface of the bedrock was found to vary between elevations 186 and 196 which corresponds to depths of 59 to 80.5' below existing ground surface.

The bedrock is composed of a reddish-brown horizontally bedded shale, which is in a sound state, as evidenced by a high percentage of rock core recovered.

5. GROUNDWATER CONDITIONS:

Groundwater level observations have been carried out, during the period of the investigation, in the open boreholes. The observations are recorded on the Record of Borehole sheets and summarized on Drawing No. 72-11033A.

The recorded observations indicate that the groundwater level in the overburden deposits generally varies between

elevations 248 and 264; i.e. some 1 to 6 feet below the existing ground surface.

6. DISCUSSION AND RECOMMENDATIONS:

6.1) General:

It is proposed to construct an 80 feet wide four-span (110' - 115' - 108' - 130') underpass structure at the crossing of the Q.E.W. and Hwy. #20; this structure will occupy the middle strip of the existing traffic circle. In the vicinity of the structure the Q.E.W. will have four 12 foot wide paved lanes in both the E.B. and W.B. directions; there will also be associated collector lane ramps. The profile grade of the Q.E.W. complex will vary from elevation 255 at the North Collector Road to 260 at the South Collector Road - i.e. ranging from 12 feet to 5 feet respectively, below the original ground surface in the area. Hwy. #20 will have two 12 foot wide paved lanes in either direction with a variable median. The profile grade of Hwy. #20, across the structure, will vary from elevation 277 to 284. At these grades the approaches will have an unsupported height of 21 feet in the longitudinal direction. In the transverse direction the fill heights will range from a maximum of 15 feet and 21 feet along the south and north approaches, respectively.

An existing east-west trending, 78 inch diameter sanitary sewer, as well as a 33 inch diameter sanitary sewer are located in the vicinity of the south abutment.

The predominant stratum across the site is a 34 to 49 feet thick clayey silt to silty clay stratum. In the northern portion of the complex the consistency of the deposit in the upper 10-12 feet ranges from soft to firm. Elsewhere the consistency of the deposit varies from firm to hard. In certain portions of the alignment, an 8 feet surficial cover of sandy silt to silt was encountered. Underlying the cohesive clayey silt to silty clay stratum is a competent 21 to 32 feet thick deposit of glacial till composed of clayey silt with sand and

gravel. The glacial till is followed by sound shale bedrock.

The presence of the relatively soft upper portion of the clayey silt stratum is of primary importance as far as foundation design is concerned, since it will be necessary to ensure that it is not "overstressed" by either the structure element or approach fill loadings. This being the case the structure elements must be supported on piled foundations. Further, it is imperative that the stability of the approaches be ensured. These will be discussed in the subsections to follow.

6.2) Approach Embankments:

6.2.1) Stability Considerations:

The critical condition for stability of an embankment on slightly over-consolidated cohesive soils, as is the case at this site, generally occurs during or immediately after construction. This being the case, a total stress stability analysis ($\phi = 0$) provides a suitable means of assessing the stability of the embankment section. In this method of analysis, stability is governed by the applied loads and by the stress-strain and undrained shear strength characteristics of the foundation and embankment soils.

Analyses have been carried out, therefore, in terms of total stresses, making use of the electronic computer, to determine the stability of the approaches.

The critical approach, as far as stability is concerned, is the north. Here the height of the embankment will be of the order of 21 feet in the longitudinal as well as transverse direction. The following assumptions were made for computational purposes.

SOIL PROPERTIES (NORTH APPROACH EMBANKMENT)

Elevation	Soil	Bulk Density (p.c.f.)	Parameters	
			Undrained Shear Strength (Cu - p.s.f.)	Effective Angle of Internal Friction (ϕ)
276 - 255	Embankment Fill	125		30°
255 - 250	Clayey Silt	120	700	
250 - 244	Clayey Silt	115	480	
Below 244	Clayey Silt	120	>2,000	

Note: 1) Tension Crack 8 feet deep.
2) Ground Water Level - Elevation 254.

In order to ensure the stability of the north approach, the following procedures should be adopted.

- 1) Fills up to 19 feet can be constructed with standard 2:1 slopes.
- ii) Fills in excess of 19 feet will require a stabilizing mid-height berm. A 21 foot high approach, for instance, will require a berm 15 feet in length. A smooth transition should be affected between the no berm and full berm sections.

The south approach is inherently more stable, since the cohesive foundation subsoil has a higher consistency in this area (minimum undrained shear strength about 1,000 p.s.f.). This approach can, therefore, be constructed to the maximum height proposed (21 feet), providing standard 2:1 slopes are employed.

6.2.2) Settlement Considerations:

The underlying more compressible zone within the clayey silt stratum will undergo settlement due to consolidation, over a period of time, under the weight of the approach embankments. In addition, recompression settlement will occur within the remaining highly overconsolidated portion of the stratum for similar reasons. Settlement computations were, therefore, carried out. The maximum settlement will occur beneath the centre line of the north approach, where fills up to 21 feet are to be placed. The computations indicate that the settlement at this location

could be of the order of 4 inches. Beneath the south approach, the fill will have a maximum height of 15 feet; here the consolidation settlement should be within 2 inches.

The total amount of the consolidation settlements predicted will take place over a period of from $2\frac{1}{2}$ to 3 years. However, about 50% of the settlement should occur within six months.

6.3) Structure Foundations:

As discussed previously one existence of a relatively compressible zone, at a shallow depth below original ground precludes one economic use of spread footings for the support of structure elements. Piled foundations should, therefore, be employed.

6.3.1) Pier Foundations:

The piers can be supported on end-bearing piles driven to practical refusal within the competent glacial till deposit. The pile driving during construction should be controlled by the Hiley Dynamic Pile Driving Formula, in accordance with current practices. For estimation purposes, the pile tips can be assumed to be located at the following elevations:

		<u>Refer to</u>
North Pier	Elev. 203	B.H. #2
Centre Pier	Elev. 205	B.H. #3
South Pier	Elev. 204 to 207	B.H. #4

Piles driven as recommended could be designed for the ultimate capacity of the pile section chosen. For example, a 12 HP 7 $\frac{1}{4}$ steel H-pile may be designed for 95 tons per pile.

No major dewatering problems are anticipated for the construction of the pile caps in view of the relatively impervious nature of the cohesive subsoil. Any minor seepage or surface runoff occurring in the excavations could be handled by using conventional techniques, such as pumping from sumps.

A minimum of 4 feet of earth cover should be provided to the underside of the pile caps in order to satisfy the frost protection requirements in the area.

6.3.2) Abutment Foundations:

The abutments may be "perched" within the approach fills and supported on end-bearing piles driven to practical refusal within the competent glacial till deposit using the techniques discussed in Subsection 6.3.1). For estimating purposes the pile tips can be assumed to be located at the following elevations:

North Abutment - 205

South Abutment - 196 (to bedrock)

The cohesive foundation subsoil will settle due to the embankment fill loading. This settlement will place a negative skin frictional load on the piles supporting the "perched" abutments. This effect will have to be taken into consideration when designing the piles, this could be accomplished by using a design load equivalent to approximately 35 percent of the ultimate capacity of the pile section chosen. For example, 12BP53 steel H-piles could be designed using an allowable load of 60 tons/pile.

No bouldery or rock fill should be placed in areas where piles are to be driven.

As mentioned elsewhere the south abutment will be in close proximity to the sanitary sewers. Where piles are to be driven adjacent to existing utilities, special precautions must be taken to ensure that no damage results. We suggest that the following procedures be adopted.

- i) When piles will be 10 feet or more from the edge of a utility, no special precautions need be taken.
- ii) All piles closer than 10 feet from a utility should be pre-bored to a depth 6 ft. below the invert of the utility. The size of the hole need only be slightly larger than the pile section.

The above procedure was followed in Contracts 63-182 and 63-24 with satisfactory results.

7. MISCELLANEOUS:

The field work was carried out during the period of March 1 to 20, 1972, under the supervision of Mr. S. A. Ahmad, Project Foundations Engineer.

The equipment was owned and operated by the F. E. Johnston Drilling Co. Ltd., Toronto.

This report was written by Mr. S. A. Ahmad and reviewed by Mr. M. Devata, Supervising Foundations Engineer.

S. A. Ahmad

S. A. Ahmad, P. Eng.



M. Devata

M. Devata, P. Eng.

SAA/ao
May 9/72

APPENDIX I

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 1

JOB 72-11033

LOCATION Co-ords. 15,713,476 N. 731,546 E.

ORIGINATED BY SA

W.P. 10-57-02

BORING DATE March 13, 1972

COMPILED BY SE

DATUM Geodetic

BOREHOLE TYPE Washboring, NX Casing

CHECKED BY

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT W_L			BULK DENSITY γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	BLOWS/FOOT	20	40	60	80	100	WATER CONTENT W_p		
254.5	Ground Level														
0.0	Brown Grey		1	SS	7	250									250.0
			2	TW	PM										in open BH
			3	TW	PM										
	Clayey silt to silty clay with trace of sand and gravel.		4	TW	PM										
			5	SS	22	240									0 12 46 42
			6	TW	PM										
	Firm to Very Stiff		7	TW	PM	230									
			8	TW	PM										
220.5															
34.0	Het. mix. of clayey silt, sand & gravel		9	SS	132	220									
			10	SS	60 4"										31 23 34 12
	Glacial Till		11	SS	67	210									
	Hard		12	SS	100 5"										8 24 58 10
	Reddish Brown		13	SS	100 1/2"	200									
195.5															
59.0	Shale Bedrock		14	BX	100%	190									
190.5	Sound Red														
64.0	End of Borehole														

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 1A

JOB 72-11033

LOCATION Co-ords. 15,713,450 N; 931,622 E.

ORIGINATED BY SA

W.P. 10-57-02

BORING DATE March 20, 1972

COMPILED BY SE

DATUM Geodetic

BOREHOLE TYPE Auger

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100	LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W W_P — W — W_L WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT					
253.7 0.0	Ground Level									
	Brown		1	SS	5	250				
	Grey		2	TW	PH					
	Clayey silt to silty clay with trace of sand and gravel.		3	TW	PH	240				
			4	SS	25					
			5	SS	19					
	Firm to Very Stiff		6	SS	26	230				
			7	SS	25					
218.7						220				
35.0	Glacial Till. Hard		8	SS	80					
36.6	End of Borehole					210				

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 2

JOB 72-11033

LOCATION Co-ords. 15,713,344 N; 931,500 E.

ORIGINATED BY SA

W.P. 10-57-02

BORING DATE March 1, 1972

COMPILED BY SR

DATUM Geodetic

BOREHOLE TYPE C.M.E. Auger and Washboring

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w		BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	BLOWS / FOOT		20 40 60 80 100	100	w_p — w — w_L	WATER CONTENT %		
266.1	Ground Level											
0.0	Sandy silt to silt		1	SS	25	260						264.1 in open BH
258.1	Compact Brown		2	SS	17							
8.0	Clayey silt to silty clay with trace of sand and gravel.		3	SS	3							
			4	TW	PH							
			5	TW	PH	250						
			6	TW	PH							
			7	TW	PH	240						
	Soft to Hard		8	TW	PH							
	Grey		9	SS	52	230						3 17 46 34
			10	SS	34							
218.1			11	SS	62	220						
48.0	Het. mix. of clayey silt, sand and gravel		12	SS	60							22 17 48 13
	Glacial Till					210						
			13	SS	135/9"	200						
	Hard					190						
			14	SS	100/1"							
	Reddish Brown					180						
185.8			15	SS	100/3"							
80.3	Shale Bedrock		16	BX	100%							
176.3	Sound Red		17	BX	100%							
89.8	End of Borehole					170						

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 3

JOB 72-11033

LOCATION Co-ords. 15,713,222 N; 931,520 E.

ORIGINATED BY SA

W.P. 10-57-02

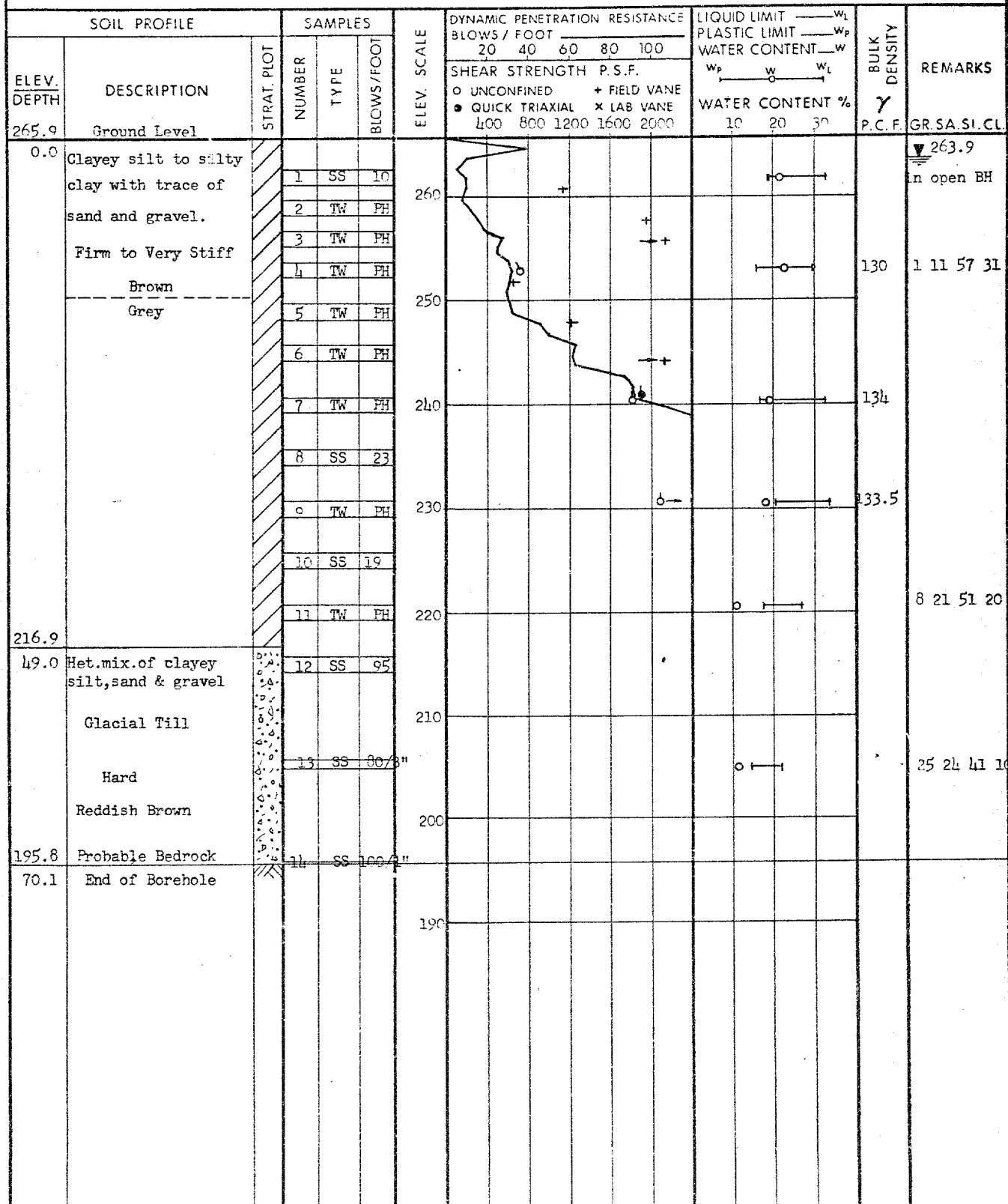
BORING DATE March 8, 1972

COMPILED BY SR

DATUM Geodetic

BOREHOLE TYPE C.M.E. Auger and Washboring

CHECKED BY



DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 4

JOB 72-11033 LOCATION Co-ords. 15,713,142 N; 931,425 E. ORIGINATED BY SA
 W.P. 10-57-02 BORING DATE March 10, 1972 COMPILED BY SP
 DATUM Geodetic BOREHOLE TYPE CME Auger and Washboring CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100	LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w w_p — w — w_L WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	BLOWS/FOOT					
265.0	Ground Level									
0.0	Clayey silt to silty clay with trace of sand and gravel.		1	SS	1					
	Firm to Hard		2	TW	PH					
	Brown		3	TW	PH					
	Grey		4	TW	PH					
			5	TW	PH					
			6	TW	PH					
			7	SS	18					
			8	TW	PH					
			9	SS	28					
			10	SS	32					
218.5			11	SS	69					
46.5	Wet mix of clayey silt, sand & gravel		12	SS	100.6"					
	Glacial Till									
	Hard		13	SS	100					
	Reddish Brown									
194.9	Probable Bedrock		14	SS	100.1"					
70.1	End of Borehole									

20
15 — 5 % STRAIN AT FAILURE
10

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 5

JOB 72-11033

LOCATION Co-ords. 15,712,986 N: 931,452 E.

ORIGINATED BY CA

W.P. 10-57-02

BORING DATE March 14, 1972

COMPILED BY SP

DATUM Geodetic

BOREHOLE TYPE CME Auger and Washboring

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT w			BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLCT	NUMBER	TYPE	BLOWS/FOOT		20	40	60	80	100	W_p	w	W_L		
267.0	Ground Level															
0.0	Clayey silt to silty clay with trace of sand and gravel. Stiff to Very Stiff		1	SS	11											
			2	TW	PH	260										
			3	TW	PH											
			4	TW	PH											
			5	TW	PH	250										
			6	TW	PH											
			7	TW	PH	240										
			8	TW	PH											
			9	TW	PH	230										
			10	SS	25											
			11	SS	75	220										
219.0			12	SS	89	210										
148.0	Glacial Till Hard Reddish Brown Het. mix. of clayey silt, sand & gravel.		13	SS	99	200										
196.7			14	SS	100	190										
70.3	Shale Bedrock		15	RC	95%											
191.7	Sound Red															
75.3	End of Borehole															

RECORD OF BOREHOLE NO 5A

FOUNDATIONS OFFICE

LOCATION_ Co-ords. 15,713,012 N; 931,371 E.

ORIGINATED BY SA

BORING DATE March 17, 1972

COMPILED BY SR

BOREHOLE TYPE Cone Penetration Test

CHECKED BY

[illegible]

15 ²⁰ 5 % STRAIN AT FAILURE
10

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 6

JOB 72-11033

LOCATION Co-ords. 15,712,946 N; 931,412 E.

ORIGINATED BY SA

W.P. 10-57-02

BORING DATE March 16, 1972

COMPILED BY SR

DATUM Geodetic

BOREHOLE TYPE CME Auger and Washboring

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT				LIQUID LIMIT —WL PLASTIC LIMIT —WP WATER CONTENT —W			BULK DENSITY γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	BLOWS/FOOT		20	40	60	80	100	WP	WL		
268.0	Ground Level														
0.0															
	Brown		1	SS	22	260									in open BH
	Grey		2	TW	PH										264.
	Clayey silt to silty		3	TW	PH	250								125	
	clay with trace of		4	TW	PH										
	sand and gravel.		5	TW	PH	240								135	Piez.2
	Stiff to Very Stiff		6	SS	31										Tip El. 246.
			7	SS	25	230									
			8	SS	43										
222.0			9	SS	53	220									Piez. Tip
46.0	Het. mix. of clayey														El. 220.
	silt, sand & gravel.														
216.4	Glacial Till.		10	SS	92										4 21 54 21
51.6	End of Borehole					210									

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 7

JOB 72-11033

LOCATION Co-ords. 15,713,590 N; 931,587 E.

ORIGINATED BY SA

W.P. 10-57-02

BORING DATE March 17, 1972

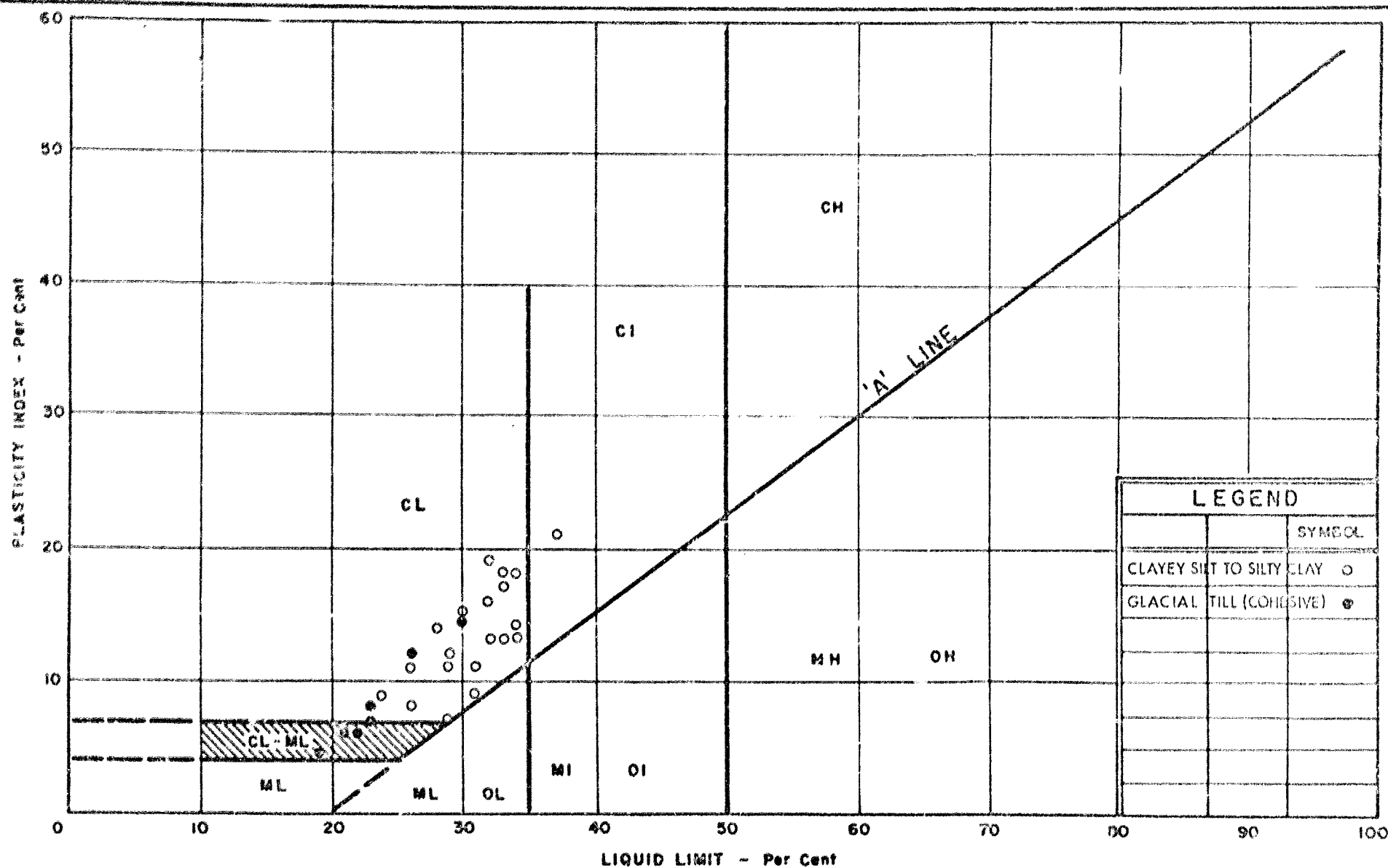
COMPILED BY SR

DATUM Geodetic

BOREHOLE TYPE Diamond Drill

CHECKED BY SP

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	BLOWS / FOOT		20	40	60	80	100	w_p	w	w_L		
253.0	Ground Level															
0.0																
	brown grey		1	SS	3	250			+							
	Clayey silt to silty clay with trace of sand and gravel.		2	TW	PM		+	+								
			3	TW	PM	240						+	+			
	Soft to Hard		4	SS	26											
			5	SS	26	230										
			6	SS	30											
218.0						220										
35.0	Het. mix. of clayey silt, sand & gravel.		7	SS	17											
211.4	Glacial Till.															
	Reddish Brown		8	SS	78											
41.6	End of Borehole					210										
						200										



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

PLASTICITY CHART

W.P. No. 10-57-02

JOB No. 72-11033

FIG. 2

VOID RATIO - PRESSURE CURVES

JOB NO. 72 - 11033

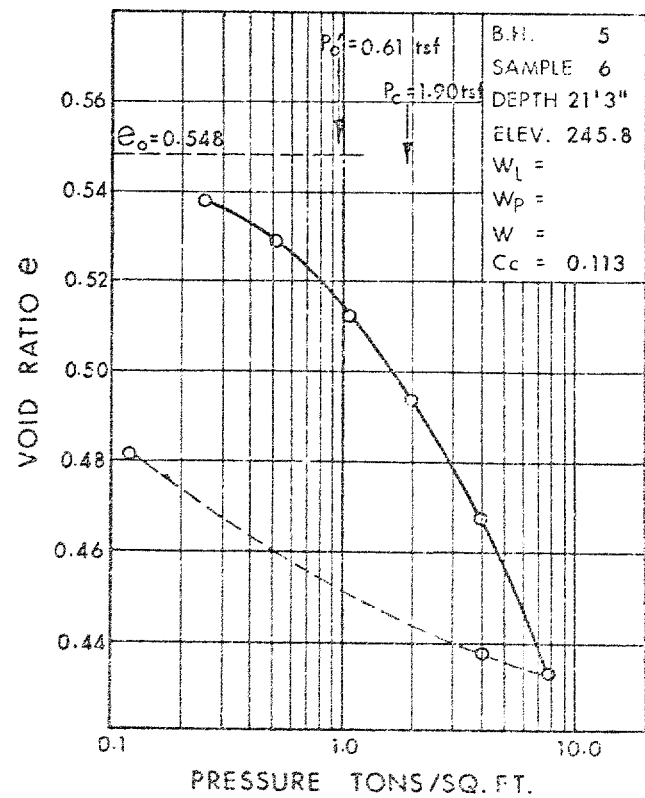
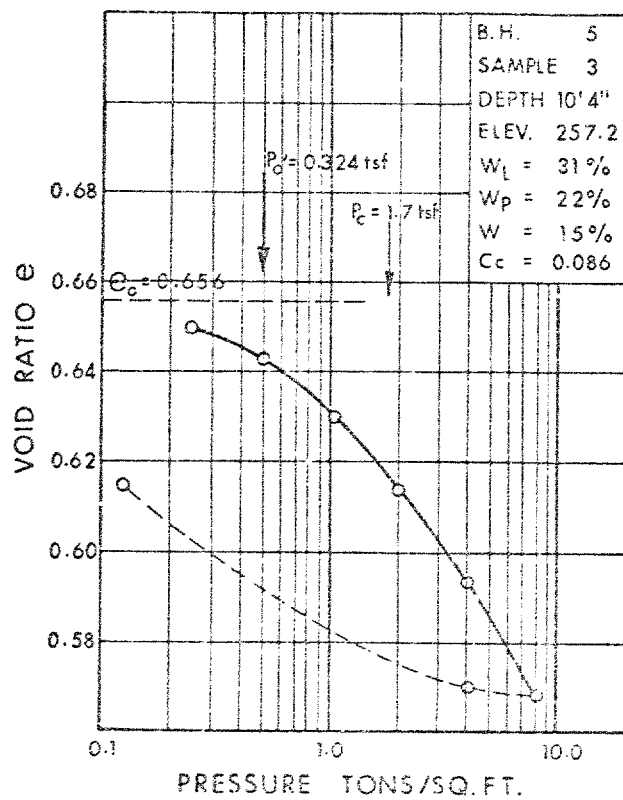
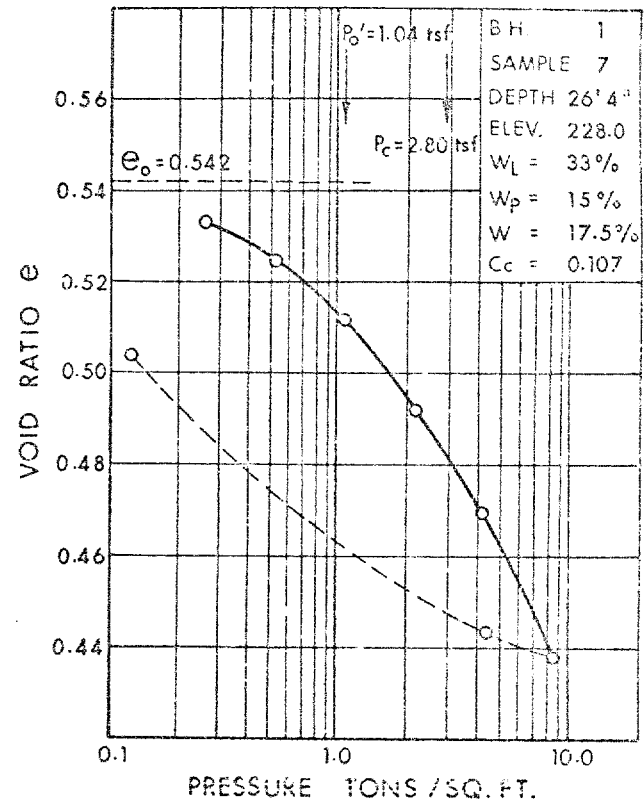
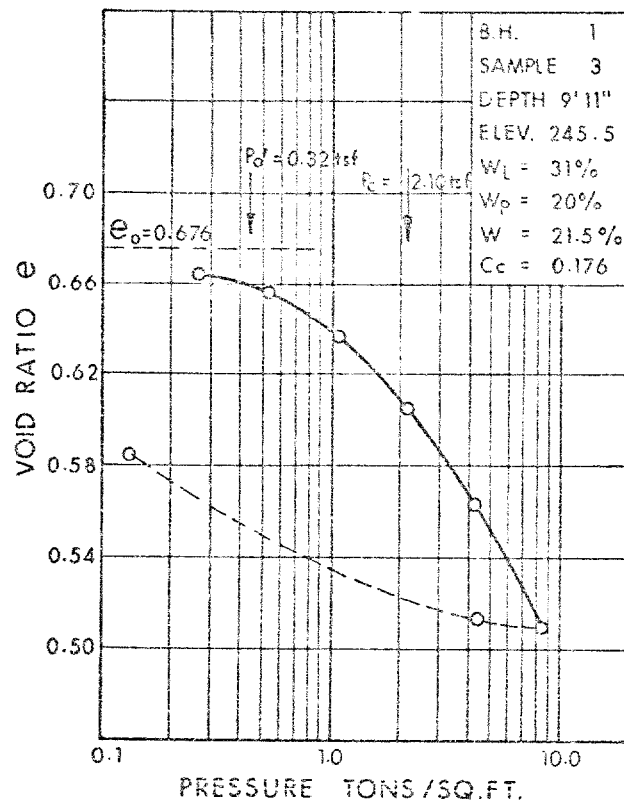
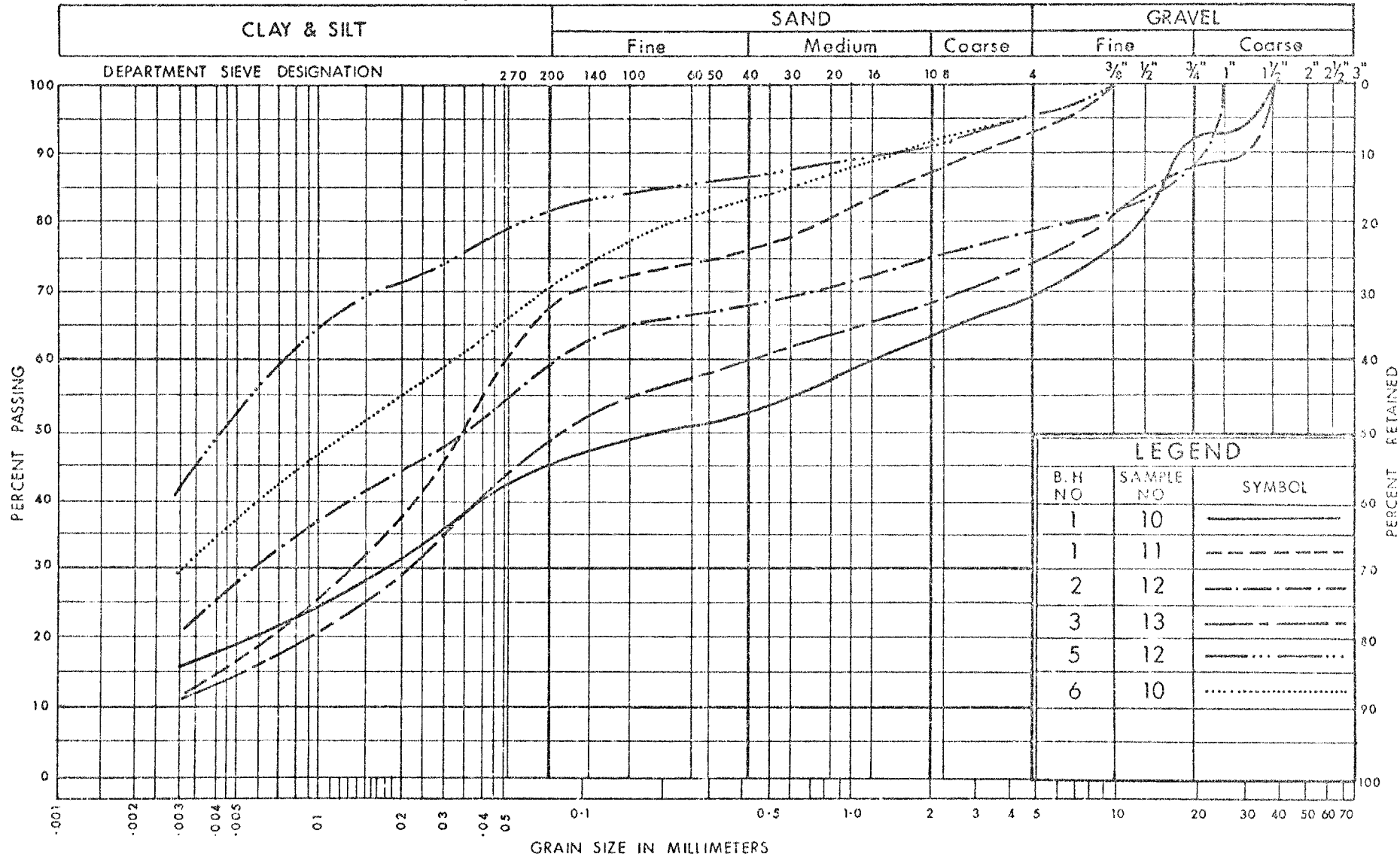


FIG. 3

UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT
OF
TRANSPORTATION AND COMMUNICATIONS



DESIGN SERVICES
BRANCH

GRAIN SIZE DISTRIBUTION GLACIAL TILL

W.P. No. 10-57-02

JOB No. 72-11033

FIG. 4

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

Qu	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V.	FIELD VANE
Qcu	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL	S	SENSITIVITY

ABBREVIATIONS 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_{soil}	UNIT WEIGHT OF SOIL (BULK DENSITY)
γ_{sub}	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
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
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
σ'_1	MAJOR EFFECTIVE STRESS
σ'_3	MINOR EFFECTIVE STRESS
ϕ	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
c_u	APPARENT COHESION
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
μ	COEFFICIENT OF FRICTION
S_i	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 (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

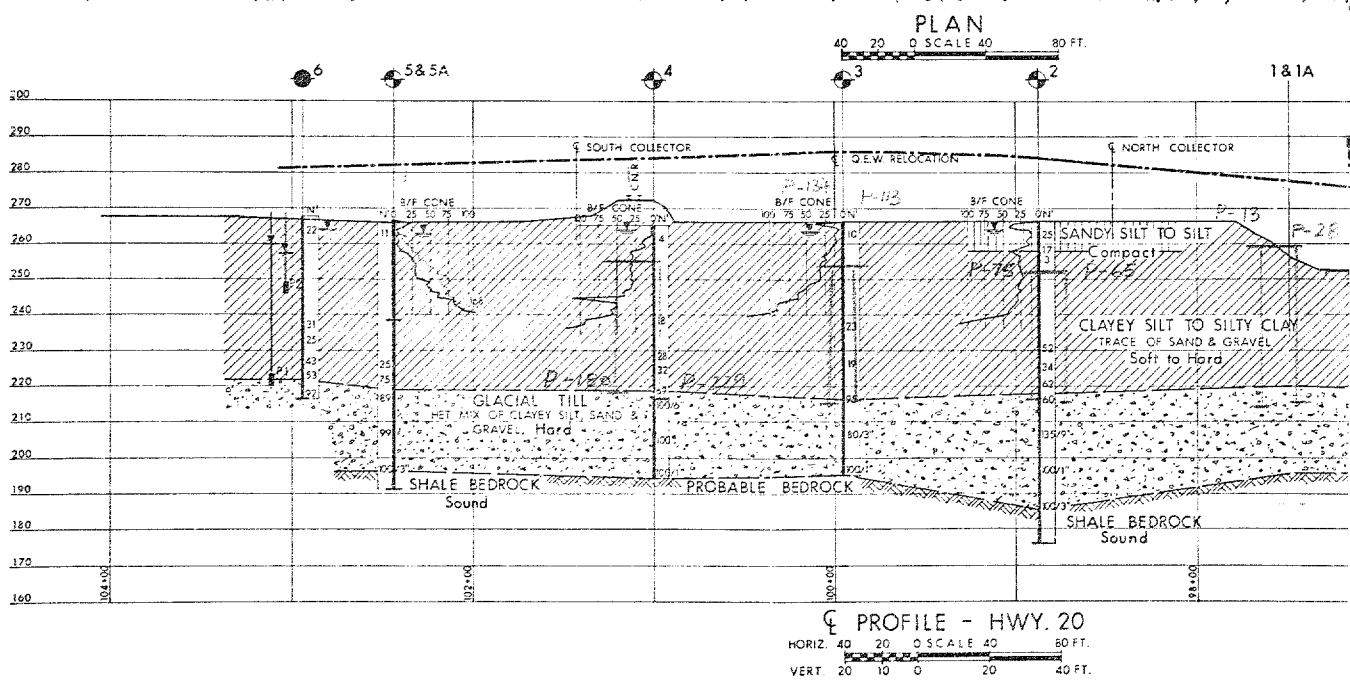
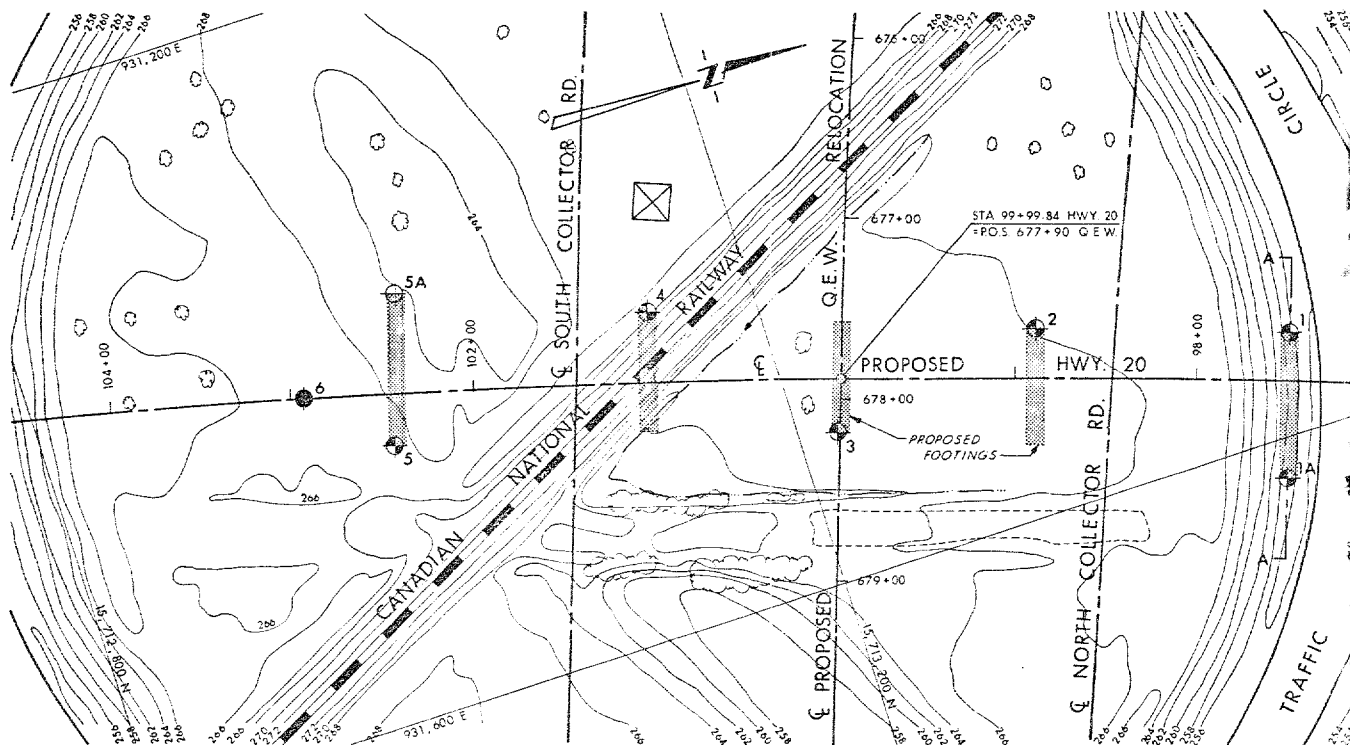
d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_0	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

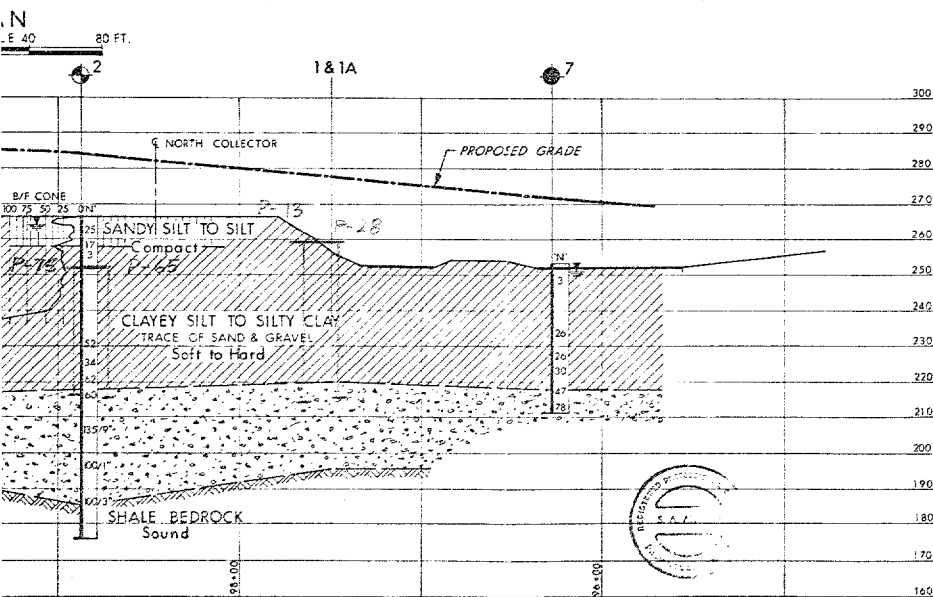
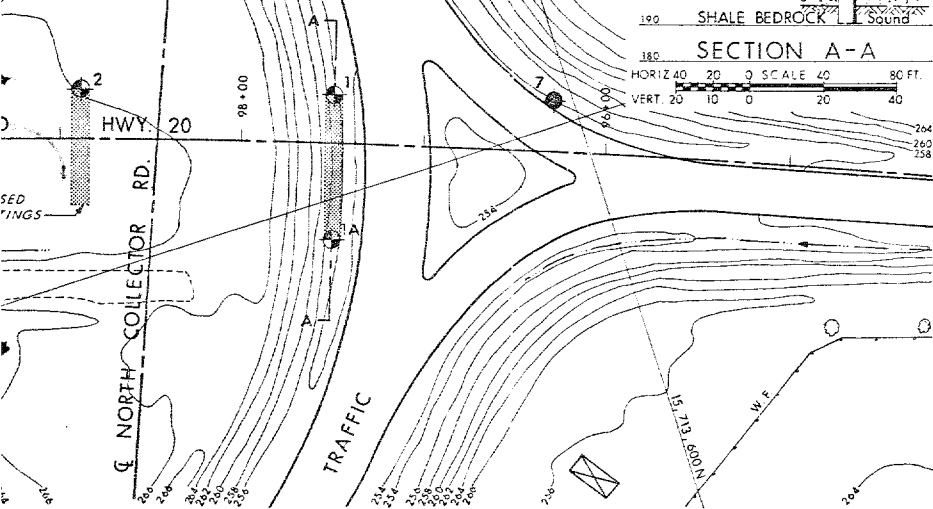
B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
k_s	MODULUS OF SUBGRADE REACTION

SLOPES

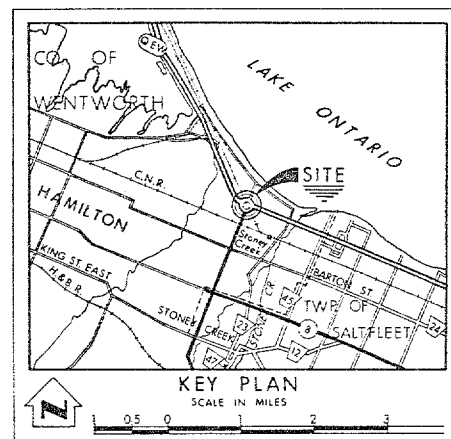
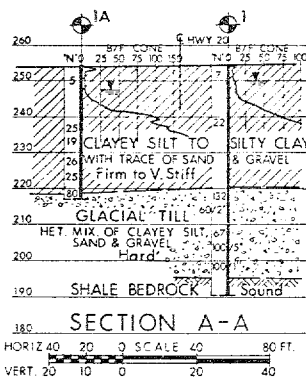
H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
β	ANGLE OF SLOPE TO HORIZONTAL



STA. 99+99.84 HWY. 20
+POS. 677+90 GEW



- HWY. 20
E 40 80 FT.
20 40 FT.



LEGEND				
	Bore Hole			
	Cone Penetration Test			
	Bore Hole & Cone Test			
	Water Levels established at time of field investigation, March 1972.			
	Piezometers			
NO	ELEVATION	CG - ORDINATES		
		NORTH	EAST	
1	254.5	15,713,476	931,546	
1A	253.7	15,713,450	931,622	
2	266.1	15,713,344	931,500	
3	265.9	15,713,222	931,520	
4	265.0	15,713,142	931,425	
5	267.0	15,712,986	931,452	
5A	264.1	15,713,012	931,371	
6	268.0	15,712,945	931,412	
7	253.0	15,713,590	931,587	

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

MINISTRY OF TRANSPORTATION & COMMUNICATIONS
DESIGN SERVICES BRANCH — FOUNDATIONS OFFICE

HIGHWAY No. 20
(AT STONEY CREEK TRAFFIC CIRCLE)

HIGHWAY NO. Q.E.W. DIST. NO. 4
CO. WENTWORTH
CITY OF HAMILTON LOT CON

BORE HOLE LOCATIONS & SOIL STRATA

SUBMD 3 A [CHECKED]	WP NO. 10 - 57 - 02	DRAWING NO.
DRAWN 5 R [CHECKED]	JOB NO. 72 - 11033	72 - 11033A
DATE APRIL 18, 1972	SITE NO.	BRIDGE DRAWING NO.
APPROVED <i>[Signature]</i>	CONT. NO.	

REF No. B-1336-9

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

MEMORANDUM

TO: Mr. G.C.E. Burkhardt, (4) FROM: Foundations Office,
Regional Structural Planning Eng., Design Services Branch,
Central Region, Central Bldg., Downsview.
90 Floral Parkway,
ATTENTION: Downsview, Ontario. DATE: May 25, 1972.

OUR FILE REF.

IN REPLY TO

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For
Proposed Underpass Structure at
The Crossing of the Reconstructed
Q.E.W. 8, Hwy. #20, Stoney Creek
Traffic Circle, Co. of Wentworth
District No. 4 (Hamilton)
W.O. 72-11033 -- W.P. 10-57-02

Please refer to the above-mentioned report dated
May 10, 1972, and remove Page 10 replacing it with the
revised copy attached to this letter.

AGS/ao
Attach.

cc: Messrs. D. W. Farren
B. R. Davis
A. Rutka
P. J. Harvey
C. R. Robertson
B. J. Giroux
T. J. Kovich
G. A. Wrong
B. A. Singh
M. M. Dillon & Co. Ltd.

A. G. Stermac
A. G. Stermac,
PRINCIPAL FOUNDATIONS ENGINEER.

Foundations Files
Documents

MR. C.S. Grebski,
Structural Design Engineer.
West Bldg. Downsview.

Foundation Office.
Central Bldg. Downsview

Date: June 8/72.

Re: Proposed Underpass Structure at the
crossing of the Reconstructed Q.E.W.
& Hwy. #2, Stoney Creek Traffic
Circle, County of Wentworth
District No 4 (Hamilton)
~~W.O. 72-4033~~ W.P. 10-57-02.

We have reviewed the ~~etc~~ ~~etc~~ preliminary
Bridge Plan Drawing No 36-144-P3 ~~and submit the following~~
~~etc~~ for the above-mentioned structure and submit the
following comments:-

- i) ~~Revised~~ ~~for~~ Pile tip elevation for South pier
will be approximately 204 to 207 as per
page 11 of our Report W.O. 72-4033.
- ii) Design load for the 12HP53 steel H-piles
at the abutment locations should be reduced
by 15% due to the anticipated negative
skin friction forces. i.e. design load should
be 60 Tons/pile.

C.C. G.

Shahem Ahmad
PROJECT. FOUNDATION ENGINEER
for M. G. Grebski
SUPERVISING Foundation Engineer

SUMMARY OF PILE DRIVING RECORDS

W.O. 72-11033 W.P. 10-57-02 CONT. 74-110 DIST. 4
 SITE HWY # 20 UNDERPASS AT STONEY CREEK & Q.E.W.
 DATE DRIVEN JAN 22 - MAR 6/75 WEIGHT OF ANVIL 1249 LB
 HAMMER TYPE VULCAN 30C WEIGHT 8000 LB ENERGY 24450 Ft/Lb

LOCATION OF PILES	PILE				ESTIMATED TIP EL. (ft.)	DIFFERENCE Longer(+) Shorter(-) Than Estimated (ft.)	REMARKS
	TYPE	NO.	LENGTH (ft.)	TIP EL. (ft.)			
NORTH ABUTMENT	HP 12x74	9	45.7	214.6	205.0	- 9.6	
		13	46.1	214.2	- - -	- 9.2	
		28	45.3	215.1	- - -	- 10.1	
SOUTH ABUTMENT	HP 12x74	250			196.0		INCOMPLETE INFORMATION
		269			196.0		
		271			196.0		
NORTH PIER	HP 12x53	65	36.7	215.05	203.0	- 12.05	
		75	35.6	216.15	- - -	- 13.15	
		92	36.3	215.95	- - -	- 12.95	
CENTRE PIER	HP 12x53	113	37.8	216.55	205.0	- 11.55	
		134	37.8	215.95	- - -	- 10.95	
		161	38.6	215.75	- - -	- 10.75	
SOUTH PIER	HP 12x53	180	37.4	217.85	204.0 - 207.0	- 10.85	
		197	37.0	217.75	- - -	- 10.75	
		229	39.0	216.35	- - -	- 9.35	
		238	37.7	217.65	- - -	- 10.65	

Mr. C.R. Robertson,
District Engineer,
District 4, Hamilton.

Mr. D. Waller,
Construction Engineer.

Mr. W.T. Hashizume,
Reg. Construction Engineer,
Structures.

September 15th, 1975.




RE: CONTRACT NO. 74-110
W.P. 10-57-02
Hwy. #20 U'pass at Stoney Creek,
Site #36-144
Hwy. Q.E.W. District 4

This will confirm our telephone conversation
with you (11 Sept. 1975) regarding the berms at the
abutments of the above bridge.

It will be permissible to lower the berms at
both abutments by one (1) foot.

WTH/jm


W.T. Hashizume,
Reg. Construction Engineer,
Structures.

c.c. B. Davis
M. Devata ✓ (Soil Mechanics Section)

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

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

September 20, 1972.

Hwy. #20 Underpass at Stoney Creek
W.P. 10-57-02 Site 36-144
District #4 (Hamilton)

72-11033

~~69-1-71~~

We have reviewed the final bridge drawings for the above-mentioned structure and submit the following comment.

The pile lengths in the table of 'Pile Data' should be amended as follows:

<u>Location</u>	<u>Pile Length</u>
North Abutment	56 feet
North Pier	50 feet
Centre Pier	50 feet
South Pier	50 feet
South Abutment	72 feet

M. Devata

M. Devata,
SUPERVISING FOUNDATIONS ENGINEER.

MD/ao

cc: Foundations Files ✓
Documents

General Layout.
Working Pt. 4
Footing 11

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRE No. 304-19

DIST. 4 REGION Central

W.P. No. 10-51-02

CONT. No. 74-10

W. O. No. _____

STR. SITE No. 36-144

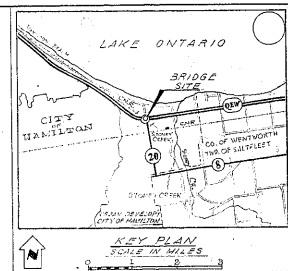
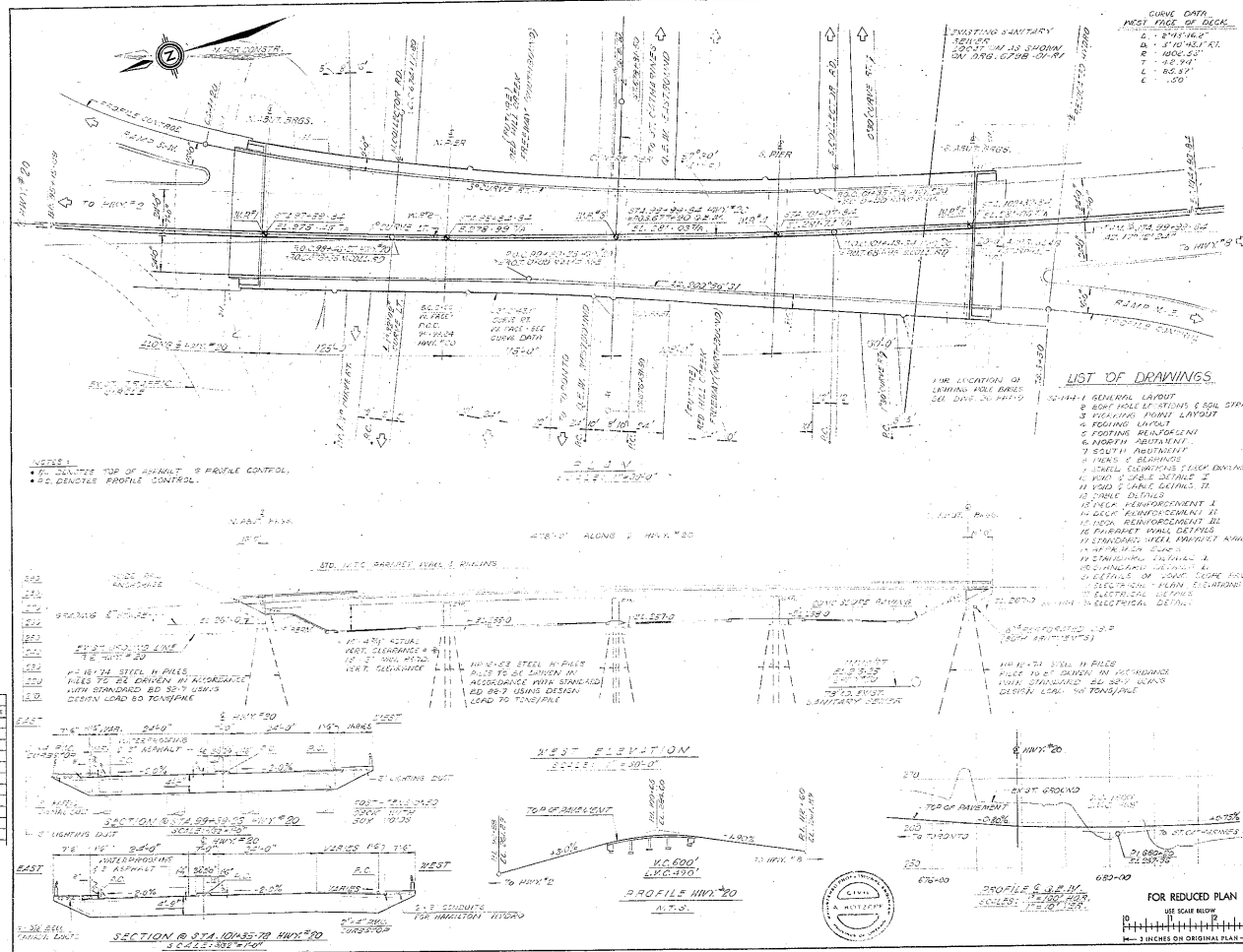
HWY. No. 20

LOCATION See Highway Story
Creek Traffic Circle

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT 3

REMARKS: Documents to be unfolded

6-1958 SEPT. 1978



NOTES:

GRADE OF CONCRETE
DECK, SIDEWALKS & PARAPET WALLS - 3000 P.S.I.
INTER DRIVING - 3000 P.S.I.
ALUMINUM - 3000 P.S.I.
AND/OR AS NOTED ON DRAWINGS.

STEEL SHALL BE AINTE STEEL

1. ALL STEEL SHALL BE AINTE STEEL
2. ALL STEEL SHALL BE AINTE STEEL
3. ALL STEEL SHALL BE AINTE STEEL
4. ALL STEEL SHALL BE AINTE STEEL
5. ALL STEEL SHALL BE AINTE STEEL
6. ALL STEEL SHALL BE AINTE STEEL
7. ALL STEEL SHALL BE AINTE STEEL
8. ALL STEEL SHALL BE AINTE STEEL
9. ALL STEEL SHALL BE AINTE STEEL
10. ALL STEEL SHALL BE AINTE STEEL
11. ALL STEEL SHALL BE AINTE STEEL
12. ALL STEEL SHALL BE AINTE STEEL
13. ALL STEEL SHALL BE AINTE STEEL
14. ALL STEEL SHALL BE AINTE STEEL
15. ALL STEEL SHALL BE AINTE STEEL
16. ALL STEEL SHALL BE AINTE STEEL
17. ALL STEEL SHALL BE AINTE STEEL
18. ALL STEEL SHALL BE AINTE STEEL
19. ALL STEEL SHALL BE AINTE STEEL
20. ALL STEEL SHALL BE AINTE STEEL

LIST OF DRAWINGS

1. GENERAL LAYOUT
2. BRIDGE DECK LAYOUT
3. BRIDGE PIER LAYOUT
4. BRIDGE ABUTMENT LAYOUT
5. BRIDGE PARAPET LAYOUT
6. BRIDGE SIDEWALK LAYOUT
7. BRIDGE SIDEWALK DETAIL
8. BRIDGE SIDEWALK DETAIL
9. BRIDGE SIDEWALK DETAIL
10. BRIDGE SIDEWALK DETAIL
11. BRIDGE SIDEWALK DETAIL
12. BRIDGE SIDEWALK DETAIL
13. BRIDGE SIDEWALK DETAIL
14. BRIDGE SIDEWALK DETAIL
15. BRIDGE SIDEWALK DETAIL
16. BRIDGE SIDEWALK DETAIL
17. BRIDGE SIDEWALK DETAIL
18. BRIDGE SIDEWALK DETAIL
19. BRIDGE SIDEWALK DETAIL
20. BRIDGE SIDEWALK DETAIL

DATE: 11/11/77
DRAWN BY: [Signature]
CHECKED BY: [Signature]
APPROVED BY: [Signature]

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS
ONTARIO

72-11033

HWY #20 UNDERPASS
OVER SAN JOSE CREEK

KING'S HIGHWAY No. 20 E.W. CITY OF HAMILTON
CO. OF HAMILTON

GENERAL LAYOUT

APPROVED: [Signature] 10-14-77
DESIGN: A.K. CHECK: V.F.B.
DRAWING: J.S. CHECK: A.K.
DATE: 11/11/77

