

#69-F-70

W.P.'S 10-57-03 AND 10-57-04

Q.E.W. AND HW.Y. #20

STONEY CREEK

SUBWAY STRUCTURES

MEMORANDUM

To: Mr. B. R. Davis,
Bridge Engineer,
Bridge Office,
Admin. Bldg.

FROM: Foundation Section,
Materials & Testing Office,
Room 107, Lab. Bldg.

ATTENTION: Mr. S. McCombie

DATE: November 12, 1969

OUR FILE REF.

IN REPLY TO

SUBJECT:

FOUNDATION INVESTIGATION REPORT

For

Proposed Subway Structures at the
Crossings of the Reconstructed Q.E.W.
and Revised Hwy. #20 - Stoney Creek
Traffic Circle - County of Wentworth
District No. 4 (Hamilton)
W.J.69-F-70, W.P.'s 10-57-03 & 10-57-04

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

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/KdeF
Attach.

cc: Messrs. B. R. Davis (2)
H. A. Tregaskes
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Foundations Files
Gen. Files

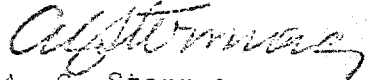

A. G. Stermac
PRINCIPAL FOUNDATION ENGINEER

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FOUNDATION INVESTIGATION REPORT

For

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Crossings of the Reconstructed Q.E.W.
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W.J.69-F-70, W.P.'s 10-57-03 & 10-57-04

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. In conjunction with this project the Foundation Section was requested to carry out an investigation at the locations of two proposed C.N.R. Subway structures, which will cross the Q.E.W. and Hwy.#20, respectively. The request for this foundation investigation was contained in a memo from Mr.W.S.Melinyshyn, Regional Bridge Location Engineer, Central Region, dated August 12, 1969. An investigation was subsequently carried out by this Section to determine the subsoil, bedrock and groundwater conditions at the respective structure sites.

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

The foundation investigation for the proposed underpass structure, at the crossing of reconstructed Q.E.W. and Hwy.#20, will be discussed in a separate report (W.J.#69-F-71, W.P.#10-57-02).

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. Q.E.W. and Hwy.#20 have two paved traffic 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 about elevations 264 and 267. The sites are 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. legs of the Q.E.W. The heights of the associated approaches, in the forward direction, are approximately 18 to 19 feet.

Physiographically the sites are situated in the "Iroquois Plain", specifically in the "Niagara Fruit Belt" sub-section. 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" sub-section the upper most stratum is composed of a

2. DESCRIPTION OF THE SITE AND GEOLOGY: (cont'd.) ...

silty clay of lacustrine origin; the thickness of this cohesive subsoil generally varies between 25 and 45 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 twenty-six boreholes, all of which were accompanied by a dynamic cone penetration test, were put down at the two proposed structure sites. Nineteen of the boreholes were put down in the vicinity of the C.N.R. - Q.E.W. crossing, while the remaining 7 were located in the vicinity of the C.N.R. - Hwy.#20, crossing. The borings were advanced by either a conventional diamond drill rig or a pendrill(power auger machine), both of which were adapted for soil sampling purposes.

Samples of the fill and 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 tests. Wherever possible these samples were supplemented by obtaining 2" I.D. shelby tubes, 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 this stratum. Bedrock was proven in eight of the borings, by obtaining BXL size rock core samples.

3. FIELD AND LABORATORY WORK: (cont'd.) ...

The ground water 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 the following drawings.

- i) C.N.R. crossing of Reconstructed Q.E.W. -
Drawing No. 69-F-70A-1
- ii) C.N.R. crossing of Reconstructed Hwy.#20 -
Drawing No. 69-F-70B.

An estimated stratigraphical profile along the proposed centre line of the C.N.R. is shown on the aforementioned drawings. Additional stratigraphic sections, in the vicinity of the proposed C.N.R. - Q.E.W. structure, are shown on Drawing No.69-F-70A-2. All elevations given 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 Densities

Natural Moisture Contents

Grain-size Distributions

Atterberg Limits

Undrained Shear Strengths

Consolidation Characteristics

3. FIELD AND LABORATORY WORK: (cont'd.) ...

The results of this testing are plotted on the Record of Borelog sheets and summarized on Figures 3 to 8 , inclusive, all contained in Appendix I of this report.

4. SUBSOIL CONDITIONS:

4. 1) General:

The predominant stratum across the area is composed of a clayey silt to silty clay with a trace of sand and gravel, the overall thickness of which varies from 17 to 51 feet. The upper 3 to 10 feet of this stratum has been desiccated forming a hard crust. The cohesive deposit is underlain by a hard cohesive glacial till which is between 12 and 24 feet thick. The glacial till is followed by sound shale bedrock.

In the northern portion of the area under investigation the clayey silt stratum is overlaid by as much as 9.5 feet of silty sand. At a few random locations, in the vicinity of the existing Traffic Circle, up to 6.5 feet of fill, composed primarily of clayey silt with some related organic matter, overlies the cohesive stratum.

The boundaries of the various deposits, as determined in the boreholes, are shown on the accompanying borelog sheets. The stratigraphical sections, plotted on Drawings No. 69-F-70A1, -2 and 69-F-70B, have been inferred from this data.

From ground surface downwards, the various soil types encountered are as follows:-

4. SUBSOIL CONDITIONS: (cont'd.) ...

4. 2) Surficial Deposits:

4. 2.1) Fill Material

Some fill was placed at random location, in conjunction with the original construction of the Stoney Creek Traffic Circle. The depth of this material where encountered, varies from 3 to 6.5 feet. The composition of this fill is variable. At some locations it is composed of a stiff to very stiff ('N' values 9 to 39 blows/ft.) clayey silt to silty clay with some sand and gravel, and a trace of organic matter. Occasional partings and seams of silt and sand were encountered throughout this cohesive material. Elsewhere, however, it is granular in nature being composed of a loose to dense ('N' values 7 to 40 blows/ft.) silty sand, with a trace of clay and gravel.

4.2.2.) Silty Sand:

In the northern portion of the area under investigation (B.H.'s #39, 40, and 40A) the surficial deposit is composed of a loose to dense ('N' values 8 to 39 blows/ft.) silty sand, with a trace of gravel and organic matter. The thickness of this deposit varies between 6 and 9.5 feet. Occasional seams and partings of clayey silt, up to 1/4 inch thick were encountered randomly throughout the granular subsoil. Grain-size distribution curves for samples from this deposit, as well as the granular fill discussed in sub-section 4.2.1.), are plotted on Figure # 3.

4. SUBSOIL CONDITIONS: (cont'd.) ...

4.3) Clayey Silt to Silty Clay

Directly underlying the surficial cover is the predominant overburden stratum across the site, composed of a grey silty clay to clayey silt with a trace of sand and gravel. The overall thickness of this stratum varies from 17 feet at B.H.#14OA to 31 feet at B.H.#30C, being typically in the 30 to 40 foot range. The upper 3 to 10 feet of the stratum has been desiccated; this crust zone can easily be differentiated from the underlying subsoil by its characteristic brown colour. Numerous random seams and layers of sandy silt, varying anywhere from a fraction of an inch to up to $2\frac{1}{2}$ feet in thickness (refer to B.H.#30C), are present throughout the deposit. These granular layers were not found to be continuous across the area; it is inferred, therefore, that they are localized pockets. Grain-size distribution curves for samples of the clayey silt to silty clay, as well as from the granular layers, are plotted on Figure # 4.

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

4. SUBSOIL AND BEDROCK CONDITIONS: (cont'd.) ...
 4.3) Clayey Silt to Silty Clay: (cont'd.) ...

		Overall Stratum (Excluding More Compressible Zone)	Relatively More Compressible Zone Encountered North of Sta. 13 + 50 (Between Elev.'s 253 and 235)
<u>Identity Tests</u>		<u>Range(Average)</u>	<u>Range Average)</u>
Bulk Density	(ρ)	120 - 137 (133)	116 - 132 (124)
(p.c.f.)			
Liquid Limit	(w_L)	25 - 44 (31)	26 - 39 (33)
%			
Plastic Limit	(w_p)	14 - 23 (17)	12 - 21 (18)
%			
Natural Moisture Content	(W)	12 - 28 (18)	21 - 37 (26)
Liquidity Index	(I_L)	-0.2 -0.4 (0)	0.3- 0.9 (0.7)
<u>Consolidation Characteristics</u>			
Initial Void Ratio	(e_o)	-	(0.60 - 0.97
Compression Index	(C_c)	-	(0.13 - 0.37
Degree of Preconsolidation		-	(1,500- 3,000
(p.s.f.)	($P_c - p_o'$)		(
<u>Undrained Shear Strength (C_u)</u>			
(p.s.f.)			
1) Field Vanes		1,300 -> 2,000	450 - 1,500
2) Lab. Tests		1,000 -> 2,000	450 - 1,000
<u>Standard Penetration Tests 'N'</u>			
Blows /ft.		9 - 70 (35)	Up to 6

..... 9

4. SUBSOIL AND BEDROCK CONDITIONS: (cont'd.) ...
4.3) Clayey Silt to Silty Clay: (cont'd.) ...

The resumé of engineering properties, presented in the table, conclusively indicates the presence of a relatively soft, compressible zone within the cohesive deposit, north of Station 13 + 50 (C.N.R. chainage). The significance of this compressible zone, with regard to foundation design, will be discussed throughout this report.

The Atterberg Limit tests, summarized above are also plotted on the Plasticity Chart, Figure # 6 . 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. In the compressible zone, however, the natural moisture content exceeds the liquid limit by a considerable degree, as represented by liquidity indices between 0.3 and 0.9.

The undrained shear strength testing carried out gave values which vary from 1,000 p.s.f. to greater than 2,000 p.s.f., throughout the overall deposit. In the relatively more compressible zone, encountered north of Station 13 + 50, however, the values range from 450 p.s.f., increasing with depth, to 1,500 p.s.f. Based on these results it is estimated that the consistency of the majority of the stratum varies from stiff to hard, being typically in the very stiff range. The consistency of the compressible zone varies from soft to stiff. The aforementioned patterns were corroborated by the standard penetration testing carried out within the cohesive deposit.

4. SUBSOIL AND BEDROCK CONDITIONS: (cont'd.) ...
4.3) Clayey Silt to Silty Clay: (cont'd.) ...

The sensitivity of the stratum, as determined by the field and laboratory testing, was found to vary between 2 and 10, being on the average about 4.

The consolidation characteristics of the stratum were determined by carrying out six laboratory consolidation tests, the results of which are shown as Void Ratio vs. Pressure plots, on Figures # 7 and 8. The results of this testing indicated that the clayey silt to silty clay subsoil, located within the relatively compressible zone, is preconsolidated by about 1,500 to 3,000 p.s.f. in excess of the existing overburden pressure. The remainder of the stratum is preconsolidated by a magnitude which would be in excess of the values aforementioned. The values for the 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 penetrated, was found to vary from 12 feet (B.H.#29) to 24 feet (B.H.#40A). Occasional random granular zones are present throughout the glacial till; in these areas the subsoil is composed of silt and sand binding gravel. Grain-size distribution curves, for samples of the glacial till, obtained with 2" O.D. sampling equipment, are shown on Figure # 5.

4. SUBSOIL AND BEDROCK CONDITIONS: (cont'd.) ...
4.4) Clayey Silt with Sand and Gravel --(Glacial Till)(cont'd.)

Atterberg limit tests were carried out on samples from the deposit, the results are plotted on the plasticity Chart, Figure # 6 . This testing gave values for the liquid and plastic limits which range from 18 to 27 and 14 to 17, respectively. The corresponding natural moisture content is consistently 4 to 7 percent below the plastic limit.

The Standard Penetration Tests, carried out in this deposit, gave 'N' values which range from 50 blows/ft. to 100 blows/2 inches, being typically greater than 100 blows/ft. Based on these results, it is estimated that the consistency of the basically cohesive glacial till is hard.

4.5) Shale Bedrock

The glacial till is directly underlain by bedrock, which was proven in eight of the borings, by obtaining from 4 to 9 feet of BXL size rock core samples. Over the area under investigation the surface of the bedrock was found to vary between elevations 193 and 208.

The bedrock is composed of a reddish brown horizontally bedded shale. In general, bedrock is sound throughout; however, some signs of fracturing and weathering were observed in the upper 1 to 2 feet, at some of the boring locations.

5. GROUNDWATER CONDITIONS:

Groundwater level observations have been carried out, during the period of the investigation, in the open holes. The observations are recorded on the borelog sheets and summarized on Drawings No. 69-F-70A-1, - 2, and 69-F-70B.

The recorded observations indicate that the groundwater level in the overburden deposits varies between elevations 247 and 258, being generally at a higher elevation in the northern areas.

At B.H.'s #15, 30A and 31 the groundwater level was encountered at elevations below 236. These borings were carried out with the Pendrill(power auger) i.e. the borings were carried out in a dry state. The subsoil is relatively impervious. This being the case it is inferred that the water levels, recorded in these open holes, did not have sufficient time, during the period of the investigation, to reach their true equilibrium level. This was confirmed by a more recent set of readings taken on November 5, 1969, which indicated that the water levels were, in fact, rising in the open holes at these locations.

6. DISCUSSION AND RECOMMENDATIONS:

It is proposed to realign the C.N.R. in the vicinity of the Stoney Creek Traffic Circle. This will necessitate the construction of separate subway structures at the crossing of the reconstructed Q.E.W. and Hwy.#20, respectively. The proposed details for this scheme are given on Drawing No.6465-01-R1, dated July, 1969; this drawing was prepared by M.M.Dillon Limited, Consulting Engineers.

6. DISCUSSION AND RECOMMENDATIONS:(cont'd.) ...

The recommendations pertaining to structure foundations, as well as the stability and settlement of the associated approaches, at the respective structure sites, will be discussed separately in the following sub-sections.

6.1) C.N.R. crossing at Reconstructed Hwy.#20.

6.2) C.N.R. crossing at Reconstructed Q.E.W.

6.1) C.N.R. Subway Structure at Revised Hwy.#20:

6.1.1) General:

In the vicinity of the structure Hwy.#20 will have two 12 foot-wide paved lanes in both the N.B. and S.B. directions separated by a median. The profile grade of the highway will vary from elevation 255 to 256 - i.e. 3 to 9 below the surrounding ground surface.

The proposed profile grade of the C.N.R., in the vicinity of the structure, varies from elevations 276 to 279, which is some 12 to 15 feet above existing ground surface.

The structure scheme proposed calls for three spans (40' - 90' - 40' - approx.), incorporating two piers and 'perched' abutments, within the approach fills. The bridge deck will be about 31 feet wide, this will allow for two sets of tracks.

The predominant deposit across the site is a 17 to 51 foot thick clayey silt to silty clay stratum, which is underlain by a hard glacial till deposit, followed by shale bedrock. The majority of the clayey silt stratum is competent. Between elevations 253 and 235, however, a relatively compressible zone was encountered within this subsoil.

6. DISCUSSION AND RECOMMENDATIONS:(cont'd.) ...

6.1.1) General: (cont'd.) ...

The presence of this zone 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 surcharge 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 sub-sections to follow.

6.1.2) Approach Embankments:

1) Stability Considerations:

The critical condition for stability of an embankment on slightly overconsolidated cohesive subsoils, as in the case at this site, generally occurs during or immediately after construction. This being the case, a total stress analysis ($\phi = 0$) provides a suitable means of assessing the stability of the embankment sections. 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, both manually and by the use of the electronic computer, to determine the stability of the approaches. The geometric sections at the approaches, and the soil properties for the fill and subsoil, assumed for computation purposes, are presented on Figure 1, in the Appendix I of this report. The results of the analyses, presented on the aforementioned figure, are summarized in the following paragraphs.

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.1.2) Approach Embankments: (cont'd.) ...

i) Stability Considerations: (cont'd.) ...

The critical approach, as far as stability is concerned, is the north. Here the height of the embankment will be 22 and 19 feet in the longitudinal and transverse directions, respectively. The compressible zone, underlying this approach at a shallow depth, has an average undrained shear strength which varies from 650 p.s.f., increasing with depth to 800 p.s.f. The computations carried out indicate that this approach will be stable with standard 2:1 slopes. The minimum factor of safety will be of the order of 1.4 (in the longitudinal direction).

The south approach is inherently more stable since

i) the undrained shear strength of the compressible zone is consistently higher than the values quoted above (minimum value $c_u = 850$ p.s.f.) and

ii) the height of the approach, in the longitudinal direction, is comparable to that along the north approach.

ii) 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 results of which are summarized on Figure #1.

6. DISCUSSION AND RECOMMENDATIONS:(cont'd.) ...

6.1.2) Approach Embankments:(cont'd.) ...

ii) Settlement Considerations:(cont'd.) ...

In the immediate vicinity of the structure the maximum consolidation settlement will occur under the centre line of the north approach, where the height of fill will be of the order of 19 feet above ground surface. The computations indicate that the settlement, at this location, could be of the order of $4\frac{1}{2}$ to 5 inches. Beneath the south approach, where the height of fill is approximately 15 feet, the consolidation settlement should be within 3 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 consolidation settlement should occur within 6 months (refer to graph on Figure #1).

6.1.3) Structure Foundations:

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

i) Pier Foundations:

The piers can be supported on end-bearing piles driven to practical refusal into the competent glacial till deposit. For estimating purposes, it can be assumed that the pile tips will be located between elevations 200 and 205. The piles can be designed for the maximum allowable load for the respective pile section selected (e.g. 12BP74 steel H.-piles may be designed for 90 tons/pile.)

6. DISCUSSION AND RECOMMENDATIONS:(cont'd.) ...

6.1.3)Structure Foundations: (cont'd.) ...

i) Pier Foundations: (cont'd.) ...

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

i.i) Abutment Foundations:

The abutments may be 'perched' within the approach fills; they can be supported on end-bearing piles driven to practical refusal within the glacial till deposit, as discussed in sub-section 6.1.3.i). It is estimated that the pile tips will be located at about elevation 205.

No bouldery or rock fill should be used in areas in which piles are to be driven.

6.2) C.N.R. SUBWAY . STRUCTURE CROSSING AT RECONSTRUCTED Q.E.W.

6.2.1) General

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 ramps. In addition there will be North and South Collector Roads in this complex. The profile grade of the Q.E.W. will be between elevations 255 and 256,- i.e. be of the order of 10 to 12 feet below the original ground surface.

The proposed profile grade of the C.N.R., in the vicinity of the structure, varies from elevation 282, along the north approach, to 277 along the south, which is some 16 to 11 feet respectively, above original ground surface.

The structure scheme proposed, calls for six spans (90' - 90' - 100' - 105' - 105' - 95' - approx.), incorporating five piers and abutments. It is understood that the abutments may be 'perched' within the approach fills, or alternatively, be of the closed - type. Because of the space restrictions between the alignment of the C.N.R. and the Ramp E-N, a retaining wall will be required. This retaining wall will be approximately 175 feet along, and run in a northerly direction starting from the end of the north abutment.

An existing east-west trending, 78" diameter sanitary sewer crosses the proposed location of the south end-pier (Station 9+50). In addition a buried hydro cable crosses the northern extremity of the proposed retaining wall. (Sta. 16+00).

6.2 C.N.R. SUBWAY STRUCTURE CROSSING AT RECONSTRUCTED Q.E.W.

6.2.1) General - (Cont'd.)

The subsoil sequence encountered at this site is similar to that discussed in Sub-Section 6.1.1) (C.N.R. - Hwy. #20 Structure Site). The more compressible zone, encountered within the clayey silt stratum, however, is only present north of Station 13+50. South of this station the upper cohesive stratum is competent throughout (minimum undrained shear strength $c_u = 1,000$ p.s.f.).

It will be imperative, at this site, to adopt measures to ensure the stability of the approach embankments. This will be particularly true along the north approach, which will be underlain at a shallow depth, by the compressible zone within the clayey silt stratum. The undrained shear strength, within this zone, typically ranges between 500 p.s.f., increasing with depth to about 1,100 p.s.f. The stability and settlement considerations are discussed in the following paragraphs. Recommendations pertaining to foundation design are also given.

6.2.2) Stability Considerations - North Approach

The north approach, in the vicinity of this structure, will have a height of 23 feet in the longitudinal, as well as to the west in the transverse direction (towards the Ramp E-N, S). Towards the east, in the transverse direction, the final height will be of the order of 16 feet. This approach may be of the spill-through type, in the longitudinal direction, in which case the abutment would be 'perched' within the fill, (this is designated as Case 'A'). Alternatively, the abutment could be

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.2) C.N.R. SUBWAY Structure Crossing at Reconstructed J.E.W.

6.2.2) Stability Considerations - North Approach: (cont'd.)

designed as a 'closed-type', in which case the fill would be retained in the forward direction (Case 'B').

The design criteria, adopted in the stability analyses, were presented elsewhere in this report (Sub-section 6.1.2) i). The geometric sections at this approach, and the soil properties for the fill and subsoil, assumed for computational purposes, are presented on Figure #2. The results of the analyses, presented on the aforementioned figure, are summarized in the following paragraphs. In the analyses two types of fill material have been considered:

- i) locally available earth material (total bulk weight 125 p.c.f. assumed for design purposes).
- ii) lightweight fill, such as slag (total unit weights 95 p.c.f. and 80 p.c.f. assumed, depending on the type of material finally selected).

1) Case 'A' = Spill-Through Type of Approach -

a) Longitudinal Direction -

The computations carried out indicate that the factor of safety with respect to a deep-seated failure of the approach, with a 2:1 slope, would be approximately 0.9. If lightweight fill is utilized for the construction of the approaches, the factor of safety would be of the order of 1.1 ($\gamma = 95$ p.c.f.) and 1.25 ($\gamma = 80$ p.c.f.). These results are summarized on Figure #2, Section #1). The aforementioned factors of safety

6.2 C.N.R. SUBWAY STRUCTURE CROSSING AT RECONSTRUCTED Q.E.W.

6.2.2) Stability Considerations - North Approach (cont'd.) ...

1) Case 'A' Spill-Through Type of Approach (Cont'd.) ...

a) Longitudinal Direction (cont'd.) ...

are lower than the value acceptable to the Department (minimum factor of safety equal to 1.3).

The stability in the longitudinal direction would be improved by constructing a berm of the following requirements. (refer to Section #1).

<u>Type of Fill</u>		<u>Berm Length (L)</u>
locally available	(γ = 125 p.c.f.)	30 ft.
Light Weight -	γ = 95 p.c.f.	20 ft.
-	γ = 80 p.c.f.	10 ft.

b) Retaining Wall Section

The profile grade of the Q.E.W. is about elevation 256. The retaining wall foundations could, therefore, be supported on end-bearing piles, with the pile caps located at as high an elevation as 251. Alternatively, the retaining wall could be supported on spread footings, located at or below elevation 239. Recommendations pertaining to structure foundations will be discussed in detail in Sub-section 6.2.5).

The retaining wall, founded at the higher of the two elevations quoted above, would be underlain by the more compressible zone of the clayey silt. This retaining wall section would be inherently unstable, in this the transverse direction, even if light weight fill is employed (refer to Figure #2,

6.2 C.N.R. SUBWAY STRUCTURE CROSSING AT RECONSTRUCTED Q.E.W.

6.2.2) Stability Considerations - North Approach (Cont'd.)

b) Retaining Wall Section (Cont'd.)

Section 2). The stability of the wall will be improved by carrying the foundations down to the lower competent portion of the cohesive stratum, -i.e. foundations located at or below elevation 239. Under these circumstances the potential cylindrical failure surface would be forced through the competent subsoil. The computations carried out, which are summarized on Figure #2 Section #2, indicate that this wall section will be stable, even if locally available fill is used.

The stability analyses, carried out for that portion of the approach immediately north of the retaining wall, indicate that, if locally available fill is employed, the factor of safety with respect to stability, would be below the minimum acceptable value (F.S. = 1.1). Therefore, it would be necessary to extend the retaining structure by some 105 feet - i.e. from station 16+25 to 17+30 (refer to Figure #2, Sections 3 and 4).

If light weight fill is used then the approach, north of the proposed retaining wall, would be stable with respect to a deep-seated rotational type failure (F.S. = 1.35 for $\gamma = 95$ p.c.f.). Therefore, it would not be necessary to extend the wall beyond Station 16+25. In this case light weight fill will be required, in embankment construction, between Stations 16+25 and 17+30.

As an alternative to the aforementioned possibilities the structure could be extended to a point where the abutment would

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.2) C.N.R. SUBWAY.. Structure Crossing at Reconstructed Q.E.W.

6.2.2) Stability Consideration - North Approach: (cont'd.)

b) Retaining Wall Section - (cont'd.) ...

be located at approximately Station 17+30. At this location the proposed approaches (with 2:1 slopes) would be stable in both the longitudinal and transverse directions (refer to Figure #2, Section #4).

If the structure is extended to Station 17+30 the proposed retaining wall can be eliminated. Further, the use of lightweight fill can be avoided.

ii) Case 'B' - Closed-Type Abutment Retaining Fill:

For reasons discussed previously, (refer to Case 'A' Sub-section b) - Retaining Wall Section), the closed-type abutment wall will be unstable in the longitudinal direction, if the foundations are located at about elevation 250, and supported on end-bearing piles. To ensure stability in this direction it will be necessary to place the foundations within the lower competent portion of the clayey silt stratum - i.e., at or below elevation 239. The results of these computations are summarized on Figure #2 (Case 'B', Section #1).

The retaining wall section will be required in this scheme also. The retaining wall requirements, with regard to stability considerations, will be identical to those discussed under Case 'A', Sub-section b).

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.2) C.N.R. SUBWAY Structure Crossing at Reconstructed Q.E.W.

6.2.3) Stability Consideration - South Approach:

In the immediate vicinity of the structure the approach will have a maximum height of 25 feet, in both the longitudinal and transverse directions. The south approach will be inherently more stable than the north approach, since the more compressible zone within the clayey silt stratum is not encountered in this area.

For a spill-through scheme the computations carried out indicate that this approach will be stable with standard 2:1 slopes. The minimum factor of safety will be approximately 1.7, in both the longitudinal and transverse directions.

If a closed-type abutment is employed it will be necessary to locate the footings at or below elevation 244, in order to ensure the stability in the longitudinal direction.

6.2.4) Settlement Considerations - Approaches:

Consolidation and recompression settlements will occur within the cohesive subsoil due to the induced surcharge loading of the embankment fill. Settlement computations were, therefore, carried out, the results of which are summarized on Figure #2 - (North Approach).

In the immediate vicinity of the structure the maximum consolidation settlement will occur under the centre-line of the north approach, since this approach is underlain by the more compressible zone of the upper cohesive stratum. At this location

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.2) C.N.R. SUBWAY Structure Crossing at Reconstructed Q.E.W.

6.2.4) Settlement Considerations - Approaches: (cont'd.)

the maximum height of fill will be 28 feet - i.e., the crest of the embankment will extend 16 feet above original ground level, which was at approximately elevation 266. The computations indicate that the settlement could be of the order of $4\frac{1}{2}$ to 5 inches, if locally available fill is utilized. If lightweight fill is used the magnitude of the settlement will be less than the values quoted above. Beneath the south approach, where -

i) the more compressible zone is not present, and

ii) the height of fill will be comparable to that at the north approach -

the consolidation settlement should be within 4 inches.

The time-rate of settlement will be similar to that discussed in Sub-section 6.1.2 ii) (refer to graph on Figure #1).

6.2.5) Structure Foundations:

At a shallow depth below ground surface, north of Station 13+50, a relatively soft, compressible zone is present. The existence of this zone precludes the economic use of spread footings for the support of the structure elements, in this area. South of this station, however, consideration can be given to founding the elements on spread footings. Recommendations, pertaining to the specific foundation elements will be given in the sub-sections to follow.

6. DISCUSSION AND RECOMMENDATIONS: (Cont'd.) ...

6.2) C.N.R. Subway Structure Crossing at Reconstructed Q.E.W.

6.2.5) Structure Foundations: (cont'd.) ...

1) Pier Foundations -

All of the piers, with the exception of the north end-pier, can be founded on spread footings, located within the competent portion of the upper cohesive stratum. Spread footings, founded at or below the elevations tabulated below, could be designed using an allowable bearing value of 2.5 t.s.f.

<u>Location</u>	<u>Recommended Elevation of Footing</u>
South End-Pier	244
South Intermediate Pier	242
Centre Pier	250
North Intermediate Pier	251

The foundation subsoil will undergo settlement due to the induced footing pressure. For the size of footings contemplated, which will impose the aforementioned pressure, the total settlement should not exceed 1 inch. Since the cohesive subsoil, at and below the proposed footing level, is highly preconsolidated this settlement will be of a recompression nature - i.e., take place during or immediately following the construction period.

As an alternative to a spread footing solution, the piers mentioned above could be supported on end-bearing piles driven to practical refusal into the competent glacial till deposit. It

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.2) C.N.R. Subway Structure Crossing at Reconstructed Q.E.W.

6.2.5) Structure Foundations: (cont'd.) ...

i) Pier Foundations - (cont'd.) ...

should be noted that, in any event, the north end-pier will have to be pile supported, for reasons discussed previously. For estimating purposes, it can be assumed that the pile tips will be located within the following elevation range:

<u>Location</u>	<u>Estimated Pile Tip Elevation (ft.)</u>
South End-Pier	210 - 215
South Intermediate Pier	
Centre Pier	
North Intermediate Pier	205 - 210
North End-Pier	

The piles can be designed for the maximum allowable load for the pile section selected (e.g., 12 BP 74 steel H-piles may be designed for 90 tons/pile).

The excavations for the spread footings, if this foundation type is adopted, will extend approximately 3 feet (at North Intermediate Pier) to 12 feet (at South Intermediate Pier) below the proposed Q.E.W. profile grade. Further, the base of these excavations could be carried down some 3 to 8 feet below the groundwater level, recorded at the time of the investigation. Because of the impervious nature of the subsoil,

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.2) C.N.R. Subway Structure Crossing at Reconstructed Q.E.W.

6.2.5) Structure Foundations: (cont'd.) ...

i) Pier Foundations - (cont'd.) ...

however, no major dewatering problems are anticipated during the construction period. Any minor seepage or surface run-off occurring in the excavations could be handled by employing conventional techniques, such as pumping from sumps. If the foundations are supported on piles, no dewatering problems are anticipated during the construction of the pile caps.

ii) Abutment Foundations -

a) Closed-Type Abutments

To ensure the stability in the longitudinal direction, it will be necessary to locate the foundations of the closed end abutments within the competent portion of the clayey silt stratum - i.e., at or below the following elevations:

North Abutment - Elevation 239

South Abutment - Elevation 244

(Refer to Sub-Section 6.2.2 ii) for discussion of the stability considerations.)

If founded at these elevations, the abutments can be supported on spread footings, using an allowable bearing pressure of 2.5 t.s.f. in design.

Settlement of the foundation subsoil, due to the induced footing pressure, will be similar in nature and magnitude to that

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.2) C.N.R. Subway Structure Crossing at Reconstructed Q.E.W.

6.2.5) Structure Foundations: (cont'd.) ...

ii) Abutment Foundations - (cont'd.) ...

a) Closed-Type Abutments - (cont'd.) ...

discussed in the 'Pier Foundations' section. No major dewatering problems are anticipated.

In computing the passive resistance, below the profile grade of the Q.E.W., an average value for the undrained shear strength (C_u) of 500 p.s.f. should be used down to footing foundation level. If the structure is designed as a rigid frame then a coefficient of earth pressure at rest (K_0) of 0.5 should be assumed for the granular fill material behind the wall, when designing the abutments. However, if some movement of the top of the wall is permitted, then a coefficient of active earth pressure (K_a) of 0.33 can be used. Below the granular fill material a value of 500 p.s.f. can be assumed for the undrained shear strength (C_u), when computing the active earth pressure distribution behind the walls.

It is recommended that a value of 2,000 p.s.f. be used in the computations to determine the sliding resistance between the base of the footing and the underlying cohesive stratum.

b) Spill-Through Type Abutment -

If this solution is adopted, the north abutment will have to be shifted, from its proposed location at Station 14+45,

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.2) C.N.R. Subway Structure Crossing at Reconstructed Q.E.W.

6.2.5) Structure Foundations: (cont'd.) ...

ii) Abutment Foundations - (cont'd.) ...

b) Spill-Through Type Abutment - (cont'd.) ...

to the north, in order to accommodate the berm required in the longitudinal direction (refer to Sub-section 6.2.2) i)).

The abutments may be 'perched' within the approach fills; they can be supported on end-bearing piles driven to practical refusal within the glacial till deposit, as discussed in Sub-section 6.2.5) i). It is estimated that the piles will meet refusal at the following elevations:

North Abutment	-	Elevation 210
South Abutment	-	Elevation 215

No bouldery or rock fill should be used in areas in which piles are to be driven.

In order to:

- eliminate the necessity of constructing the retaining wall section, and
- improve the stability of the north approach (with 2:1 slopes), in the longitudinal and transverse directions,

consideration can be given to extending the structure to the north - (refer to Sub-section 6.2.2) i) b)). An extension of 300 feet is recommended - i.e., the proposed north abutment would be situated at Station 17+30.

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.2) C.N.R. Subway Structure Crossing at Reconstructed Q.E.W.

6.2.5) Structure Foundations: (cont'd.) ...

ii) Abutment Foundations - (cont'd.) ...

b) Spill-Through Type Abutment - (cont'd.) ...

The additional structure elements, required for this extension, could be supported on end-bearing piles, as discussed in the previous paragraphs.

Whether or not this is a feasible alternative, will be dependent on economic considerations.

iii) Differential Settlement Considerations for the Structure Foundations -

If some of the structure elements are supported on spread footings, while others are supported on end-bearing piles, some differential settlements can be anticipated. The magnitude of the differential settlement, between foundations supported on end-bearing piles and on spread footings, would be between 1/2 and 1 inch.

iv) Retaining Wall Foundations -

The stability considerations for this retaining wall were discussed in Sub-section 6.2.2) i) b). In order to improve the stability in the transverse direction (west side), the foundations for the retaining wall will have to be located within the competent portion of the cohesive stratum, at or below elevation 239. The wall can be supported on spread footings

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.2) C.N.R. Subway Structure Crossing at Reconstructed Q.E.W.

6.2.5) Structure Foundations: (cont'd.) ...

iv) Retaining Wall Foundations - (cont'd.) ...

using an allowable bearing value of 2.5 t.s.f. in design. The same recommendation will be applicable if the retaining wall has to be extended to Station 17+30, as discussed previously.

The settlement, that will occur in the foundation subsoil, due to the induced footing pressure, will be of a recompression nature. The magnitude of this settlement will not exceed 1 inch.

The recommendations pertaining to earth pressure computations, as well as sliding resistance between the base of the footing and the underlying subsoil, will be similar to those discussed in the Sub-section on Abutment Foundations (Sub-section 6.2.5) ii).

In order to relieve the build-up of excess hydrostatic pressure behind the retaining structure, suitable drainage measures should be provided. If the embankments are not constructed of a relatively free-draining type of granular material, the following measures should be taken:

An 8-ft. wide vertical strip of free-draining granular material should be provided behind the wall; the remainder of the backfill could consist of locally available earth material similar to that used for embankment construction. In addition to the 8-ft. wide gravel strip behind the wall, a horizontal layer of gravel, 4 ft. thick should be built into the backfill at half the height

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.2) C.N.R. Subway Structure Crossing at Reconstructed Q.E.W.

6.2.5) Structure Foundations: (cont'd.) ...

iv) Retaining Wall Foundations - (cont'd.) ...

of the wall, and should extend for a distance equal to one-half the height of the wall. No horizontal drains would be required for that portion of the retaining wall having a height of less than 12 ft. Suitable weep holes should be provided at the base of the wall at a maximum spacing of 10 ft. D.H.O. Standard SD-4-58, prepared for various retaining wall backfilling requirements for Hwy. 401 Toronto Bypass, may be used for design and construction purposes.

6.2.6) Foundations In Close Proximity to Buried Utilities:

If spread footing foundations are proposed in an area that is in close proximity to a buried utility, they could be supported on end-bearing steel H-piles driven to practical refusal into the competent glacial till deposit, as discussed in Sub-section 6.2.5).

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 procedure be adopted:

1) When piles will be 12 feet or more from the edge of a utility, no special precautions need be taken.

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.2) C.N.R. Subway... Structure Crossing at Reconstructed Q.E.W.

6.2.6) Foundations in Close Proximity to Buried Utilities: (cont'd.) ...

2) All piles, closer than 12 feet from a utility should be prebored to a depth of 6 feet below the invert of the utility. The size of the augered hole need only be slightly larger than the pile section.

The above procedure was followed in Contracts 63-182 and 68-24 with satisfactory results.

7. MISCELLANEOUS:

The field work, performed during the period of August 21 to September 25, 1969, was under the immediate supervision of Mr. V. Korlu, Project Foundation Engineer. The equipment was owned and operated by Dominion Soil Investigation Ltd., Toronto.

This report was written by Mr. E. T. Darch, Senior Foundation Engineer, who was assisted by Mr. Korlu. This project was carried out under the general supervision of Mr. M. Devata, Supervising Foundation Engineer, who reviewed this report.

November 1969

APPENDIX I

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 13

FOUNDATION SECTION

JOB 69-F-70

LOCATION Co-ord. 712, 827N., 931, 789 E

ORIGINATED BY **VK**

W.P. 10-57-03

BORING DATE September 17, 1969

COMPILED BY HR

DATUM Geodetic

BOREHOLE TYPE Washboring, NX Casing, Tricone Bit, BXL Rock Core CHECKED BY

[illegible]

FOUNDATION SECTION

ORIGINATED BY VK

COMPILED BY GP

CHECKED BY

[illegible]

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 16

FOUNDATION SECTION

JOB 69-F-70 LOCATION CO-ORD. 712, 936N., 931,745E
W.P. 10-57-03 BORING DATE September 12, 1969
DATUM Geodetic BOREHOLE TYPE Pen DrillORIGINATED BY VKCOMPILED BY GPCHECKED BY 4.6

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. PLOT	NUMBER	TYPE	BLOWS / FOOT		20	40	60	80	100	w_p	w	w_L		
265.3	Ground Level															
0.0	Clayey silt with sand & gravel trace of org.		1	SS	20											
261.3	(v. stiff) (fill mat'l)		2	SS	12	260										
4.0	Clayey silt with traces of sand and occ. gravel		3	SS	37											
	Stiff to Hard		4	SS	43											
			5	SS	30											
248.3	(Brown)		6	SS	24	250										
17.0	(Grey)		7	TW	PH											
			8	TW	PH	240										
			9	TW	PH											
			10	SS	73											
228.3			11	SS	81	230										
37.0	End of Bore Hole					220										

W.L.
251.7

130

136.5

 ρ $\times S = 5$

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 17

FOUNDATION SECTION

JOB 69-F-70

LOCATION GO-ORD. 713, 027N., 931, 776E

ORIGINATED BY VK

W.P. 10-57-03

BORING DATE September 10, 1969

COMPILED BY GP

DATUM Geodetic

BOREHOLE TYPE Washboring, NX, Casing

CHECKED BY *JK*

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT ——— w_L PLASTIC LIMIT ——— w_p WATER CONTENT ——— w			BULK DENSITY γ P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT					SHEAR STRENGTH P.S.F.					WATER CONTENT %
							20	40	60	80	100	○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE					
							500	1000	1500	2000	2500	w_p ——— w ——— w_L					
253.1	Ground Level																
0.0	Clayey silt & some sand & org. Stiff (fill mat'l)		1	SS	13	250											
250.1																	
3.0	Clayey silt to silty clay with sand and occ. gravel		2	SS	26												
			3	SS	27												
			4	SS	30												
	V. stiff to hard (Brown)		5	SS	36	240											
239.1																	
14.0	(Grey)		6	SS	29												
			7	SS	28												
			8	SS	26												
			9	SS	27	230											
222.1			10	SS	67												
31.0	Clayey silt with sand and gravel					220											
	Hard - Reddish		11	SS	89												
	(Glacial Till)																
212.6			12	SS	100/6"												
40.5	End of Bore Hole					210											

W.L.
247.0

17-26-40-17

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 18

FOUNDATION SECTION

JOB 69-F-70

LOCATION

CO-ORD. 713, 041N., 931, 738E

ORIGINATED BY VK

W.P. 10-57-03

BORING DATE

September 11, 1969

COMPILED BY GP

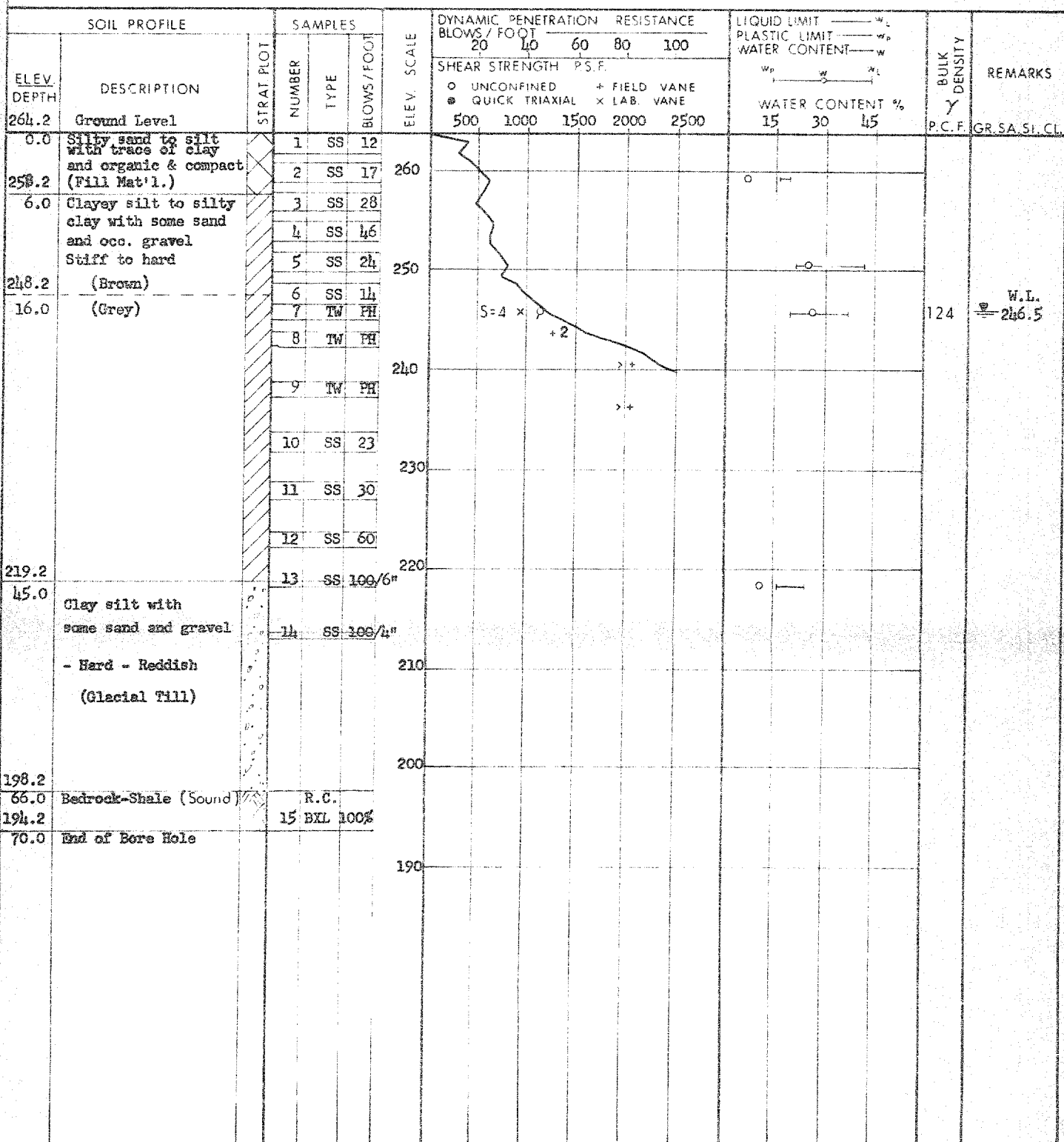
DATUM Geodetic

BOREHOLE TYPE

Washboring, Tricone Bit, BXL Coring

CHECKED BY

LK



DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 19

FOUNDATION SECTION

JOB 69-F-70 LOCATION CO-ORD. 713, 140N., 931, 779E ORIGINATED BY VK
 W.P. 10-57-03 BORING DATE September 16, 1969 COMPILED BY GP
 DATUM Geodetic BOREHOLE TYPE Washboring, NX, Casing CHECKED BY *LR*

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. PLOT	NUMBER	TYPE	BLOWS / FOOT		20	40	60	80	100	WATER CONTENT % w_p — w — w_L				
258.8	Ground Level						SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE									
0.0	Clayey silt with traces of sand V. stiff to hard		1	SS	27	250										W.L. 249.4
			2	SS	32											
			3	SS	32											
			4	SS	19											
			5	SS	17											
242.8	(Brown)		6	SS	18	240										
16.0	(Grey)				70											
			7	SS	29											
			8	SS	35											
			9	SS	39											
227.8						230										
31.0	End of Bore Hole															
						220										

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 20

FOUNDATION SECTION

JOB 69-F-70

LOCATION

CO-ORD. 713, 152N., 931, 731E

ORIGINATED BY VK

W.P. 10-57-03

BORING DATE

September 9, 1969

COMPILED BY GP

DATUM Geodetic

BOREHOLE TYPE

Washboring, NX, Casing

CHECKED BY *HL*

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 PLOT	NUMBER	TYPE		BLOWS / FOOT	20	40	60	80	100	15	30		
262.1	Ground Level														
0.0			1	SS	13	260									
	Clayey silt to silty clay with traces of sand and occ. gravel V. stiff to Hard		2	SS	25										
			3	SS	15										
			4	SS	15										
			5	SS	21	250									
			6	SS	28										
			7	SS	26										
244.6	(Brown)		8	SS	30	240									
17.5	(Grey)		9	TW	PT										
			10	SS	26										
			11	SS	44	230									
			12	SS	38										
221.6			13	SS	70	220									
40.5	End of Bore Hole					210									

FOUNDATION SECTION

ORIGINATED BY VK

COMPILED BY GP

CHECKED BY *AK*

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT ——— w_L PLASTIC LIMIT ——— w_p WATER CONTENT ——— w			BULK DENSITY γ P.C.F.	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BLOWS / FOOT	20	40	60	80	100	WATER CONTENT % w_p ——— w ——— w_L		
264.9	Ground Level														
0.0			1	SS	12										
			2	SS	15										
			3	TW	PH										
			4	TW	PH										
			5	SS	33										
248.4			6	SS	26										
16.5															
			7	SS	21										
			8	SS	19										
232.0			9	SS	32										
32.0	End of Bore Hole														

Q x S = 3.5

W.L.
255.4

FOUNDATION SECTION

CHECKED BY *[Signature]*

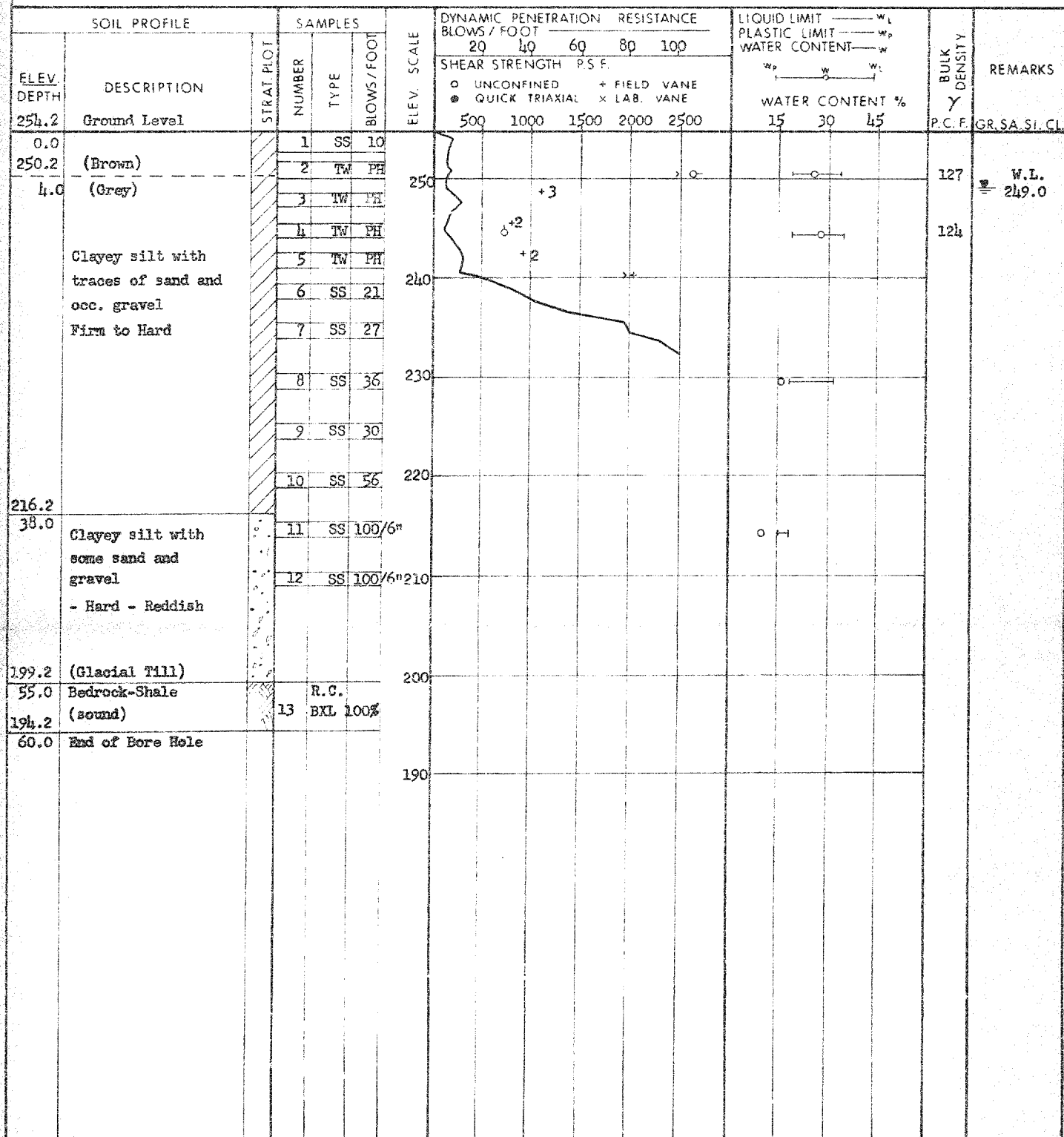
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DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 25

FOUNDATION SECTION

JOB 69-F-70 LOCATION 00-ORD. 713, 330N., 931, 761E. ORIGINATED BY VK
 W.P. 10-57-03 BORING DATE September 4, 1969 COMPILED BY GP
 DATUM Geodetic BOREHOLE TYPE Washboring, NX Casing, BXL, Rock Core CHECKED BY *HK*

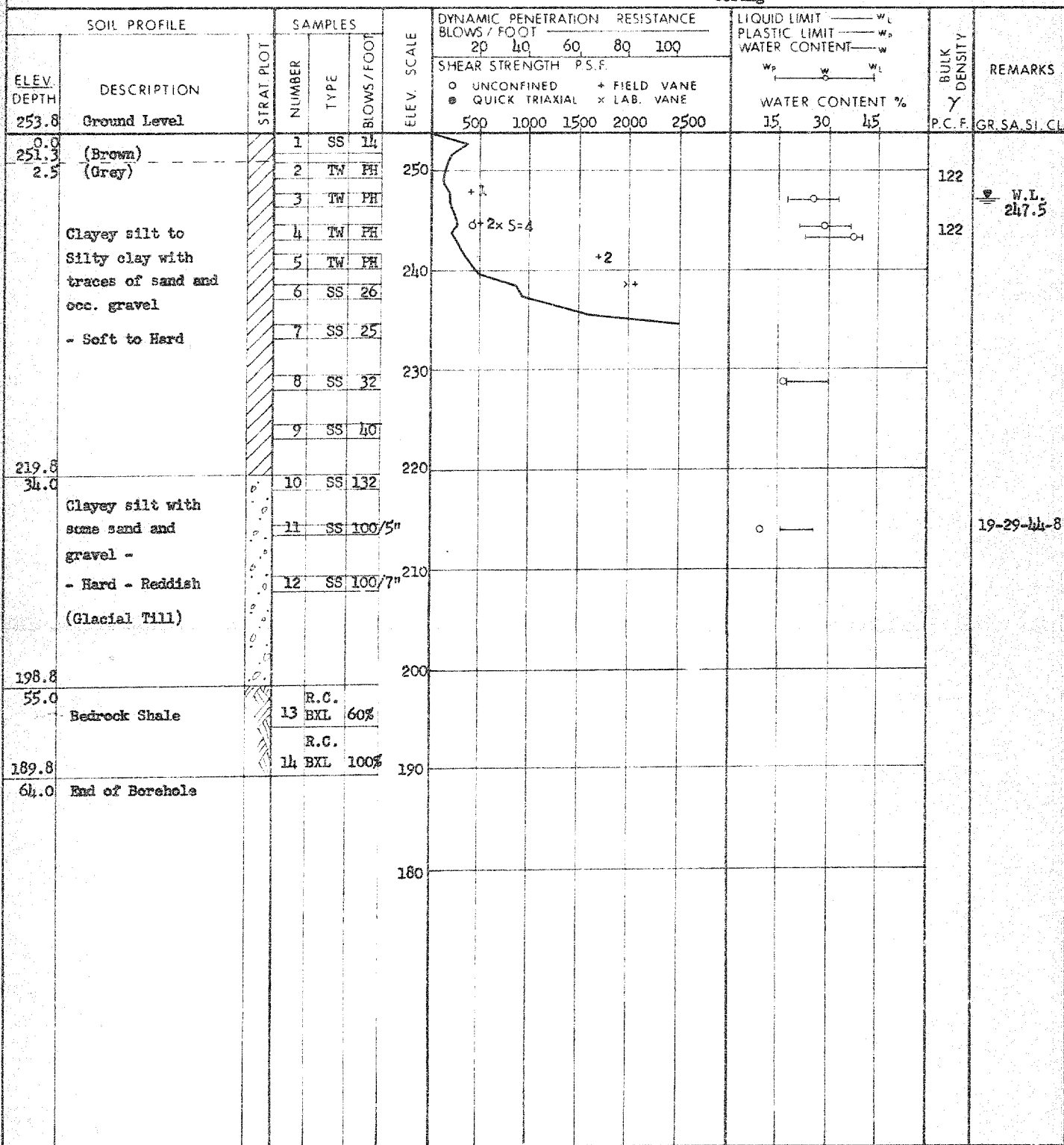



DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 26

FOUNDATION SECTION

JOB 69-F-70 LOCATION OO-ORD. 713, 413N., 931, 697E ORIGINATED BY VK
 W.P. 10-57-03 BORING DATE September 2, 1969 COMPILED BY GP
 DATUM Geodetic BOREHOLE TYPE Washboring-NX Casing, Tricone Bit, BXL Rock CHECKED BY *LR*
 Coring



CHECKED BY 

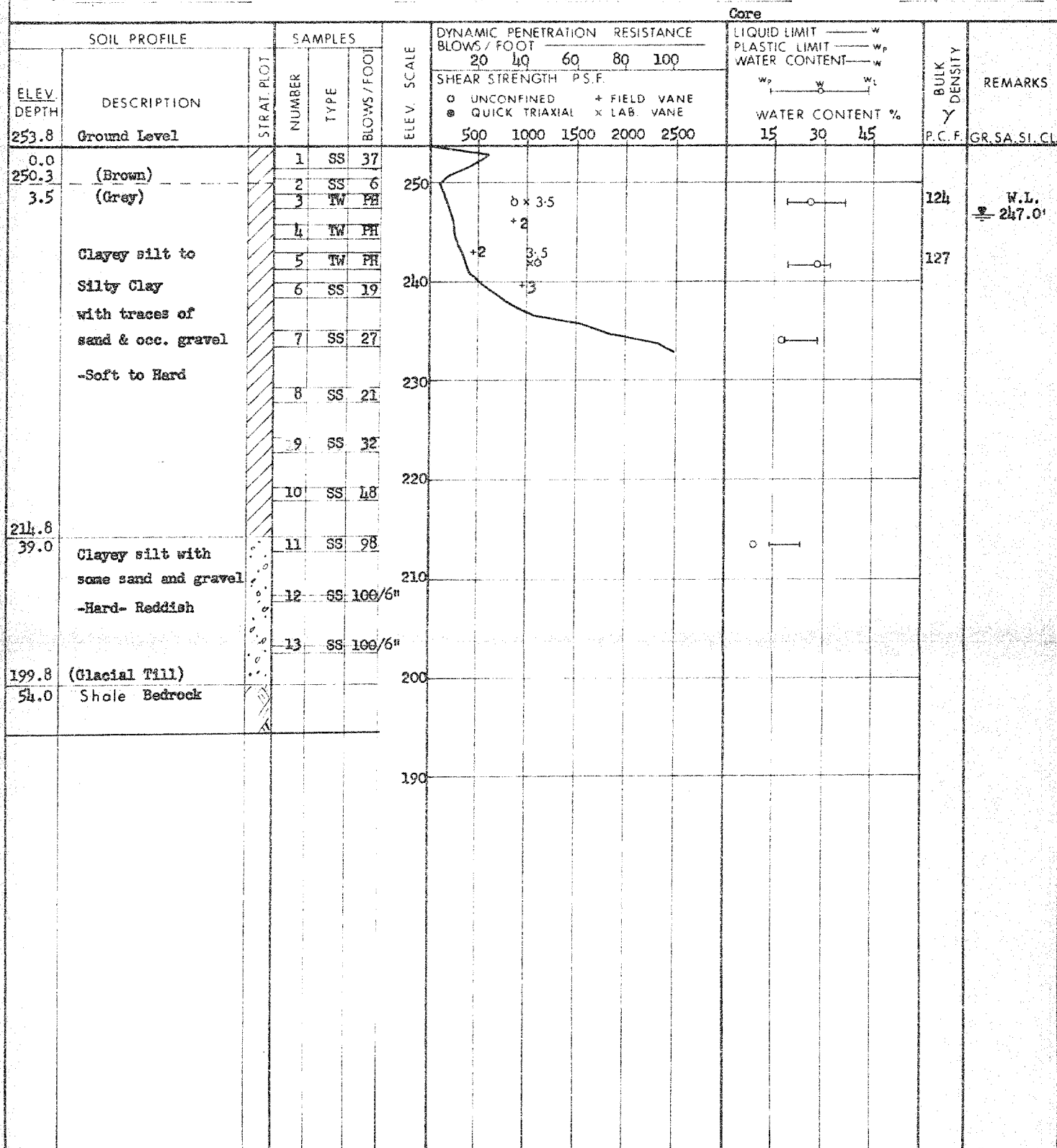
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DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 28

FOUNDATION SECTION

JOB 69-F-70 LOCATION CO-ORD. 713, 511N., 931, 693E ORIGINATED BY VK
 W.P. 10-57-07 BORING DATE August 27, 1969 COMPILED BY GP
 DATUM Geodetic BOREHOLE TYPE Washboring, NX Casing, Tricone Bit, Art Rock CHECKED BY *AK*

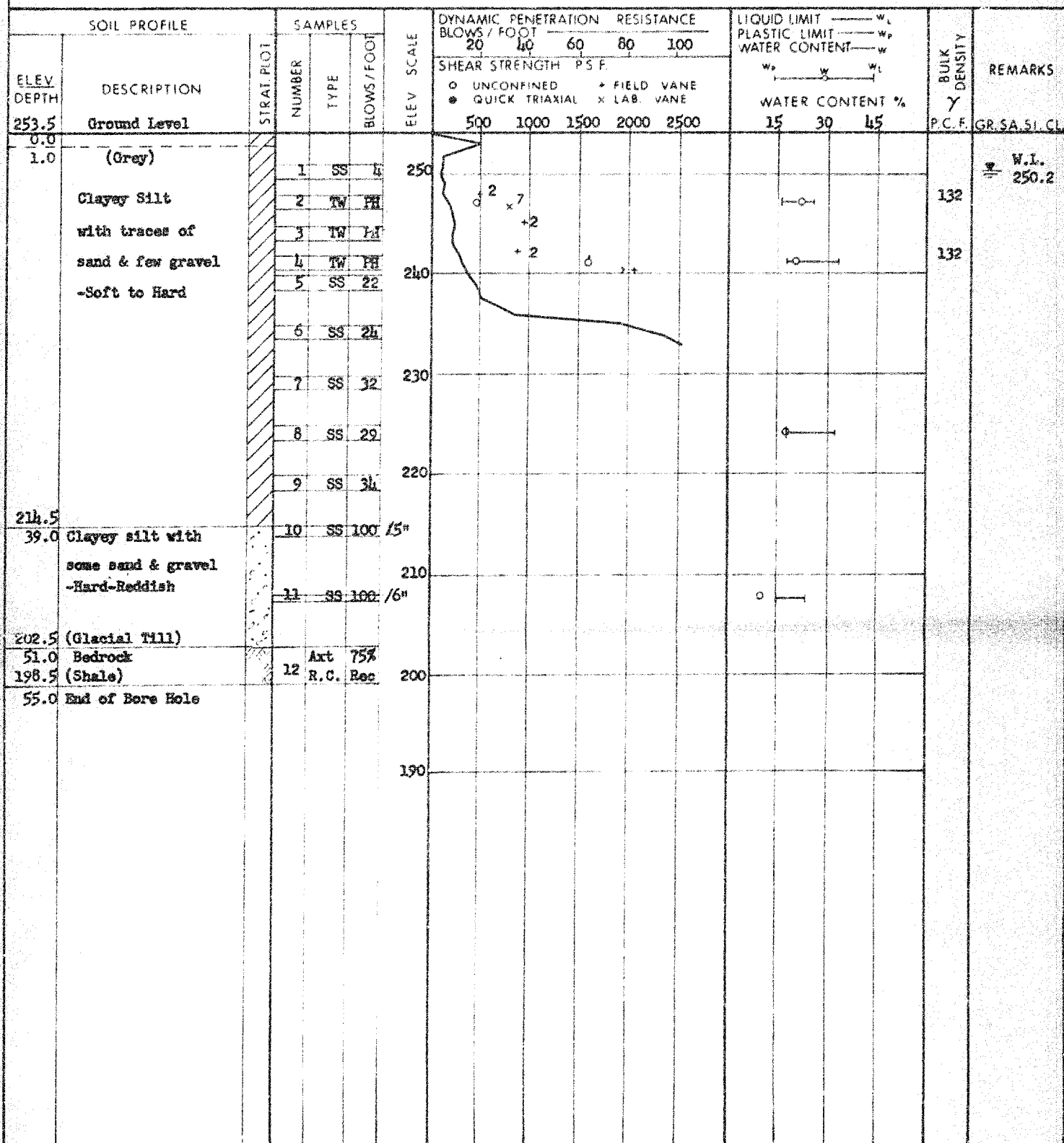


DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 29

FOUNDATION SECTION

JOB 69-F-70 LOCATION CO-ORD. 713, 594N., 931, 669E ORIGINATED BY VK
W.P. 10-57-03 BORING DATE August 25, 1969 COMPILED BY GP
DATUM Geodetic BOREHOLE TYPE Washboring, NX Casing, Axt. Rock Core CHECKED BY 41



DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 30

FOUNDATION SECTION

JOB 69-F-70 LOCATION Co-Ord. 713, 602N., 931, 718E

ORIGINATED BY VK

W.P. 10-57-03 BORING DATE Aug. 21, 1969

COMPILED BY GP

DATUM Geodetic BOREHOLE TYPE Washboring, NX Casing

CHECKED BY

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			BULK DENSITY γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	20	40	60	80	100	w_p	w	w_L		
266.2	Ground Level														
0.0	Silty sand & trace of organics - loose (Probable fill)		1	SS	7										
261.2			2	SS	28										0.70 (30) W.L. 260.7
5.0			3	SS	22										
256.2	(Brown)		4	TW	PH										126.5
16.0	(Grey)		5	TW	PH										
	Clayey silt to Silty Clay		6	TW	PH										118
	with traces of sand & occ. gravel		7	TW	PH										
	Soft to Hard		8	SS	28										
			9	SS	36										
			10	SS	35										
			11	SS	28										
217.2			12	SS	89										
19.0	Clayey silt with some sand & gravel - Hard		13	SS	100 3/4"										
212.6	Reddish (Glacial Till)														
53.6	End of Bore Hole														
	Probable Bedrock														

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 30A

FOUNDATION SECTION

JOB	69-F-70	LOCATION	CO-ORD. 713, 700N., 931, 699E	ORIGINATED BY	VK
W.P.	10-57-03	BORING DATE	September 16, 1969	COMPILED BY	GP
DATUM	Geodetic	BOREHOLE TYPE	Pen Drill	CHECKED BY	WFF

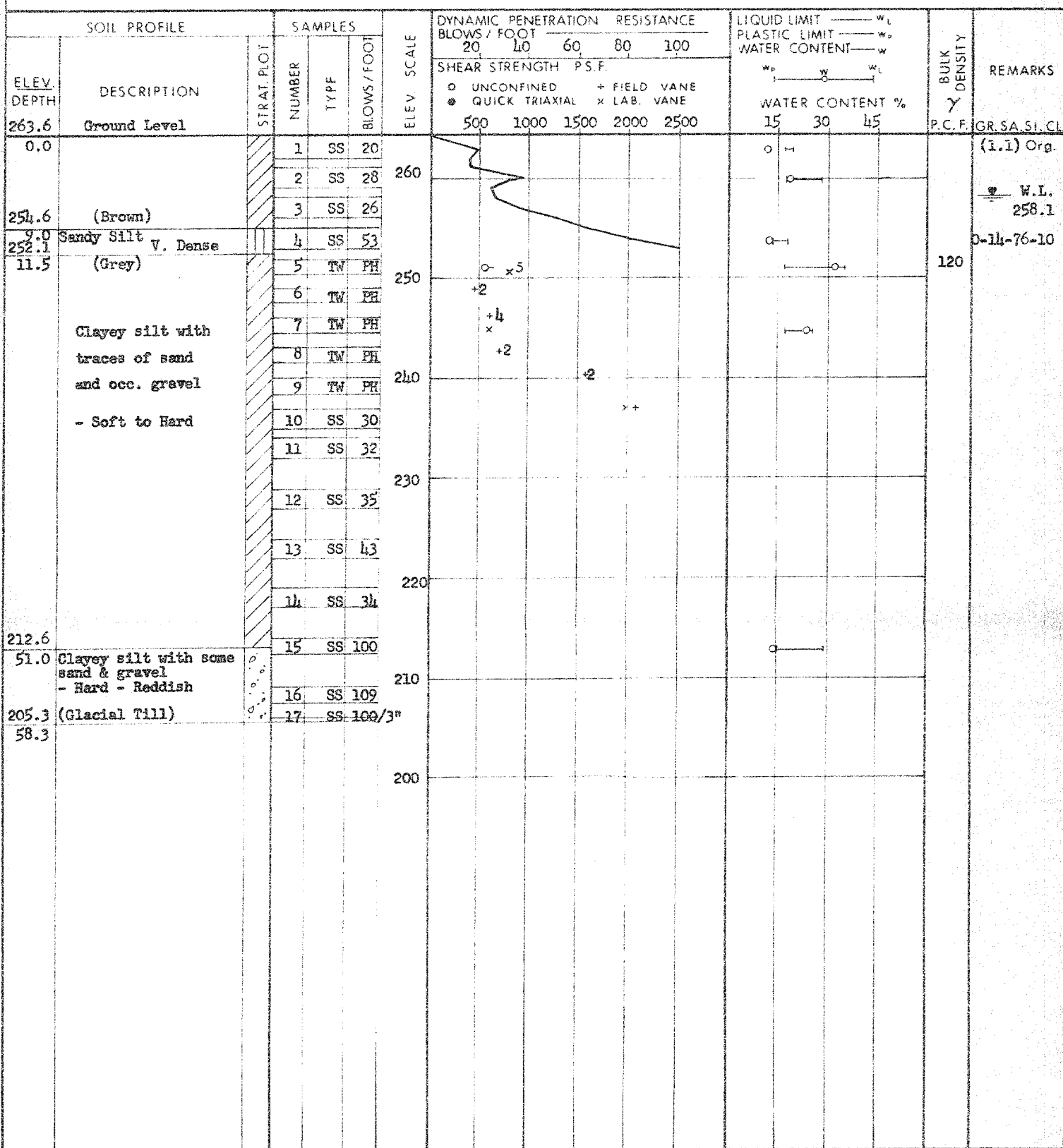
[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 300

FOUNDATION SECTION

JOB 69-P-70 LOCATION CO-ORD 714, 069N., 931, 565E ORIGINATED BY VK
 W.P. 10-57-04 BORING DATE September 29, 1969 COMPILED BY GP
 DATUM Geodetic BOREHOLE TYPE Pen Drill CHECKED BY *HL*

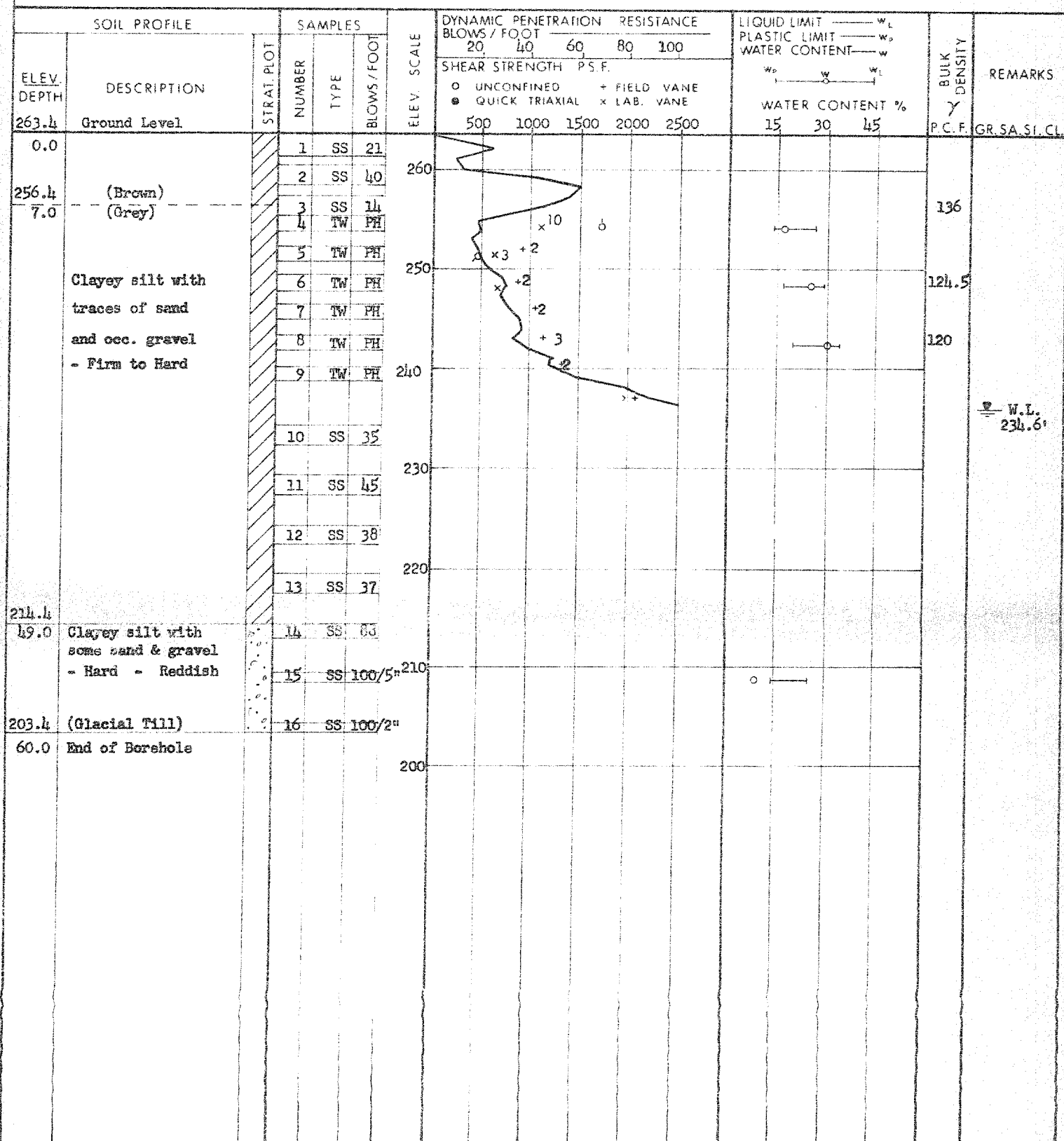


DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 31

FOUNDATION SECTION

JOB 69-F-70 LOCATION CO-ORD. 714, 291N., 931, 457E ORIGINATED BY VK
 W.P. 10-57-04 BORING DATE September 18, 1969 COMPILED BY GP
 DATUM Geodetic BOREHOLE TYPE Pen Drill CHECKED BY *AK*



DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 32

FOUNDATION SECTION

JOB 69-F-70

LOCATION

CO-ORD 714, 312N., 931, 413E

ORIGINATED BY VK

W.P. 10-57-04

BORING DATE

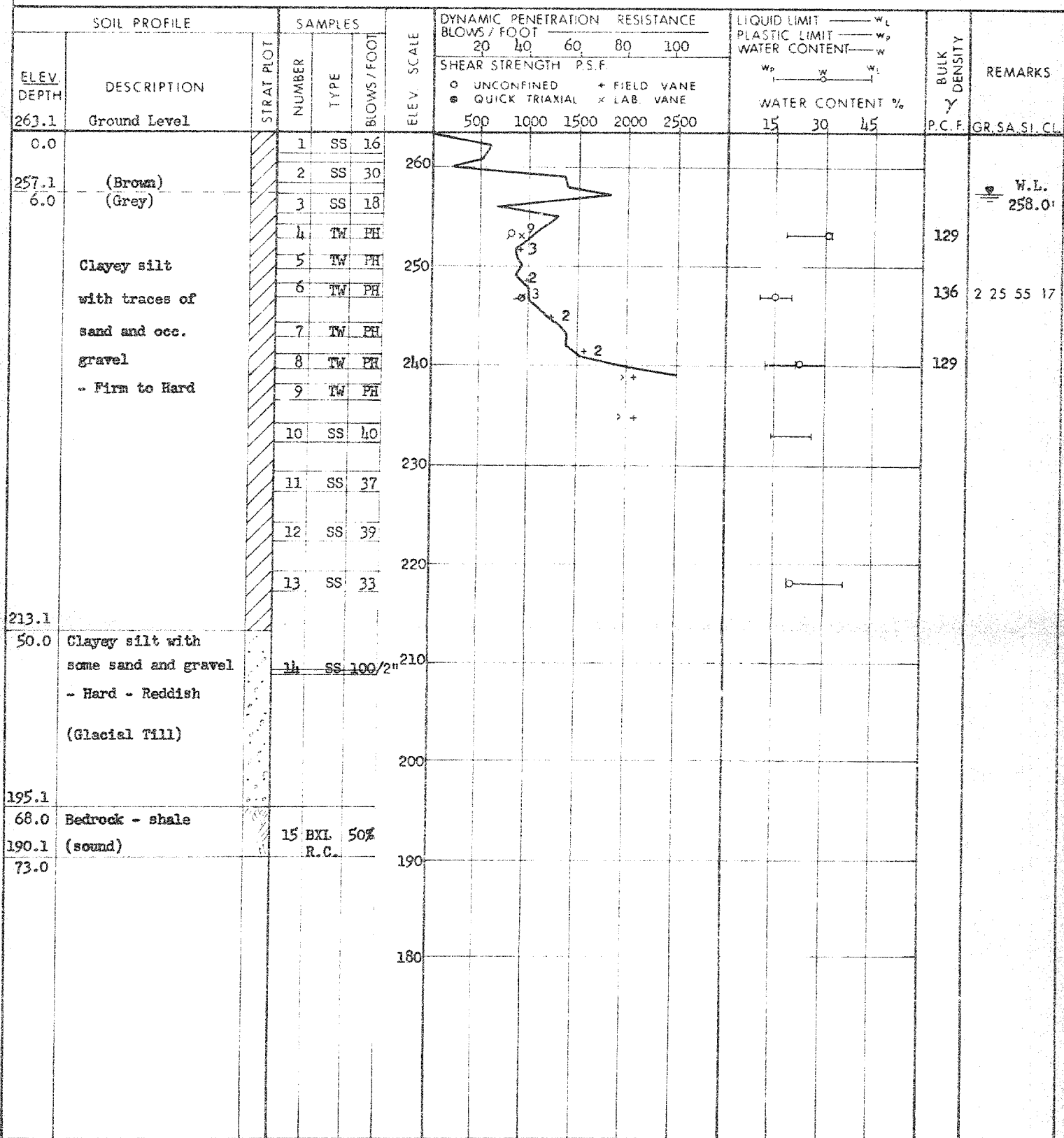
September 17, 1969

COMPILED BY GP

DATUM Geodetic

BOREHOLE TYPE

Pen Drill

CHECKED BY *AK*

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 35

FOUNDATION SECTION

JOB 69-F-70

LOCATION

CO-ORD. 714, 389N., 931, 407E

ORIGINATED BY VK

W.P. 10-57-04

BORING DATE

September 19, 1969

COMPILED BY GP

DATUM Geodetic

BOREHOLE TYPE

Pen Drill

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		20	40	60	80	100	w_p	w	w_L		
262.0	Ground Level														
0.0			1	SS	36										(.8) Org.
255.0	(Brown)		2	SS	37										
7.0	(Grey)		3	SS	24										W.L. 253.7
	Clayey silt		4	SS	22										
	with traces of		5	TW	PH									121	
	sand and		6	TW	PH										
	occ. gravel		7	TW	PH									120.5	
	- Firm to Hard		8	TW	PH										
			9	TW	PH										
			10	TW	PH										
			11	SS	39										
			12	SS	56										
			13	SS	40										
			14	SS	34										
208.0			15	SS	164										
54.0	Clayey silt with some sand and gravel		16	SS	100/5"										
	- Hard - Reddish -														
194.0	(Glacial Till)														
68.0	Bedrock - Shale		17	BXL	Rec										
189.0	(sound)														
73.0	End of Borehole														

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 36

FOUNDATION SECTION

JOB 69-F-70 LOCATION CNR & HWY. 20 714.416N, 931.339E ORIGINATED BY VK
 W.P. 10-57-04 BORING DATE September 22, 1969 COMPILED BY VK
 DATUM Geodetic BOREHOLE TYPE Pen Drill 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	NUMBER	TYPE	BLOWS / FOOT		20	40	60	80	100	w_p	w	w_L		
259.2	Ground Level														
0.0		1	SS	20											(1.3) Org.
		2	SS	23											
		3	SS	14											
249.2	(Brown)	4	SS	30	250										W.L. 251.7'
10.0	(Grey)	5	SS	22											
	Clayey silt to silty clay with traces of sand and occ. gravel	6	TW	PH										120	
		7	TW	PH	240										
		8	TW	PH										127	
	Firm to Hard	9	SS	31											
		10	SS	38	230										
		11	SS	34											
		12	SS	39	220										
213.7		13	SS	46											
45.5	Clayey silt with some sand & gravel - Hard - Reddish	14	SS	100/2"	210										
205.0	(Glacial Till)	15	SS	100/2"											
54.2	End of Borehole				200										

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 39

FOUNDATION SECTION

JOB 69-F-70 LOCATION C.N.R. & HWY. 20 714,489 N., 931,321 E. ORIGINATED BY VK
 W.P. 10-57-04 BORING DATE September 23, 1969 COMPILED BY VK
 DATUM Geodetic BOREHOLE TYPE Drive BX Casing & Wash & Tricone Drill CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			BULK DENSITY γ P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT					SHEAR STRENGTH P.S.F.					WATER CONTENT %
							20	40	60	80	100	UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE					
						500	1000	1500	2000	2500	15 30 45						
256.9	Ground Level		1	SS	13												
0.0	Silty sand with traces of organics - compact to dense		2	SS	10											-2 -73 -25	
247.4			3	SS	39											W.L. 251.2	
9.5			4	TW	PH											-0 -93 -7	
	Clayey silt to silty clay with traces of sand and occ. gravel		5	TW	PH												
			6	TW	PH												
			7	TW	PH												
			8	TW	PH												
	Firm to Hard Grey		9	SS	14												
			10	SS	23												
			11	SS	40												
			12	SS	23												
212.9			13	SS	72												
44.0	Clayey silt with some sand and gravel - Hard - Reddish (Glacial Till)		14	SS	100/2"												
			15	SS	100/6"												
192.9																	
64.0	Bedrock - shale (Sound)		16	BXL	100% Rec												
187.9																	
69.0	End of Borehole																

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 40

FOUNDATION SECTION

JOB 69-F-70 LOCATION C.N.R. & HWY. 20 714,520 N., 931,288 E. ORIGINATED BY VK
 W.P. 10-57-04 BORING DATE September 23, 1969 COMPILED BY VK
 DATUM Geodetic BOREHOLE TYPE Pen-drill CHECKED BY VK

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			BULK DENSITY γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	20	40	60	80	100	w_p	w	w_L		
260.3	Ground Level					SHEAR STRENGTH P.S.F.					WATER CONTENT %			P.C.F.	GR. SA. SI. CL.
0.0	Silty sand with traces of clay and organics - compact		1	SS	14	500 1000 1500 2000 2500					15	30	45		
254.3			2	SS	19										0-44-51-5
6.0			3	SS	32										
247.8	(Brown)		4	SS	17									116	
12.5	(Grey)		5	TW	PH										W.L. 245.6'
	Clayey silt to silty clay with traces of sand and occ. gravel		6	TW	PH									122	
			7	TW	PH									132.5	
			8	TW	PH										
			9	TW	PH										
			10	SS	28										
	Firm to Hard		11	SS	48										
			12	SS	54										
			13	SS	50										
211.3			14	SS	51										7 19 48 26
49.0	Clayey silt, some sand & gravel		15	SS	100/3"										
206.8	(Glacial Till) Hard														
53.5	End of Borehole														

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

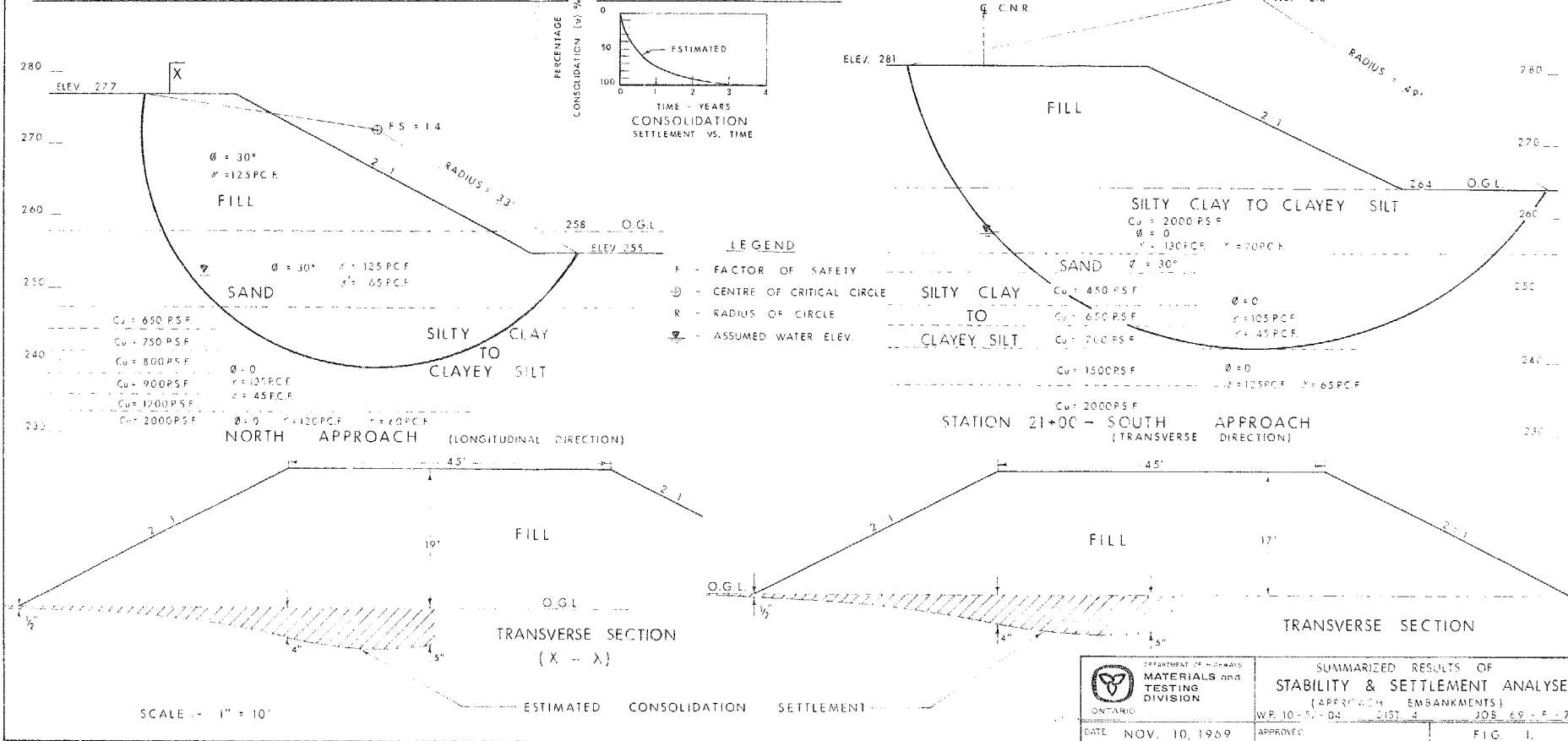
RECORD OF BOREHOLE No. 40A

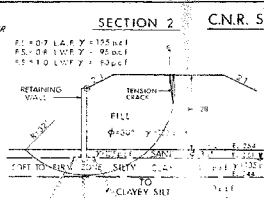
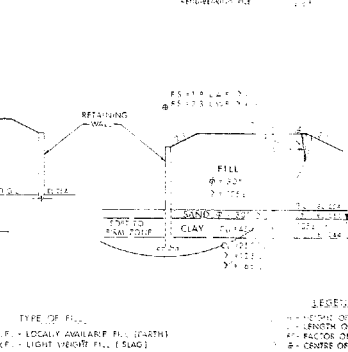
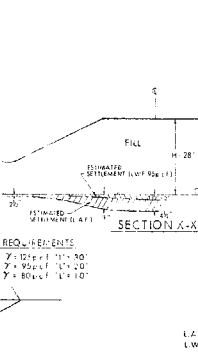
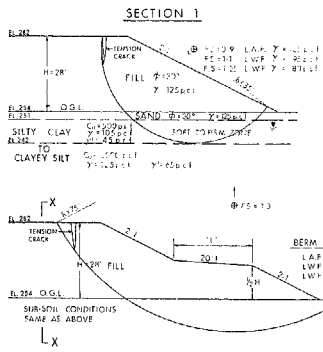
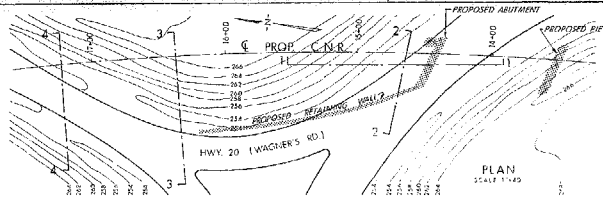
FOUNDATION SECTION

JO# 69-F-70 LOCATION C.N.R. & HWY. 20 714,752N., 931,032E. ORIGINATED BY VK
 W.P. 10-57-04 BORING DATE September 30, 1969 COMPILED BY VK
 DATUM Geodetic BOREHOLE TYPE Pen-drill CHECKED BY *[Signature]*

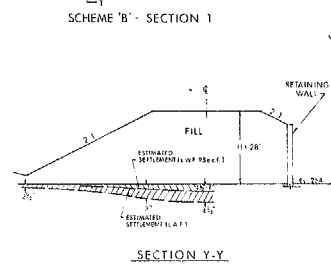
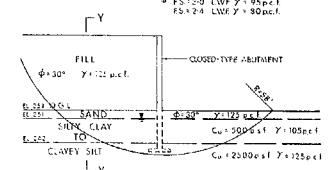
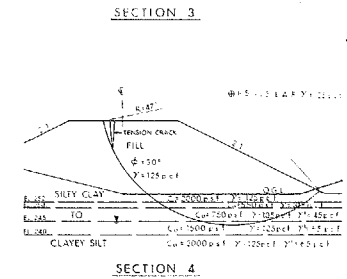
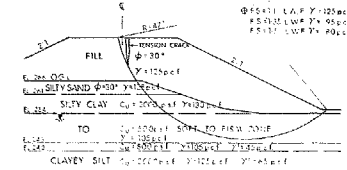
SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			BULK DENSITY γ P.C.F.	REMARKS				
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT					SHEAR STRENGTH P.S.F.					w_p — w — w_L	WATER CONTENT % 15 30 45		
							20	40	60	80	100	PS F.								
												○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE								
251.7	Ground Level						500	1000	1500	2000	2500	15	30	45						
0.0	Silty sand with clayey silt and traces of organics		1	SS	17	250										W.L. 249.7				
244.2	- Compact		2	SS	8															
7.5			3	SS	10															
238.2	(Brown)		4	SS	31	240														
13.5	(Grey)		5	SS	18															
	Clayey silt with traces of sand and occ. gravel		6	TW	PH		3-5 x o								128					
			7	TW	PH			+2												
227.2	- Firm to Hard		8	TW	PH	230									136					
24.5			9	SS	44															
	Clayey silt with some sand and gravel		10	SS	100															
	- Hard		11	SS	100/5"	220														
	- Reddish		12	SS	100/3"															
	(Glacial Till)		13	SS	100/3"	210														
203.2			14	SS	100/6"															
48.5	End of Borehole					200														

C.N.R. SUBWAY STRUCTURE AT THE CROSSING OF REVISED HWY. 20





C.N.R. SUBWAY STRUCTURE AT THE CROSSING OF RECONSTRUCTED Q.E.W. (NORTH APPROACH)



TYPE OF FILL
L.A.F. = LOCALLY AVAILABLE FILL (GARTH)
L.W.F. = LIGHT WEIGHT FILL (SLAG)

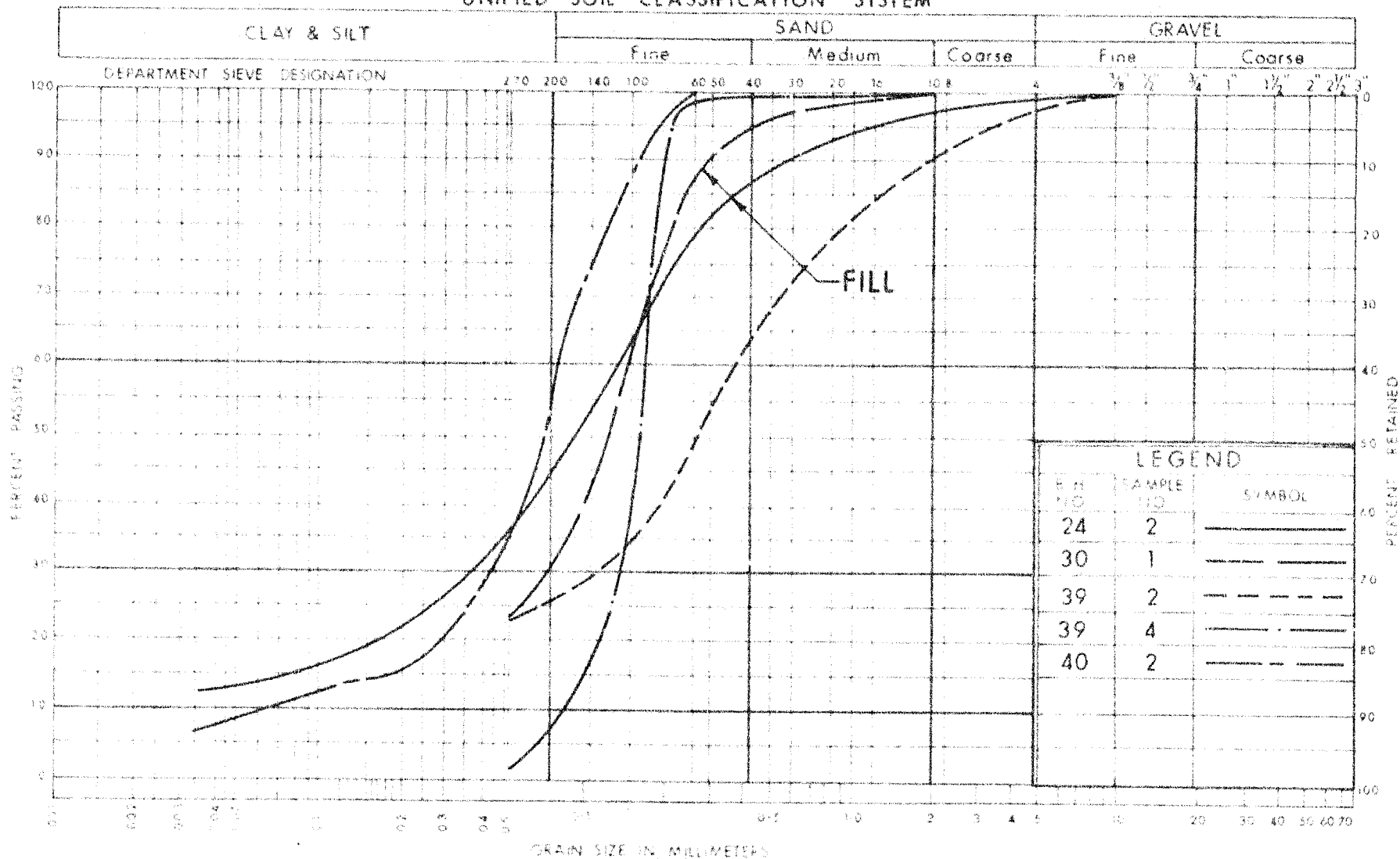
LEGEND
H = HEIGHT OF FILL (FT)
L = LENGTH OF BERM (FT)
F.S. = FACTOR OF SAFETY
C = CENTRE OF CRITICAL CIRCLE
R = RADIUS OF CIRCLE (FT)

ONTARIO
DEPARTMENT OF HIGHWAYS
MATERIALS AND
TESTING
DIVISION
DATE 11 NOV. 1969

SUMMARIZED RESULTS OF
STABILITY & SETTLEMENT ANALYSES
(APPROACH EMBANKMENTS)
W.F. 10-57-03
APPROVED

FIGURE NO. 2

UNIFIED SOIL CLASSIFICATION SYSTEM



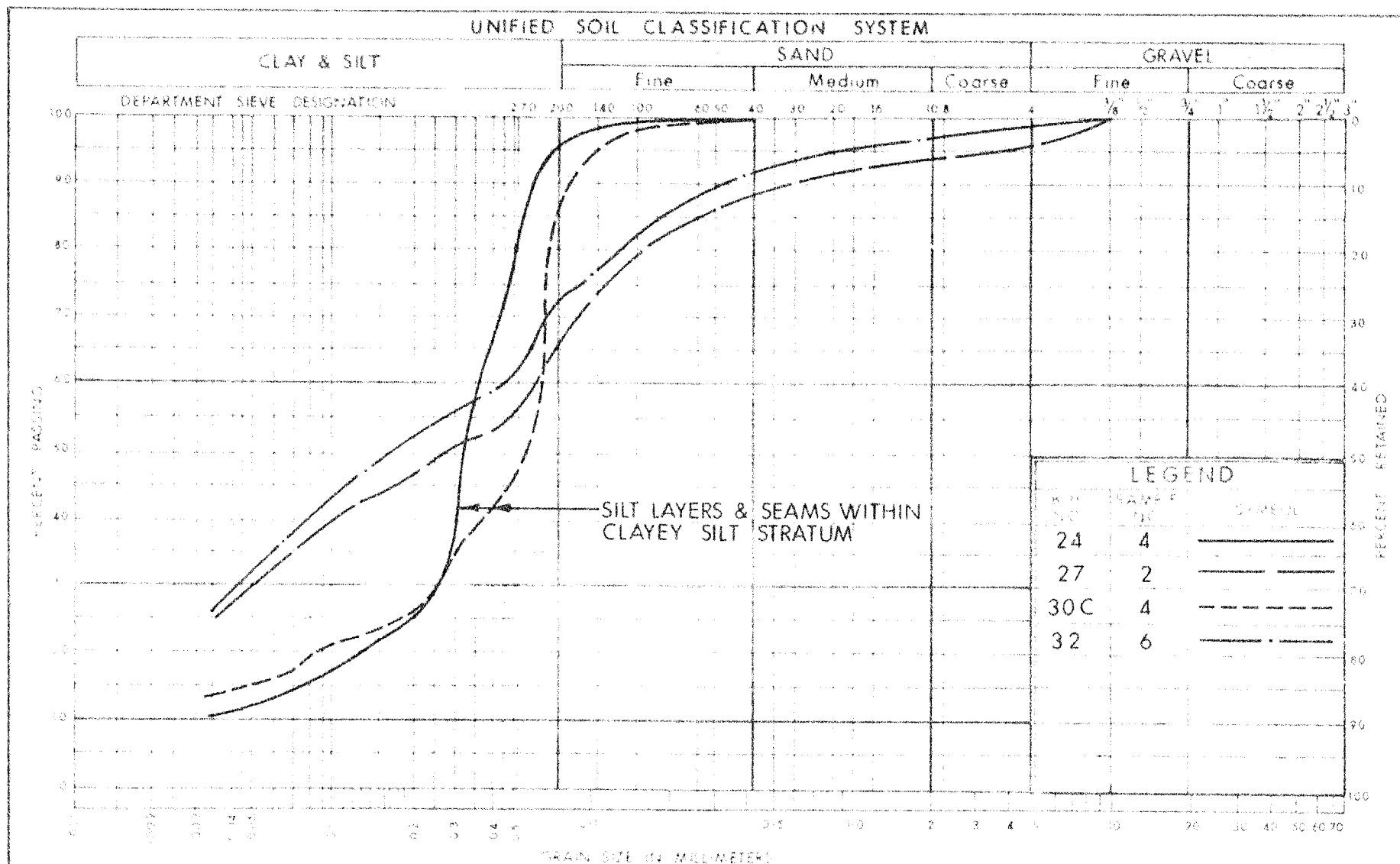
DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

GRAIN SIZE DISTRIBUTION
SILTY SAND
TRACE OF CLAY

WP No. 10-57-03

JOB No. 69-F-70

FIG 3



SILT LAYERS & SEAMS WITHIN CLAYEY SILT STRATUM



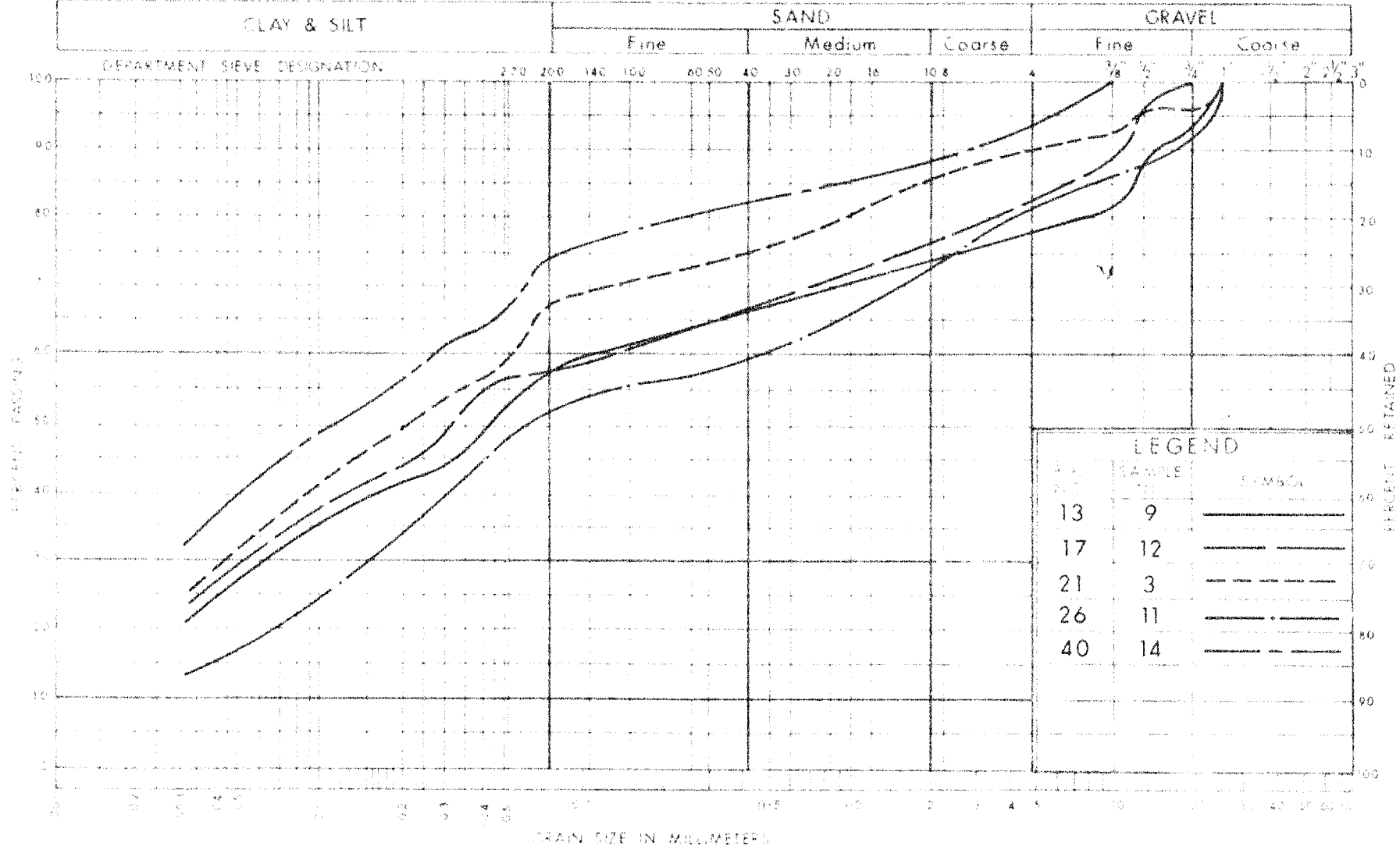
DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

GRAIN SIZE DISTRIBUTION CLAYEY SILT TO SILTY CLAY

W.P. No. 10-57-03
JOB No. 69-F-70

FIG. 4

UNIFIED SOIL CLASSIFICATION SYSTEM



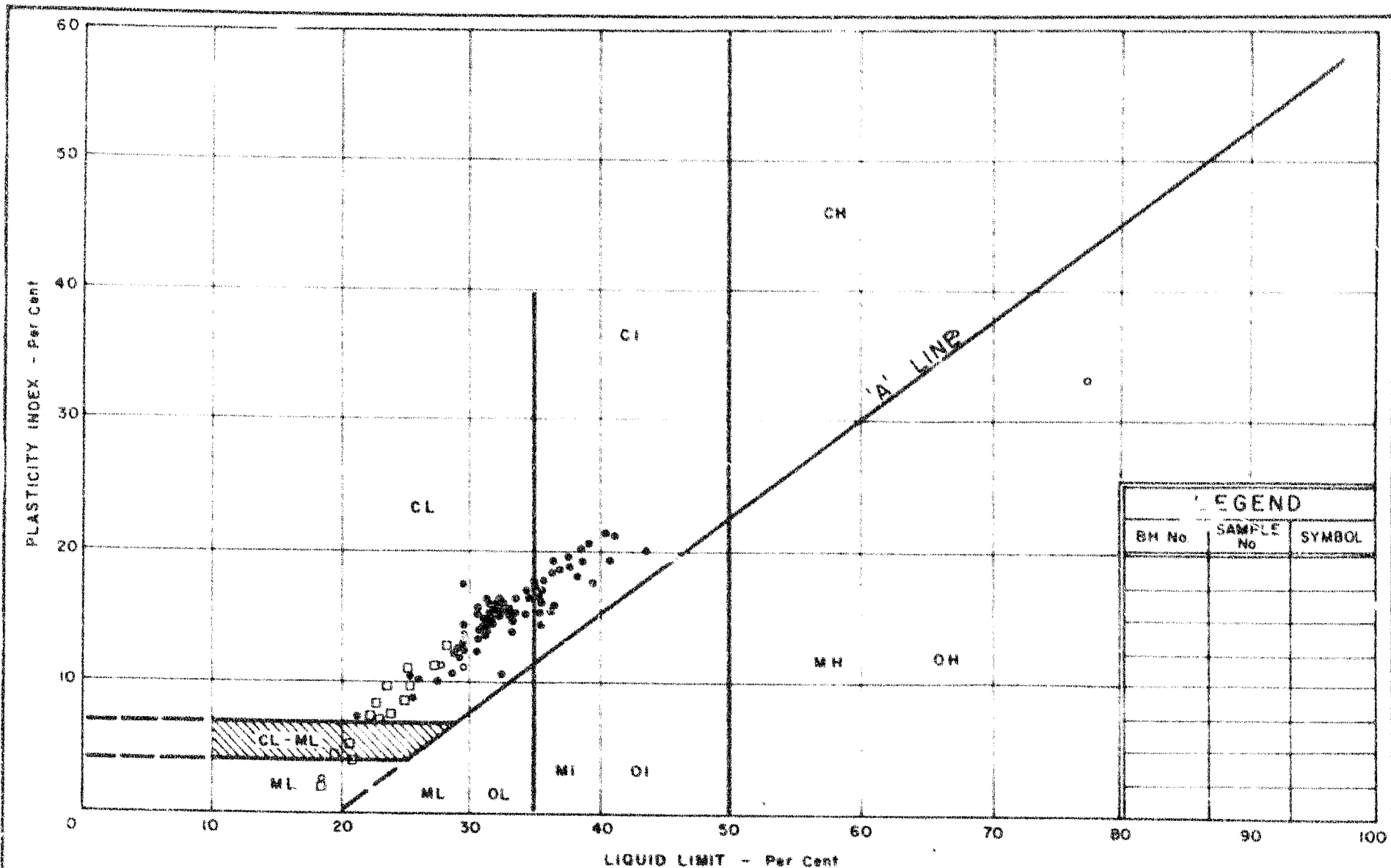
DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

GRAIN SIZE DISTRIBUTION
GLACIAL TILL
CLAYEY SILT WITH SAND & GRAVEL

WP No 10-57-03

JOB No 69-F-70

FIG. 5



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

PLASTICITY CHART

○ FILL ● CLAYEY SILT TO SILTY CLAY □ GLACIAL TILL

WP No. 10-57-03

JOB No. 69-F-70

FIG 6

VOID RATIO - PRESSURE CURVES

JOB NO. 69-F-70

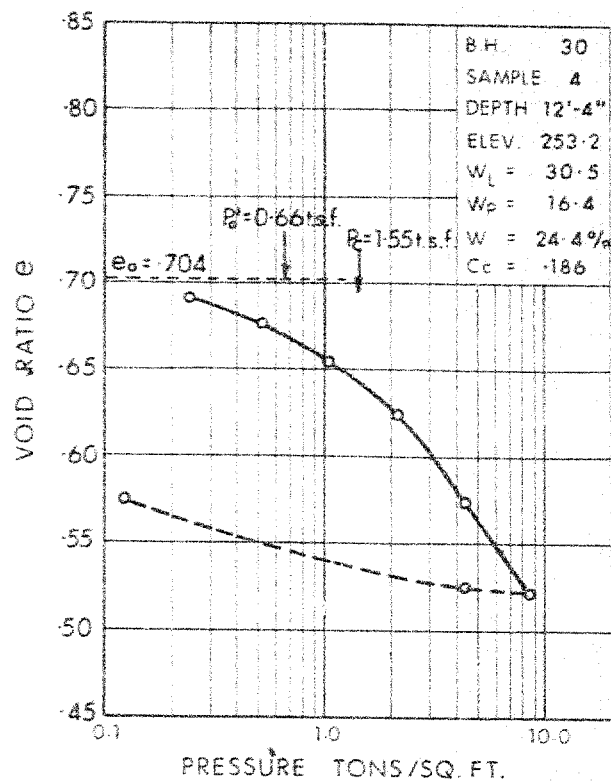
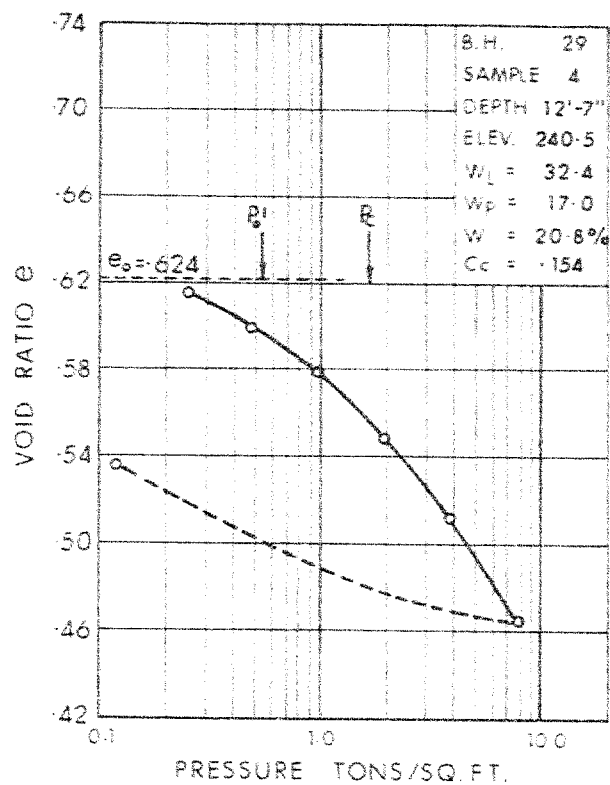
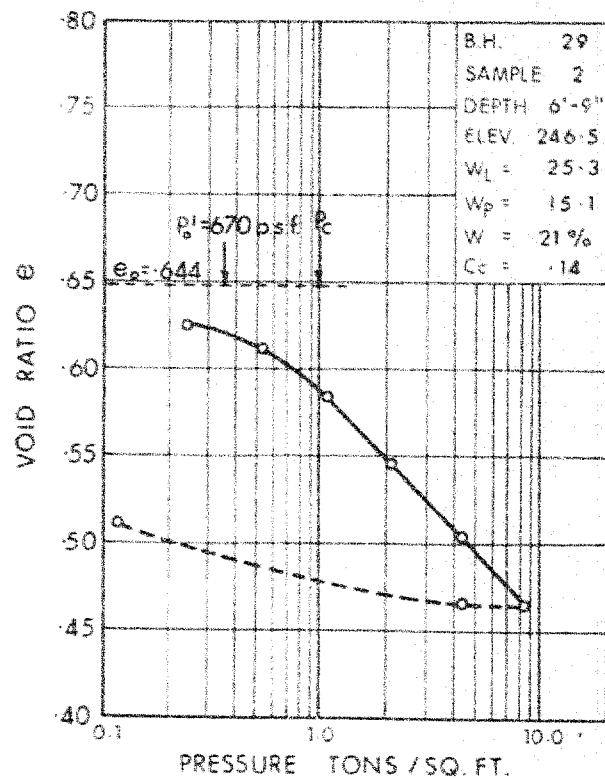
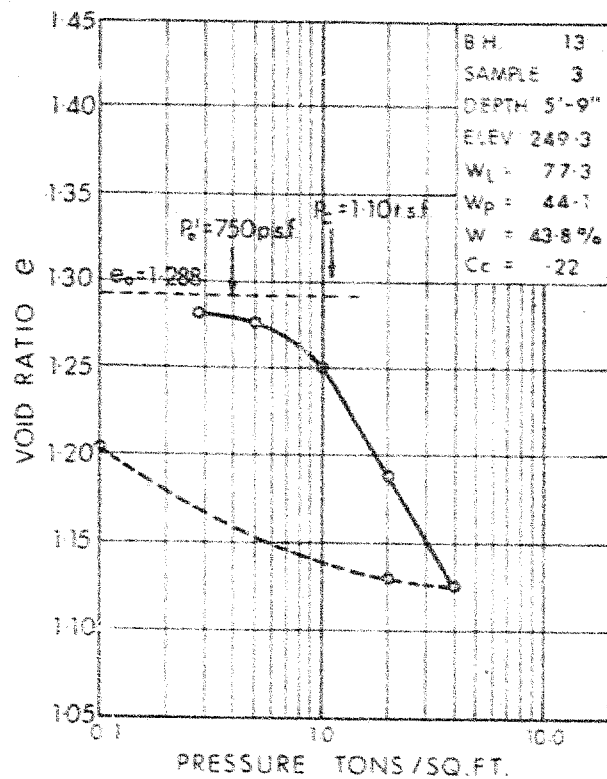


FIG. 7

VOID RATIO - PRESSURE CURVES

JOB NO. 69-F-70

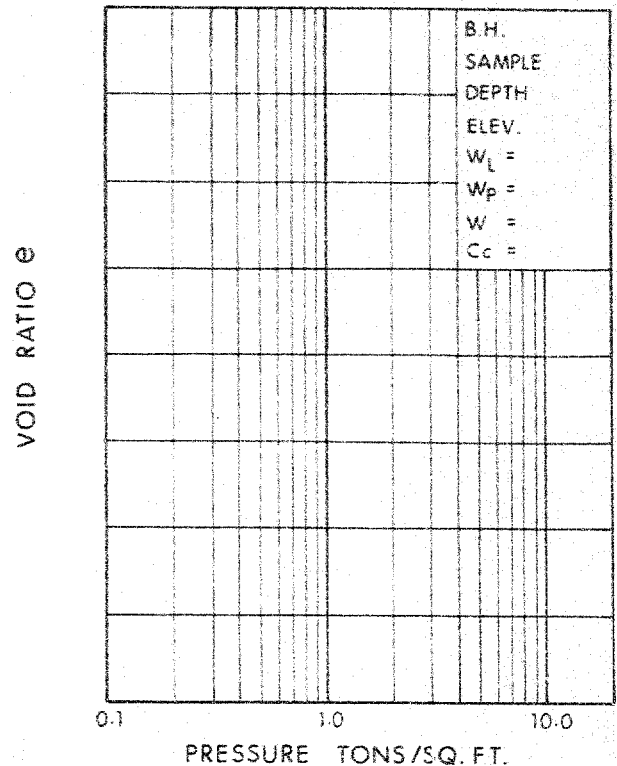
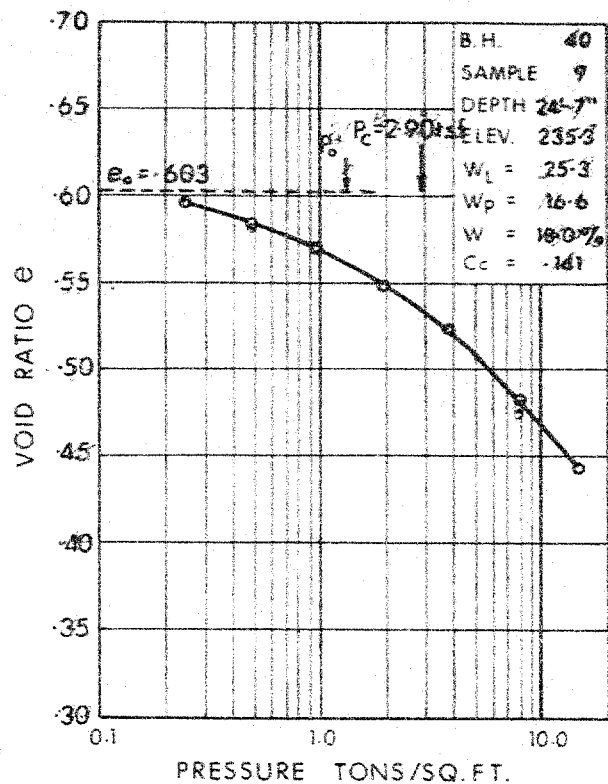
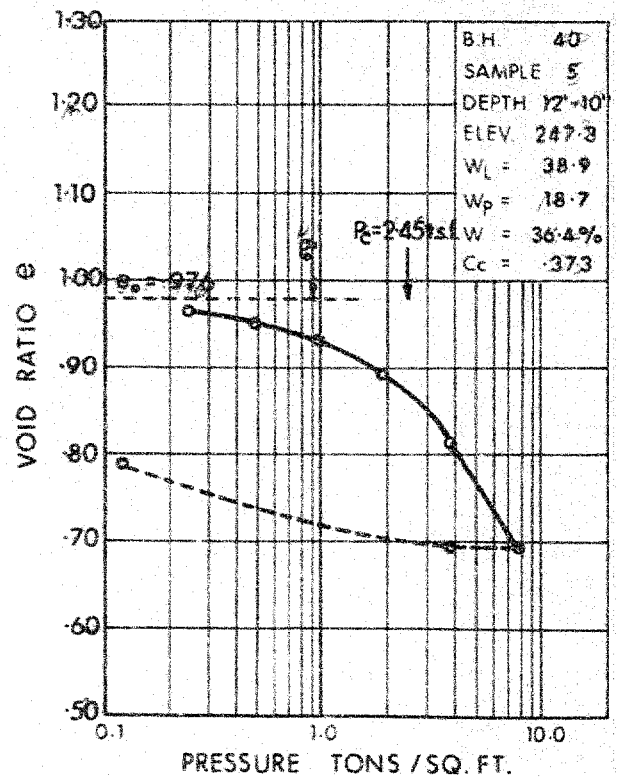
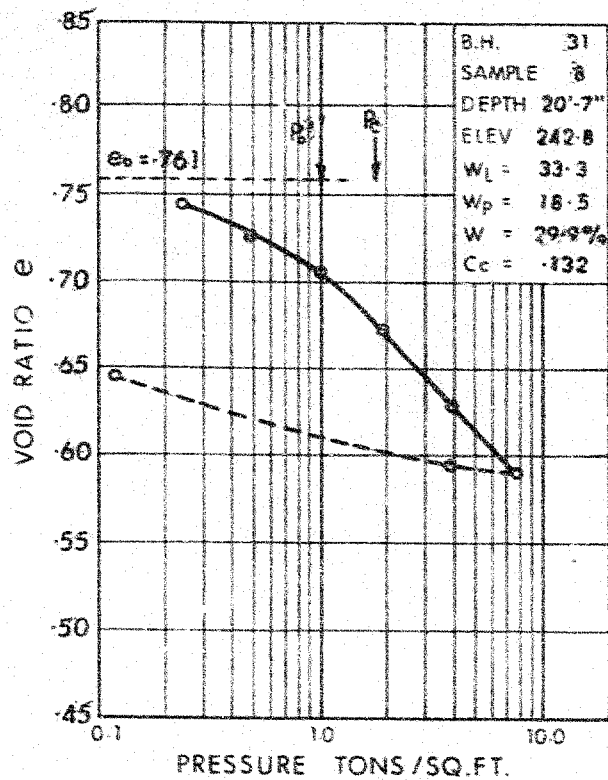


FIG. 8

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

SS	SPLIT SPOON	T.W	THINWALL OPEN
WS	WASHED SAMPLE	T.P	THINWALL PISTON
SB	SCRAPER BUCKET SAMPLE	OS	OESTERBERG SAMPLE
AS	AUGER SAMPLE	FS	FOIL SAMPLE
CS	CHUNK SAMPLE	RC	ROCK CORE
ST	SLOTTED TUBE SAMPLE		
	P.H	SAMPLE ADVANCED HYDRAULICALLY	
	P.M	SAMPLE ADVANCED MANUALLY	

SOIL TESTS

QU	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
C	UNDRAINED TRIAXIAL	F.V	FIELD VANE
QUU	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
QUd	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_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
s	SHRINKAGE LIMIT
I_L	LIQUIDITY INDEX $= \frac{w - w_p}{I_p}$
I_C	CONSISTENCY INDEX $= \frac{w_L - w}{I_p}$
e_{max}	VOID RATIO IN LOOSEST STATE
e_{min}	VOID RATIO IN DENSEST STATE
I_D	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
	RELATIVE DENSITY D_r IS ALSO USED
h	HYDRAULIC HEAD OR POTENTIAL
Q	RATE OF DISCHARGE
v	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
k	COEFFICIENT OF PERMEABILITY
j	SEEPAGE FORCE PER UNIT VOLUME
m_v	COEFFICIENT OF VOLUME CHANGE $= \frac{-\Delta e}{(1+e)\Delta\sigma}$
C_v	COEFFICIENT OF CONSOLIDATION
C_c	COMPRESSION INDEX $= \frac{\Delta e}{\Delta \log_{10} \sigma}$
T_v	TIME FACTOR $= \frac{C_v t}{d^2}$ (d, DRAINAGE PATH)
U	DEGREE OF CONSOLIDATION
τ_f	SHEAR STRENGTH
c'	EFFECTIVE COHESION
	INTERCEPT
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
c_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
$\bar{\sigma}$	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
ϵ	EAR 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

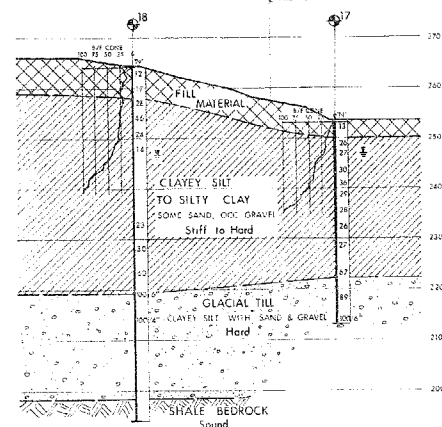
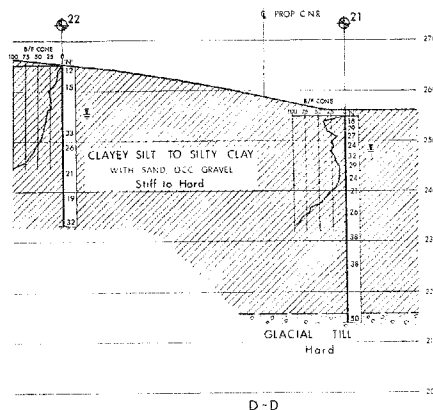
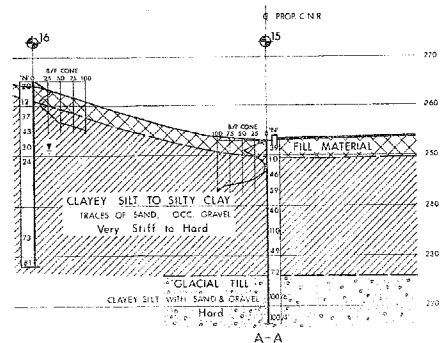
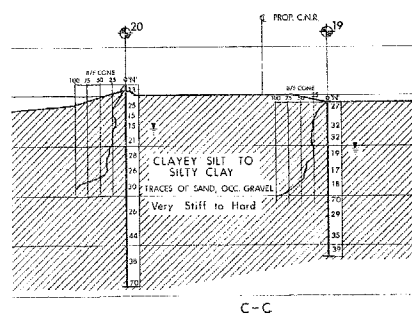
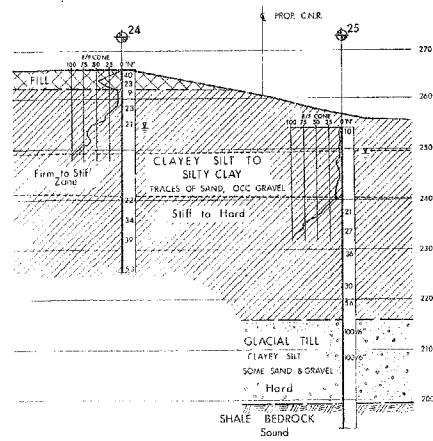
d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_0	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

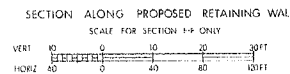
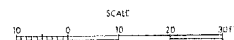
B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC IN THE FORMULA FOR BEARING CAPACITY
K_s	MODULUS OF SUBGRADE REACTION

SLOPES

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



SECTIONS







SEE DWG.

69 - F - 70A - 1

KEY PLAN
SCALE IN MILES

LEGEND

-  Bore Hole
 Cone Penetration Hole
 Bore & Cone Penetration Hole
 Water Levels established at time of field investigation. SPT 10-

NO.	ELEVATION	STATION	OFFSET

- 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.

[illegible]

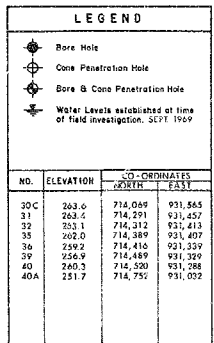
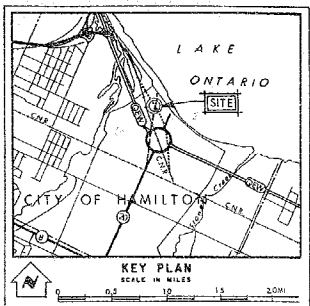
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE - FOUNDATION SECTION

RELOCATED
CANADIAN NATIONAL RAILWAYKING'S HIGHWAY NO. Q.E.W. (RECONST'N.) DIST. NO. 4

CITY OF HAMILTON - STONEY CREEK TRAFFIC CIRCLE

SECTIONS AND SOIL STRATA

SUBM'D V.K.	CHECKED <i>LM</i>	W.F. NO. 10-57-03	W.B.T. DRAWING NO.
DRAWN T.U.	CHECKED <i>LM</i>	JOB NO. 69-F-70	69-F-70A-2
DATE 3 NOV 1969		SITE NO.	BRIDGE DRAWING NO.
APPROVED <i>LM</i>		CONT NO.	



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.



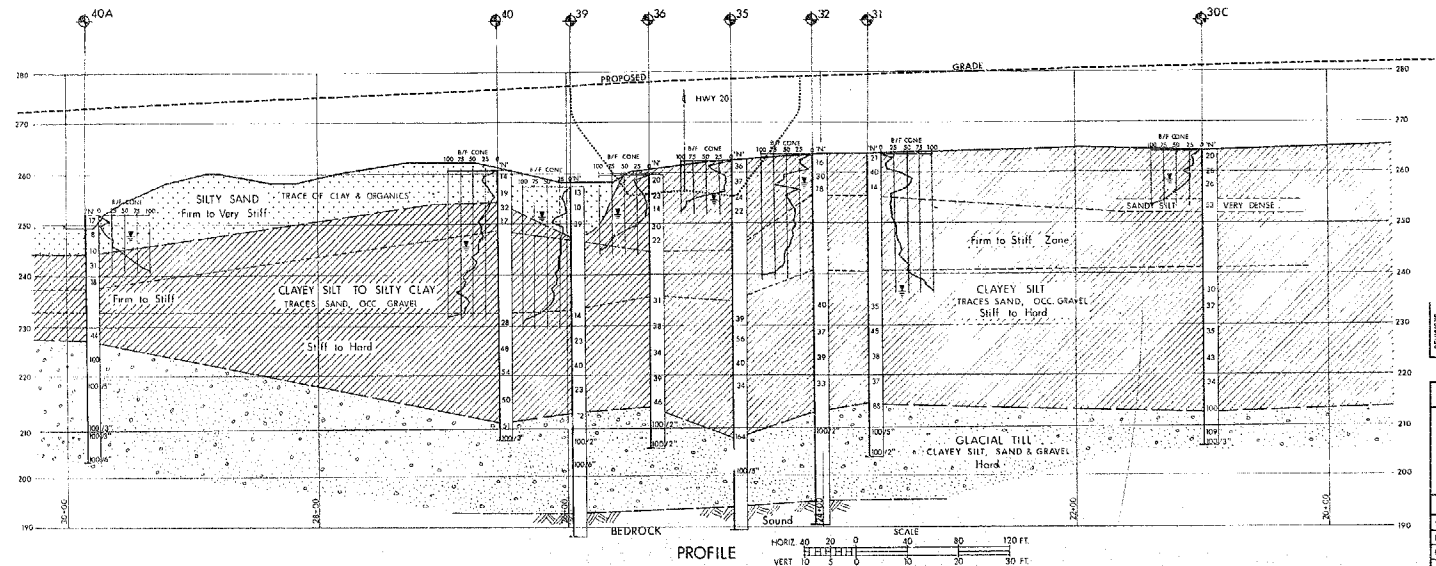
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE - FOUNDATION SECTION

RELOCATED
CANADIAN NATIONAL RAILWAY

KING'S HIGHWAY NO. Q.E.W. (RECONST'N) DIST. NO. 4
CITY OF HAMILTON - STONEY CREEK TRAFFIC CIRCLE
TWP. _____ LOT _____ CON. _____

BORE HOLE LOCATIONS AND SOIL STRATA

SUBWDWK.	CHECKED <i>ED</i>	W.P. NO. 10-57-03	M.B.T. DRAWING NO. 69-F-70B
DRAWN T.U.	CHECKED <i>JA</i>	JOB NO. 69-F-70	
DATE 4 NOV. 1969		SITE NO.	BRIDGE DRAWING NO.
APPROVED <i>W.B. Williams</i>		CONT. NO.	



MEMORANDUM

To: Mr. B. R. Davis,
Bridge Engineer,
Bridge Office,
Main Bldg.
ATTENTION: Mr. S. McComble.

FROM: Foundation Section,
Materials & Testing Office,
Room 107, Lab. Bldg.

DATE: June 19, 1970.

Our File Ref.

IN REPLY TO

SUBJECT:

Location of existing 78" I.D. Sanitary
Sewer C.N.R. Relocation, Stoney Creek
Traffic Circle Q.E.W. District #6
Hamilton
H.O. 69-110701 - H.P. 10-57-03

Further to the request of Mr. W. Melinyshyn, Regional Bridge Location Engineer, recently this section completed a field investigation to locate the existing 78" Ø sanitary sewer at the Stoney Creek Traffic Circle in the vicinity of the proposed C.N.R. subway. A drawing #69-F-704-3 showing the location of sewer is enclosed and this should be included with our foundation report submitted to you on November 12th 1969 for the C.N.R. subway at Q.E.W.

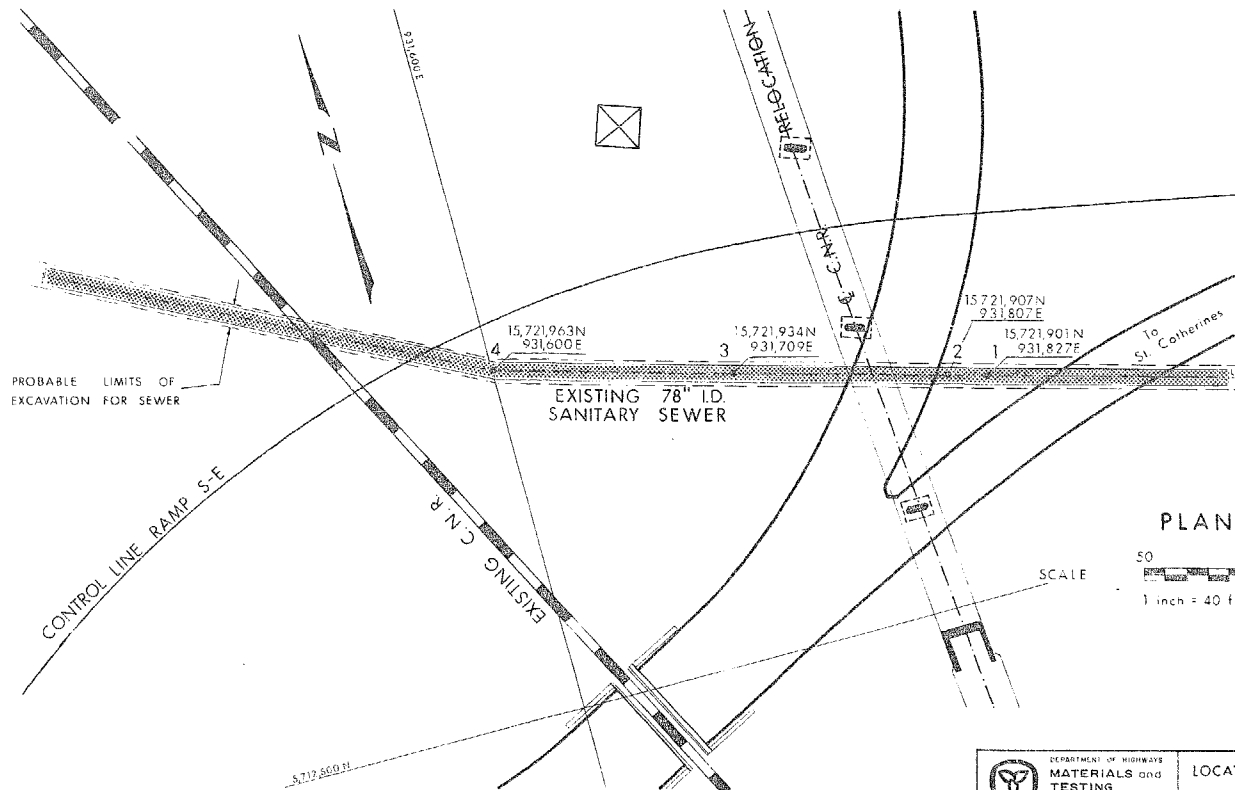
If you need any further information with regard to this, please contact our office.

W. Devata
W. Devata,
SUPERVISING FOUNDATION ENGINEER
For:
A. C. Stermac,
PRINCIPAL FOUNDATION ENGINEER

AGG/hnd
Attach.

cc: Messrs. B. R. Davis (2)
H. A. Tregaskes
D. W. Farren
C. K. Hunter (2)
R. Greenland
W. S. Melinyshyn
T. J. Kovich
B. A. Singh

✓ Foundation Files
General Files



PLAN

SCALE

50 0 50 100 FT.

1 inch = 40 feet



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

LOCATION OF EXISTING 78" I.D. SANITARY SEWER
C.N.R. RELOCATION
STONEY CREEK TRAFFIC CIRCLE

DATE 12 JUNE 1970

W.P. NO. 10-57-03

DRAWING NO. 69-F-70A-3

MEMORANDUM

To: Mr. B. R. Davis,
Bridge Engineer,
Bridge Office,
Admin. Bldg.

FROM: Foundation Section,
Materials & Testing Office,
Room 107, Lab. Bldg.

ATTENTION: Mr. S. McCombie

DATE: February 26, 1970

OUR FILE REF.

IN REPLY TO

SUBJECT: Additional Stability Analyses for the Revised Scheme -
Proposed Subway Structure at the Crossing of the Q.E.W.
And the Relocated C.N.R. -- Stoney Creek Traffic Circle
District No. 4 (Hamilton)
W.J. 69-P-70 W.P. 10-57-03

A meeting was held on November 18, 1969, at M. M. Dillon Ltd. to discuss the various foundation problems at the proposed subway locations, namely: C.N.R. and Q.E.W. crossing, and C.N.R. and Hwy. 20 crossing. Discussion took place between representatives of the Department, C.N.R., and the highway design consultants - (M. M. Dillon Ltd.). The discussions resulted in the following conclusions:

- i) C.N.R. relocation and bridge structure will be built to contain only one track (C.N.R. and Q.E.W. structure).
- ii) Proposed grade for Ramp E.-N.S. can be revised. This revision will result in reducing the height of fills for the North approach embankments of the C.N.R. subway at Q.E.W.

The Foundation Section recently received the necessary drawings incorporating the aforementioned revisions from M. M. Dillon Ltd. Taking these revisions into account, we have carried out further stability analyses in terms of total stresses by the use of the electronic computer. The geometric sections at the approaches, and the soil properties for the fill and subsoil, assumed for computational purposes, are presented on Fig. 2A. The design consultant suggested that the toe of the railway fill should not be closer than 5 ft. from the face of the curb of Ramp E.-N.S. (future). The results of the analyses, presented on Fig. 2A, are summarized as follows:

The critical condition, as far as stability is concerned, is the North approach embankment (Q.E.W. - C.N.R. subway structure). The North approach, in the vicinity of the structure, will have a height of 22 ft. in the longitudinal, as well as 21 ft. in the transverse direction. The results of the computations indicate that the factor of safety with respect to stability in the longitudinal and transverse directions, will be of the order of 1.3 or

Mr. B. R. Davis,
Bridge Engineer,
Bridge Office, Admin. Bldg.
Attn: Mr. S. McCombie

2

February 26, 1970

Re: Additional Stability Analyses for the Revised Scheme -
Proposed Subway Structure at the Crossing of the Q.E.W.
And the Relocated C.N.R. - Stoney Creek Traffic Circle
District No. 4 (Hamilton) - W.J. 69-F-70, W.P. 10-57-03

more for the proposed geometry as shown on Fig. 2A. These factors of safety indicate that the proposed fills with berms as required (Ref. Fig. 2A) will be stable, even if they are constructed with locally available earth material. The factors of safety for various sections using lightweight fill, are also given on Fig. 2A. This memo, together with the enclosed drawing (Fig. 2A), should be included with our Foundation Report W.J. 69-F-70.

We believe that the aforementioned information will be adequate for your design requirements. If you need any further information with regard to this project, please contact our office.

MD/MdeF
Attach.

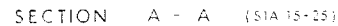
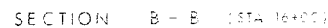
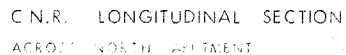
cc: Messrs. B. R. Davis
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W. S. Melinyshyn (2)
T. J. Kovich
B. A. Singh
M. M. Dillon Ltd.

Foundations Files ✓
Gen. Files

M. Devata
M. Devata,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.



PROPOSED SCHEME
$ \begin{array}{l} R_1 = 1.1 \\ R_2 = 1.2 \\ R_3 = 1.3 \end{array} $



SCALE 1" = 10'



SUMMARIZED RESULTS OF STABILITY ANALYSES
REVISED SCHEME
NORTH APPROACH EMBANKMENT

W P 10-57-03

DIST. 4 JOB 69 -- F -- 70

DATE: 20 JAN. 1970

APPROVED

Fig. NO 2 A

HIGHWAY 20 - Q.E.W. INTERCHANGE

AT STONEY CREEK

W.P. 10-57

Minutes of meeting held in Board Room of M. M. Dillon Limited at
2:00 P.M., Tuesday 18 November 1969

PRESENT:-

C. S. Grebski)	D.H.O.
J. F. Walshe)	
W. S. Melinyshyn)	
M. Devata)	
C. S. Dunn)	C.N.R.
J. W. Stzelecki)	
W. L. Higgins)	
P. A. Weber)	
L. Peckover)	
F. Z. Sobolak)	M.M.D.
J. Kozel)	
I. Hausmanis)	

ITEM

ACTION BY

1. C.N.R. relocation and bridge structures will be built to contain only one track. There is a remote possibility of another track being required in future.
2. Alternate designs for C.N.R. structure were discussed. Meeting agreed that C.N.R. would examine designs of structure in:
 - a) reinforced concrete
 - b) composite construction
 - c) reinforced and prestressed concrete

CNR
DHO

D.H.O. would prefer a design in concrete. This type of design would blend best with the many highway bridges to be built in the immediate area.

CNR

(continued)

ITEM

ACTION BY

3. C.N.R. profile was discussed. C.N.R. have approached Allen Industries as to location of their rail access. Allen Industries are agreeable to relocating their access location. This will enable railway profile to be raised if required. M. M. Dillon Limited feel that profile can be adjusted such that bridge depth (base of rail to underside of bridge deck) can approach 8'-0". A 15'-6" clearance is required at all pavement crossings.

Alignment of C.N.R. relocation is final, all coordinates are shown on site plan. Ties to D.H.O. monuments will be provided for layout.

CNR
MMD

4. Soil and foundations were discussed. M. Devata presented foundation reports for both C.N.R. structures. A detailed examination of soil and foundation problems followed.

Department of Highways Foundation and Bridge Offices would prefer a 3-span arrangement with spill through abutments for C.N.R. overpass at Highway 20.

Soils conditions in this area necessitate use of piles driven to refusal. Settlement of approach fills is anticipated. The use of light-weight fill material was examined.

Soils problems at C.N.R. - Q.E.W. overpass were examined. Soils report shows a layer of weak material at north abutment of bridge. Slip type failure is anticipated if approach fills constructed as planned. M. Devata presented two remedial coarces:

- a) Use of retaining wall up to 45' in height to contain the weak layer.
- b) Extending bridge spans for 300 feet to pass over weak layer.

J. Kozel proposed to substitute the end 90' span by two spans (75' each approximately) providing possibility for a perched north abutment and a

continued

ITEM

ACTION BY

4 continued

- lower retaining wall along ramp E-NS. M. M. Dillon Limited will send M. Devata elevations and typical sections along ramp E-N.S. and proposed location of retaining wall. The use of light-weight fill material with a 1.75:1 or 2:1 slope was also recommended. M. Devata will examine this proposal and forward recommendations to C.N.R. DHO
MMD
5. C.N.R. will review foundation conditions and make design recommendations. CNR
6. D.H.O. should approach Canadian Transport Commission. DHO
7. Scheduling was discussed. C.N.R. feel they will be able to have their contract complete and be off the site by July 1971. CNR
8. C.N.R. will prepare their property requirements and send them to D.H.O. CNR
9. An accurate location of the existing 78 inch sanitary sewer on the south side of Q.E.W. passing under bridge abutments is required. DHO
10. Location of Telegraph lines in construction area is required. If relocation is necessary underground relocation would be preferred. CNR
MMD
11. C.N.R. will produce General Arrangements drawings for both structures as soon as possible and forward them to D.H.O. This will enable profiles and property requirements to be finalized. CNR

IH:mab

I. Hausmanis,

DISTRIBUTION: All Present

Also: J. Bonsall
P. Weber
H. Adams

D. Smith
M. Robinson
W. C. Friedmann
G. McMillan

MEMORANDUM

To: Mr. A.G. Stermac,
Principal Foundation Engineer,
Room 107,
Lab. Building.

ATTENTION: M. DEVATA
Mr. [REDACTED].

Our File Ref.

FROM: W.S. Melinyshyn,
Bridge Office.

DATE: August 12th, 1969.

IN REPLY TO

SUBJECT: W.P. 10-57-2, -3, -4,
Hwy. 20 Interchange at Stoney Creek,
Site 36-144, Hwy. 20 over Q.E.W.,
Site 36-228, CNR Subway over Q.E.W.,
Site 36-229, CNR Subway over Hwy. 20,
District No. 4.

→ 69-F-70
→ 69-F-71

Herewith are two prints of drawing 6465-01-R1 and 6465-01-R2, on which the probable location of footings are shown for the above projects.

Please arrange for a foundation investigation of sufficient scope to enable us to proceed with the design.

Should difficulties be found in locating the proposed locations we suggest that you request assistance from J. Barclay, Reg. Sup. of Surveys, Local 3559.

Also attached for your information is a copy of BA 735 a soils investigation carried out in the area during 1958.

WK/cew
Encl.

W. Killin
W. Killin,
for:
W.S. Melinyshyn,
REGIONAL BRIDGE LOCATION ENGINEER.