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**PRELIMINARY FOUNDATION
INVESTIGATION AND DESIGN REPORT
NORTH CANAL / CANAL ROAD BRIDGES
STRUCTURE SITE 30-334N/S
HIGHWAY 400 WIDENING
FROM YORK / SIMCOE BOUNDARY
TO 1 KM SOUTH OF HIGHWAY 89
G.W.P. 40-00-00**

Submitted to:

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PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
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Appendix A Records of Boreholes and Test Results – 1970 Subsurface Investigation

1.0 INTRODUCTION

Golder Associates Ltd. has been retained by URS Cole, Sherman (Cole, Sherman) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the ultimate widening of Highway 400 from the York / Simcoe Boundary northerly to 1 km south of Highway 89, in Simcoe County, Ontario. Foundation engineering services are required for the widening and / or replacement of six existing overpass and underpass structures, as well as four structural culverts.

This report addresses the widening and / or replacement of the North Canal / Canal Road bridges. Existing subsurface data for this site from a report prepared by the Department of Highways, Ontario, referenced below, were used to determine the subsurface conditions for this preliminary design study:

- “Foundation Investigation Report for Proposed Extensions to the Overpass Structures at the Crossing of Highway 400 and the North Canal Road and Drainage Canal, Township of West Gwillimbury, County of Simcoe”, dated December 1970 – GEOCRE File No. 31D-30.

The terms of reference for the scope of work are outlined in the MTO's Request for Quotation (RFQ) dated September 5, 2000, and in Golder Associates' subsequent letters dated December 13, 2000 and February 15, 2001.

2.0 SITE DESCRIPTION

The existing twin North Canal and Canal Road bridge structures are located almost 3 km north of the Highway 9 interchange, in the Township of West Gwillimbury, County of Simcoe. The MTO has designated these bridges as Structure Site Nos. 30-334N and 30-334S.

The original ground surface at this site was between Elevation 220 m and 221 m. At the twin bridge structures, the North Canal channel is about 20 m wide, with its base at about Elevation 218 m; steel sheet piling is present along the sides of the channel. Canal Road runs immediately south of the drainage canal, with its profile grade at about Elevation 223.5 m. A low retaining wall is present along the south side of Canal Road, at the toe of the foreslope. Highway 400 has been constructed on embankment fill, with its grade rising northward from about Elevation 226.7 m to 228.5 m within the limits of these structures. The existing Highway 400 approach embankments are up to about 7 m in height, with side slopes ranging from about 2.5 to 3 horizontal to 1 vertical (2.5H:1V to 3H:1V).

The existing six-span twin bridges were originally constructed in 1948. The “perched” abutments and piers are supported on driven timber piles. The twin bridges were widened outward by approximately 8 m in 1971, under Contract 71-12. The widened portions of the structures have been supported on driven steel H-piles. At the south and north abutments, the tops of the pile caps are at about Elevation 224.9 m and 226.6 m, respectively. The tops of the pile caps at the five pier locations are at about Elevation 219 m. The middle pier of both structures is located within the North Canal channel.

These foundation conditions were determined from the general arrangement drawing for Contract 71-12, which was provided by Morrison Hershfield, the structural designers for this preliminary study.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at this site by the Department of Highways, Ontario (DHO). In October and November of 1970, seven boreholes were advanced at the site to between 20 m and 40 m depth. Boreholes 1, 2, 3 and 7 were advanced on the south side of the canal, and Boreholes 4, 5 and 6 were advanced on the north side.

Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. Thin-walled tube samples were also obtained. In-situ vane shear testing was carried out to determine the undrained shear strength of cohesive soils where possible. The groundwater conditions in the open boreholes were observed during and following the drilling operations. Laboratory testing, consisting of water contents, Atterberg Limits, bulk unit weights, grain size distributions, undrained shear strengths, and oedometer testing, were carried out on selected soil samples.

The borehole locations and elevations, referenced to the geodetic datum, were established by the DHO. Approximate northing and easting coordinates consistent with the MTM NAD83 survey system, currently in use on this project, have been determined based on the borehole locations given in the 1970 report. The approximate borehole locations and northing and easting coordinates are shown on the attached Drawing 1.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

This 15 km section of Highway 400 traverses, from south to north, the following physiographic regions as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, Third Edition, 1984): the Simcoe Lowlands; the Schomberg Clay Plains; the Peterborough Drumlin Field; and a second lobe of the Simcoe Lowlands. Along Highway 400, the southern lobe of the Simcoe Lowlands is present at the North Canal / Canal Road site. The Schomberg Clay Plains are present north of this site, to 2 km north of Simcoe Road 88 (formerly Highway 88). The Peterborough Drumlin Field extends from 2 km north of Simcoe Road 88 to about 3 km south of Highway 89. The northern lobe of the Simcoe Lowlands extends from about 3 km south of Highway 89 to beyond the northern limit of this project.

The surficial soils in the Schomberg Clay Plains consist primarily of varved clay and silt deposits. These varved deposits overlie till with drumlins as found in the Peterborough Drumlin Field. The drumlins (glacially-shaped hills) are completely or partially buried by the clay and silt deposits, depending on the size of the drumlin. The varved clay and silt deposits are typically about 5 m thick, although deeper deposits have been found in some locations.

The surficial soils in the Peterborough Drumlin Field consist primarily of gravelly sand till or sand and gravel deposits. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins.

Along Highway 400, the Simcoe Lowlands include the Holland River valley, the shores of Kempenfelt Bay, the Nottawasaga River, and Innisfil Creek. The Holland River valley at the southern end of this project extends southwest from Cook Bay, at the south end of Lake Simcoe; it was once a shallow extension of the lake. The floor of the valley is covered by extensive deposits of loose silts and soft clays, which overlie a till sheet. In localized areas, these silts and clays are overlain by a thin, poorly graded sand of deltaic origin. Because the valley is depressed and poorly drained, a surficial cover of peat has formed in many areas. The surficial soils of the northern lobe of the Simcoe Lowlands consist primarily of sand, although silt, clay or peat may be found in low-lying areas.

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes, and the results of the laboratory testing carried out on selected soil samples, are given on the Record of Borehole sheets and Figures 1 to 5 contained in Appendix A. The stratigraphic boundaries shown

on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

Boreholes 1, 2, 3 and 7 were advanced on the south side of the canal and Canal Road, and Boreholes 4, 5 and 6 were advanced on the north side of the canal. The approximate locations and ground surface elevations for these borings are shown on the attached Drawing 1.

In summary, the subsoils below the Highway 400 embankment fill consist of a 24 m to 32 m thick clayey silt deposit, containing a 2.5 m to 6 m thick silty sand interlayer. In localized areas, the clayey silt is overlain by a loose to compact deposit of silty sand. The clayey silt is underlain by a very dense / hard sand and silt to clayey silt till deposit; the top of this till deposit was encountered between Elevations 187 m and 190 m. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Embankment Fill

All of the boreholes penetrated the existing Highway 400 embankment fill. The fill is comprised of clayey silt containing trace sand and gravel. Grain size distribution test results are plotted on Figure 1 of Appendix A.

The fill consistency is variable, with measured Standard Penetration Test (SPT) 'N' values ranging from 5 to 40 blows per 0.3 m of penetration. In-situ and laboratory vane shear strength testing and undrained triaxial testing were carried out in the "softer" zones of the fill. This testing measured undrained shear strengths ranging from about 25 kPa to greater than 100 kPa, but typically about 40 kPa. The SPT and shear strength testing indicate that the clayey silt fill has a firm to very stiff consistency.

4.2.2 Surficial Silty Sand to Sandy Silt

A 1 m to 3 m thick surficial layer of silty sand to sandy silt was encountered below the embankment fill in several of the boreholes; it was not encountered in Boreholes 2 and 6 on the west side of Highway 400. The surficial silty sand to sand silt layer contains trace quantities of organic matter; in Borehole 3, a 0.3 m thick organic layer was encountered within the silty sand. Grain size test results obtained for this surficial deposit are plotted on Figure 3 of Appendix A. This layer has a loose to dense relative density, with measured SPT 'N' values ranging from 6 to 34 blows per 0.3 m of penetration, with an average of 16 blows per 0.3 m of penetration.

4.2.3 Clayey Silt

A deposit of clayey silt underlies the embankment fill and surficial layer of sand / silt, where present. The clayey silt ranges from about 24 m to 32 m in thickness; its base was encountered at about Elevation 187 m to 188 m on the south side of the Canal, and at about Elevation 190 m to 193.5 m on the north side of the Canal.

The clayey silt generally contains trace quantities of sand, with lenses, seams and interlayers of silty sand, between 6 mm and 300 mm in thickness, throughout the stratum. A 450 mm diameter boulder was encountered within the clayey silt at about Elevation 195 m, in Borehole 4. An envelope of grain size distribution test results obtained from samples of the clayey silt is shown on Figure 2 in Appendix A. A 2.5 m to 6 m thick interlayer of silty sand to sand silt is present within the deposit; this interlayer is discussed further in Section 4.2.4.

The upper and lower portions of the clayey silt deposit differ in consistency. Above the silty sand to sandy silt interlayer, the SPT results ranged from 6 to 11 blows per 0.3 m of penetration, and in-situ vane, laboratory vane and triaxial testing measured undrained shear strengths ranging from about 30 kPa to greater than 100 kPa. Below the silty sand to sandy silt interlayer, the SPT results ranged from 12 to 43 blows per 0.3 m of penetration, with an average 'N' value of about 35 blows per 0.3 m of penetration. The results of the Standard Penetration, in-situ and laboratory shear strength testing indicate that the upper portion of the clayey silt generally has a firm to stiff consistency, while the lower clayey silt has a stiff to hard, but generally very stiff to hard, consistency.

The natural moisture contents measured on samples of the clayey silt ranged from 17 to 28 per cent. Atterberg Limits testing measured plastic limits ranging from about 13 to 20 per cent. Liquid limits measured on samples of the upper and lower clayey silt ranged from 19 to 30 per cent and 22 to 39 per cent, respectively. The plasticity indices ranged from about 6 to 11 per cent in the upper clayey silt, and from about 7 to 20 per cent in the lower clayey silt. The results of the Atterberg Limits testing are plotted on Figure 5 in Appendix A; these results indicate that the clayey silt is inorganic and generally of low plasticity.

4.2.4 Silty Sand to Sandy Silt Interlayer

A 2.5 m to 6 m thick interlayer of silty sand to sandy silt was encountered between the upper and lower clayey silt deposits. Thin clayey silt seams, up to about 12 mm in thickness, are present throughout the interlayer. Grain size distribution test results obtained on samples of the silty sand to sandy silt interlayer are shown on Figure 3 in Appendix A.

In the boreholes, the top of the interlayer was encountered between about Elevation 212 m and 210.5 m; the surface of the interlayer was lower in the immediate vicinity of the Canal, and higher to the north and south. The interlayer has a compact to very dense relative density, based on measured SPT 'N' values ranging from 21 to 157 blows per 0.3 m of penetration. Typically the SPT 'N' values range from 30 to 50 blows per 0.3 m, indicating a dense relative density.

4.2.5 Sand and Silt Till to Clayey Silt Till

The clayey silt deposit is underlain by a till deposit which ranges in composition from sand and silt to clayey silt. The deeper boreholes terminated in this stratum, with a maximum penetration of about 5 m in Borehole 7. The top of this till stratum was encountered at about Elevation 187 m to 188 m in Boreholes 2, 3 and 7 on the south side of the Canal, and at about Elevation 190 m to 193.5 m in Boreholes 4 and 5, on the north side of the Canal.

The glacial till is primarily composed of silt and sand containing variable quantities of clay and gravel. In some areas, however, the till is plastic, consisting of clayey silt containing sand and trace gravel. Grain size distribution curves are plotted on Figure 4 in Appendix A.

The measured SPT 'N' values range from 27 blows to greater than 100 blows per 0.3 m of penetration, but are typically greater than 75 blows per 0.3 m of penetration, indicative of a very dense relative density / hard consistency.

4.3 Groundwater Conditions

The groundwater conditions were observed in the open boreholes in the 1970 investigation; these conditions are indicated on the Records of Boreholes contained in Appendix A. In Boreholes 2, 3 and 7 in the vicinity of the canal, the water levels were similar to the canal water level, at about Elevation 218 m to 218.5 m. In Boreholes 1 and 6 drilled further away from the canal through the embankment fill, the water levels in the open boreholes were observed at Elevations 222.6 m and 225 m, respectively. These levels are above the ground surface elevation (and therefore above the base of the fill) and it is likely that the high levels are related to artesian conditions in the silty sand / sandy silt interlayer within the clayey silt deposit. Boreholes 1 and 6 just penetrated this interlayer and were terminated at about Elevations 204 m and 208 m, respectively.

Artesian groundwater pressures were encountered in Boreholes 4 and 5 at about Elevation 195 m. It is indicated that the artesian conditions are associated with granular interlayers at depth within


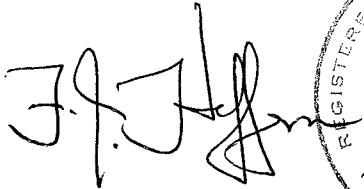
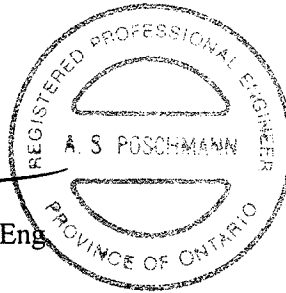
the clayey silt soil and possibly with the underlying sand / silt till. The artesian head is indicated to be at Elevation 225.1 m in Borehole 4 and at Elevation 223.1 m in Borehole 5.

It should be noted that groundwater levels are expected to fluctuate seasonally and are expected to be higher during wet periods of the year.

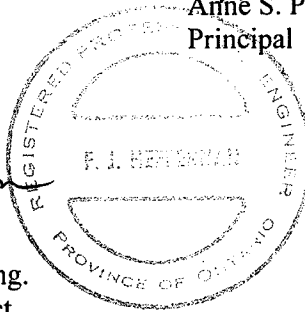
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PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
NORTH CANAL / CANAL ROAD BRIDGES
STRUCTURE SITE 30-334N/S
HIGHWAY 400 WIDENING
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5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides preliminary foundation design recommendations for the widening and / or replacement of the existing North Canal / Canal Road bridge structures, associated with the widening of Highway 400. The recommendations are preliminary only and are based on interpretation of the factual data obtained from a limited number of boreholes advanced during a 1970 subsurface investigation at this site. The interpretation and recommendations provided are intended for planning purposes only, to provide the information necessary at this stage of the study. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the planning of the project. Further foundation investigation will be required at this bridge site as part of the detailed design stage of the project.

It is understood that Highway 400 will be widened from its existing six-lane configuration to an ultimate configuration of ten lanes. The primary options under consideration involve widening into the median, or using a 22 m wide open median with widening on the outside of the existing highway. Depending on which option is adopted, it is expected that the existing highway will be widened by between 10 m and 29 m. Widening or replacement of the existing North Canal / Canal Road bridges will, therefore, be necessary.

At the structure site, the base of the North Canal channel is at about Elevation 218 m. Canal Road runs immediately south of the drainage canal, with its profile grade at about Elevation 223.5 m. Highway 400 has been constructed on embankment fill, with its grade at about Elevation 226.7 m to 228.5 m, rising northward, within the limits of the structures. The existing Highway 400 approach embankments are up to about 7 m in height, with side slopes ranging from about 2.5 to 3 horizontal to 1 vertical (2.5H:1V to 3H:1V).

Based on the general arrangement and layout drawings for the existing six-span structures, the “perched” abutments and piers are supported on deep foundations. At the south and north abutments, the tops of the pile caps are at about Elevation 224.9 m and 226.6 m, respectively, and the tops of the pile caps at the pier locations are at about Elevation 219 m. The original portions of the bridges are supported on driven timber piles; it is considered likely that these piles are founded within the dense to very dense silty sand interlayer encountered at about Elevation 210 m. The outside portions of both bridges, which were constructed under a 1971 widening contract, are supported on steel H-piles; the note on the General Plan drawing indicates that the piles were to be driven to practical refusal in the glacial till. This implies that the piles would be founded on the very dense sand and silt till to hard clayey silt till deposit encountered at about Elevations 193.5 m to 187 m.

5.2 Bridge Foundation Options

The native subsoils encountered in the boreholes advanced during the 1970 investigation consist of surficial sands and silts overlying an extensive deposit of clayey silt, in turn underlain by a very dense sand and silt till to hard clayey silt till deposit.

The presence of relatively weak clayey silt at shallow depth precludes the use of spread footings for the new or widened structures. Therefore, it is recommended that the foundation elements for the new or widened structures be supported on steel H-piles driven to found within the very dense sand and silt till / hard clayey silt till deposit which underlies the site. Preliminary recommendations for driven steel H-pile foundations are provided in the following section.

5.3 Driven Steel H-Piles

Based on the results of the 1970 boreholes, the top of the till deposit is at about Elevation 187 m to 188 m on the south side of the Canal, and at about Elevation 190 m to 193.5 m on the north side of the Canal. Based on the measured Standard Penetration Test (SPT) results, it is unlikely that the piles would penetrate the till by more than about 2 m before reaching practical refusal. Assuming that the new pile caps are placed at the same elevations as the existing pile caps, the pier piles would be about 27 m to 33 m long, and the abutment piles would be about 35 m to 40 m long.

In order to minimize the potential for significant vibrations impacting the existing bridges, consideration should be given to driving the piles to a reduced set just within the sand and silt till to clayey silt till founding stratum. This option should be considered where the works involve widening rather than replacement of the bridges. If a reduced capacity and set are adopted, the pier piles would be about 25 m to 31 m long and the abutment piles would be about 33 m to 38 m long.

It is noted that additional borehole investigation will be required at the detailed design stage in order to confirm the pile tip elevations at the new structure locations depending on the configuration of the widening.

5.3.1 Axial Geotechnical Resistance

For preliminary design, and subject to confirmation following the detailed design stage of subsurface investigation, the factored axial resistance at ULS for steel HP 310 x 110 piles driven

to found just within the sand and silt till to clayey silt till deposit may be taken as 1,100 kN. A factored axial resistance of 1,600 kN may be used for piles driven to practical refusal. The corresponding axial resistances at SLS for 25 mm of settlement may be taken as 800 kN and 1,100 kN, respectively.

To achieve the above design resistance of 1,600 kN at ULS, the piles should be driven to a final set of no less than 15 blows per 25 mm of penetration using a hammer with rated energy of about 50 kJ, and not exceeding 60 kJ. Provision should be made to re-tap selected piles to confirm the set after adjacent piles have been driven, in accordance with MTO's current Special Provision. To achieve the lower design resistance of 1,100 kN at ULS, the piles should be driven to at least Elevation 190.5 m on the north side of the Canal and to at least Elevation 187 m on the south side of the Canal.

The additional load due to widened embankment construction outside of the existing highway platform will induce consolidation settlement of the 5 m to 8.5 m thick, firm to stiff upper clayey silt soils under the widened portions of the embankments. At the abutments and at piers which are located within the embankments, this consolidation will induce a downward movement of the soils adjacent to the piles, and negative skin friction will develop along the portion of the pile shaft embedded within the clayey silt. The negative skin friction load on a single pile may be taken as 150 kN. This represents an unfactored value; an appropriate load factor would have to be applied in the structural analysis. If widening is carried out between the existing structures, only minor filling will be necessary and negative skin friction will not be a factor as the settlement will be less than the elastic compression of the piles.

To minimize the amount of negative skin friction that may develop on a pile, consideration could be given to placing new approach embankment fill for the widening as early as possible to allow as much settlement as possible to occur prior to the driving of the piles. The embankment could also be surcharged to increase the magnitude of settlement prior to pile installation.

5.3.2 Resistance to Lateral Loads

The lateral loading could be resisted fully or partially by the use of battered piles. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles. If integral abutments are under consideration, there may also be a requirement for the piles to move sufficiently to accommodate the bridge deck deflections.

The resistance to lateral loading in front of the pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h , is based on the following equations.

For the existing clayey silt embankment fill, and for the native clayey silt soils:

$$k_h = \frac{k_{s1}}{5B} \quad \text{where} \quad B \text{ is the pile diameter (m) and}$$

$$k_{s1} \text{ is the coefficient of horizontal subgrade reaction, as given below.}$$

For the new embankment fill (assuming compacted granular fill), and for the surficial silty sand to sandy silt deposit and the silty sand to sandy silt interlayer:

$$k_h = \frac{n_h z}{B} \quad \text{where} \quad n_h \text{ is the constant of subgrade reaction, as given below;}$$

$$z \text{ is the depth (m); and}$$

$$B \text{ is the pile diameter (m}^2\text{).}$$

The following ranges for the value of n_h and k_{s1} may be assumed in the structural analysis; these values will have to be confirmed following the detailed design stage of the subsurface investigation.

<i>Soil Unit</i>	<i>n_h</i>	<i>k_{s1}</i>
Existing embankment fill, above Elevation 219 m	—	10 to 35 MPa/m
New embankment fill (assumed to be compacted granular fill), above Elevation 219 m	5 to 15 MPa/m	—
Surficial silty sand to sandy silt layer, between Elevation 219 m and 217 m	5 to 15 MPa/m	—
Upper clayey silt, above Elevation 211 m	—	10 to 25 MPa/m
Silty sand to sandy silt interlayer, between Elevation 211 m and 207 m	10 to 25 MPa/m	—
Lower clayey silt, below Elevation 207 m	—	25 to 60 MPa/m

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor, R , as follows:

<i>Pile Spacing in Direction of Loading d = Pile Diameter</i>	<i>Subgrade Reaction Reduction Factor R</i>
8d	1.00
6d	0.70
4d	0.40
3d	0.25

5.3.3 Frost and Erosion Protection

The pile caps should be provided with 1.5 m of soil cover for frost protection. Additional cover may be required at pier pile caps located within and immediately adjacent to the North Canal channel to protect against scour action and resultant undermining of the pile caps.

5.4 Lateral Earth Pressures

The lateral pressures acting on the bridge abutments and associated retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. The following recommendations are made concerning the design of the abutments, in accordance with the OHBDC:

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the abutments and walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00.
- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the abutment wall, in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be used in accordance with OPSS 501.06.
- The granular fill may be placed either in a zone with width equal to at least 1.5 m behind the back of the stem (Case I from OHBDC Figure 6-7.4.1) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II from OHBDC Figure 6-7.4.4).

- For Case I, the pressures are based on the existing and proposed embankment fill materials and the following parameters (unfactored) may be assumed:

Soil unit weight:	20 kN/m ³
Coefficients of lateral earth pressure:	
Active, K_a	0.35
At rest, K_o	0.50

- For Case II, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

	Granular 'A'	Granular 'B' Type 2
Soil unit weight:	22 kN/m ³	21 kN/m ³
Coefficients of lateral earth pressure:		
Active, K_a	0.27	0.31
At rest, K_o	0.43	0.47

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the wall support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.

It should be noted that the above design recommendations and parameters assume level backfill and ground surface behind the abutment and retaining walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

5.5 Approach Embankment Design and Construction

Based on topographic information and site reconnaissance, the existing Highway 400 embankment side slopes in the vicinity of the North Canal / Canal Road bridges are formed at a gradient of 2.5 to 3 horizontal to 1 vertical (2.5H:1V to 3H:1V). For preliminary design purposes, the side slopes of the widened or new embankments should be formed at a maximum gradient of 2.5H:1V. This recommended slope configuration will have to be reviewed during detailed design, once the configuration of the structures and embankments is established. To

ensure the stability of the abutment foreslopes, erosion and scour protection should be placed at the banks of the North Canal channel.

For the proposed embankment widening, fill heights of up to about 7 m will be required within the limits of the new approach embankments. This embankment loading will induce consolidation settlement of the clayey silt soils below the widened portions of the embankment, as well as below the existing embankment. Based on correlations between plasticity indices and consolidation parameters, it is estimated that 100 mm to 150 mm of settlement will occur within the native clayey silt soils under the widened embankment, and under the outside shoulder and driving lanes of the existing embankment. However, for the option of widening in the median, the settlements would be minor.

The time for the settlement to occur is dependent on the extent of sandy seams within the clayey silt as well as the applied loading. It is expected that the majority of the settlement would occur within the first two or three years after fill placement. It is recommended that the new embankment fill be placed as early as possible to allow most of the settlement to take place prior to paving of the new lanes, and opening of the widened portion of the highway. Some maintenance of the existing outside driving lanes and shoulders should be expected during this initial period. Alternatively, in order to reduce the amount of consolidation settlement of the underlying clayey silt deposit, consideration could be given to the use of light-weight fill for the widened approach embankments; also, the use of light-weight fill may eliminate downdrag loads on the piles. The use of light-weight fill would require further review during detailed design for this structure site. It is recommended that oedometer testing be carried out during the final design stage of the bridge to establish consolidation parameters and allow a refinement of settlement predictions.

Settlement of the newly placed fill will also occur as a consequence of consolidation of the fill itself. In order to minimize the differential settlement (due to consolidation of the fill itself) between the widened portions of Highway 400 and the existing embankments, the use of granular fill may be considered for the widening. The majority of settlement of granular fills will occur during construction whereas the majority of settlement of cohesive fills, if used, would occur post-construction. The construction of the new embankments should be carried out using conventional fill placement and compaction practices. Benching of the existing embankment side slopes, in accordance with OPSD 208.01, should be carried out to key in the new fill and, again, minimize differential settlement between the existing and new portions of the embankments.

Fill containing boulders or other obstructions should not be placed within the limits of where piles are to be driven.

5.6 Design and Construction Considerations

5.6.1 Groundwater and Surface Water Control

The middle three piers are located within or immediately adjacent to the North Canal channel. A dewatering scheme will be necessary at these pier locations to divert or otherwise control the canal water flow and allow pile driving and pile cap construction in dry conditions. This will likely comprise driven closed steel sheet piling.

Elsewhere at the site, groundwater seepage into the pile cap excavations may occur from the surficial silty sand to sandy silt deposit, where present, and from water-bearing lenses and interlayers of granular soil within the clayey silt deposit, although the quantity is expected to be minor. Pumping from properly filtered sumps or a filtered drain placed at the base of the excavation should provide sufficient groundwater control during foundation works. Surface water run-off should be directed away from the footing excavations.

The soils in which the pile cap excavations will be formed are susceptible to disturbance from ponded water and construction traffic. Provision should be made in the Contract Documents for the placement of a lean concrete mat to protect the soils from such disturbance.

5.6.2 Excavation

Pile cap excavations will extend through embankment fill, surficial sands and silts (where present) and firm to stiff clayey silt. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. The fill, surficial sands and firm to stiff clayey silt soils at this site would be classified as Type 3 soil. Temporary open-cut slopes should be maintained no steeper than 1 horizontal to 1 vertical (1H:1V). Where space restrictions dictate, pile cap excavation could also be carried out within a braced excavation.

5.6.3 Obstructions

One 450 mm diameter boulder was encountered in Borehole 4 during the 1970 investigation. It should be recognized that cobbles and boulders are inherent in glacially-derived materials. In addition, cobbles, boulders or other obstructions could be present within the existing embankment fill. As noted in Section 5.5, fill containing boulders or other obstructions should not be used for embankment construction within the limits of where piles are to be driven.

The presence of cobbles and boulders may create obstructions to H-pile installation at this site. The piles should be provided with flange plates to limit damage to the pile tip during installation.

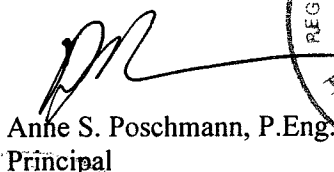
5.6.4 Vibrations

Given the close proximity of the new construction to the existing structures, it is recommended that vibration monitoring be carried out during pile installation. A Non-Standard Special Provision (NSSP) should be included in the Contract Documents to require the Contractor to monitor vibrations on the existing bridge and maintain the measured vibration levels below a peak particle velocity of 50 mm per second.

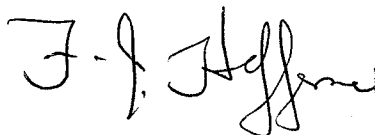
GOLDER ASSOCIATES LTD.



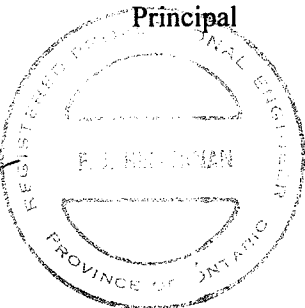
Lisa C. Coyne, P.Eng.
Geotechnical Engineer



Anne S. Poschmann, P.Eng.
Principal

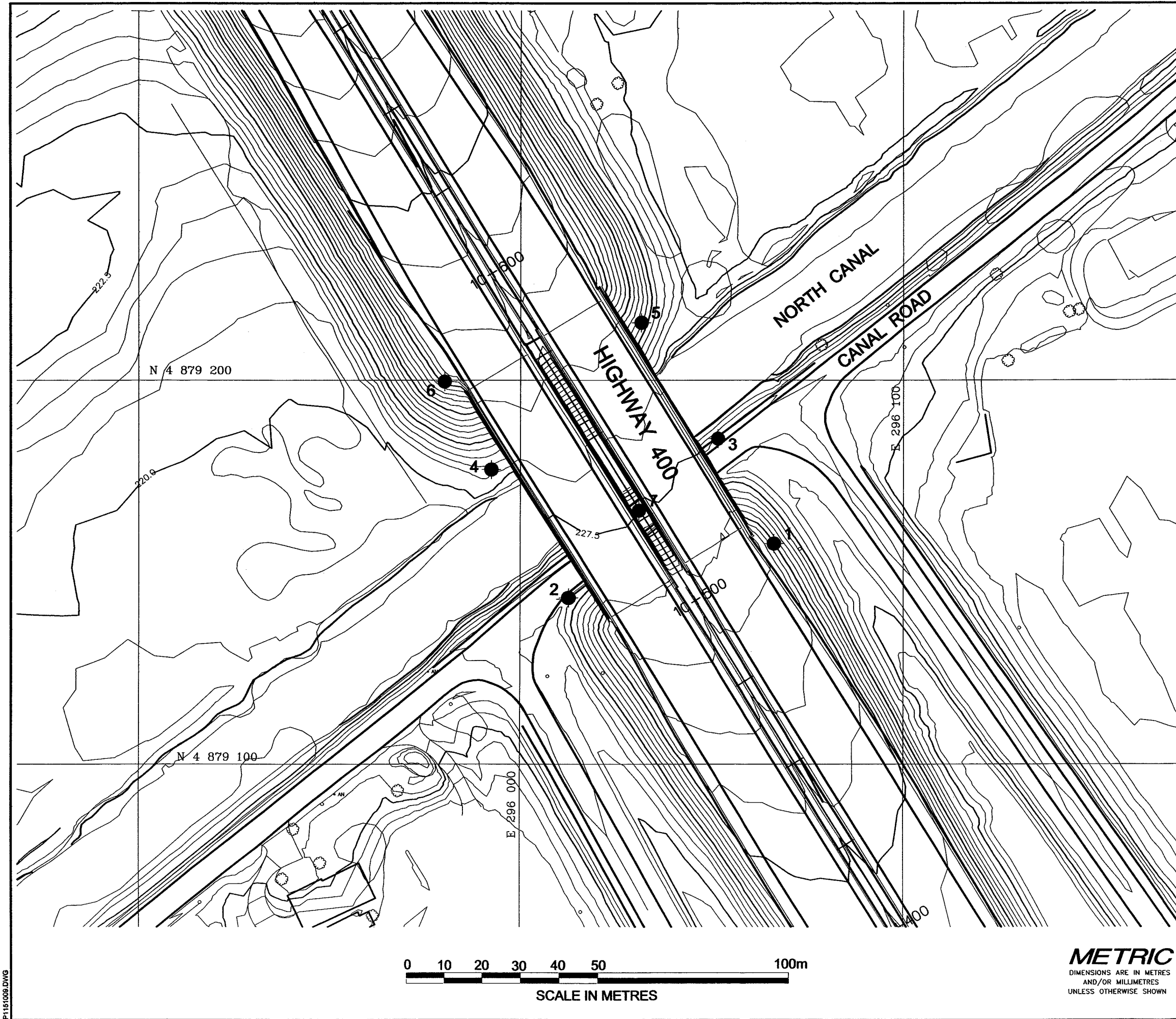


Fintan J. Heffernan, P.Eng.
Designated MTO Contact



LCC/ASP/FJH/clg

\\MIS_MAIN\DATA\ACTIVE\1100\001-1151\RPT01-01DEC-NORTHCANAL.DOC



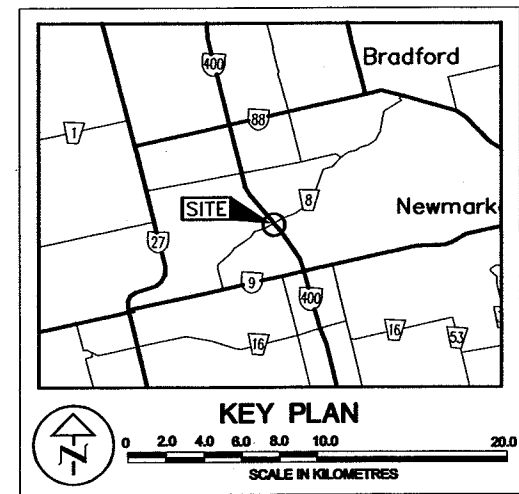
DIST HWY 400

CONT. No.

GWP No. 40-00-00

SHEET

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



LEGEND			
	Borehole, previous investigation		
	Borehole, present investigation		
No.	ELEVATION	LOCATION	
		NORTHING	EASTING
1	226.0	4,879,157	296,066
2	221.6	4,879,143	296,013
3	221.5	4,879,185	296,051
4	220.9	4,879,177	295,992
5	223.2	4,879,215	296,031
6	227.7	4,879,200	295,980
7	221.6	4,879,166	296,031

REFERENCE
This drawing was created from digital file "33822.dwg"
provided by URS Cole Sherman

P1151008.DWG

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

NO.	DATE	BY	REVISION
Geocres No.			
HWY. No. 400		PROJECT NO.: 001-1151	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-334N/S
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

APPENDIX A

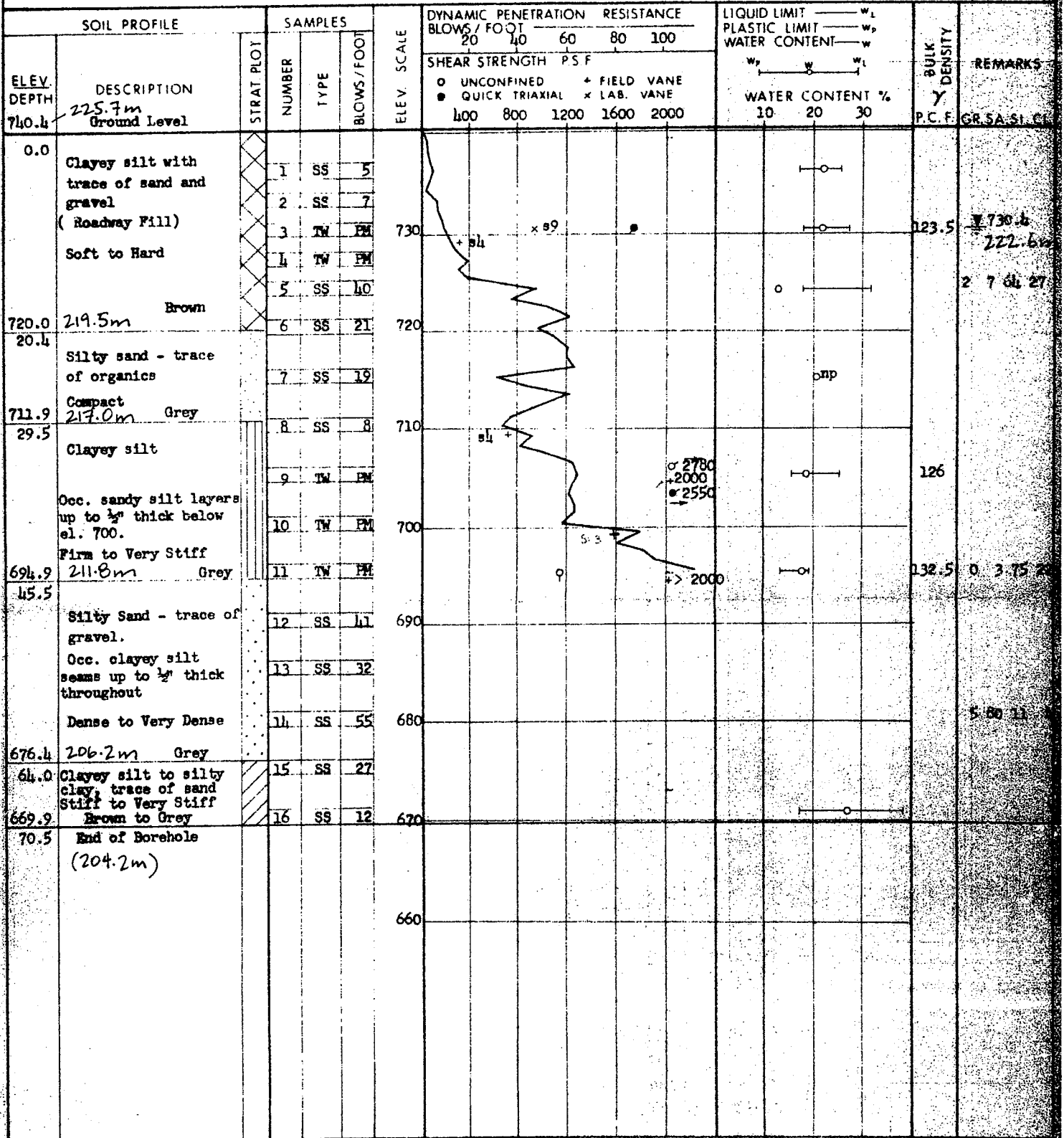
RECORDS OF BOREHOLES AND TEST RESULTS
1970 SUBSURFACE INVESTIGATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 1

FOUNDATION SECTION

JOB 70-11090 LOCATION Sta. 116+73 @ Hwy. 400 o/s 52.5' Rt. ORIGINATED BY WK
W.P. 105-70-05 BORING DATE Nov. 2-4, 1970 COMPILED BY WH
DATUM Geodetic BOREHOLE TYPE Diamond Drill-Washboring CHECKED BY LK

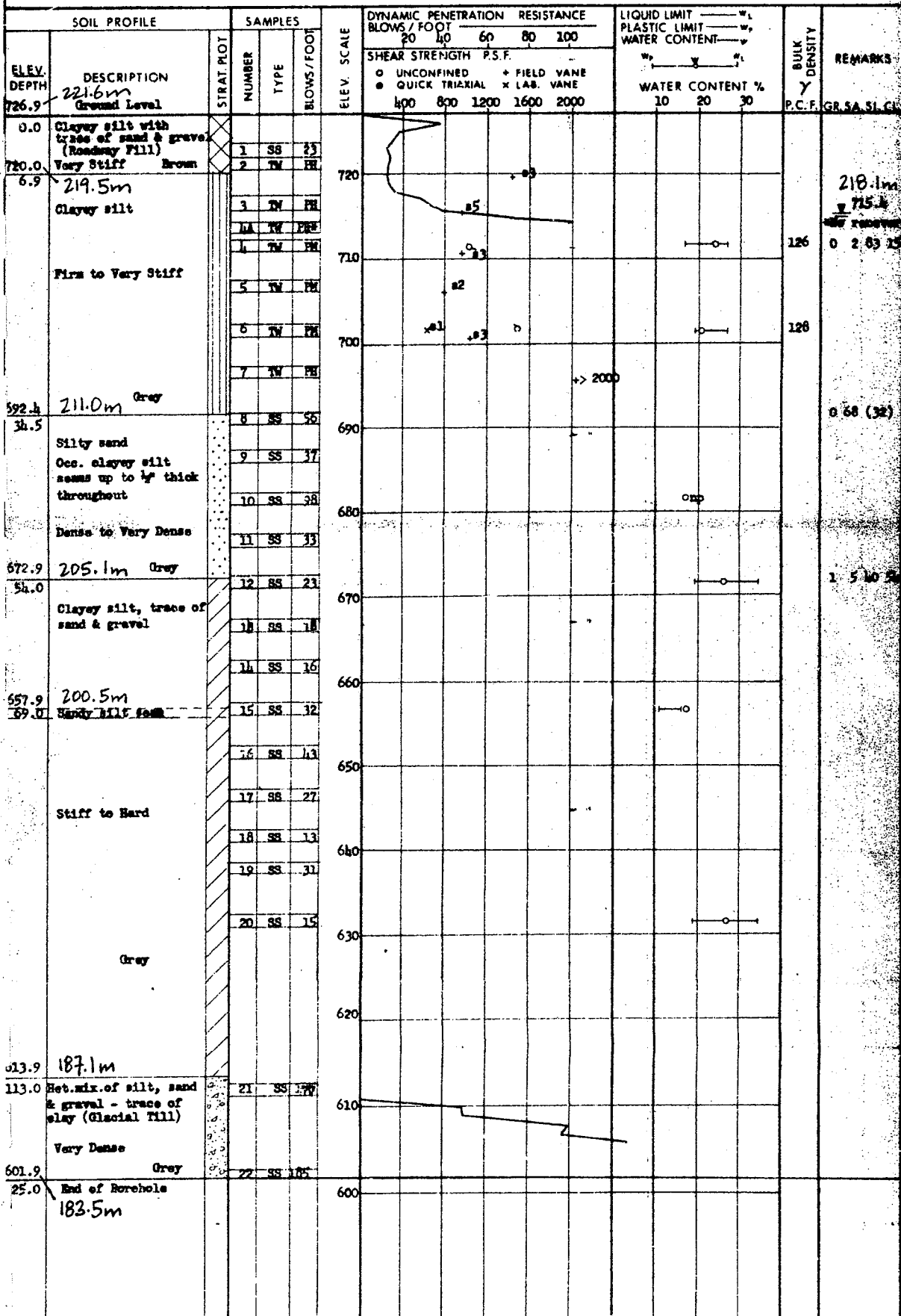


DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 2

FOUNDATION SECTION

JOB 70-11090 LOCATION Sta. 117 + 07 @ Hwy. 400 @/s 56.5' Lt. ORIGINATED BY W
 W.P. 105-70-05 BORING DATE Oct. 26, 1970 COMPILED BY W
 DATUM Geodetic BOREHOLE TYPE Cont. Flight Auger CHECKED BY W

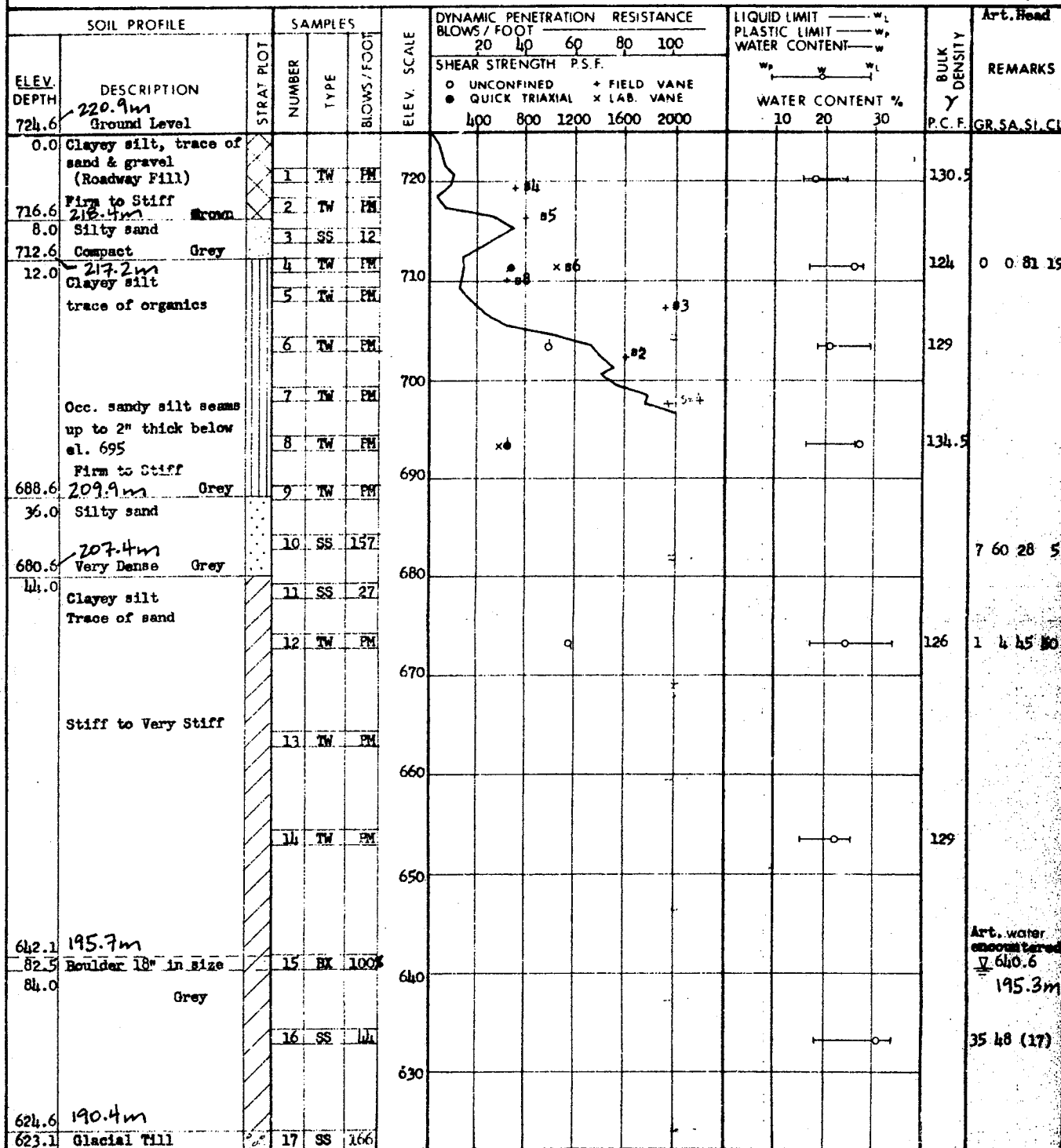


DEPARTMENT OF HIGHWAYS- ONTARIO
 MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 4

FOUNDATION SECTION

JOB 70-11090	LOCATION Sta. 118 + 20 @ Hwy. 400 o/s 52.5' Lt.	ORIGINATED BY VK
W.P. 105-70-05	BORING DATE Nov. 12-13, 1970	COMPILED BY WE
DATUM Geodetic	BOREHOLE TYPE Diamond Drilling-Washboring	CHECKED BY <u>AK</u> 738.6


 20
 15 5 % STRAIN AT FAILURE
 in

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 5

FOUNDATION SECTION

JOB 70-11090

LOCATION Sta. 118 + 63 @ Hwy. 400 o/s 52.5' Rt.

ORIGINATED BY VK

W.P. 105-70-05

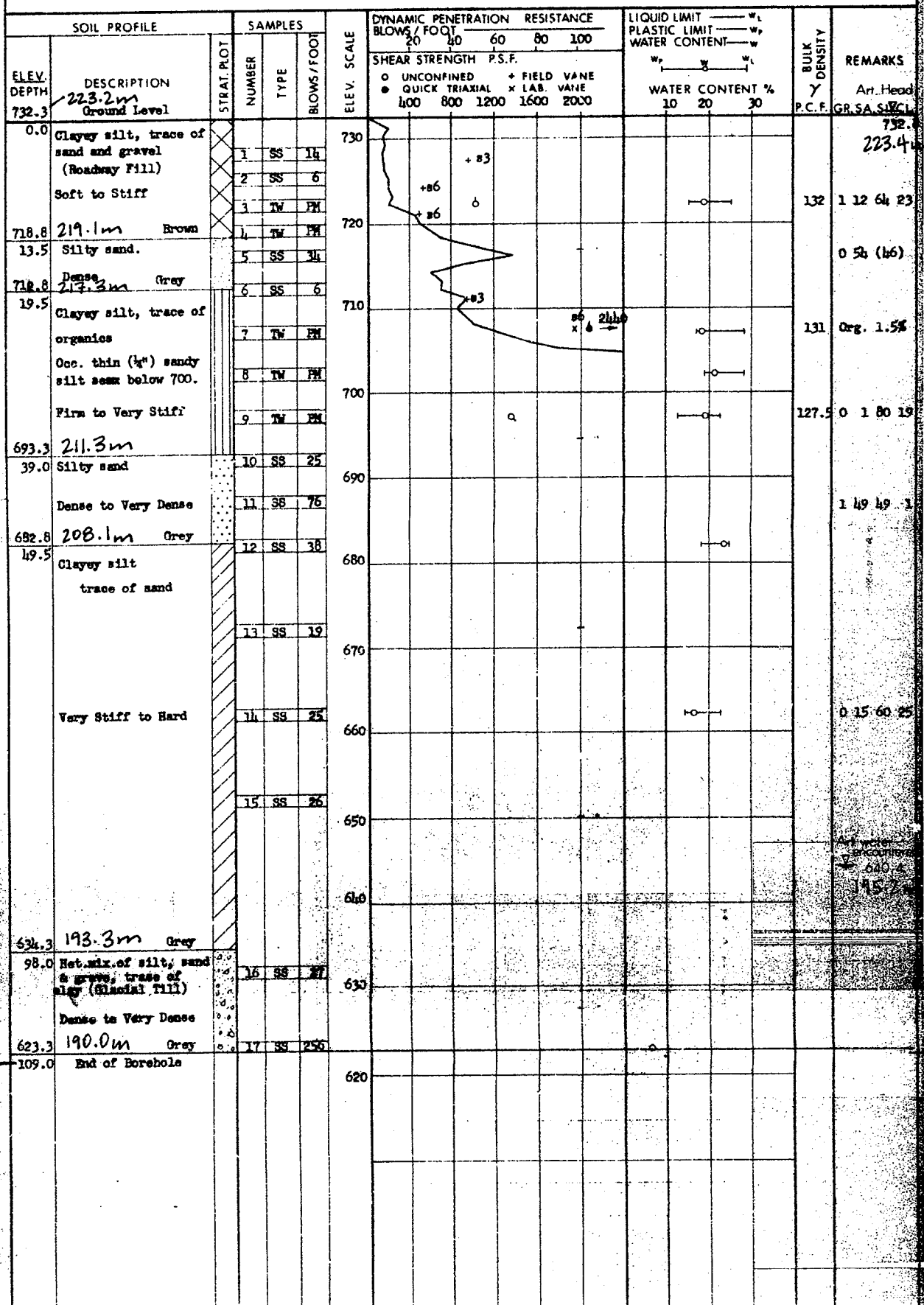
BORING DATE Nov. 5-6, 1970

COMPILED BY MH

DATUM Geodetic

BOREHOLE TYPE Diamond Drill - Washboring

CHECKED BY



DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 6

FOUNDATION SECTION

JOB 70-11090

LOCATION

Sta. 118 + 98 @ Hwy. 400 o/s 51' Lt.

ORIGINATED BY VK

W.P. 105-70-05

BORING DATE

Nov. 9-10, 1970

COMPILED BY WH

DATUM Geodetic

BOREHOLE TYPE

Diamond Drill-Washboring

CHECKED BY AK

SOIL PROFILE		SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT — W _L PLASTIC LIMIT — W _P WATER CONTENT — W			BULK DENSITY Y P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		BLOWS / FOOT	20	40	60	80	100	W _p	W _L		
747.1	227.7m Ground Level														
0.0	Clayey silt - trace of sand & gravel (Roadway Fill)		1	SS	5										
	Firm to Very Stiff		2	TW	PM	740								126	225.0m 738.1
			3	SS	2										
			4	TW	PM									126	0 9 75 16
			5	SS	16	730									
			6	SS	10										
722.1	220.1m Brown		7	SS	11	720									
25.0	Clayey silt		8	TW	PM										
	Traces of organics		9	TW	PM	710								126	
	Occ. thin (1/8") silt seams below 702.		10	TW	PM									129	
	Firm to Stiff		11	TW	PM	700								130	0.3 79 18 0.8% Org.
695.1	211.9m Grey		12	TW	PM										
52.0	Silty sand, occ. clayey silt seams up to 1/2" thick throughout		13	SS	104	690									
	Dense to Very Dense		14	SS	74										
681.6	207.8m Grey		15	SS	43										
65.5	End of Borehole					680									

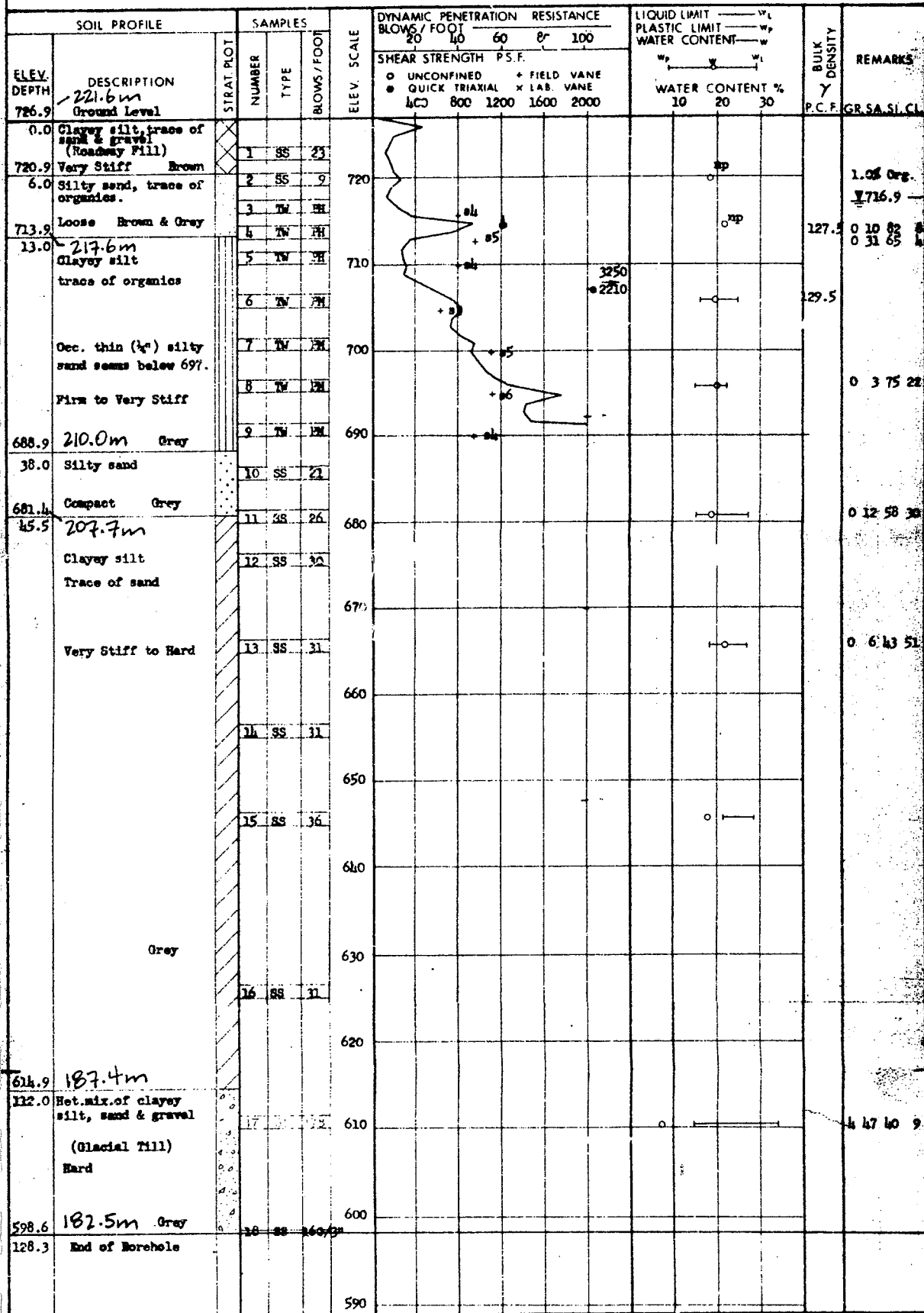
20
10 5 % STRAIN AT FAILURE
10

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

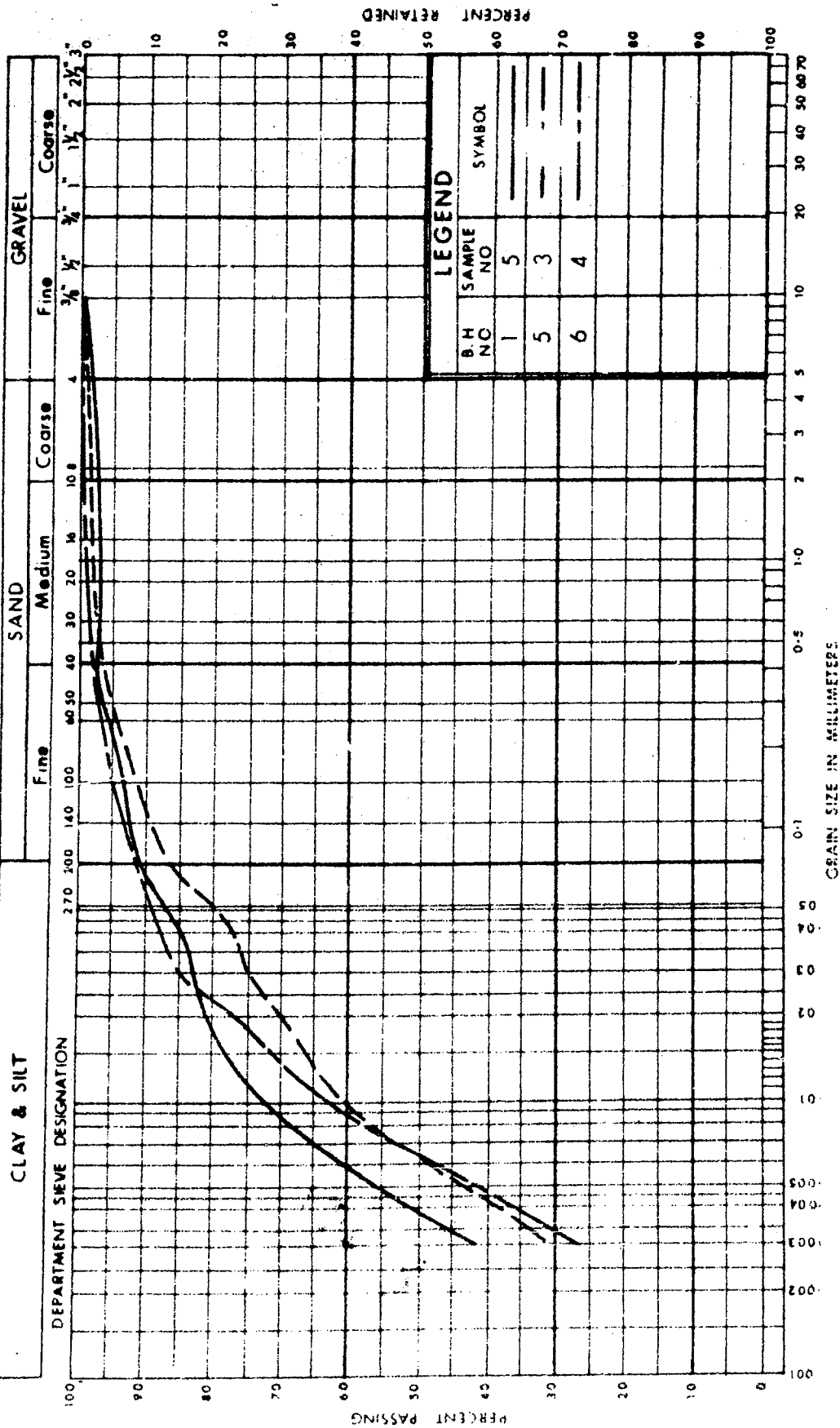
RECORD OF BOREHOLE No. 7

FOUNDATION SECTION

JOB 70-11090 LOCATION Sta. 117 + 33 E. Hwy. 400 ORIGINATED BY VK
 W.P. 105-70-05 BORING DATE Oct. 29, 1970 COMPILED BY WH
 DATUM Geodetic BOREHOLE TYPE Cont. Flight Auger CHECKED BY JR



UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

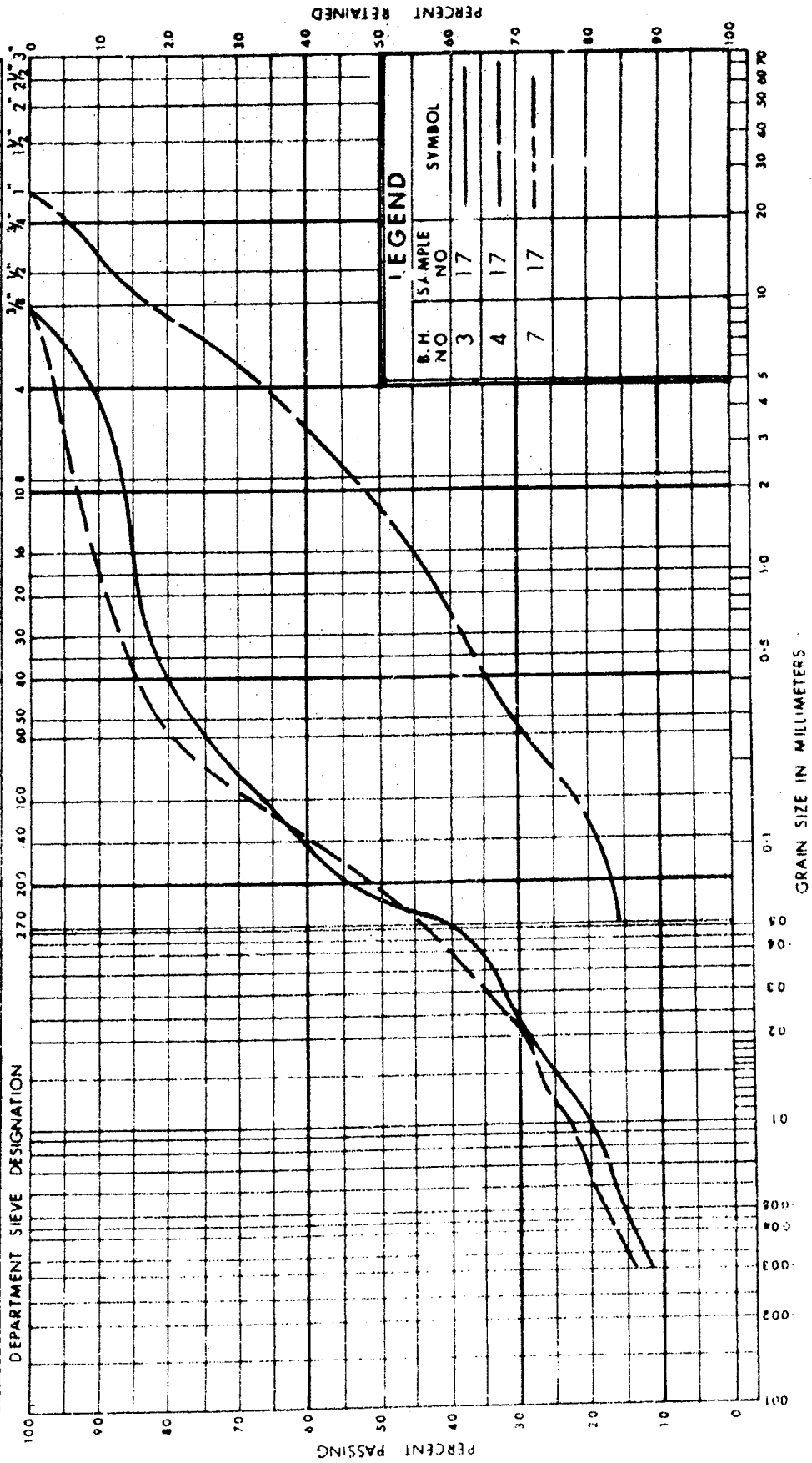
GRAIN SIZE DISTRIBUTION
CLAYEY SILT (FILL)
TRACE OF SAND & GRAVEL

W.P. No. 105-70-05

JOB No. 70-11090

FIG. 1

CLAY & SILT	SAND		GRAVEL	
	Fine	Medium	Fine	Coarse



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TESTING
DIVISION

GRAIN SIZE DISTRIBUTION

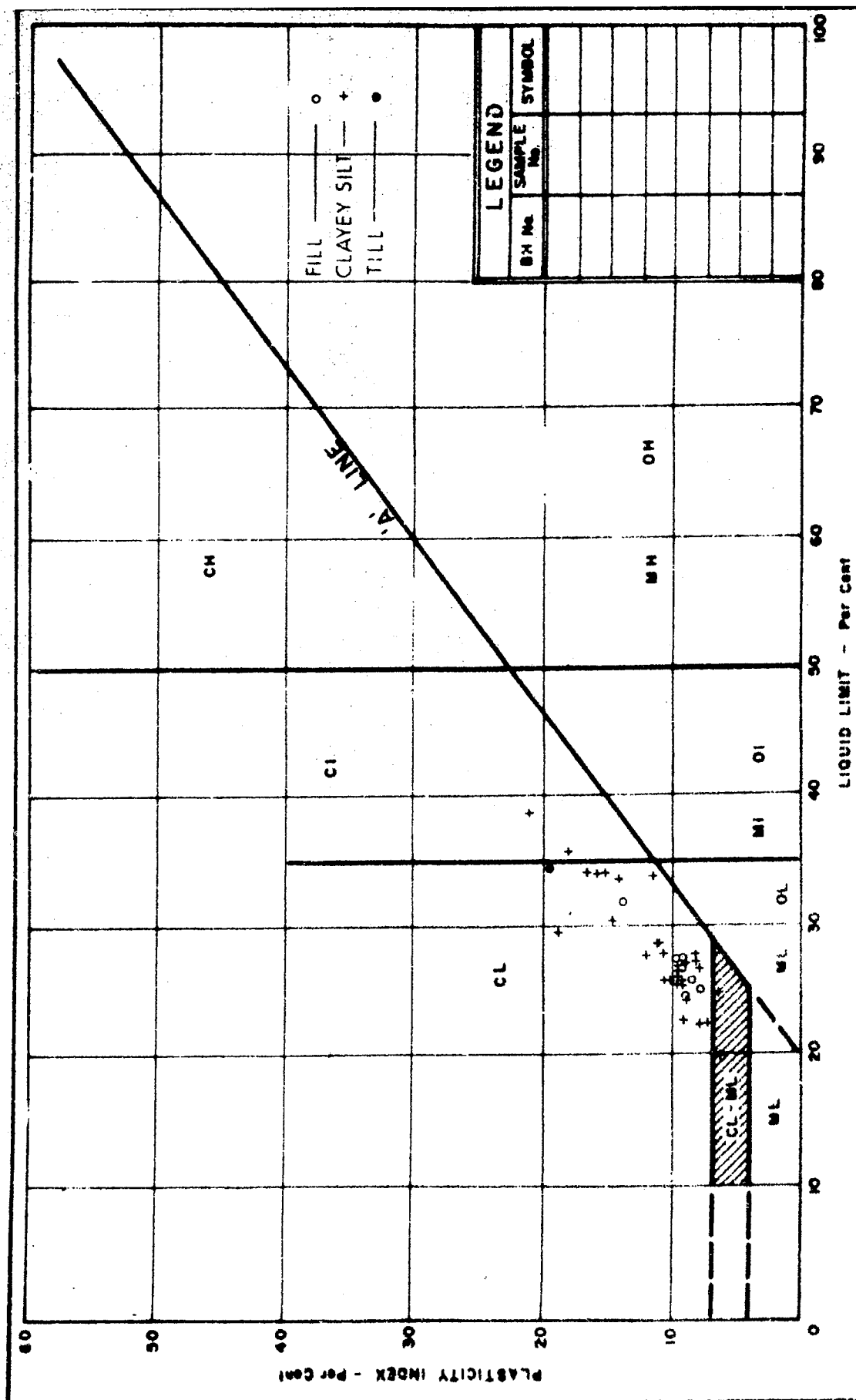
GLACIAL TILL

HET MIXTURE OF SILT, SAND & GRAVEL, SOME CLAY

W.P. No. 105-70-05

JOB No: 70-11090

FIG. 4



DEPARTMENT OF MATERIALS and
TESTING
DIVISION

PLASTICITY CHART

W.P. No. 105-70-05

70-11090

FIG. 5

