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**PRELIMINARY  
FOUNDATION INVESTIGATION AND DESIGN REPORT  
SOUTH CANAL BRIDGES  
HIGHWAY 400 WIDENING FROM NORTH OF  
MAJOR MACKENZIE DRIVE TO SOUTH CANAL ROAD  
G.W.P. 222-97-00, AGREEMENT NO. 2005-A-000106**

Submitted to:

McCormick Rankin Corporation  
2655 North Sheridan Way  
Mississauga, Ontario  
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June 2001



001-1122F-8

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**PART A**

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

## 1.0 INTRODUCTION

Golder Associates Ltd. has been retained by McCormick Rankin Corporation (McCormick Rankin) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the ultimate widening of Highway 400 from north of Major Mackenzie Drive, northerly 23 km to the north side of South Canal Road, in the Regional Municipality of York, Ontario. Foundations engineering services are required for the widening and / or replacement of eight structures at the following locations: Teston Road, Kirby Sideroad, Vaughan / King Road, the Humber River, King Road, Sixteenth Sideroad, Aurora Sideroad and the South Canal.

This report addresses the widening and / or replacement of the South Drainage Canal / Road bridges. Existing subsurface data from a report for the site prepared by the MTO, listed 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 South Drainage Canal and Road, Township of King, County of York, dated December 14, 1970 – GEOCRE File No. 31D-29.*

The terms of reference for the scope of work are outlined in Golder Associates Proposal No. P91-8118 / P91-1459, dated December 1999.

## **2.0 SITE DESCRIPTION**

The existing twin South Canal bridge structures are located about 0.6 km north of the Highway 9 interchange in King Township, in the Regional Municipality of York.

At the site, the original ground surface was at about Elevation 221 m. At the twin bridge structures, the South Canal channel is about 15 m wide, with its base at about Elevation 217.5 m; steel sheet piling is present along the sides of the channel. South Canal Bank Road runs immediately north of the drainage canal, with its profile grade at about Elevation 221 m, near the original ground surface. A low retaining wall is present along the north side of South Canal Bank Road, at the toe of the foreslope. Highway 400 has been constructed in fill, with its grade at about Elevation 228 m to 226 m, declining 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 to 3 horizontal to 1 vertical (2H:1V to 3H:1V).

The existing six-span twin bridges were originally constructed in 1948. The "perched" abutments and piers were supported on 12 m to 18 m long, 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 north and south abutments, the top of the pile caps are at about Elevation 224.5 m and 226 m, respectively. The top of the pile caps at the five pier locations is at about Elevation 219 m. The middle pier of both structures is located within the South Canal channel.

These foundation conditions were determined from the general arrangement drawing for Contract 71-12, which was provided by McCormick Rankin.

### **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, eight boreholes were advanced at the site to between 7 m and 20 m depth. Boreholes 1, 2 and 9 were advanced in the vicinity of the south abutment; Boreholes 3, 4 and 8 were advanced adjacent to the banks of the South Canal; and Boreholes 5 and 6 were advanced in the vicinity of the north abutment.

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. In-situ vane shear testing was carried out to determine the undrained shear strength of cohesive soils, where practicable. The groundwater conditions in the open borehole 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 23 km section of Highway 400 traverses, from south to north, the physiographic regions known as the South Slope, Oak Ridges Moraine and Simcoe Lowlands (as delineated in the Third Edition of *The Physiography of Southern Ontario*, by Chapman and Putnam). Along Highway 400, the South Slope of the Oak Ridges Moraine is present south of King Road; the Oak Ridges Moraine extends from north of King Road to south of Highway 9; and the Simcoe Lowlands occupy a 4 km wide strip extending from south of Highway 9 to the Holland River. The South Drainage Canal and South Canal Bank Road site is located within the Simcoe Lowlands physiographic region.

The surficial soils of the South Slope region are comprised of generally cohesive tills. The Oak Ridges Moraine is comprised predominantly of sand and gravel, although in the King Township area these soils are often overlain by till. It is understood that during grading for the initial construction of Highway 400 in this area, deep cuts exposed up to about 10 m of till overlying the sands and gravels.

The Holland River valley, which crosses Highway 400 in the vicinity of Highway 9 and South Canal Road, comprises part of the Simcoe Lowlands region. This valley 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. It is understood that during initial construction of Highway 400, between 2 m and 3 m of peat was removed in order to construct the road upon the underlying sands, silts and clays.

### **4.2 Site Stratigraphy**

The detailed subsurface soil and groundwater conditions encountered in the 1970 boreholes, together with the results of the laboratory testing carried out on selected soil samples, are given on the Record of Borehole sheets and the figures from the DHO report included in the attached 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 and 9 were advanced in the vicinity of the south abutment; Boreholes 3, 4 and 8 were advanced adjacent to the banks of the South Canal; and Boreholes 5 and 6 were advanced in



the vicinity of the north abutment. The approximate locations and ground surface elevations for these borings are shown on the attached Drawing 1.

In summary, the native subsoils at the site consist of surficial sand and organic material overlying an extensive deposit of clayey silt, in turn underlain by a dense to very dense sand and silt till deposit. A detailed description of the subsurface conditions encountered in the 1970 boreholes is provided in the following sections.

#### **4.2.1 Embankment Fill**

Four of the boreholes (Boreholes 1, 2, 5 and 6) penetrated the existing approach embankment fill. The fill is comprised of clayey silt containing trace quantities of sand and gravel. The fill consistency is variable, with measured Standard Penetration Test (SPT) 'N' values ranging from 4 to 48 blows per 0.3 m of penetration. In-situ vane shear strength testing and undrained triaxial testing was carried out in the "softer" zones of the fill. This testing measured undrained shear strengths ranging from about 35 kPa to greater than 100 kPa, with an average of about 60 kPa.

#### **4.2.2 Organic Clay**

In Borehole 5, drilled on the east side of Highway 400 adjacent to the South Canal Bank Road, a 1 m thick layer of soft, black organic clay was encountered below 2 m of fill (associated with the toe of the highway embankment). The sample had a measured organic content of 47 percent, and a natural water content of 97 percent.

#### **4.2.3 Surficial Silty Sand to Sandy Silt**

A 2 m to 3 m thick surficial layer of silty sand, containing trace quantities of organic matter, was encountered in several of the boreholes; it was not encountered in Boreholes 3, 5 and 9 on the east side of the highway. Grain size test results obtained for this surficial deposit are plotted on Figure 1 of Appendix A. This layer has a loose to very dense relative density, with measured SPT 'N' values ranging from 6 to 66 blows per 0.3 m of penetration. Where embankment fill was present overlying the silty sand (in Boreholes 1, 2 and 6) the SPT 'N' values ranged from 23 to 66 blows per 0.3 m of penetration. Where the surficial silty sand was not overlain by embankment fill, the measured SPT 'N' values varied from 6 to 11 blows per 0.3 m of penetration.

#### 4.2.4 Clayey Silt

A deposit of clayey silt underlies the embankment fill and surficial layer of sand, where present, and extends from ground surface elsewhere. This deposit ranges from about 5.5 m to 14.5 m in thickness; its base was encountered at about Elevation 211 m to 210.5 m in Boreholes 1 to 3, on the south side of the Canal, and at about Elevation 208 m to 206 m in Boreholes 4 to 6 and 8, on the north side of the Canal.

The clayey silt generally contains trace quantities of sand. The grain size distribution test results obtained from samples of the clayey silt are shown on Figure 2 in Appendix A. Lenses, interlayers and seams of silty sand, between 6 mm and 300 mm in thickness, were encountered throughout the stratum. Grain size distribution test results for the silty sand are also shown on Figure 2 in Appendix A. The bulk unit weight of the clayey silt was determined to range from 20.3 kN/m<sup>3</sup> to 21.5 kN/m<sup>3</sup>.

The Standard Penetration Test (SPT) results typically ranged from 5 to 28 blows per 0.3 m of penetration. In-situ vane testing measured undrained shear strengths which typically ranged from 30 kPa to greater than 100 kPa. Laboratory vane and triaxial testing also measured undrained shear strengths ranging from about 35 kPa to greater than 100 kPa. The exception to this is the presence of a "softened" layer of clayey silt encountered on the north side of the canal in Boreholes 4, 5 and 8. The thickness of the "softened" zone is typically about 4 m with top surface at about Elevation 210 m to 212 m. The in-situ vane testing gave undrained shear strengths within this "softened" zone of 14 kPa to 29 kPa. The results of the Standard Penetration, in-situ vane and laboratory vane shear testing indicate that the clayey silt generally has a firm to hard consistency, and that the "softened" zone encountered on the north side of the canal has a soft to firm consistency.

Atterberg Limits testing measured plastic limits ranging from about 11 to 19 percent, and liquid limits ranging from 18 to 28 percent. The plasticity indices were between 7 and 13 percent. The results of the Atterberg Limits testing are shown on Figure 4 in Appendix A; these limits indicate that the clayey silt is inorganic and of low plasticity. The natural moisture content measured on samples of the clayey silt ranged from 12 to 24 percent in the firm to hard portions of the deposit, and from 19 to 29 percent in the soft portion of the deposit. Generally, the natural moisture content was near or above the liquid limit in the softened portions of the deposit; elsewhere, in the firm to hard portions of the clayey silt deposit, the measured moisture contents tended to be closer to the plastic limit.

Two oedometers tests were carried out to determine the compressibility characteristics of the "softened" portion of the clayey silt stratum encountered on the north side of the canal. The results are plotted on Figure 5 of Appendix A. For the two tested samples, the initial void ratios were 0.6 and 0.85, and the compression indices,  $c_c$ , were calculated to be 0.075 and 0.22. Within this

more compressible zone, the preconsolidation pressure is about 100 kPa greater than the effective overburden pressure.

#### **4.2.5 Sand and Silt Till**

The clayey silt stratum is underlain by a dense to very dense deposit of sand and silt till. The boreholes were terminated in this stratum, with a maximum penetration of about 5.5 m in Boreholes 2 and 8 (with the exception of Borehole 9). The top of this till stratum was encountered at about Elevation 211 m to 210.5 m in Boreholes 1 to 3, on the south side of the Canal, and at about Elevation 208 m to 206 m in Boreholes 4 to 6 and 8, 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 quantities of gravel. Grain size distribution curves are plotted on Figure 3 in Appendix A.

The measured SPT 'N' values in this deposit range from 30 blows to greater than 100 blows per 0.3 m of penetration, indicating a dense to very dense relative density. Generally, the SPT 'N' values in the upper 1 m to 2 m of the deposit range from 30 to 89 blows per 0.3 m of penetration. In the lower portion of the deposit, all measured SPT 'N' values were greater than 100 blows per 0.3 m of penetration.

#### **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 the boreholes in the vicinity of the canal, the water levels were at or slightly higher than the canal water level at the time of the investigation which was at about Elevation 219 m. In boreholes drilled further away from the canal, the water levels in the open boreholes were observed to be as high as Elevation 224 m. The levels recorded for Boreholes 1, 2 and 6 are at or above the base of the embankment fills and it is noted that these water levels may not be representative of the groundwater table. Based on the available information, however, it is expected that the groundwater table slopes downward toward the canal.

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

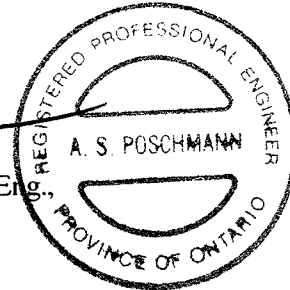
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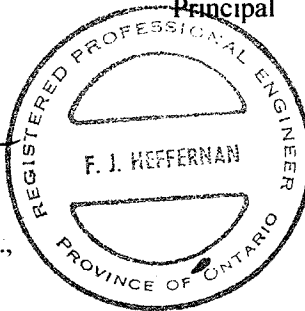
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**PART B**

**PRELIMINARY FOUNDATION DESIGN REPORT  
SOUTH CANAL BRIDGES  
HIGHWAY 400 WIDENING  
HIGHWAY 400 WIDENING FROM NORTH OF  
MAJOR MACKENZIE DRIVE TO SOUTH CANAL ROAD  
G.W.P. 222-97-00, AGREEMENT NO. 2005-A-000106**

## **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 South Canal 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. Additional foundation investigation work will be required at this bridge site during the detailed design stage of the project.

It is understood that Highway 400 will be widened from its existing six-lane configuration to an interim configuration of eight lanes, and an ultimate configuration of ten lanes, with the possible inclusion of additional high-occupancy vehicle lanes. The existing highway platform will therefore be widened by between 13 m and 31 m, necessitating widening or replacement of the existing South Canal bridge structures.

At the structure site, the South Canal channel is about 15 m wide, with its base at about Elevation 217.5 m; the sides of the channel are formed of driven steel sheet-piling. South Canal Bank Road runs immediately north of the drainage canal, with its profile grade at about Elevation 221 m, near the original ground surface. A low retaining wall is present along the north side of South Canal Bank Road, at the toe of the foreslope. Highway 400 has been constructed in fill, with its grade at about Elevation 228 m to 226 m, declining 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 to 3 horizontal to 1 vertical (2H: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 north and south abutments, the top of the pile caps are at about Elevation 224.5 m and 226 m, respectively, and the top of the pile caps at the pier locations is at about Elevation 219 m. The original portions of the bridges are supported on 12 m to 18 m long, driven timber piles; it is considered likely that these piles are founded on the very dense sand and silt till stratum. The outside portions of both bridges, which were constructed under a 1971 widening contract, are supported on steel H-piles which have been driven to practical refusal within the very dense sand and silt till deposit.

## **5.2 Bridge Foundation Options**

The subsoils encountered in the boreholes put down during the 1970 investigation consist of surficial sand and organic material overlying an extensive deposit of clayey silt, in turn underlain by a dense to very dense sand and silt till deposit.

The presence of relatively weak clayey silt at shallow depth and the fact that the existing structure is supported on piles 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 practical refusal within the very dense sand and silt till 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 very dense till deposit is at about Elevation 211 m to 210 m on the south side of the Canal, and at about Elevation 208 m to 206 m on the north side of the canal. Based on the measured Standard Penetration Test (SPT) results, which indicate that the upper 2 m of the till are less dense than the lower portion of the deposit, it is anticipated that the pile tips will be at or below about Elevation 208 m on the south side of the Canal, and at or below about Elevation 204 m on the north side of the Canal.

Assuming that the new pile caps are placed at the same founding elevations as the existing pile caps, the pier piles would be about 11 m to 15 m long, and the abutment piles would be about 18 m to 20 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.

### **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 H-piles driven to practical refusal within the sand and silt till deposit may be taken as 1,600 kN. The axial resistance at SLS for 25 mm of settlement may be taken as 1,000 kN.

The additional load due to embankment construction will induce consolidation settlement of the clayey silt soils under the widened portions of the embankments. On the south side of the canal, the clayey silt is 6 m to 10 m thick, while on the north side of the canal, the encountered clayey silt was 12 m to 14.5 m thick. 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 follows:

<i>Location</i>	<i>Unfactored Negative Skin Friction Load</i>
Abutments and piers on south side of canal	150 kN
Piers on north side of canal	200 kN
North abutments	350 kN

The above represent unfactored values; an appropriate load factor would need to be applied in the structural analysis.

To minimize the amount of negative skin friction that may develop on a pile, consideration could be given to placing the new approach embankment fill 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.

To achieve the above design resistances, the piles should be driven to at least Elevation 208 m and 204 m on the south and north sides of the Canal, respectively, and 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. The appropriate note to be shown on the General Arrangement drawing is:

- Piles to be driven in accordance with Standard SS103-11 using an ultimate capacity of 4,000 kN per pile, to at least Elevation 208 m on the south side of the Canal and Elevation 204 m on the north side of the Canal.

### **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.

It is anticipated that a portion of the pile length would be within the existing and new embankment fill and the remainder of the pile length would be within the clayey silt deposit.



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 existing fill and native soils:

$$k_h = \frac{k_{s1}}{5B} \quad \text{where} \quad \begin{array}{l} B \text{ is the pile diameter (m) and} \\ k_{s1} \text{ is the coefficient of horizontal subgrade reaction, as given below} \end{array}$$

For new embankment, assuming granular fill

$$k_h = \frac{n_h z}{B} \quad \text{where} \quad \begin{array}{l} n_h \text{ is the constant of subgrade reaction} \\ z \text{ is the depth (m)} \\ B \text{ is the pile diameter (m}^2\text{)} \end{array}$$

For the existing fill and native clayey silt, the range in value of  $k_{s1}$  may be taken as 10 MPa to 35 MPa in the structural analysis. For the new embankment, assuming granular fill is used, the range in value of  $n_h$  may be taken as 5 MPa to 10 MPa in the structural analysis.

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</i> <i>D = Pile Diameter</i>	<i>Subgrade Reaction Reduction</i> <i>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 1.2 m soil cover for frost protection. Additional cover may be required at pier pile caps located within and immediately adjacent to the South 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 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 percent 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 percent 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 1.2 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 upward at 1.5 horizontal to 1 vertical (1.5H:1V) from the bottom of 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 <sup>3</sup>
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:

	<b>Granular "A"</b>	<b>Granular "B"</b>
		<b>Type II</b>
Soil unit weight:	22 kN/m <sup>3</sup>	21 kN/m <sup>3</sup>
Coefficients of lateral earth pressure:		
Active, $K_a$	0.27	0.31
At rest, $K_o$	0.43	0.47

- If the wall support allows lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment 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.

## 5.5 Approach Embankments

Based on site reconnaissance, the existing embankment side slopes in the vicinity of the South Canal bridges are formed at a gradient of 2 to 3 horizontal to 1 vertical (2H: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. To ensure the stability of the abutment foreslopes, erosion and scour protection should be placed at the banks of the South Canal channel.

It is anticipated that a grade raise of up to about 7 m will be required within the limits of the new approach embankments. This increased 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 the results of oedometer testing carried out during the 1970 MTO investigation, it is estimated that 50 mm to 100 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.

The majority of the settlement is expected to occur within one year of placement of the fill to the full embankment height. 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.

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 is recommended 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 Dewatering**

The middle three piers are located within or immediately adjacent to the South Canal channel. A dewatering scheme will be necessary at these pier locations to divert / control the canal water flow and allow pile driving and pile cap construction in dry conditions. This will likely comprise driven 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, which is expected to be more significant than groundwater seepage, should be directed away from the footing excavations.

The clayey silt 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 a minimum of 1.2 m below lowest surrounding grade, through embankment fill, surficial sands (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**

Although no cobbles or boulders were noted on the borehole records from 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 placed within the limits of where piles are to be driven.

Cobbles and boulders should be expected within the glacially derived deposits at this site and these may create obstructions to H-pile installation. 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.



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Principal

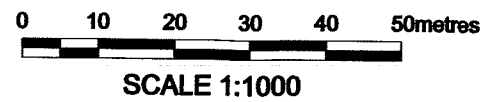
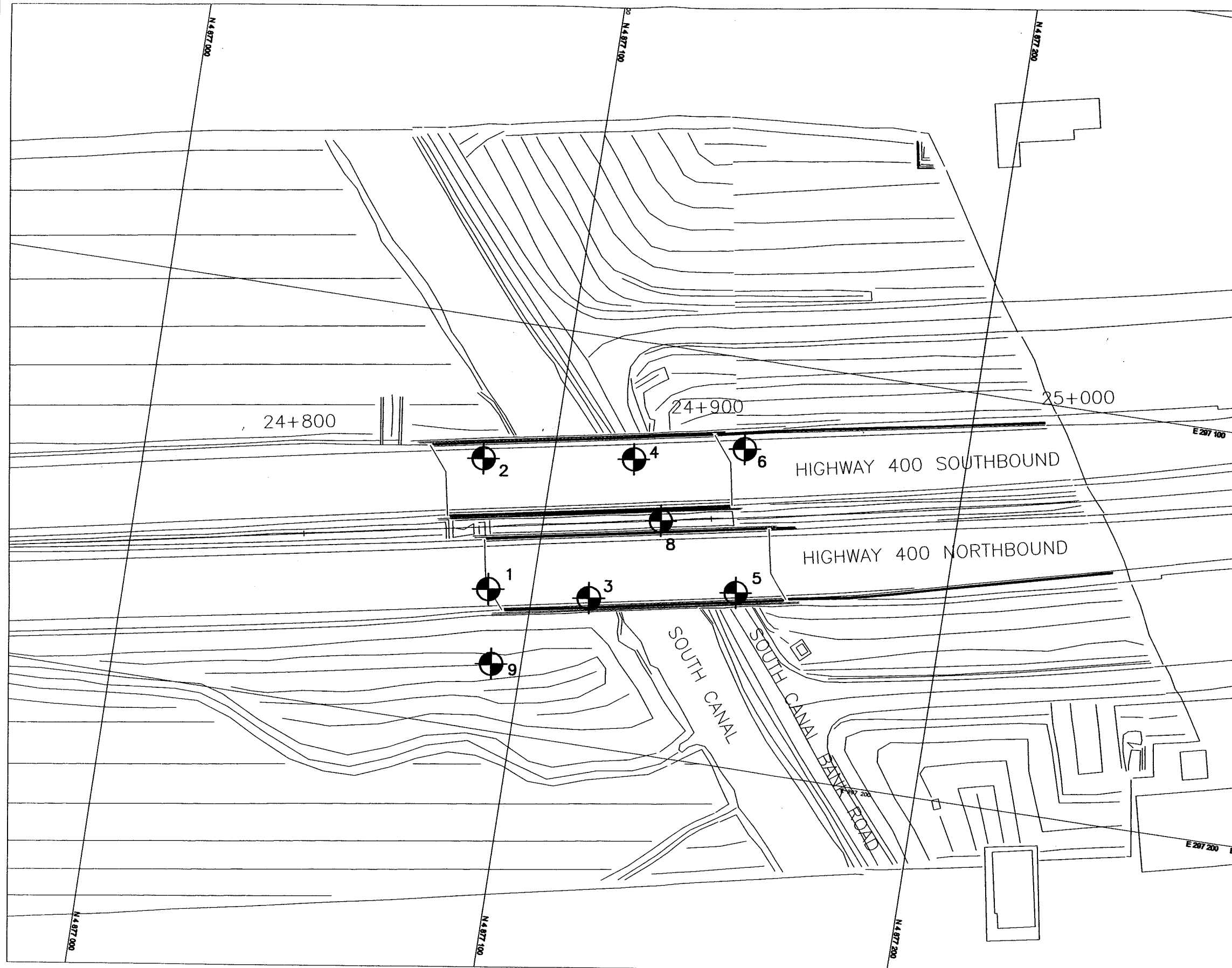


Fintan J. Heffernan, P.Eng.,  
Designated MTO Contact



LCC/ASP/FJH/clg

WMIS\_MAIN\DATA\ACTIVE\1100\001-1122\2001\RPT-JUN-01-SOUTHCANAL.DOC

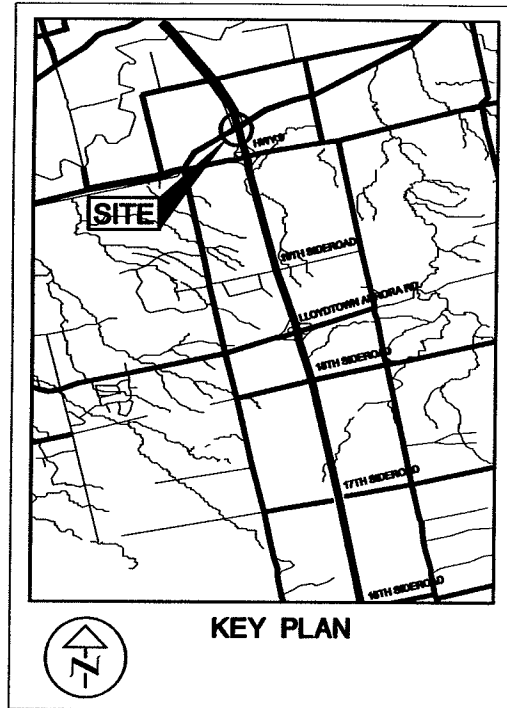


**METRIC**  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

CONT. No.  
GWP No. 222-97-00

SOUTH CANAL ROAD OVERPASS  
HIGHWAY 400  
BOREHOLE LOCATION PLAN

Golder Associates Ltd.  
MISSISSAUGA, ONTARIO, CANADA



LEGEND  
 Approximate Borehole Location

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
1	227.4	4877089	297166
2	223.9	4877083	297134
3	220.6	4877114	297164
4	221.1	4877119	297129
5	221.1	4877149	297157
6	225.8	4877146	297122
8	221.1	4877128	297143
9	221.1	4877093	297184

REFERENCE  
This drawing was created from digital base mapping provided by McCormick Rankin Corporation, dated September 16, 1998.

NO.	DATE	BY	REVISION
Geocres No.			
HWY. No. 400		PROJECT NO.: 001-1122F	
SUBM'D. LCC	CHKD: ASP	DATE: 2000 12 15	SITE 37-34
DRAWN: PS	CHKD. LCC	APPD. ASP	DWG. 1

P1122F07.DWG

June 2001

001-1122F-8

## **APPENDIX A**

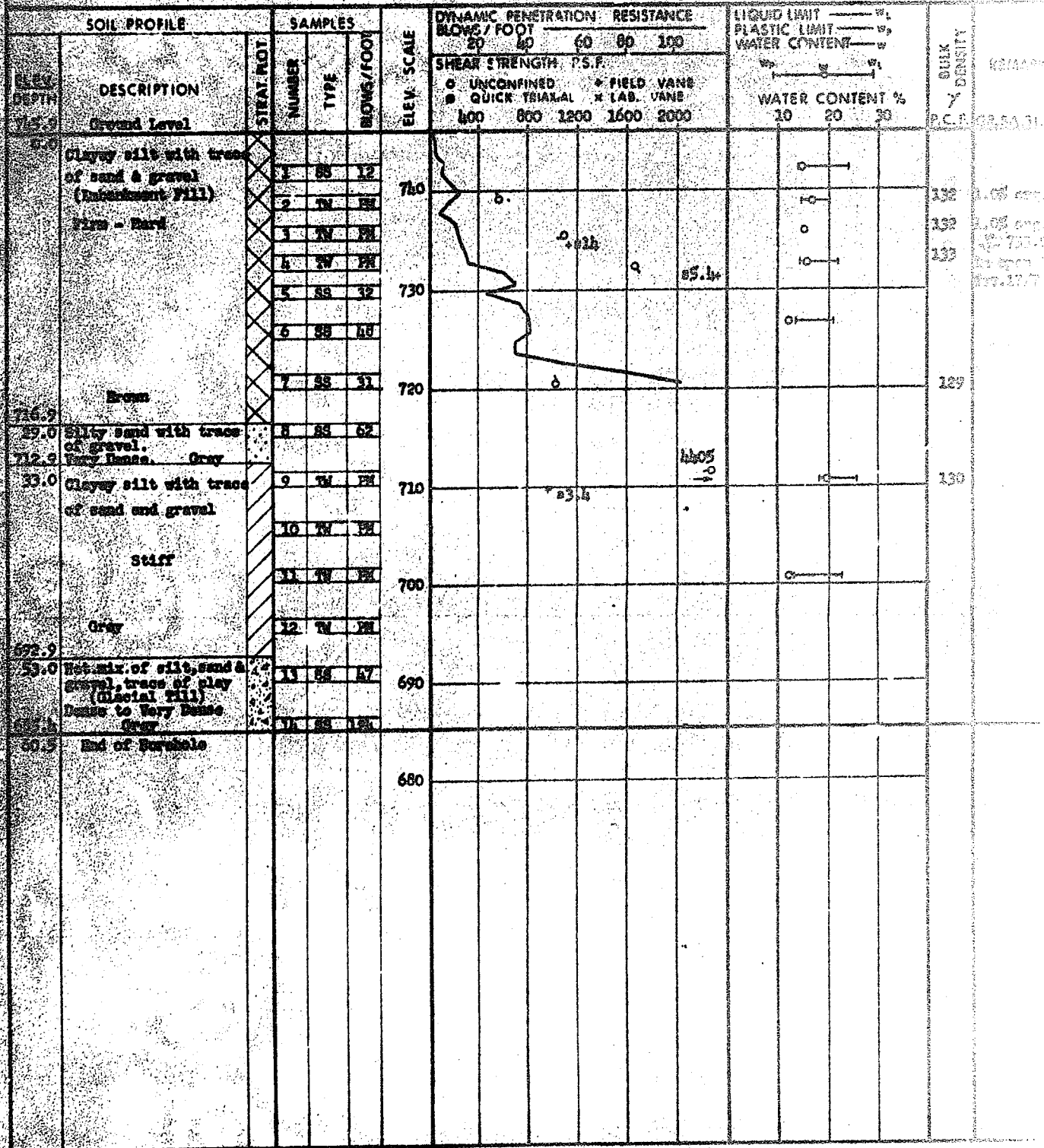
### **RECORDS OF BOREHOLES AND LABORATORY TEST RESULTS 1970 MTO INVESTIGATION**

DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS & TESTING OFFICE

# RECORD OF BOREHOLE No. 1

FOUNDATION DESIGN

JOB 70-13089 LOCATION Sta. 587 + 10 o/s 50.5' Rt. ORIGINATED BY JK  
 W.P. 105-70-04 BORING DATE Nov. 12, 1970 COMPILED BY SA  
 DATUM Geodetic BOREHOLE TYPE Washboring, BX Casing CHECKED BY JK



20  
10-5 % STRAIN AT FAILURE  
10



DEPARTMENT OF HIGHWAYS- ONTARIO  
MATERIALS & TESTING OFFICE

## RECORD OF BOREHOLE No.2

FOUNDATION SECTION

JOB 70-11089

LOCATION Sta. 587 + 07 s/s 2d Lk.

ORIGINATED BY

W.R. 105-70-04

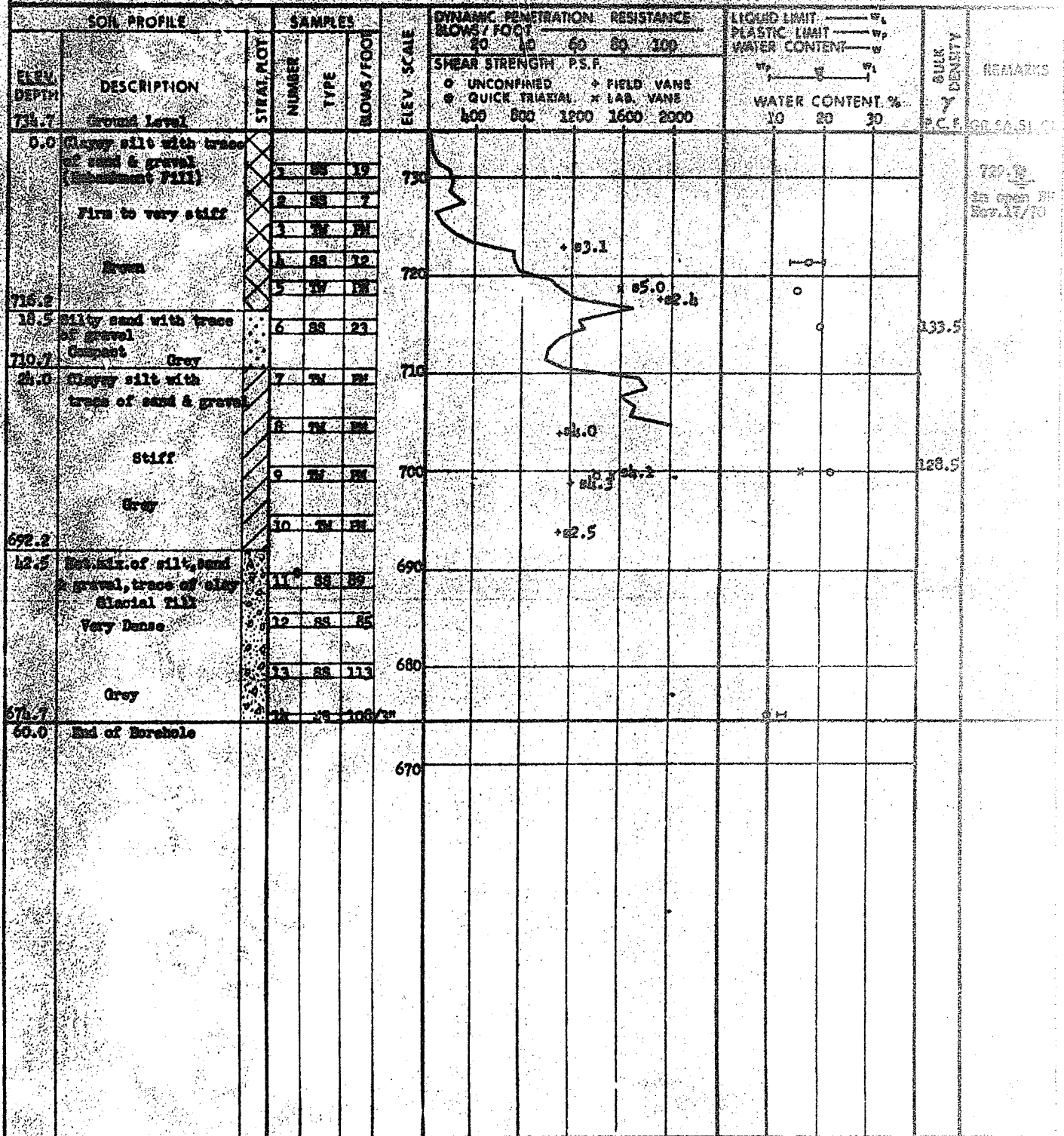
BORING DATE Nov. 13, 1970

COMPILED BY

DATUM Hamilton

BOREHOLE TYPE Washboring-MX casing

CHECKED BY



DEPARTMENT OF HIGHWAYS- ONTARIO  
MATERIALS & TESTING OFFICE

## RECORD OF BOREHOLE No. 3

FOUNDATION SECTION

JOB 70-11089

LOCATION Sta. 587 + 87 O/B 59' Et.

ORIGINATED BY VI

W.P. 105-70-04

BORING DATE Nov. 10, 1970

COMPILED BY SAA

DATUM Sandatic

BOREHOLE TYPE Weathering and RL Casing

CHECKED BY

SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT — $w_L$ PLASTIC LIMIT — $w_p$ WATER CONTENT — $w$			BULK DENSITY $\gamma$	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. NO.	NUMBER	TYPE	BLOWS/FOOT	20	40	60	80	100	10			20
723.6	Ground Level													
0.0	Clayey silt with trace of sand & gravel. Stiff to Very Stiff		1	SS	11									
715.1	Brown		2	SS	19									
8.5	Grey		3	SS	24									
			4	SS	28									
			5	SS	25									
			6	SS	25									
			7	SS	24									
			8	SS	12									
691.6	End of silt, sand & gravel, trace of clay. Glacial Till Very Dense		9	SS	25									
			10	SS	100A									
678.1	Grey		11	SS	200A									
45.5	End of Borehole													



DEPARTMENT OF HIGHWAYS- ONTARIO  
MATERIALS & TESTING OFFICE

## RECORD OF BOREHOLE No. 5

FOUNDATION SECTION

JOB NO. 11059

LOCATION Sta. 559 + 08 a/b 57' E.

ORIGINATED BY TE

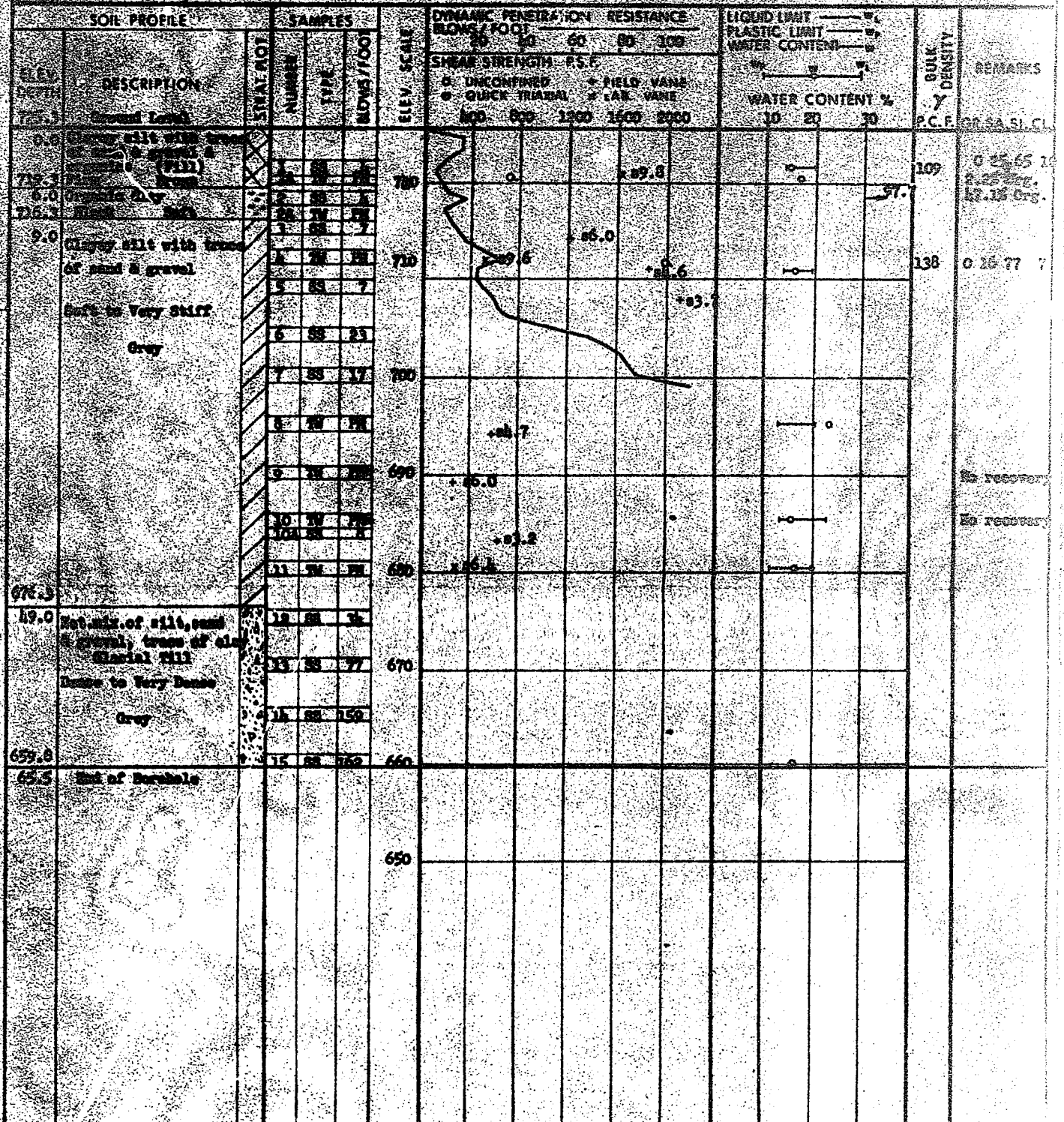
W.P. 105-70-02

BORING DATE Oct. 20/70

COMPILED BY SAA

DAYTON Roadside

BOREHOLE TYPE Bore Flight Auger

CHECKED BY *SK*



DEPARTMENT OF HIGHWAYS- ONTARIO  
MATERIALS & TESTING OFFICE

## RECORD OF BOREHOLE No. 6

FOUNDATION SECTION

JOB 75-21009

LOCATION

Sta. 589 + 15 o/s S.A. 5' 14.

ORIGINATED BY

W.P. 105-70-04

BORING DATE

Nov. 19/70

COMPILED BY

SAA

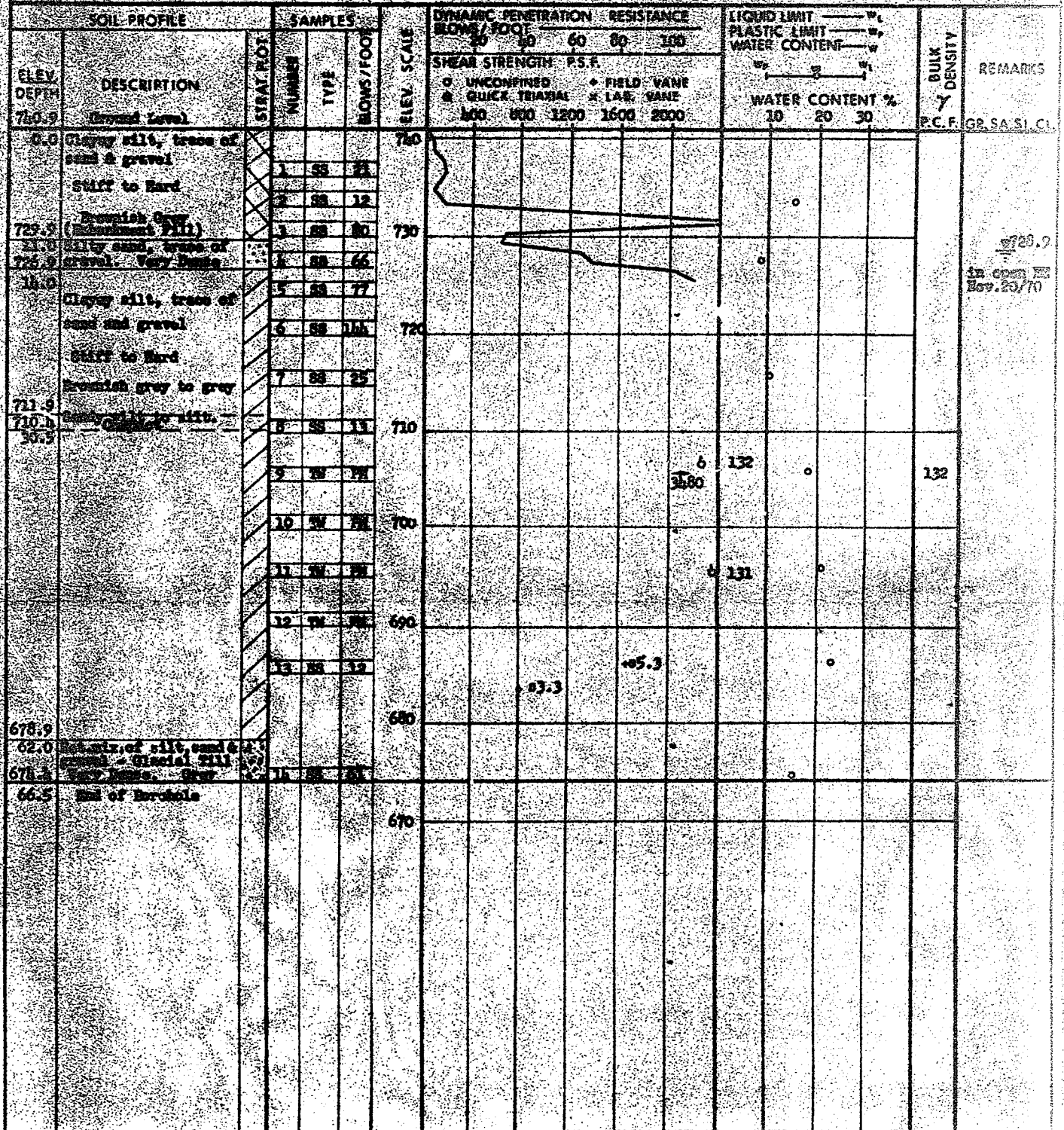
DATE 1/25/71

BOREHOLE TYPE

Weathering-III Casing

CHECKED BY

J.R.



DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS & TESTING OFFICE

## RECORD OF BOREHOLE No. 8

FOUNDATION SECTION

JOB 70-11000

LOCATION Sta. 588 + 15 o/s 2<sup>nd</sup> Lt.

ORIGINATED BY ES

W.P. 105-70-05

BORING DATE Oct. 21/70

COMPILED BY

DATUM Canadian

BOREHOLE TYPE Cons. Flight Auger

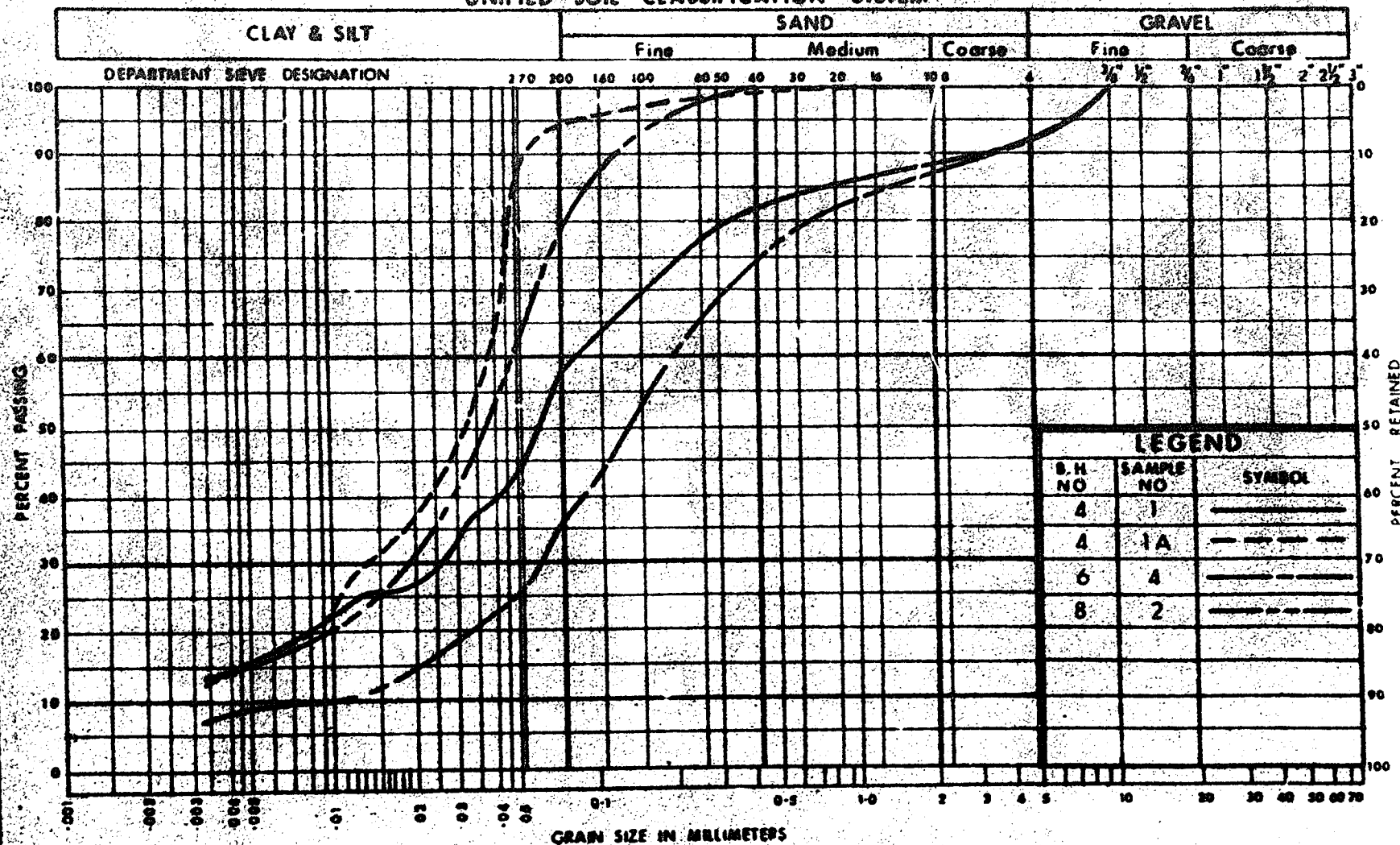
CHECKED BY

SOIL PROFILE		SAMPLES		ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT					LIQUID LIMIT — % PLASTIC LIMIT — % WATER CONTENT — %			BULK DENSITY γ	REMARKS
DEPT.	DESCRIPTION	NUMBER	TYPE		20	40	60	80	100	10	20	30		
75.5	Ground Level													
0.0	Heavy silt, trace of gravel	1	SS	11										Nov. 17/70
	Loose to - open	2	SS	7										720.8
75.5	Brown	3	SS	7										0 21.69 10
0.0	Clayey silt with trace of sand & gravel	4	SS	11										0 1.87 12
	Soft to Stiff	5	TV	11										
	Gray	6	SS	11										
		7	SS	11										
		8	SS	11										
		9	TV	11										
		10	TV	11										
		11	TV	11										
65.5		12	SS	11.5										
60.0	Ext. mix. of silt, sand & gravel; trace of clay - Glacial Till	13	SS	20										
	Dense to Very Dense	14	SS	100/5										
55.0		15	SS	100/5										
50.5	End of Borehole													





# UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT OF HIGHWAYS  
MATERIALS and  
TESTING  
DIVISION

**GRAIN SIZE DISTRIBUTION**  
**SANDY SILT TO SILTY SAND**

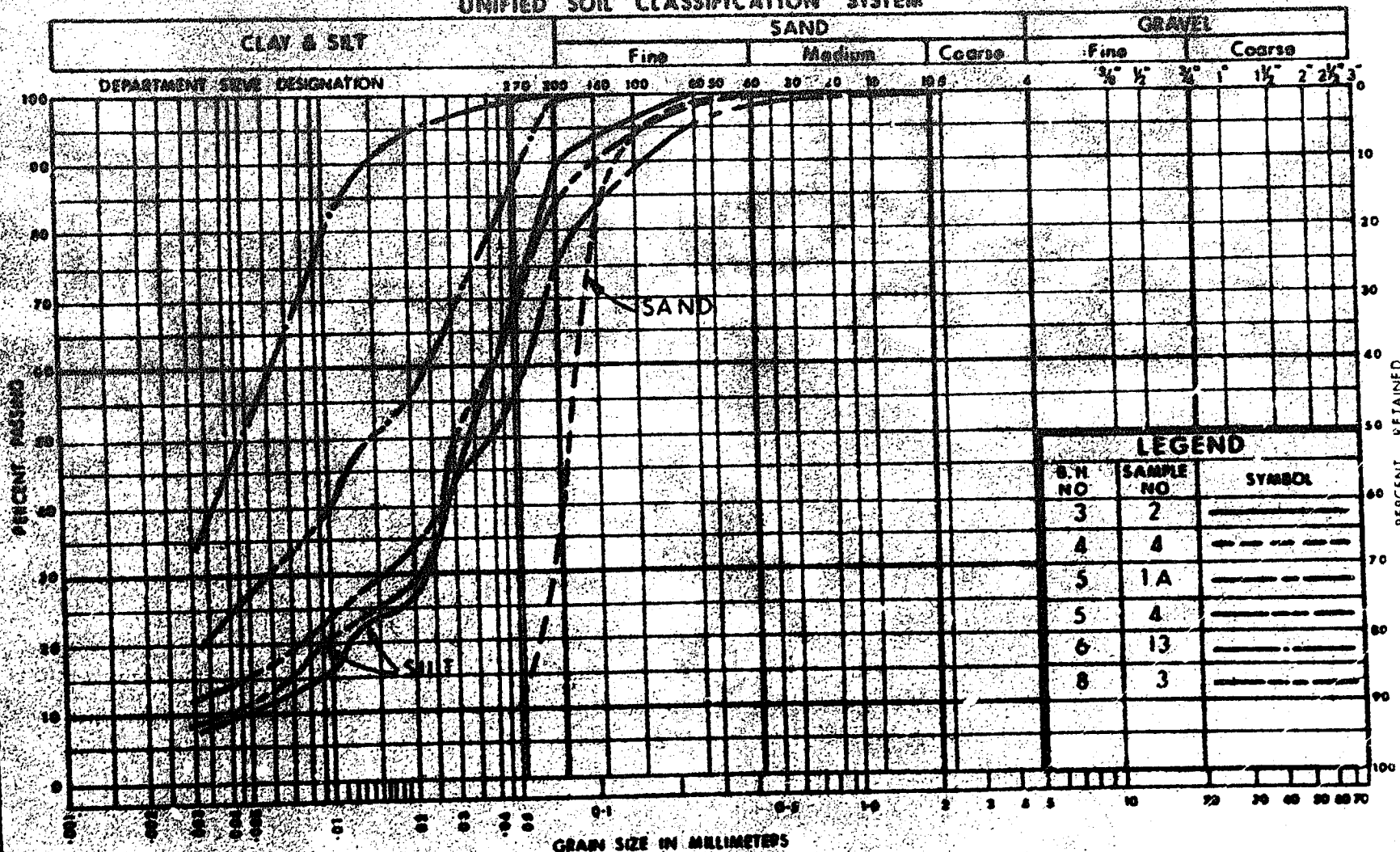
W.P. No. 105-70-04

JOB No. 70-11089

FIG. 1



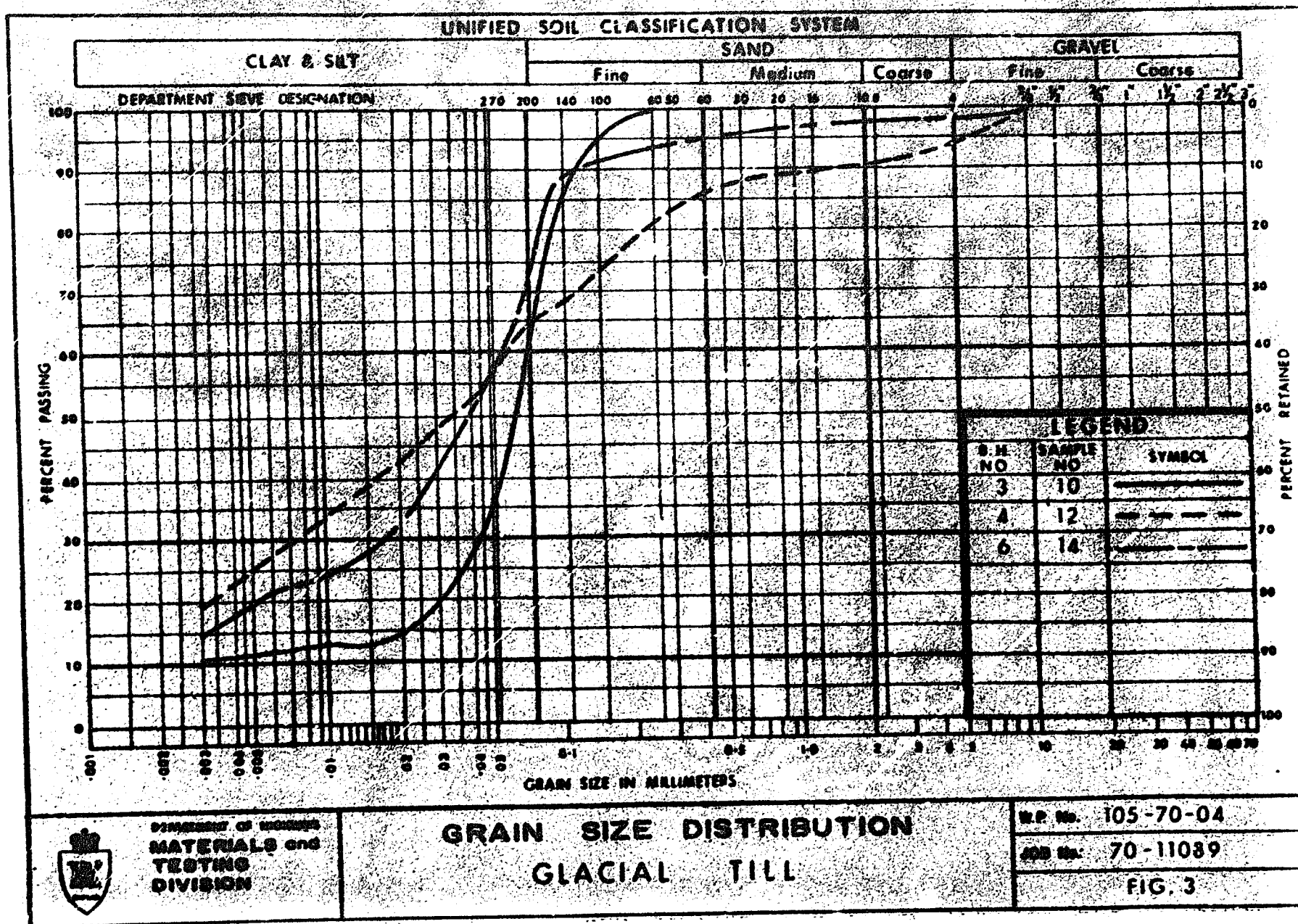
# UNIFIED SOIL CLASSIFICATION SYSTEM

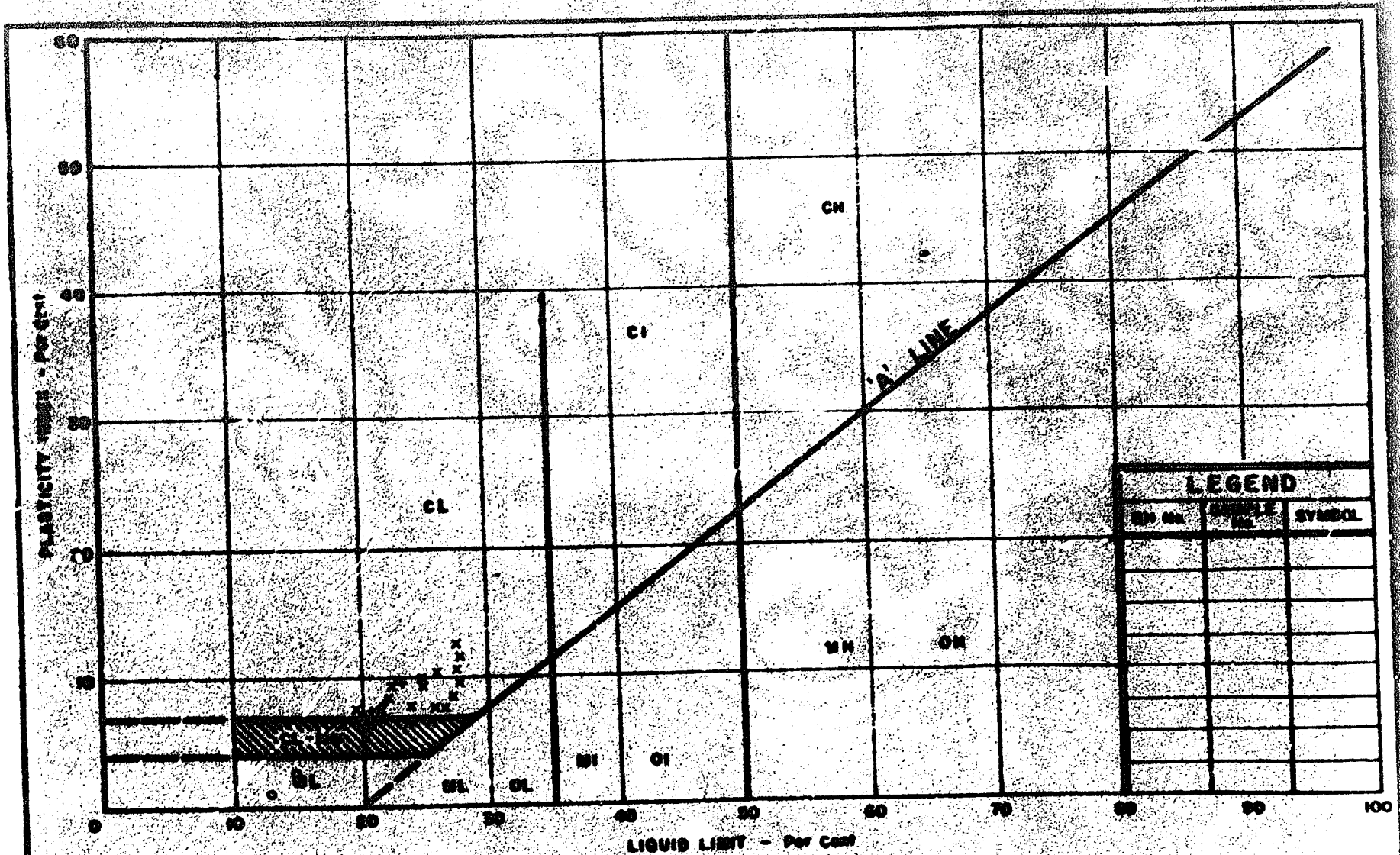


DEPARTMENT OF PUBLIC WORKS AND ENGINEERING  
MATERIALS AND TESTING DIVISION

## GRAIN SIZE DISTRIBUTION CLAYEY SILT

W.P. No. 105-70-04  
JOB No. 70-11089  
FIG. 2





DEPARTMENT OF DEFENSE  
 MATERIALS and  
 TESTING  
 DIVISION

# PLASTICITY CHART

FILL (CLAYEY SILT) — +      CLAYEY SILT STRATUM — x  
 GLACIAL TILL — o

SP. No. 105-70-04

JOB No. 70-11089

FIG. 4

# VOID RATIO-PRESSURE CURVES

JOB NO. 70-11085

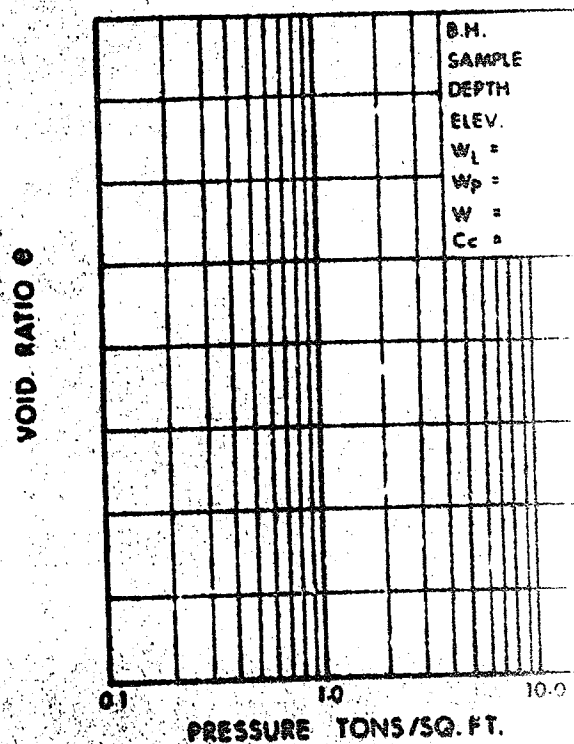
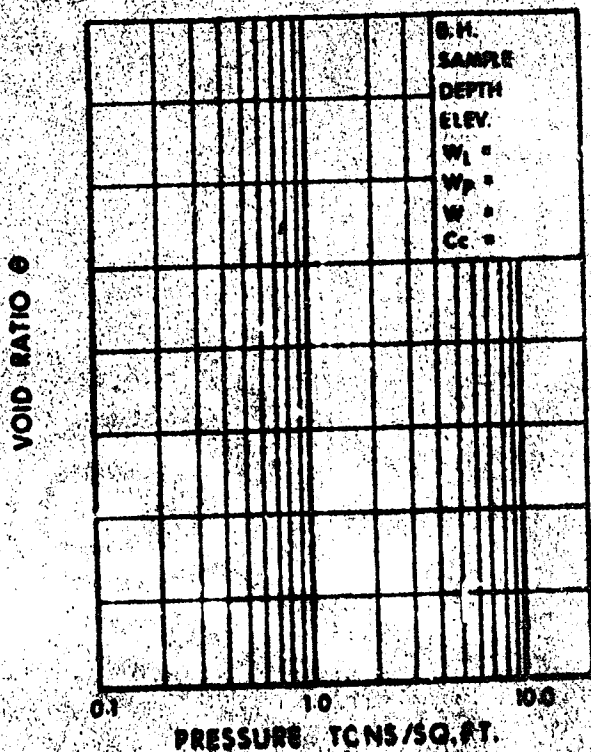
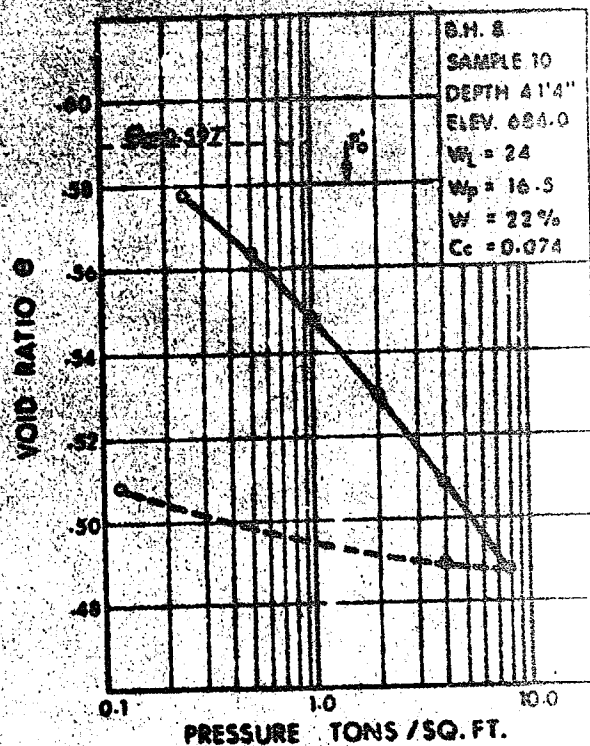
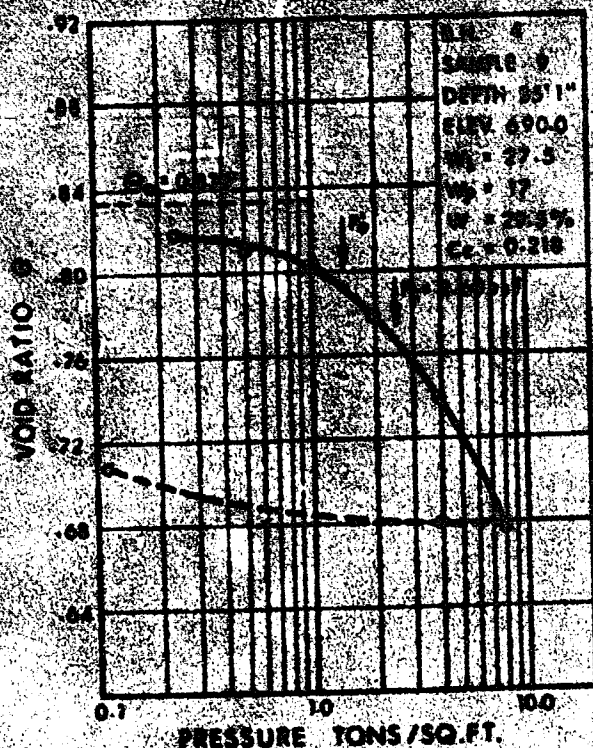


FIG. 5