

GEOCRES No:  
31G-202



**Golder Associates Ltd.**  
CONSULTING ENGINEERS

REPORT ON

*CONT 96-59*

FOUNDATION INVESTIGATION  
PROPOSED COUNTY ROAD 19 UNDERPASS  
HIGHWAY 416  
W.P. 373-89-04, SITE 16-320  
DISTRICT 9 (OTTAWA) EASTERN REGION  
*GEOCRES # 31G-202*

Submitted to:

McCormick Rankin  
1148 Hunt Club Road  
Ottawa, Ontario  
K1V 0Y3

Distribution:

2 copies - McCormick Rankin  
12 copies - Ministry of Transportation Ontario  
2 copies - Golder Associates Ltd.

July 1991

901-2064B-2

TABLE OF CONTENTS

	<u>Page No.</u>
1. INTRODUCTION	1
2. SITE DESCRIPTION AND GEOLOGY	1
3. PROCEDURE	2
4. SUBSURFACE CONDITIONS	3
4.1 General	3
4.2 Topsoil, Fill	4
4.3 Sandy Silt, Silty Sand, Sand	5
4.4 Sensitive Silty Clay	5
4.5 Sandy Silt with Clay, Gravel, Cobbles and Boulders	6
4.6 Silt, Sand	7
4.7 Bedrock	8
4.8 Groundwater	8
5. PROPOSED COUNTY ROAD 19 UNDERPASS	9
5.1 General	9
5.2 Bridge Foundations	10
5.3 Abutment Wall Backfill and Earth Pressures	13
5.4 Approach Embankments	15
5.4.1 Settlement	15
5.4.2 Stability of Approach Embankments	18
5.4.3 Light Weight Embankment Fill	20
5.4.4 Excavation of Silty Clay and Replacement	20
5.5 Corrosion of Buried Structures	21
5.6 Construction Considerations	22

EXPLANATION OF TERMS AND SYMBOLS

RECORD OF BOREHOLE SHEETS

FIGURES 1 TO 10 AND DRAWINGS 3738904-A AND 3738904-B

APPENDIX A AND B

LIST OF FIGURES

1. Plasticity Chart - Silty Clay
2. Plasticity Chart - Silty Clay
3. Grain Size Distribution - Silty Sand
4. Grain Size Distribution - Sandy Silt
5. Grain Size Distribution - Sandy Silt trace clay (glacial till)
6. Grain Size Distribution - Silt, trace to some sand and gravel
7. Void Ratio - Pressure Curves, Consolidation Test
8. Void Ratio - Pressure Curves, Consolidation Test
9. Void Ratio - Pressure Curves, Consolidation Test
10. Summary of Vane Shear Strength vs. Elevation

## 1. INTRODUCTION

Golder Associates Ltd. has been retained by McCormick Rankin, consultants to the Ministry of Transportation Ontario (MTO), to carry out a subsurface investigation at the site of a proposed underpass for County Road 19 at Highway 416 (see key plan on Drawing 3738904-A). The purpose of this investigation was to determine the subsurface conditions at the site of the proposed underpass and approach embankments between Stations 39+800 and 40+200 along County Road 19.

## 2. SITE DESCRIPTION AND GEOLOGY

The site is located along Highway 16 some 160 metres to the southeast of the Rideau River. The topography across the site is relatively flat, although the existing County Road 19 and Highway 16 are raised relative to the adjacent existing ground surface. The area across the underpass and approach embankment site presently consists of cleared land, except for a treed section east of County Road 19 and south of Highway 16. Drainage ditches are present along both sides of County Road 19; several corrugated steel pipe drains cross the highway in the proposed bridge and embankment area.

A previous preliminary subsurface investigation was carried out by MTO along County Road 19 at Highway 16. The results of that work are provided in MTO report entitled "Preliminary Foundation Investigation Report for Proposed Underpass at the Crossing of Highway #416 and Relocated County Road #19, County of Grenville - Township of South Gower, District No. 8 (Kingston) W.O. 70-11078, W.P. 6-66", dated 1970. The subsurface conditions in this area were shown to consist of thin surficial deposits of sand, followed by a thin layer of clayey silt, followed by a thick deposit of firm to stiff, sensitive, grey silty clay. The silty clay was indicated to be underlain by glacial till extending to about 23 metres below ground surface, followed by very dense silt. Dolomitic limestone bedrock was previously encountered at about 25 to 26 metres below ground surface.

### 3. PROCEDURE

The field work for this investigation was carried out between May 2 and November 8, 1990. During this time, five (5) boreholes, numbered 1-1 to 1-5 inclusive, were advanced in the area of the proposed bridge, and seven (7) boreholes, numbered 1-6 to 1-12 inclusive, were put down within about 150 metres of the proposed bridge abutments for embankment design purposes. The boreholes were put down using a track mounted hollow stem auger drill rig. Two of the boreholes put down in the area of the bridge abutments and the borehole put down at the centre pier were advanced to bedrock and the bedrock was core drilled using BXL size diamond drilling equipment. The other two boreholes advanced in the abutment areas were taken to practical auger refusal. The boreholes advanced for embankment design purposes were terminated in glacial till at depths ranging from 11.3 to 14.3 metres below ground surface. Standard penetration and in-situ vane shear strength tests were carried out in the boreholes and samples of the soils encountered were recovered using drive open sampling equipment. In addition, relatively undisturbed 73 millimetre diameter Shelby tube samples of the silty clay were recovered for oedometer consolidation testing. Standpipes were sealed into most of the boreholes to determine the groundwater levels at the site. One groundwater sample was obtained from borehole 1-4 and was sent to a laboratory for basic chemical testing relating to corrosion of buried steel and concrete. The field work was supervised throughout by a member of our engineering staff.

Samples of the soil and bedrock encountered in the boreholes were taken to our laboratory for examination and classification testing. Samples of the soil were tested for moisture content, liquid and plastic limit, and grain size distribution. Oedometer consolidation tests were carried out on three samples of the sensitive, grey silty clay.

Logs of the soil, bedrock, and groundwater conditions encountered in boreholes 1-1 to 1-12, inclusive are provided on the Record of Borehole sheets following the text of this report. The locations of the boreholes are given on the Borehole Location and Soil

Strata, Drawing 3738904-A. The subsurface profile together with the proposed bridge and approach embankment grades are provided on Drawings 3738904-A and 3738904-B. The results of the laboratory and field testing are provided on Figures 1 to 10, inclusive and on the Record of Borehole sheets. The results of the chemical analysis on the groundwater sample are shown on the Report of Analyses No. A1-0472 provided in Appendix A.

The borehole locations and elevations were determined by McCormick Rankin. The elevations are referenced to Geodetic datum.

#### 4. SUBSURFACE CONDITIONS

##### 4.1 General

As previously indicated, the detailed soil, bedrock and groundwater conditions determined from the boreholes are given on the Record of Borehole sheets following the text of this report.

The borehole logs indicate the approximate subsurface conditions at the specific test locations only. Boundaries between zones on the logs are often not distinct, but rather are transitional and have been interpreted. The precision with which subsurface conditions are indicated depends on the method of boring, the frequency of sampling, the method of sampling and the uniformity of the subsurface conditions. Subsurface conditions between the boreholes may vary significantly from conditions encountered at the boreholes.

Groundwater conditions described in this report refer only to those observed at the place and time of observation noted in the report. These conditions may vary seasonally or as a consequence of construction activities.

The soil and bedrock descriptions in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification and identification of soil and bedrock involves judgement and Golder Associates Ltd. does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice.

In general, the site was found to be underlain by surficial deposits of roadway sand and gravel fill, followed by thin deposits of sandy silt, silty sand and sand, followed by thick deposits of sensitive, grey silty clay. Beneath the silty clay, sandy silt containing clay, gravel, cobbles and boulders (glacial till) was encountered. In the area of the proposed underpass, deposits of silt containing sand, gravel, cobbles and boulders and deposits of sand were encountered beneath the glacial till and above dolomitic limestone bedrock. For the most part, the groundwater levels were found to be at about 0.7 to 1.9 metres below existing ground surface (elevation 86.0 to 87.2 metres).

The following sections present descriptions of the soil, bedrock, and groundwater conditions encountered in the boreholes.

#### 4.2 Topsoil, Fill

Surficial deposits consisting of roadway fill were encountered at all of the borehole locations. The fill is composed of sand and gravel containing a trace to some silt, and some cobbles. The fill was found to have a thickness of about 0.3 to 1.8 metres and extends to between elevation 85.8 and 87.3 metres. Standard penetration tests carried out in the sand and gravel fill at boreholes 1-1 and 1-4 gave N values of 10 and 14 blows per 0.3 metres, which reflect a compact relative density.

Borehole 1-8 encountered a 0.9 metre thick deposit of sandy silt (non cohesive) topsoil beneath the surficial fill. A standard penetration test in the topsoil at this location gave an N value of 10 blows per 0.3 metres, which reflects a compact relative density.

#### 4.3 Sandy Silt, Silty Sand, Sand

Deposits composed of sandy silt, silty sand, and/or sand were encountered beneath the surficial fill and/or topsoil at the borehole locations. These deposits were found to have a thickness of about 0.8 to 2.2 metres and to extend to depths of 2.1 to 4.0 metres below ground surface (elevation 84.5 to 85.9 metres). Grain size distribution curves for samples of the silty sand and sandy silt are provided on Figures 3 and 4, respectively. Standard penetration testing carried out within these deposits gave N values of 5 to 13 blows per 0.3 metres, which reflect a loose to compact relative density. The moisture content of these deposits ranges from 21 to 32 percent.

#### 4.4 Sensitive Silty Clay

The surficial fill/topsoil and sandy deposits are underlain by a thick deposit of sensitive, grey silty clay. The silty clay was found to have a thickness of 7.6 to 10.0 metres and to extend to depths of 9.8 to 13.0 metres below ground surface (elevation 75.5 to 77.5 metres). With the exception of boreholes 1-5 and 1-6, the grey silty clay was found to be mottled with black organic material. Atterberg limit tests carried out on samples of the silty clay gave liquid limit values ranging from 28 to 49 percent and plasticity indices of 11 to 23, which indicate a clay of low to intermediate plasticity. The Atterberg limit test results are summarized on the Plasticity Chart, Figures 1 and 2. The moisture content of the grey silty clay ranges from about 30 to 62 percent, which is near or above the measured liquid limit.

In situ vane testing carried out in the silty clay generally gave shear strengths of about 20 to 30 kilopascals between elevation 85.0 and 80.5 metres, and about 24 to 60 kilopascals below elevation 80.5 metres. The wide range in vane shear test results could be due to the presence of sandy silt seams in this deposit. A summary of the vane shear strength information is provided on Figure 10. Based on the remoulded vane shear



strengths, the sensitivity of the silty clay was found to range from about 1 to 31; the lower sensitivity values may be due to the presence of sandy silt seams in the silty clay.

Three oedometer consolidation tests were carried out on relatively undisturbed Shelby tube samples of the silty clay. The results of the consolidation tests are provided on the Void Ratio - Pressure Curves, Figures 7 to 9, inclusive. This testing shows that the apparent past preconsolidation pressure for the samples ranges from about 110 to 135 kilopascals, which is about 45 to 75 kilopascals in excess of the existing overburden pressure at the respective sample depths and locations.

#### 4.5 Sandy Silt with Clay, Gravel, Cobbles and Boulders

A deposit of sandy silt containing clay, gravel, cobbles and boulders (glacial till) was encountered beneath the silty clay at all of the borehole locations. Where the deposit was fully penetrated with the sampling equipment (boreholes 1-1, 1-2, 1-4), the glacial till was found to have a thickness of about 11.1 to 12.8 metres and to extend to depths of about 21.8 to 22.9 metres below ground surface (elevation 64.4 to 65.6 metres). Auger refusal was encountered at boreholes 1-3 and 1-5 at depths of 14.0 to 20.5 metres (elevation 66.8 to 74.2 metres) within this deposit.

The glacial till is slightly cohesive and consists of a heterogeneous mixture of all grain sizes but may be generally described as a sandy silt containing clay, gravel, cobbles, and boulders. The results of grain size distribution tests carried out on samples of the glacial till are given on Figure 5. It should be noted that the gradation tests were carried out on 38 millimetre I.D. split barrel samples and so do not reflect the presence of cobbles and boulders which exist within the glacial till.

Standard penetration tests carried out within the glacial till at the proposed bridge location gave N values of 4 to over 100 blows per 0.3 metres, which reflect a variable,

loose to very dense relative density. The average penetration resistance N value obtained in this deposit is 21 blows per 0.3 metres.

In two out of the three boreholes taken to bedrock, it was necessary to use diamond drilling techniques in the glacial till due to the presence of cobbles and boulders.

The moisture content of the glacial till is between 5 and 12 percent.

#### 4.6 Silt, Sand

Deposits of silt and sand were encountered beneath the glacial till deposits and above bedrock at boreholes 1-1, 1-2 and 1-4. These deposits have a thickness of between about 2.5 and 3.9 metres. At borehole 1-1, the deposit consists of sand with trace amounts of gravel and silt, whereas at boreholes 1-2 and 1-4, the deposit is composed of silt with gravel, sand, and occasional silty clay seams. Grain size distribution curves for samples of the silt encountered at boreholes 1-2 and 1-4 are provided on Figure 6. It should be noted that the grain size analyses were carried out on standard split barrel samples of the material and so do not reflect the presence of cobbles and boulders which exist within this deposit.

Standard penetration testing carried out within the silt gave N values of 67 to over 100 blows per 0.3 metres which reflect a very dense relative density. For the sand deposit at borehole 1-1, the split spoon sank under the weight of the rods, possibly due to some disturbance of the deposit during sampling due to an upward groundwater seepage into the open auger.

Diamond drilling techniques were required to penetrate the cobbles and boulders encountered in the silt at borehole 1-4.

#### 4.7 Bedrock

The three cored boreholes at the abutment and pier locations encountered dolomitic limestone bedrock at depths of 25.1 to 26.4 metres below ground surface (elevation 61.7 to 61.9 metres).

The bedrock consists of fresh, thinly to thickly bedded, dolomitic limestone with occasional shaly limestone layers, typical of the Oxford formation.

A measure of the quality of the bedrock recovered from the boreholes is shown on the Record of Borehole sheets as the percent core recovery (REC) and Rock Quality Designation (RQD). Except for the upper, fractured bedrock zone at borehole 1-4, the core loss was low, resulting in core recovery values of 98 to 100 percent. The RQD values range from 45 to 91 percent, which reflect a poor to excellent quality bedrock.

#### 4.8 Groundwater

Groundwater levels were obtained from standpipes sealed in the completed boreholes. On November 14, 1990, the groundwater levels in the silty clay were found to range from 0.7 to 2.1 metres below existing grade (elevation 86.0 to 87.2 metres). Multiple standpipe installations in the silty clay indicate a slight downward hydraulic gradient in this deposit. The water levels in the glacial till at borehole 1-3 and in the underlying silt at borehole 1-2 deposit were found to be at 5.2 and 1.2 metres below ground surface, respectively. It should be noted that the open hole water level in borehole 1-3 was found to be at 0.9 metres below ground surface and, therefore, the observed water level in the standpipe of 5.2 metres below ground surface may be anomalous.

The groundwater levels are expected to be higher during wet periods of the year such as the early spring.

The results of the chemical analyses on the groundwater sample from this site are as follows:

pH	- 7.70
Conductivity	- 1920 umhos per centimetre
Soluble Sulphate (SO <sub>4</sub> )	- 40 milligrams per litre (mg/L)
Soluble Chloride (Cl)	- 386 milligrams per litre (mg/L)

## 5. PROPOSED COUNTY ROAD 19 UNDERPASS

### 5.1 General

This section of the report provides engineering guidelines on the geotechnical design aspects of the project based on our interpretation of the borehole information and project requirements. It is stressed that the information in this portion of the report is provided for the guidance of the design engineers. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction and make their own interpretation of the factual data as it affects their proposed construction techniques, schedule and equipment capabilities.

It is understood that the proposed underpass will consist of a two span concrete bridge structure having a total length of 79 metres and a width of 12 metres. The proposed maximum roadway level is understood to be elevation 95.0 metres, which will result in approach embankments with a height of up to about 8.0 metres above existing ground surface (elevation of intersection of County Road 19 and existing Highway 16 is about 87.0 metres). The abutments for the bridge will likely be perched above ground surface.

## 5.2 Bridge Foundations

Due to the relatively low shear strength of the sensitive silty clay, the shallow overburden deposits are not considered suitable for the support of the proposed underpass structure on conventional spread footings. It is recommended therefore that the proposed bridge be founded on deep foundations, such as driven piles deriving support in end bearing.

Since the glacial till and underlying silt deposit contain cobbles and boulders, a heavy steel H pile is suggested to minimize pile bending problems. The piles should be equipped with a cast steel driving shoe (such as those manufactured by Titus Steel Company or by Associated Pile and Fitting Corporation) to minimize damage to the tips of the piles.

The piles should be driven to refusal on bedrock. Based on the borehole information, the bedrock elevation at the abutment and pier locations can be taken as follows:

South Abutment - 61.7 metres

Centre Pier - 61.9 metres

North Abutment - 61.8 metres

The piles should be set in accordance with standard MTO procedure using the Hiley Formula as outlined on the attached Standard Drawings SS103-10 and SS103-11 in Appendix B. As a design example, for an HP310 x 110 steel H-pile, the SLS and ULS loads could be taken as 1150 and 1600 kilonewtons, respectively if the H-piles are set to a termination of 10 blows for the last 12 millimetres of penetration using a hammer transferring about 60 kilojoules of energy per blow to the pile. Depending on the rate of resistance build-up, and the tip elevation of the piles, it may be necessary to restrike some of the piles at least once to confirm the set.

The lateral capacity of HP310 x 110 steel H-piles could be taken as 70 and 50 kilonewtons for ULS and SLS conditions, respectively.

Skin friction loads can be induced on the abutment piles due to consolidation settlement of the silty clay beneath the approach embankments. The negative skin friction loads on the piles can be determined using an effective stress approach as described in the Commentary to the Ontario Highway Bridge Design Code (OHBD) Section C6-8.3.3.2.

The skin friction force per unit area of pile can be determined by:

$$f_s = 0.25 p'_z$$

where  $p'_z$  = unit effective vertical stress at depth  $z$ .

For design purposes, the effective unit weight of the grey silty clay can be taken as 7 kilonewtons per cubic metre.

The negative skin friction loads should be assumed to act only with the permanent (dead) loads.

The skin friction load depends to some extent on the adjacent embankment loading. For instance, if the embankment loading is 70 kilopascals adjacent to the abutments, the negative skin friction load on a HP310x110 steel H pile driven through 8 metres of silty clay could be taken as 360 kilonewtons at Serviceability Limit States and 450 kilonewtons at Ultimate Limit States. The negative skin friction load can be reduced by coating the pile section within silty clay with an asphalt emulsion (such as Bakelite 700-01). Further recommendations could be provided if required.

For snow cleared areas, the pile caps should be provided with at least 1.8 metres of earth cover for frost protection purposes.

Alternatively, deep foundations, such as socketed concrete piles (caissons), or cast in place concrete piers which derive support from end bearing could also be considered. Although lateral soil movement can also negatively affect steel piles, caisson foundation types are relatively rigid compared with steel piles and, therefore, more stringent measures would have to be taken to ensure that no significant lateral movement occurs in the silty clay beneath the abutments following foundation installation. To this end, the embankment should be pre-loaded well in advance of foundation construction (at least 1 year) and the embankment load should be such that the stress in the grey silty clay near the abutments is kept below the past preconsolidation pressure for the silty clay. The lateral and vertical movements beneath the abutment areas should be carefully monitored prior to foundation installation.

Drilled piles could consist of either large diameter shafts (piers) taken to end bearing at a shallow depth in the bedrock, or smaller diameter shafts (caissons) socketed into the bedrock. In either case, a minimum shaft diameter of about 1 metre will be required to allow for visual inspection of the bearing surface or socket. From a design point of view, the piers or caissons should be cased full length within the soil overburden to prevent soil loss and/or "necking" of the pile shaft and the casing should be seated into the bedrock to allow inspection of the bedrock. Due to the presence of cobbles and boulders, churn drilling will be required to advance the shaft casing through the glacial till and silt deposits. The bedrock socket could be formed in the underlying relatively hard bedrock type by means of a churn drill. Based on past experience with socketed piles in this rock formation, significant drilling effort will be required to form and advance the bedrock socket.

For the larger diameter piers which will derive support by end bearing on the bedrock, it is recommended that the base of the pier be keyed at least 0.3 metres into the bedrock. Provided the bedrock beneath the piers is competent and does not have significant open or soil filled seams, an Ultimate Limit States bearing pressure of 4000 kilopascals may be used for design purposes.

Socketed caissons which derive support through the concrete to bedrock bond should have a socket length to socket diameter ratio of at least 2. For design purposes, a Ultimate Limit States bond value of 1000 kilopascals could be used between concrete and bedrock for good quality, unfractured bedrock. The capacity of the caissons should be calculated on the basis of bond stress only, neglecting end bearing on the base of the sockets since most of the load will be transferred to the bedrock through adhesion between the concrete and the bedrock.

Negative skin friction forces should also be considered in the design of the abutment piers or caissons. As a design example, if the embankment load near the abutments is about 70 kilopascals, the negative skin friction load on a 1.0 metre diameter concrete pier or socketed pile could be taken as 780 kilonewtons at Serviceability Limit States and 980 kilonewtons at Ultimate Limit States.

Water inflow from seams or bedding planes in the bedrock should be expected and socket dewatering will be required if inspection and concreting are to take place in the dry. Unwatering of the piers or sockets within bedrock having a high groundwater yield could impact on nearby bedrock water wells. Alternatively, inspection could be carried out under water and concreting completed using tremie techniques.

### 5.3 Abutment Wall Backfill and Earth Pressures

The abutments should be backfilled with compacted non frost susceptible, free draining backfill such as that meeting Ontario Provincial Standard Specifications (OPSS) for Granular B Type I or II. The granular fill should extend horizontally at least 1.5 metres beyond the inside face of the abutments and should be compacted in thin lifts to at least 95 percent of standard Proctor density. If lateral movement at the top of the abutment of about 0.05 percent of the retained height is expected to occur, "active" earth pressure coefficients ( $K_a$ ) should be used in determining the horizontal load on the abutments.



If the wall movement is expected to be less, then "at rest" pressure coefficients ( $K_o$ ) should be used.

Assuming that a well graded sand and gravel backfill material meeting OPSS Granular B Type I material is used behind the abutments, a material unit weight of 21.2 kilonewtons per cubic metre may be used together with the following earth pressure coefficients in determining the lateral load on the abutments.

	Earth Pressure Coefficient
At Ultimate Limit State (ULS)	
"at rest" condition	0.55
"active" condition	0.38
At Serviceability Limit State (SLS)	
"at rest" condition	0.47
"active" condition	0.31

Earth pressure parameters for other materials could be provided if necessary.

To reduce compaction induced stress on the abutment walls, the granular fill near the abutments should be compacted with walk behind compaction equipment in accordance with present MTO procedures.

If light weight styrofoam fill is used near the abutments or wing walls, drainage of the wall section adjacent to the styrofoam could be provided by means of prefabricated wick drains, such as those manufactured by Alidrain, installed on 1 metre centres against the face of the wall.

Highway live loads should be considered on the abutments unless approach slabs are used.

## 5.4 Approach Embankments

### 5.4.1 Settlement

The following section provides design guidelines on the expected settlements for the embankment fill options which have been considered; these fill options include:

- clean sand fill
- unprocessed pelletized slag
- structural coarse slag
- light weight styrofoam

As a basis for design of the embankments, it has been assumed that the allowable post construction (in service) settlement of the roadway is about 0.15 metres. The settlements were determined assuming that the existing roadway fills would be left in place rather than excavated and replaced with embankment fill and that the proposed roadway granular fill (or its equivalent in earth fill) would be placed well in advance of final paving to allow settlement to occur under the design embankment load.

With the above post construction settlement constraint, the height that the embankment could be constructed depends on the type(s) of embankment fill material used and its bulk unit weight, and the amount of preload time available. The embankment height can obviously be increased if the preload period is increased while still maintaining the post construction settlement within acceptable limits, for any one of the material types.

As a guide to evaluating the relative costs of the various embankment design options, the following table provides the maximum possible elevation of the top of the embankment for clean sand fill, pelletized slag, and structural coarse slag for a 1, 3 and 10 year preload period.

Embankment Fill Material Type	In Situ Bulk Unit Weight (KN/m <sup>3</sup> )	Maximum Elevation of Top of Embankment for an in Service Settlement of About 0.15 Metres		
		Approximate Preload Period (Years)		
		1	3	10
Clean Sand Fill	18.6	91.5	92.3	95.0
Pelletized Slag	11.8 <sup>1)</sup>	93.2	94.4	95.0
Structural Coarse Slag	10.8 <sup>1)</sup>	93.6	95.0	95.0

The time rate of settlement of the various design options outlined above are based on settlement monitoring at the existing Rideau River crossing embankment over a 20 year period. The settlement information indicates that about 50 percent of the total settlement occurred within the first year, and 70 percent within 4 to 5 years.

As a guide to determining the amounts of additional fill that may be required due to settlement of the embankments, it is estimated that the settlement of a sand embankment constructed to elevation 95.0 metres (including roadway fills) would be about 0.9 metres. Similar embankments constructed with pelletized slag and structural coarse slag are expected to settle about 0.5 and 0.45 metres, respectively. Embankment settlement will occur over the design life of the structure and is expected to be less towards the sides of the approach fills.

For some of the above embankment fill options, the required bridge grade cannot be obtained through the use of earth or slag fill alone. In these cases, the bridge structure could either be extended or light weight fill (such as styrofoam) could be used above the earth or slag fill adjacent to the bridge abutments achieve the required grade.

- 1) Based on design information provided by MTO Foundation Design Section in memo dated April 10, 1990, RE: Lightweight Fill Requirements, Standherd Drive to Jock River, W.P. 128-87-00, Hwy 416, District 9, Ottawa.

Based on previous experience with pile supported abutments in areas of sensitive silty clay, it is recommended that the embankment loading near to the pile supported abutments and wing walls be kept below the apparent past preconsolidation pressure to reduce the potential for rotation of the abutments toward the centre of settlement. In addition, batter piles should be installed in a direction away from the bridge, as well as in the normal direction towards the centre of the bridge, to resist movement of the abutments towards the centre of settlement below the approach embankments. However, if a considerable preload period is available, it may be possible to increase the embankment loading near the abutments somewhat, provided that continued monitoring (using slope indicators and settlement plates) shows that no significant lateral displacement is occurring in the silty clay beneath the abutment area prior to foundation installation.

The following table provides the maximum embankment elevations that could be obtained near the abutments and wing walls, while maintaining the stress level ( $p_o^1 + \Delta p$ ) in the silty clay below the apparent preconsolidation pressures ( $p_c$ ):

Material Type	In Situ Bulk Unit Weight (kN/m <sup>3</sup> )	Maximum Elevation of Embankment for $p_o^1 + \Delta p < p_c$ (metres)
Clean sand Fill	18.6	90.6
Unprocessed Pelletized Slag	11.8 <sup>1)</sup>	92.7
Structural Coarse Slag	10.8 <sup>1)</sup>	93.2

- 1) Based on design information provided by MTO Foundation Design Section in memo dated April 10, 1990, RE: Lightweight Fill Requirements, Standherd Drive to Jock River, W.P. 128-87-00, Hwy 416, District 9, Ottawa.

The above embankment loading is expected to result in a long term consolidation settlement of about 150 millimetres. Based on embankment monitoring information for the existing Rideau River crossing, the time for 90 percent of the settlement to occur will be about 10 years; it is expected to take about 1 year for about 50 percent of the settlement to occur.

To maintain the embankment load near the abutments and wing walls below acceptable levels and still achieve the required fill height, light weight fill (such as styrofoam) could be used above the lower height embankment behind the abutments. Where overall rotational stability constraints dictate, the lower height fill could form part of the front embankment berm. To reduce the effect of embankment sections which impose a stress beyond the preconsolidation pressure, such embankment sections should be located at least 9 metres away from the pile supported abutments. The embankment section near the abutments should be constructed at least 6 months and preferably 1 year in advance of foundation construction.

#### 5.4.2 Stability of Approach Embankments

In evaluating the rotational stability of the proposed approach embankments, an undrained (total stress) approach was used.

Where the embankment loading is such that the stress level in the silty clay is kept below the apparent preconsolidation pressure (as discussed in section 5.4.1), the factor of safety of the approach embankment would be adequate if 2 horizontal to 1 vertical slopes are used on the sides of the embankment and in front of the abutments.

If higher embankment loadings are used, the overall embankment stability will have to be satisfied through the use of shallower side slopes or berms on the sides and front of the embankments. The following table provides a guide for the side and front slope

requirements for clean sand fill, pelletized slag, and structural coarse slag with the roadway granular materials in place.

Material Type	In Situ Bulk Unit Weight (kN/m <sup>3</sup> )	Elevation of Top of Embankment (m)	Side Slope and Berm Width* Requirements for F.S. = 1.3	Top of Berm Elevation at Embankment (m)
Clean sand Fill	18.6	90.7	2H:1V SIDE SLOPES	n/a
		91.1	4 m wide berm	89.0
		91.5	8 m wide berm	89.5
		92.3	13 m wide berm	90.0
		94.3	27 m wide berm	91.0
Pelletized* Slag	11.8 <sup>1)</sup>	92.1	2H:1V SIDE SLOPES	n/a
		93.2	6 m wide berm	90.0
		94.4	10 m wide berm	90.5
		95.0	11.5 m wide berm	90.5
Structural Coarse Slag	10.8 <sup>1)</sup>	92.6	2H:1V SIDE SLOPES	n/a
		93.6	5 m wide berm	90.0
		95.0	9.5 wide berm	90.5

\* berm width refers only to the top of the berm.

These guidelines assume that the berm fill material consists of clean sand having a unit weight of about 18.6 kilonewtons per cubic metre and that the slope of the top of the berm is not more than 5 percent.

The above berm requirements could be reduced if the embankment is constructed in stages over a period of 5 to 10 years.

- 1) Based on design information provided by MTO Foundation Design Section in memo dated April 10, 1990, RE: Lightweight Fill Requirements, Standherd Drive to Jock River, W.P. 128-87-00, Hwy 416, District 9, Ottawa.

#### 5.4.3 Light Weight Embankment Fill

As indicated previously, light weight styrofoam fill could be used to reduce the embankment loading near the pile supported abutments or to increase the embankment height without significantly affecting the settlement of the embankment. To this end, expanded polystyrene blocks, such as that produced by Durofoam (Type II with a density of 0.24 kilonewtons per cubic metre) could be considered. The styrofoam blocks should be placed on a level surface and cut/arranged to provide a staggered pattern in both horizontal directions. The styrofoam should be completely covered (both top and sides) with 8 mil thick polyethylene sheeting to prevent possible deterioration due to contact with petroleum based liquids. All of the insulation sheets should be interconnected with suitable gang nails.

In the roadway area, the insulation should be covered with a 125 millimetre thick reinforced concrete slab. Further, it is suggested that the roadway granular subbase and base layers above the styrofoam and concrete have a total thickness of at least 450 millimetres to minimize icing of the roadway and to provide adequate spread of the wheel loads onto the underlying concrete and styrofoam.

#### 5.4.4 Excavation of Silty Clay and Replacement

For cost comparison purposes, excavation and removal of the sensitive silty clay in the embankment areas could also be considered. The silty clay would have to be replaced with acceptable, compacted earth borrow material. The silty clay would either be removed in areas where the embankment loading would otherwise cause excessive settlement of the embankment, or only in the embankment area adjacent to the bridge abutments to eliminate both differential settlement between the embankment and the pile supported abutments and to eliminate the risk of rotational movement of the abutments. In the latter case, the cost of excavation and replacement of the silty clay would have to

be weighed against the cost of providing light weight styrofoam fill next to the abutments and wing walls.

For preliminary costing purposes, it is suggested that excavations up to about 5 metres below ground surface be carried out using 2 horizontal to 1 vertical or flatter side slopes. For 8 to 9 metre deep excavations, preliminary analyses indicate that the slopes could be excavated at about 3 horizontal to 1 vertical.

Groundwater inflow should be expected from the upper sandy deposits, the silty clay, and the glacial till. For the most part, it is expected that groundwater inflow could be controlled by pumping from sumps within the excavation. However, groundwater inflow from the upper sandy deposits may cause sloughing of these materials and it may be necessary in some areas to either reduce the slope angle along the upper part of the excavation or to use groundwater lowering techniques in the upper sandy deposits.

The excavation should be taken to a uniform grade across the entire width of the top of the proposed embankment. To provide a smooth transition between embankment areas underlain by native silty clay and areas where excavation and replacement is carried out, a 5 horizontal to 1 vertical (or flatter) taper is suggested.

### 5.5 Corrosion of Buried Structures

As previously indicated, the sulphate content of the groundwater was found to be 40 milligrams per litre. According to CSA CAN3 A23.1-M90, the measured level of sulphate should not be corrosive to concrete where normal (Type 10) Portland Cement is used.

Based on the measured conductivity and pH value of the groundwater, this site can be classified as slightly aggressive toward unprotected steel. Corrosion of driven piles in the native, homogenous and undisturbed soil below the groundwater level is not expected



to be a problem. However, the potential exists at this site for corrosion of the driven piles along that portion of the pile within the perched abutment fill, at the interface of the existing fills and the native subsoil, and within the groundwater fluctuation zone. To reduce this corrosion potential, it is suggested that the upper part of the piles at this site be provided with a bituminous coating (such as Bakelite 700-1) and that the pile cap be designed such that the steel pile is electrically isolated from the remainder of the bridge structure i.e. no steel to steel contact with the piles in the pile cap.

#### 5.6 Construction Considerations

During construction of the embankment, careful monitoring will be required in areas where the embankment load exceeds about 60 kilopascals. The monitoring instrumentation would include pneumatic type piezometers to measure the porewater pressure in the silty clay, slope indicators to monitor horizontal displacement in the silty clay deposit with depth, and settlement plates.

It is recommended that the pile driving equipment proposed by the contractor be reviewed in light of the contract pile type and set criteria and accepted by the geotechnical engineer well in advance of any pile driving operations. Also, all piling operations should be inspected throughout by qualified geotechnical personnel.

To facilitate pile driving, the fill material used beneath the abutments should not contain cobble or boulder size material (i.e. not larger than 75 millimetres).

The soils at this site are highly susceptible to frost heaving. Therefore, the native soils around the piles should be protected from freezing during construction to prevent pile jacking due to adfreeze effects.

A licensed welding inspector should be retained during the pile driving to periodically inspect the welding procedures used by the contractor if welded pile splices are used.

Yours truly,

GOLDER ASSOCIATES LTD.

ORIGINAL SIGNED BY

A.F. Chevrier, P.Eng.

ORIGINAL SIGNED BY

R.A. Montgomery, P.Eng.

AFC:RAM:cp  
RCP1

## EXPLANATION OF TERMS USED IN REPORT

**N VALUE:** THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS  $\bar{N}$ .

**DYNAMIC CONE PENETRATION TEST:** CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

**CONSISTENCY:** COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH ( $c_u$ ) AS FOLLOWS:

$c_u$ (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

**DENSENESS:** COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND /OR STRENGTH.

**RECOVERY:** SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

**MODIFIED RECOVERY:** SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

**JOINTING AND BEDDING:**

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

### STRESS AND STRAIN

$u_w$	kPa	PORE WATER PRESSURE
$r_u$	1	PORE PRESSURE RATIO
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### MECHANICAL PROPERTIES OF SOIL

$m_v$	kPa <sup>-1</sup>	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_\alpha$	1	RATE OF SECONDARY CONSOLIDATION
$C_v$	m <sup>2</sup> /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
$T_v$	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
$\sigma'_{vo}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_f$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\phi'$	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	-°	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_R$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_r$	kPa	REMOULDED SHEAR STRENGTH
$S_l$	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

### PHYSICAL PROPERTIES OF SOIL

$\rho_s$	kg/m <sup>3</sup>	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	$e_{min}$	1, %	VOID RATIO IN DENSEST STATE
$\gamma_s$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$\rho_w$	kg/m <sup>3</sup>	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
$\gamma_w$	kN/m <sup>3</sup>	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	n PERCENT - DIAMETER
$\rho$	kg/m <sup>3</sup>	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	kg/m <sup>3</sup>	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	q	m <sup>3</sup> /s	RATE OF DISCHARGE
$\gamma_d$	kN/m <sup>3</sup>	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
$\rho_{sat}$	kg/m <sup>3</sup>	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
$\gamma_{sat}$	kN/m <sup>3</sup>	UNIT WEIGHT OF SATURATED SOIL	$I_C$	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	kg/m <sup>3</sup>	DENSITY OF SUBMERGED SOIL	$e_{max}$	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m <sup>3</sup>	SEEPAGE FORCE
$\gamma'$	kN/m <sup>3</sup>	UNIT WEIGHT OF SUBMERGED SOIL						

# RECORD OF BOREHOLE No I-I

METRIC

W P 373-89-04 LOCATION Co-ords N4 993 586; E 373 023 ORIGINATED BY DM  
DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger, BXL Rock Core COMPILED BY AFC  
DATUM Geodetic DATE May 2, 1990 CHECKED BY AFC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
87.0	Ground Surface																
0.0	Fill, sand and gravel, trace to some silt																
85.8	Compact Brown		1	SS	14		86										
1.2	Sandy silt, some silty clay layers		2	SS	7												
84.8	Loose Grey																
2.2			3	SS	2												
	Silty clay, some sandy silt seams, trace black organic mottling		4	SS	1												
			5	TP	PH												
	Firm to stiff Grey		6	SS	PM												
			7	TP	PH												
			8	SS	PM												
77.2																	
9.8	Sandy silt and gravel, trace clay, occasional cobble and boulder (glacial till)		9	SS	18												
	Loose to compact Grey		10	SS	7												
			11	SS	10												
	Continued																

# RECORD OF BOREHOLE No I-I

METRIC

W P 373-89-04 LOCATION Co-ords N 4 993 586; E 373 023 ORIGINATED BY DM  
 DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger, BXL Rock Core COMPILED BY AFC  
 DATUM Geodetic DATE May 2, 1990 CHECKED BY AFC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100									
								SHEAR STRENGTH kPa									
								○ UNCONFINED + FIELD VANE									
								● QUICK TRIAXIAL × LAB VANE									
							20 40 60										
Continued																	
	Sandy silt and gravel, trace clay, occasional cobble and boulder  (glacial till)		12	SS	16		72										
			13	SS	14		70										
	Loose to compact  Grey		14	SS	26												
67.9																	
19.1	Sandy silt and gravel, trace clay, some cobbles and boulders (glacial till)		15	SS	54		68										
			16	SS	25		66										
64.4	Compact to very dense  Grey																
22.6	Sand, trace gravel and silt		17	SS	WR*		64										
61.9																	
25.1	Dolomitic limestone bedrock, fresh, thinly to thickly bedded, core fractured from 25.1 to 25.3 m and 25.7 to 25.9 m		18	RC BXL	Rec=100% RQD=45%		62										
			19	RC BXL	Rec=100% RQD=83%		60										
59.6																	
27.4	End of Hole  *Sank under weight of rods  **REC = Recovery RQD = Rock Quality Designation						58										

# RECORD OF BOREHOLE No 1-2

METRIC

W P 373-89-04 LOCATION Co-ords N 4 993 548; E 373 025 ORIGINATED BY DM  
DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger, BXL Rock Core COMPILED BY AFC  
DATUM Geodetic DATE October 23 to 25, 1990 CHECKED BY AFC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100		
87.4	Ground Surface							SHEAR STRENGTH kPa						
0.0	Fill, sand and gravel							O UNCONFINED + FIELD VANE						
87.1	some silt							• QUICK TRIAXIAL x LAB VANE						
0.3	Silty sand							20	40	60	80	100		
	Loose	Brown	1	SS	5			WATER CONTENT (%)						
86.2								20	40	60				
1.2	Sandy silt, some silty clay layers							Plastic Limit Liquid Limit						
85.3	Loose	Grey	2	SS	8			W <sub>p</sub>	W	W <sub>L</sub>				
2.1	Silty clay, some sandy silt seams, trace black organic mottling		3	SS	PM									
			4	SS	PM									
			5	TP	PH									
			6	SS	PM									
			7	SS	WH*									
	Soft to stiff		8	SS	WH*									
76.7														
10.7	Sandy silt some gravel to sandy silt and gravel, trace clay, some sand seams, some cobbles and boulders (glacial till)		9	SS	13									
	Loose to very dense		10	SS	4									
			11	SS	4									
	Continued													
	*Sank under weight of hammer													

OFFICE REPORT ON SOIL EXPLORATION

## METRIC

W P 373-89-04 LOCATION Co-ords N 4 993 548; E 373 025 ORIGINATED BY DM  
DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger, BXL Rock Core COMPILED BY AFC  
DATUM Geodetic DATE October 23 to 25, 1990 CHECKED BY AFC

[illegible]

+3, x5: Numbers refer to Sensitivity

20  
15   
10

## METRIC

W P 373-89-04 LOCATION Co-ords N 4 993 553; E 373 015 ORIGINATED BY DM  
DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger COMPILED BY AFC  
DATUM Geodetic DATE October 31, 1990 CHECKED BY AFC

[illegible]

OFFICE REFORMS IN SOIL EXPLORATION

+3, x<sup>5</sup>: Numbers refer to Sensitivity





## METRIC

W P 373-89-04 LOCATION Co-ords N 4 993 553; E 373 015 ORIGINATED BY DM  
DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger COMPILED BY AFC  
DATUM Geodetic DATE October 31, 1990 CHECKED BY AFC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL					
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100							WATER CONTENT (%)				
								SHEAR STRENGTH kPa							20 40 60				

+3, x5: Numbers refer to Sensitivity

# RECORD OF BOREHOLE No 1-4

METRIC

W P 373-89-04 LOCATION Co-ords: N 4 993 623; E 373 055 ORIGINATED BY DM  
 DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger, BXL Rock Core COMPILED BY AFC  
 DATUM Geodetic DATE October 26 to 29, 1990 CHECKED BY AFC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
88.2	Ground Surface																
0.0	Fill, sand and gravel, some silt																
86.7	Compact Brown		1	SS	10												
1.5	Sand, some silt becoming silty sand		2	SS	10												
85.9	Compact Brown																
2.3	Silty clay, some silty sand seams		3	SS	10												
85.3	Stiff Grey																
2.9			4	SS	PM												
			5	TP	PH												
	Silty clay, some sandy silt seams, trace black organic mottling		6	TP	PH												
			7	SS	WH*												
			8	SS	WH*												
	Soft to stiff Grey		9	SS	WH*												
			10	SS	PM												
76.6																	
11.6																	
	Sandy silt, some gravel trace clay, some cobbles and boulders (glacial till)		11	SS	50 for 0.08 m												
			12	RC BXL	Rec= 100% **												
	Loose to very dense Grey		13	SS	8												
	Continued																

+<sup>3</sup>, x<sup>5</sup>: Numbers refer to  
Sensitivity

20  
15  
10  
s (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

# RECORD OF BOREHOLE No I-4

METRIC

W P 373-89-04 LOCATION Co-ords N 4 993 623; E 373 055 ORIGINATED BY DM  
DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger, BXL Rock Core COMPILED BY AFC  
DATUM Geodetic DATE October 26 to 29, 1990 CHECKED BY AFC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100									
								SHEAR STRENGTH kPa									
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
							20 40 60 80 100					WATER CONTENT (%) 20 40 60					
Continued																	
65.3	Sandy silt, some gravel, trace clay, some cobbles and boulders (glacial till)		14	SS	8												
			15	SS	8												
			16	SS	29												
			17	RC BXL	Rec= 65%												
			18	SS	31												
			19	SS	44												
22.9	Silt, trace sand, some gravel, occasional silty clay seam, some cobbles and boulders		20	RC BXL	Rec= 85%												
21			SS	76													
22			RC BXL	Rec=30%													
23			SS	94 for 0.18 m													
24			RC BXL	Rec= 18%													
25			SS	75 for 0.08 m													
61.8	Very dense		26	RC BXL	Rec=80% RQD=83% ***												
27			RC BXL	Rec= 98% RQD= 91%													
26.4	Dolomitic limestone bedrock, fresh, medium to thickly bedded, occasional shaley limestone layer, fractured core at 26.9 metres																
58.4																	
29.8	End of hole																
*Sank under weight of hammer **Rec = Recovery RQD = Rock Quality Designation																	

+3, x5: Numbers refer to Sensitivity 20  
15 5 (%) STRAIN AT FAILURE  
10

SOIL REPORT ON SOIL EXPLORATION

# RECORD OF BOREHOLE No 1-5

METRIC

W P 373-89-04 LOCATION Co-ords N 4 993 631; E 373 042 ORIGINATED BY DM  
DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger COMPILED BY AFC  
DATUM Geodetic DATE October 30, 1990 CHECKED BY AFC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				NATURAL MOISTURE CONTENT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	W <sub>p</sub>	W	W <sub>L</sub>		
88.2	Ground Surface															
0.0	Fill, sand and gravel, some silt and cobbles					*										
87.2	Brown															
1.0	Sandy silt															
86.1	Brown															
2.1	Silty clay															
85.1	Stiff Grey															
3.1	Silty clay, some sandy silt seams		1	SS	PM											
			2	TP	PH											
	Soft to stiff Grey															
			3	SS	WH*											
77.5																
10.7	Sandy silt some gravel, trace clay, occasional cobble and boulder (glacial till)		4	SS	34											
	Dense. Grey															
74.2																
14.0	Auger refusal End of hole															
	*Water level not established **Sank under weight of hammer															

## METRIC

W P 373-89-04 LOCATION Co-ords N 4.993 655; E 373.053 ORIGINATED BY DM  
DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger COMPILED BY APC  
DATUM Geodetic DATE November 2, 1990 CHECKED BY APC

[illegible]

## METRIC

W P 173-89-04 LOCATION Co-ords N 4 993 525; E 373 005 ORIGINATED BY DM  
DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger COMPILED BY AFC  
DATUM Geodetic DATE November 1, 1990 CHECKED BY AFC

[illegible]

## METRIC

w p 373-89-04

LOCATION Co-ords N 4 993 688: E 373 077

ORIGINATED BY DM

DIST 9 HWY 416

BOREHOLE TYPE      Hollow Stem Auger

COMPILED BY AFC

DATUM Geodetic

DATE November 2, 1990

CHECKED BY AFC

[illegible]

+3, x5: Numbers refer to Sensitivity

## METRIC

W P 373-89-04

LOCATION Co-ords N 4 993 731; E 373 080

ORIGINATED BY DM

DIST 9 HWY 416

BOREHOLE TYPE      Hollow Stem Auger

COMPILED BY AFC

DATUM Geodetic

DATE      November 5, 1990

CHECKED BY AFC

[illegible]

+3, x5: Numbers refer to Sensitivity





## METRIC

W P 373-89-04 LOCATION Co-ords N 4 993 764; E 373 105 ORIGINATED BY DM  
DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger COMPILED BY AFC  
DATUM Geodetic DATE November 5 and 6, 1990 CHECKED BY AFC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH kPa					
								20 40 60 80 100	W <sub>p</sub>	W	W <sub>L</sub>		
88.1	Ground Surface												
0.0	Fill, sand and gravel, some cobbles and silt	X											
87.0	Brown	X											
1.1	Sand, some silt												
86.6	Brown												
1.5	Sandy silt, occasional silty clay seam		1	SS	8								
85.8	Loose Grey brown												
2.3	Silty clay, some silty sand and sandy silt layers		2	SS	5								
84.7	Stiff Grey												
3.4							+ S=4						
							+ S=5						
							+ S=4						
	Silty clay, some sandy silt seams, trace black organic mottling		3	TP	PM				I	O			
							+ S=10						
							+ S=11						
							+ S=9						
							+ S=16						
							+ S=6						
							+ S=9						
	Soft to firm Grey						+ S=17						
							+ S=18						
							+ S=18						
			4	SS	WR*				I	O			
							+ S=23						
							+ S=9						
							+ S=9						
			5	TP	WR*								
							+ S=18						
75.8							+ S=11						
12.3	Sandy silt some gravel, clay, cobbles & boulders (glacial till)		6	SS	75 for 0.1 m								
75.6													
12.5	End of Hole												
	*Water level not established **Sank under weight of rods												
							74						

+<sup>3</sup>, x<sup>5</sup>: Numbers refer to Sensitivity

20  
15  $\phi$  5 (%) STRAIN AT FAILURE  
10



## METRIC

LOCATION Co-ords N 4.993 483; E 373 001

ORIGINATED BY DM

BOREHOLE TYPE      Hollow Stem Auger

COMPILED BY AFC

DATUM \_\_\_\_\_ Geodetic

DATE November 6 to 8, 1990

CHECKED BY AFC

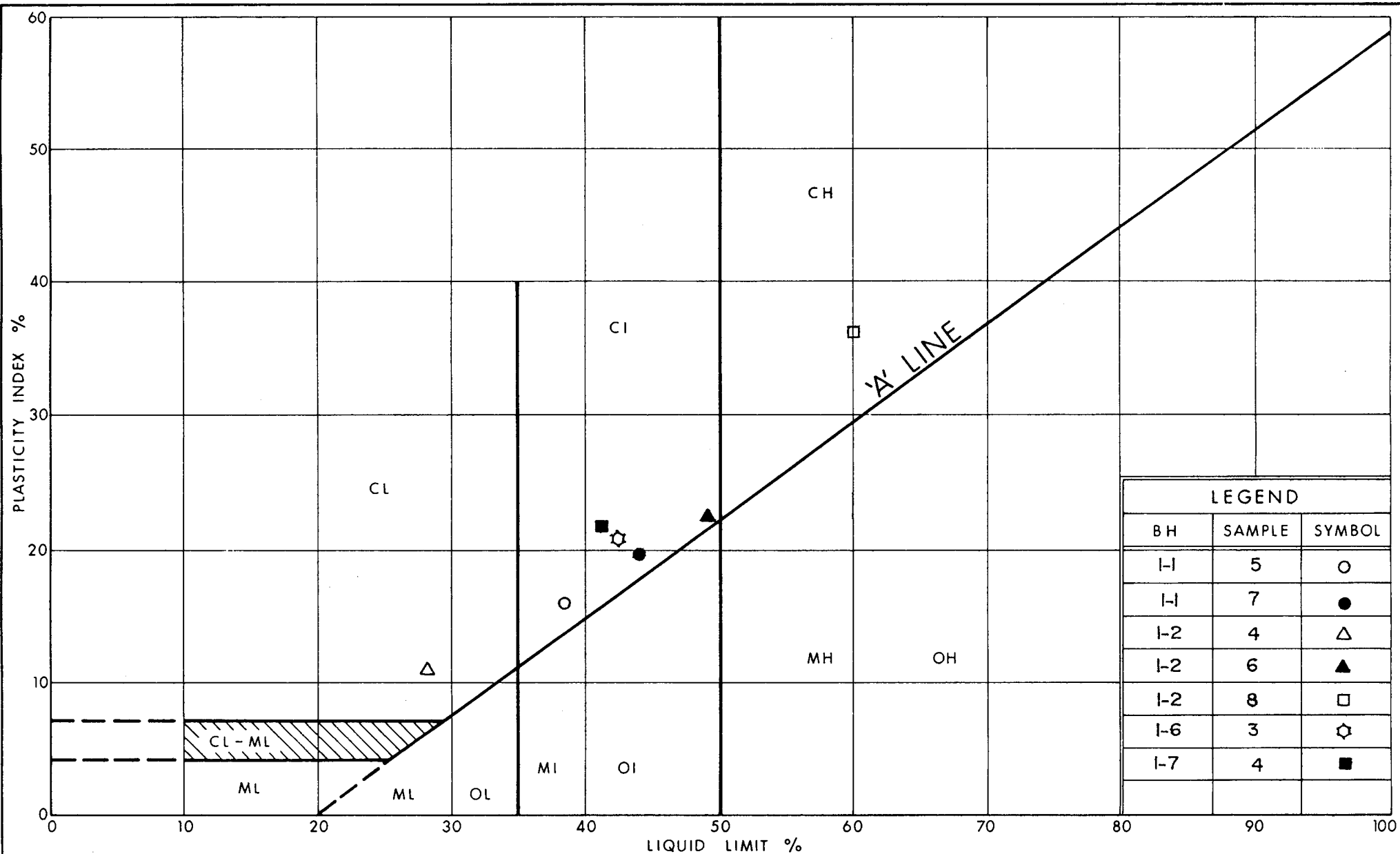
+3, x5: Numbers refer to Sensitivity

# RECORD OF BOREHOLE No I-12

METRIC

W P 373-89-04 LOCATION Co-ords N 4 993 430; E 372 970 ORIGINATED BY DM  
 DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger COMPILED BY AFC  
 DATUM Geodetic DATE November 8, 1990 CHECKED BY AFC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				NATURAL MOISTURE CONTENT			UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100				PLASTIC LIMIT W <sub>p</sub>	W	LIQUID LIMIT W <sub>L</sub>		
88.8	Ground Surface							SHEAR STRENGTH kPa				WATER CONTENT (%)				GR SA SI CL
0.0	Fill, sand and gravel, some silt and cobbles							O UNCONFINED + FIELD VANE • QUICK TRIAXIAL x LAB VANE								
87.0	Brown							20 40 60 80 100				20 40 60				
1.8	Silty sand		1	SS	12			Water level in standpipe at elev. 87.2 metres on Nov. 14, 1990				O				0 68 28 4
86.7	Brown															
2.1	Sandy silt with silty clay layers		2	SS	9			Native backfill				O				
85.7	Loose															
3.1			3	SS	1							O				
	Silty clay, some sandy silt seams trace black organic mottling							Bentonite								
								+ S=3								
								Sand								
								+ S=3								
								Standpipe								
								+ S=3								
								Bentonite								
								+ S=8								
								+ S=12								
								+ S=21								
								+ S=25								
								82								
								Native backfill								
								+ S=14								
								+ S=31								
								+ S=11								
								+ S=29								
								+ S=29								
								+ S=27								
								+ S=25								
								+ S=25								
								+ S=25								
								+ S=25								
								+ S=23								
								78								
								+ S=23								
								+ S=9								
								+ S=13								
								+ S=36								
								+ S=19								
								+ S=31								
								+ S=16								
75.8																
13.0	Sandy silt, some gravel, trace clay (glacial till)															
74.5	Compact		5	SS	10											
14.3	End of hole															
	*Sank under weight of rods															



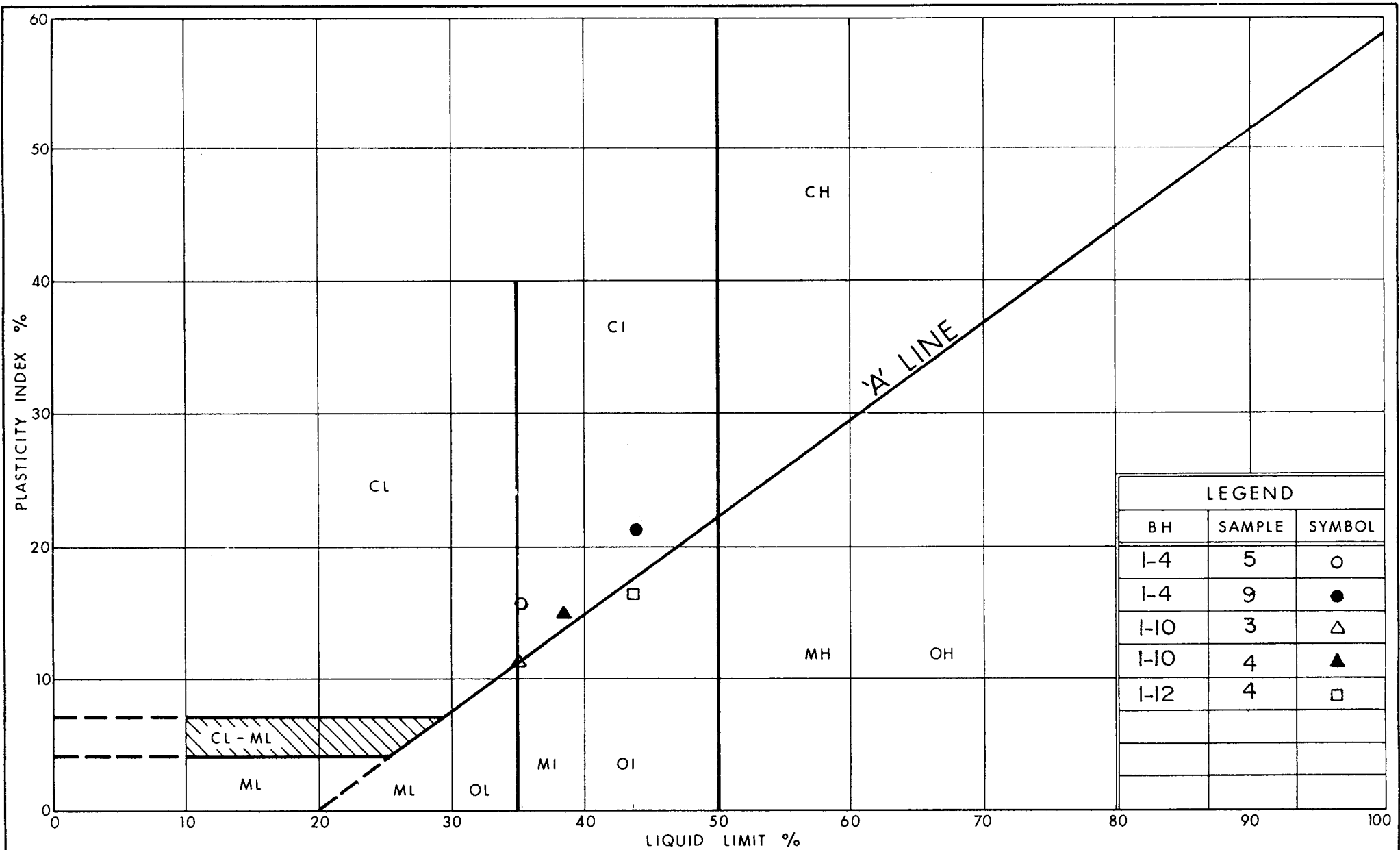
Ministry of  
Transportation

Ontario

# PLASTICITY CHART SILTY CLAY

FIG No 1

W P 373-89-04



Ministry of  
Transportation

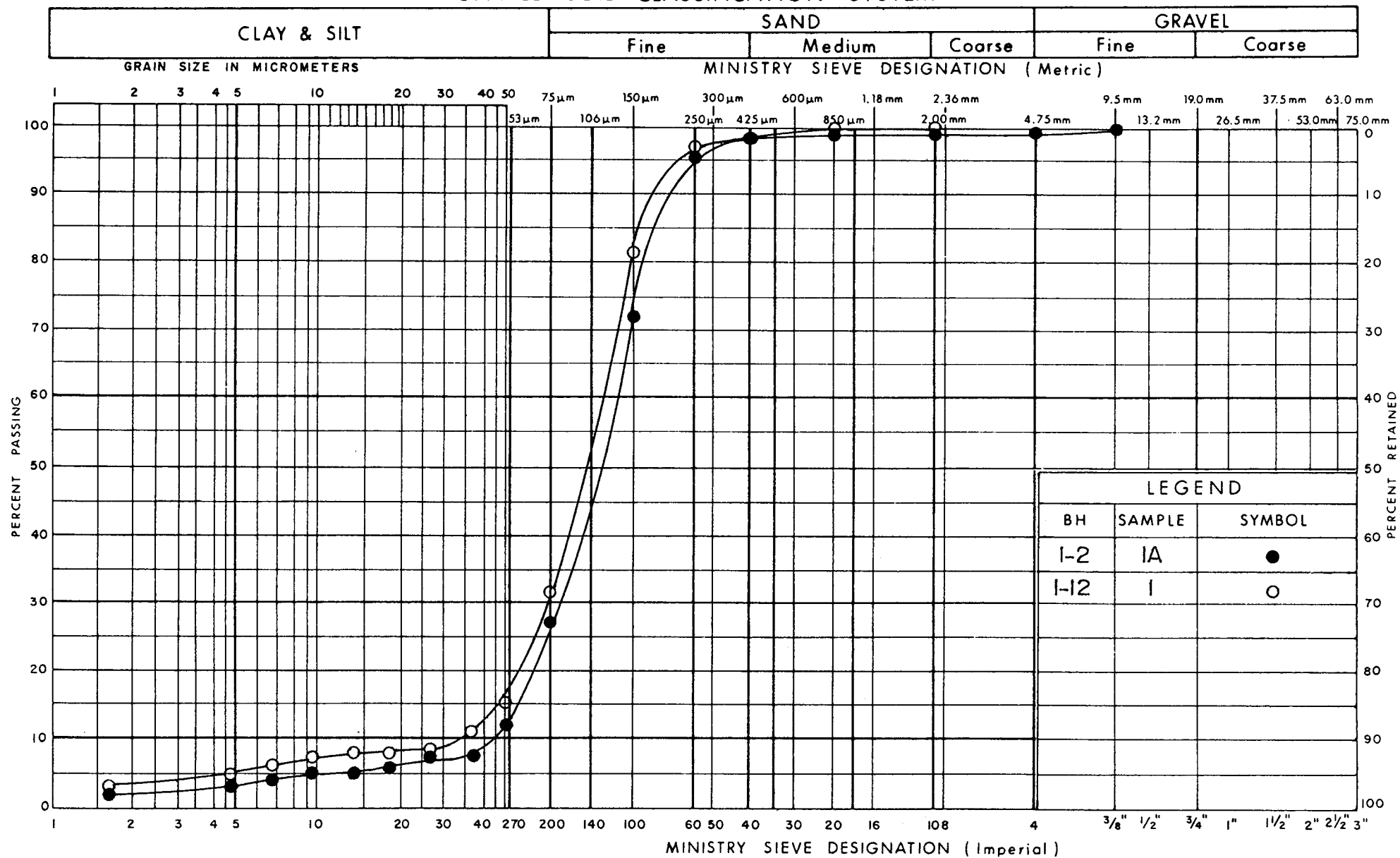
Ontario

# PLASTICITY CHART SILTY CLAY

FIG No 2

W P 373-89-04

## UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

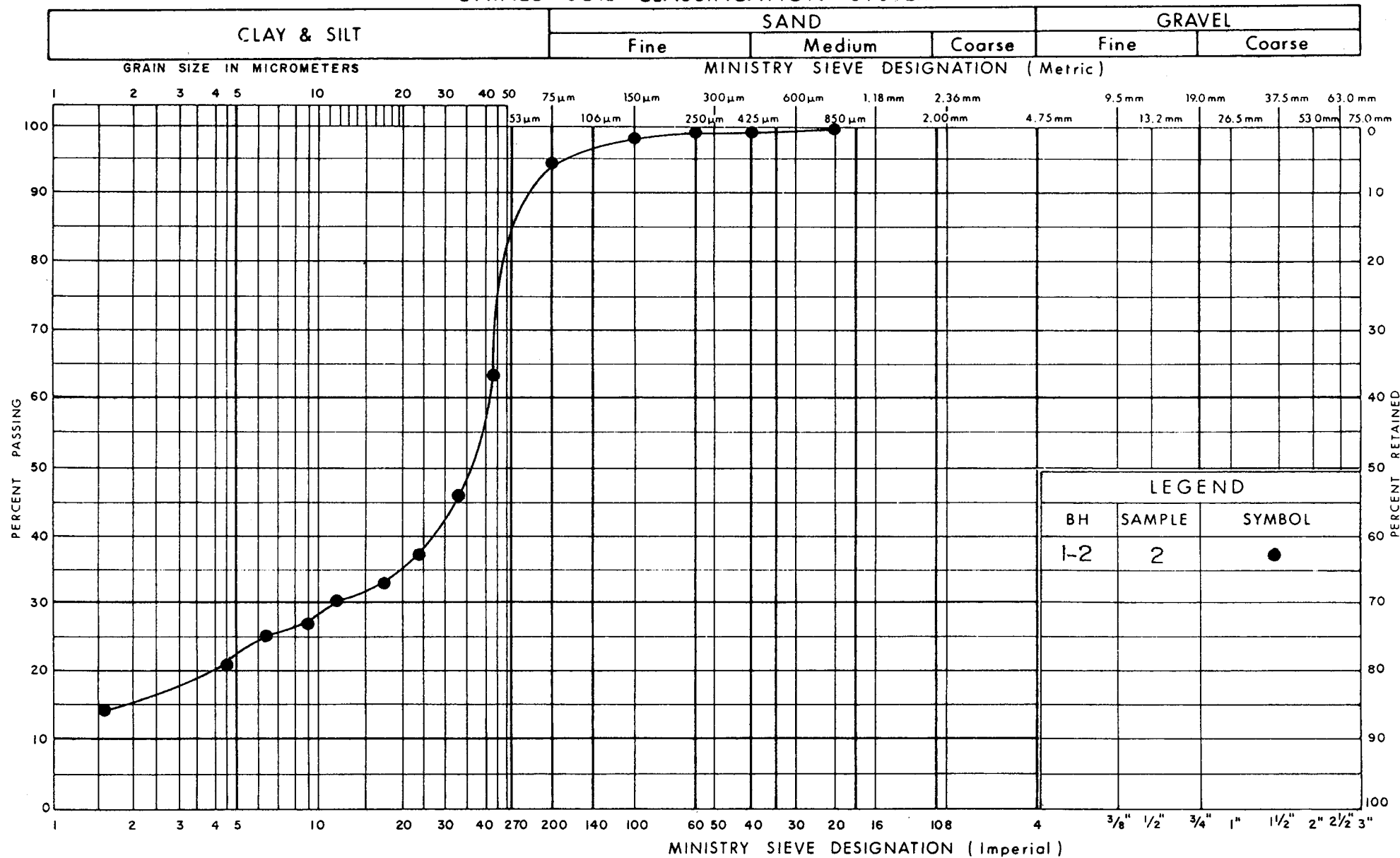
Ontario

GRAIN SIZE DISTRIBUTION  
SILTY SAND

FIG No 3

W P 373-89-04

## UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

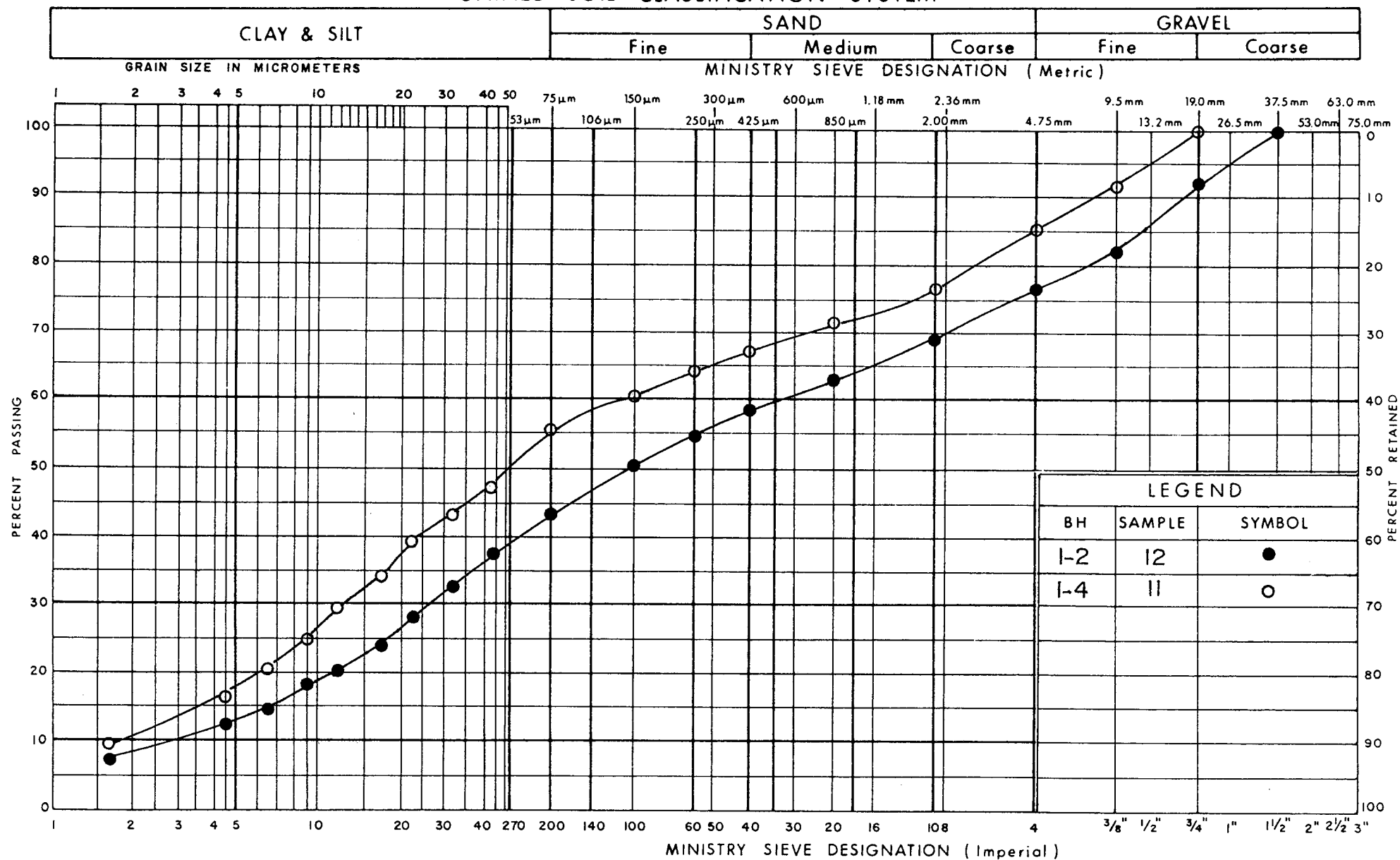
## GRAIN SIZE DISTRIBUTION

### SANDY SILT

FIG No 4

W P 373-89-04

## UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

## GRAIN SIZE DISTRIBUTION

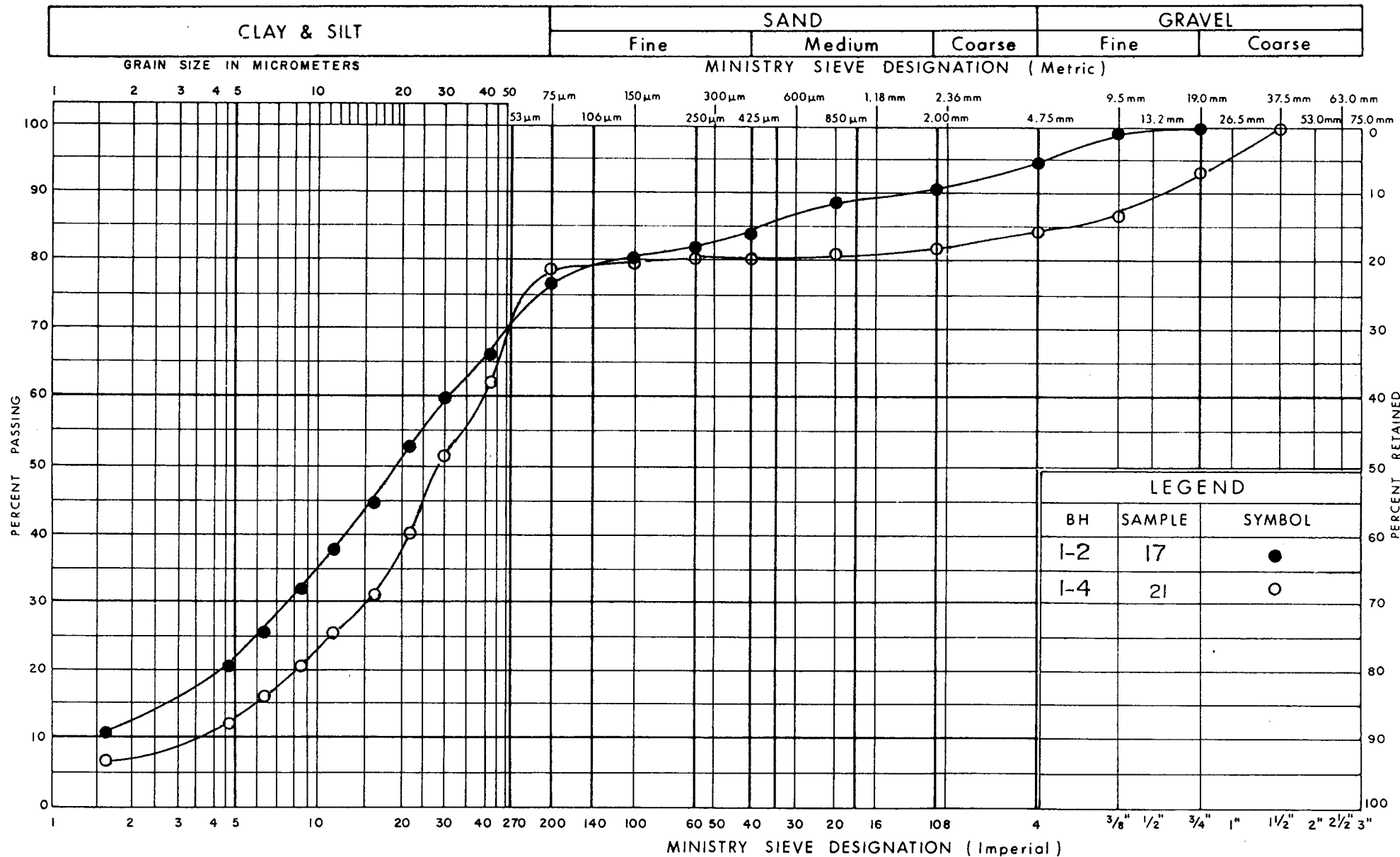
SANDY SILT, trace clay  
(GLACIAL TILL)

FIG No 5

W P 373-89-04



## UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

## GRAIN SIZE DISTRIBUTION

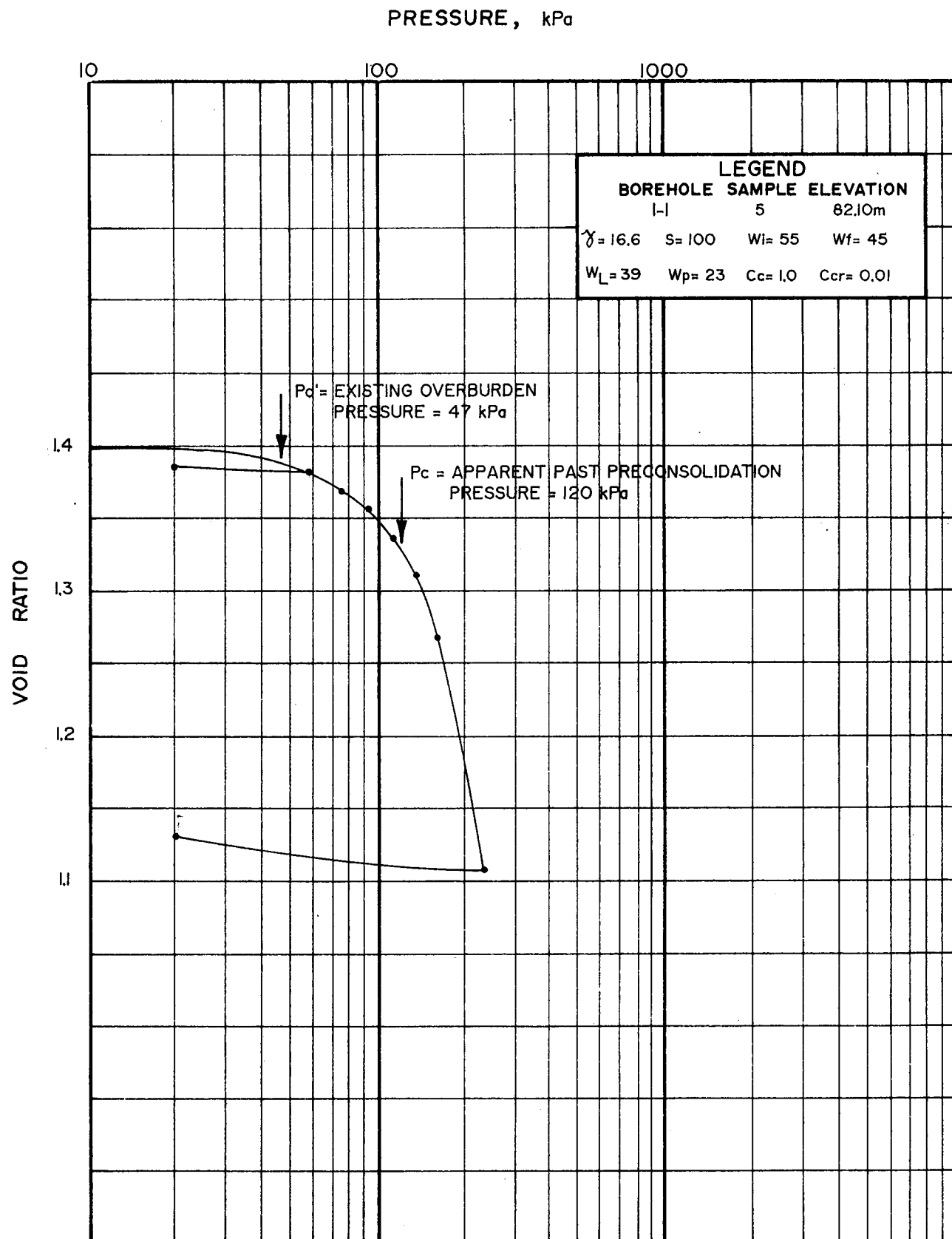
SILT, trace to some  
sand and gravel

FIG No 6

W P 373-89-04

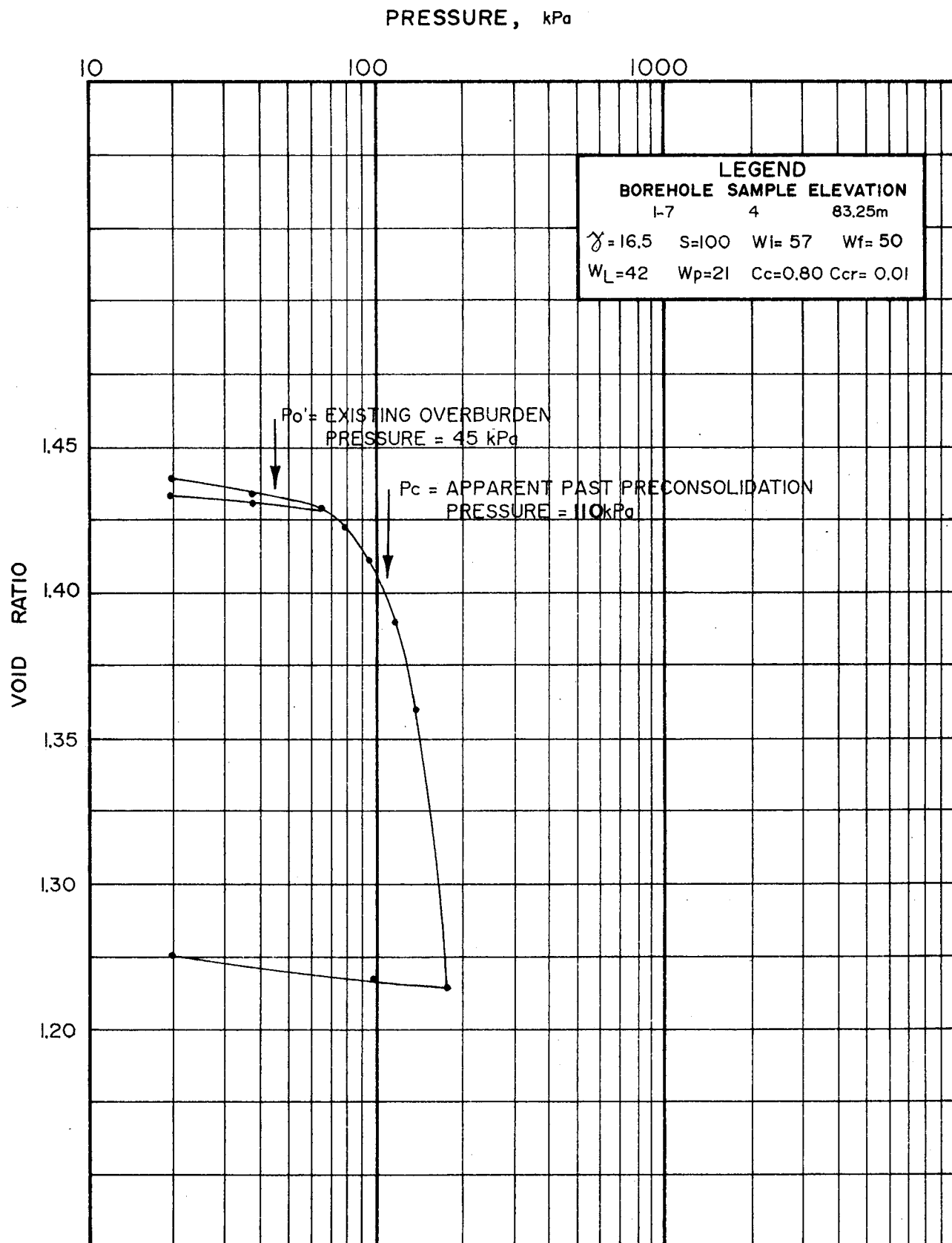
# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 7  
WP 373-89-04



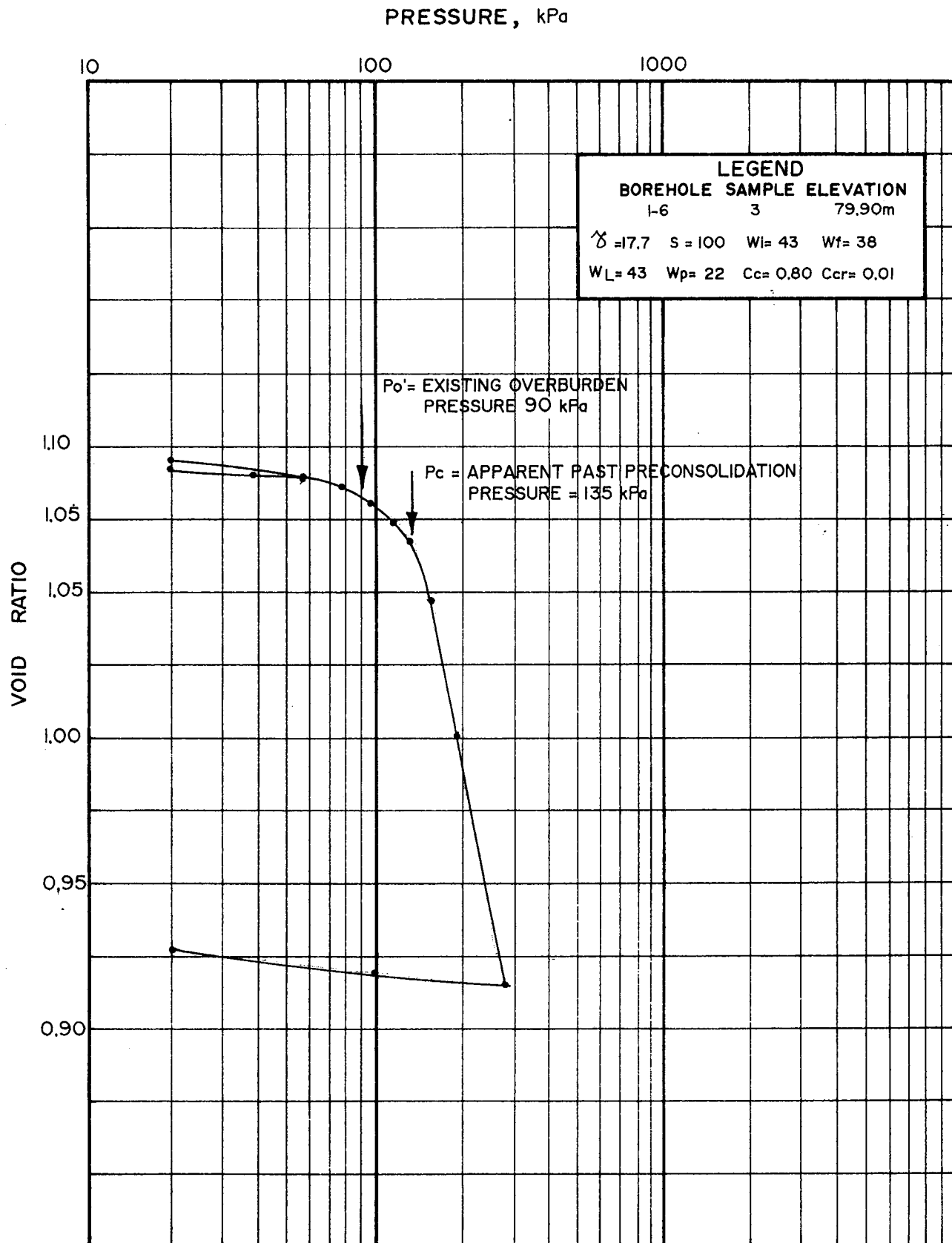
# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 8  
WP 373-89-04



# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

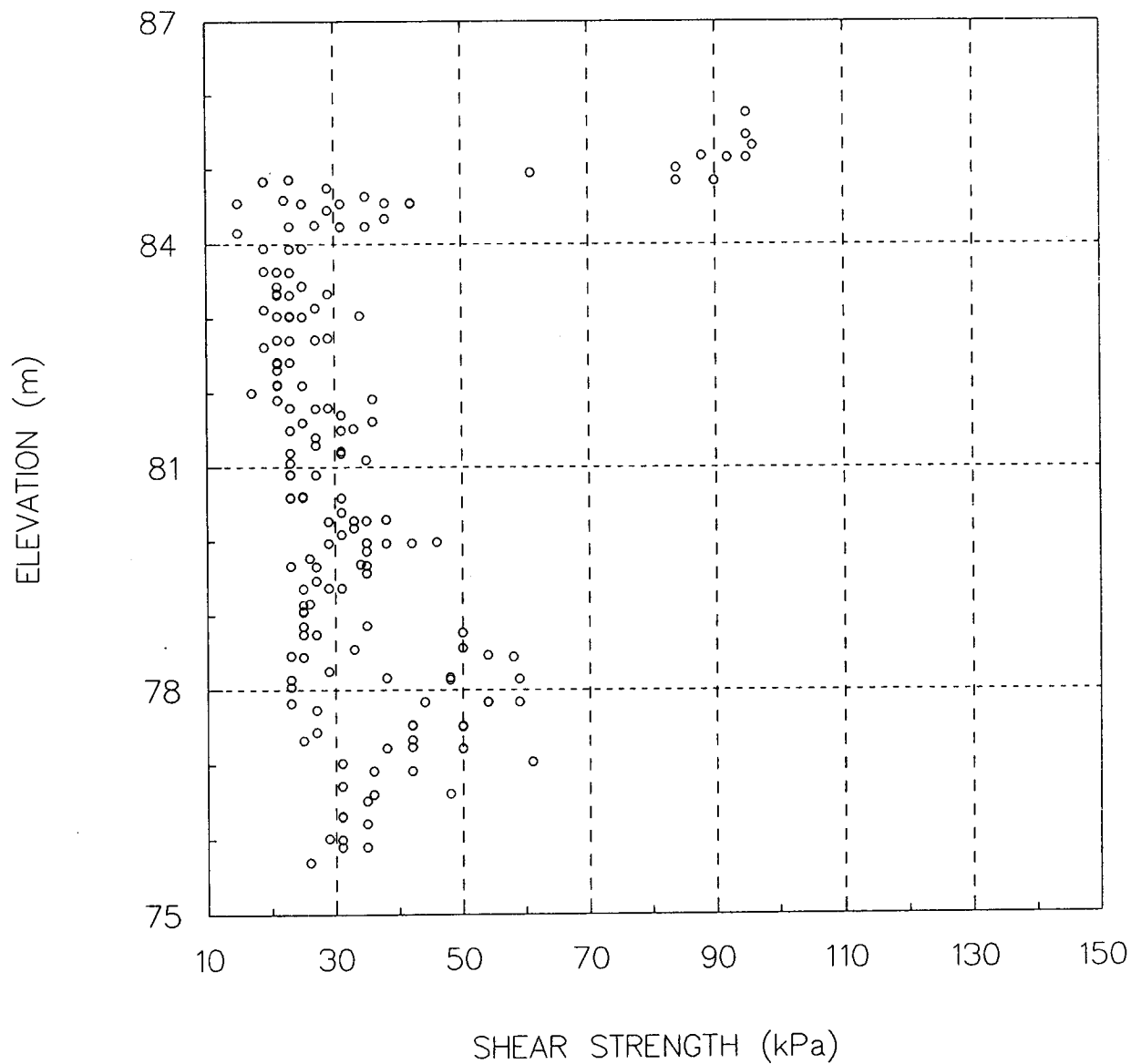
FIGURE 9  
WP 373-89-04



# SUMMARY OF VANE SHEAR STRENGTH vs. ELEVATION

FIGURE 10

WP 373-89-04



Date JAN. 10, 1991  
Project 90I-2064B

Golder Associates

Drawn JC  
Chkd. AC

**METRIC**  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES UNLESS  
OTHERWISE SHOWN. STATIONS  
IN KILOMETRES + METRES

CONT No  
WP No 373-89-04

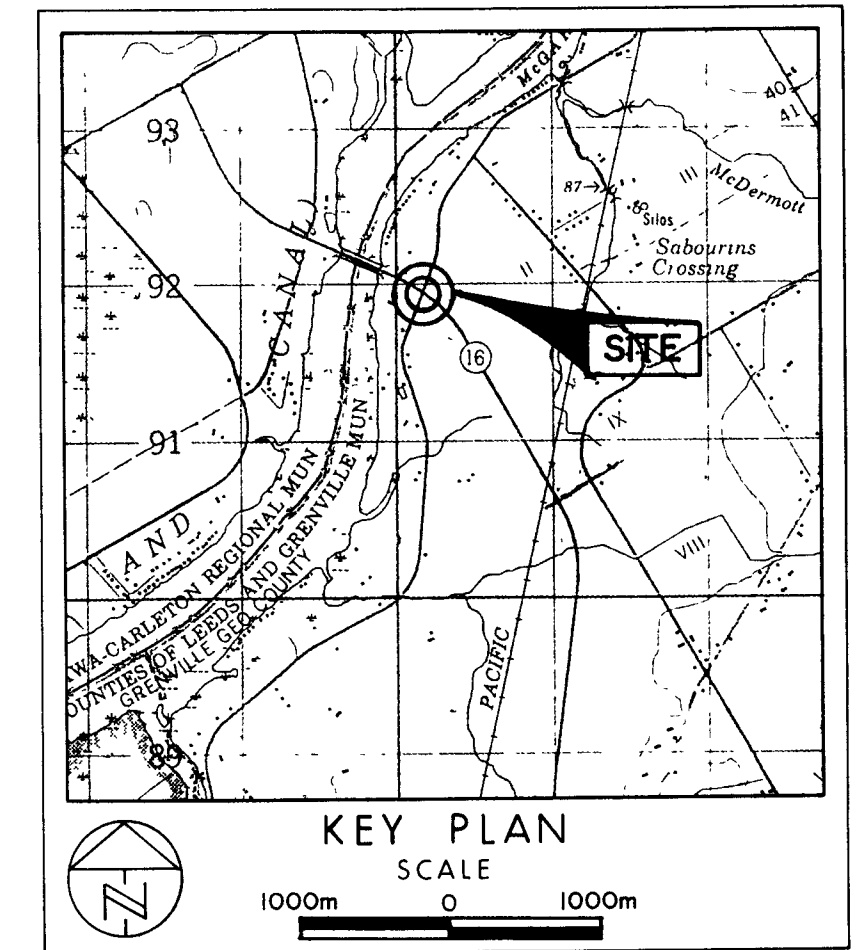
COUNTY ROAD 19

BORE HOLE LOCATIONS & SOIL STRATA

SHEET

**COPY**

GOLDER ASSOCIATES LTD.



**LEGEND**

- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊙ Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- W.L. at time of investigation (November 1990)
- Standpipe

No	ELEVATION (m)	CO-ORDINATES	
		NORTH	EAST
I-1	87.0	4 993 586	373 023
I-2	87.4	4 993 548	373 025
I-3	87.3	4 993 553	373 015
I-4	88.2	4 993 623	373 055
I-5	88.2	4 993 631	373 042
I-6	88.5	4 993 655	373 053
I-7	87.6	4 993 525	373 005
I-8	88.5	4 993 688	373 077
I-9	88.3	4 993 731	373 080
I-10	88.1	4 993 764	373 105
I-11	87.9	4 993 483	373 001
I-12	88.8	4 993 430	372 970

**NOTE**

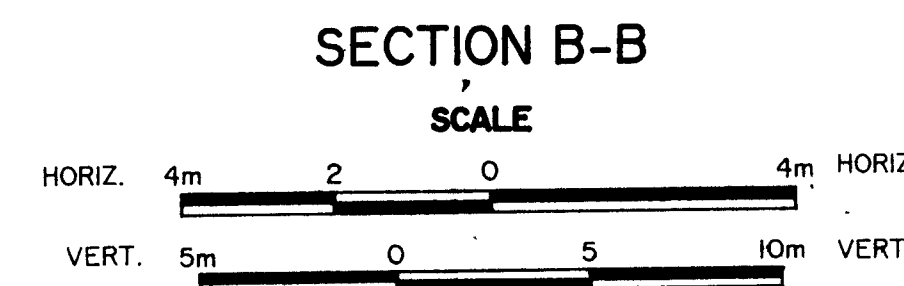
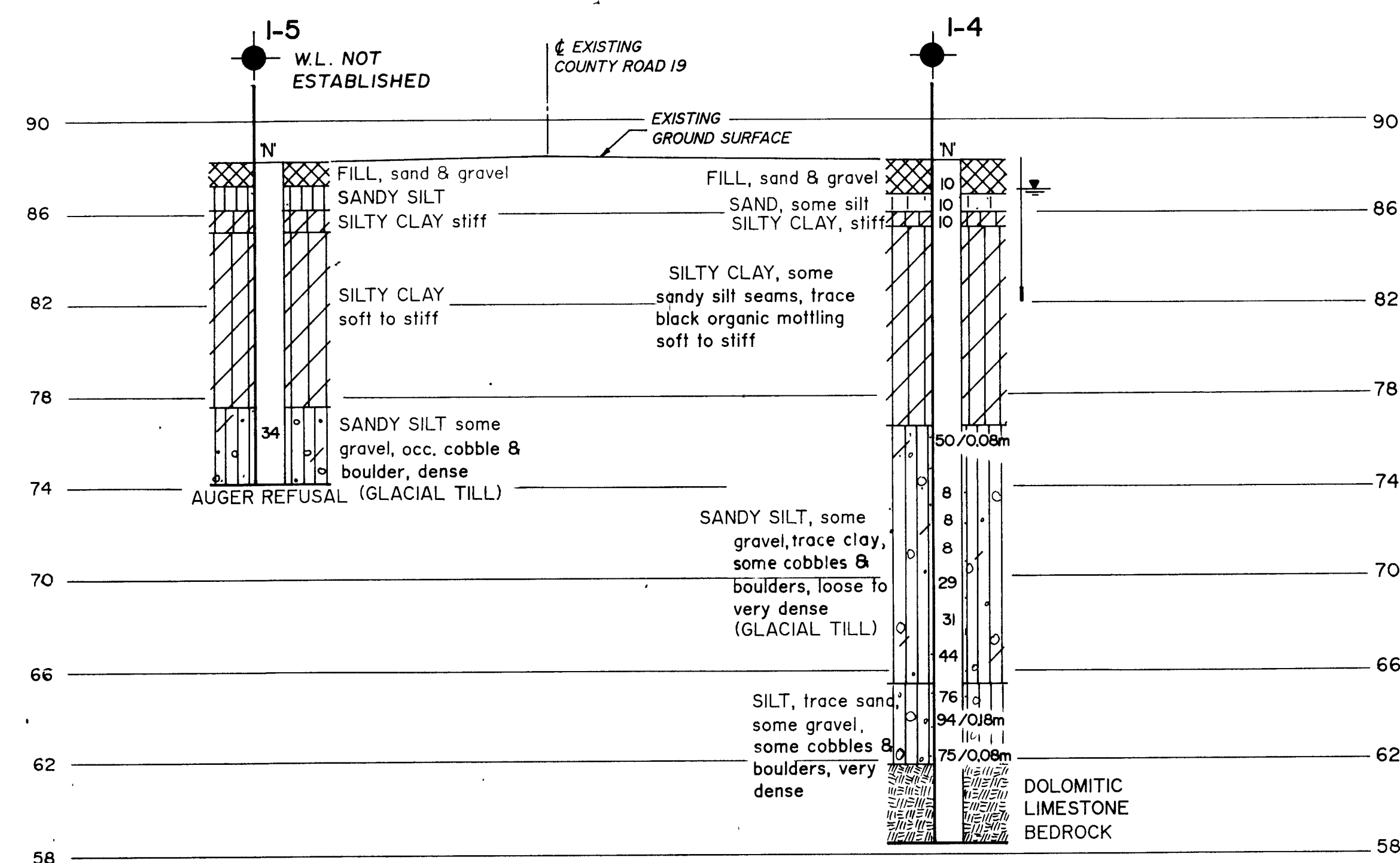
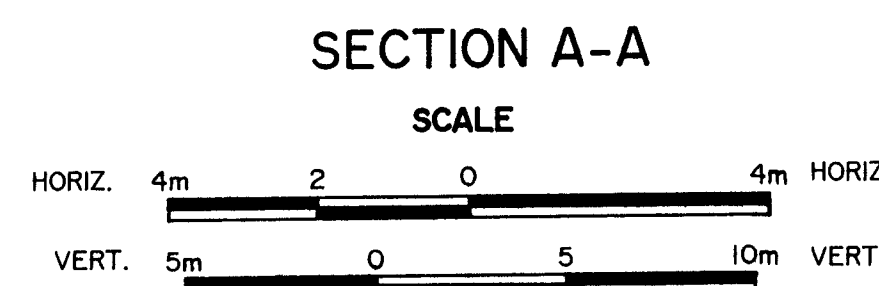
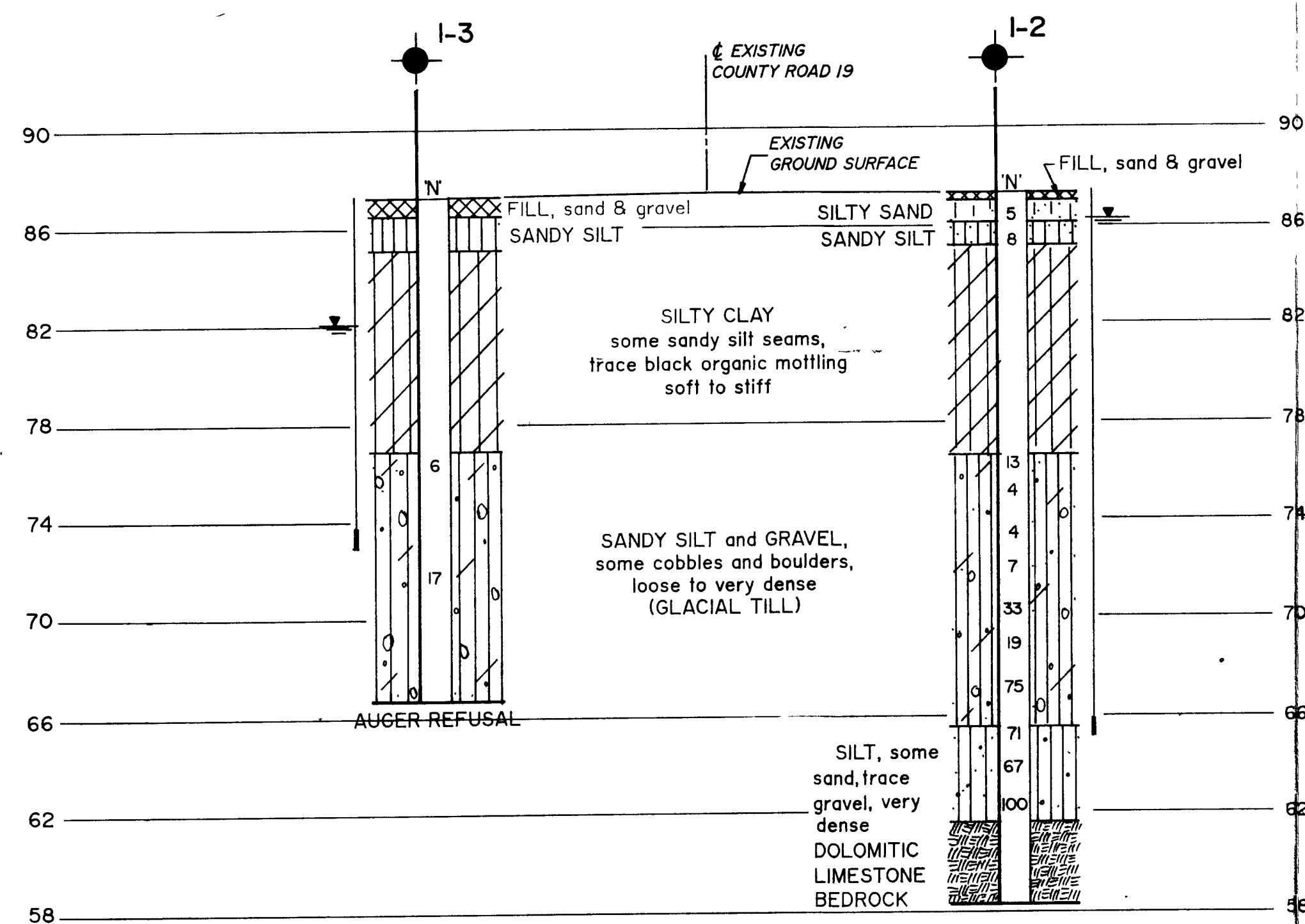
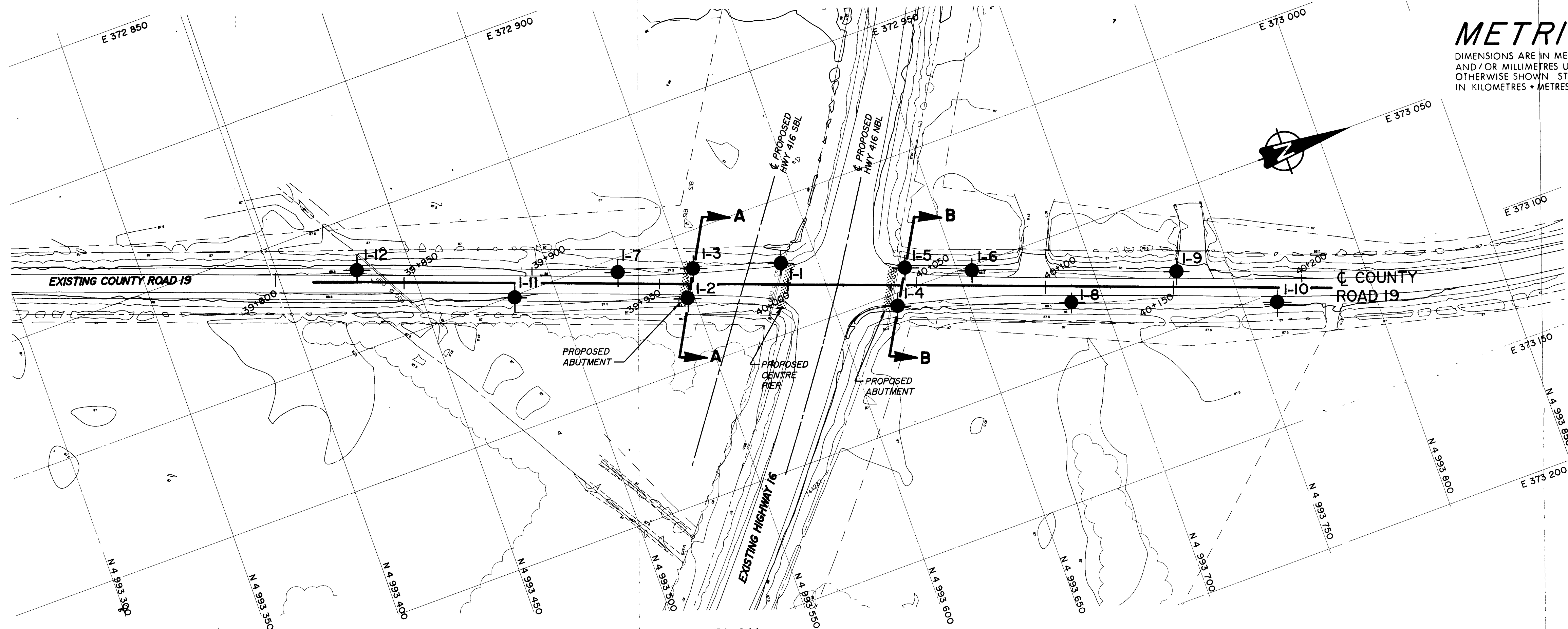
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

DATE	BY	DESCRIPTION

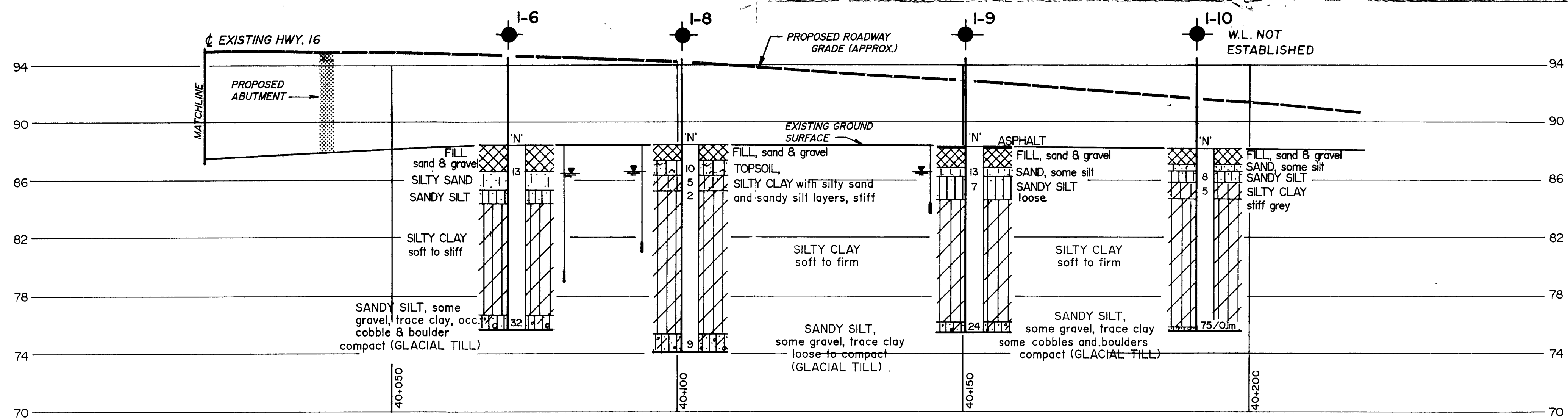
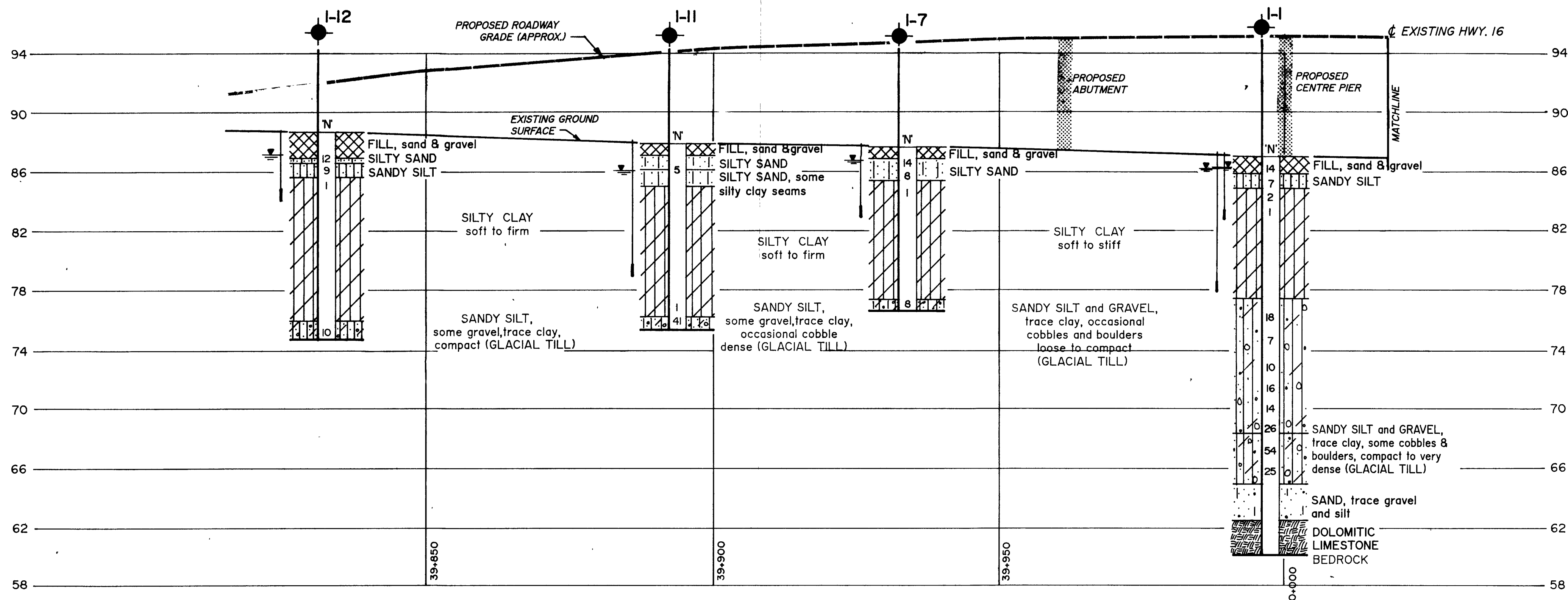
Geocres No 316-202

HWY No 416	SUBMD AC	CHECKED AC	DATE 91/01/08	DIST 9
				SITE 16-320
DRAWN JC	CHECKED	APPROVED		DWG 3738904-A

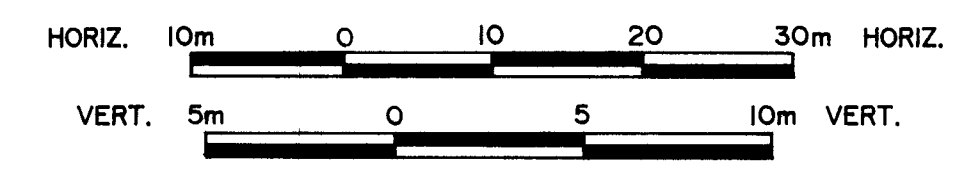


NOTE REFER TO DRAWING No. 3738904-B  
FOR CL PROFILE COUNTY ROAD 19

REF. No. -



PROFILE ALONG COUNTY ROAD 19  
SCALE

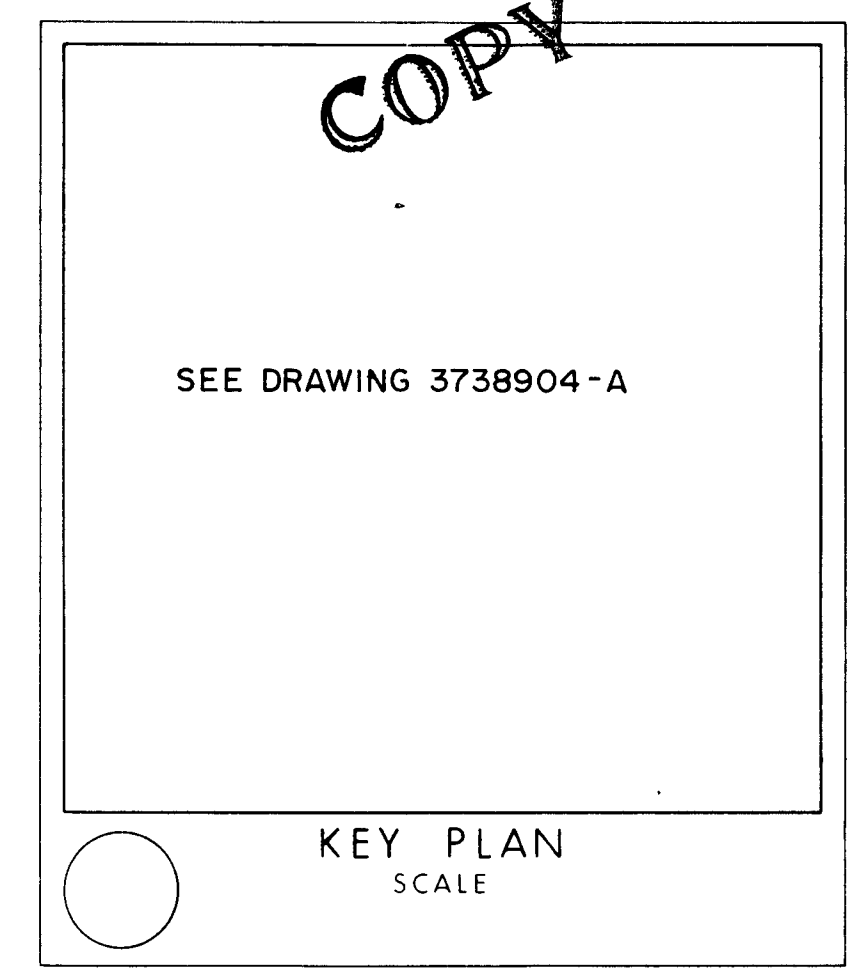


**METRIC**  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES UNLESS  
OTHERWISE SHOWN. STATIONS  
IN KILOMETRES + METRES

CONT No  
WP No 373-89-04  
COUNTY ROAD 19  
BORE HOLE LOCATIONS & SOIL STRATA

SHEET

GOLDER ASSOCIATES LTD.



LEGEND	
●	Bore Hole
⊕	Dynamic Cone Penetration Test (Cone)
⊗	Bore Hole & Cone
N	Blows/0.3m (Std Pen Test, 475 J/blow)
CON	Blows/0.3m (60° Cone, 475 J/blow)
W.L.	W.L. at time of investigation (November 1990)
—	Standpipe

No	ELEVATION (m)		
I-1	87.0		
I-6	88.5		
I-7	87.6		
I-8	88.5		
I-9	88.3		
I-10	88.1		
I-11	87.9		
I-12	88.8		

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

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

NOTE: REFER TO DRAWING No. 3738904-A FOR BOREHOLE LOCATION IN PLAN.

REV	DATE	BY	DESCRIPTION

Geocres No 316-202

HWY No 416	CHECKED AC	DATE 91/01/08	DIST 9
SUBM'D AC	CHECKED	APPROVED	SITE 16-320
DRAWN JC	CHECKED		DWG 3738904-B

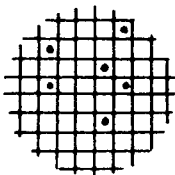
July 1991

901-2064B-2

APPENDIX A

CHEMICAL ANALYSIS OF GROUNDWATER SAMPLE





# ACCUTEST LABORATORIES LTD.

146 Colonnade Road, Unit 8, Nepean, Ontario K2E 7Y3 Tel.: (613) 727-5692 Fax: (613) 727-5222

## REPORT OF ANALYSES

CLIENT: Golder Assoc.

LAB REPORT NO: A1-0472

DATE: March 22, 1991

Attention: A.Chevrier

PROJECT: 901-2064B

PARAMETER	UNITS	Sample	Sample	Sample	Sample	Sample
Fe	mg/L					
Mn	mg/L					
Hardness	mg/L CaCO <sub>3</sub>					
Alkalinity	mg/L CaCO <sub>3</sub>					
pH		7.77				
Conductivity	umhos/cm	1920				
F	mg/L					
Na	mg/L					
N-NO <sub>3</sub>	mg/L					
N-NO <sub>2</sub>	mg/L					
N-NH <sub>3</sub>	mg/L					
SO <sub>4</sub>	mg/L	40				
Cl	mg/L	386				
Phenols	mg/L					
Turbidity	NTU					
Colour	Pt/Co Units					
Ca	mg/L					
Mg	mg/L					
Tann./Lig.	mg/L					
Total N	mg/L					
K	mg/L					

ANALYST: 

July 1991

901-2064B-2

APPENDIX B

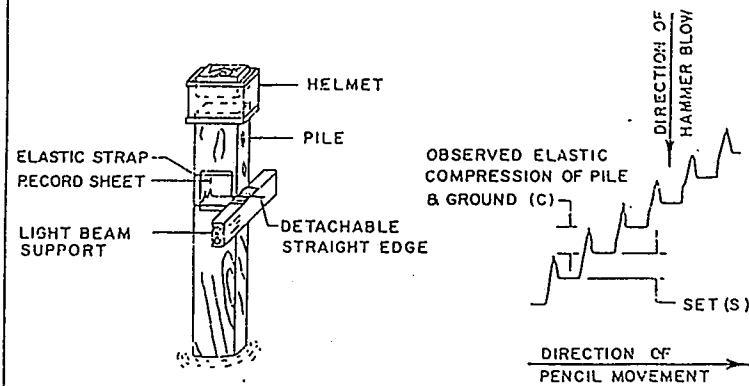
STANDARD DRAWINGS SS103-10 AND SS103-11

PRE DRIVING-DROP, STEAM AND DIESEL HAMMERS

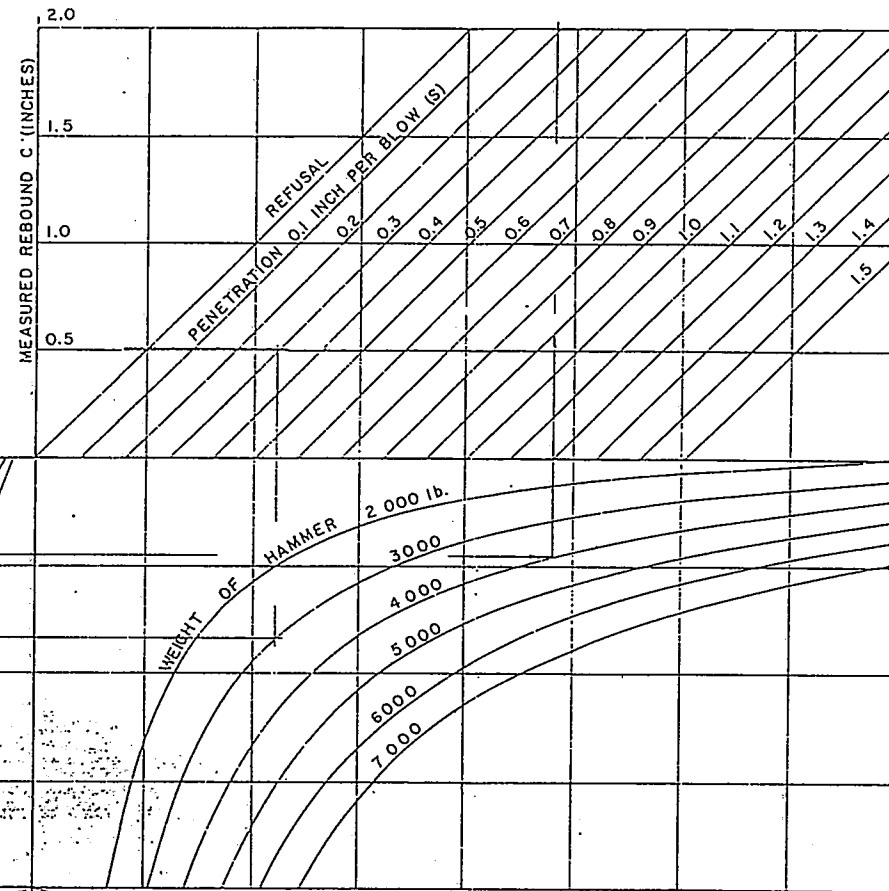
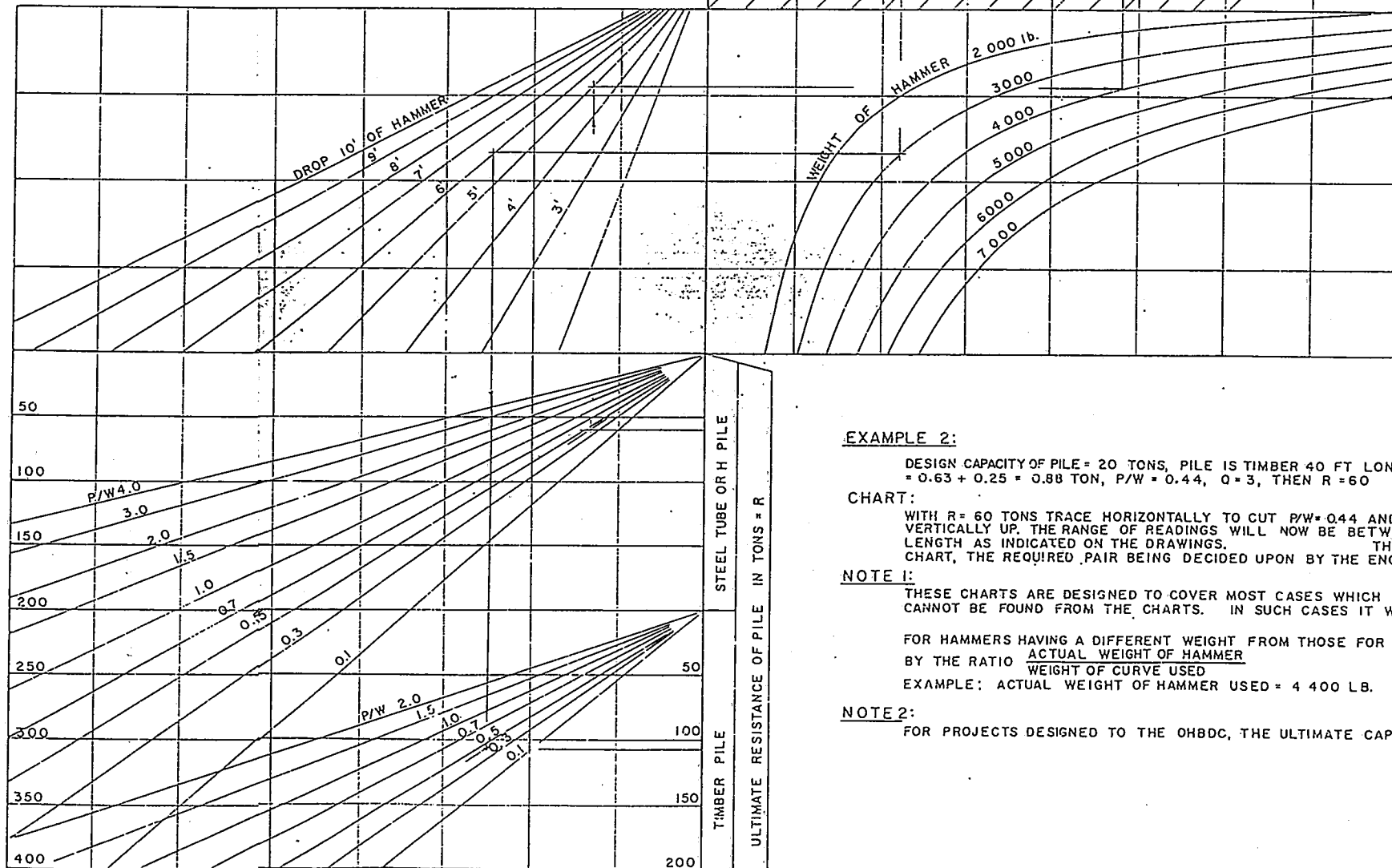
CONT No  
WP No 373-89-04

PILE DRIVING-DROP  
HAMMERS

SHEET



DROP HAMMERS ACTUATED BY FRICTION WINCH



$$R = \frac{nWh}{S + C/2} \text{ TONS (HILEY FORMULA)}$$

WHERE R = ULTIMATE CAPACITY IN TONS  
S = MEASURED PENETRATION OF PILE PER BLOW OF HAMMER IN INCHES  
C = MEASURED REBOUND OF PILE PER BLOW OF HAMMER IN INCHES  
W = GROSS WEIGHT OF HAMMER WITH A REDUCTION DUE TO THE EFFECT OF THE FRICTION WINCH AS AGAINST A PERFECT FREE FALL. THIS REDUCTION IS INCLUDED IN PLOTTING OF THE CURVES  
h = FALL OF HAMMER  
n = EFFICIENCY OF BLOW =  $\frac{W + P}{W + P}$   
WHERE e = 32 FOR STEEL (THESE VALUES OF e HAVE BEEN FOUND BY EXPERIMENT)  
= 0.25 FOR TIMBER  
P = WEIGHT OF PILE + 0.25 TON FOR HELMET  
W = WEIGHT OF HAMMER IN TONS  
THE P/W CURVES FORM THE REQUIRED REDUCTION OF TOTAL ENERGY (WH) OF THE HAMMER BLOW ACCORDING TO THE RATIO OF P/W  
L = R/Q TONS  
WHERE L = DESIGN CAPACITY OF PILE IN TONS  
Q = FACTOR OF SAFETY  
USE Q = 3 UNLESS OTHERWISE AUTHORIZED BY THE ENGINEER

EXAMPLE 1:

OBSERVED MEASURED REBOUND = C = 0.5 IN  
OBSERVED MEASURED SET PER BLOW = S = 0.3  
12 IN STEEL TUBE PILE 30 FT LONG AT 28 LBS PER FT PLUS HELMET  
WEIGHING 0.25 TON GIVING P = 0.67 TON: 3 000 LB HAMMER  
DROPPING 5 FT  
 $R = \frac{0.67}{1.5} = 0.45$

CHART:

WITH C = 0.5 IN PROCEED HORIZONTALLY TO THE RIGHT TO CUT LINE S = 0.3 AND VERTICALLY DOWN TO CUT CURVE 3 000 LBS THEN HORIZONTALLY TO THE LEFT TO CUT CURVE 6 FT AND VERTICALLY DOWN TO CUT P/W=0.45 FOR STEEL PILES AND READ R = 114 TONS APPROX

EXAMPLE 2:

DESIGN CAPACITY OF PILE = 20 TONS, PILE IS TIMBER 40 FT LONG, HAMMER 4 000 LBS DROPPING 5 FT, W = 2 TONS, P (MEAN DIA OF PILE 12 IN, TIMBER AT 40 LBS PER CU FT) = 0.63 + 0.25 = 0.88 TON, P/W = 0.44, Q = 3, THEN R = 60

CHART:

WITH R = 60 TONS TRACE HORIZONTALLY TO CUT P/W=0.44 AND VERTICALLY UP TO CUT 5 FT THEN HORIZONTALLY TO THE RIGHT TO CUT CURVE 4 000 LBS AND VERTICALLY UP. THE RANGE OF READINGS WILL NOW BE BETWEEN C=0 AND S=1.2 IN AND C=2.0 IN AND S=0.2 IN. A TEST PILE MUST BE DRIVEN OF A LENGTH AS INDICATED ON THE DRAWINGS. THE DRIVING MUST CONTINUE UNTIL A PAIR OF READINGS IS OBTAINED CORRESPONDING TO A PAIR ON THE CHART, THE REQUIRED PAIR BEING DECIDED UPON BY THE ENGINEER

NOTE 1:

THESE CHARTS ARE DESIGNED TO COVER MOST CASES WHICH WILL BE ENCOUNTERED ON NORMAL CONSTRUCTION PROJECTS. OCCASIONALLY IT WILL BE FOUND THAT R CANNOT BE FOUND FROM THE CHARTS. IN SUCH CASES IT WILL BE NECESSARY TO CALCULATE R USING THE ORIGINAL FORMULA  $R = \frac{nWh}{S + C/2}$

FOR HAMMERS HAVING A DIFFERENT WEIGHT FROM THOSE FOR WHICH CURVES ARE PLOTTED USE THE CURVE CLOSEST IN VALUE AND MULTIPLY THE RESULT OBTAINED BY THE RATIO  $\frac{\text{ACTUAL WEIGHT OF HAMMER}}{\text{WEIGHT OF CURVE USED}}$

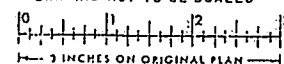
EXAMPLE: ACTUAL WEIGHT OF HAMMER USED = 4 400 LB. USE CURVE FOR 4 000 TO DETERMINE R AND MULTIPLY RESULT BY  $\frac{4 400}{4 000}$

NOTE 2:

FOR PROJECTS DESIGNED TO THE OHBDC, THE ULTIMATE CAPACITY (R) IS SHOWN ON THE CONTRACT DRAWINGS AND L AND Q ARE NOT REQUIRED

STANDARD DRAWING  
SEPT 1981 SS 103 - 10

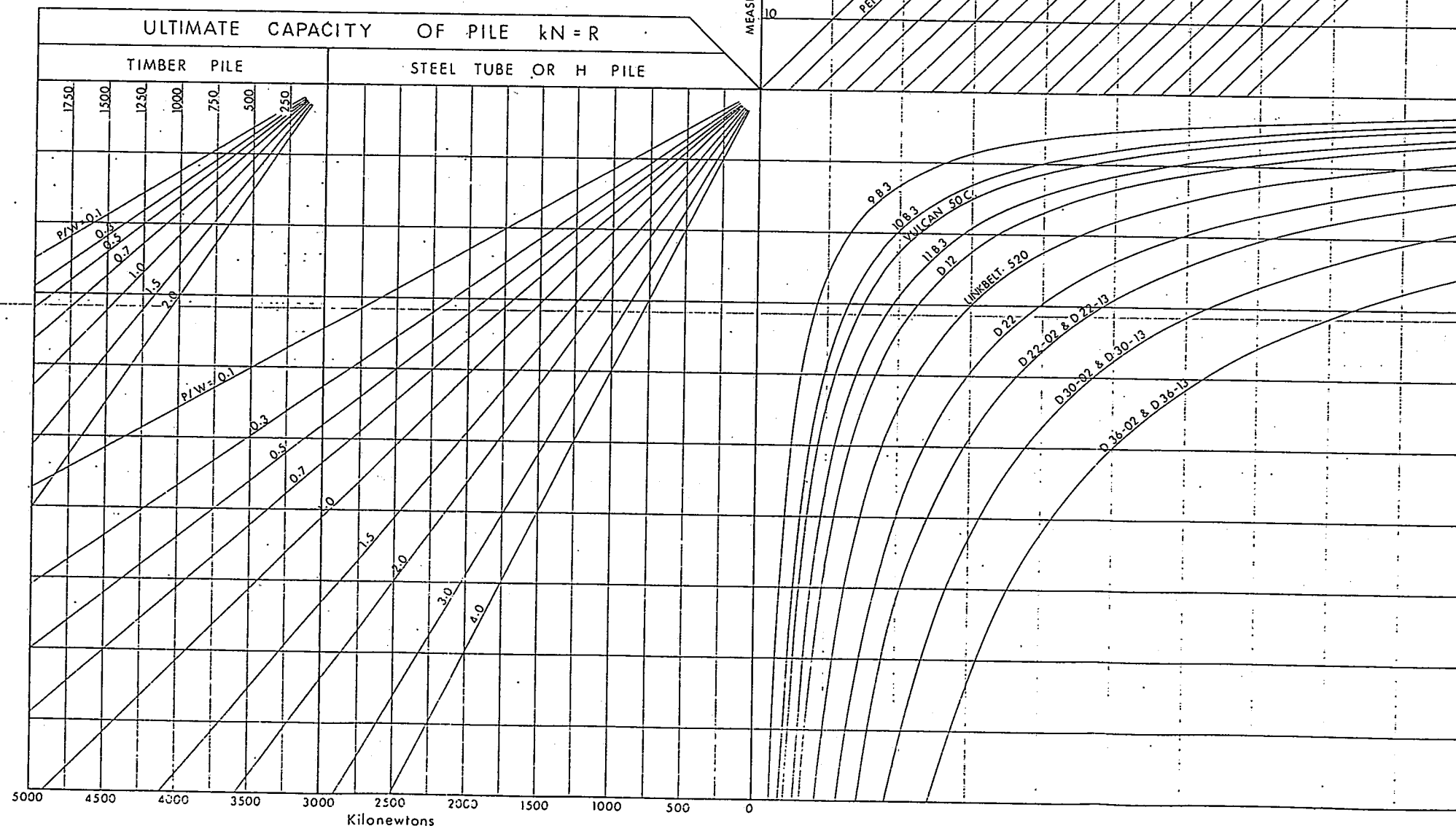
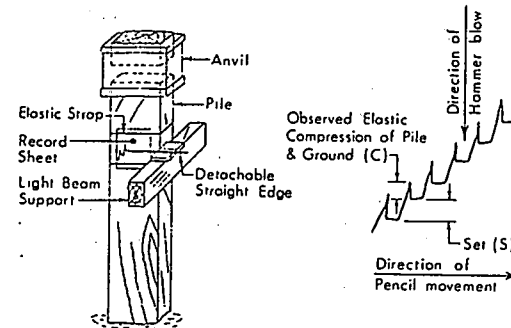
DRAWING NOT TO BE SCALED



REVISIONS	DATE	BY	DESCRIPTION
DESIGN	CHECK	LOADING	DATE
DRAWING	CHECK	SITE No	DWG

HAMMERS		
TYPE	MASS OF RAM W Kilograms	MAXIMUM ENERGY Joules/blow
9B3	726	12 419
10B3	1 361	16 948
50C	2 268	20 337
11B3	2 268	26 005
D12	1 250	30 506
B 225	1 360	39 300
L B 520	2 300	40 675
B 300	1 700	46 100
D 22	2 200	53 826
B 400	2 268	62 400
D22-02	2 200	67 000
D22-13	2 200	67 000
D30-02	3 000	91 000
D30-13	3 000	91 000
B 500	3 129	107 100
D36-02	3 600	115 000
D36-13	3 600	115 000

NOTE:  
Ram may also be referred to as Piston



METRIC

DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES UNLESS  
OTHERWISE SHOWN

CONT No  
WP No 373-89-04

SHEET

PILE DRIVING-STEAM & DIESEL HAMMERS

### METHOD OF APPLYING THE HILEY FORMULA

$$R = \frac{nWgh}{S + \frac{C}{2}} \quad (\text{Hiley Formula}) \quad g = 9.80665 \text{ m/s}^2$$

Where R = Ultimate pile capacity in kilonewtons  
S = Measured penetration of pile per hammer blow in millimetres  
C = Measured rebound of pile per hammer blow in millimetres  
Wgh = Energy of hammer blow in joules  
n = Efficiency of blow =  $\frac{W + Pe^2}{W + P}$

where e = 0.32 for steel (These values of e have been determined by experiment)  
= 0.25 for timber  
P = Mass of pile + anvil in kilograms  
W = Mass of ram (piston) in kilograms

The P/W curves form the required reduction of total energy of the hammer blow according to the value of P/W

L = R/Q kilonewtons

Where L = Design capacity of pile

Q = Factor of safety

Use Q = 3 unless otherwise authorized by the Engineer

#### EXAMPLE 1:

Steel tube pile, O.D. = 323.90 mm linear density = 49.73 kg/m,  
20 m long plus anvil of mass 600 kg, giving P = 994.6 + 600 = 1594.6 kg

Delmag D12 hammer W = 1250 kg P/W =  $\frac{1594.6}{1250} = 1.28$

Observed measured rebound C = 10 mm

Observed measured penetration S = 5 mm

USING CHART: With C = 10 proceed horizontally to right to cut line S = 5 then vertically down to cut curve D12 then horizontally to left to cut P/W = 1.28 then vertically down to read ultimate capacity R = 1512 kN L =  $\frac{1512}{3} = 504$  kN

#### EXAMPLE 2:

HP 310x110, 50 m long plus anvil of mass 600 kg giving  
P = 5500 + 600 = 6100 kg. The hammer is a Delmag D22-13

W = 2200 kg, n =  $\frac{W + Pe^2}{W + P} = \frac{2200 + (6100 \times 0.32 \times 0.32)}{2200 + 6100} = \frac{2824}{8300} = 0.34$

Energy of hammer (Wgh) = 67 000 J/blow

Observed measured rebound C = 10 mm

Observed measured penetration S = 5 mm

#### USING HILEY FORMULA

Ultimate capacity R =  $\frac{nWgh}{S + \frac{C}{2}} \text{ kN} = \frac{0.34 \times 67000}{10} = 2278 \text{ kN}$

Design capacity L =  $\frac{2278}{3} = 759 \text{ kN}$

#### NOTE 1:

These charts are designed to cover most cases which will be encountered on normal construction projects. Occasionally it will be found that R cannot be obtained from the charts, for instance when C = 5 mm and S = 2 mm using a Delmag D22 hammer. In such cases it will be necessary to calculate R using the original equation  $R = \frac{nWgh}{S + \frac{C}{2}}$

In cases where the energy of the hammer being used is slightly different from the hammer energy for which curves are drawn the curves may still be used but the result should be reduced or increased according to the energy ratios. Example use Linkbelt 520 curve (Energy 40 675 J) for Berminghammer 225 (Energy 39 300 J) but reduce result by multiplying by  $\frac{39300}{40675}$

#### NOTE 2:

For projects designed to the OHBDC, the ultimate capacity (R) is shown on the contract drawings and L and Q are not required

STANDARD DRAWING  
JULY 1981 SS 103-11

REVISIONS	DATE	BY	DESCRIPTION
DESIGN K S	CHECK	LOADING	DATE
DRAWING C I	CHECK	SITE No	DWG