GEOCRES No. 30M15-89 / 30M13-146.

DIST. CR REGION

W.P. No. 2505-98-00
2506-98-00

CONT. No.

W. O. No.

STR. SITE No.

HWY. No. 401

LOCATION Hwy 401, W of Bennett Rd.
MTO Truck Inspection Station

No of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

G.I.-30 SEPT. 1976

Golder Associates Ltd.

100 Scotia Court
Whitby, Ontario, Canada L1N 8Y6
Telephone (905) 723-2727
Fax (905) 723-2182

Golder Associates

GEOCRES # 30M15-89

REPORT ON

FOUNDATION INVESTIGATION
MTO TRUCK INSPECTION STATION
HWY 401 WBL, WEST OF BENNETT ROAD
NEWCASTLE, ONTARIO
WP 2505-98-00

Submitted to:

Delcan Corporation
133 Wynford Drive
North York, Ontario
M3C 1K1

Distribution:

4 copies - Delcan Corporation
2 copies - Golder Associates Ltd.

June, 1998

981-8018B

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LIST OF FIGURES

1 INTRODUCTION

Golder Associates Ltd. has been retained by Delcan Corporation (Delcan), on behalf of the Ministry of Transportation, Ontario (MTO), to carry out a foundation investigation at the site of the MTO Truck Inspection Station (TIS) and Weight Scale, Highway 401 (WBL), west of Bennett Road, Newcastle.

The purpose of this investigation is to determine the subsurface conditions at the site for the reconstruction of the TIS building by means of a limited number of boreholes, in-situ tests and laboratory testing on selected samples.

The terms of reference for the scope of work are outlined in our proposal letter P81-8051, dated April 21, 1998.

2 SITE DESCRIPTION AND GEOLOGY

The site is situated north of Highway 401 WBL within the MTO corridor between stations 500+00 and 512+00. It is understood that the existing building at the TIS, was originally built in the 70's and upgraded in the early 80's. The land surrounding the site is generally flat to the north and south of the highway right of way with a gradual increase in the grade to the east along Highway 401. The site location is shown on the Key Plan, Figure 1.

Physiographically, the site is located within the region known as the Iroquois Plain. The Iroquois Plain is the product of the advance and retreat of the Wisconsin ice sheet, which covered the area during the Pleistocene epoch (over 2000 years ago). The lowland bordering Lake Ontario, was inundated by the glacial lake called Lake Iroquois when the last glacier was receding at the site. Conditions in the old lake plain vary greatly within the Iroquois Plain. Within the site location, the former lake bottom consisted of silty clay to clay which is underlain by till plains and by glaciofluvial sands to silty sands.

The overburden at the site is underlain by shale bedrock of the Queenston Formation at an elevation ranging from approximately 56 to 68 m suggesting a highly variable and irregular bedrock surface. The overburden thickness is believed to range from approximately 8 to 23 m.

3 INVESTIGATION PROCEDURE

The fieldwork was carried out on May 8, 1998 using a truck mounted CME 55 equipped with solid stem augers to advance the boreholes. The fieldwork consisted of the advancement of two (2) boreholes to a depth of 6.55 m.

Samples were retrieved at regular intervals using a standard 35 mm I.D. split spoon sampler driven in accordance with the Standard Penetration Test (SPT – ASTM D 1586).

Groundwater levels were determined by monitoring the water level in the open boreholes throughout the duration of the field investigation. All boreholes were backfilled upon completion of fieldwork.

The fieldwork was supervised on a full time basis by a member of our technical staff who located the boreholes, directed the drilling, sampling and in-situ testing operations and logged the boreholes. All subsoil samples were identified in the field and then properly sealed in plastic containers to preserve the insitu moisture content in the soil. The samples were then transported to the Golder laboratory in Whitby, where additional visual classification was carried out and pertinent laboratory tests were conducted.

Sample preparation and laboratory tests were conducted in accordance with the respective procedures outlined in the MTO Laboratory Testing Manual and as described in the MTO Soil Classification Manual.

The borehole locations and elevations were provided by Delcan and we understand that the elevations are referenced to Geodetic Datum. The northing and easting co-ordinates of the boreholes are shown on the Record of Borehole sheets attached.

4 SUBSURFACE CONDITIONS

4.1 SITE STRATIGRAPHY

The detailed subsurface soil and groundwater conditions encountered in the boreholes together with the results of the laboratory testing carried out on selected soil samples are presented on the appended Record of Borehole sheets and Figures following the text of this report. Within the proposed building foundation area, the subsurface conditions consist of a sand and gravel and/or silty sand fill material ranging in thickness between 1.75 m to 2.14 m. The fill is underlain by a wet silty fine sand deposit inturn underlain by a silty clay and clayey silt till at the lower depths in each borehole.

The locations of the boreholes are shown on Figure 2. A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2 FILL

The fill in both boreholes consisted primarily of a brown gravelly or silty sand and sand & gravel, typical of granular base and subbase materials. The fill is in a relatively compact to dense state with "N" values of 13 to 46 blows per 0.3 m penetration. The insitu moisture content of the sand fill ranges from 2 to 5 per cent.

4.3 SILTY SAND

A wet brown silty fine sand deposit was fully penetrated in each borehole below the fill. The silty sand is in a relatively loose to compact state of packing with standard penetration test (SPT) "N" values of 8 and 14 blows for 0.3 m penetration. The measured natural moisture content of the samples tested varied between 17 and 19 per cent.

A grey silty sand, with a trace of clay and gravel (till-like) layer was encountered above the clayey silt within the lower depths of Borehole 401-2. The 'N' value derived from the SPT conducted in this deposit was 13 blows for 0.3 m indicating a relatively compact state of denseness. The natural moisture content of the grey silty sand was 10 per cent.

4.4 SILTY CLAY/CLAYEY SILT

The silty sand in each borehole was underlain by a brown to grey silty clay to clayey silt containing a trace to some fine sand and wet silty sand seams. The consistency of the silty clay/clayey silt is firm to stiff with "N" values ranging from 5 to 12 blows per 0.3 m penetration. The natural moisture content of the clayey silt/silty clay deposit varies from 28 to 33 per cent.

4.5 CLAYEY SILT TILL

A grey clayey silt till was encountered within the lower depths of each borehole. The measured "N" value in Borehole 401-1 was 27 blows per 0.3 m indicating a very stiff consistency. The natural moisture content of the clayey silt samples tested ranged from 6 to 10 per cent.

4.6 GROUNDWATER CONDITIONS

The water level in Borehole 401-1 was recorded in the open borehole upon completion of drilling at a depth of 2.74 m, below the existing ground surface. A piezometer was installed in Borehole 401-2 and the water level in the piezometer upon completion of drilling was 2.15 m below the existing ground surface. It should be noted that the water levels are subject to seasonal fluctuations.

5 ENGINEERING RECOMMENDATIONS

This section of the report provides our recommendations on the geotechnical aspects for the design of the new TIS building and the associated servicing, based on our interpretation of the factual information obtained during the investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction method and scheduling.

The works described in this report are associated with the proposed demolition and reconstruction of the TIS building. It is understood that a new pre-fabricated building supported on piers, with an elevated floor, will be erected in the location as shown on the borehole location plan, Figure 2.

5.1 BUILDING FOUNDATIONS

The subsoils encountered in the boreholes put down at this site are not suitable for support of shallow spread footings. It is recommended that the footings be supported on an engineered fill placed on the native clayey silt/silty clay strata. A minimum one metre thick layer of well compacted Granular A material should be provided below founding level. However, an allowance for additional Granular A should be made to account for excavation within the wet silty sand in Borehole 401-1.

The engineered fill envelope should extend at least one metre beyond the perimeter at the base of the footings and outward at a 1:1 slope for the full depth of the engineered fill. Footings supported by the engineered fill can be designed for an allowable soil bearing pressure of 150 kPa. The Granular A engineered fill should be placed in maximum 300 mm loose lifts and be uniformly compacted to at least 100 per cent of standard Proctor maximum dry density. Exterior footings should be provided with at least 1.2 m of earth cover for frost protection after final grading. Alternatively, spread footings can be carried down to below the fill and founded on the native soil at an allowable soil bearing value of 150 kPa.

Prior to placing the engineered fill, the exposed base should be proof rolled and inspected by the geotechnical engineer to verify that it is suitable for the support of the engineered fill. The site excavated materials, which are free of organics or other deleterious materials may be used for backfill around the engineered fill and outside the zone of influence of footings. Assuming this fill will be required to perform as the subgrade for the restored pavement, it should be placed in maximum 300 mm loose lifts and uniformly compacted to at least 95 per cent of standard Proctor maximum dry density.

The excavated materials from the site will mainly consist of sand and gravel and silty sand. These materials are considered suitable for reuse as compacted fill provided the moisture content is strictly controlled during compaction. The backfill material should be placed in maximum 300 mm loose lifts and uniformly compacted to at least 95 per cent of standard Proctor maximum dry density.

1.1 EXCAVATION AND GROUND WATER CONTROL

Based on the results of the field work, relatively shallow excavations for the placement of the engineered fill to support spread footings can be carried out using conventional open cut procedures. The side slopes of the excavations should comply with the Occupational Health and Safety Act. Some sloughing of the side slopes through the wet silty sand should be expected. Blanketing of the cut slopes with coarse granular or local slope flattening may be required to maintain stability. Removal of groundwater within the wet silty sand should be readily controlled by conventional pumping from sump holes.

Prior to construction, the geotechnical aspects of the final design drawings and specifications should be reviewed by this office to confirm that the intent of this report has been met.

We trust that this report provides sufficient geotechnical information for you to proceed with the design of this project. If you have any questions regarding the contents of this report, please do not hesitate to contact this office.

Yours truly,

GOLDER ASSOCIATES LTD.

V. Aurilio, P.Eng.
Senior Materials Engineer

Fin Heffernan, P.Eng.
Consultant

REGISTERED PROFESSIONAL ENGINEER
F. J. HEFFERNAN
PROVINCE OF ONTARIO

VA:FJH:va

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.).

Dynamic Penetration Resistance; N_d:
The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT):
An electronic cone penetrometer with a 60° conical tip and a projected end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	c_u, s_u kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_S)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane test (LV-laboratory vane test)
γ	unit weight

Note:
1. Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π = 3.1416
ln x, natural logarithm of x
$\log_{10}$ x or log x, logarithm of x to base 10
g acceleration due to gravity
t time
F factor of safety
V volume
W weight

II. STRESS AND STRAIN

γ shear strain
Δ change in, e.g. in stress: $\Delta \sigma$
ε linear strain
ε_v volumetric strain
η coefficient of viscosity
ν Poisson's ratio
σ total stress
σ' effective stress ($\sigma' = \sigma - u$)
σ'_{vo} initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$ principal stresses (major, intermediate, minor)
σ_{oct} mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ shear stress
u porewater pressure
E modulus of deformation
G shear modulus of deformation
K bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$ bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$ dry density (dry unit weight)
$\rho_w(\gamma_w)$ density (unit weight) of water
$\rho_s(\gamma_s)$ density (unit weight) of solid particles
γ' unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R relative density (specific gravity)of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e void ratio
n porosity
S degree of saturation

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

(a) Index Properties (con't.)

w water content
w_l liquid limit
w_p plastic limit
I_P plasticity Index = $(w_l - w_p)$
w_S shrinkage limit
I_L liquidity index = $(w - w_p) / I_p$
I_C consistency index = $(w_l - w) / I_p$
e_{max} void ratio in loosest state
e_{min} void ratio in densest state
I_D density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(c) Hydraulic Properties

h hydraulic head or potential
q rate of flow
v velocity of flow
i hydraulic gradient
k hydraulic conductivity (coefficient of permeability)
j seepage force per unit volume

(d) Consolidation (one-dimensional)

C_c compression index (normally consolidated range)
C_r recompression index (overconsolidated range)
C_S swelling index
C_α coefficient of secondary consolidation
m_v coefficient of volume change
c_v coefficient of consolidation
T_v time factor (vertical direction)
U degree of consolidation
σ'_p pre-consolidation pressure
OCR Overconsolidation ratio $= \sigma'_p / \sigma'_{vo}$

(e) Shear Strength

τ_p, τ_r peak and residual shear strength
ϕ' effective angle of internal friction
δ angle of interface friction
μ coefficient of friction = $\tan \delta$
c' effective cohesion
c_u, s_u undrained shear strength ($\phi = 0$ analysis)
p mean total stress $(\sigma_1 + \sigma_3)/2$
p' mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q $(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u compressive strength $(\sigma_1 - \sigma_3)$
S_t sensitivity

Notes: 1. $\tau = c' + \sigma' \tan \phi'$
2. Shear strength = (Compressive strength)/2

PROJECT: 981-8018B

RECORD OF BOREHOLE 401-1

SHEET 1 OF 1

LOCATION: N 4861189.5, E 331691.4

BORING DATE: May 8, 1998

DATUM: Geodetic

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: POWER AUGER, 114 mm Dia. Solid Stem Augers

Soil Profile Description	Elev. (m) / Depth (m)	Sample Number	Type	Blows/0.3m
Pavement Surface	83.90			
ASPHALT	0.00			
Gravelly sand, trace silt Compact to dense Brown Moist (GRANULAR BASE)	0.20	1	50 DO	32
		2	50 DO	46
SILTY fine SAND Loose, grey/brown Wet (FILL)	82.68 / 1.22	3A	50 DO	8
CLAYEY SILT, trace to some fine sand Firm to stiff Brown to grey Moist	82.15 / 1.75	3B		
		4	50 DO	12
SILTY CLAY, trace fine sand, Firm to stiff Light brown to grey Moist to wet	81.16 / 2.74	5	50 DO	11
— Wet at 4.57 m		6	50 DO	5
CLAYEY SILT, some sand, trace gravel Very stiff, grey, wet to moist	77.57 / 6.33	7A	50 DO	27
	77.35 / 6.55	7B		
END OF BOREHOLE				

Chart columns: DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - O); HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT (Wp, W, Wl; 10, 20, 30, 40); ADDITIONAL LAB. TESTING; PIEZOMETER OR STANDPIPE INSTALLATION

Depth scale (metres): 0 – 10

Piezometer or standpipe installation notes:

Water encountered at approx. 4.57 m during drilling, May 8/98

Borehole caved to 5.49 m upon completion of drilling, May 8/98

Water level at approx. 2.74 m upon completion of drilling, May 8/98

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: RJS

CHECKED: [signature]

N8018801.BH

DATA INPUT: ph n8018801.bh 5/98

SOILMS

PROJECT: 981-8018B

RECORD OF BOREHOLE 401-2

SHEET 1 OF 1

LOCATION: N 4861191.0, E 331675.2

BORING DATE: May 8, 1998

DATUM: Geodetic

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: POWER AUGER, 114 mm Dia. Solid Stem Augers

DESCRIPTION	ELEV. / DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m
Ground Surface	83.70			
Silty sand, trace gravel, trace organics, Compact, Brown, Dry (FILL)	0.00	1	50 DO	18
Sand and gravel, trace silt, Dense to compact, Brown, Dry (FILL)	82.99 / 0.71	2	50 DO	45
		3	50 DO	13
SILTY fine SAND, some gravel, Compact, Brown, Wet	81.56 / 2.14	4A	50 DO	12
SILTY CLAY, trace fine sand, wet, sandy silt seams, Firm to stiff, Brown to grey, Moist to wet	81.11 / 2.59	4B		
		5	50 DO	10
		6	50 DO	6
SILTY SAND, trace clay, trace gravel (TILL-LIKE), Compact, Grey, Moist				
CLAYEY SILT, some fine sand, trace gravel, Stiff, grey, moist (TILL)	77.60 / 6.10	7A	50 DO	13
	77.25 / 6.45	7B		
END OF BOREHOLE	6.55			

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

SHEAR STRENGTH Cu, kPa — nat V - + Q - ● rem V - ⊕ U - O

HYDRAULIC CONDUCTIVITY, k, cm/s

WATER CONTENT, PERCENT — Wp, W, Wl — 10 20 30 40

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

Bentonite Seal

Water encountered during drilling

Borehole open to 6.4 m upon completion of drilling, May 8/98

Water level at approx. 2.15 m upon completion of installation, May 8/98

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: RJS

CHECKED:

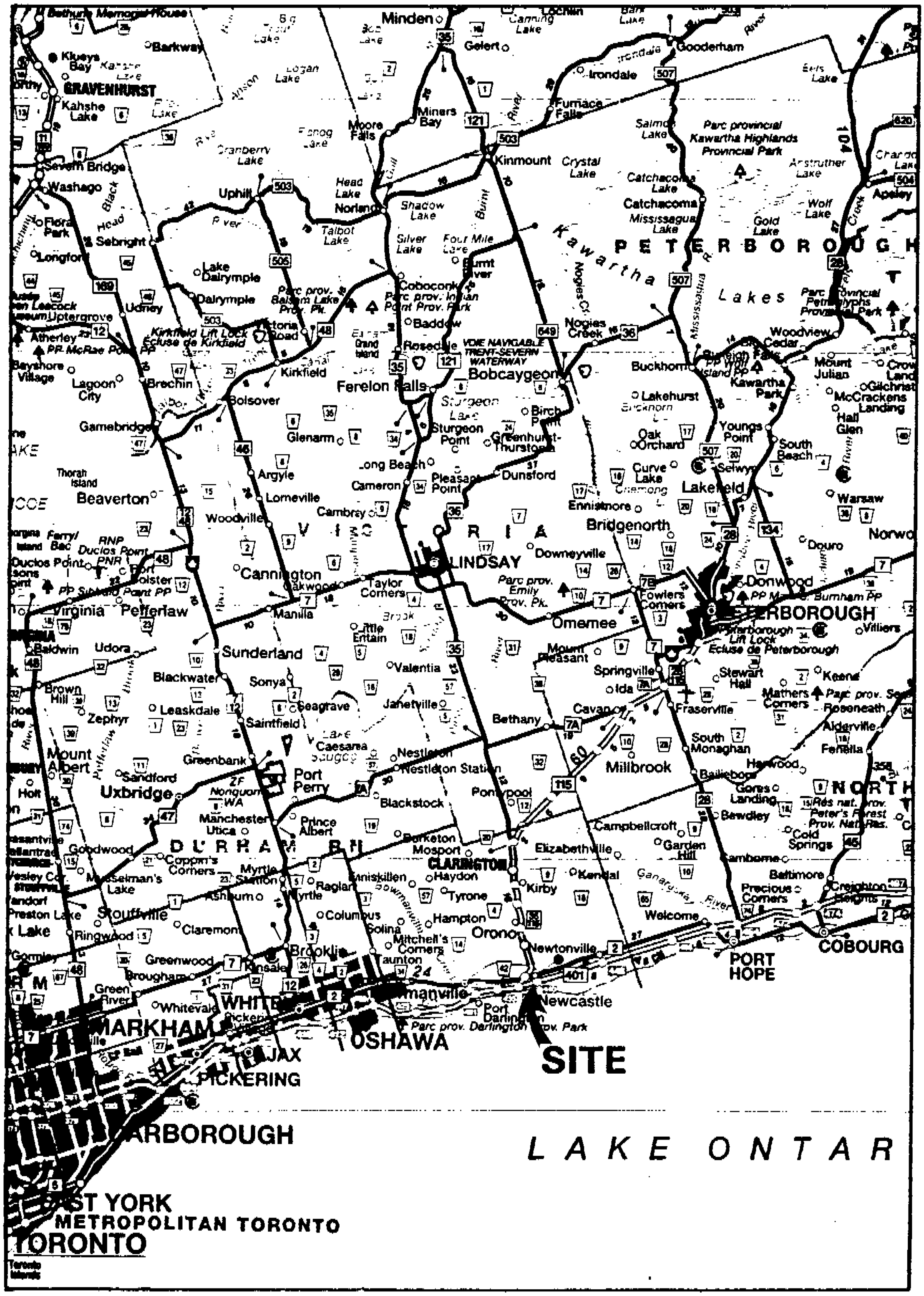
N6018B02.BH

DATA INPUT ph n6018b02.bh 5/98

SOILM8

KEY PLAN

FIGURE 1



Date May, 1998

Project 981-8018B

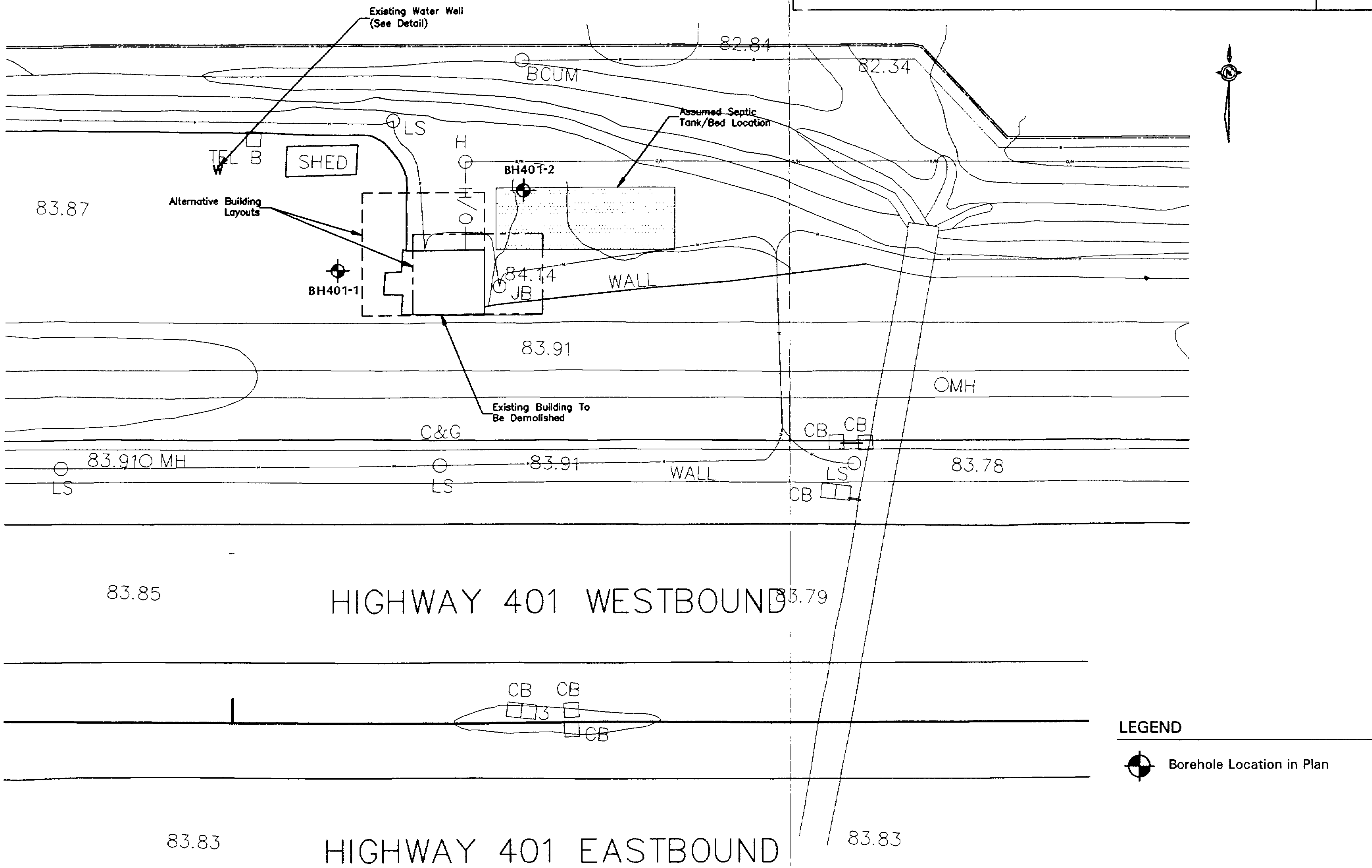
Golder Associates

Drawn PH

Chkd.

BOREHOLE LOCATION PLAN

FIGURE 2



Date May. 22, 199

Project 981-8018B

Golder Associates

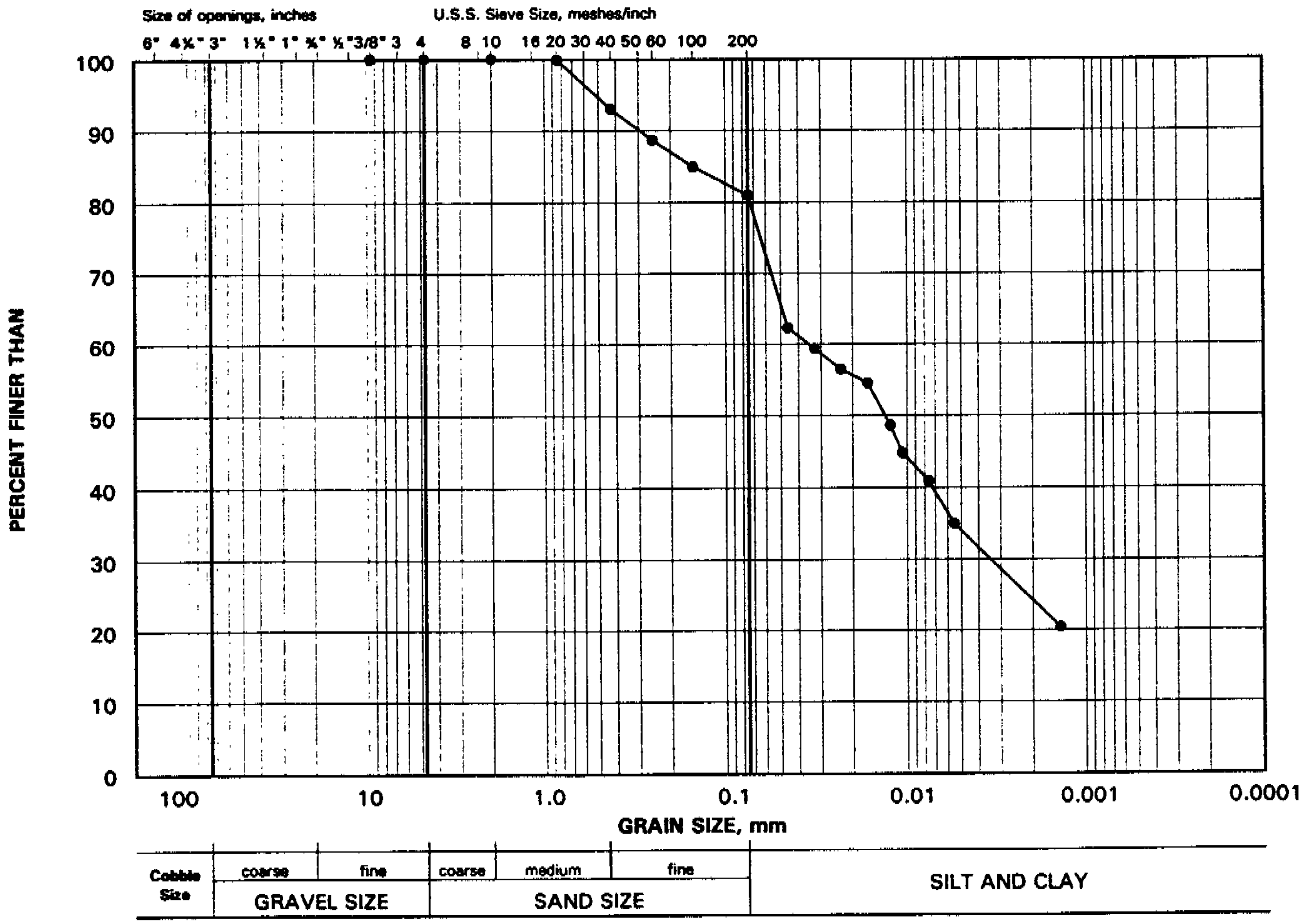
Drawn ph

Chkd

GRAIN SIZE DISTRIBUTION

FIGURE 3

CLAYEY SILT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	401-1	4B	2.28 - 2.74

ProGrain ver. 2.01

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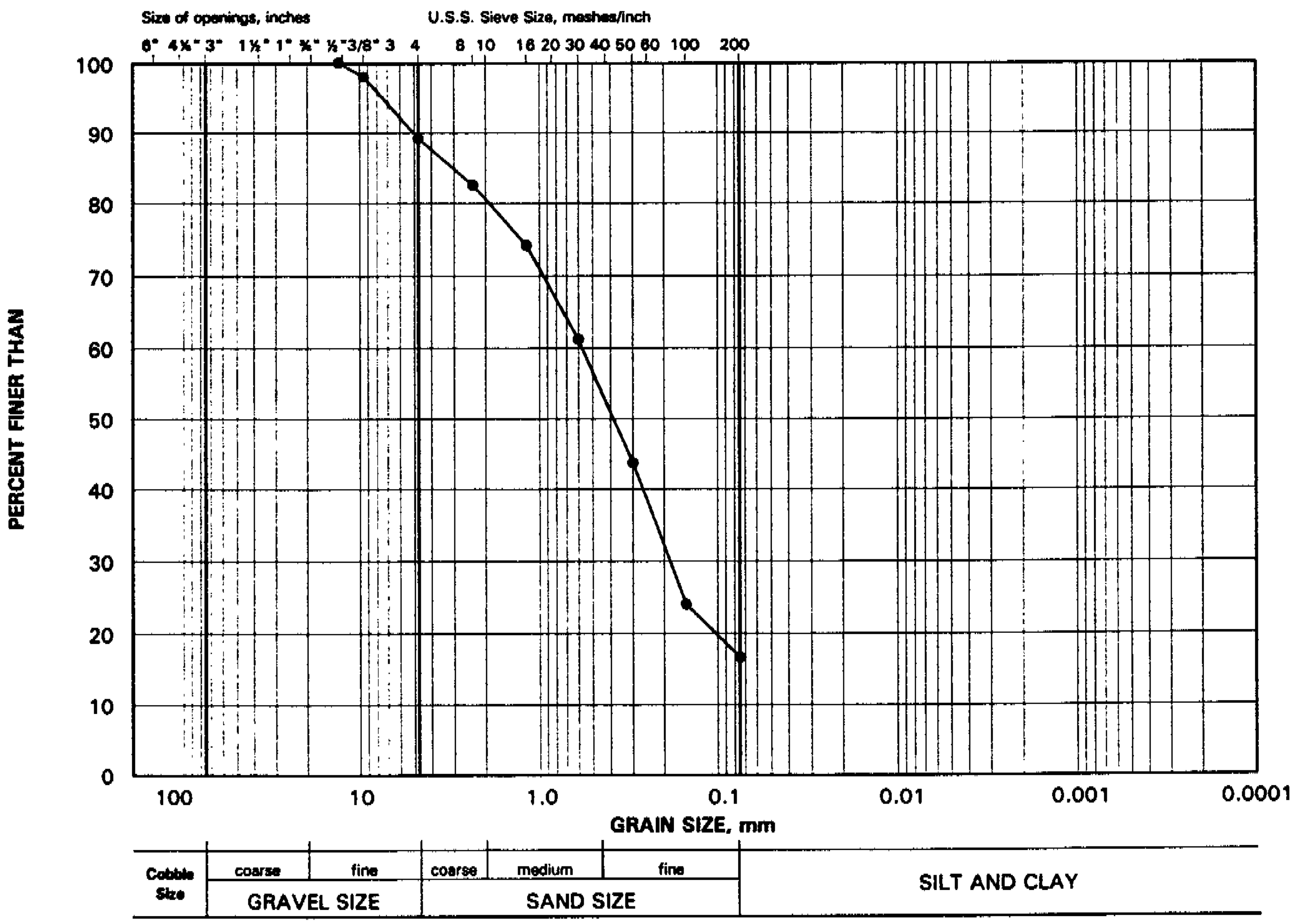
GRAIN SIZE DISTRIBUTION

FIGURE 4

Sand and gravel (FILL)



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	401-2	3	0.76 - 1.22

ProGrain ver. 2.01

Project 981-8018B

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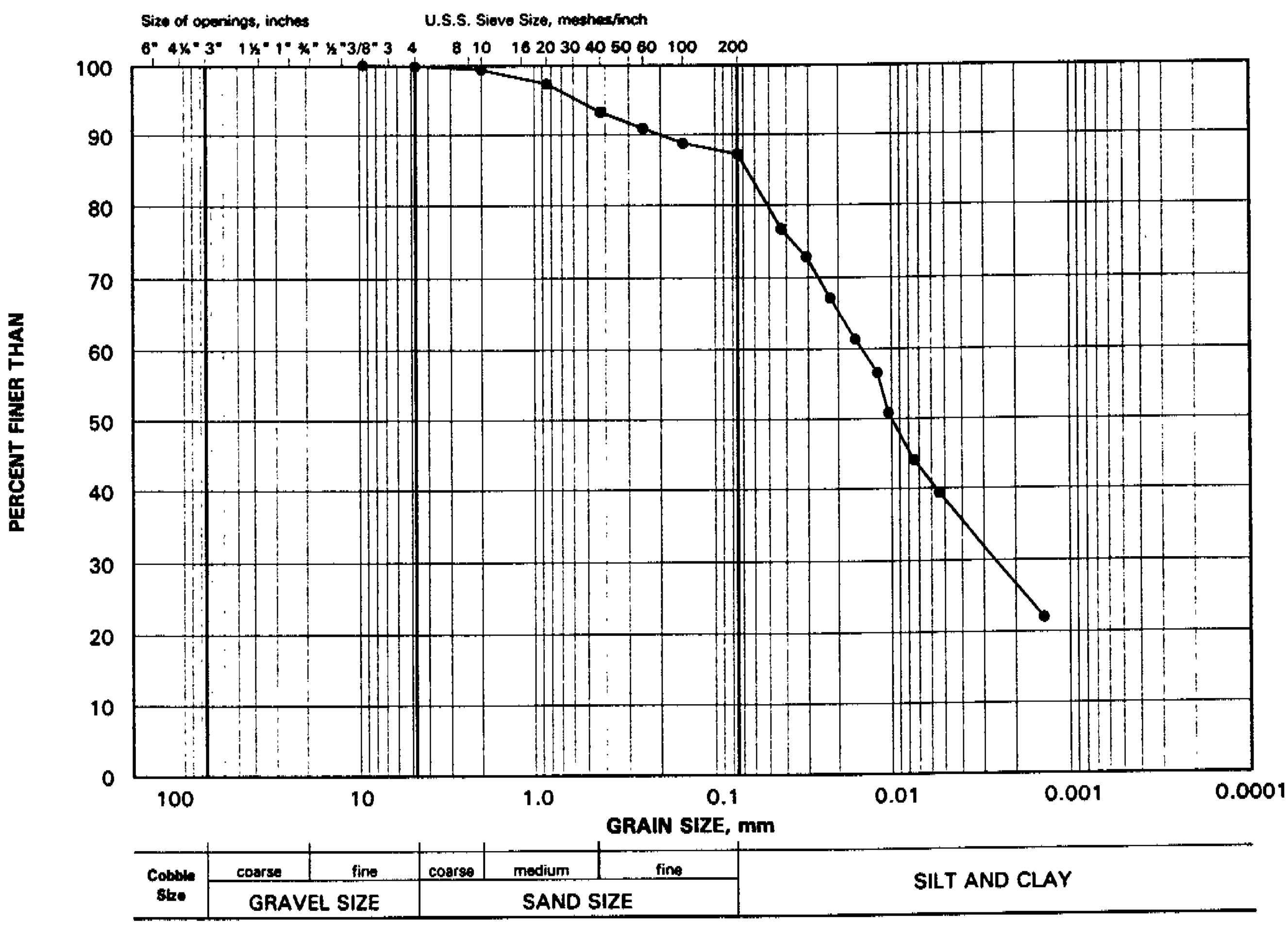
GRAIN SIZE DISTRIBUTION

FIGURE 5

SILTY CLAY



LEGEND			
SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	401-2	5	3.05 - 3.50

ProGrain ver. 2.01

Project 981-8018 B

Golder Associates

Golder Associates Ltd.

100 Scotia Court
Whitby, Ontario, Canada L1N 8Y6
Telephone (905) 723-2727
Fax (905) 723-2182

Golder Associates

GEOCRES # 30M13-146

REPORT ON

FOUNDATION INVESTIGATION
MTO TRUCK INSPECTION STATION
HWY 400 NBL, NORTH OF KING SIDEROAD
TOWNSHIP OF KING, ONTARIO
WP 2506-90-00

Submitted to:

Delcan Corporation
133 Wynford Drive
North York, Ontario
M3C 1K1

Distribution:

4 copies - Delcan Corporation
2 copies - Golder Associates Ltd.

June, 1998

981-8018A

TABLE OF CONTENTS

LIST OF FIGURES

1.0 INTRODUCTION

Golder Associates Ltd. has been retained by Delcan Corporation (Delcan), on behalf of the Ministry of Transportation, Ontario (MTO), to carry out a foundation investigation at the site of the MTO Truck Inspection Station (TIS) and Weight Scale, Highway 400 (NBL), north of King Sideroad, Township of King.

The purpose of this investigation is determine the subsurface conditions at the site for the reconstruction of the TIS building by means of a limited number of boreholes, in-situ tests and laboratory tests on selected samples.

The terms of reference for this work scope are outlined in our proposal letter P81-8051, dated April 21, 1998.

2.0 SITE DESCRIPTION AND GEOLOGY

The site is situated east of Highway 400 NBL within the MTO corridor between stations 242+50 to 244+50. The existing building at the TIS, was constructed in 1972. The land surrounding the site is generally flat to the east and west of the right of way with a gradual grade increase to the north along Highway 400. The site location is shown on the attached Key Plan, Figure 1.

Physiographically, the site is located within the region known as the Oak Ridges Moraine. The moraine is the product of the advance and retreat of the Wisconsin glacier, which covered the area during the Pleistocene stage of the Quaternary period. The surface of the moraine is typically hilly with a "knob-and basin relief" with fairly level tracts of sand between the hills (Chapman and Putnam). The hills generally consist of sandy or gravely materials with the exception of some of the highest ridges, which are formed of boulder clay above the outwash. Grading for the construction of Highway 400, in deep cut areas, exposed till within proximity of the surface. At the site location, the moraine consists predominately of clay till.

Based on available Paleozoic geologic information, the overburden at the site is underlain by grey shales and limestones of the Meaford-Dundas Formation deposited during the Ordovician geologic time. The bedrock/overburden contact is believed to occur at depths of in excess of 120 m.

3.0 INVESTIGATION PROCEDURE

The fieldwork for this project was carried out on May 8, 1998 when three boreholes were completed. Two (2) boreholes were drilled to a depth of 6.55 m using a truck mounted CME 55 drill rig equipped with solid stem augers for the foundation investigation. In the boreholes, samples were retrieved at regular intervals using a standard 35 mm I.D. split spoon sampler driven in accordance with the Standard Penetration Test (SPT – ASTM D 1586). Groundwater levels were determined by monitoring the water level in the open boreholes throughout the duration of the field investigation. All boreholes were backfilled upon completion of fieldwork.

The fieldwork was supervised on a full time basis by a member of our technical staff who located the boreholes, directed the drilling, sampling and in-situ testing operations and logged the boreholes. All recovered samples were identified in the field and then properly sealed in plastic containers to preserve in situ moisture content in the soil. The samples were then transported to the Golder laboratory in Whitby, where additional visual classification was carried out and pertinent laboratory testing was conducted.

Sample preparation and laboratory tests were conducted in accordance with the respective procedures outlined in the MTO Laboratory Testing Manual and as described in the MTO Soil Classification Manual.

The borehole locations and elevations were provided by Delcan and we understand that the elevations are referenced to Geodetic Datum. The northing and easting co-ordinates of the boreholes are shown on the Record of Borehole sheets attached.

4.0 SUBSURFACE CONDITIONS

4.1 SITE STRATIGRAPHY

The detailed subsurface soil and shallow groundwater conditions encountered in the boreholes together with the results of the laboratory tests carried out on selected soil samples are presented on the appended Record of Borehole sheets and on Figures following the text of this report. Within the proposed building foundation, the subsurface conditions consist of variable surficial fill material comprised of a mixture of old granular base and subbase, fine to medium sand and clayey silt ranging in thickness at the borehole locations from 1.8 m to 3.1 m. In each borehole, a 150 mm thick layer of asphaltic concrete was encountered at the surface. The fill is underlain by silty fine sand (Borehole 400-2) and/or clayey silt to silty clay till (Boreholes 400-1).

The locations of the boreholes are shown on Figure 2. A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2 FILL

A silty sand with some gravel or sandy gravel (granular base material) fill, underlain by a fine to medium sand with some silt and a trace of gravel (subbase) fill was encountered in Boreholes 400-1 and 400-2 below the asphaltic concrete. These strata were in a relatively compact to dense state with "N" values of 12 to 30 blows per 0.3 m penetration. In Borehole 400-2 the sand fill was underlain by a brown to dark grey clayey silt fill with a trace of sand, some gravel and a trace of organics and pieces of asphalt. The measured "N" values for the clayey silt fill ranged from 16 to 19, indicating a very stiff consistency. The in situ moisture content of the sand fill ranged from 5 to 16 per cent, while the clayey silt fill moisture contents varied from 13 to 16 per cent.

4.3 SAND

A wet silty fine sand deposit was fully penetrated in Borehole 400-2 at a depth of 3.81 m. The sand is in a relatively compact state of packing with a standard penetration test (SPT) "N" value of 14 blows for 0.3 m penetration. The measured natural moisture content of the sample tested was 18 per cent.

4.4 CLAYEY SILT AND SILTY CLAY

Underlying the fill in Boreholes 400-1 is a deposit of brown clayey silt, containing a trace to some sand and a trace gravel. The silty sand in Borehole 400-2 is underlain by a brown silty clay stratum with a trace of sand and gravel. The consistency of the clayey silt is very stiff to hard (Borehole 400-1) with "N" values ranging from 16 to 33 blows per 0.3 m penetration. Wet silty fine sand seams and/or medium to coarse sand deposits were encountered in Boreholes 400-1 and 400-2.

One measured "N" value for the silty clay was 15 blows for 0.3 m penetration indicating a stiff consistency. The natural moisture content for the clayey silt/silty clay deposits varied from 14 to 18 per cent.

4.5 GROUNDWATER CONDITIONS

The water levels in the open boreholes ranged from 3.20 to 3.35 m depth below the existing ground surface during drilling operations.

The boreholes were open upon completion of drilling. It should be noted that the water levels are subject to seasonal fluctuations

5.0 ENGINEERING RECOMMENDATIONS

This section of the report provides our recommendations on the geotechnical aspects for the design of the new TIS building and the associated servicing based on our interpretation of the factual information obtained during the investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction method and scheduling.

The works described in this report are associated with the proposed demolition and reconstruction of the TIS building. It is understood that a new pre-fabricated building supported on piers, with an elevated floor, will be erected in the location as shown on the borehole location plan.

5.1 BUILDING FOUNDATIONS

The subsoils encountered in the boreholes put down at this site are not suitable for support of shallow spread footings, at the nominal 1.2 m depth required for frost protection, due to the extent of the existing fill. Therefore, it is recommended that the footings should be supported on an engineered fill consisting of a minimum one metre thick layer of well compacted Granular A material. The engineered fill envelope should extend at least one metre beyond the perimeter at the base of the footing and outward at a 1:1 slope for the full depth of the engineered fill. Prior to placing the engineered fill, the exposed base should be proof rolled and inspected by the geotechnical engineer to verify that it is suitable for the support of the engineered fill. The site excavated materials, which are free of organics or other deleterious materials may be used for backfill around the engineered fill and outside the zone of influence of footings. Assuming this fill will be required to perform as the subgrade for the restored pavement, it should be placed in maximum 300 mm loose lifts and uniformly compacted to at least 95 per cent of standard Proctor maximum dry density. Footings supported by the engineered fill can be designed for an allowable soil bearing pressure of 150 kPa. The Granular A engineered fill should be placed in loose lifts not exceeding 300 mm in thickness and be uniformly compacted to at least 100 % standard Proctor maximum dry density. Exterior footings should be provided with at least 1.2 m of earth cover for frost protection.

Alternatively, the building can be supported on augured caissons founded at least 1.5 caisson diameters into the clayey silt till and using an allowable soil bearing pressure of 400 kPa. A minimum caisson diameter of 800 mm and the use of a casing liner for the entire length of the caisson are required to allow inspection of the caissons.

The caisson excavations should be cleaned of all loose or disturbed soil and inspected by geotechnical personnel from Golder Associates to ensure that founding subsoils are consistent with those identified in the boreholes and are capable of supporting the design loads.

The excavated materials from the site will be variable but mainly consist of sand and gravel, silty sand and clayey silt and are considered suitable for reuse as compacted fill. The backfill material should be placed in maximum 300 mm loose lifts and uniformly compacted to at least 95 per cent of standard Proctor maximum dry density. Any existing fill to be removed from the site will require environmental testing to establish suitable disposal options.

5.2 EXCAVATION AND GROUND WATER CONTROL

Based on the results of the field work, excavations for the placement of the engineered fill to support spread footings and for site servicing may be carried out using conventional open cut procedures. Removal of the foundations of the existing demolished building will be required. In addition, it may be necessary to underpin the existing footings of the weigh scale if the footings are founded above the excavation required for the engineered fill. The side slopes of the excavation should comply with the Occupational Health and Safety Act.

In general, ground water is not expected to pose any significant problems for the placement of the engineered fill or the caissons. Seepage water or surface run-off that enters the excavation should be readily controlled by conventional pumping. Removal (pumping) of water from the base of the caissons may be required prior to pouring concrete.

Prior to construction the geotechnical aspects of the final design drawings and specifications should be reviewed by this office to confirm that the intent of this report has been met.

We trust that this report provides sufficient geotechnical information for you to proceed with the design of this project. If you have any questions regarding the contents of this report, please do not hesitate to contact this office.

Yours truly,

GOLDER ASSOCIATES LTD.

V. Aurilio, P.Eng.
Senior Materials Engineer

REGISTERED PROFESSIONAL ENGINEER
F. J. HEFFERNAN
PROVINCE OF ONTARIO

Fin Heffernan, P.Eng.
Consultant

VA:FJH:va

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.).

Dynamic Penetration Resistance; N_d:
The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT):
An electronic cone penetrometer with a 60° conical tip and a projected end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	c_u, s_u kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_S)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane test (LV-laboratory vane test)
γ	unit weight

Note:
1. Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π = 3.1416
ln x, natural logarithm of x
$\log_{10}$ x or log x, logarithm of x to base 10
g acceleration due to gravity
t time
F factor of safety
V volume
W weight

II. STRESS AND STRAIN

γ shear strain
Δ change in, e.g. in stress: $\Delta\sigma$
ε linear strain
ε_v volumetric strain
η coefficient of viscosity
ν Poisson's ratio
σ total stress
σ' effective stress ($\sigma' = \sigma - u$)
σ'_{vo} initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$ principal stresses (major, intermediate, minor)
σ_{oct} mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ shear stress
u porewater pressure
E modulus of deformation
G shear modulus of deformation
K bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$ bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$ dry density (dry unit weight)
$\rho_w(\gamma_w)$ density (unit weight) of water
$\rho_s(\gamma_s)$ density (unit weight) of solid particles
γ' unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R relative density (specific gravity)of solid particles ($D_R = \rho_s/\rho_w$) (formerly G_s)
e void ratio
n porosity
S degree of saturation

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

(a) Index Properties (con't.)

w water content
w_l liquid limit
w_p plastic limit
I_P plasticity Index = $(w_l - w_p)$
w_S shrinkage limit
I_L liquidity index = $(w - w_p)/I_p$
I_C consistency index = $(w_l - w)/I_p$
e_{max} void ratio in loosest state
e_{min} void ratio in densest state
I_D density index = $(e_{max} - e)/(e_{max} - e_{min})$ (formerly relative density)

(c) Hydraulic Properties

h hydraulic head or potential
q rate of flow
v velocity of flow
i hydraulic gradient
k hydraulic conductivity (coefficient of permeability)
j seepage force per unit volume

(d) Consolidation (one-dimensional)

C_c compression index (normally consolidated range)
C_r recompression index (overconsolidated range)
C_S swelling index
C_α coefficient of secondary consolidation
m_v coefficient of volume change
c_v coefficient of consolidation
T_v time factor (vertical direction)
U degree of consolidation
σ'_p pre-consolidation pressure
OCR Overconsolidation ratio $=\sigma'_p/\sigma'_{vo}$

(e) Shear Strength

τ_p, τ_r peak and residual shear strength
ϕ' effective angle of internal friction
δ angle of interface friction
μ coefficient of friction = $\tan\delta$
c' effective cohesion
c_u, s_u undrained shear strength ($\phi = 0$ analysis)
p mean total stress $(\sigma_1 + \sigma_3)/2$
p' mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q $(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u compressive strength $(\sigma_1 - \sigma_3)$
S_t sensitivity

Notes: 1. $\tau = c' + \sigma' \tan\phi'$
2. Shear strength = (Compressive strength)/2

PROJECT: 981-8018A

RECORD OF BOREHOLE 400-1

SHEET 1 OF 1

LOCATION: N 4868818.2, E 299133.6

BORING DATE: May 8, 1998

DATUM: Geodetic

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

Boring method: POWER AUGER, 114 mm Dia. Solid Stem Augers

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	Pavement Surface	311.90					
0	ASPHALT	0.00					
	Silty sand, some gravel; Compact, brown, dry (FILL)	0.15	1	50 DO	12		
	Fine to medium sand, some silt, trace gravel; Compact to dense; Brown; Moist to wet (FILL)	0.30	2	50 DO	30		
		310.07 / 1.83	3A / 3B	50 DO	17		
2	CLAYEY SILT, trace to some sand, trace gravel; Very stiff to hard; Brown to grey; Moist (TILL)		4	50 DO	33	MH	
3	— Wet medium to coarse sand seam at 3.28 m		5	50 DO	16		▽
5	— Wet gravelly coarse sand seam at 4.88 m		6	50 DO	32		
6		305.35 / 6.55	7	50 DO	17		Water encountered at approx. 3.35 m during drilling, May 8/98
	END OF BOREHOLE						Borehole open upon completion of drilling, May 8/98

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - +, Q - ●, rem V - ⊕, U - O); HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT (Wp, W, Wl; 10, 20, 30, 40)

Depth scale: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

DEPTH SCALE 1 to 50

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N8018A01.BH

DATA INPUT: ph n8018a01.bh 5/98

SOILMS

PROJECT: 981-8018A

RECORD OF BOREHOLE 400-2

SHEET 1 OF 1

LOCATION: N 4868804.0, E 299134.6

BORING DATE: May 8, 1998

DATUM: Geodetic

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

Boring method: POWER AUGER, 114 mm Dia. Solid Stem Augers

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	WATER CONTENT (%)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Pavement Surface		312.00						
	ASPHALT		0.00						
	Sand and gravel, some silt Compact, reddish-brown, dry to moist (FILL)		0.15	1A	50 DO	17	○		
	Fine to medium sand, some silt, trace gravel Compact, brown (FILL)		311.55 0.45	1B			○		
				2A	50 DO	19	○		
1	Clayey silt, trace sand, trace gravel, some organics, trace roots Very stiff Brown to dark grey Moist (FILL)		311.03 0.97	2B			○		
				3	50 DO	16	○		
2				4	50 DO	16	○		
3	SILTY fine SAND, trace gravel Compact Brown to grey Wet		308.95 3.05	5	50 DO	14	○	M	▽
4	SILTY CLAY, trace sand, trace gravel Stiff to very stiff Grey Moist to wet (TILL)		308.19 3.81	6	50 DO	15	○		
6	CLAYEY SILT, trace sand, trace to some gravel Very stiff, grey, moist to wet (TILL)		305.90 6.10	7	50 DO	23	○		
	END OF BOREHOLE		305.45 6.55						Water encountered at approx. 3.20 m during drilling, May 8/98 Borehole open to 6.40 upon completion of drilling

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

SHEAR STRENGTH Cu, kPa — nat V - + Q - ● rem V - ⊕ U - ○

HYDRAULIC CONDUCTIVITY, k, cm/s

WATER CONTENT, PERCENT — Wp ├──○W──┤ Wl — 10 20 30 40

DEPTH SCALE 1 to 50

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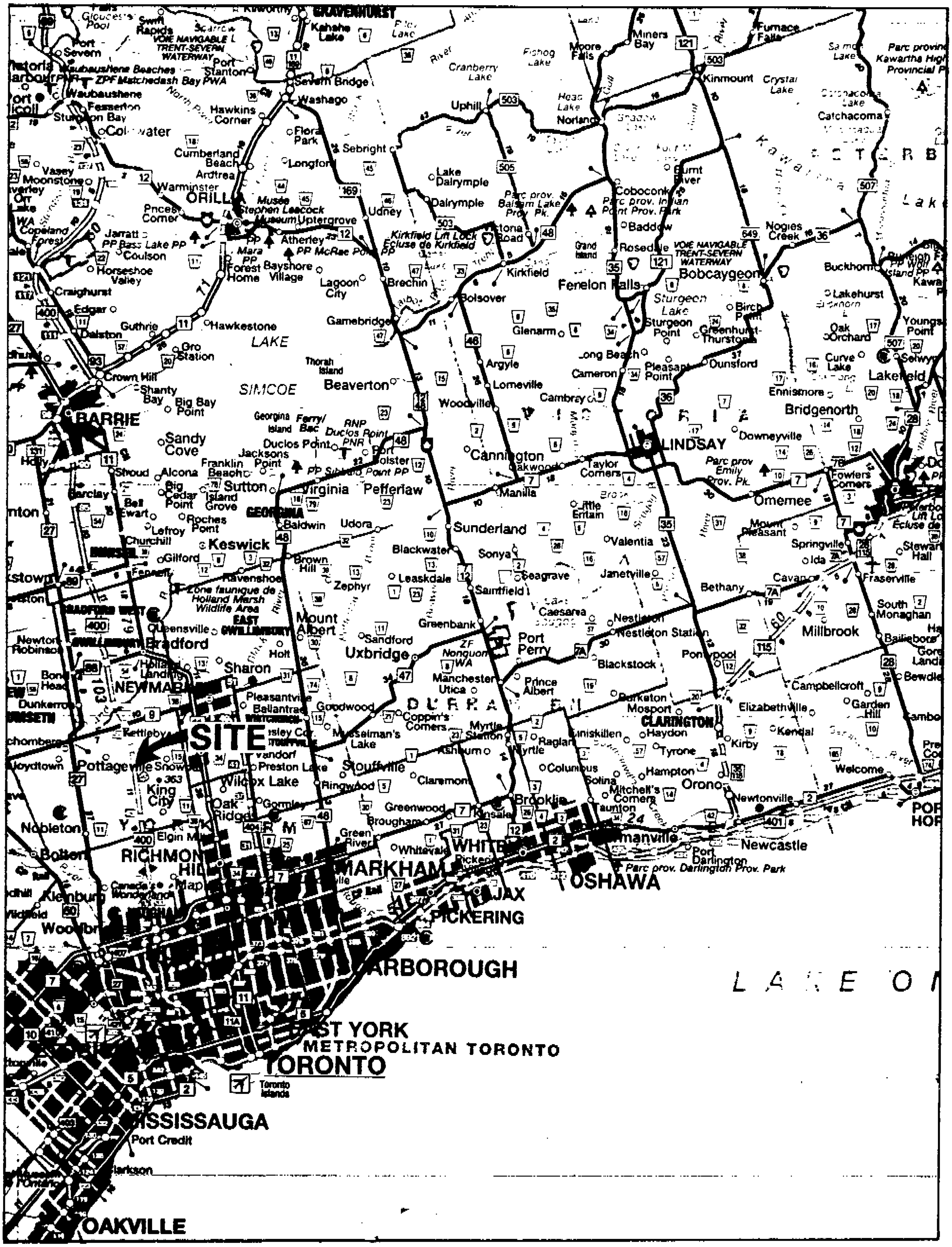
KEY PLAN

FIGURE 1



Date May, 1998

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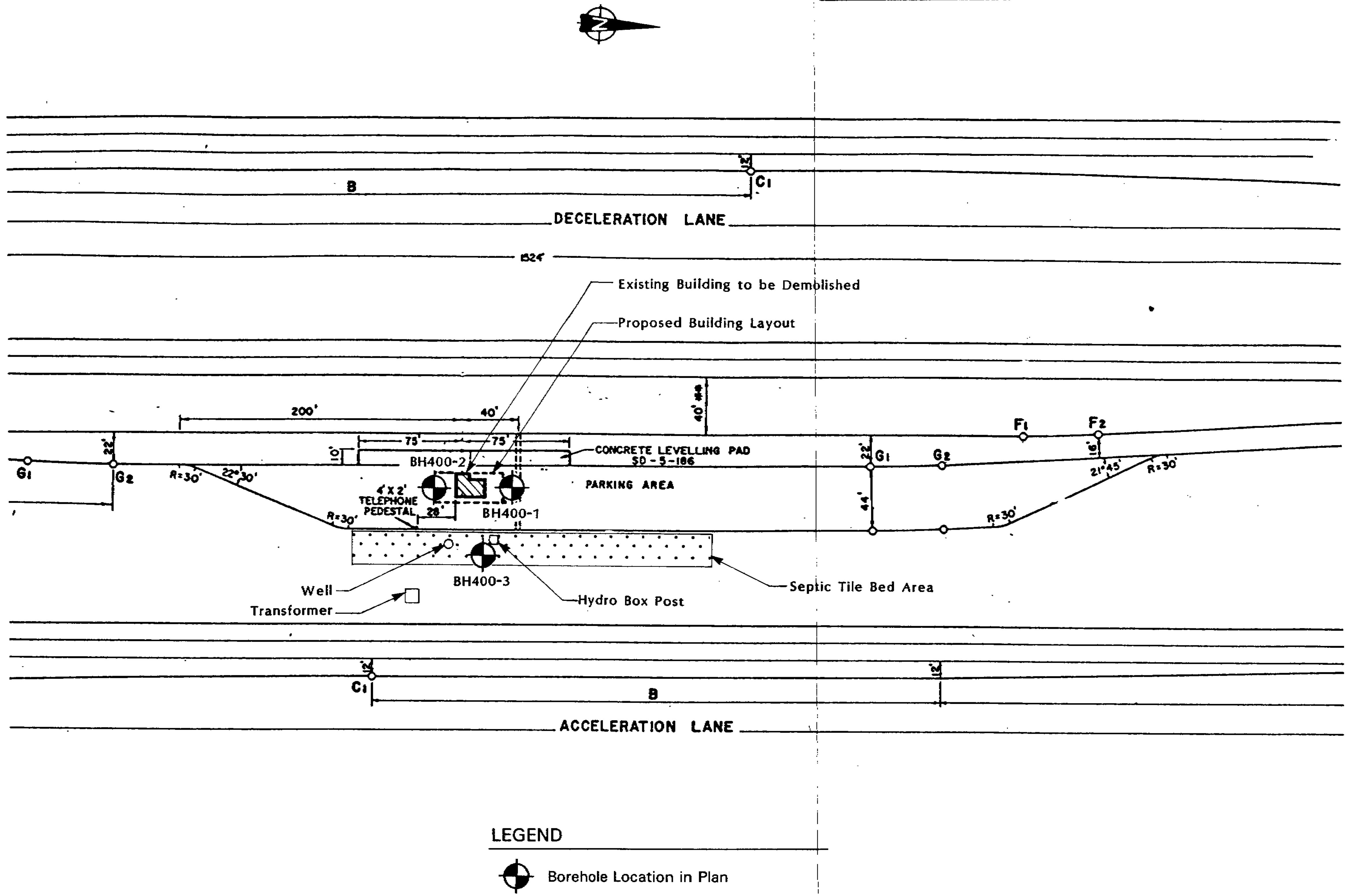
Drawn PH

Chkd.

BOREHOLE LOCATION PLAN — FIGURE 2



Date: May, 1998
Project: 981-8018A

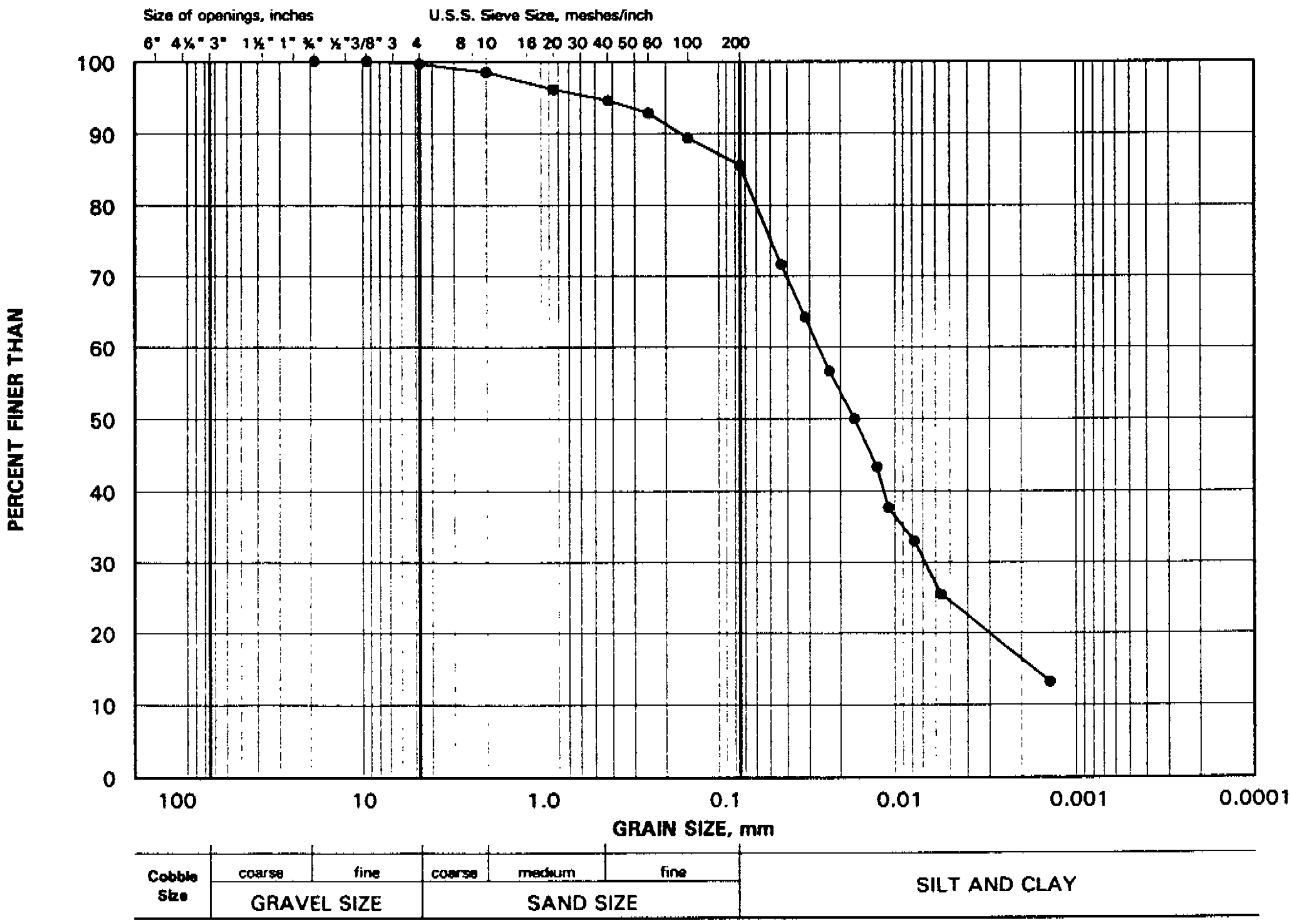
Golder Associates

Drawn: PH
Chkd:

GRAIN SIZE DISTRIBUTION

FIGURE 3

CLAYEY SILT TILL



LEGEND			
SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
•	400-1	4	2.29 - 2.74

ProGrain ver. 2.01

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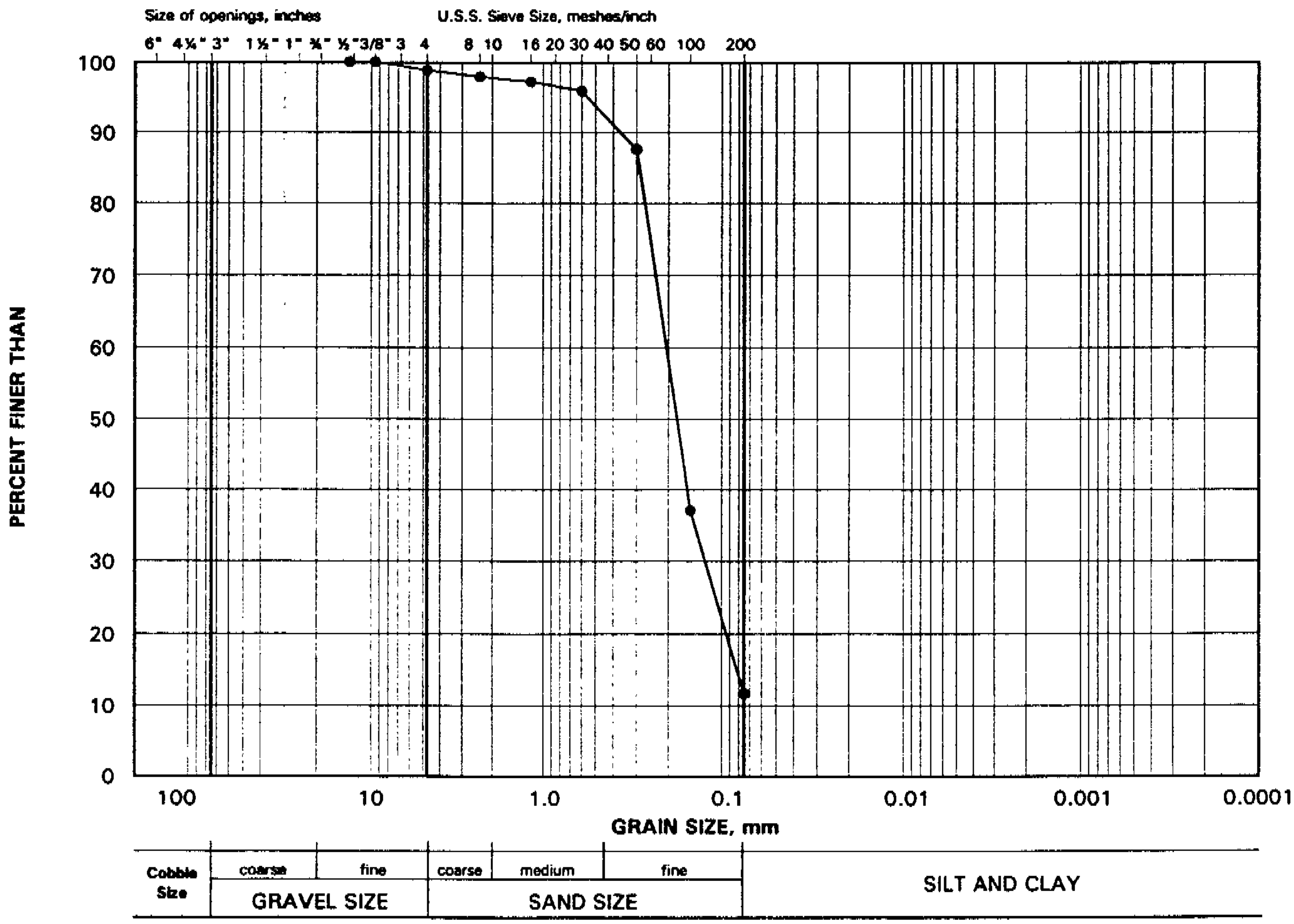
GRAIN SIZE DISTRIBUTION

FIGURE 4

Sand (FILL)



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	400-3	2A	0.76 - 1.22

ProGrain ver. 2.01

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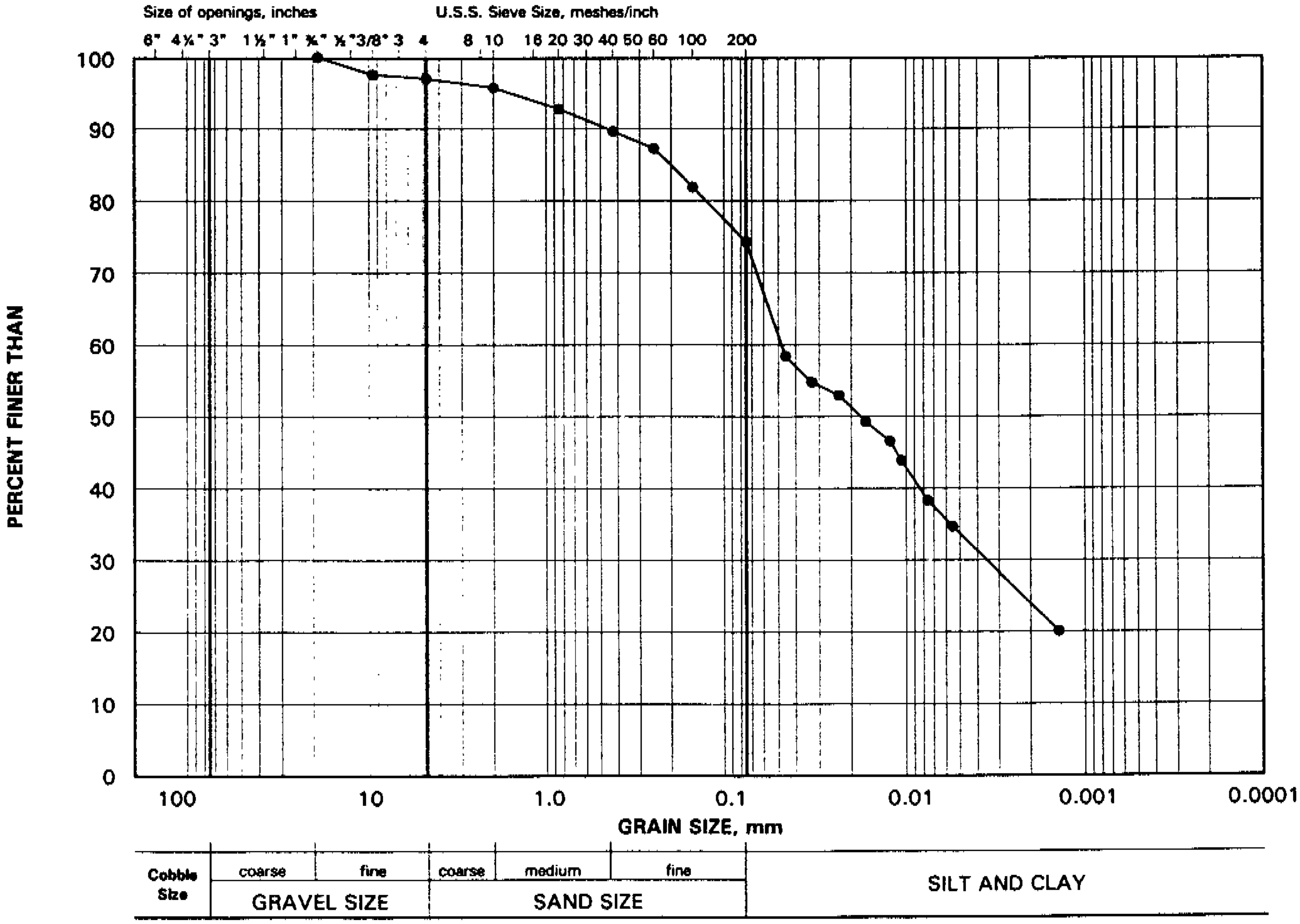
GRAIN SIZE DISTRIBUTION

FIGURE 5

CLAYEY SILT TILL



LEGEND			
SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
•	400-3	2B	1.22 - 1.52

ProGrain ver. 2.01

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