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**GEOTECHNICAL REPORT
PROPOSED SANITARY SEWER REPLACEMENT
ALONG HARBORN ROAD
FROM GRANGE DRIVE TO HURONTARIO STREET
MISSISSAUGA, ONTARIO
G.W.P. 566-90-00**

Submitted to:

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December 2005

04-1181-142



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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) was retained by Giffels Associates Limited (Giffels) to carry out a geotechnical investigation and provide geotechnical engineering recommendations for the proposed sanitary sewer replacement along Harborn Road, from Grange Drive to Hurontario Street, as part of the reconstruction of the Queen Elizabeth Way (QEW) – Hurontario Street interchange in Mississauga, Ontario. The results of the borehole investigation and geotechnical engineering recommendations for the proposed sanitary sewer replacement are documented in this report. The scope of work for the geotechnical investigation and engineering services is outlined in Golder's letter to Giffels dated August 16, 2005.

The reader is referred to the "Important Information and Limitations of This Report" that follows the text of this report but forms an integral part of this document. In this regard, it should be noted that Golder's professional services for this assignment address only the geotechnical (physical) aspects of the subsurface conditions at this site. The geoenvironmental (chemical) aspects, including consequences of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources, are outside the terms of reference for this report and have not been investigated or addressed by Golder.

2.0 PROJECT AND SITE DESCRIPTION

The site is located immediately north of the existing Queen Elizabeth Way (QEW) – Hurontario Street interchange, extending along Harborn Road from the east side of Hurontario Street westward for one block to the east side of Grange Road, as shown on Drawing 1. Residential and light commercial buildings are present along the local roads.

The existing ground surface is relatively flat along the proposed sewer alignment, at about Elevation 101 m. It is understood that the existing 1,050 mm diameter sanitary sewer in this area has been installed with its invert at approximately Elevation 96 m.

3.0 INVESTIGATION PROCEDURES

The field work for this geotechnical investigation was carried out on October 25, 2005 and November 9, 2005, during which time three boreholes (Boreholes 1 to 3) were drilled to investigate the subsurface conditions along the proposed sewer alignment. The borehole locations, which were proposed by Giffels and which are understood to be in the vicinity of the proposed new manholes, are shown on Drawing 1.

The borehole investigation was carried out using a truck-mounted Diedrich D-50 drill rig, supplied and operated by a specialist drilling contractor. The boreholes were advanced using a 108 mm outside diameter solid stem augers. Samples were obtained at 0.76 m intervals of depth using a 50 mm outside diameter split-spoon sampler driven by an automatic hammer in accordance with the Standard Penetration Test (SPT) procedure. The boreholes were advanced to depths of between 6.4 m and 7.9 m, and terminated within the shale bedrock at the site.

The water level was measured in the open boreholes prior to backfilling with bentonite pellets. A piezometer was installed in Borehole 2 to permit further monitoring of the groundwater level at the site. The piezometer consists of a 1.5 m long, 50 mm diameter slotted screen installed within a 3.7 m long sand pack, then backfilled to ground surface with bentonite pellets. The piezometer installation details and the water level measurements are shown on the borehole records.

The field work was supervised throughout by a member of Golder's technical staff, who located the boreholes in the field, arranged for the clearance of underground services, supervised the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder's Mississauga geotechnical laboratory where the samples underwent further visual examination and laboratory testing. Classification tests (water content determinations and Atterberg limit tests) were carried out on selected soil samples.

The borehole locations proposed by Giffels were measured by Golder personnel relative to existing site features. The following table summarizes the borehole locations, which are based on the NAD83 MTM (Zone 12) co-ordinate system, and the ground surface elevations, which have been determined using the digital terrain model (DTM) for this project and which are referenced to the geodetic datum.

<i>Borehole Number</i>	<i>MTM NAD83 Northing (m)</i>	<i>MTM NAD83 Easting (m)</i>	<i>Ground Surface Elevation (m)</i>
1	4825555.0	296608.1	101.8
2	4825509.5	296557.3	101.4
3	4825485.6	296506.7	101.3

4.0 SUBSURFACE CONDITIONS

4.1 Regional Geology

The study area for this project lies within the Iroquois Plain physiographic region, as delineated in *The Physiography of Southern Ontario*¹ and *Urban Geology of Canadian Cities*².

The Iroquois Plain stretches along the northern shoreline of Lake Ontario, extending from the Niagara Escarpment in the west to the Scarborough Bluffs in the east. The Iroquois Plain soils consist of glaciolacustrine sediments – mainly coarse-grained sands and gravels – deposited in glacial Lake Iroquois.

The bedrock underlying the Toronto and Mississauga area consists of three shale-dominated units: from oldest to youngest, they are the Blue Mountain, Georgian Bay and Queenston Formations. These bedrock formations are essentially horizontally bedded, although on a regional scale, they dip gently to the south. The Georgian Bay Formation, which underlies this study area, consists mainly of a blue-grey shale, containing siltstone, sandstone and limestone interbeds.

4.2 Subsoil Conditions Along Proposed Sewer Alignment

As part of the subsurface investigation for the proposed sanitary sewer replacement, three boreholes were advanced at the locations shown on Drawing 1. The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of geotechnical laboratory testing are shown on the Record of Borehole sheets and on Figure 1. The stratigraphic boundaries shown on the borehole records are inferred from drilling observations and non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In general, the subsoil conditions consist of fill material overlying a loose to very dense sand to silty sand deposit, underlain at some locations by a stiff to hard clayey silt till deposit that is in turn underlain by shale bedrock. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

¹ Chapman, L.J. and D.F. Putnam. *The Physiography of Southern Ontario*, Ontario Geological Survey Special Volume 2, Third Edition, 1984. Accompanied by Map P.2715, Scale 1:600,000.

² C.L.Baker, L.R.Lahti and D.C.Roumbanis: *Urban Geology of Toronto and Surrounding Area*, P.F.Karrow and O.L.White *Urban Geology of Canadian Cities*, Geological Association of Canada Special Paper , 1998.

4.2.1 Topsoil

Approximately 100 mm to 200 mm of topsoil was encountered immediately below ground surface at all three borehole locations.

4.2.2 Fill

A brown fill material was encountered underlying the topsoil in all three boreholes. The base of this layer was encountered in the boreholes between Elevations 97.5 m and 99.9 m, at about 1.5 m to 4 m depth. The fill material consists of sand, trace silt to silty sand, containing trace to some gravel. Brick fragments were noted within the fill samples in Borehole 2.

The measured Standard Penetration Test (SPT) 'N' values within the fill material were between 7 and 22 blows per 0.3 m of penetration, indicating that the fill has a loose to compact relative density.

4.2.3 Silty Sand to Sand

Below the fill lies a deposit of grey sand, trace silt to silty sand, containing trace gravel and trace wood fragments (as observed in Borehole 1). The surface of the silty sand to sand deposit was encountered between Elevations 97.5 m and 99.9 m in all the boreholes, and the deposit varied between 1.5 m and 3.8 m in thickness.

The measured SPT 'N' values within the silty sand to sand deposit were typically between 22 and 53 blows per 0.3 m of penetration, indicating a generally compact to very dense relative density, except at Borehole 1 where an SPT 'N' value of 7 blows per 0.3 m of penetration was measured which indicates that the silty sand to sand in this area has a loose relative density. The measured natural water contents of two selected samples of the silty sand to sand deposit are between 10 and 15 per cent.

4.2.4 Clayey Silt Till

A deposit of grey clayey silt till, containing some sand and trace gravel, was encountered underlying the silty sand to sand deposit in Boreholes 1 and 2. The surface of the till deposit was encountered between Elevations 95.4 m and 96.1 m, and the deposit is 1.2 m to 2.3 m thick as encountered in these boreholes.

An Atterberg limits test was conducted on one sample of the till and the result is shown on a plasticity chart on Figure 1. The test measured a liquid limit of 23 percent, a plastic limit of 15 percent, and a corresponding plasticity index of 8 percent, which confirms that this till deposit is a clayey silt of low plasticity.

The SPT 'N' values measured within the clayey silt till were between 30 and 44 blows per 0.3 m of penetration, indicative of a hard consistency, except at Borehole 1 where an SPT 'N' value of 9 blows per 0.3 m of penetration was measured at the transition zone between the overlying silty sand deposit and the clayey silt till. The natural water content measured on two selected samples of the clayey silt till is between 9 and 15 per cent.

4.2.5 Bedrock

Bedrock was encountered in all three boreholes between Elevations 93.8 m and 96.0 m (between about 5.3 m and 7.6 m depth). The boreholes were advanced into the shale bedrock by augering and split spoon sampling.

The recovered split-spoon samples indicate that the bedrock is grey shale of the Georgian Bay Formation. It is noted that stronger seams and interlayers of limestone are known to be present within the Georgian Bay Formation.

The SPT 'N' values measured within the upper 0.3 m to 1.1 m of bedrock ranged from 52 blows to more than 100 blows per 0.3 m of penetration.

4.3 Groundwater Conditions

Groundwater Level

The water level in the open boreholes was observed to be between 4.6 m and 5.2 m depth (Elevations 96.1 m to 96.6 m) upon completion of drilling. A standpipe piezometer was installed and sealed within the sand deposit in Borehole 2. The water level in the piezometer was measured at 3.7 m depth (Elevation 97.7 m) on November 16, 2005, and at 3.65 m depth (Elevation 97.75 m) on December 5, 2005. Details of the installation are shown on the Record of Borehole sheet.

It should be noted that seasonal fluctuations of the groundwater level are anticipated; the groundwater level is expected to be higher during the spring and following periods of heavy precipitation.

Results of Rising Head Test in Piezometer

A rising head test was carried out in the standpipe piezometer that was installed in Borehole 2, and the test data are included in Appendix A.

Applying the Hvorslev method of analysis, the hydraulic conductivity of the aquifer in the vicinity of the well screen at Borehole 2 is calculated to be approximately 4×10^{-4} cm/s. This hydraulic conductivity is typical of sand to silty sand. It is noted that the hydraulic conductivity determined from this testing applies to the soils in the immediate vicinity of the well screen. The aquifer (i.e. silty sand to sand deposit) composition and permeability will vary beyond the piezometer location.

5.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

5.1 General

The recommendations provided in this report are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at this site. The interpretation and recommendations provided are intended to provide the designers with sufficient information to carry out the design and plan for construction. Where comments are made on construction, they are provided in order to highlight those aspects which could affect the design of the project and for which special provision could be required during construction. Those requiring detailed information on aspects of construction should make their own interpretation of the factual information provided, as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

The sanitary sewer replacement work includes a new 1,050 mm diameter sewer that will be approximately 73.8 m in length, and a 1,500 mm diameter sewer that will be 50.9 m in length. Both sewers will be constructed along Harborn Road from Grange Drive to Hurontario Street. In the preparation of this report, it has been assumed that the invert elevation for the replacement sewer will be similar to that for the existing 1,050 mm diameter sewer, which varies from approximately Elevation 96.1 m at Harborn Road to Elevation 95.9 m near Hurontario Street.

5.2 Trench Excavations

Assuming an invert level of approximately Elevation 96 m, trench excavations for the replacement sewer will extend to about 5.5 m to 6 m below the existing ground surface. The excavations will extend below the water level at the site, and will terminate near the silty sand-till or till-bedrock interface.

5.2.1 Surface Preparation

It is recommended that provision be made in the contract for the existing pavement to be saw-cut to minimize disturbance to the adjacent sections of the roadway and to facilitate removal of the asphalt and concrete where the excavations will cross Hurontario Street and Harborn Drive.

5.2.2 Dewatering

Excavations for installation of the new sewers will extend below the groundwater level at this site, which has been measured at approximately 3.7 m depth (Elevation 97.7 m) in the piezometer installed in Borehole 2. Thus, unless the excavation is supported by a shoring system that provides a complete cut-off to groundwater flow through the sand to silty sand (such as an interlocking steel sheet-pile wall driven to penetrate into the clayey silt till and/or the shale

bedrock) it should be specified that the sand deposit be dewatered prior to the excavation being advanced to below the preconstruction groundwater level in order to control the excavation base and sides. It is estimated³ that the total flow rate into the excavations for the sewer replacements will approach, or may exceed, 50 m³/day, depending on the staging of the excavation and dewatering operations. A permit to take water is required from the Ministry of the Environment when a groundwater taking exceeds 50 m³/day. Given the anticipated groundwater flow rates and the inherent uncertainty in predicting such rates from limited subsurface data, it is recommended that a permit to take water be obtained for this project, unless a cut-off wall shoring system is adopted.

It is noted that the excavations will terminate near the silty sand-till interface, and even with closely spaced dewatering wells, it will not be possible to fully dewater the silty sand soils immediately above the till interface. Provision will, therefore, have to be made during construction for handling and removing saturated sand at the interface with the till and/or bedrock.

5.2.3 Open-Cut Excavations

All excavations should be carried out in accordance with latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities. The surficial fill, silty sand to sand and clayey silt till soils are classified as Type 3 soils according to the OHSA. Where space permits (for example, for the portion of the replacement sewer that is located south of Harborn Street), temporary open-cut excavations through these materials should be made with side slopes not steeper than 1 horizontal to 1 vertical (1H:1V).

5.2.4 Temporary Excavation Support

It is expected that temporary excavation support will be required for the portions of the trench that cross Hurontario Street and Harborn Road. A prefabricated support system (liner box) could be used for the trench support, provided that some degree of movement in the adjacent utilities can be tolerated. If a trench liner box is adopted, the contract should require that the longitudinal open sections of the trench be kept to a minimum and backfilling of the trench should be carried out immediately behind the liner box.

If any utilities adjacent to the trench excavation are intolerant of settlement, the use of a sheeted and braced excavation may be required to minimize ground and utility deformation. In this case, the lateral movement of the temporary shoring system should be specified to meet the settlement tolerances for adjacent utilities (for example, the lateral movement of the shoring system could be

³ Using Jacob's modified non-equilibrium equation, from F.G. Driscoll's *Groundwater and Wells*, Second Edition, 1986.

specified to meet Performance Level 2 as per MTO's Special Provision 105S19, which allows for up to 25 mm of deformation adjacent to the excavation). In addition, it is recommended that settlement-sensitive utilities within the zone of influence of the excavation be monitored and corrective action taken should excessive ground movement be observed.

5.2.5 Bedrock Excavation

The shale bedrock at the site is a member of the Georgian Bay Formation, which is known to contain stronger limestone, dolostone and siltstone interlayers. If excavation into the bedrock is required for the sanitary sewer replacement, it is recommended that the contract documents include a Non-Standard Special Provision to warn the contractor of this condition for excavation planning purposes.

5.2.6 Subgrade Protection

Based on the results of the borehole investigation and assuming that the invert elevation of the replacement sewer and manholes will be similar to that for the existing sewer, the excavation subgrade will consist predominantly of clayey silt till or shale bedrock, but could include a relatively thin layer of saturated sand. Provision should be made to remove any remaining sand from below the planned subgrade level, with replacement of the subexcavated material with lean mix concrete or compacted sewer bedding. The clayey silt till and shale bedrock materials are susceptible to weathering on exposure to the air or water (for the shale), and disturbance due to construction traffic and ponded water. It is recommended that the bedding, pipe and cover material be placed within four hours following exposure of the excavation subgrade to minimize such degradation.

5.3 Pipe Bedding and Cover Materials

The bedding material for the sewer pipe should consist of an approved granular material consistent with the type and class of pipe to be used, and in accordance with the standards of the City of Mississauga and the Region of Peel. Provided that the manufacturer's specifications are met, the bedding and cover for the sewer could consist of OPSS 1010 Granular 'A' material extending from approximately 150 mm below the pipe invert to 300 mm above the pipe obvert. The bedding and cover material should be placed in 150 mm thick lifts and uniformly compacted to 95 per cent of the material's standard Proctor maximum dry density. Hand tamping around the pipe may be required to ensure that no voids are present below the spring line of the pipe. It is also important to provide a well compacted granular base for the pipe within the approach zone of the pipe at the manhole areas.

Clear stone should not be used as a bedding material, given the fine-grained nature of the soils at the site and the resultant potential for loss of fine soil particles into the void spaces in the clear

stone, with consequent ground surface settlement. If clear stone is adopted within the sewer trench, it would be necessary to use a geotextile filter fabric. The filter fabric should be placed to completely surround the clear stone, with an overlap of at least 300 mm to minimize the potential for loss of granular cover material or surrounding native soils into the clear stone voids. Care must be taken during placement of soil over the geotextile to ensure that the overlaps are completely compressed together. Otherwise, a potential still exists for fine-grained soil to move into the clear stone through wrinkles and openings in the overlap. The use of stitched overlaps is recommended if clear stone is adopted.

5.4 Trench Backfill

Depending on the Region of Peel and City of Mississauga requirements, consideration could be given to the use of “unshrinkable fill” (as per OPSS 1359) for trench backfill within the limits of the paved roadways and sidewalks, to avoid settlement of the pavements that would be associated with post-construction compression of other types of trench backfill materials. If permitted, consideration could also be given to the use of Granular “A” or “B” backfill above the pipe cover material within the roadway limits.

Outside of the roadway crossing areas, it is anticipated that the majority of the excavated soils from above the water table will be suitable for re-use as general trench backfill above the pipe cover material, provided that the soils are free of organic matter, foreign material, snow, ice, frozen soil, and other deleterious materials. The placement water content of the backfill material should be within about 2 per cent of the optimum water content for compaction purposes to avoid difficulties in achieving the specified degree of compaction. It is expected that some drying of the excavated soils from below the water table will be necessary in order to reuse this material as backfill; excessively wet materials should be wasted.

The backfill material should be placed in loose lifts not exceeding 300 mm in thickness, and be uniformly compacted to at least 95 per cent of the material’s standard Proctor maximum dry density. The upper 1 m of the trench backfill below the pavement sub-base should be compacted to at least 98 per cent of the material’s standard Proctor maximum dry density.

Post-construction settlement of the compacted trench backfill should be anticipated, with the majority of such settlement taking place within approximately six months following the completion of the trench backfilling. This settlement will be reflected by some subsidence of the ground surface and/or restored pavement surface, and may require local repairs. It is recommended that the reconstruction of surface coat asphalt and sidewalks be delayed until such time as post-construction settlement of the trench backfill has occurred. Alternatively, as noted above, consideration should be given to the use of lean concrete as backfill to the trench.

6.0 CLOSURE

The factual data, interpretations and recommendations contained in this report pertain to a specific project as described in the report and are not applicable to any other project or site location. If the project is modified in concept, location or elevation, or if the project is not initiated within twelve months of the date of this report, Golder Associates Ltd. should be given an opportunity to confirm that the recommendations are still valid.

GOLDER ASSOCIATES LTD.

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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
SS	Split-spoon
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

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

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.)

Consistency

	<u>kPa</u>	<u>psf</u>
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

(b) Cohesive Soils

c_u, s_u

Dynamic Cone 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)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² 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.

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 (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 stress (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 (continued)

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)

(b) 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

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_a	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	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$



(d) 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

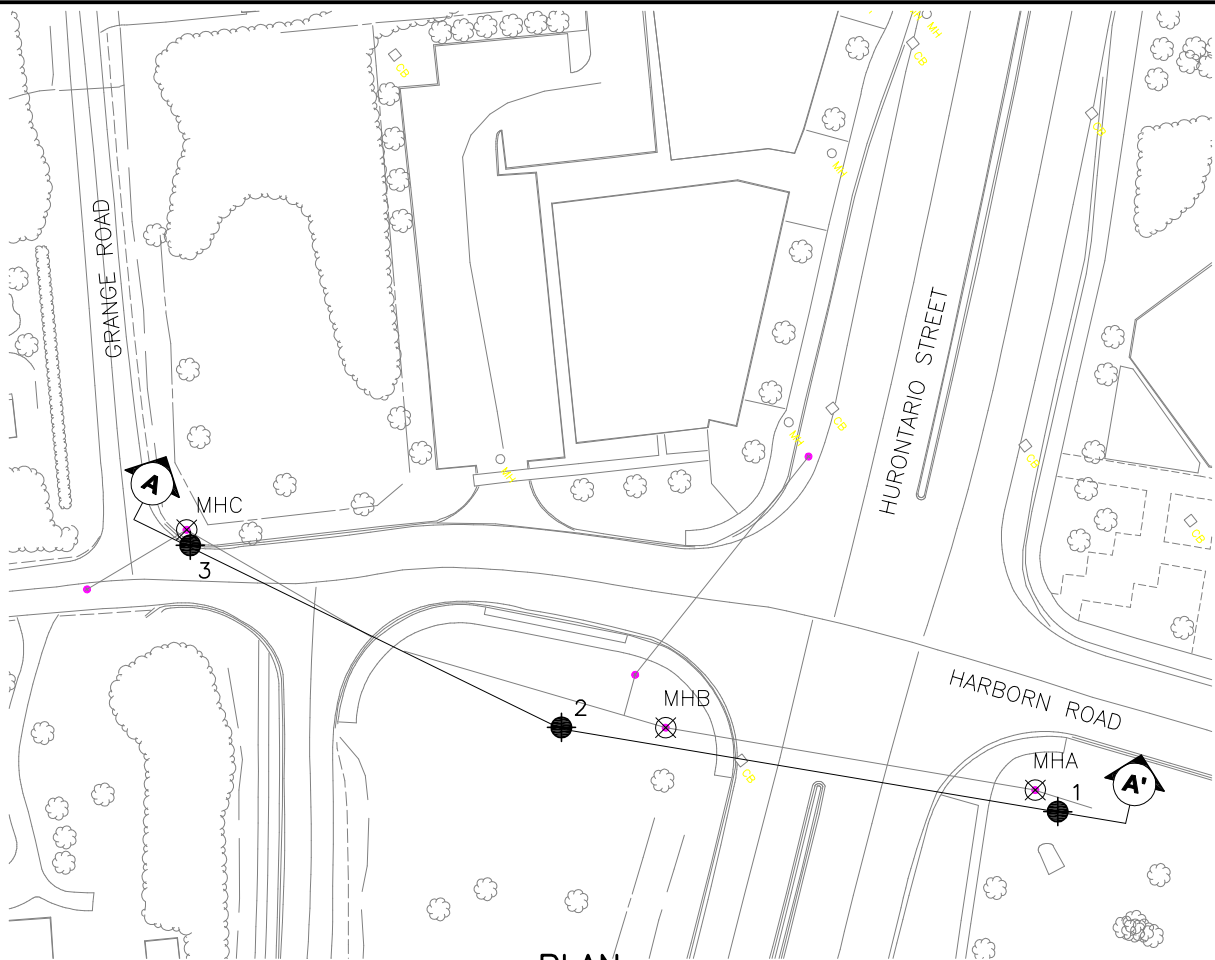
- Notes:** 1 $\tau = c' + \sigma' \tan \phi'$
2 Shear strength = (Compressive strength)/2

PROJECT		04-1181-142		RECORD OF BOREHOLE No 1				1 OF 1 METRIC									
W.P.		566-90-00		LOCATION		N 4825555.0 ; E 296608.1		ORIGINATED BY									
DIST		HWY QEW		BOREHOLE TYPE		Power Auger, 108 mm O.D. Solid Stem Augers		COMPILED BY									
DATUM		Geodetic		DATE		November 9, 2005		CHECKED BY									
								BLT									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
101.8	GROUND SURFACE																
8.9	TOPSOIL Silty SAND, trace gravel (FILL) Loose to compact Brown Moist		1	SS	7												
			2	SS	22												
			3	SS	7												
			4	SS	7												
			5	SS	5												
97.8	Silty SAND, trace gravel, containing wood fragments Loose Grey Wet		6	SS	7												
4.0																	
95.4	Clayey SILT, some sand, trace gravel (TILL) Stiff Grey Wet		7	SS	9												
6.4																	
94.2	SHALE (BEDROCK) Grey		8	SS	>75												
93.9	END OF BOREHOLE																
7.9	NOTE: 1. Water level at 5.2m depth (Elev. 96.6m) upon completion of drilling.																

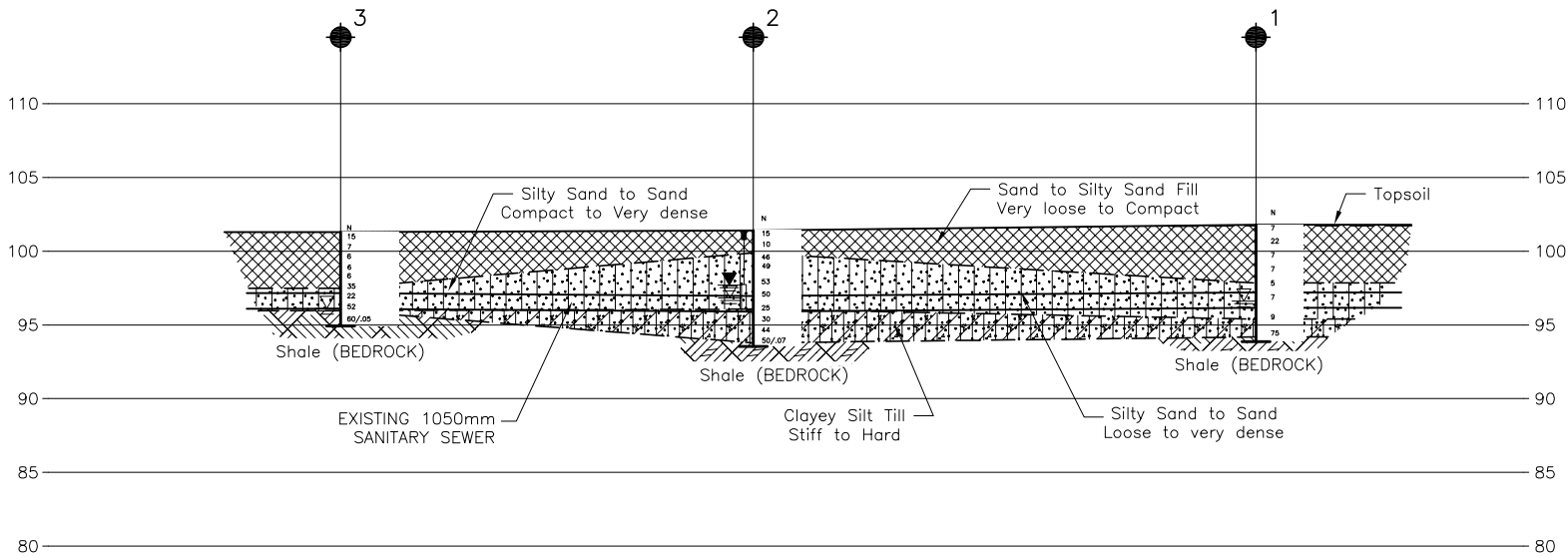
PROJECT 04-1181-142			RECORD OF BOREHOLE No 2				1 OF 1 METRIC								
W.P. 566-90-00			LOCATION N 4825509.5 ; E 296557.3				ORIGINATED BY GD								
DIST _____ HWY QEW			BOREHOLE TYPE Power Auger, 108 mm O.D. Solid Stem Augers				COMPILED BY NK/BLT								
DATUM Geodetic			DATE October 25, 2005				CHECKED BY BLT								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
101.4	GROUND SURFACE														
0.0	TOPSOIL		1	SS	15										
0.2	SAND, some gravel, trace silt, containing brick fragments (FILL) Compact Brown Moist		2	SS	10										
99.9															
1.5	SAND, trace silt Compact to very dense Brown Moist to wet		3	SS	46										
			4	SS	49										
			5	SS	53										
			6	SS	50										
			7	SS	25										
96.1															
5.3	Clayey SILT, some sand, trace gravel (TILL) Hard Grey Moist		8	SS	30										
			9	SS	44										
93.8															
7.9	SHALE (BEDROCK) Grey END OF BOREHOLE		10	SS	50/.07										
NOTES: 1. Water level at 4.6m depth (Elev. 96.8m) upon completion of drilling. 2. Standpipe piezometer installed at 5.5m depth (Elev. 95.9m). 3. Water level was measured at 3.68m depth (Elev. 97.7m on 16 Nov. 2005.															

PROJECT <u>04-1181-142</u>				RECORD OF BOREHOLE No 3				1 OF 1 METRIC									
W.P. <u>566-90-00</u>				LOCATION <u>N 4825485.6 ; E 296506.7</u>				ORIGINATED BY <u>GD</u>									
DIST <u> </u> HWY <u>QEW</u>				BOREHOLE TYPE <u>Power Auger, 108 mm O.D. Solid Stem Augers</u>				COMPILED BY <u>NK/BLT</u>									
DATUM <u>Geodetic</u>				DATE <u>October 25, 2005</u>				CHECKED BY <u>BLT</u>									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
101.3	GROUND SURFACE						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div>										
8.0	TOPSOIL		1	SS	15	▽	101										
	SAND, trace silt, trace gravel (FILL)		2	SS	7		100										
	Loose		3	SS	6		99										
	Brown		4	SS	6		98										
	Moist to wet		5	SS	6		97										
97.5	Silty SAND, trace clay	6	SS	35													
3.8	Dense to compact	7	SS	22													
	Grey																
	Moist to wet																
96.0	SHALE (BEDROCK)		8	SS	52	96											
5.3	Grey																
94.9	END OF BOREHOLE		9	SS	60/.05	95											
6.4	NOTE: 1. Water level at 5.2m depth (Elev. 96.1m) upon completion of drilling.																

MISS_MTO 041181142.GPJ ON_MOT.GDT 11/25/05 DD



PLAN
SCALE
10 0 10 20 m



PROFILE A-A'
SCALE
10 0 10 20 m
SCALE
5 0 5 10 m

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

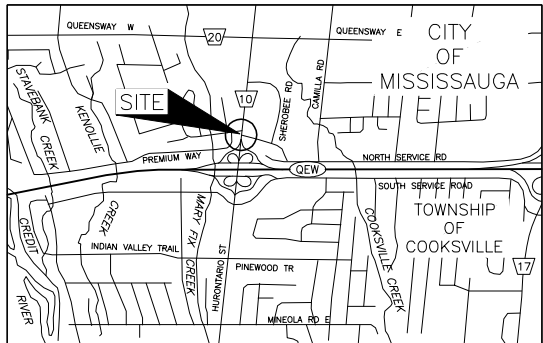
CONT No.
WP No. 566-90-00



SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

SCALE
500 0 500 m

LEGEND

- Borehole - Current Investigation
- Indicative Proposed Manhole
- Seal
- Piezometer
- Standard Penetration Test Value
- Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- Rock Quality Designation (RQD)
- WL in piezometer, measured on Nov 16, 2005
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
1	101.8	4825555.0	296608.1
2	101.4	4825509.5	296557.3
3	101.3	4825485.6	296506.7

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

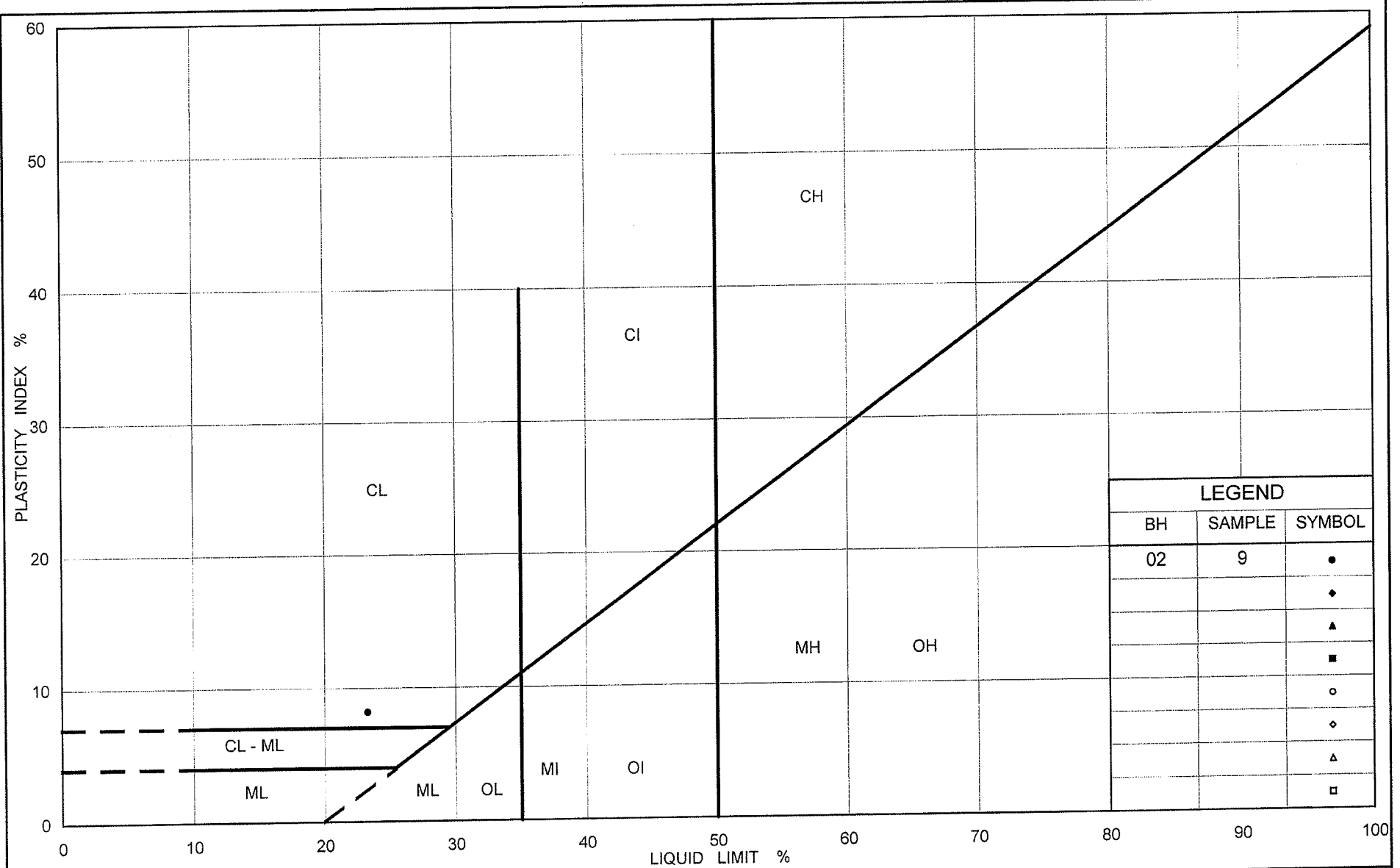
The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by GIFFELS, drawing file Harbourn.dwg, received October 30, 2005 and obrn-2004-1.dwg, dated Feb 25, 2005, received Feb 28, 2005.

NO.	DATE	BY	REVISION
Geocres No.			
HWY. HARBORN ROAD		PROJECT NO. 04-1181-142	DIST.
SUBM'D. NK	CHKD. BLT	DATE: NOV 2005	SITE:
DRAWN: MSM	CHKD. LCC	APPD. LCC	DWG. 1



Ministry of Transportation

Ontario

PLASTICITY CHART Clayey Silt Till

FIG No. 1

Project No. 04-1181-142

APPENDIX A

RESULTS OF RISING HEAD TEST IN BOREHOLE 2



Job No.: 04-1181-142
 Borehole No: 2
 Test Date : December 5, 2005
 Elevation : 101.4 m
 Well Installation : 5.5 m depth (Elevation 95.9 m)

Static Water Level =	3.65 m
H =	97.75 m
Ho =	96.32 m

r =	0.0508 m
Le =	1.524 m
R =	0.10795 m
To =	9.28 min

Using Hvorslev's Law:
$$K = \frac{r^2 \ln\left(\frac{Le}{R}\right)}{2 Le T_o}$$

K =	2.42E-04 m/min
	4.03E-04 cm/s

Δt (min)	Reading (m BGS)	h	H-h/H-Ho
0	5.08	96.32	1
0.5	5.05	96.35	0.97902098
1	5.02	96.38	0.95804196
1.5	4.97	96.43	0.92307692
2	4.90	96.50	0.87412587
2.5	4.86	96.54	0.84615385
3			
3.5	4.78	96.62	0.79020979
4	4.72	96.68	0.74825175
4.5	4.66	96.74	0.70629371
5	4.55	96.85	0.62937063
6	4.52	96.88	0.60839161
7	4.50	96.90	0.59440559
8	4.36	97.04	0.4965035
9	4.30	97.10	0.45454545
10	4.20	97.20	0.38461538
12	4.10	97.30	0.31468531
14	3.96	97.44	0.21678322
16	3.89	97.51	0.16783217
18	3.82	97.58	0.11888112
20	3.78	97.62	0.09090909
25	3.71	97.69	0.04195804
30	3.68	97.72	0.02097902
35	3.65	97.75	0
40	3.65	97.75	0

Rising Head Test for BH No. 2

