Ontario

Ministry of Transportation
Southwestern Region
Planning and Design Section

HIGHWAY 400 PLANNING AND PRELIMINARY DESIGN STUDY

FROM 1 KM SOUTH OF HIGHWAY 89 NORTHERLY 30 KM
TO THE JUNCTION AT HIGHWAY 11

COUNTY OF SIMCOE
TOWN OF BRADFORD WEST GWILLIMBURY
TOWN OF INNISFIL
CITY OF BARRIE
TOWNSHIP OF SPRINGWATER

G.W.P. 30-95-00

PRELIMINARY DESIGN REPORT

VOLUME 4 – APPENDIX J TO APPENDIX T

MAY 2005

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G.W.P. 30-95-00

APPENDIX J

PRELIMINARY GEOTECHNICAL / PAVEMENT DESIGN REPORT

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REPORT ON

PRELIMINARY GEOTECHNICAL/PAVEMENT DESIGN REPORT HIGHWAY 400 ULTIMATE WIDENING BETWEEN HIGHWAYS 89 AND 11 GWP 30-95-00, AGREEMENT #3005-A-000074 MINISTRY OF TRANSPORTATION, ONTARIO - SOUTHWESTERN REGION

Submitted to:

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75 Commerce Valley Drive East
Thornhill, Ontario
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May 23, 2003 001-3203

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BEST MANAGED PRIVATE COMPANIES

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May 23, 2003 001-3203

URS Cole, Sherman
75 Commerce Valley Drive East
Thornhill, Ontario
L3T 7N9

Attention: Mr. L.V.S. Kozachuk, P. Eng.

RE: PRELIMINARY GEOTECHNICAL/PAVEMENT DESIGN REPORT HIGHWAY 400 ULTIMATE WIDENING BETWEEN HIGHWAYS 89 AND 11 GWP 30-95-00, AGREEMENT #3005-A-000074 MINISTRY OF TRANSPORTATION, ONTARIO - SOUTHWESTERN REGION

Dear Sirs:

This report presents the results of the review of the readily available background information carried out for the preliminary design for the interim and ultimate widening of the section of Highway 400 described as GWP 30-95-00. This section of Highway 400 is about 30 kilometres in length and extends from approximately 1.0 kilometre south of Highway 89 northerly to the junction of Highway 11. It also includes improvements to interchanges and service centres, replacement or widening of bridges and culvert extensions.

The purpose of the review was to assess the existing pavement structure(s) and the near surface soil and groundwater conditions along the alignment of this portion of Highway 400 and to provide preliminary geotechnical engineering recommendations for the design of the interim and ultimate widening and rehabilitation of the highway as well as for the rehabilitation of interchange ramps and the construction of new ramps.

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OFFICES ACROSS NORTH AMERICA, SOUTH AMERICA, EUROPE, ASIA, AUSTRALASIA

We trust that this pavement design report provides the geotechnical information that you require. If any point requires further clarification, or if you need further information, please contact this office.

Yours truly,

GOLDER ASSOCIATES LTD.

Michael E. Beadle, P. Eng.

Philip R. Bedell, P. Eng.

MEB/PRB/MLJM/cl
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1.0 PROJECT DESCRIPTION

GWP 30-95-00 involves the preliminary design for the interim eight lane and ultimate ten lane widening of a portion of Highway 400. The project is about 30 kilometres in length extending from approximately 1 kilometre south of Highway 89 northerly to junction of Highway 11. The project also includes improvements to interchanges. The location of the project is shown on the Plan, Figure 1 and a Site Location Plan is shown on Figure 2.

1.1 Existing Design

Highway 400, within the project limits, is a Class I, six lane divided freeway with 3.66 metre lanes and 3.05 metre wide outside shoulders and 1.2 to 1.8 metre wide median paved shoulders. A box beam guiderail barrier is present in much of the median. The existing median width varies from 3.0 metres to 9.1 metres. A summary is provided below:

	MEDIAN WIDTH (m)			
LOCATION (Station)	3.0	4.3	9.1	Transition
Township of West Gwillimbury				
25+154 to 25+911			✓	
25+911 to 26+154				✓
26+154 to 27+552	✓			
Township of Innisfil				
10+000 to 22+276	✓			
22+276 to 22+550		✓		
22+550 to 26+780*	✓			
26+780 to 27+100*				✓
27+100 to 10+975*			✓	
27+975 to 28+200*				✓
28+200 to 29+789*	✓			
Township of Vespra				
10+100 to 10+100*	✓			
10+100 to 10+260*				✓
10+260 to 10+920*			✓	
10+920 to 11+110*				✓

Township of Vespra				
11+110 to 13+536*	✓			
13+536 to 13+719				✓
13+719 to 18+687			✓	

NOTES:
1. Station 27+552, Township of West Gwillimbury = Station 10+000, Township of Innisfil.
2. Station 29+739, Township of Innisfil = Station 10+000, Township of Vespra.

1.2 Historical Pavement Data

The original grading for this portion of Highway 400 was carried out under Contracts 46-19, 50-10, 50-11 and 50-12 to accommodate two lanes in each direction. During the original grading, 300 millimetres of granular materials, on average, were placed and earth shoulders were constructed. Subsequently, additional granular materials were placed in the main lanes; however, the thickness of these granulars was apparently not documented. The original grading was such as to provide a 9.1 metre wide median.

Paving was carried out under Contracts 51-36, 52-03 and 52-04. Later investigation by the Ministry of Transportation, Ontario (MTO) for the 1971 widening and resurfacing indicated that the average asphalt thickness was about 200 millimetres. Details were not provided.

Under Contract 66-509, from just south of Essa Road to Dunlop Street, the north southbound lanes were resurfaced together with localized sections north of Dunlop Street.

Within the project limits, in 1971, the highway was widened to the inside south of Bayfield Street and to the outside north of Bayfield Street and resurfaced to accommodate six 3.66 metre lanes, three in each direction, with 3.05 metre outside shoulders (Contracts 71-145, 71-146, 71-147, 71-148 and 71-151). The widening and resurfacing reportedly consisted of:

- south of Bayfield Street
  - outside plug widening 255 mm (10") HL 4

- inside widening — 130 mm (5") HL 4
  150 mm (6") minimum Granular A
  600 mm (24") minimum Granular C starting 0.6 metres from median edge of pavement

- resurfacing — 30 mm (1 ¼") HL 1 surface course
  30 mm (1 ¼") HL 4 binder course
  25 mm (1") minimum HL 4 padding

- paved outer shoulder — 30 mm (1 ¼") HL 3 surface course
  50 mm (2") HL 4 binder course
  Granular A continuous with pavement granular

- paved median shoulder — 30 mm (1 ¼") HL 3 surface course
  45 (1 ¾") mm HL 4 binder course
  Granular A and Granular C continuous with pavement granulars

- north of Bayfield Street

- outside widening — 100 mm (4") HL 4
  150 mm (6") Granular A
  600 mm (24") Granular C

- inside plug widening — 50 mm (2") HL 4
  230 mm (9") minimum Granular A

- resurfacing — 30 mm (1 ¼") HL 1 surface course
  30 mm (1 ¼") HL 4 binder course
  25 mm (1") minimum HL 4 padding

- paved outer shoulder — 30 (1 ¼") mm HL 3 surface course
  50 mm (2") HL 4 binder course
  Granular A continuous with pavement granular

- paved median shoulder — 30 mm (1 ¼") HL 3 surface course
  30 mm (1 ¼") HL 4 binder course
  25 mm (1") minimum HL 4 padding
  50 mm (2") HL 4
  230 mm (9") minimum Granular A

Typical cross-sections are provided on Figures 3 and 4 for the inside and outside widenings, respectively. A graphical summary of the available pavement structure information within the project limits is provided on Figures 5, 6, 7 and 8.

1.3 Maintenance History

Subsequent to the 1971 widening, Contract 90-07 involved the rehabilitation of the north and southbound driving lanes due to severe wheel track rutting between Highway 89 and Essa Road. One isolated section in the southbound median and centre lanes was also resurfaced, however, the location was not documented. The resurfacing reportedly consisted of milling 90 millimetres and paving 50 millimetres modified HL 1 binder course and 40 millimetres modified HL 1 surface course. Exempt from Contract 90-07 were 3 kilometres of the southbound driving lane between Stations 14+400 and 17+400, Township of Innisfil, which contains a trial section for five polymer modified asphalt mixes designed to resist rutting.

GWP 198-89-01, from 1.4 kilometres north of Highway 89 to 0.7 kilometres south of Essa Road, recommended resurfacing the north and southbound lanes. This involved:

- mill 40 millimetres in the driving lane and pave 40 millimetres dense friction course (DFC); and
- mill 90 millimetres from median and centre lanes and pave 50 millimetres modified heavy duty binder course (HDBC) and 40 millimetres DFC.
- to correct pavement failures, severe cracking and rutting distortions, an additional 50 millimetres was to be milled and replaced with modified HDBC across all three lanes at the following locations:

NORTHBOUND LANES	SOUTHBOUND LANES
Township of Innisfil	
Station 14+110 to 14+140	Station 14+122 to 14+124
Station 22+530 to 22+630	Station 24+075 to 24+085
	Station 24+095 to 24+105
	Station 24+125 to 24+135
	Station 27+005 to 27+035
	Station 27+165 to 27+185

- In areas prone to frost heaving, 100 millimetre diameter subdrains were installed adjacent to the outside edge of the driving lane pavement at the following locations:

NORTHBOUND LANES	SOUTHBOUND LANES
Township of Innisfil	
Station 14+110 to 14+140	Station 24+070 to 24+140
Station 22+430 to 22+500	Station 27+010 to 27+030
Station 25+125 to 25+145	

Under Contract 96-11, between Highbury 89 and Essa Road, milling and paving was carried out at selected locations on Highway 400 and the Innisfil Beach Road and Mapleview Drive interchange ramp. In addition, longitudinal and transverse crack repairs were also carried out. Details are provided in Tables I and II attached.

Contract 98-65 involved both full and partial depth milling and resurfacing of Highway 400 and the interchange ramps at Melson Park Drive. Details of the main lane works are provided below:

LOCATION	MILLING	PAVING
24+400 to 25+000	Mill 40 mm	40 mm DFC
25+000 to 25+850	Full depth removal	40 mm DFC, 100 mm HDBC, 100 mm HL 4
25+850 to 26+450	Mill 40 mm	40mm DFC

Partial and full depth milling and paving was also carried out on selected sections of the interchange ramps in similar fashion.

Under Contract 2001-3004, the majority of the work consisted of partial depth milling and paving of predominately the centre lane throughout the contract limits. In addition, the interchange ramps between Essa Road and Duckworth Street were rehabilitated. Details of the locations and treatments for the rehabilitation are provided in Tables III and IV attached.

2.0 DESIGN CRITERIA

2.1 Traffic Volumes

Estimates of the traffic data for the main lanes and interchange ramps for this portion of Highway 400 were provided by URS Cole Sherman. The data are summarized in Tables V and VI, attached.

The directional split is considered to be 50 per cent.

3.0 PAVEMENT PERFORMANCE (EXISTING CONDITIONS)

A visual pavement condition survey was conducted by Golder Associates Ltd. (Golder Associates) staff in January 2001. The survey was carried out along the Highway 400 northbound and southbound lanes and all associated interchange ramps within the project limits.

The most significant pavement distress features noted along the evaluated sections were as follows:

DISTRESS FEATURE	SEVERITY	FREQUENCY
Highway 400 Northbound		
Ravelling and Coarse Aggregate Loss	Slight	Extensive
Flushing	Slight	Extensive
Wheel Track Rutting	Slight	Extensive
Single and Multiple Centreline Cracking	Slight	Throughout
Transverse Cracking	Slight	Extensive
Longitudinal Meander and Midlane Cracking	Slight	Intermittent
Map Cracking	Slight	Frequent
Highway 400 Southbound		
Single and Multiple Centreline Cracking	Slight	Throughout
Transverse Cracking	Slight	Throughout
Longitudinal Meander and Midlane Cracking	Slight	Intermittent

In general, the pavement along the existing Highway 400 north and southbound lanes is in fair to good condition having a Pavement Condition Rating (PCR) in the range of 67 to 71 and a Ride Condition Rating (RCR) of 6.5 to 6.8 based on the 2000 MTO data provided.

The condition of the pavement is described based on the PCR descriptions given on page 266 of the MTO Pavement Design and Rehabilitation Manual and the MTO Manual for Condition Rating of Flexible pavements.

A visual pavement condition survey of the associated interchange ramps was carried out. In general, the interchange ramps were noted to be in good condition, however, the most significant pavement distress features are summarized below. It should be noted that the Highway 89, Innisfil Beach Road and Dunlop Street interchange ramps were in good condition.

LOCATION	DISTRESS FEATURE	SEVERITY	FREQUENCY
Essa Road Interchange			
E-S Ramp	Wheel Track Rutting	Slight	Throughout
	Longitudinal Alligator Wheel Track Cracking	Slight to Moderate - intermittent severe areas	Throughout
	Transverse Cracking	Severe	Extensive
W-N and S-E Ramps	Longitudinal Wheel Track Cracking	Moderate	Extensive
	Transverse Cracking	Severe to V. Severe	Throughout
Dunlop Street Interchange			
E/W-S Ramp	Longitudinal Wheel Track Cracking	Moderate - intermittent V. Severe areas	Extensive
	Transverse Cracking	Moderate to Severe	Extensive
	Transverse Alligator Cracking	Severe to V. Severe	Extensive
N-E/W Ramp	Longitudinal Wheel Track and Alligator Cracking	Moderate to Severe	Throughout
	Transverse Cracking	Severe	Throughout
S-W and E/W-N Ramps	Longitudinal Wheel Track and Alligator Cracking	Moderate	Throughout
	Transverse Cracking	Moderate	Throughout
S-E Ramp	Longitudinal Wheel Track and Alligator Cracking	Severe to V. Severe	Throughout
	Transverse	Severe to V. Severe	Extensive
	Longitudinal Midlane and Meander Cracking	Moderate to Severe	Throughout
	Map Cracking	Severe	Extensive
Bayfield Street Interchange			
E-N/S Ramp	Transverse Cracking	Slight to Moderate	Extensive
N/S-E Ramp	Longitudinal Wheel Track and Alligator - severe potholing and alligator cracking on north radius	Severe	Intermittent
Duckworth Street Interchange			
N-S Ramp	Transverse Cracking	Severe	Frequent
	Transverse Alligator Cracking	Moderate	Frequent
N-N/S Ramp	Transverse and Transverse Alligator Cracking	Moderate	Few

Photographs in Appendix A show selected features along the main lanes.

4.0 PHYSIOGRAPHY AND SOILS DATA

4.1 Physiography (Chapman & Putnam, 1984)

The predominant physiographic regions in the area of the site are outlined on Figure 9. From the project south limit to about Essa Road, Highway 400 crosses the Peterborough drumlin field. The underlying bedrock is reported to be limestone of the Lindsay and Verulam formations and is described as highly fossiliferous and easily disintegrated. The general orientation of the drumlin axes is from northeast to southwest. The drumlins are reportedly composed of highly calcareous till of a sandy nature.

As Highway 400 rounds the western point of Kempenfelt Bay, between Essa Road and Bayfield Street, for a distance of about 3.5 kilometres, it traverses the Simcoe lowlands, specifically an area within the Nottawasaga Basin known as the Camp Borden sand plain. The soils are reportedly comprised of the Tioga sand or sandy loam which is described as loose, coarse textured and well drained.

The majority of the northern portion of the project lies within the Simcoe Uplands. The area is described as broad, rolling till plains separated by steep-sided, flat-floored valleys. The till is reportedly derived from Precambrian rock with a gritty loam texture.

4.2 Quaternary Geology

Quaternary geologic mapping is available for the southern 5.8 kilometres of the site. The predominant quaternary geologic regions in this area of the site are outlined on Figure 10.

Based on the available quaternary geologic mapping, the southern 0.9 kilometres of the project lies within glaciolacustrine shallow water sand deposits described as Lake Algonquin deposits.

From 0.8 kilometres to 5.7 kilometres north of Highway 89, the subsurface soils consist predominantly of Newmarket Till. The Newmarket Till is described as a sand to silty sand till.

From 5.7 kilometres north of Highway 89, bedrock topography, and drift thickness mapping is available; however, quaternary geologic mapping is not available.

4.3 Soil Survey of Simcoe County

The surficial soils, according to the Soil Survey of Simcoe County, are outlined on Figure 11. In general, the surficial soils consist of sandy loams. Details of the various soil series and soil types are provided on Figure 11 and are summarized below:

SERIES	SYMBOL	DESCRIPTION	NATURAL DRAINAGE
Alliston sandy loam	Ans	grey calcareous outwash sand	imperfect
Tioga loamy sand	Tis		good
Tioga sandy loam	Tisl		good
Bondhead loam	Bl	light grey calcareous loam and sandy loam till	good
Bondhead sandy loam	Bs		good
Bondhead sandy loam - steep phase	Bs.s		good
Dundonald sandy loam	Ds	outwash sand underlain by grey calcareous loam or sandy loam till at depth of 0.9 metres or less	good
Muck	M	well decomposed organic material over 0.3 metres thick	very poor
Schomberg clay loam	Shsc	calcareous lacustrine varved silt loam and clay	good
Smithfield silty clay loam	Smsc		imperfect
Vasey sandy loam	Vasl	light grey, calcareous and non-calcareous sandy loam till	good

Reference - Soil Map of Simcoe County, South Sheet, Soil Survey Report No. 29.

The sandy loam materials are predominant throughout the project. The Schomberg clay loam and Smithfield silty clay loam are reportedly located to the northeast of Barrie and about 3 to 5 kilometres north of Highway 89. Muck deposits are reportedly located from about 1 kilometre south to 0.2 kilometres north of Willow Creek.

4.4 Hydrogeology

A hydrogeological assessment was carried out for GWP 30-95-00 by Golder Associates and the results were summarized in a letter from Golder Associates entitled "Hydrogeological Assessment, Preliminary Geotechnical Engineering Services, Proposed Highway 400 Widening, GWP 30-95-00", dated May 8, 2001. The pertinent results of the hydrogeological assessment are provided in the following sections.

4.4.1 Groundwater Investigation

A preliminary groundwater investigation was conducted for the project. The purpose of the investigation was to identify areas of high water table and upwelling, groundwater recharge and discharge and high overburden permeability and the locations and usage of high volume producing water wells.

The tasks completed as part of this investigation included a review of topographic mapping, geological mapping, available Ontario Ministry of Environment (MOE) water well records, and any readily available borehole information from the MTO and Golder Associates' files.

The groundwater investigation presents a preliminary, generalized interpretation of hydrogeologic conditions and has been based on limited available background information. Actual hydrogeological conditions within the project limits will vary and are subject to confirmation with actual site specific investigations including boreholes, test pits, groundwater hydraulic testing, chemical analyses and the like.

Site Hydrogeology

The majority of the site is anticipated to represent an area of groundwater recharge. The most significant recharge zone is expected to be in the area of the Camp Borden sand plain between Essa Road and Bayfield Street. Groundwater discharge is expected to be confined to stream channels and floodplains and wetland areas within the project limits. Based on the MOE water well records and a cursory site reconnaissance, the Cookstown Hollow forest swamp at the southern project limit, and the Little Lake marsh area at the northern limit are anticipated to be groundwater discharge zones.

Based on the information provided on the Surveys and Mapping Branch, Dept. of Energy, Mines and Resources, Alliston Topographic Map 31 D/4 and Barrie Topographic Map 310/5, surface flows from the Innisfil Township 5th Line south generally flow to the southeast north of Highway 89 and northwest from south of Highway 89 to Innisfil Creek.

Between about the 5th Line and Essa Road, surface flows generally flow east and ultimately report to Lover's Creek. Lover's Creek flows northerly from a marsh area on the east side of Highway 400 between the 6th Line and Innisfil Beach Road to outlet at Kempenfelt Bay.

Through Barrie, surface water generally flows east to Kempenfelt Bay.

North of Bayfield Street, surface flows report to the Little Lake marsh area.

Also noted on the mapping, a sewage lagoon is present west of Highway 400, just north of the 4th Line, Innisfil Township and another sewage lagoon is located in the southeast corner of Innisfil Beach Road and Highway 400. A junk yard is located west of Highway 400 and just south of the 14th Concession, West Gwillimbury Township (approximately Station 25+900). A pumping station is located in the northwest corner of Highway 400 and Highway 89. In the southeast corner of Highway 400 and Big Bay Point Road just south of Barrie between Essa Road and Molson Park Drive, a junk yard, a filtration plant and poultry farm are present. Another junk yard is located at the southwest corner of Highway 400 and the Innisfil/Vespra Township boundary. An underground reservoir is located northwest of Highway 400 between Dunlop Street and Cundles Road.

During our field reconnaissance, service centres with refuelling stations were noted at the northwest corner of Highway 400 and Highway 89 and on the east side of Highway 400 just north of Molson Park Drive.

Adjacent to the Highway 400 right-of-way throughout Barrie, several commercial and industrial properties are present. It is understood that a trichloroethylene (TCE) plume is present at the intersection of Tiffin Street and Highway 400. Investigation and implications of the TCE plume will be addressed under separate cover.

Areas of groundwater upwelling or high water table (i.e. less than 1 metre below ground surface) have been identified in the southern, central and northern portions of the project based on the topographic mapping and the MOE water well records examined. Interpreted areas of shallow water table are provided on Figure 12.

Groundwater Usage

Existing groundwater uses in the vicinity of the site consist of private domestic and commercial water wells associated with private residences, farms and commercial properties along the right-of-way. A plot of the locations of water wells within adjacent concessions along the Highway 400 right-of-way, based on MOE water well records, is shown on Figure 12. It should be noted that the MOE water well records are based on records current as of 1977. Additional wells which have been drilled subsequent to the compilation of the well records in 1977 or wells for which records were never filed will be present along the right-of-way. Water well records for large diameter, shallow

bored/dug wells commonly do not exist. A summary of the water well records reviewed is provided in Table VII.

Existing well uses in the area of the site generally consist of drilled wells which utilize relatively shallow overburden aquifers. Of the 257 water well records reviewed, 70 of the wells utilize shallow aquifers (less than 15 metres deep) and a further 4 wells utilize bedrock aquifers. One hundred seventy four wells utilize deep overburden aquifers greater than 15 metres below the ground surface. Thirteen well records indicated flowing conditions and 13 were dry to the depth drilled. Thirteen water well records offered no water level information.

A total of 14 large volume producing water wells (190 litres per minute (L/min) or 50 Imperial gallons per minute (Igpm)), municipal, commercial or public supply water wells were identified from the MOE water well records in the vicinity of the project:

MOE WELL NUMBER	UTM Easting	UTM Northing	TESTED PRODUCTION RATE (L/min)	USAGE
57-1177	604158	4906189	189	domestic
57-1517	603235	4910774	27	public supply
57-1536	604544	4911568	227	public supply
57-1538	604297	4912456	91	public supply
57-1720	602800	4912894	15	public supply
57-4920	605965	4918859	8	municipal
57-4929	607247	4919838	8	public supply
57-4934	607109	4919681	-	public supply
57-4940	607109	4919576	924	public supply
57-4941	607079	4919551	598	public supply
57-8108	605480	4904650	768	commercial
57-9006	605100	4904220	227	commercial
57-11777	606217	4894421	76	municipal
57-11853	605600	4904900	1930	municipal

Groundwater Impact Assessment

Based on the above information, a groundwater impact assessment has been carried out. The purpose of the assessment has been to determine, to the extent possible based on the accessible information, the following:

- Expected areas of interception, draw-down, and impoundment of groundwater;
- Expected areas of obstruction to groundwater recharge and discharge;
- Likelihood and significance of detrimental effects of interception, draw-down, impoundment and obstruction to recharge and discharge of groundwater;
- Likelihood and significance of release of contaminants to groundwater;

- Likelihood and significance of interference with private/municipal wells;
- Impacts of areas of high groundwater table on the project; and
- Temporary and permanent construction impacts.

The project involves the widening of the existing six lane divided highway to an interim eight lane and ultimate ten lane divided highway resulting in expanded paved surfaces. The specific geometry of the proposed widening has not yet been decided.

Subsurface works expected in conjunction with the widening are anticipated to consist of topsoil stripping, cutting and filling earthwork, culvert replacements, and ditch excavation. No significant excavations are anticipated. In general, it is expected that construction impacts are likely to stem from reduced recharge of groundwater under roadway pavement, quality impacts from road runoff, including automotive fluids and road salt, and construction impacts, either temporary or long-term, from grade changes.

Impact on Groundwater Recharge and Discharge

A reduction of groundwater recharge to the subsurface will occur as a result of the construction of relatively impermeable roadway pavements. The quantum of this reduction is expected to be variable throughout the project limits, but will be highest in areas of most significant recharge, namely within the Camp Borden sand plain area of the site between about Station 29+000, Township of Innisfil and Station 13+000, Township of Vespra.

In general, the relatively impermeable area covered by the pavement surface, in the context of the regional recharge in the vicinity of the project, is not considered to be significant. Recharge lost to impermeable surfaces will, in part, be mitigated by direction of runoff to ditches where some additional recharge above what is currently occurring can be anticipated.

Based on a relatively large area from which the deep overburden water wells and streams derive groundwater recharge, this reduction in overall groundwater recharge is not expected to be significant. The potential reduction in recharge is not anticipated to produce a measurable effect on groundwater recharge and discharge functions including base flow in streams and water well supply quantity.

The construction of enhanced infiltration structures along the road allowance could further mitigate the potential groundwater recharge reduction. In general, these structures function best in areas of permeable soils and deep water table conditions. Such

conditions typically occur in the Camp Borden sand plain area of the project within the City of Barrie area.

Water Well Interference

The most susceptible wells to either quantity or quality interference related to the proposed highway construction are the shallow overburden wells. As shown on Figure 12, these wells are distributed along much of the project area. In general, deep overburden and bedrock wells should not be susceptible to short or long-term quality or quantity interference effects related to the construction.

The nature of the construction expected, as outlined above, is such that no extensive excavation below the water table is expected outside of identified areas of high water table. As such, no significant lowering of the water table is anticipated and no corresponding reduction in available water supply in shallow overburden wells should be experienced. If deep excavations or permanent service installations below the water table were to be carried out, then some long-term lowering of the water table in the vicinity of these installations may result. This may result in a corresponding reduction in groundwater supplies in shallow wells, but only generally in those wells located within approximately 50 metres of the installation. If such excavations or installations are anticipated, the actual subsurface soil and groundwater conditions in those areas should be assessed in conjunction with the nearby wells to provide a prediction of potential impacts.

During construction, vibration of heavy equipment could potentially produce some mobilization of sediment within nearby wells. Equipment vibration has the potential to affect both shallow and deep, drilled or dug wells. Mobilization of sediment could also result in elevated bacteriological analyses on well water. This condition should be temporary and will subside after the cessation of the vibrations.

The application of road de-icing salt has the potential to affect neighbouring wells, in particular shallow wells (i.e. less than 15 metres deep) and is discussed in further detail below.

4.4.2 Potential For Groundwater Contamination

Shallow wells located near the proposed right-of-way may be susceptible to impact by road de-icing salt application. Chloride, which is highly mobile in the subsurface, is a major constituent of road salt. Chloride at high concentrations (greater than 250 milligrams per litre) may produce an impact on the taste of water. Sodium, which is the

other major constituent of road salt is less mobile in the subsurface, but is known to contribute to hypertension at elevated concentrations.

Because of the mobility of road salt constituents, mitigation of road salt impacts is difficult. To the extent possible, road salt application in areas of significant groundwater recharge within the right-of-way should be closely monitored and managed to prevent over application of road salt. Further, the presence of dissolved salt in runoff may lessen the desirability of enhanced groundwater recharge structures. The overall impact of road salt application can be mitigated by diverting, to the extent possible, surface runoff to surface water bodies in areas of groundwater discharge.

Mobile vehicle re-fuelling during construction presents a risk of impact to local wells as a result of accidental releases of fuel. In general, only catastrophic, large volume releases (i.e. greater than 100 litres) are likely to have an adverse impact on local water well supplies. The risk can all but be eliminated or managed by allowing re-fuelling only in designated areas preferably situated on a paved, impermeable surface and by having an emergency response plan in place to clean up any releases of fuel. Shallow wells in areas of relatively higher water table such as the northern and southern portions of the project area are potentially the most susceptible to fuel impacts. As well, the Camp Borden sand plain area of the project area forms a significant regional groundwater recharge area. These factors should be considered in the management of fuel impacts.

Impact of Areas of High Water Table

Areas of high water table, as noted above in Section 4.1 and as indicated on Figure 12, may affect construction progress and technique. These areas are, in general, expected to be limited to the northern and southern portions of the site, in particular, the low lying areas associated with the Cookstown Hollow swamp just south of Highway 89, and the Little Lake pond area. The significance of the areas as they may affect construction should be re-assessed once site specific borehole data for the detail design of the project have been obtained.

As discussed above, dewatering activities in areas of high water table, either active or as part of construction, or passive ongoing dewatering as a result of the construction of buried utilities have the potential to adversely impact nearby shallow water wells.

Temporary and Permanent Construction Impacts

As outlined in the foregoing sections, the proposed Highway 400 widening has the potential to produce some relatively minor impacts on existing groundwater resources.

These impacts are expected to be localized and should affect primarily only those wells/resources located in the immediate vicinity of the project works (i.e. within 100 metres of actual proposed construction works). Precautions to mitigate the potential impacts have been outlined in the foregoing sections of the report. All of these issues should be re-examined in light of actual subsurface borehole data and a detailed inventory of water wells obtained at the detail design stage of the project. In summary, the expected potential groundwater impacts from the proposed construction works are as follows:

- Some reduction in potential groundwater recharge within the project limits as a result of expanded relatively impermeable pavement surfaces is expected. Given the anticipated area to be paved relative to the regional recharge area, the reduction is not expected to be measurable;
- Some physical interference with shallow water well supplies in the immediate vicinity of the work should be expected to occur as a result of dewatering through excavation and disturbance of sediment from equipment vibration. These impacts are expected to be temporary, unless permanent buried utility excavations, which could have a dewatering effect, are proposed in areas of high water table; and
- Groundwater quality impacts should be anticipated from the proposed work resulting from road salt application and vehicle re-fuelling activities during construction.

4.5 Previous Geotechnical Investigations

The results of previous geotechnical investigations carried out by others on this section of Highway 400 were reviewed and the pertinent results are included in this report. The documents reviewed included:

- John Emery Geotechnical Engineering Ltd. Report No. 97178 entitled, "Geotechnical Design Report, GWP 25-96-00, Highway 400, From 1.0 km south of Molson Park Drive to 1.0 km north of Molson Park Drive," dated June 5, 1998;
- MTO Geotechnical Design Report entitled, "Highway 400, WP 198-89-01, Resurfacing, Interchange and Drainage Improvements, From 1.4 km north of Highway 89 northerly to 0.7 km south of Highway 27 (Essa Road), Barrie," dated May 25, 1994;
- MTO Memorandum entitled, "WP 198-89-00, Highway 400, From Highway 89 to Essa Road, District 5, Owen Sound," dated December 6, 1994; and
- MTO Memorandum entitled, "WP199-89-00, Highway 400, Hot Mix Paving, From Essa Road-Highway 27 (Barrie) to Highway 400 Extension (Willow Creek Bridge)," dated September 6, 1996.

The results of the coring and boreholes drilled as part of these investigations are summarized on Figures 5, 6, 7 and 8 for the northbound and southbound lanes, respectively. Also included on the figures are the design pavement component thicknesses based on the available information.

For all of the above noted investigations, where subgrade materials were encountered in the boreholes, these subgrade materials consisted primarily of fine sands, silty sands and sandy silts.

5.0 AGGREGATE MATERIALS

The project crosses three physiographic regions, as shown in the Physiography of Ontario by L.J. Chapman and D.F. Putnam. The southern part of the project is in the Peterborough Drumlin Field region. The surficial deposits in this area are generally clayey till and therefore, are generally void of aggregate material. There are a few drumlins that consist of stoney till material which could be used as earth borrow or Select Subgrade Material (SSM).

The central part of the project crosses the Simcoe Lowland region at the centre of the City of Barrie. In this area, the surficial deposits of this region consist mainly of poorly drained fine sands, except at the old shoreline Glacial Lake Algonquin, which has deposits of sand and some gravel. Most of these deposits are either depleted or have been built over. However, there are some remnants of sand deposit about 10 kilometres west of Highway 400 along Highway 90.

The northerly part of the project is in the Simcoe Uplands region where the surficial deposit are clayey till.

In the northern and western parts of Oro Township lies the Oro Moraine, also called the "sandhills". This formation contains significant good quality sand and gravel deposits which are excellent sources of aggregate materials.

Granular B Type I and SSM may be located in the following licensed pits:

- Lafarge Canada Inc., 1.5 kilometres northeast of Grenfell;
- Midhurst Sand and Gravel, 2.5 kilometres southwest of Midhurst;
- Cliff Varcoe Ltd., 5.5 kilometres southeast of Craighurst;
- Beamish Construction Co. Ltd., 2.5 kilometres east of Dalston;
- D. Ross, 4.0 kilometres northwest of Cookstown; and
- Georgian Aggregates and Construction, 5.5 kilometres northwest of Alliston.

The approximate haul distances to the centre of the project are 15, 16, 22, 25, 25, and 36 kilometres, respectively. Granular B Type I material may also be available in three unlicensed sources, the first under an aggregate and the other two under a wayside permit:

- MTO (Holy) pit, 5.0 kilometres north of Thornton;
- Hydro pit, 7.0 kilometres west of the junction of Highways 400 and 90; and
- G. Fullerton property, 5.0 kilometres northwest of Cookstown.

The approximate haul distances to the centre of the project are 10, 12 and 25 kilometres, respectively.

Granular A aggregates may be available from the following licensed pits:

- Cliff Varcoe Ltd., 5.5 kilometres southeast of Craighurst;
- Beamish Construction Co. Ltd., 2.5 kilometres east of Dalston;
- Seeley and Arnill Aggregates Ltd., 4.0 kilometres northeast of Edgar;
- The Sarjeant Co. Ltd., 5.0 kilometres northeast of Edgar;
- Lafarge Canada Inc., 5.0 kilometres northeast of Edgar; and
- Georgian Aggregates and Construction, 5.5 kilometres northwest of Alliston.

The approximate haul distances to the centre of the project are 22, 25, 32, 35, 36 and 36 kilometres, respectively. The first five sources would also be acceptable for HL 3, HL 4 and HL 8 coarse and fine aggregates. Granular A may also be obtained from the Nelson Aggregates Uhthoff quarry at New Uhthoff at an approximate haul distance to the centre of the project of 52 kilometres.

For HDBC coarse and fine aggregates there are the following possible alternatives from licensed quarries:

- Blue Circle Aggregates Canada, 2.5 kilometres east of Acton;
- Dufferin Aggregates, 6.4 kilometres northwest of Milton;
- Nelson Aggregates, 4.0 kilometres, southeast of Cedar Springs; and
- Lafarge Canada Inc., on Highway 5, north of Dundas.

The approximate haul distances of the quarries to the centre of the project are 115, 125, 135 and 150 kilometres, respectively.

Sources for DFC coarse and fine aggregates are shown in the Designated Sources Manual in the DSM #3.05.25 list, maintained by the Ontario Good Roads Association. The closest DFC sources to the project are the Fowler Construction C. Ltd. meta-arkose quarry, 4.0 kilometres south of Washago and the Lafarge Canada Inc. gneiss quarry, 5.0 kilometres east of Waubaushene, with approximate haul distance to the centre of the project of 65 kilometres for both sources. The next closest source is the Fowler Construction Co. Ltd. Rosewarne quarry, 3.0 kilometres northeast of Bracebridge at the approximate haul distance to the centre of the project of 95 kilometres.

It is anticipated that the sources noted above could also supply aggregate for the SuperPave mixes subject to the materials meeting the additional test requirements.

Hot mix asphalt for pavements may be provided by portable or stationary plants.

The closest sources for structural concrete coarse and fine aggregates are:

- Cliff Varcoe Ltd., 5.5 kilometres southeast of Craighurst,
- Sarjeant Ready Mix, 1.5 kilometres southwest of Phelpston, and
- CBM, Orillia Aggregates, 4.0 kilometres southwest of Orillia.

The approximate haul distances to the centre of the project are 22, 32 and 36 kilometres, respectively. It is most likely that structural concrete will be supplied by ready mix plants.

6.0 ALIGNMENT AND GRADELINE

No overall alignment changes are proposed. Grade changes are expected to be minimal. This portion of Highway 400 is to be widened to an interim eight lane cross section with an ultimate ten lane cross-section. The proposed widening is to be constructed to accommodate a median tall wall barrier. The proposed cross-section is to have 3.75 metre lane widths, 3.0 metre paved median shoulders, 3.0 metre paved outer shoulders and 2.5 metre outside gravel shoulders. It is anticipated that embankment fills will consist of native borrow material obtained from adjacent cuts. The anticipated subgrade soils have been based on the available borehole data, surficial soils and quaternary geology mapping and are expected to be predominantly sandy silt tills and sands.

Various widening alternatives were considered for each of the three sections of the project and are discussed below. Note that the discussions pertain to an ultimate ten lane section.

6.1 South Limit to Essa Road

This section currently has a 3 metre wide median. The three widening alternatives being considered are discussed below.

The centred 8.8 metre median with tall wall alternative will require the construction of three additional outer lanes on each side and associated outer shoulders. This would render the existing median lanes redundant and utilize the two existing outside lanes on each side. This would utilize the existing highway platform and increase the width of the total highway footprint width by about 22.5 metres.

Provide an 8.8 metre median and holding the east edge of shoulder, this alternative will utilize the existing southbound outside and centre lanes. All three of the existing northbound lanes would be utilized. This alternative will require the construction of five southbound lanes and two additional northbound lanes. All of the existing highway platform is salvaged and the total width of the highway footprint is widened by about 18.8 metres.

The 22 metre open median holding the east edge of shoulder would abandon all of the existing northbound lanes and utilize only the existing northbound lanes. This alternative will require the construction of five new southbound lanes and two new outside northbound lanes. Only half of the existing highway platform is salvaged and the total width of the highway footprint increases by about 30.0 metres.

The 8.8 metre median holding the east edge of pavement is the preferred alternative in this section.

6.2 Essa Road to Bayfield Street

This section has an existing 4.3 metre median. The centered 8.8 metre median with tall wall alternative will require the construction of three additional lanes on each side and associated outer shoulders. This alternative would abandon the median lanes on both sides and utilize the existing outside and centre lanes on each side. The existing highway platform is salvaged and the total width of the highway footprint is increased by about 23 metres.

The 8.8 metre median alternatives holding the east edge of shoulder will utilize the outside and centre southbound lanes along with all three of the northbound lanes. This alternative will require construction of three southbound lanes and two northbound lanes. All of the existing highway platform is salvaged and the total width of the highway footprint would be increased by about 19 metres.

The 22 metre open median holding the east edge of shoulder will preclude the use of any of the existing northbound lanes and utilize all three of the existing southbound lanes. The construction of five new southbound lanes and two new northbound lanes will be required to achieve the ten lane cross-section. Only half of the existing highway platform is salvaged and the total width of the highway footprint would be increased by about 26 metres.

Based on the above, the 8.8 metre median holding the east edge of pavement is the preferred alternative for this section.

6.3 Bayfield Street to Junction of Highway 11

This section has an existing 9.1 metre median. The centered 8.8 metre median with tall wall alternative will utilize all the existing northbound and southbound lanes and require the construction of two new outside lanes on each side and associated shoulders. The existing highway platform is salvaged and the total width of the highway footprint would be increased by about 15 metres.

The 8.8 metre median holding the east edge of shoulder will utilize all the existing northbound and southbound lanes and require the construction of two new outside lanes on each side and associated shoulders. The existing highway platform is salvaged and the total width of the highway footprint would be increased by about 15 metres.

The 22 metre median holding the east edge of shoulder will preclude the use of any of the existing southbound lanes and utilize all three of the northbound lanes. The construction of five new southbound lanes and two new outside northbound lanes will be required with this alternative. Only half of the existing highway platform is salvaged and the total width of the highway footprint would be increased about 26 metres.

Based on the above, either 8.8 metre median alternative is preferable since they are essentially equivalent.

6.4 Preferred Alternatives

Currently, the preferred widening alternatives are:

i) from project south limit to Essa Road - widen to eight lanes with a ten lane platform to the west holding the east edge of pavement;
ii) from Essa Road to Bayfield Street - widen to ten lanes about the centreline;
iii) from Bayfield Street to project north limit - widen to eight lanes about the centreline.

These alternatives are acceptable based on geotechnical considerations.

7.0 ALTERNATIVE PRELIMINARY PAVEMENT DESIGNS CONSIDERED

7.1 General

This portion of Highway 400 is to be widened to an interim eight lane cross section with an ultimate ten lane cross-section. The proposed widening is to be constructed to accommodate a median tall wall barrier. The proposed cross-section is to have 3.75 metre lane widths, 3.0 metre paved median shoulders, 3.0 metre paved outer shoulders and 2.5 metre outside gravel shoulders. It is anticipated that embankment fills will consist of native borrow material obtained from adjacent cuts. The anticipated subgrade soils have been based on the available borehole data, surficial soils and quaternary geology mapping and are expected to be predominantly sandy silt tills and sands.

The pavement sections have been analysed using the American Association of State Highway and Transportation Officials (AASHTO, 1993) design method and the Asphalt Institute (1991) design method together with our knowledge of previous MTO experiences and standards and employing good engineering judgement. A discussion of the various pavement design methods used for this project is provided in Section 7.2. Detail Pavement Design calculations are provided in Appendix B.

The MTO has indicated that this project is to proceed to detail design and construction using SuperPave mixes. The following sections of the report provide pavement designs which were prepared using the traditional hot mix components and which were subsequently converted to SuperPave mixes.

The following are the MTO's traditional mixes and the corresponding SuperPave mixes together with the lift thicknesses specified by MTO:

CONVENTIONAL MIX	SUPERPAVE MIX	LIFT THICKNESS (mm)
DFC	SuperPave 12.5 FC 2	38 to 50
HDBC	SuperPave 19.0	50 to 75
HL 4 Surface	SuperPave 12.5	38 to 50
HL 4 Binder	SuperPave 19.0	50 to 75
HL 1	SuperPave 12.5 FC 1	38 to 50
HL 3	SuperPave 12.5	38 to 50
Heavy Duty Padding	SuperPave 9.5	nil to 30

7.2 Discussion of Pavement Design Methods

7.2.1 AASHTO (1993) Design Method

Flexible Pavements

The AASHTO design method is a statistically based method calibrated using the results from the AASHTO Road Test carried out in the late 1950's and early 1960's. Thus, there are some limitations to the application of this design method. The limitations are discussed in Section 1.4.2 of the AASHTO Guide and are briefly summarized below:

a) limited flexible pavement types (asphaltic concrete, granular base and subbase structure only);
b) traffic loadings (road test terminated at 1,114,000 axle applications, equivalent to about eight million ESALs and design in excess of this value requires extrapolation beyond the equations developed);
c) age (aging effects are ignored as the road test was terminated after two years); and
d) environment (road test carried out in Ottawa, Illinois, U.S.A.).

The design method has been updated several times since inception, and recently the MTO undertook a study to adapt the AASHTO method to Ontario conditions.

The results of the AASHTO design method are sensitive to the input parameters, particularly subgrade resilient modulus and the initial and terminal serviceability factors. In addition, the scaling of layer coefficients is not consistent with the Granular Base Equivalency (GBE) concept. Designing for a specific GBE or structural number (SN) will cause discrepancies with other GBE based methods.

The benefits of employing the AASHTO method, particularly when properly calibrated to Ontario conditions, are that advanced conditions such as reliability, drainage, frost susceptible subgrades and swelling may be incorporated directly.

Since the AASHTO design method has not, until very recently, been scrutinized under Ontario conditions, the computed pavement structure must be verified using experience, judgement and comparison with other accepted pavement design methods.

7.2.2 Asphalt Institute (AI) Design Method

The Asphalt Institute (AI) design method is based on a theoretical multilayered elastic system. However, the results of analyses using the AI method tend to provide asphaltic thicknesses well in excess of those typically employed in current practice. The main

reason is the perceived notion that base and/or subbase courses greater than 300 millimetres in thickness do not provide additional structural capacity to the pavement structure as stated in the following excerpt from the AI design guide.

"Some agencies employ greater thicknesses of untreated aggregate base to try to minimize the potential for frost damage in the pavement. Although thicknesses greater than 300 millimetres are used for this purpose, pavement analysis studies have shown that this additional thickness adds little benefit if any to the structural behavior of the pavement system....Reducing the asphalt concrete thickness because of very large thicknesses of untreated aggregate is discouraged by the Asphalt Institute and most pavement designers because the potential for other pavement problems will increase."

The results of the AI design method will be scaled using typical GBE factors to develop a more typical pavement section based on past MTO experience and current design philosophies.

7.3 Preliminary Pavement Design

For analysis purposes, the following equivalency factors and AASHTO layer coefficients were used (based on the MTO Pavement Design and Rehabilitation Manual, the AASHTO Design Guide and the Adaptation of AASHTO for Ontario Conditions Guide) based on a ten lane cross-section:

MATERIAL	EQUIVALENCY FACTOR	AASHTO LAYER COEFFICIENT	AASHTO DRAINAGE COEFFICIENT
HL Asphalt	2.0	0.42	-
Granular A Base	1.0	0.14	1.0
Granular B Type I Subbase	0.67	0.09	1.0

As shown in Table V eighteen year traffic volumes, assuming completion of construction in 2006, of 43 to 65 million equivalent single axle loads (ESAL's) were calculated for the various sections of highway based on the traffic data summarized in Section 2.1, 12 per cent commercial traffic and a truck factor of 2.0. This results in a required SN of about 173 to 192 millimetres and a GBE of 1015 to 1150 millimetres. The Asphalt Institute design method yields SNs of 210 to 225 millimetres and GBEs of 1120 to 1190 millimetres.

For preliminary design, the pavement structure required to support the expected traffic loading is:

COMPONENT	THICKNESS (mm)
Dense Friction Course (SP 12.5 FC2)	40
Heavy Duty Binder Course (SP 19.0)	100 (2x50)
HL 4 Binder Course (SP19.0)	100 (2x50)
Granular A Base	200
Granular B Type I Subbase	700
Granular Base Equivalent	1150
Structural Number (SN)	192

The existing travelled lanes are to be salvaged and utilized, to the extent possible, as a significant portion of the proposed cross-section. The existing travelled lanes, as indicated in previous sections, are conventional flexible pavement structures and it is considered that a flexible pavement structure should be incorporated for the proposed widenings. The reasons for selecting a flexible structure rather than a rigid structure for the proposed widenings are summarized below:

a) Differential performance - A significant potential for differential performance exists when two very different engineering materials are utilized in conjunction with one another.
b) Lateral drainage - Some 1.1 metres of total structure thickness is required in order to provide continuous lateral drainage of the existing pavement structure. It is considered that a rigid structure would provide only about 0.6 metres of total thickness and an additional 0.5 metres of granular materials would be required for drainage purposes only. This would result in an artificially inflated initial cost for a concrete alternative in the life cycle costing.
c) Aesthetics - For aesthetic purposes, a two lift hot mix overlay would probably be placed on the completed concrete pavement to provide a visually unified cross-section and to accommodate potential crossfall correction and crown shifts.

Based on the above, rigid and composite pavement structures were not considered to be viable alternatives for the proposed widenings. However, if it is determined during design that the existing flexible pavements cannot be utilized and reconstruction of the pavements is considered necessary, then Alternative Bid contracts will be required.

8.0 PRELIMINARY RECOMMENDATIONS AND CONSTRUCTION FEATURES

8.1 Recommended Main Lane Widening Pavement Structure - Preliminary Design

Based on the information provided, coupled with the field observations and considering the results of the analyses, the following pavement structures are required to support the anticipated main lane traffic and to provide structure continuity and lateral drainage:

COMPONENT	THICKNESS (mm)
Dense Friction Course (SP 12.5 FC2)	40
Heavy Duty Binder Course (SP 19.0)	100 (2x50)
HL 4 Binder Course (SP 19.0)	100 (2x50)
Granular A Base	200
Granular B Type I Subbase	700

The fully paved shoulders should consist of:

COMPONENT	THICKNESS (mm)
Dense Friction Course (SP 12.5 FC2)	40
Heavy Duty Binder Course (SP 19.0)	50
Granular A Base	350
Granular B Type I Subbase	700

In any areas where the new shoulders are to support through traffic for staging purposes, a minimum of 200 millimetres asphaltic concrete should be provided over 300 millimetres Granular A over Granular B to proposed subgrade level.

Pavement design calculations are provided in Appendix B.

The existing topsoil, any deleterious materials and the existing shoulders should be excavated, as required, to provide the appropriate pavement structure. Approved earth borrow may be used for the embankments. Any existing subdrains adjacent to the existing main lanes should be removed and replaced with approved granular or earth borrow. The area should be proofrolled in conjunction with an inspection by the geotechnical engineer and any areas exhibiting distress or containing deleterious materials should be further excavated. The roadway should then be brought to subgrade using approved earth borrow or Granular B, Type I. All granular materials should be placed full width and should be daylighted to the ditches or subdrains provided.

Since construction is not anticipated in the near future, these preliminary recommendations should be revisited during detail design.

8.2 Preliminary Existing Main Lane Rehabilitation

Based on the results of the pavement condition survey and assuming the timing of construction beyond 2008, it is recommended that the rehabilitation of the existing main lanes consist of milling the existing surface 50 millimetres throughout in conjunction with a two lift overlay consisting of 50 millimetres HDBC and 40 millimetres DFC. In addition, padding may be required to address crown shifts and crossfall corrections. Due to the extensive distribution of transverse cracks within the project limits, and considering the potential for these cracks to further deteriorate prior to construction, it is considered that full depth crack repairs will be required in conjunction with the pavement rehabilitation.

8.3 Preliminary Ramp Rehabilitation and Reconstruction

Based on the results of the pavement condition survey and the anticipated timing of construction it is recommended that the rehabilitation of the existing interchange ramps consist of milling the existing surface throughout in conjunction with a two lift overlay consisting of 50 millimetres HDBC and 40 millimetres DFC. In addition, padding may be required to address crown shifts and crossfall corrections. Further, the need to carry out transverse crack repairs should be addressed.

Where new construction is required, based on current poor performance or realignment, and based on the traffic data provided, as many as 15 to 30 million ESALs may be expected on the various interchange ramps for an eighteen year period.

For the preliminary design, the new ramp pavement structures should consist of:

	THICKNESS (mm)		
COMPONENT	Highway 89 Interchange	Essa Road Interchange	Other
DFC (SP 12.5 FC2)	40	40	40
HDBC (SP 19.0)	100 (2x50)	120 (2x60)	100 (2x50)
HL 4 (SP 19.0)	50	50	50
Granular A Base	200	200	200
Granular B Type I Subbase	450	550	550

Since construction is not anticipated in the near future, these preliminary recommendations should be revisited during detail design.

9.0 CUT MATERIALS AND DESIGN

In conjunction with the proposed pavement widenings, cuts greater than 4.5 metres are anticipated in the following locations:

NORTHBOUND LANE	SOUTHBOUND LANE
Township of Innisfil	
Station 13+300 to 13+900	Station 12+450 to 12+040
Station 18+660 to 18+900	Station 13+375 to 13+900
Station 22+450 to 22+600	Station 18+660 to 18+825
Station 23+800 to 24+080	Station 22+425 to 22+600
Station 27+500 to 27+950	Station 23+780 to 24+100
	Station 27+500 to 27+975
Township of Vespra	
Station 12+600 to 12+725	Station 12+430 to 12+530
Station 13+250 to 13+450	Station 12+570 to 12+750
Station 17+000 to 17+400	Station 12+970 to 13+200

Cuts which are greater than 4.5 metres will require detailed investigation for stability purposes during detail design.

For preliminary design, any new cut slopes should not exceed an inclination of 2 horizontal to 1 vertical. In areas of sandy soils, erosion control blankets should be provided to address surficial erosion problems. All cut slopes should be provided with a pre-seeded erosion protection blanket. Any areas of cut slopes which are wet due to high groundwater levels or seepage should be cut as flat as possible and provided with a blanket of non-woven geotextile and 500 millimetres of rip-rap. Should localized areas of groundwater flows be encountered during construction in the cut slopes, granular blankets, French drains, edge drainage systems or the like may be required.

Concern has been expressed that cutback slopes in drumlin features may be subject to instability as a result of seasonally high groundwater levels. While visual inspections of the existing slopes did not indicate any such distress, the original construction may have included the installation of subsurface drainage works and the potential for water bearing layers and/or zones within the drumlins should be investigated and mitigative works developed during detail design.

To the extent possible, a drainage swale should be provided behind the crest of all slopes. These swales should be outletted down the slope to the ditch at regular intervals by means of rock lined channels. Where the drainage swale is precluded by property boundaries, rock lined channels should be provided on the slope at regular intervals and at any areas exhibiting runoff features.

The anticipated ditch inverts should be reviewed to ensure that the final ditch inverts are a minimum of 0.5 metres below subgrade elevation or that subdrains are provided.

9.1 Subexcavation of Frost Susceptible Soils

Based on the background information provided, subexcavation of frost susceptible materials should be anticipated in the following locations:

NORTHBOUND LANES	SOUTHBOUND LANES
Township of Innisfil	
Station 14+110 to 14+140	Station 24+070 to 24+140
Station 22+430 to 22+500	Station 27+010 to 27+030
Station 25+125 to 25+145	

Frost susceptible material encountered within the frost penetration zone of 1.4 metres below profile grade is to be removed and replaced with suitable native material. Subexcavated frost susceptible material can be used as fill below the frost line. If granular backfill is placed on any subexcavated areas, subdrains should be provided at the lower subgrade level to provide drainage.

9.2 Swamp Excavation

The boreholes drilled adjacent to the Willow Creek bridge structure, included in the Contract 71-151 drawings, indicated the presence of up to 3 metres of organics at the borehole locations. No additional information was available to determine the extent of this organic zone. Based on 1:10,000 scale topographic mapping, swamp/marsh conditions are present in the Little Lake area for a length of about 0.9 kilometres or from Station 17+500 to the north project limit. Highway widening in the vicinity of Willow Creek, and most likely Little Lake, may require swamp excavation. The areas where swamp excavation will be required can be better assessed during the investigation for detail design.

Following subexcavation of the organics in the area of the new pavements for the proposed widening, the new highway platform may be constructed. Shallow groundwater conditions should be anticipated. Therefore, in areas of standing water and/or wet conditions imported material consistent with Ontario Provincial Standard Specifications (OPSS) Granular B, Type II gradation should be placed in a single lift a minimum of 0.5 metres, or to at least 300 millimetres above the water level.

Subsequently, the remainder of the new platform may be constructed using approved earth borrow as described in Section 10.0.

10.0 EMBANKMENT MATERIALS AND DESIGN

Embankment fills greater than 4.5 metres are anticipated in the following locations for the widenings:

Northbound Lane	Southbound Lane
Township of Innisfil	
Station 14+580 to 15+200	Station 14+500 to 15+125
Station 26+050 to 26+120	Station 26+050 to 26+140
Station 29+200 to 29+700	Station 26+550 to 26+830
	Station 29+300 to 29+750
Township of Vespra	
Station 12+750 to 12+875	Station 12+800 to 12+960

Embankments which are greater than 4.5 metres will require detailed investigation for stability purposes during final design.

In areas where widening of the roadway platform is required, the widening may be constructed by stripping all topsoil and otherwise deleterious material from within the area of the widening to expose competent materials. The exposed subgrade should be carefully proofrolled and any softened areas subexcavated. The widening may then be constructed by placing suitable granular or earth borrow in 300 millimetre lifts working upwards, benching into the existing slope at 300 millimetre intervals of height consistent with Ontario Provincial Standard Drawing (OPSD) 208.010. The bench width will be dependent on the existing slope inclination. Upon completion, the fill may be trimmed to the desired width and inclination. For preliminary purposes, slopes may be designed with a 2 horizontal to 1 vertical inclination.

The materials placed for the widening fills should be provided with 100 millimetres of topsoil and properly seeded consistent with Ontario Provincial Standard Specifications (OPSS) 570. The anticipated ditch inverts should be reviewed to ensure that the final ditch inverts are a minimum of 0.5 metres below subgrade elevation.

11.0 DRAINAGE

11.1 Ditching and Subdrains

It is recommended that the ditch inverts throughout the project limits be reviewed to ensure that the ditch invert is a minimum of 0.5 metres below the subgrade elevation. Subdrains should be provided in the areas where this increased ditch depth cannot be provided or curb and gutter is required. Pipe subdrains should be perforated, 150 millimetres in diameter and placed in a geotextile-lined trench 300 millimetres wide by 300 millimetres deep. The trench should be backfilled with 19 millimetre clear stone.

Although no specific problem areas were noted during the site inspections, any ditch areas which are historically wet from conditions other than poor grading, i.e. cat tails, etc., should be provided with an approved geotextile and appropriately sized rip-rap consisting of angular stone or recycled concrete conforming to OPSS 1004.

11.2 Geotextile

For preliminary design, geotextile used for pipe subdrains should be Class I, non-woven with a FOS of 40 to 80 microns. Geotextile used under rip-rap should be Class II, non-woven with a FOS of 40 to 80 microns.

REFERENCES

i) "Pavement Design and Rehabilitation Manual", Ministry of Transportation, Ontario. AASHTO Guide for Design of Pavement Structures, 1993.

ii) "The Physiography of Southern Ontario", 3rd Edition, Chapman & Putnam, 1984.

iii) "Highway 400 Widening, Major Mackenzie Drive to Holland River, Hydrogeology Component, Section B.1.10", draft report by Goffco Limited, dated October 2000.

iv) Soils Design Report, WP's 105-70-03 and –14, Highway 400, 1.6 Mi South of Aurora Sideroad to Highway 88.

v) Soils Design Report, WP 105-70-02, Highway 400, 0.7 Mi North of Maple Sideroad to 1.6 Mi South of Aurora Sideroad.

vi) Pavement Performance Records and Flexible Pavement Condition Evaluation Forms.

vii) Soils Design Report, WP 105-70-03, Highway 400, 1.8 Mi South of Aurora Sideroad to 0.4 Mi South of Highway 9.

viii) Soils Design Report, WP 105-70-14, Highway 400, 0.4 Mi South of Highway 9 to 0.2 Mi North of Highway 88.

ix) Environmental Scoping Report, WP 222-97-00, Highway 400.

x) 1:5,000 Aerial Photographs, Highway 400, From Maple Leaf Drive to the Holland River, 1988.

xi) 1:50,000 Topographic Mapping, Bolton, Ontario.

xii) 1in to 1mi Soil Map of York County (Regional Municipality of York), Ontario.

xiii) Soil Survey of York County, Report No. 19 of the Ontario Soil Survey.

xiv) 1:50,000 Pleistocene Geology of the Bolton Area, Ontario Department of Mines, Preliminary Geological Map No. P.477.

xv) 1:50,000 Quaternary Geology of the Alliston Area, Ontario Division of Mines, Preliminary Map P.835, Geological Series.

REFERENCES CONTINUED

xvi) 1:10,000 Ontario Base Mapping.

xvii) Various Ministry of Transportation, Ontario memoranda.

xviii) Regional Geotechnical Pavement Condition Reports.

xix) John Emery Geotechnical Engineering Ltd. Report No. 97178 entitled "Geotechnical Design Report, GWP 25-96-00, Highway 400, From 1.0 KM South of Molson Park Drive to 1.0 KM North of Molson Park Drive," dated June 5, 1998.

xx) MTO Geotechnical Design Report entitled "Highway 400, WP 198-89-01, Resurfacing, Interchange and Drainage Improvements, From 1.4 KM North of Highway 89 Northerly to 0.7 KM South of Highway 27 (Essa Road) Barrie," dated May 25, 1994.

TABLE I

DETAILS OF CONTRACT 96-11 REHABILITATION

Highway 400 Planning Study
GWP 30-95-00

LOCATION	40 mm DENSE FRICTION SURFACE COURSE	50 mm DENSE FRICTION SURFACE COURSE	50 mm HEAVY DUTY BINDER COURSE	60 mm HEAVY DUTY BINDER COURSE	ASPHALT PAVEMENT REMOVAL				PADDING (tonnes)
					PARTIAL DEPTH 40 mm	PARTIAL DEPTH 50 mm	PARTIAL DEPTH 100 mm	REMOVE FULL DEPTH	
HIGHWAY 400 NORTHBOUND									
MEDIAN & CENTRE LANES									
(extend treatment 40 mm into driving lane)									
11+400 - 14+110		X				X			
14+110 - 14+140	X			X			X		
14+140 - 22+530		X				X			
22+530 - 22+650	X			X			X		
22+650 - 27+600		X				X			
DRIVING LANE									
13+650 - 13+770		X				X			
14+110 - 14+140	X			X			X		
16+880 - 16+950		X				X			
19+592 - 19+710		X				X			
20+140 - 20+270		X				X			
20+680 - 20+800		X				X			
MEDIAN SHOULDER (1m width)									
11+400 - 16+880		X				X			
16+950 - 27+600		X				X			
SPEED CHANGE LANES									
18+927 - 19+287		X				X			
19+564 - 19+834		X				X			
19+734 - 19+834 see note 1		X	X	X					
19+868 - 20+190		X				X			
24+391 - 24+926 see note 1	X		2 LIFTS					X	
24+607 - 24+926	X				X				
25+648 - 26+102 (Mapleview Dr.)		X				X			
26+348 - 26+704		X				X			
27+373 - 27+600		X				X			
RAMPS (INNISFIL BEACH ROAD)									
N-W, S-E, W-S, N-E, W-N, E-N		X					X		
DISTRESSED AREAS		X	X					X	
S-W, E-S									

TABLE II

CRACK REPAIR DETAILS - CONTRACT 96-11

Highway 400 Planning Study
GWP 30-95-00

LONGITUDINAL PAVING JOINT CRACK
Located between either median and centre lane or between centre lane and driving lane.

LOCATION			LENGTH (metres)
14+180	-	14+290	NBL 110
17+450	-	17+500	SBL 50
18+950	-	19+020	SBL 70
22+960	-	23+000	and
23+070	-	23+120	SBL 90
24+000	-	26+000	SBL 2700
22+840	-	26+040	NBL 3200
26+340	-	26+380	SBL 40
26+410	-	26+450	SBL 40
26+500	-	26+560	SBL 60
26+620	-	26+670	SBL 50
27+120	-	26+920	SBL 200
27+600	-	27+420	SBL 70
TOTAL			6680

TRANSVERSE CRACK LOCATIONS
ACROSS THE CENTRE & MEDIAN LANES ONLY

LOCATION			NUMBER OF CRACKS REQUIRING TREATMENT
13+350	-	13+650	NBL 4
14+200	-	15+500	SBL 11
14+472			NBL 1
14+920			NBL 1
15+022			NBL 1
15+590			NBL 1
16+100	-	16+650	SBL 11
16+950	-	18+220	SBL 18
18+220	-	18+500	SBL 3
19+800	-	20+100	SBL 11
20+300	-	21+650	SBL 7
20+310			NBL 1
20+610			NBL 1
21+050	-	22+010	SBL 40
21+330	-	22+400	NBL 10
22+010	-	22+380	SBL 20
22+380	-	23+380	SBL 45
22+650	-	22+950	NBL 3
23+380	-	23+761	SBL 30
23+671	-	24+550	SBL 36
24+550	-	25+000	SBL 22
25+000	-	25+700	SBL 23
25+700	-	26+300	SBL 20
25+555			NBL 1
25+700	-	26+000	NBL 4
26+000	-	26+300	NBL 4
26+300	-	26+682	SBL 19
26+300	-	27+000	NBL 12
26+682	-	27+070	SBL 20
27+070	-	27+600	SBL 20
27+000	-	27+600	NBL 8
TOTAL			408

TABLE III

DETAILS OF CONTRACT 2001-3004 REHABILITATION
MAIN LANES

Highway 400 Planning Study
GWP 30-95-00

LOCATION	MEDIAN LANE + 0.5 SHLD	CENTRE LANE	DRIVING LANE	SPEED CHANGE LANE	OUTSIDE SHOULDER	40 mm DENSE FRICTION COURSE	50 mm DENSE FRICTION COURSE	50 mm UPPER HEAVY DUTY BINDER COURSE	ASPHALT PAVEMENT REMOVAL				PADDING 50 mm HDBC
									PARTIAL DEPTH 40 mm	PARTIAL DEPTH 50 mm	PARTIAL DEPTH 90 mm	PARTIAL DEPTH 100 mm	
N.B.L													
27+600 - 28+125	X	X	X	X	X		X			X			
28+125 - 28+477	X		X	X	X		X			X			
28+125 - 28+477		X					X	X				X	
28+477 - 28+500	X	X	X	X	X	X			X				
28+500 - 29+728	X		X	X	X		X			X			
28+500 - 29+728		X					X	X					
29+728 - 10+006	X	X	X	X	X	X				X		X	
10+006 - 11+802	X		X	X	X		X			X			
10+006 - 11+802		X					X	X				X	
11+802 - 11+825	X	X	X	X	X	X			X				
11+825 - 12+500	X		X	X	X		X			X			
11+825 - 12+500		X					X	X				X	
12+500 - 12+650	X	X	X	X	X		X			X			
12+650 - 13+800	X		X	X	X		X			X			
12+650 - 13+800		X					X	X				X	
13+800 - 17+075	X	X	X	X	X		X			X			
S.B.L													
27+600 - 28+457	X		X	X	X		X			X			
27+600 - 28+457		X					X	X				X	
28+197 - 28+203					X								X
28+457 - 28+483	X	X	X	X	X	X			X				
28+483 - 29+240	X		X	X	X		X			X			
28+483 - 29+240		X					X	X				X	
29+240 - 29+739	X	X	X	X	X		X			X			
10+000 - 11+100	X	X	X	X	X		X			X			
11+100 - 11+812	X		X	X	X		X			X			
11+100 - 11+812		X					X	X				X	
11+812 - 11+836	X	X	X	X	X		X			X			
11+836 - 12+644	X		X	X	X		X			X			
11+836 - 12+644		X					X	X				X	
12+644 - 13+520	X	X	X	X	X		X			X			
13+520 - 13+900	X		X	X	X		X			X			
13+530 - 13+900		X					X	X				X	
13+900 - 14+225	X	X	X	X	X		X			X			
14+225 - 14+475	X		X	X	X		X			X			
14+225 - 14+475		X					X	X				X	
14+475 - 16+656	X	X	X	X	X		X			X			
16+565 - 17+150	X		X	X	X		X			X			
16+565 - 17+150		X					X	X				X	
17+150 - 18+200	X	X	X	X	X		X			X			

DETAILS OF CONTRACT 2001-3004 REHABILITATION
MAIN LANES

LOCATION	MEDIAN LANE + 0.5 SHLD	CENTRE LANE	DRIVING LANE	SPEED CHANGE LANE	OUTSIDE SHOULDER	40 mm DENSE FRICTION COURSE	50 mm DENSE FRICTION COURSE	50 mm UPPER HEAVY DUTY BINDER COURSE	ASPHALT PAVEMENT REMOVAL				PADDING 50 mm HDBC
									PARTIAL DEPTH 40 mm	PARTIAL DEPTH 50 mm	PARTIAL DEPTH 90 mm	PARTIAL DEPTH 100 mm	
SERVICE CENTRE RAMP													
10+000 - 10+319						X		X			X		
10+329 - 10+445							X			X			
10+445 - 10+675						X		X			X		

NOTE: Table to be read in conjunction with accompanying report.

TABLE IV

DETAILS OF CONTRACT 2001-3004 REHABILITATION INTERCHANGE RAMPS

Highway 400 Planning Study
GWP 30-95-00

LOCATION	MEDIAN LANE + 0.5 SHLD	CENTRE LANE	DRIVING LANE	SPEED CHANGE LANE	OUTSIDE SHOULDER	40 mm DENSE FRICTION COURSE	50 mm HL 4 SURFACE COURSE	50 mm HL 4 UPPER BINDER COURSE	50 mm HL 4 LOWER BINDER COURSE	40 mm HL 4 SURFACE COURSE	40 mm HL 4 UPPER BINDER COURSE	ASPHALT PAVEMENT REMOVAL		
												PARTIAL DEPTH 50 mm	PARTIAL DEPTH 100 mm	FULL DEPTH
ESSA ROAD														
ESSA ROAD STRUCTURE						X								
S-E RAMP 10+000 - 10+399			X							X	X	X		
W-N RAMP 10+000 - 10+325			X							X	X	X		
E-S RAMP 10+000 - 10+328			X							X	X	X		
N-W RAMP 10+000 - 10+397			X							X	X	X		
DUNLOP STREET														
S-E RAMP 10+000 - 10+320			X				X	X					X	
E/W-S RAMP 10+057 - 10+219			X							X	X	X		
N-E/W RAMP 10+000 - 10+111			X							X	X	X		
S-W RAMP 10+000 - 10+213			X							X	X	X		
W/E-N RAMP 10+000 - 10+212			X							X	X	X		
W-N RAMP			X							X	X	X		
E-N RAMP			X							X	X	X		
BAYFIELD STREET														
W-N/S RAMP 10+000 - 10+033					X		X					X		
W-N/S RAMP 10+033 - 10+400					X		X					X		
W-N/S RAMP 10+415 - 10+464			X				X					X		
N-W RAMP 9+666 - 9+996					X		X							X
N/S-E RAMP 10+000 - 10+385			X		X		X					X		
STRIP REPAIR 10+216 - 10+353								X	X				X*	
RT SHLD REPAIR 10+216 - 10+353							X	X				X**		
E-N/S RAMP 10+070 - 10+120					X		X	X				X	X	
E-N/S RAMP 10+120 - 10+175					X		X	X					X	
DUCKWORTH STREET														
S-N/S RAMP 10+000 - 10+050			X				X					X		

NOTES:
* Remove 100 mm depth in addition to 50 mm surface removal.
** Remove 50 mm depth in addition to 50 mm surface removal.
Table to be read in conjunction with accompanying report.

TABLE V

MAIN LINE TRAFFIC VOLUMES

Highway 400 Planning Study
GWP 30-95-00

LOCATION	AADT 1999	AADT 2001	AADT 2006	AADT 2011	SADT 1999	SADT 2001	SADT 2006	SADT 2011	% COMM.	ESALs (millions)
Simcoe Road 88	64,800	70,400	84,300	98,200	80,400	87,300	104,600	121,800	12	49.7
Highway 89	65,550	71,700	87,000	102,200	81,300	88,900	107,900	126,800	12	50.6
Innisfil Beach Road	70,500	77,200	93,800	110,400	87,400	95,700	116,300	136,900	12	54.7
Molson Park Drive	69,500	76,100	92,500	109,000	86,200	94,400	114,700	135,200	12	54.2
Essa Road	84,000	91,600	110,700	129,800	104,200	113,600	137,300	161,000	12	64.8
Dunlop Street	84,700	93,300	114,800	136,300	105,000	115,700	142,300	169,000	9.5	53.3
Bayfield Street	63,000	67,500	78,700	89,900	78,100	83,700	97,600	111,400	12	45.1
Duckworth Street	58,500	63,200	74,800	86,500	72,500	78,300	92,700	107,200	12	43.3
North Project Limit										

NOTES:
1. Traffic data provided by URS Cole, Sherman by e-mail, December 3, 2001.
2. Table to be read in conjunction with accompanying report.

TABLE VI

INTERCHANGE RAMP TRAFFIC VOLUMES

Highway 400 Planning Study
GWP 30-95-00

LOCATION	AADT 1999	2001	2006	2011	% COMM.
Highway 89					
S-E/W	5,100	5,500	6,600	7,600	12
E/W-N	4,500	4,900	5,800	6,700	11
N-E/W	4,700	5,100	6,100	7,000	11
E/W-S (1)	4,700	5,100	6,100	7,000	11
E/W-S (2)	1,000	1,100	1,300	1,500	13
Innisfil Beach Road					
S-E/W	4,200	4,800	6,200	7,600	15
E-N	800	1,400	1,700	2,100	11
W-N	4,100	4,300	6,000	7,800	9
N-E/W (1)	2,500	4,700	6,300	7,900	8
N-E/W (2)	1,700	4,000	6,700	9,400	10
E-S	5,100	2,400	3,000	3,600	14
Molson Park Drive					
S-E/W	8,600	9,100	10,200	11,300	9
E/W-N	9,500	10,000	11,300	12,500	9
N-E/W	8,400	8,800	10,000	11,100	11
E/W-S	5,200	5,500	6,200	6,900	11
Essa Road					
S-E/W	4,800	5,100	6,000	6,800	10
W-N	4,800	5,100	6,000	6,800	10
E-N	6,900	7,400	8,600	9,800	9
N-E/W	14,100	15,100	17,600	20,100	9
E-W/S	4,100	4,400	5,100	5,800	10
Dunlop Street					
S-E	4,900	5,300	6,300	7,300	10
S-W	8,400	9,100	10,800	12,600	11
E/W-N	7,200	7,800	9,300	10,800	10
N-E/W	7,300	7,900	9,400	10,900	9
E/W-S	9,900	10,700	12,800	14,800	8

INTERCHANGE RAMP TRAFFIC VOLUMES

LOCATION	AADT 1999	2001	2006	2011	% COMM.
Bayfield Street					
S-E/W	13,100	13,400	14,000	14,600	10
E/W-N	4,300	4,400	4,600	4,800	10
N-E/W	4,400	4,500	4,700	4,900	9
E-S	2,500	2,500	2,700	2,800	10
W-S	11,400	11,600	12,200	12,700	8
Duckworth Street					
S-E/W	8,400	9,500	12,100	14,800	9
E/W-N	3,600	4,100	5,200	6,300	11
N-E/W	3,600	3,900	4,800	5,700	12
E-S	6,200	6,800	8,300	9,800	9
W-S	1,300	1,400	1,700	2,100	12

NOTES:
1. Traffic data provided by URS Cole, Sherman by e-mail, December 3, 2001.
2. Table to be read in conjunction with accompanying report.

TABLE VII

MOE WATER WELL RECORDS

Highway 400 Planning Study
GWP 30-95-00

CONCESSION	LOT	WELL NO.	ELEVATION (m)	UTM EASTING	UTM NORTHING	WELL DIAMETER (mm)	WATER FOUND (m)	STATIC LEVEL (m)	PUMP LEVEL (m)	TEST RATE (l/min)	WELL DEPTH (m)	OVERBURDEN/BEDROCK (O/B)	WATER USE
West Gwillimbury Township													
14	5	57-5370	225.6	606501	4894143	101.6	33.5	FLW			45.7	O	DO
14	5	57-7471	225.6	606700	4893340	762.0	3.0	2.1	6.1	75.7	7.6	O	IR
14	6	57-5396	226.2	607116	4894606	177.8	77.7	FLW			77.7	O	CO
14	7	57-8408	230.1	607900	4893245	177.8	89.9	FLW			92.0	O	DO
Innisfil Township													
1	5	57-8774	236.2	606160	4894490	152.4	DRY				61.0	O	
1	5	57-11777	228.6	606217	4894421	101.6	63.7	FLW		75.7	81.7	B	MU
1	5	57-11778	228.6	606244	4894389	254.0	DRY				70.7	O	
1	5	57-866	245.7	606439	4895100	127.0	89.9	7.6	15.2	37.9	90.2	O	ST DO
1	6	57-865	227.1	607316	4894857	152.4	9.8	2.4	23.2	94.6	40.2	O	CO
1	6	57-867	227.1	607321	4894917	152.4	35.1	3.0	30.5	30.3	39.6	O	CO
1	7	57-868	227.1	607315	4895177	203.2	97.5	FLW			103.6	B	CO
1	7	57-869	227.1	607321	4894917	762.0	0.9	0.9		15.1	4.9	O	IN
1	7	57-870	227.1	607471	4895002	101.6	36.6	3.7	13.4	37.9	36.6	O	IN
2	5	57-9351	266.7	606170	4896550	127.0	DRY				56.4	O	
2	5	57-9698	251.5	606180	4896550	127.0	106.7	26.2	45.7	22.7	106.7	O	DO
2	5	57-895	236.2	606186	4895831	762.0	13.4	7.0		7.6	13.7	O	ST DO
2	5	57-14033	251.5	606230	4896500	127.0	147.8	27.1	144.8	15.1	147.8	B	DO
2	6	57-6141	266.7	606340	4896950	762.0	15.2	9.8			15.2	O	DO
3	6	57-6454	280.4	605970	4898160	762.0	3.7	1.8	2.7		3.7	O	DO
3	6	57-7257	283.5	606000	4898080	127.0	12.2	6.7	12.2	7.6	14.6	O	DO
3	7	57-910	263.7	606993	4897606	762.0	15.2	11.6		7.6	15.2	O	ST DO
4	5	57-13595	291.1	605250	4899750	152.4	16.5	6.4	10.4	37.9	16.5	O	DO
4	6	57-938	289.6	605891	4899632	762.0	6.1	6.1		7.6	9.8	O	ST DO
4	7	57-941	291.1	606694	4899028	762.0	19.8	16.8		11.4	19.8	O	ST DO
5	6	57-11926	291.1	605550	4901350	762.0	10.7	4.9			11.3	O	DO
5	7	57-996	294.1	606447	4900356		10.7	9.1		3.8	18.9	O	ST DO
6	6	57-5418	304.8	605150	4902550	762.0	7.6	3.7			13.7	O	DO
6	7	57-1049	294.1	606256	4901938	762.0	11.6	6.7		11.4	11.6	O	ST DO
7	5	57-11502	303.3	604750	4902800	762.0	10.1	3.7			13.4	O	ST
7	5	57-1071	303.3	604777	4903527	152.4	20.1	6.4	18.3	30.3	21.9	O	DO
7	6	57-10894	303.3	604976	4903291	152.4	63.4	26.8		75.7	88.4	O	CO
7	6	57-5630	304.8	605050	4904020	177.8		3.7	31.7	94.6	33.5	O	CO
7	6	57-10050	303.3	605055	4903084		DRY				91.4	O	
7	6	57-1072	304.8	605140	4903971	762.0	9.1	9.1		7.6	13.7	O	ST DO
7	7	57-8737	297.2	605670	4904190	152.4	59.1				94.5	O	
7	7	57-8736	300.2	605680	4903870	152.4	63.7				94.5	O	
7	7	57-8735	300.2	605750	4903900	152.4					74.7	O	
7	7	57-8738	304.8	605850	4903450	152.4		32.3	33.2	41.6	74.7	O	
7	7	57-7421	286.5	605870	4904170	152.4	58.8	23.8		227.1	97.5	O	CO

MOE WATER WELL RECORDS

CONCESSION	LOT	WELL NO.	ELEVATION (m)	UTM EASTING	UTM NORTHING	WELL DIAMETER (mm)	WATER FOUND (m)	STATIC LEVEL (m)	PUMP LEVEL (m)	TEST RATE (l/min)	WELL DEPTH (m)	OVERBURDEN/BEDROCK (O/B)	WATER USE
8	5	57-5725	304.8	604050	4905150	152.4	20.4	8.5	20.4	15.1	21.3	O	DO
8	6	57-10594	303.3	604623	4904750	127.0	65.2	28.0	33.5	22.7	65.2	O	DO
8	6	57-1127	303.3	604639	4904684	762.0	7.3	6.4		7.6	16.2	O	DO
8	6	57-1126	304.8	604774	4904008	762.0	10.4	7.6		7.6	13.7	O	DO
8	6	57-9006	304.8	605100	4904220	177.8	73.2	26.8		227.1	78.6	O	CO
8	6	57-9007	304.8	605140	4904230	177.8					78.6	O	
8	7	57-8108	301.8	605480	4904650	152.4	73.2	24.4	35.4	768.4	118.9	O	CO
8	7	57-8107	301.8	605480	4904720	50.8		21.9			79.2	O	
8	7	57-8739	303.3	605500	4904825	50.8	57.9	24.4			88.4	O	
8	7	57-11853	298.7	605600	4904900	254.0	57.9	25.6	53.9	1930.4	91.4	O	MU
9	5	57-1176	312.4	603656	4906406	101.6	40.2	31.7	32.6	11.4	40.2	O	ST DO
9	5	57-1177	310.9	604158	4906189	152.4	54.6	33.5	49.4	189.3	77.7	O	DO
10	5	57-14396	310.9	603600	4906550	152.4	44.8	30.5	41.1	22.7	44.8	O	DO
10	5	57-13133	312.4	603660	4907850	914.4	39.9	35.7	39.6	37.9	42.7	O	DO
10	5	57-14349	310.9	603700	4906550	152.4		30.5	39.6	22.7	50.9	O	DO
10	5	57-7034	310.9	603850	4906600	762.0	12.5	7.9			13.1	O	DO
10	5	57-8465	304.8	604090	4907300	152.4	44.2	34.4	42.1	30.3	45.7	O	DO
10	5	57-1252	307.8	604102	4907076	152.4	57.3	31.1	48.8	56.8	58.2	O	DO
10	5	57-1251	310.9	604180	4906760	101.6	39.3	31.7	35.4	30.3	39.3	O	DO
10	5	57-5928	310.9	604200	4906700	152.4	48.8	31.7	36.6	26.5	58.5	O	DO
10	6	57-1255	306.3	604156	4907281		34.7	34.7	37.8	34.1	40.8	O	DO
10	6	57-1253	309.4	604365	4906772	101.6	DRY				42.7	O	
10	6	57-1254	309.4	604415	4906757		42.1	32.0	41.1	26.5	43.0	O	DO
10	7	57-1256	304.8	605240	4907058	101.6	21.3	15.8	16.8	22.7	27.4	O	DO
11	5	57-1390	309.4	603548	4909041	101.6	9.1	9.1	13.1	11.4	15.2	O	DO
11	5	57-1389	309.4	603603	4909042	863.6	7.6	7.6		15.1	11.3	O	DO
11	5	57-1388	309.4	603753	4908316	101.6	36.6	36.6	44.2	30.3	47.9	O	ST DO
11	5	57-1391	310.9	603823	4908442	152.4	43.6	37.2	42.7	30.3	44.5	O	ST DO
11	6	57-8895	312.4	603870	4908800	127.0	41.1	38.7	41.1		46.9	O	DO
11	6	57-1392	307.8	604027	4909405	101.6	13.7	3.7	12.2	30.3	13.7	O	DO
11	8	57-1394	289.6	605197	4909744		33.2	19.5	33.5	7.6	34.1	O	ST DO
12	5	57-14922	313.9	602900	4910550	762.0	4.6	3.0			7.6	O	DO
12	5	57-1443	313.9	602967	4910524	762.0	4.6	4.6		11.4	9.1	O	DO
12	5	57-1435	313.9	603003	4910522		18.0	14.3	16.8	15.1	20.1	O	DO
12	5	57-12158	313.9	603100	4910600	762.0	7.0				10.7	O	DO
12	5	57-1444	313.9	603107	4910602	762.0	7.6	4.6	6.1	7.6	7.6	O	CO
12	5	57-1445	313.9	603190	4910671	152.4	23.5	14.6	21.3	37.9	24.4	O	ST DO
12	5	57-14107	312.4	603400	4910570	127.0	46.6	42.7	44.2	22.7	50.0	O	DO
12	5	57-1441	312.4	603441	4910492	101.6	DRY				36.6	O	
12	5	57-1442	312.1	603446	4910480	127.0	DRY				145.4	O	
12	5	57-1446	309.4	603495	4910215		49.1	42.4	47.2	37.9	52.1	O	DO
12	5	57-1440	310.9	603516	4910035		50.9	42.4	46.9	15.1	50.9	O	DO
12	5	57-8231	307.8	603600	4909630	101.6	17.7	6.7	13.7	18.9	18.3	O	DO
12	6	57-13455	304.8	603740	4910850	127.0	32.0	32.0	39.6	18.9	44.5	O	DO

MOE WATER WELL RECORDS

CONCESSION	LOT	WELL NO.	ELEVATION (m)	UTM EASTING	UTM NORTHING	WELL DIAMETER (mm)	WATER FOUND (m)	STATIC LEVEL (m)	PUMP LEVEL (m)	TEST RATE (l/min)	WELL DEPTH (m)	OVERBURDEN/BEDROCK (O/B)	WATER USE
12	6	57-8284	304.8	604100	4909550	152.4	42.7	35.4	39.9	53.0	43.9	O	DO
12	6	57-7578	306.3	604150	4909580	762.0	4.6	1.5	4.0	18.9	4.6	O	DO
12	7	57-1447	300.2	604620	4909888	762.0	1.8	1.8		7.6	6.1	O	DO
12	8	57-1448	283.5	605079	4911239	101.6	27.4	13.1	14.6	15.1	27.4	O	ST DO
13	5	57-1708	297.2	602611	4911922	177.8	37.2	28.0	36.0	26.5	38.4	O	DO
13	5	57-1522	262.1	602616	4911918	152.4	15.2	0.6	13.4	26.5	15.5	O	DO
13	5	57-1518	266.7	602702	4911952	101.6	14.9	3.7	8.5	15.1	14.9	O	DO
13	5	57-14481	266.7	602750	4911950	127.0	18.3	3.4	9.1	56.8	19.2	O	DO
13	5	57-7925	266.7	602750	4911980	152.4	21.6	3.7	18.9	37.9	22.6	O	DO
13	5	57-14024	265.2	602900	4912010	152.4	17.7	3.4	13.7	37.9	17.7	O	DO
13	5	57-1523	263.7	602921	4912027	762.0	2.7	2.7		7.6	5.5	O	DO
13	5	57-10466	256.0	602955	4912040	762.0	4.6	2.4		11.4	6.1	O	DO
13	5	57-5942	237.7	602980	4912550	101.6	15.2	5.2	10.7	18.9	15.2	O	DO
13	5	57-13139	297.2	603100	4911680	152.4	33.5	32.0	36.6	75.7	40.8	O	DO
13	5	57-1519	281.9	603108	4911854	50.8	28.7	21.3		37.9	30.5	O	DO
13	5	57-1520	286.5	603117	4911814	101.6	30.5	27.4	30.5	18.9	35.1	O	DO
13	5	57-7953	274.3	603130	4911950	101.6	18.3	11.6	11.6	37.9	18.3	O	DO
13	5	57-1524	285.0	603145	4911809	101.6	27.4	27.4	30.5	15.1	36.6	O	DO
13	5	57-9561	281.9	603150	4911860	101.6	29.9	17.7	21.3	18.9	29.9	O	DO
13	5	57-1525	289.6	603155	4911717	101.6	27.4	24.4	27.4	37.9	30.5	O	DO
13	5	57-1521	298.7	603167	4911787	762.0	DRY				27.4	O	
13	5	57-6557	313.9	603200	4910760	762.0	15.8	12.8	14.6		15.8	O	DO
13	5	57-1516	307.8	603212	4911082	101.6	19.8	17.4	17.4	15.1	22.9	O	DO
13	5	57-1517	313.9	603235	4910774	101.6	20.4	15.5	17.1	15.1	26.8	O	PS
13	5	57-13465	310.9	603250	4910950	127.0	15.2	13.7	17.7	15.1	22.3	O	DO
13	5	57-14003	310.9	603340	4910800	762.0	2.4	1.8	6.1	11.4	7.9	O	DO
13	5	57-14002	310.9	603350	4910840	762.0	2.4	1.8	9.1	11.4	11.3	O	DO
13	6	57-7677	306.3	603390	4911350	762.0	12.2	8.5	11.0	22.7	12.2	O	DO
13	6	57-13194	307.8	603400	4911010	127.0	14.0	11.3	13.7	15.1	18.6	O	DO
13	6	57-8794	306.3	603500	4911330	762.0	5.5	5.5	7.9	7.6	9.1	O	DO
13	6	57-1528	307.8	603510	4910878	101.6	40.2	40.2	44.2		47.5	O	DO
13	6	57-1529	286.5	603547	4911943	101.6	20.7	12.8	16.2	22.7	20.7	O	DO
13	6	57-1532	307.8	603573	4911395	152.4	53.3	44.8	54.3	22.7	54.6	O	CO
13	6	57-7449	306.3	603600	4910900	101.6	51.8	42.7	45.7	18.9	51.8	O	DO
13	6	57-14186	307.8	603600	4910900	127.0	36.6	36.6	44.2	22.7	48.2	O	DO
13	6	57-7567	306.3	603600	4911350	762.0	14.0	11.0	13.4	22.7	14.0	O	DO
13	6	57-11221	307.8	603612	4910827	127.0	40.5	40.5	42.7	26.5	48.8	O	DO
13	6	57-1526	307.8	603619	4911470	101.6	53.9	42.1	43.6	18.9	53.9	O	DO
13	6	57-1527	304.8	603670	4911501	101.6	22.6	22.6	26.2	7.6	28.0	O	DO
13	6	57-1531	265.2	603685	4912112		15.5	9.1	15.2	53.0	16.8	O	ST DO
13	6	57-11172	306.3	603698	4910858	127.0	40.5	40.5	42.7	26.5	49.1	O	DO
13	6	57-8282	289.6	603700	4911750	203.2	36.0	23.8	32.9	75.7	38.4	O	DO
13	6	57-1530	281.9	603764	4911980	152.4	DRY				12.2	O	
13	6	57-13884	306.3	603800	4911400	152.4	64.0	43.9	53.3	30.3	74.7	O	CO DO

MOE WATER WELL RECORDS

CONCESSION	LOT	WELL NO.	ELEVATION (m)	UTM EASTING	UTM NORTHING	WELL DIAMETER (mm)	WATER FOUND (m)	STATIC LEVEL (m)	PUMP LEVEL (m)	TEST RATE (l/min)	WELL DEPTH (m)	OVERBURDEN/BEDROCK (O/B)	WATER USE
13	7	57-1533	262.1	603715	4912243	152.4	68.3	45.7	51.8	30.3	68.3	O	CO
13	7	57-1538	281.9	604297	4912456	101.6	17.1	13.4	13.7	90.8	23.2	O	PS
13	7	57-1534	292.6	604496	4911434	101.6	30.5	25.9	32.0	37.9	33.8	O	IR
13	7	57-1536	292.6	604544	4911568	203.2	35.1	18.3	27.4	227.1	44.5	O	PS
13	7	57-1535	292.6	604556	4911620	101.6	38.1	18.3	42.7	151.4	43.6	O	IR
13	7	57-1537	280.4	604559	4911970	101.6	29.0	20.1	21.0	60.6	33.8	O	DO
14	5	57-1692	234.7	602375	4913451	152.4	71.0	2.4	56.4	15.1	91.4	O	CO
14	5	57-11200	233.2	602500	4913450	127.0	25.9	7.6	13.7	37.9	28.0	O	DO
14	5	57-1694	234.7	602541	4913510	50.8	18.3	2.4	6.7	30.3	19.8	O	DO
14	5	57-11395	233.2	602550	4913450	127.0	18.9	7.3	14.0	18.9	20.1	O	DO
14	5	57-1711	259.1	602575	4911990	762.0	1.8	1.8		15.1	6.1	O	DO
14	5	57-7116	236.2	602600	4913240	762.0	5.8	3.0	4.3		5.8	O	DO
14	5	57-1693	234.7	602608	4913515	50.8	18.3	2.4	6.7	30.3	19.8	O	DO
14	5	57-1710	262.1	602641	4912049	152.4	16.8	0.3	9.1	26.5	16.8	O	DO
14	5	57-10573	236.2	602650	4913230	762.0	6.1	5.5			9.4	O	DO
14	5	57-10574	236.2	602650	4913230	762.0	2.1	2.1			4.3	O	DO
14	5	57-10575	236.2	602650	4903230	762.0	4.9	4.3			6.7	O	DO
14	5	57-6395	236.2	602650	4913500	127.0	18.9	7.6	16.8	22.7	21.6	O	DO
14	5	57-6999	236.2	602660	4913240	762.0		3.0	5.5		6.1	O	DO
14	5	57-1697	234.7	602682	4913481	127.0	DRY				77.1	O	
14	5	57-1696	234.7	602682	4913512	50.8	19.8	3.7	7.3	94.6	71.6	O	IR
14	5	57-11461	236.2	602700	4913150	127.0	22.9	9.1	12.2	37.9	25.6	O	DO
14	5	57-12388	236.2	602700	4913250	101.6	22.9	11.0	13.7	18.9	24.1	O	DO
14	5	57-7451	234.7	602710	4913520	101.6	21.6	4.6	9.1	37.9	22.9	O	CO
14	5	57-6796	236.2	602720	4913530	127.0	18.9	7.6	16.8	7.6	21.9	O	DO
14	5	57-12390	236.2	602730	4913250	101.6	25.9	11.6	22.9	18.9	27.4	O	DO
14	5	57-12325	239.3	602750	4912850	101.6	24.4	10.7	16.8	37.9	25.9	O	DO
14	5	57-12406	236.2	602750	4913150	101.6	28.0	9.8	25.9	18.9	29.6	O	DO
14	5	57-12557	236.2	602750	4913150	101.6	29.0	12.2	24.4	22.7	29.9	O	DO
14	5	57-12391	236.2	602750	4913250	101.6	25.3	10.1	21.3	26.5	27.4	O	DO
14	5	57-1713	259.1	602774	4912075	101.6	11.3	0.9	1.2	56.8	12.8	O	DO
14	5	57-1720	239.3	602800	4912894	152.4	17.4	6.1	19.8	15.1	20.7	O	PS
14	5	57-12392	234.7	602800	4913270	101.6	28.3	9.1	21.3	30.3	29.9	O	DO
14	5	57-1715	259.1	602806	4912097	101.6	11.0	2.7	6.1	11.4	11.6	O	DO
14	5	57-5552	234.7	602810	4913370	101.6	21.9	10.7	16.8	18.9	23.8	O	DO
14	5	57-12393	234.7	602820	4913300	101.6	24.4	11.9	21.9	22.7	26.5	O	DO
14	5	57-1706	239.3	602838	4912799	152.4	19.8	6.7	17.4	26.5	20.7	O	DO
14	5	57-1704	239.3	602839	4912728	152.4	16.2	4.6	15.2	56.8	17.4	O	DO
14	5	57-9524	240.8	602850	4912770	127.0	17.4	7.6	13.7	37.9	21.6	O	DO
14	5	57-14607	240.8	602850	4912900	101.6	25.9	10.1	15.2	56.8	27.1	O	DO
14	5	57-12553	236.2	602850	4913200	101.6	29.6	10.1	18.3	37.9	30.5	O	DO
14	5	57-12389	234.7	602850	4913300	101.6	20.7	14.0	18.3	22.7	22.3	O	DO
14	5	57-1705	240.8	602853	4912619	152.4	15.8	6.1	14.6	53.0	17.1	O	DO
14	5	57-1703	240.8	602855	4912642	152.4	15.5	4.6	13.7	37.9	16.5	O	DO

MOE WATER WELL RECORDS

CONCESSION	LOT	WELL NO.	ELEVATION (m)	UTM EASTING	UTM NORTHING	WELL DIAMETER (mm)	WATER FOUND (m)	STATIC LEVEL (m)	PUMP LEVEL (m)	TEST RATE (l/min)	WELL DEPTH (m)	OVERBURDEN/BEDROCK (O/B)	WATER USE
14	5	57-1718	240.8	602865	4912591	152.4	19.5	9.1	20.4	15.1	21.3	O	DO
14	5	57-1716	237.7	602865	4913011	101.6	17.7	6.7	9.1	37.9	17.7	O	DO
14	5	57-1712	237.7	602868	4913055	152.4	12.8	6.1	12.8	15.1	13.7	O	DO
14	5	57-1698	240.8	602882	4912568	152.4	14.0	6.1	13.1	34.1	14.0	O	DO
14	5	57-1699	240.8	602886	4912508	152.4	14.0	6.1	13.7	37.9	14.0	O	DO
14	5	57-9738	236.8	602900	4913050	152.4	12.8	6.1	12.2	15.1	19.2	O	DO
14	5	57-7887	243.8	602940	4912360	762.0		5.5	8.2		9.1	O	DO
14	5	57-14479	251.5	602950	4912300		16.8	FLW	1.5	18.9	20.7	O	DO
14	5	57-11036	237.7	602950	4912850	101.6	12.2	8.5	13.7	11.4	16.8	O	DO
14	5	57-1717	254.5	602955	4912230	152.4	19.2	10.4	20.1	7.6	20.7	O	DO
14	5	57-1707	237.7	602960	4912778	152.4	18.6	5.8	18.3	18.9	19.8	O	DO
14	5	57-9522	237.7	602960	4912790	127.0	18.9	9.1	14.3	15.1	18.9	O	DO
14	5	57-1695	243.8	602963	4912145	152.4	14.3	FLW	9.1	37.9	14.3	O	DO
14	5	57-1700	240.8	602978	4912694	152.4	15.5	4.6	11.9	18.9	16.5	O	DO
14	5	57-1701	237.7	602982	4912715	152.4	13.1	4.3	12.2	37.9	14.0	O	DO
14	5	57-1709	243.8	602994	4912466	101.6	43.9	13.4	16.8	22.7	43.9	O	DO
14	5	57-1714	243.8	603007	4912463	152.4	14.9	7.6	15.8	15.1	17.7	O	DO
14	5	57-7768	243.8	603010	4912390	762.0	9.1	5.5	7.9	22.7	9.1	O	DO
14	5	57-1691	240.8	603015	4912541	101.6	61.0	3.0		15.1	61.0	O	DO
14	5	57-5572	251.5	603050	4912300	152.4	19.5	10.1	19.8	7.6	20.4	O	DO
14	5	57-1719	262.1	603058	4912172	152.4	13.4	1.8	12.2	37.9	14.6	O	DO
Vespra Township													
3	21	57-4613	265.2	604528	4918399	762.0	9.8	9.8		7.6	14.0	O	
3	21	57-4612	262.1	604697	4918520	152.4	DRY				48.2	O	
3	21	57-4611	262.1	604712	4918525	152.4	DRY				86.9	O	
4	20	57-9959	268.2	603970	4918780	152.4	46.6	29.0	46.6	15.1	47.5	O	DO
4	20	57-6133	266.7	604010	4918700	101.6	36.6	32.0	35.1	30.3	41.1	O	DO
4	20	57-5662	266.7	604020	4918700	152.4	62.5	28.3	64.0	11.4	64.9	O	DO
4	20	57-4724	266.7	604027	4918689	152.4	62.5	32.0	45.7	56.8	63.4	O	DO
4	20	57-9419	266.7	604046	4918712	762.0	14.3	14.3	16.2	11.4	18.9	O	DO
4	20	57-4725	269.7	604096	4918601	152.4	47.5	35.1	46.3	22.7	48.8	O	DO
Vespra Township - West of Penetanguishene Road													
1	4	57-4919	266.7	607582	4917937	152.4	39.0	21.3	36.0	30.3	41.1	O	DO
1	4	57-9618	260.6	607600	4918010		DRY				20.7	O	
1	4	57-4918	266.7	607633	4918032	127.0	29.6	29.0	33.5	30.3	44.2	O	IN
1	5	57-4922	243.8	605950	4918787	762.0	2.1	2.1		7.6	7.0	O	DO
1	5	57-4920	236.2	605965	4918859	762.0	1.8	1.8		7.6	5.2	O	MU
1	5	57-4923	239.3	605978	4918829	762.0		3.4		7.6	6.7	O	DO
1	5	57-4921	236.2	605998	4918859	762.0	2.4	2.4		11.4	5.5	O	DO
1	5	57-13399	259.1	606190	4918840	127.0	21.3	21.3	24.4	18.9	29.9	O	DO
1	5	57-4924	239.3	606223	4918854	762.0	12.8	12.8		7.6	20.1	O	DO
1	5	57-10838	242.3	606250	4918850	152.4	21.3	7.3	18.6	75.7	25.9	O	DO

MOE WATER WELL RECORDS

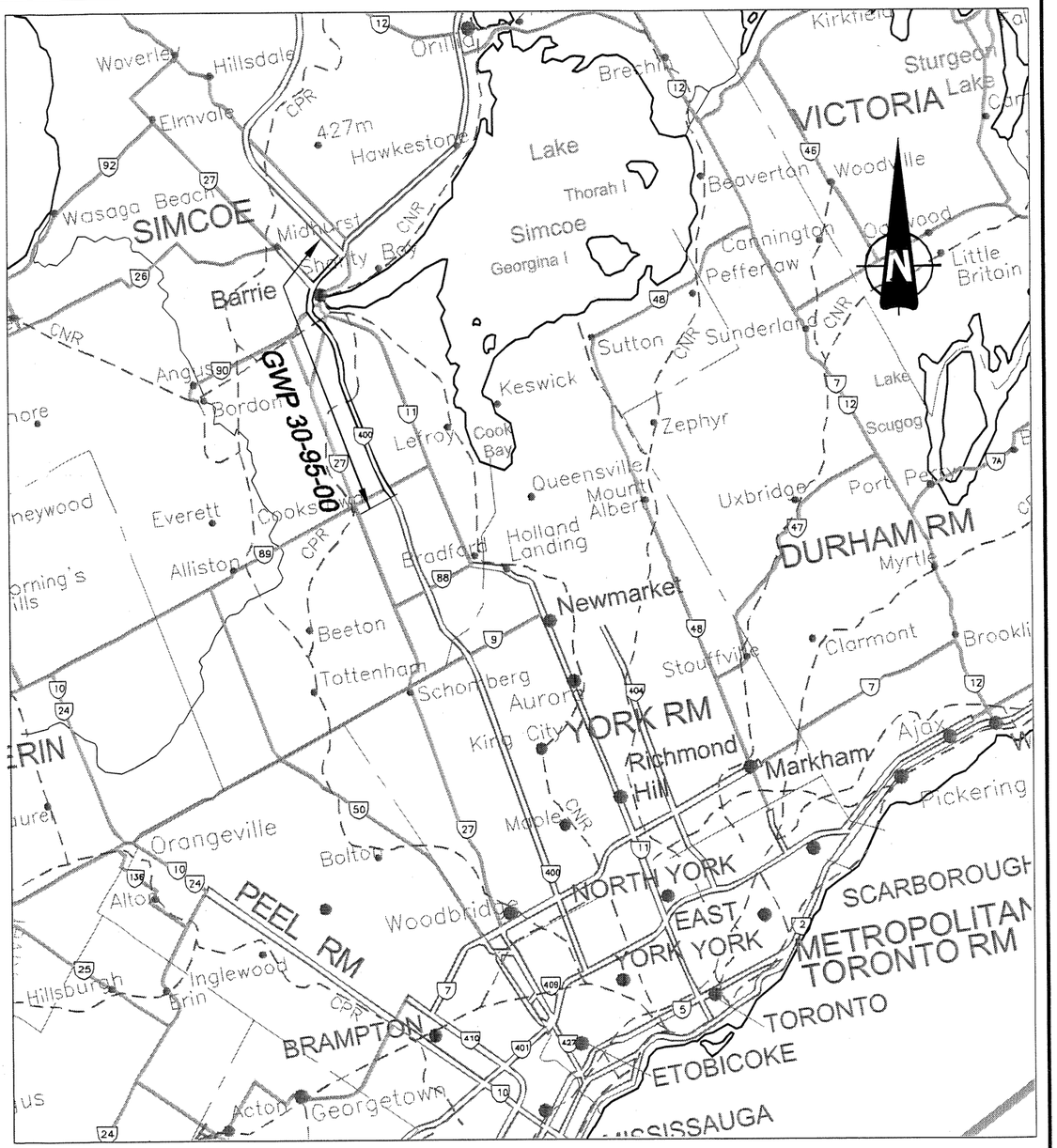
CONCESSION	LOT	WELL NO.	ELEVATION (m)	UTM EASTING	UTM NORTHING	WELL DIAMETER (mm)	WATER FOUND (m)	STATIC LEVEL (m)	PUMP LEVEL (m)	TEST RATE (l/min)	WELL DEPTH (m)	OVERBURDEN/BEDROCK (O/B)	WATER USE
1	5	57-9039	240.8	606250	4918860	762.0	4.6	4.6	6.1	15.1	8.5	O	DO
1	5	57-9041	240.8	606260	4918850	762.0	5.5	5.5	7.3	15.1	9.1	O	DO
1	5	57-10036	242.3	606265	4918850	762.0	3.0	3.4	5.5	15.1	7.0	O	DO
1	5	57-8875	262.1	606400	4918500	762.0	23.8	2.1	24.7	7.6	25.9	O	DO
1	5	57-3533	259.1	607950	4918394	152.4	42.4	24.4	35.1	56.8	46.6	O	ST DO
1	5	57-6645	261.2	607680	4918580	152.4	34.7	25.9	33.5	37.9	35.1	O	DO
1	6	57-4925	233.2	605770	4918994	101.6	9.4	FLW			9.4	O	CO
1	6	57-4991	233.2	605863	4918974	101.6	9.8	FLW			9.8	O	DO
1	6	57-7297	236.2	606220	4918970	762.0	4.6	3.0	4.0	18.9	4.6	O	DO
1	6	57-6807	236.2	606270	4918940	762.0	1.5	0.3	1.5		3.7	O	DO
1	6	57-4926	236.2	606378	4918909	1397.0	1.5	1.5		7.6	4.0	O	DO
1	6	57-7253	259.1	606710	4918860	152.4	27.4	27.4	32.0	37.9	38.4	O	DO
1	6	57-13400	259.1	606750	4918860	127.0	32.0	29.0	32.9	26.5	42.1	O	DO
1	6	57-6549	251.5	606750	4918880	101.6	45.7	27.4	33.5	30.3	45.7	O	DO
1	6	57-5941	259.1	606850	4918860	101.6	38.1	27.4	30.5	37.9	41.1	O	DO
1	6	57-10077	290.2	606900	4918860	127.0	32.0	30.5	32.3	18.9	36.9	O	DO
1	6	57-4584	262.1	606908	4918868	101.6	36.6	27.4	33.5	18.9	36.6	O	DO
1	6	57-4928	259.7	606913	4918890	101.6	35.1	27.4	30.5	37.9	35.1	O	DO
1	6	57-9525	262.1	606930	4918850	127.0	44.8	30.5	35.1	30.3	44.8	O	DO
1	6	57-8445	262.1	606970	4918840	127.0	54.3	29.3	45.7	18.9	55.5	O	DO
1	6	57-9364	262.1	607010	4918840	101.6	46.9	30.2	33.5	37.9	58.8	O	DO
1	6	57-5646	262.1	607050	4918850	101.6	54.3	36.6	38.1	37.9	54.3	O	DO
1	6	57-5906	262.1	607100	4918830	101.6	27.4	8.2	24.4	11.4	27.4	O	DO
1	6	57-10822	262.1	607150	4918800	127.0	33.5	30.5	33.5	26.5	41.5	O	DO
1	6	57-5905	262.1	607150	4918850	101.6	27.4	25.0	30.5	37.9	35.1	O	DO
1	6	57-6221	262.1	607150	4918820	101.6	36.6	25.9	29.0	37.9	36.6	O	DO
1	6	57-4927	239.3	607724	4918769	101.6	38.1	27.4	27.4	30.3	38.1	O	DO
1	7	57-12307	242.3	607800	4919150	127.0	28.3	9.8	14.3	37.9	30.5	O	DO
1	7	57-14222	236.2	607800	4919300	127.0	14.0	3.7	12.2	18.9	16.5	O	DO
1	7	57-9613	234.7	607800	4919400	762.0	1.8			34.1	5.5	O	DO
1	8	57-4938	233.2	607069	4919621	152.4					73.2	O	
1	8	57-4941	233.2	607079	4919551	203.2		FLW	0.9	598.0	69.8	O	PS
1	8	57-4940	233.2	607109	4919576	203.2		FLW	0.6	923.5	67.7	O	PS
1	8	57-4934	233.2	607109	4919681	50.8					70.7	O	PS
1	8	57-4937	233.2	607127	4919806	152.4					73.2	O	
1	8	57-4935	233.2	607134	4919673	152.4					73.5	O	
1	8	57-4936	233.2	607140	4919786	152.4					70.4	O	
1	8	57-4932	233.2	607207	4919823	152.4					39.6	O	
1	8	57-4931	233.2	607227	4919836	152.4					33.5	O	
1	8	57-4930	233.2	607245	4919866	152.4					72.2	B	
1	8	57-4939	233.2	607245	4919871	304.8					57.9	O	
1	8	57-4929	233.2	607247	4919838	304.8	32.3	FLW	15.2	7.6	32.3	O	PS
1	8	57-4933	233.2	607252	4919871	152.4					25.9	O	
1	10	57-4943	240.8	607893	4920538	152.4					77.4	O	
1	10	57-4942	240.8	607906	4920530	127.0		FLW	3.0	41.6	29.6	O	
1	11	57-4944	234.7	607285	4920058	101.6	26.8	19.5	20.4	11.4	32.0	O	DO

NOTES:
1. DO - Domestic
2. PS - Public Supply
3. IR - Irrigation
4. CO - Commercial
5. ST - Stock
6. MU - Municipal
7. IN - Industrial
8. FLW - Flowing
9. Table to be read in conjunction with accompanying report.

KEY PLAN

FIGURE 1

GWP 30-95-00 – HIGHWAY 400



Date NOV. 7, 2002

Project 001-3203

Golder Associates

LONDON, ONTARIO, CANADA

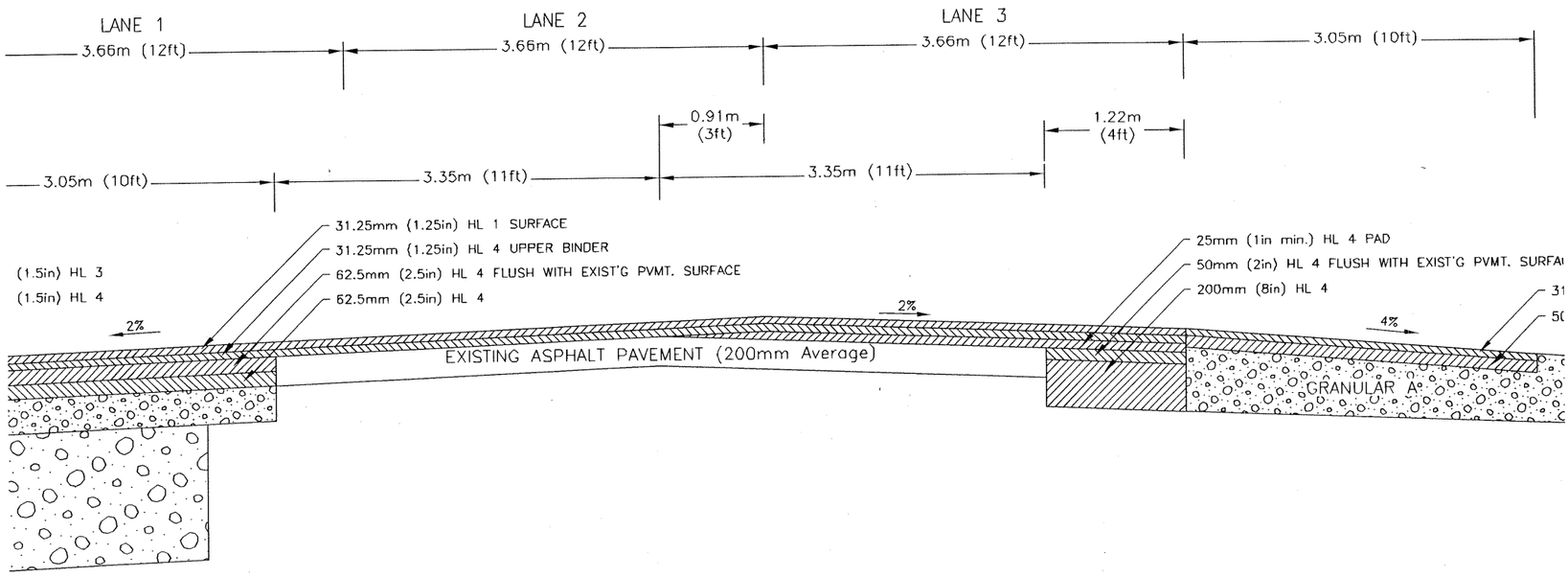
Drawn WDF

Chkd .....

001321 2D001.DWG WDF 11/07/02

SCALE 1:50,000

THIS DRAWING IS TO BE READ IN CONJU
WITH ACCOMPANYING TEXT.



REFERENCE

DRAWING BY DEPARTMENT OF
MATERIALS and TESTING DIVISI
ENTITLED W.P. 105-70 08&-17
TYPICAL SECTION - INSIDE WID
HWY. 400 WIDENING TO SIX LAI
DRAWING NO. APPENDIX II
DECEMBER 30, 1970

VERTICAL SCALE 1:25
HORIZONTAL SCALE 1:50

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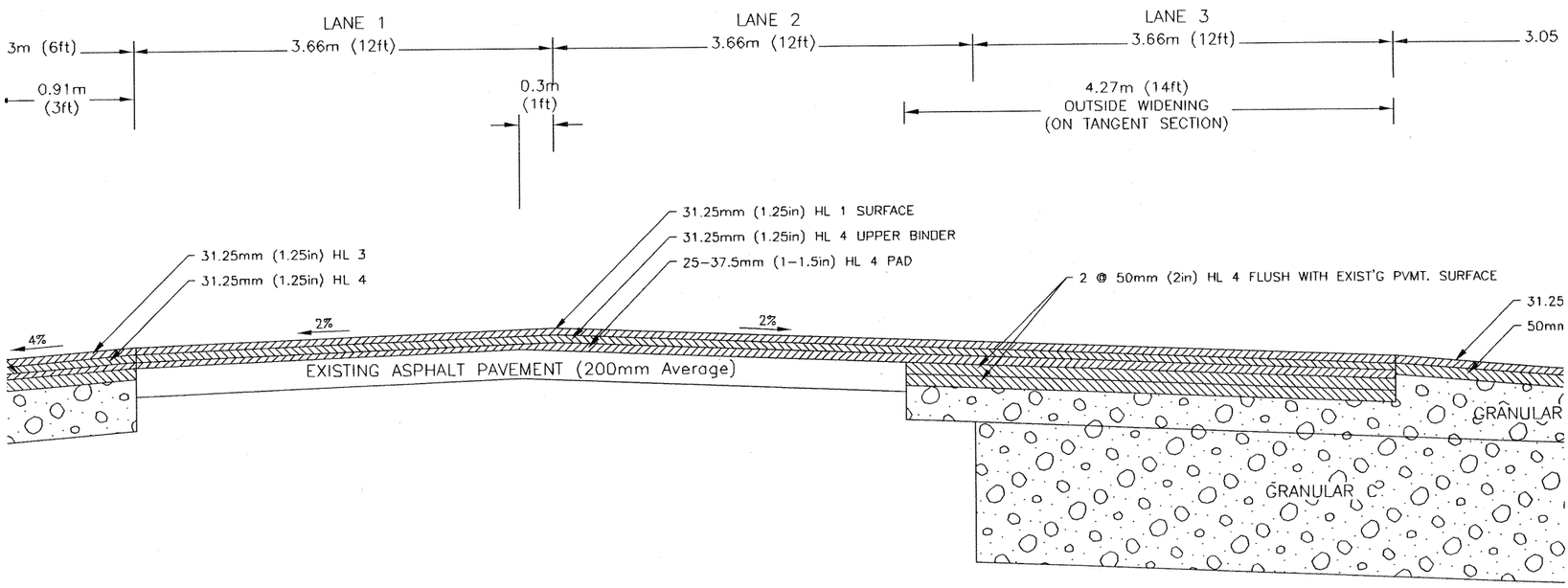

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DRAWING BY DEPARTMENT OF
MATERIALS and TESTING DIVISI
ENTITLED W.P. 105-70 08&-17
TYPICAL SECTION - OUTSIDE W
HWY. 400 WIDENING TO SIX LAI
DRAWING NO. APPENDIX III
DECEMBER 30, 1970

VERTICAL SCALE 1:25
HORIZONTAL SCALE 1:50

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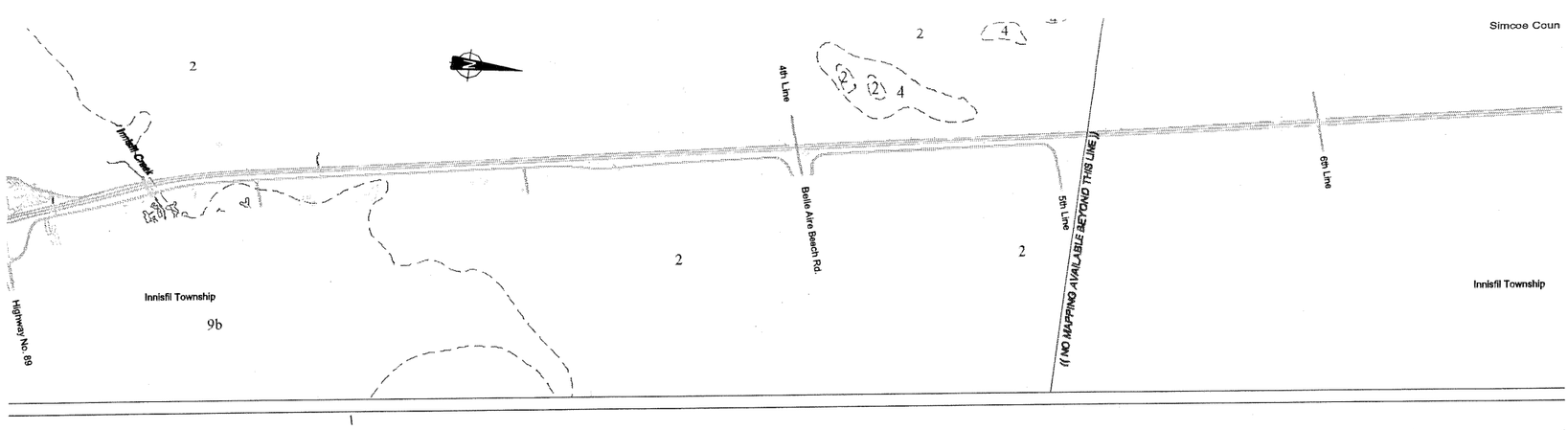
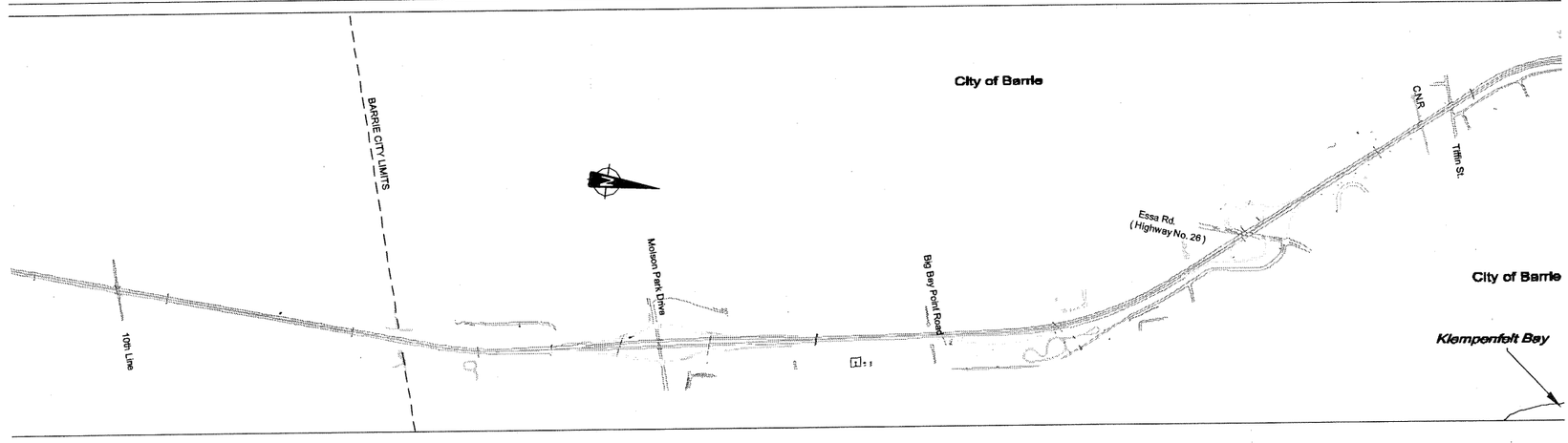
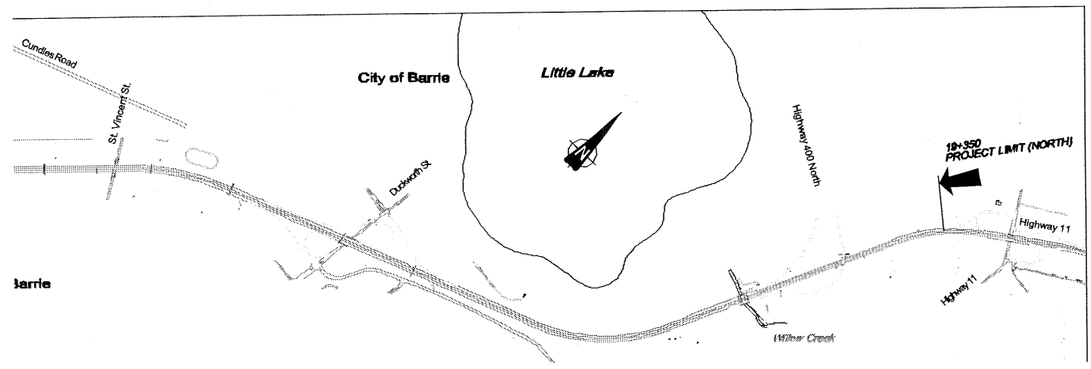






LEGEND

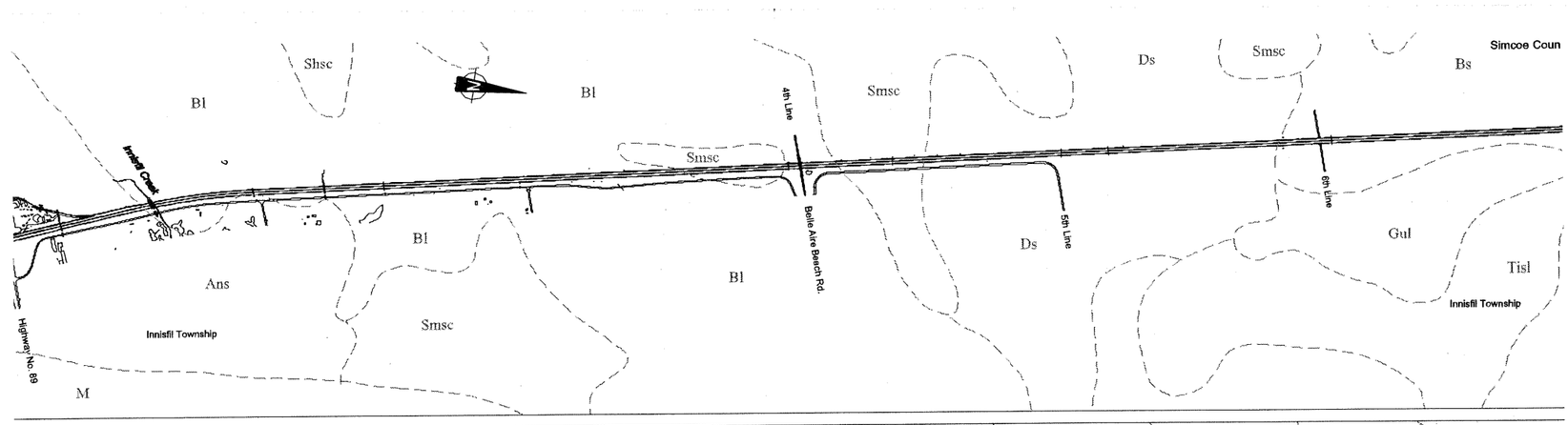
CENOZOIC
QUATERNARY

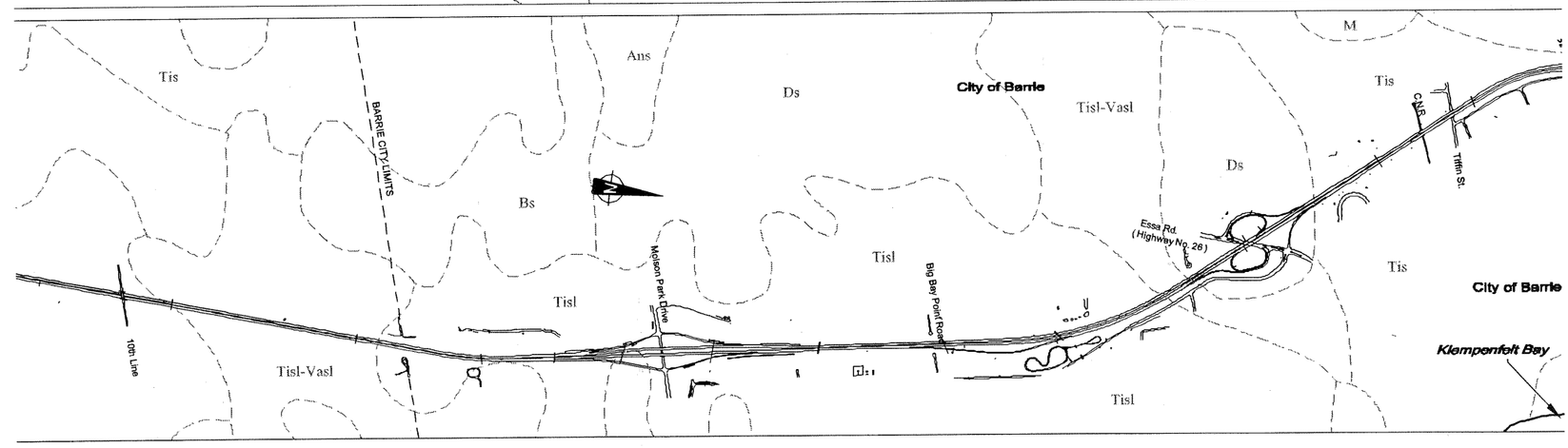
Pleistocene

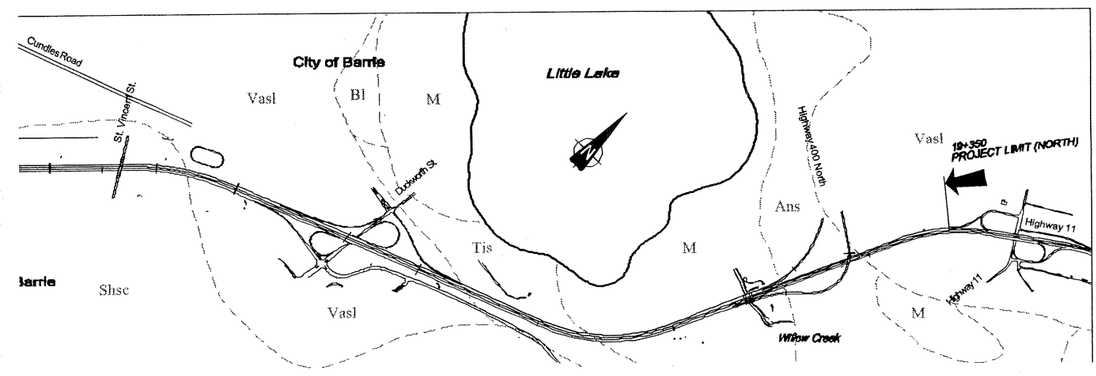
9 Glaciolacustrine shallow water; sand
9b Lake Algonquin deposits

4

2



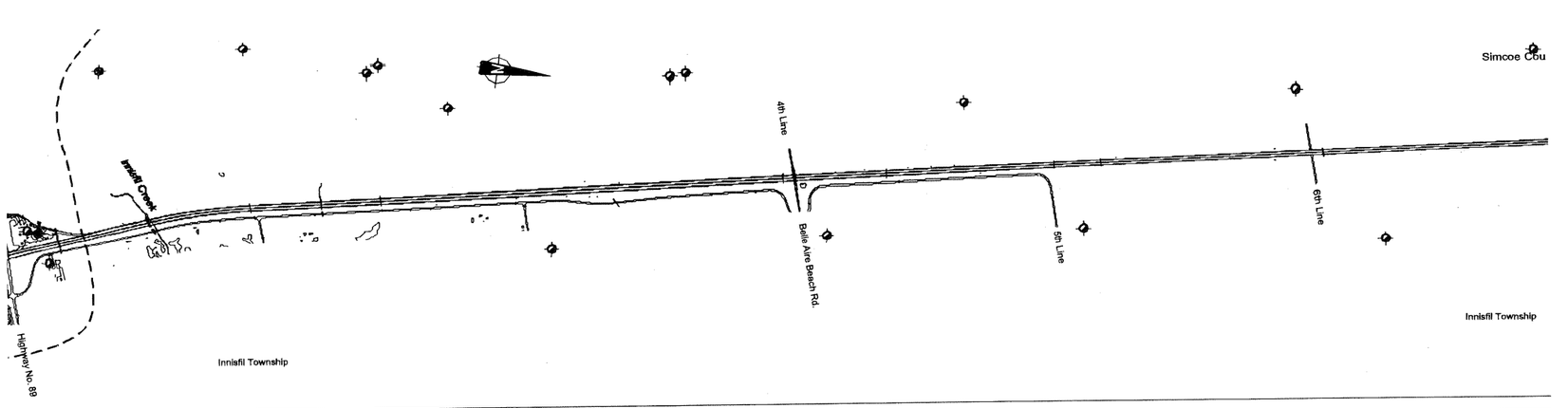


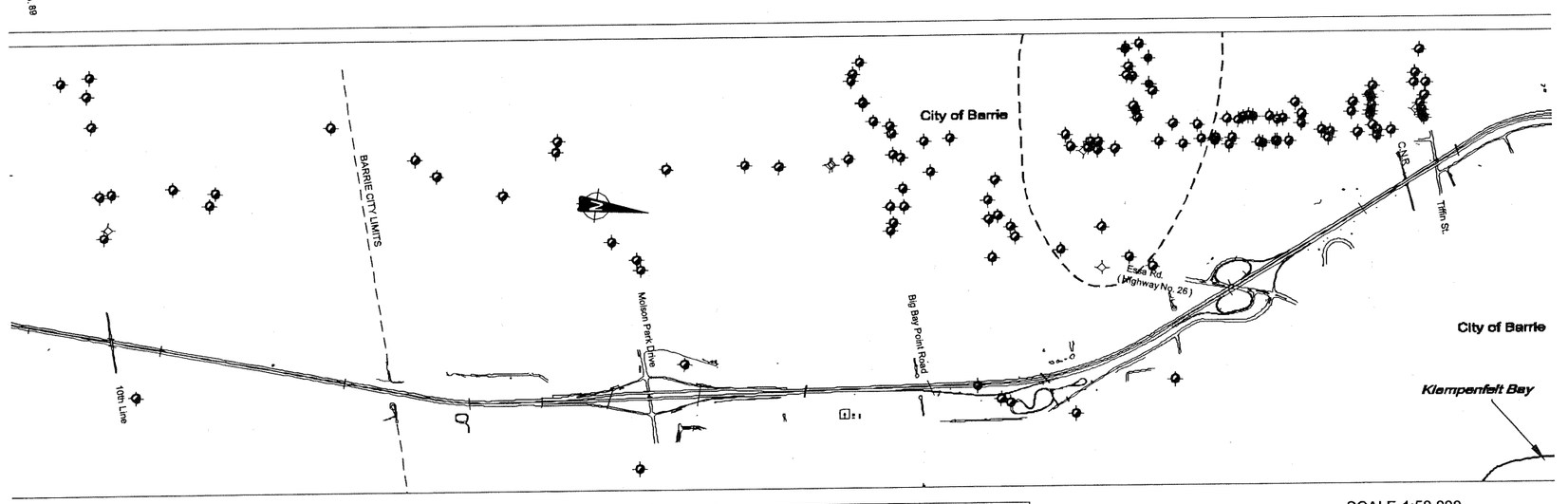


LEGEND

Map Symbol	Soil Series	Soil Type
Ans	ALLISTON	sandy loam
Bl	BONDHEAD	loam
Bs	BONDHEAD	sandy loam
Bs-b	BONDHEAD	sandy loam stoney phase
Ds	DUNDONALD	sandy loam
Gul	GUERIN	loam

Ma





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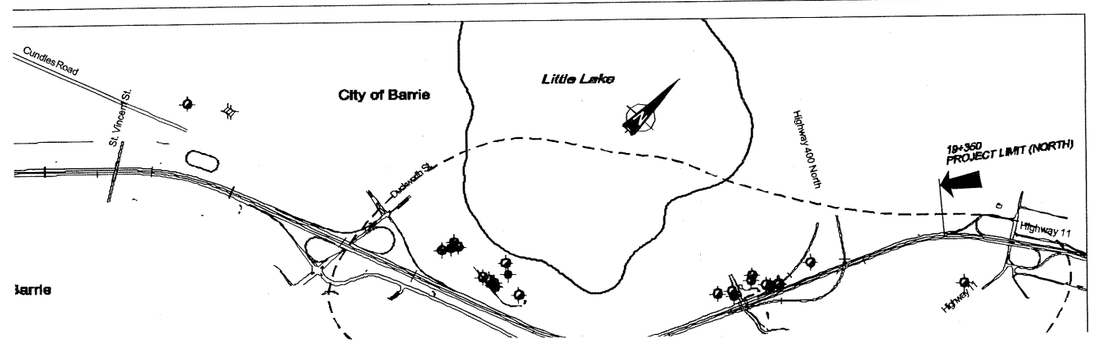
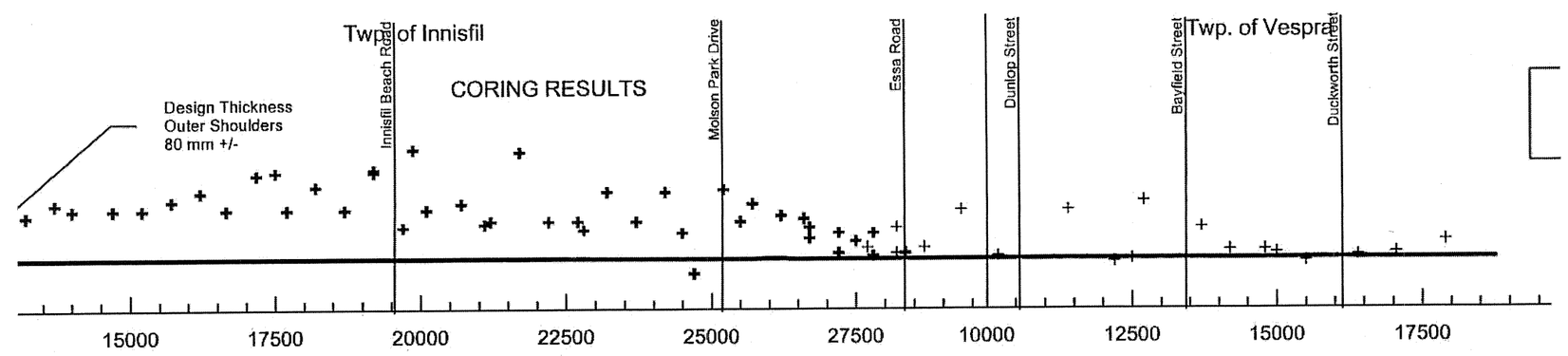
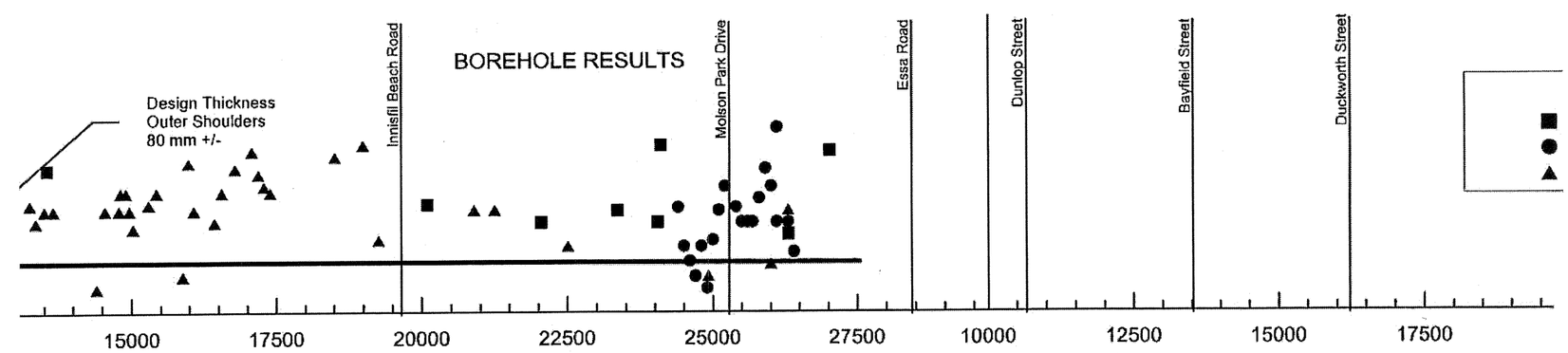


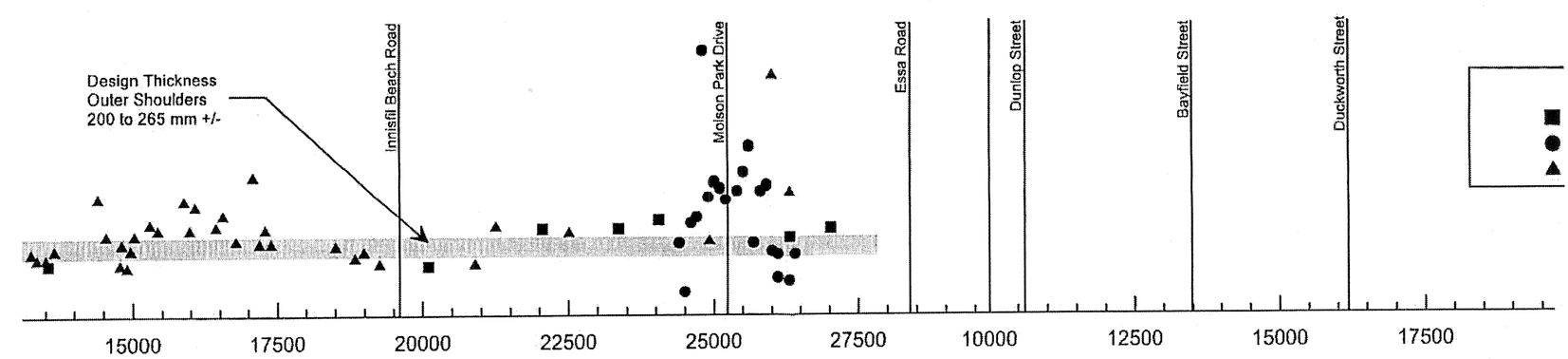
THIS DRAWING IS TO BE READ IN CONJU
WITH ACCOMPANYING TEXT.

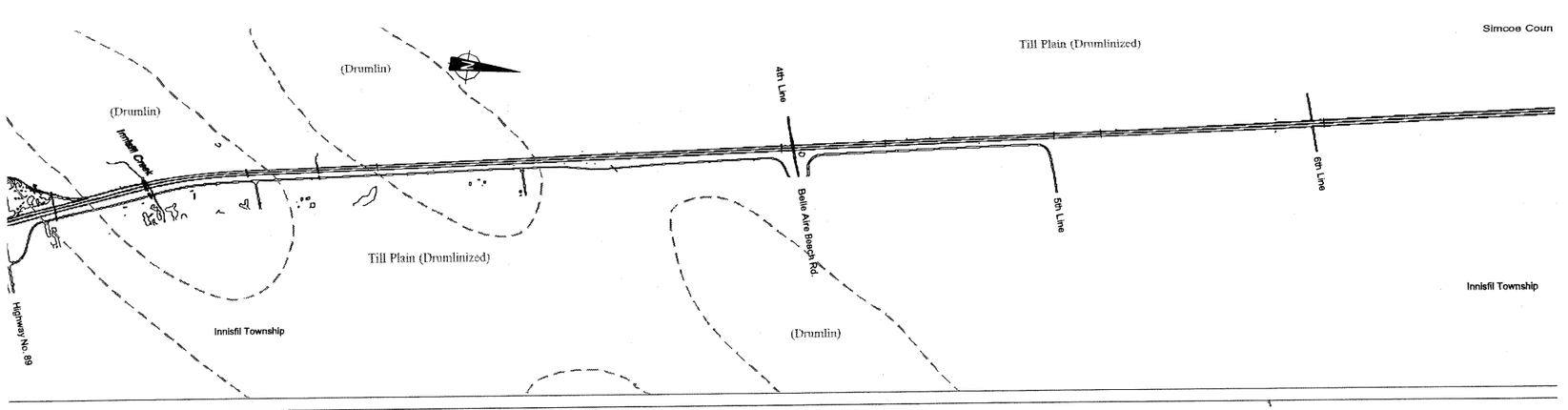
LEGEND

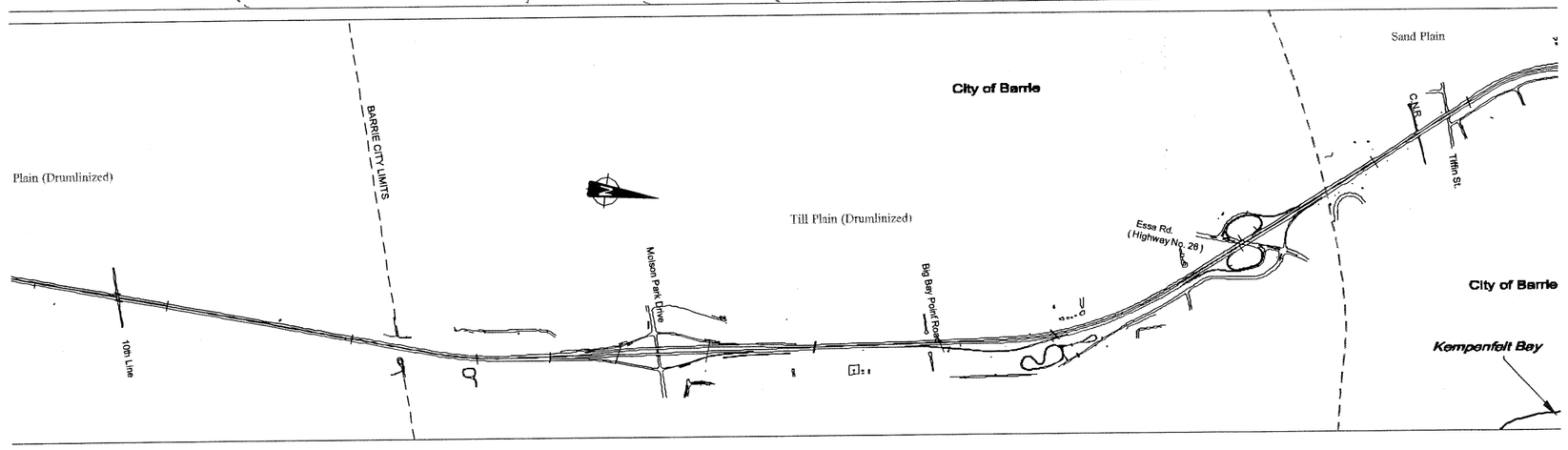
(MOE WATER WELL LOCATION AND DES
- FLOWING WELLS (ARTESIAN CONDI
- WATER TABLE LESS THAN 1.5 METR
- WATER TABLE GREATER THAN 1.5 M
- DRY

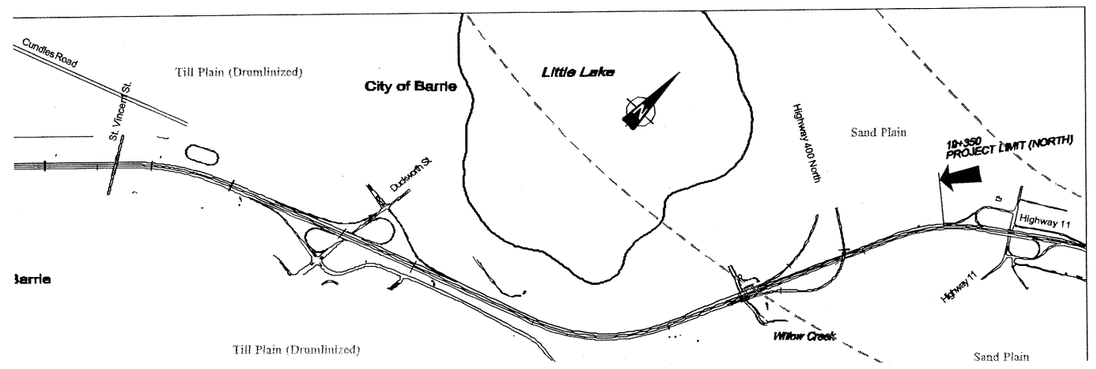












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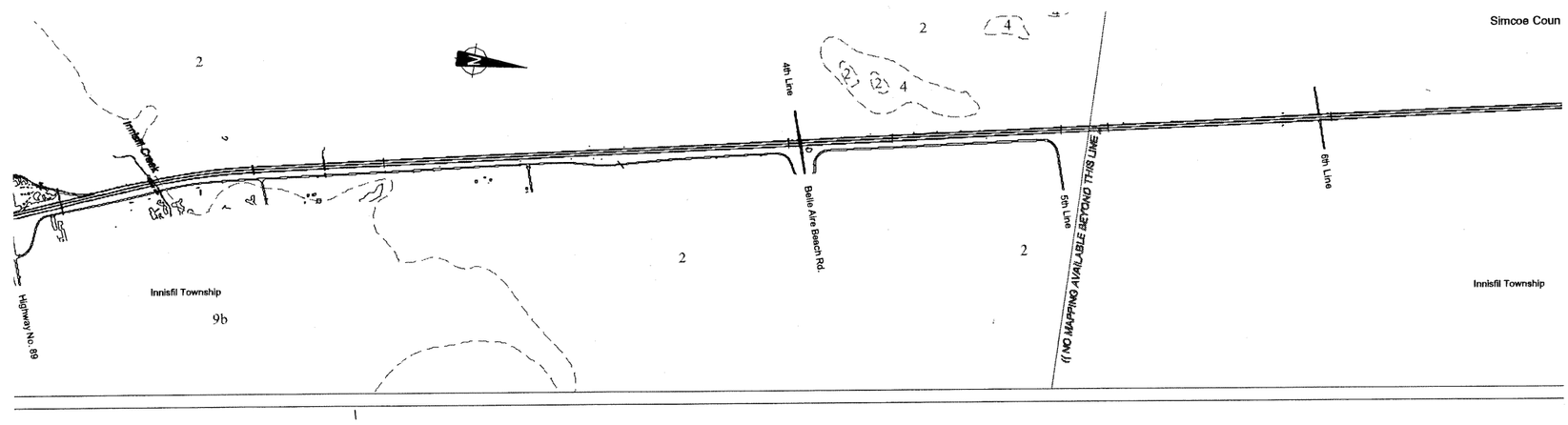
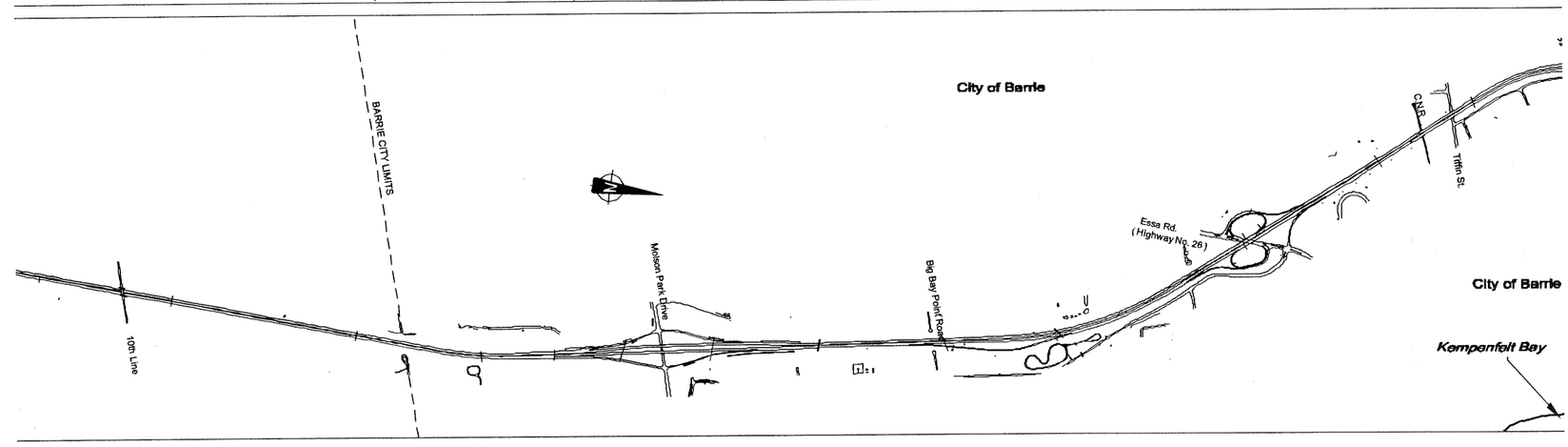
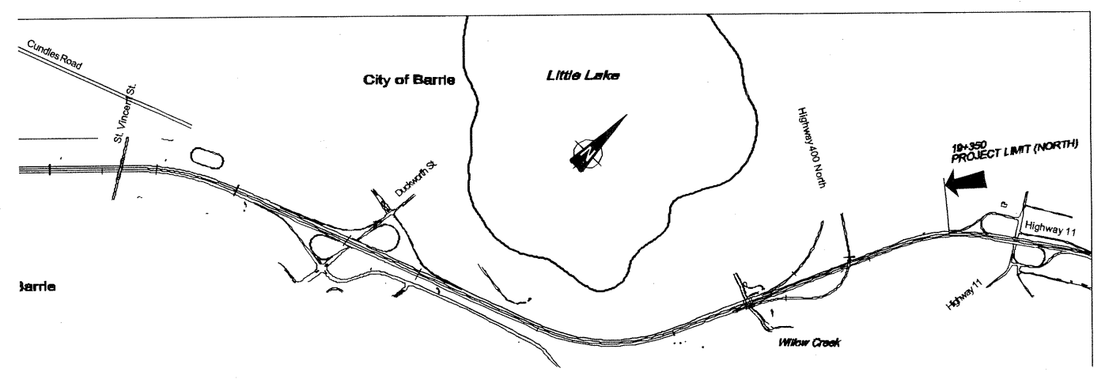
REFERENCE

CHAPMAN, L.J., AND PUTNAM, D.F.
1984: PHYSIOGRAPHY OF SOUTHERN (
ONTARIO GEOLOGICAL SURVEY
SCALE 1:600 000.







LEGEND

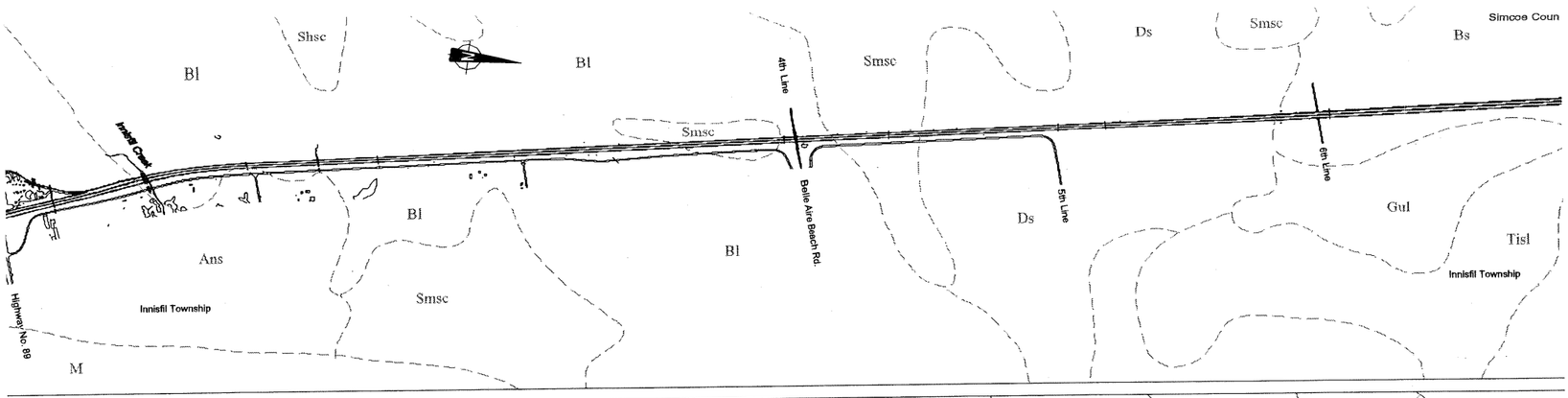
CENOZOIC
QUATERNARY

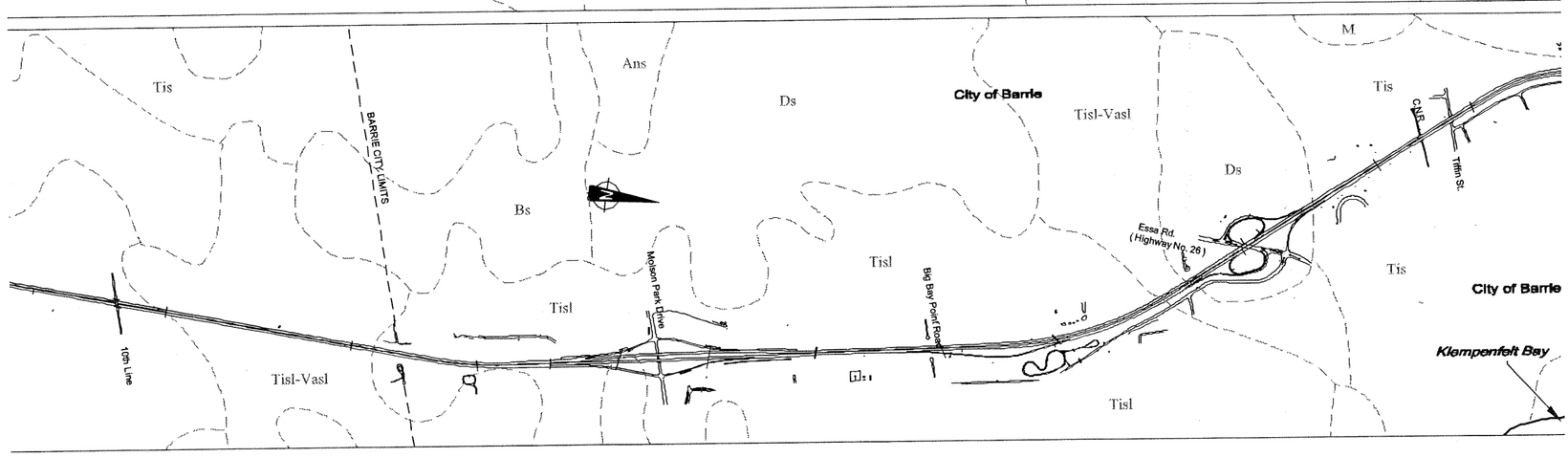
Pleistocene

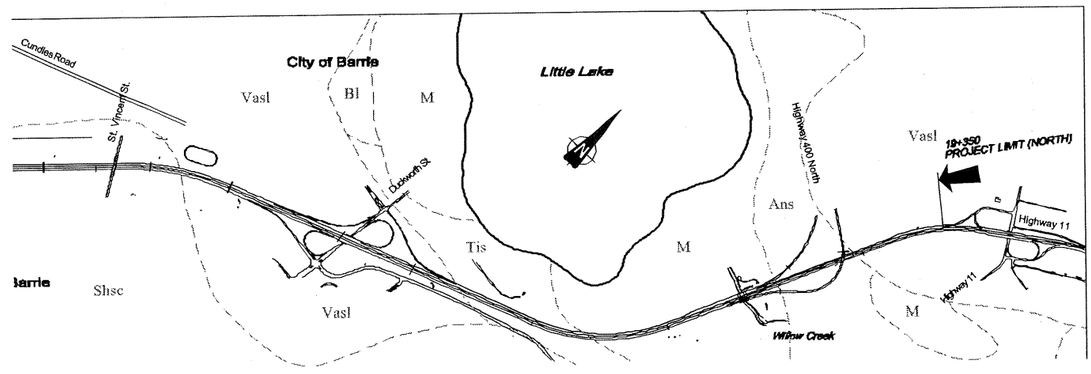
9	Glaciolacustrine shallow water; sand 9b Lake Algonquin deposits

4

2

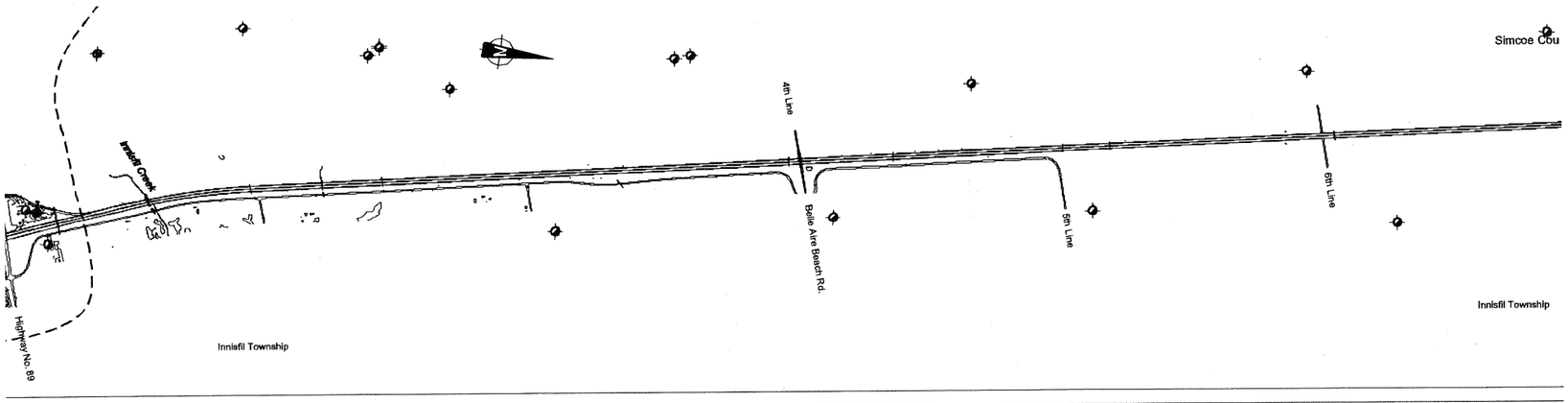


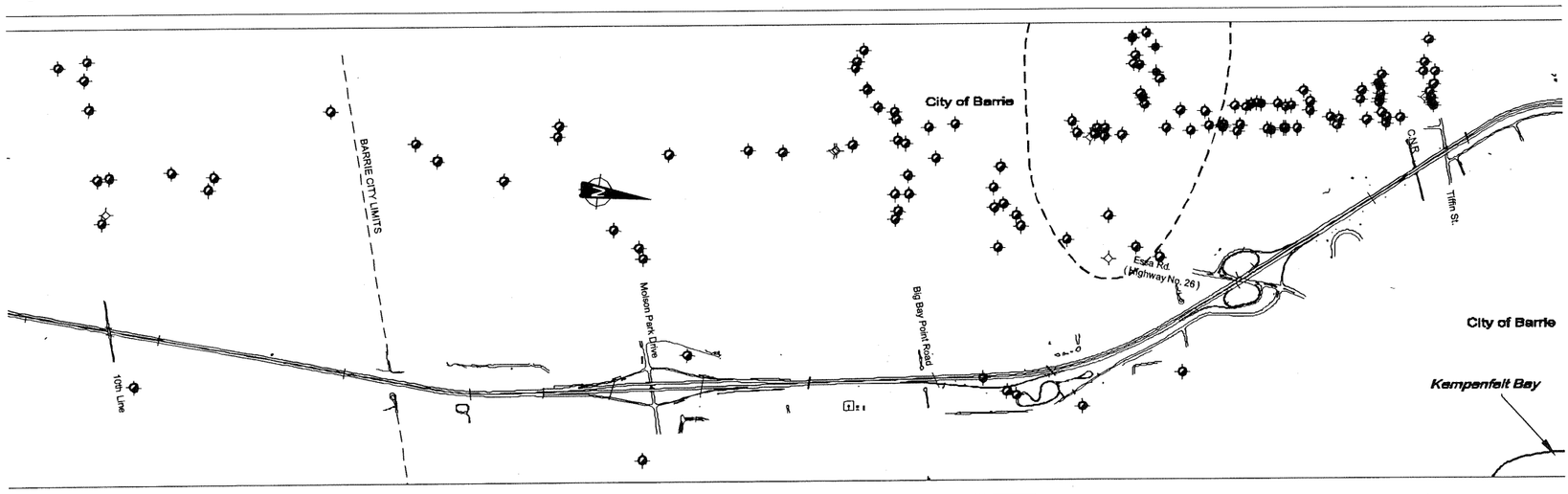


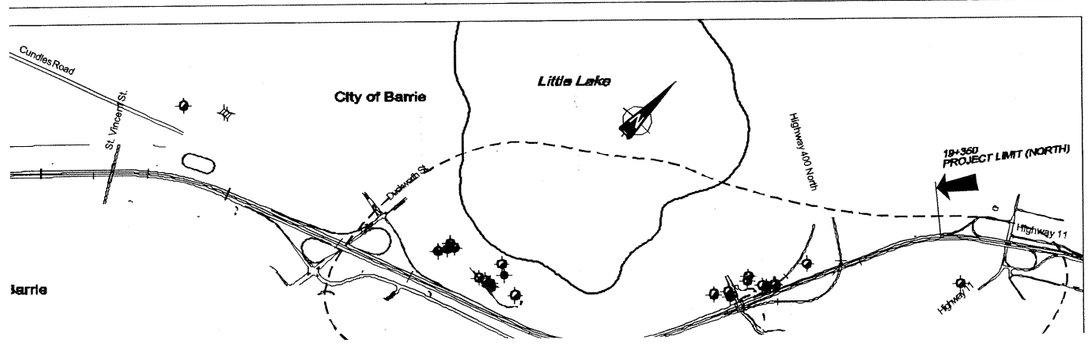


LEGEND

Map Symbol	Soil Series	Soil Type
Ans	ALLISTON	sandy loam
Bl	BONDHEAD	loam
Bs	BONDHEAD	sandy loam
Bs-b	BONDHEAD	sandy loam stoney phase
Ds	DUNDONALD	sandy loam
Gul	GUERIN	loam







SCALE 1:50,000

THIS DRAWING IS TO BE READ IN CONJU
WITH ACCOMPANYING TEXT.

LEGEND

(MOE WATER WELL LOCATION AND DES

- FLOWING WELLS (ARTESIAN CONDI
- WATER TABLE LESS THAN 1.5 METR
- WATER TABLE GREATER THAN 1.5 M
- DRY
- INTERPRETED AREA OF HIGH WATE

APPENDIX A

SELECTED SITE PHOTOGRAPHS

SITE PHOTOGRAPHS - GWP 30-95-00

Photo 1

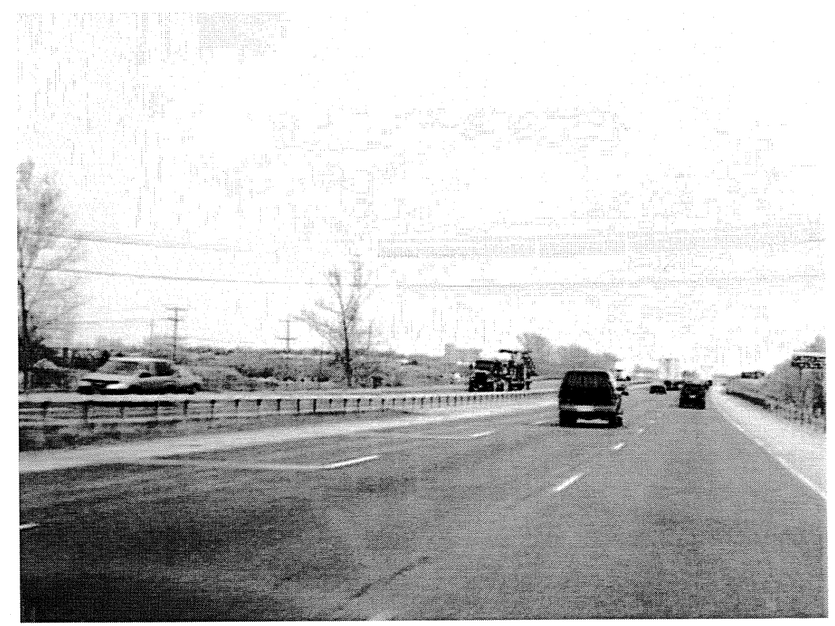


Photo 2

SITE PHOTOGRAPHS - GWP 30-95-00

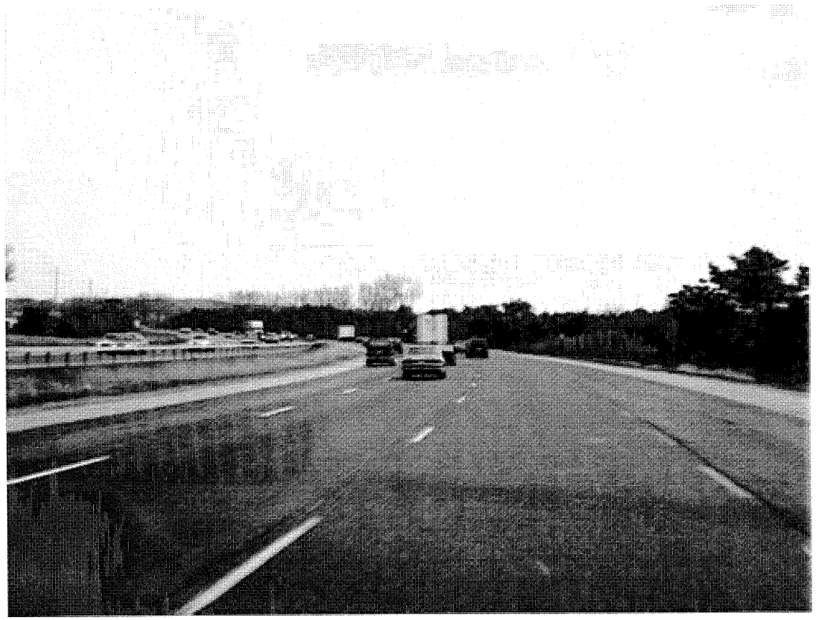


Photo 3

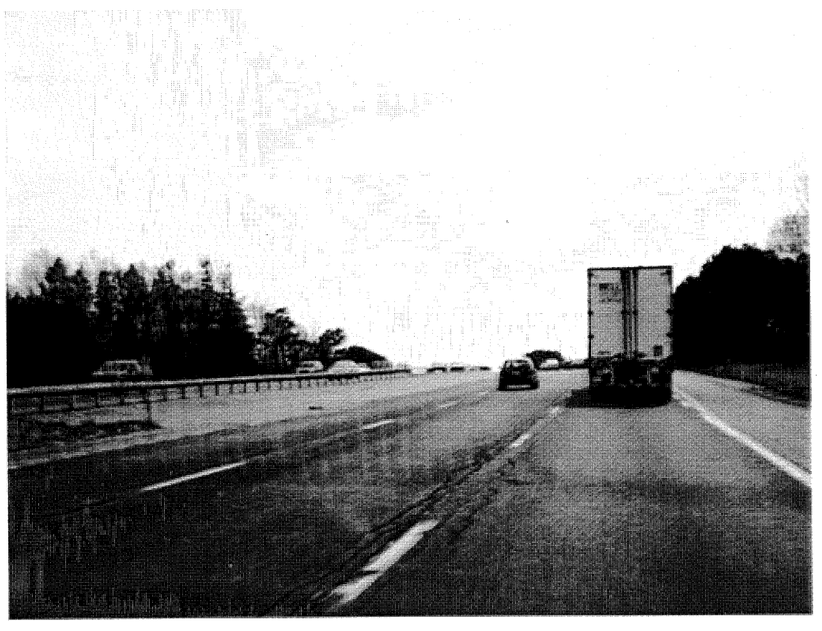


Photo 4

APPENDIX B

PAVEMENT DESIGN CALCULATIONS

Table 1

Equivalent Single Axle Load Calculations

November 8, 2002

Project No : 001-3203
Project Name : CS/GWP30/Hwy 400
Description : Highway 400 Planning Study

	OPAC Designation	2006	2006	Year 2011	2026
Analysis period	Ap		0	5	15
Average annual daily traffic, two directions	AADT	110,700	110,700	129,771	161,649
Average rate of increase in traffic (%)			0.000	3.230	1.450
Directional split	DF	0.5	0.5	0.5	0.5
Truck fraction of total traffic	T	0.12	0.12	0.12	0.12
Lane distribution factor	LDF	0.6	0.6	0.6	0.6
Truck Factor, equivalent standard axle loads per truck	TF	2	2	2	2
Daily ESAL's	ESALs/Day	7,970	7,970	9,344	11,596
Days per year for truck traffic	Days	365	365	365	365
ESAL's for analysis period	Nf			15,798,937	57,320,598
Design period, years	Y			5	13
Total ESAL's for design period	N				64,764,399
	Total Nf				73,119,535
					SN
					181.9

	OPAC Designation	2006	2006	Year 2011	2026
Analysis period	Ap		0	5	15
Average annual daily traffic, two directions	AADT	74,800	74,800	86,503	107,512
Average rate of increase in traffic (%)			0.000	2.950	1.460
Directional split	DF	0.5	0.5	0.5	0.5
Truck fraction of total traffic	T	0.12	0.12	0.12	0.12
Lane distribution factor	LDF	0.6	0.6	0.6	0.6
Truck Factor, equivalent standard axle loads per truck	TF	2	2	2	2
Daily ESAL's	ESALs/Day	5,386	5,386	6,228	7,741
Days per year for truck traffic	Days	365	365	365	365
ESAL's for analysis period	Nf			10,597,636	38,240,407
Design period, years	Y			5	13
Total ESAL's for design period	N				43,260,835
	Total Nf				48,838,043

N:\Notes\3200\01-3200 (CS - OAC) 01-331 Consultant\[AASHTO Results - Non-Linear ESALs]

Table 2

AASHTO Method of Pavement Design

November 8, 2002

Project No : 001-3203
Project Name : CS/GWP30/Hwy 400
Description : Highway 400 Planning Study

Proposed Pavement Structure	**Required Pavement**
Terminal Sevicenbility Index	2.5
Desired Initial Serviceability Index	4.5
Allowable Total Loss in Serviceability Index	2.0
Desired Reliability (per cent)	95.0
Standard Normal Deviate for Desired Reliability	-1.645
Overall Standard Deviation	0.45
Estimated Elastic Modulus for Subgrade (MPa):	35
Estimated Single Axle Load Applications	64,764,399

Loss In Serviceability due to Environmental Causes during Initial Period

Initial Design Period (years)	18.0
Analysis Period	20.0
1) Swelling	
Swell Rate Constant	0.00
Potential Vertical Rise of Natural Soils	0.0
Swell Probability	0.0
Loss in Serviceability Index due to Swell (PSIsw)	**0.000**
2) Heave	
Drainage Quality	
Frost Heave Rate (mm/day)	0.0
Maximum Potential Serviceability Loss	0.0
Heave Probability	0.0
Loss in Serviceability Index due to Heave (PSIfh)	**0.000**
Total Loss in Serviceability due to Environment	**0.000**
Allowable Serviceability Loss due to Traffic	**2.000**
Design Structural Number for Subgrade (in)	**7.16**
Design Structural Number for Subgrade (mm)	**182**
$\log_{10}W_{18}$	7.811
	7.811

Pavement Structure	Component Thickness (mm)	GBE	Layer Coefficient	Drainage Coefficient
Asphaltic Concrete	240	2.00	0.42	1.00
Granular A Base	200	1.00	0.14	1.00
Granular B Subbase (Type I)	700	0.67	0.09	1.00
	0	0.00	0.00	0.00
	0	0.00	0.00	0.00
	0	0.00	0.00	0.00
	0	0.00	0.00	0.00

Granular Base Equivalent (mm)	**1149**
Structural Number (mm)	**192**

Table 3
AASHTO Method of Pavement Design

Project No : 001-3203
Project Name : CS/GWP30/Hwy 400
Description : Highway 400 Planning Study

November 8, 2002

Proposed Pavement Structure	**Required Pavement**
Terminal Serviceability Index	2.5
Desired Initial Serviceability Index	4.5
Allowable Total Loss in Serviceability Index	2.0
Desired Reliability (per cent)	95.0
Standard Normal Deviate for Desired Reliability	-1.645
Overall Standard Deviation	0.45
Estimated Elastic Modulus for Subgrade (MPa):	35
Estimated Single Axle Load Applications	43,120,000
Loss in Serviceability due to Environmental Causes during Initial Period	
Initial Design Period (years)	18.0
Analysis Period	20.0
1) Swelling	
Swell Rate Constant	0.00
Potential Vertical Rise of Natural Soils	0.0
Swell Probability	0.0
Loss In Serviceability Index due to Swell (PSIsw)	**0.000**
2) Heave	
Drainage Quality	
Frost Heave Rate (mm/day)	0.0
Maximum Potential Serviceability Loss	0.0
Heave Probability	0.0
Loss in Serviceability Index due to Heave (PSIfh)	**0.000**
Total Loss in Serviceability due to Environment	**0.000**
Allowable Serviceability Loss due to Traffic	**2.000**
Design Structural Number for Subgrade (in)	**6.81**
Design Structural Number for Subgrade (mm)	**173**
$\log_{10}W_{18}$	7.635
	7.635

Pavement Structure	Component Thickness (mm)	GBE	Layer Coefficient	Drainage Coefficient
Asphaltic Concrete	240	2.00	0.42	1.00
Granular A Base	200	1.00	0.14	1.00
Granular B Subbase (Type I)	500	0.67	0.09	1.00
	0	0.00	0.00	0.00
	0	0.00	0.00	0.00
	0	0.00	0.00	0.00
	0	0.00	0.00	0.00
Granular Base Equivalent (mm)	**1015**			
Structural Number (mm)	**174**			

Table 4
Equivalent Single Axle Load Calculations

Project No : 001-3203
Project Name : CS/GWP 30/Hwy 400
Description : Highway 400 Planning Study

November 8, 2002

Highway 89	**OPAC Designation**	2006	2006	**Year** 2011	2026
Analysis period	Ap		0	5	15
Average annual daily traffic, two directions	AADT	6,600	6,600	7,600	11,604
Average rate of increase in traffic (%)			0.000	2.862	2.862
Directional split	DF	1	1	1	1
Truck fraction of total traffic	T	0.12	0.12	0.12	0.12
Lane distribution factor	LDF	1	1	1	1
Truck Factor, equivalent standard axle loads per truck	TF	2	2	2	2
Daily ESAL's	ESALs/Day	1,584	1,584	1,824	2,785
Days per year for truck traffic	Days	365	365	365	365
ESAL's for analysis period	Nf			3,109,800	12,617,289
Design period, years	Y			5	13
Total ESAL's for design period	N				13,740,770
	Total Nf				15,727,089

Innisfil Beach Road	**OPAC Designation**	2006	2006	**Year** 2011	2026
Analysis period	Ap		0	5	15
Average annual daily traffic, two directions	AADT	6,200	6,200	7,600	13,998
Average rate of increase in traffic (%)			0.000	4.156	4.156
Directional split	DF	1	1	1	1
Truck fraction of total traffic	T	0.15	0.15	0.15	0.15
Lane distribution factor	LDF	1	1	1	1
Truck Factor, equivalent standard axle loads per truck	TF	2	2	2	2
Daily ESAL's	ESALs/Day	1,860	1,860	2,280	4,200
Days per year for truck traffic	Days	365	365	365	365
ESAL's for analysis period	Nf			3,777,750	17,737,710
Design period, years	Y			5	13
Total ESAL's for design period	N				18,543,221
	Total Nf				21,515,460

Molson Park Drive	**OPAC Designation**	2006	2006	**Year** 2011	2026
Analysis period	Ap		0	5	15
Average annual daily traffic, two directions	AADT	10,000	10,000	11,100	15,181
Average rate of increase in traffic (%)			0.000	2.109	2.109
Directional split	DF	1	1	1	1
Truck fraction of total traffic	T	0.11	0.11	0.11	0.11
Lane distribution factor	LDF	1	1	1	1
Truck Factor, equivalent standard axle loads per truck	TF	2	2	2	2
Daily ESAL's	ESALs/Day	2,200	2,200	2,442	3,340
Days per year for truck traffic	Days	365	365	365	365
ESAL's for analysis period	Nf			4,235,825	15,827,554
Design period, years	Y			5	13
Total ESAL's for design period	N				17,669,049
	Total Nf				20,063,379

Essa Road	**OPAC Designation**	2006	2006	**Year** 2011	2026
Analysis period	Ap		0	5	15
Average annual daily traffic, two directions	AADT	17,000	17,000	20,100	29,940
Average rate of increase in traffic (%)			0.000	2.692	2.692
Directional split	DF	1	1	1	1
Truck fraction of total traffic	T	0.1	0.1	0.1	0.1
Lane distribution factor	LDF	1	1	1	1
Truck Factor, equivalent standard axle loads per truck	TF	2	2	2	2
Daily ESAL's	ESALs/Day	3,520	3,520	4,020	5,988
Days per year for truck traffic	Days	365	365	365	365
ESAL's for analysis period	Nf			6,880,250	27,396,690
Design period, years	Y			5	13
Total ESAL's for design period	N				30,001,528
	Total Nf				34,276,940

Dunlop Street	OPAC Designation	2006	2006	Year 2011	2026
Analysis period	Ap		0	5	15
Average annual daily traffic, two directions	AADT	10,800	10,800	12,600	20,003
Average rate of increase in traffic (%)			0.000	3.131	3.131
Directional split	DF	1	1	1	1
Truck fraction of total traffic	T	0.11	0.11	0.11	0.11
Lane distribution factor	LDF	1	1	1	1
Truck Factor, equivalent standard axle loads per truck	TF	2	2	2	2
Daily ESAL's	ESALs/Day	2,376	2,376	2,772	4,402
Days per year for truck traffic	Days	365	365	365	365
ESAL's for analysis period	Nf			4,697,550	19,638,369
Design period, years	Y			5	13
Total ESAL's for design period	N				21,201,899
	Total Nf				24,335,919

Bayfield Street	OPAC Designation	2006	2006	Year 2011	2026
Analysis period	Ap		0	5	15
Average annual daily traffic, two directions	AADT	14,000	14,000	14,600	16,559
Average rate of increase in traffic (%)			0.000	0.843	0.843
Directional split	DF	1	1	1	1
Truck fraction of total traffic	T	0.1	0.1	0.1	0.1
Lane distribution factor	LDF	1	1	1	1
Truck Factor, equivalent standard axle loads per truck	TF	2	2	2	2
Daily ESAL's	ESALs/Day	2,800	2,800	2,920	3,312
Days per year for truck traffic	Days	365	365	365	365
ESAL's for analysis period	Nf			5,219,500	17,059,411
Design period, years	Y			5	13
Total ESAL's for design period	N				19,880,400
	Total Nf				22,278,911

Duckworth Street	OPAC Designation	2006	2006	Year 2011	2026
Analysis period	Ap		0	5	15
Average annual daily traffic, two directions	AADT	12,000	12,000	14,800	22,231
Average rate of increase in traffic (%)			0.000	3.131	3.131
Directional split	DF	1	1	1	1
Truck fraction of total traffic	T	0.09	0.09	0.09	0.09
Lane distribution factor	LDF	1	1	1	1
Truck Factor, equivalent standard axle loads per truck	TF	2	2	2	2
Daily ESAL's	ESALs/Day	2,160	2,160	2,520	4,002
Days per year for truck traffic	Days	365	365	365	365
ESAL's for analysis period	Nf			4,270,500	17,853,063
Design period, years	Y			5	13
Total ESAL's for design period	N				19,274,454
	Total Nf				22,123,563

maximum	30,001,528
minimum	13,740,770
average	20,044,474

Table 5

AASHTO Method of Pavement Design

November 8, 2002

Project No : 001-3203
Project Name : CS/GWP 30/Hwy 400
Description : Highway 400 Planning Study

Proposed Pavement Structure	**Required Pavement**
Terminal Serviceability Index	2.5
Desired Initial Serviceability Index	4.5
Allowable Total Loss in Serviceability Index	2.0
Desired Reliability (per cent)	95.0
Standard Normal Deviate for Desired Reliability	-1.645
Overall Standard Deviation	0.45
Estimated Elastic Modulus for Subgrade (MPa):	35
Estimated Single Axle Load Applications	15,000,000
Loss in Serviceability due to Environmental Causes during Initial Period	
Initial Design Period (years)	18.0
Analysis Period	20.0
1) Swelling	
Swell Rate Constant	0.00
Potential Vertical Rise of Natural Soils	0.0
Swell Probability	0.0
Loss in Serviceability Index due to Swell (PSIsw)	**0.000**
2) Heave	
Drainage Quality	
Frost Heave Rate (mm/day)	0.0
Maximum Potential Serviceability Loss	0.0
Heave Probability	0.0
Loss in Serviceability Index due to Heave (PSIfh)	**0.000**
Total Loss in Serviceability due to Environment	**0.000**
Allowable Serviceability Loss due to Traffic	**2.000**
Design Structural Number for Subgrade (in)	**5.95**
Design Structural Number for Subgrade (mm)	**151**
$\log_{10} W_{18}$	7.176
	7.176

Pavement Structure	Component Thickness (mm)	GBE	Layer Coefficient	Drainage Coefficient
Asphaltic Concrete	190	2.00	0.42	1.00
Granular A Base	200	1.00	0.14	1.00
Granular B Subbase (Type I)	450	0.67	0.09	1.00
	0	0.00	0.00	0.00
	0	0.00	0.00	0.00
	0	0.00	0.00	0.00
	0	0.00	0.00	0.00

Granular Base Equivalent (mm)	**882**
Structural Number (mm)	**148**

Table 6

AASHTO Method of Pavement Design

November 8, 2002

Project No : 001-3203
Project Name : CS/GWP 30/Hwy 400
Description : Highway 400 Planning Study

Proposed Pavement Structure	**Required Pavement**
Terminal Serviceability Index	2.5
Desired Initial Serviceability Index	4.5
Allowable Total Loss in Serviceability Index	2.0
Desired Reliability (per cent)	95.0
Standard Normal Deviate for Desired Reliability	-1.645
Overall Standard Deviation	0.45
Estimated Elastic Modulus for Subgrade (MPa):	35
Estimated Single Axle Load Applications	20,000,000

Loss in Serviceability due to Environmental Causes during Initial Period	
Initial Design Period (years)	18.0
Analysis Period	20.0
1) Swelling	
Swell Rate Constant	0.00
Potential Vertical Rise of Natural Soils	0.0
Swell Probability	0.0
Loss in Serviceability Index due to Swell (PSIsw)	**0.000**
2) Heave	
Drainage Quality	
Frost Heave Rate (mm/day)	0.0
Maximum Potential Serviceability Loss	0.0
Heave Probability	0.0
Loss in Serviceability Index due to Heave (PSIfh)	**0.000**
Total Loss in Serviceability due to Environment	**0.000**
Allowable Serviceability Loss due to Traffic	**2.000**
Design Structural Number for Subgrade (in)	6.18
Design Structural Number for Subgrade (mm)	**157**
$\log_{10}W_{18}$	7.301
	7.301

Pavement Structure	Component Thickness (mm)	GBE	Layer Coefficient	Drainage Coefficient
Asphaltic Concrete	190	2.00	0.42	1.00
Granular A Base	200	1.00	0.14	1.00
Granular B Subbase (Type I)	550	0.67	0.09	1.00
	0	0.00	0.00	0.00
	0	0.00	0.00	0.00
	0	0.00	0.00	0.00
	0	0.00	0.00	0.00

Granular Base Equivalent (mm)	**949**
Structural Number (mm)	**157**

Table 7

AASHTO Method of Pavement Design

November 8, 2002

Project No : 001-3203
Project Name : CS/GWP 30/Hwy 400
Description : Highway 400 Planning Study

Proposed Pavement Structure	**Required Pavement**
Terminal Serviceability Index	2.5
Desired Initial Serviceability Index	4.5
Allowable Total Loss in Serviceability Index	2.0
Desired Reliability (per cent)	95.0
Standard Normal Deviate for Desired Reliability	-1.645
Overall Standard Deviation	0.45
Estimated Elastic Modulus for Subgrade (MPa):	35
Estimated Single Axle Load Applications	50,000,000

Loss in Serviceability due to Environmental Causes during Initial Period	
Initial Design Period (years)	18.0
Analysis Period	20.0
1) Swelling	
Swell Rate Constant	0.00
Potential Vertical Rise of Natural Soils	0.0
Swell Probability	0.0
Loss in Serviceability Index due to Swell (PSIsw)	**0.000**
2) Heave	
Drainage Quality	
Frost Heave Rate (mm/day)	0.0
Maximum Potential Serviceability Loss	0.0
Heave Probability	0.0
Loss in Serviceability Index due to Heave (PSIfh)	**0.000**
Total Loss in Serviceability due to Environment	**0.000**
Allowable Serviceability Loss due to Traffic	**2.000**
Design Structural Number for Subgrade (in)	6.94
Design Structural Number for Subgrade (mm)	**176**
$\log_{10}W_{18}$	7.699
	7.699

Pavement Structure	Component Thickness (mm)	GBE	Layer Coefficient	Drainage Coefficient
Asphaltic Concrete	240	2.00	0.42	1.00
Granular A Base	200	1.00	0.14	1.00
Granular B Subbase (Type I)	550	0.67	0.09	1.00
	0	0.00	0.00	0.00
	0	0.00	0.00	0.00
	0	0.00	0.00	0.00
	0	0.00	0.00	0.00

Granular Base Equivalent (mm)	**1049**
Structural Number (mm)	**178**

Ontario

Ministry of Transportation
Southwestern Region
Planning and Design Section

HIGHWAY 400 PLANNING AND PRELIMINARY DESIGN STUDY

FROM 1 KM SOUTH OF HIGHWAY 89 NORTHERLY 30 KM
TO THE JUNCTION AT HIGHWAY 11

COUNTY OF SIMCOE
TOWN OF BRADFORD WEST GWILLIMBURY
TOWN OF INNISFIL
CITY OF BARRIE
TOWNSHIP OF SPRINGWATER

G.W.P. 30-95-00

APPENDIX K

PRELIMINARY FOUNDATION INVESTIGATION & DESIGN REPORTS

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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT HIGHWAY 89 UNDERPASS STRUCTURE SITE 30-256 HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11 G.W.P. 30-95-00

Submitted to:

URS Cole, Sherman
75 Commerce Valley Drive East
Thornhill, Ontario
L3T 7N9

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January 2002

001-1143F-1

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PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
HIGHWAY 89 UNDERPASS
STRUCTURE SITE 30-256
HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00

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LIST OF DRAWINGS

Drawing 1 Highway 89 Underpass, Highway 400, Borehole Location Plan

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Figure 1 Grain Size Distribution Test Result, Clayey Silt Till

1.0 INTRODUCTION

Golder Associates Ltd. has been retained by URS Cole, Sherman (Cole, Sherman) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the ultimate widening of Highway 400 from 1 km south of Highway 89, northerly 30 km to Highway 11, in Simcoe County, Ontario. Foundation engineering services are required for the widening and / or replacement of eighteen existing overpass and underpass structures, as well as five structural culverts.

This report addresses the replacement of the existing Highway 89 underpass structure. A foundation investigation has been carried out, in which two boreholes were advanced and in-situ and laboratory testing was conducted, to determine the subsurface conditions at the site for this preliminary design study.

The terms of reference for the scope of work are outlined in Golder Associates' Proposal No. P01-1192, dated June 2000.

2.0 SITE DESCRIPTION

The existing Highway 89 underpass structure is located about 11 km north of the Simcoe Road 88 interchange, and about 7 km south of the Innisfil Beach Road interchange, in the Town of Innisfil, Simcoe County. The MTO has designated this underpass as Structure Site No. 30-256.

At this existing structure, the Highway 400 grade is at about Elevation 229 m to 229.5 m. Highway 89 has been constructed in fill, with approach embankments up to about 7 m in height. The Highway 89 grade over Highway 400 is at about Elevation 236 m.

According to the general layout drawing for this existing single-span underpass structure (provided by Morrison Hershfield, the structural designers for this preliminary design study), the abutments and associated wing walls are supported on spread footings which are founded at about Elevation 227.7 m.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at this site in December 2000, at which time two boreholes were drilled. Boreholes B1-1 and B1-2 were advanced on the east and west sides of Highway 400 to depths of 28 m and 37 m, respectively, below the Highway 400 grade.

The investigation was carried out using bombardier-mounted D-50 and B-57 drill rigs supplied and operated by Master Soil Investigations Ltd. of Weston, Ontario. Borehole B1-1 was advanced using solid stem augers; this borehole was terminated at 28 m depth due to "tightening" of the augers in the soil. In Borehole B1-2, hollow stem augers were used initially to 7.5 m depth, then the borehole was advanced using casing with wash boring. Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. The water levels in the open boreholes were observed throughout the drilling operations, and a piezometer was installed in Borehole B1-2 to permit monitoring of the groundwater levels at the site.

The field work was supervised on a full-time basis by members of our staff who located the boreholes in the field, directed the drilling, sampling, and in-situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder Associates' laboratory in Mississauga for further examination. Index and classification tests consisting of water content determinations, Atterberg Limits tests and grain size distribution analyses were carried out on selected soil samples.

The borehole locations and elevations were surveyed by Callon Dietz, Ontario Land Surveyors. The borehole elevations are referenced to geodetic datum, and the northing and easting coordinates are referenced to the MTM NAD83 survey system. The borehole locations, together with elevations and northing and easting coordinates, are shown on the attached Drawing 1.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

This 30 km section of Highway 400 traverses, from south to north, the following physiographic regions as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, Third Edition, 1984): the Simcoe Lowlands; the Peterborough Drumlin Field; a second lobe of the Simcoe Lowlands; and the Simcoe Uplands. Along Highway 400, the Simcoe Lowlands are present from the southern limit of the project to just south of Innisfil Creek (about 1 km north of Highway 89), and again from Essa Road (Simcoe Road 30, formerly Highway 27) to about 1 km north of Dunlop Street (Simcoe Road 90, formerly Highway 90). The Peterborough Drumlin Field occupies the belt between these lobes of the Simcoe Lowlands, extending from just south of Innisfil Creek to Essa Road. The Simcoe Uplands extend from about 1 km north of Dunlop Street to beyond the northern limit of the project at Highway 11.

The two sections where Highway 400 crosses the Simcoe Lowlands consist of two lobes of a sand plain which include the shores of Kempenfelt Bay, the Nottawasaga River and Innisfil Creek. The surficial soils of these sections of the Simcoe Lowlands consist primarily of sand, although silt, clay or peat may be found in low-lying areas. The Highway 89 site is located within the southern of these two lobes of the Simcoe Lowlands.

The surficial soils in the Peterborough Drumlin Field consist primarily of gravelly sand till or sand and gravel deposits. Drumlins (glacially-shaped hills) are more frequent in the southern portion of the section of the Peterborough Drumlin Field traversed by Highway 400. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins.

The surficial soils in the Simcoe Uplands physiographic region are primarily sandy silt till deposits, known to contain occasional boulders. Low-lying areas may be infilled with shallow sand and gravel deposits, which are shoreline deposits of a former glacial lake that once flooded the area.

4.2 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 given on the Record of Borehole sheets and Figure 1. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

Boreholes B1-1 and B1-2 were advanced on the east and west sides of Highway 400, respectively, from approximately Highway 400 grade. The locations and ground surface elevations for these borings are shown on the attached Drawing 1.

In summary, the Highway 89 embankment fill at the site overlies a deposit of silty sand to sand silt, underlain by a silty clay deposit, which in turn overlies a lower silty sand deposit. A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Fill

About 1.4 m and 2.3 m of fill and/or reworked soil was encountered in the boreholes drilled at the north toe of the Highway 89 embankments on the east and west sides of Highway 400, respectively. The fill consists of silty sand, containing minor quantities of gravel, pockets of silty clay, and organics. The measured Standard Penetration Test (SPT) 'N' values ranged from 8 to 20 blows per 0.3 m of penetration, indicating that this fill has a loose to compact relative density.

4.2.2 Silty Sand to Sandy Silt

A silty sand to sandy silt deposit, 17.5 m to 19.5 m thick, was encountered below the fill in both boreholes at about Elevation 226 m. The base of the deposit was encountered between Elevation 209 m and 206.5 m. The silty sand to sandy silt contains trace clay and gravel. Layers of silty clay, 25 mm to 100 mm in thickness, were encountered within the silty sand to sandy silt in some of the samples. The grain size distribution test results obtained on representative samples of this deposit are shown on Figure 1.

The silty sand to sandy silt soils are wet, with measured natural moisture contents ranging from 19 to 21 per cent.

The measured SPT 'N' values ranged from 13 to 46 blows per 0.3 m of penetration, with an average of 25 blows per 0.3 m of penetration. The silty sand to sandy silt deposit has a compact to dense, but predominantly compact, relative density.

4.2.3 Silty Clay

A deposit of silty clay containing trace sand underlies the silty sand to sandy silt deposit. The top of the silty clay was encountered at about 20 m to 22 m depth (Elevation 209 m to 206.5 m). The deposit was fully penetrated in Borehole B1-2, where it was about 11.5 m thick. In Borehole B1-1, the silty clay is at least 8 m thick.

The natural moisture contents measured on samples of the silty clay ranged from 23 to 36 per cent. Atterberg Limits testing on two samples measured plastic limits of 15 and 18 per cent, liquid limits of 33 and 43 per cent, and plasticity indices of 18 and 25 per cent. The results of the Atterberg Limits testing indicate that the silty clay is inorganic and of low to intermediate plasticity.

The measured SPT 'N' values ranged from 6 to 40 blows, but were typically between 6 and 19 blows per 0.3 m of penetration. Field vane testing carried out within the softer portion of the deposit, below about 25 m in Borehole B1-2, measured undrained shear strengths of 80 kPa to 90 kPa. The results of the SPT and in-situ vane shear strength testing indicate that the silty clay deposit has a generally stiff to very stiff consistency.

4.2.4 Lower Silty Sand

A lower deposit of silty sand, containing thin silty clay layers, was encountered below 33.5 m depth (Elevation 195 m) in Borehole B1-2. The deposit was not fully penetrated by this boring, but it is at least 3.5 m thick. The recovered silty sand samples were wet, with a measured natural moisture content of 21 per cent on one sample.

This lower silty sand deposit is very dense, with measured SPT 'N' values of 52 and 72 blows per 0.3 m of penetration.

4.3 Groundwater Conditions

Recovered samples from both the upper silty sand to sandy silt deposit and the lower silty sand deposit were wet; both deposits comprise aquifers at the site, separated by the silty clay deposit. The water levels were observed in the open boreholes following drilling operations in December 2000. At that time, the measured water levels were at 2.3 m and 2.7 m depth, at or slightly below the top of the silty sand to sandy silt deposit in the boreholes. The groundwater level measured in the piezometer installed in Borehole B1-2 was at 1.8 m depth (Elevation 226.6 m) in January 2001, and at 1.3 m depth (Elevation 227.1 m) in March 2001.

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

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
HIGHWAY 89 UNDERPASS
STRUCTURE SITE 30-256
HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides preliminary foundation design recommendations for the replacement of the existing Highway 89 underpass structure, associated with the widening of Highway 400. The recommendations are preliminary only and are based on interpretation of the factual data obtained from a limited number of boreholes advanced during the subsurface investigation at this site. The interpretation and recommendations provided are intended for planning purposes only, to provide the information necessary at this stage of the study. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the planning of the project. Further foundation investigation will be required at this bridge site as part of the detailed design stage of the project.

It is understood that Highway 400 will be widened from its existing six-lane configuration to an interim configuration of eight lanes, and an ultimate configuration of ten lanes, and that an alternative for a twelve-lane express / collector system is under consideration between Molson Park Drive and Duckworth Street in Barrie. Throughout the project length, it is expected that the existing highway platform will be widened by between 13 m and 30 m, necessitating replacement of the existing Highway 89 underpass structure.

Based on the general layout drawing for the existing single-span structure, the abutments and associated wing walls are supported on spread footings founded at about Elevation 227.7 m. The Highway 400 grade is at about Elevation 229 m to 229.5 m, and Highway 89 has been constructed in fill with approach embankments up to about 7 m in height. The Highway 89 grade over Highway 400 is at about Elevation 236 m.

5.2 Bridge Foundation Options

The soils at the site consist of fill overlying a generally compact silty sand to sandy silt deposit, which is about 18 m to 20 m thick. This upper cohesionless deposit is water-bearing, with the measured groundwater level at about Elevation 227 m, near the top of the deposit and about 2 m to 2.5 m below Highway 400 grade. This silty sand to sandy silt overlies a generally stiff to very stiff silty clay deposit, which is in turn underlain by a lower, very dense silty sand deposit. The top of this very dense silty sand is at about Elevation 195 m.

Based on these subsurface conditions, it is recommended that the new structure be founded on spread footings placed on the upper silty sand to sandy silt stratum. The spread footings should be placed as high as possible, to minimize the requirements for dewatering and the potential for

disturbance to the founding soils due to the high groundwater level. In this regard, consideration could also be given to the use of perched abutments, founded on spread footings placed on a compacted granular pad within the approach fill.

Consideration could also be given to the use of deep foundations, such as steel "H" piles driven to found within the very dense, lower silty sand deposit at this site. It is noted that this lower silty sand deposit is almost 40 m below the Highway 89 grade.

Preliminary recommendations for spread footings, including perched abutments, and for driven steel "H" pile foundations are provided in the following sections.

5.3 Spread Footings

For preliminary design of the bridge abutment and pier footings, spread footings may be placed at a design founding level of Elevation 227 m, to be founded below the fill / reworked soil, on the generally compact silty sand to sandy silt deposit. Any associated wing wall or retaining wall footings may be stepped upward away from the abutments such that a minimum soil cover of 1.5 m is maintained above the underside of the footings; in this case, a well-compacted granular pad will be required for support of the wall footings. It is noted that the design founding level given herein is affected by the base of fill elevation as encountered in Borehole B1-2, drilled at the north toe of the Highway 89 embankment. This founding elevation is coincident with the groundwater level measured in Borehole B1-2 in March 2001. If, during the detailed design stage of subsurface investigation, native soil is encountered higher at the proposed foundation locations, consideration could be given to raising the design founding level to minimize the dewatering requirements and the potential for disturbance of the founding soils.

To avoid the difficulties associated with groundwater control and potential disturbance to the silty sand to sandy silt soils, consideration should also be given to the use of abutment footings perched on the embankment fill.

5.3.1 Axial Geotechnical Resistance

Spread footings placed on the properly prepared silty sand to sandy silt deposit at Elevation 227 m may be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 500 kPa, assuming a 4 m wide footing. The settlement of footings founded on the silty sand to sandy silt will be dependent on the footing size and configuration, on the degree of disturbance caused by construction activities, and on the applied loads. The majority of this settlement is

expected to take place during the construction period; for preliminary design purposes, the geotechnical resistance at Serviceability Limit States (SLS) may be taken as 250 kPa for a design founding level of Elevation 227 m. The geotechnical resistance at SLS will have to be reviewed following the detailed design stage of subsurface investigation, once the footing size, configuration and loadings are known.

For spread footings placed within the approach embankments on a compacted Granular 'A' core, a factored geotechnical resistance at ULS of 900 kPa may be assumed for preliminary design. The geotechnical resistance at SLS will depend on the thickness of Granular 'A' and the consistency and thickness of the underlying soils; a value of 350 kPa may be assumed for preliminary design.

The geotechnical resistances provided herein are given under the assumption that the loads will be applied perpendicular to the surface of the footings; where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the Ontario Highway Bridge Design Code (OHBDC).

5.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footing and the subsoils should be calculated in accordance with Section 6-8.4.3 of the OHBDC. The angle of friction between the concrete and the undisturbed founding soils should be taken as 24 degrees; the corresponding coefficient of friction, tan δ, would then be 0.45. Where "perched" footings are adopted, the angle of friction between the concrete footings and the compacted Granular 'A' pad should be taken as 30 degrees; the corresponding coefficient of friction would be 0.58.

5.3.3 Frost Protection

The footings should be provided with a minimum of 1.5 m of soil cover for frost protection.

5.4 Driven Steel H-Piles

Consideration could be given to supporting the abutments and pier of the replacement structure on steel H-piles driven to found within the very dense lower silty sand deposit, which was encountered below Elevation 195 m in Borehole B1-2. For preliminary design, a pile tip elevation of 190 m may be assumed.

5.4.1 Axial Geotechnical Resistance

For preliminary design of deep foundations, the factored axial resistance at ULS for steel HP 310 x 110 H-piles driven into the very dense lower silty sand may be taken as 900 kN. The axial resistance at SLS for 25 mm of settlement, for a single pile, may be taken as 800 kN.

To achieve the above design resistances, the piles should be driven to a tip elevation of at least 190 m. Provision should be made to re-tap selected piles to confirm the set after adjacent piles have been driven, in accordance with MTO's current Special Provision.

5.4.2 Resistance to Lateral Loads

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

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

$$k_h = \frac{n_h z}{B}$$

where
n_h is the constant of subgrade reaction
z is the depth (m)
B is the pile diameter (m)

Based on the borehole data, the subsoils in front of the upper portion of the piles will consist of embankment fill overlying generally compact silty sand to sand silt soils. For the embankment fill, the range in value of n_h may be taken as 5 MPa/m to 10 MPa/m in the structural analysis. For the water-bearing, generally compact silty sand to sandy silt deposit, the range in value of n_h may also be taken as 5 MPa/m to 10 MPa/m. These values will have to be confirmed following the detailed design stage of the subsurface investigation.

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

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

5.4.3 Frost Protection

All pile caps should be provided with 1.5 m of soil cover for frost protection.

5.5 Lateral Earth Pressures

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

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the abutments and walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00

- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the abutment wall, in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be used in accordance with OPSS 501.06.

- The granular fill may be placed either in a zone with width equal to at least 1.5 m behind the back of the stem (Case I from OHBDC Figure 6-7.4.1) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II from OHBDC Figure 6-7.4.4).

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

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

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

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

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

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

5.6 Embankment Design

Based on the topographic information on the Engineering and Title Records plates and on site reconnaissance, the existing Highway 89 embankment side slopes are formed at a gradient of about 2 horizontal to 1 vertical (2H:1V). If any widening of the local road embankment will be required, the new side slopes should be formed at a maximum gradient of 2H:1V. The construction of the embankment should be carried out using conventional fill placement and compaction practices, and benching of the existing embankment side slopes should be carried out to key in the new fill.

5.7 Design and Construction Considerations

5.7.1 Groundwater Control

The abutment and pier footing excavations will extend close to water-bearing silty sand to sandy silt soils (unless the abutment footings are designed to be perched on a granular pad within the approach embankment fill). A dewatering scheme will be necessary to facilitate footing excavation and concrete placement in dry conditions. In this regard, it will be necessary to lower the groundwater level to at least 0.5 m below the design founding level prior to excavation and construction of the footings.

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

5.7.2 Excavation

Footing excavations will extend a minimum of 1.5 m below lowest surrounding grade, through existing fill and into generally compact, water-bearing silty sand to sandy silt soils. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. With proper dewatering or a temporary cut-off wall, the silty sand to sandy silt deposits at this site would be classified as Type 3 soil. Temporary open-cut slopes in the dewatered soil should be maintained no steeper than 1 horizontal to 1 vertical (1H:1V). Where space restrictions dictate, the foundation works could also be carried out within a braced excavation.

5.7.3 Settlement

Foundation loading will result in deformation of the ground and consequent settlement of the foundation elements and superstructure. Differential settlement could occur between the abutments and pier locations, depending on the variability and consistency/relative density of the foundation soils, on the types of foundations selected, and on the degree of disturbance caused by activities during construction. The potential for differential settlement should be reassessed during the detailed design stage, once the proposed bridge configuration is established.

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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

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 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 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	c_u, s_u 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 (LV-laboratory vane test)
γ	unit weight

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

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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

RECORD OF BOREHOLE No B1-1

1 OF 2 **METRIC**

PROJECT 001-1143F
W.P. 30-95-00
DIST SW HWY 400
DATUM Geodetic

LOCATION N 4895635.8; E 292452.1
BOREHOLE TYPE 108mm DIAMETER SOLID STEM AUGERS
DATE Dec.14-18/2000

ORIGINATED BY PKS
COMPILED BY LCC
CHECKED BY ASP

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_p / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_l — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
228.9	GROUND SURFACE										
0.0	Silty Sand, trace to some gravel with pockets of clay (Fill) Loose to compact Moist Brown		1	SS	8						
227.7			2	SS	20		228				
1.2	Silty Sand to Sandy Silt, trace gravel, trace clay, containing silty clay layers Compact to dense Wet Brown to grey		3	SS	13		227				
			4	SS	39		226				0 55 43 2
			5	SS	46						
	Becoming grey below 3.8m depth		6	SS	27		225				
			7	SS	32		224				0 24 74 2
							223				
			8	SS	27		222				
			9	SS	30		221				
							220				
			10	SS	37		219				
	Increasing frequency of silty clay layers, 25mm to 100mm in thickness, below 10.7m depth.		11	SS	33		218				
							217				
			12	SS	34		216				
			13	SS	22		215				
							214				

Continued Next Page

+³, ×³: Numbers refer to Sensitivity O³% STRAIN AT FAILURE

ON_MOT 001143F.GPJ ON_MOT.GDT 14/1/02

RECORD OF BOREHOLE No B1-1

2 OF 2 **METRIC**

PROJECT 001-1143F
W.P. 30-95-00
DIST SW HWY 400
DATUM Geodetic
LOCATION N 4895635.8; E 292452.1
BOREHOLE TYPE 108mm DIAMETER SOLID STEM AUGERS
DATE Dec.14-18/2000
ORIGINATED BY PKS
COMPILED BY LCC
CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---						
	Silty Sand to Sandy Silt, trace gravel, trace clay, containing silty clay layers Compact to dense Wet Brown to grey	14	SS	29			
		15	SS	38			
209.1 19.8	Silty Clay, trace sand Stiff to hard Wet Grey	16	SS	40			
		17	SS	12			
		18	SS	19			
200.9 28.0	END OF BOREHOLE						

Notes:
1. Hole terminated due to tightening of soil around augers and resulting difficulties in advancing/withdrawing augers.
2. Water level in open borehole on December 15 and 18, 2000 at 2.7m depth (Elev.226.2m)

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED: 20 40 60 80 100
PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L
WATER CONTENT (%): 10 20 30
UNIT WEIGHT γ kN/m³

ELEVATION SCALE: 213, 212, 211, 210, 209, 208, 207, 206, 205, 204, 203, 202, 201

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

ON_MOT 0011143F.GPJ ON_MOT.GDT 14/1/02

RECORD OF BOREHOLE No B1-2

1 OF 3 **METRIC**

PROJECT 001-1143F
W.P. 30-95-00
DIST SW HWY 400
DATUM Geodetic
LOCATION N 4895623.5; E 292394.6
BOREHOLE TYPE 108mm ID HOLLOW STEM AUGERS AND CASING
DATE Dec.14-18/2000
ORIGINATED BY GPD
COMPILED BY LCC
CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
228.4	GROUND SURFACE						
0.0	Silty Sand, trace gravel, trace organics (Fill) Loose to compact Moist Brown	1	SS	14			
		2	SS	8			
226.1 2.3	Silty Sand to Sandy Silt, trace clay Compact Wet Brown	3	SS	13			
		4	SS	23			0 71 29 0
		5	SS	16			
		6	SS	13			0 34 65 1
		7	SS	21			
		8	SS	19			
		9	SS	18			
		10	SS	21			
		11	SS	19			
		12	SS	14			

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED: 20 40 60 80 100
PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L
WATER CONTENT (%): 10 20 30
UNIT WEIGHT γ kN/m³

ELEVATION SCALE: 228, 227, 226, 225, 224, 223, 222, 221, 220, 219, 218, 217, 216, 215, 214

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

ON_MOT 0011143F.GPJ ON_MOT.GDT 14/1/02

RECORD OF BOREHOLE No B1-2

2 OF 3

METRIC

PROJECT 001-1143F
W.P. 30-95-00
DIST SW HWY 400
DATUM Geodetic

LOCATION N 4895623.5; E 292394.6
BOREHOLE TYPE 108mm ID HOLLOW STEM AUGERS AND CASING
DATE Dec.14-18/2000

ORIGINATED BY GPD
COMPILED BY LCC
CHECKED BY ASP

SOIL PROFILE		SAMPLES										
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Silty Sand to Sandy Silt, trace clay Compact Wet Brown		13	SS	13		213				
							212				
							211				
			14	SS	21		210		o		
							209				
							208				
			15	SS	18		207				
206.6 21.8	Silty Clay, trace sand Firm to very stiff Moist Grey						206				
							205				
			16	SS	18		204		43		
							203				
			17	SS	8		202	× +			
			18	SS	6		201				
							200	× +			
							199				

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No B1-2

3 OF 3

METRIC

PROJECT 001-1143F
W.P. 30-95-00
DIST SW HWY 400
DATUM Geodetic

LOCATION N 4895623.5; E 292394.6
BOREHOLE TYPE 108mm ID HOLLOW STEM AUGERS AND CASING
DATE Dec.14-18/2000

ORIGINATED BY GPD
COMPILED BY LCC
CHECKED BY ASP

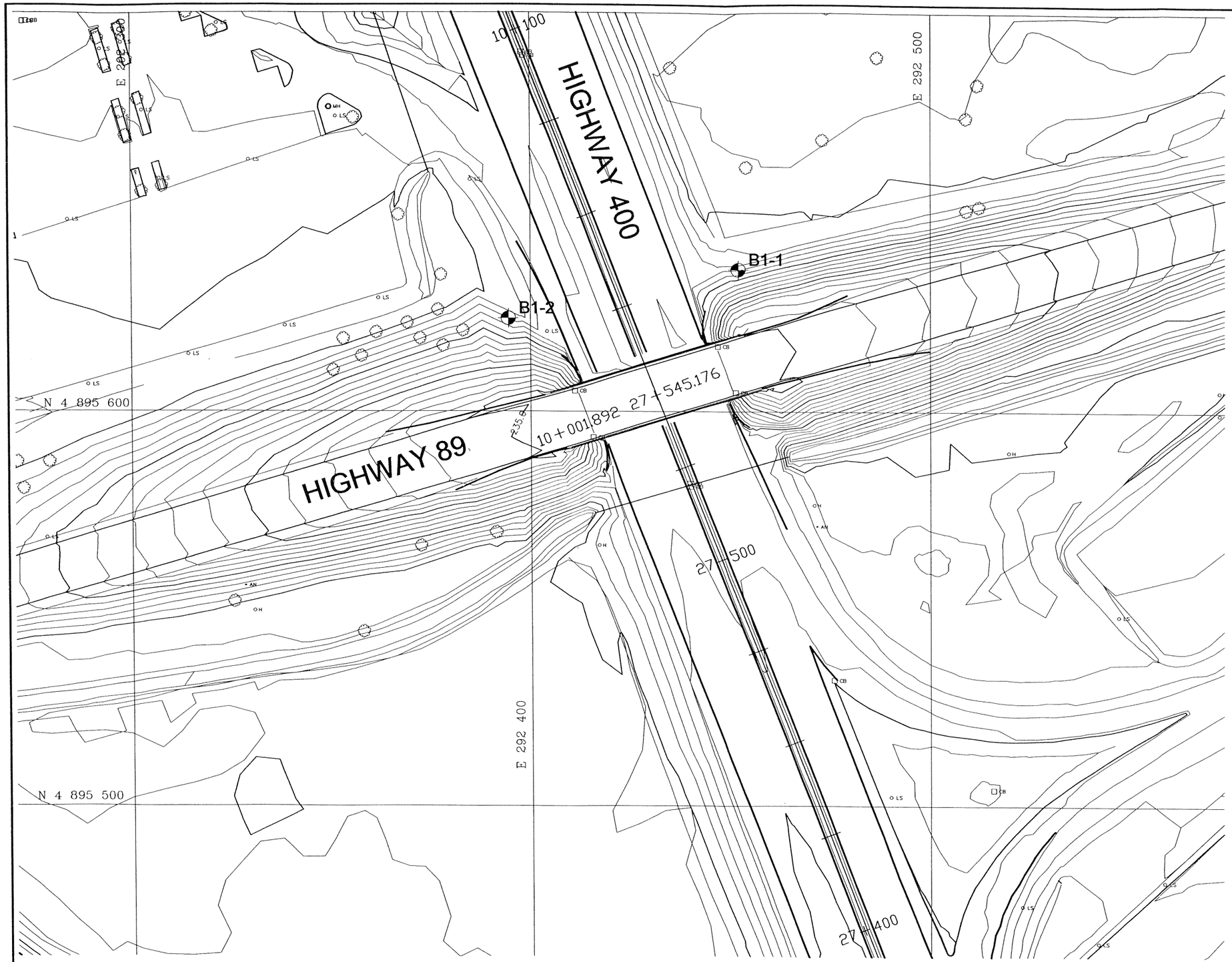
SOIL PROFILE		SAMPLES										
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Silty Clay, trace sand Firm to very stiff Moist Grey						198				
			19	SS	9				o		
							197				
							196				
194.9 33.5	Silty Sand containing silty clay layers Very dense Wet Grey		20	SS	52		195				
							194				
							193				
							192				
191.4 37.0			21	SS	72				o		
	END OF BOREHOLE Notes 1. Water level in open borehole at 2.3m depth (Elev.226.1m) on completion of drilling operations 2. Water level in piezometer at 1.8m depth (Elev.226.6m) on January 19, 2001, and at 1.3m depth (Elev.227.1m) on March 15, 2001.										

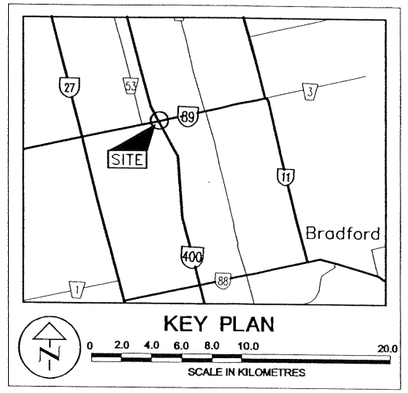
+ 3, × 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

DIST HWY 400	
CONT. No.	
GWP No. 30-95-00	
HIGHWAY 89 UNDERPASS HWY 400 BOREHOLE LOCATION PLAN	SHEET

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA





KEY PLAN

LEGEND

- Borehole, previous investigation
- Borehole, present investigation

No.	ELEVATION	LOCATION NORTHING	LOCATION EASTING
B1-1	228.9	4,895,635.8	292,452.1
B1-2	228.4	4,895,623.5	292,394.6

REFERENCE

This drawing was created from digital file "33811.dwg" provided by URS Cole Sherman

0 10 20 30 40 50 100m

SCALE IN METRES

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

NO.	DATE	BY	REVISION

Geocres No.

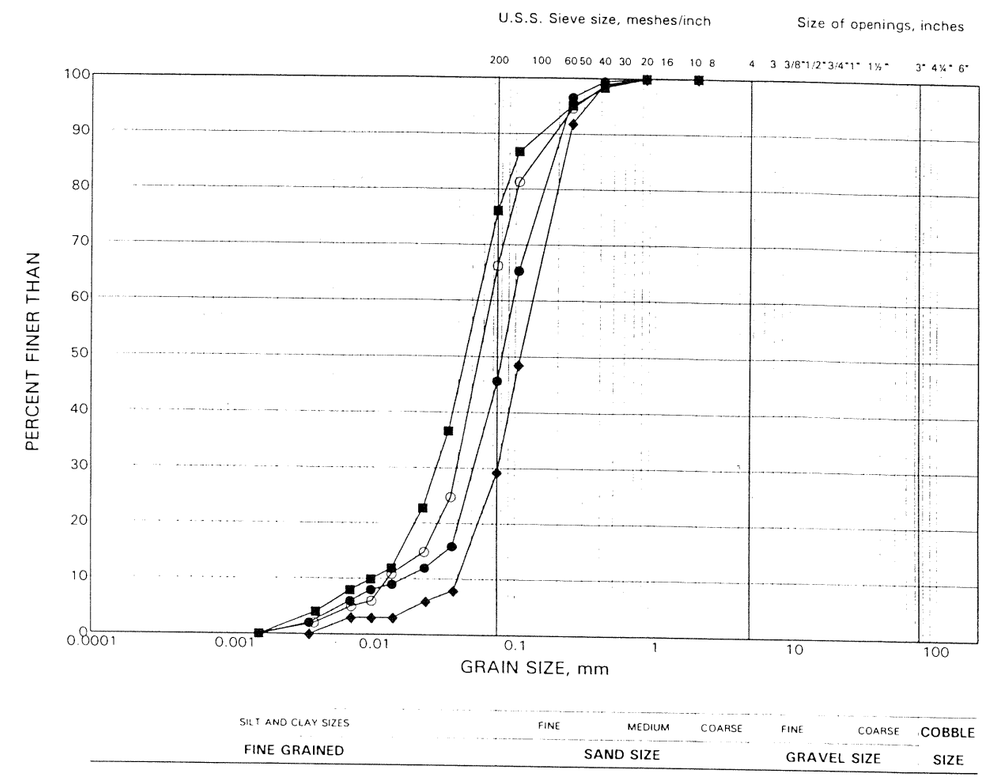
HWY. No. 400	PROJECT NO.: 001-1143F		
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-256
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

P1143F01.DWG

plot scale 1:2mm.

GRAIN SIZE DISTRIBUTION TEST RESULTS
Silty Sand to Sandy Silt Deposit

FIGURE 1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	B1-1	4	226.3
■	B1-1	7	224.0
◆	B1-2	4	225.0
○	B1-2	6	223.5

Project 001-1143F

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Golder Associates

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT KILLARNEY BEACH ROAD UNDERPASS STRUCTURE SITE 30-212 HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11 G.W.P. 30-95-00

Submitted to:

URS Cole, Sherman
75 Commerce Valley Drive East
Thornhill, Ontario
L3T 7N9

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January 2002 001-1143F-2

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PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
KILLARNEY BEACH ROAD UNDERPASS
STRUCTURE SITE 30-212
HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00

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LIST OF DRAWINGS
Drawing 1 Killarney Beach Road Underpass, Highway 400, Borehole Location Plan

LIST OF FIGURES
Figure 1 Grain Size Distribution Test Result, Silty Clay Till
Figure 2 Grain Size Distribution Test Result, Sand and Silt

1.0 INTRODUCTION

Golder Associates Ltd. has been retained by URS Cole, Sherman (Cole, Sherman) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the ultimate widening of Highway 400 from 1 km south of Highway 89, northerly 30 km to Highway 11, in Simcoe County, Ontario. Foundation engineering services are required for the widening and / or replacement of eighteen existing overpass and underpass structures, as well as five structural culverts.

This report addresses the replacement of the existing Killarney Beach Road underpass structure. A foundation investigation has been carried out, in which two boreholes were advanced and in-situ and laboratory testing was conducted, to determine the subsurface conditions at the site for this preliminary design study.

The terms of reference for the scope of work are outlined in Golder Associates' Proposal No. P01-1192, dated June 2000.

2.0 SITE DESCRIPTION

The existing Killarney Beach Road (Innisfil Fourth Line, also formerly Churchill Sideroad) underpass structure is located about 4 km north of the Highway 89 interchange and 5.5 km south of the Innisfil Beach Road (Simcoe Road 21) interchange in the Town of Innisfil, in Simcoe County. The MTO has designated this underpass as Structure Site No. 30-212.

At this site, the original ground surface varies from about Elevation 285 m to 287 m. The Highway 400 grade is at about Elevation 285.5 m to 287 m, rising northward. Killarney Beach Road has been constructed in fill, with approach embankments up to about 7 m in height. The local road grade over Highway 400 is at about Elevation 292.5 m.

According to the general layout drawing for the existing single-span underpass structure, provided by Morrison Hershfield (the structural designers for this preliminary study), the abutments and associated retaining walls are supported on spread footings which are founded at about Elevation 284.5 m.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at this site in October 2000, at which time two boreholes were drilled. Boreholes B2-1 and B2-2 were advanced in the vicinity of the east and west abutments to depths of about 11 m and 9.5 m, respectively, below the Highway 400 grade.

The investigation was carried out using a bombardier-mounted B-57 drill rig supplied and operated by Master Soil Investigations Ltd. of Weston, Ontario. The boreholes were advanced using solid stem augers. Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. The water levels in the open boreholes were observed during and following the drilling operations, and a piezometer was installed in Borehole B2-1, in the vicinity of the west abutment, to permit monitoring of the groundwater level at the site.

The field work was supervised on a full-time basis by a member of our staff who located the boreholes in the field, directed the drilling, sampling, and in-situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder Associates' laboratory in Mississauga for further examination. Index and classification tests consisting of water content determinations, Atterberg Limits tests and grain size distribution analyses were carried out on selected soil samples.

The borehole locations and elevations were surveyed by Callon Dietz, Ontario Land Surveyors. The borehole elevations are referenced to geodetic datum, and the northing and easting co-ordinates are referenced to the MTM NAD83 survey system. The borehole locations, together with elevations and northing and easting co-ordinates, are shown on the attached Drawing 1.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

This 30 km section of Highway 400 traverses, from south to north, the following physiographic regions as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, Third Edition, 1984): the Simcoe Lowlands; the Peterborough Drumlin Field; a second lobe of the Simcoe Lowlands; and the Simcoe Uplands. Along Highway 400, the Simcoe Lowlands are present from the southern limit of the project to just south of Innisfil Creek, and again from Essa Road (Simcoe Road 30, formerly Highway 27) to about 1 km north of Dunlop Street (Simcoe Road 90, formerly Highway 90). The Peterborough Drumlin Field occupies the belt between these lobes of the Simcoe Lowlands, extending from just south of Innisfil Creek, which is located about 1 km north of Highway 89, to Essa Road. The Simcoe Uplands extend from about 1 km north of Dunlop Street to beyond the northern limit of the project at Highway 11.

The two sections where Highway 400 crosses the Simcoe Lowlands consist of two lobes of a sand plain which include the shores of Kempenfelt Bay, the Nottawasaga River and Innisfil Creek. The surficial soils of these sections of the Simcoe Lowlands consist primarily of sand, although silt, clay or peat may be found in low-lying areas.

The surficial soils in the Peterborough Drumlin Field physiographic region, in which the Killarney Beach Road site is located, consist primarily of gravelly sand till or sand and gravel deposits. Drumlins (glacially-shaped hills) are more frequent in the southern portion of the section of the Peterborough Drumlin Field traversed by Highway 400. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins.

The surficial soils in the Simcoe Uplands physiographic region are primarily sandy silt till deposits, known to contain occasional boulders. Low-lying areas may be infilled with shallow sand and gravel deposits, which are shoreline deposits of a former glacial lake that once flooded the area.

4.2 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 given on the Record of Borehole sheets and Figures 1 and 2. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

Boreholes B2-1 and B2-2 were advanced in the vicinity of the east and west abutments, respectively, from approximately Highway 400 grade. The locations and ground surface elevations for these borings are shown on the attached Drawing 1.

In summary, at the toe of the existing embankments, the boreholes encountered fill and topsoil, overlying a variable till deposit consisting of stiff to hard silty clay to clayey silt grading to very dense silty sand with depth. A significant interlayer of silty sand to sand and silt was encountered within this till deposit. A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Fill and Topsoil

A layer of fill, 1.5 m to 2.1 m thick, was encountered in the boreholes. The fill ranges in composition from silty sand to sand and gravel, containing minor quantities of organics, clay and, in Borehole B2-2, asphalt pieces. The measured Standard Penetration Test (SPT) 'N' values ranged from 7 to 25 blows per 0.3 m of penetration, indicating that the fill has a loose to compact relative density.

A layer of topsoil was found underlying the fill in both boreholes; this topsoil is 800 mm and 300 mm thick in Boreholes B2-1 and B2-2, respectively. An organic content test on a sample of this topsoil gave an organic content of 7.3 per cent.

4.2.2 Silty Clay Till

An upper deposit of silty clay till, between 0.8 m and 3.6 m in thickness, was penetrated below the fill and topsoil in both boreholes. The top of the silty clay till was encountered between Elevation 283.9 m and 283.5 m. In Borehole B2-1 on the east side of the highway, the base of the silty clay till was encountered at about Elevation 283 m, overlying a silty sand to sand and silt interlayer. In Borehole B2-2 on the west side of the highway, the base of the silty clay till was encountered at about Elevation 280 m, directly overlying the lower hard clayey silt till.

The silty clay till contains trace to some sand and gravel. The results of a grain size distribution test carried out on a representative sample are shown on Figure 1. The natural moisture contents measured on samples of the silty clay till ranged from 22 to 24 per cent. Atterberg Limits testing on two samples measured plastic limits of 15 and 19 per cent, liquid limits of 35 and 39 per cent, and plasticity indices of about 20 per cent. The results of the Atterberg Limits testing indicate that this silty clay till is inorganic and of intermediate plasticity.

The SPT 'N' values measured in the silty clay till ranged from 15 to 23 blows per 0.3 m of penetration, indicating that this till has a stiff to very stiff consistency.

4.2.3 Silty Sand to Sand and Silt Interlayer

A 3.5 m thick layer of silty sand to sand and silt, containing trace clay and gravel, was encountered below a thin upper portion of silty clay till in Borehole B2-1, between about Elevation 283 m and 279.5 m. No corresponding layer was encountered in Borehole B2-2. It is considered that the silty sand to sand and silt may represent an interlayer or lens within the till deposit.

The results of a grain size distribution test carried out on a representative sample are shown on Figure 2. The natural moisture contents measured on two samples of the silty sand to sand and silt till were 13 and 14 per cent. The measured SPT 'N' values ranged from 9 to 82 blows per 0.3 m of penetration; however, it is considered that the measurement of 9 blows could have been impacted by groundwater inflow into the borehole. This interlayer is therefore considered to have a dense to very dense relative density, based on the other measured SPT 'N' values of 44 to 82 blows per 0.3 m of penetration.

4.2.4 Clayey Silt Till

A deposit of clayey silt till, containing trace to some sand and gravel, was encountered in the boreholes below the silty clay till and, where present, the silty sand to sand and silt interlayer. The surface of the clayey silt till deposit was encountered in the boreholes between Elevation 280 m and 279.5 m. The till extended to the maximum depth investigated in Borehole B2-1 (to about Elevation 275 m), and graded to silty sand till at about Elevation 277 m in Borehole B2-2.

Natural moisture contents of 7 and 8 per cent were measured on two samples of the clayey silt till. Atterberg Limits testing on one sample measured a plastic limit of 9 per cent, a liquid limit of 16 per cent, and a plasticity index of 7 per cent. The results of the Atterberg Limits testing indicate that this clayey silt till is inorganic and of low plasticity.

The SPT 'N' values measured in the clayey silt till were greater than 100 blows per 0.3 m of penetration, indicating that this deposit has a hard consistency.

4.2.5 Silty Sand Till

Silty sand till, containing trace gravel, was encountered at the base of Borehole B2-2. This layer was explored for a depth of 0.7 m. The measured natural moisture content on one sample of this till was 13 per cent. The measured SPT 'N' value of greater 100 blows per 0.3 m of penetration indicates that this till has a very dense relative density.

4.3 Groundwater Conditions

Water levels were observed in the open boreholes following completion of the drilling operations in October 2000. In Borehole B2-1, the measured water level was at 2.1 m depth (Elevation 284.1 m), about 1 m above the top of the water-bearing silty sand to sand and silt interlayer within the till. In Borehole B2-2, the water level was measured at 6 m depth (Elevation 279.9 m), at the top of the clayey silt portion of the till deposit.

The groundwater level measured in the piezometer sealed at depth in Borehole B2-2 was at 1.9 m depth (Elevation 284 m) in January 2001 and at 1.1 m depth (Elevation 284.8 m) in March 2001. These readings most probably represent the piezometric groundwater level at the site.

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

GOLDER ASSOCIATES LTD.

Lisa C. Coyne, P.Eng.,
Geotechnical Engineer

for Anne S. Poschmann, P.Eng.,
Principal

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

LCC/JLS/ASP/FJH/clg
N:\ACTIVE\1100\001-1143F\RPT02-02JAN-KILLARNEYBEACH.DOC

PART B

PRELIMINARY FOUNDATION DESIGN REPORT
KILLARNEY BEACH ROAD UNDERPASS
STRUCTURE SITE 30-212
HIGHWAY 400 WIDENING FROM 1 KM SOUTH
OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

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

It is understood that Highway 400 will be widened from its existing six-lane configuration to an interim configuration of eight-lanes, and an ultimate configuration of ten-lanes, and that an alternative for a twelve-lane express / collector system is under consideration between Molson Park Drive and Duckworth Street in Barrie. Throughout the project length, it is expected that the existing highway platform will be widened by between 13 m and 30 m. Replacement of the existing Killarney Beach Road underpass structure will therefore be necessary.

Based on the available general layout drawing for the existing single-span structure, the abutments and associated retaining walls are supported on spread footings founded at about Elevation 284.5 m. The Highway 400 grade is at about Elevation 285.5 m to 287 m, while the surrounding grade varies from about Elevation 285 m to 287 m. Killarney Beach Road has been constructed on embankment fill, with its grade at about Elevation 292.6 m over Highway 400.

5.2 Bridge Foundation Options

The subsoils at the site consist of fill and topsoil overlying an upper, stiff to very stiff silty clay till deposit which is underlain in the borehole on the east side of Highway 400 by a loose to very dense silty sand interlayer. These deposits are in turn underlain by a deposit of hard clayey silt till grading with depth to very dense silty sand till. The top of the hard / very dense till was encountered in the boreholes at about Elevation 280 m to 279.5 m, about 5 m to 7 m below Highway 400 grade. The measured groundwater level was as high as 1 m below ground surface, at approximately Elevation 285 m.

Consideration could be given to the use of perched abutments for the replacement structure, founded on spread footings which are placed on a compacted granular pad within the approach

embankment fill. Spread footings placed on native soils will be more difficult to adopt for the replacement structure, due to the relatively high groundwater table and the variable thickness of the stiff to very stiff silty clay till deposit across the site. However, if the design can accommodate lower design bearing values, consideration could be given to placing the footings on this stiff to very stiff silty clay till. If this option is adopted, further investigation will be required for final design to assess the variability of the silty clay till and the extent of the silty sand / sand and silt interlayer. Prior groundwater control will be required if this deposit is present at the proposed founding level. It is noted that it is not considered practicable to found spread footings on the hard clayey silt till deposit, as the top of this deposit is about 5 m to 7 m below the Highway 400 grade.

Alternatively, the abutments and any piers could be supported on steel H-piles driven to found within the hard lower clayey silt till. The soil conditions at this site are favourable for the use of an integral abutment replacement structure.

Preliminary recommendations for spread footings, including perched abutments, and for deep foundations are provided in the following sections.

5.3 Spread Footings

For preliminary design of the abutment and pier footings for the replacement structure, spread footings could be founded on the stiff to very stiff silty clay till or the silty sand to sand and silt interlayer where present, at or below Elevation 283.5 m. A minimum soil cover of 1.5 m should be provided. Any associated wing wall or retaining wall footings could be stepped upward away from the abutments such that a minimum soil cover of 1.5 m is maintained above the underside of the footings. In this case, a well-compacted granular material would be necessary as a subgrade above Elevation 283.5 m.

Alternatively, consideration could be given to the use of abutment footings perched within the approach embankments.

5.3.1 Axial Geotechnical Resistance

Spread footings placed on the properly prepared silty clay till or, where present, silty sand to sand and silt interlayer, at or below the design elevation indicated above may be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 300 kPa, assuming a 3 m wide footing. The settlement of the footings will be dependent on the footing size and configuration,

the applied loads, and the thickness and distribution of the stiff to very stiff silty clay till deposit. For preliminary design purposes, the geotechnical resistance at Serviceability Limit States (SLS) may be taken as 150 kPa. The geotechnical resistance at SLS will have to be reviewed following the detailed design stage of subsurface investigation, once the footing size, configuration and loadings are known.

For spread footings placed within the approach embankments on a compacted Granular 'A' core, a factored geotechnical resistance at ULS of 900 kPa may be assumed for preliminary design. The geotechnical resistance at SLS will depend on the thickness of Granular 'A' and the consistency and thickness of the underlying soils; a value of 350 kPa may be assumed for preliminary design.

The geotechnical resistances provided herein are given under the assumption that the loads will be applied perpendicular to the surface of the footings; where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the Ontario Highway Bridge Design Code (OHBDC).

5.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footing and the subsoils should be calculated in accordance with Section 6-8.4.3 of the OHBDC. The angle of friction between the concrete and the undisturbed founding soils should be taken as 22 degrees; the corresponding coefficient of friction, tan δ, would then be 0.4. Where "perched" footings are adopted, the angle of friction between the concrete footings and the compacted Granular "A" pad should be taken as 30 degrees; the corresponding coefficient of friction would be 0.58.

5.3.3 Frost Protection

The footings should be provided with a minimum of 1.5 m of soil cover for frost protection.

5.4 Driven Steel H-Piles

Consideration could be given to supporting the new underpass structure on steel H-piles driven to found within the hard lower clayey silt till deposit. The surface of the clayey silt till was encountered in the boreholes at about Elevation 280 m, some 6 m below Highway 400 grade and

12 m below Killarney Beach Road grade. A design pile tip level of Elevation 278 m may be assumed for preliminary design.

5.4.1 Axial Geotechnical Resistance

For preliminary design, the factored axial resistance at ULS for steel HP 310 x 110 H-piles driven to found within the hard clayey silt till at or below Elevation 278 m may be taken as 1,600 kN. The axial resistance at SLS, for a single pile, for 25 mm of settlement may be taken as 1,400 kN.

To achieve the above design resistances, the piles should be driven to a final set of no less than 15 blows per 25 mm of penetration using a hammer with rated energy of about 50 kJ, and not exceeding 60 kJ. Provision should be made to re-tap selected piles to confirm the set after adjacent piles have been driven, in accordance with MTO's current Special Provision.

5.4.2 Resistance to Lateral Loads

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

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

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

where
n_h is the constant of subgrade reaction
z is the depth (m)
B is the pile diameter (m)

For cohesive soils:

$$k_h = \frac{k_{s1}}{5B}$$

where
B is the pile diameter (m) and
k_{s1} is the constant of horizontal subgrade reaction, as given below

If necessary, the pile caps could be perched within the embankment fill to provide a greater driven length. In this case, the piles would extend through the embankment fill, the stiff to very stiff upper silty clay till, and the generally dense to very dense silty sand to sand and silt interlayer, where present. For well-compacted cohesionless embankment fill and for the silty sand to sand and silt interlayer, the range in value of n_h may be taken as 5 MPa/m to 10 MPa/m in the structural analysis. For the stiff to very stiff silty clay till, below Elevation 283.5 m, the range in value of k_{s1} may be taken as 20 MPa/m to 45 MPa/m in the structural analysis.

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

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

5.4.3 Frost Protection

The pile caps should be provided with 1.5 m of soil cover for frost protection.

5.5 Lateral Earth Pressures

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

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the abutments and walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00

- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the abutment wall, in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be used in accordance with OPSS 501.06.

- The granular fill may be placed either in a zone with width equal to at least 1.5 m behind the back of the stem (Case I from OHBDC Figure 6-7.4.1) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II from OHBDC Figure 6-7.4.4).

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

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

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

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

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

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

5.6 Embankment Design

Based on the topographic information on the Engineering and Title Records plates and on site reconnaissance, the existing Killarney Beach Road embankment side slopes are formed at a

gradient of about 2 horizontal to 1 vertical (2H:1V). If any widening of the local road embankment will be required, the new side slopes also should be formed at a maximum gradient of 2H:1V. The widening of the embankment should be carried out using conventional fill placement and compaction practices, and benching of the existing embankment side slopes should be carried out to key in the new fill.

5.7 Design and Construction Considerations

5.7.1 Groundwater Control

Groundwater seepage into the footing or pile cap excavations will occur from the fill and from water-bearing lenses or interlayers of granular soil within the till deposit. Pumping from properly-filtered sumps or a filtered drain placed at the base of the excavation should provide sufficient groundwater control during excavation to foundation level within the stiff to very stiff silty clay till. Where the in-situ integrity of the silty sand to sand and silt interlayer is to be maintained, dewatering would be required prior to excavation.

The silty subgrade soils at the site are susceptible to disturbance from ponded water and construction traffic. Surface water run-off should be directed away from the footing excavations, and care must be exercised to avoid disturbance, particularly at and below founding level. Provision should be made in the Contract Documents for the placement of a lean concrete mat to protect the soils from such disturbance once the subgrade has been inspected and approved.

5.7.2 Excavation

Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities, using proper groundwater control. The footing or pile cap excavations will extend a minimum of 1.5 m below lowest surrounding grade, generally through fill, the upper silty clay till and the silty sand to sand and silt interlayer, where present. The upper silty clay till and silty sand to sand and silt interlayer would be classified as Type 2 to 3 soils. Temporary open-cut slopes should be maintained no steeper than 1 horizontal to 1 vertical (1H:1V). For excavations formed within the silty clay till, the lower 1.2 m of the excavation sides may be formed near-vertical. Where space restrictions dictate, footing or pile cap excavations could also be carried out within a braced excavation.

5.7.3 Settlement

Deformation of the ground due to foundation loading will result in settlement of the bridge piers, abutments and superstructure. The resulting differential settlement across the structure is a critical consideration in the design. The magnitude of deformation will be dependent on the type of foundations, the rigidity of the structural elements, the variability and consistency / relative density of the founding soils, and the activities during construction. The total and differential settlement will have to be reassessed during final design.

5.7.4 Obstructions

Although no cobbles or boulders were encountered during the borehole investigation, it should be noted that cobbles and boulders are inherent in glacially-derived materials. Cobbles and boulders should therefore be expected during driving of steel H-piles if a deep foundation option is selected.

GOLDER ASSOCIATES LTD.

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for Anne S. Poschmann, P.Eng.,
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LCC/JLS/ASP/FJH/clg
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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

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 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.).

FH:	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 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	c_u, s_u 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 (LV-laboratory vane test)
γ	unit weight

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

S:\FINALDAT\ABBREV\2000\LOFA-D00.DOC

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 δ
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

RECORD OF BOREHOLE No B2-1 — 1 OF 1 — METRIC

PROJECT 001-1143F | LOCATION N 4899694.3; E 291491.8 | ORIGINATED BY AZ
W.P. 30-95-00 | BOREHOLE TYPE 108mm DIAMETER SOLID STEM AUGERS | COMPILED BY LCC
DIST SW HWY 400 | DATE Oct.23/2000 | CHECKED BY ASP
DATUM Geodetic

ELEV / DEPTH	SOIL PROFILE — DESCRIPTION	SAMPLES — NUMBER	TYPE	'N' VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
286.2	GROUND SURFACE					
0.0	Silty Sand, some organics, trace clay and gravel (Fill) Compact Moist Brown/black	1	SS	18		
284.7						
1.5	Topsoil	2	SS	13	OC=7.3%	
283.9						
2.3	Silty Clay, trace sand and gravel (Till) Very stiff Brown Moist	3	SS	21		
283.1						
3.1	Silty Sand to Sand and Silt, trace clay and gravel Loose to very dense Brown Moist to wet	4	SS	9		
		5	SS	51		2 43 50 5
		6	SS	44		
		7	SS	82		
279.6						
6.6	Clayey Silt, some sand, trace gravel. (Till) Hard Grey Moist	8	SS	108/.15		
		9	SS	100/.08		
		10	SS	100/.15		
275.2						
11.0	END OF BOREHOLE Note: Water level in open borehole at 2.1m depth (Elev.284.1m) on completion of drilling operations.					

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100
ELEVATION SCALE: 286, 285, 284, 283, 282, 281, 280, 279, 278, 277, 276
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30

+³, ×³: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No B2-2 — 1 OF 1 — METRIC

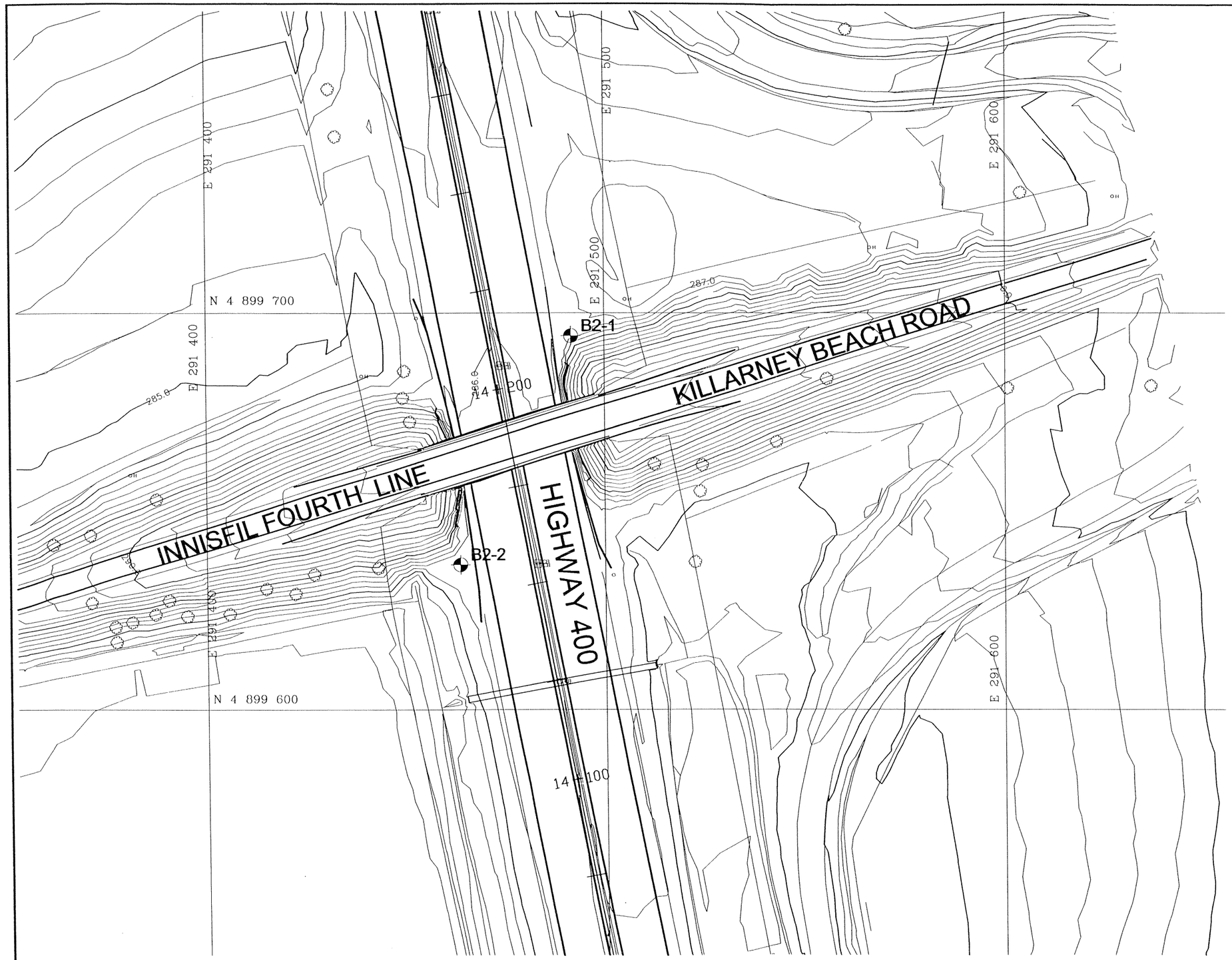
PROJECT 001-1143F | LOCATION N 4899636.5; E 291463.6 | ORIGINATED BY PKS
W.P. 30-95-00 | BOREHOLE TYPE 108mm DIAMETER SOLID STEM AUGERS | COMPILED BY LCC
DIST SW HWY 400 | DATE Oct.26/2000 | CHECKED BY ASP
DATUM Geodetic

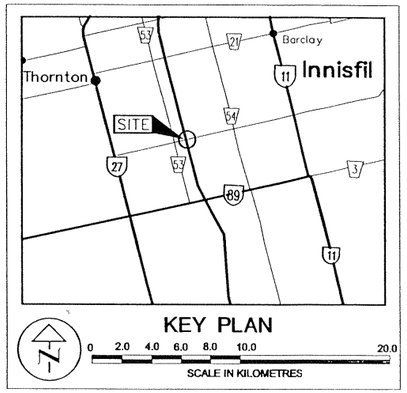
ELEV / DEPTH	SOIL PROFILE — DESCRIPTION	SAMPLES — NUMBER	TYPE	'N' VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
285.9	GROUND SURFACE					
0.0	Sand and Gravel, trace silt, trace clay pockets and asphalt pieces (Fill) Loose to compact Moist Brown	1	SS	7		
		2	SS	25		
		3	SS	16		
283.8						
283.5	Topsoil					
2.4	Silty Clay, trace sand and gravel (Till) Stiff to very stiff Moist Grey	4	SS	15		
		5	SS	15		1 24 36 39
		6	SS	23		
		7	SS	15		
279.9						
6.0	Clayey Silt, trace to some sand and gravel (Till) Hard Moist Grey	8	SS	121		
		9	SS	108		
277.1						
8.8	Silty Sand, trace clay and gravel (Till) Very dense Moist Grey	10	SS	100/.23		
276.4						
9.5	END OF BOREHOLE Notes: 1. Water level in open borehole at 6m depth (Elev.279.9m) on completion of drilling operations. 2. Water level in piezometer at 1.9m depth (Elev.284.0m) on January 19, 2001, and at 1.1m depth (Elev.284.8m) on March 15, 2001.					

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100
ELEVATION SCALE: 285, 284, 283, 282, 281, 280, 279, 278, 277
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30

+³, ×³: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

DIST HWY 400 CONT. No. GWP No. 30-95-00	
KILLARNEY BEACH RD. UNDERPASS HWY 400 BOREHOLE LOCATION PLAN	SHEET
Golder Associates	Golder Associates Ltd. MISSISSAUGA, ONTARIO, CANADA





LEGEND

- Borehole, previous investigation
- Borehole, present investigation

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
B2-1	286.2	4,899,694.3	291,491.8
B2-2	285.9	4,899,636.5	291,463.6

REFERENCE

This drawing was created from digital file "33808, 33809.dwg" provided by URS Cole Sherman

0 10 20 30 40 50 100m

SCALE IN METRES

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

NO.	DATE	BY	REVISION

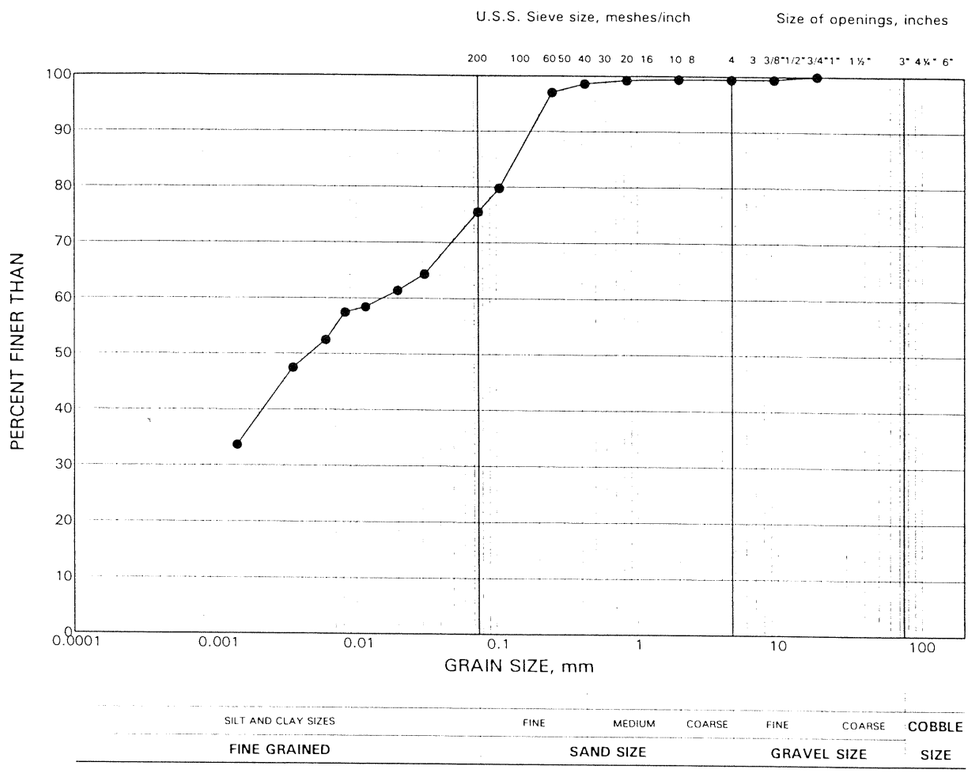
Geocres No.

HWY. No. 400		PROJECT NO.: 001-1143F	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-212
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

P1143F06.DWG

GRAIN SIZE DISTRIBUTION TEST RESULT
Silty Clay Till

FIGURE 1



LEGEND

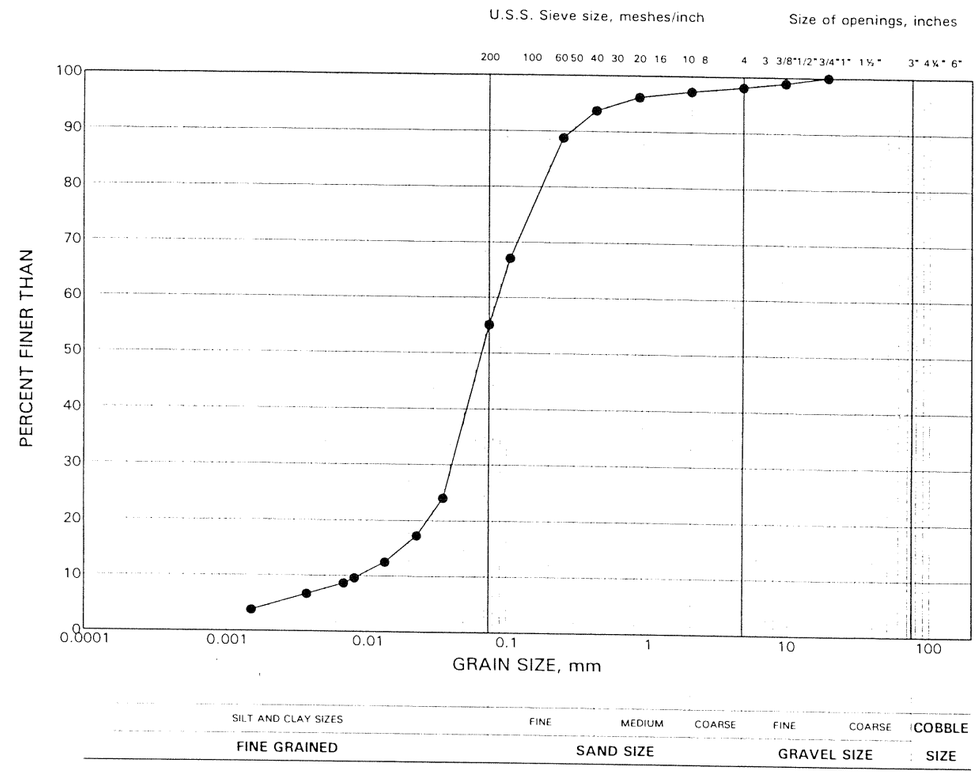
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	B2-2	5	282.5

Project 001-1143F

Golder Associates

GRAIN SIZE DISTRIBUTION TEST RESULT
Sand and Silt

FIGURE 2



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	B2-1	5	282.1

Project 001-1143F

Golder Associates

Golder Associates Ltd.

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Mississauga, Ontario, Canada L5N 5S3
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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT INNISFIL SIXTH LINE OVERPASS STRUCTURE SITE 30-211 HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11 G.W.P. 30-95-00

Submitted to:

URS Cole, Sherman
75 Commerce Valley Drive East
Thornhill, Ontario
L3T 7N9

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January 2002 001-1143F-3

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PRELIMINARY FOUNDATION INVESTIGATION REPORT
INNISFIL SIXTH LINE OVERPASS
STRUCTURE SITE 30-211
HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11
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Drawing 1 Innisfil Sixth Line Overpass, Highway 400, Borehole Location Plan

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Figure 1 Grain Size Distribution Test Result, Clayey Silt Till

1.0 INTRODUCTION

Golder Associates Ltd. has been retained by URS Cole, Sherman (Cole, Sherman) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the ultimate widening of Highway 400 from 1 km south of Highway 89, northerly 30 km to Highway 11, in Simcoe County, Ontario. Foundation engineering services are required for the widening and / or replacement of eighteen existing overpass and underpass structures, as well as five structural culverts.

This report addresses the widening and / or replacement of the existing Innisfil Sixth Line overpass structure. A foundation site investigation has been carried out, in which two boreholes were advanced and in-situ and laboratory testing was conducted, to determine the subsurface conditions at the site for this preliminary design study.

The terms of reference for the scope of work are outlined in Golder Associates' Proposal No. P01-1192, dated June 2000.

2.0 SITE DESCRIPTION

The existing Innisfil Sixth Line overpass structure is located about 7 km north of the Highway 89 interchange and about 8.5 km south of the Molson Park Drive interchange, in the Town of Innisfil, Simcoe County. The MTO has designated this overpass as Structure Site No. 30-211.

At this site, the original ground surface was at about Elevation 294 m to 295 m. Innisfil Sixth Line has been constructed in a cut up to 4 m deep, with its grade at about Elevation 291 m under Highway 400. The Highway 400 grade is at about Elevation 296.5 m at the structure site.

The existing single-span overpass structure was constructed in the early 1950s. According to the general layout drawings for this existing structure, which was provided by Morrison Hershfield (the structural designers for this preliminary study), the abutments and associated retaining walls are supported on spread footings which are founded at about Elevation 289.4 m.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at this site in October 2000, at which time two boreholes were drilled. Boreholes B3-1 and B3-2 were advanced in the vicinity of the north and south abutments, on the east and west sides of the highway, respectively. The boreholes were advanced to between 8 m and 11 m below the Innisfil Sixth Line cut grade.

The investigation was carried out using a bombardier-mounted B-57 drill rig supplied and operated by Master Soil Investigations Ltd. of Weston, Ontario. The boreholes were advanced using solid stem augers. Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. The groundwater conditions in the open boreholes were observed throughout the drilling operations, and a piezometer was installed in Borehole B3-1 to permit monitoring of the groundwater levels at the site.

The field work was supervised on a full-time basis by a member of our staff who located the boreholes in the field, directed the drilling, sampling, and in-situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder Associates' laboratory in Mississauga for further examination. Index and classification tests consisting of water content determinations, Atterberg Limits tests and grain size distribution analyses were carried out on selected soil samples.

The borehole locations and elevations were surveyed by Callon Dietz, Ontario Land Surveyors. The borehole elevations are referenced to geodetic datum, and the northing and easting co-ordinates are referenced to the MTM NAD83 survey system. The borehole locations, together with elevations and northing and easting co-ordinates, are shown on the attached Drawing 1.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

This 30 km section of Highway 400 traverses, from south to north, the following physiographic regions as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, Third Edition, 1984): the Simcoe Lowlands; the Peterborough Drumlin Field; a second lobe of the Simcoe Lowlands; and the Simcoe Uplands. Along Highway 400, the Simcoe Lowlands are present from the southern limit of the project to just south of Innisfil Creek (about 1 km north of Highway 89) and again from Essa Road (Simcoe Road 30, formerly Highway 27) to about 1 km north of Dunlop Street (Simcoe Road 90, formerly Highway 90). The Peterborough Drumlin Field occupies the belt between these lobes of the Simcoe Lowlands, extending from just south of Innisfil Creek to Essa Road. The Simcoe Uplands extend from about 1 km north of Dunlop Street to beyond the northern limit of the project at Highway 11.

The two sections where Highway 400 crosses the Simcoe Lowlands consist of two lobes of a sand plain which include the shores of Kempenfelt Bay, the Nottawasaga River and Innisfil Creek. The surficial soils of these sections of the Simcoe Lowlands consist primarily of sand, although silt, clay or peat may be found in low-lying areas.

The surficial soils in the Peterborough Drumlin Field, in which the Innisfil Sixth Line site is located, consist primarily of gravelly sand till or sand and gravel deposits. Drumlins (glacially-shaped hills) are more frequent in the southern portion of the section of the Peterborough Drumlin Field traversed by Highway 400. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins.

The surficial soils in the Simcoe Uplands physiographic region are primarily sandy silt till deposits, known to contain occasional boulders. Low-lying areas may be infilled with shallow sand and gravel deposits, which are shoreline deposits of a former glacial lake that once flooded the area.

4.2 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 given on the Record of Borehole sheets and Figure 1. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

Boreholes B3-1 and B3-2 were advanced on the east and west sides of Highway 400, respectively, from approximately Innisfil Sixth Line grade. The locations and ground surface elevations for these borings are shown on the attached Drawing 1.

In summary, the site is underlain at the borehole locations by sand and gravel to silty sand fill, overlying clayey silt till. A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Fill

Beneath the asphalt in Boreholes B3-1 and B3-2, 300 mm to 500 mm of sand and gravel road base fill was encountered.

Underlying this road base fill in Borehole B3-2, a 1.8 m thick layer of silty sand fill, containing some gravel and trace clay, was encountered. It is likely that this silty fill is associated with utility trench backfill, as Borehole B3-2 is located near a catch basin. The measured Standard Penetration Test 'N' values were 11 and 28 blows per 0.3 m of penetration, indicating that the silty sand fill has a compact relative density.

4.2.2 Clayey Silt Till

A deposit of clayey silt till was encountered below the fill in both boreholes. The surface of the till is at Elevation 290.7 m in Borehole B3-1, on the east side of the highway. On the west side of the highway, Borehole B3-2 encountered utility trench backfill; outside of the utility trench areas, it is expected that the surface of the till deposit will be encountered immediately below the road base fill. The till deposit extends to the maximum depth investigated, to about Elevation 283 m and 279.5 m in the boreholes on the east and west sides of Highway 400, respectively. The till deposit is at least 8 m to 11 m thick.

The clayey silt till contains a significant proportion of sand, and trace to some gravel. The result of a grain size distribution test carried out on a representative sample is shown on Figure 1. The natural moisture contents measured on samples of the clayey silt till ranged from 6 to 9 per cent. Atterberg Limits testing was carried out on three samples. The plastic limits ranged from 11 to 12 per cent, the liquid limits from 14 to 15 per cent and the plasticity indices from 3 to 4 per cent. The results of the Atterberg Limits testing indicate that the clayey silt till is inorganic and of low plasticity.

The measured SPT 'N' values ranged from 67 to 138 blows, but were typically greater than 100 blows per 0.3 m of penetration, indicating that the clayey silt till is hard.

4.3 Groundwater Conditions

The groundwater conditions were observed in the open boreholes following drilling operations in October 2000. At that time, the water level measured in Borehole B3-1 was at 6.9 m depth (about Elevation 284.5 m) and rising; the water level measured in Borehole B3-2 was at 4 m depth (about Elevation 287 m). The piezometer which was installed in Borehole B3-1 could not be founded in January or March 2001. This piezometer is presumed to have been destroyed. Therefore, the stabilized groundwater level could not be determined

The colour change in the soil from brown to grey at a relatively shallow depth indicates that the piezometric groundwater level is likely in the upper portion of the till.

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

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
INNISFIL SIXTH LINE OVERPASS
STRUCTURE SITE 30-211
HIGHWAY 400 WIDENING FROM 1 KM SOUTH
OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

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

It is understood that Highway 400 will be widened from its existing six-lane configuration to an interim configuration of eight-lanes, and an ultimate configuration of ten-lanes, and that an alternative for a twelve-lane express / collector system is under consideration between Molson Park Drive and Duckworth Street in Barrie. Throughout the project length, it is expected that the existing highway platform will be widened by between 13 m and 30 m. Widening and / or replacement of the existing Innisfil Sixth Line overpass will therefore be necessary.

Based on the general layout drawing for the existing single-span structure, the abutments and associated retaining walls are supported on spread footings founded at about Elevation 289.4 m. The Highway 400 grade is at about Elevation 296.5 m at the structure site, while the surrounding grade is at about Elevation 294 m to 295 m. Innisfil Sixth Line has been constructed in a cut up to 4 m deep, with its grade at about Elevation 291 m under Highway 400.

5.2 Bridge Foundation Options

The native subsoils at the site consist of hard clayey silt till, having Standard Penetration Test (SPT) 'N' values typically greater than 100 blows per 0.3 m of penetration. Based on these subsurface conditions and on the existing foundation conditions, it is recommended that the widening or replacement structure be founded on spread footings placed on the hard clayey silt till below the Innisfil Sixth Line grade. Consideration could also be given to the use of perched abutments, founded on spread footings placed on a compacted granular pad within the approach embankments.

Alternatively, for a new overpass or structurally-separate widening, the abutments could be supported on steel H-piles driven to practical refusal within the hard clayey silt till.

It should be noted that the boreholes put down during this preliminary phase of field work were drilled from the Innisfil Sixth Line cut level. If perched footings or integral abutments supported on deep foundations are considered viable options, determination of the subsoil conditions between the Highway 400 grade and the Innisfil Sixth Line cut will be required during the detailed design stage.

Preliminary recommendations for spread footings, including perched abutments, and for driven steel H-pile foundations are provided in the following sections.

5.3 Spread Footings

For preliminary design of the abutment foundations, spread footings may be placed on the hard clayey silt till deposit at or below Elevation 289.4 m; a minimum of 1.5 m of soil cover should be provided. For widening of the existing overpass structure, the founding level of the existing footings should be matched. Where fill is encountered below footing founding level, it should be removed and replaced with lean concrete. Any associated wing wall or retaining wall footings may be stepped upward away from the abutments such that a minimum soil cover of 1.5 m is maintained above the underside of the footings. In this case, a well-compacted granular pad will be required to support the wing wall or retaining wall footings above Elevation 289.4 m.

Alternatively, consideration could be given to the use of abutment footings perched on a granular pad within well compacted approach embankment fill.

5.3.1 Axial Geotechnical Resistance

Spread footings placed within the undisturbed clayey silt till deposit at the design elevation given above may be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 900 kPa, assuming a 3 m wide footing. The groundwater conditions at the site will have to be confirmed during the detailed design stage.

The settlement of footings founded on the clayey silt till will be dependent on the footing size and configuration, and on the applied loads. This settlement will be differential with respect to the existing overpass structure where widening of the structure is considered. For preliminary design purposes, the geotechnical resistance at Serviceability Limit States (SLS) may be taken as 600 kPa. The geotechnical resistance at SLS will have to be reviewed following the detailed

design stage of subsurface investigation, once the footing size, configuration and loadings are known, and the groundwater conditions at the site are confirmed.

For spread footings placed within the approach embankments on a compacted Granular 'A' pad, a factored geotechnical resistance at ULS of 900 kPa may be assumed for preliminary design. The geotechnical resistance at SLS will depend on the thickness of well compacted Granular 'A' and the consistency and thickness of the underlying soils; a value of 350 kPa may be assumed for preliminary design.

The geotechnical resistances provided herein are given under the assumption that the loads will be applied perpendicular to the surface of the footings; where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the Ontario Highway Bridge Design Code (OHBDC).

5.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footing and the subsoils should be calculated in accordance with Section 6-8.4.3 of the OHBDC. The angle of friction between the concrete and the undisturbed founding soils should be taken as 30 degrees; the corresponding coefficient of friction, tan δ, would then be 0.58. Where "perched" footings are adopted, the angle of friction between the concrete footings and the compacted Granular "A" pad should be taken as 30 degrees; the corresponding coefficient of friction would be 0.58.

5.3.3 Frost Protection

All footings should be provided with a minimum of 1.5 m of soil cover for frost protection.

5.4 Driven Steel H-Piles

Consideration could be given to supporting a new or structurally-separate overpass structure on steel H-piles driven to found within the hard clayey silt till deposit. The pile tip level is expected to be variable at this site and could be as low as Elevation 286 m on the west side and as high as Elevation 288 m on the east side. It is assumed that the abutment pile caps would be perched above the Innisfil Sixth Line grade in order to achieve the required pile length for rotational flexibility purposes. In this regard, it is noted that additional borehole investigation will be

required at the proposed abutment locations during the detailed design stage in order to confirm the soil conditions above the Innisfil Sixth Line cut grade.

5.4.1 Axial Geotechnical Resistance

For preliminary design, the factored axial resistance at ULS for steel HP 310 x 110 H-piles driven to found within the hard clayey silt till may be taken as 1,600 kN. The axial resistance at SLS for a single pile, for 25 mm of settlement, may be taken as 1,400 kN. For design, a pile tip level at Elevation 287 m may be assumed.

To achieve the above design resistances, the piles should be driven to a final set of no less than 15 blows per 25 mm of penetration using a hammer with rated energy of about 50 kJ, and not exceeding 60 kJ. Provision should be made to re-tap selected piles to confirm the set after adjacent piles have been driven, in accordance with MTO's current Special Provision.

5.4.2 Resistance to Lateral Loads

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

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

$$k_h = \frac{k_{s1}}{5B}$$

where B is the pile diameter (m) and k_{s1} is the constant of horizontal subgrade reaction, as given below

It is assumed that the abutment pile caps will be perched above the Innisfil Sixth Line cut grade to provide a greater driven length. The native soils above the cut grade were not investigated during this stage of field work. Based on regional geological mapping, it is likely that these soils will consist of clayey silt till, similar to that encountered below the cut grade. For very stiff to hard clayey silt till, as might be expected in the upper portion of the deposit above Elevation 291 m, the range in value of k_{s1} may be taken as 25 MPa/m to 60 MPa/m. Below Elevation 291, the range in value of k_{s1} should be taken as 50 MPa/m to 100 MPa/m in the structural analysis. These values will have to be confirmed following the detailed design stage of the subsurface investigation.

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

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

5.4.3 Frost Protection

All pile caps should be provided with 1.5 m of soil cover for frost protection.

5.5 Lateral Earth Pressures

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

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the abutments and walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular

backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00

- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the abutment wall, in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be used in accordance with OPSS 501.06.

- The granular fill may be placed either in a zone with width equal to at least 1.5 m behind the back of the stem (Case I from OHBDC Figure 6-7.4.1) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II from OHBDC Figure 6-7.4.4).

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

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

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

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

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

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

5.6 Embankment and Permanent Cut Slope Design

Based on the topographic information on the Engineering and Title Records plates and on site reconnaissance, the existing embankment side slopes along Highway 400 and permanent cut slopes along Innisfil Sixth Line are formed at a gradient of approximately 2 horizontal to 1 vertical (2H:1V). For widening of the Highway 400 embankment, the new side slopes should be formed at a maximum gradient of 2H:1V. The embankment widening should be carried out using conventional fill placement and compaction practices, and benching of the existing embankment side slopes should be carried out to key in the new fill.

If additional permanent cut work is required along Innisfil Sixth Line, a maximum gradient of 2H:1V may be assumed for preliminary design.

5.7 Design and Construction Considerations

5.7.1 Groundwater Control

Groundwater seepage into the footing or pile cap excavations will occur from the surficial fill or from water-bearing lenses or interlayers of granular soil within the clayey silt till deposit, although the quantity is expected to be relatively minor. Pumping from properly-filtered sumps or a filtered drain placed at the base of the excavation should provide sufficient groundwater control during foundation construction works. Surface water run-off, which is expected to be more significant than groundwater seepage, should be directed away from the footing excavations.

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

5.7.2 Excavation

Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. The footing excavations will extend a minimum of 1.5 m below lowest surrounding grade, generally through hard clayey silt till soils although it is expected that interlayers of granular soil could be encountered within the till. The hard clayey silt till soils would be classified as Type 1 soil, but granular interlayers, if encountered, would be classified as Type 2 soil. Temporary open-cut slopes should therefore

be maintained no steeper than 1 horizontal to 1 vertical (1H:1V) to within 1.2 m of the excavation base; below this, the excavation sides may be maintained near-vertical. Where space restrictions dictate, foundation works could also be carried out within a braced excavation.

5.7.3 Settlement

Deformation of the ground due to foundation loading will result in settlement of the bridge abutments and superstructure. This settlement will be differential with respect to the existing overpass structure, and also could vary along the new structure depending on the variability and consistency/relative density of the founding soils as well as the activities during construction. The potential for differential settlement should be reassessed during the detailed design stage once the proposed bridge configuration is established.

5.7.4 Obstructions

Cobbles were encountered within the clayey silt till deposits as noted on the records for Boreholes B3-1 and B3-2. It is noted that cobbles and boulders are inherent in glacially-derived soils, and should therefore be expected during footing excavation, pile driving and / or temporary shoring system installation. Where boulders are encountered within footing excavations, they should be removed and the sub-excavated areas should be backfilled with well-compacted Granular 'A' or lean concrete.

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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

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 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 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	c_u, s_u 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 (LV-laboratory vane test)
γ	unit weight

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

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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 δ
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

RECORD OF BOREHOLE No B3-1

1 OF 1 **METRIC**

PROJECT 001-1143F
W.P. 30-95-00
DIST SW HWY 400
DATUM Geodetic
LOCATION N 4902360.3; E 290986.9
BOREHOLE TYPE 108mm DIAMETER SOLID STEM AUGERS
DATE Oct.25/2000
ORIGINATED BY AZ
COMPILED BY LCC
CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
291.2	GROUND SURFACE									
0.0	Asphalt									
290.7	Sand and Gravel (Fill)									
0.5	Clayey Silt with sand, trace to some gravel (Till) Hard Brown becoming grey at 1.4m depth Moist	1	SS	67						
		2	SS	130						
		3	SS	138						3 52 31 14
		4	SS	104						
		5	SS	138						
		6	SS	135						
	Cobble at 6.4m depth	7	SS	100/.10						
		8	SS	101						
283.0 / 8.2	END OF BOREHOLE									

Notes:
1. Water level in open borehole at 7.4m depth (Elev.283.8m) immediately after completion of drilling. Water level rose to 6.9m depth (Elev.284.3m) about 10 minutes after completion of drilling.
2. Piezometer could not be found on January 19 or March 15, 2001; piezometer presumed destroyed.

ON_MOT 0011143F.GPJ ON_MOT.GDT 14/1/02

+³, ×³: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No B3-2

1 OF 1 **METRIC**

PROJECT 001-1143F
W.P. 30-95-00
DIST SW HWY 400
DATUM Geodetic
LOCATION N 4902360.3; E 290986.9
BOREHOLE TYPE 108mm DIAMETER SOLID STEM AUGERS
DATE Oct.25/2000
ORIGINATED BY AZ
COMPILED BY LCC
CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
290.9	GROUND SURFACE									
0.0	Asphalt									
	Sand and Gravel (Fill)									
0.3	Silty Sand, some gravel, trace clay (Fill) Compact Brown Moist	1	SS	11						
	Cobble at 2.1m depth	2	SS	28						
288.8 / 2.1	Clayey Silt with sand, trace to some gravel (Till) Hard Brown becoming grey at 4.6m depth Moist	3	SS	77						
		4	SS	78						
		5	SS	106						
		6	SS	130						
		7	SS	71						
		8	SS	135						
	Cobble at 8.7m depth	9	SS	104						
		10	SS	105						
279.6 / 11.3	END OF BOREHOLE									

Notes:
1. Water level in open borehole at 4.0m depth (Elev.286.9m) on completion of drilling operations.

ON_MOT 0011143F.GPJ ON_MOT.GDT 14/1/02

+³, ×³: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

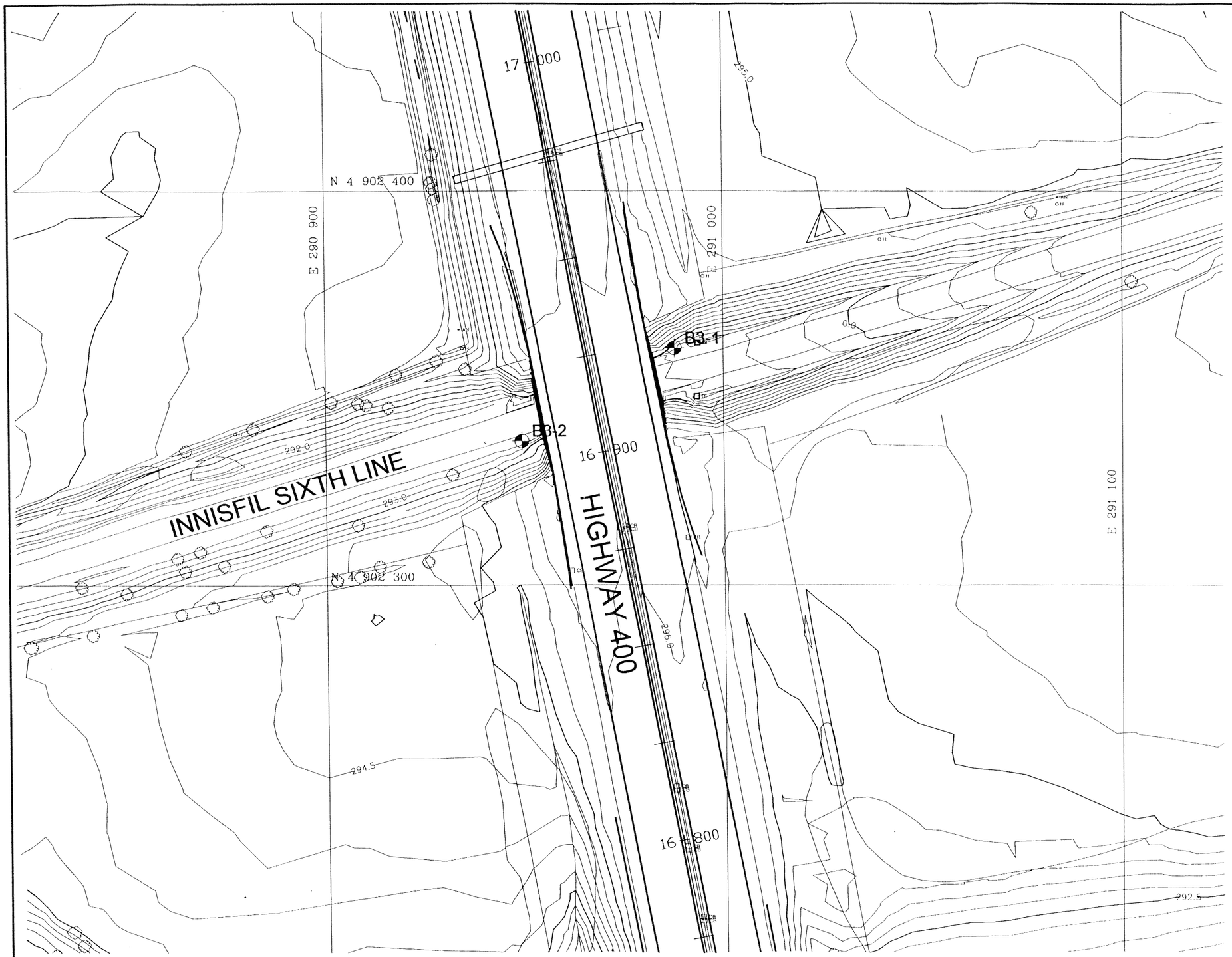
DIST HWY 400
CONT. No.
GWP No. 30-95-00

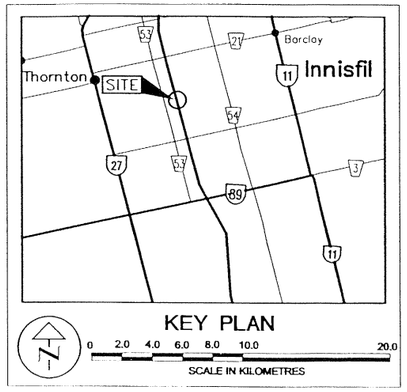
INNISFIL SIXTH LINE OVERPASS
HWY 400
BOREHOLE LOCATION PLAN

SHEET

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA





KEY PLAN

0 2.0 4.0 6.0 8.0 10.0 20.0
SCALE IN KILOMETRES

LEGEND

Borehole, previous investigation

Borehole, present investigation

No.	ELEVATION	LOCATION NORTHING	LOCATION EASTING
B3-1	291.2	4,902,360.3	290,986.9
B3-2	290.9	4,902,336.7	290,949.2

REFERENCE

This drawing was created from digital file "33807.dwg" provided by URS Cole Sherman

0 10 20 30 40 50 100m
SCALE IN METRES

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

NO.	DATE	BY	REVISION

Geocres No.

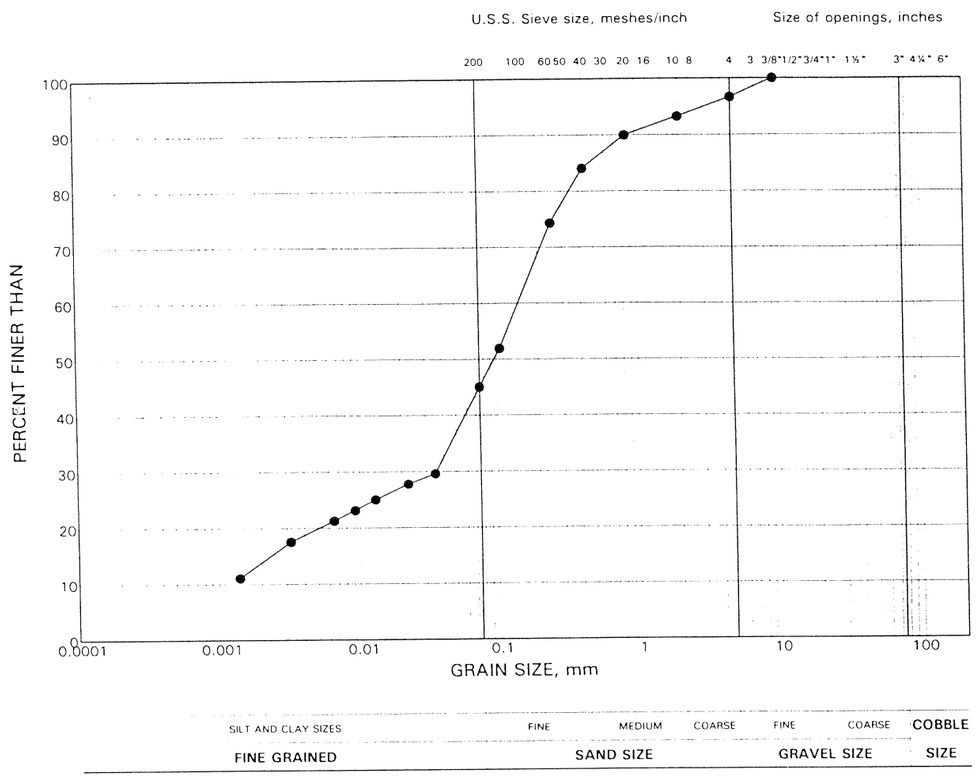
HWY. No. 400		PROJECT NO.: 001-1143F	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-211
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

P1143F08.DWG

GRAIN SIZE DISTRIBUTION TEST RESULT

Clayey Silt Till

FIGURE 1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	B3-1	3	288.7

Project 001-1143F

Golder Associates

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Golder Associates

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT INNISFIL BEACH ROAD OVERPASS STRUCTURE SITE 30-210 HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11 G.W.P. 30-95-00

Submitted to:

URS Cole, Sherman
75 Commerce Valley Drive East
Thornhill, Ontario
L3T 7N9

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PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
INNISFIL BEACH ROAD OVERPASS
STRUCTURE SITE 30-210
HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00

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LIST OF DRAWINGS
Drawing 1 Innisfil Beach Road Overpass, Highway 400, Borehole Location Plan

LIST OF FIGURES
Figure 1 Grain Size Distribution Test Result, Clayey Silt Till

1.0 INTRODUCTION

Golder Associates Ltd. has been retained by URS Cole, Sherman (Cole, Sherman) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the ultimate widening of Highway 400 from 1 km south of Highway 89, northerly 30 km to Highway 11, in Simcoe County, Ontario. Foundation engineering services are required for the widening and / or replacement of eighteen existing overpass and underpass structures, as well as five structural culverts.

This report addresses the widening and / or replacement of the existing Innisfil Beach Road (also called Simcoe Road 21 and Thornton Sideroad) overpass structure. A foundation investigation has been carried out, in which two boreholes were advanced and in-situ and laboratory testing was conducted, to determine the subsurface conditions at the site for this preliminary design study.

The terms of reference for the scope of work are outlined in Golder Associates' Proposal No. P01-1192, dated June 2000.

2.0 SITE DESCRIPTION

The existing Innisfil Beach Road overpass structure is located about 9.5 km north of the Highway 89 interchange and about 6 km south of the Molson Park Drive interchange, in the Town of Innisfil, Simcoe County. Innisfil Beach Road is also known locally as Simcoe Road 21 and Thornton Sideroad. The MTO has designated this overpass as Structure Site No. 30-210.

At this existing structure, the Highway 400 grade is at about Elevation 308 m, about 3 m higher than the surrounding grade. Innisfil Beach Road has been constructed in a cut up to about 2.5 m deep, with its grade at about Elevation 302.5 m under Highway 400.

The existing single-span overpass structure was constructed in the early 1950s under Contract 49-65. According to the general layout drawing for this contract, which was provided by Morrison Hershfield (the structural designers for this preliminary study), the abutments are supported on 3.7 m wide spread footings founded below the Innisfil Beach Road grade, at Elevation 300.4 m at the east end, rising to Elevation 300.6 m at the west end of the footings. The associated retaining walls are supported on 1.4 m wide spread footings at the same founding levels.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at this site in October 2000, at which time two boreholes were drilled. Borehole B4-1 was advanced on the east side of Highway 400, on the north side of Innisfil Beach Road, to a depth of about 6 m below the local road grade. Borehole B4-2 was advanced in the southeast quadrant of the interchange to a depth of 11 m, extending to about 7.5 m below the Innisfil Beach Road cut grade.

The investigation was carried out using a bombardier-mounted B-57 drill rig supplied and operated by Master Soil Investigations Ltd. of Weston, Ontario. Both boreholes were advanced using 108 mm diameter solid stem augers. Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. The water levels in the open boreholes were observed throughout the drilling operations, and a piezometer was installed in Borehole B4-2 to permit monitoring of the groundwater levels at the site.

The field work was supervised on a full-time basis by members of our staff who located the boreholes in the field, directed the drilling, sampling, and in-situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder Associates' laboratory in Mississauga for further examination. Index and classification tests consisting of water content determinations, Atterberg Limits tests and grain size distribution analyses were carried out on selected soil samples.

The borehole locations and elevations were surveyed by Callon Dietz, Ontario Land Surveyors. The borehole elevations are referenced to geodetic datum, and the northing and easting co-ordinates are referenced to the MTM NAD83 survey system. The borehole locations, together with elevations and northing and easting co-ordinates, are shown on the attached Drawing 1.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

This 30 km section of Highway 400 traverses, from south to north, the following physiographic regions as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, Third Edition, 1984): the Simcoe Lowlands; the Peterborough Drumlin Field; a second lobe of the Simcoe Lowlands; and the Simcoe Uplands. Along Highway 400, the Simcoe Lowlands are present from the southern limit of the project to just south of Innisfil Creek (about 1 km north of Highway 89), and again from Essa Road (Simcoe Road 30, formerly Highway 27) to about 1 km north of Dunlop Street (Simcoe Road 90, formerly Highway 90). The Peterborough Drumlin Field occupies the belt between these lobes of the Simcoe Lowlands, extending from just south of Innisfil Creek to Essa Road. The Simcoe Uplands extend from about 1 km north of Dunlop Street to beyond the northern limit of the project at Highway 11.

The two sections where Highway 400 crosses the Simcoe Lowlands consist of two lobes of a sand plain which include the shores of Kempenfelt Bay, the Nottawasaga River and Innisfil Creek. The surficial soils of these sections of the Simcoe Lowlands consist primarily of sand, although silt, clay or peat may be found in low-lying areas.

The surficial soils in the Peterborough Drumlin Field, in which the Innisfil Beach Road site is located, consist primarily of gravelly sand till or sand and gravel deposits. Drumlins (glacially-shaped hills) are more frequent in the southern portion of the section of the Peterborough Drumlin Field traversed by Highway 400. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins.

The surficial soils in the Simcoe Uplands physiographic region are primarily sandy silt till deposits, known to contain occasional boulders. Low-lying areas may be infilled with shallow sand and gravel deposits, which are shoreline deposits of a former glacial lake that once flooded the area.

4.2 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 given on the Record of Borehole sheets and Figure 1. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

Boreholes B4-1 and B4-2 were advanced on the east and west sides of Highway 400, respectively. The locations and ground surface elevations for these borings are shown on the attached Drawing 1.

In summary, the subsoils at the site consist of topsoil and fill / reworked soils overlying a deposit of hard clayey silt till, which contains interlayers of silty sand. A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil and Fill / Reworked Soil

Between 200 mm and 500 mm of topsoil was encountered in the boreholes. In Borehole B4-2, 1.7 m of fill or reworked soil was encountered below the topsoil. This fill consists of clayey silt to silty sand containing trace quantities of organics. Standard Penetration Test (SPT) 'N' values of 11 and 27 blows per 0.3 m of penetration were measured in the fill, indicating that the clayey silt fill has a stiff consistency and the silty sand fill has a compact relative density.

4.2.2 Clayey Silt Till

A deposit of clayey silt till underlies the topsoil and, where present, the fill or reworked soil. The till deposit extends to the maximum depth investigated (Elevation 294.8 m, approximately 7.5 m below Innisfil Beach Road grade).

The clayey silt till contains a significant proportion of sand and trace to some gravel; the grain size distribution test result for a representative sample of this till is shown on Figure 1. The till deposit grades to a non-plastic silty sand containing trace quantities of clay and gravel, as recovered in one sample from Borehole B4-2, at about Elevation 299 m. The deposit also contains a 600 mm thick interlayer of silty sand containing some gravel, the surface of which was encountered in Borehole B4-1 at about Elevation 300.6 m. Cobbles were inferred within the till deposit from grinding and resistance to augering, as noted on the borehole records.

The natural moisture contents measured on samples of the clayey silt till ranged from 6 to 9 per cent. Atterberg Limits testing measured plastic limits of 10 to 11 per cent, liquid limits of 13 to 14 per cent, and plasticity indices of 3 to 4 per cent. The results of the Atterberg Limits testing indicate that the clayey silt till is inorganic and of low plasticity.

The measured SPT 'N' values ranged from 43 to greater than 100 blows per 0.3 m of penetration, but were typically in excess of 100 blows per 0.3 m of penetration, indicating that this till deposit has a hard consistency.

4.3 Groundwater Conditions

Both boreholes were dry during and on completion of the drilling operations, and recovered samples of the clayey silt till were dry to moist, with measured natural moisture contents between 6 and 9 per cent. The groundwater level in the piezometer installed in Borehole B4-2 was measured at Elevation 297.9 m, approximately 4.5 m below the Innisfil Beach Road grade in March 2001.

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

GOLDER ASSOCIATES LTD.

Lisa C. Coyne, P.Eng.,
Geotechnical Engineer

for Anne S. Poschmann, P.Eng.,
Principal

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

LCC/ASP/FJH/clg
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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
INNISFIL BEACH ROAD OVERPASS
STRUCTURE SITE 30-210
HIGHWAY 400 WIDENING FROM 1 KM SOUTH
OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

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

It is understood that Highway 400 will be widened from its existing six-lane configuration to an interim configuration of eight lanes, and an ultimate configuration of ten lanes, and that an alternative for a twelve-lane express / collector system is under consideration between Molson Park Drive and Duckworth Street in Barrie. Throughout the project length, it is expected that the existing highway platform will be widened by between 13 m and 30 m. Widening and / or replacement of the existing Innisfil Beach Road overpass will be necessary.

Based on the general layout drawing for the existing single-span structure, the abutments and associated retaining walls are supported on spread footings founded between Elevation 300.4 m and 300.6 m, below the Innisfil Beach Road cut grade of about Elevation 302.5 m. The Highway 400 grade at the structure site is at about Elevation 308 m.

5.2 Bridge Foundation Options

The soils at the site consist of topsoil and fill / reworked soils overlying a deposit of hard clayey silt till, having Standard Penetration Test (SPT) 'N' values typically greater than 100 blows per 0.3 m of penetration. Based on these subsurface conditions, it is recommended that the widening or replacement structure be founded on spread footings placed on the hard clayey silt till below the Innisfil Beach Road grade. Consideration could also be given to the use of perched abutments, founded on spread footings placed within the approach embankments.

If the overpass structure is to be replaced, integral abutments could be considered at this site. In that case, the abutments could be supported on driven steel H-pile foundations. The subsoils at and below about Elevation 302 m (approximately 6 m below the Highway 400 grade, at the Innisfil Beach Road cut grade) are generally very hard, with measured Standard Penetration Test

'N' values in excess of 100 blows per 0.3 m of penetration. Heavy pile driving will be encountered within this deposit. In order to achieve the necessary pile length for integral abutments, consideration should be given to perching the pile caps within the Highway 400 approach embankments. Alternatively, pre-augering could be carried out within the hard clayey silt till to ensure an adequate pile length is achieved.

Preliminary recommendations for spread footings, including perched abutments, and for driven steel H-pile foundations are provided in the following sections.

5.3 Spread Footings

For preliminary design of the bridge abutment footings, spread footings may be founded on the hard clayey silt till deposit at or below Elevation 302.5 m; a minimum soil cover of 1.5 m should be provided. Where widening is proposed, the widened footing should match the existing founding level (i.e Elevation 300.4 m to 300.6 m). Any associated wing wall or retaining wall footings may be stepped upward away from the abutments such that a minimum soil cover of 1.5 m is maintained above the underside of the footings.

Alternatively, consideration could be given to the use of abutment footings perched on a granular pad within the approach embankment.

5.3.1 Axial Geotechnical Resistance

Spread footings placed on the properly prepared clayey silt till deposit at the design elevation given above may be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 1,200 kPa, assuming a 3.5 m wide footing. The settlement of footings founded on the clayey silt till will be dependent on the footing size and configuration, and on the applied loads. For preliminary design purposes, the geotechnical resistance at Serviceability Limit States (SLS) may be taken as 900 kPa. The geotechnical resistance at SLS will have to be reviewed following the detailed design stage of subsurface investigation, once the footing size, configuration and loadings are known.

For spread footings placed within the approach embankments on a compacted Granular 'A' pad, a factored geotechnical resistance at ULS of 900 kPa may be assumed for preliminary design. The geotechnical resistance at SLS will depend on the thickness of Granular 'A' and the consistency and thickness of the underlying soils; a value of 350 kPa may be assumed for preliminary design.

The geotechnical resistances provided herein are given under the assumption that the loads will be applied perpendicular to the surface of the footings; where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the Ontario Highway Bridge Design Code (OHBDC).

5.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footing and the subsoils should be calculated in accordance with Section 6-8.4.3 of the OHBDC. The angle of friction between the concrete and the undisturbed clayey silt till founding soils should be taken as 24 degrees; the corresponding coefficient of friction, tan δ, would then be 0.45. Where "perched" abutment footings are adopted, the angle of friction between the concrete footings and the compacted Granular 'A' pad should be taken as 30 degrees; the corresponding coefficient of friction would be 0.58.

5.3.3 Frost Protection

The footings should be provided with a minimum of 1.5 m of soil cover for frost protection.

5.4 Driven Steel H-Piles

Consideration could be given to supporting the replacement structure or widenings on steel H-piles driven to found within the hard clayey silt till deposit. The surface of the hard clayey silt till was encountered at about Elevation 302 m in Boreholes B4-1 and B4-2; this level is about 6 m below the Highway 400 grade, at approximately the Innisfil Beach Road cut grade. Because of the hard nature of the till and the presence of cobbles and boulders, hard driving conditions should be anticipated. Consideration should be given to perching the pile caps within the Highway 400 approach embankments in order to maximize the driven pile length. If the piles are driven from the Innisfil Beach Road cut grade, it is expected to be necessary to pre-auger the upper 2 m of the pile length within the till in order to provide a starting guide for driving of the piles. Further, it may be necessary to pre-auger deeper in order to achieve an adequate pile length for integral abutment considerations. As discussed in Section 5.7.3, cobbles and boulders are anticipated within the till deposit, and provision must be made in both the Contract Documents and the contractor's methods and equipment to handle such obstructions.

For preliminary design where the abutment pile caps are placed at the Innisfil Beach Road cut grade, a pile tip level of Elevation 296 m may be assumed to allow a 5 m pile length to be achieved. If the pile caps are perched within the Highway 400 approach embankments, the pile tip level should be taken at Elevation 299 m for preliminary design.

5.4.1 Axial Geotechnical Resistance

For preliminary design, the factored axial resistance at ULS for steel HP 310 x 110 H-piles driven to found within the hard clayey silt till at or below the tip elevations given above may be taken as 1,400 kN. The axial resistance at SLS for 25 mm of settlement may be taken as 1,200 kN.

To achieve the above design resistances, the piles should be driven to a final set of no less than 15 blows per 25 mm of penetration using a hammer with rated energy of about 50 kJ, and not exceeding 60 kJ. Provision should be made to re-tap selected piles to confirm the set after adjacent piles have been driven, in accordance with MTO's current Special Provision.

5.4.2 Resistance to Lateral Loads

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

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

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

where n_h is the constant of subgrade reaction
z is the depth (m)
B is the pile diameter (m)

For cohesive soils:

$$k_h = \frac{k_{s1}}{5B}$$

where B is the pile diameter (m) and
k_{s1} is the constant of horizontal subgrade reaction, as given below

The piles will be driven through embankment fill (if the pile caps are perched within the Highway 400 approach embankments) and into the clayey silt till. The following ranges for the value of n_h may be assumed in the structural analysis; these values will have to be confirmed following the detailed design stage of the subsurface investigation.

Soil Unit	n_h	k_{s1}
Embankment Fill above Elevation 303.5 m	5 to 15 MPa/m	—
Clayey Silt Till between Elevations 303.5 m and 302 m	—	25 to 60 MPa/m
Clayey Silt Till below Elevation 302 m	—	50 to 100 MPa/m

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

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

5.4.3 Frost Protection

The pile caps should be provided with 1.5 m soil of cover for frost protection.

5.5 Lateral Earth Pressures

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

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the abutments and walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00

- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the abutment wall, in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be used in accordance with OPSS 501.06.

- The granular fill may be placed either in a zone with width equal to at least 1.5 m behind the back of the stem (Case I from OHBDC Figure 6-7.4.1) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II from OHBDC Figure 6-7.4.4).

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

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

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

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

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

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

5.6 Embankment and Permanent Cut Slope Design

Based on the topographic information on the Engineering and Title Records plates and on site reconnaissance, the existing embankment side slopes along Highway 400 and permanent cut slopes along Innisfil Beach Road are formed at a gradient of approximately 2 horizontal to 1 vertical (2H:1V). For widening of the Highway 400 embankment, the new side slopes should be formed at a maximum gradient of 2H:1V. The embankment widening should be carried out using conventional fill placement and compaction practices, and benching of the existing embankment side slopes should be carried out to key in the new fill.

If additional permanent cut work is required along Innisfil Beach Road, a maximum gradient of 2H:1V may be assumed for preliminary design.

5.7 Design and Construction Considerations

5.7.1 Dewatering

Groundwater seepage into the footing excavations could occur from water-bearing lenses or interlayers of granular soil within the clayey silt till deposit, although the quantity is expected to be minor. Pumping from properly-filtered sumps or a filtered drain placed at the base of the excavation should provide sufficient groundwater control during foundation works. Surface water run-off, which is expected to be more significant than groundwater seepage, should be directed away from the footing excavations.

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

5.7.2 Excavation

The footing excavations will extend a minimum of 1.5 m below lowest surrounding grade, generally through hard clayey silt till soils although interlayers of granular soil could be encountered within the till. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. The hard clayey silt till soils would be classified as Type 1 soil, but granular interlayers, if encountered, would be classified as Type 2 soil. Temporary open-cut slopes should therefore be maintained no steeper than 1 horizontal to 1 vertical (1H:1V) to within 1.2 m of the excavation base; below this depth, the excavation sides may be formed near-vertical. Where space restrictions dictate, foundation works could also be carried out within a braced excavation.

5.7.3 Obstructions

An obstruction was encountered at 1.4 m depth in Borehole B4-1, and cobbles were noted within the hard clayey silt till deposit in both boreholes. Cobbles and boulders should be expected excavation and pile driving for deep foundations or temporary shoring systems, if these options are adopted.

GOLDER ASSOCIATES LTD.

Lisa C. Coyne, P.Eng.,
Geotechnical Engineer

for Anne S. Poschmann, P.Eng.,
Principal

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

LCC/ASP/FJH/clg
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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

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 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 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 (LV-laboratory vane test)
γ	unit weight

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

S:\FINALDAT\ABBREV\2000\LOFA-D00.DOC

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

RECORD OF BOREHOLE No B4-1

1 OF 1 **METRIC**

PROJECT 001-1143F
W.P. 30-95-00
DIST SW HWY 400
DATUM Geodetic
LOCATION N 4905036.5; E 290509.0
BOREHOLE TYPE 108mm DIAMETER SOLID STEM AUGERS
DATE Oct.24/2000
ORIGINATED BY AZ
COMPILED BY LCC
CHECKED BY ASP

SOIL PROFILE: ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED)	PLASTIC LIMIT w_p / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L (WATER CONTENT (%))	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
302.7	GROUND SURFACE										
0.0	Topsoil										
0.2	Clayey Silt with sand, some gravel (Till) Hard Brown Moist		1	SS	120		302				
			2	SS	125		301				11 37 43 9
300.6											
2.1	Silty Sand, some gravel, trace clay Very dense Brown Dry		3	SS	103/.15						
300.0							300				
2.7	Clayey Silt with sand, some gravel (Till) Hard Brown Moist		4	SS	151/.15						
			5	SS	101/.15		299				
			6	SS	109/.15		298				
	Cobbles at 5.5m and 5.8m depth						297				
296.5			7	SS	113/.15						
6.2	END OF BOREHOLE										

Notes:
1. Refusal to auger advance was encountered at 1.4m depth. Borehole was relocated 1m west and drilling continued.
2. Borehole dry on completion of drilling operations.

+³, ×³: Numbers refer to Sensitivity O³% STRAIN AT FAILURE

ON_MOT 0011143F.GPJ ON_MOT.GDT 14/1/02

RECORD OF BOREHOLE No B4-2

1 OF 1

METRIC

PROJECT 001-1143F

W.P. 30-95-00

DIST SW HWY 400

DATUM Geodetic

LOCATION N 4904989.7; E 290437.7

BOREHOLE TYPE 108mm DIAMETER SOLID STEM AUGERS

DATE Oct.24/2000

ORIGINATED BY AZ

COMPILED BY LCC

CHECKED BY ASP

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (o UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × REMOULDED)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L (WATER CONTENT (%) 10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
305.6	GROUND SURFACE									
0.0 / 305.1	Topsoil									
0.5 / 304.1	Clayey Silt with sand, some organics (Fill) Stiff Dark brown Moist	1	SS	11		305				
1.5 / 303.4	Silty Sand, trace gravel, clay and organics (Fill) Compact Dark brown Moist	2	SS	27		304		o		
2.2	Clayey Silt with sand, trace gravel (Till) Hard Brown becming grey at 7.3m depth Moist	3	SS	48		303				
		4	SS	141/.25				o		
	Cobbles at 3.7m depth	5	SS	101/.15		302				
		6	SS	115/.15		301		o		
	Silty sand, trace clay and gravel (Till) encountered in Sample 7.	7	SS	43		300				
						299		o		
		8	SS	210	▽	298		o		
						297				
		9	SS	240		296				
294.8 / 10.8		10	SS	135/.15		295				
	END OF BOREHOLE									

Notes:
1. Borehole dry on completion of drilling operations.
2. Water level in piezometer measured at 7.7m depth (Elev.297.9m) on March 15, 2001.

+ ³, × ³: Numbers refer to Sensitivity

o ³% STRAIN AT FAILURE

DIST HWY 400
CONT. No.
GWP No. 30-95-00

INNISFIL BEACH ROAD OVERPASS
HWY 400
BOREHOLE LOCATION PLAN

SHEET

Golder Associates

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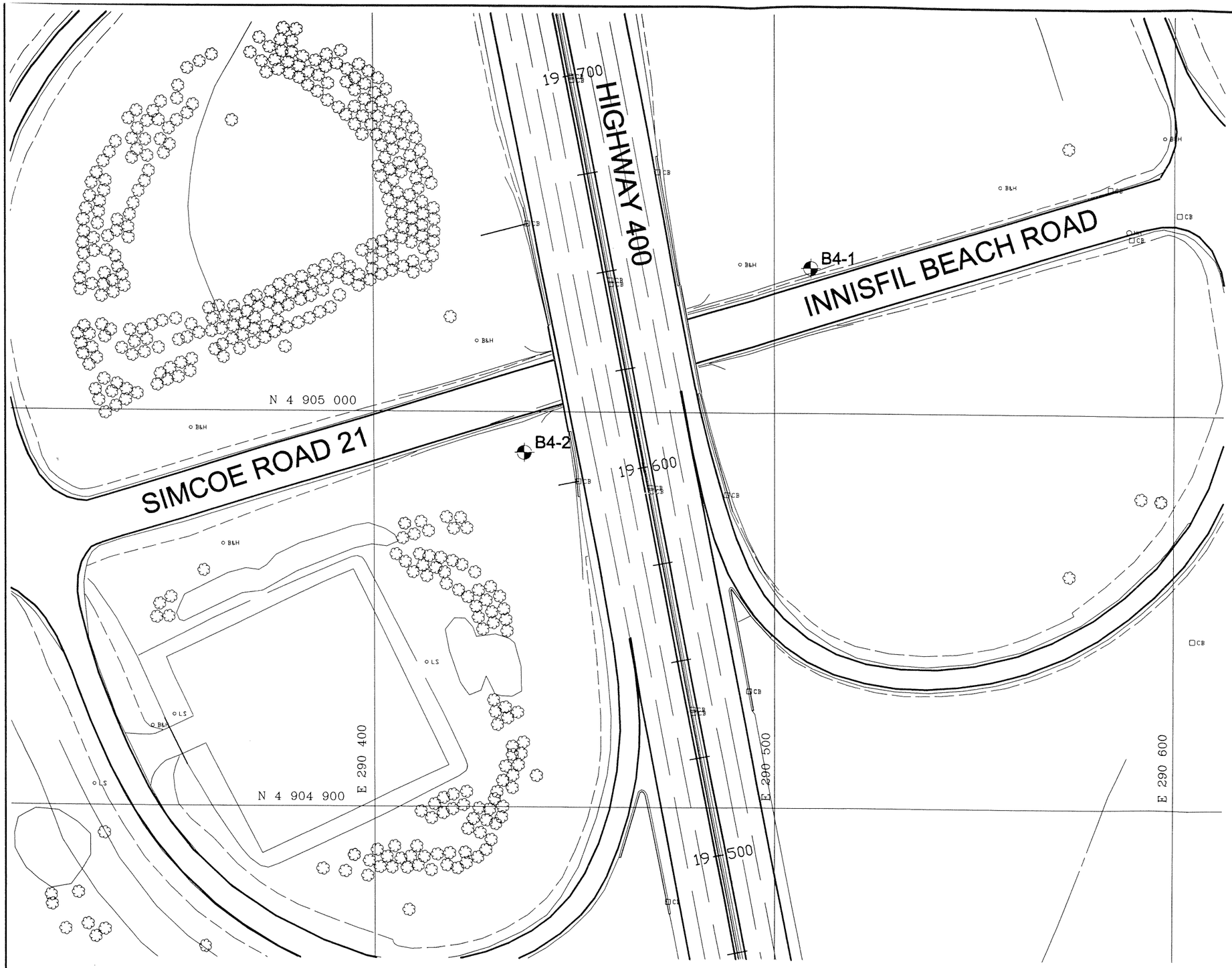
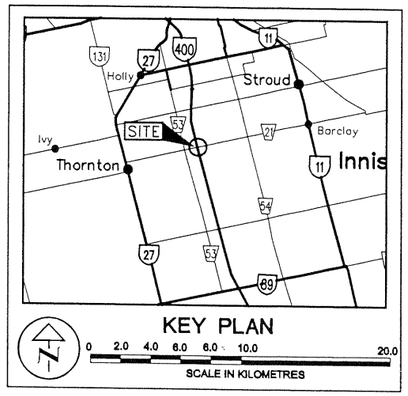
MISSISSAUGA, ONTARIO, CANADA





LEGEND

- Borehole, previous investigation
- Borehole, present investigation

No.	ELEVATION	LOCATION NORTHING	LOCATION EASTING
B4-1	302.7	4,905,036.5	290,509.0
B4-2	305.6	4,904,989.7	290,437.7

REFERENCE
This drawing was created from digital file "50201.dwg" provided by URS Cole Sherman

0 10 20 30 40 50 100m
SCALE IN METRES

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

NO.	DATE	BY	REVISION

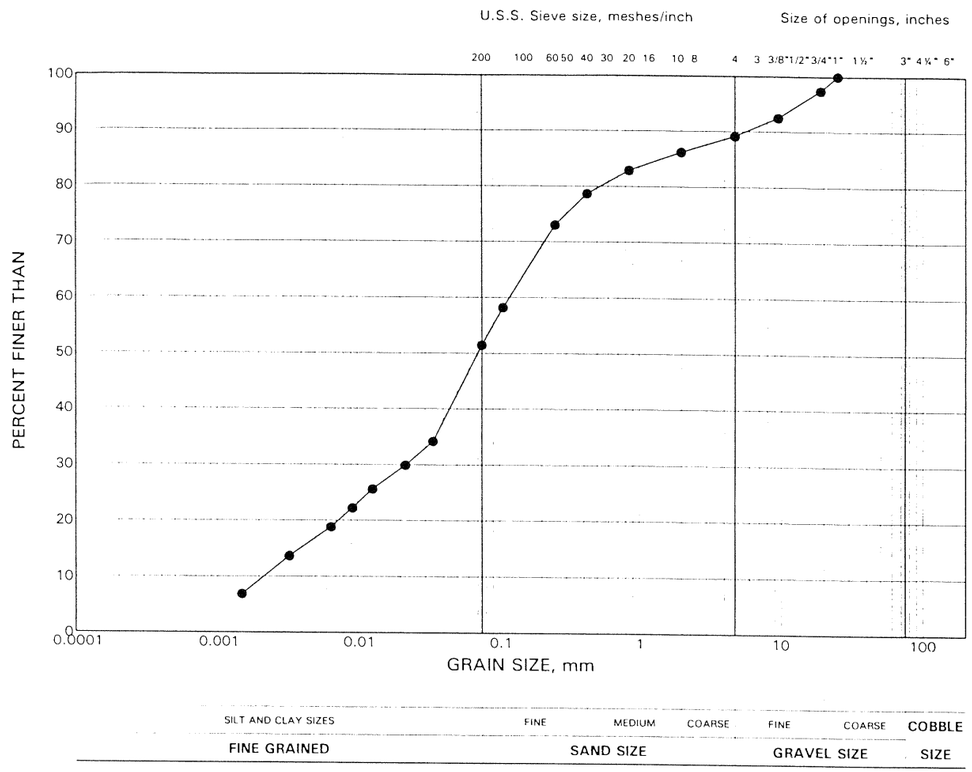
Geocres No.

HWY. No. 400	PROJECT NO.: 001-1143F		
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-210
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

P1143709.DWG

GRAIN SIZE DISTRIBUTION TEST RESULT
Clayey Silt Till

FIGURE 1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	B4-1	2	300.9

Project 001-1143F

Golder Associates

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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT BARRIE / COLLINGWOOD RAILWAY OVERPASS STRUCTURE SITE 30-209 HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11 G.W.P. 30-95-00

Submitted to:

URS Cole, Sherman
75 Commerce Valley Drive East
Thornhill, Ontario
L3T 7N9

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January 2002 001-1143F-5

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PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
BARRIE / COLLINGWOOD RAILWAY OVERPASS
STRUCTUE SITE 30-209
HIGHWAY 400 WIDENING FROM 1 KM SOUTH
OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00

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Drawing 1
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LIST OF DRAWINGS

Drawing 1 Barrie/Collingwood Railway Overpass, Highway 400, Borehole Location Plan

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Appendix A Records of Boreholes – 1958 Investigation

1.0 INTRODUCTION

Golder Associates Ltd. has been retained by URS Cole, Sherman (Cole, Sherman) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the ultimate widening of Highway 400 from 1 km south of Highway 89, northerly 30 km to Highway 11, in Simcoe County, Ontario. Foundation engineering services are required for the widening and / or replacement of eighteen existing overpass and underpass structures, as well as five structural culverts.

This report addresses the widening and / or replacement of the existing Barrie / Collingwood Railway (formerly CN Rail) overpass structure that is located south of Barrie. Existing subsurface data for this site from an investigation conducted for the Department of Highways, Ontario (DHO) in 1958 were used to determine the subsurface conditions for this preliminary design study. The 1958 report was prepared by Racey, MacCallum and Associates Limited (*"Foundation Investigation for the Proposed Canadian National Railway Crossing on Highway 400, Innisfil Township, Near Barrie"*, dated April 1958 – GEOCRES File No. 31D-186).

The terms of reference for the scope of work are outlined in Golder Associates' Proposal No. P01-1192, dated June 2000.

2.0 SITE DESCRIPTION

The existing single-span Barrie / Collingwood Railway (formerly CN Rail) overpass structure is located about 600 m north of Innisfil Beach Road (Simcoe Road 21) in the Town of Innisfil, Simcoe County. The MTO has designated this overpass as Structure Site 30-209.

At this structure site, the Highway 400 grade is at about Elevation 312.5 m, declining to the north and south. Highway 400 has been constructed in fill with approach embankments up to 8.5 m in height. The Barrie / Collingwood Railway grade is at about Elevation 304 m, near the original ground surface which varied from Elevation 303 m to 304 m in the vicinity of the site.

According to the general layout drawing for this contract, which was provided by Morrison Hershfield (the structural designers for this preliminary study), the abutments and associated retaining walls are supported on spread footings which are founded at about Elevation 301.3 m. The existing spread footings at the abutments are approximately 3 m wide.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at this site for the Department of Highways, Ontario (DHO) in March 1958, by Racey, MacCallum and Associates Limited. At that time, a total of three boreholes with accompanying dynamic cone test (probeholes) and three separate probeholes were advanced in the vicinity of the north and south abutments of the then-proposed overpass structure. Boreholes 1 and 2 and Probehole P3 were advanced at the south abutment, and Probeholes P4 and P5 and Borehole 6 were advanced at the north abutment. The three boreholes were advanced to between 8 m and 9 m depth below the original ground surface, to approximately Elevation 295 m to 296 m. The probeholes were advanced to between 2 m and 3 m below the ground surface.

In the boreholes, samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. The groundwater conditions in the open borehole were observed during and following the drilling operations. No laboratory testing was carried out in conjunction with this 1958 investigation.

The borehole locations and elevations, referenced to the geodetic datum, were established by Racey, MacCullum and Associates Limited. Approximate northing and easting co-ordinates consistent with the MTM NAD83 survey system, currently in use on this project, have been determined by Golder Associates based on the borehole locations given in the 1958 report. The approximate borehole locations and northing and easting co-ordinates are shown on the attached Drawing 1.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

This 30 km section of Highway 400 traverses, from south to north, the following physiographic regions as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, Third Edition, 1984): the Simcoe Lowlands; the Peterborough Drumlin Field; a second lobe of the Simcoe Lowlands; and the Simcoe Uplands. Along Highway 400, the Simcoe Lowlands are present from the southern limit of the project to just south of Innisfil Creek, and again from Essa Road (Simcoe Road 30, formerly Highway 27) to about 1 km north of Dunlop Street (Simcoe Road 90, formerly Highway 90. The Peterborough Drumlin Field occupies the belt between these lobes of the Simcoe Lowlands, extending from just south of Innisfil Creek, which is located about 1 km north of Highway 89, to Essa Road. The Simcoe Uplands extend from about 1 km north of Dunlop Street to beyond the northern limit of the project at Highway 11.

The two sections where Highway 400 crosses the Simcoe Lowlands consist of two lobes of a sand plain which include the shores of Kempenfelt Bay, the Nottawasaga River and Innisfil Creek. The surficial soils of these sections of the Simcoe Lowlands consist primarily of sand, although silt, clay or peat may be found in low-lying areas.

The surficial soils in the Peterborough Drumlin Field consist primarily of gravelly sand till or sand and gravel deposits. Drumlins (glacially-shaped hills) are more frequent in the southern portion of the section of the Peterborough Drumlin Field traversed by Highway 400. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins. The Barrie / Collingwood Railway overpass site is located within the Peterborough Drumlin Field physiographic region.

The surficial soils in the Simcoe Uplands physiographic region are primarily sandy silt till deposits, known to contain occasional boulders. Low-lying areas may be infilled with shallow sand and gravel deposits, which are shoreline deposits of a former glacial lake that once flooded the area.

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes are given on the Record of Borehole sheets contained in Appendix A. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

Boreholes 1 and 2 and Probehole P3 were advanced in the vicinity of the south abutment, and Borehole 6 and Probeholes P4 and P5 were advanced in the vicinity of the north abutment. All of the test holes were advanced from approximately the Barrie / Collingwood Railway grade. The approximate locations and ground surface elevations for these borings and probeholes are shown on the attached Drawing 1.

In summary, the soils below the rail grade at this site consist of a generally compact to very dense sand and gravel to gravelly sand deposit, overlying very dense sand. A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Gravelly Sand to Sand and Gravel

The boreholes encountered an upper deposit consisting of gravelly sand to sand and gravel, noted to contain boulders. This deposit ranged from about 5.5 m to greater than 7.5 m in thickness at the borehole locations.

In the upper 2 m of the deposit, the measured Standard Penetration Test (SPT) "N" values ranged from 4 to 48 blows, but were typically between 12 and 28 blows per 0.3 m of penetration. These SPT results indicate that the upper 2 m of the deposit is loose to dense, but has a predominantly compact relative density. The dynamic cone test results are in accordance with these SPT values. Below 2 m depth, the measured SPT "N" values ranged from 54 to greater than 100 blows per 0.3 m of penetration, indicating that the gravelly sand to sand and gravel has a very dense relative density. The dynamic cone tests were terminated at about 2 m to 3 m depth, in this very dense material, on reaching 80 to 100 blows per 0.3 m of penetration.

4.2.2 Sand

The gravelly sand to sand and gravel overlies a deposit of sand, which was noted to contain boulders. The sand deposit was encountered below Elevations 296 m and 298.5 m in Boreholes 2 and 6, respectively, and it is at least 1 m to 2.5 m thick as encountered in these boreholes. The measured SPT "N" values were greater than 100 blows per 0.3 m of penetration, indicating that the sand has a very dense relative density.

4.3 Groundwater Conditions

The 1958 report indicates that no water table was encountered during the borehole investigation, which was carried out in March of that year. However, it should be noted that groundwater levels are expected to fluctuate seasonally and are expected to rise during wet periods of the year.

GOLDER ASSOCIATES LTD.

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Designated MTO Contact

LCC/ASP/FJH/clg
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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
BARRIE / COLLINGWOOD RAILWAY OVERPASS
STRUCTURE SITE 30-209
HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides preliminary foundation design recommendations for the widening and / or replacement of the existing Barrie / Collingwood Railway overpass structure, associated with the widening of Highway 400. The recommendations are preliminary only and are based on interpretation of the factual data obtained from a limited number of boreholes advanced during a 1958 subsurface investigation at this site. The interpretation and recommendations provided are intended for planning purposes only, to provide the information necessary at this stage of the study. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the planning of the project. Further foundation investigation will be required at this structure site as part of the detailed design stage of the project.

It is understood that Highway 400 will be widened from its existing six-lane configuration to an interim configuration of eight lanes, and an ultimate configuration of ten lanes, and that an alternative for a twelve-lane express / collector system is under consideration between Molson Park Drive and Duckworth Street in Barrie. Throughout the project length, it is expected that the existing highway platform will be widened by between 13 m and 30 m. Widening and / or replacement of the existing rail overpass structure will be necessary.

Based on the general layout drawing for the existing single-span structure, the abutments and associated retaining and wing walls are supported on spread footings founded at about Elevation 301.3 m. Highway 400 has been constructed in fill, with its grade at about Elevation 312.5 m and approach embankments up to about 8.5 m in height. The CN Rail grade is at about Elevation 304 m.

5.2 Bridge Foundation Options

The soils below the rail grade consist of a compact to very dense deposit of gravelly sand to sand and gravel, overlying a deposit of very dense sand. Boulders were noted on the borehole records within both deposits.

Based on these subsurface conditions and on the existing structure foundations, it is recommended that the widened portions of the structure be founded on spread footings placed on the very dense gravelly sand to sand and gravel deposit. Consideration could also be given to the use of perched abutments, founded on spread footings placed on a compacted granular pad within the approach embankments.

Alternatively, if integral abutments are under consideration for the replacement structure, the abutments could be supported on steel H-piles driven to found within the very dense sand to gravelly sand deposit. It should be noted that heavy driving will be encountered due to the presence of cobbles and boulders (as encountered in the 1958 borehole investigation) and the very dense nature of the native soil deposits.

Preliminary recommendations for spread footings, including perched abutments, and for deep foundations are provided in the following sections.

5.3 Spread Footings

For preliminary design of the abutment foundations, spread footings may be placed at a design founding level of Elevation 301.3 m, to be founded on the very dense gravelly sand to sand and gravel deposit. Any associated wing wall or retaining wall footings may be stepped upward away from the abutments such that a minimum soil cover of 1.5 m is maintained above the underside of the footings.

Alternatively, consideration could be given to the use of abutment footings perched within the approach embankments.

5.3.1 Axial Geotechnical Resistance

Spread footings placed on the properly prepared gravelly sand to sand and gravel deposit at the design elevation given above may be designed for a factored geotechnical resistance at Ultimate Limit States (ULS) of 1,000 kPa, assuming a 3 m wide footing. Where boulders are encountered within footing areas, they should be removed and the sub-excavated area should be backfilled with well-compacted Granular 'A' fill or lean concrete. The settlement of footings founded on the gravelly sand to sand and gravel soils will be dependent on the footing size and configuration, and on the applied loads. The majority of this settlement will take place during construction itself; for preliminary design purposes, the geotechnical resistance at Serviceability Limit States (SLS) may be taken as 650 kPa. The geotechnical resistances at ULS and SLS will have to be reviewed following the detailed design stage of subsurface investigation, once the groundwater conditions at the site are confirmed and the footing size, configuration and loadings are known.

For spread footings placed within the approach embankments on a compacted Granular 'A' core, a factored geotechnical resistance at ULS of 900 kPa may be assumed for preliminary design. The

geotechnical resistance at SLS will depend on the thickness of Granular 'A' and the consistency and thickness of the underlying soils; a value of 350 kPa may be assumed for preliminary design.

The geotechnical resistances provided herein are given under the assumption that the loads will be applied perpendicular to the surface of the footings; where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the Ontario Highway Bridge Design Code (OHBDC).

5.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footing and the subsoils should be calculated in accordance with Section 6-8.4.3 of the OHBDC. The angle of friction between the concrete and the undisturbed, very dense gravelly sand to sand and gravel founding soils should be taken as 27 degrees; the corresponding coefficient of friction, tan δ, would then be 0.5. Where "perched" abutment footings are adopted, the angle of friction between the concrete footings and the compacted Granular 'A' pad should be taken as 30 degrees; the corresponding coefficient of friction would be 0.58.

5.3.3 Frost Protection

The footings should be provided with a minimum of 1.5 m of soil cover for frost protection.

5.4 Driven Steel H-Piles

Consideration could be given to supporting the widened or replacement overpass structure on steel H-piles driven to found within the very dense sand to gravelly sand deposit. Based on the Highway 400 grade at Elevation 312.5 m, it is expected that the highest pile cap level would be about Elevation 309 m. The results of the 1958 boreholes indicate that refusal to further cone penetration was met at about Elevation 301 m. At or below Elevation 297 m, the measured SPT 'N' values are typically greater than 100 blows per 0.3 m of penetration. Based on this information, it is anticipated that the pile founding level would be below about Elevation 295 m, approximately 9 m below the rail grade and about 17 m to 18 m below the Highway 400 grade.

It is noted that additional borehole investigation will be required at the proposed abutment locations during detailed design in order to confirm the nature of the overburden, to establish the

relative density of the soils below Elevation 295 m, and to confirm the groundwater conditions at the site.

5.4.1 Axial Geotechnical Resistance

For preliminary design, the factored axial resistance at ULS for steel HP 310 x 110 H-piles driven to found within the very dense sand to gravelly sand deposit below about Elevation 295 m may be taken as 1,400 kN. The axial resistance at SLS for a single pile, for 25 mm of settlement, may be taken as 1,200 kN.

As a guide, to achieve the above design resistances, the piles should be driven to a final set of no less than 10 blows per 25 mm of penetration using a hammer with rated energy of about 50 kJ, and not exceeding 60 kJ. The actual set requirements will have to be established based on the Contractor's driving equipment. Provision should be made to re-tap selected piles to confirm the set after adjacent piles have been driven, in accordance with MTO's current Special Provision.

5.4.2 Resistance to Lateral Loads

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

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

$$k_h = \frac{n_h z}{B}$$

Where n_h is the constant of subgrade reaction
z is the depth (m)
B is the pile diameter (m)

The following ranges in the value of n_h may be used in the structural analysis; these values will require confirmation following the detailed design stage of subsurface investigation:

Stratum / Elevation	n_h
Gravelly Sand to Sand and Gravel Above Elevation 301.5 m	10 to 20 MPa/m
Gravelly Sand to Sand Below Elevation 301.5 m	15 to 25 MPa/m

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

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

5.4.3 Frost Protection

The pile caps should be provided with 1.5 m soil cover for frost protection.

5.5 Lateral Earth Pressures

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

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the abutments and walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00

- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the abutment wall, in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be used in accordance with OPSS 501.06.

- The granular fill may be placed either in a zone with width equal to at least 1.5 m behind the back of the stem (Case I from OHBDC Figure 6-7.4.1) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II from OHBDC Figure 6-7.4.4).

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

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

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

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

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

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

5.6 Approach Embankment Design

The existing Barrie / Collingwood Railway overpass approach embankments are up to 8.5 m in height. For preliminary design purposes, the side slopes of the widened or new Highway 400 embankments should be formed at a maximum gradient of 2 horizontal to 1 vertical (2H:1V). In order to minimize differential settlement between the widened portions of Highway 400 and the existing embankments, the use of granular fill is recommended for the widening. The majority of settlement of granular fills will occur during construction whereas the majority of settlement of cohesive fills, if used, would occur post-construction. The construction of the new embankments

should be carried out using conventional fill placement and compaction practices. Benching of the existing embankment side slopes, in accordance with OPSD 208.01, should be carried out to key in the new fill and, again, minimize differential settlement between the existing and new portions of the embankments.

5.7 Design and Construction Considerations

5.7.1 Dewatering

The 1958 subsurface investigation report indicates that no water table was encountered during the subsurface investigation; this will have to be confirmed during the detailed design stage of subsurface investigation. Based on the information available, groundwater seepage into the footing excavations is expected to be minor. Pumping from properly-filtered sumps or a filtered drain placed at the base of the excavation should provide sufficient groundwater control during foundation works. Surface water run-off, which is expected to be more significant than groundwater seepage, should be directed away from the footing excavations.

5.7.2 Excavation

The footing or pile cap excavations will extend a minimum of 1.5 m below lowest surrounding grade, generally through compact gravelly sand to sand and gravel soils into the very dense gravelly sand to sand and gravel deposit. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. The compact soils below the Barrie / Collingwood Railway grade would be classified as Type 2 soil; temporary open-cut slopes should therefore be maintained no steeper than 1 horizontal to 1 vertical (1H:1V). Where space restrictions dictate, footing excavations could also be carried out within a braced excavation.

5.7.3 Obstructions

Boulders were noted on the borehole records in the 1958 subsurface investigation. Cobbles and boulders should therefore be expected during footing excavation and temporary shoring system installation, if such a system is required.

GOLDER ASSOCIATES LTD.

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Designated MTO Contact

LCC/ASP/FJH/clg
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DIST	HWY 400
CONT. No.	
GWP No.	30-95-00

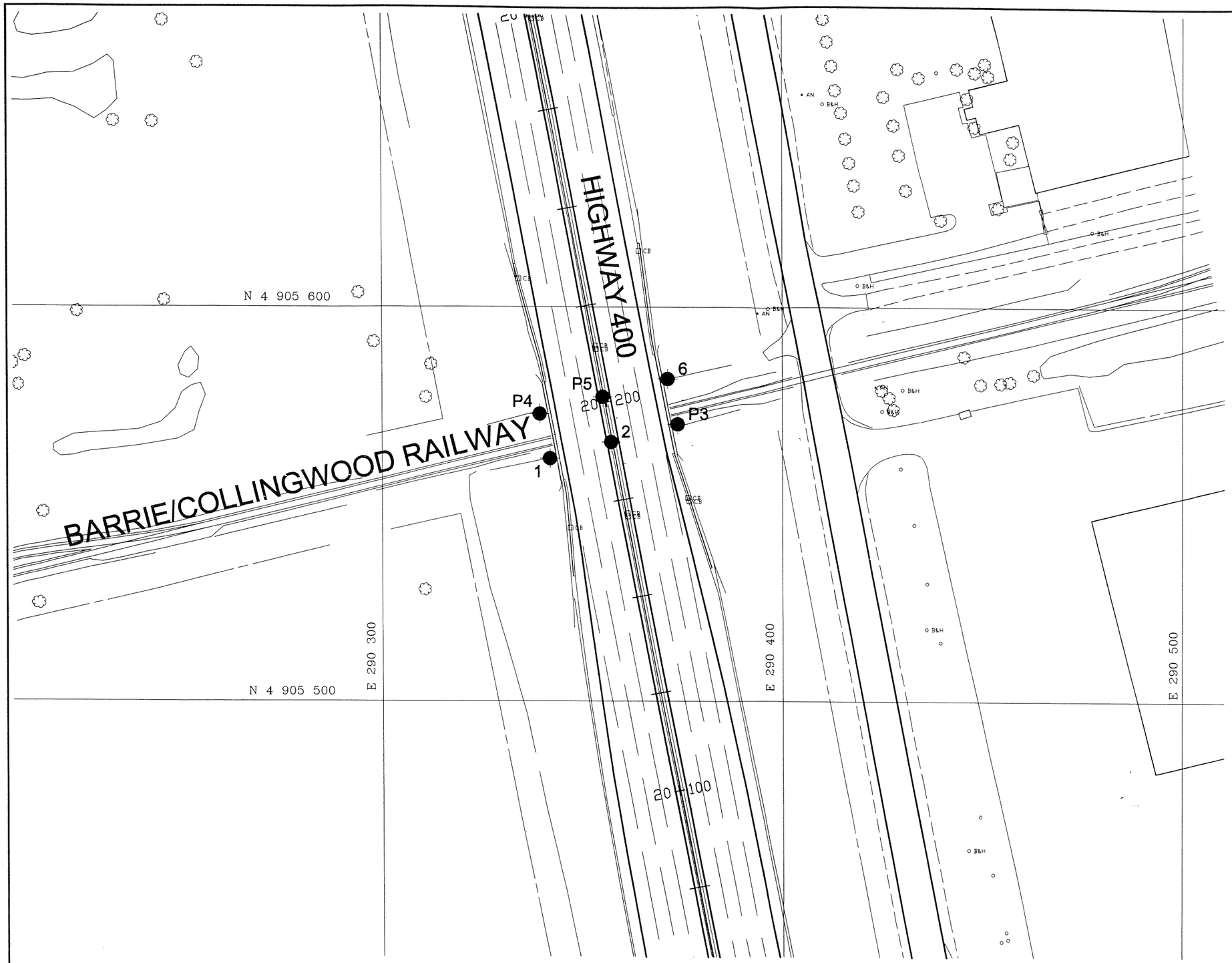
SHEET

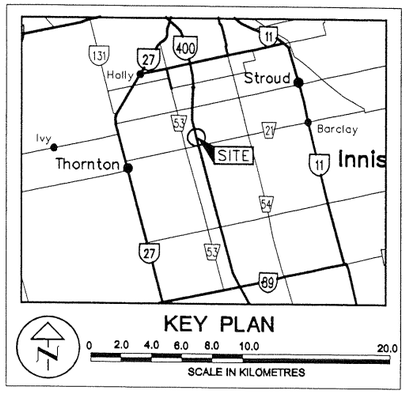
BARRIE/COLLINGWOOD RAILWAY OVERPASS HWY 400

BOREHOLE LOCATION PLAN

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA





KEY PLAN

0 2.0 4.0 6.0 8.0 10.0 20.0
SCALE IN KILOMETRES

LEGEND

Borehole, previous investigation

Borehole, present investigation

No.	ELEVATION	LOCATION NORTHING	LOCATION EASTING
1	303.9	4,905,562	290,342
2	303.6	4,905,566	290,357
P3	303.9	4,905,570	290,374
P4	303.9	4,905,573	290,340
P5	303.9	4,905,577	290,355
6	303.8	4,905,582	290,371

REFERENCE

This drawing was created from digital file "50202.dwg" provided by URS Cole Sherman

0 10 20 30 40 50 100m

SCALE IN METRES

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

NO.	DATE	BY	REVISION

Geocres No.

HWY. No. 400		PROJECT NO.: 001-1143F	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-209
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

P1143F10.DWG

APPENDIX A

RECORDS OF BOREHOLES
1958 INVESTIGATION

Order No. S-500/T-1128 Enclosure No. 3

RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

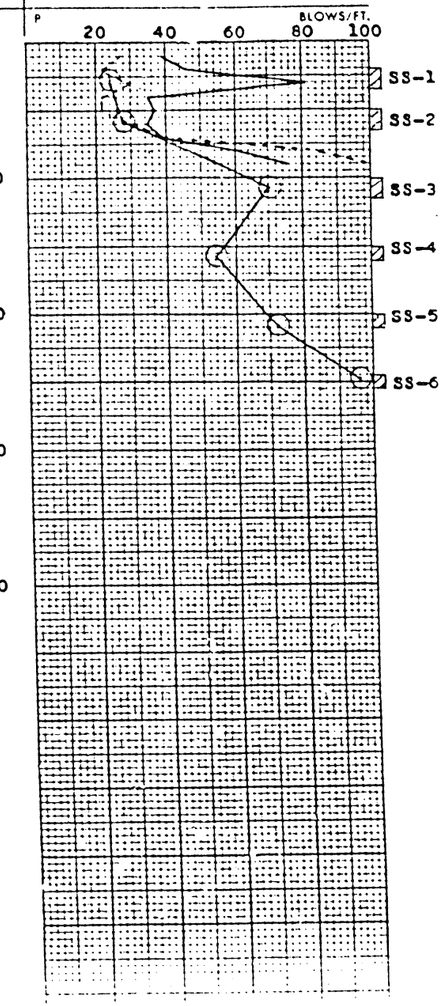
Engineering Data Sheet for Borehole: 1

Project: FOUNDATION INVESTIGATION FOR PROPOSED C.N.R. OVER-PASS
Location: HWY #400, INNISFIL TWP.,NEAR BARRIE, ONT.
Hole Location: See Enclosure No 2
Hole Elevation and Datum: 997 Based on contours shown on D.H.O. Plan E-3247-1
Field Supervisor: H.G. Prep.: P.M. Date: 26. 3.'58
Driller: M.C. Checked:

LEGEND
Shear Strength (C)
Unconfined compression
Vane test and sensitivity (S)
Penetration Resistance (P)
2" Split tube
2" Dia. Cone
Casing

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
303.9m		997	
	Medium dense coarse gravelly sand and boulders		
	Medium dense fine brown sand and gravel		
301.1m	END OF CONE TEST AND CASED HOLE	988	10
	Very dense fine brown gravelly sand		
	Very dense brown gravelly medium sand		20
296.3m	END OF HOLE	972	
			30
			40

STRENGTH AND PENETRATION RESISTANCE



Order No. S-500/T-1128 Enclosure No. 4

RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

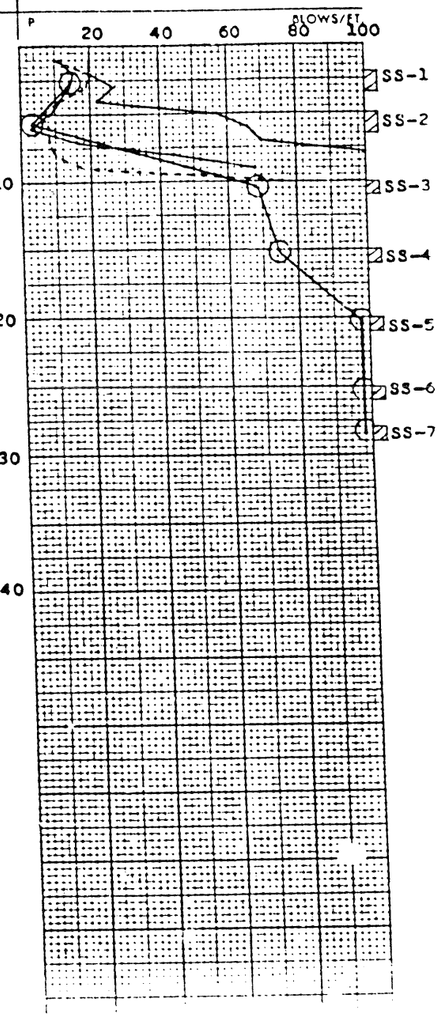
Engineering Data Sheet for Borehole: 2

Project: FOUNDATION INVESTIGATION FOR PROPOSED C.N.R.OVER-
Location: HWY #400, INNISFIL TWP.,NEAR BARRIE, ONT. PASS
Hole Location: See Enclosure No 2
Hole Elevation and Datum: 996 Based on contours on D.H.O. Plan
Field Supervisor: H.G. Prep.: P.M. E-3247-1
Driller: M.C. Checked: Date: 26.3.'58

LEGEND
Shear Strength (C)
Unconfined compression
Vane test and sensitivity (S)
Penetration Resistance (P)
2" Split tube
2" Dia. Cone
Casing

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
303.6m		996	
	Loose brown medium sand and gravel		
301.1m	END OF FIRST CONE TEST	988	
300.8m	END OF SECOND CONE TEST	987	
300.5m	END OF CASED HOLE	986	10
	Dense light brown gravelly fine sand		20
	Very dense fine light brown sand		30
294.7m	END OF HOLE	967	40

STRENGTH AND PENETRATION RESISTANCE



Order No. S-500/T-1128 Enclosure No. 5

RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

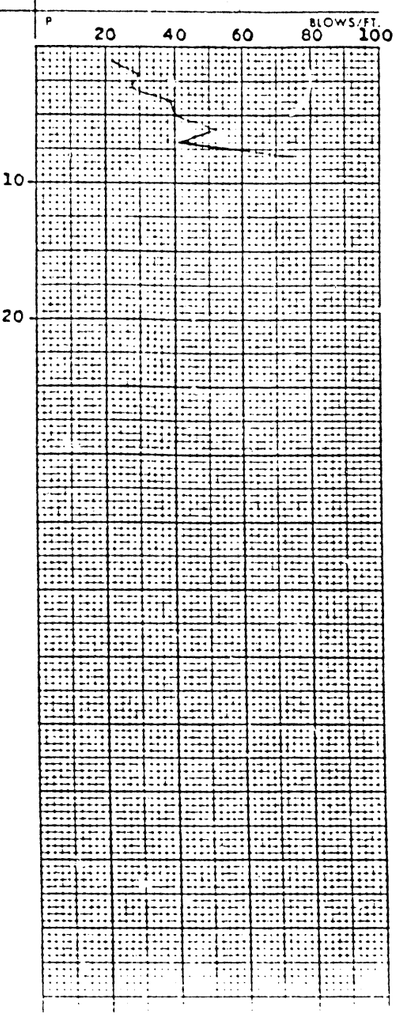
Engineering Data Sheet for Borehole: P3

Project: FOUNDATION INVESTIGATION FOR PROPOSED C.N.R.OVER-
Location: ON HWY #400,INNISFIL TWP.NEAR BARRIE,ONT. PASS
Hole Location: See Enclosure No 2
Hole Elevation and Datum: 997 Based on Contours on D.H.O.plan
Field Supervisor: H.G. Prep.: P.M. E-3247-1
Driller: M.C. Checked: Date: 26.3.'58

LEGEND
Shear Strength (C)
Unconfined compression
Vane test and sensitivity (S)
Penetration Resistance (P)
2" Split tube
2" Dia. Cone
Casing

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
303.9m		997	
301.4m	END OF CONE TEST	989	10
			20

STRENGTH AND PENETRATION RESISTANCE



Order No. S-500/T-1128 Enclosure No. 6

RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

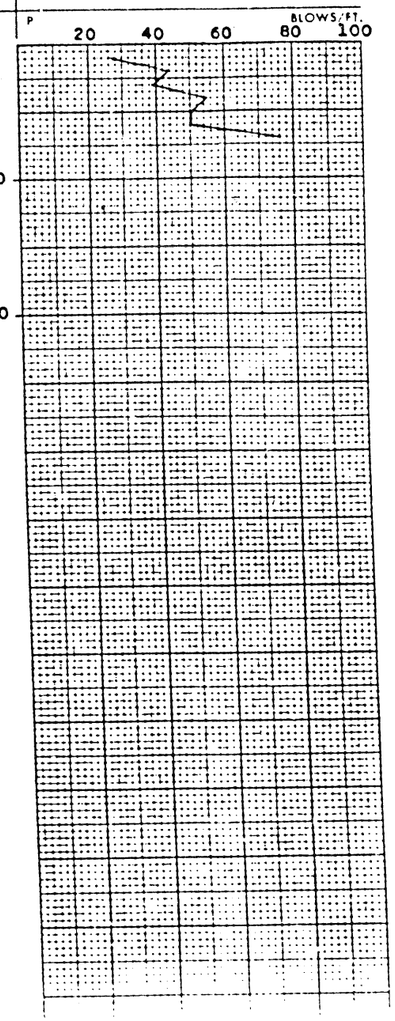
Engineering Data Sheet for Borehole: P4

Project: FOUNDATION INVESTIGATION FOR PROPOSED C.N.R.OVER-
Location: ON HWY.#400,INNISFIL TWP.NEAR BARRIE,ONT. PASS
Hole Location: See Enclosure No 2
Hole Elevation and Datum: 997 Based on contours on D.H.O.Plan
Field Supervisor: H.G. Prep.: P.M. E-3247-1
Driller: M.C. Checked: Date: 26.3.'58

LEGEND
Shear Strength (C)
Unconfined compression
Vane test and sensitivity (S)
Penetration Resistance (P)
2" Split tube
2" Dia. Cone
Casing

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
303.9 m		997	
301.8m	END OF CONE TEST	990	10 20

STRENGTH AND PENETRATION RESISTANCE



Order No. S-500/T-1128 Enclosure No. 7

RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

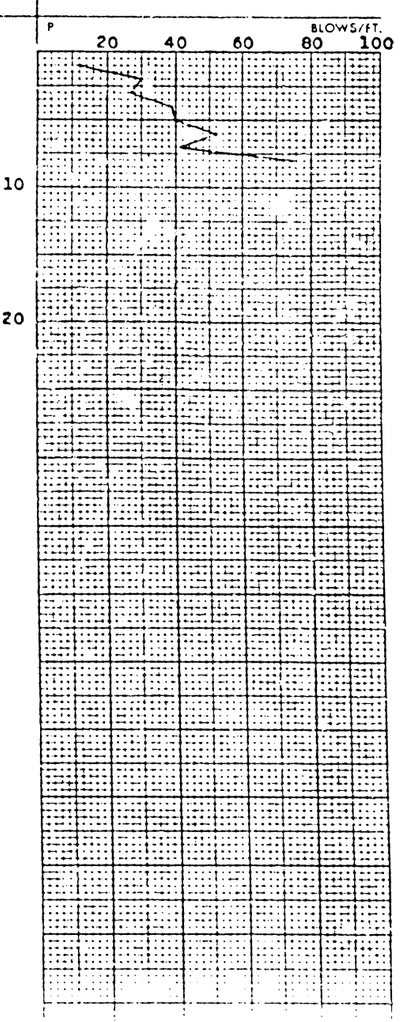
Engineering Data Sheet for Borehole: P5

Project: FOUNDATION INVESTIGATION FOR PROPOSED C.N.R.OVER-
Location: ON HWY.#400,INNISFIL TWP.,NEAR BARRIE,ONT. PASS
Hole Location: See Enclosure No 2
Hole Elevation and Datum: 997 Based on contours shown on D.H.O.
Field Supervisor: H.G. Prep.: P.M. plan E-3247-1
Driller: M.C. Checked: Date: 26.3.'58

LEGEND
Shear Strength (C)
Unconfined compression
Vane test and sensitivity (S)
Penetration Resistance (P)
2" Split tube
2" Dia. Cone
Casing

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
303.9 m		997	
301.4m	END OF CONE TEST	989	10 20

STRENGTH AND PENETRATION RESISTANCE



Order No. S-500/T-1128

Enclosure No. 8

RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

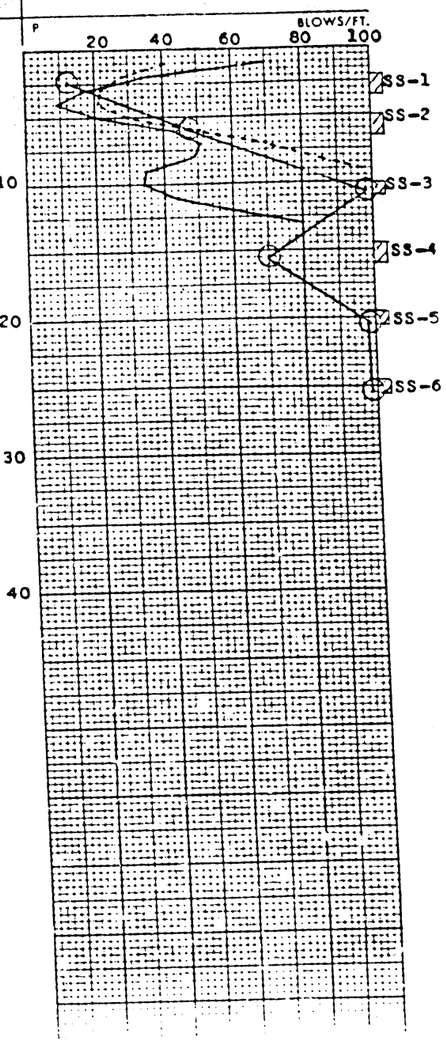
Engineering Data Sheet for Borehole: 6

Project: FOUNDATION INVESTIGATION FOR PROPOSED C.N.R.OVER-
Location: ON HWY.#400,INNISFIL TWP.,NEAR BARRIE,ONT. PASS
Hole Location: See Enclosure No 2
Hole Elevation and Datum: 996.7 Based on contours shown on D.H.O.plan E-3247-1
Field Supervisor: H.G. Prep.: P.M.
Driller: M.C. Checked: Date: 26.3.'58

LEGEND
Shear Strength (C)
Unconfined compression
Vane test and sensitivity (S)
Penetration Resistance (P)
2" Split tube
2" Dia. Cone
Casing

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
303.8m		996.7	
	Medium brown sand and fill		
	Fine light brown gravelly sand		
301.1m	END CASED HOLE	987.7	10
299.8m	END OF CONE TEST	983.7	
	Fine light brown sand and boulders		20
296.2m	END OF HOLE	971.7	
			30
			40

STRENGTH AND PENETRATION RESISTANCE



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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT VICTORIA STREET UNDERPASS STRUCTURE SITE 30-208 HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11 G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

Submitted to:

URS Cole, Sherman
75 Commerce Valley Drive East
Thornhill, Ontario
L3T 7N9

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PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
VICTORIA STREET UNDERPASS
STRUCTURE SITE 30-208
HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

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Drawing 1 Innisfil Tenth Line Underpass, Highway 400, Borehole Location Plan

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Figure 1 Grain Size Distribution Test Results, Silty Sand to Silt, Some Sand

1.0 INTRODUCTION

Golder Associates Ltd. has been retained by URS Cole, Sherman (Cole, Sherman) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the ultimate widening of Highway 400 from 1 km south of Highway 89, northerly 30 km to Highway 11, in Simcoe County, Ontario. Foundation engineering services are required for the widening and / or replacement of eighteen existing overpass and underpass structures, as well as five structural culverts.

This report addresses the replacement of the existing Victoria Street (Innisfil Tenth Line, formerly Stroud Road) underpass structure. A foundation investigation has been carried out, in which two boreholes were advanced and in-situ and laboratory testing was conducted, to determine the subsurface conditions at the site for this preliminary design study.

The terms of reference for the scope of work are outlined in Golder Associates' Proposal No. P01-1192, dated June 2000.

2.0 SITE DESCRIPTION

The existing Victoria Street (Innisfil Tenth Line, also formerly known as Stroud Road) underpass structure is located about 2.8 km north of the Innisfil Beach Road (Simcoe Road 21) interchange and 2.8 km south of the Molson Park Drive interchange, in the Town of Innisfil, Simcoe County. The MTO has designated this underpass as Structure Site No. 30-208.

In the vicinity of this structure site, Highway 400 has been constructed in a 3 m to 9 m deep cut. The Highway 400 grade declines northward from about Elevation 304 m to 303.5 m within the structure limits. Victoria Street is near the original ground surface, with its grade at about Elevation 310 m over Highway 400.

The two-span underpass structure was constructed in the late 1940s under Contract 46-48. According to the general layout drawing for this contract, which was provided by Morrison Hershfield (the structural designers for this preliminary study), the abutments, associated wing walls and the pier are supported on spread footings founded at about Elevation 302.5 m to 302.2 m, declining toward the east. The existing spread footings are approximately 2.7 m wide at the abutments, and 1.8 m wide at the centre pier.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at this site in October 2000, at which time two boreholes were drilled. Boreholes B6-1 and B6-2 were advanced on the east and west sides of Highway 400 to depths of 5 m and 8 m, respectively, below the Highway 400 cut grade.

The investigation was carried out using bombardier-mounted D-50 and B-57 drill rigs supplied and operated by Master Soil Investigations Ltd. of Weston, Ontario. Both boreholes were advanced using solid stem augers. Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. The water levels in the open boreholes were observed throughout the drilling operations, and a piezometer was installed in Borehole B6-1 to permit monitoring of the groundwater levels at the site.

The field work was supervised on a full-time basis by members of our staff who located the boreholes in the field, directed the drilling, sampling, and in-situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder Associates' laboratory in Mississauga for further examination. Index and classification tests consisting of water content determinations, Atterberg Limits tests and grain size distribution analyses were carried out on selected soil samples.

The borehole locations and elevations were surveyed by Callon Dietz, Ontario Land Surveyors. The borehole elevations are referenced to geodetic datum, and the northing and easting co-ordinates are referenced to the MTM NAD83 survey system. The borehole locations, together with elevations and northing and easting co-ordinates, are shown on the attached Drawing 1.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

This 30 km section of Highway 400 traverses, from south to north, the following physiographic regions as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, Third Edition, 1984): the Simcoe Lowlands; the Peterborough Drumlin Field; a second lobe of the Simcoe Lowlands; and the Simcoe Uplands. Along Highway 400, the Simcoe Lowlands are present from the southern limit of the project to just south of Innisfil Creek, and again from Essa Road (Simcoe Road 30, formerly Highway 27) to about 1 km north of Dunlop Street (Simcoe Road 90, formerly Highway 90). The Peterborough Drumlin Field occupies the belt between these lobes of the Simcoe Lowlands, extending from just south of Innisfil Creek, which is located about 1 km north of Highway 89, to Essa Road. The Simcoe Uplands extend from about 1 km north of Dunlop Street to beyond the northern limit of the project at Highway 11.

The two sections where Highway 400 crosses the Simcoe Lowlands consist of two lobes of a sand plain which include the shores of Kempenfelt Bay, the Nottawasaga River and Innisfil Creek. The surficial soils of these sections of the Simcoe Lowlands consist primarily of sand, although silt, clay or peat may be found in low-lying areas.

The surficial soils in the Peterborough Drumlin Field physiographic region, in which the Victoria Street site is located, consist primarily of gravelly sand till or sand and gravel deposits. Drumlins (glacially-shaped hills) are more frequent in the southern portion of the section of the Peterborough Drumlin Field traversed by Highway 400. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins.

The surficial soils in the Simcoe Uplands physiographic region are primarily sandy silt till deposits, known to contain occasional boulders. Low-lying areas may be infilled with shallow sand and gravel deposits, which are shoreline deposits of a former glacial lake that once flooded the area.

4.2 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 given on the Record of Borehole sheets and Figure 1. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

Boreholes B6-1 and B6-2 were advanced on the east and west sides of Highway 400, respectively, from approximately Highway 400 grade. The locations and ground surface elevations for these borings are shown on the attached Drawing 1.

In summary, the soils below the Highway 400 cut at this site consist of a very dense deposit of silty sand to silt. On the east side of the highway, 1.2 m of silty clay till was encountered underlying the cohesionless deposit, while on the west side of the highway, the silty sand to silt deposit grades to sand and gravel at about 7 m depth (approximately Elevation 297.5 m). A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil and Silty Sand, Some Organics

About 300 mm of topsoil was encountered in Borehole B6-1 on the east side of Highway 400. On the west side of the highway, Borehole B6-2 encountered an upper 600 mm layer of silty sand containing some organics. This silty sand is loose, with a measured Standard Penetration Test (SPT) 'N' value of 7 blows per 0.3 m of penetration.

4.2.2 Silty Sand to Silt, Some Sand

A cohesionless soil deposit, about 3.5 m to 6.5 m thick, was encountered below the topsoil and loose, organic silty sand. The deposit ranges in composition from silty sand to silt containing some sand. Layers of silty clay, up to 75 mm in thickness, were encountered within the silty sand to silt deposit in some of the samples. The grain size distribution test results obtained on representative samples of this deposit are shown on Figure 1.

The silty sand to silt soils were generally moist, with measured natural moisture contents ranging from 5 to 15 per cent. The soils became wet below about 3.2 m depth (Elevation 301.7 m) in Borehole B6-2.

The measured SPT 'N' values ranged from 50 to 190 blows per 0.3 m of penetration, but were typically greater than 100 blows per 0.3 m of penetration. The silty sand to silt deposit therefore has a very dense relative density.

4.2.3 Silty Clay Till

A deposit of silty clay till containing trace quantities of sand and gravel, as well as silt and sand seams, underlies the silty sand to silt deposit in Borehole B6-1. The top of the silty clay till was encountered at 3.7 m depth (Elevation 300.1 m). The deposit was not fully penetrated by the boring, but it is at least 1.2 m thick.

Atterberg Limits testing on a sample of the till measured a plastic limit of 16 per cent, a liquid limit of 28 per cent, and a plasticity index of 12 per cent. The results of the Atterberg Limits testing indicate that the silty clay till is inorganic and of low plasticity. The natural moisture content measured on a sample of the till was 23 per cent; this water content is considered to have been influenced by the presence of sand and silt seams within the till.

The measured SPT 'N' values were greater than 100 blows per 0.3 m of penetration, indicating that this till deposit has a hard consistency.

4.2.4 Sand and Gravel

A 0.7 m thick layer of sand and gravel was encountered at the base of Borehole B6-2, underlying the silty sand to silt deposit. This sand and gravel layer is very dense, with a measured SPT 'N' value of greater than 100 blows per 0.3 m of penetration.

4.3 Groundwater Conditions

Recovered samples from the silty sand to silt deposit were generally moist, although the samples became wet below about 3.2 m depth in Borehole B6-1. This increase in moisture content likely reflects perched water atop the silty clay till that was encountered in this borehole. It is possible that a shallow surficial aquifer exists within the sandy silt to silt deposit during wet periods of the year. However, both boreholes were dry on completion of the drilling operations in October 2000, and the piezometer sealed within the sandy silt to silt deposit in Borehole B6-1 was dry when measured in March 2001.

GOLDER ASSOCIATES LTD.

Lisa C. Coyne, P.Eng.,
Geotechnical Engineer

Anne S. Poschmann, P.Eng.,
Principal

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

LCC/ASP/FJH/clg
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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
VICTORIA STREET UNDERPASS
STRUCTURE SITE 30-208
HIGHWAY 400 WIDENING FROM 1 KM SOUTH
OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

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

It is understood that Highway 400 will be widened from its existing six-lane configuration to an interim configuration of eight lanes, and an ultimate configuration of ten lanes, and that an alternative for a twelve-lane express / collector system is under consideration between Molson Park Drive and Duckworth Street in Barrie. Throughout the project length, it is expected that the existing highway platform will be widened by between 13 m and 30 m. Replacement of the existing Victoria Street underpass structure will therefore be necessary.

Based on the general layout drawing for the existing two-span structure, the pier, abutments and associated retaining and wing walls are supported on spread footings founded at about Elevation 302.5 m to 302.2 m, declining eastward. Highway 400 has been constructed in cut, with its grade at about Elevation 304 m to 303.5 m within the structure limits. Victoria Street is near the original ground surface at the site, with its grade at about Elevation 310 m over Highway 400.

5.2 Bridge Foundation Options

The soils below the Highway 400 level consist of a thin layer of topsoil and organic silty sand overlying a deposit of very dense silty sand to silt, which in turn overlies deposits of hard silty clay till and very dense sand and gravel.

Based on these subsurface conditions, it is recommended that the new structure be founded on spread footings placed on the silty sand to silt stratum. Consideration could also be given to the use of perched abutments, founded on spread footings placed on a compacted granular pad within the approach embankments.

Alternatively, if integral abutments are under consideration for the replacement structure, the abutments could be supported on steel H-piles driven to found within the very dense silty sand to silt deposit. In this case, it is assumed that the pile cap would be perched above the existing Highway 400 grade in order to achieve the required pile length.

It should be noted that the boreholes put down during this preliminary phase of field work were drilled from the Highway 400 cut level. If perched footings or integral abutments supported on deep foundations are considered viable options, the subsoil conditions between the Victoria Street level and the Highway 400 grade will be required during the detailed design stage.

Preliminary recommendations for spread footings, including perched abutments, and for deep foundations are provided in the following sections.

5.3 Spread Footings

For preliminary design of the bridge abutment and pier footings, spread footings may be founded on the very dense silty sand to silt deposit at or below Elevation 303 m; a minimum soil cover of 1.5 m should be provided. Any associated wing wall or retaining wall footings may be stepped upward away from the abutments such that a minimum soil cover of 1.5 m is maintained above the underside of the footings.

Alternatively, consideration could be given to the use of abutment footings perched within the approach embankments.

5.3.1 Axial Geotechnical Resistance

Spread footings placed on the properly prepared silty sand to silt deposit at the design elevation given above may be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 1,000 kPa, assuming a 3 m wide footing. The settlement of footings founded on the silty sand to sandy silt will be dependent on the footing size and configuration, and on the applied loads. The majority of this settlement will take place during construction itself; for preliminary design purposes, the geotechnical resistance at Serviceability Limit States (SLS) may be taken as 800 kPa. The geotechnical resistance at SLS will have to be reviewed following the detailed design stage of subsurface investigation, once the footing size, configuration and loadings are known.

For spread footings placed within the approach embankments on a compacted Granular 'A' core, a factored geotechnical resistance at ULS of 900 kPa may be assumed for preliminary design. The geotechnical resistance at SLS will depend on the thickness of Granular 'A' and the consistency and thickness of the underlying soils; a value of 350 kPa may be assumed for preliminary design.

The geotechnical resistances provided herein are given under the assumption that the loads will be applied perpendicular to the surface of the footings; where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the Ontario Highway Bridge Design Code (OHBDC).

5.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footing and the subsoils should be calculated in accordance with Section 6-8.4.3 of the OHBDC. The angle of friction between the concrete and the undisturbed silty sand to silt founding soils should be taken as 24 degrees; the corresponding coefficient of friction, tan δ, would then be 0.45. Where "perched" abutment footings are adopted, the angle of friction between the concrete footings and the compacted Granular 'A' pad should be taken as 30 degrees; the corresponding coefficient of friction would be 0.58.

5.3.3 Frost Protection

The footings should be provided with a minimum of 1.5 m of soil cover for frost protection.

5.4 Driven Steel H-Piles

Consideration could be given to supporting the new underpass structure on steel H-piles driven to found within the very dense silty sand to silt deposit or the hard silty clay deposit. Based on the results of Boreholes B6-1 and B6-2, which were drilled from Highway 400 grade, it is anticipated that an adequate driving resistance would not be achieved until at least Elevation 300 m, i.e. 3 m of penetration into the deposit. This would put the pile founding level more than 10 m below Victoria Street grade and 4 m below Highway 400 grade.

It is noted that additional borehole investigation will be required at the proposed abutment locations during detailed design in order to determine the composition and consistency of the upper soils through which the piles would be driven.

5.4.1 Axial Geotechnical Resistance

For preliminary design, a pile founding level of Elevation 299 m may be assumed. The factored axial resistance at ULS for steel HP 310 x 110 H-piles driven to this design tip elevation may be taken as 1,400 kN. The axial resistance at SLS for a single pile, for 25 mm of settlement, may be taken as 1,200 kN.

As a guide, to achieve the above design resistances, the piles should be driven to a final set of no less than 10 blows per 25 mm of penetration using a hammer with rated energy of about 50 kJ, and not exceeding 60 kJ. The actual set criteria should be established based on the Contractor's pile driving equipment. Provision should be made to re-tap selected piles to confirm the set after adjacent piles have been driven, in accordance with MTO's current Special Provision.

5.4.2 Resistance to Lateral Loads

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

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

$$k_h = \frac{n_h z}{B}$$

where n_h is the constant of subgrade reaction
z is the depth (m)
B is the pile diameter (m)

It is noted that the soils above the Highway 400 cut level were not investigated during this preliminary subsurface investigation stage. Based on regional geological mapping, it is expected that the surficial subsoils will consist of silts, sands and gravels. For these materials, the range in value n_h may be taken as 10 MPa/m to 20 MPa/m in the structural analysis. This estimated range of values will require confirmation following the detailed design stage of investigation.

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

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

5.4.3 Frost Protection

The pile caps should be provided with 1.5 m soil cover for frost protection.

5.5 Lateral Earth Pressures

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

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the abutments and walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00

- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the abutment wall, in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be used in accordance with OPSS 501.06.

- The granular fill may be placed either in a zone with width equal to at least 1.5 m behind the back of the stem (Case I from OHBDC Figure 6-7.4.1) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II from OHBDC Figure 6-7.4.4).

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

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

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

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

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

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

5.6 Permanent Cut Slopes

Boreholes B6-1 and B6-2 were advanced from the Highway 400 cut grade at about Elevation 304 m to 305 m, and so no borehole information is available regarding the soils which will comprise the permanent cut slopes along the east and west sides of Highway 400. Based on regional geological mapping, it is anticipated that the cut will be formed in silt, sand and gravel soils, similar to that encountered below Elevation 304 m.

Based on the topographic information on the Engineering and Title Records plates and on site reconnaissance, the existing slopes are generally formed at a gradient of 2 to 3 horizontal to 1 vertical (2H:1V to 3H:1V). For preliminary design purposes, a maximum gradient of 2.5H:1V may be assumed for the new permanent cut slopes. This design recommendation will have to be confirmed during the detailed design stage of the subsurface investigation program.

It is noted that the fine sands and silts which will comprise the cut slopes will be readily erodible. The slope surfaces should be grassed, and drainage should be provided from the top of the slope to the ditch line.

5.7 Design and Construction Considerations

5.7.1 Dewatering

Groundwater seepage into the footing or pile cap excavations is expected to be minor. Pumping from properly-filtered sumps or a filtered drain placed at the base of the excavation should provide sufficient groundwater control during foundation works. Surface water run-off, which is expected to be more significant than groundwater seepage, should be directed away from the footing excavations.

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

5.7.2 Excavation

The footing or pile cap excavations will extend a minimum of 1.5 m below lowest surrounding grade, generally through very dense silty sand to silt soils below the Highway 400 grade. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. The soils below the Highway 400 grade would be classified as Type 2 to 3 soil; for the soils below Victoria Street grade, the type and consistency / relative density will have to be confirmed during the detailed design stage. Temporary open-cut slopes should be maintained no steeper than 1 horizontal to 1 vertical (1H:1V). Where space restrictions dictate, footing or pile cap excavations could also be carried out within a braced excavation.

5.7.3 Obstructions

Although no cobbles or boulders were encountered during the borehole investigation, it should be noted that cobbles and boulders are inherent in glaciolacustrine materials. Cobbles and boulders should therefore be expected during driving of steel H-piles if a deep foundation option is selected.

GOLDER ASSOCIATES LTD.

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

LCC/ASP/FJH/clg
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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

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 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 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	c_u, s_u 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 (LV-laboratory vane test)
γ	unit weight

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

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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_f	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
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

RECORD OF BOREHOLE No B6-1

1 OF 1 **METRIC**

PROJECT 001-1143F
W.P. 30-95-00
DIST SW HWY 400
DATUM Geodetic
LOCATION N 4907794.9; E 290179.1
BOREHOLE TYPE 108mm DIAMETER SOLID STEM AUGERS
DATE Oct.23/2000
ORIGINATED BY AZ
COMPILED BY LCC
CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
303.8	GROUND SURFACE					
0.0 / 303.5	Topsoil					
0.3	Sandy Silt to Silt, some sand, containing thin clay layers below 3.1m depth Very dense Brown Moist becoming wet at 3.2m depth	1	SS	87	303	
		2	SS	125	302	
		3	SS	190	301	0 19 78 3
		4	SS	99/.15		
300.1 / 3.7	Silty Clay, trace sand and gravel (Till), containing silt and sand seams Hard Brown Moist	5	SS	165	300	
		6	SS	128/.15	299	
298.9 / 4.9	END OF BOREHOLE Notes: 1. Borehole dry on completion of drilling operations. 2. Piezometer dry on March 15, 2001.					

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED: 20 40 60 80 100
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%): 10 20 30
UNIT WEIGHT γ kN/m³

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

ON_MOT 0011143F.GPJ ON_MOT.GDT 14/1/02

Ministry of Transportation
Ontario

Foundation Design

RECORD OF BOREHOLE No B6-2

1 OF 1 **METRIC**

PROJECT 001-1143F
W.P. 30-95-00
DIST SW HWY 400
DATUM Geodetic
LOCATION N 4907743.3; E 290135.1
BOREHOLE TYPE 108mm DIAMETER SOLID STEM AUGERS
DATE Oct.26/2000
ORIGINATED BY PKS
COMPILED BY LCC
CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
304.9	GROUND SURFACE					
0.0	Silty Sand, trace gravel, some organics Loose Moist Brown	1	SS	7		
304.3 / 0.6	Silty Sand Very dense Moist Brown	2	SS	50	304	
		3	SS	100	303	0 84 16 0
		4	SS	147	302	
		5	SS	114	301	
		6	SS	100/.15		
		7	SS	95/.23	300	
	Contains 50mm to 75mm thick layers of clayey silt below 4.9m depth				299	
		8	SS	105	298	
297.6 / 7.3	Sand and Gravel Very dense Moist Brown	9	SS	105/.25	297	
296.9 / 8.0	END OF BOREHOLE Note: Borehole dry on completion of drilling.					

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED: 20 40 60 80 100
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%): 10 20 30
UNIT WEIGHT γ kN/m³

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

ON_MOT 0011143F.GPJ ON_MOT.GDT 14/1/02

0 10 20 30 40 50 100m

SCALE IN METRES

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

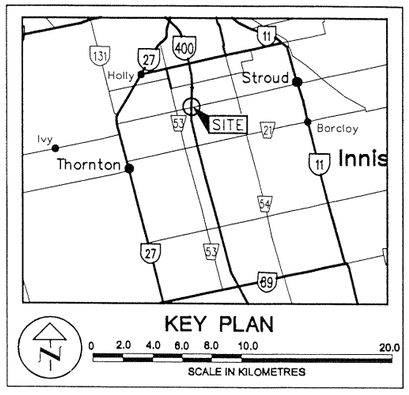
DIST	HWY 400
CONT. No.	
GWP No.	30-95-00

INNISFIL TENTH LINE UNDERPASS
HWY 400
BOREHOLE LOCATION PLAN

SHEET

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

LEGEND

- Borehole, previous investigation
- Borehole, present investigation

No.	ELEVATION	LOCATION NORTHING	LOCATION EASTING
B6-1	303.8	4,907,794.9	290,179.1
B6-2	304.9	4,907,743.3	290,135.1

REFERENCE

This drawing was created from digital file "50203.dwg" provided by URS Cole Sherman

NO.	DATE	BY	REVISION

Geocres No.

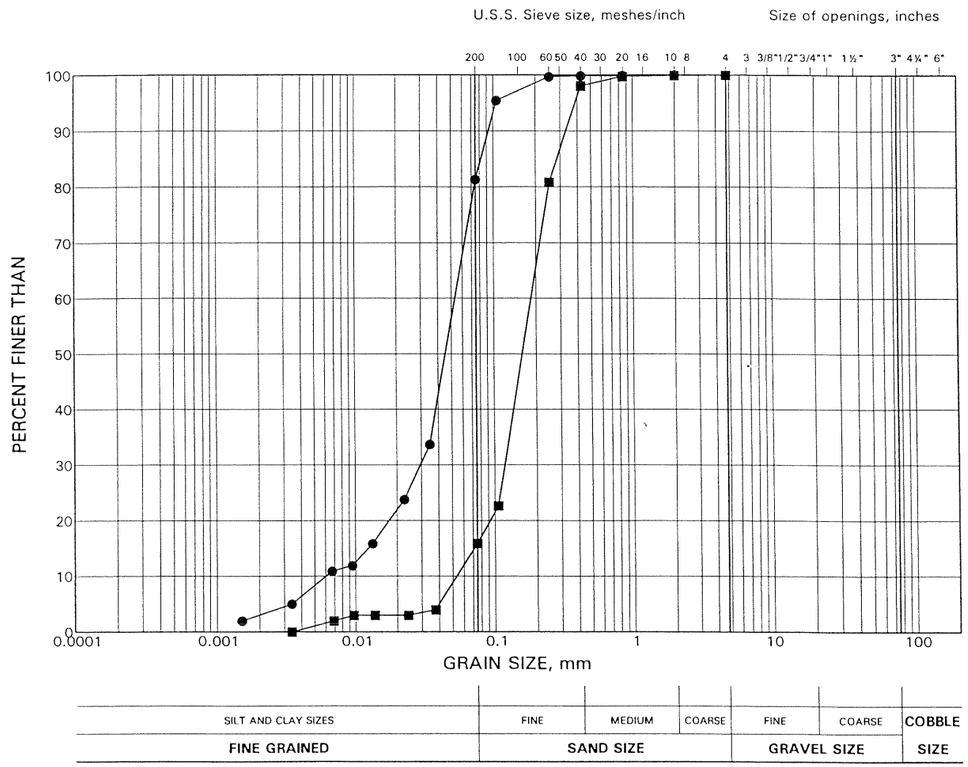
HWY. No. 400		PROJECT NO.: D01-1143F	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-208
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

P1143F11.DWG

GRAIN SIZE DISTRIBUTION TEST RESULTS

Silty Sand to Silt, some sand

FIGURE 1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	B6-1	3	301.3
■	B6-2	3	303.2

Project 001-1143F

Golder Associates

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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT MOLSON PARK DRIVE OVERPASS STRUCTURE SITE 30-179 HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11 G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

Submitted to:

URS Cole, Sherman
75 Commerce Valley Drive East
Thornhill, Ontario
L3T 7N9

DISTRIBUTION:

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January 2002 001-1143F-7

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Drawing 1 Molson Park Drive Overpass, Highway 400, Borehole Location Plan

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Appendix A Records of Boreholes and Test Results – 1997 Investigation

PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
MOLSON PARK DRIVE OVERPASS
STRUCTURE SITE 30-179
HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

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LIST OF DRAWINGS

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Appendix A Records of Boreholes and Test Results – 1997 Investigation

1.0 INTRODUCTION

Golder Associates Ltd. has been retained by URS Cole, Sherman (Cole, Sherman) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the ultimate widening of Highway 400 from 1 km south of Highway 89, northerly 30 km to Highway 11, in Simcoe County, Ontario. Foundation engineering services are required for the widening and / or replacement of eighteen existing overpass and underpass structures, as well as five structural culverts.

This report addresses the widening of the existing Molson Park Drive overpass structure. Existing subsurface data for this site from an investigation conducted by the MTO in 1997 (*"Foundation Investigation Report for Molson Park Drive Replacement Bridge, Highway 400"*, dated July 1997 – GEOCRES File No. 31D-362) were used to determine the subsurface conditions for this preliminary design study.

The terms of reference for the scope of work are outlined in Golder Associates' Proposal No. P01-1192, dated June 2000.

2.0 SITE DESCRIPTION

The existing Molson Park Drive overpass structure is located about 3 km south of the Essa Road (Simcoe Road 30, formerly Highway 27) interchange, in Barrie, Ontario. The MTO has designated this overpass as Structure Site 30-179.

At the site, the original ground surface was at about Elevation 298 m to 301 m. The Highway 400 grade is at about Elevation 302 m to 302.5 m. Molson Park Drive has been constructed in cut, with its grade at about Elevation 296 m under Highway 400.

The existing overpass is a single span, steel box girder structure with integral abutments supported on driven steel H-piles. The contract drawings indicate that the pile cap underside is at Elevation 300 m at the south abutment, and Elevation 300.5 m at the north abutment. The design pile lengths shown on the contract drawings — 22.6 m and 23.1 m at the south and north abutments, respectively — correspond to a pile tip level of Elevation 277.4 m.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at this site by the Ministry of Transportation, Ontario (MTO) in April 1997. At that time, a total of nine boreholes were drilled – Boreholes 1, 2, 5 and 6 were advanced in the vicinity of the north abutments, and Boreholes 3, 4, 7, 8 and 9 were advanced in the vicinity of the south abutment. The boreholes were extended to between 19 m and 28 m below the original ground surface, to between Elevations 283 m and 273 m.

The boreholes were advanced using hollow stem augers. Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. The groundwater conditions in the open borehole were observed following the drilling operations. Laboratory index and classification tests, consisting of natural moisture contents, Atterberg Limits and grain size distributions, were carried out on selected soil samples.

The borehole locations, referenced to the MTM NAD83 survey system, and elevations, referenced to the geodetic datum, were established by MTO. The approximate borehole locations and northing and easting co-ordinates are shown on the attached Drawing 1.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

This 30 km section of Highway 400 traverses, from south to north, the following physiographic regions as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, Third Edition, 1984): the Simcoe Lowlands; the Peterborough Drumlin Field; a second lobe of the Simcoe Lowlands; and the Simcoe Uplands. Along Highway 400, the Simcoe Lowlands are present from the southern limit of the project to just south of Innisfil Creek, and again from Essa Road (Simcoe Road 30, formerly Highway 27) to about 1 km north of Dunlop Street (Simcoe Road 90, formerly Highway 90). The Peterborough Drumlin Field occupies the belt between these lobes of the Simcoe Lowlands, extending from just south of Innisfil Creek, which is located about 1 km north of Highway 89, to Essa Road. The Simcoe Uplands extend from about 1 km north of Dunlop Street to beyond the northern limit of the project at Highway 11.

The two sections where Highway 400 crosses the Simcoe Lowlands consist of two lobes of a sand plain which include the shores of Kempenfelt Bay, the Nottawasaga River and Innisfil Creek. The surficial soils of these sections of the Simcoe Lowlands consist primarily of sand, although silt, clay or peat may be found in low-lying areas.

The surficial soils in the Peterborough Drumlin Field, in which the Molson Park Drive overpass site is located, consist primarily of gravelly sand till or sand and gravel deposits. Drumlins (glacially-shaped hills) are more frequent in the southern portion of the section of the Peterborough Drumlin Field traversed by Highway 400. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins.

The surficial soils in the Simcoe Uplands physiographic region are primarily sandy silt till deposits, known to contain occasional boulders. Low-lying areas may be infilled with shallow sand and gravel deposits, which are shoreline deposits of a former glacial lake that once flooded the area.

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the 1997 boreholes, together with the results of in-situ and laboratory testing, are given on the Record of Borehole sheets and figures contained in Appendix A. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

Boreholes 1, 2, 5 and 6 were advanced in the vicinity of the north abutment, and Boreholes 3, 4, 7, 8 and 9 were advanced in the vicinity of the south abutment. The approximate locations and ground surface elevations for these borings are shown on the attached Drawing 1.

In summary, the Highway 400 fill is underlain by compact to very dense silty sand and silt deposits, overlying stiff to hard silty clay to clayey silt, in turn underlain by a deposit of very dense silty sand to sand. All nine boreholes were terminated in the lower silty sand to sand. A detailed description of the subsurface conditions encountered in the 1997 boreholes is provided in the following sections.

4.2.1 Fill

About 1 m to 2 m of silty sand fill, which was placed to raise the grade of Highway 400, was encountered in Boreholes 5 to 9. Measured Standard Penetration Test (SPT) 'N' values of 7 to 27 blows per 0.3 m of penetration indicate that the fill has a loose to compact relative density.

Below the silty sand fill in Boreholes 5, 7 and 8, a thin layer of organics was encountered.

4.2.2 Upper Silty Sand

Silty sand was encountered at ground surface or beneath a thin topsoil layer in Boreholes 1 to 4, and below the fill and organics (where present) in Boreholes 5 to 9. The silty sand ranges from 0.8 m in thickness in the drainage ditch areas, to a maximum of 6.9 m in thickness. The base of the deposit varies from Elevation 299 m to 293 m, typically declining from west to east.

Thin layers of sandy silt were encountered within the silty sand deposit, as noted on the records of Boreholes 6 and 8. The grain size distribution test results obtained for representative samples of the silty sand are shown on Figure 1 in Appendix A.

In the upper 1 m to 2 m of the silty sand deposit in Boreholes 6, 7 and 8, SPT 'N' values of 3 to 10 blows per 0.3 m of penetration indicate a very loose to loose relative density. Elsewhere, the measured SPT 'N' values in the silty sand ranged from 12 to greater than 100 blows, but were typically more than 30 to greater than 100 blows per 0.3 m of penetration, indicating that this portion of the silty sand deposit has a predominantly dense to very dense relative density.

4.2.3 Silt

A thin layer of silt, ranging in thickness from about 1.5 m to 3.5 m, was encountered in all of the boreholes. The base of the silt layer was encountered between Elevation 296 m and 291 m, generally declining from west to east across the site.

The silt layer contains trace to some sand. Grain size distribution test results obtained on the coarse portion of the deposit are shown on Figure 2 in Appendix A.

The measured SPT 'N' values in the silt ranged from 23 to greater than 100 blows per 0.3 m of penetration, indicating that the silt layer typically has a compact to very dense relative density. However, where the silt layer is closer to the ground surface in Borehole 1, the measured SPT 'N' values of 9 and 11 blows per 0.3 m of penetration are indicative of a loose relative density.

4.2.4 Silty Clay to Clayey Silt

A 1 m to 2.5 m thick deposit of silty clay to clayey silt was encountered in all of the boreholes. The base of the deposit varies between Elevations 295 m and 288.5 m and, like the overlying deposits, generally declines from west to east at the site.

The results of Atterberg Limits testing on samples of the silty clay and clayey silt portions of this deposit are shown on Figures 3 and 4 of Appendix A. The test results, which are summarized in the table below, indicate that both the clayey silt and silty clay are inorganic and of low to intermediate plasticity, respectively. The measured natural moisture contents in samples of the silty clay to clayey silt ranged from 18 to 27 per cent, and were typically near the plastic limit of the material.

Soil Type	*Plastic Limits*	*Liquid Limits*	*Plasticity Indices*
Silty Clay	17 to 20 %	35 to 47 %	18 to 27 %
Clayey Silt	16 to 18 %	26 to 34 %	10 to 16 %

The measured SPT 'N' values ranged from 14 to 69 blows per 0.3 m of penetration, indicating that the silty clay to clayey silt has a stiff to hard consistency.

4.2.5 Lower Silty Sand to Sand

In all of the boreholes, a lower deposit of very dense silty sand to sand was encountered underlying the silty clay to clayey silt deposit. The surface of this deposit was typically encountered between Elevation 292 m and 288.5 m, generally declining from west to east. In Borehole 1, in the north western portion of the structure site, the surface of this lower silty sand to sand deposit was encountered at Elevation 295 m. This deposit extended to the maximum investigated depth, corresponding to approximately Elevation 273 m.

The silty sand to sand deposit contains trace quantities of gravel. Gravelly sand interlayers, between 0.5 m and 3 m in thickness, were encountered in Boreholes 1, 2, 4, 5, 7 and 8. These gravelly sand interlayers were typically encountered at or near the surface of the deposit. Grain size distribution envelopes for the silty sand to sand deposit and for the gravelly sand interlayers are shown on Figures 5 and 6, respectively.

The measured SPT 'N' values ranged from 85 to greater than 100 blows per 0.3 m of penetration, but were typically well in excess of 100 blows per 0.3 m of penetration. The lower silty sand to sand deposit has a very dense relative density.

4.3 Groundwater Conditions

The groundwater conditions were observed in the open boreholes following the 1997 drilling operations. There were no piezometers installed in the boreholes. Perched water was encountered in all of the boreholes between about Elevation 297.5 m and 294 m, typically just above the silt layer within the upper silty sand (in Boreholes 2 to 4 and 6 to 9), or within the silt layer (in Boreholes 1 and 5). It is considered that this shallow water table is perched by the silty clay to clayey silt deposit which underlies the upper silty sand and silt layers. This upper aquifer therefore has a saturated thickness of about 1 m to 4 m, although it may be up to about 6 m in saturated thickness as encountered in Borehole 6.

The 1997 borehole records note that the boreholes, which all terminated in the lower silty sand to sand stratum, were dry on completion of drilling, with the exception of Borehole 4 where the water level was at Elevation 272.8 m, about 0.1 m above the base of the borehole. It is noted that the dry conditions and low water level in Borehole 4 may not represent the stabilized conditions at the site.

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

GOLDER ASSOCIATES LTD.

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Principal

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

DJE/LCC/ASP/FJH/clg
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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
MOLSON PARK DRIVE OVERPASS
STRUCTURE SITE 30-179
HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides preliminary foundation design recommendations for the widening of the existing Molson Park Drive overpass structure, associated with the widening of Highway 400. The recommendations are preliminary only and are based on interpretation of the factual data obtained during the subsurface investigation carried out in 1997 at this site. The interpretation and recommendations provided are intended for planning purposes only, to provide the information necessary at this stage of the study. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the planning of the project. Further foundation investigation will be required at this structure site as part of the detailed design stage of the project.

It is understood that Highway 400 will be widened from its existing six-lane configuration to an interim configuration of eight lanes, and an ultimate configuration of ten lanes, and that an alternative for a twelve-lane express / collector system is under consideration between Molson Park Drive and Duckworth Street in Barrie. Throughout the project length, it is expected that the existing highway platform will be widened by between 13 m and 30 m. Widening of the Molson Park Drive overpass structure will therefore be necessary.

At the Molson Park Drive site, the Highway 400 grade is at about Elevation 302 m (about 2 m to 3 m above the adjacent ground), while Molson Park Drive has been constructed in cut and is at about Elevation 296 m under the highway. The existing overpass is a single-span, steel box girder structure with integral abutments supported on driven steel H-piles. The contract drawings indicate that the underside of the pile cap is at Elevation 300 m to 300.5 m. The HP 310 x 110 piles are approximately 23 m long, with their tips at Elevation 277.4 m.

5.2 Bridge Foundation Options

According to the 1997 borehole investigation data, the Highway 400 platform fill is underlain by compact to very dense silty sand and silt deposits, overlying stiff to hard silty clay to clayey silt, in turn underlain by a deposit of very dense silty sand to sand. A perched groundwater table exists at the site, atop the clayey silt to silty clay deposit; the perched water was encountered in the boreholes near the base of the upper silty sand or within the silt layer.

Based on these subsurface conditions and the configuration of the existing structure, it is recommended that any widened portion of this structure also be supported on driven steel H-piles. Consideration could be given to the use of spread footings if the widened structure can be

maintained separate from the existing structure; however, in this case perched abutments supported on spread footings within the approach embankments should be adopted. The native soils at shallow depth are variable and spread footings would have to be extended to below the perched groundwater level over at least a portion of the footing length. The extent and elevation of the loose to compact silt deposit in the area of the proposed widening should be determined to assess the feasibility of the shallow footing option. Groundwater control will be required; however, because of the presence of the silt subsoil, well-point dewatering would be difficult.

Preliminary recommendations for spread footings for perched abutments, and for driven steel H-pile foundations are provided in the following sections.

5.3 Spread Footings

Consideration could be given to the use of abutment footings perched within the approach embankments. Given the existing road grades, perched abutments may not be feasible since the approach embankment is only about 2 m to 3 m high. Some sub-excavation of the loose upper sands and the silt deposit, where present, would be required.

5.3.1 Axial Geotechnical Resistance

For spread footings placed within the approach embankments on a compacted Granular 'A' core, a factored geotechnical resistance at ULS of 900 kPa may be assumed for preliminary design. The geotechnical resistance at SLS will depend on the thickness of Granular 'A' and the consistency and thickness of the underlying soils; a value of 350 kPa may be assumed for preliminary design. These values assume that the granular pad has a thickness of at least one footing width.

The geotechnical resistances provided herein are given under the assumption that the loads will be applied perpendicular to the surface of the footings; where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the Ontario Highway Bridge Design Code (OHBDC).

5.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footing and the subsoils should be calculated in accordance with Section 6-8.4.3 of the OHBDC. The angle of friction

between the concrete footings and the compacted Granular 'A' pad should be taken as 30 degrees; the corresponding coefficient of friction would be 0.58.

5.3.3 Frost Protection

The footings should be provided with a minimum of 1.5 m of soil cover for frost protection.

5.4 Driven Steel H-Piles

Consideration should be given to supporting the widened portions of the overpass on steel H-piles driven to found within the very dense, lower silty sand to sand deposit. Based on the 1997 borehole investigation results and the configuration of the existing deep foundations, it is anticipated that the pile founding level would be at about Elevation 277.5 m. It is noted that heavy driving is anticipated, given the high Standard Penetration Test 'N' values and the presence of the gravelly sand interlayers which may be difficult to penetrate.

5.4.1 Axial Geotechnical Resistance

For preliminary design, the factored axial resistance at ULS for steel HP 310x110 H-piles driven to found within the very dense, lower silty sand to sand deposit may be taken as 1,400 kN. The axial resistance at SLS for 25 mm of settlement may be taken as 1,200 kN.

To achieve the above design resistances, the piles should be driven to about Elevation 277.5 m. The piles should be driven to a final set of no less than 15 blows per 25 mm of penetration using a hammer with rated energy of about 50 kJ, and not exceeding 60 kJ. Provision should be made to re-tap selected piles to confirm the set after adjacent piles have been driven, in accordance with MTO's current Special Provision.

5.4.2 Resistance to Lateral Loads

The lateral loading could be resisted fully or partially by the use of battered piles. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles.

If integral abutments are under consideration, there may also be a requirement for the piles to move sufficiently to accommodate the bridge deck deflections.

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

$$k_h = \frac{n_h z}{B}$$

where n_h is the constant of subgrade reaction
z is the depth (m)
B is the pile diameter (m)

The piles will extend through the generally dense to very dense upper silty sand and silt layers, the relatively thin silty clay to clayey silt deposit, and into the very dense lower silty sand to sand deposit. Above Elevation 294 m at the north abutment and above Elevation 292 m at the south abutment, the range in value of n_h may be taken as 10 MPa/m to 20 MPa/m in the structural analysis. Below these elevations, the range of n_h may be taken as 16 MPa/m to 20 MPa/m in the structural analysis.

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

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

5.4.3 Frost Protection

The pile caps should be provided with 1.5 m of soil cover for frost protection.

5.5 Lateral Earth Pressures

The lateral pressures acting on the bridge abutments and associated retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill and on the subsequent lateral movement of the structure. The following

recommendations are made concerning the design of the abutments, in accordance with the OHBDC:

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the abutments and walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00

- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the abutment wall, in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be used in accordance with OPSS 501.06.

- The granular fill may be placed either in a zone with width equal to at least 1.5 m behind the back of the stem (Case I from OHBDC Figure 6-7.4.1) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II from OHBDC Figure 6-7.4.4).

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

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

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

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

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

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

5.6 Design of Permanent Cut Slopes

In the vicinity of this overpass, the Molson Park Drive cut is up to about 5 m in depth, with the road grade at about Elevation 296 m and the drainage ditch invert level as low as Elevation 295 m. Based on the topographic information on the Engineering and Title Records plates and on site reconnaissance, the existing cut slopes are formed at a gradient of about 2 horizontal to 1 vertical (2H:1V). For preliminary design purposes, a maximum gradient of 2H:1V may be assumed for any new permanent cut slopes.

It is noted that the base of the cut will extend below the perched groundwater level at the site. Protection of the lower portion of the cut slope, for example using a drainage blanket, will be necessary to prevent sloughing of the toe of the permanent cut slopes. It is understood that prior to the reconstruction of this structure, seepage and sloughing were evident in some areas of the permanent cut slope toe.

5.7 Design and Construction Considerations

5.7.1 Dewatering

A perched water table was encountered in the 1997 boreholes, between Elevation 297.5 m and 294 m. This shallow aquifer appears to be perched by the silty clay to clayey silt deposit, and has a saturated thickness of about 1 m to 4 m associated with the upper silty sand and silt layers. A groundwater control scheme will be required for pile cap construction if excavations extend below these elevations.

Where encountered at the subgrade or pile cap underside level, the silt soils will be particularly susceptible to disturbance from ponded water and construction traffic. Provision should be made in the Contract Documents for the placement of a lean concrete mat to protect the soils from such disturbance.

5.7.2 Excavation

The footing or pile cap excavations will extend a minimum of 1.5 m below lowest surrounding grade, generally through loose to dense silty sand to silt soils. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. The loose to very dense silty sand to silt soils would be classified as Type 2 to 3 soil, assuming that appropriate groundwater control is in place. Temporary open-cut slopes should therefore be maintained no steeper than 1 horizontal to 1 vertical (1H:1V). Where space restrictions dictate, footing or pile cap excavations could also be carried out within a braced excavation.

5.7.3 Obstructions

Although no cobbles or boulders were noted on the borehole records from the 1997 subsurface investigation, it should be recognized that cobbles and boulders are inherent in glaciolacustrine materials. Such obstructions should therefore be expected during driving of steel H-piles for deep foundations or for temporary roadway protection.

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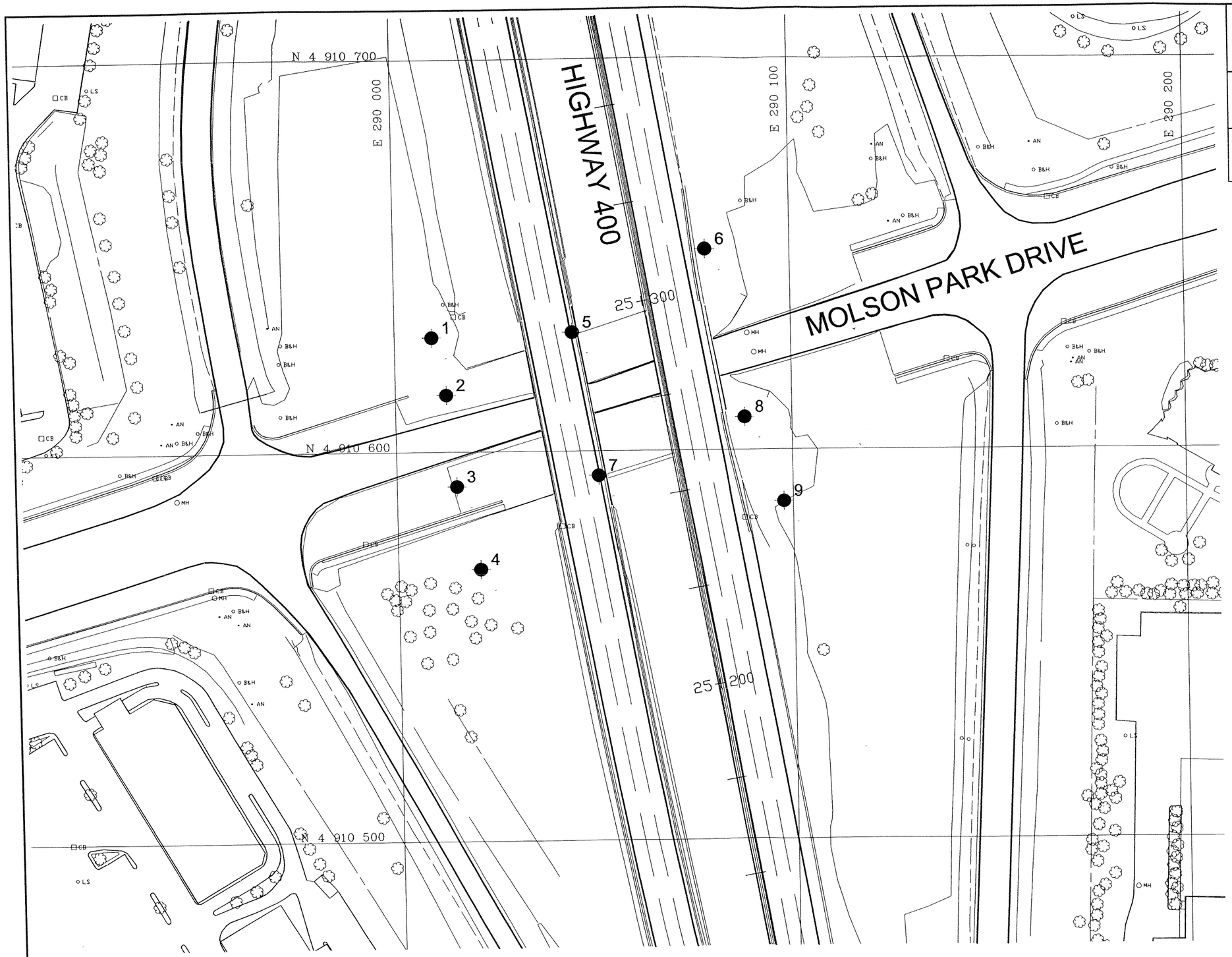
DIST	HWY 400
CONT. No.	
GWP No.	30-95-00

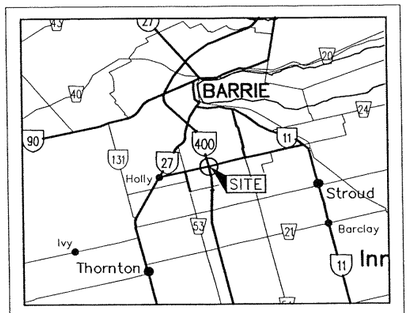
MOLSON PARK DRIVE OVERPASS
HWY 400
BOREHOLE LOCATION PLAN

SHEET

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA





KEY PLAN

0 2.0 4.0 6.0 8.0 10.0 20.0
SCALE IN KILOMETRES

LEGEND

Borehole, previous investigation

Borehole, present investigation

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
1	300.9	4,910,629.3	290,009.5
2	296.1	4,910,614.6	290,013.0
3	296.1	4,910,591.1	290,015.3
4	300.4	4,910,569.8	290,021.0
5	301.8	4,910,630.5	290,044.8
6	302.0	4,910,651.4	290,078.7
7	301.3	4,910,593.7	290,050.8
8	301.3	4,910,608.4	290,088.2
9	300.2	4,910,586.7	290,097.7

REFERENCE

This drawing was created from digital file "50205.dwg" provided by URS Cole Sherman

0 10 20 30 40 50 100m
SCALE IN METRES

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

NO.	DATE	BY	REVISION

Geocres No.

HWY. No. 400		PROJECT NO.: 001-1143F			
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-179		
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1		

P1143F12.DWG

plot scale 1:2imp.

APPENDIX A

RECORDS OF BOREHOLES AND TEST RESULTS – 1997 INVESTIGATION

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

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL

e	1, %	VOID RATIO
n	1, %	POROSITY
w	1, %	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX = $w_L - w_P$
I_L	1	LIQUIDITY INDEX = $\frac{w - w_P}{I_P}$
I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_P}$
e_{max}	1, %	VOID RATIO IN LOOSEST STATE

e_{min}	1, %	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_U	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

RECORD OF BOREHOLE No 1 — 1 OF 1 — METRIC

W.P. 26 - 96 - 01 | LOCATION CO - ORDS: N 4 910 629.3; E 290 009.5 | ORIGINATED BY M V&P C

DIST 33 HWY 400 | BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST | COMPILED BY M V

DATUM GEODETIC | DATE 1997 04 02 | CHECKED BY T C K

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
300.9	Ground Surface					
0.0	Topsoil				*	
299.4	SILTY SAND, Loose					
1.5	SILT, Trace of Sand, Loose to Compact	1	SS	9		
		2	SS	11		0 5 (95)
296.2						
4.7	SILTY CLAY, Stiff to Very Stiff	3	SS	13		
295.0		4	SS	21		
5.9	Gravelly Sand	5	SS	156		24 66 (10)
		6	SS	128		
		7	SS	133		
		8	SS	100	/13cm	31 52 (17)
	SILTY SAND TO SAND, Trace of Gravel, Very Dense	9	SS	123		
		10	SS	128		
		11	SS	145		
		12	SS	113		
		13	SS	125		
		14	SS	161		
		15	SS	154		
		16	SS	177		4 86 (10)
276.1		17	SS	130		
24.8	End of Borehole					

* Note:
Borehole Dry on Completion
Perched Water Level Encountered
at El. 297.4

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa: o UNCONFINED, + FIELD VANE, • QUICK TRIAXIAL, × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%) 15 30 45

UNIT WEIGHT γ kN/m³

ELEVATION SCALE: 300, 298, 296, 294, 292, 290, 288, 286, 284, 282, 280, 278

120/18cm

+3, ×5: Numbers refer to Sensitivity

20 15◇5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 2 — 1 OF 1 — METRIC

W.P. 26 - 96 - 01 | LOCATION CO - ORDS: N 4 910 614.6; E 290 013.0 | ORIGINATED BY M V

DIST 33 HWY 400 | BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST | COMPILED BY M V

DATUM GEODETIC | DATE 1997 04 01 | CHECKED BY T C K

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
296.1	Drainage Ditch					
0.0	SILTY SAND, Trace of Organic, Compact	1	SS	21	*	
295.3						
0.8	SILT, Some Sand, Compact	2	SS	30		2 18 (80)
293.8		3	SS	24		
2.3	SILTY CLAY, Stiff to Very Stiff	4	SS	14		
292.4		5	SS	23		
3.7		6	SS	125		5 81 (14)
		7	SS	125	/25cm	
	Gravelly Sand	8	SS	141		15 71 (14)
		9	SS	100	/10cm	
		10	SS	150	/20cm	
		11	SS	168	/20cm	
		12	SS	102	/15cm	
	SILTY SAND TO SAND, Trace of Gravel, Very Dense	13	SS	137		3 86 (11)
		14	SS	177		
277.4		15	SS	146		
18.7	End of Borehole					

* Note:
Borhole Dry on Completion
Perched Water Level Encountered
at El. 296.0

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa: o UNCONFINED, + FIELD VANE, • QUICK TRIAXIAL, × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%) 15 30 45

UNIT WEIGHT γ kN/m³

ELEVATION SCALE: 294, 292, 290, 288, 286, 284, 282, 280, 278

120/28cm

+3, ×5: Numbers refer to Sensitivity

20 15◇5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 3

1 OF 1 METRIC

W.P. 26 - 96 - 01 LOCATION CO - ORDS: N 4 910 591.1; E 290 015.3 ORIGINATED BY M V

DIST 33 HWY 400 BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST COMPILED BY M V

DATUM GEODETIC DATE 1997 04 14 CHECKED BY T C K

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
296.1	Drainage Ditch						
0.0	Silt and Organic						
	SILTY SAND, Very Dense		1	SS	54	*	0 79 (21)
294.2			2	SS	71		
1.9	Clayey Silt, Hard		3	SS	43		0 5 (95)
	SILT, Trace of Sand, Dense to Very Dense		4	SS	59		
292.4							
3.7	SILTY CLAY, Hard		5	SS	31		
291.1			6	SS	55		
5.0			7	SS	156		6 84 (10)
			8	SS	168		10 78 (12)
			9	SS	118		
			10	SS	108		
			11	SS	115		
	SILTY SAND TO SAND, Trace of Gravel, Very Dense		12	SS	147		0 94 (6)
			13	SS	140		
274.3			14	SS	130		
21.8	End of Borehole • Note: Borehole Dry on Completion Perched Water Level Encountered at El. 295.9						

ELEVATION SCALE: 294, 292, 290, 288, 286, 284, 282, 280, 278, 276

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (o UNCONFINED, + FIELD VANE, • QUICK TRIAXIAL, × LAB VANE) (20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (15 30 45); UNIT WEIGHT γ kN/m^3

120/23cm

$+^{3}_{}$, $\times^{5}_{}$: Numbers refer to Sensitivity

$15 \diamond^{20}_{5}$ (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 4

1 OF 1 METRIC

W.P. 26 - 96 - 01 LOCATION CO - ORDS: N 4 910 569.8; E 290 021.0 ORIGINATED BY M V

DIST 33 HWY 400 BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST COMPILED BY M V

DATUM GEODETIC DATE 1997 04 10 CHECKED BY T C K

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
300.4	Ground Surface						
0.0	Topsoil						
	SILTY SAND, Compact to Very Dense		1	SS	28		
			2	SS	38		
295.2			3	SS	54		0 81 (19)
5.2	SILT, Trace of Sand, Dense to Very Dense		4	SS	63		
			5	SS	45		
292.9			6	SS	55		0 9 (91)
7.5	CLAYEY SILT, Hard		7	SS	30		
			8	SS	32		
291.0			9	SS	133	/23cm	
9.4			10	SS	102	/10cm	
	Gravelly Sand		11	SS	136	/5cm	38 55 (7)
	SILTY SAND TO SAND, Trace of Gravel, Very Dense		12	SS	85		2 93 (5)
			13	SS	96		
			14	SS	88		
			15	SS	120		
			16	SS	128		
			17	SS	119	/15cm	
272.7			18	SS	112	/15cm	1 91 (8)
27.7	End of Borehole • Note: Water Level on Completion at El. 272.8 Perched Water Level Encountered at El. 295.9						

ELEVATION SCALE: 300, 298, 296, 294, 292, 290, 288, 286, 284, 282, 280, 278, 276, 274

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (o UNCONFINED, + FIELD VANE, • QUICK TRIAXIAL, × LAB VANE) (20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (15 30 45); UNIT WEIGHT γ kN/m^3

120/23cm

$+^{3}_{}$, $\times^{5}_{}$: Numbers refer to

$15 \diamond^{20}_{5}$ (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No 5

1 OF 1 METRIC

W.P. 26 - 96 - 01 LOCATION CO - ORDS: N 4 910 630.5; E 290 044.8 ORIGINATED BY M V&P C
DIST 33 HWY 400 BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST COMPILED BY M V
DATUM GEODETIC DATE 1997 04 02 & 03 CHECKED BY T C K

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
301.8	Ground Surface					
0.0	SILTY SAND, Compact, (Fill)	1	SS	25	*	
299.9	Organic	2	SS	10		
1.9	SILTY SAND, Compact to Very Dense	3	SS	14		0 80 (20)
		4	SS	52		
		5	SS	96		0 62 (38)
		6	SS	97		
295.9		7	SS	31		
5.9	SILT, Trace of Sand, Compact to Dense	8	SS	36		
		9	SS	23		0 6 (94)
		10	SS	29		
293.3		11	SS	23		
8.5	CLAYEY SILT, Very Stiff	12	SS	28		
292.0		13	SS	148	/20cm	
9.8	Gravelly Sand	14	SS	200	/28cm	24 67 (9)
	SILTY SAND TO SAND, Trace of Gravel, Very Dense	15	SS	210	/25cm	
		16	SS	195	/28cm	
		17	SS	210	/28cm	
		18	SS	230	/25cm	
		19	SS	235	/25cm	8 72 (20)
		20	SS	268	/23cm	
277.8		21	SS	257	/25cm	0 92 (8)
24.0	End of Borehole					

• Note:
Borehole Dry on Completion
Perched Water Level Encountered
at El. 295.6

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100
ELEVATION SCALE: 300, 298, 296, 294, 292, 290, 288, 286, 284, 282, 280, 278
PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L
WATER CONTENT (%) 15 30 45
UNIT WEIGHT γ kN/m^3

$+^3_{\cdot}$ $\times^5$: Numbers refer to Sensitivity

$15\overset{20}{\diamond}5$ (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No 6

1 OF 1 METRIC

W.P. 26 - 96 - 01 LOCATION CO - ORDS: N 4 910 651.4; E 290 078.7 ORIGINATED BY M V
DIST 33 HWY 400 BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST COMPILED BY M V
DATUM GEODETIC DATE 1997 04 08 CHECKED BY T C K

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
302.0	Ground Surface					
0.0	Topsoil					
	SILTY SAND, Trace of Gravel, Trace of Organic, Compact (Fill)	1	SS	27	*	
300.1		2	SS	19		
1.9	Loose	3	SS	9		
		4	SS	41		
	Sandy Silt	5	SS	50		0 35 (65)
	SILTY SAND, Dense to Very Dense	6	SS	60		
		7	SS	250		
		8	SS	269		
293.3		9	SS	144		0 71 (29)
8.7	SILT, Trace of Sand, Very Dense	10	SS	103		
		11	SS	62		0 10 (90)
291.0		12	SS	162	/25cm	0 12 (88)
11.0	SILTY CLAY, Hard	13	SS	30		
		14	SS	69		
288.5		15	SS	154		9 76 (13)
13.5	SILTY SAND TO SAND, Trace of Gravel, Very Dense	16	SS	125	/10cm	
		17	SS	154	/23cm	
		18	SS	176	/15cm	
		19	SS	123	/15cm	
277.8		20	SS	131	/15cm	0 90 (10)
24.2	End of Borehole					

• Note:
Borehole Dry on Completion
Perched Water Level Encountered
at El. 297.3

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100
ELEVATION SCALE: 300, 298, 296, 294, 292, 290, 288, 286, 284, 282, 280, 278
120/25cm
PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L
WATER CONTENT (%) 15 30 45
UNIT WEIGHT γ kN/m^3

$+^3_{\cdot}$ $\times^5$: Numbers refer to

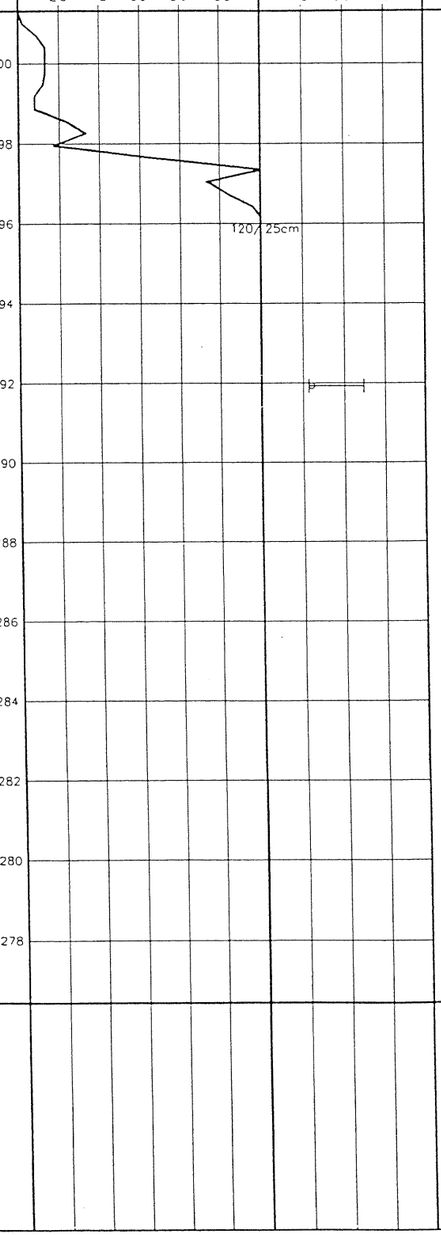
$15\overset{20}{\diamond}5$ (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 7 — 1 OF 1 — METRIC

W.P. 26 - 96 - 01 | LOCATION CO - ORDS: N 4 910 593.7; E 290 050.8 | ORIGINATED BY M V
DIST 33 HWY 400 | BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST | COMPILED BY M V
DATUM GEODETIC | DATE 1997 04 11 | CHECKED BY T C K

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
301.3	Ground Surface					
0.0	SILTY SAND, Loose (Fill)	1	SS	7	*	
300.2						
1.1	Organic	2	SS	7		0 71 (29)
	Very Loose to Loose	3	SS	3		
	SILTY SAND, Dense	4	SS	49		
		5	SS	40		
295.4						
5.9	SILT, Trace of Sand, Dense to Very Dense	6	SS	46		0 16 (84)
		7	SS	52		
293.1		8	SS	*0		0 8 (92)
8.2	SILTY CLAY, Hard	9	SS	38		
		10	SS	37		
291.3						
10.0		11	SS	138		
		12	SS	104	/15cm	
		13	SS	155		
	Gravelly Sand	14	SS	126	/15cm	26 63 (11)
	SILTY SAND TO SAND, Trace of Gravel, Very Dense	15	SS	175		6 86 (8)
		16	SS	115		
		17	SS	119		
		18	SS	117		
276.5		19	SS	159		
24.8	End of Borehole					

- Note: Borehole Dry on Completion. Perched Water Level Encountered at El. 295.7



UNIT WEIGHT γ kN/m³

$+^3$, $\times^5$: Numbers refer to Sensitivity

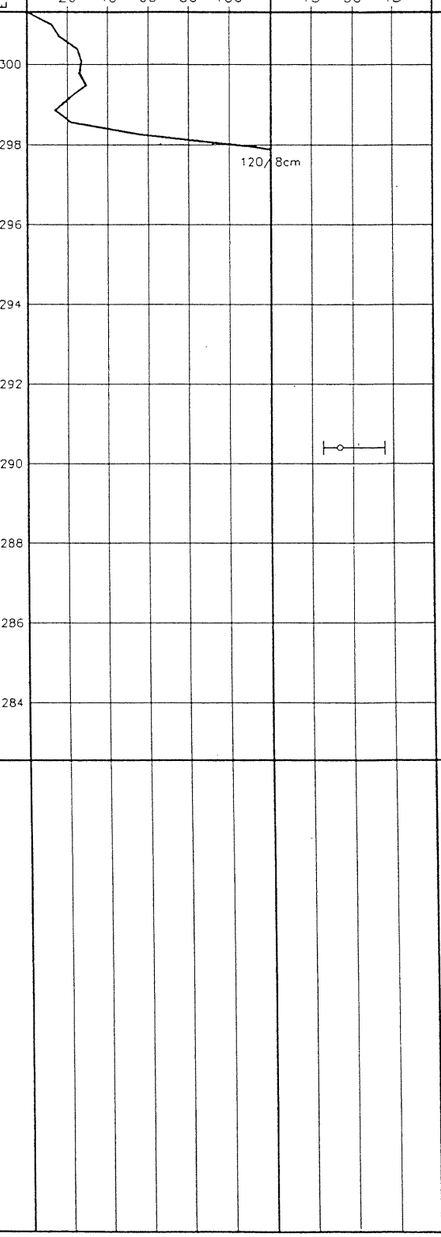
$15 \diamond^{20} 5$ (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 8 — 1 OF 1 — METRIC

W.P. 26 - 96 - 01 | LOCATION CO - ORDS: N 4 910 608.4; E 290 088.2 | ORIGINATED BY P C
DIST 33 HWY 400 | BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST | COMPILED BY M V
DATUM GEODETIC | DATE 1997 04 04 | CHECKED BY T C K

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
301.3	Ground Surface					
0.0	SILTY SAND, Compact (Fill)	1	SS	14	*	
299.9						
1.4	Organic	2	SS	5		
	Loose	3	SS	10		
	SILTY SAND, Very Dense	4	SS	49		
		5	SS	85		0 82 (18)
		6	SS	78		
		7	SS	98		
		8	SS	62		
		9	SS	61		
	Sandy Silt	10	SS	72		0 32 (68)
293.0						
8.3	SILT, Trace of Sand, Dense	11	SS	40		0 8 (92)
		12	SS	36		
291.3						
10.0	SILTY CLAY, Hard	13	SS	40		
290.1						
11.2	Gravelly Sand	14	SS	251		15 76 (9)
	SILTY SAND TO SAND, Trace of Gravel, Very Dense	15	SS	254	/25cm	
		16	SS	165		
		17	SS	241		
282.6		18	SS	239		0 83 (17)
18.7	End of Borehole					

- Note: Borehole Dry on Completion. Perched Water Level Encountered at El. 295.1



UNIT WEIGHT γ kN/m³

$+^3$, $\times^5$: Numbers refer to Sensitivity

$15 \diamond^{20} 5$ (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No 9 — 1 OF 1 — METRIC

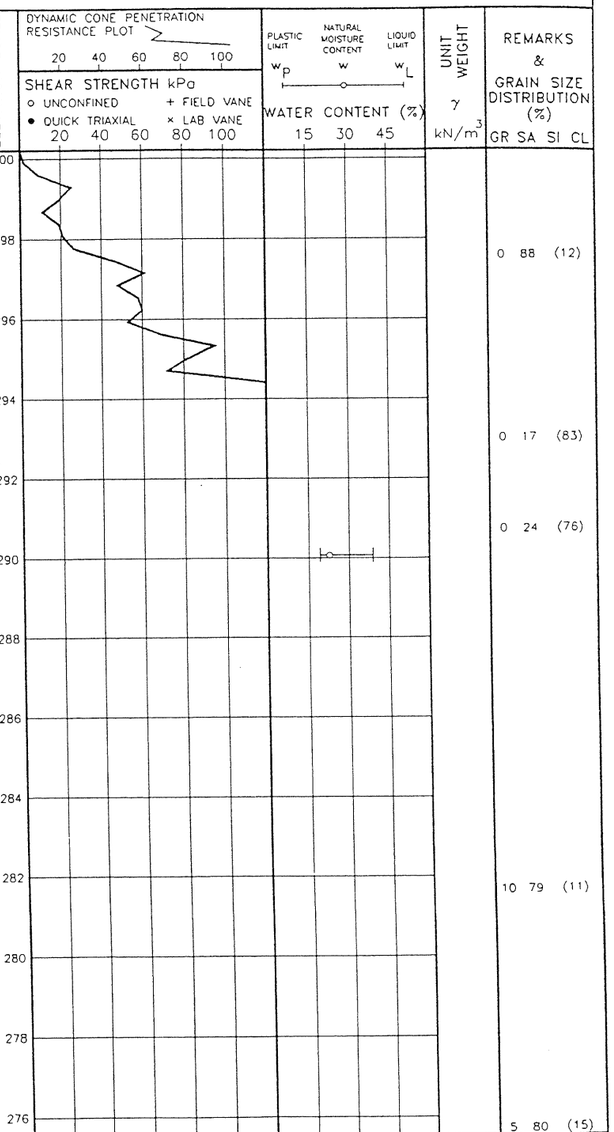
W.P. 26 - 96 - 01 | LOCATION CO - ORDS: N 4 910 586.7; E 290 097.7 | ORIGINATED BY M V

DIST 33 | HWY 400 | BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST | COMPILED BY M V

DATUM GEODETIC | DATE 1997 04 07 | CHECKED BY T C K

ELEV / DEPTH	SOIL PROFILE — DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
300.2	Ground Surface					
0.0 / 299.2	SILTY SAND, Trace of Organic. Loose (Fill)				*	
1.0	SILTY SAND. Compact to Dense	1	SS	12		
		2	SS	12		
		3	SS	35		0 88 (12)
		4	SS	42		
		5	SS	31		
293.5		6	SS	29		
6.7	SILT, Some Sand. Compact to Dense	7	SS	51		0 17 (83)
		8	SS	29		
		9	SS	35		
		10	SS	43		0 24 (76)
290.1		11	SS	30		
10.1 / 289.2	SILTY CLAY. Hard	12	SS	67		
11.0	SILTY SAND TO SAND. Trace of Gravel. Very Dense	13	SS	181	/28cm	
		14	SS	135	/23cm	
		15	SS	116	/15cm	
		16	SS	155	/23cm	
		17	SS	105	/15cm	10 79 (11)
		18	SS	156	/15cm	
275.5		19	SS	112	/15cm	5 80 (15)
24.7	End of Borehole					

• Note: Borehole Dry on Completion. Perched Water Level Encountered at El. 294.0



$+^{3}$, $\times^{5}$: Numbers refer to Sensitivity

$15 \diamond^{20} 5$ (%) STRAIN AT FAILURE

78 12 M

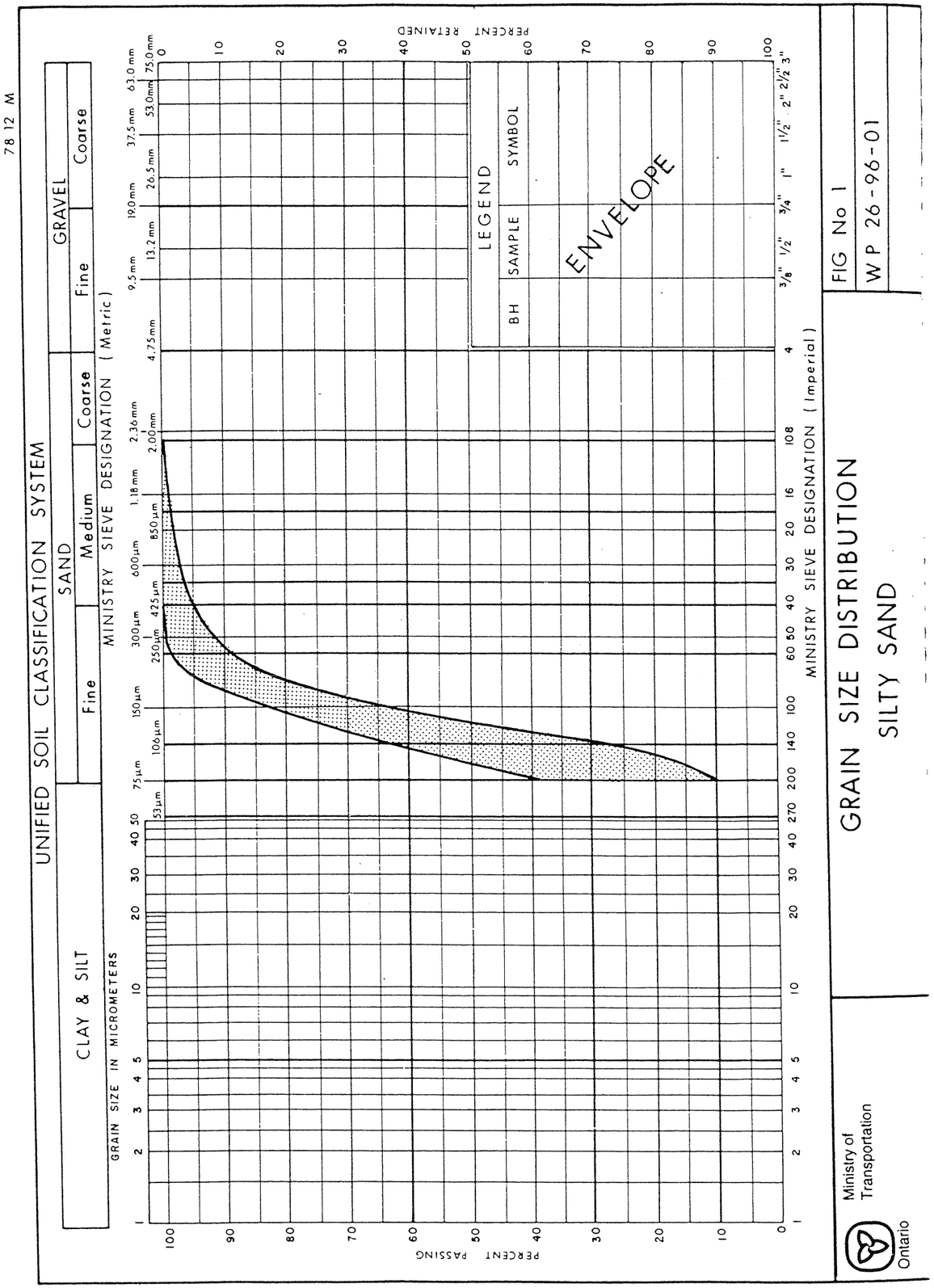


FIG No 1

W P 26-96-01

GRAIN SIZE DISTRIBUTION

SILTY SAND

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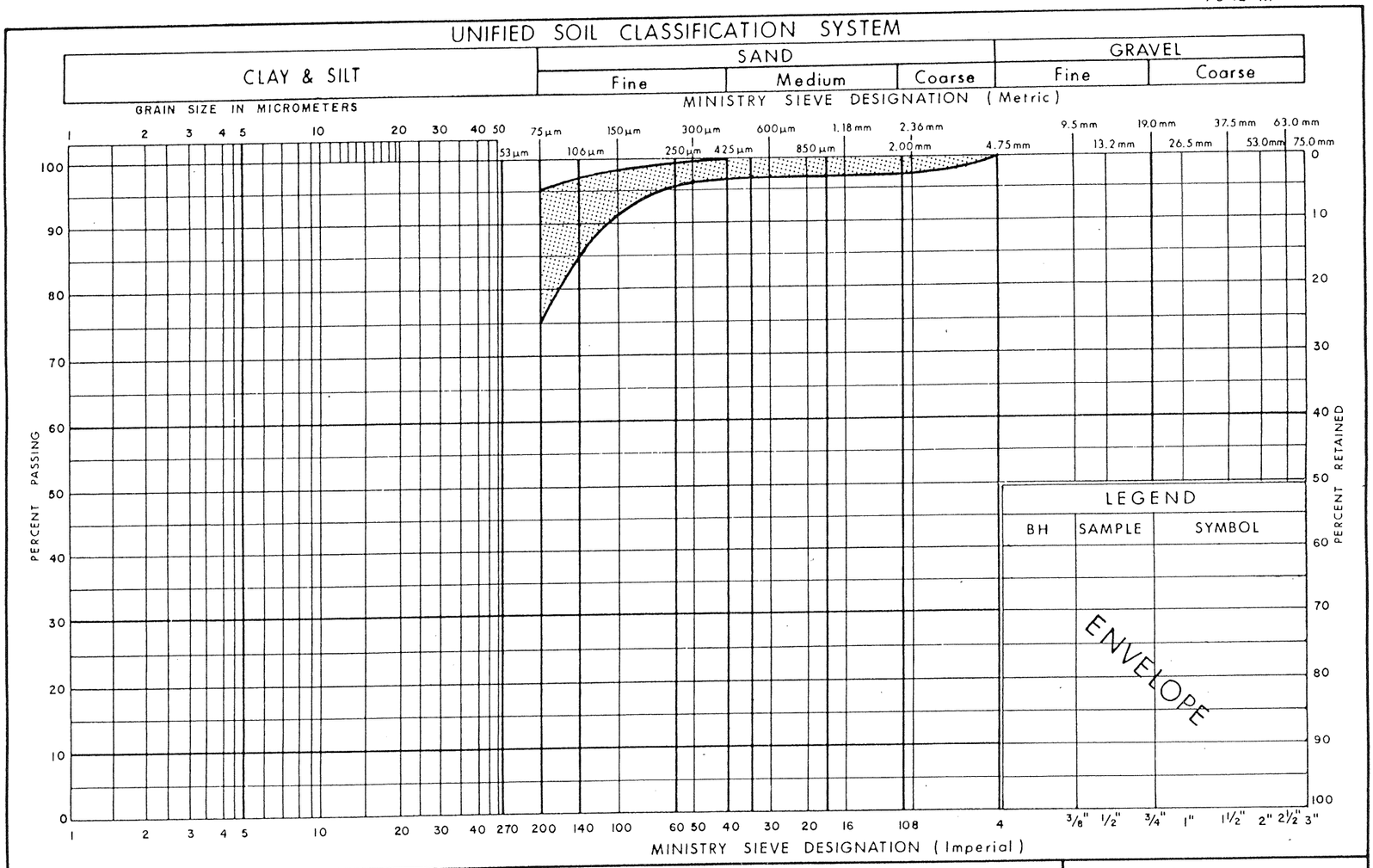
78 12 M



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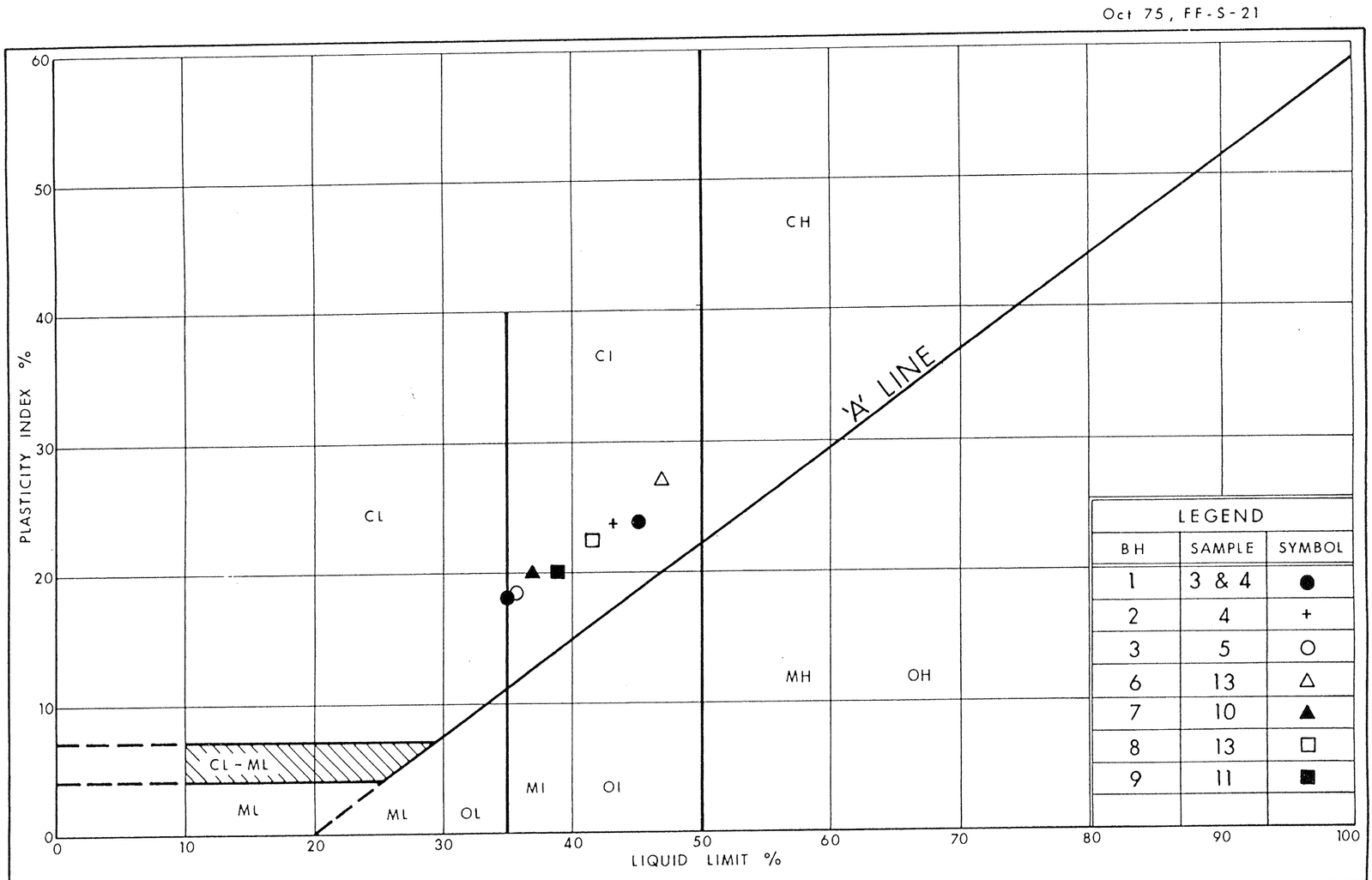
GRAIN SIZE DISTRIBUTION

SILT, TRACE / SOME SAND

FIG No 2

W P 26-96-01

Oct 75, FF-S-21



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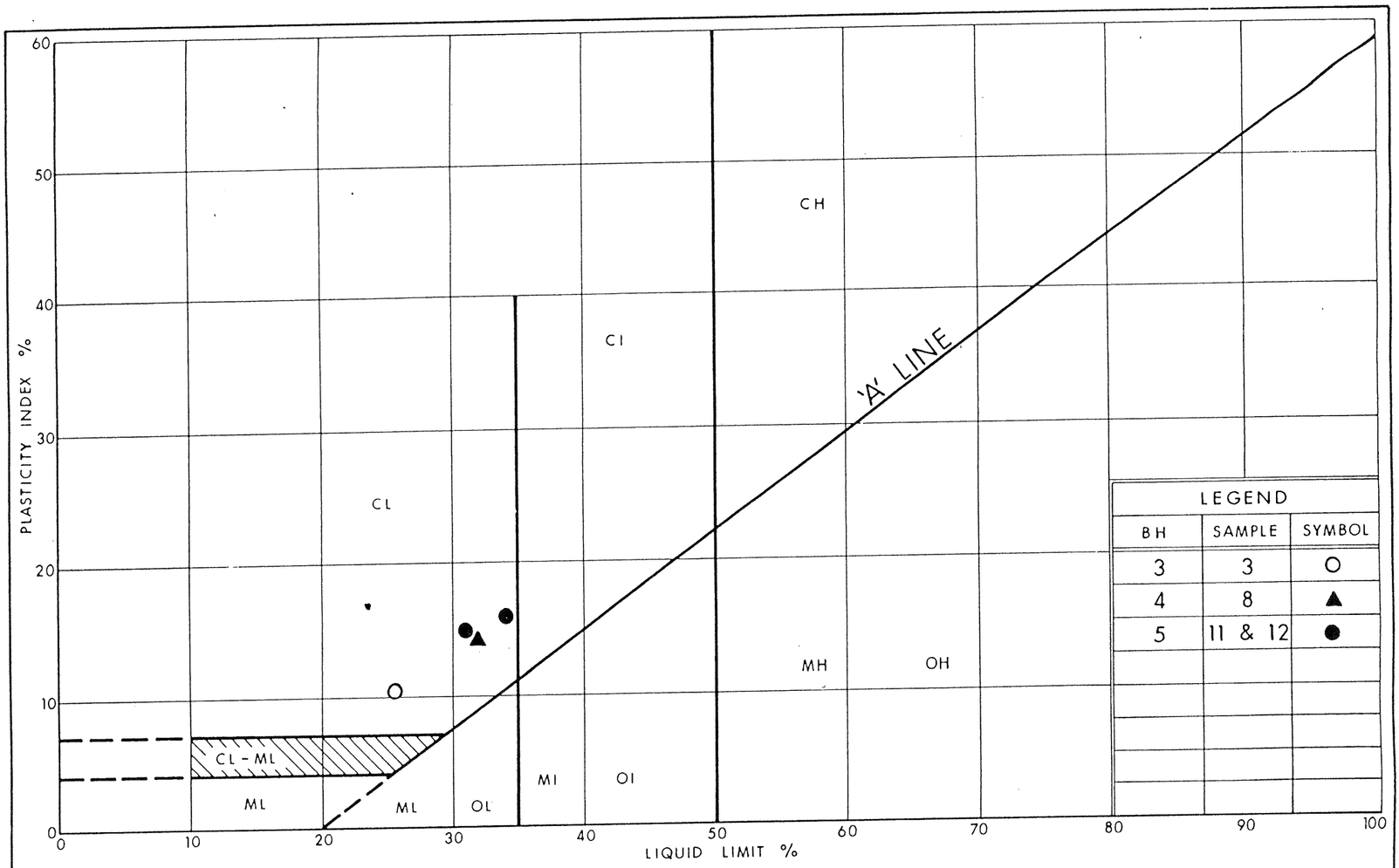
PLASTICITY CHART

SILTY CLAY

FIG No 3

W P 26-96-01

Oct 75, FF-S-21



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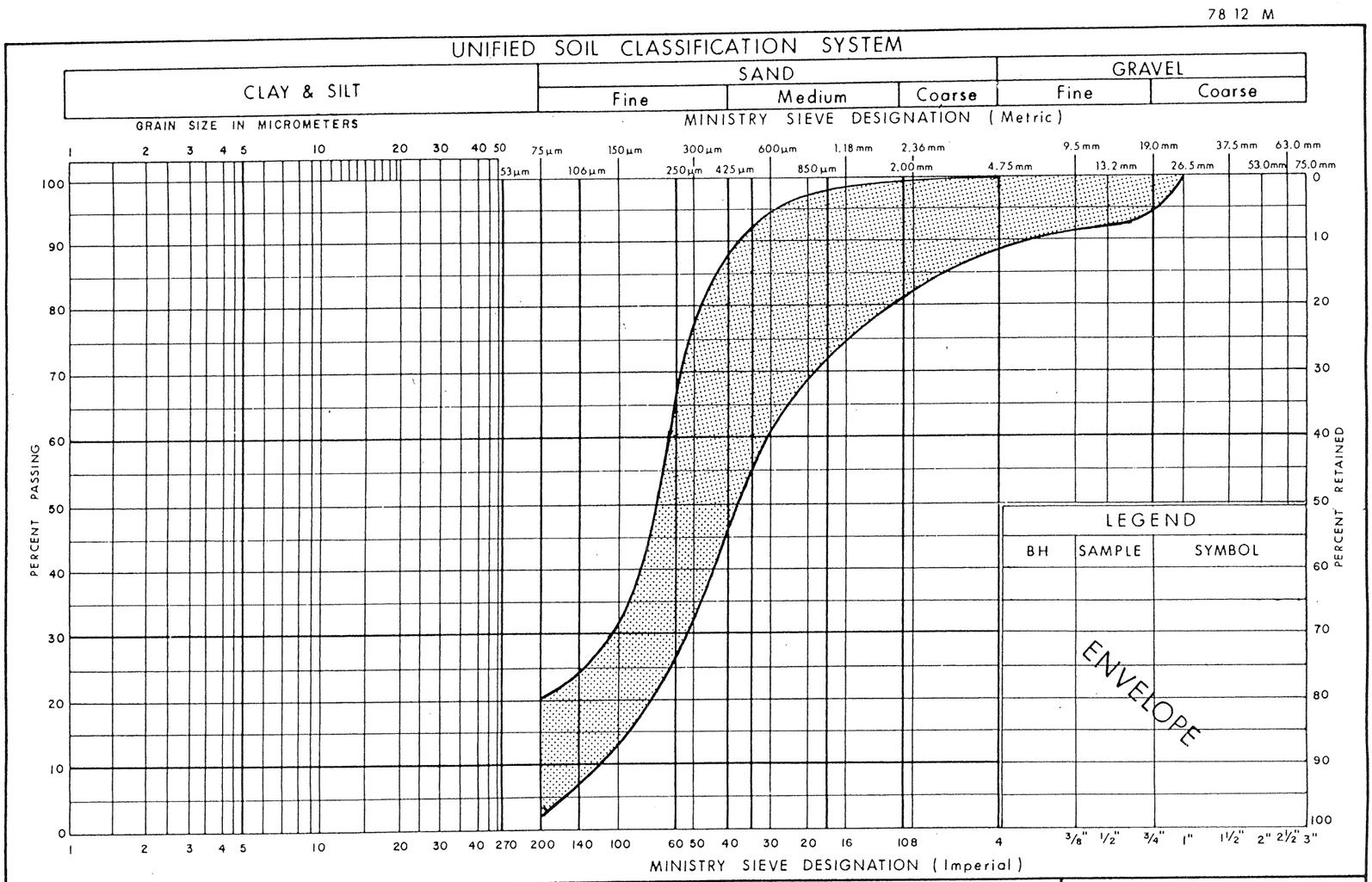
PLASTICITY CHART
CLAYEY SILT

FIG No 4
W P 26-96-01

78 12 M



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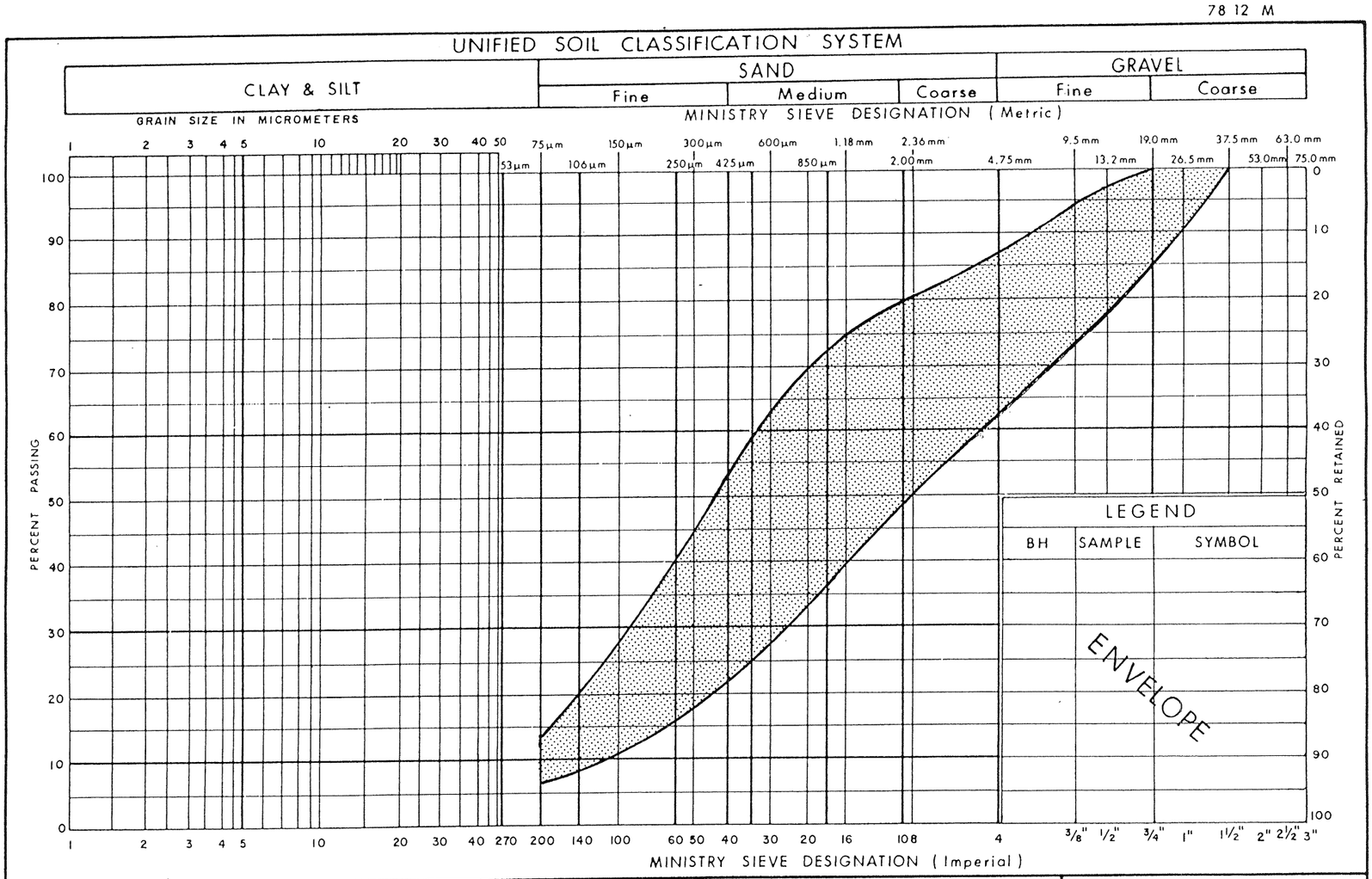
GRAIN SIZE DISTRIBUTION
SILTY SAND TO SAND,
TRACE OF GRAVEL

FIG No 5
W P 26-96-01



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GRAIN SIZE DISTRIBUTION
GRAVELLY SAND

FIG No 6
W P 26-96-01

Golder Associates Ltd.

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Telephone (905) 567-4444
Fax (905) 567-6561

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT ESSA ROAD (SIMCOE ROAD 30) OVERPASS STRUCTURE SITE 30-178 HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11 G.W.P. 30-95-00. AGREEMENT NO. 3005-A-000074

Submitted to:

URS Cole, Sherman
75 Commerce Valley Drive East
Thornhill, Ontario
L3T 7N9

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January 2002

001-1143F-8

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LIST OF DRAWINGS

Drawing 1 Essa Road Overpass, Highway 400, Borehole Location Plan

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Appendix A Records of Boreholes and Test Results – 1970 Investigation

PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
ESSA ROAD (SIMCOE ROAD 30) OVERPASS
STRUCTURE SITE 30-178
HIGHWAY 400 WIDENING FROM 1 KM SOUTH
OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

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Drawing 1
Appendix A

LIST OF DRAWINGS

Drawing 1 Essa Road Overpass, Highway 400, Borehole Location Plan

LIST OF APPENDICES

Appendix A Records of Boreholes and Test Results – 1970 Investigation

1.0 INTRODUCTION

Golder Associates Ltd. has been retained by URS Cole, Sherman (Cole, Sherman) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the ultimate widening of Highway 400 from 1 km south of Highway 89, northerly 30 km to Highway 11, in Simcoe County, Ontario. Foundation engineering services are required for the widening and / or replacement of eighteen existing overpass and underpass structures, as well as five structural culverts.

This report addresses the widening and / or replacement of the Essa Road (Simcoe Road 30, formerly Highway 27) overpass structure in Barrie, Ontario. Existing subsurface data for this site from an investigation conducted by the Department of Highways, Ontario in 1970 (*"Foundation Investigation Report for Proposed Extensions to the Overpass Structures at the Crossing of Highway 400 and Highway 27, City of Barrie"*, dated December 1970 – GEOCRES File No. 31D-169) were used to determine the subsurface conditions for this preliminary design study.

The terms of reference for the scope of work are outlined in Golder Associates' Proposal No. P01-1192, dated June 2000.

2.0 SITE DESCRIPTION

The existing Essa Road (Simcoe Road 30, formerly Highway 27) overpass is located about 3 km north of the Molson Park Drive interchange and 2.5 km south of Dunlop Street (Simcoe Road 90, formerly Highway 90) in Barrie, Ontario. The MTO has designated this overpass as Structure Site 30-178.

The Highway 400 embankment is up to 6.5 m high, with the highway grade at about Elevation 253.5 m to 253 m at the structure site, declining northward. The Essa Road grade is at about Elevation 246.5 m.

The existing overpass structures were widened in the early 1970s under Contract 71-150. According to the general layout drawing for this contract, which was provided by Morrison Hershfield (the structural designers for this preliminary study), the abutments and associated wing walls and retaining walls are supported on spread footings. The spread footings are founded at about Elevation 244.8 m and 245.3 m at the north and south abutments, respectively.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was previously carried out at this site by the Department of Highways, Ontario (DHO) in October 1970. At that time, two boreholes were advanced to determine the subsurface conditions for widening of the then-existing northbound and southbound lane overpass structures. Boreholes 1 and 2 were extended to 12 m and 20 m depth on the east and west sides of Highway 400, respectively.

Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. The groundwater conditions in the open boreholes were observed during and following the drilling operations. Laboratory index and classification tests, consisting of natural moisture content determinations, Atterberg Limits testing and grain size distribution analyses, were carried out on selected soil samples.

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

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

This 30 km section of Highway 400 traverses, from south to north, the following physiographic regions as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, Third Edition, 1984): the Simcoe Lowlands; the Peterborough Drumlin Field; a second lobe of the Simcoe Lowlands; and the Simcoe Uplands. Along Highway 400, the Simcoe Lowlands are present from the southern limit of the project to just south of Innisfil Creek, and again from Essa Road (Simcoe Road 30, formerly Highway 27) to about 1 km north of Dunlop Street (Simcoe Road 90, formerly Highway 90). The Peterborough Drumlin Field occupies the belt between these lobes of the Simcoe Lowlands, extending from just south of Innisfil Creek, which is located about 1 km north of Highway 89, to Essa Road. The Simcoe Uplands extend from about 1 km north of Dunlop Street to beyond the northern limit of the project at Highway 11.

The two sections where Highway 400 crosses the Simcoe Lowlands consist of two lobes of a sand plain which include the shores of Kempenfelt Bay, the Nottawasaga River and Innisfil Creek. The surficial soils of these sections of the Simcoe Lowlands consist primarily of sand, although silt, clay or peat may be found in low-lying areas. The Essa Road site is located within the Simcoe Lowlands physiographic region, just north of the Peterborough Drumlin Field boundary.

The surficial soils in the Peterborough Drumlin Field consist primarily of gravelly sand till or sand and gravel deposits. Drumlins (glacially-shaped hills) are more frequent in the southern portion of the section of the Peterborough Drumlin Field traversed by Highway 400. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins.

The surficial soils in the Simcoe Uplands physiographic region are primarily sandy silt till deposits, known to contain boulders. Low-lying areas may be infilled with shallow sand and gravel deposits, which are shoreline deposits of a former glacial lake that once flooded the area.

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the 1970 boreholes, and the results of in-situ and laboratory testing, are given on the Record of Borehole sheets and figures contained in Appendix A. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

Boreholes 1 and 2 were advanced on the east and west sides of Highway 400, from approximately Essa Road grade. The approximate locations and ground surface elevations for these boreholes are shown on the attached Drawing 1.

In summary, the subsoils at the site consist of surficial deposits of loose to very dense sand and very stiff clayey silt, overlying a deposit of compact to very dense silty sand till. A detailed description of the subsurface conditions encountered in the boreholes is provided in the following section.

4.2.1 Sand

A surficial sand deposit, 2.3 m to 8.5 m in thickness, was encountered in both boreholes. The sand extended to about Elevation 240.5 m in Borehole 1, and to about Elevation 238.5 m in Borehole 2, on the east and west sides of Highway 400, respectively. The sand contains trace to some silt and trace gravel; grain size distribution test results for samples of this surficial sand layer are presented on Figure 1 in Appendix A. A layer of clayey silt was encountered within the sand; this sub-layer is discussed in Section 4.2.2.

The sand layer was moist, with measured natural water contents of 5 to 8 per cent. The measured Standard Penetration Test (SPT) 'N' values ranged from 6 to 65 blows, but were typically between about 20 and 40 blows per 0.3 m of penetration. The sand therefore has a predominantly compact to dense relative density.

4.2.2 Clayey Silt

A 1.8 m thick layer of clayey silt was encountered within the sand deposit in Borehole 2, on the west side of Highway 400. The surface of the clayey silt layer was at about Elevation 245.5 m.

The clayey silt was moist, with measured natural water contents of 17 and 23 per cent. Atterberg Limits testing on both samples of this material measured plastic limits of 16 and 21 per cent, liquid limits of 25 and 26 per cent, and plasticity indices of 5 and 9 per cent; the results of the Atterberg Limits testing indicate that the clayey silt is inorganic and of low plasticity. Measured SPT 'N' values of about 27 blows per 0.3 m of penetration for both samples indicate that the clayey silt layer has a very stiff consistency.

4.2.3 Silty Sand Till

The 1970 boreholes encountered a silty sand till deposit below the surficial sand deposit. The surface of the silty sand till was encountered at Elevation 240.5 m and 238.5 m in the boreholes on the east and west sides of Highway 400, respectively. The till extended to the maximum depth investigated (approximately 20 m depth, corresponding to about Elevation 227.5 m).

The silty sand till contains trace clay and gravel. An envelope of grain size distribution test results for samples of the till is shown on Figure 2 in Appendix A.

The measured natural moisture contents of two samples of the silty sand till were 13 and 17 per cent. The measured SPT 'N' values ranged from 14 to 79 blows per 0.3 m of penetration, but were typically between 14 and 50 blows per 0.3 m of penetration, indicating that the till has a predominantly compact to dense relative density.

4.3 Groundwater Conditions

The groundwater conditions were observed in the open boreholes during the October 1970 drilling investigation. The water levels were measured at Elevation 245 m and 243 m on the east and west sides of Highway 400, respectively. These water levels are between 1.5 m and 3.5 m below the Essa Road grade.

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

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
ESSA ROAD (SIMCOE ROAD 30) OVERPASS
STRUCTURE SITE 30-178
HIGHWAY 400 WIDENING FROM 1 KM SOUTH
OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides preliminary foundation design recommendations for the widening and / or replacement of the existing Essa Road (Simcoe Road 30, formerly Highway 27) overpass structure, associated with the widening of Highway 400. The recommendations are preliminary only and are based on interpretation of the factual data obtained from a limited number of boreholes advanced during a 1970 subsurface investigation at this site. The interpretation and recommendations provided are intended for planning purposes only, to provide the information necessary at this stage of the study. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the planning of the project. Further foundation investigation will be required at this structure site as part of the detailed design stage of the project.

It is understood that Highway 400 will be widened from its existing six-lane configuration to an interim configuration of eight lanes, and an ultimate configuration of ten lanes, and that an alternative for a twelve-lane express / collector system is under consideration between Molson Park Drive and Duckworth Street in Barrie. Throughout the project length, it is expected that the existing highway platform will be widened by between 13 m and 30 m. Widening and / or replacement of the existing Essa Road overpass structure will therefore be necessary.

Based on the available general layout drawing for the existing single-span overpass, the abutments and associated retaining and wing walls are supported on spread footings. The north and south abutment footings are founded at about Elevation 244.8 m and 245.3 m, respectively. The Essa Road grade is at about Elevation 246.5 m, and the Highway 400 grade is at about Elevation 253.5 m to 253 m at the structure site, declining northward.

5.2 Bridge Foundation Options

The soils below the Essa Road level consist of a surficial deposit of predominantly compact to dense sand, containing a very stiff clayey silt interlayer, overlying a compact to very dense silty sand till deposit. The water levels measured in the open boreholes in October 1970 were at Elevation 245 m and 243 m on the east and west sides of Highway 400, respectively, between 1.5 m and 3.5 m below the Essa Road grade, but they could be higher particularly during wet weather periods.

Based on these subsurface conditions and on the existing overpass foundation conditions, the widened structures could be founded on spread footings, placed on the generally compact to

dense surficial sand deposit to match the founding levels for the existing spread footings. If a new replacement structure is to be constructed, consideration could also be given to the use of perched abutments, founded on spread footings placed on a compacted granular pad within the approach embankments. Alternatively the structure could be supported on deep foundations, such as driven steel H-piles.

Preliminary recommendations for spread footings, including perched abutments, and for deep foundations are provided in the following sections.

5.3 Spread Footings

For preliminary design of the abutment foundations, spread footings may be placed at or below Elevation 245.3 m, on the generally compact to dense sand deposit or the very stiff clayey silt interlayer that exists at Borehole 2. Where the existing structure is being widened, the new footings should match the existing founding levels. A minimum of 1.5 m of soil cover should be provided to the footings. Specific precautions must be taken during the excavation and construction period to avoid disturbance and loosening of granular subsoil due to inadequate groundwater control.

Alternatively, if the existing structure is to be replaced, consideration could be given to the use of abutment footings perched within the approach embankments.

5.3.1 Axial Geotechnical Resistance

Spread footings placed on a properly prepared base at or below the design elevation given above may be designed for a factored geotechnical resistance at Ultimate Limit States (ULS) of 500 kPa, assuming a 2.5 m wide footing.

Differential settlement will occur between the new footings and the existing structure. Due to the granular nature of the subsoils, the majority of this settlement is expected to occur during or shortly after the construction period; however, some longer-term consolidation settlement will occur in the clayey silt interlayer, if this is present at the locations of the new foundation elements. The settlement of the footings will be dependent on the footing size and configuration, on the applied loads, and on the distribution and thickness of any cohesive interlayers. For preliminary design purposes, the geotechnical resistance at Serviceability Limit States (SLS) may be taken as 300 kPa. The geotechnical resistances at ULS and SLS will have to be reviewed

following the detailed design stage of subsurface investigation, once the piezometric groundwater conditions at the site are confirmed and the footing size, configuration and loadings are known.

For spread footings placed within the approach embankments on a compacted Granular 'A' core, a factored geotechnical resistance at ULS of 900 kPa may be assumed for preliminary design. The geotechnical resistance at SLS will depend on the thickness of Granular 'A' and the consistency and thickness of the underlying soils; a value of 350 kPa may be assumed for preliminary design.

The geotechnical resistances provided herein are given under the assumption that the loads will be applied perpendicular to the surface of the footings; where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the Ontario Highway Bridge Design Code (OHBDC).

5.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footing and the subsoils should be calculated in accordance with Section 6-8.4.3 of the OHBDC.

The angle of friction between the concrete and the sand founding soils should be taken as 27 degrees; the corresponding coefficient of friction, tan δ, would then be 0.5. It should be assumed that the clayey silt interlayer will be present on the west side of the highway for preliminary design purposes; the angle of friction between the concrete and the clayey silt interlayer should be taken as 24 degrees, with a corresponding coefficient of friction of 0.45. Where "perched" abutment footings are adopted, the angle of friction between the concrete footings and the compacted Granular 'A' pad should be taken as 30 degrees; the corresponding coefficient of friction would be 0.58.

5.3.3 Frost Protection

The footings should be provided with a minimum of 1.5 m of soil cover for frost protection.

5.4 Driven Steel H-Piles

Consideration could be given to supporting a replacement structure or structurally-separate widening on steel H-piles. The recommendations that follow assume that the piles would be

driven to about Elevation 229 m, terminating in the silty sand till deposit; however, the available subsurface information is not sufficient to confirm this assumption. Further borehole investigation will be required in the detailed design stage to determine the probable founding elevation. It is assumed that the pile caps would be perched above the Essa Road grade, to minimize the potential requirement for appropriate groundwater control at the site.

5.4.1 Axial Geotechnical Resistance

For preliminary design of deep foundations at this site, the factored axial resistance at ULS for steel HP 310 x 110 H-piles driven to the design elevation given above may be taken as 1,000 kN. The axial resistance at SLS for a single pile, for 25 mm of settlement, may be taken as 800 kN. Provision should be made to re-tap selected piles to confirm the set after adjacent piles have been driven, in accordance with MTO's current Special Provision.

5.4.2 Resistance to Lateral Loads

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

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

$$k_h = \frac{n_h z}{B}$$

Where
n_h is the constant of subgrade reaction
z is the depth (m)
B is the pile diameter (m)

It is expected that the steel H-piles would be driven through embankment fill and the compact to dense sand layer, into the compact to dense silty sand till. For well-compacted granular embankment fill and native sand above the water table (i.e. above Elevation 245 m), the range in value of n_h may be taken as 10 MPa/m to 20 MPa/m in the structural analysis. For the predominantly compact to dense soils below the water table (i.e. below Elevation 245 m), the range in value of n_h may be taken as 5 MPa/m to 10 MPa/m in the structural analysis. These values will have to be confirmed following the detailed design stage of the subsurface investigation.

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

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

5.4.3 Frost Protection

The pile caps should be provided with 1.5 m of soil cover for frost protection.

5.5 Lateral Earth Pressures

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

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the abutments and walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00

- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the abutment wall, in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be used in accordance with OPSS 501.06.

- The granular fill may be placed either in a zone with width equal to at least 1.5 m behind the back of the stem (Case I from OHBDC Figure 6-7.4.1) or within the wedge-shaped zone

defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II from OHBDC Figure 6-7.4.4).

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

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

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

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

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

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

5.6 Embankment Design

Based on the topographic information on the Engineering and Title Records plates and on site reconnaissance, the existing Highway 400 embankment side slopes are formed at a gradient of about 2 horizontal to 1 vertical (2H:1V). For the embankment widening, the new side slopes should be formed at a maximum gradient of 2H:1V. The embankment widening should be carried out using conventional fill placement and compaction practices, and benching of the existing embankment side slopes should be carried out to key in the new fill.

5.7 Design and Construction Considerations

5.7.1 Groundwater Control

The water levels observed in the open boreholes in the October 1970 investigation were at Elevation 245 m to 243 m, about 1.5 m to 3.5 m below the Essa Road grade and near the design founding elevation for spread footings. Provision should, however, be made for higher groundwater levels during the construction period. Groundwater control will be required to prevent disturbance to the subgrade soils for spread footings founded on the native soils. To this end, it is recommended that the groundwater level be lowered prior to excavation and maintained 0.5 m below the founding level for the footings during the construction period.

The stabilized groundwater conditions at the Essa Road site will have to be determined during the detailed design stage; further assessment of the groundwater control requirements (such as well points or properly filtered sumps) will be necessary at that time.

5.7.2 Excavation

Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. The footing excavations will extend a minimum of 1.5 m below lowest surrounding grade, through generally compact to dense sand. The sand soils below the Essa Road grade would be classified as Type 2 / 3 soils. Temporary open-cut slopes should therefore be maintained no steeper than 1 horizontal to 1 vertical (1H:1V). Where space restrictions dictate, footing excavations could also be carried out within a braced excavation.

5.7.3 Settlement

Deformation of the ground due to foundation loading will result in settlement of the bridge abutments and superstructure. This settlement will be differential with respect to the existing overpass structure, and also could vary along the new structure depending on the variability and consistency/relative density of the founding soils. The potential for differential settlement should be reassessed during the detailed design stage once the proposed bridge configuration is established.

5.7.4 Obstructions

It is noted that cobbles and boulders are inherent in glacially-derived soils, and should therefore be expected during pile driving and / or temporary shoring system installation.

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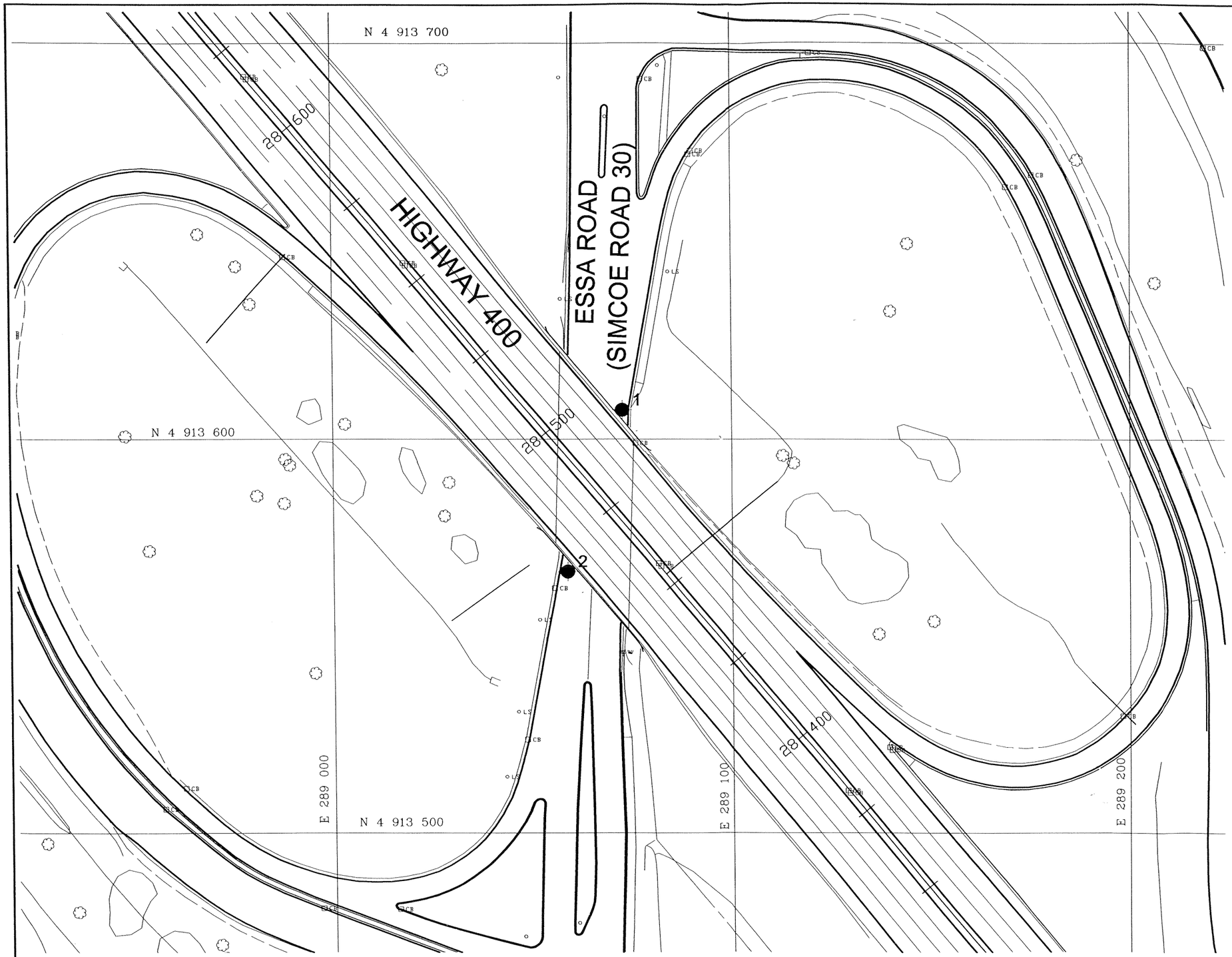
DIST HWY 400
CONT. No.
GWP No. 30-95-00

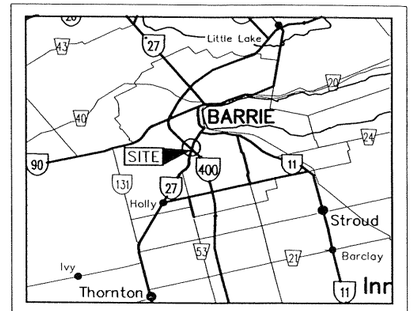
SHEET

ESSA ROAD OVERPASS
HWY 400
BOREHOLE LOCATION PLAN

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA





KEY PLAN

0 2.0 4.0 6.0 8.0 10.0 20.0
SCALE IN KILOMETRES

LEGEND

- Borehole, previous investigation
- Borehole, present investigation

No.	ELEVATION	LOCATION NORTHING	LOCATION EASTING
1	245.8	4,913,607	289,072
2	247.1	4,913,566	289,058

REFERENCE
This drawing was created from digital file "50207.dwg" provided by URS Cole Sherman

0 10 20 30 40 50 100m
SCALE IN METRES

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

NO.	DATE	BY	REVISION

Geocres No.

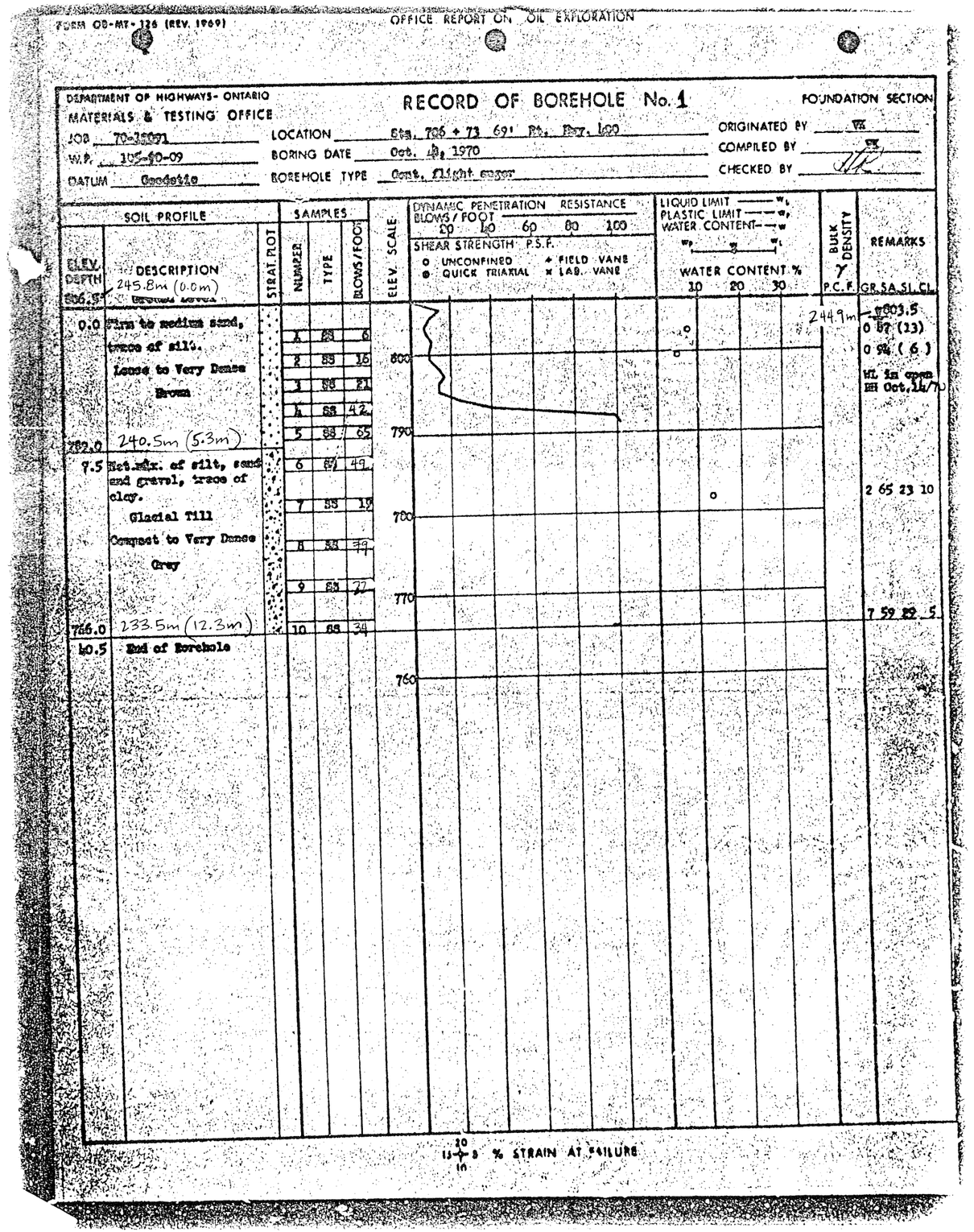
HWY. No. 400		PROJECT NO.: 001-1143F	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-178
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

P1143F13.DWG

plot scale 1:2/mp.

APPENDIX A

RECORDS OF BOREHOLES AND TEST RESULTS – 1970 INVESTIGATION



FORM OB-MT-126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 1

FOUNDATION SECTION

JOB 70-35091
W.P. 105-90-09
DATUM Geodetic

LOCATION Sta. 706 + 73 69' Rt. Hwy. 400
BORING DATE Oct. 4, 1970
BOREHOLE TYPE Cont. flight auger

ORIGINATED BY VK
COMPILED BY VK
CHECKED BY [illegible]

ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/FOOT	WATER CONTENT %	REMARKS
806.5	245.8m (0.0m) Ground Level					
0.0	Firm to medium sand, trace of silt. Loose to Very Dense Brown	1	SS	6		0 87 (13)
		2	SS	16		0 94 (6)
		3	SS	21		WL in open BH Oct. 14/70
		4	SS	42		
782.0	240.5m (5.3m)	5	SS	65		
7.5	Net mix. of silt, sand and gravel, trace of clay. Glacial Till Compact to Very Dense Grey	6	SS	49		2 65 23 10
		7	SS	19		
		8	SS	79		
		9	SS	22		
766.0	233.5m (12.3m)	10	SS	34		7 59 29 5
40.5	End of Borehole					

Elev. scale: 800, 790, 780, 770, 760

Dynamic penetration resistance: Blows/foot 20 40 60 80 100

Shear strength P.S.F.: o Unconfined, + Field vane, ● Quick triaxial, × Lab. vane

Liquid limit w_L, Plastic limit w_P, Water content w

Water content % 10 20 30

Bulk density γ P.C.F.

244.9m

GR. SA. SI. CL.

% STRAIN AT FAILURE

FORM CS MT-126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

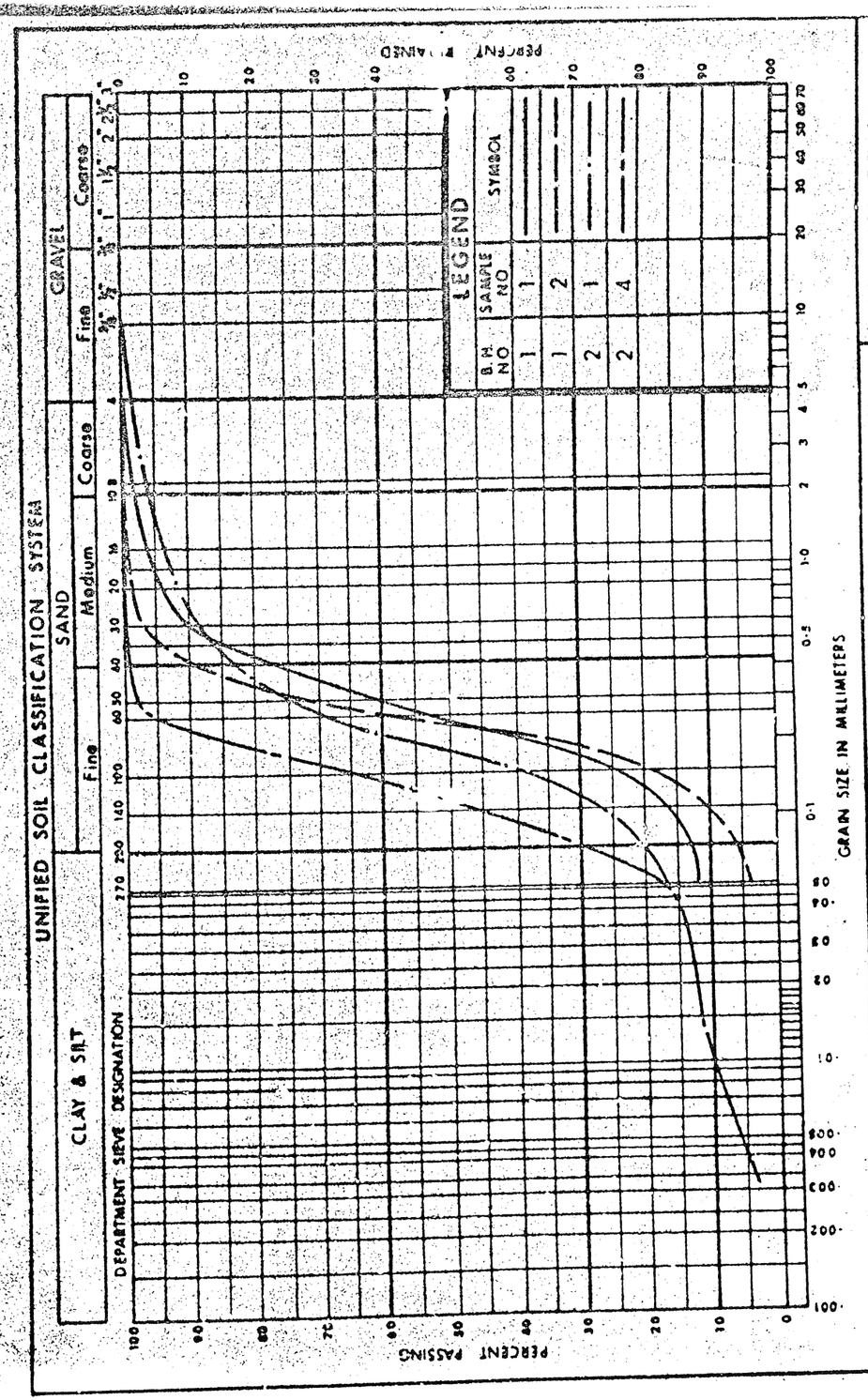
RECORD OF BOREHOLE No. 2

FOUNDATION SECTION

JOB 70-11091 LOCATION Bdg. 706 + 03 Grd. Lt. Ptr. 100 ORIGINATED BY [illegible]
W.P. 105-70-09 BORING DATE Oct. 15/70 COMPILED BY [illegible]
DATUM Geodetic BOREHOLE TYPE Cont. Flight Auger CHECKED BY [illegible]

SOIL PROFILE: ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT 20 40 60 80 100 / SHEAR STRENGTH P.S.F. ○ UNCONFINED ● QUICK TRIAXIAL ◆ FIELD VANE × LAB. VANE	LIQUID LIMIT w_L / PLASTIC LIMIT w_P / WATER CONTENT w — WATER CONTENT % 20 40 60	BULK DENSITY γ P.C.F.	REMARKS GR SA SI CL
242.2m (0.0m)										
0.0	Fine to medium sand with silt. Compact to Dense		1	SS	37	810				2 18 (20)
6.4	Clayey silt with trace of sand and gravel. Very Stiff		2							
			3	SS	37					
12.0	Brown-Grey		4	SS	42	800			242.9m	0 70 22 1
	240.5m (3.7m)		5	SS	21					▽797.0 in open BH Oct. 15/70
			6	SS	20					
			7	SS	20	790				
203.0	238.7m (8.5m)									
28.0			8	SS	16	780				1 87 26 6
	Bot. mix. of silt, sand and gravel, trace of clay. Compact to Very Dense Grey		9	SS	51					
			10	SS	14	770				
						760				
						750				
46.0	227.4m (19.8m)									
55.0	End of Borehole					740				

10 15 → 5 % STRAIN AT FAILURE 10

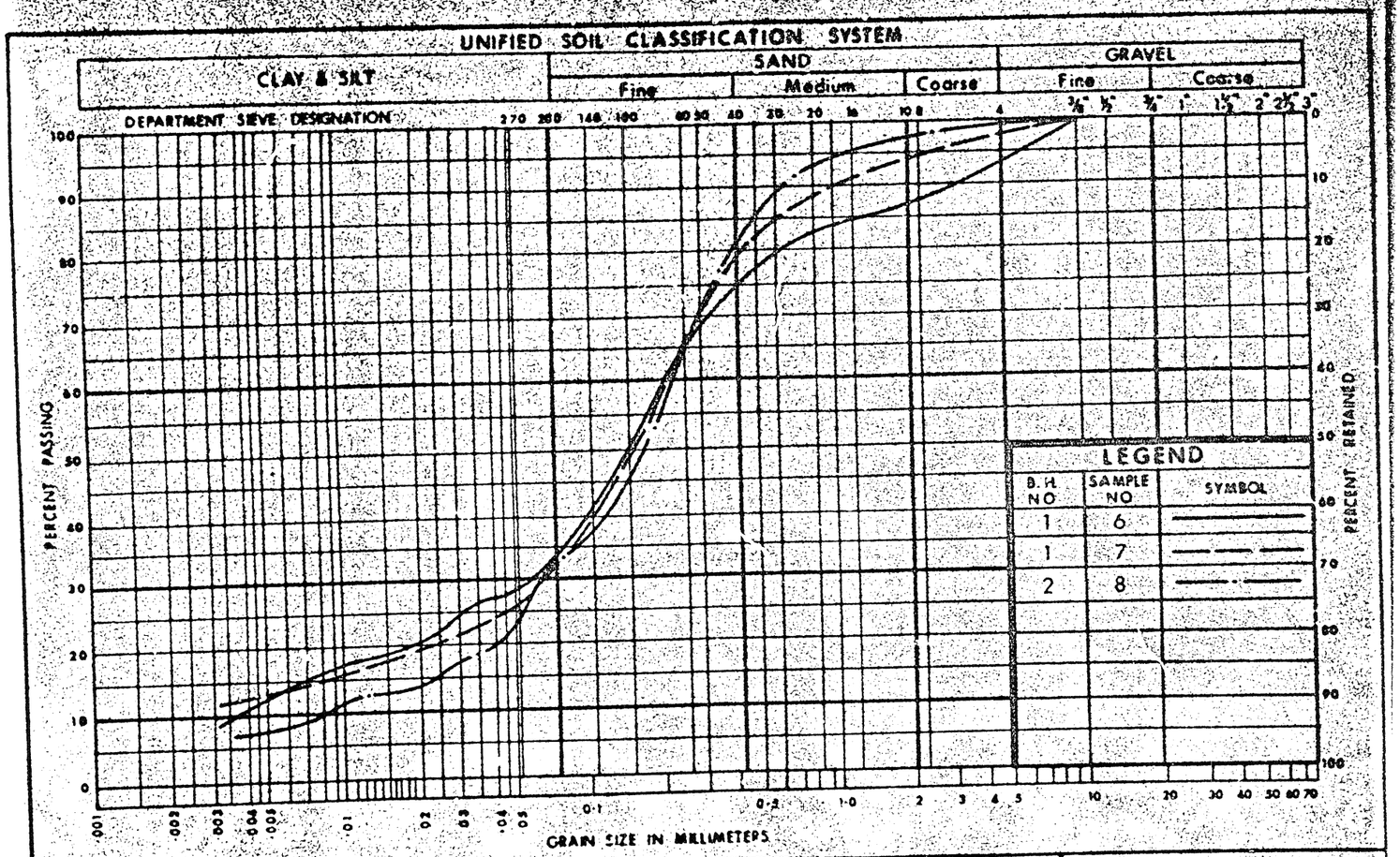


W.P. No. 105-70-09
JOB No. 70-11091
FIG. 1

GRAIN SIZE DISTRIBUTION
SAND WITH SILT

DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION

FORM CS MT-
REV. 1968



DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION

GRAIN SIZE DISTRIBUTION
GLACIAL TILL

W.P. No. 105-70-09
JOB No. 70-11091
FIG. 2

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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT BARRIE / COLLINGWOOD RAILWAY OVERPASS STRUCTURE SITE 30-177 HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11 G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

Submitted to:

URS Cole, Sherman
75 Commerce Valley Drive East
Thornhill, Ontario
L3T 7N9

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January 2002 001-1143F-9

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PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
BARRIE / COLLINGWOOD RAILWAY OVERPASS
STRUCTURE SITE 30-177
HIGHWAY 400 WIDENING FROM 1 KM SOUTH
OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

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1.0 INTRODUCTION

Golder Associates Ltd. has been retained by URS Cole, Sherman (Cole, Sherman) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the ultimate widening of Highway 400 from 1 km south of Highway 89, northerly 30 km to Highway 11, in Simcoe County, Ontario. Foundation engineering services are required for the widening and / or replacement of eighteen existing overpass and underpass structures, as well as five structural culverts.

This report addresses the widening and / or replacement of the Barrie / Collingwood Railway (former CN Rail) overpass structure in Barrie, Ontario. A subsurface investigation has been carried out, in which one borehole was advanced and in-situ and laboratory testing was conducted, to determine the subsurface conditions at the site for this preliminary design study.

The terms of reference for the scope of work are outlined in Golder Associates' Proposal No. P01-1192, dated June 2000. Under the original scope of work, two boreholes were to be advanced at this site for preliminary investigation purposes. However, during drilling of Borehole B10-1 at the Tiffin Street site, located less than 200 m north of the Barrie / Collingwood Railway site, the City of Barrie informed Golder Associates of the presence of trichloroethylene (TCE) contamination in the area encompassing the Barrie / Collingwood Railway, Tiffin Street and Dunlop Street sites. Based on review of the then-available subsurface information for the Essa Road and Anne Street sites to the south and north of this area, respectively, and following discussions with the MTO Foundations Section, the scope of work was revised as follows:

- Borehole B10-1, which had already been advanced at the Tiffin Street site, would be used to provide preliminary subsurface information for both the Tiffin Street site and the Barrie / Collingwood Railway (former CN Rail) site, less than 200 m to the south. No further drilling would be carried out at these two sites during the preliminary foundation investigation stage.

- One borehole would be drilled at the Dunlop Street (Simcoe Road 90, formerly Highway 90) site, located about 1 km north of the Tiffin Street site. This borehole would be advanced using environmental drilling procedures and telescoping of casing to minimize the potential for drilling-induced downward movement of any contaminants at the site.

The above revised scope of work was approved by the MTO Foundations Section.

2.0 SITE DESCRIPTION

The existing Barrie / Collingwood Railway (former CN Rail) overpass is located about 200 m south of the Tiffin Street overpass and 1.2 km south of the Dunlop Street (Simcoe Road 90, formerly Highway 90) interchange, in Barrie, Ontario. The MTO has designated this overpass as Structure Site 30-177.

At the site, the Barrie / Collingwood Railway grade is at about Elevation 235 m, near the original ground surface. Highway 400 has been constructed on embankment fill up to about 8 m high; the highway grade is at about Elevation 243 m, declining toward both the south and north.

The existing single-span overpass was constructed in the early 1950s under Contract 50-10. According to the general layout drawing for the existing structure, which was provided by Morrison Hershfield (the structural designers for this preliminary study), the abutments are supported on spread footings which step upward from about Elevation 233.5 m to 233.9 m between the east and west ends, respectively. The associated wing walls and retaining walls are also supported on spread footings founded to match the abutment footings.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at the Tiffin Street overpass site, located less than 200 m north of the Barrie / Collingwood Railway site, in October 2000. Due to the presence of trichloroethylene (TCE) contamination in the area encompassing the Barrie / Collingwood Railway and Tiffin Street sites, it was determined in conjunction with the MTO Foundations Section that the borehole that was advanced at the Tiffin Street site would be used to provide preliminary subsurface information for both the Tiffin Street site and the Barrie / Collingwood Railway site; no further drilling would be carried out at these two sites during the preliminary foundation investigation stage.

Borehole B10-1 was located on the west side of Highway 400, south of Tiffin Street. This borehole was advanced from slightly above Tiffin Street grade to about 23 m depth (approximately Elevation 212 m).

The investigation was carried out using a bombardier-mounted CME-55 drill rig supplied and operated by Master Soil Investigations Ltd. of Weston, Ontario. The borehole was advanced using hollow stem augers. Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. The water level in the open borehole was observed throughout the drilling operations, and a piezometer was installed to permit monitoring of the groundwater level at the site.

The field work was supervised on a full-time basis by a member of our staff who located the borehole in the field, directed the drilling, sampling, and in-situ testing operations, and logged the borehole. The soil samples were identified in the field, placed in labelled containers and transported to Golder Associates' laboratory in Mississauga for further examination. Index and classification tests consisting of water content determinations, Atterberg Limits testing and grain size distribution analyses were carried out on selected soil samples.

In addition, a sample of the stockpiled cuttings was obtained for chemical analysis, in order to determine the waste classification for disposal of this soil. This sample was submitted to Philip Analytical Services for leachate analysis in accordance with Ontario Regulation 347.

The borehole location and elevation were surveyed by Callon Dietz, Ontario Land Surveyors. The borehole elevation is referenced to geodetic datum, and the northing and easting co-ordinates are referenced to the MTM NAD83 survey system. The borehole location, ground surface elevation, and northing and easting co-ordinates are shown on the attached Drawing 1.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

This 30 km section of Highway 400 traverses, from south to north, the following physiographic regions as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, Third Edition, 1984): the Simcoe Lowlands; the Peterborough Drumlin Field; a second lobe of the Simcoe Lowlands; and the Simcoe Uplands. Along Highway 400, the Simcoe Lowlands are present from the southern limit of the project to just south of Innisfil Creek, and again from Essa Road (Simcoe Road 30, formerly Highway 27) to about 1 km north of Dunlop Street (Simcoe Road 90, formerly Highway 90). The Peterborough Drumlin Field occupies the belt between these lobes of the Simcoe Lowlands, extending from just south of Innisfil Creek, which is located about 1 km north of Highway 89, to Essa Road. The Simcoe Uplands extend from about 1 km north of Dunlop Street to beyond the northern limit of the project at Highway 11.

The two sections where Highway 400 crosses the Simcoe Lowlands consist of two lobes of a sand plain which include the shores of Kempenfelt Bay, the Nottawasaga River and Innisfil Creek. The surficial soils of these sections of the Simcoe Lowlands consist primarily of sand, although silt, clay or peat may be found in low-lying areas. The Barrie / Collingwood Railway site within Barrie is located in this physiographic region.

The surficial soils in the Peterborough Drumlin Field consist primarily of gravelly sand till or sand and gravel deposits. Drumlins (glacially-shaped hills) are more frequent in the southern portion of the section of the Peterborough Drumlin Field traversed by Highway 400. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins.

The surficial soils in the Simcoe Uplands physiographic region are primarily sandy silt till deposits, known to contain boulders. Low-lying areas may be infilled with shallow sand and gravel deposits, which are shoreline deposits of a former glacial lake that once flooded the area.

4.2 Site Stratigraphy

The subsurface soil and groundwater conditions encountered in the borehole and the results of in-situ and laboratory testing are given on the Record of Borehole sheets and Figures 1 and 2. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

Borehole B10-1 was advanced in the southwest quadrant of the Tiffin Street structure site, from slightly above the Tiffin Street grade; the borehole is located between 170 m and 190 m from the Barrie / Collingwood Railway site. The location and elevation for this boring is shown on the attached Drawing 1. In summary, the soils encountered in Borehole B10-1 consist of topsoil and fill overlying deposits of silty sand and silt, underlain by a deposit of clayey silt to silty clay, in turn underlain by a lower silty sand deposit. A more detailed description of the subsurface conditions encountered in the borehole is provided in the following sections.

It is noted that no odour, staining or other evidence of contamination was observed in the recovered samples, or in the auger cuttings produced during drilling of Borehole B10-1. A sample of the stockpiled cuttings was submitted for Regulation 347 leachate testing in order to determine the disposal requirements for this soil. The analyzed material met the Schedule 4 concentrations set out in the regulation; the stockpiled soil was therefore classified as non-registerable, non-hazardous waste for disposal purposes.

4.2.1 Topsoil and Fill

Borehole B10-1 encountered about 100 mm of topsoil overlying 1.4 m of silty sand fill, extending to Elevation 233.5 m. The silty sand fill contains trace to some gravel, and trace organics. The measured Standard Penetration Test (SPT) 'N' values were 13 and 30 blows per 0.3 m of penetration, indicating that the fill has a compact relative density.

4.2.2 Silty Sand

A 3 m thick deposit of silty sand was encountered below the fill, with its surface at Elevation 233.5 m. A grain size distribution test result obtained for a representative sample of this material is shown on Figure 1. The recovered samples were moist, with measured natural water contents of 8 and 12 per cent. The measured SPT 'N' values ranged from 38 to 53 blows per 0.3 m of penetration, indicating that this silty sand has a dense to very dense relative density.

4.2.3 Silt

Underlying the silty sand deposit, Borehole B10-1 encountered a 7 m thick deposit of silt. The surface of the silt was at about Elevation 230.5 m.

The silt deposit contains trace to some sand and trace clay. Grain size distribution test results obtained for two samples of this material are shown on Figure 2. The recovered silt samples were wet and exhibited dilatant behaviour. The measured natural water contents ranged from 18 to 23 per cent.

The measured SPT 'N' values ranged from 48 to 20 blows per 0.3 m of penetration, decreasing with depth. It is noted that the lower samples may have been disturbed by groundwater inflow to the borehole during sampling. The silt deposit is therefore considered to have a predominantly dense relative density.

4.2.4 Clayey Silt to Silty Clay

The upper silty sand and silt deposits overlie a deposit ranging in composition from clayey silt to silty clay. The surface of the clayey silt to silty clay was encountered at about Elevation 223.5 in Borehole B10-1. The deposit is about 8 m thick at the borehole location, extending to about Elevation 215.5 m.

The clayey silt to silty clay contains trace quantities of sand. Silt interlayers were noted within the lower half of the deposit, below about 15 m depth (approximately Elevation 220 m). The recovered samples were moist, with measured natural water contents ranging from 20 to 29 per cent. Atterberg Limits testing on one representative sample of clayey silt measured plastic and liquid limits of 14 and 20 per cent, respectively, and a plasticity index of 6 per cent; the clayey silt is therefore inorganic and of low plasticity. It is noted that the measured natural water content on this particular sample was slightly above the liquid limit of the clayey silt, which is not consistent with an SPT 'N' value of 27 blows per 0.3 m of penetration; it is possible that the portion of the sample tested for moisture content contained more silty clay than the portion of the sample used for Atterberg Limits testing.

The clayey silt to silty clay has a very stiff to hard consistency, with measured SPT 'N' values ranging from 15 to 40 blows per 0.3 m of penetration.

4.2.5 Lower Silty Sand

A lower deposit of silty sand was encountered below the clayey silt to silty clay deposit in Borehole B10-1. The surface of this lower silty sand is at about Elevation 215.5 m. The deposit was not fully penetrated by the boring, which was terminated at Elevation 211.7 m; it is at least 4 m thick.

The recovered samples of the lower silty sand deposit were wet, with one measured natural water content of 17 per cent. The measured SPT 'N' values were greater than 100 blows per 0.3 m of penetration, indicating that this material has a very dense relative density.

4.3 Groundwater Conditions

The water level in Borehole B10-1 was at about 5 m depth (Elevation 230 m) during and on completion of the drilling operations. The groundwater level in the piezometer sealed within the upper silt layer was measured at 4.9 m depth in January 2001, and at 5 m depth in March 2001 (Elevations 230.1 m and 230 m, respectively). These measured water levels are near the surface of the silt layer. The groundwater level associated with the lower silty sand deposit was not determined during this preliminary investigation.

It should be noted that groundwater levels are expected to fluctuate seasonally and are expected to rise during wet periods of the year. With a relatively impervious clay deposit underlying the upper silty sand and silt deposits, it is possible for an appreciable rise in the groundwater level to occur during periods of heavy precipitation.

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
BARRIE / COLLINGWOOD RAILWAY OVERPASS
STRUCTURE SITE 30-177
HIGHWAY 400 WIDENING FROM 1 KM SOUTH
OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides preliminary foundation design recommendations for the widening and / or replacement of the existing Barrie / Collingwood Railway (former CN Rail) overpass structure in Barrie, Ontario, associated with the widening of Highway 400. The recommendations are preliminary only and are based on interpretation of the factual data obtained from a single borehole advanced during subsurface investigation in the vicinity of this site. The interpretation and recommendations provided are intended for planning purposes only, to provide the information necessary at this stage of the study. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the planning of the project. Further foundation investigation will be required at this structure site as part of the detailed design stage of the project.

It is understood that Highway 400 will be widened from its existing six-lane configuration to an interim configuration of eight-lanes, and an ultimate configuration of ten-lanes, and that an alternative for a twelve-lane express / collector system is under consideration between Molson Park Drive and Duckworth Street in Barrie. Throughout the project length, it is expected that the existing highway platform will be widened by between 13 m and 30 m. Widening and / or replacement of the existing Barrie / Collingwood Railway overpass structure will therefore be necessary.

Based on the available general layout drawing for the existing single-span structure, the abutments are supported on spread footings which step upward from about Elevation 233.5 m to 233.9 m between the west and east ends, respectively. The associated wing walls and retaining walls are also supported on spread footings founded to match the abutment footings. The Barrie / Collingwood Railway grade is at about Elevation 235 m, near the original ground surface. Highway 400 has been constructed on embankment fill up to about 8 m high; the highway grade is at about Elevation 243 m at the structure site.

5.2 Bridge Foundation Options

Based on the subsurface information obtained from Borehole B10-1, advanced at the Tiffin Street site less than 200 m to the north, the native soils consist of silty sand and silt overlying a clayey silt to silty clay deposit, in turn underlain by a lower deposit of silty sand. The groundwater level associated with the upper, unconfined aquifer at the site was at about Elevation 230 m in March 2001, approximately 5 m below the Barrie / Collingwood Railway grade. The presence of the relatively impervious clayey silty to silty clay deposit at depth creates a perched aquifer

condition within the upper silty sand and silt deposits. It is understood from the City of Barrie that TCE contamination is present within the upper soils (i.e. above the clayey silt to silty clay deposit) in the vicinity of this structure site.

Based on these subsurface conditions and on the foundation conditions for the existing overpass structure, it is recommended that the widened portions of the structure be founded on spread footings to match the existing conditions. If a new or structurally separate overpass is to be constructed, consideration could also be given to the use of perched abutments, founded on spread footings placed on a well-compacted granular pad within the approach fill embankments. Preliminary recommendations for spread footings, including perched abutments, are provided in the following sections. However, it is noted that further borehole investigation will be required within the foundation footprints during detailed design in order to confirm the soil types and relative densities / consistencies. Future boreholes will have to be advanced using accepted environmental drilling, sampling and abandonment procedures.

Deep foundations, such as driven steel H-piles, are not recommended at this site due to the reported presence of TCE contamination in the upper aquifer. Trichloroethylene is a dense, non-aqueous phase liquid; it does not float atop the groundwater table, but instead migrates downward through an aquifer until it reaches a confining layer, such as the clayey silt to silty clay deposit. If driven steel H-piles were adopted at this site, they would have to be driven through the upper aquifer and the clayey silt to silty clay deposit, into the very dense lower silty sand deposit. The introduction of piles through the clayey silt to silty clay confining layer could create pathways for downward migration of the TCE, thereby introducing the contaminants to the lower aquifer at the site.

5.3 Spread Footings

For preliminary design of the abutment foundations, spread footings may be placed on the dense to very dense silty sand at or below Elevation 233.5 m; the footings should be maintained as high as possible to minimize the requirements for groundwater control during excavation and construction. For widening, the founding level of the existing overpass footings (about Elevation 233.5 m to 233.9 m) should be matched to prevent possible undermining of the existing foundations, provided that a minimum soil cover of 1.5 m is maintained at all footing locations.

Alternatively, consideration could be given to the use of abutment footings perched within the approach embankments.

5.3.1 Axial Geotechnical Resistance

Spread footings placed on the properly prepared, undisturbed silty sand deposit at or below the design elevation given above, but above the groundwater level, may be designed for a factored geotechnical resistance at Ultimate Limit States (ULS) of 700 kPa, assuming a 3 m wide footing. The settlement of footings founded on the dense to very dense silty sand soils will be dependent on the footing size and configuration, and on the applied loads. The majority of the settlement should occur during construction; however, the settlement will be differential with respect to the existing overpass structure. For preliminary design purposes, the geotechnical resistance at Serviceability Limit States (SLS) may be taken as 450 kPa. The geotechnical resistances at ULS and SLS will have to be reviewed during detailed design, once the footing size, configuration and loadings are known, and once further drilling has been carried out within the foundation footprints during the detailed design stage.

For spread footings placed within the approach embankments on a compacted Granular 'A' core, a factored geotechnical resistance at ULS of 900 kPa may be assumed for preliminary design. The geotechnical resistance at SLS will depend on the thickness of Granular 'A' and the consistency and thickness of the underlying soils; a value of 350 kPa may be assumed for preliminary design. These resistances will have to be confirmed during detailed design, once the composition and consistency of the upper soils at the site are confirmed.

The geotechnical resistances provided herein are given under the assumption that the loads will be applied perpendicular to the surface of the footings; where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the Ontario Highway Bridge Design Code (OHBDC).

5.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footing and the subsoils should be calculated in accordance with Section 6-8.4.3 of the OHBDC. For preliminary design purposes, and subject to confirmation following the detailed design stage of drilling at this site, the angle of friction between the concrete and the undisturbed, dense to very dense silty sand founding soils should be taken as 24 degrees; the corresponding coefficient of friction, tan δ, would then be 0.45. Where "perched" abutment footings are adopted, the angle of friction between the concrete footings and the compacted Granular 'A' pad should be taken as 30 degrees; the corresponding coefficient of friction would be 0.58.

5.3.3 Frost Protection

All footings should be provided with a minimum of 1.5 m of soil cover for frost protection.

5.4 Lateral Earth Pressures

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

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the abutments and walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00

- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the abutment wall, in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be used in accordance with OPSS 501.06.

- The granular fill may be placed either in a zone with width equal to at least 1.5 m behind the back of the stem (Case I from OHBDC Figure 6-7.4.1) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II from OHBDC Figure 6-7.4.4).

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

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

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

	Granular 'A'	Granular 'B' Type II
Soil unit weight:	22 kN/m³	21 kN/m³
Coefficients of lateral earth pressure:		
Active, K_a	0.27	0.31
At rest, K_o	0.43	0.47

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

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

5.5 Embankment Design

Based on the topographic information on the Engineering and Title Records plates and on site reconnaissance, the existing Highway 400 embankment side slopes are formed at a gradient of about 2 horizontal to 1 vertical (2H:1V) at the Barrie / Collingwood Railway site. For widening of the Highway 400 embankments, the new side slopes should be formed at a maximum gradient of 2H:1V. The embankment widening should be carried out using conventional fill placement and compaction practices, and benching of the existing embankment side slopes should be carried out to key in the new fill.

5.6 Design and Construction Considerations

5.6.1 Groundwater Control

The groundwater level during the investigation period was found to be at about 5 m depth (Elevation 230 m), near the interface of the silty sand and silt deposits. The presence of the relatively impervious clayey silt to silty clay deposit could, during periods of heavy precipitation, create a marked rise in the groundwater level within the upper sand deposit, on which the spread footings are to be founded. It will, therefore, be important to have provision of appropriate groundwater and surface water control measures during construction to maintain the in-situ integrity of the silty sand and silt deposits.

5.6.2 Excavation

Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. The footing excavations will extend a minimum of 1.5 m below lowest surrounding grade, through existing fill and into the dense to very dense silty sand. The fill and silty sand would be classified as Type 2 to 3 soils. Temporary open-cut slopes should therefore be maintained no steeper than 1 horizontal to 1 vertical (1H:1V). Where space restrictions dictate, adjacent to the new permanent cut slope, footing excavations could also be carried out within a braced excavation.

5.6.3 Settlement

Deformation of the ground due to foundation loading will result in settlement of the bridge abutments and superstructure. Settlement of the new, widened portions of the structure will be differential with respect to the existing overpass structure, and also could vary between the new foundation elements depending on the variability and consistency / relative density of the founding soils. The potential for differential settlement will have to be reassessed during the detailed design stage once the proposed bridge configuration is established.

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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

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 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 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	c_u, s_u 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 (LV-laboratory vane test)
γ	unit weight

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

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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 δ
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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No B10-1 1 OF 2 METRIC

PROJECT 001-1143F
W.P. 30-95-00
DIST SW HWY 400
DATUM Geodetic
LOCATION N 4914534.5; E 288241.1
BOREHOLE TYPE 108mm ID HOLLOW STEM AUGERS
DATE Oct.24-25/2000
ORIGINATED BY PKS
COMPILED BY LCC
CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
235.0	GROUND SURFACE					
0.0 / 0.1	Topsoil					
	Silty Sand, trace to some gravel, trace organics (Fill) Compact Brown Moist	1	SS	13		
		2	SS	30	234	
233.5 / 1.5	Silty Sand Dense to very dense Brown Moist	3	SS	38	233	
		4	SS	42	232	
		5	SS	51		0 76 24 0
		6	SS	53	231	
230.4 / 4.6	Silt, trace to some sand, trace clay Dense to compact Dilatant Brown to grey below 6m depth Wet	7	SS	48	230	
		8	SS	37	229	0 1 95 4
					228	
	SPT "N" values may be impacted by groundwater inflow to borehole during sampling.	9	SS	23	227	
		10	SS	25	226	0 15 84 1
					225	
		11	SS	20	224	
223.4 / 11.6	Clayey Silt to Silty Clay, trace sand Very stiff to hard Grey Moist				223	
		12	SS	27	222	
		13	SS	15	221	

Continued Next Page

+3, ×3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

ON_MOT 0011143F.GPJ ON_MOT.GDT 14/1/02

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No B10-1 2 OF 2 METRIC

PROJECT 001-1143F
W.P. 30-95-00
DIST SW HWY 400
DATUM Geodetic
LOCATION N 4914534.5; E 288241.1
BOREHOLE TYPE 108mm ID HOLLOW STEM AUGERS
DATE Oct.24-25/2000
ORIGINATED BY PKS
COMPILED BY LCC
CHECKED BY ASP

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
	Clayey Silt to Silty Clay, trace sand Very stiff to hard Grey Moist Contains silt interlayers below 15.2m depth	14	SS	18	219
		15	SS	40	218
					217
		16	SS	35	216
215.5 / 19.5	Silty Sand Very dense Grey Wet	17	SS	132	215
					214
		18	SS	102	213
		19	SS	100	212
211.7 / 23.3	END OF BOREHOLE				

Notes:
1. Water level in open borheole on compeltion of drilling at about 5m depth (Elev.230.0m).
2. Water level in piezometer at 4.9m depth (Elev.230.1m) on January 19, 2001, and at 5m depth (Elev.230.0m) on March 15, 2001.

+3, ×3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

ON_MOT 0011143F.GPJ ON_MOT.GDT 14/1/02

DIST	HWY 400
CONT. No.	
GWP No.	30-95-00

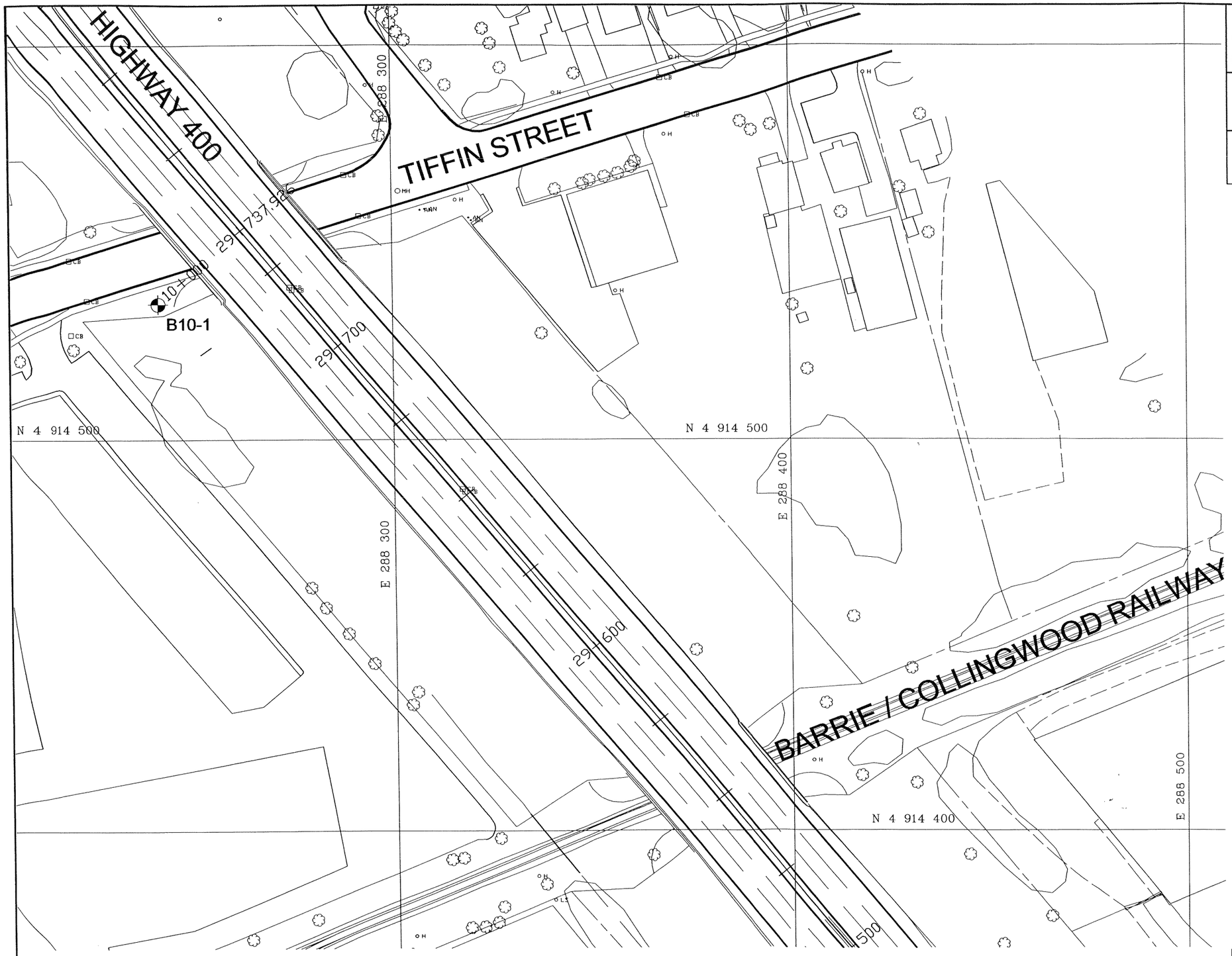
SHEET

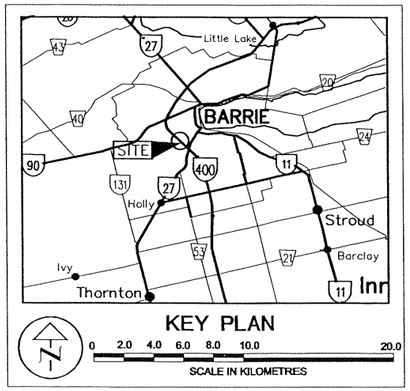
BARRIE/COLLINGWOOD RAILWAY OVERPASS, HWY 400

BOREHOLE LOCATION PLAN

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA





LEGEND

- Borehole, previous investigation
- Borehole, present investigation

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
B10-1	235.0	4,914,534.5	288,241.1

REFERENCE

This drawing was created from digital file "50208.dwg" provided by URS Cole Sherman

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

NO.	DATE	BY	REVISION

Geocres No.

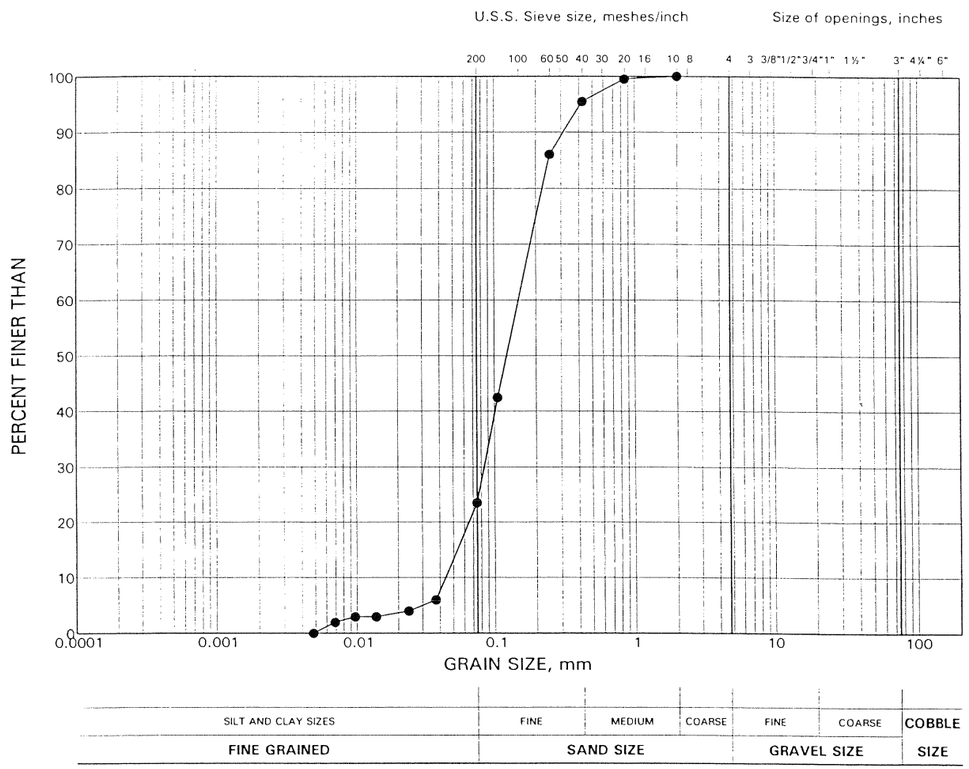
HWY. No. 400		PROJECT NO.: 001-1143F	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-177
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

P1143F14.DWG

plot scale 1:20mp.

GRAIN SIZE DISTRIBUTION TEST RESULT
Silty Sand

FIGURE 1



LEGEND

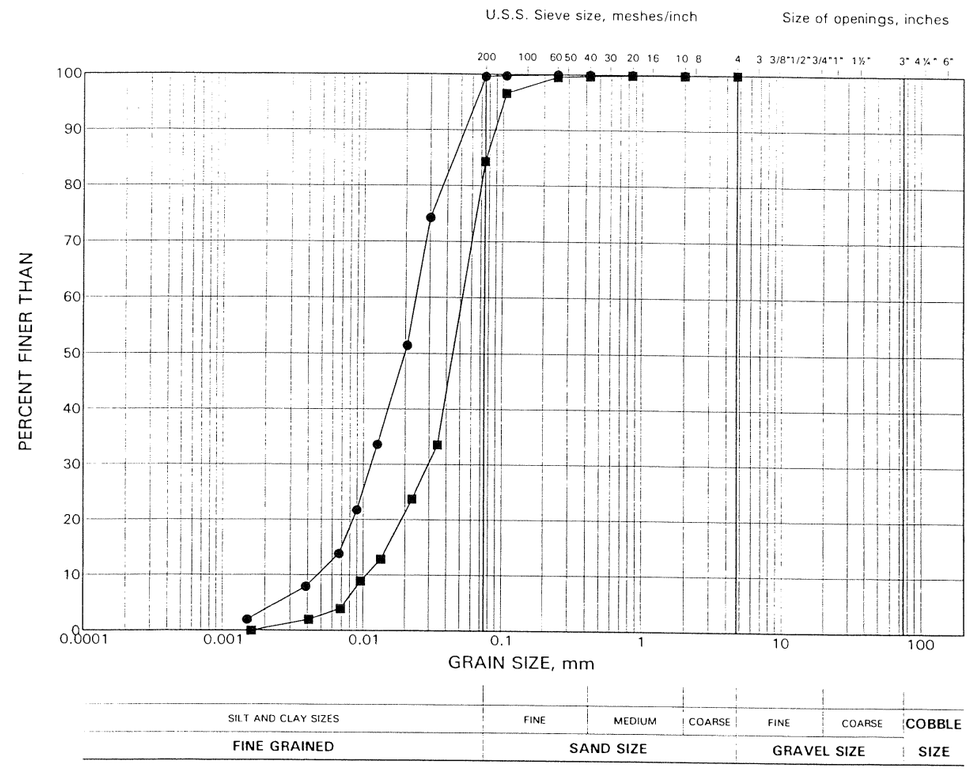
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	B10-1	5	231.6

Project 001-1143F

Golder Associates

GRAIN SIZE DISTRIBUTION TEST RESULTS
Silt

FIGURE 2



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	B10-1	8	228.6
■	B10-1	10	225.5

Project 001-1143F

Golder Associates

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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT TIFFIN STREET OVERPASS STRUCTURE SITE 30-176 HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11 G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

Submitted to:

URS Cole, Sherman
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Thornhill, Ontario
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January 2002 001-1143F-10

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PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
TIFFIN STREET OVERPASS
STRUCTURE SITE 30-176
HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

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Figure 1 Grain Size Distribution Test Result – Silty Sand
Figure 2 Grain Size Distribution Test Results – Silt

1.0 INTRODUCTION

Golder Associates Ltd. has been retained by URS Cole, Sherman (Cole, Sherman) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the ultimate widening of Highway 400 from 1 km south of Highway 89, northerly 30 km to Highway 11, in Simcoe County, Ontario. Foundation engineering services are required for the widening and / or replacement of eighteen existing overpass and underpass structures, as well as five structural culverts.

This report addresses the widening and / or replacement of the Tiffin Street overpass structure in Barrie, Ontario. A site investigation has been carried out, in which one borehole was advanced and in-situ and laboratory testing was conducted, to determine the subsurface conditions at the site for this preliminary design study.

The terms of reference for the scope of work are outlined in Golder Associates' Proposal No. P01-1192, dated June 2000. Under the original scope of work, two boreholes were to be advanced at this site for preliminary investigation purposes. However, during drilling of Borehole B10-1 at the Tiffin Street site, the City of Barrie informed Golder Associates of the presence of trichloroethylene (TCE) contamination in the area encompassing the Barrie / Collingwood Railway (former CN Rail), Tiffin Street and Dunlop Street sites. Based on a review of the then-available subsurface information for the Essa Road and Anne Street sites to the south and north of this area, respectively, and following discussions with the MTO Foundations Section, the scope of work was revised as follows:

- Borehole B10-1, which had already been advanced at the Tiffin Street site, would be used to provide preliminary subsurface information for both the Tiffin Street site and the Barrie / Collingwood Railway (former CN Rail) site, less than 200 m to the south. No further drilling would be carried out at these two sites during the preliminary foundation investigation stage.

- One borehole would be drilled at the Dunlop Street (Simcoe Road 90, formerly Highway 90) site, located about 1 km north of the Tiffin Street site. This borehole would be advanced using environmental drilling procedures and telescoping of casing to minimize the potential for drilling-induced downward movement of any contaminants at the site.

The above revised scope of work was approved by the MTO Foundations Section.

2.0 SITE DESCRIPTION

The existing Tiffin Street overpass is located about 200 m north of the Barrie / Collingwood Railway (former CN Rail) crossing and 1 km south of the Dunlop Street (Simcoe Road 90, formerly Highway 90) interchange, in Barrie, Ontario. The MTO has designated this overpass as Structure Site 30-176.

At the site, the Tiffin Street grade is at about Elevation 234 m to 234.5 m, near the original ground surface. Highway 400 has been constructed on embankment fill up to about 6.5 m high; the highway grade is at about Elevation 240.5 m to 240 m, declining northward.

The existing single-span overpass was constructed in the early 1950s under Contract 50-10. According to the general layout drawing for the existing structure, which was provided by Morrison Hershfield (the structural designers for this preliminary study), the abutments and associated wing walls and retaining walls are supported on spread footings founded at about Elevation 233.2 m.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at this site in October 2000, at which time one borehole was drilled. Borehole B10-1 was located on the west side of Highway 400, south of Tiffin Street. This borehole was advanced from slightly above Tiffin Street grade to about 23 m depth (approximately Elevation 212 m).

The investigation was carried out using a bombardier-mounted CME-55 drill rig supplied and operated by Master Soil Investigations Ltd. of Weston, Ontario. The borehole was advanced using hollow stem augers. Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. The water level in the open borehole was observed throughout the drilling operations, and a piezometer was installed to permit monitoring of the groundwater level at the site.

The field work was supervised on a full-time basis by a member of our staff who located the borehole in the field, directed the drilling, sampling, and in-situ testing operations, and logged the borehole. The soil samples were identified in the field, placed in labelled containers and transported to Golder Associates' laboratory in Mississauga for further examination. Index and classification tests consisting of water content determinations, Atterberg Limits testing and grain size distribution analyses were carried out on selected soil samples.

In addition, a sample of the stockpiled cuttings was obtained for chemical analysis, in order to determine the waste classification for disposal of this soil. This sample was submitted to Philip Analytical Services for leachate analysis in accordance with Ontario Regulation 347.

The borehole location and elevation were surveyed by Callon Dietz, Ontario Land Surveyors. The borehole elevation is referenced to geodetic datum, and the northing and easting co-ordinates are referenced to the MTM NAD83 survey system. The borehole location, ground surface elevation, and northing and easting co-ordinates are shown on the attached Drawing 1.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

This 30 km section of Highway 400 traverses, from south to north, the following physiographic regions as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, Third Edition, 1984): the Simcoe Lowlands; the Peterborough Drumlin Field; a second lobe of the Simcoe Lowlands; and the Simcoe Uplands. Along Highway 400, the Simcoe Lowlands are present from the southern limit of the project to just south of Innisfil Creek, and again from Essa Road (Simcoe Road 30, formerly Highway 27) to about 1 km north of Dunlop Street (Simcoe Road 90, formerly Highway 90). The Peterborough Drumlin Field occupies the belt between these lobes of the Simcoe Lowlands, extending from just south of Innisfil Creek, which is located about 1 km north of Highway 89, to Essa Road. The Simcoe Uplands extend from about 1 km north of Dunlop Street to beyond the northern limit of the project at Highway 11.

The two sections where Highway 400 crosses the Simcoe Lowlands consist of two lobes of a sand plain which include the shores of Kempenfelt Bay, the Nottawasaga River and Innisfil Creek. The surficial soils of these sections of the Simcoe Lowlands consist primarily of sand, although silt, clay or peat may be found in low-lying areas. The Tiffin Street site is located within this physiographic region.

The surficial soils in the Peterborough Drumlin Field consist primarily of gravelly sand till or sand and gravel deposits. Drumlins (glacially-shaped hills) are more frequent in the southern portion of the section of the Peterborough Drumlin Field traversed by Highway 400. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins.

The surficial soils in the Simcoe Uplands physiographic region are primarily sandy silt till deposits, known to contain boulders. Low-lying areas may be infilled with shallow sand and gravel deposits, which are shoreline deposits of a former glacial lake that once flooded the area.

4.2 Site Stratigraphy

The subsurface soil and groundwater conditions encountered in the borehole and the results of in-situ and laboratory testing are given on the Record of Borehole sheets and Figures 1 and 2. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

Borehole B10-1 was advanced in the southwest quadrant of the structure site, from slightly above Tiffin Street grade. The location and elevation for this boring is shown on the attached Drawing 1. In summary, the soils encountered below the Tiffin Street grade consist of topsoil and fill overlying deposits of silty sand and silt, underlain by a deposit of clayey silt to silty clay, which in turn is underlain by a lower silty sand deposit. A more detailed description of the subsurface conditions encountered in the borehole is provided in the following sections.

It is noted that no odour, staining or other evidence of contamination was observed in the recovered samples, or in the auger cuttings produced during drilling of Borehole B10-1. A sample of the stockpiled cuttings was submitted for Regulation 347 leachate testing in order to determine the disposal requirements for this soil. The analyzed material met the Schedule 4 concentrations set out in the regulation; the stockpiled soil was therefore classified as non-registerable, non-hazardous waste for disposal purposes.

4.2.1 Topsoil and Fill

Borehole B10-1 encountered about 100 mm of topsoil overlying 1.4 m of silty sand fill, extending to Elevation 233.5 m. The silty sand fill contains trace to some gravel, and trace organics. The measured Standard Penetration Test (SPT) 'N' values were 13 and 30 blows per 0.3 m of penetration, indicating that the fill has a compact relative density.

4.2.2 Silty Sand

A 3 m thick deposit of silty sand was encountered below the fill, with its surface at Elevation 233.5 m. A grain size distribution test result obtained for a representative sample of this material is shown on Figure 1. The recovered samples were moist, with measured natural water contents of 8 and 12 per cent. The measured SPT 'N' values ranged from 38 to 53 blows per 0.3 m of penetration, indicating that this silty sand has a dense to very dense relative density.

4.2.3 Silt

Underlying the silty sand layer, Borehole B10-1 encountered a 7 m thick deposit of silt. The surface of the silt was at about Elevation 230.5 m.

The silt deposit contains trace to some sand and trace clay. Grain size distribution test results obtained for two samples of this material are shown on Figure 2. The recovered silt samples were wet and exhibited dilatant behaviour. The measured natural water contents ranged from 18 to 23 per cent.

The measured SPT 'N' values ranged from 48 to 20 blows per 0.3 m of penetration, decreasing with depth. It is noted that the lower samples may have been disturbed by groundwater inflow to the borehole during sampling. The silt deposit is therefore considered to have a predominantly dense relative density.

4.2.4 Clayey Silt to Silty Clay

The upper silty sand and silt deposits overlie a deposit ranging in composition from clayey silt to silty clay. The surface of the clayey silt to silty clay was encountered at about Elevation 223.5 m in Borehole B10-1. The deposit is about 8 m thick at the borehole location, extending down to about Elevation 215.5 m.

The clayey silt to silty clay contains trace quantities of sand. Silt interlayers were noted within the lower half of the deposit, below about 15 m depth (approximately Elevation 220 m). The recovered samples were moist, with measured natural water contents ranging from 20 to 29 per cent. Atterberg Limits testing on one representative sample of clayey silt measured plastic and liquid limits of 14 and 20 per cent, respectively, and a plasticity index of 6 per cent; the clayey silt is therefore inorganic and of low plasticity. It is noted that the measured natural water content on this particular sample was slightly above the liquid limit of the clayey silt, which is not consistent with an SPT "N" value of 27 blows per 0.3 m of penetration; it is possible that the portion of the sample tested for moisture content contained more silty clay than the portion of sample used for the Atterberg Limit tests.

The clayey silt to silty clay has a very stiff to hard consistency, with measured SPT 'N' values ranging from 15 to 40 blows per 0.3 m of penetration.

4.2.5 Lower Silty Sand

A lower deposit of silty sand was encountered below the clayey silt to silty clay deposit in Borehole B10-1. The surface of this lower silty sand is at about Elevation 215.5 m. The deposit was not fully penetrated by the boring, which was terminated at Elevation 211.7 m; it is at least 4 m thick.

The recovered samples of the lower silty sand deposit were wet, with one measured natural water content of 17 per cent. The measured SPT 'N' values were greater than 100 blows per 0.3 m of penetration, indicating that this material has a very dense relative density.

4.3 Groundwater Conditions

The water level in Borehole B10-1 was at about 5 m depth (Elevation 230 m) during and on completion of the drilling operations. The groundwater level in the piezometer sealed within the upper silt layer was measured at 4.9 m depth in January 2001, and at 5 m depth in March 2001 (Elevations 230.1 m and 230 m, respectively). These measured water levels are near the surface of the silt layer. The groundwater level associated with the lower silty sand deposit was not determined during this preliminary investigation.

It should be noted that groundwater levels are expected to fluctuate seasonally and are expected to rise during wet periods of the year. With a relatively impervious clay deposit underlying the upper silty sand and silt deposits, it is possible for an appreciable rise in the groundwater level to occur during periods of heavy precipitation.

GOLDER ASSOCIATES LTD.

Lisa C. Coyne, P.Eng.,
Geotechnical Engineer

for Anne S. Poschmann, P.Eng.,
Principal

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

LCC/ASP/FJH/clg/dh
N:\ACTIVE\1100\001-1143F\2002\RPT10-02JAN-TIFFIN.DOC

PART B

PRELIMINARY FOUNDATION DESIGN REPORT
TIFFIN STREET OVERPASS
STRUCTURE SITE 30-176
HIGHWAY 400 WIDENING FROM 1 KM SOUTH
OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

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

It is understood that Highway 400 will be widened from its existing six-lane configuration to an interim configuration of eight lanes, and an ultimate configuration of ten lanes, and that an alternative for a twelve-lane express / collector system is under consideration between Molson Park Drive and Duckworth Street in Barrie. Throughout the project length, it is expected that the existing highway platform will be widened by between 13 m and 30 m. Widening or replacement of the existing Tiffin Street overpass structure will therefore be necessary.

Based on the available general layout drawing for the existing single-span structure, the abutments and associated wing walls and retaining walls are supported on spread footings which are founded at about Elevation 233.2 m. The Tiffin Street grade is at about Elevation 234 m to 234.5 m, near the original ground surface. Highway 400 has been constructed on embankment fill up to about 6.5 m high; the highway grade is at about Elevation 240.5 m to 240 m, declining northward.

5.2 Bridge Foundation Options

The native soils below the Tiffin Street grade consist of silty sand and silt overlying a clayey silt to silty clay deposit, in turn underlain by a lower deposit of silty sand. The groundwater level associated with the upper, unconfined aquifer at the site was found to be at about Elevation 230 m in March 2001, approximately 4 m to 4.5 m below the Tiffin Street grade. The presence of the relatively impervious clayey silt to silty clay deposit at depth provides a perched aquifer condition within the upper silty and silt deposits. It is understood from the City of Barrie that trichloroethylene (TCE) contamination is present within the upper soils (i.e. above the clayey silt to silty clay deposit) in the vicinity of this structure site.

Based on these subsurface conditions and on the foundation conditions for the existing overpass structure, it is recommended that the widened portions of the structure be founded on spread footings to match the existing conditions. If a new overpass is to be constructed, consideration could also be given to the use of perched abutments, founded on spread footings placed on a well-compacted granular pad within the approach fill embankments. Preliminary recommendations for spread footings, including perched abutments, are provided in the following sections.

Deep foundations, such as driven steel H-piles, are not recommended at this site due to the reported presence of TCE contamination in the upper aquifer. Trichloroethylene is a dense, non-aqueous phase liquid; it does not float atop the groundwater table, but instead migrates downward through an aquifer until it reaches a confining layer, such as the clayey silt to silty clay deposit. If driven steel H-piles were adopted at this site, they would have to be driven through the upper aquifer and the clayey silt to silty clay deposit, into the very dense lower silty sand deposit. The introduction of piles through the clayey silt to silty clay confining layer could create pathways for downward migration of the TCE, thereby introducing the contaminants to the lower aquifer at the site.

5.3 Spread Footings

For preliminary design of the abutment foundations, spread footings may be placed on the dense to very dense silty sand at or below Elevation 233.2 m; the footings should be maintained as high as possible to minimize the requirements for groundwater control during excavation and construction. For widening, the founding level of the existing overpass footings (about Elevation 233.2 m) should be matched to prevent possible undermining of the existing foundations, provided that a minimum soil cover of 1.5 m is maintained at all footing locations.

Alternatively, consideration could be given to the use of abutment footings perched within well compacted approach embankments.

5.3.1 Axial Geotechnical Resistance

Spread footings placed on the properly prepared, undisturbed silty sand deposit, at or below the design elevation given above, may be designed for a factored geotechnical resistance at Ultimate Limit States (ULS) of 700 kPa, assuming a 3 m wide footing. The settlement of footings founded on the dense to very dense silty sand soils will be dependent on the footing size and configuration, and on the applied loads. The majority of the settlement should occur during construction; however, the settlement will be differential with respect to the existing overpass

structure. For preliminary design purposes, the geotechnical resistance at Serviceability Limit States (SLS) may be taken as 450 kPa. The geotechnical resistances at ULS and SLS will have to be reviewed during detailed design, once the footing size, configuration and loadings are known.

For spread footings placed within the approach embankments on a compacted Granular 'A' core, a factored geotechnical resistance at ULS of 900 kPa may be assumed for preliminary design. The geotechnical resistance at SLS will depend on the thickness of Granular 'A' and the consistency and thickness of the underlying soils; a value of 350 kPa may be assumed for preliminary design. These resistances will have to be confirmed during detailed design, once the composition and consistency of the upper soils at the site are confirmed.

The geotechnical resistances provided herein are given under the assumption that the loads will be applied perpendicular to the surface of the footings; where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the Ontario Highway Bridge Design Code (OHBDC).

5.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footing and the subsoils should be calculated in accordance with Section 6-8.4.3 of the OHBDC. The angle of friction between the concrete and the undisturbed dense to very dense silty sand founding soils should be taken as 24 degrees; the corresponding coefficient of friction, tan δ, would then be 0.45. Where "perched" abutment footings are adopted, the angle of friction between the concrete footings and the compacted Granular 'A' pad should be taken as 30 degrees; the corresponding coefficient of friction would be 0.58.

5.3.3 Frost Protection

All footings should be provided with a minimum of 1.5 m of soil cover for frost protection.

5.4 Lateral Earth Pressures

The lateral pressures acting on the bridge abutments and associated retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the freedom of lateral movement of the structure, and on the drainage conditions

behind the walls. The following recommendations are made concerning the design of the abutments, in accordance with the OHBDC:

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the abutments and walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00

- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the abutment wall, in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be used in accordance with OPSS 501.06.

- The granular fill may be placed either in a zone with width equal to at least 1.5 m behind the back of the stem (Case I from OHBDC Figure 6-7.4.1) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II from OHBDC Figure 6-7.4.4).

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

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

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

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

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

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

5.5 Embankment Design

Based on the topographic information on the Engineering and Title Records plates and on site reconnaissance, the existing Highway 400 embankment side slopes are formed at a gradient of about 2 horizontal to 1 vertical (2H:1V) at the Tiffin Street site. For widening of the Highway 400 embankments, the new side slopes should be formed at a maximum gradient of 2H:1V. The embankment widening should be carried out using conventional fill placement and compaction practices, and benching of the existing embankment side slopes should be carried out to key in the new fill.

5.6 Design and Construction Considerations

5.6.1 Groundwater Control

The groundwater level during the investigation period was found to be at about 5 m depth (Elevation 230 m), near the interface of the upper silty sand and silt deposits. The presence of the relatively impervious clayey silt to silty clay deposit could, during periods of high precipitation, create a marked rise in the groundwater level within the relatively pervious upper sand deposit, on which the spread footings are to be founded. It is therefore important to have provision of appropriate groundwater and surface water control measures during the construction phase of the work, to preserve the existing in-situ integrity of the sands and silts.

5.6.2 Excavation

Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. The footing excavations will extend a minimum of 1.5 m below lowest surrounding grade, through existing fill and into the dense to very dense silt sand. The fill and silty sand would be classified as Type 2 to 3 soils. Temporary open-cut slopes should therefore be maintained no steeper than 1 horizontal to

1 vertical (1H:1V). Where space restrictions dictate, adjacent to the new permanent cut slope, footing excavations could also be carried out within a braced excavation.

5.6.3 Settlement

Deformation of the ground due to foundation loading will result in settlement of the bridge abutments and superstructure. Settlement of the new, widened portions of the structure will be differential with respect to the existing overpass structure, and also could vary between the new foundation elements depending on the variability and consistency/relative density of the foundation soils. The potential for differential settlement will have to be reassessed during the detailed design stage once the proposed bridge configuration is established.

GOLDER ASSOCIATES LTD.

Lisa C. Coyne, P.Eng.,
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for Anne S. Poschmann, P.Eng.,
Principal

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

LCC/ASP/FJH/clg/dh
N:\ACTIVE\1100\001-1143F\2002\RPT10-02JAN-TIFFIN.DOC

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

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 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 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	c_u, s_u 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 (LV-laboratory vane test)
γ	unit weight

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

S:\FINALDAT\ABBREV\2000\LOFA-D00.DOC

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 δ
c' effective cohesion
c_u, s_u undrained shear strength (ϕ = 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

RECORD OF BOREHOLE No B10-1

1 OF 2

METRIC

PROJECT 001-1143F
W.P. 30-95-00
DIST SW HWY 400
DATUM Geodetic
LOCATION N 4914534.5; E 288241.1
BOREHOLE TYPE 108mm ID HOLLOW STEM AUGERS
DATE Oct.24-25/2000
ORIGINATED BY PKS
COMPILED BY LCC
CHECKED BY ASP

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
235.0	GROUND SURFACE										
0.0	Topsoil										
0.1	Silty Sand, trace to some gravel, trace organics (Fill) Compact Brown Moist		1	SS	13						
			2	SS	30		234				
233.5											
1.5	Silty Sand Dense to very dense Brown Moist		3	SS	38		233				
			4	SS	42		232				
			5	SS	51			0	76	24	0
			6	SS	53		231				
230.4											
4.6	Silt, trace to some sand, trace clay Dense to compact Dilatant Brown to grey below 6m depth Wet		7	SS	48	▽	230				
			8	SS	37		229				
							228				
	SPT "N" values may be impacted by groundwater inflow to borehole during sampling.		9	SS	23		227				
							226				
			10	SS	25			0	1	95	4
							225				
			11	SS	20		224				
223.4											
11.6	Clayey Silt to Silty Clay, trace sand Very stiff to hard Grey Moist						223				
			12	SS	27		222				
			13	SS	15		221				

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa
O UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × REMOULDED
20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L
WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

Continued Next Page

+³,×³: Numbers refer to Sensitivity

O³% STRAIN AT FAILURE

ON_MOT_0011143F.GPJ ON_MOT.GDT 14/1/02

RECORD OF BOREHOLE No B10-1

2 OF 2

METRIC

PROJECT 001-1143F
W.P. 30-95-00
DIST SW HWY 400
DATUM Geodetic

LOCATION N 4914534.5; E 288241.1
BOREHOLE TYPE 108mm ID HOLLOW STEM AUGERS
DATE Oct.24-25/2000

ORIGINATED BY PKS
COMPILED BY LCC
CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Clayey Silt to Silty Clay, trace sand Very stiff to hard Grey Moist Contains silt interlayers below 15.2m depth		14	SS	18		219		○		
			15	SS	40		218				
							217				
			16	SS	35		216		○		
215.5 / 19.5	Silty Sand Very dense Grey Wet		17	SS	132		215				
							214				
			18	SS	102		213		○		
211.7 / 23.3			19	SS	100		212				
	END OF BOREHOLE Notes: 1. Water level in open borheole on compeltion of drilling at about 5m depth (Elev.230.0m). 2. Water level in piezometer at 4.9m depth (Elev.230.1m) on January 19, 2001, and at 5m depth (Elev.230.0m) on March 15, 2001.										

+³,×³: Numbers refer to Sensitivity O³% STRAIN AT FAILURE

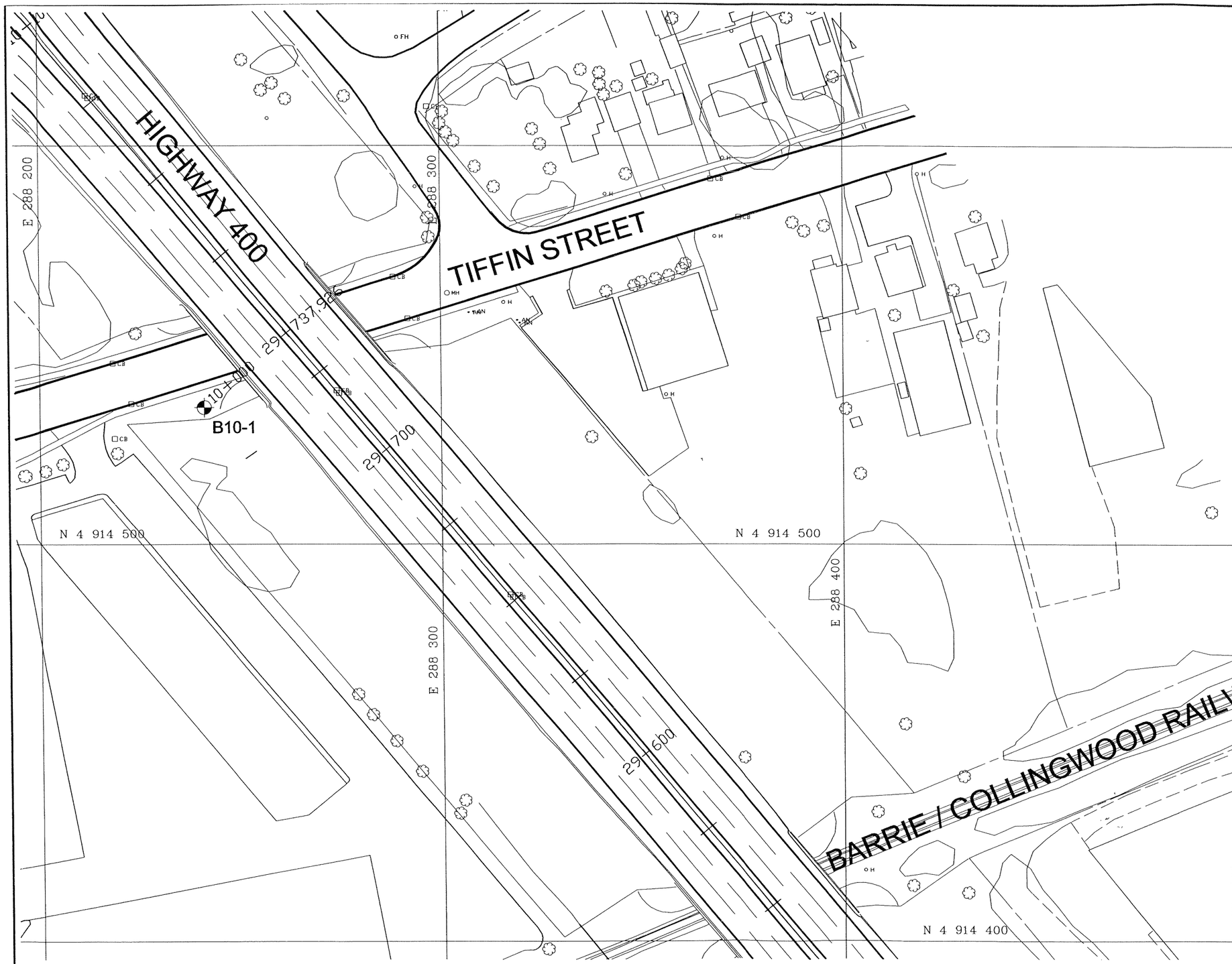
DIST	HWY 400
CONT. No.	
GWP No.	30-95-00

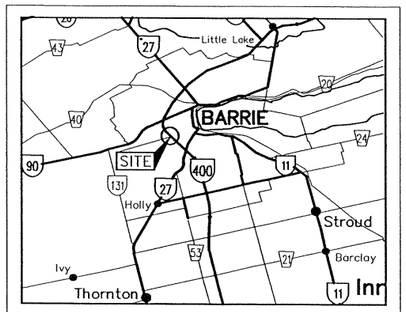
SHEET

TIFFIN STREET OVERPASS
HWY 400
BOREHOLE LOCATION PLAN

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA





LEGEND

- Borehole, previous investigation
- Borehole, present investigation

No.	ELEVATION	LOCATION NORTHING	LOCATION EASTING
B10-1	235.0	4,914,534.5	288,241.1

REFERENCE
This drawing was created from digital file "50208.dwg" provided by URS Cole Shermon

0 10 20 30 40 50 100m
SCALE IN METRES

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

NO.	DATE	BY	REVISION

Geocres No.

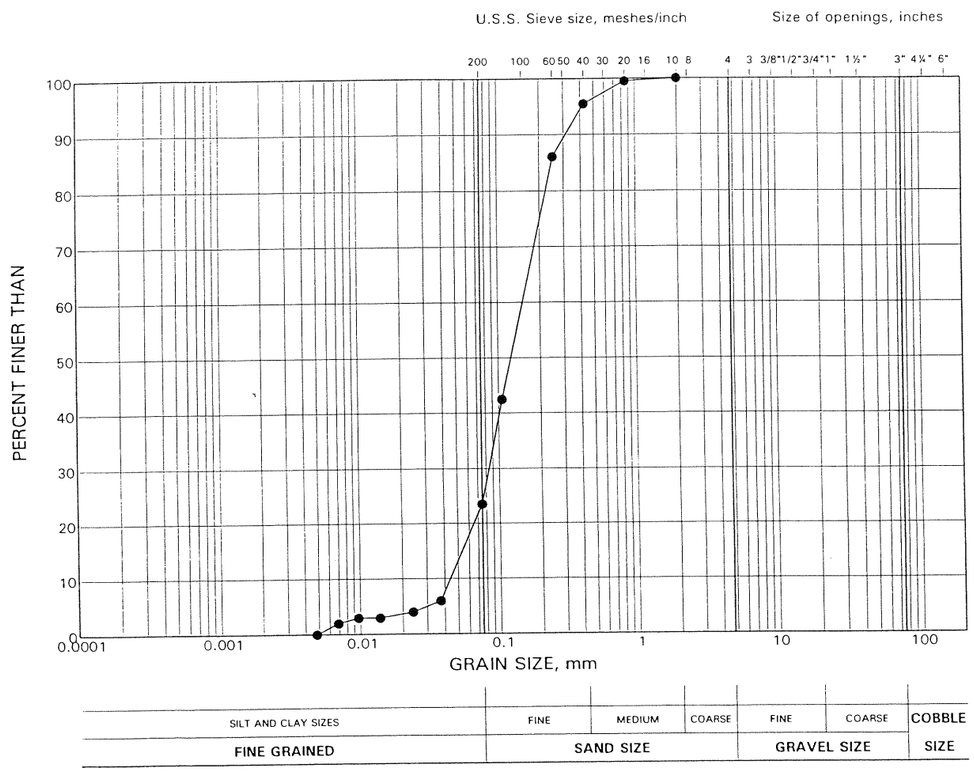
HWY. No. 400		PROJECT NO.: 001-1143F	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-176
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

P:1143F15.DWG

plot scale 1:2/mp.

GRAIN SIZE DISTRIBUTION TEST RESULT
Silty Sand

FIGURE 1



LEGEND

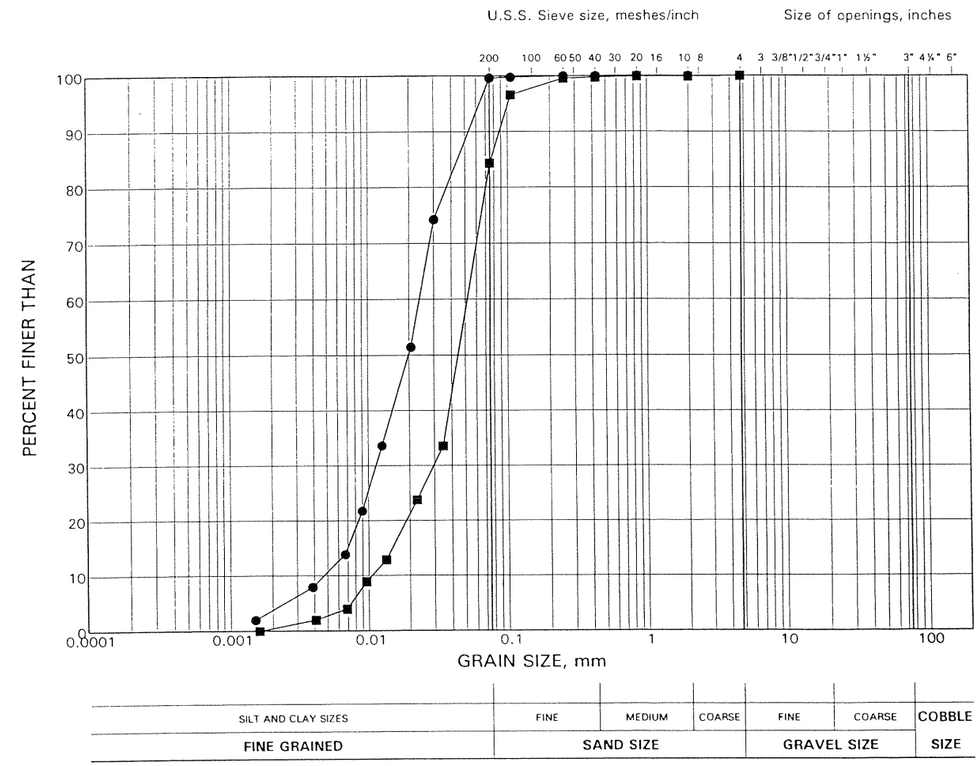
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	B10-1	5	231.6

Project 001-1143F

Golder Associates

GRAIN SIZE DISTRIBUTION TEST RESULTS
Silt

FIGURE 2



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	B10-1	8	228.6
■	B10-1	10	225.5

Project 001-1143F

Golder Associates

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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT DUNLOP STREET UNDERPASS STRUCTURE SITE 30-175 HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11 G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

Submitted to:

URS Cole, Sherman
75 Commerce Valley Drive East
Thornhill, Ontario
L3T 7N9

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January 2002 001-1143F-11

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PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
DUNLOP STREET UNDERPASS
STRUCTURE SITE 30-175
HIGHWAY 400 WIDENING FROM 1 KM SOUTH
OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

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LIST OF DRAWINGS
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LIST OF FIGURES
Figure 1 Grain Size Distribution Test Result – Clayey Silt to Silty Clay
Figure 2 Grain Size Distribution Test Results – Sandy Silt with Clayey Silt Interlayers

1.0 INTRODUCTION

Golder Associates Ltd. has been retained by URS Cole, Sherman (Cole, Sherman) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the ultimate widening of Highway 400 from 1 km south of Highway 89, northerly 30 km to Highway 11, in Simcoe County, Ontario. Foundation engineering services are required for the widening and / or replacement of eighteen existing overpass and underpass structures, as well as five structural culverts.

This report addresses the replacement of the existing Dunlop Street underpass structure in Barrie, Ontario. A foundation investigation has been carried out, in which one borehole was advanced and in-situ and laboratory testing was conducted, to determine the subsurface conditions at the site for this preliminary design study.

The terms of reference for the scope of work are outlined in Golder Associates' Proposal No. P01-1192, dated June 2000. Under the original scope of work, two boreholes were to be advanced at this site for preliminary investigation purposes. However, during drilling of Borehole B10-1 at the Tiffin Street site, the City of Barrie informed Golder Associates of the presence of trichloroethylene (TCE) contamination in the area encompassing the Barrie/Collingwood Railway (former CN Rail), Tiffin Street and Dunlop Street sites. Based on review of the then-available subsurface information for the Essa Road and Anne Street sites to the south and north of this area, respectively, and following discussions with the MTO Foundations Group, the scope of work was revised as follows:

- Borehole B10-1, which had already been advanced at the Tiffin Street site, would be used to provide preliminary subsurface information for both the Tiffin Street site and the Barrie/Collingwood Railway (former CN Rail) site, less than 200 m to the south. No further drilling would be carried out at these two sites during the preliminary foundation investigation stage.

- One borehole would be drilled at the Dunlop Street (Simcoe Road 90, formerly Highway 90) site. This borehole was to be advanced by telescoping casing, to minimize the potential for aquifer cross-contamination.

The above revised scope of work was approved by the MTO Foundations Section.

2.0 SITE DESCRIPTION

The existing Dunlop Street (Simcoe Road 90, formerly Highway 90) underpass is located about 2.5 km north of the Essa Road (Simcoe Road 30, formerly Highway 27) interchange, and about 2.5 km south of the Bayfield Street (Highway 26) interchange, in Barrie, Ontario. The MTO has designated this underpass as Structure Site 30-175.

At the site, Highway 400 has been constructed near the original ground surface, with its grade at about Elevation 231.5 m, rising toward both the north and south. Dunlop Street has been constructed on embankment fill up to about 5.5 m high; the local road grade is at about Elevation 237 m over the highway.

The existing two-span underpass was constructed in the early 1950s under Contract 50-116. According to the general layout drawing for the existing structure, which was provided by Morrison Hershfield (the structural designers for this preliminary study), the pier, abutments and associated wing walls and retaining walls are supported on spread footings which are founded at about Elevation 229 m to 229.3 m.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at this site in January 2001, at which time one borehole was drilled using a bombardier-mounted D-50 drill rig, supplied and operated by Master Soil Investigations Ltd. of Weston, Ontario. Borehole B11-1 was located on the west side of Highway 400, north of Dunlop Street. This borehole was advanced from slightly below the Highway 400 grade to about 40 m depth (approximately Elevation 191 m).

The borehole was advanced by telescoping casing in order to minimize the potential for drilling-induced movement of any contaminants at the site. The borehole was initially advanced to about 11 m depth using hollow stem augers. A bentonite seal was placed over the interval between 9 m and 11 m, the hollow stem augers were withdrawn, and 'N'-size casing was installed and socketted within the bentonite seal at about 10.7 m depth. Drilling below this depth was carried out using 'B'-size casing.

Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. The water level in the open borehole was observed throughout the drilling operations, and a piezometer was installed to permit monitoring of the groundwater level at the site. The piezometer was installed in a second shallow borehole, located about 4 m west of the deep borehole.

The field work was supervised on a full-time basis by a member of our staff who located the boreholes in the field, directed the drilling, sampling, and in-situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder Associates' laboratory in Mississauga for further examination. Index and classification tests consisting of water content determinations, Atterberg Limits testing and grain size distribution analyses were carried out on selected soil samples.

The borehole location and elevation were surveyed by Callon Dietz, Ontario Land Surveyors. The borehole elevation is referenced to geodetic datum, and the northing and easting coordinates are referenced to the MTM NAD83 survey system. The borehole location, ground surface elevation, and northing and easting coordinates are shown on the attached Drawing 1.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

This 30 km section of Highway 400 traverses, from south to north, the following physiographic regions as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, Third Edition, 1984): the Simcoe Lowlands; the Peterborough Drumlin Field; a second lobe of the Simcoe Lowlands; and the Simcoe Uplands. Along Highway 400, the Simcoe Lowlands are present from the southern limit of the project to just south of Innisfil Creek, and again from Essa Road (Simcoe Road 30, formerly Highway 27) to about 1 km north of Dunlop Street (Simcoe Road 90, formerly Highway 90). The Peterborough Drumlin Field occupies the belt between these lobes of the Simcoe Lowlands, extending from just south of Innisfil Creek, which is located about 1 km north of Highway 89, to Essa Road. The Simcoe Uplands extend from about 1 km north of Dunlop Street to beyond the northern limit of the project at Highway 11.

The two sections where Highway 400 crosses the Simcoe Lowlands consist of two lobes of a sand plain which include the shores of Kempenfelt Bay, the Nottawasaga River and Innisfil Creek. The surficial soils of these sections of the Simcoe Lowlands consist primarily of sand, although silt, clay or peat may be found in low-lying areas. The Dunlop Street site is located within this physiographic region.

The surficial soils in the Peterborough Drumlin Field consist primarily of gravelly sand till or sand and gravel deposits. Drumlins (glacially-shaped hills) are more frequent in the southern portion of the section of the Peterborough Drumlin Field traversed by Highway 400. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins.

The surficial soils in the Simcoe Uplands physiographic region are primarily sandy silt till deposits, known to contain occasional boulders. Low-lying areas may be infilled with shallow sand and gravel deposits, which are shoreline deposits of a former glacial lake that once flooded the area.

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the borehole and the results of in-situ and laboratory testing are given on the Record of Borehole sheets and Figures 1 and 2. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

Borehole B11-1 was advanced in the northwest quadrant of the structure site, from approximately Highway 400 grade. The location and elevation for this boring are shown on the attached Drawing 1. In summary, below a thin layer of topsoil and reworked silty clay, the native soils below the Highway 400 level consist of interlayered deposits of stiff to very stiff clayey silt to silty clay, and compact to dense silty sand and sandy silt. Very dense silty sand, with measured SPT 'N' values greater than 100 blows per 0.3 m of penetration, was encountered at about 39 m depth (Elevation 192.5 m).

A more detailed description of the subsurface conditions encountered in the borehole is provided in the following sections. It is noted that no odour, staining or other evidence of contamination was observed in the recovered samples, or in the auger cuttings produced during drilling of Borehole B11-1. Further, the City of Barrie confirmed during the investigation that the TCE contaminant plume is not known to extend to the Dunlop Street structure area.

4.2.1 Upper Clayey Silt to Silty Clay

A 10 m thick deposit of clayey silt to silty clay was encountered below about 200 mm of topsoil and 600 mm of silty clay fill/reworked soil. The deposit extends from about Elevation 230.5 m to 220.5 m. The clayey silt to silty clay contains trace sand, as well as significant interlayers of wet silty sand; a 1.7 m layer of compact to dense silty sand was encountered at about Elevation 227 m, and a 0.4 m thick layer of silty sand was encountered at about Elevation 224.5 m. A grain size distribution test result obtained for a sample of the clayey silt to silty clay material is shown on Figure 1.

The recovered samples were moist, with measured natural water contents ranging from 15 to 20 per cent. Atterberg limits testing carried out on three samples of the clayey silt to silty clay measured plastic limits of 13 to 16 per cent, liquid limits of 22 to 27 per cent, and plasticity indices of 7 to 12 per cent. The limit test results indicate that the soil is inorganic and of low plasticity.

A Standard Penetration Test (SPT) 'N' value of 19 blows per 0.3 m of penetration was measured at the surface of this deposit, possibly indicative of a stiffer "crust". Measured SPT 'N' values of 4 to 19 blows per 0.3 m of penetration were measured below Elevation 230 m, indicating that the clayey silt to silty clay deposit has a firm to very stiff consistency.

4.2.2 Sandy Silt

A 7 m thick sandy silt deposit was encountered at about Elevation 200.5 m, below the upper clayey silt to silty clay. The sandy silt contains thin interlayers of silty clay. The result from a grain size distribution test carried out on a representative sample of this layered deposit is shown on Figure 2.

The recovered samples of sandy silt were wet, with measured natural water contents of 14 to 17 per cent. The measured SPT 'N' values ranged from 31 to 48 blows per 0.3 m of penetration, indicating that the sandy silt has a dense relative density.

4.2.3 Lower Silty Clay

A lower deposit of silty clay was encountered in Borehole B11-1 at about 18 m depth (Elevation 213 m), extending to about 39 m depth (Elevation 192.5 m). This silty clay contains trace sand; two relatively thin interlayers of wet silty sand to sandy silt were encountered within this deposit, as noted on the borehole record. The recovered silty clay samples were moist, with measured natural water contents of 16 to 24 per cent.

The SPT 'N' values measured in the silty clay deposit ranged from 15 to 30 blows per 0.3 m of penetration, indicating that this deposit has a very stiff consistency. One measured SPT 'N' value of 54 blows per 0.3 m of penetration was obtained; however, this measurement reflects the very dense relative density of the sandy silt interlayer encountered at about Elevation 202 m.

4.2.4 Silty Sand

A lower deposit of silty sand was encountered at 38.7 m depth (Elevation 192.5 m). The recovered sample of silty sand was wet, with a measured SPT 'N' value of 175 blows for 150 mm of penetration. Based on this SPT 'N' value, this lower silty sand has a very dense relative density.

4.3 Groundwater Conditions

The water level in Borehole B11-1 was at between about 6 m and 9 m depth (Elevation 225 m to 222.5 m) during the drilling operations. The groundwater level in the piezometer, which is sealed within the silty sand interlayer in the upper clayey silt to silty clay deposit, was measured at 1.3 m depth (about Elevation 230 m) in March 2001. The groundwater levels associated with the sandy silt and lower silty sand deposits were not determined during this preliminary investigation.

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

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
DUNLOP STREET UNDERPASS
STRUCTURE SITE 30-175
HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides preliminary foundation design recommendations for the replacement of the existing Dunlop Street underpass structure, associated with the widening of Highway 400. The recommendations are preliminary only and are based on interpretation of the factual data obtained from one borehole advanced to 40 m depth during the subsurface investigation at this site. The interpretation and recommendations provided are intended for planning purposes only, to provide the information necessary at this stage of the study. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the planning of the project. Further foundation investigation will be required at this structure site as part of the detailed design stage of the project.

It is understood that Highway 400 will be widened from its existing six-lane configuration to an interim configuration of eight lanes, and an ultimate configuration of ten lanes, and that an alternative for a twelve-lane express / collector system is under consideration between Molson Park Drive and Duckworth Street in Barrie. Throughout the project length, it is expected that the existing highway platform will be widened by between 13 m and 30 m. Replacement of the existing Dunlop Street underpass structure will therefore be necessary.

Based on the general layout drawing for the existing double-span structure, it appears that the pier, abutments and associated wing walls and retaining walls are supported on spread footings which are founded at about Elevation 229 m to 229.3 m. The Highway 400 grade is at about Elevation 231.5 m, near the original ground surface. Dunlop Street has been constructed on embankment fill up to about 5.5 m high, with its grade at about Elevation 237 m over the highway.

5.2 Bridge Foundation Options

The native soils below the Dunlop Street grade consist of interlayered deposits of firm to very stiff clayey silt to silty clay, and compact to dense silty sand and sandy silt. Very dense silty sand was encountered at about 39 m depth (Elevation 192.5 m).

Consideration could be given to founding the replacement structure on spread footings, either placed on the native upper clayey silt to silty clay deposit or "perched" on a compacted granular pad within the approach embankment fill. However, settlement of the predominantly firm upper clayey silt to silty clay stratum will occur and the magnitude of this settlement could preclude the use of spread footings for the replacement structure.

It is recommended that consideration be given to the use of deep foundations, such as steel "H" piles, for support of the new structure. Such piles would be driven to found within the very dense, lower silty sand deposit, some 40 m below the Highway 400 grade at this site. The soil conditions at this site are favourable for the use of an integral abutment replacement structure.

Preliminary recommendations for spread footings and for driven steel "H" pile foundations are provided in the following sections.

5.3 Spread Footings

For preliminary design of the abutment and pier foundations, spread footings may be placed on the firm to very stiff clayey silt to silty clay deposit at or below Elevation 229 m. Alternatively, consideration could be given to the use of abutment footings perched on a compacted granular pad within the approach embankment fill.

5.3.1 Axial Geotechnical Resistance

Spread footings placed on the properly prepared clayey silt to silty clay deposit at or below Elevation 229 m may be designed for a factored geotechnical resistance at Ultimate Limit States (ULS) of 200 kPa, assuming a 3 m wide footing.

The settlement of these footings will be dependent on the footing size, configuration, and applied loads. In addition, settlement of the footings will occur due to consolidation of the founding soils under the additional embankment loading if there is a widening or raising of the existing embankment grade. For preliminary design purposes, the geotechnical resistance at Serviceability Limit States (SLS), for 25 mm of settlement, may be taken as 130 kPa for footings placed at or below Elevation 229 m. It is estimated that the abutment footings could experience up to about 25 m of additional settlement due to raising/widening of the approach embankments on the existing Dunlop Street alignment. The abutment footings, therefore, could experience up to 50 mm of settlement with the magnitude dependent on the proposed embankment configuration. It should be noted that the pier footing would not be influenced by the embankment loading and settlement of this footing would be only about 25 mm.

Due to the potential differential settlement between the abutment and pier footings, the option of shallow spread footings may not be feasible. The settlement should be confirmed at the final design stage, once the footing size, configuration and loadings as well as the proposed embankment configuration are known, to assess whether this spread footing option is feasible.

Additional field and laboratory testing should be carried out to confirm the strength and to determine the compressibility characteristics of the firm to stiff clayey silt to silty clay stratum to refine the settlement predictions.

For abutment spread footings placed within the approach embankments on a compacted Granular 'A' pad, a factored geotechnical resistance at ULS of 900 kPa may be assumed for preliminary design. The geotechnical resistance at SLS will depend on the thickness of Granular 'A' and the consistency and thickness of the underlying soils. A value of 350 kPa may be assumed for preliminary design; however, this must be reviewed at the final design stage. The concerns with respect to settlement of the abutment footings due to embankment loading as described above will apply to perched abutments as well. Since the footing load will be spread through the Granular "A" pad, the magnitude of differential settlement between the abutments and the pier will be less with this perched abutment approach and may therefore be the preferred option.

The geotechnical resistances provided herein are given under the assumption that the loads will be applied perpendicular to the surface of the footings; where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the Ontario Highway Bridge Design Code (OHBDC).

5.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footing and the subsoils should be calculated in accordance with Section 6-8.4.3 of the OHBDC. The angle of friction between the concrete and the firm to stiff clayey silt to silty clay may be taken as 22 degrees. The corresponding coefficient of friction, tan δ, would then be 0.4. Where "perched" footings are adopted, the angle of friction between the concrete and the compacted Granular 'A' pad may be taken as 30 degrees, with a corresponding coefficient of friction equal to 0.58.

5.3.3 Frost Protection

The footings should be provided with a minimum of 1.5 m of soil cover for frost protection.

5.4 Driven Steel H-Piles

Based on the results of Borehole B11-1, the surface of the very dense lower silty sand deposit is at about Elevation 192.5 m. For preliminary design, a pile tip level of Elevation 190 m may be assumed for the structure replacement. The piles would be about 40 m to 45 m long, depending on the pile cap level adopted. It is noted that additional borehole investigation will be required at the detailed design stage in order to confirm the design pile tip elevations.

5.4.1 Axial Geotechnical Resistance

For preliminary design, the factored axial resistance at Ultimate Limit States (ULS) for steel HP 310 x 110 H-piles driven into the lower silty sand deposit may be taken as 1,200 kN. The axial resistance at Serviceability Limit States (SLS) for 25 mm of settlement may be taken as 1,200 kN.

To achieve the above design resistances, the piles should be driven to a tip level of at least Elevation 190 m. The rated energy of the hammer should not exceed 60 kJ. Provision should be made to re-tap selected piles to confirm the set after adjacent piles have been driven, in accordance with MTO's current Special Provision.

With the Dunlop Street underpass lengthened, the existing embankment will likely have to be raised and widened to accommodate the new structure. This additional fill will induce consolidation settlement of the clayey silt to silty clay soils. At the abutment locations, this consolidation will induce a downward movement of the soils adjacent to the piles, and negative skin friction will develop along the upper portion of the pile shaft embedded within the clayey silt to silty clay. Down to a certain point along the pile, called the neutral point, the settlement of the soil will be larger than the downward movement of the pile. The shear stresses mobilized along the pile down to the neutral point act downward, causing a downdrag load. Below this point, the downward movement of the pile is larger than the soil settlement and the mobilized shear stresses act upward on the pile as positive skin friction. At this site, with the pile tip resting on the very dense sand deposit, downward movement of the pile will occur due to yielding at the tip as well as pile compression.

The magnitude of the downdrag load acting on the pile is a function of the skin friction that develops between the pile and the clayey silt to silty clay, the surface area of the pile within these deposits, and the embankment loading (which is only partial filling in this case). The load calculated in this manner is a nominal (unfactored) load. The structural engineer must multiply this load by a load factor of 1.25 and include it as part of the dead load effects acting on the pile, as described in the OHBDC. For preliminary design, the negative skin friction load on a single pile may be taken as 250 kN. The downdrag load will have to be reassessed during the detailed design stage, using shear strength and consolidation data which should be determined for the shallow subsoils at the abutment and pier locations.

To minimize the amount of negative skin friction that may develop on a pile, consideration could be given to placing the new fill for any widened sections of the Dunlop Street embankment as early as possible to maximize the amount of settlement that occurs prior to the driving of the piles.

5.4.2 Resistance to Lateral Loads

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

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

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

where n_h is the constant of subgrade reaction
z is the depth (m)
B is the pile diameter (m)

For cohesive soils:

$$k_h = \frac{k_{s1}}{5B}$$

where B is the pile diameter (m) and
k_{s1} is the constant of subgrade reaction

The piles will be driven through embankment fill, firm to very stiff clayey silt to silty clay, and compact to dense, wet silty sand to sandy silt. The following ranges in value of n_h and k_{s1} may be assumed in the structural analysis; these values will have to be confirmed at the detailed design stage.

Soil Type	n_h	k_{s1}
Embankment fill	5 to 15 MPa/m	–
Firm to very stiff clayey silt to silty clay	–	10 to 35 MPa/m
Dense, wet sandy silt	5 to 10 MPa/m	–
Very stiff silty clay	–	25 to 60 MPa/m

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

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

5.4.3 Frost Protection

The pile caps should be provided with a minimum 1.5 m of soil cover for frost protection.

5.5 Lateral Earth Pressures

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

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the abutments and walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00

- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the abutment wall, in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be used in accordance with OPSS 501.06.

- The granular fill may be placed either in a zone with width equal to at least 1.5 m behind the back of the stem (Case I from OHBDC Figure 6-7.4.1) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II from OHBDC Figure 6-7.4.4).

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

Soil unit weight:	20 kN/m³
Coefficients of lateral earth pressure:	
Active, K_a	0.35
At rest, K_o	0.50

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

	Granular 'A'	**Granular 'B' Type II**
Soil unit weight:	22 kN/m³	21 kN/m³
Coefficients of lateral earth pressure:		
Active, K_a	0.27	0.31
At rest, K_o	0.43	0.47

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

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

5.6 Embankment Design

Based on the topographic information on the Engineering and Title Records plates and on site reconnaissance, the existing Dunlop Street embankment side slopes are formed at a gradient of about 2 horizontal to 1 vertical (2H:1V). If any widening of the local road is required, the new side slopes should be formed at a maximum gradient of 2H:1V. The embankment widening should be carried out using conventional fill placement and compaction practices, and benching of the existing embankment side slopes should be carried out to key in the new fill.

The embankment stability and settlement should be further assessed as part of the detailed design, once the embankment configuration has been finalized and the shear strength and consolidation characteristics of the upper clayey silt to silty clay stratum have been established.

5.7 Design and Construction Considerations

5.7.1 Groundwater Control

Groundwater seepage into the footing excavations is expected to be minor. Pumping from properly-filtered sumps or a filtered drain placed at the base of the excavation should provide sufficient groundwater control during foundation works. Surface water run-off should be directed away from the footing excavations.

5.7.2 Excavation

Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. The footing excavations will extend a minimum of 1.5 m below lowest surrounding grade, through existing fill and into the firm to very stiff clayey silt to silty clay deposit. The fill and clayey silt to silty clay would be classified as Type 2 to 3 soils. Temporary open-cut slopes should therefore be maintained no steeper than 1 horizontal to 1 vertical (1H:1V). Where space restrictions dictate, footing excavations could also be carried out within a braced excavation.

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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

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 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 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	c_u, s_u 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 (LV-laboratory vane test)
γ	unit weight

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

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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 δ
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

RECORD OF BOREHOLE No B11-1 — 1 OF 3 — METRIC

PROJECT 001-1143F | LOCATION N 4915592.6; E 287961.2 | ORIGINATED BY SB/PKS
W.P. 30-95-00 | BOREHOLE TYPE SEE NOTE 1 | COMPILED BY LCC
DIST SW HWY 400 | DATE Jan.17-26/2001 | CHECKED BY ASP
DATUM Geodetic

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	γ kN/m³	GR	SA	SI	CL
231.2	GROUND SURFACE								
0.0	Topsoil								
0.2	Silty Clay, trace sand (Fill) Brown								
230.4									
0.8	Clayey Silt to Silty Clay, trace sand Firm to very stiff Moist Grey	1	SS	19					
		2	SS	4					
		3	SS	8					
		4	SS	7		0	2	72	26
227.1									
4.1	Silty Sand, trace clay and gravel Compact to dense Moist Grey	5	SS	26					
		6	SS	37					
225.4									
5.8	Silty Clay, trace sand Stiff Moist Grey Contains a layer of silty sand from 6.6m to 7m depth (Elev.224.6m to 224.2m)	7	SS	14					
		8	SS	10					
		9	SS	11					
220.5									
10.7	Sandy Silt containing silty clay interlayers Dense Wet Grey	10	SS	31					
		11	SS	42					
		12	SS	48		0	23	63	14

GROUND WATER CONDITIONS: ▽

ELEVATION SCALE: 231, 230, 229, 228, 227, 226, 225, 224, 223, 222, 221, 220, 219, 218, 217

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100)
SHEAR STRENGTH kPa: o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED (20 40 60 80 100)
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%) (10 20 30)

Continued Next Page

+³, ×³: Numbers refer to Sensitivity o³% STRAIN AT FAILURE

RECORD OF BOREHOLE No B11-1 — 2 OF 3 — METRIC

PROJECT 001-1143F | LOCATION N 4915592.6; E 287961.2 | ORIGINATED BY SB/PKS
W.P. 30-95-00 | BOREHOLE TYPE SEE NOTE 1 | COMPILED BY LCC
DIST SW HWY 400 | DATE Jan.17-26/2001 | CHECKED BY ASP
DATUM Geodetic

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	γ kN/m³	GR	SA	SI	CL
	-- CONTINUED FROM PREVIOUS PAGE --								
	Sandy Silt containing silty clay interlayers Dense Wet Grey	13	SS	39					
		14	SS	40					
213.2									
18.0	Silty Clay, trace sand Very stiff Moist Grey	15	SS	16					
	Silty sand layer encountered between 19.8m and 20.1m depth (Elev.211.4m to 211.1m)	16	SS	30					
		17	SS	19					
		18	SS	15					
	Sandy Silt layer encountered between 29.3m and 29.6m depth (Elev.201.9m to 201.6m)	19	SS	22					
		20	SS	54					

ELEVATION SCALE: 216, 215, 214, 213, 212, 211, 210, 209, 208, 207, 206, 205, 204, 203, 202

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100)
SHEAR STRENGTH kPa: o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED (20 40 60 80 100)
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%) (10 20 30)

Continued Next Page

+³, ×³: Numbers refer to Sensitivity o³% STRAIN AT FAILURE

RECORD OF BOREHOLE No B11-1 3 OF 3 METRIC

PROJECT 001-1143F
W.P. 30-95-00
DIST SW HWY 400
DATUM Geodetic

LOCATION N 4915592.6; E 287961.2
BOREHOLE TYPE SEE NOTE 1
DATE Jan.17-26/2001

ORIGINATED BY SB/PKS
COMPILED BY LCC
CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	-- CONTINUED FROM PREVIOUS PAGE --										
	Silty Clay, trace sand Very stiff Moist Grey						201				
							200				
			21	SS	24		199				
							198				
							197				
							196				
							195				
							194				
							193				
192.5											
38.7	Silty Sand Very dense Brown Wet						192				
191.4			22	SS	175/.15				o		
39.8	END OF BOREHOLE										

Notes:
1. Hollow stem augers used to advance to 10.7m depth. After sampling, bentonite seal was placed between 11.3m and 8.8m depth, and augers were withdrawn. "N" casing was installed to 10.7m depth, then "B" casing was used for the remainder of the borehole.
2. During drilling operations, water level in open borehole was typically between 6.2m and 8.8m depth (Elev.225.0m to 222.4m).
3. Piezometer installed in second borehole drilled 4m west. Water level in piezometer measured at 1.3m depth (Elev.229.9m) on March 15, 2001.

+³, X³: Numbers refer to Sensitivity O³% STRAIN AT FAILURE

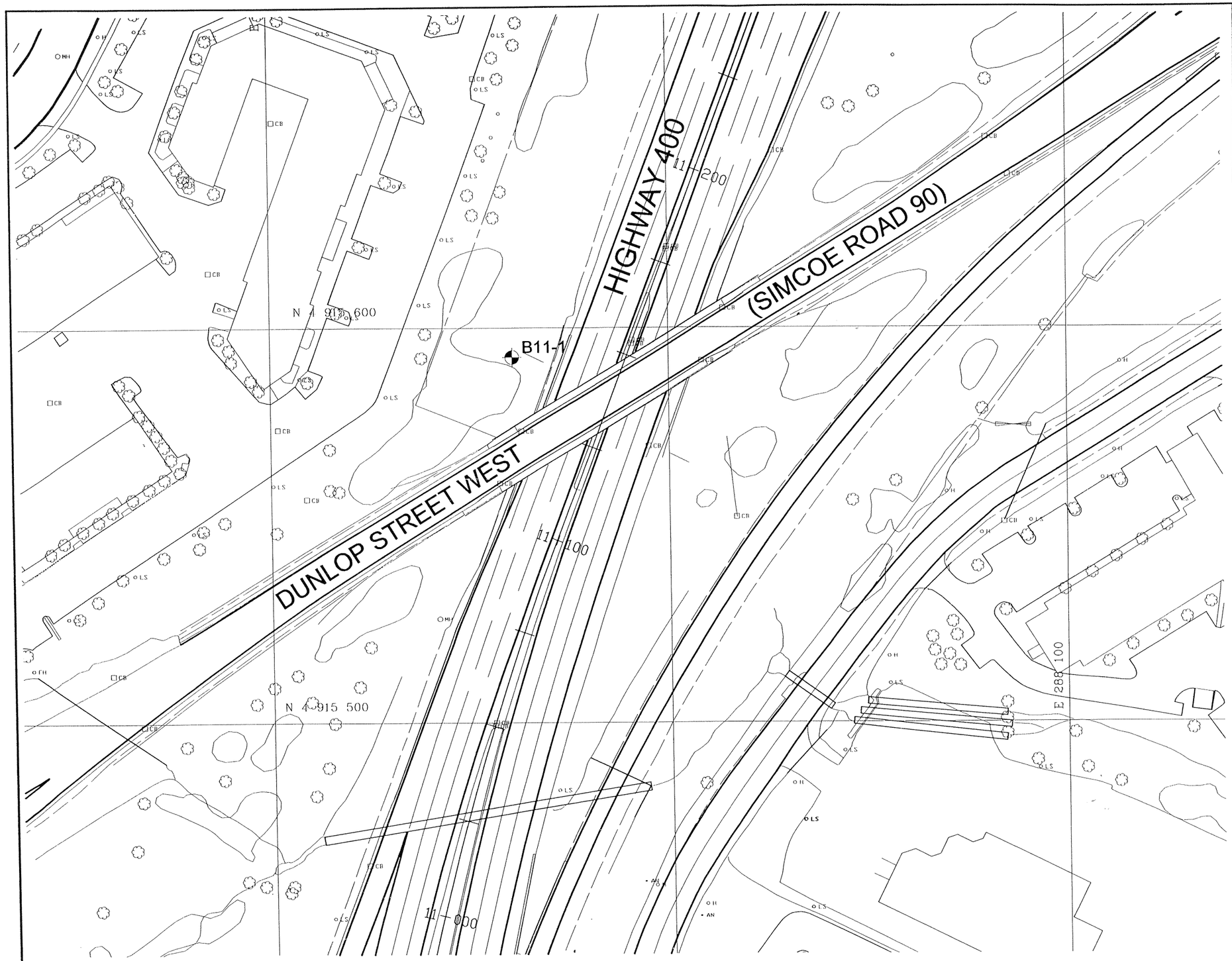
DIST	HWY 400
CONT. No.	
GWP No.	30-95-00

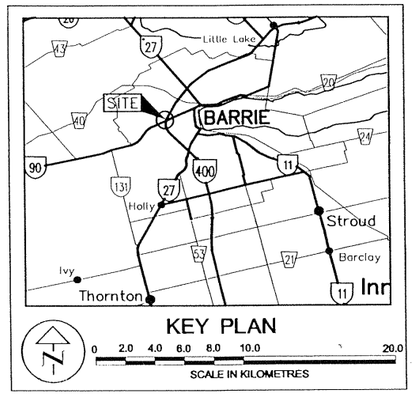
SHEET

DUNLOP STREET UNDERPASS
HWY 400
BOREHOLE LOCATION PLAN

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA





LEGEND

- Borehole, previous investigation
- Borehole, present investigation

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
B11-1	231.2	4,915,592.6	287,961.2

REFERENCE

This drawing was created from digital file "50209.dwg" provided by URS Cole Sherman

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

NO.	DATE	BY	REVISION

Geocres No.

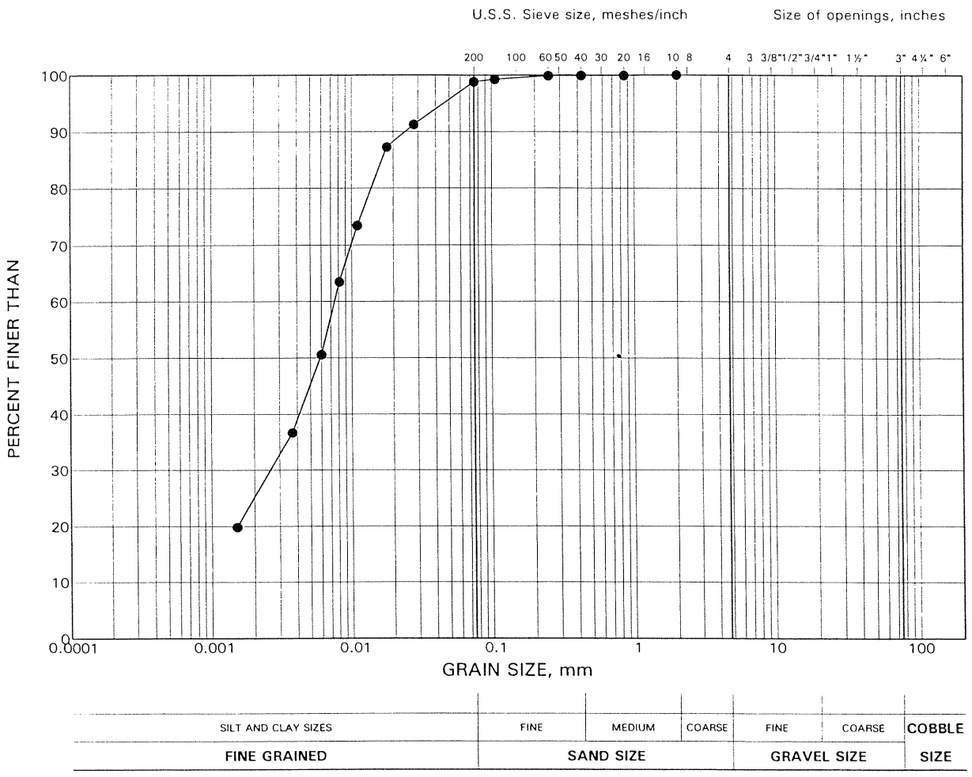
HWY. No. 400		PROJECT NO.: 001-1143F	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-175
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

P1143F16.DWG

plot scale 1:2imp.

GRAIN SIZE DISTRIBUTION
Clayey Silt to Silty Clay

FIGURE 1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	B11-1	4	227.8

Project 001-1143F

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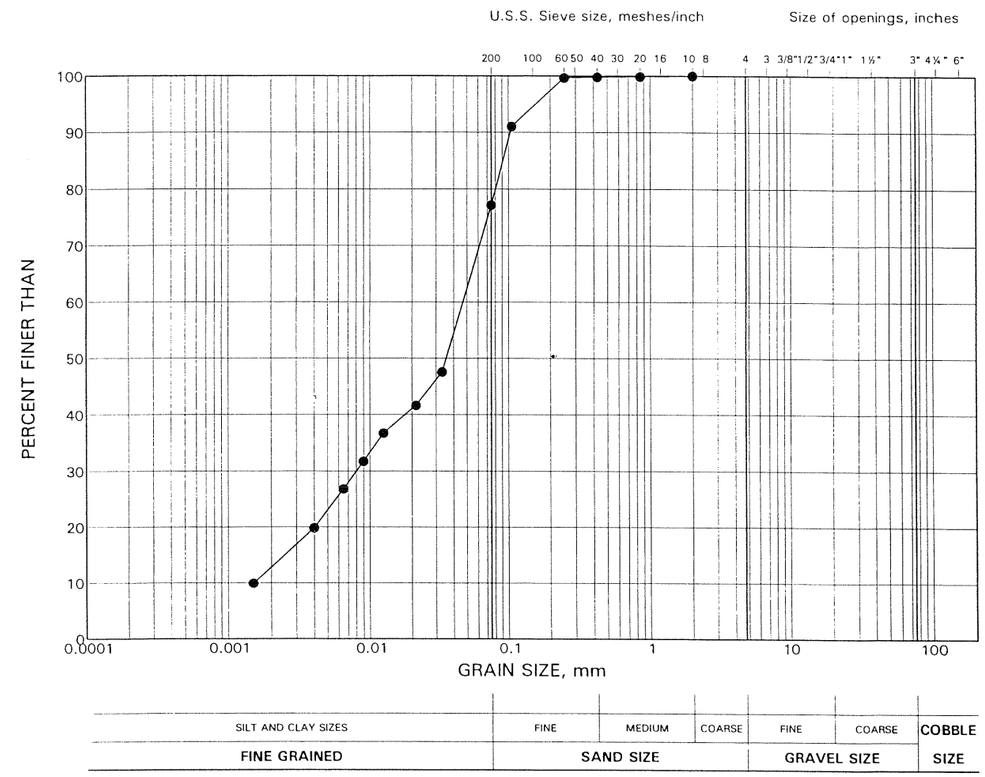
GRAIN SIZE DISTRIBUTION
Sandy Silt with Clayey Silt Interlayers

FIGURE 2



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	B11-1	12	217.2

Project 001-1143F

Golder Associates

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Golder Associates

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT ANNE STREET UNDERPASS STRUCTURE SITE 30-347 HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11 G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

Submitted to:

URS Cole, Sherman
75 Commerce Valley Drive East
Thornhill, Ontario
L3T 7N9

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Appendix A Records of Boreholes – 1957 Subsurface Investigation

PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
ANNE STREET UNDERPASS
STRUCTURE SITE 30-347
HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

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Drawing 1
Appendix A

LIST OF DRAWINGS

Drawing 1 Anne Street Underpass, Highway 400, Borehole Location Plan

LIST OF APPENDICES

Appendix A Records of Boreholes – 1957 Subsurface Investigation

1.0 INTRODUCTION

Golder Associates Ltd. has been retained by URS Cole, Sherman (Cole, Sherman) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the ultimate widening of Highway 400 from 1 km south of Highway 89, northerly 30 km to Highway 11, in Simcoe County, Ontario. Foundation engineering services are required for the widening and / or replacement of eighteen existing overpass and underpass structures, as well as five structural culverts.

This report addresses the replacement of the existing Anne Street underpass structure which is located in Barrie. Existing subsurface data for this site from an investigation conducted for the Department of Highways, Ontario (DHO) in 1957 were used to determine the subsurface conditions for this preliminary design study. The 1957 report was prepared by Universal Geotechnique Limited (*"Report on Subsurface Exploration for Proposed Overpass at Anne Street and Highway 400, Barrie Ontario"*, dated 1957 – GEOCRES File No, 31D-182).

The terms of reference for the scope of work are outlined in Golder Associates' Proposal No. P01-1192, dated June 2000.

2.0 SITE DESCRIPTION

The existing two-span Anne Street underpass structure is located about 1 km north of Dunlop Street (Simcoe Road 90, formerly Highway 90) and less than 2 km south of Bayfield Street (Highway 26) in Barrie, Ontario. The MTO has designated this underpass as Structure Site 30-347.

At this structure site, the Highway 400 grade is at about Elevation 236 m, rising northward. Anne Street has been constructed in fill with approach embankments up to about 6 m in height; the Anne Street grade is at about Elevation 242 m over Highway 400.

The existing two-span underpass structure was constructed in the late 1950s. According to the general layout drawing for this contract, which was provided by Morrison Hershfield (the structural designers for this preliminary study), the existing structure is supported by driven steel H-piles. The underside of the abutment and pier pile caps is at about Elevation 232.5 m. The tip elevation of the H-piles is not known.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at this site for the Department of Highways, Ontario (DHO) in June and July of 1957, by Universal Geotechnique Limited. At that time, a total of six boreholes were advanced in the vicinity of the abutments and pier for the then-proposed structure. Boreholes 1 and 2 were located in the vicinity of the east abutment, Boreholes 3 and 4 were drilled near the west abutment, and Boreholes 5 and 6 were advanced at the approximate location of the central pier. Boreholes 1, 4, 5 and 6 were advanced to about 7.5 m depth, while Boreholes 2 and 3 were advanced to about 19 m and 15 m depth at the east and west abutments, respectively. All of the boreholes were advanced from approximately Highway 400 grade.

Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. The groundwater conditions in the open borehole were observed during and following the drilling operations. No laboratory testing was carried out in conjunction with this 1957 investigation.

The borehole locations and elevations, referenced to the geodetic datum, were established by DHO. Approximate northing and easting co-ordinates consistent with the MTM NAD83 survey system, currently in use on this project, have been determined by Golder Associates based on the borehole locations given in the 1957 report. The approximate borehole locations and northing and easting co-ordinates are shown on the attached Drawing 1.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

This 30 km section of Highway 400 traverses, from south to north, the following physiographic regions as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, Third Edition, 1984): the Simcoe Lowlands; the Peterborough Drumlin Field; a second lobe of the Simcoe Lowlands; and the Simcoe Uplands. Along Highway 400, the Simcoe Lowlands are present from the southern limit of the project to just south of Innisfil Creek, and again from Essa Road (Simcoe Road 30, formerly Highway 27) to about 1 km north of Dunlop Street (Simcoe Road 90, formerly Highway 90). The Peterborough Drumlin Field occupies the belt between these lobes of the Simcoe Lowlands, extending from just south of Innisfil Creek, which is located about 1 km north of Highway 89, to Essa Road. The Simcoe Uplands extend from about 1 km north of Dunlop Street to beyond the northern limit of the project at Highway 11.

The two sections where Highway 400 crosses the Simcoe Lowlands consist of two lobes of a sand plain which include the shores of Kempenfelt Bay, the Nottawasaga River and Innisfil Creek. The surficial soils of these sections of the Simcoe Lowlands consist primarily of sand, although silt, clay or peat may be found in low-lying areas. The Anne Street underpass site is located within the Simcoe Lowlands physiographic region.

The surficial soils in the Peterborough Drumlin Field consist primarily of gravelly sand till or sand and gravel deposits. Drumlins (glacially-shaped hills) are more frequent in the southern portion of the section of the Peterborough Drumlin Field traversed by Highway 400. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins.

The surficial soils in the Simcoe Uplands physiographic region are primarily sandy silt till deposits, known to contain occasional boulders. Low-lying areas may be infilled with shallow sand and gravel deposits, which are shoreline deposits of a former glacial lake that once flooded the area.

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes are given on the Record of Borehole sheets contained in Appendix A. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

Boreholes 1 and 2 were advanced in the vicinity of the east abutment, Boreholes 3 and 4 were drilled near the west abutment, and Boreholes 5 and 6 were advanced at the approximate location of the central pier. The approximate locations and ground surface elevations for these borings are shown on the attached Drawing 1.

In summary, the soils below Highway 400 grade at this site consist of fill overlying a generally compact to dense deposit of sand and gravel to sandy silt, in turn underlain by a deposit of hard silty clay till. A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Fill

The 1957 boreholes encountered between 2 m and 2.5 m of sand fill containing gravel, clay, organics and wood fragments. The base of this fill layer, as encountered in the boreholes, was at about Elevation 233 m to 234 m. The measured Standard Penetration Test (SPT) "N" values ranged from 9 to 29 blows per 0.3 m of penetration, indicating that the fill has a loose to compact relative density.

4.2.2 Sand and Gravel to Sandy Silt

Below the fill at about Elevation 233 m to 234 m, the boreholes encountered a deposit which contains layers ranging in composition from sand and gravel to sand to sandy silt. Evidence of bedding, and seams or interlayers of silty clay (particularly within the sandy silt portions of the deposit) are noted on the borehole records. The deposit was fully penetrated by the two deeper borings (Boreholes 2 and 3, at the east and west abutments, respectively) where it was about 12 m to 15 m in thickness.

The measured SPT "N" values ranged from 22 to 55 blows, but were typically between 22 and 40 blows per 0.3 m of penetration; the deposit therefore has a predominantly compact to dense relative density.

4.2.3 Silty Clay Till

In the deeper borings which were advanced at the east and west abutments, a silty clay deposit was encountered below the sand and gravel to sandy silt stratum. This deposit was described as a

till in the text of the 1957 report. The surface of the till was at Elevations 218 m and 222 m in Boreholes 2 and 3, respectively. The deposit was not fully penetrated by either boring, but was at least 1 m to 2 m thick at these borehole locations. The measured SPT "N" values of 52 and 62 blows per 0.3 m of penetration indicate that this till deposit has a hard consistency.

4.3 Groundwater Conditions

The water levels observed in the open boreholes following completion of the June / July 1957 drilling operations were measured to be between Elevation 233.5 m and 234.5 m, approximately 1 m to 2 m below then-existing ground surface and 1.5 m to 2.5 m below the current Highway 400 grade. It should be noted that groundwater levels are expected to fluctuate seasonally and are expected to rise during wet periods of the year.

GOLDER ASSOCIATES LTD.

Lisa C. Coyne, P.Eng.,
Geotechnical Engineer

for Anne S. Poschmann, P.Eng.,
Principal

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

LCC/ASP/FJH/clg
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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
ANNE STREET UNDERPASS
STRUCTURE SITE 30-347
HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

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

It is understood that Highway 400 will be widened from its existing six-lane configuration to an interim configuration of eight lanes, and an ultimate configuration of ten lanes, and that an alternative for a twelve-lane express / collector system is under consideration between Molson Park Drive and Duckworth Street in Barrie. Throughout the project length, it is expected that the existing highway platform will be widened by between 13 m and 30 m. Replacement of the existing Anne Street underpass structure will be necessary.

Based on the general layout drawing for the existing two-span structure, the abutments and pier are supported on driven steel H-piles, with the underside of the abutment and pier pile caps at about Elevation 232.5 m. The Highway 400 grade is at about Elevation 236 m, while Anne Street has been constructed in fill with its grade at about Elevation 242 m over Highway 400.

5.2 Bridge Foundation Options

The soils below the Highway 400 level consist of about 2 m to 2.5 m of fill overlying a deposit of sand and gravel, sand, and silt, in turn underlain by a hard silty clay till deposit. The water level observed in the open boreholes at the time of the 1957 investigation was at about Elevation 233.5 m to 234.5 m (between 1.5 m and 2.5 m below the current Highway 400 grade).

Based on the thickness of the fill and the relatively high groundwater table in the granular subsoils, it is considered that shallow foundations (spread footings) are not a suitable option for support of the new underpass structure at this site. It is recommended that the new two-span structure be supported on driven steel H-piles; preliminary recommendations for these deep foundations are provided in the following sections.

5.3 Driven Steel H-Piles

It is recommended that the new two-span underpass structure be supported on steel H-piles driven to found within the hard silty clay till stratum, which was encountered below Elevation 218 m and 222 m at the east and west abutments, respectively. The top of this founding stratum, as encountered in the 1957 boreholes, is between 14 m and 18 m below Highway 400 grade, and about 20 m to 24 m below Anne Street grade. Based on the existing borehole information, it is expected that the depth where practical refusal to pile penetration will occur will be located below the base of the 1957 boreholes (i.e. below Elevation 216 m). In this regard, it is noted that additional borehole investigation work will be require at the proposed foundation locations during detailed design in order to confirm the thickness and consistency of the silty clay till founding stratum and establish design tip elevations.

It is recommended that the pile cap be maintained as high as possible, due to the relatively high groundwater level at the site. The pile length will vary due to the variable depth to this founding stratum; however, based on the above considerations and recommendations, for preliminary estimating purposes the piles can be considered to range from about 20 m to 25 m in length.

5.3.1 Axial Geotechnical Resistance

For preliminary design, the factored axial resistance at Ultimate Limit States (ULS) for steel HP 310 x 110 H-piles driven to found within the hard silty clay till stratum may be taken as 1,600 kN. The axial resistance at Serviceability Limit States (SLS) for 25 mm of settlement may be taken as 1,400 kN.

To achieve the above design resistances, the piles should be driven to a final set of no less than 15 blows per 25 mm of penetration using a hammer with rated energy of about 50 kJ, and not exceeding 60 kJ. Provision should be made to re-tap selected piles to confirm the set after adjacent piles have been driven, in accordance with MTO's current Special Provision.

5.3.2 Resistance to Lateral Loads

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

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

$$k_h = \frac{n_h z}{B}$$

where
n_h is the constant of subgrade reaction
z is the depth (m)
B is the pile diameter (m)

For the embankment fill and generally compact to dense sand and gravel to sandy silt soils through which the piles would be driven, the range in value of n_h may be taken as 5 MPa to 10 MPa in the structural analysis.

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

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

5.3.3 Frost Protection

The pile caps should be provided with 1.5 m soil cover for frost protection.

5.4 Lateral Earth Pressures

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

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the abutments and walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00

- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the abutment wall, in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be used in accordance with OPSS 501.06.

- The granular fill may be placed either in a zone with width equal to at least 1.5 m behind the back of the stem (Case I from OHBDC Figure 6-7.4.1) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II from OHBDC Figure 6-7.4.4).

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

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

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

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

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

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

5.5 Embankment Design

Based on the topographic information on the Engineering and Title Records plates and on site reconnaissance, the existing approach embankment side slopes are formed at a gradient of about 2 horizontal to 1 vertical (2H:1V). If any widening of the local road embankment will be required, the new side slopes should be formed at a maximum gradient of 2H:1V. The widening of the embankment should be carried out using conventional fill placement and compaction practices, and benching of the existing embankment side slopes should be carried out to key in the new fill.

5.6 Design and Construction Considerations

5.6.1 Dewatering

The sand and gravel to sandy silt stratum which underlies the site is water-bearing; based on observation of the water levels in the open boreholes following the June / July 1957 drilling operations, the groundwater level varies from Elevation 233.5 m to 234.5 m at the site and may be higher during wet periods of the year. If the pile caps are placed below about Elevation 235 m, a dewatering scheme will be necessary to facilitate pile cap excavation, pile driving and concrete placement in dry conditions. The groundwater level should be lowered to at least 0.5 m below the underside of the pile caps prior to excavation and pile driving.

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

5.6.2 Excavation

The pile cap excavations will extend a minimum of 1.5 m below lowest surrounding grade, and would be carried out through the existing embankment fill and possibly the compact to dense sand and gravel to sandy silt deposit. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. The fill and cohesionless native soils would be classified as Type II soils, assuming that proper dewatering control is in place for excavations taken below about Elevation 235 m. Temporary open-cut slopes should therefore be maintained no steeper than 1 horizontal to 1 vertical (1H:1V). Where space restrictions dictate, in particular adjacent to the existing Anne Street embankment, pile cap excavations could also be carried out within a braced excavation.

5.6.3 Obstructions

Although no cobbles or boulders were recorded on the records for the 1957 boreholes, it should be noted that cobbles and boulders are inherent in glaciolacustrine materials. Cobbles and boulders should therefore be expected during driving of the steel H-piles.

GOLDER ASSOCIATES LTD.

Lisa C. Coyne, P.Eng.,
Geotechnical Engineer

for Anne S. Poschmann, P.Eng.,
Principal

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

LCC/ASP/FJH/clg
N:\ACTIVE\1100\001-1143F\2002\RPT12-02JAN-ANNE.DOC

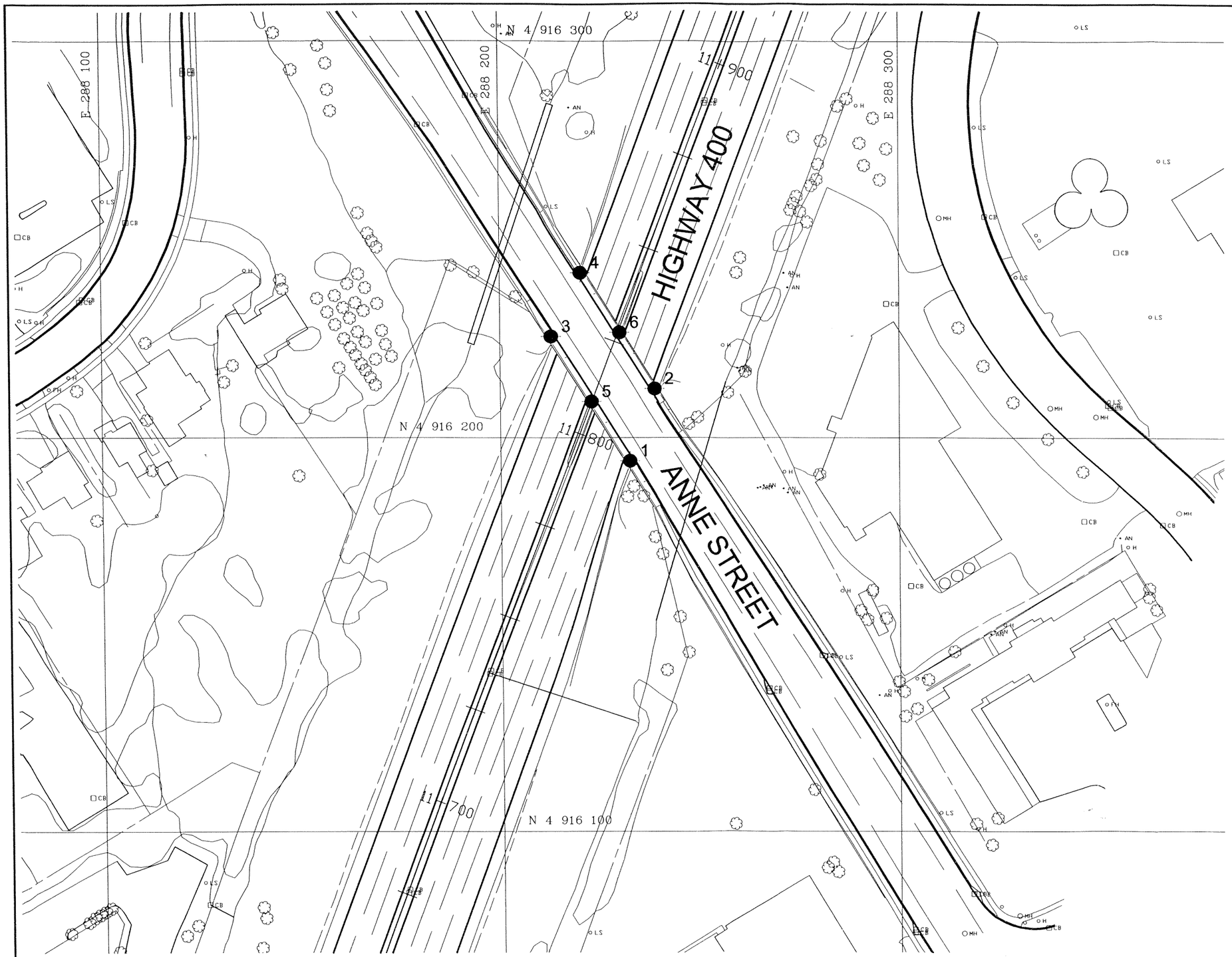
DIST HWY 400
CONT. No.
GWP No. 30-95-00

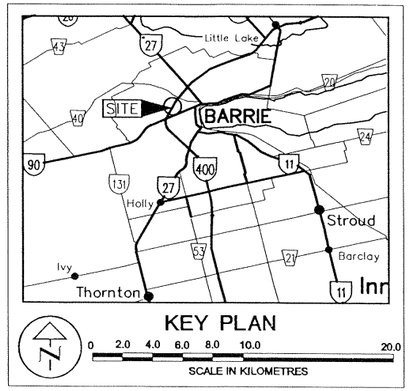
SHEET

ANNE STREET UNDERPASS
HWY 400
BOREHOLE LOCATION PLAN

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA





LEGEND

- Borehole, previous investigation
- Borehole, present investigation

No.	ELEVATION	LOCATION NORTHING	LOCATION EASTING
1	235.3	4,916,194	288,232
2	235.5	4,916,212	288,239
3	235.8	4,916,226	288,213
4	236.0	4,916,242	288,220
5	235.1	4,916,209	288,223
6	235.5	4,916,227	288,229

REFERENCE
This drawing was created from digital file "50210.dwg" provided by URS Cole Sherman

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

NO.	DATE	BY	REVISION

Geocres No.

HWY. No. 400		PROJECT NO.: 001-1143F	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-347
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

P1143F17.DWG

plot scale 1:20mp.

APPENDIX A

RECORDS OF BOREHOLES
1957 SUBSURFACE INVESTIGATION

UNIVERSAL **GEOTECHNIQUE** LIMITED

SOIL MECHANICS LABORATORY

BOREHOLE LOG

PRO Anne Street Overpass, Barrie, Ontario. ORDER NO. T.227/57

CLIENT Department of Highways, Ontario.

BOREHOLE NO. BH.1 DIAMETER 2-1/2" CASING 2-1/2"

BOREHOLE LOCATION See Plan INCLINATION Vertical BEARING ——

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
Firm brown to grey clayey sand with some organic matter and fine to medium gravel. Probably FILL.	771.82 235.3m		• 1	Zero – 0.0m Free Water		12	Moist Low to medium dry strength.
Loose brown to grey sand with fine to medium gravel and some organic matter. Probably FILL.			• 2			9	Moist Low dry strength.
Dense brown to grey fine to coarse SAND with generally subrounded fine to medium gravel.			• 3			30	do
Dense brown grey generally fine calcareous SAND with fine to medium subrounded gravel.			• 4			33	do
Firm do			• 5			22	do
Dense brown grey fine to medium calcareous SAND with fine to medium subrounded gravel.	227.7m		• 6	25'-1" – 7.6m		37 (7")	do
			End of Borehole				

SCALE: 1" = 5'-0" • DISTURBED SAMPLE ▮ UNDISTURBED SAMPLE

UNIVERSAL **GEOTECHNIQUE** LIMITED

SOIL MECHANICS LABORATORY

BOREHOLE LOG

PROJECT Anne Street Overpass, Barrie, Ontario, ORDER NO. T.227/37

CLIENT Department of Highways, Ontario.

BOREHOLE NO. BH.2 DIAMETER 2-1/2" CASING 2-1/2"

BOREHOLE LOCATION See Plan INCLINATION Vertical BEARING ——

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
Firm brown grey sand, clayey concentrations. Black organic matter. Probably FILL.	772.65			Zero	0.0m		
	235.5m		● 1			20	Moist
Firm do			● 2	Free Water		15	do
do			● 3			12	do
Iron staining							
	232.9m			8'-6"	2.6m		
Dense grey brown fine to coarse calcareous SAND and fine to medium generally subrounded GRAVEL.			● 4			32	Moist Low dry strength.
	231.5m			13'-0"	4.0m		
Dense brown sandy SILT with lenses of fine to medium SAND. Traces of bedding.			● 5			31	Moist. Low to medium dry strength.
do Some iron staining			● 6			48	do
	228.5m			23'-0"	7.0m		
Firm brown grey fine to medium calcareous SAND. Lenses of fine subrounded to rounded gravel embedded in clay.			● 7			26	Moist Low dry strength.
Dense brown grey fine to medium calcareous SAND with fine to medium gravel, generally subrounded.			● 8			30	do
do			● 9			35	do
do			● 10			-	Wash Sample
	221.2m			47'-0"	14.3m		
Dense grey generally fine calcareous SAND.			● 11			47	Moist Low dry strength.

SCALE: 1" = 5'-0" ● DISTURBED SAMPLE ▮ UNDISTURBED SAMPLE

UNIVERSAL **GEOTECHNIQUE** LIMITED

SOIL MECHANICS LABORATORY

BOREHOLE LOG

PROJECT Anne Street Overpass, Barrie, Ontario, ORDER NO. T.227/37

CLIENT Department of Highways, Ontario.

BOREHOLE NO. BH.2 DIAMETER 2-1/2" CASING 2-1/2"

BOREHOLE LOCATION See Plan INCLINATION Vertical BEARING ——

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
Dense grey generally fine calcareous SAND.	220.3m			50'-0"	15.2m		
	217.7m		● 12	58'-6"	17.8m	62	Moist. Sand: Low dry strength. Clay: High dry strength.
Very stiff grey calcareous silty CLAY.	216.8m		End of Borehole	61'-6"	18.7m		

SCALE: 1" = 5'-0" ● DISTURBED SAMPLE ▮ UNDISTURBED SAMPLE

UNIVERSAL **GEOTECHNIQUE** LIMITED

SOIL MECHANICS LABORATORY

BOREHOLE LOG

PROJECT: Anne Street Overpass, Barrie, Ontario. ORDER NO. T.227/57

CLIENT: Department of Highways, Ontario.

BOREHOLE NO. BH.3 DIAMETER 2-1/2" CASING 2-1/2"

BOREHOLE LOCATION: See Plan INCLINATION: Vertical BEARING: ——

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
	773.58			Zero	0.0m		
Firm brown sand, gravel, little clay and bits of wood. FILL.	235.8m		1			29	Moist
Firm brown sand and black organic matter. Probably FILL.			2			21	do
	233.8m			6'-7"	Free Water 2.0m		Wet
Firm grey to iron-stained yellow fine to medium SAND with fine to medium generally subrounded gravel.			3			27	Low dry strength.
do No iron staining.			4			33	Moist Low dry strength.
do			5			23	do
Dense grey generally fine calcareous silty SAND.	230.2m		6	18'-6"	5.6m	37	do
do Slight iron staining.			7			37	do
do No iron staining.			8			22	Wet Low dry strength.
	225.4m			34'-0"	10.4m		
Brown grey fine to medium calcareous SAND.			9				Wash Sample
Grey generally fine calcareous SAND.			10				Wash Sample
do Firm grey silty CLAY.	222.1m		11	45'-0"	13.7m	22	Moist Low dry strength.
Hard do	220.6m		12	50'-0"	15.2m	52	Lost sample

End of Borehole

SCALE: 1" = 5'-0" • DISTURBED SAMPLE ▮ UNDISTURBED SAMPLE

UNIVERSAL **GEOTECHNIQUE** LIMITED

SOIL MECHANICS LABORATORY

BOREHOLE LOG

PROJECT: Anne Street Overpass, Barrie, Ontario. ORDER NO. T.227/57

CLIENT: Department of Highways, Ontario.

BOREHOLE NO. BH.4 DIAMETER 2-1/2" CASING 2-1/2"

BOREHOLE LOCATION: See Plan INCLINATION: Vertical BEARING: ——

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
	774.26			Zero	0.0m		
Firm grey brown fine to medium somewhat clayey sand with gravel. Probably FILL.	236.0m		1			22	Moist
do With traces of organic matter. Probably FILL.			2	Free Water		17	Wet
Firm grey brown fine to coarse calcareous SAND and fine to medium generally subrounded GRAVEL.			3			22	Wet No dry strength.
	232.0m			13'-0"	4.0m		
Dense brown sandy SILT with thin lenses of clay. Exhibits bedding.			4			33	Damp Low to medium dry strength.
Dense grey generally fine calcareous SAND.			5			34	Wash sample
Dense grey brown generally fine calcareous SAND with occasional fine gravel. Exhibits faint bedding and some iron staining.	228.2m		6	25'-6"	7.8m	55	Moist. Low to medium dry strength.

End of Borehole

SCALE: 1" = 5'-0" • DISTURBED SAMPLE ▮ UNDISTURBED SAMPLE

UNIVERSAL **GEOTECHNIQUE** LIMITED

SOIL MECHANICS LABORATORY

BOREHOLE LOG

PROJECT Anne Street Overpass, Barrie, Ontario. ORDER NO. T.227/57

CLIENT Department of Highways, Ontario.

BOREHOLE NO. BH.5 DIAMETER 2-1/2" CASING 2-1/2"

BOREHOLE LOCATION See Plan INCLINATION Vertical BEARING ——

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
	771.38			Zero	0.0m		
Firm grey brown sand, gravel and little clay. Probably FILL.	235.1m		• 1			25	Moist
do With some organic matter.			• 2	Free Water		26	Moist
	233.0m			7'-0"	2.1m		
Firm grey brown fine to coarse SAND with fine to medium generally subrounded GRAVEL.			• 3			26	Wet No dry strength.
	231.4m			12'-0"	3.7m		
Firm brown sandy SILT with some gravel and clay bands.			• 4			24	Moist Low to medium dry strength.
Dense grey brown fine to medium calcareous SAND.			• 5			39	Moist Low dry strength.
do	227.5m		• 6 End of Borehole	25'-1"	7.6m	39 (7")	Wet Low dry strength.

SCALE: 1" = 5'-0" • DISTURBED SAMPLE ▮ UNDISTURBED SAMPLE

UNIVERSAL **GEOTECHNIQUE** LIMITED

SOIL MECHANICS LABORATORY

BOREHOLE LOG

PROJECT Anne Street Overpass, Barrie, Ontario. ORDER NO. T.227/57

CLIENT Department of Highways, Ontario.

BOREHOLE NO. BH.6 DIAMETER 2-1/2" CASING 2-1/2"

BOREHOLE LOCATION See Plan INCLINATION Vertical BEARING ——

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
	772.48			Zero	0.0m		
Firm grey sand and black organic matter. Probably FILL.	235.5m		• 1			12	Moist
Firm grey and iron-stained yellow sand, little clay, Probably FILL.			• 2	Free Water		29	Moist
Dense medium to coarse calcareous SAND and fine to medium generally subrounded GRAVEL.			• 3			39	Wet No dry strength.
	231.5m			13'-0"	4.0m		
Very stiff brown sandy silty calcareous CLAY with fine to medium subangular to subrounded gravel.			• 4			30	Moist High dry strength.
Firm grey brown fine to coarse SAND and fine to medium subangular to subrounded GRAVEL.			• 5			25	Wet No dry strength.
Dense grey brown fine to medium calcareous SAND with generally subrounded gravel.	227.7m		• 6 End of Borehole	25'-6"	7.8m	37	Moist Low dry strength.

SCALE: 1" = 5'-0" • DISTURBED SAMPLE ▮ UNDISTURBED SAMPLE

Golder Associates Ltd.

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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT SUNNIDALE ROAD UNDERPASS STRUCTURE SITE 30-173 HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11 G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

Submitted to:

URS Cole, Sherman
75 Commerce Valley Drive East
Thornhill, Ontario
L3T 7N9

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January 2002 001-1143F-13

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PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
SUNNIDALE ROAD UNDERPASS
STRUCTURE SITE 30-173
HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

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Lists of Abbreviations and Symbols
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LIST OF DRAWINGS
Drawing 1 Sunnidale Road Underpass, Highway 400, Borehole Location Plan

LIST OF FIGURES
Figure 1 Grain Size Distribution Test Result – Silty Sand Till
Figure 2 Grain Size Distribution Test Result – Sand, Some Silt

1.0 INTRODUCTION

Golder Associates Ltd. has been retained by URS Cole, Sherman (Cole, Sherman) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the ultimate widening of Highway 400 from 1 km south of Highway 89, northerly 30 km to Highway 11, in Simcoe County, Ontario. Foundation engineering services are required for the widening and / or replacement of eighteen existing overpass and underpass structures, as well as five structural culverts.

This report addresses the replacement of the Sunnidale Road underpass structure in Barrie, Ontario. A subsurface investigation has been carried out, in which two boreholes were advanced, and in-situ and laboratory testing was conducted, to determine the subsurface conditions at the site for this preliminary design study.

The terms of reference for the scope of work are outlined in Golder Associates' Proposal No. P01-1192, dated June 2000.

2.0 SITE DESCRIPTION

The existing Sunnidale Road underpass is located about 1.5 km north of the Dunlop Street (Simcoe Road 90, formerly Highway 90) interchange and 800 m south of the Bayfield Street (Highway 26) interchange, in Barrie, Ontario. The MTO has designated this underpass as Structure Site 30-173.

In the immediate vicinity of the structure site, Highway 400 has been constructed in a cut up to 6 m deep. The Highway 400 grade is at about Elevation 248.5 m to 249 m, rising toward the north. The Sunnidale Road grade rises westward, from about Elevation 255 m to 257 m over Highway 400.

The existing single-span underpass was constructed in the early 1950s under Contracts 50-11 and 50-66. According to the general layout drawing for the existing structure, which was provided by Morrison Hershfield (the structural designers for this preliminary study), the abutments and associated wing walls and retaining walls are supported on spread footings which are founded at about Elevation 247.2 m.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at this site in February 2001, at which time two boreholes were drilled. Boreholes B13-1 and B13-2 were advanced on the west and east sides of Highway 400, respectively, from Sunnidale Road grade. The boreholes were extended to between 12.5 m and 13.5 m depth, about 5 m to 6 m below the Highway 400 grade.

The investigation was carried out using a truck-mounted D-90 drill rig supplied and operated by Master Soil Investigations Ltd. of Weston, Ontario. Both boreholes were advanced using hollow stem augers. Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. The water levels in the open boreholes were observed throughout the drilling operations, and a piezometer was installed in Borehole B13-2 to permit monitoring of the groundwater level at this location.

The field work was supervised on a full-time basis by a member of our staff who located the boreholes in the field, directed the drilling, sampling, and in-situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder Associates' laboratory in Mississauga for further examination. Index and classification tests consisting of water content determinations, Atterberg Limits testing and grain size distribution analyses were carried out on selected soil samples.

The borehole locations and elevations were surveyed by Callon Dietz, Ontario Land Surveyors. The borehole elevations are referenced to geodetic datum, and the northing and easting co-ordinates are referenced to the MTM NAD83 survey system. The borehole locations, together with elevations and northing and easting co-ordinates, are shown on the attached Drawing 1.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

This 30 km section of Highway 400 traverses, from south to north, the following physiographic regions as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, Third Edition, 1984): the Simcoe Lowlands; the Peterborough Drumlin Field; a second lobe of the Simcoe Lowlands; and the Simcoe Uplands. Along Highway 400, the Simcoe Lowlands are present from the southern limit of the project to just south of Innisfil Creek, and again from Essa Road (Simcoe Road 30, formerly Highway 27) to about 1 km north of Dunlop Street (Simcoe Road 90, formerly Highway 90). The Peterborough Drumlin Field occupies the belt between these lobes of the Simcoe Lowlands, extending from just south of Innisfil Creek, which is located about 1 km north of Highway 89, to Essa Road. The Simcoe Uplands extend from about 1 km north of Dunlop Street to beyond the northern limit of the project at Highway 11.

The two sections where Highway 400 crosses the Simcoe Lowlands consist of two lobes of a sand plain which include the shores of Kempenfelt Bay, the Nottawasaga River and Innisfil Creek. The surficial soils of these sections of the Simcoe Lowlands consist primarily of sand, although silt, clay or peat may be found in low-lying areas.

The surficial soils in the Peterborough Drumlin Field consist primarily of gravelly sand till or sand and gravel deposits. Drumlins (glacially-shaped hills) are more frequent in the southern portion of the section of the Peterborough Drumlin Field traversed by Highway 400. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins.

The surficial soils in the Simcoe Uplands physiographic region, in which the Sunnidale Road site is located, are primarily sandy silt till deposits, known to contain boulders. Low-lying areas may be infilled with shallow sand and gravel deposits, which are shoreline deposits of a former glacial lake that once flooded the area.

4.2 Site Stratigraphy

The subsurface soil and groundwater conditions encountered in the boreholes and the results of in-situ and laboratory testing are given on the Record of Borehole sheets and Figures 1 and 2. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

Boreholes B13-1 and B13-2 were advanced on the west and east sides of Highway 400, from Sunnidale Road grade. The locations and ground surface elevations for these boreholes are shown on the attached Drawing 1.

In summary, the soils below the Sunnidale Road approach fill (behind the existing abutment walls) consist of a thin layer of silty sand, overlying silty sand to clayey silt till. In Borehole B13-2, on the east side of Highway 400, the silty sand till is underlain by a deposit of very dense silty sand to sand. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Approach Fill

The boreholes were advanced through the approach fill behind the abutments. This fill extended from just below the pavement structure to approximately 7 m depth, to about Elevation 249.5 m on the west side of Highway 400, and about Elevation 248 m on the east side of the highway.

The approach fill is comprised of sand and gravel; the presence of cobbles was noted in Borehole B13-1. The measured Standard Penetration Test (SPT) 'N' values ranged from 9 to 62 blows, but were typically between 10 and 30 blows per 0.3 m of penetration, indicating that the fill has a predominantly compact relative density.

4.2.2 Surficial Silty Sand

A 0.7 m to 1.1 m thick layer of silty sand was encountered below the approach fill. This silty sand contains trace clay and gravel, and trace quantities of wood fragments and organics. It is considered that this deposit could represent a fill or reworked native soil, due to the presence of wood fragments in the samples. The samples were moist, with measured natural water contents of 8 and 10 per cent. The measured SPT 'N' values were 19 and 31 blows, indicating that this silty sand has a compact relative density.

4.2.3 Silty Sand to Clayey Silt Till

The surficial silty sand layer overlies a deposit of silty sand till. The top of this deposit was encountered at about Elevation 248.5 m and 247.5 m in the boreholes on the west and east sides of the highway, respectively. The silty sand till is about 1.5 m to 2 m thick at the borehole

locations; on the west side of the highway, the silty sand till grades to a clayey silt till, while on the east side of the highway, the silty sand till overlies a deposit of silty sand to sand.

The silty sand till contains trace clay and trace to some gravel. A grain size distribution test result for a representative sample of this till is shown on Figure 1. This silty sand till was moist, with two measured water contents of 7 and 8 per cent. The measured SPT 'N' values ranged from 21 to greater than 100 blows, but were typically over 80 blows per 0.3 m of penetration. The relative density therefore ranges from compact to very dense, but is predominantly very dense.

The clayey silt portion of the till, encountered below about Elevation 246.5 m in Borehole B13-1 on the west side of the highway, contains trace to some sand and gravel. The measured natural water content was 9 per cent, and Atterberg Limits testing measured plastic and liquid limits of 11 and 14 per cent, and a plasticity index of 3 per cent. The till is therefore inorganic and of low plasticity. It has a hard consistency, with measured SPT 'N' values of greater than 100 blows per 0.3 m of penetration.

4.2.4 Silty Sand to Sand, Some Silt

A silty sand to sand layer was encountered below the silty sand till in the borehole on the east side of the highway. The surface of this layer was found to be at about Elevation 246 m. The silty sand to sand was not fully penetrated by the boring, which was terminated at Elevation 241.7 m; it is at least 4.3 m thick. This material may represent an interlayer or lens, of limited lateral extent, within the till deposit, as it was not encountered within the investigated depth on the west side of the highway.

The layer ranges in composition from silty sand containing trace gravel, to sand containing some silt and trace gravel; in one sample, thin silty clay layers were noted within the cohesionless soil. A grain size distribution test result is shown on Figure 2. The samples were wet, with measured natural water contents ranging from 17 to 19 per cent. The measured SPT 'N' values ranged from 69 to greater than 100 blows per 0.3 m of penetration, indicating that this material has a very dense relative density.

4.3 Groundwater Conditions

Borehole B13-1 on the west side of the highway was dry on completion of the drilling operations. In Borehole B13-2 on the east side of the highway, in which wet silty sand was encountered

below Elevation 246 m, the water level on completion of drilling was at 11 m depth, or about Elevation 244 m.

The groundwater level in the piezometer in Borehole B13-2, located east of Highway 400, was measured in March 2001 at about Elevation 246 m, about 9 m below the Sunnidale Road grade and 2.5 m below the Highway 400 grade. This is approximately coincident with the top of the silty sand to silt layer encountered in this boring.

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

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
SUNNIDALE ROAD UNDERPASS
STRUCTURE SITE 30-173
HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

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

It is understood that Highway 400 will be widened from its existing six-lane configuration to an interim configuration of eight lanes, and an ultimate configuration of ten lanes, and that an alternative for a twelve-lane express / collector system is under consideration between Molson Park Drive and Duckworth Street in Barrie. Throughout the project length, it is expected that the existing highway platform will be widened by between 13 m and 30 m. Replacement of the existing Sunnidale Road underpass structure will therefore be necessary.

Based on the available general layout drawing for the existing single-span structure, the abutments and associated wing walls and retaining walls are supported on spread footings which are founded at about Elevation 247.2 m. Highway 400 has been constructed in cut, with its grade at about Elevation 248.5 m to 249 m, rising toward the north. The Sunnidale Road grade rises westward, from about Elevation 255 m to 257 m over Highway 400.

5.2 Bridge Foundation Options

The soils below the embankment approach fill (encountered behind the abutments in Boreholes B13-1 and B13-2) consist of a thin layer of silty sand (probably fill), overlying silty sand till to clayey silt till. On the east side of the highway, the silty sand till is underlain by a layer of very dense silty sand to sand. It is noted that the native soils above the Highway 400 cut were not investigated during this stage of drilling. The groundwater level associated with the lower, very dense silty sand to sand layer was found to be at about Elevation 246 m in March 2001, approximately 2.5 m to 3 m below the Highway 400 cut grade.

Based on these subsurface conditions, the new structure can be founded on spread footings placed in the dense silty sand till deposit. Consideration could also be given to the use of perched abutments, founded on spread footings placed on a compacted granular pad within the approach embankments. Alternatively, deep foundations, such as driven steel H-piles, could be adopted at this site. The soil conditions at this site are favourable for the use of an integral abutment replacement structure.

Preliminary recommendations for spread footings, including perched abutments, and for driven steel H-pile foundations are provided in the following sections. It should be noted that the boreholes put down during this preliminary phase of field work are located within the excavation area for the existing bridge and encountered fill materials extending to below the Highway 400 cut level. If perched footings or perched pile caps for deep foundations are considered viable options, the native subsoil conditions between the Sunnidale Road and Highway 400 grades will have to be investigated during the detailed design stage.

5.3 Spread Footings

For preliminary design of the abutment and pier foundations, spread footings may be placed on the silty sand till at or below Elevation 248 m on the west side of the highway, and at or below Elevation 247 m on the east side of the highway; for any pier footings, a design founding level of Elevation 247 m should be assumed at this preliminary design stage. A minimum soil cover of 1.5 m should be provided at all footing locations. Any associated wing wall or retaining wall footings may be stepped upward within the till away from the abutments, such that a minimum soil cover of 1.5 m is maintained above the underside of the footings.

Alternatively, consideration could be given to the use of abutment footings perched within the approach embankments.

5.3.1 Axial Geotechnical Resistance

Spread footings placed on a properly prepared base within the silty sand till deposit at the preliminary design elevations given above may be designed for a factored geotechnical resistance at Ultimate Limit States (ULS) of 800 kPa, assuming a 3 m wide footing. The settlement of footings founded on the silty sand till soils will be dependent on the footing size and configuration, and on the applied loads. The majority of the settlement should occur during construction. For preliminary design purposes, the geotechnical resistance at Serviceability Limit States (SLS) may be taken as 500 kPa provided proper groundwater control is exercised during

construction to prevent reduction of the existing in situ density of the foundation subsoil. The geotechnical resistances at ULS and SLS will have to be reviewed during detailed design, once the footing size, configuration and loadings are known.

For spread footings placed within the approach embankments on a compacted Granular 'A' core, a factored geotechnical resistance at ULS of 900 kPa may be assumed for preliminary design. The geotechnical resistance at SLS will depend on the thickness of Granular 'A' and the consistency and thickness of the underlying soils; a value of 350 kPa may be assumed for preliminary design. These resistances will have to be confirmed during detailed design, once the composition and consistency of the upper soils at the site are confirmed.

The geotechnical resistances provided herein are given under the assumption that the loads will be applied perpendicular to the surface of the footings; where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the Ontario Highway Bridge Design Code (OHBDC).

5.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footing and the subsoils should be calculated in accordance with Section 6-8.4.3 of the OHBDC. The angle of friction between the concrete and the undisturbed, very dense silty sand till founding soils should be taken as 24 degrees; the corresponding coefficient of friction, tan δ, would then be 0.45. Where "perched" abutment footings are adopted, the angle of friction between the concrete footings and the compacted Granular 'A' pad should be taken as 30 degrees; the corresponding coefficient of friction would be 0.58.

5.3.3 Frost Protection

All footings should be provided with a minimum of 1.5 m of soil cover for frost protection.

5.4 Driven Steel H-Piles

Consideration could be given to supporting the abutments of the replacement structure on steel H-piles driven to found within the very dense silty sand deposit on the east side and the very dense/hard till deposits on the west side. The surface of these deposits was encountered in the

borings at about Elevation 248.5 m and 246.5 m on the west and east sides of the highway, respectively (only slightly below the Highway 400 grade). Based on the nature of these deposits, a suitable driving resistance may be achieved at about Elevation 246 m on the west side, within the hard clayey silt till. On the east side, suitable driving resistance will likely not be achieved until the piles have advanced well into the sand deposit. For preliminary design, pile tip levels at Elevation 246 m and 242 m may be assumed for the west and east sides, respectively. If necessary, the pile caps could be perched within the approach embankments above Highway 400 grade in order to achieve the required length for abutment flexibility purposes.

5.4.1 Axial Geotechnical Resistance

For preliminary design of deep foundations on the west side of the highway, the factored axial resistance at ULS for steel HP 310 x 110 H-piles driven to found within the very dense silty sand till / hard clayey silt till may be taken as 1,600 kN. The axial resistance at SLS for 25 mm of settlement may be taken as 1,400 kN.

On the east side of the highway, where at least 4.5 m of water-bearing silty sand to sand was encountered below a thin till layer in Borehole B13-2, the factored axial resistance at ULS for driven steel HP 310 x 110 piles may be taken as 800 kN. The axial resistance at SLS for 25 mm of settlement may be taken as 800 kN. It is noted that this silty sand to sand layer may represent an interlayer, of limited lateral extent, within the till. The presence or absence of this interlayer at the east abutment location, its thickness and the nature and consistency of the underlying soils, will require investigation during the detailed design stage. This information should be used to confirm / refine the load carrying capacity.

To achieve the above design resistances, the piles should be driven to at least the above design tip elevations. Provision should be made to re-tap selected piles to confirm the set after adjacent piles have been driven, in accordance with MTO's current Special Provision.

5.4.2 Resistance to Lateral Loads

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

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

$$k_h = \frac{n_h z}{B}$$

where
n_h is the constant of subgrade reaction
z is the depth (m)
B is the pile diameter (m)

Based on borehole data obtained to date, the subsoils in front of the piles will consist of sand and gravel fills, sand till and silty sand deposits. For the compact to dense sand and gravel fill above Elevation 249 m, the range in value of n_h may be taken as 7 MPa/m to 20 MPa/m in the structural analysis. The range in value of n_h may be taken as 10 MPa/m to 25 MPa/m between Elevations 249 m and 246 m, and as 6 MPa/m to 12 MPa/m below Elevation 246 m. These values will have to be confirmed following the detailed design stage of the subsurface investigation.

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

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

5.4.3 Frost Protection

All pile caps should be provided with 1.5 m of soil cover for frost protection.

5.5 Lateral Earth Pressures

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

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the abutments and walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00

- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the abutment wall, in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be used in accordance with OPSS 501.06.

- The granular fill may be placed either in a zone with width equal to at least 1.5 m behind the back of the stem (Case I from OHBDC Figure 6-7.4.1) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II from OHBDC Figure 6-7.4.4).

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

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

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

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

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

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

5.6 Design of Permanent Cut Slopes

In the vicinity of the Sunnidale Road underpass, the Highway 400 cut is up to about 6 m in depth. Boreholes B13-1 and B13-2 were advanced through the approach fill behind the existing abutments, and so no borehole information is available regarding the native soils which will comprise the permanent cut slopes along the east and west sides of the widened Highway 400. Based on regional geological mapping, it is anticipated that the cut will be formed in silty sand to clayey silt till, similar to that encountered below the highway grade.

Based on the topographic information on the Engineering and Title Records plates and on site reconnaissance, the existing cut slopes are formed at a gradient of 1.7 to 2.5 horizontal to 1 vertical (1.7H:1V to 2.5H:1V). For preliminary design purposes, a gradient no steeper than 2H:1V may be assumed for the new permanent cut slopes. This design recommendation will have to be confirmed during the detailed design stage of the subsurface investigation program.

The groundwater level measured in Borehole B13-2 on the east side of the highway is at about Elevation 246 m, about 2.5 m below the Highway 400 grade. Based on this preliminary information, no major subsurface seepage concerns are anticipated for the permanent cut slopes. However, surficial erosion protection will be required to maintain stability over the long term.

5.7 Design and Construction Considerations

5.7.1 Groundwater Control

Based on the available groundwater level readings, foundation excavations for footings and/or pile caps should be above the groundwater level. However, provision should be made for appropriate groundwater and surface water control during wet periods to prevent loosening and loss of in-situ support within the foundation subsoil.

The silty sand till soils in which the footing excavations will be formed are susceptible to disturbance from ponded water and construction traffic. Provision should be made for a lean concrete mat placed at subgrade level, to protect the soils from such disturbance.

5.7.2 Excavation

Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. If spread footings or pile caps placed below the Highway 400 grade are selected for the replacement structure, the footing excavations will extend a minimum of 1.5 m below lowest surrounding grade, through compact silty sand and into compact to very dense silty sand till. The silty sand would be classified as Type 3 soil, while the silty sand till soils below Highway 400 grade would be classified as Type 2 soil. It is recommended that temporary open-cut slopes, where adopted, be maintained no steeper than 1 horizontal to 1 vertical (1H:1V).

Where space restrictions dictate, footing excavations could also be carried out within a braced excavation.

5.7.3 Settlement

Foundation loading will result in deformation of the ground and consequent settlement of the foundation elements and superstructure. Differential settlement could occur between the abutments and pier locations, depending on the variability and consistency/relative density of the foundation soils, and on the types of foundations selected. The potential for differential settlement should be reassessed during the detailed design stage, once the proposed bridge configuration is established.

5.7.4 Obstructions

Cobbles were observed within the approach fill but not within the native subsoils in the boreholes advanced at this site. However, it is noted that cobbles and boulders are inherent in glacial soils, and should therefore be expected during footing excavation and temporary shoring system installation, if such a system is required at the site. Where boulders are encountered within footing excavations, they should be removed and the sub-excavated areas should be backfilled with well-compacted Granular 'A' or lean concrete.

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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

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 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 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	c_u, s_u 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 (LV-laboratory vane test)
γ	unit weight

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

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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 δ
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

Ministry of Transportation
Ontario

Foundation Design

RECORD OF BOREHOLE No B13-1 1 OF 1 METRIC

PROJECT 001-1143F
W.P. 30-95-00
DIST SW HWY 400
DATUM Geodetic
LOCATION N 4916980.9; E 288547.3
BOREHOLE TYPE 108mm ID HOLLOW STEM AUGERS
DATE Feb.6/2001
ORIGINATED BY PKS
COMPILED BY LCC
CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L / WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
256.8	GROUND SURFACE									
0.0	Asphalt									
0.2	Sand and Gravel, some cobbles (Fill) Compact to dense Brown Moist					256				
		1	SS	32		255				
						254				
		2	SS	21		253				
						252				
						251				
		3	SS	9		250				
249.5 / 7.3	Silty Sand, trace gravel Compact Brown Moist	4	SS	19		249		o		
248.4 / 8.4	Silty Sand, trace clay, trace gravel (Till) Very dense Brown Moist	5	SS	104		248				
		6	SS	100/.15				o		4 57 30 9
		7	SS	101		247				
246.3 / 10.5	Clayey Silt, trace to some sand and gravel (Till) Hard Brown Moist	8	SS	102		246		o H		
						245				
244.4 / 12.4	END OF BOREHOLE	9	SS	100/.18						

Notes:
1. Borehole dry on completion of drilling.
2. Borehole backfilled with bentonite and surface cold patch on sand bedding.

+³, ×³: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

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Ministry of Transportation
Ontario

Foundation Design

RECORD OF BOREHOLE No B13-2 1 OF 1 METRIC

PROJECT 001-1143F
W.P. 30-95-00
DIST SW HWY 400
DATUM Geodetic
LOCATION N 4916967.4; E 288585.1
BOREHOLE TYPE 108mm ID HOLLOW STEM AUGERS
DATE Feb.6-7/2001
ORIGINATED BY PKS
COMPILED BY LCC
CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L / WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
255.1	GROUND SURFACE									
0.0	Asphalt					255				
0.2	Sand and Gravel (Fill) Compact to very dense Brown Moist					254				
		1	SS	62		253				
		2	SS	11		252				
						251				
		3	SS	15		250				
		4	SS	25		249				
248.2 / 6.9	Silty Sand, trace clay, some gravel, trace wood and organics Dense Brown Moist	5	SS	31		248		o		
247.5 / 7.6	Silty Sand, trace clay, trace to some gravel (Till) Compact to very dense Brown Moist	6	SS	21		247		o		
		7	SS	84						
246.0 / 9.1	Silty Sand to Sand, some silt, trace gravel Very dense Wet Brown	8	SS	69		246		o		0 85 15 0
		9	SS	129		245				
		10	SS	100/.15		244		o		
		11	SS	108		243				
		12	SS	96				o		
	Thin silty clay layers present in Sample 13.	13	SS	110		242				
241.7 / 13.4	END OF BOREHOLE									

Notes:
1. Water level on completion of drilling at 11m depth (Elev.244.1m).
2. Water level in piezometer measured at 9m depth (Elev.246.1m) on March 15, 2001.

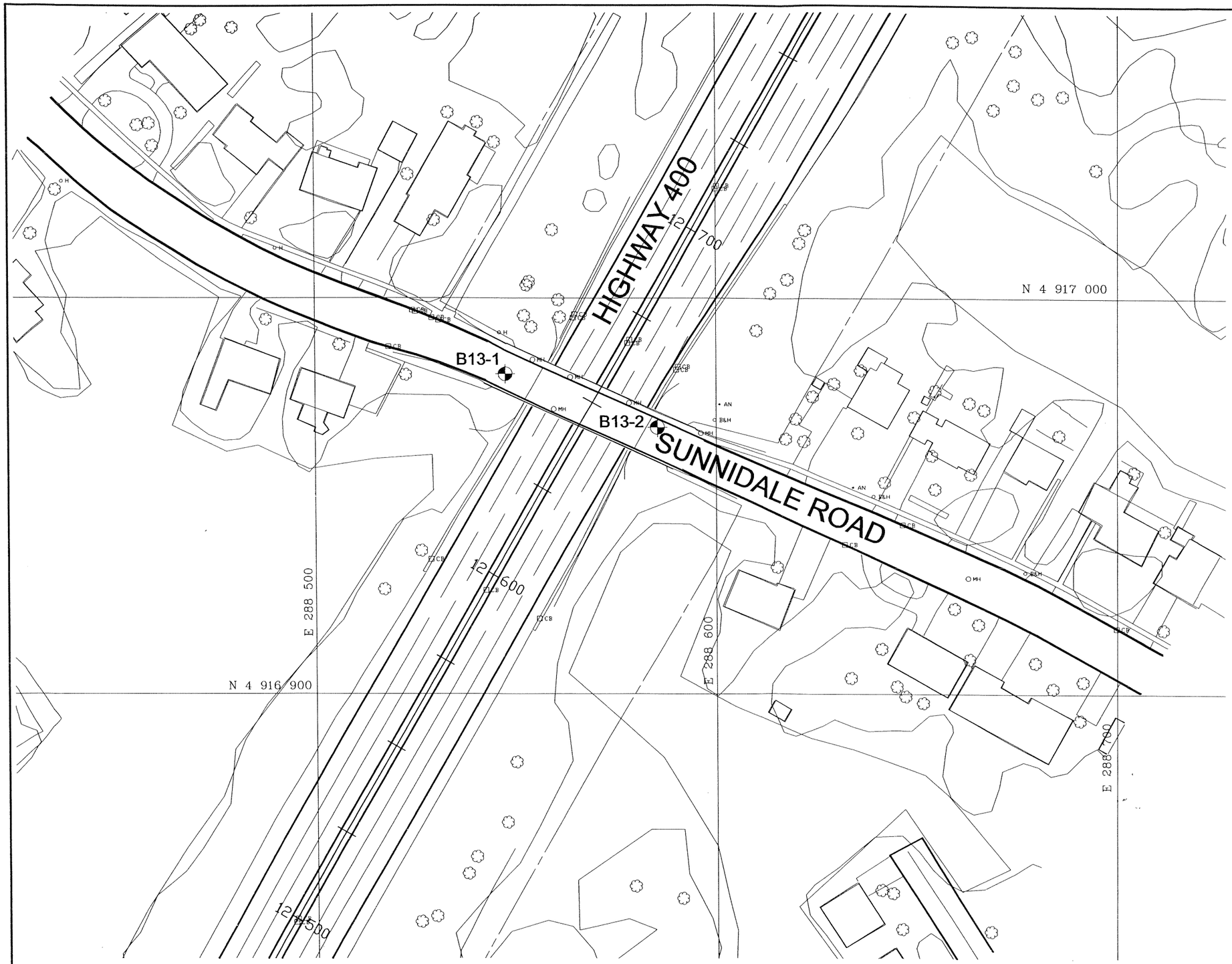
+³, ×³: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

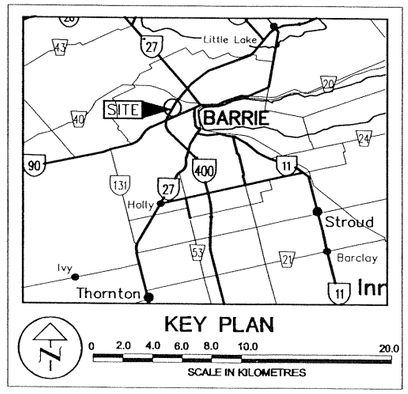
ON_MOT 0011143F.GPJ ON_MOT.GDT 14/1/02

DIST HWY 400 CONT. No. GWP No. 30-95-00	N
SUNNIDALE ROAD UNDERPASS HWY 400 BOREHOLE LOCATION PLAN	SHEET

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA





LEGEND

- Borehole, previous investigation
- Borehole, present investigation

No.	ELEVATION	LOCATION NORTHING	LOCATION EASTING
B13-1	256.8	4,916,980.9	288,547.3
B13-2	255.1	4,916,967.4	288,585.1

REFERENCE
This drawing was created from digital file "50210.dwg" provided by URS Cole Sherman

0 10 20 30 40 50 100m
SCALE IN METRES

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

NO.	DATE	BY	REVISION

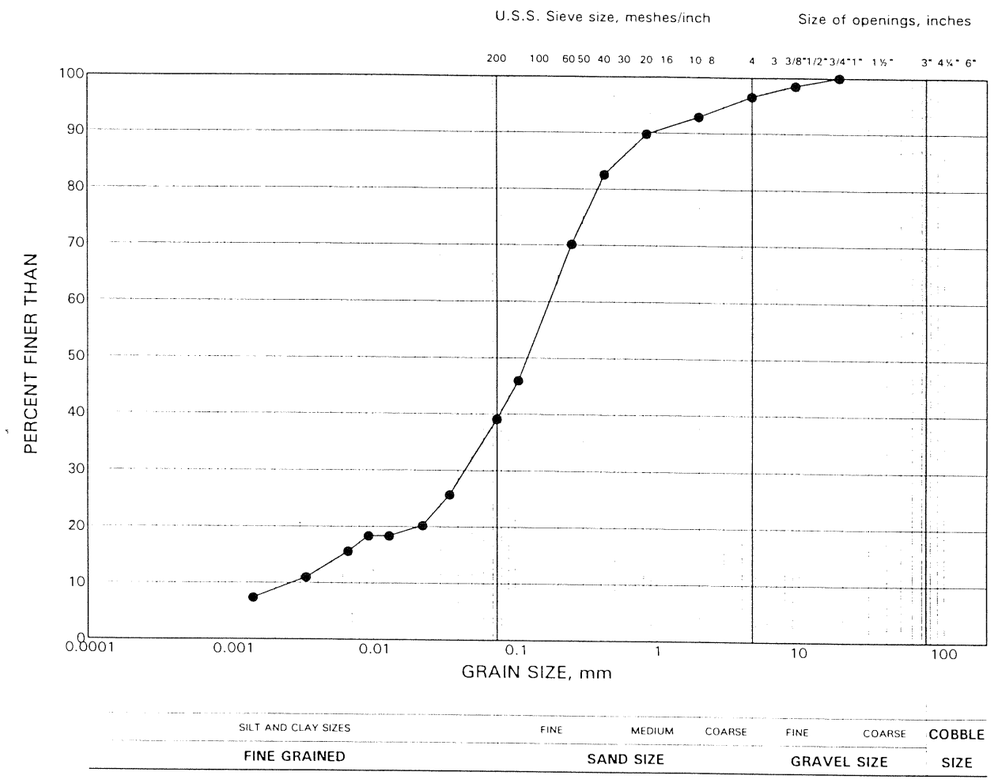
Geocres No.

HWY. No. 400		PROJECT NO.: 001-1143F	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-173
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

P1143F18.DWG

GRAIN SIZE DISTRIBUTION TEST RESULT
Silty Sand Till

FIGURE 1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	B13-1	6	247.5

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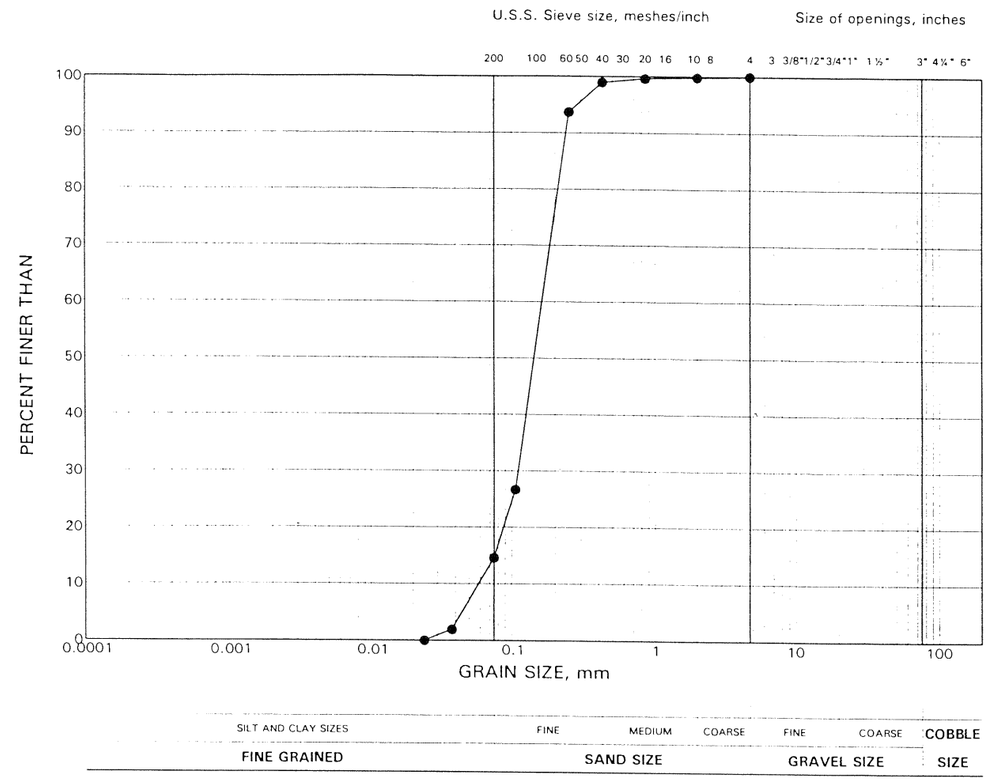
GRAIN SIZE DISTRIBUTION TEST RESULT
Sand, some silt

FIGURE 2



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	B13-2	8	245.6

Project 001-1143F

Golder Associates

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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT BAYFIELD STREET (HIGHWAY 26) UNDERPASS STRUCTURE SITE 30-172 HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11 G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

Submitted to:

URS Cole, Sherman
75 Commerce Valley Drive East
Thornhill, Ontario
L3T 7N9

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February 2002 001-1143F-14

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Appendix A Records of Boreholes and Test Results – 1979 Investigation

PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
BAYFIELD STREET (HIGHWAY 26) UNDERPASS
STRUCTURE SITE 30-172
HIGHWAY 400 WIDENING FROM 1 KM SOUTH
OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

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LIST OF DRAWINGS

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LIST OF APPENDICES

Appendix A Records of Boreholes and Test Results – 1979 Investigation

1.0 INTRODUCTION

Golder Associates Ltd. has been retained by URS Cole, Sherman (Cole, Sherman) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the ultimate widening of Highway 400 from 1 km south of Highway 89, northerly 30 km to Highway 11, in Simcoe County, Ontario. Foundation engineering services are required for the widening and / or replacement of eighteen existing overpass and underpass structures, as well as five structural culverts.

This report addresses the replacement of the Bayfield Street (Highway 26) underpass structure in Barrie, Ontario. Existing subsurface data for this site from an investigation conducted by the Ministry of Transportation, Ontario in 1979 [*"Foundation Investigation Report for Highway 26/27 (Bayfield Street) Underpass Widening"*, dated August 1979 – GEOCRES File No. 31D-268] were used to determine the subsurface conditions for this preliminary design study.

The terms of reference for the scope of work are outlined in Golder Associates' Proposal No. P01-1192, dated June 2000.

2.0 SITE DESCRIPTION

The existing Bayfield Street (Highway 26) underpass is located about 2.5 km north of Dunlop Street (Simcoe Road 90, formerly Highway 90) and 5 km south of Highway 11, in Barrie, Ontario. The MTO has designated this underpass as Structure Site 30-172.

Highway 400 has been constructed in cut, with its grade at about Elevation 266 m to 267 m at the structure site. The Bayfield Street grade is at about Elevation 273.5 m.

According to the general layout drawing for the existing structure, which was provided by Morrison Hershfield (the structural designers for this preliminary study), the abutments and associated wing walls and retaining walls are supported on spread footings. The spread footings are founded at about Elevation 264.6 m and 264.9 m at the west and east abutments, respectively.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at this site by the MTO in June 1979. At that time, two boreholes were advanced on the south side of the existing structure, associated with the then-proposed southward widening of the existing underpass to accommodate widening of Bayfield Street. Boreholes 1 and 2 were advanced to between 7.5 m and 8 m depth below Highway 400 grade, to about Elevation 258.5 m.

Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. The groundwater conditions in the open borehole were observed following the drilling operations. Laboratory index and classification testing, consisting of natural moisture contents and grain size distributions, was carried out on selected soil samples.

The borehole locations and elevations, referenced to the geodetic datum, were established by the MTO. Approximate northing and easting co-ordinates consistent with the MTM NAD83 survey system, currently in use on this project, have been determined by Golder Associates based on the borehole locations given in the 1979 report. The approximate borehole locations and northing and easting co-ordinates are shown on the attached Drawing 1.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

This 30 km section of Highway 400 traverses, from south to north, the following physiographic regions as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, Third Edition, 1984): the Simcoe Lowlands; the Peterborough Drumlin Field; a second lobe of the Simcoe Lowlands; and the Simcoe Uplands. Along Highway 400, the Simcoe Lowlands are present from the southern limit of the project to just south of Innisfil Creek, and again from Essa Road (Simcoe Road 30, formerly Highway 27) to about 1 km north of Dunlop Street (Simcoe Road 90, formerly Highway 90). The Peterborough Drumlin Field occupies the belt between these lobes of the Simcoe Lowlands, extending from just south of Innisfil Creek, which is located about 1 km north of Highway 89, to Essa Road. The Simcoe Uplands extend from about 1 km north of Dunlop Street to beyond the northern limit of the project at Highway 11.

The two sections where Highway 400 crosses the Simcoe Lowlands consist of two lobes of a sand plain which include the shores of Kempenfelt Bay, the Nottawasaga River and Innisfil Creek. The surficial soils of these sections of the Simcoe Lowlands consist primarily of sand, although silt, clay or peat may be found in low-lying areas.

The surficial soils in the Peterborough Drumlin Field consist primarily of gravelly sand till or sand and gravel deposits. Drumlins (glacially-shaped hills) are more frequent in the southern portion of the section of the Peterborough Drumlin Field traversed by Highway 400. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins.

The surficial soils in the Simcoe Uplands physiographic region, in which the Bayfield Street underpass site is located, are primarily sandy silt till deposits, known to contain occasional boulders. Low-lying areas may be infilled with shallow sand and gravel deposits, which are shoreline deposits of a former glacial lake that once flooded the area.

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes are given on the Record of Borehole sheets and figures contained in Appendix A. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

Boreholes 1 and 2 were advanced on the south side of the existing underpass structure, from Highway 400 grade. The approximate locations and ground surface elevations for these boreholes are shown on the attached Drawing 1.

In summary, the soils below the Highway 400 pavement structure at this site consist of very stiff to hard clayey silt till. A detailed description of the subsurface conditions encountered in the boreholes is provided in the following section.

4.2.1 Pavement Structure

No samples were obtained in the upper 0.7 m of both 1979 boreholes. The borehole records identify the surficial 0.3 m as asphalt pavement; however, the lower portion of this pavement structure may be granular fill.

4.2.2 Clayey Silt Till

The boreholes encountered a clayey silt till deposit below the pavement structure at 0.3 m depth. This deposit extended to the maximum depth investigated (approximately 8 m). The clayey silt till contains a significant fraction of sand, and trace quantities of gravel. The presence of cobbles within the till deposit is noted on both borehole records. An envelope of grain size distribution test results for samples of the till is shown on the figure in Appendix A.

The measured natural moisture contents of samples of the clayey silt till varied from 7 to 9 per cent.

In the upper 1 m of the till deposit, the measured Standard Penetration Test (SPT) 'N' values were 13 and 17 blows per 0.3 m of penetration, indicating that this portion of the till has a stiff to very stiff consistency. Below Elevation 265 m, the measured SPT 'N' values ranged from 42 to greater than 100 blows, but were generally between 60 and greater than 100 blows per 0.3 m of penetration; this portion of the deposit has a hard consistency. A dynamic cone penetration test adjacent to Borehole 2 met refusal at about 2.5 m depth (Elevation 263.8 m) at a resistance of 100 blows per 0.08 m of penetration.

4.3 Groundwater Conditions

The 1979 report indicates that no water table was encountered during the borehole investigation, which was carried out in June of that year. The 1979 report further notes that the groundwater level is likely below the base of the boreholes (i.e. below Elevation 258 m). However, no piezometers were installed in the 1979 boreholes to permit monitoring of the groundwater level within the clayey silt till deposit.

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

GOLDER ASSOCIATES LTD.

for Lisa C. Coyne, P.Eng.,
Geotechnical Engineer

Anne S. Poschmann, P.Eng.,
Principal

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

LCC/ASP/FJH/clg
N:\ACTIVE 1100 001-1143F 2002 RPT14-02FEB-BAYFIELD DOC

PART B

PRELIMINARY FOUNDATION DESIGN REPORT
BAYFIELD STREET (HIGHWAY 26) UNDERPASS STRUCTURE SITE 30-172
HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides preliminary foundation design recommendations for the replacement of the existing Bayfield Street (Highway 26) underpass structure, associated with the widening of Highway 400. The recommendations are preliminary only and are based on interpretation of the factual data obtained from a limited number of boreholes advanced during a 1979 subsurface investigation at this site. The interpretation and recommendations provided are intended for planning purposes only, to provide the information necessary at this stage of the study. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the planning of the project. Further foundation investigation will be required at this structure site as part of the detailed design stage of the project.

It is understood that Highway 400 will be widened from its existing six-lane configuration to an interim configuration of eight lanes, and an ultimate configuration of ten lanes, and that an alternative for a twelve-lane express / collector system is under consideration between Molson Park Drive and Duckworth Street in Barrie. Throughout the project length, it is expected that the existing highway will be widened by between 13 m and 30 m. Replacement of the existing Bayfield Street (Highway 26) underpass structure will therefore be necessary.

Based on the general layout drawing for the existing single-span structure, the abutments and associated retaining and wing walls are supported on spread footings which are founded at about Elevation 264.6 m and 264.9 m on the west and east sides of the highway, respectively. Highway 400 has been constructed in cut, with its grade at about Elevation 266 m to 267 m at the structure site. The Bayfield Street grade is at about Elevation 273.5 m.

5.2 Bridge Foundation Options

The soils below the Highway 400 level consist of a deposit of stiff to hard clayey silt till containing sand and trace quantities of gravel; cobbles were noted on the borehole records within this till deposit. It is noted that the soils above the Highway 400 cut were not investigated during the 1979 subsurface drilling program.

Based on these subsurface conditions, it is recommended that the new structure be founded on spread footings placed on the hard clayey silt till deposit. Consideration could also be given to the use of perched abutments, founded on spread footings placed on a compacted granular pad within the approach embankments.

Alternatively, if integral abutments are under consideration for the replacement structure, the abutments could be supported on steel H-piles driven to found within the hard clayey silt till deposit. In this case, it is assumed that the pile cap would be perched above the existing Highway 400 grade. It should be noted that heavy driving will be encountered due to the presence of cobbles (as encountered in the 1979 borehole investigation) and boulders within the till deposit. In this regard, depending on the proposed pile cap level, some preaugering may be needed to achieve the required pile length.

It should be noted that the boreholes put down during this preliminary phase of field work were drilled from the Highway 400 cut level. If perched footings or integral abutments supported on deep foundations are considered viable options, the subsoil conditions between the Bayfield Street level and the Highway 400 grade will have to be determined during the detailed design stage.

Preliminary recommendations for spread footings, including perched abutments, and for driven steel H-pile foundations are provided in the following sections.

5.3 Spread Footings

For preliminary design of the abutment and pier foundations, spread footings may be placed at a design founding level of Elevation 264.5 m, to be founded on the hard clayey silt till deposit. Any associated wing wall or retaining wall footings may be stepped upward away from the abutments such that a minimum soil cover of 1.5 m is maintained above the underside of the footings.

Alternatively, consideration could be given to the use of abutment footings perched within the approach embankments.

5.3.1 Axial Geotechnical Resistance

Spread footings placed on the properly prepared clayey silt till deposit at the design elevation given above may be designed for a factored geotechnical resistance at Ultimate Limit States (ULS) of 1,100 kPa, assuming a 2.5 m wide footing. The settlement of footings founded on the clayey silt till soils will be dependent on the footing size and configuration, and on the applied loads. For preliminary design purposes, the geotechnical resistance at Serviceability Limit States (SLS) may be taken as 800 kPa. The geotechnical resistances at ULS and SLS will have to be reviewed following the detailed design stage of subsurface investigation, once the groundwater conditions at the site are confirmed and the footing size, configuration and loadings are known.

For spread footings placed within the approach embankments on a compacted Granular 'A' core, a factored geotechnical resistance at ULS of 900 kPa may be assumed for preliminary design. The geotechnical resistance at SLS will depend on the thickness of Granular 'A' and the consistency and thickness of the underlying soils; a value of 350 kPa may be assumed for preliminary design. These resistances will have to be confirmed during detailed design, once the composition and consistency of the upper soils at the site (through which the permanent cut will be made) are confirmed.

The geotechnical resistances provided herein are given under the assumption that the loads will be applied perpendicular to the surface of the footings; where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the Ontario Highway Bridge Design Code (OHBDC).

5.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footing and the subsoils should be calculated in accordance with Section 6-8.4.3 of the OHBDC. The angle of friction between the concrete and the undisturbed, hard clayey silt till founding soils should be taken as 24 degrees; the corresponding coefficient of friction, tan δ, would then be 0.45. Where "perched" abutment footings are adopted, the angle of friction between the concrete footings and the compacted Granular 'A' pad should be taken as 30 degrees; the corresponding coefficient of friction would be 0.58.

5.3.3 Frost Protection

The footings should be provided with a minimum of 1.5 m of soil cover for frost protection.

5.4 Driven Steel H-Piles

Consideration could be given to supporting the replacement structure on steel H-piles driven to found within the hard clayey silt till deposit. Based on the results of the 1979 boreholes that were drilled from the Highway 400 cut grade, it is anticipated that an adequate driving resistance may be achieved with pile tip as high as about Elevation 264 m, although it may be possible to advance the piles to about Elevation 260 m before achieving a suitable set. These pile tip levels correspond to depths of approximately 2 m to 6 m below Highway 400 grade and 9.5 m to 13.5 m

below the Bayfield Street grade. Consideration should be given to perching the pile caps above the Highway 400 cut grade in order to maximize the driven pile length.

Because of the hard nature of the till and the presence of cobbles and boulders, hard driving conditions should be anticipated. As discussed in Section 5.7.3, provision must be made in both the Contract Documents and the contractor's methods and equipment to handle these obstructions.

It should be noted that additional borehole investigation will be required at the proposed abutment locations during detailed design in order to determine the composition and consistency of the upper soils through which the piles would be driven, as well as to confirm the nature of the soils at and below the anticipated pile tip level.

5.4.1 Axial Geotechnical Resistance

For preliminary design, a pile founding level of Elevation 260 m should be assumed; however, the design should be checked with a pile tip level at Elevation 264 m to confirm that this shorter pile length would also be feasible. The factored axial resistance at ULS for steel HP 310 x 110 H-piles driven to the design tip elevation may be taken as 1,400 kN. The axial resistance at SLS for a single pile, for 25 mm of settlement, may be taken as 1,200 kN.

As a guide, to achieve the above design resistances, the piles should be driven to a final set of no less than 10 blows per 25 mm of penetration using a hammer with rated energy of about 50 kJ, and not exceeding 60 kJ. The actual set criteria should be established based on the Contractor's pile driving equipment. Provision should be made to re-tap selected piles to confirm the set after adjacent piles have been driven, in accordance with MTO's current Special Provision.

5.4.2 Resistance to Lateral Loads

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

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

$$k_h = \frac{k_{s1}}{5B}$$

where B is the pile diameter (m) and k_{s1} is the constant of horizontal subgrade reaction, as given below

It is anticipated that the pile caps would be perched above the Highway 400 cut grade to provide a greater driven length. The soils above the Highway 400 cut level were not investigated during the 1979 drilling program. Based on regional geological mapping, it is likely that the soils above the cut will consist of clayey silt till, similar to that below the cut grade. The following ranges for the value of ks1 may be assumed in the structural analysis; these values will have to be confirmed following the detailed design stage of the subsurface investigation.

Soil Unit	k_{s1}
Clayey Silt Till above Elevation 264 m	25 to 60 MPa/m
Clayey Silt Till below Elevation 264 m	50 to 100 MPa/m

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

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

5.4.3 Frost Protection

The pile caps should be provided with 1.5 m soil of cover for frost protection.

5.5 Lateral Earth Pressures

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

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the abutments and walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00

- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the abutment wall, in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be used in accordance with OPSS 501.06.

- The granular fill may be placed either in a zone with width equal to at least 1.5 m behind the back of the stem (Case I from OHBDC Figure 6-7.4.1) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II from OHBDC Figure 6-7.4.4).

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

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

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

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

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

It should be noted that the above design recommendations and parameters assume level backfill and ground surface behind the abutment and retaining walls. Where there is sloping ground

behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

5.6 Design of Permanent Cut Slopes

In the vicinity of the Bayfield Street underpass, the Highway 400 cut is up to about 7.5 m in depth. The 1979 boreholes were advanced from highway grade at about Elevation 266 m, and so no borehole information is available regarding the soils which will comprise the permanent cut slopes along the east and west sides of the widened Highway 400. Based on regional geological mapping, it is anticipated that the cut will be formed in sandy silt to clayey silt till, similar to that encountered below the highway grade, although a surficial deposit of sands or silts could also be present atop the till. If such a water-bearing deposit is present, protection of the slope faces by means of a drainage blanket will be required. This will have to be investigated during the detailed design stage.

Based on the topographic information on the Engineering and Title Records plates and on site reconnaissance, the existing cut slopes are formed at a gradient of 1.7 to 2.5 horizontal to 1 vertical (1.7H:1V to 2.5H:1V). For preliminary design purposes, a maximum gradient of 2.H:1V may be assumed for the new permanent cut slopes. This design recommendation will have to be confirmed during the detailed design stage of the subsurface investigation program.

5.7 Design and Construction Considerations

5.7.1 Dewatering

The 1979 subsurface investigation report indicates that no water table was encountered during the investigation; this will have to be confirmed during the detailed design stage of subsurface investigation. Based on the available information, groundwater seepage into the footing excavations is expected to be minor. Pumping from properly-filtered sumps or a filtered drain placed at the base of the excavation should provide sufficient groundwater control during foundation works. Surface water run-off, which is expected to be more significant than groundwater seepage, should be directed away from the footing excavations.

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

5.7.2 Excavation

The footing excavations will extend a minimum of 1.5 m below lowest surrounding grade, through stiff to hard clayey silt till. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. The clayey silt till soils below Highway 400 grade would be classified as Type 2 soil. Temporary open-cut slopes should therefore be maintained no steeper than 1 horizontal to 1 vertical (1H:1V) to within 1.2 m of the excavation base; below this, the excavation walls may be maintained near-vertical. Where space restrictions dictate, adjacent to the new permanent cut slope, footing excavations could also be carried out within a braced excavation.

5.7.3 Obstructions

Cobbles were noted on the borehole records from the 1979 subsurface investigation. It is noted that cobbles and boulders are inherent in glacial soils, and should therefore be expected during footing or pile cap excavation, driven pile installation and temporary shoring system installation, if such a system is required at the site. Where boulders are encountered within footing excavations, they should be removed and the sub-excavated areas should be backfilled with well-compacted Granular 'A' or lean concrete.

GOLDER ASSOCIATES LTD.

for Lisa C. Coyne, P.Eng.,
Geotechnical Engineer

Anne S. Poschmann, P.Eng.,
Principal

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

LCC/ASP/FJH/clg
N:\ACTIVE\1100\001-1143F\2002\RPT14-02FEB-BAYFIELD.DOC

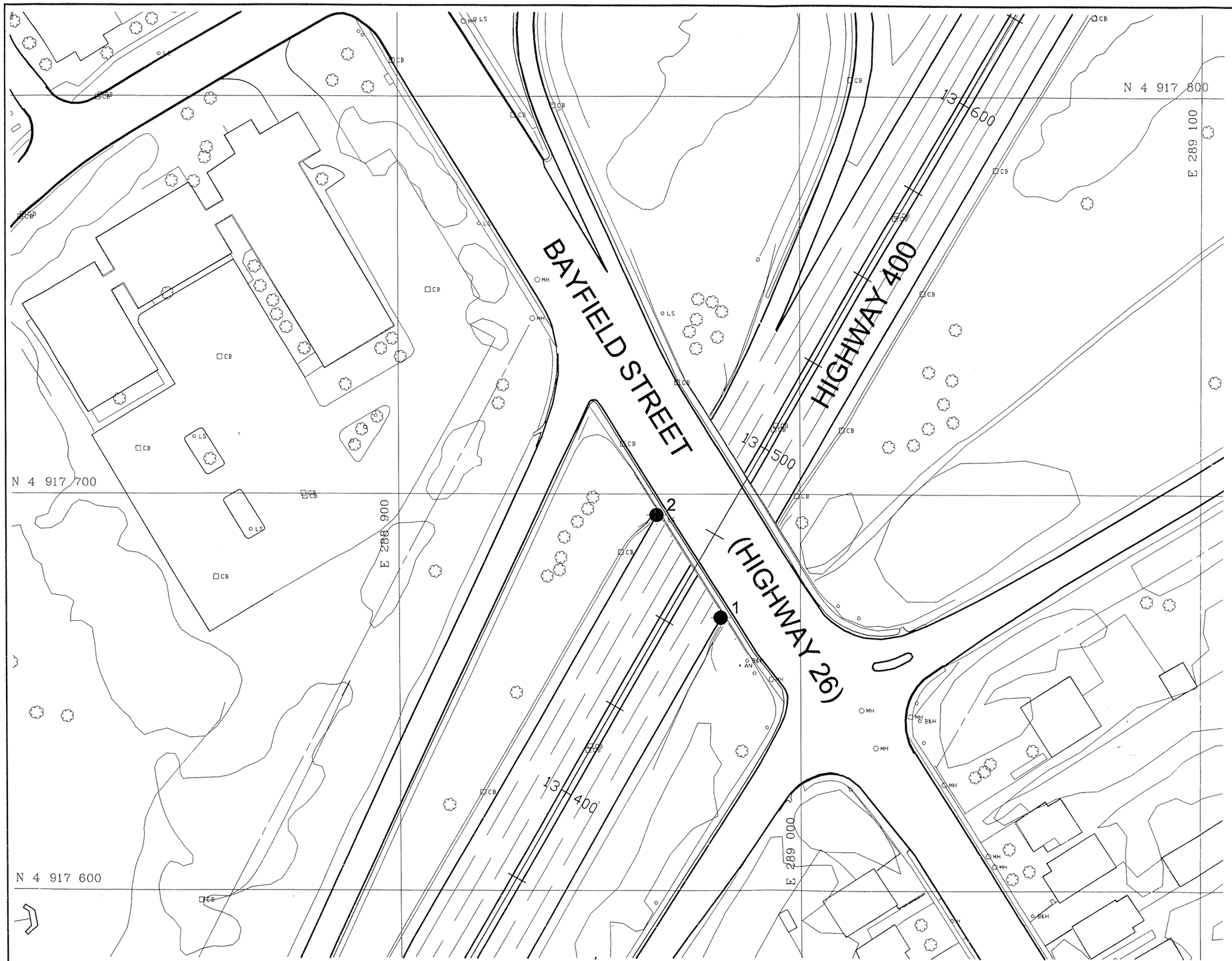
DIST HWY 400
CONT. No.
GWP No. 30-95-00

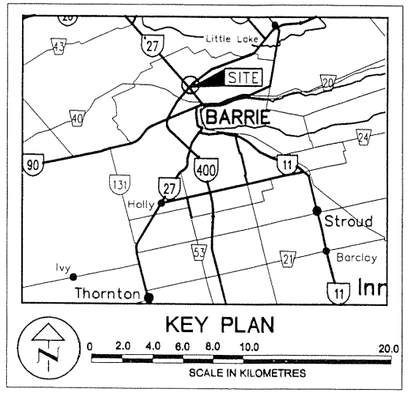
BAYFIELD STREET UNDERPASS
HWY 400
BOREHOLE LOCATION PLAN

SHEET

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA





KEY PLAN

0 2.0 4.0 6.0 8.0 10.0 20.0
SCALE IN KILOMETRES

LEGEND

- Borehole, previous investigation
- Borehole, present investigation

No.	ELEVATION	LOCATION NORTHING	LOCATION EASTING
1	266.1	4,917,669	288,980
2	266.4	4,917,695	288,964

REFERENCE
This drawing was created from digital file "50211.dwg" provided by URS Cole Sherman

0 10 20 30 40 50 100m
SCALE IN METRES

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

NO.	DATE	BY	REVISION

Geocres No.

HWY. No. 400	PROJECT NO.: 001-1143F		
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-172
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

P1143F19.DWG

APPENDIX A

RECORDS OF BOREHOLES AND TEST RESULTS
1979 INVESTIGATION

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 (R.Q.D.), 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

S.S.	SPLIT SPOON	T.P.	THINWALL PISTON
W.S.	WASH SAMPLE	O.S.	OSTERBERG SAMPLE
S.T.	SLOTTED TUBE SAMPLE	R.C.	ROCK CORE
B.S.	BLOCK SAMPLE	P.H.	T.W. ADVANCED HYDRAULICALLY
C.S.	CHUNK SAMPLE	P.M.	T.W. ADVANCED MANUALLY
T.W.	THINWALL OPEN	F.S.	FOIL SAMPLE

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL

e	1, %	VOID RATIO
n	1, %	POROSITY
w	1, %	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX = $w_L - w_P$
I_L	1	LIQUIDITY INDEX = $\frac{w - w_P}{I_P}$

e_{min}	1, %	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_U	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT

Ontario Ministry of Transportation and Communications

HIGHWAY ENGINEERING DIVISION-ENGINEERING MATERIALS OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 1 METRIC

W P 28-78-02 LOCATION Sta. 10+446.3 o/s 13.4 m Lt. ℄ Hwy. 26 & 27 ORIGINATED BY PRK

DIST 5 HWY 400 BOREHOLE TYPE Hollow Stem Auger 82 mm I.D. COMPILED BY PRK

DATUM Geodetic DATE 1979 06 06 CHECKED BY RS

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
266.1	Ground Level							
0.0	Asphalt Pavement				*	266		
0.3	Very Stiff	1	SS	17			o	6 63 21 10
	Clayey Silt to Silt With Sand Trace of Gravel Occasional Cobbles Hard (Glacial Till)	2	SS	42		264		
		3	SS	84			o	
		4	SS	98			o	9 58 23 10
		5	SS	88		262		
		6	SS	90				
		7	SS	58			o	4 59 25 12
		8	SS	70		260		
258.6		9	SS	67			o	5 59 25 11
7.5	End of Borehole * Water Level Not Observed at Completion of Augering							

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, x LAB VANE

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%) 5 10 15

UNIT WEIGHT γ

HIGHWAY ENGINEERING DIVISION-ENGINEERING MATERIALS OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 2 METRIC

W P 28-78-02 LOCATION Sta. 10+477.1 o/s 12.5 m Lt. ℄ Hwy. 26 & 27 ORIGINATED BY PRK

DIST 5 HWY 400 BOREHOLE TYPE Solid Stem Auger, Cone Test COMPILED BY PRK

DATUM Geodetic DATE 1979 06 07 CHECKED BY RS

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI
266.4	Ground Level							
0.0	Asphalt Pavement				*	266	0.3 m augered through pavement	
0.3	Stiff Clayey Silt to Silt With Sand Traces of Gravel Occasional Cobbles Hard (Glacial Till)	1	SS	13			o	
		2	SS	77			o	8 55 26 1
		3	SS	106		264	o	
		4	SS	63			100/0.08 m	2 59 27 1
		5	SS	110		262	o	
		6	SS	111/	0.20 m			3 62 23 1
		7	SS	128				
		8	SS	74		260		
258.3		9	SS	147/	0.25 m		o	
8.1	End of Borehole Water Level Not * Observed at Completion of Augering							

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

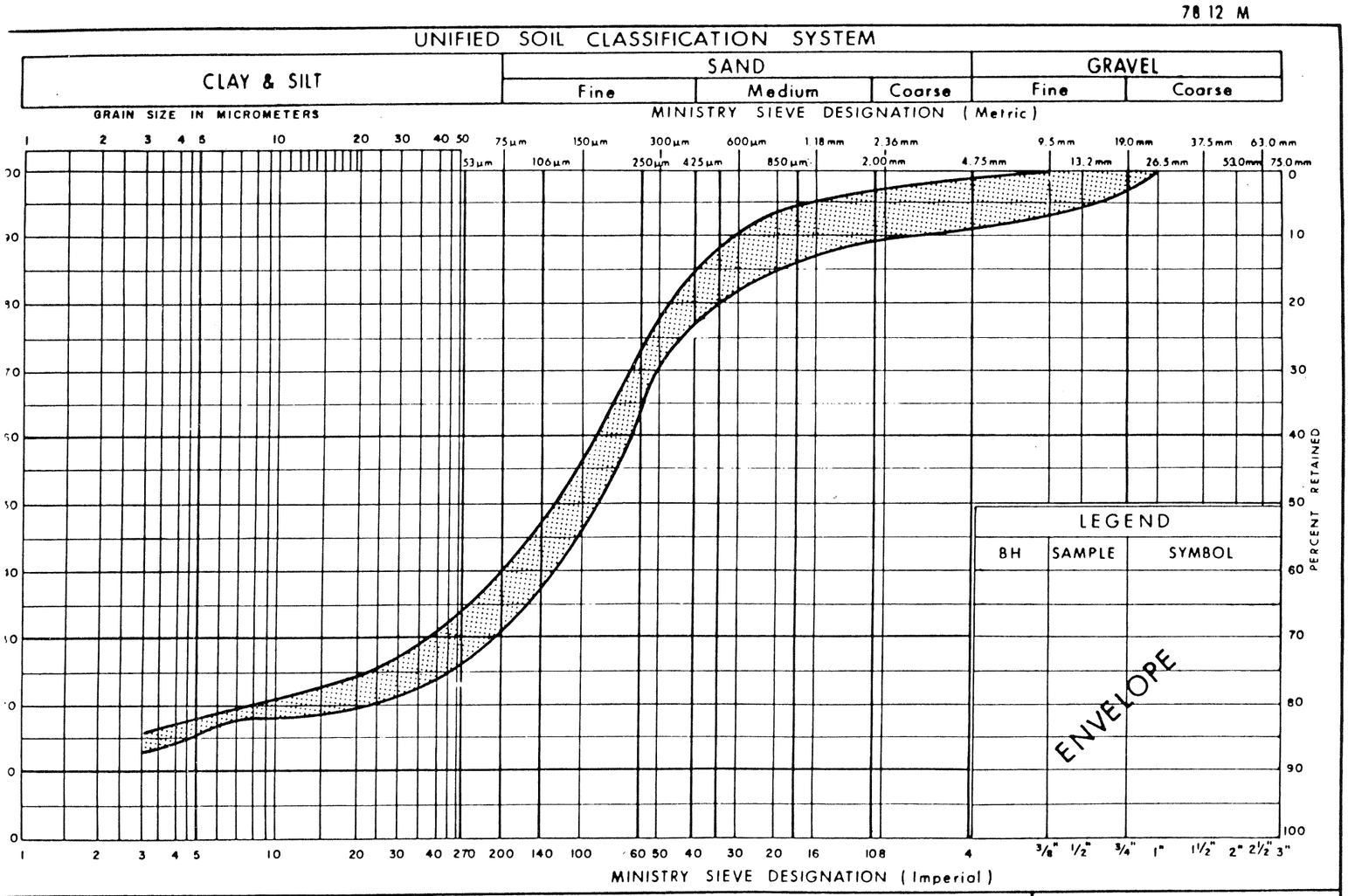
SHEAR STRENGTH: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, x LAB VANE

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%) 5 10 15

UNIT WEIGHT γ

78 12 M



Ministry of Transportation and Communications

GRAIN SIZE DISTRIBUTION
CLAYEY SILT TO SILT
WITH SAND, TRACES OF GRAVEL, OCC COBBLES (GLACIAL TILL)

FIG No 1

W P 28 - 78 - 02

Golder Associates Ltd.

2180 Meadowvale Boulevard
Mississauga, Ontario, Canada L5N 5S3
Telephone (905) 567-4444
Fax (905) 567-6561

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT ST. VINCENT STREET UNDERPASS STRUCTURE SITE 30-394 HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11 G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

Submitted to:

URS Cole, Sherman
75 Commerce Valley Drive East
Thornhill, Ontario
L3T 7N9

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Appendix A

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Drawing 1 St. Vincent Street Underpass, Highway 400, Borehole Location Plan

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PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
ST. VINCENT STREET UNDERPASS
STRUCTURE SITE 30-394
HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

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Drawing 1
Appendix A

LIST OF DRAWINGS

Drawing 1 St. Vincent Street Underpass, Highway 400, Borehole Location Plan

LIST OF APPENDICES

Appendix A Records of Boreholes and Test Results – 1964 Investigation

1.0 INTRODUCTION

Golder Associates Ltd. has been retained by URS Cole, Sherman (Cole, Sherman) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the ultimate widening of Highway 400 from 1 km south of Highway 89, northerly 30 km to Highway 11, in Simcoe County, Ontario. Foundation engineering services are required for the widening and / or replacement of eighteen existing overpass and underpass structures, as well as five structural culverts.

This report addresses the subsurface conditions at the St. Vincent Street underpass structure in Barrie, Ontario. Existing subsurface data for this site from an investigation conducted for the Department of Highways, Ontario (DHO) in 1964 were used to determine the subsurface conditions for this preliminary design study. The 1964 report was prepared by H.Q. Golder and Associates Ltd. ("*Soil Conditions and Foundations, Proposed Highway 400 – St. Vincent Street Underpass, Barrie, Ontario*", dated December 1964 – GEOCRES File No. 31D-207).

The terms of reference for the scope of work are outlined in Golder Associates' Proposal No. P01-1192, dated June 2000.

2.0 SITE DESCRIPTION

The existing four-span St. Vincent Street underpass structure is located about 1.5 km north of Bayfield Street (Highway 26) and 5 km south of Highway 11, in Barrie, Ontario. The MTO has designated this underpass as Structure Site 30-394.

At this structure site, the Highway 400 grade is at about Elevation 261.5 m, rising slightly northward. St. Vincent Street has been constructed in fill, with approach embankments up to 6 m in height. The St. Vincent Street grade is at about Elevation 267.5 m over Highway 400.

The existing four-span underpass structure was constructed in the mid-1960s under Contract 66-149. According to the general layout drawing for this contract, which was provided by Morrison Hershfield (the structural designers for this preliminary study), the abutments, wing walls and piers are supported on spread footings which are founded at Elevation 259.2 m.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at this site for the Department of Highways, Ontario (DHO) in November 1964, by H.Q. Golder and Associates Ltd. At that time, a total of five boreholes, with accompanying dynamic cone penetration tests, were advanced – one borehole at each of the then-proposed abutment and pier locations. Borehole 1 was advanced at the central pier location; Boreholes 2 and 3 were drilled at the west pier and abutment locations, respectively; and Boreholes 4 and 5 were located at the east abutment and pier locations, respectively. The boreholes were advanced to between 5 m and 10.5 m below Highway 400 grade.

Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. The water levels in the open borehole were observed during and following the drilling operations, and a piezometer was installed in each borehole to monitor the groundwater conditions at the site. Laboratory testing, consisting of natural moisture content and bulk unit weight determinations, and grain size distribution analyses, was carried out on selected soil samples.

The borehole locations and elevations, referenced to the geodetic datum, were established by H.Q. Golder and Associates Ltd. Approximate northing and easting co-ordinates consistent with the MTM NAD83 survey system, currently in use on this project, have been determined by Golder Associates based on the borehole locations given in the 1964 report. The approximate borehole locations and northing and easting co-ordinates are shown on the attached Drawing 1.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

This 30 km section of Highway 400 traverses, from south to north, the following physiographic regions as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, Third Edition, 1984): the Simcoe Lowlands; the Peterborough Drumlin Field; a second lobe of the Simcoe Lowlands; and the Simcoe Uplands. Along Highway 400, the Simcoe Lowlands are present from the southern limit of the project to just south of Innisfil Creek, and again from Essa Road (Simcoe Road 30, formerly Highway 27) to about 1 km north of Dunlop Street (Simcoe Road 90, formerly Highway 90). The Peterborough Drumlin Field occupies the belt between these lobes of the Simcoe Lowlands, extending from just south of Innisfil Creek, which is located about 1 km north of Highway 89, to Essa Road. The Simcoe Uplands extend from about 1 km north of Dunlop Street to beyond the northern limit of the project at Highway 11.

The two sections where Highway 400 crosses the Simcoe Lowlands consist of two lobes of a sand plain which include the shores of Kempenfelt Bay, the Nottawasaga River and Innisfil Creek. The surficial soils of these sections of the Simcoe Lowlands consist primarily of sand, although silt, clay or peat may be found in low-lying areas.

The surficial soils in the Peterborough Drumlin Field consist primarily of gravelly sand till or sand and gravel deposits. Drumlins (glacially-shaped hills) are more frequent in the southern portion of the section of the Peterborough Drumlin Field traversed by Highway 400. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins.

The surficial soils in the Simcoe Uplands physiographic region, in which the St. Vincent Street underpass site is located, are primarily sandy silt till deposits, known to contain occasional boulders. Low-lying areas may be infilled with shallow sand and gravel deposits, which are shoreline deposits of a former glacial lake that once flooded the area.

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes, together with the results of in-situ and laboratory testing, are given on the Record of Borehole sheets and figures contained in Appendix A. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

Borehole 1 was advanced at the central pier location; Boreholes 2 and 3 were drilled at the west pier and abutment locations, respectively; and Boreholes 4 and 5 were located at the east abutment and pier locations, respectively. These boreholes were advanced to between 5 m and 10.5 m below Highway 400 grade. The approximate locations and ground surface elevations for these borings and probeholes are shown on the attached Drawing 1.

In summary, the soils below the Highway 400 grade at this site consist of road base fill or topsoil underlain by a compact to very dense silty sand till deposit. A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Fill

Up to 1.2 m of silty sand to sand and gravel fill was encountered in the 1964 boreholes; this fill comprised the St. Vincent Street road base. The base of the fill was encountered at about Elevation 260 m to 261 m. This cohesionless fill was considered to have a compact relative density, based on a measured Standard Penetration Test (SPT) 'N' value of 15 blows per 0.3 m of penetration.

4.2.2 Silty Sand Till

Below the surficial fill, the boreholes encountered a till deposit which extended to the maximum investigated depth of 10 m (approximately Elevation 250.5 m). The till composition ranges from silty sand to sand and silt, containing trace to some clay and gravel; layers or lenses of silty clay were noted within the till in one of the boreholes. The grain size distribution test results for samples of the till are shown on the figures contained in Appendix A.

The SPT 'N' values measured in the upper 3 m to 5 m of the till deposit range from 18 to greater than 100 blows per 0.3 m of penetration, but were typically between 20 and 50 blows per 0.3 m of penetration; the upper portion of the deposit has a generally compact to dense relative density. Below this depth (below about Elevation 256 m), the measured SPT 'N' values were generally greater than 100 blows per 0.3 m of penetration, indicating that this portion of the till deposit has a very dense relative density. The SPT 'N' values measured in the 1964 boreholes are summarized on Figure 4 in Appendix A.

4.3 Groundwater Conditions

Piezometers were installed in each borehole to monitor the groundwater levels at the site. The groundwater levels measured in November 1964 were between about 4.5 m and 7.5 m below ground surface, at approximately Elevation 254 m to 257 m. The 1964 report by H.Q. Golder and Associates Ltd. indicates that the year 1964 was relatively dry and consequently the water levels at the site were considered to be below normal levels.

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

GOLDER ASSOCIATES LTD.

for Lisa C. Coyne, P.Eng.,
Geotechnical Engineer

Anne S. Poschmann, P.Eng.,
Principal

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

LCC/ASP/FJH/clg
N:\ACTIVE\1100\001-1143F 2002 RPT15-02FEB-STVINCENT.DOC

PART B

PRELIMINARY FOUNDATION DESIGN REPORT
ST. VINCENT STREET UNDERPASS
STRUCTURE SITE 30-394
HIGHWAY 400 WIDENING FROM 1 KM SOUTH
OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

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

It is understood that Highway 400 will be widened from its existing six-lane configuration to an interim configuration of eight lanes, and an ultimate configuration of ten lanes, and that an alternative for a twelve-lane express / collector system is under consideration between Molson Park Drive and Duckworth Street in Barrie. Throughout the project length, it is expected that the existing highway will be widened by between 13 m and 30 m.

The existing St. Vincent Street underpass is a four-span structure, built in 1964 to accommodate future widening of Highway 400. It is understood that the existing structure will accommodate widening to the ultimate ten-lane configuration. However, if a twelve-lane express / collector system is adopted, or if there are changes in the median width, replacement of the existing structure will be necessary. In addition, if widening of the local road is required, it will be necessary to widen or twin the existing structure.

Based on the general layout drawing for the existing four-span structure, the abutments, wing walls and piers are supported on spread footings which are founded at about Elevation 259.2 m. The Highway 400 grade is at about Elevation 261.5 m, and St. Vincent Street has been constructed on embankment fill, with approach embankments up to 6 m in height and grade at about Elevation 267.5 m over Highway 400.

5.2 Bridge Foundation Options

The soils below the Highway 400 level consist of road base fill overlying a deposit of compact to very dense silty sand to sand and silt till. Based on these subsurface conditions, consideration could be given to founding the replacement structure on spread footings placed on the till stratum.

Consideration could also be given to the use of perched abutments, founded on spread footings placed on a compacted granular pad within the approach embankments.

Alternatively, if integral abutments are under consideration for the replacement structure, the abutments could be supported on steel H-piles driven to found within the very dense silty sand to sand and silt till deposit.

Preliminary recommendations for spread footings, including perched abutments, and for deep foundations are provided in the following sections.

5.3 Spread Footings

For preliminary design, spread footings for the abutments and pier may be placed at a design founding level of Elevation 259 m. The footings would be founded on the compact to very dense silty sand to sand and silt till deposit. Any associated wing wall or retaining wall footings may be stepped upward away from the abutments such that a minimum soil cover of 1.5 m is maintained above the underside of the footings. To minimize requirements for groundwater control, the footings should be maintained above the groundwater level, which was measured during the 1964 investigation to be between Elevation 254 m and 257 m; it is noted that 1964 was a relatively dry year and these measured water levels could be lower than the normal levels at the site.

Alternatively, consideration could be given to the use of abutment footings perched within the approach embankments.

5.3.1 Axial Geotechnical Resistance

Spread footings placed on the properly prepared silty sand to sand and silt till deposit at the design elevation given above may be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 900 kPa, assuming a 3 m wide footing. The settlement of footings founded on the silty sand to sandy silt will be dependent on the footing size and configuration, and on the applied loads. The majority of this settlement will take place during construction itself; however, in the case of widening, it should be noted that the settlement will be differential with respect to the existing footings. For preliminary design purposes, the geotechnical resistance at Serviceability Limit States (SLS) may be taken as 400 kPa. The geotechnical resistance at SLS will have to be reviewed following the detailed design stage of

subsurface investigation, once the proposed footing size and loadings as well as the configuration with respect to the existing footings are known.

For spread footings placed within the approach embankments on a compacted Granular 'A' core, a factored geotechnical resistance at ULS of 900 kPa may be assumed for preliminary design. The geotechnical resistance at SLS will depend on the thickness of Granular 'A' and the consistency and thickness of the underlying soils; a value of 350 kPa may be assumed for preliminary design.

The geotechnical resistances provided herein are given under the assumption that the loads will be applied perpendicular to the surface of the footings; where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the Ontario Highway Bridge Design Code (OHBDC).

5.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footing and the subsoils should be calculated in accordance with Section 6-8.4.3 of the OHBDC. The angle of friction between the concrete and the undisturbed silty sand to sand and silt till founding soils should be taken as 24 degrees; the corresponding coefficient of friction, tan δ, would then be 0.45. Where "perched" abutment footings are adopted, the angle of friction between the concrete footings and the compacted Granular 'A' pad should be taken as 30 degrees; the corresponding coefficient of friction would be 0.58.

5.3.3 Frost Protection

The footings should be provided with a minimum of 1.5 m of soil cover for frost protection.

5.4 Driven Steel H-Piles

Consideration could be given to supporting a new underpass structure on steel H-piles driven to found within the very dense silty sand to sand and silt till deposit. In the 1964 borings, the measured Standard Penetration Test (SPT) 'N' values were generally greater than 100 blows per 0.3 m of penetration below Elevation 256 m. It is possible that a suitable driving resistance may not be achieved above the base of the 1964 boreholes. In this regard, it should be noted that additional borehole investigation work will be required at the proposed foundation locations

during detailed design in order to confirm the thickness and relative density of the till stratum and establish design tip elevations.

For preliminary design, the pile tip elevation may be taken at Elevation 250 m, about 17.5 m below St. Vincent Street grade and 11.5 m below Highway 400 grade.

5.4.1 Axial Geotechnical Resistance

For preliminary design, the factored axial resistance at ULS for steel HP 310 x 110 H-piles driven to found within the very dense silty sand to sand and silt till at a design tip elevation of 250 m may be taken as 1,400 kN. The axial resistance at SLS for 25 mm of settlement may be taken as 1,200 kN.

As a guide, to achieve the above design resistances, the piles should be driven to a final set of no less than 10 blows per 25 mm of penetration using a hammer with rated energy of about 50 kJ, and not exceeding 60 kJ. The actual set criteria should be established based on the Contractor's pile driving equipment. Provision should be made to re-tap selected piles to confirm the set after adjacent piles have been driven, in accordance with MTO's current Special Provision.

5.4.2 Resistance to Lateral Loads

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

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

$$k_h = \frac{n_h z}{B}$$

where n_h is the constant of subgrade reaction
z is the depth (m)
B is the pile diameter (m)

For the embankment fill and the upper, compact to dense portion of the silty sand to sand and silt till soils through which the piles would be driven, the range in value of n_h may be taken as 5 MPa/m to 10 MPa/m in the structural analysis.

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

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

5.4.3 Frost Protection

The pile caps should be provided with 1.5 m soil cover for frost protection.

5.5 Lateral Earth Pressures

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

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the abutments and walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00

- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the abutment wall, in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be used in accordance with OPSS 501.06.

- The granular fill may be placed either in a zone with width equal to at least 1.5 m behind the back of the stem (Case I from OHBDC Figure 6-7.4.1) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II from OHBDC Figure 6-7.4.4).

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

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

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

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

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

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

5.6 Embankment Design

Based on the topographic information on the Engineering and Title Records plates and on site reconnaissance, the existing approach embankment side slopes are formed at a gradient of about 2 horizontal to 1 vertical (2H:1V). For raising and widening of the existing St. Vincent Street embankment to accommodate a widened underpass structure, the new embankment side slopes should be formed at a maximum gradient of 2H:1V. The widening of the embankment should be carried out using conventional fill placement and compaction practices, and benching of the existing embankment side slopes should be carried out to key in the new fill.

5.7 Design and Construction Considerations

5.7.1 Dewatering

Provided that the spread footing founding level or pile cap underside is maintained above the groundwater level, seepage into the footing or pile cap excavations is expected to be minor. Pumping from properly-filtered sumps or a filtered drain placed at the base of the excavation should provide sufficient groundwater control during foundation works. Surface water run-off, which is expected to be more significant than groundwater seepage, should be directed away from the footing excavations.

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

5.7.2 Excavation

The footing or pile cap excavations will extend a minimum of 1.5 m below the lowest surrounding grade, through embankment fill or road base fill, into the compact to dense portion of the silty sand to sand and silt till deposit. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. The embankment fill and compact to dense till would be classified as Type 2 soil. Temporary open-cut slopes should therefore be made to within 1.2 m of the excavation base, and maintained at a minimum gradient of 1 horizontal to 1 vertical (1H:1V). The remaining depth of 1.2 m may be excavated near-vertical. Where space restrictions dictate, footing excavations could also be carried out within a braced excavation.

5.7.3 Obstructions

Although no cobbles or boulders were encountered during the borehole investigation, it should be noted that cobbles and boulders are inherent in glacially-derived materials. Cobbles and boulders should therefore be expected during driving of steel H-piles if a deep foundation option is selected.

GOLDER ASSOCIATES LTD.

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Geotechnical Engineer

Anne S. Poschmann, P.Eng.,
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Designated MTO Contact

LCC/ASP/FJH/clg
N:\ACTIVE\1100\001-1143F\2002\RPT15-02FEB-ST VINCENT.DOC

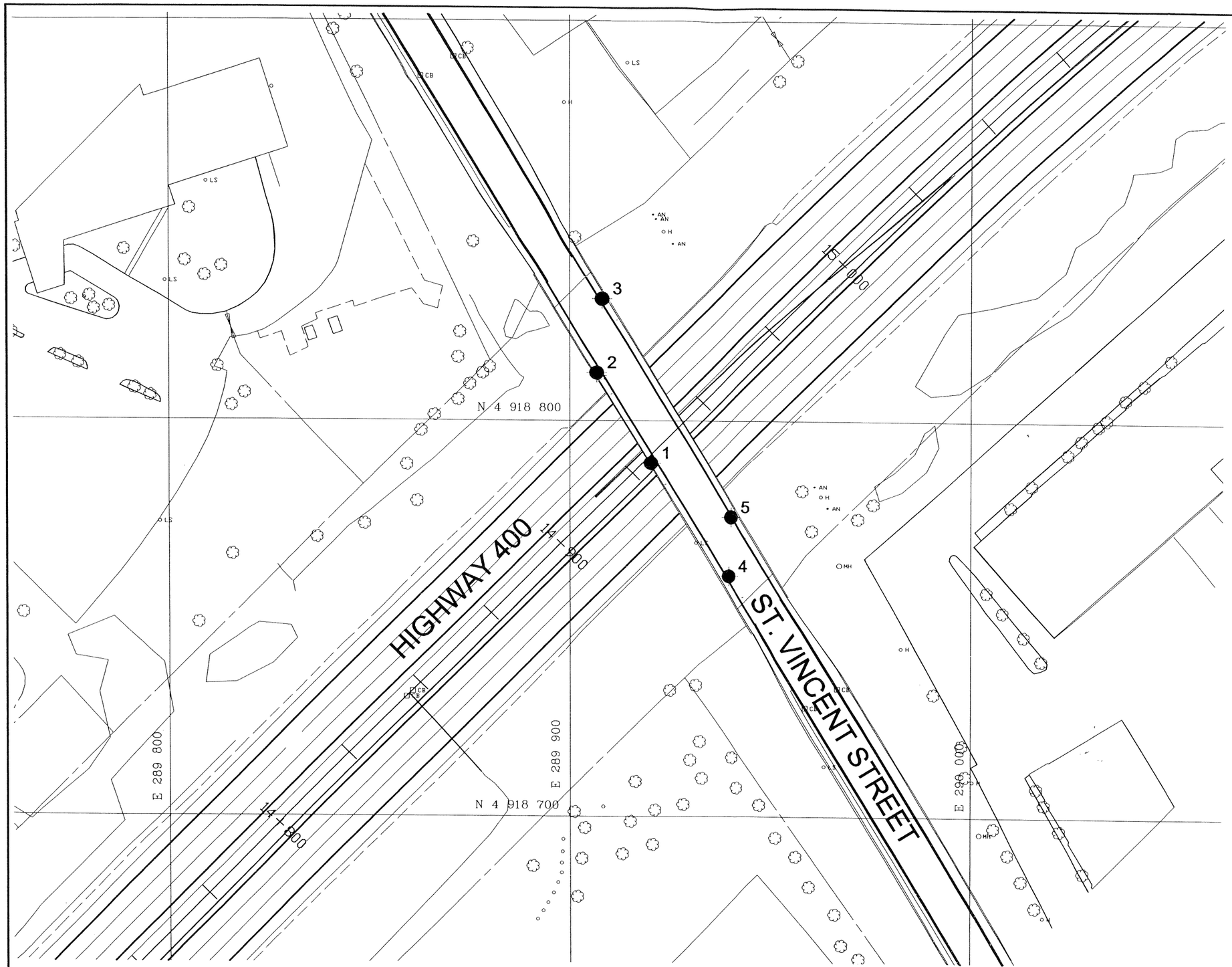
DIST HWY 400
CONT. No.
GWP No. 30-95-00

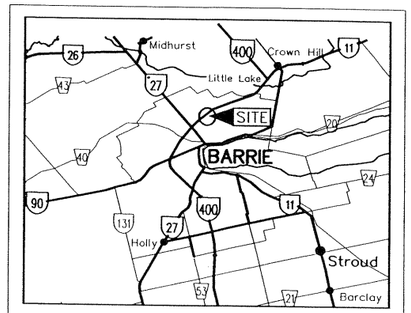
ST. VINCENT STREET UNDERPASS
HWY 400
BOREHOLE LOCATION PLAN

SHEET

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA





KEY PLAN

0 2.0 4.0 6.0 8.0 10.0 20.0
SCALE IN KILOMETRES

LEGEND

- Borehole, previous investigation
- Borehole, present investigation

No.	ELEVATION	LOCATION NORTHING	LOCATION EASTING
1	261.3	4,918,789	289,920
2	261.2	4,918,812	289,907
3	261.5	4,918,831	289,908
4	260.5	4,918,761	289,940
5	261.0	4,918,776	289,940

REFERENCE
This drawing was created from digital file "50212.dwg" provided by URS Cole Sherman

0 10 20 30 40 50 100m
SCALE IN METRES

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

NO.	DATE	BY	REVISION

Geocres No.

HWY. No. 400		PROJECT NO.: 001-1143F	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-394
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

P1143F20.DWG

plot scale 1:2mm.

APPENDIX A

RECORDS OF BOREHOLES AND TEST RESULTS
1964 INVESTIGATION

LIST OF ABBREVIATIONS

The abbreviations commonly employed on each "Record of Borehole," on the figures and in the text of the report, are as follows:

I. SAMPLE TYPES

AS auger sample
CS chunk sample
DO drive open
DS Denison type sample
FS foil sample
RC rock core
ST slotted tube
TO thin-walled, open
TP thin-walled, piston
WS wash sample

II. PENETRATION RESISTANCES

Dynamic Penetration Resistance: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch diameter, 60 degree cone one foot, where the cone is attached to 'A' size drill rods and casing is not used.

Standard Penetration Resistance, *N*: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch drive open sampler one foot.

WH sampler advanced by static weight—weight, hammer

PH sampler advanced by pressure—pressure, hydraulic

PM sampler advanced by pressure—pressure, manual

III. SOIL DESCRIPTION

(a) *Cohesionless Soils*

Relative Density	*N, 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, *lb./sq. ft.*
Very soft	Less than 250
Soft	250 to 500
Firm	500 to 1,000
Stiff	1,000 to 2,000
Very stiff	2,000 to 4,000
Hard	over 4,000

IV. SOIL TESTS

C consolidation test
H hydrometer analysis
M sieve analysis
MH combined analysis, sieve and hydrometer[1]
Q undrained triaxial[2]
R consolidated undrained triaxial[2]
S drained triaxial
U unconfined compression
V field vane test

NOTES:

[1]Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.
[2]Undrained triaxial tests in which pore pressures are measured are shown as $\bar{Q}$ or $\bar{R}$.

LIST OF SYMBOLS

I. GENERAL

π = 3.1416
e = base of natural logarithms 2.7183
$\log_e a$ or $\ln a$, natural logarithm of a
$\log_{10} a$ or $\log a$, logarithm of a to base 10
t time
g acceleration due to gravity
V volume
W weight
M moment
F factor of safety

II. STRESS AND STRAIN

u pore pressure
σ normal stress
σ' normal effective stress ($\bar{\sigma}$ is also used)
τ shear stress
ϵ linear strain
ϵ_{xy} shear strain
ν Poisson's ratio (μ is also used)
E modulus of linear deformation (Young's modulus)
G modulus of shear deformation
K modulus of compressibility
η coefficient of viscosity

III. SOIL PROPERTIES

(a) *Unit weight*

γ unit weight of soil (bulk density)
γ_s unit weight of solid particles
γ_w unit weight of water
γ_d unit dry weight of soil (dry density)
γ' unit weight of submerged soil
G_s specific gravity of solid particles $G_s = \gamma_s/\gamma_w$
e void ratio
n porosity
w water content
S_r degree of saturation

(b) *Consistency*

w_L liquid limit
w_P plastic limit
I_P plasticity index
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
D_r relative density $= (e_{max} - e)/(e_{max} - e_{min})$

(c) *Permeability*

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

(d) *Consolidation (one-dimensional)*

m_v coefficient of volume change $= -\Delta e/(1+e)\Delta\sigma'$
C_c compression index $= -\Delta e/\Delta \log_{10} \sigma'$
c_v coefficient of consolidation
T_v time factor $= c_v t/d^2$ (d, drainage path)
U degree of consolidation

(e) *Shear strength*

τ_f shear strength
c' effective cohesion intercept
ϕ' effective angle of shearing resistance, or friction
} in terms of effective stress $\tau_f = c' + \sigma' \tan \phi'$
c_u apparent cohesion*
ϕ_u apparent angle of shearing resistance, or friction
} in terms of total stress $\tau_f = c_u + \sigma \tan \phi_u$
μ coefficient of friction
S_t sensitivity

*For the case of a saturated cohesive soil, $\phi_u = 0$ and the undrained shear strength $\tau_f = c_u$ is taken as half the undrained compressive strength.

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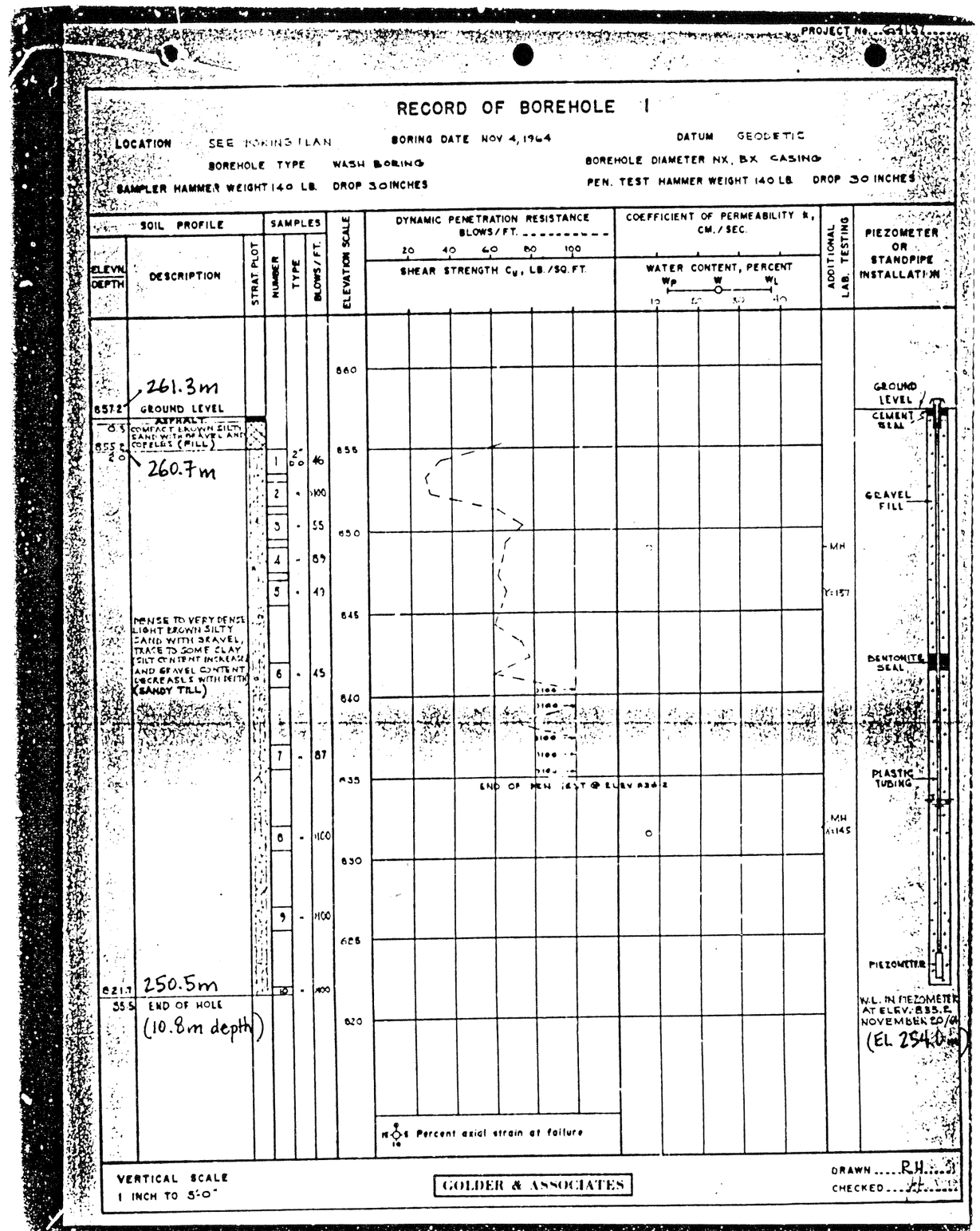


PROJECT No. [illegible]

RECORD OF BOREHOLE 2

LOCATION SEE BORING PLAN BORING DATE NOV. 5, 1964 DATUM GEODETIC

BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER NX BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/FT.	ADDITIONAL LAB. TESTING
856.8	261.2m GROUND LEVEL				
0.0 / 0.4	TOPSOIL				
	261.0m	1	2" DO	62	
	COMPACT TO DENSE LIGHT BROWN SILTY SAND WITH TRACE TO SOME GRAVEL AND CLAY (SANDY TILL)	2	"	26	MH
		3	"	24	
		4	"	23	
		5	"	27	MH
841.8 / 15.8	256.4m END OF HOLE (4.7m depth)	6	"	46	

ELEVATION SCALE: 860, 855, 850, 845, 840

DYNAMIC PENETRATION RESISTANCE BLOWS/FT. 20 40 60 80 100

SHEAR STRENGTH C_u, LB./SQ.FT.

COEFFICIENT OF PERMEABILITY k, CM./SEC.

WATER CONTENT, PERCENT W_P W W_L 10 20 30 40

PIEZOMETER OR STANDPIPE INSTALLATION: GROUND LEVEL; CEMENT SEAL; BENTONITE SEAL; PLASTIC TUBING; GRAVEL; SLOTTED PIPE

W.L. IN STANDPIPE AT ELEV. 842.8 NOVEMBER 20/64 (El. 256.9m)

>100 >100

END OF PEN. TEST @ ELEV 836.8

Percent axial strain at failure

VERTICAL SCALE 1 INCH TO 5'-0"

GOLDER & ASSOCIATES

DRAWN RH CHECKED [illegible]

PROJECT No. [illegible]

RECORD OF BOREHOLE 3

LOCATION SEE BORING PLAN BORING DATE NOV 6, 1964 DATUM GEODETIC

BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER NX, BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/FT.	ADDITIONAL LAB. TESTING
857.8	261.5m GROUND LEVEL				
0.0	COMPACT LIGHT BROWN, SILTY SAND WITH GRAVEL NUMEROUS SMALL LUMPS OF GREY SILTY CLAY (FILL)	1	2" DO	15	
853.8 / 4.0	260.2m	2	"	49	MH
		3	"	54	
		4	"	35	MH
	DENSE TO VERY DENSE LIGHT BROWN SILTY SAND TRACE TO SOME GRAVEL AND CLAY (SANDY TILL) LAYERS UP TO SEVERAL FEET IN THICKNESS OF VERY STIFF TO HARD SILTY CLAY WITH TRACE TO SOME SAND AND TRACE OF GRAVEL AT ABOUT EL. 849 AND 840.	5	"	34	
		6	"	46	
		7	CA CA	—	
		8	"	>100	MH >150
		9	"	>100	
828.8 / 29.5	252.5m END OF HOLE (9.0m depth)	10	"	>100	

ELEVATION SCALE: 860, 855, 850, 845, 840, 835, 830, 825

DYNAMIC PENETRATION RESISTANCE BLOWS/FT. 20 40 60 80 100

SHEAR STRENGTH C_u, LB./SQ.FT.

COEFFICIENT OF PERMEABILITY k, CM./SEC.

WATER CONTENT, PERCENT W_P W W_L 10 20 30 40

PIEZOMETER OR STANDPIPE INSTALLATION: GROUND LEVEL; CEMENT SEAL; PLASTIC TUBING; GRAVEL; BENTONITE SEAL; PIEZOMETER

W.L. IN PIEZOMETER AT ELEV. 841.8 NOVEMBER 20/64. (El. 256.6m)

>100 >100

END PEN TEST @ ELEV 836.8

Percent axial strain at failure

VERTICAL SCALE 1 INCH TO 5'-0"

GOLDER & ASSOCIATES

DRAWN RH CHECKED [illegible]

PROJECT No. 54157

RECORD OF BOREHOLE 4

LOCATION SEE BORING PLAN BORING DATE NOV 9, 1964 DATUM GEODETIC

BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER NX, BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	BLOWS/FT.	ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT. / SHEAR STRENGTH C_u, LB./SQ.FT.	COEFFICIENT OF PERMEABILITY k, CM./SEC. / WATER CONTENT, PERCENT (W_P, W, W_L)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
854.5 / 0.0	260.5m GROUND LEVEL SAND AND GRAVEL (FILL)					855				CAP; GROUND LEVEL; SURFACE SEAL
0.5	260.3m		1	2" D.O.	18					
			2	"	52	850			MH	BENTONITE SEAL
			3	"	36					
			4	"	15	845				
	COMPACT TO DENSE (VERY DENSE BELOW EL. 841) LIGHT BROWN SILTY SAND WITH TRACE TO SOME GRAVEL AND CLAY (SANDY TILL)		5	"	37				MH	GRAVEL
						840	>100			
			6	"	>100		END OF PEN. TEST @ ELEV. 839.5			
						835			MH	STANDPIPE
			7	"	>100					
830.5 / 24.0	253.1m END OF HOLE (7.3m depth)					830				W.L. IN STANDPIPE BELOW ELEV. 835.5 NOVEMBER 20/64. (COULD NOT LOWER WATER LEVEL FINDER PAST ELEV 835.5)
						825				

DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT

Percent axial strain at failure

VERTICAL SCALE 1 INCH TO 5'-0"

GOLDER & ASSOCIATES

DRAWN ...

CHECKED ...

PROJECT No. 54157

RECORD OF BOREHOLE 5

LOCATION SEE BORING PLAN BORING DATE NOV. 10, 1964 DATUM GEODETIC

BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER NX, BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	BLOWS/FT.	ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT. / SHEAR STRENGTH C_u, LB./SQ.FT.	COEFFICIENT OF PERMEABILITY k, CM./SEC. / WATER CONTENT, PERCENT (W_P, W, W_L)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
						860				CAP; GROUND LEVEL; SURFACE SEAL
856.4 / 0.0	261.0m GROUND LEVEL SAND AND GRAVEL (FILL)									
0.5	260.9m		1	2" D.O.	46	855				
			2	"	25				MH	BENTONITE SEAL
	COMPACT TO DENSE LIGHT BROWN SILTY SAND WITH SOME GRAVEL TRACE TO SOME CLAY (SANDY TILL)		3	"	26	850				
			4	"	24					GRAVEL
			5	"	21	845			MH	STANDPIPE
839.9 / 16.5	256.0m END OF HOLE (5.0m depth)		6	"	30	840	>100 END OF PEN. TEST @ ELEV. 838.4			W.L. IN STANDPIPE AT ELEV. 842.4 NOVEMBER 20/64 (El. 256.8m)
						835				

Percent axial strain at failure

VERTICAL SCALE 1 INCH TO 5'-0"

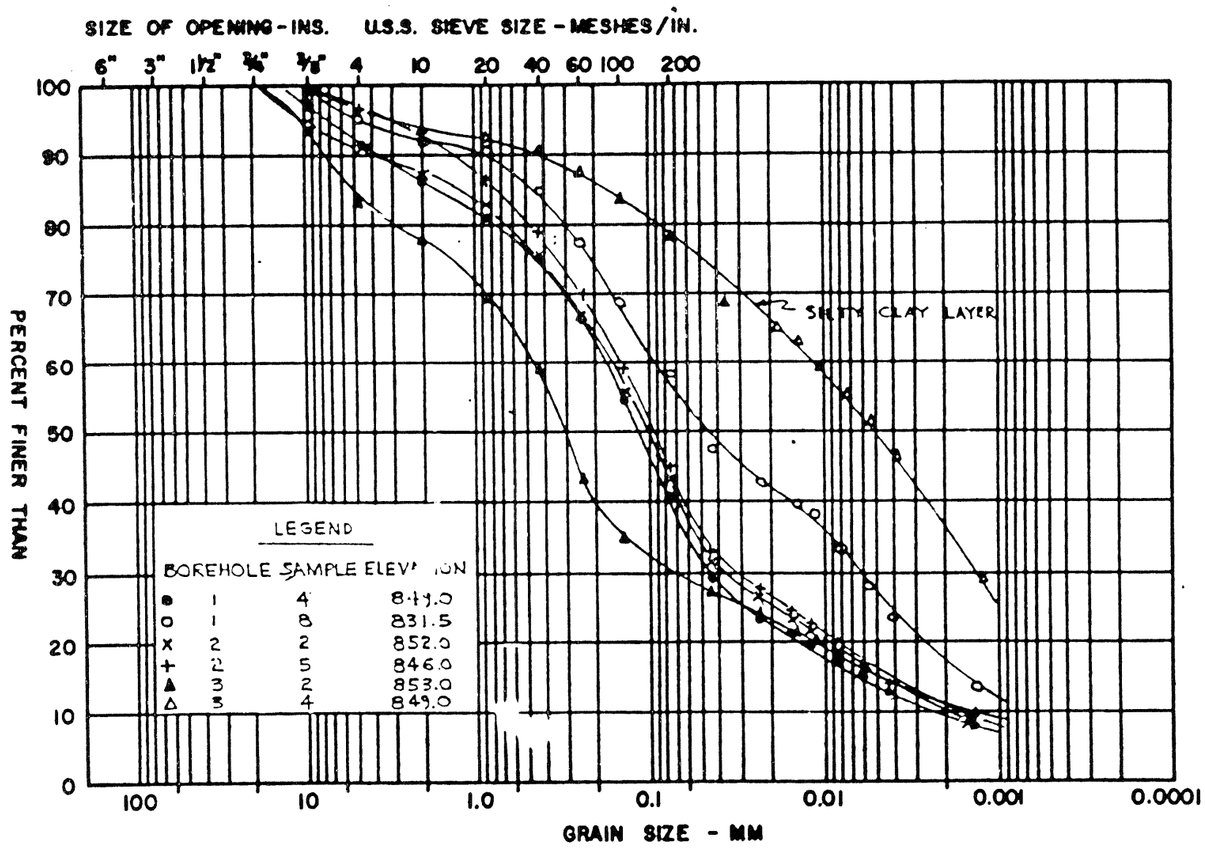
GOLDER & ASSOCIATES

DRAWN ...

CHECKED ...

PROJECT No. 6417

M.I.T. GRAIN SIZE SCALE



COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

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GRAIN SIZE DISTRIBUTION
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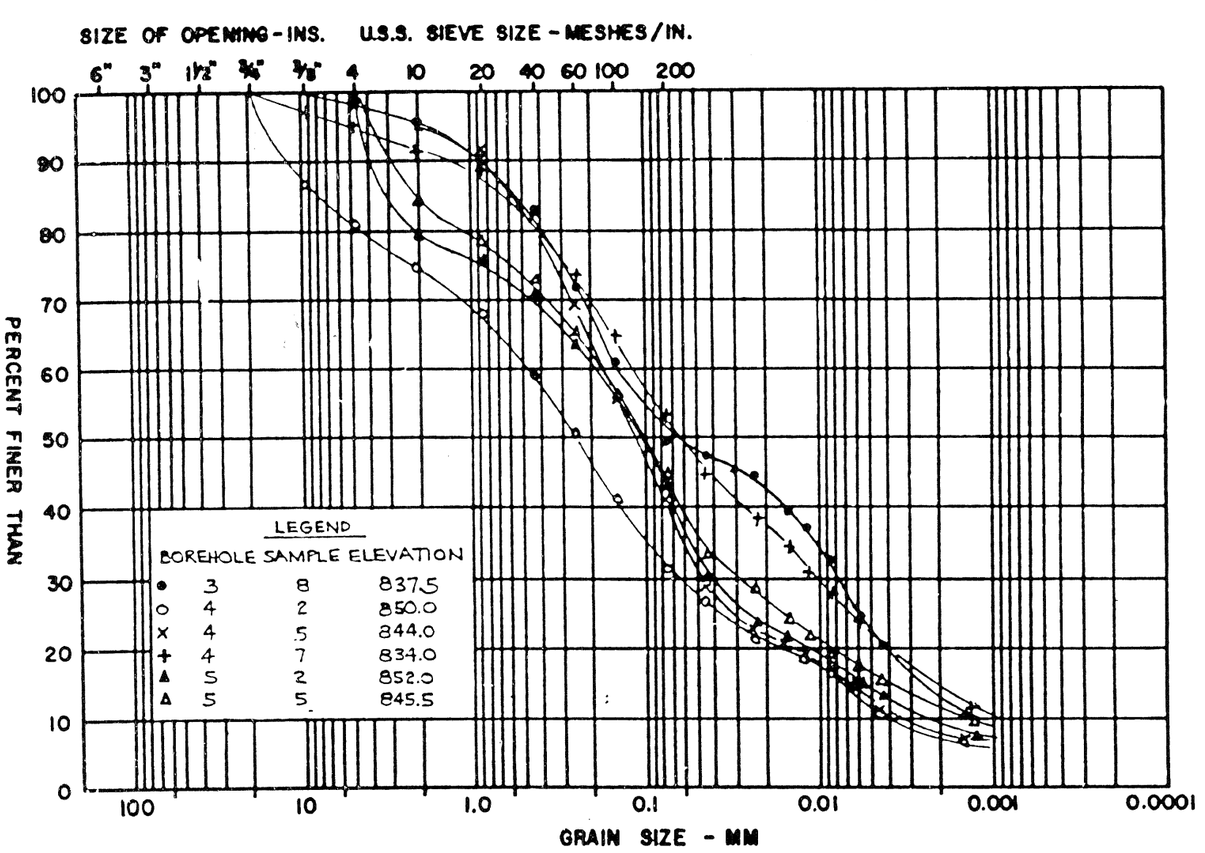
FIGURE 2

PROJECT No. 6417

M.I.T. GRAIN SIZE SCALE



COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

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GRAIN SIZE DISTRIBUTION
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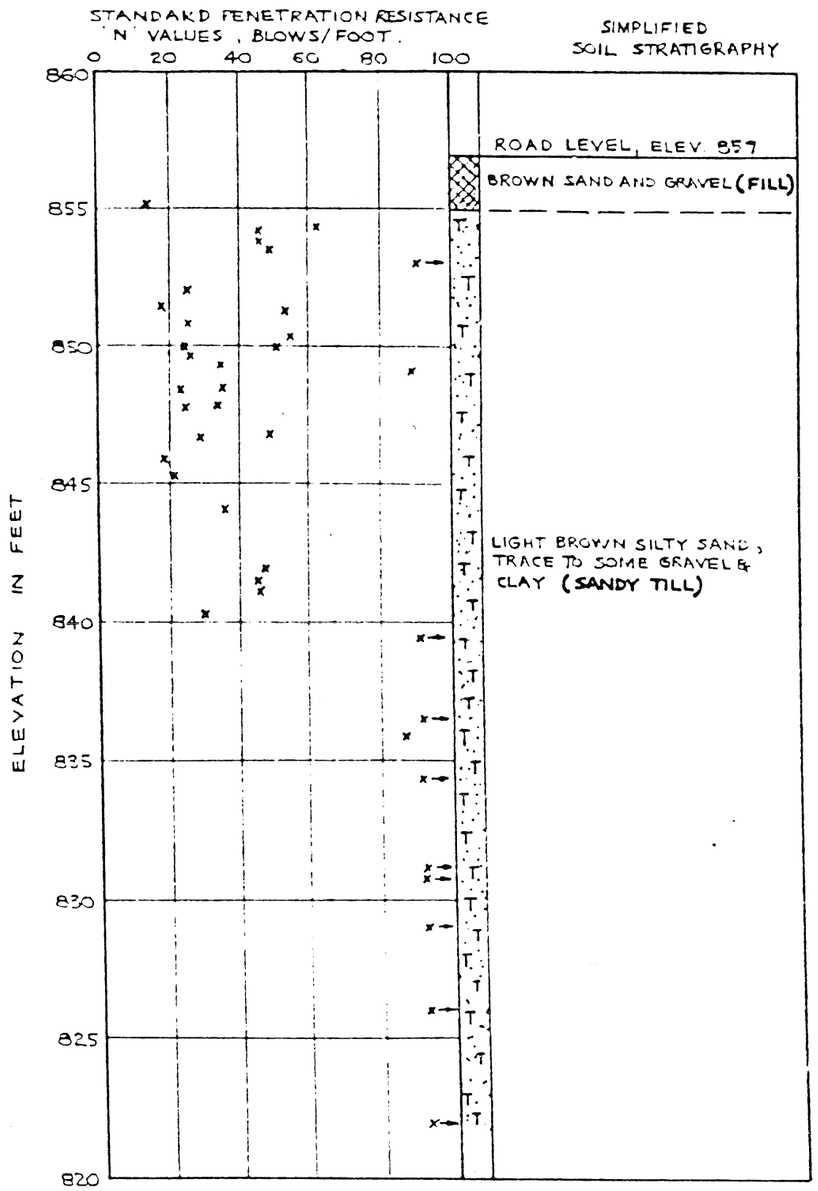
FIGURE 3

PROJECT No. 6413.7

SUMMARY PLOT OF STANDARD PENETRATION RESISTANCE VALUES VS ELEVATION

FIGURE 4



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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT DUCKWORTH STREET OVERPASS STRUCTURE SITE 30-140 HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11 G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

Submitted to:

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February 2002

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PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
DUCKWORTH STREET OVERPASS
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Figure 1 Grain Size Distribution Test Results – Silt

1.0 INTRODUCTION

Golder Associates Ltd. has been retained by URS Cole, Sherman (Cole, Sherman) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the ultimate widening of Highway 400 from 1 km south of Highway 89, northerly 30 km to Highway 11, in Simcoe County, Ontario. Foundation engineering services are required for the widening and / or replacement of eighteen existing overpass and underpass structures, as well as five structural culverts.

This report addresses the widening and / or replacement of the Duckworth Street overpass structure in Barrie, Ontario. A foundation investigation has been carried out, in which two boreholes were advanced and in-situ and laboratory testing was conducted, to determine the subsurface conditions at the site for this preliminary design study.

The terms of reference for the scope of work are outlined in Golder Associates' Proposal No. P01-1192, dated June 2000.

2.0 SITE DESCRIPTION

The existing Duckworth Street overpass is located about 2.5 km north of the Bayfield Street (Highway 26) interchange and 2.5 km south of the Highway 11 interchange in Barrie, Ontario. The MTO has designated this underpass as Structure Site 30-140.

At the site, Duckworth Street has been constructed in a cut up to about 5 m deep, with its grade between Elevations 257.5 m and 259 m under the highway; the Duckworth Street grade rises toward the south. The Highway 400 grade is at about Elevation 264.5 m, with embankments up to about 2 m in height in the vicinity of the overpass structure.

According to the general layout drawing for the existing single-span structure, which was provided by Morrison Hershfield (the structural designers for this preliminary study), the abutments and associated wing walls and retaining walls are supported on spread footings. The founding elevations rise toward the south, with the east abutment footings founded at Elevation 256.2 m to 257.6 m, and the west abutment footings founded at Elevation 256.3 m to 257.9 m. The abutment spread footings are about 2.3 m wide.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at this site in March 2001, at which time two boreholes were drilled. Highway 400 is oriented in an approximately east-west direction at this site; Boreholes B16-1 and B16-2 were advanced on the south and north sides of the highway, respectively, to between 6 m and 6.5 m depth below Duckworth Street grade.

The investigation was carried out using a bombardier-mounted B-57 drill rig supplied and operated by Master Soil Investigations Ltd. of Weston, Ontario. Both boreholes were advanced using hollow stem augers. Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. The water levels in the open boreholes were observed throughout the drilling operations, and a piezometer was installed in Borehole B16-1 to permit monitoring of the groundwater levels at the site.

The field work was supervised on a full-time basis by a member of our staff who located the boreholes in the field, directed the drilling, sampling, and in-situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder Associates' laboratory in Mississauga for further examination. Index and classification tests consisting of water content determinations and grain size distribution analyses were carried out on selected soil samples.

The borehole locations and elevations were surveyed by Callon Dietz, Ontario Land Surveyors. The borehole elevations are referenced to geodetic datum, and the northing and easting co-ordinates are referenced to the MTM NAD83 survey system. The borehole locations, together with elevations and northing and easting co-ordinates, are shown on the attached Drawing 1.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

This 30 km section of Highway 400 traverses, from south to north, the following physiographic regions as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, Third Edition, 1984): the Simcoe Lowlands; the Peterborough Drumlin Field; a second lobe of the Simcoe Lowlands; and the Simcoe Uplands. Along Highway 400, the Simcoe Lowlands are present from the southern limit of the project to just south of Innisfil Creek, and again from Essa Road (Simcoe Road 30, formerly Highway 27) to about 1 km north of Dunlop Street (Simcoe Road 90, formerly Highway 90). The Peterborough Drumlin Field occupies the belt between these lobes of the Simcoe Lowlands, extending from just south of Innisfil Creek, which is located about 1 km north of Highway 89, to Essa Road. The Simcoe Uplands extend from about 1 km north of Dunlop Street to beyond the northern limit of the project at Highway 11.

The two sections where Highway 400 crosses the Simcoe Lowlands consist of two lobes of a sand plain which include the shores of Kempenfelt Bay, the Nottawasaga River and Innisfil Creek. The surficial soils of these sections of the Simcoe Lowlands consist primarily of sand, although silt, clay or peat may be found in low-lying areas.

The surficial soils in the Peterborough Drumlin Field consist primarily of gravelly sand till or sand and gravel deposits. Drumlins (glacially-shaped hills) are more frequent in the southern portion of the section of the Peterborough Drumlin Field traversed by Highway 400. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins.

The surficial soils in the Simcoe Uplands physiographic region, in which the Duckworth Street site is located, are primarily sandy silt till deposits, known to contain occasional boulders. Low-lying areas may be infilled with shallow sand and gravel deposits, which are shoreline deposits of a former glacial lake that once flooded the area.

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in-situ and laboratory testing are given on the Record of Borehole sheets and Figure 1. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

Boreholes B16-1 and B16-2 were advanced on the south and north sides of the highway, respectively, from the Duckworth Street grade. The locations and ground surface elevations for these boreholes are shown on the attached Drawing 1. In summary, the soils below the Duckworth Street cut grade consist of minor fill overlying a deposit of dense to very dense sandy silt to silt, which is in turn underlain by a hard clayey silt till. A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil and Fill

About 100 mm of topsoil was encountered in both boreholes. The topsoil was underlain by silty sand or silty clay fill, containing trace rootlets and organics. The measured Standard Penetration Test (SPT) 'N' values were 28 and 12 blows per 0.3 m of penetration, indicating that the silty sand fill has a compact relative density, and the silty clay fill has a stiff consistency.

4.2.2 Sandy Silt to Silt

About 3 m to 4.5 m of sandy silt to silt was encountered below the fill / reworked soil in both boreholes. The surface of this deposit was at about Elevation 259 m on the south side of the highway, and at about Elevation 256 m on the north side.

The grain size distribution test results measured on two samples from this deposit are shown on Figure 1. The soils became wet below about 1 m and 3 m depth in Boreholes B16-1 and B16-2, respectively, at about Elevation 257 m to 256 m. The measured natural moisture contents ranged from 16 to 23 per cent.

The measured SPT 'N' values ranged from 42 to 136 blows per 0.3 m of penetration, but were generally greater than 80 blows per 0.3 m of penetration. The sandy silt to silt deposit therefore has a predominantly very dense relative density.

4.2.3 Clayey Silt Till

A deposit of clayey silt till, containing some sand, trace gravel, and cobbles, was encountered below the sandy silt to silt stratum in both boreholes. The surface of the till was encountered at about Elevation 254.5 m and 253 m in the boreholes on the south and north sides of the highway,

respectively. The deposit was not fully penetrated, but has a measured thickness of 1.5 m to 2.5 m at the borehole locations.

The measured natural moisture content on two samples of the till were 13 and 15 per cent. The SPT 'N' values measured in this deposit were greater than 100 blows per 0.3 m of penetration, indicating that the clayey silt till has a hard consistency. Atterberg Limits testing conducted on two samples of this deposit measured plastic limits of 13 and 17 per cent, liquid limits of 21 and 22 per cent, and plasticity indices of about 5 and 8 per cent. These test results indicate that the clayey silt is inorganic and of low plasticity.

4.3 Groundwater Conditions

The sandy silt to silt soils were wet below about Elevations 257 m and 256 m in the boreholes on the south and north sides of the highway, respectively. The groundwater level measured in the piezometer installed in Borehole B16-1 was at 1.9 m depth (Elevation 257.9 m) within the first few days after drilling. The water level in the piezometer rose to 1.6 m depth (Elevation 258.2 m) about one week later on March 19, 2001. It is likely that the groundwater level slopes downward to the north at this site, following the topography.

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

GOLDER ASSOCIATES LTD.

for Lisa C. Coyne, P.Eng.,
Geotechnical Engineer

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Principal

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

LCC/ASP/FJH/clg/dh
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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
DUCKWORTH STREET OVERPASS
STRUCTURE SITE 30-140
HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

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

It is understood that Highway 400 will be widened from its existing six-lane configuration to an interim configuration of eight lanes, and an ultimate configuration of ten lanes, and that an alternative for a twelve-lane express / collector system is under consideration between Molson Park Drive and Duckworth Street in Barrie. Throughout the project length, it is expected that the existing highway platform will be widened by between 13 m and 30 m. Widening and / or replacement of the existing Duckworth Street overpass structure will therefore be necessary.

Based on the general layout drawing for the existing single-span structure, the abutments and associated retaining and wing walls are supported on spread footings. The spread footings step upward to the south, with the east abutment footings founded at 256.2 m to 257.6 m, and the west abutment footings founded at Elevation 256.3 m to 257.9 m. Duckworth Street has been constructed in a cut up to about 5 m deep, with its grade between Elevations 257.5 m and 259 m under the highway. The Highway 400 grade is at about Elevation 264.5 m, with embankments up to about 2 m in height in the vicinity of the overpass structure.

5.2 Bridge Foundation Options

The native soils below the Duckworth Street cut level consist of very dense sandy silt to silt, overlying a deposit of hard clayey silt till, observed to contain cobbles. The groundwater level measured in the sandy silt to silt stratum was as high as Elevation 258 m in March 2001, about 1 m below the Duckworth Street grade on the south side of Highway 400.

Based on these subsurface conditions, consideration could be given to founding the widened portions of the structure on spread footings placed on the very dense sandy silt to silt stratum. If foundation excavation is carried out near to or below the groundwater level existing at the time of construction, groundwater control will be required to achieve the design bearing capacities and prevent loosening/disturbance of the foundation soil and undermining of the existing structure foundations. Consideration could also be given to the use of perched abutments, founded on spread footings placed on a compacted granular pad within the approach embankments, although this option is most suitable for a replacement structure or a structurally separate widening.

Also, if a new structure is to be built or the widening can be maintained structurally separate from the existing overpass, consideration could be given to supporting the abutments on steel H-pile foundations driven to found within the hard clayey silt till deposit. In this case, the pile caps would have to be perched above the Duckworth Street grade and some preaugering may be required in order to achieve the required pile length.

Preliminary recommendations for spread footings, including perched abutments, and for deep foundations are provided in the following sections.

5.3 Spread Footings

For preliminary design, spread footings may be founded on the very dense sandy silt to silt deposit. For preliminary design, the founding elevation should be taken at or below Elevation 256.3 m at the current north limit of Highway 400, and at or below Elevation 257.9 m at the south side of the highway. For widening of the existing overpass structure, the founding levels at the limits of the existing footings should be matched; the footings may step downward to the north, and upward to the south, provided that a minimum soil cover of 1.5 m is maintained along the footings. Any associated wing wall or retaining wall footings may be stepped upward away from the abutments such that a minimum soil cover of 1.5 m is maintained above the underside of the footings. In this case, a well-compacted granular pad will be required to support wing wall footings.

As noted elsewhere in this report, if the footing level is below the groundwater level at the time of construction, groundwater control in the water-bearing sandy silt to silt stratum will be imperative to achieve the geotechnical resistances recommended herein, and to prevent undermining of the foundations for the existing overpass structure.

Alternatively, consideration could be given to the use of abutment footings perched within well compacted granular "A" approach embankments. If the Duckworth Street structure is to be replaced or the widening can be maintained structurally separate from the existing overpass, this

option would be advantageous in that groundwater control in the very fine-grained sandy silt to silt stratum would be minimized or eliminated in the abutment areas. If this option is considered viable, investigation of the subsoil conditions above the Duckworth Street grade will be necessary during the detailed design stage.

5.3.1 Axial Geotechnical Resistance

Spread footings placed on a properly prepared base within the sandy silt to silt deposit at the design elevation given above may be designed for a factored geotechnical resistance at Ultimate Limit States (ULS) of 500 kPa, assuming a 2.5 m wide footing. The settlement of footings founded on the sandy silt to silt stratum will be dependent on the footing size and configuration, and on the applied loads. The settlement will be differential with respect to the existing structure; the majority of the settlement will occur during construction. For preliminary design purposes, the geotechnical resistance at Serviceability Limit States (SLS) may be taken as 400 kPa. The geotechnical resistances at ULS and SLS will have to be reviewed following the detailed design stage of subsurface investigation, once the footing size, configuration with respect to the existing structure, and loadings are known.

For spread footings placed within the approach embankments on a properly compacted Granular 'A' core, a factored geotechnical resistance at ULS of 900 kPa may be assumed for preliminary design. The geotechnical resistance at SLS will depend on the thickness of Granular 'A' and the consistency and thickness of the underlying soils; a value of 350 kPa may be assumed for preliminary design. These resistances will have to be confirmed during detailed design, once the composition and consistency of the upper soils at the site are confirmed.

The geotechnical resistances provided herein are given under the assumption that the loads will be applied perpendicular to the surface of the footings; where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the Ontario Highway Bridge Design Code (OHBDC).

5.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footing and the subsoils should be calculated in accordance with Section 6-8.4.3 of the OHBDC. The angle of friction between the concrete and the undisturbed silty sand to sand and silt till founding soils should be taken as 22 degrees; the corresponding coefficient of friction, tan δ, would then be 0.4. Where "perched" abutment footings are adopted, the angle of friction between the concrete footings and

the compacted Granular 'A' pad should be taken as 30 degrees; the corresponding coefficient of friction would be 0.58.

5.3.3 Frost Protection

All footings should be provided with a minimum of 1.5 m of soil cover for frost protection.

5.4 Driven Steel H-Piles

Consideration could be given to supporting the widened portion of the overpass structure on steel H-piles driven to found within the hard clayey silt till deposit. The surface of the till was encountered in the boreholes at about Elevation 254.5 m on the south side of Highway 400, and at about Elevation 253 m on the north side of the highway. Given the very dense nature of the overlying sandy silt deposit, it is expected that refusal would be met just below the surface of the clayey silt till. It is also possible that preaugering through the very dense sandy silt deposit may be required in order to achieve the required pile length for rotational flexibility purposes. This requirement will depend on how high the pile caps could be perched above the Duckworth Street grade.

5.4.1 Axial Geotechnical Resistance

For preliminary design, a pile founding level of Elevation 252 m, within the hard clayey silt till deposit, may be assumed. The factored axial resistance at ULS for steel HP 310 x 110 H-piles driven to this tip elevation may be taken as 1,600 kN. The axial resistance at SLS for a single pile, for 25 mm of settlement, may be taken as 1,400 kN.

As a guide, to achieve the above design resistances, the piles should be driven to a final set of 15 blows per 25 mm of penetration using a hammer with rated energy of about 50 kJ, and not exceeding 60 kJ. The actual set criteria should be established based on the Contractor's pile driving equipment. Provision should be made to re-tap selected piles to confirm the set after adjacent piles have been driven, in accordance with MTO's current Special Provision.

5.4.2 Resistance to Lateral Loads

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

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

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

where n_h is the constant of subgrade reaction
z is the depth (m)
B is the pile diameter (m)

For cohesive soils:

$$k_h = \frac{k_{s1}}{5B}$$

where B is the pile diameter (m) and
k_{s1} is the constant of horizontal subgrade reaction, as given below

Pile caps can be perched above the Duckworth Street grade to provide a greater driven length. The soils above the Duckworth Street grade were not investigated during this stage of field work. Based on regional geological mapping, it is likely that these soils will consist of sandy silt to silt soils, or clayey silt till, similar to that encountered below the cut grade. For compact to dense sandy silt to silt soils, the range in value of n_h may be taken as 10 MPa/m to 20 MPa/m in the structural analysis. For very stiff to hard clayey silt till, the range in value of k_{s1} may be taken as 25 MPa/m to 60 MPa/m in the structural analysis. These values will have to be confirmed during following the detailed design stage of the subsurface investigation.

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

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

5.4.3 Frost Protection

The pile caps should be provided with 1.5 m soil of cover for frost protection.

5.5 Lateral Earth Pressures

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

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the abutments and walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00

- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the abutment wall, in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be used in accordance with OPSS 501.06.

- The granular fill may be placed either in a zone with width equal to at least 1.5 m behind the back of the stem (Case I from OHBDC Figure 6-7.4.1) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II from OHBDC Figure 6-7.4.4).

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

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

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

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

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

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

5.6 Design of Embankment and Permanent Cut Slopes

The Highway 400 embankments in the vicinity of the Duckworth Street overpass are up to about 2 m in height. For the widening, the new side slopes should be formed at a maximum gradient of 2 horizontal to 1 vertical (2H:1V). The construction of the embankment should be carried out using conventional fill placement and compaction practices, and benching of the existing embankment side slopes should be carried out to key in the new fill.

The Duckworth Street cut is up to about 5 m in depth, with the road grade at about Elevation 258 m to 258.5 m. It is noted that the boreholes were advanced from approximately Duckworth Street grade, and so no subsurface information is available regarding the soils which will comprise the upper portion of the permanent cut slopes. Based on regional geological mapping, it is anticipated that the cut will be formed in sandy silt or clayey silt till, similar to that encountered below the Duckworth Street grade.

Based on the topographic information on the Engineering and Title Records plates and on site reconnaissance, the existing cut slopes are formed at a gradient of about 2 to 2.5 horizontal to 1 vertical (2H:1V to 2.5H:1V). For preliminary design purposes, a maximum gradient of 2.5H:1V may be assumed for any new permanent cut slopes. This design recommendation will have to be confirmed during the detailed design stage of the subsurface investigation.

It is noted that the groundwater level in the piezometer installed in Borehole B16-1 was measured at Elevation 258.2 m in March 2001. The base of the cut will therefore extend below this measured groundwater level. Protection of the lower portion of the slope face, for example using a drainage blanket, may be necessary to prevent sloughing of the toe of the permanent cut slopes.

5.7 Design and Construction Considerations

5.7.1 Groundwater Control

Based on available information, if spread footings are founded below about Elevation 258 m, the excavations will extend into water-bearing sandy silt to silt soils immediately adjacent to the spread footings for the existing overpass. For footing excavations carried down close to or below the groundwater level, an appropriate dewatering scheme will be necessary to prevent disturbance/loosening of the foundation subgrade, to facilitate footing excavation and concrete placement in dry conditions. Given the silty nature of the stratum, a well point system could be used to lower the groundwater level to at least 0.5 m below the founding level prior to excavation and construction of the footings; for this option, proper design of the screen and filter pack is critical to ensure there is no loss of fine soil particles and consequent settlement of the foundations for the existing overpass structure. Alternatively, interlocking steel sheeting may be driven around the foundation areas to a depth below founding level equal to the height of the groundwater above founding level. It must be ensured that a full enclosure is achieved around the footing areas in order for the steel sheeting option to provide the required groundwater control. This groundwater control alternative may be impractical considering the dense nature of the subsoil.

For perched abutments or for pile caps above the Duckworth Street cut grade, groundwater seepage into the excavations is expected to be minor. Pumping from properly-filtered sumps or a filtered drain placed at the base of the excavation should provide sufficient groundwater control during foundation works. Surface water run-off, which is expected to be more significant than groundwater seepage, should be directed away from the footing excavations.

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

5.7.2 Excavation

The footing excavations will extend a minimum of 1.5 m below the lowest surrounding grade through the sandy silt to silt stratum, which is water-bearing below about Elevation 258 m. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. The sandy silt to silt soils below Duckworth Street grade would be classified as Type 2 soil, assuming proper groundwater control is in place. Temporary open-cut slopes should therefore be maintained no steeper than 1 horizontal to 1 vertical (1H:1V) provided that proper groundwater control is implemented to stabilize the ground at and below excavation level. Where space restrictions dictate, adjacent to the new permanent cut slope, footing excavations could also be carried out within a braced excavation.

5.7.3 Settlement

Deformation of the ground due to foundation loading will result in settlement of the bridge abutments and superstructure. This settlement will be differential with respect to the existing overpass structure, and also could vary along the new structure depending on the variability and consistency/relative density of the founding soils. The potential for differential settlement should be reassessed during the detailed design stage, once the proposed bridge configuration is established.

5.7.4 Obstructions

Cobbles were noted within the clayey silt till deposit in Borehole B16-1. Cobbles and boulders are inherent in glacially-derived soils, and should therefore be expected during footing or pile cap excavation, steel H-pile installation and temporary shoring system installation, if such a system is required at the site. Where boulders are encountered within footing excavations, they should be removed and the sub-excavated areas should be backfilled with well-compacted Granular 'A' or lean concrete.

GOLDER ASSOCIATES LTD.

for Lisa C. Coyne, P.Eng.,
Geotechnical Engineer

Anne S. Poschmann, P.Eng.,
Principal

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

LCC/JLS/ASP/FJH/clg/dh
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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

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 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 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	c_u, s_u 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 (LV-laboratory vane test)
γ	unit weight

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

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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 δ
c' effective cohesion
c_u, s_u undrained shear strength (ϕ = 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

RECORD OF BOREHOLE No B16-1

1 OF 1 **METRIC**

PROJECT 001-1143F
W.P. 30-95-00
DIST SW HWY 400
DATUM Geodetic
LOCATION N 4919304.4; E 291019.9
BOREHOLE TYPE 108mm ID HOLLOW STEM AUGERS
DATE March 12/2001
ORIGINATED BY PKS
COMPILED BY LCC
CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
259.8	GROUND SURFACE				
0.0	Topsoil				
0.1	Silty Sand, trace organics (Fill) Compact Brown Moist	1	SS	28	
259.0					259
0.8	Sandy Silt to Silt, trace sand, trace gravel below 3m depth Very dense Moist becoming wet at 1m Brown to grey-brown	2	SS	84	
		3	SS	136	258
		4	SS	122	
					257
		5	SS	113	
		6	SS	110	256
		7	SS	122	255
254.6					
5.2	Clayey Silt, some sand, trace gravel (Till) Hard Grey Moist				254
	Cobbles inferred between 5.2m and 5.3m depth due to grinding of augers.	8	SS	100	
253.2					
6.6	END OF BOREHOLE				

Notes:
1. Water level in open borehole at 3m depth (Elev.256.8m) on completion of drilling.
2. Water level in piezometer measured as follows:
March 13, 2001: 1.9m depth (Elev.257.9m)
March 15, 2001: 1.9m depth (Elev.257.9m)
March 19, 2001: 1.6m depth (Elev.258.2m)

SOIL PROFILE | STRAT PLOT | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100)
SHEAR STRENGTH kPa (20 40 60 80 100)
O UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × REMOULDED

PLASTIC LIMIT w_p | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w_l
WATER CONTENT (%) (10 20 30)

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL
2 3 93 2

+ 3, × 3: Numbers refer to Sensitivity
O 3% STRAIN AT FAILURE

ON_MOT 0011143F.GPJ ON_MOT.GDT 1/2/02

Ministry of Transportation
Ontario

Foundation Engineering

RECORD OF BOREHOLE No B16-2 1 OF 1 METRIC

PROJECT 001-1143F
W.P. 30-95-00 LOCATION N 4919380.9; E 291015.5 ORIGINATED BY PKS
DIST SW HWY 400 BOREHOLE TYPE 108mm ID HOLLOW STEM AUGERS COMPILED BY LCC
DATUM Geodetic DATE March 13/2001 CHECKED BY ASF

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
256.8	GROUND SURFACE						
0.0	Topsoil						
0.1	Silty Clay, some sand, trace gravel, trace rootlets (Fill) Stiff Brown Moist	1	SS	12			
256.0 / 0.8	Sandy Silt to Silt, trace sand, trace gravel below 3m depth Dense to very dense Brown Moist to wet below 3m depth	2	SS	42	256		
		3	SS	77	255		0 7 92 1
		4	SS	97	254		
		5	SS	122			
253.1 / 3.7	Clayey Silt, some sand, trace gravel (Till) Hard Brown Moist	6	SS	145	253		
		7	SS	75/.08	252		
		8	SS	100/.15	251		
250.6 / 6.2	END OF BOREHOLE						

Note:
Base of borehole wet (at 6.2m depth or Elev.250.6m) on completion of drilling operations.

GROUND WATER CONDITIONS

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa
O UNCONFINED + FIELD VANE
● QUICK TRIAXIAL X REMOULDED
20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%) 10 20 30

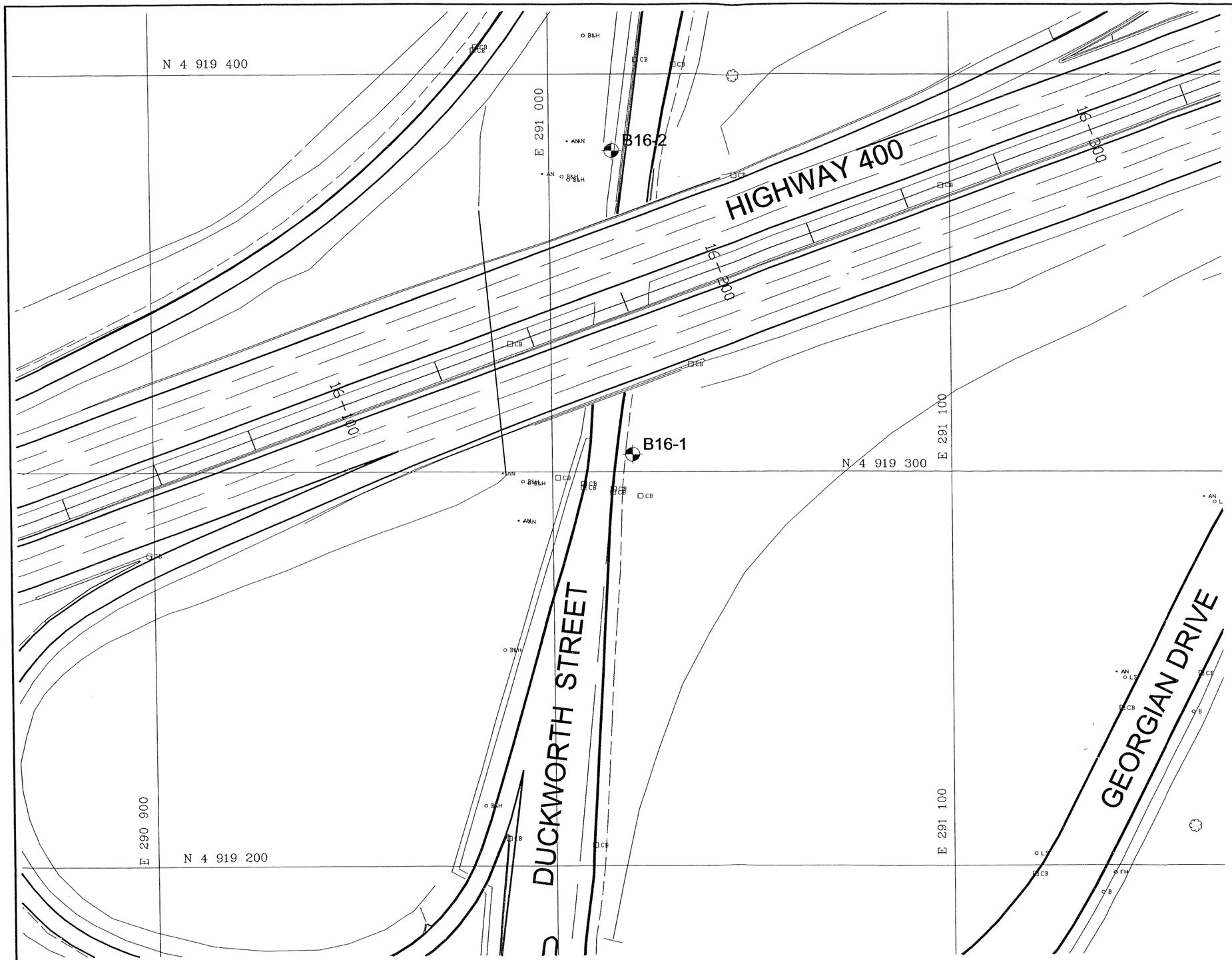
STRAT PLOT

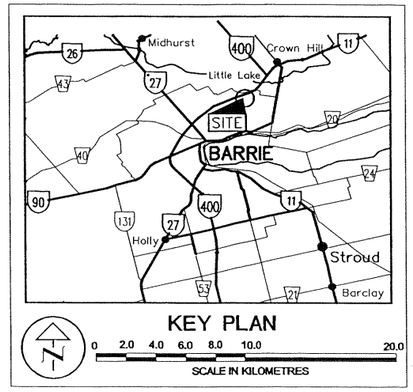
$+^3, \times^3$: Numbers refer to Sensitivity

$O^{3\%}$ STRAIN AT FAILURE

ON_MOT_0011143F.GPJ ON_MOT.GDT 1/2/02

DIST HWY 400 CONT. No. GWP No. 30-95-00	
DUCKWORTH STREET OVERPASS HWY 400 BOREHOLE LOCATION PLAN	SHEET
Golder Associates **Golder Associates Ltd.** MISSISSAUGA, ONTARIO, CANADA	





KEY PLAN

LEGEND

Borehole, previous investigation

Borehole, present investigation

No.	ELEVATION	LOCATION NORTHING	LOCATION EASTING
B16-1	259.8	4,919,304.4	291,019.9
B16-2	256.8	4,919,380.9	291,015.5

REFERENCE

This drawing was created from digital file "50213.dwg" provided by URS Cole Sherman

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

NO.	DATE	BY	REVISION

Geocres No.

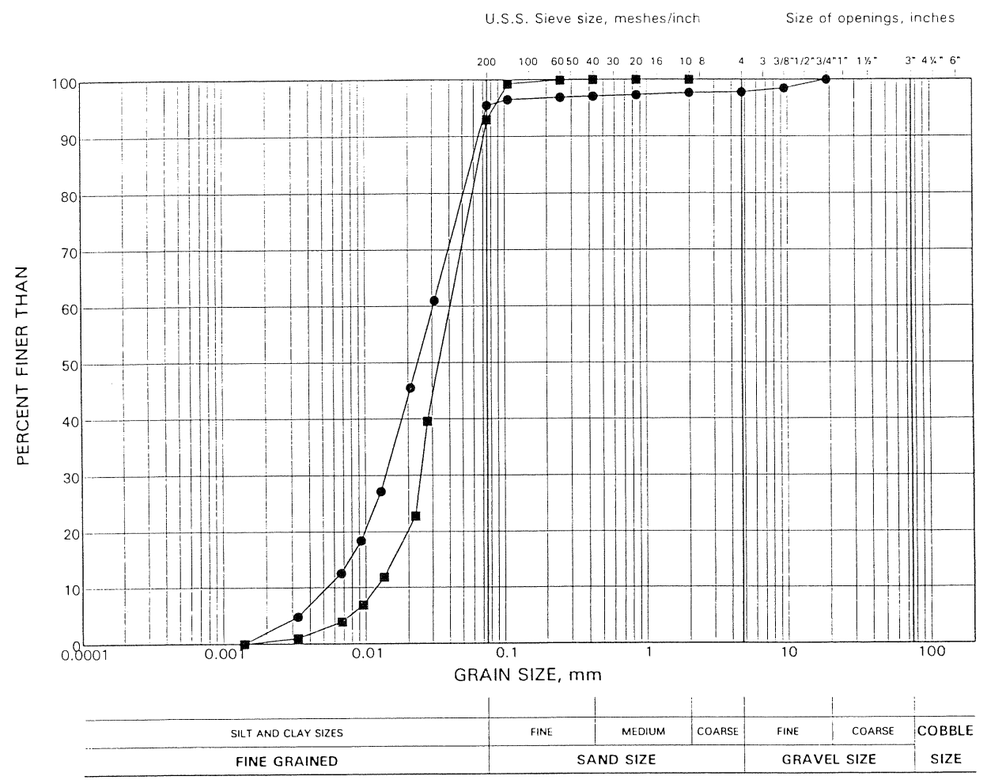
HWY. No. 400		PROJECT NO.: 001-1143F			
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-140		
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1		

P1143F21.DWG

plot scale 1:2mp.

GRAIN SIZE DISTRIBUTION TEST RESULTS
Silt

FIGURE 1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	16-1	4	257.3
■	16-2	3	255.0

Project 001-1143F

Golder Associates

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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT WILLOW CREEK BRIDGE STRUCTURE SITE 30-139 HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11 G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

Submitted to:

URS Cole, Sherman
75 Commerce Valley Drive East
Thornhill, Ontario
L3T 7N9

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January 2002 001-1143F-17

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Drawing 1 Willow Creek Bridge, Highway 400, Borehole Location Plan

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PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
WILLOW CREEK BRIDGE
STRUCTURE SITE 30-139
HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

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LIST OF DRAWINGS

Drawing 1 Willow Creek Bridge, Highway 400, Borehole Location Plan

LIST OF APPENDICES

Appendix A Records of Boreholes – 1970 Subsurface Investigation

1.0 INTRODUCTION

Golder Associates Ltd. has been retained by URS Cole, Sherman (Cole, Sherman) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the ultimate widening of Highway 400 from 1 km south of Highway 89, northerly 30 km to Highway 11, in Simcoe County, Ontario. Foundation engineering services are required for the widening and / or replacement of eighteen existing overpass and underpass structures, as well as five structural culverts.

This report addresses the widening and/or replacement of the existing Willow Creek bridge which is located at the northern end of Barrie. Existing subsurface data for this site from an investigation conducted for the Department of Highways, Ontario (DHO) in 1970 were used to determine the subsurface conditions for this preliminary design study. The DHO report was entitled "*Foundation Investigation Report for the Proposed Extension to the Structure at the Crossing of Hwy. 400 and Willow Creek – Township of Vespra – County of Simcoe (GEOCRES File No. 31D-166)*", dated January 1971.

The terms of reference for the scope of work are outlined in Golder Associates' Proposal No. P01-1192, dated June 2000.

2.0 SITE DESCRIPTION

The existing three-span Willow Creek bridge is located about 500 m south of the existing overpass structure at the Highway 11 interchange. The MTO has designated the Willow Creek bridge as Structure Site 30-139.

At this structure site, the Highway 400 grade is at about Elevation 232 m and the original ground surface was at about Elevation 230 m. At the bridge structure, the Willow Creek channel is about 15 m wide, with its base at about Elevation 229 m.

The existing three-span bridge was constructed in the late 1950s, and widened in the 1970s under Contract 77-112. According to the General Arrangement drawing for the structure widening, provided by Morrison Hershfield (the structural designers for this study), the existing structure is founded on driven steel H-piles. The underside of the abutment and pier pile caps is at about Elevation 228.5 m. The H-pile tips are at about Elevation 190 m.

3.0 INVESTIGATION PROCEDURES

The Department of Highways, Ontario (DHO) carried out a subsurface investigation at this site in November 1970. At that time, four boreholes were advanced in the vicinity of the abutments and piers for the then-proposed widening of the bridge. Boreholes 1 and 2 were located near the south abutment and pier, respectively; Boreholes 3 and 4 were located near the north pier and abutment, respectively. The boreholes were advanced to depths ranging from about 14 m to 34 m below ground surface, which varied from about Elevation 229.5 m to 231 m.

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

The borehole locations and elevations, referenced to the geodetic datum, were established by the DHO. Approximate northing and easting co-ordinates consistent with the MTM NAD83 survey system, currently in use on this project, have been determined by Golder Associates based on the borehole locations given in the 1971 report. The approximate borehole locations and northing and easting co-ordinates are shown on the attached Drawing 1.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

This 30 km section of Highway 400 traverses, from south to north, the following physiographic regions as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, Third Edition, 1984): the Simcoe Lowlands; the Peterborough Drumlin Field; a second lobe of the Simcoe Lowlands; and the Simcoe Uplands. Along Highway 400, the Simcoe Lowlands are present from the southern limit of the project to just south of Innisfil Creek, and again from Essa Road (Simcoe Road 30, formerly Highway 27) to about 1 km north of Dunlop Street (Simcoe Road 90, formerly Highway 90). The Peterborough Drumlin Field occupies the belt between these lobes of the Simcoe Lowlands, extending from just south of Innisfil Creek, which is located about 1 km north of Highway 89, to Essa Road. The Simcoe Uplands extend from about 1 km north of Dunlop Street to beyond the northern limit of the project at Highway 11.

The two sections where Highway 400 crosses the Simcoe Lowlands consist of two lobes of a sand plain which include the shores of Kempenfelt Bay, the Nottawasaga River and Innisfil Creek. The surficial soils of these sections of the Simcoe Lowlands consist primarily of sand, although silt, clay or peat may be found in low-lying areas.

The surficial soils in the Peterborough Drumlin Field consist primarily of gravelly sand till or sand and gravel deposits. Drumlins (glacially-shaped hills) are more frequent in the southern portion of the section of the Peterborough Drumlin Field traversed by Highway 400. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins.

The surficial soils in the Simcoe Uplands physiographic region are primarily sandy silt till deposits, known to contain occasional boulders. Low-lying areas may be infilled with shallow sand and gravel deposits, which are shoreline deposits of a former glacial lake that once flooded the area. The Willow Creek bridge site is located within the Simcoe Uplands region.

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes are given on the Record of Borehole sheets contained in Appendix A. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

Boreholes 1 and 2 were located near the south abutment and pier, respectively; Boreholes 3 and 4 were located near the north pier and abutment, respectively. The boreholes were advanced to depths ranging from about 14 m to 34 m. The approximate locations and ground surface elevations for these borings are shown on the attached Drawing 1.

In summary, the soils below Highway 400 grade at this site consist of very loose to dense silty sand overlying soft to firm silty clay, which in turn overlies a lower deposit of very dense silty sand. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Upper Silty Sand

The 1970 boreholes encountered a stratum of silty sand extending from ground surface to about 19.5 m depth (Elevation 210 m). Trace organics were encountered in the upper 3 m of the deposit. Layers of clayey silt were found between about 3 m and 6 m depth in Boreholes 1 and 4, and below about 4 m depth in Borehole 3. Typical grain size distribution curves for the silty sand are plotted on Figure 1 in Appendix A. The test results from Borehole 3 show the grading for clayey silt layers that were encountered within the silty sand.

The measured Standard Penetration Test (SPT) 'N' values ranged from 2 to 50 blows per 0.3 m of penetration, but were typically between about 5 and 20 blows per 0.3 m of penetration. The SPT results indicate that the silty sand has a predominantly loose to compact relative density.

4.2.2 Silty Clay to Clay

Below the silty sand at about Elevation 210 m, the boreholes encountered a 10 m thick deposit of silty clay to clay. The results of grain size distribution testing carried out on samples of this deposit are plotted on Figure 2 in Appendix A.

Atterberg Limits testing carried out on two samples of this material measured plastic limits of 15 and 22 per cent, liquid limits of 30 and 54 per cent, and plasticity indices of 15 and 32 per cent. These test results indicate that the clay is inorganic and of low to high plasticity. The natural moisture contents measured on the same samples of silty clay were 33 and 53 per cent, typically near the liquid limit of the material. In-situ vane shear testing measured undrained shear strengths of about 20 kPa to 35 kPa, indicating that the silty clay has a soft to firm consistency.

4.2.3 Lower Silty Sand

A lower deposit of silty sand underlies the silty clay deposit at about Elevation 199 m to 200 m. The deposit was not fully penetrated by any of the boreholes, but was at least 4 m thick as encountered in Borehole 2. Measured SPT 'N' values ranged from 20 to 133 blows per 0.3 m of penetration, but were typically greater than 100 blows per 0.3 m of penetration. These SPT results indicate that the lower silty sand generally has a very dense relative density.

4.3 Groundwater Conditions

The water levels observed in the open boreholes following completion of the November 1970 drilling operations were measured to be between Elevation 228.9 m and 229.2 m, approximately 0.3 m to 1.5 m below the then-existing ground surface and about 3 m below the current Highway 400 grade. The river water level was observed at the time of the investigation to be at about Elevation 229.2 m. It should be noted that groundwater levels are expected to fluctuate seasonally and are expected to rise during wet periods of the year.

GOLDER ASSOCIATES LTD.

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
WILLOW CREEK BRIDGE
STRUCTURE SITE 30-139
HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

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

It is understood that Highway 400 will be widened from its existing six-lane configuration to an interim configuration of eight lanes, and an ultimate configuration of ten lanes, and that an alternative for a twelve-lane express / collector system is under consideration between Molson Park Drive and Duckworth Street in Barrie. Throughout the project length, it is expected that the existing highway platform will be widened by between 13 m and 30 m, necessitating widening or replacement of the existing Willow Creek bridge structure.

At the structure site, the Willow Creek channel is about 15 m wide, with its base at about Elevation 229 m; the sides of the channel are formed of driven steel sheet-piling. Based on the general arrangement and layout drawings, the existing three-span structure is founded on driven steel H-piles. The underside of the abutment and pier pile caps is at about Elevation 228.5 m, and the tips of the H-piles are at about Elevation 190 m.

5.2 Bridge Foundation Options

The subsoils encountered in the boreholes put down during the 1970 investigation consist of very loose to dense silty sand overlying soft to firm silty clay, which in turn overlies a lower deposit of very dense silty sand.

Shallow foundations are not recommended for widening or replacement of the existing bridge, due to the presence of the relatively loose, water-bearing upper silty sand deposit and the underlying soft to firm silty clay deposit. It is recommended that the foundation elements for the new or widened structures be supported on steel H-piles driven to found within the lower, very dense silty sand deposit which underlies the site. Preliminary recommendations for driven steel H-pile foundations are provided in the following section.

5.3 Driven Steel H-Piles

Based on the results of the 1970 boreholes, the top of the very dense lower silty sand deposit is at about Elevation 200 m to 199 m. For preliminary design, a pile tip level of Elevation 190 m (similar to the existing pile tip elevation) may be assumed for the widening or replacement. Assuming that the new pile caps are placed at the same elevations as the existing pile caps, the piles would be about 35 m to 40 m long. It is pointed out that additional borehole investigation will be required at the detailed design stage in order to confirm the soil conditions at and below the pile tip elevations.

5.3.1 Axial Geotechnical Resistance

For preliminary design, the factored axial resistance at Ultimate Limit States (ULS) for steel HP 310 x 110 H-piles driven to found within the very dense lower silty sand deposit at or below the tip elevation given above may be taken as 1,400 kN.

The additional load due to embankment construction will induce consolidation settlement of the silty clay soils under the widened portions of the embankments. At the abutments and at piers which are located within the embankments, this consolidation will induce a downward movement of the soils adjacent to the piles, and negative skin friction will develop along the portion of the pile shaft embedded within the silty clay. The negative skin friction load on a single pile may be taken as 100 kN. This value is unfactored; an appropriate load factor should be applied in the structural analysis. To minimize the amount of negative skin friction that may develop on a pile, consideration could be given to placing the new approach embankment fill as early as possible to allow as much settlement as possible to occur prior to the driving of the piles. The embankment could also be surcharged to increase the magnitude of settlement prior to pile installation.

The axial resistance at Serviceability Limit States (SLS) for 25 mm of settlement may be taken as 1,000 kN.

To achieve the above design resistances, the piles should be driven using a hammer with rated energy not exceeding 60 kJ. Provision should be made to re-tap selected piles to confirm the set after adjacent piles have been driven, in accordance with MTO's current Special Provision.

5.3.2 Resistance to Lateral Loads

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

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

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

where n_h is the constant of subgrade reaction
z is the depth (m)
B is the pile diameter (m)

For cohesive soils:

$$k_h = \frac{k_{s1}}{5B}$$

where B is the pile diameter (m) and
k_{s1} is the constant of horizontal subgrade reaction, as given below

The piles will be driven through embankment fill, loose to compact silty sand, and soft to firm silty clay. For the embankment fill and loose to compact silty sand above the groundwater level (above about Elevation 229 m), the range in value of n_h may be taken as 5 MPa/m to 15 MPa/m in the structural analysis. For the loose to compact silty sand soils below the groundwater table, between Elevations 229 m and 210 m, the range in value of n_h may be taken as 2 MPa/m to 10 MPa/m. Between Elevations 210 m and 200 m, the range in value of k_{s1} may be taken as 5 MPa/m to 20 MPa/m.

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

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

5.3.3 Frost Protection

The pile caps should be provided with a minimum 1.5 m of soil cover for frost protection.

5.4 Lateral Earth Pressures

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

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the abutments and walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00

- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the abutment wall, in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be used in accordance with OPSS 501.06.

- The granular fill may be placed either in a zone with width equal to at least 1.5 m behind the back of the stem (Case I from OHBDC Figure 6-7.4.1) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II from OHBDC Figure 6-7.4.4).

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

Soil unit weight:	20 kN/m³
Coefficients of lateral earth pressure:	
Active, K_a	0.35
At rest, K_o	0.50

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

	Granular 'A'	**Granular 'B' Type II**
Soil unit weight:	22 kN/m³	21 kN/m³
Coefficients of lateral earth pressure:		
Active, K_a	0.27	0.31
At rest, K_o	0.43	0.47

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

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

5.5 Embankment Design

Based on the topographic information on the Engineering and Title Records plates and on site reconnaissance, the existing approach embankment side slopes are formed at a gradient of about 2 horizontal to 1 vertical (2H:1V). For the highway widening, the new side slopes should be formed at a maximum gradient of 2H:1V. The widening of the embankment should be carried out using conventional fill placement and compaction practices, and benching of the existing embankment side slopes should be carried out to key in the new fill. To ensure the stability of the abutment foreslopes, erosion and scour protection should be placed at the banks of the Willow Creek channel.

During detailed design, once the configuration of the structures and embankments is established, the recommended slope configuration will have to be reviewed to confirm the stability, and the embankment settlement will have to be assessed.

5.6 Design and Construction Considerations

5.6.1 Groundwater and Surface Water Control

The new pier pile caps will be located adjacent to Willow Creek, and the excavations for both the abutment and pier pile caps will extend into the water-bearing upper silty sand soils. A dewatering scheme will be necessary to facilitate pile cap excavation, pile driving and concrete placement in dry conditions. It is noted that excavation without groundwater control could result in loss of ground and consequent undermining of the existing pile caps.

The groundwater level should be lowered to at least 0.5 m below the pile cap underside prior to excavation and pile driving. In this regard, a well point or eductor system could be employed in the pile cap areas. Alternatively, interlocking steel sheeting may be driven around the foundation areas to a depth below founding level equal to the height of the groundwater above the founding level. It must be ensured that a full enclosure is achieved around the pile cap areas in order for the steel sheeting option to provide the required groundwater control.

5.6.2 Excavation

The pile cap excavations will extend a minimum of 1.5 m below lowest surrounding grade, and would be carried out through the existing embankment fill and the generally loose to compact, water-bearing upper silty sand deposit. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. Any fill and cohesionless native soils would be classified as Type 3 soils, assuming that proper groundwater control is in place prior to excavation. Temporary open-cut slopes should therefore be maintained no steeper than 1 horizontal to 1 vertical (1H:1V). Where space restrictions dictate, pile cap excavations could also be carried out within a braced excavation.

5.6.3 Vibrations

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

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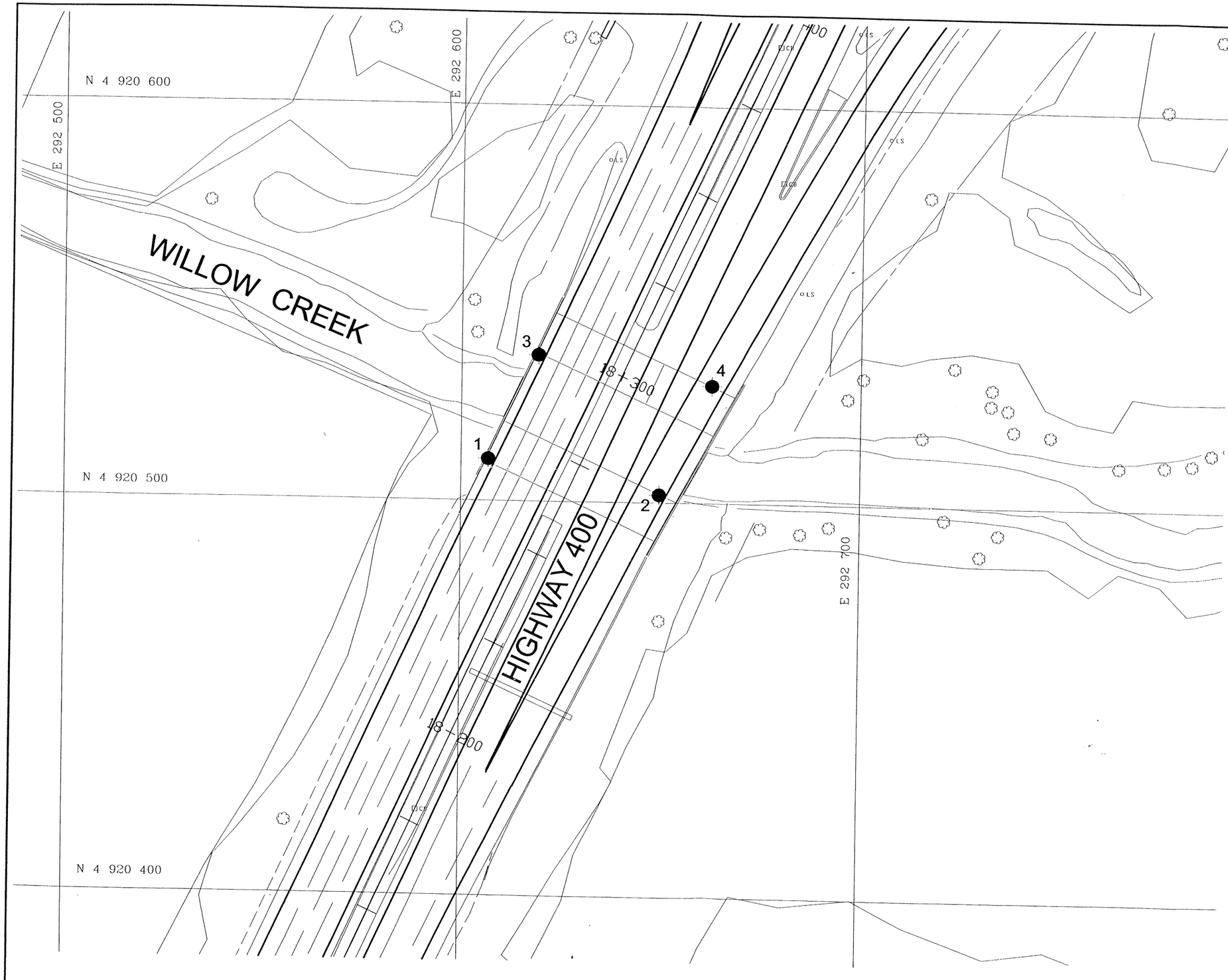
DIST HWY 400
CONT. No.
GWP No. 30-95-00

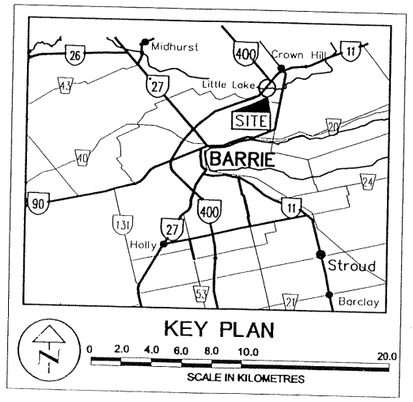
WILLOW CREEK BRIDGE
HWY 400
BOREHOLE LOCATION PLAN

SHEET

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA





LEGEND

- Borehole, previous investigation
- Borehole, present investigation

No.	ELEVATION	LOCATION NORTHING	LOCATION EASTING
1	230.7	4,920,511	292,607
2	229.4	4,920,502	292,650
3	229.5	4,920,537	292,619
4	230.8	4,920,530	292,663

REFERENCE

This drawing was created from digital file "50214.dwg" provided by URS Cole Sherman

0 10 20 30 40 50 100m

SCALE IN METRES

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

NO.	DATE	BY	REVISION

Geocres No.

HWY. No. 400		PROJECT NO.: 001-1143F	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-139
DRAWN: MHW	CHKD: LCC		

P1143F22.DWG

APPENDIX A

RECORDS OF BOREHOLES
1970 SUBSURFACE INVESTIGATION

FORM OB-MT-126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

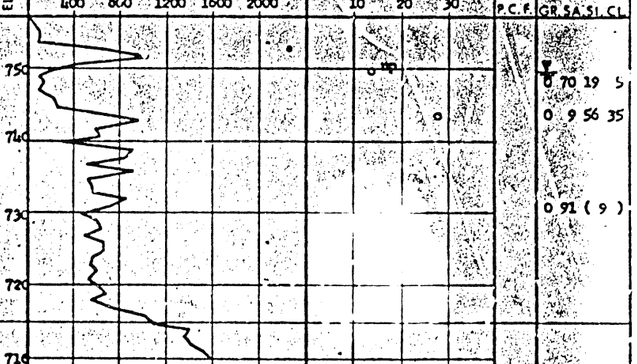
RECORD OF BOREHOLE No. 1

FOUNDATION SECTION

JOB 70-11097 LOCATION Hwy. 400 & Willow Creek Sta. 371 + 24 70' Lt. ORIGINATED BY VK
W.P. 105-70-08 BORING DATE Nov. 25, 1970 COMPILED BY VK
DATUM Geodetic BOREHOLE TYPE Washboring with BX & NX Casing CHECKED BY

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLE NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	REMARKS GR. SA. SI. CL.
230.7m (0.0m) 757.0	Ground Level						
0.0	with organic matter		1	SS	21		
			2	SS	7	750	0 70 19 5
	with layers of clayey silt		3	SS	4		
			4	SS	4		0 9 56 35
			5	SS	8	740	
			6	SS	16		
	Silty fine sand		7	SS	12	730	0 91 (9)
	Loose to compact		8	SS	15		
218.1m (12.6m) 715.5			9	SS	21	720	
41.5	End of Borehole						
216.4m (14.3m) 710.0						710	
47.0	End of D.C.P.						

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE 400 800 1200 1600 2000
LIQUID LIMIT w_L PLASTIC LIMIT w_P WATER CONTENT w WATER CONTENT % 10 20 30
BULK DENSITY γ P.C.F.



20 1.5 S 10 % STRAIN AT FAILURE

FORM OB-MT-126 (REV 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 2

FOUNDATION SECTION

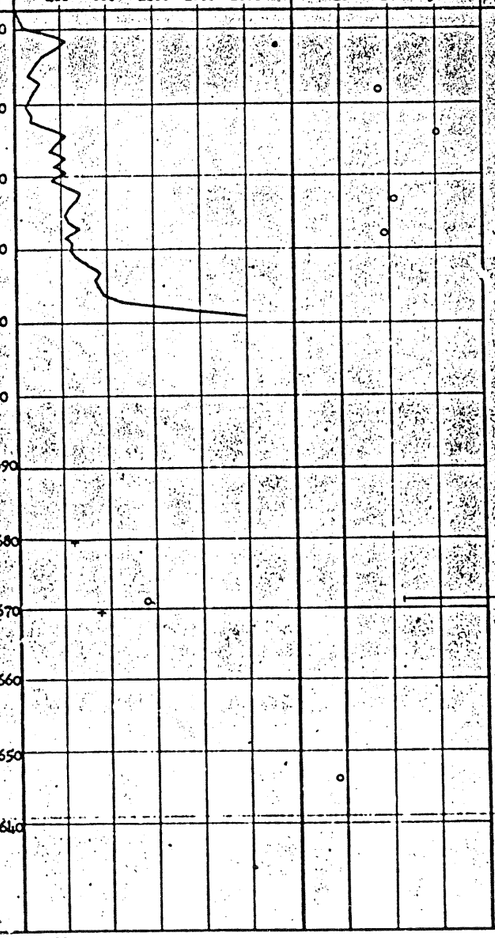
JOB 70-11097 LOCATION Hwy. 400 & Willow Cr. Sta. 371 + 61 70' Rt. ORIGINATED BY VK
W.P. 105-70-08 BORING DATE Nov. 5, 1970 COMPILED BY VK
DATUM Geodetic BOREHOLE TYPE Washboring with BX & NX Casing, Pendrill CHECKED BY //

ELEV. DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT	REMARKS
752.8	Ground Level				
0.0	with organic matter	1	SS	19	
		2	SS	14	
		3	SS	7	5 62 27 6
	Silty fine sand Loose to Dense	4	SS	3	
		5	SS	2	
		6	SS	8	
		7	SS	6	
		8	SS	10	0 62 34 4
		9	SS	14	
		10	SS	36	
		11	SS	9	
		12	SS	7	
		13	SS	50	
		14	SS	38	
688.8 / 64.0	Clayey silt to clay Soft to Firm	15	TW	PH	
		16	TW	PH	104.5 0 5 32 63
		17	TW	PH	
654.8 / 98.0	Silty sand Compact to Very Dense	18	SS	20	
		19	SS	101	8 61 (31)
641.3 / 111.5	End of Borehole	20	SS	133	

229.4m (0.0m) 209.9m (19.5m) 199.6m (29.9m) 195.5m (34.0m)

ELEV. SCALE: 750, 740, 730, 720, 710, 700, 690, 680, 670, 660, 650, 640

DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F. o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE 400 800 1200 1600 2000
LIQUID LIMIT w_L PLASTIC LIMIT w_P WATER CONTENT w WATER CONTENT % 10 20 30
BULK DENSITY γ P.C.F.



20 1.5 5 % STRAIN AT FAILURE 10

FORM OB-MT-126 (REV 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 3

FOUNDATION SECTION

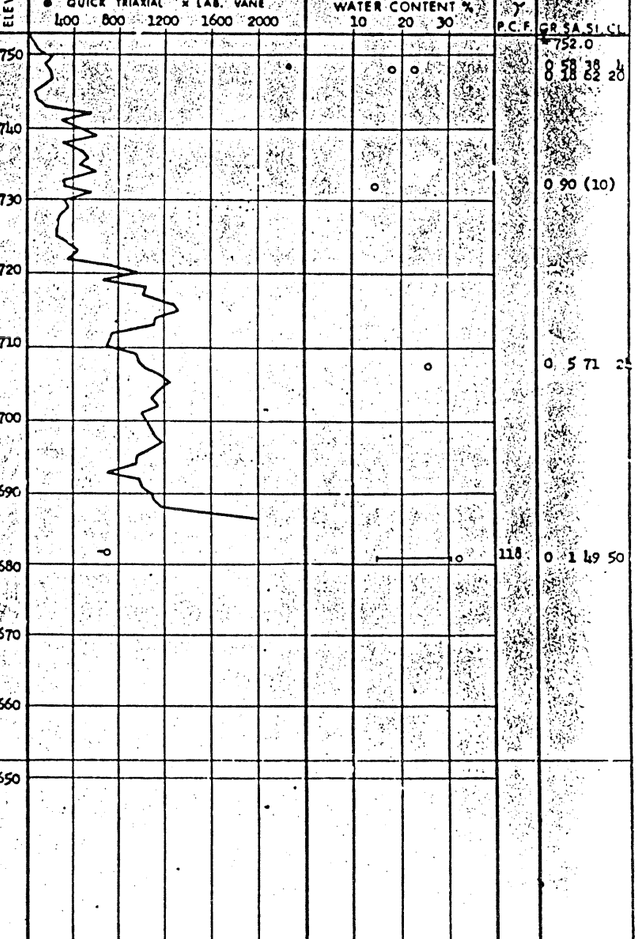
JOB 70-11097 LOCATION Hwy. 400 & Willow Cr. Sta. 372 + 24 70' Lt. ORIGINATED BY VK
W.P. 105-70-08 BORING DATE Nov. 23, 1970 COMPILED BY VK
DATUM Geodetic BOREHOLE TYPE Washboring with NX & BX Casing CHECKED BY //

ELEV. DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT	REMARKS
753.0	Ground Level				
0.0	with organic mater	1	SS	7	*752.0
		2	SS	3	0 58 38 4
		3	SS	12	0 18 62 20
	Silty sand, layers of clayey silt up to 10" thick below elev. 740.	4	SS	18	
		5	SS	20	
		6	SS	5	0 90 (10)
		7	SS	4	
		8	SS	6	
		9	SS	16	
		10	SS	6	0 5 71 25
	Loose to Compact	11	SS	13	
689.0 / 64.0	Clayey silt Firm	12	TW	PH	
		13	TW	PH	118 0 1 49 50
		14	TW	PH	
		15	TW	PH	
655.0 / 652.5 / 100.5	Silty Sand, Very Dense End of Borehole	16	SS	111	

229.5m (0.0m) 210.0m (19.5m) 199.6m 198.9m (30.6m)

ELEV. SCALE: 750, 740, 730, 720, 710, 700, 690, 680, 670, 660, 650

DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F. o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE 400 800 1200 1600 2000
LIQUID LIMIT w_L PLASTIC LIMIT w_P WATER CONTENT w WATER CONTENT % 10 20 30
BULK DENSITY γ P.C.F.



20 1.5 5 % STRAIN AT FAILURE 10

FORM OB-MT-126 (REV 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 4

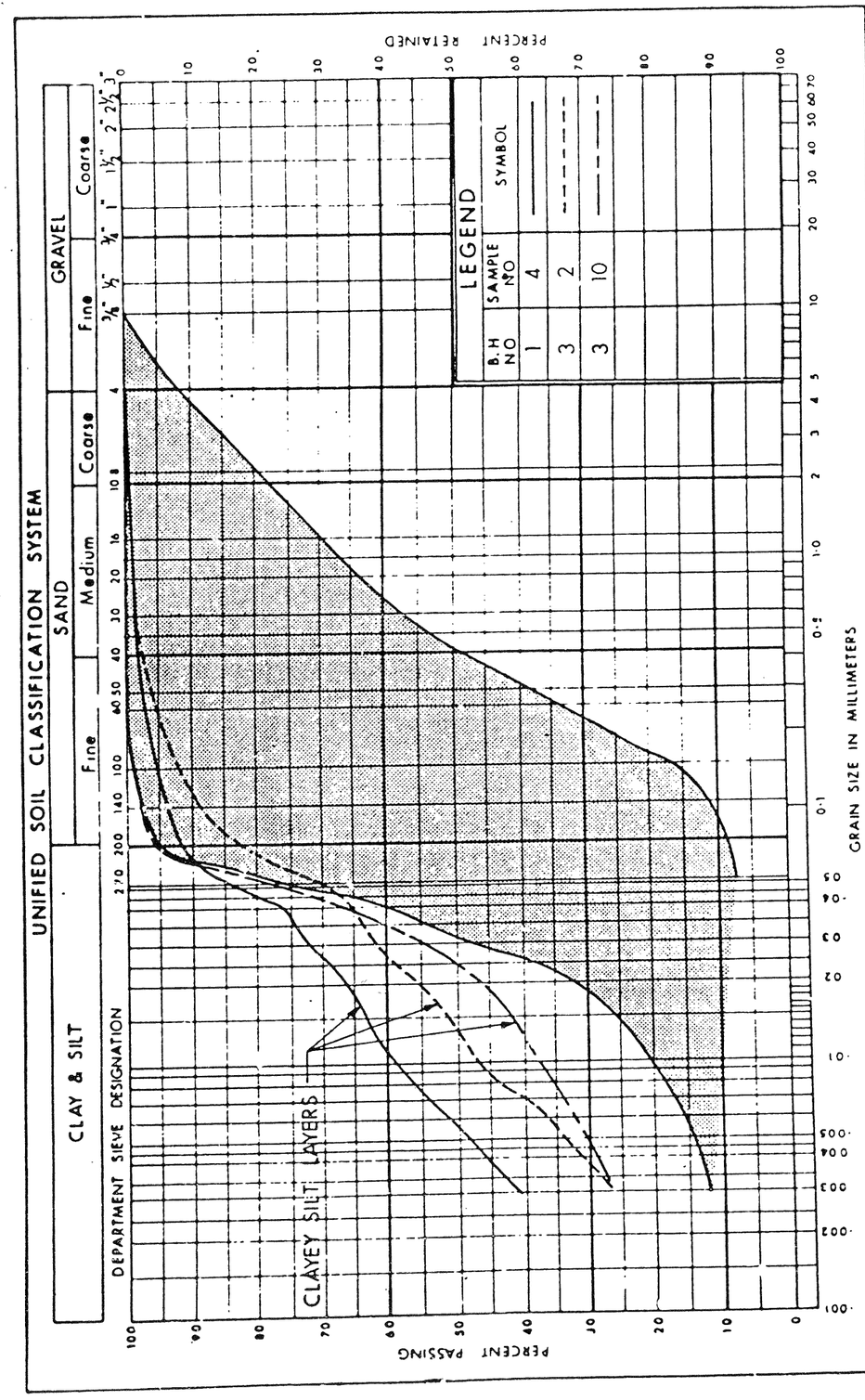
FOUNDATION SECTION

JOB 70-11097 LOCATION Hwy. 400 & Willow Cr. Sa. 372 + 60 70' Rt. ORIGINATED BY VK
W.P. 105-70-08 BORING DATE Nov. 23, 1970 COMPILED BY VK
DATUM Geodetic BOREHOLE TYPE Washboring with BX & NE Casing CHECKED BY

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLE NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS GR. SA. SI. CL.
230.8m (0.0m) 757.2	Ground Level							
0.0	with organic matter		1	SS	10			▼ 752.0
			2	SS	27	750		
	layers of clayey silt		3	TW	PM			0 5 84 11
	soft to firm		4	TW	PM		118	0 68 29 3
			5	TW	PM	740		
			6	SS	18			
			7	SS	16	730		2 72 (26)
			8	SS	19			
	Silty fine sand					720		
	Loose to Compact		9	SS	29			
						710		
215.1m (15.7m) 705.7			10	SS	7			
51.5	End of Borehole					700		
210.8m (20.0m) 691.7								
65.5	End of D.C.P.					690		

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE 400 800 1200 1600 2000
LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w
WATER CONTENT % 10 20 30

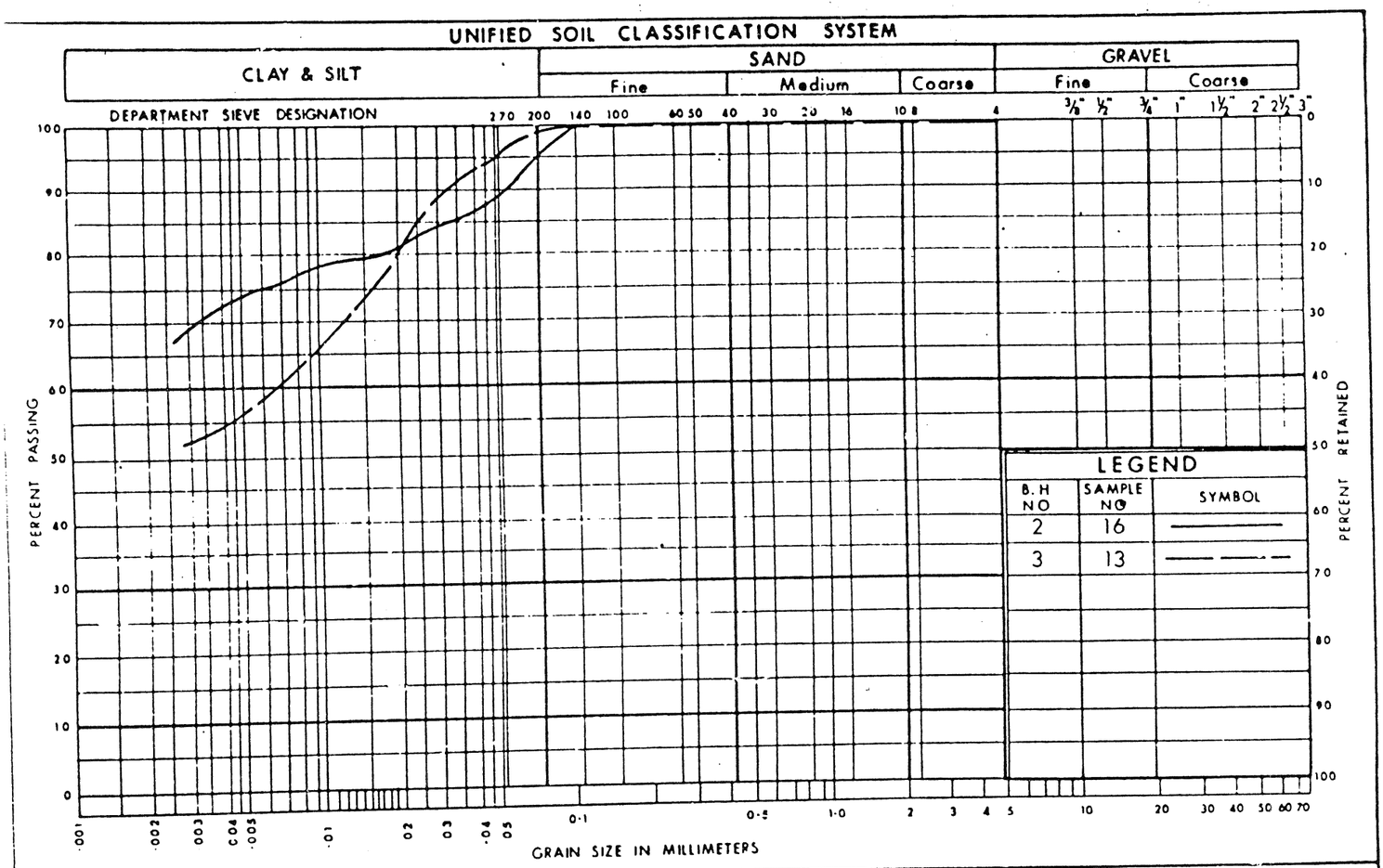
20 13 5 10 % STRAIN AT FAILURE



W.P No. 105 - 70 - 08
JOB No: 70 - 11097
FIG. 1

GRAIN SIZE DISTRIBUTION
SILTY SAND

DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION



DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION

GRAIN SIZE DISTRIBUTION
CLAYEY SILT TO CLAY

W.P. No.	105 - 70 - 08
JOB No.	70 - 11097
FIG.	2

Golder Associates Ltd.

2180 Meadowvale Boulevard
Mississauga, Ontario, Canada L5N 5S3
Telephone (905) 567-4444
Fax (905) 567-6561

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT CULVERTS STRUCTURE SITES 30-399, 571, 572, 573 & 415 HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11 G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

Submitted to:

URS Cole, Sherman
75 Commerce Valley Drive East
Thornhill, Ontario
L3T 7N9

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December 2001

001-1143F-18

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PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
CULVERTS
STRUCTURE SITES 30-399, 571, 572, 573 & 415
HIGHWAY 400 WIDENING FROM 1 KM SOUTH
OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

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1.0 INTRODUCTION

Golder Associates Ltd. has been retained by URS Cole, Sherman (Cole, Sherman) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the ultimate widening of Highway 400 from 1 km south of Highway 89, northerly 30 km to Highway 11, in Simcoe County, Ontario. Foundation engineering services are required for the widening and / or replacement of eighteen existing overpass and underpass structures, as well as five structural culverts.

This report addresses the five structural culverts. A foundation investigation has been carried out, in which two boreholes were advanced at each culvert location and in-situ and laboratory testing was conducted, to determine the subsurface conditions at the sites for this preliminary design study.

The terms of reference for the scope of work are outlined in Golder Associates' Proposal No. P01-1192, dated June 2000.

2.0 INVESTIGATION PROCEDURES

A subsurface investigation at the five culvert sites was carried out between October 26, 2000 and March 15, 2001, at which time ten boreholes were drilled. Boreholes C-1 and C-2 were advanced at the east and west side, respectively, of the existing culvert at Station 10+780. Boreholes C-3 and C-4 were advanced at the east and west side, respectively, of the existing culvert at Station 11+675. Boreholes C-5 and C-6 were advanced at the east and west side, respectively, of the existing culvert at Station 12+900. Boreholes C-7 and C-8 were advanced at the east and west side, respectively, of the existing culvert at Station 13+100. Boreholes C-9 and C-10 were advanced at the east and west side of the existing culvert at Station 16+730, respectively. The boreholes were advanced to between 8.1 m and 21.5 m depth below ground surface.

The investigation was carried out using a bombardier-mounted B-57 drill rig supplied and operated by Master Soil Investigations Ltd. of Weston, Ontario. The boreholes were advanced using 108 mm diameter solid stem augers. Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. The water levels in the open boreholes were observed throughout the drilling operations, and piezometers were installed in Boreholes C-1, C-4, C-5 and C-10, to permit monitoring of the groundwater levels at the site.

The field work was supervised on a full-time basis by members of our staff who located the boreholes in the field, directed the drilling, sampling, and in-situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder Associates' laboratory in Mississauga for further examination. Index and classification tests consisting of water content determinations, Atterberg Limits tests and grain size distribution analyses were carried out on selected soil samples.

The borehole locations and elevations were surveyed by Callon Dietz, Ontario Land Surveyors. The borehole elevations are referenced to geodetic datum, and the northing and easting co-ordinates are referenced to the MTM NAD83 survey system. The approximate borehole locations, together with elevations and northing and easting co-ordinates, are shown on the attached Drawings 1 to 5.

3.0 SITE GEOLOGY AND STRATIGRAPHY

3.1 Regional Geological Conditions

This 30 km section of Highway 400 traverses, from south to north, the following physiographic regions as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, Third Edition, 1984): the Simcoe Lowlands; the Peterborough Drumlin Field; a second lobe of the Simcoe Lowlands; and the Simcoe Uplands. Along Highway 400, the Simcoe Lowlands are present from the southern limit of the project to just south of Innisfil Creek (about 1 km north of Highway 89) and again from Highway 27 (Essa Road) to about 1 km north of Highway 90 (Dunlop Street). The Peterborough Drumlin Field occupies the belt between these lobes of the Simcoe Lowlands, extending from just south of Innisfil Creek to Highway 27 (Essa Road). The Simcoe Uplands extend from about 1 km north of Highway 90 (Dunlop Street) to beyond the northern limit of the project at Highway 11.

The two sections where Highway 400 crosses the Simcoe Lowlands consist of two lobes of a sand plain which include the shores of Kempenfelt Bay, the Nottawasaga River and Innisfil Creek. The surficial soils of these sections of the Simcoe Lowlands consist primarily of sand, although silt, clay or peat may be found in low-lying areas.

The surficial soils in the Peterborough Drumlin Field consist primarily of gravelly sand till or sand and gravel deposits. Drumlins (glacially-shaped hills) are more frequent in the southern portion of the section of the Peterborough Drumlin Field traversed by Highway 400. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins.

The surficial soils in the Simcoe Uplands physiographic region are primarily sandy silt till deposits, known to contain occasional boulders. Low-lying areas may be infilled with shallow sand and gravel deposits, which are shoreline deposits of a former glacial lake that once flooded the area.

3.2 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 given on the Record of Borehole Sheets and Figures 1 to 4. The approximate locations and ground surface elevations for the borings are shown on the attached Drawings 1 to 5. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

In summary, the subsoils at the sites consist of topsoil and fill overlying a deposit of hard clayey silt till. At some locations, a deposit of sand overlies the clayey silt till and at other locations, a deposit of silty sand till is present below the fill or clayey silt till. A detailed description of the subsurface conditions encountered in the boreholes at each of the culvert locations is provided in the following sections.

3.3 Culvert at Station 10+780, Structure Site 30-399

Boreholes C-1 and C-2 were drilled on the east and west sides of the existing culvert which extends under Highway 400 at Station 10+780. The ground surface at the borehole locations is at Elevations 226.5 m and 227.2 m in Boreholes C-1 and C-2, respectively. The approximate grade of Highway 400 at this location is about Elevation 228.5 m.

3.3.1 Topsoil and Fill

Between 300 mm and 600 mm of topsoil was encountered in the boreholes. In both boreholes, about 1.7 m of fill soil was encountered below the topsoil. The fill generally consists of clayey silt containing trace to some sand, trace gravel and trace to some organics. Standard Penetration Test (SPT) 'N' values of 8 and 11 blows per 0.3 m of penetration were measured in the fill, indicating that the clayey silt fill has a stiff consistency. The natural moisture content measured on one sample of the fill was about 16 percent.

3.3.2 Peat

A 1.1 m layer of fibrous peat was encountered below the fill in Borehole C-1 at about Elevation 224.2 m. The SPT 'N' value measured within the peat was 4 blows per 0.3 m of penetration indicating that the peat has a firm consistency. The natural moisture content measured on one sample of the peat was about 61 percent.

3.3.3 Sand

A 3.9 m thick deposit of sand containing some silt and trace gravel was encountered below the peat in Borehole C-1 at about Elevation 223.1 m. The SPT 'N' values measured within the sand

were between 44 and 80 blows per 0.3 m of penetration indicating that the sand has a dense to very dense relative density. The grain size distribution test result for a representative sample of this deposit is shown on Figure 1. The natural moisture content measured on samples of the sand range from 16 and 21 percent.

3.3.4 Clayey Silt Till

A deposit of clayey silt till underlies the sand in Borehole C-1 and the topsoil in Borehole C-2 at about Elevations 219.2 m and 225.1 m in the two boreholes, respectively. The till deposit extends to the maximum depth investigated (about Elevation 217 m).

The clayey silt till contains trace to some sand and gravel; the grain size distribution test result for a representative sample of this till is shown on Figure 2. The deposit also contains occasional sand seams / partings. The measured SPT 'N' values ranged from 31 blows to greater than 100 blows per 0.3 m of penetration indicating that this till deposit has a hard consistency.

The natural moisture contents measured on samples of the clayey silt till ranged from 14 to 25 percent. Atterberg Limits testing measured a plastic limit of 14 percent, a liquid limit of 32 percent, and a plasticity index of about 18 percent. The results of the Atterberg Limits testing indicate that the till is inorganic and of low plasticity.

3.4 Culvert at Station 11+675, Structure Site 30-571

Boreholes C-3 and C-4 were drilled on the east and west sides of the existing culvert which extends under Highway 400 at Station 11+675. The ground surface at the borehole locations is at Elevations 230.5 m and 230.8 m in Boreholes C-3 and C-4, respectively. The approximate grade of Highway 400 at this location is about Elevation 232.5 m.

3.4.1 Topsoil and Fill

Approximately 300 mm of topsoil was encountered below the ground surface in Borehole C-4. In Borehole C-3, 1.5 m of fill material was encountered below the ground surface. The fill consists of silty sand containing some gravel and trace organics. One SPT 'N' value measured within the fill was 3 blows per 0.3 m of penetration indicating a very loose relative density.

3.4.2 Clayey Silt

Beneath the topsoil in Borehole C-4, a 1.2 m thick layer of clayey silt with sand and trace organics was encountered. One SPT 'N' value measured within the clayey silt was 6 blows per 0.3 m of penetration indicating a firm consistency.

3.4.3 Silty Sand to Sand and Gravel

A 3.5 m thick deposit of silty sand was encountered below the clayey silt in Borehole C-4 at about Elevation 229.3 m. The silty sand contains trace clay, trace gravel and occasional clayey silt layers. The SPT 'N' values measured within the silty sand were between 10 and 77 blows per 0.3 m of penetration indicating that the silty sand has a compact to very dense relative density. The natural moisture content measured on representative samples of the deposit range from 14 to 25 percent.

The lower 0.7 m of the deposit consists of sand and gravel containing some silt and trace clay. One SPT 'N' value measured within the sand and gravel was 16 blows per 0.3 m of penetration indicating that the sand and gravel has a compact relative density.

3.4.4 Clayey Silt Till

A deposit of clayey silt till underlies the fill in Borehole C-3 and sand and gravel in Borehole C-4 at about Elevations 229.0 m and 225.8 m in the boreholes, respectively. The till deposit extends to the maximum depth investigated (between Elevations 221 m and 223 m).

The clayey silt till contains trace to some sand and gravel and occasional sand partings. The upper 4 m of the deposit in Borehole C-3 had measured SPT 'N' values ranged from 2 to 24 blows per 0.3 m of penetration indicating that this upper portion of the till deposit has a soft to very stiff consistency. Within the lower 4 m of the deposit in Borehole C-3 (below Elevation 225 m) and for the full stratum depth in Borehole C-4, the measured SPT 'N' values range from 32 blows to greater than 100 blows per 0.3 m of penetration indicating that this portion of the till has a hard consistency.

The natural moisture contents measured on samples of the clayey silt till ranged from 12 to 23 percent. Atterberg Limits testing measured plastic limits of 12 and 16 percent, liquid limits of 26 and 32 percent, and plasticity indices of 14 and 18 percent. The results of the Atterberg Limits testing indicate that the till is inorganic and of low plasticity.

In Borehole C-4, a 0.3 m thick layer of wet sand and gravel containing trace silt was present within the till deposit at about Elevation 223.5 m.

3.5 Culvert at Station 12+900, Structure Site 30-572

Boreholes C-5 and C-6 were drilled on the east and west sides of the existing culvert which extends under Highway 400 and the Service Road at Station 12+900. The ground surface at the borehole locations is at Elevation 249.9 m. The approximate grade of Highway 400 at this location is about Elevation 255.5 m.

3.5.1 Topsoil and Fill

Approximately 300 mm of topsoil was encountered below the ground surface in Borehole C-6. In both boreholes, 2.4 m to 6.9 m of fill material was encountered below the ground surface or topsoil. The fill in Borehole C-5 drilled through the Service Road consists of 0.8 m of sand and gravel underlain by 4.1 m of silty sand containing trace gravel and clay underlain by 2.0 m of sand and gravel containing trace silt and clay. In Borehole C-6, the 2.4 m of fill consisted of silty sand containing some gravel, trace clay and trace organics. The SPT 'N' values measured within the fill were between 8 and 68blows per 0.3 m of penetration indicating a very loose to very dense relative density.

3.5.2 Clayey Silt Till

A deposit of clayey silt till underlies the fill in both boreholes at about Elevations 243.0 m and 247.2 m in Boreholes C-5 and C-6, respectively. The till deposit extends to the maximum depth investigated (Elevation 235.6 m).

The clayey silt till contains trace of sand and trace to some gravel. The SPT 'N' values measured in the till ranged from 9 blows to greater than 100 blows per 0.3 m of penetration indicating that the till has a stiff to hard consistency. In general, the SPT 'N' values are greater than 40 blows per 0.3 m of penetration indicating a hard consistency.

The natural moisture contents measured on samples of the clayey silt till ranged from 7 to 13 percent. Atterberg Limits testing measured plastic limits of 9 and 11 percent, liquid limits

of 15 and 16 percent, and plasticity indices of about 5 and 7 percent. The results of the Atterberg Limits testing indicate that the clayey silt till is inorganic and of low plasticity.

3.6 Culvert at Station 13+100, Structure Site 30-573

Boreholes C-7 and C-8 were drilled on the east and west sides of the existing culvert which extends under Highway 400 at Station 13+100. The ground surface at the borehole locations is at Elevations 255.5 m and 260.6 m in Boreholes C-7 and C-8, respectively. The approximate grade of Highway 400 at this location is about Elevation 261.5 m.

3.6.1 Fill

About 10.7 m of fill material was encountered below the ground surface in Borehole C-8 which was drilled from the Highway 400 grade. The upper 4.6 m of fill consisted of clayey silt containing some sand and trace gravel. The SPT 'N' value measured in this fill was 39 blows per 0.3 m of penetration indicating a hard consistency. The lower 6.1 m of the fill consisted of silty sand containing trace to some gravel. The SPT 'N' values measured in of this fill were about 45 blows per 0.3 m of penetration indicating a dense relative density.

3.6.2 Clayey Silt Till

A 4.6 m thick deposit of clayey silt with sand till containing trace gravel was encountered below the ground surface in Borehole C-7. The SPT 'N' values measured in the till ranged from 20 to 26 blows per 0.3 m of penetration indicating that the till has a very stiff consistency.

The natural moisture contents measured on one sample of the clayey silt till was 12 percent. Atterberg Limits testing measured a plastic limit of 10 percent, a liquid limit of 16 percent, and a plasticity index of about 6 percent. The results of the Atterberg Limits testing indicate that the clayey silt till is inorganic and of low plasticity.

3.6.3 Silty Sand Till

A deposit of silty sand till underlies the fill or clayey silt till in the boreholes at about Elevations 250.9 m and 249.9 m in Boreholes C-7 and C-8, respectively. The till deposit extends to the maximum depth investigated (between Elevations 243.1 m and 239.1 m).

The silty sand till contains trace to some clay and trace to some gravel. Grain size distribution test results for representative samples of this till are shown on Figure 3. The natural moisture contents measured on samples of the silty sand till ranged from 5 to 11 percent. The SPT 'N' values measured in the silty sand till ranged from 19 blows to greater than 100 blows per 0.3 m of penetration indicating that the till has a compact to very dense relative density. In general the deposit becomes more dense with depth.

3.7 Culvert at Station 16+730, Structure Site 30-415

Boreholes C-9 and C-10 were drilled on the east and west sides of the existing culvert which extends under Highway 400 at Station 16+730. The ground surface at the borehole locations is at Elevations 288.0 m and 291.8 m in Boreholes C-9 and C-10, respectively. The approximate grade of Highway 400 at this location is about Elevation 296.0 m.

3.7.1 Topsoil

A layer of topsoil, between 600 mm and 700 mm thick was encountered below the ground surface in both boreholes. The SPT 'N' values measured within the topsoil were 8 and 10 blows per 0.3 m of penetration.

3.7.2 Clayey Silt Till

A deposit of clayey silt till underlies the topsoil in the boreholes. The till deposit extends to the maximum depth investigated (about Elevation 279.5 m).

The clayey silt till contains some to with sand and some gravel; the grain size distribution test result for a representative sample of this till is shown on Figure 4. The measured SPT 'N' values ranged from 14 blows to greater than 100 blows per 0.3 m of penetration, but typically greater than 40 bows per 0.3 m of penetration indicating that the till has a hard consistency.

The natural moisture contents measured on samples of the clayey silt till range from 6 to 8 percent. Atterberg Limits testing measured plastic limits of 9 and 10 percent, liquid limits of 15 and 17 percent, and plasticity indices of 5 and 7 percent. The results of the Atterberg Limits testing indicate that the clayey silt till is inorganic and of low plasticity.

3.8 Groundwater Conditions

The water levels in the boreholes were observed during and upon completion of drilling. Piezometers were installed in Boreholes C-1, C-5 and C-10 and sealed into the clayey silt till. A piezometer was installed in Borehole C-4 and was sealed into the silty sand to sand and gravel deposit. The details of the piezometer installations are shown on the Record of Borehole Sheets and the results of the water level readings are given in the table below.

Culvert Location	*Borehole Number*	*Water Level On Completion of Drilling*		*Water Level in Piezometer*					
				After Installation		*March 19, 2001*		*June 19, 2001*	
		Depth (m)	*Elevation (m)*	*Depth (m)*	*Elevation (m)*	*Depth (m)*	*Elevation (m)*	*Depth (m)*	*Elevation (m)*
10+780	C-1	6.6	221.9	---	---	3.6	224.9	---	---
	C-2	Dry	---	N/A	N/A	N/A	N/A	N/A	N/A
11+675	C-3	4.6	225.9	N/A	N/A	N/A	N/A	N/A	N/A
	C-4	2.3	228.5	4.1	226.7	1.4	229.4	---	229.4
12+900	C-5	---	---	6.9	243.0	5.3	244.6	---	244.6
	C-6	4.1	245.8	N/A	N/A	N/A	N/A	N/A	N/A
13+100	C-7	4.6	250.9	N/A	N/A	N/A	N/A	N/A	N/A
	C-8	---	---	N/A	N/A	N/A	N/A	N/A	N/A
16+730	C-9	5.2	282.8	N/A	N/A	N/A	N/A	N/A	N/A
	C-10	3.3	288.5	11.4	280.4	---	---	2.6	289.2

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

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
CULVERTS
STRUCTURE SITES 30-399, 571, 572, 573 & 415
HIGHWAY 400 WIDENING FROM 1 KM SOUTH
OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

4.0 ENGINEERING RECOMMENDATIONS

4.1 General

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

It is understood that Highway 400 will be widened from its existing six-lane configuration to an interim configuration of eight-lanes, and an ultimate configuration of ten-lanes, and that an alternative for a twelve-lane express / collector system is under consideration between Molson Park Drive and Duckworth Street in Barrie. Throughout the project length, it is expected that the existing highway platform will be widened by between 13 m and 30 m. Widening and / or replacement of five structural culverts in the area will be necessary.

The Highway 400 grade at the sites varies from Elevation 228.5 m at the southernmost site to about Elevation 296.0 m at the northernmost culvert site.

4.2 Foundation Options, Culvert at Station 10+780

The soils at the east side of the site consist of topsoil and fill overlying a peat deposit which is underlain by a dense to very dense sand deposit in turn underlain by hard clayey silt till. On the west side of the existing structure, the topsoil and fill is underlain by hard clayey silt till. Based on these subsurface conditions, it is recommended that the widening or replacement structure be supported on the hard clayey silt till or the dense to very dense sand.

The use of deep foundations is not recommended at this site due to the presence of suitable material at or just below the founding elevation of the existing culverts.

The invert of the existing twin corrugated steel pipe arch (CSPA) culverts varies between Elevation 224.4 m on the east side of Highway 400 to Elevation 224.1 m on the west side. Just north of the twin culverts exists a corrugated steel pipe (CSP) culvert with an invert at about Elevation 223.5 m. For preliminary design of the culvert, it is recommended that the peat deposit

as encountered on the east side of the highway extending to about Elevation 223.1 m, be sub-excavated and replaced as required. At the west side, with the base of the culvert at or below Elevation 224.4 m, the culvert will be founded on the hard clayey silt.

4.2.1 Axial Geotechnical Resistance

For the culvert founded on the properly prepared clayey silt till or on compacted granular placed on the underlying very dense sand deposit, a factored geotechnical resistance at Ultimate Limit States (ULS) of 600 kPa may be assumed. The geotechnical resistance at Serviceability Limit States (SLS) may be taken as 400 kPa. The geotechnical resistance at SLS will have to be reviewed following the detailed design stage of subsurface investigation once the culvert size, configuration and loadings are known.

4.3 Foundation Options, Culvert at Station 11+675

The soils on the east side of the existing structure consist of fill which is underlain by clayey silt till which has a firm to hard consistency below Elevation 228 m but is soft above this elevation. The soils at the west side of the site consist of topsoil overlying a deposit of compact to very dense silty sand to sand and gravel which is underlain by hard clayey silt till. Based on these subsurface conditions, it is recommended that the widening or replacement structure be founded on the compact silty sand or the firm clayey silt till. There is potential for differential settlement along the culvert due to the presence of the firm upper portion of the till on the east side. In this regard, consideration could be given to sub-excavating the firm clayey silt till to Elevation 226 m and replacing with compacted granular.

The use of deep foundations is not recommended at this site due to the presence of suitable material at or just below the founding elevation of the existing culvert.

The invert of the existing corrugated steel pipe arch (CSPA) culvert varies between 228.7 m on the east side of Highway 400 to 228.0 m on the west side. The soft clayey silt till must be sub-excavated to at least Elevation 228 m on the east side of the highway. The founding soils will therefore vary from firm to very stiff clayey silt till on the east side of the highway to compact silty sand on the west side.

4.3.1 Axial Geotechnical Resistance

For the culvert founded on the properly prepared compact silty sand or firm to very stiff clayey silt till deposits at or below Elevation 228.0 m a factored geotechnical resistance at Ultimate Limit States (ULS) of 450 kPa may be assumed. For preliminary design purposes, the geotechnical resistance at Serviceability Limit States (SLS) may be taken as 300 kPa. The geotechnical resistance at SLS will have to be reviewed following the detailed design stage of subsurface investigation, once the culvert size, configuration and loadings are known.

For the option of sub-excavating and replacing the firm clayey silt till (to Elevation 226 m) on the east side of the highway, a factored geotechnical resistance at ULS of 600 kPa and a corresponding SLS value of 450 kPa may be assumed.

4.4 Foundation Options, Culvert at Station 12+900

The soils at the site of the existing culvert structure consist of topsoil and fill underlain by generally hard clayey silt till.

The invert of the existing concrete box culvert varies between Elevation 245.2 m on the east side of the Service Road to Elevation 245.4 m on the west side of Highway 400. These invert levels are within the hard clayey silt till on the west side of the highway; however, sand and gravel fill was found extending from the culvert invert level to Elevation 243 m on the east side. This fill may be related to the existing culvert or the Service Road construction. For preliminary design, it should be assumed that sub-excavation to Elevation 243 m and replacement will be required on the east side of the highway.

The use of deep foundations is not recommended at this site due to the presence of suitable material at or just below the founding elevation of the existing culvert.

4.4.1 Axial Geotechnical Resistance

For the box culvert on the properly prepared hard clayey silt till deposit or on compacted granular fill placed on the underlying till, a factored geotechnical resistance at Ultimate Limit States (ULS) of 600 kPa may be assumed. For preliminary design purposes, the geotechnical resistance at Serviceability Limit States (SLS) may be taken as 400 kPa. The geotechnical resistance at SLS will have to be reviewed following the detailed design stage of subsurface investigation, once the culvert size, configuration and loadings are known.

4.5 Foundation Options, Culvert at Station 13+100

At the west end of the existing culvert, compact to very dense silty sand till is expected at and below about Elevation 250 m. At the east end of the culvert, a layer of very stiff clayey silt till is present below the ground surface and overlies the silty sand till encountered at Elevation 251 m. The invert of the existing concrete box culvert varies between Elevation 252.2 m on the east side of Highway 400 to Elevation 250.0 m on the west side. Based on the subsurface conditions encountered the widening or replacement structure will be founded within the compact to very dense silty sand till and / or very stiff clayey silt till.

The use of deep foundations is not recommended at this site due to the presence of suitable material at or just below the founding elevation of the existing culvert.

4.5.1 Axial Geotechnical Resistance

For the box culvert placed at or below Elevation 252 m on the east side and 250 m on the west side, a factored geotechnical resistance at Ultimate Limit States (ULS) of 450 kPa may be used. For preliminary design purposes, the geotechnical resistance at Serviceability Limit States (SLS) may be taken as 300 kPa. The road embankment is up to about 10 m in height in the vicinity of the culvert, and the embankment loading itself will have an impact on the settlement. The geotechnical resistance at SLS will have to be reviewed following the detailed design stage of subsurface investigation, once the footing size, configuration and loadings are known.

4.6 Foundation Options, Culvert at Station 16+730

The soils at the site of the existing culvert structure consist of topsoil underlain by generally hard clayey silt till. Based on these subsurface conditions, it is recommended that the widening or replacement structure be founded on spread footings placed on the hard clayey silt till. The invert of the existing concrete arch culvert varies between Elevation 285.5 m on the east side of Highway 400 to Elevation 286.0 m on the west side. A design founding level at or below these invert levels will be within the hard clayey silt till. Preliminary recommendations for spread footings are provided in the following sections.

The use of deep foundations is not recommended at this site due to the presence of suitable material at or just below the founding elevation of the existing culvert.

4.6.1 Axial Geotechnical Resistance

Spread footings placed on the properly prepared hard clayey silt till deposit at the design elevations given above may be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 650 kPa. This value assumes a footing width of 2 m and founding depth of 1.5 m. The settlement of footings founded on the clayey silt till will be dependent on the footing size and configuration, and on the applied loads. For preliminary design purposes, the geotechnical resistance at Serviceability Limit States (SLS) may be taken as 450 kPa. The geotechnical resistance at SLS will have to be reviewed following the detailed design stage of subsurface investigation, once the footing size, configuration and loadings are known.

The geotechnical resistances provided herein are given under the assumption that the loads will be applied perpendicular to the surface of the footings; where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the Ontario Highway Bridge Design Code (OHBDC).

4.6.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footing and the subsoils should be calculated in accordance with Section 6-8.4.3 of the OHBDC. The angle of friction between the concrete footings and the undisturbed hard clayey silt till founding soil should be taken as 25 degrees; the corresponding coefficient of friction, tan δ, would be 0.47.

4.6.3 Frost Protection

The footings should be provided with a minimum of 1.5 m of soil cover for frost protection.

4.7 Frost Protection

The design frost depth for the five culvert sites is 1.5 m.

4.8 Pipe Culverts, Bedding and Backfill

Circular pipe and pipe arch culverts should be designed for the full overburden pressure and live load assuming a soil unit weight of 21 kN/m^3.

The culverts should be provided with at least 400 mm of OPSS Granular 'A' bedding shaped to the underside of the pipe culverts. In accordance with OHBDC Clause 7-8.4, a 200 mm layer of that portion of the bedding which is in direct contact with the pipe arch should be left uncompacted to allow proper embedment of the pipe profile. The remaining portion of the bedding should be compacted to at least 95 percent of the Standard Proctor maximum dry density using suitable vibratory compaction equipment. Specifications provided by the pipe manufacturer with respect to bedding requirements should be adhered to if different from the above general requirements. The minimum depth of bedding will depend on the size of the pipe and should adhere to the manufacturer's specifications.

The backfill to the culvert should be free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B', Type II but with less than 5 percent passing the 200 sieve. Adequate and careful compaction of the backfill under the haunches is essential for the performance of the culvert. The backfill should be placed in lifts not exceeding 200 mm loose thickness and compacted to 95 percent Standard Proctor dry density. The fill depth during placement should be maintained equal on both sides of the culvert with one side not exceeding the other by more than 400 mm.

Backfill to the culverts should be in accordance with OPSD 803.010 and 803.02. For general backfilling within the future roadway limits, granular fill meeting the specifications for OPSS Granular B Type 2 should be used. Representative samples of the materials proposed for use should be submitted to a qualified laboratory for suitable testing and determination of laboratory Proctor values prior to placement and compaction. Inspection and testing should be carried out by qualified geotechnical personnel during fill placement and compaction.

Since the performance of the arch culvert is highly dependant on the compaction of the haunches, equipment restrictions exist. The suppliers' specifications should be followed with respect to compaction equipment tolerances in close proximity to the culvert. As a guide, small air operated tampers or vibrating pads are typically required for the compaction of the fill within 300 mm of the culvert sides while hand rollers or packers are typically specified for distances of up to 1.5 m from the culvert sides. Heavier compactors may generally be used for compaction of the remaining areas. Inspection and field density testing should be carried out by qualified geotechnical personnel during all earthfill placement operations to ensure that appropriate materials are used and adequate levels of compaction have been achieved.

4.9 Lateral Earth Pressures

The lateral pressures acting on the concrete arch culvert extensions for the culvert at Station 16+730 will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill and on the subsequent lateral movement of the structure. The following recommendations are made concerning the design of the abutments, in accordance with the OHBDC:

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' but with less than 5 percent passing the 200 sieve should be used as backfill behind the walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 percent of the material's Standard Proctor maximum dry density in accordance with OPSS501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the culvert granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 803.010 and 803.02.

- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the culvert, in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be used in accordance with OPSS 501.06.

- The granular fill may be placed either in a zone with width equal to at least 1.5 m behind the back of the stem (Case I from OHBDS Figure 6-7.4.1) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the footing (Case II from OHBDS Figure 6-7.4.3).

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

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

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

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

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

It should be noted that the above design recommendations and parameters assume level backfill and ground surface behind the culvert walls. Where there is sloping ground behind the walls (i.e. sloping upwards away from the culvert), the coefficient of lateral earth pressure must be increased to account for the slope.

4.10 Erosion Protection

Erosion protection to the culvert should be provided as appropriate. Consideration could be given to use of suitable non-woven geotextiles and rip-rap as required to provide erosion protection based on hydraulic requirements. An upstream clay seal and / or a cut-off wall should be provided to control seepage through the bedding below the culvert. The material specification for a clay seal should be as per OPSS 1205. In addition, sediment control such as silt fences, erosion control blanket may be required during construction and diversion of the creek to mitigate migration of fine soil particles in to the creek.

4.11 Construction Considerations

4.11.1 Subgrade Protection

For protection of the founding stratum a working mat of lean concrete should be placed as soon as practical after reaching the base of the excavation and following completion of inspection. This mat could be 150 mm thick and could serve as the bedding for the culvert provided drainage

under the culvert is not required and provided that an adequate level surface can be achieved with the lean concrete. Where drainage is required or where additional levelling is required, granular bedding should be provided.

4.11.2 Dewatering

Groundwater seepage into the excavations for culvert construction should be expected particularly at the sites where water-bearing deposits are present or interlayers of granular soil are present within the till deposits. In general, pumping from properly-filtered sumps or a filtered drain placed at the base of the excavation should provide sufficient groundwater control during foundation works. The creek / ditch waters will have to be diverted in order to permit construction in the dry. Surface water run-off, which is expected to be more significant than groundwater seepage, should be directed away from the footing excavations.

On the east side of the culvert at Station 10+780, sub-excavation of the peat to the surface of the sand will be extended some 2 m below the measured groundwater level. Dewatering will be required at this site to allow proper compaction on the granular backfill material.

Depending on the time of year for construction, the groundwater level at the culvert at Station 11+675 may be up to 1.5 m above the proposed founding level. In this case, groundwater control / dewatering would be required to maintain the integrity of the founding stratum. Groundwater control could consist of closed steel sheeting driven into the underlying clayey silt till.

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

4.11.3 Excavation

Culvert at Station 10+780

Based on the results of the investigation, the excavations may extend to a depth of up to 5.5 m below existing grade, through clayey silt fill, peat, and clayey silt till. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. The fill and peat would be classified as Type 3

soil and the hard clayey silt till soils would be classified as Type 1 soil. Temporary open-cut slopes should therefore be maintained no steeper than 2 horizontal to 1 vertical (2H:1V) through the sand and fills and 1H:1V through the till. Where space restrictions dictate, excavations could also be carried out within a braced excavation or trench box.

Culvert at Station 11+675

Based on the results of the investigation, the excavations may extend to a depth of up to 5.5 m below existing grade. The excavation would extend through silty sand fill, clayey silt, silty sand and clayey silt till. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. The fill would be classified as Type 3 soils. The clayey silt and silty sand would be classified as Type 4 soil. Temporary open-cut slopes should therefore be maintained no steeper than 3 horizontal to 1 vertical (3H:1V) through the clayey silt and sand deposits and 2H:1V through the fill. Where space restrictions dictate, excavations could also be carried out within a braced excavation or trench box.

Culvert at Station 12+900

Based on the results of the investigation, the excavations may extend to a depth of up to 12.5 m below existing grade depending on the proximity to the existing embankment. The excavation would extend through silty sand to sand and gravel fill and stiff to hard clayey silt till. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. The stiff to hard clayey silt till soils would be classified as Type 1 soil. The fill deposits would be classified as Type 3 soil. Temporary open-cut slopes should therefore be maintained no steeper than 2 horizontal to 1 vertical (2H:1V) through fill and 1H:1V through the till. Where space restrictions dictate, excavations could also be carried out within a braced excavation or trench box.

Culvert at Station 13+100

Based on the results of the investigation, the excavations will extend to a depth of up to 11.5 m below existing grade depending on the proximity to the existing embankment. The excavation would extend through clayey silt to silty sand fill, very stiff clayey silt till and compact to very

dense silty sand till. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. The very stiff clayey silt till and compact to very dense silty sand till soils would be classified as Type 1 soil. The fill would be classified as Type 3 soil. Temporary open-cut slopes should therefore be maintained no steeper than 2 horizontal to 1 vertical (2H:1V) through fill and 1H:1V through the tills. Where space restrictions dictate, excavations could also be carried out within a braced excavation or trench box.

Culvert at Station 16+730

Based on the results of the investigation, excavations for culvert footing construction will extend to a depth of up to 2.5 m below existing grade. The excavations would generally extend through stiff to hard clayey silt till. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. The clayey silt till soils would be classified as Type 1 soil. Temporary open-cut slopes should therefore be maintained no steeper than 1 horizontal to 1 vertical (1H:1V) through the tills. Where space restrictions dictate, footing excavations could also be carried out within a braced excavation or trench box.

GOLDER ASSOCIATES LTD.

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SEMP/ASP/FJH/clg
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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

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 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 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	c_u, s_u 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 (LV-laboratory vane test)
γ	unit weight

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

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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 δ
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

RECORD OF BOREHOLE No C-1

1 OF 1

METRIC

PROJECT 001-1143F
W.P. 30-95-00
DIST Central HWY 400
DATUM Geodetic

LOCATION N 4896326.9; E 292177.2
BOREHOLE TYPE 108mm ID SOLID STEM AUGERS AND CASING
DATE Oct.26/2000

ORIGINATED BY PKS
COMPILED BY LCC
CHECKED BY ASP

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_p / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_l — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
226.5	GROUND SURFACE								
0.0	Topsoil	1	AS						
225.9					226				
0.6	Clayey Silt, trace to some sand, trace gravel, trace to some organics (Fill) Stiff Brown Moist	2	SS	11			o		
		3	SS	8	225				
224.2									
2.3	Fibrous Peat Firm Black Moist	4	SS	4	224		61		
223.1		5	SS	66					
3.4	Sand, some silt, trace gravel Dense to very dense Grey Wet	6	SS	80	223		o		0 83 17 0
		7	SS	65	222				
		8	SS	44	221		o		
219.2					220				
7.3	Clayey Silt, trace sand and gravel, occasional sand seams/partings (Till) Hard Grey Moist	9	SS	57	219				
		10	SS	111	218		o		
216.9					217				
9.6	END OF BOREHOLE								

Notes:
1. Water level in open borehole at 4.6m depth (Elev.221.9m) upon completion of drilling.
2. Water level in piezometer at 1.6m depth (Elev.224.9m) on March 19, 2001.

+ 3, × 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

ON_MOT 001 1143F.GPJ ON_MOT.GDT 25/9/01

RECORD OF BOREHOLE No C-2 — 1 OF 1 — METRIC

PROJECT 001-1143F
W.P. 30-95-00
DIST Central HWY 400
DATUM Geodetic

LOCATION N 4896322.8; E 292130.0
BOREHOLE TYPE 108mm ID SOLID STEM AUGERS AND CASING
DATE Nov.1/2000

ORIGINATED BY AZ
COMPILED BY LCC
CHECKED BY ASP

ELEV / DEPTH	SOIL PROFILE — DESCRIPTION	SAMPLES — NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
227.2	GROUND SURFACE				227		
0.0 / 226.9	Topsoil	1	SS	7			
0.3	Clayey Silt, some sand (Fill)				226		
225.1					225		
2.1	Clayey Silt, some sand, trace gravel (Till) Hard Grey Moist	2	SS	37	224		
		3	SS	54	223		2 11 55 32
		4	SS	55	222		
		5	SS	46	221		
		6	SS	37	220		
		7	SS	31	219		
		8	SS	33	218		
217.4 / 9.8	END OF BOREHOLE						

Note:
1. Open borehole dry upon completion of drilling.

DYNAMIC CONE PENETRATION RESISTANCE PLOT — SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100)

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)

+³, ×³: Numbers refer to Sensitivity O³% STRAIN AT FAILURE

RECORD OF BOREHOLE No C-3 — 1 OF 1 — METRIC

PROJECT 001-1143F
W.P. 30-95-00
DIST Central HWY 400
DATUM Geodetic

LOCATION N 4897209.0; E 291968.1
BOREHOLE TYPE 108mm ID SOLID STEM AUGERS AND CASING
DATE Oct.26/2000

ORIGINATED BY AZ
COMPILED BY LCC
CHECKED BY ASP

ELEV / DEPTH	SOIL PROFILE — DESCRIPTION	SAMPLES — NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
230.5	GROUND SURFACE					230		
0.0	Silty Sand, some gravel, trace organics (Fill) Very loose Brown Moist	1						
229.0		2	SS	3		229		
1.5	Clayey Silt, trace to some sand, trace gravel (Till) Soft to very stiff Grey Moist	3	SS	2		228		
		4	SS	15				
		5	SS	8		227		
		6	SS	7				
		7	SS	24	▽	226		
225.0								
5.5	Clayey Silt, trace sand and gravel, occasional sand partings (Till) Hard Grey Moist	8	SS	104		225		
		9	SS	68		224		
		10	SS	96		223		
220.9 / 9.6	END OF BOREHOLE					222		
						221		

Note:
1. Water level in open borehole at 4.6m depth (Elev.225.9m) upon completion of drilling.

DYNAMIC CONE PENETRATION RESISTANCE PLOT — SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100)

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)

+³, ×³: Numbers refer to Sensitivity O³% STRAIN AT FAILURE

RECORD OF BOREHOLE No C-4

1 OF 1

METRIC

PROJECT 001-1143F
W.P. 30-95-00
DIST Central HWY 400
DATUM Geodetic

LOCATION N 4897193.6; E 291920.1
BOREHOLE TYPE 108mm ID SOLID STEM AUGERS AND CASING
DATE Oct.31/2000

ORIGINATED BY AZ
COMPILED BY LCC
CHECKED BY ASP

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³
230.8	GROUND SURFACE				
0.0 / 230.5	Topsoil				
0.3	Clayey Silt with sand, trace organics Firm Brown Moist	1	SS	6	
229.3					
1.5	Silty Sand, trace clay, trace gravel, occ. clayey silt layers Compact to very dense Brown to grey Moist to wet	2	SS	12	
		3	SS	10	
		4	SS	20	Non-Plastic
226.5		5	SS	77	
4.3	Sand and Gravel, some silt, trace clay Compact Grey Wet	6	SS	16	
225.8 / 5.0	Clayey Silt, trace to some sand, trace gravel (Till) Very stiff to hard Grey Moist	7	SS	32	
		8	SS	38	
	Wet sand and gravel, trace silt from 7.3m to 7.6m depth.	9	SS	60	
222.6 / 8.2	END OF BOREHOLE				

GROUND WATER CONDITIONS

ELEVATION SCALE: 230, 229, 228, 227, 226, 225, 224, 223

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100)
SHEAR STRENGTH kPa
O UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × REMOULDED
(20 40 60 80 100)

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%) (10 20 30)

REMARKS & GRAIN SIZE DISTRIBUTION (%)
GR SA SI CL

Notes:
1. Water level in open borehole at 2.3m depth (Elev.228.5m) upon completion of drilling.
2. Water level in piezometer after installation at 4.1m depth (Elev.226.7m).
3. Water level in piezometer at 1.4m depth (Elev.229.4m) on March 19, 2001.

+³, ×³: Numbers refer to Sensitivity O³% STRAIN AT FAILURE

RECORD OF BOREHOLE No C-5

1 OF 2

METRIC

PROJECT 001-1143F
W.P. 30-95-00
DIST Central HWY 400
DATUM Geodetic

LOCATION N 4898392.1; E 291770.3
BOREHOLE TYPE 108mm ID SOLID STEM AUGERS AND CASING
DATE Nov.1/2000

ORIGINATED BY AZ
COMPILED BY LCC
CHECKED BY ASP

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³
249.9	GROUND SURFACE				
0.0	Sand and Gravel (Fill) Loose Grey Moist	1	SS	8	
249.1					
0.8	Silty Sand, trace gravel and clay (Fill) Very dense Brown Moist	2	SS	68	
245.0					
4.9	Sand and Gravel, trace silt and clay (Fill) Dense Grey Wet	3	SS	48	
243.0					
6.9	Clayey Silt, trace sand and gravel (Till) Very stiff to hard Brown to grey below 8.2m depth Moist	4	SS	19	
		5	SS	47	
		6	SS	44	
		7	SS	67	
		8	SS	66	
		9	SS	182	
		10	SS	101	
235.7 / 14.2					

GROUND WATER CONDITIONS

ELEVATION SCALE: 249, 248, 247, 246, 245, 244, 243, 242, 241, 240, 239, 238, 237, 236

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100)
SHEAR STRENGTH kPa
O UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × REMOULDED
(20 40 60 80 100)

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%) (10 20 30)

REMARKS & GRAIN SIZE DISTRIBUTION (%)
GR SA SI CL

Continued Next Page

+³, ×³: Numbers refer to Sensitivity O³% STRAIN AT FAILURE

RECORD OF BOREHOLE No C-5 — 2 OF 2 — METRIC

PROJECT 001-1143F
W.P. 30-95-00
DIST Central HWY 400
DATUM Geodetic

LOCATION N 4898392.1; E 291770.3
BOREHOLE TYPE 108mm ID SOLID STEM AUGERS AND CASING
DATE Nov.1/2000

ORIGINATED BY AZ
COMPILED BY LCC
CHECKED BY ASP

SOIL PROFILE: ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	-- CONTINUED FROM PREVIOUS PAGE --										
	END OF BOREHOLE Notes: 1. Water level in piezometer after installation at 6.9m depth (Elev.243.0m). 2. Water level in piezometer at 5.3m depth (Elev.244.6m) on March 19, 2001.										

+ 3, X 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No C-6 — 1 OF 2 — METRIC

PROJECT 001-1143F
W.P. 30-95-00
DIST Central HWY 400
DATUM Geodetic

LOCATION N 4898408.7; E 291680.3
BOREHOLE TYPE 108mm ID SOLID STEM AUGERS AND CASING
DATE Oct.31/2000

ORIGINATED BY AZ
COMPILED BY LCC
CHECKED BY ASP

SOIL PROFILE: ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
249.9	GROUND SURFACE										
0.0	Topsoil										
249.6			1	SS	10						
0.3	Silty Sand, some gravel, trace clay, trace organics (Possible Fill) Compact Brown Moist						249				
							248				
247.2											
2.7	Clayey Silt, with to some sand, trace to some gravel (Till) Stiff to hard Brown becoming grey at 5.3m depth Moist		2	SS	9	▽	247		o		
							246				
			3	SS	40		245				
			4	SS	53		244		o⊢⊣		
			5	SS	70						
			6	SS	53		243		o		
			7	SS	103		242				
							241				
			8	SS	61				o		
							240				
			9	SS	130		239				
							238				
			10	SS	98				o		
							237				
			11	SS	110		236				
235.6											
14.3											

Continued Next Page

+ 3, X 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No C-6 — 2 OF 2 — METRIC

PROJECT 001-1143F
W.P. 30-95-00
DIST Central HWY 400
DATUM Geodetic

LOCATION N 4898408.7; E 291680.3
BOREHOLE TYPE 108mm ID SOLID STEM AUGERS AND CASING
DATE Oct.31/2000

ORIGINATED BY AZ
COMPILED BY LCC
CHECKED BY ASP

SOIL PROFILE: ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SOIL PROFILE: STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: 'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	END OF BOREHOLE Note: 1. Water level in open borehole at 4.1m depth (Elev.245.8m) upon completion of drilling.										

+ 3, × 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No C-7 — 1 OF 1 — METRIC

PROJECT 001-1143F
W.P. 30-95-00
DIST Central HWY 400
DATUM Geodetic

LOCATION N 4898636.1; E 291670.5
BOREHOLE TYPE 108mm ID SOLID STEM AUGERS AND CASING
DATE Oct.30/2000

ORIGINATED BY PKS
COMPILED BY LCC
CHECKED BY ASP

SOIL PROFILE: ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SOIL PROFILE: STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: 'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
255.5	GROUND SURFACE										
0.0	Clayey Silt with sand, trace gravel (Till) Very Stiff Grey-brown Moist		1	SS	20						
			2	SS	25						
			3	SS	26						
250.9						▽					
4.6	Silty Sand, trace to some clay, trace to some gravel (Till) Compact to very dense Grey brown becoming grey below 6.9m depth Moist		4	SS	24						
			5	SS	42						4 59 34 3
			6	SS	50						
			7	SS	125						
			8	SS	74						
			9	SS	65/.15						
			10	SS	50/.08						
243.1			11	SS	50/.10						
12.4	END OF BOREHOLE Note: 1. Water level in open borehole at 4.6m depth (Elev.250.9m) upon completion of drilling.										

+ 3, × 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No C-8

1 OF 2 **METRIC**

PROJECT 001-1143F
W.P. 30-95-00
DIST Central HWY 400
DATUM Geodetic
LOCATION N 4898571.1; E 291670.5
BOREHOLE TYPE 108mm ID SOLID STEM AUGERS AND CASING
DATE Mar.14-15/2001
ORIGINATED BY GD
COMPILED BY LCC
CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L / WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
260.6	GROUND SURFACE									
0.0	Silty Clay, some sand, trace gravel (Fill) Hard Brown Moist					260				
						259				
						258				
		1	SS	39		257				
256.0 / 4.6	Silty Sand, trace to some gravel (Fill) Dense Brown Moist					256				
						255				
		2	SS	45		254				
						253				
						252				
		3	SS	44		251				
249.9 / 10.7	Silty Sand, trace to some gravel, trace clay (Till) Compact to very dense Grey Moist					250				
		4	SS	21		249				
		5	SS	19		248				3 56 34 7
						247				
		6	SS	36		246				

Continued Next Page

+³, ×³: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

ON_MOT 0011143F.GPJ ON_MOT.GDT 25/9/01

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No C-8

2 OF 2 **METRIC**

PROJECT 001-1143F
W.P. 30-95-00
DIST Central HWY 400
DATUM Geodetic
LOCATION N 4898571.1; E 291670.5
BOREHOLE TYPE 108mm ID SOLID STEM AUGERS AND CASING
DATE Mar.14-15/2001
ORIGINATED BY GD
COMPILED BY LCC
CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L / WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---									
	Silty Sand, trace to some gravel, trace clay (Till) Compact to very dense Grey Moist	7	SS	53		245				
						244				
		8	SS	57		243				
		9	SS	48		242				
						241				
						240				
239.1 / 21.5		10	SS	98/.15						
	END OF BOREHOLE									

+³, ×³: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

ON_MOT 0011143F.GPJ ON_MOT.GDT 25/9/01

RECORD OF BOREHOLE No C-9

1 OF 1 **METRIC**

PROJECT 001-1143F
W.P. 30-95-00
DIST Central HWY 400
DATUM Geodetic

LOCATION N 4902176.0; E 291036.9
BOREHOLE TYPE 108mm ID SOLID STEM AUGERS AND CASING
DATE Oct.30/2000

ORIGINATED BY AZ
COMPILED BY LCC
CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GR	SA	SI	CL
288.0	GROUND SURFACE							
0.0 / 287.4	Topsoil	1	SS	8				
0.6	Clayey Silt, some to with sand, some gravel (Till) Stiff to hard Brown becoming grey below 3.7m depth Moist	2	SS	14				
		3	SS	50				
		4	SS	56				
		5	SS	78	4	56	27	13
		6	SS	123				
	Cobbles below 5.5m depth inferred from grinding/resistance to augering	7	SS	140				
279.9 / 8.1		8	SS	105				
	END OF BOREHOLE Note: 1. Water level in open borehole at 5.2m depth (Elev.282.8m) upon completion of drilling.							

GROUND WATER CONDITIONS: ▽

ELEVATION SCALE: 287, 286, 285, 284, 283, 282, 281, 280

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100)
SHEAR STRENGTH kPa (20 40 60 80 100)
o UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × REMOULDED

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%) (10 20 30)

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

$+^{3}, \times^{3}$: Numbers refer to Sensitivity $o^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No C-10

1 OF 1 **METRIC**

PROJECT 001-1143F
W.P. 30-95-00
DIST Central HWY 400
DATUM Geodetic

LOCATION N 4902139.0; E 290980.2
BOREHOLE TYPE 108mm ID SOLID STEM AUGERS AND CASING
DATE Oct.30/2000

ORIGINATED BY AZ
COMPILED BY LCC
CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GR	SA	SI	CL
291.8	GROUND SURFACE							
0.0 / 291.1	Topsoil	1	SS	10				
0.7	Clayey Silt, some sand and gravel (Till) Hard Brown becoming grey at 9.1m depth Moist	2	SS	46				
		3	SS	59				
		4	SS	40				
		5	SS	86				
		6	SS	56				
		7	SS	83				
		8	SS	133				
		9	SS	103/.15				
		10	SS	98				
279.5 / 12.3		11	SS	107/.15				
	END OF BOREHOLE Notes: 1. Water level in open borehole at 3.3m depth (Elev.288.5m) upon completion of drilling. 2. Water level in piezometer after installation at 11.4m depth (Elev.280.4m). 3. Water level in piezometer measured at 2.6m depth (Elev.289.2m) on June 19, 2001.							

GROUND WATER CONDITIONS: ▽

ELEVATION SCALE: 291, 290, 289, 288, 287, 286, 285, 284, 283, 282, 281, 280

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100)
SHEAR STRENGTH kPa (20 40 60 80 100)
o UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × REMOULDED

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%) (10 20 30)

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

$+^{3}, \times^{3}$: Numbers refer to Sensitivity $o^{3\%}$ STRAIN AT FAILURE

DIST HWY 400 CONT. No. GWP No. 30-95-00	
CULVERT AT STATION 10+780 HWY 400 BOREHOLE LOCATION PLAN	SHEET

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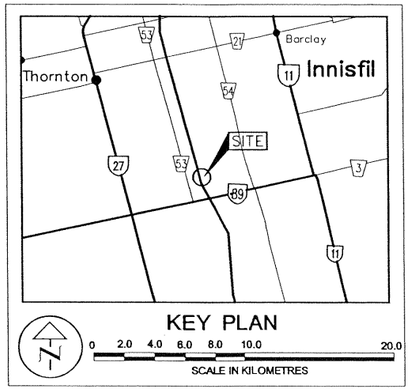
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

LEGEND

- Borehole, previous investigation
- Borehole, present investigation

No.	ELEVATION	LOCATION NORTHING	LOCATION EASTING
C-1	226.5	4,896,326.9	292,177.2
C-2	227.2	4,896,322.8	292,130.0

REFERENCE

This drawing was created from digital file "33811.dwg" provided by URS Cole Sherman

0 10 20 30 40 50 100m

SCALE IN METRES

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

NO.	DATE	BY	REVISION

Geocres No.

HWY. No. 400		PROJECT NO.: 001-1143F	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-399
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

P1143F02.DWG

plot scale 1:2tmp.

DIST HWY 400 CONT. No. GWP No. 30-95-00	N
CULVERT AT STATION 11+675 HWY 400 BOREHOLE LOCATION PLAN	SHEET

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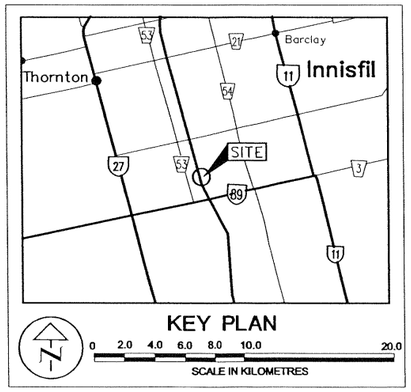

KEY PLAN

LEGEND

● Borehole, previous investigation

⊕ Borehole, present investigation

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
C-3	230.5	4,897,209.0	291,968.1
C-4	230.8	4,897,193.6	291,920.1

REFERENCE
This drawing was created from digital file "33810.dwg" provided by URS Cole Sherman

0 10 20 30 40 50 100m
SCALE IN METRES

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

NO.	DATE	BY	REVISION

Geocres No.

HWY. No. 400		PROJECT NO.: 001-1143F	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-571
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 2

P1143F03.DWG

plot scale 1:2mm.

DIST HWY 400 CONT. No. GWP No. 30-95-00	N
CULVERT AT STATION 12+900 HWY 400 BOREHOLE LOCATION PLAN	SHEET

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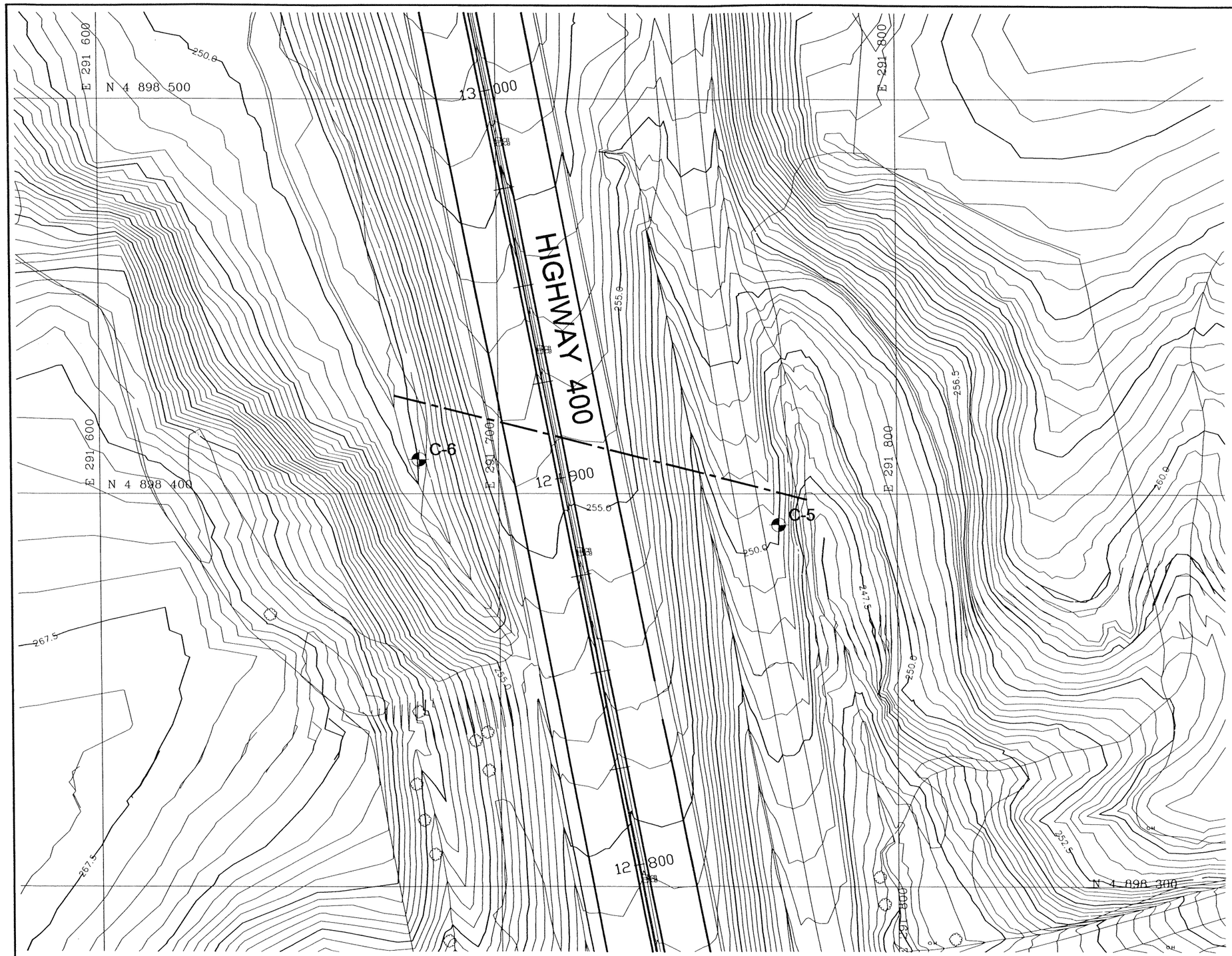

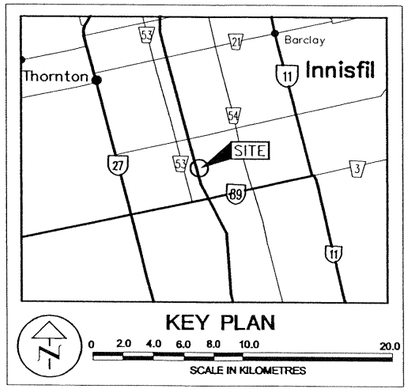

KEY PLAN

LEGEND

- Borehole, previous investigation
- Borehole, present investigation

No.	ELEVATION	LOCATION NORTHING	LOCATION EASTING
C-5	249.9	4,898,392.1	291,770.3
C-6	249.9	4,898,408.7	291,680.3

REFERENCE

This drawing was created from digital file "33810.dwg" provided by URS Cole Sherman

0 10 20 30 40 50 100m

SCALE IN METRES

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

NO.	DATE	BY	REVISION

Geocres No.

HWY. No. 400		PROJECT NO.: 001-1143F	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-572
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 3

P1143F04.DWG

plot scale 1:20mp.

DIST HWY 400
CONT. No.
GWP No. 30-95-00

CULVERT AT STATION 13+100
HWY 400
BOREHOLE LOCATION PLAN

SHEET

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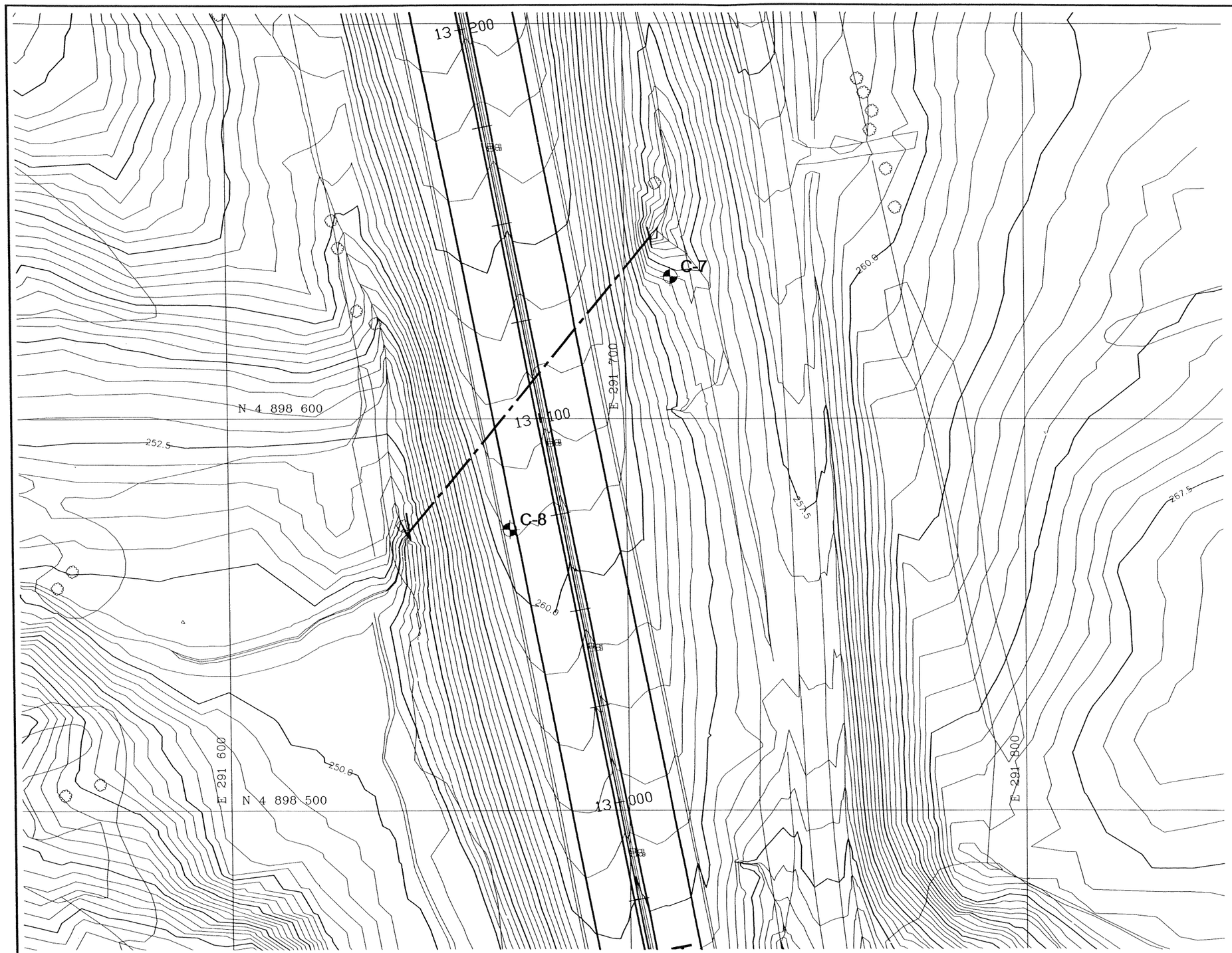

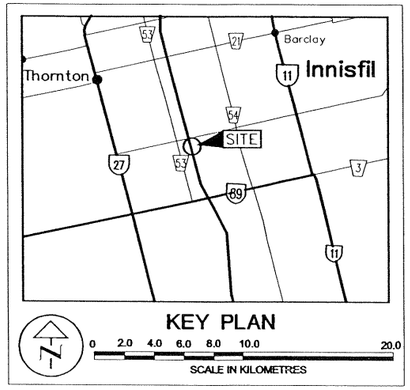

KEY PLAN

0 2.0 4.0 6.0 8.0 10.0 20.0
SCALE IN KILOMETRES

LEGEND

Borehole, previous investigation

Borehole, present investigation

No.	ELEVATION	LOCATION NORTHING	LOCATION EASTING
C-7	255.5	4,898,636.1	291,711.3
C-8	260.6	4,898,571.1	291,670.5

REFERENCE

This drawing was created from digital file "33809.dwg" provided by URS Cole Sherman

0 10 20 30 40 50 100m
SCALE IN METRES

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

NO.	DATE	BY	REVISION

Geocres No.

HWY. No. 400		PROJECT NO.: 001-1143F	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-573
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 4

P1143F06.DWG

plot scale 1:2mp.

DIST	HWY 400
CONT. No.	
GWP No.	30-95-00

SHEET

CULVERT AT STATION 16+730
HWY 400
BOREHOLE LOCATION PLAN

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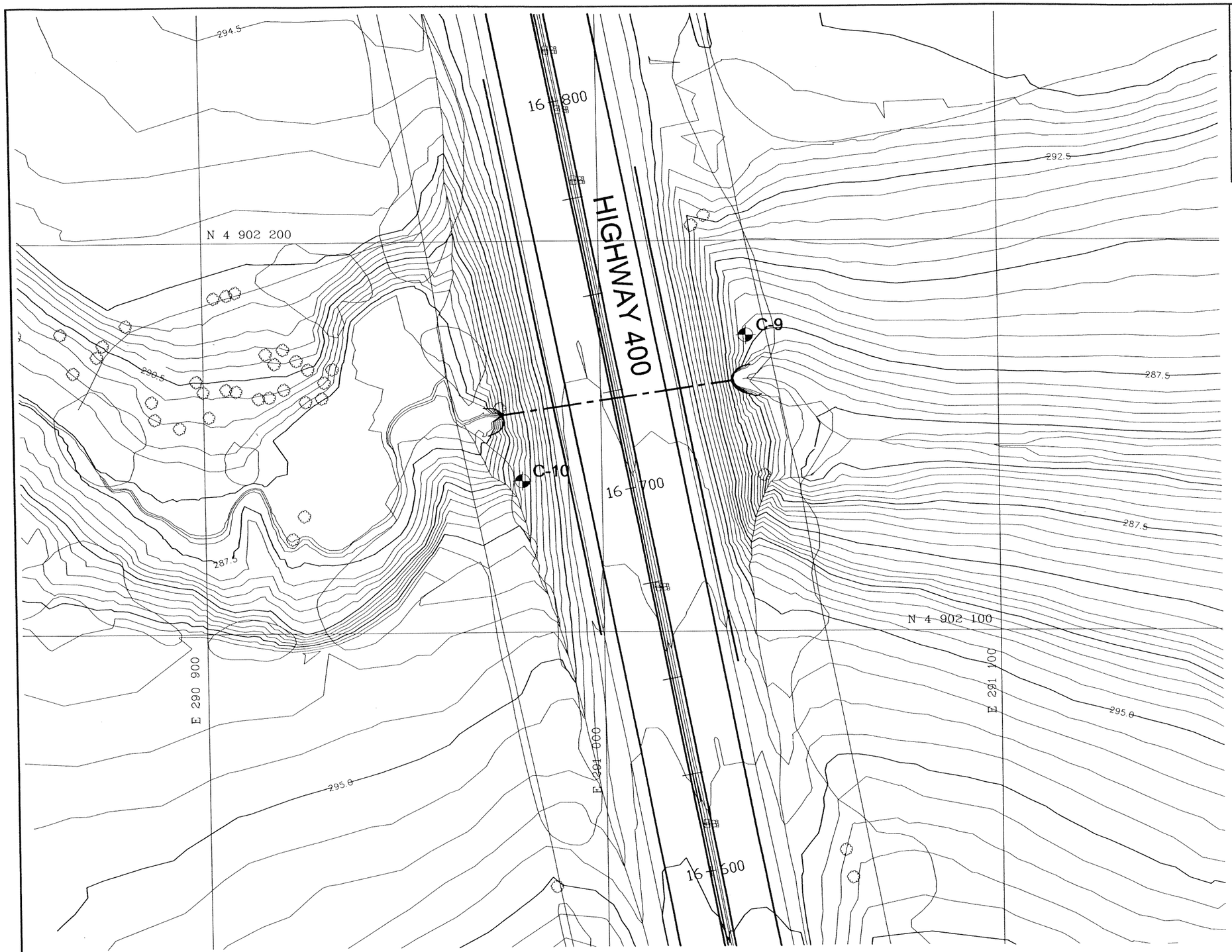
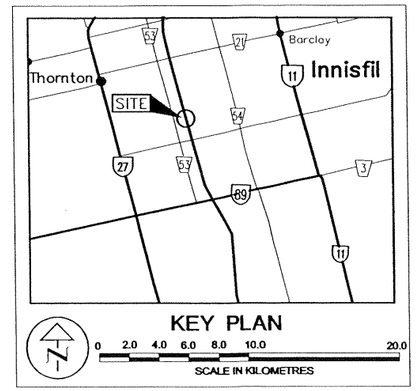




LEGEND

- Borehole, previous investigation
- Borehole, present investigation

No.	ELEVATION	LOCATION NORTHING	LOCATION EASTING
C-9	288.0	4,902,176.0	291,036.9
C-10	291.8	4,902,139.0	290,980.2

REFERENCE
This drawing was created from digital file "33808.dwg" provided by URS Cole Sherman

0 10 20 30 40 50 100m
SCALE IN METRES

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

NO.	DATE	BY	REVISION

Geocres No.

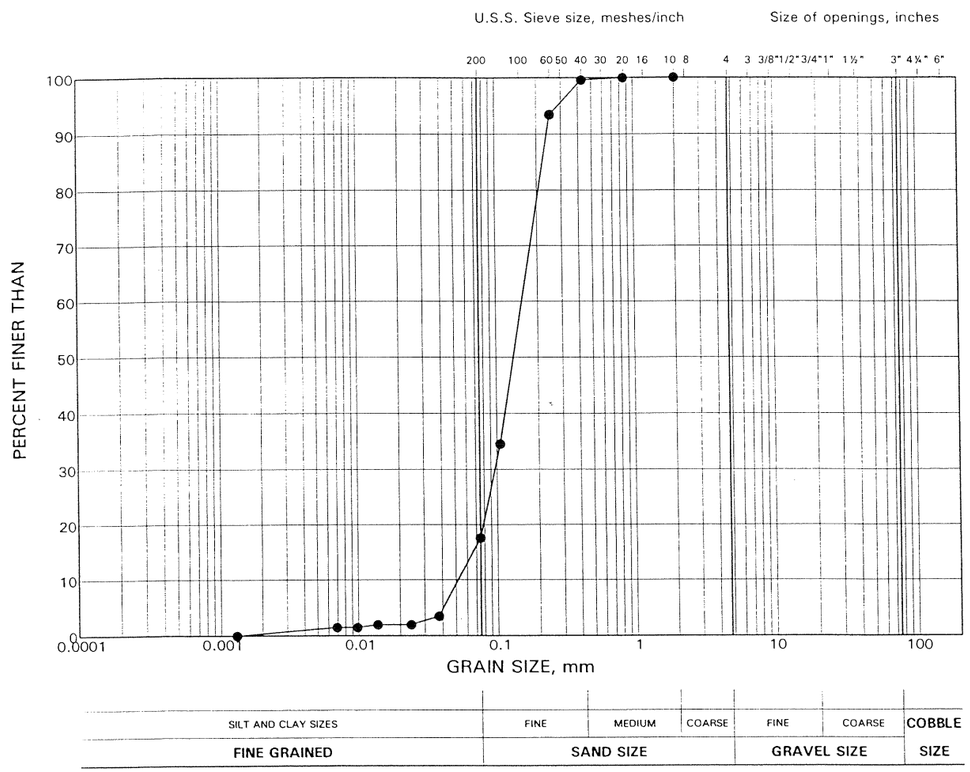
HWY. No. 400		PROJECT NO.: 001-1143F	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-415
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 5

P1143F07.DWG

plot scale 1:2mp.

GRAIN SIZE DISTRIBUTION
Sand

FIGURE 1



LEGEND

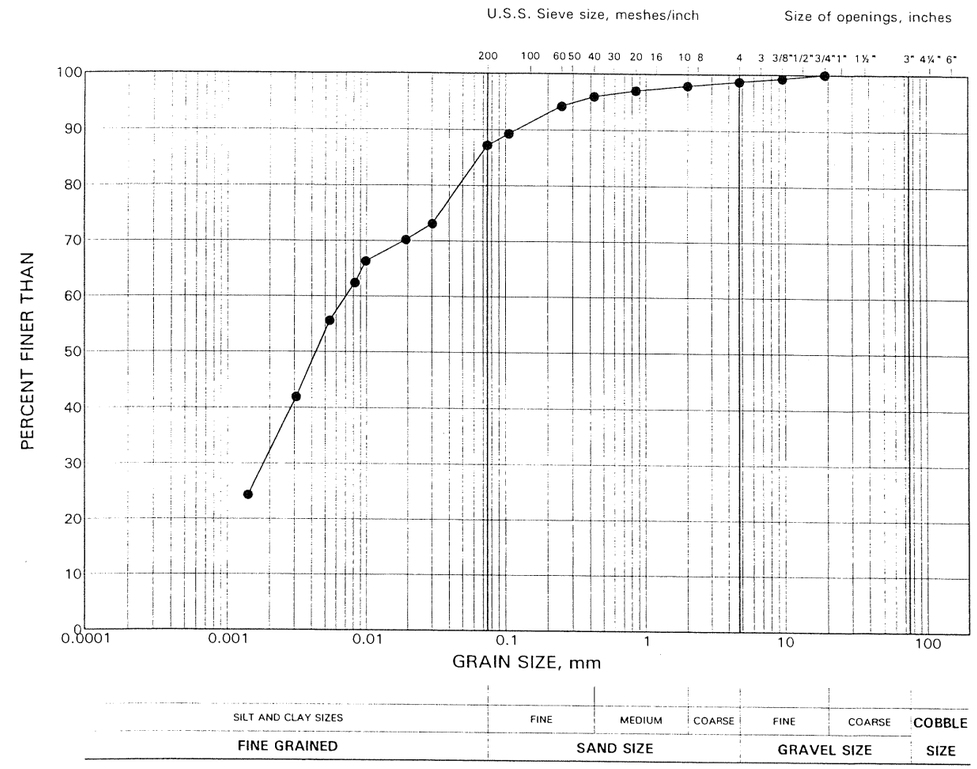
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	C-1	6	222.2

Project 001-1143F-18

Golder Associates

GRAIN SIZE DISTRIBUTION
Clayey Silt (Till)

FIGURE 2



LEGEND

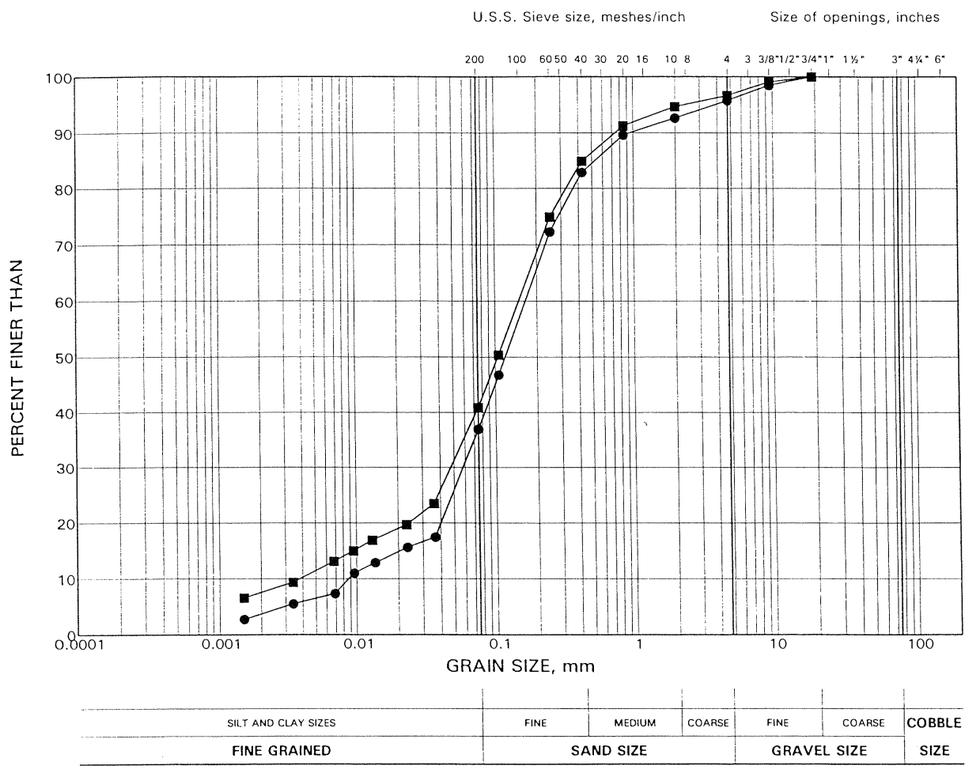
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	C-2	3	222.8

Project 001-1143F-18

Golder Associates

GRAIN SIZE DISTRIBUTION
Silty Sand (Till)

FIGURE 3



LEGEND

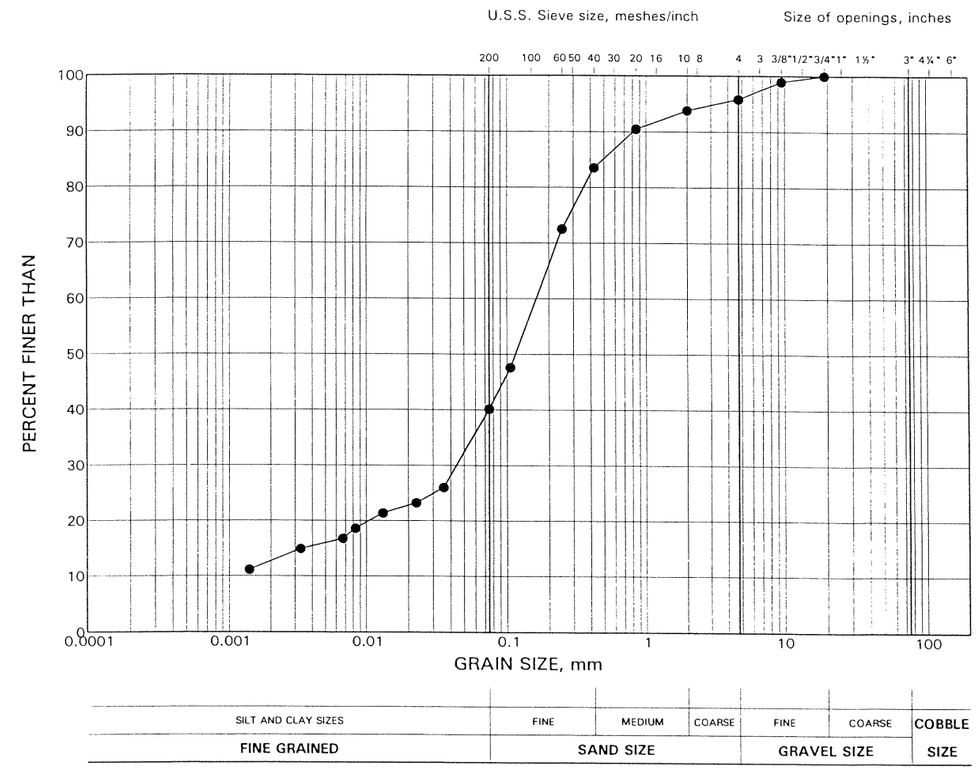
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	C-7	5	249.6
■	C-8	5	247.8

Project 001-1143F-18

Golder Associates

GRAIN SIZE DISTRIBUTION
Clayey Silt (Till)

FIGURE 4



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	C-9	5	283.6

Project 001-1143F-18

Golder Associates

Ontario

Ministry of Transportation
Southwestern Region
Planning and Design Section

HIGHWAY 400 PLANNING AND PRELIMINARY DESIGN STUDY

FROM 1 KM SOUTH OF HIGHWAY 89 NORTHERLY 30 KM TO THE JUNCTION AT HIGHWAY 11

COUNTY OF SIMCOE
TOWN OF BRADFORD WEST GWILLIMBURY
TOWN OF INNISFIL
CITY OF BARRIE
TOWNSHIP OF SPRINGWATER

G.W.P. 30-95-00

APPENDIX L

PRELIMINARY DESIGN CRITERIA

WORK PROJECT NO.: 30-95-00 **OPERATIONS:** London **HWY. NO.:** 400
TYPE OF PROJECT: Widening, Interchange, Structure Replacement, Structure Widening and Illumination **LENGTH:** 30 km

LOCATION: From 1 km south of Highway 89 to the Junction at Highway 11

COUNTY / REGIONAL MUNICIPALITY OF: County of Simcoe.
CITY / TOWNSHIP OF Town of West Gwillimbury, Town of Innisfil, City of Barrie, Township of Springwater

LIMITS: From Sta. 26+550 (Town of Bradford West Gwillimbury) **PLAN:** 076-400/55-0 **TO STA.:** 18+807 (Township of Springwater) **PLAN:** 078-400/33-0

CHAINAGE EQUATION: 27+552.3 (Town of Bradford West Gwillimbury) = 10+000 (Town of Innisfil)
29+737.9 (City of Barrie) = 10+000 (City of Barrie)

MAINLINE HIGHWAY 400

	PRESENT CONDITIONS	DESIGN STANDARDS	PROPOSED STANDARDS
HIGHWAY CLASSIFICATION	RFD 110	RFD 120	RFD 120
MIN STOPPING SIGHT DISTANCE	160 m	245 m	205 m (a)(b)
EQUIVALENT MIN 'K' FACTOR - Crest	50	120	120 (a)
EQUIVALENT MIN 'K' FACTOR - Sag	50	60	60
GRADES MAXIMUM	3%	3%	3%
MINIMUM RADIUS	806 m	650 m	780 m (b)
PAVEMENT WIDTH	22 m	37 m	40.8 m 44.5 m 37.0 m (c)
SHOULDER WIDTH (Outside)	3.0 m, Paved	3.0 m, Paved	3.0m, Paved (d)
SHOULDER ROUNDING	Varies	1.0m	1.0m
R.O.W. WIDTH	Varies	Varies	Varies
POSTED SPEED	100 km/h	100 km/h	100 km/h

MANAGER, OPERATIONS

MANAGER, ENGINEERING

MANAGER, CONSTRUCTION

REGIONAL DIRECTOR, DATE OF APPROVAL

CROSS ROADS AND INTERCHANGES

Highway 89

Realign Highway 89 to accommodate construction staging, improve sight distance and modify existing lanes to accommodate new interchange ramps.

Highway 89	Present Conditions	Design Standards	Proposed Standards
HIGHWAY CLASSIFICATION	RAU 100	RAU 100	RAU 100
MIN STOPPING SIGHT DISTANCE	100 m	185 m	185 m
EQUIVALENT MIN 'K' FACTOR – Sag	10	30	30
EQUIVALENT MIN 'K' FACTOR – Crest	20	70	70
GRADES MAXIMUM	5.0%	8.0%	4.0%
MINIMUM RADIUS	N/A	420 m	420 m
PAVEMENT WIDTH	11.0 m (3 x 3.66 m lanes)	3.5 m through lanes	15.0 m (4 x 3.75 m lanes)
SHOULDER WIDTH	2.0 m	2.5 m	2.5 m
R.O.W. WIDTH	20 m	-	30 m
POSTED SPEED	80 km/h	80 km/h	80 km/h

WORK PROJECT NO.: 30-95-00 **OPERATIONS:** London **HWY. NO.:** 400
TYPE OF PROJECT: Widening, Interchange, Structure Replacement, Structure Widening and Illumination **LENGTH:** 30 km

LOCATION: From 1 km south of Highway 89 to the Junction at Highway 11

Fourth Line (Churchill Side Road)

Realign Fourth Line to accommodate construction staging.

Fourth Line	Present Conditions	Design Standards	Proposed Standards
HIGHWAY CLASSIFICATION	RAU 90	RAU 90	RAU 90
MIN STOPPING SIGHT DISTANCE	110 m	160 m	160 m
EQUIVALENT MIN 'K' FACTOR – Sag	-	40	40
EQUIVALENT MIN 'K' FACTOR – Crest	25	50	50
GRADES MAXIMUM	4.5%	8.0%	4.0%
MINIMUM RADIUS	N/A	340 m	340 m
PAVEMENT WIDTH	6.0 m (2 x 3.0 m lanes)	3.25 m through lanes	6.5 m (2 x 3.25 m lanes)
SHOULDER WIDTH	1.5 m	2.0 m	2.0 m
R.O.W. WIDTH	20 m to 65 m	-	20 m to 80 m
POSTED SPEED	80 km/h	80 km/h	80 km/h

Sixth Line (Innisfil Township Road)

Lower profile to accommodate vertical clearance under new structure.

Sixth Line	Present Conditions	Design Standards	Proposed Standards
HIGHWAY CLASSIFICATION	RAU 90	RAU 90	RAU 90
MIN STOPPING SIGHT DISTANCE	65 m	160 m	> 160 m
EQUIVALENT MIN 'K' FACTOR – Sag	18	40	100
EQUIVALENT MIN 'K' FACTOR – Crest	10	50	100
GRADES MAXIMUM	3.5%	8.0%	0.6%
MINIMUM RADIUS	N/A	340 m	N/A
PAVEMENT WIDTH	6.0 m (2 x 3.0 m lanes)	3.25 m through lanes	6.5 m (2 x 3.25 m lanes)
SHOULDER WIDTH	-	1.0 m	1.0 m
R.O.W. WIDTH	20 m to 50 m	-	20 m to 60 m
POSTED SPEED	80 km/h	80 km/h	80 km/h

Innisfil Beach Road

Lower profile to accommodate vertical clearance under new structure and modify existing lanes to accommodate new interchange ramps.

Innisfil Beach Road	Present Conditions	Design Standards	Proposed Standards
HIGHWAY CLASSIFICATION	RAU 80	RAU 80	RAU 80
MIN STOPPING SIGHT DISTANCE	410 m	135 m	140 m
EQUIVALENT MIN 'K' FACTOR – Sag	N/A	30	80
EQUIVALENT MIN 'K' FACTOR – Crest	100	35	40
GRADES MAXIMUM	2.3%	8%	2.3%
MINIMUM RADIUS	N/A	250 m	N/A
PAVEMENT WIDTH	11.25 M (3 x 3.75 lanes)	3.5 m through lanes	7.5 m (2 x 3.75 m lanes)
SHOULDER WIDTH	2.0 m	2.5 m	2.5 m
R.O.W. WIDTH	26 m	-	40 m
POSTED SPEED	60 km/h	60 km/h	60 km/h

WORK PROJECT NO.: 30-95-00 **OPERATIONS:** London **HWY. NO.:** 400
TYPE OF PROJECT: Widening, Interchange, Structure Replacement, Structure Widening and Illumination **LENGTH:** 30 km

LOCATION: From 1 km south of Highway 89 to the Junction at Highway 11

Tenth Line (Stroud Road)

Raise profile to accommodate vertical clearance under new structure.

Tenth Line	Present Conditions	Design Standards	Proposed Standards
HIGHWAY CLASSIFICATION	RAU 90	RAU 90	RAU 90
MIN STOPPING SIGHT DISTANCE	100 m	160 m	160 m
EQUIVALENT MIN 'K' FACTOR – Sag	-	40	40
EQUIVALENT MIN 'K' FACTOR – Crest	20	50	50
GRADES MAXIMUM	2.5%	8.0%	3.5%
MINIMUM RADIUS	N/A	340	N/A
PAVEMENT WIDTH	6.0 m (2 x 3.0 m)	3.5 m through lanes	7.0 m (2 x 3.5 m lanes)
SHOULDER WIDTH	2.0 m	2.5 m	2.5 m
R.O.W. WIDTH	21 m	-	21 m to 40 m
POSTED SPEED	80 km/h	80 km/h	80 km/h

Molson Park Drive

Implement operational improvements (add right turn lanes to the E/W-S and E/W-N Ramps).

Molson Park Drive	Present Conditions	Design Standards	Proposed Standards
HIGHWAY CLASSIFICATION	UAD 80	UAD 80	UAD 80
MIN STOPPING SIGHT DISTANCE	135 m	135 m	135 m
EQUIVALENT MIN 'K' FACTOR – Sag	30	15	30
EQUIVALENT MIN 'K' FACTOR – Crest	35	35	35
GRADES MAXIMUM	3.5%	8%	3.5%
MINIMUM RADIUS	N/A	250m	N/A
PAVEMENT WIDTH	Varies (3.5m through lanes, 3.0m median left turn lane, 3.25m second left turn lane, 2m median)	3.5 m median through lane, 3.75 m all other through lanes, 3.25 m left turn lane, 3.0 m median left turn lane, 3.25m right turn lane	Varies (3.5m through lanes, 3.0m median left turn lane, 3.25m second left turn lane, 2m median, 3.25m right turn lane)
SHOULDER WIDTH	N/A	N/A	N/A
R.O.W. WIDTH	47m	-	47m
POSTED SPEED	60 km/h	60 km/h	60 km/h

WORK PROJECT NO.: 30-95-00 **OPERATIONS:** London **HWY. NO.:** 400
TYPE OF PROJECT: Widening, Interchange, Structure Replacement, Structure Widening and Illumination **LENGTH:** 30 km

LOCATION: From 1 km south of Highway 89 to the Junction at Highway 11

Essa Road

Lower profile to accommodate vertical clearance under new structure and modify existing lanes to accommodate new interchange ramps.

Essa Road	Present Conditions	Design Standards	Proposed Standards
HIGHWAY CLASSIFICATION	UAD 70	UAD 70	UAD 70
MIN STOPPING SIGHT DISTANCE	N/A	110 m	135 m
EQUIVALENT MIN 'K' FACTOR – Sag	30	12	15
EQUIVALENT MIN 'K' FACTOR – Crest	N/A	25	35
GRADES MAXIMUM	4%	8%	6%
MINIMUM RADIUS	470m	190m	470m
PAVEMENT WIDTH	Varies (3.75 m through lanes, 3.0 m median left turn lane, 3.25m right turn lane)	3.5 m median through lane, 3.75 m all other through lanes, 3.0 m median left turn lane, 3.25m right turn lane	Varies (3.75 m through lanes, 3.0 m median left turn lane, 3.25m right turn lane)
SHOULDER WIDTH	N/A	N/A	N/A
R.O.W. WIDTH	30 m	-	30 m
POSTED SPEED	50 km/h	50 km/h	50 km/h

Tiffin Street

Replace Highway 400 structure.

Tiffin Street	Present Conditions	Design Standards	Proposed Standards
HIGHWAY CLASSIFICATION	UAU 70	UAU 70	UAU 70
MIN STOPPING SIGHT DISTANCE	N/A	110 m	N/A
EQUIVALENT MIN 'K' FACTOR – Sag	N/A	12	N/A
EQUIVALENT MIN 'K' FACTOR – Crest	N/A	25	N/A
GRADES MAXIMUM	0.5%	8%	0.5%
MINIMUM RADIUS	N/A	190m	N/A
PAVEMENT WIDTH	11.0 m (2 x 3.75 m through lanes, 1 x 3.5 m left turn lane)	3.5 m to 3.75 m through lanes, 3.5 m left turn lane	11.0 m (2 x 3.75 m through lanes, 1 x 3.5 m left turn lane)
SHOULDER WIDTH	N/A	N/A	N/A
R.O.W. WIDTH	20 m	-	20 m
POSTED SPEED	50 km/h	50 km/h	50 km/h

WORK PROJECT NO.: 30-95-00 **OPERATIONS:** London **HWY. NO.:** 400
TYPE OF PROJECT: Widening, Interchange, Structure Replacement, Structure Widening and Illumination **LENGTH:** 30 km

LOCATION: From 1 km south of Highway 89 to the Junction at Highway 11

Dunlop Street

Raise profile to accommodate vertical clearance under new structure and modify existing lanes to accommodate new interchange ramps.

Dunlop Street	Present Conditions	Design Standards	Proposed Standards
HIGHWAY CLASSIFICATION	UAU 70	UAU 70	UAU 70
MIN STOPPING SIGHT DISTANCE	110 m	110 m	110 m
EQUIVALENT MIN 'K' FACTOR – Sag	30	12	15
EQUIVALENT MIN 'K' FACTOR – Crest	25	25	25
GRADES MAXIMUM	4%	8%	5%
MINIMUM RADIUS	N/A	190m	N/A
PAVEMENT WIDTH	Varies(3.3 m through lanes, 3.0 m median left turn lane)	3.5 m median through lane, 3.75 m all other through lanes, 3.25 m left turn lane, 3.0 m median left turn lane, 3.25m right turn lane	Varies (3.5 m median through lane, 3.75 m all other through lanes, 3.25 m left turn lane, 3.0 m median left turn lane, 3.25m right turn lane)
SHOULDER WIDTH	N/A	N/A	N/A
R.O.W. WIDTH	20 to 30 m	-	25 m to 40 m
POSTED SPEED	50 km/h	50 km/h	50 km/h

Anne Street

Raise profile to accommodate vertical clearance under new structure.

Anne Street	Present Conditions	Design Standards	Proposed Standards
HIGHWAY CLASSIFICATION	UAU 70	UAU 70	UAU 70
MIN STOPPING SIGHT DISTANCE	65 m	110 m	65 m (e)
EQUIVALENT MIN 'K' FACTOR – Sag	8	12	15
EQUIVALENT MIN 'K' FACTOR – Crest	8	25	8 (e)
GRADES MAXIMUM	7.5%	8%	7.5%
MINIMUM RADIUS	N/A	190m	N/A
PAVEMENT WIDTH	Varies (3.25 m through lanes, 3.5 m left turn lane)	3.5 m to 3.75 m through lanes, 3.5 m left turn lane	Varies (3.5 m through lanes, 3.5 m left turn lane)
SHOULDER WIDTH	N/A	N/A	N/A
R.O.W. WIDTH	30 m to 50 m	-	30 m to 55 m
POSTED SPEED	50 km/h	50 km/h	50 km/h

WORK PROJECT NO.: 30-95-00 **OPERATIONS:** London **HWY. NO.:** 400
TYPE OF PROJECT: Widening, Interchange, Structure Replacement, Structure Widening and Illumination **LENGTH:** 30 km

LOCATION: From 1 km south of Highway 89 to the Junction at Highway 11

Sunnidale Road

Raise profile to accommodate vertical clearance under new structure.

Sunnidale Road	Present Conditions	Design Standards	Proposed Standards
HIGHWAY CLASSIFICATION	UAU 70	UAU 70	UAU 70
MIN STOPPING SIGHT DISTANCE	135 m	110 m	135 m
EQUIVALENT MIN 'K' FACTOR – Sag	N/A	15	N/A
EQUIVALENT MIN 'K' FACTOR – Crest	35	35	35
GRADES MAXIMUM	4.5%	8%	5.4%
MINIMUM RADIUS	190 m	190 m	190 m
PAVEMENT WIDTH	10.5 m (2 x 5.25 m through lanes)	3.5 m to 3.75m through lanes	10.5 m (2 x 5.25 m through lanes)
SHOULDER WIDTH	N/A	N/A	N/A
R.O.W. WIDTH	19 m to 31 m	-	19 m to 35 m
POSTED SPEED	50 km/h	50 km/h	50 km/h

Bayfield Street

Realign Bayfield Street to accommodate construction staging, improve sight distance and modify existing lanes to accommodate new interchange ramps.

Bayfield Street	Present Conditions	Design Standards	Proposed Standards
HIGHWAY CLASSIFICATION	UAU 70	UAU 70	UAU 70
MIN STOPPING SIGHT DISTANCE	90 m	110 m	135 m
EQUIVALENT MIN 'K' FACTOR – Sag	50	15	30
EQUIVALENT MIN 'K' FACTOR – Crest	18	35	35
GRADES MAXIMUM	6%	8%	5.8 %
MINIMUM RADIUS	N/A	190 m	500 m
PAVEMENT WIDTH	Varies (3.5 m through lanes, 3.5 m left turn lane)	3.5 m to 3.75 m through lanes, 3.5 m left turn lane	Varies (3.75 m through lanes, 3.5 m left turn lane)
SHOULDER WIDTH	N/A	N/A	N/A
R.O.W. WIDTH	18 m	-	30 m
POSTED SPEED	50 km/h	50 km/h	50 km/h

WORK PROJECT NO.: 30-95-00 **OPERATIONS:** London **HWY. NO.:** 400
TYPE OF PROJECT: Widening, Interchange, Structure Replacement, Structure Widening and Illumination **LENGTH:** 30 km

LOCATION: From 1 km south of Highway 89 to the Junction at Highway 11

Duckworth Street

Lower profile to accommodate vertical clearance under new structure and modify existing lanes to accommodate new interchange ramps.

Duckworth Street	Present Conditions	Design Standards	Proposed Standards
HIGHWAY CLASSIFICATION	UAU 70	UAU 70	UAU 70
MIN STOPPING SIGHT DISTANCE	85 m	110 m	135 m
EQUIVALENT MIN 'K' FACTOR – Sag	30	15	15
EQUIVALENT MIN 'K' FACTOR – Crest	15	35	35
GRADES MAXIMUM	5.5%	8%	6%
MINIMUM RADIUS	N/A	190 m	N/A
PAVEMENT WIDTH	Varies (3.5 m through lanes, 3.25 m median left turn lane)	3.5 m to 3.75 m through lanes, 3.0 m median left turn lane	Varies (3.75 m through lanes, 3.0 m median left turn lane)
SHOULDER WIDTH	N/A	N/A	N/A
R.O.W. WIDTH	20 m to 40 m	-	20 m to 50 m
POSTED SPEED	50 km/h	50 km/h	50 km/h

TRAFFIC

HIGHWAY 400 CORRIDOR SEGMENT	1999		2001		2006		2011		Percent Commercial
	AADT	SADT	AADT	SADT	AADT	SADT	AADT	SADT	
Highway 88 to Highway 89	64,800	80,400	70,400	87,300	84,300	104,600	98,200	121,800	12
Highway 89 to Innisfil Beach Road	65,550	81,300	71,700	89,000	87,000	107,900	102,200	126,800	12
Innisfil Beach Road to Molson Park Drive	70,500	87,500	77,200	95,800	93,800	116,400	110,400	136,900	12
Molson Park Drive to Essa Road	69,500	86,200	76,100	94,400	92,600	114,900	109,000	135,200	12
Essa Road to Dunlop Street	84,000	104,200	91,700	113,800	110,800	137,400	129,800	161,000	12
Dunlop Street to Bayfield Street	84,700	105,100	93,300	115,700	114,800	142,400	136,300	169,100	9.5
Bayfield Street to Duckworth Street	63,000	78,200	67,500	83,700	78,700	97,600	89,900	111,500	12
Duckworth Street to Highway 11	58,500	72,600	63,200	78,400	74,900	92,900	86,500	107,300	12

WORK PROJECT NO.: 30-95-00 **OPERATIONS:** London **HWY. NO.:** 400
TYPE OF PROJECT: Widening, Interchange, Structure Replacement, Structure Widening and Illumination **LENGTH:** 30 km

LOCATION: From 1 km south of Highway 89 to the Junction at Highway 11

CROSSING ROAD	1999		2001		2006		2011		Percent Commercial
	AADT	SADT	AADT	SADT	AADT	SADT	AADT	SADT	
Highway 89	11,000	13,640	11,920	14,790	14,170	17,580	16,420	20,370	6
4th Line	900	1,120	940	1,170	1,200	1,490	1,450	1,800	Not Available
6th Line	200	250	220	280	270	340	330	410	Not Available
Innisfil Beach Road	10,920	13,550	13,750	17,050	19,590	24,300	25,420	31,530	5
10th Line	3,100	3,850	3,390	4,210	4,470	5,550	5,550	6,890	Not Available
Molson Park Drive	21,340	26,470	22,500	27,900	25,340	31,430	28,170	34,940	7
Essa Road	25,500	31,620	27,250	33,790	31,750	39,370	36,340	45,070	4
Tiffin Street	14,700	18,230	16,070	19,930	19,900	24,680	23,730	29,430	Not Available
Dunlop Street West	20,840	25,850	22,590	28,020	26,920	33,390	31,250	38,750	8
Anne Street	22,100	27,410	23,170	28,740	26,100	32,370	29,030	36,000	Not Available
Sunnidale Road	11,000	13,640	11,820	14,660	14,090	17,480	16,360	20,290	Not Available
Bayfield Street	26,420	32,770	30,670	38,040	32,090	39,800	33,590	41,660	3
St. Vincent Street	22,800	28,280	24,330	30,170	28,520	35,370	32,720	40,580	Not Available
Duckworth Street	13,250	16,430	14,750	18,290	18,420	22,850	22,170	27,500	3

NOTES

a) The minimum stopping sight distance of 160m occurs at a vertical crest curve location (Curve no. 33, approximate VPI station 29+560 City of Barrie), and corresponds to a design speed of approximately 90 km/h. To increase the sight distance to 245m, the K value of the curve will be increased from 50 to 120, which will provide for a design speed of 120km/h. To achieve the K value of 120, the approaches on both sides of the crest will be raised between 1.5m and 2.0m. A complete listing of the vertical curves within the study limits is listed in a table below. Eight curves, as highlighted in the table below, do not satisfy a 120 km/h design speed.

The crest curves at station 13+885 (Curve no. 10) and station 24+065 (Curve no. 25) are located within Collision Prone Areas (CPA - a specific location on Highway 400 where the collision rate is above the provincial average). As such, increasing the K value of these crests to 120 to satisfy a design speed of 120 km/h is recommended. Achieving this K value will involve excavating 0.2m (maximum) and 0.5m (maximum) from Curve no. 10 and Curve no. 25 respectively. Such an undertaking can be incorporated, as a part of the typical construction operations and staging to be implemented for the widening and resurfacing of Highway 400.

Vertical Curve No.	Township	Approx. VPI	Curve Type C=Crest S=Sag	Approx. Grade In (%)	Approx. Grade Out (%)	Approx. K	Approx. Design Speed
1	Town of Innisfil	26+555	S	-0.5	+0.5	200	>120
2	Town of Innisfil	26+800	C	+0.5	-0.5	180	>120
3	Town of Innisfil	27+125	S	-0.5	0.7	180	>120
4	Town of Innisfil	27+400	C	0.7	-0.8	120	120
5	Town of Innisfil	10+115	S	-0.8	+0.8	200	>120
6	Town of Innisfil	10+445	C	+0.8	-0.5	120	120
7	Town of Innisfil	10+760	S	-0.5	+0.5	120	>120
8	Town of Innisfil	11+830	S	+0.5	+1.8	80	>120
9	Town of Innisfil	12+690	S	+1.8	+3.0	100	>120
10	Town of Innisfil	13+885	C	+3.0	+0.3	100	110
11	Town of Innisfil	14+190	S	+0.3	+0.5	120	>120
12	Town of Innisfil	15+173	S	+0.5	+1.0	250	>120
13	Town of Innisfil	16+255	C	+1.0	-1.2	150	>120
14	Town of Innisfil	16+760	S	-1.2	+0.6	150	>120
15	Town of Innisfil	17+505	C	+0.6	-0.5	120	120
16	Town of Innisfil	18+255	S	-0.5	+1.0	150	>120
17	Town of Innisfil	19+400	C	+1.2	-0.2	400	>120
18	Town of Innisfil	19+875	S	-0.2	+2.0	70	>120
19	Town of Innisfil	20+280	C	+2.0	-3.0	80	100
20	Town of Innisfil	20+670	S	-3.0	+0.8	50	110
21	Town of Innisfil	21+380	C	+0.8	-1.5	90	110
22	Town of Innisfil	21+745	S	-1.5	+1.5	60	120
23	Town of Innisfil	22+205	C	+1.5	-3.0	70	100
24	City of Barrie	23+355	S	-3.0	+2.8	80	>120
25	City of Barrie	24+065	C	+2.8	-0.5	90	110
26	City of Barrie	24+630	S	-0.5	+1.9	100	>120
27	City of Barrie	25+150	C	+1.9	+0.6	120	120
28	City of Barrie	25+595	C	+0.6	-0.8	120	120
29	City of Barrie	26+325	C	-0.8	-1.7	180	>120
30	City of Barrie	26+755	S	-1.7	-0.5	180	>120
.31	City of Barrie	27+330	C	-0.5	-3.0	120	120
32	City of Barrie	29+095	S	-3.0	+1.9	50	110
33	City of Barrie	29+560	C	+1.9	-2.0	50	90
34	City of Barrie	10+090	S	-2.0	-0.7	70	>120
35	City of Barrie	11+225	S	-0.8	+1.0	350	>120
36	City of Barrie	12+315	S	+1.0	+2.2	200	>120
37	City of Barrie	13+880	C	+2.2	-2.5	150	>120
38	City of Barrie	14+510	S	-2.5	+0.5	180	>120
39	City of Barrie	15+045	C	+0.5	-1.3	120	>120
40	City of Barrie	16+920	C	-1.3	-3.0	100	>120
41	City of Barrie and Township of Springwater	17+585	S	-3.0	-0.5	120	>120
42	Township of Springwater	18+525	S	-0.5	+3.0	100	>120

Identifies vertical curves which do not satisfy a 120 km/h design speed

While the remaining four vertical curves (curves 19, 20, 21 and 23) match or exceed the posted speed (100 km/h) and are not located within CPA's, upgrading these curves to a 120 km/h design speed is recommended. Curve no. 19, a crest curve, will require a maximum grade raise of 1.0 metres (approximate) over a distance of 1 kilometre to improve the K value to 120. Improving the K value of Curve no. 20, a sag curve, to a K value of 60 can be implemented as part of the grade raise required to improve curve no. 19. Upgrading curve no. 21, a crest curve, to K value of 120 will require a maximum

excavation of 0.3 metres over a distance of 290 metres and upgrading curve no. 23, a crest curve, to a K value of 120 will require a maximum excavation of 1.5 metres over a distance of 520 metres. Implementing upgrades to these vertical curves can be coordinated with the proposed work necessary to widen Highway 400 to ten lanes, which includes grading, installation of median concrete barrier and a new drainage system.

b) There are a total of nine horizontal curves located within the study limits. All but three curves meet sight distance requirements for a 120 km/h design speed.

The three curves exceed the minimum radius requirements for 120 km/h design speed. The proposed tall wall median, however, will reduce the sight distance around the curves such that the design speeds are less than 120 km/h. An R=780 curve will replace the R=806 curve at PI station 10+715 in the City of Barrie to accommodate a Highway 400 centreline realignment from Dunlop Street to Sunnidale Road. This curve, with the tall wall median will provide for a sight distance of 180m, which corresponds to a design speed of approximately 90 km/h. To increase the sight distance, the barrier will be shifted to the east to provide a 5.1m southbound shoulder. (Additional pavement will be provided in the northbound lanes to maintain a standard 3.0m shoulder). This will result in a sight distance of approximately 205 m and a corresponding 100 km/h design speed.

On the R=1164 curve at PI station 15+258, the barrier will be shifted to the east to provide a 4.7m southbound shoulder. (Additional pavement will be provided in the northbound lanes to maintain a standard 3.0m shoulder). This will increase the sight distance from approximately 220m to 250m and the design speed from 110km/h to over 120km/h.

Just north of St. Vincent Street on Highway 400 (Sta. 15+258), is an R=1164 metre curve, which provides a sight distance of 221 metres for southbound traffic in Lane 1. This distance provides for a design speed of 110 km/h. To increase the sight distance to 248 metres, which satisfies a 120 km/h design speed, the Ontario Tall Wall will be located 5.1m from the inside edge of southbound Lane 1, leaving a 4.7 metre southbound median shoulder. Additional pavement will be provided on the other median to maintain a standard shoulder width. The 4.7 metre southbound shoulder will provide for the increased sight distance of 248 metres.

Since there is sufficient property in the area of the R=873 curve at PI station 17+560, the northbound shoulder will be flared to the east to increase sight distance. This will be achieved by increasing the radius of the southbound control to 950m and maintaining the R=873m curve on the northbound control. By applying this flare and positioning the barrier on the R=950 southbound control, the sight distance will improve from 200m to 255m, which will improve the design speed from 100km/h to over 120km/h. This will result in a 9.9m (maximum) northbound shoulder.

c) From the south project limit to Essa Road (station 28+500), Highway 400 will be widened from six to eight lanes for a total pavement width that will vary from 37m to 40.8m. The granular platform and grading in this area, however, will be sufficiently wide to accommodate one additional lane per direction to provide for additional capacity beyond 2011. Highway 400 will be widened from six to ten lanes from Essa Road (station 28+500) to Bayfield Street (station 13+500), and six to eight lanes from Bayfield Street to the north project limit. The pavement widths will measure 44.5m and 37m respectively. The cross

section includes an 8.8m barrier median and a 1.25m buffer with HOV provision from the south project limit to just south of Molson Park Drive. In an area just south of Molson Park Drive, the 1.25m buffer and HOV provision become phased out and the barrier median is transitioned from 8.8m to 7.5m. The following table summarizes the cross section information:

Station		Proposed Pavment Width (m)	Median Width (m)	Number of Through Lanes After Widening	Provision for HOV Lane
From	To				
26+550 (South Project Limits)	28+500 (Essa Road)	37 to 40.8	7.5 to 8.8	8*	Yes (South Project limits to Barrie City Limits)
28+500 (Essa Road)	13+500 (Bayfield Street)	44.5	7.5	10	No
13+500 (Bayfield Street)	18+800 (North Project Limits)	37m	7.5	8	No

*Granular platform sufficient to accommodate one additional lane per direction.

d) Structural extensions and replacements will incorporate 3.0 m shoulders where possible. All other sections on Highway 400 will incorporate 3.0 m shoulders.

e) To satisfy vertical clearance requirements on Highway 400 under the Anne Street structure, a three metre grade raise will be required on Anne Street. To limit the impacts of the grading to between Henry Street (located east of Highway 400) and Edgehill Drive (located west of Higwhay 400), the existing K value (8) of the crest curve, and therefore the sight distance, over Highway 400 will be maintained. Implementing a vertical curve to a higher K value will create significantly greater impacts to the adjacent properties. Further improvements to Anne Street should be undertaken in a separate municipal Environmental Assessment as Anne Street is under the jurisdiction of the City of Barrie.

REMARKS

1. Scope of Work

The purpose of this study is to meet the demands of existing and future traffic operations on Highway 400 and improve geometrics, roadside safety, drainage and illumination to reflect current Ministry standards.

2. Limits of Project:

- South Limit: Sta. 26+550 (Town of West Gwillimbury), 1 km south of Highway 89
- North Limit: Sta. 18+807 (Township of Springwater), Junction at Highway 11

3. Adjacent Projects:

- GWP 172-91-00, Highway 89 Assessment Study, Alliston to Highway 400;
- GWP 377-90-00, Highway 400 – Highway 400 to Highway 404 Extension Link "Bradford Bypass";

- GWP 222-97-00, Planning Study (Part A) from North of Major MacKenzie Drive to South Canal Bridge (Within MTO Central Region);
- GWP 40-00-00, Highway 400 (Part B) from the South Canal Bridge northerly to 1 km south of Highway 89 (Within MTO Southwestern Region);
- Simcoe Area Transportation Network Needs Assessment;
- Highway 11 Corridor Study;
- Contract 98-65, G.W.P. 199-85-00, Highway 400/Molson Park Drive Interchange Improvements (completed fall of 2002);
- Contract 2000-0061, G.W.P. 38-99-00, Highway 89 from 1.1 km west to 0.5 km east of Highway 400 (completed the Fall of 2000);
- GWP 295-99-00, Highway 400/County Road 21 Interim Interchange Improvements; and
- GWP 199-89-00, Highway 400 Resurfacing, Molson Park Drive to Duckworth Street Interchange.

4. Structures:

There are 18 structure locations within the project limits. A review is recommended prior the implementation of the structural works to confirm the selected alternative. The preliminary structural recommendations are as follows:

Site Name	Site #	Bridge Classi-fication	Previous W.P. No.	Description of Existing Conditions	Recommendations
Highway89 Underpass	30-256	RA_HU	96-85-01, concrete overlay and repave	Single span rigid frame bridge, carrying 3 lanes of traffic over 6+ lanes of Hwy400 traffic; Spread footing foundations in abutments; Approx. 35m span, 15m width and 6°skew.	New 2.70m CPCI girder; 2 x 42.5m span; integral abutments with H-piles; spread footing at pier.
Churchill Sideroad (4th Line) Underpass	30-212	RA_HU	Built in 1948, no records for rehab.	Single span rigid frame box girder bridge, carrying 2 lanes of Churchill Side Road traffic soffit over 6 lanes of Hwy 400 traffic; Spread footing foundations in both abutments; Approx. 29 m span, 11.79 m width and 6° skew.	New 2.30m CPCI girder; 2 x 35.5m span; integral abutments with H-piles; H-pile at pier.
Innisfil Township Road (6th Line) Overpass	30-211	RA_HU	Built in 1949, no records for rehab. See concrete patch repairs on site	Single span rigid frame slab bridge, carrying 6 lanes of Hwy 400 traffic over 2 lanes of Innisfil Road; Spread footing foundations in both abutments; Approx. 11.4 m span, 30.5 m width and 6° skew; The bridge was rehabilitated in 1992 with concrete overlay on deck.	New 0.75m pre-tensioned hollow slab; 15.3m span; spread footings.
Innisfil Beach Road Overpass	30-210	RA_HU	88-85-01, deck patch repairs and repave	Single span rigid frame bridge, carrying 6 lanes of Hwy 400 traffic over 2 lane Innisfil Beach Road; Spread footing foundations in abutments; Approx. 14m span, 30m width and 6° skew.	New 1.85m CPCI girder; 29.4 m span; integral abutments with H-piles.
Barrie/Colling-wood Railway Overhead	30-209	N/A	88-85-02, deck patch repairs and repave	Single span rigid frame bridge, allowing a single track to pass under 6 lanes of Hwy 400 traffic; Spread footing foundations in both abutments; Approx. 11 m span, 30.5 m width and 3° skew to Hwy 400;Concrete patch repair and asphalt and waterproofing system replaced in 1988.	New0.75m pre-tensioned hollow slab; 13.5 m span; spread footings.
Stroud Road (10th Line) Underpass	30-208	RA_HU	87-85-05,deck patch repair and new barrier walls	2 span rigid frame bridge with a center pier, carrying 2 lanes of Stroud Road traffic over 3 lanes of Hwy 400 traffic in each direction; Spread footing foundations in both abutments and pier; Approx. 16 m-16 m spans, 11 m width and 20° skew.	New 2.30m CPCI girder; 2x37.5 m span; integral abutments with H-piles; spread footing at pier.
Molson Park Drive Overpass	30-179	UA_HD	26-96-01, replaced with a new bridge	Single span concrete slab on steel bathtub girder structure with integral abutment and RSS abutments, recently reconstructed; Approx. 39 m span, 49.4m width and 7B skew.	Widen existing structure by 2.2m; 39.0m span; integral abutments with H-piles.
Essa Road Overpass	30-178	UA_HD	105-70-09, deck widening	Single span rigid frame bridge, carrying 6 lanes of Hwy400 traffic over 4 lanes of Essa Road traffic; Spread footing foundations in	New 2.35m CPCI girder structure; 2 x 39.0m span;

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LOCATION: From 1 km south of Highway 89 to the Junction at Highway 11

Site Name	Site #	Bridge Classi-fication	Previous W.P. No.	Description of Existing Conditions	Recommendations
				both abutments; Approx. 28m span, 50.2m width and 48° skew to Hwy 400.	semi-integral abutments with spread footings; spread footing at pier.
Barrie/Col-lingwood Railway Overhead	30-177	N/A	Built in 1950, no records for rehab.	Single span rigid frame, allowing a single track to pass under 6 lanes of Hwy 400 traffic; Spread footing foundations at both abutments; Approx. 11 m span, 30.69 m width and 17° skew.	New 0.75m pre-tensioned hollow slab; 14.1m span; spread footings.
Essa Road W-S Ramp Overpass	-	UA_HD		N/A	New 0.8m reinforced concrete rigid frame; 11.7m span; spread footings.
Tiffin Street Overpass	30-176	UA_HU	Built in 1950, no records for rehab.	Single span concrete rigid frame, carrying 6 lanes of Hwy 400 traffic over 3-lane of Tiffin Street traffic; Spread footing foundations at both abutments; Approx. 15.5 m span, 30.5 m width and 23° skew to Hwy 400.	New 0.95m pre-tensioned box girder; 22.0m span; spread footings.
Culvert at 10+780	30-399	N/A	N/A	A group of CSP culverts, 5.03 x 3.05 x 43.0m, 5.03 x 3.05 x 43.0m, 2.3m (dia.) x 61m.	Rehabilitate and extend existing culvert.
Culvert at 11+675	30-571	N/A	N/A	CSP culvert, 5.0 x 3.0 x 42.7m.	
Culvert at 12+900	30-572	N/A	N/A	Concrete culvert with CSP culvert extension, (4.25 x 1.5 + CSP culvert dia. 4.5m). Total length 105m.	Rehabilitate and extend existing culvert.
Culvert at 13+100	30-573	N/A	N/A	Concrete Box Culvert, 4.25 x 1.50 x 95.0m.	Rehabilitate and extend existing culvert.
Culvert at 16+730	30-415	N/A	N/A	Concrete Arch Culvert, 7.3 x 4.0 x 60.0m.	Rehabilitate and extend existing culvert.
Dunlop Street Underpass	30-175	UA_HD	Built in 1950, no records for rehab.	2 span concrete deck on steel girders rigid frame bridge, carrying 2 lanes of traffic over 6 lanes of Hwy400 traffic; Spread footing foundations for both abutments and pier; Approx. 27 m–27m spans, 12.8m width and 52° skew.	New 2.30m steel box girder; 26+46+46+26m span; semi-integral abutments with spread footings; spread footings at piers.
Anne Street Underpass	30-347	UA_HD	Built in 1958, no records for rehab. See concrete patch repairs on site	2 span rigid frame with a center pier, carrying 4 lanes of Anne Street traffic over 3 lanes of Hwy 400 traffic in each direction; Spread footing foundations at both abutments and pier; Approx. 17+17 m span, 17.2 m width and 37.5° skew.	New 2.30m steel box girder; 41.5+48.5m span; semi-integral abutments with H-pile foundation; caisson footing at pier.
Sunnidale Road Underpass	30-173	UA_HU	Built in 1951 no records for rehab. See concrete patch repairs on site	Single span concrete rigid frame, carrying 2 lanes of Sunnidale Road traffic over 6 lanes of Hwy 400 traffic; Spread footing foundations at both abutments; Approx. 29 m span, 12 m width and 5° skew.	New 1.75m steel box girder; 2x33.1m span; integral abutments with H-piles; spread footing at pier.
Bayfield Street Underpass	30-172	UA_HD	Built in 1949o records for rehab.	Single span rigid frame, carrying 4 lane of Bayfield Street traffic over 6 lanes of Hwy400 traffic; Spread footing foundations in abutments; Approx. 33m span, 17.2 width and 29° skew.	New 2.20m steel box girder; 43+37.5 span; semi-integral abutments with spread footing; spread footing at pier.
St. Vincent Street U/P	30-394	UA_HU	Built in 1966, no records for rehab.	4 span post-tensioned continuous concrete slab structure supported on column piers, carrying 2 lanes of St. Vincent Street traffic over 6 lanes of Hwy400 traffic; Spread footing foundations at all abutments and piers; Approx. 20-24-24-20 m span layout, 13 m width and 14° skew.	Rehabilitate existing structure (to be undertaken by municipality).
Duckworth Street Overpass	30-140	UA_HD	166-67-02, deck widening	Single span concrete rigid frame, allowing 2 lanes to pass under 400; Spread footing foundations in abutments; Approx. 13m span, 48m width and 27° skew.	New 2.0m steel box girder; 38.0m span; semi-integral abutments with spread footings.
Willow Creek Bridge	30-139	N/A	99-85-06, deck widening 707-89-01, deck replacement	3 span concrete slab on steel beams bridge, carrying 8 lanes of Hwy 400 traffic over Willow Creek; Steel H pile foundations at abutments and piers; Approx.12-18-12 m span layout, 48.5 m width; The bridge was rehabilitated in 1991 with deck replacement for original portion and concrete for extended portion.	Rehabilitate existing structure

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5. Interchanges / Crossing Roads

The following interchange/crossing road improvements have been identified in this study:

Interchange Location	Existing Interchange Configuration	Proposed Improvements
Highway 89	Diamond Southbound/ Parclo A Northbound	o Realign/improve geometrics of ramps and upgrade interchange to full Parclo A-4. o Realign Highway 89 approximately 30m to the north to accommodate construction staging.
4th Line	N/A	o Replace structure to accommodate Highway 400 widening. o Realign 4th Line alignment to accommodate construction staging. o Raise 4th Line profile to comply with vertical clearance requirements for the structure over Highway 400.
6th Line	N/A	o Replace structure to accommodate Highway 400 widening. o Lower 6th Line profile to comply with vertical clearance requirements underneath Highway 400 structure.
Innisfil Beach Road	Parclo A	o Maintain interchange configuration but improve geometrics of existing ramps. o Lower Innisfil Beach Road profile to comply with vertical clearance requirements underneath Highway 400 structure.
10th Line	N/A	o Replace structure to accommodate Highway 400 widening. o Raise 10th Line profile to comply with vertical clearance requirements for the structure over Highway 400.
Molson Park Drive	Diamond	o Ramps and interchange configuration to be maintained with the addition of right turn lanes on the S-E/W ramp, the N-E/W ramp, the eastbound approach to the E/W-S ramp, and the westbound approach to the E/W-N ramp.
Essa Road	Partial Parclo A	o Add W-S ramp to upgrade to full Parclo A-4. o Improve geometrics of existing ramps. o Provide for direct connection from interchange to immediately adjacent lands in the southwest quadrant. o Lower Essa Road profile to comply with vertical clearance requirements underneath Highway 400 structure.
Tiffin Street	N/A	o Replace structure to accommodate Highway 400 widening.
Dunlop Street	Partial Parclo B	o Maintain interchange configuration but improve geometrics of existing ramps. o Raise Dunlop Street profile to comply with vertical clearance requirements for the structure over Highway 400.
Anne Street	N/A	o Replace structure to accommodate Highway 400 widening. o Raise Anne Street profile to comply with vertical clearance requirements for the structure over Highway 400.
Sunnidale Road	N/A	o Replace structure to accommodate Highway 400 widening. o Raise Sunnidale Road profile to comply with vertical clearance requirements for the structure over Highway 400.
Bayfield Street	Parclo A Southbound/ Diamond Northbound	o Maintain interchange configuration but improve geometrics of existing ramps. o Realign Highway 89 approximately 20m to the north to accommodate construction staging. o Raise Bayfield Street profile to comply with vertical clearance requirements for the structure over Highway 400.
St. Vincent Street	N/A	o No improvements
Duckworth Street	Partial Parclo A	o Maintain interchange configuration but improve geometrics of existing ramps. o Lower Duckworth Street profile to comply with vertical clearance requirements underneath Highway 400 structure.

6. Construction Staging:

Long-term lane width reductions, single lane closures and localized overnight closures are required on Highway 400 to accommodate the construction staging. Lane width reductions, lane closures and overnight closures are anticipated on all cross roads, with the exception of Sixth line, Tenth line and Sunnidale Road, which are expected to be closed at Highway 400 during the construction of the new structure. Conceptual staging plans were developed as a part of this study and are included in the Preliminary Design Report.

7. Property Requirements:

Property will be required from numerous areas adjacent to the Highway 400 right-of-way and at cross road and interchange locations to accommodate the proposed widening and improvements. Refer to the preliminary design plates for the location and extent of these areas. The property requirements are based on the area required to accommodate the ultimate configurations of the improvements.

8. Illumination:

Based on Directive PLNG-B-05, full illumination is recommended for all interchanges within the project limits. As well, fill in illumination for the mainline from the south Barrie city limits, just south of Molson Park Drive, to Duckworth Street is recommended.

9. Traffic Signals:

Signalized intersections are proposed for at the following locations:

- Highway 89 S-E/W Ramp terminal;
- Highway 89 N-E/W Ramp terminal;
- Highway 89/Reive Road intersection;
- Innisfil Beach Road S-E/W Ramp terminal;
- Innisfil Beach Road N-E/W Ramp terminal;
- Innisfil Beach Road/industrial Park Drive intersection;
- Molson Park Drive S-E/W Ramp terminal*;
- Molson Park Drive N-E/W Ramp terminal*;
- Molson Park Drive/Barrie View Drive intersection*;
- Essa Road S-E/W Ramp terminal*;
- Essa Road N-E/W Ramp terminal*;
- Essa Road/Fariview Road intersection*;

- Dunlop Street Northbound Ramp terminal;
- Dunlop Street Southbound Ramp terminal*;
- Dunlop Street/Anne Street intersection*;
- Anne Street/Wellington Street intersection*;
- Anne Street/Edgehill Drive intersection*:
- Bayfield Street S-E/W Ramp*;
- Bayfield Street N-E/W Ramp*;
- Duckworth Street S-E/W Ramp*;
- Duckworth Street N-E/W Ramp; and
- Duckworth Street/Bell Farm Road intersection*.

** indicates existing signalized intersections*

10. Rail Crossings

There are no at-grade railway crossings within the project limits. There are, however, two railway crossings, that Highway 400 crosses over:

- Barrie/Collingwood Railway Overhead (Site 30-209); and
- Barrie/Collingwood Railway Overhead (Site 30-177).

To accommodate the widening of Highway 400, the structures at these locations will be replaced. The new structures will be designed and constructed in accordance with the requirements of the Barrie/Collingwood Railway.

11. Utilities:

The following utilities are located within or adjacent to the Highway 400 right-of-way, and may be impacted:

- Bell Canada;
- Barrie Hydro;
- Innisfil Hydro;
- Hydro One;
- Town of Innisfil (water);
- City of Barrie (water);
- Simcoe County Area Network;
- Rogers Cable; and
- Enbridge Consumers Gas.

12. Drainage:

All existing culverts will be extended / replaced where they are affected by highway widening. Median storm sewer will be replaced as part of the recommended highway improvements.

Details of the above noted drainage work, as well as information relating to existing drainage features and watercourses are available in the Preliminary Drainage and Hydrology Report, which was prepared as a part of this study under separate cover.

13. Signing and Guiderail

Guiderail and signing, throughout the project limits, will be updated and/or replaced to meet current MTO warrants and standards.

14. Assumptions, Designations, Transfers, Road Closings:

The realignments of Reive Road, Cedar Pointe Drive and Coulter Street will be undertaken by the Ministry of Transportation and then transferred to the respective municipalities.

Access to Rose Street, via the Bayfield Street E/W-N, will be closed.

15. Environmental Assessment:

The environmental category is Group 'B' as outlined in the Class Environmental Assessment for Provincial Transportation Facilities, (2000).

16. Miscellaneous:

Commuter parking lots are located in the southeast quadrant at the Highway 89 interchange, the southwest quadrant of the Innisfil Beach Road interchange, and in the northwest quadrant of the Essa Road interchange. Proposed improvements to the Highway 89 and Innisfil Beach Road interchanges will displace the commuter parking lots. Opportunities for relocating the parking lot at Highway 89 will be investigated through MTO Corridor Control. Relocation of the Innisfil Beach Road parking lot to the northeast quadrant has been approved. Opportunities for expanding and/or adding an additional parking lot at Essa Road will be investigated through MTO Corridor Control.

Also, a total of five potential snow drifting areas have been identified along the Highway 400 corridor within the project limits. Proposed property requirements to accommodate remedial treatments are included in the overall right-of-way requirement.

TYPICAL STRUCTURE CROSS SECTIONS AND ELEVATIONS (N.T.S.)

Figure 1 - Highway 89 Underpass (Site 30-256)

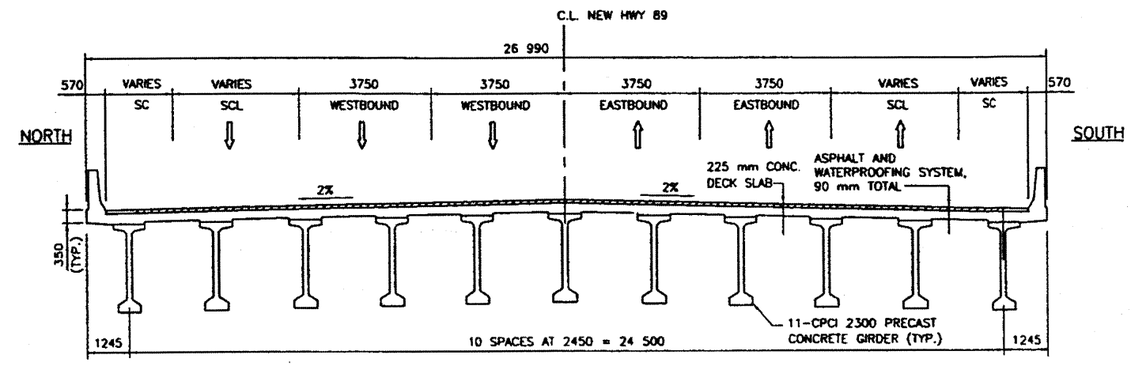

Figure 2 - 4th Line (Churchill Side Road) Underpass (Site 30-212)

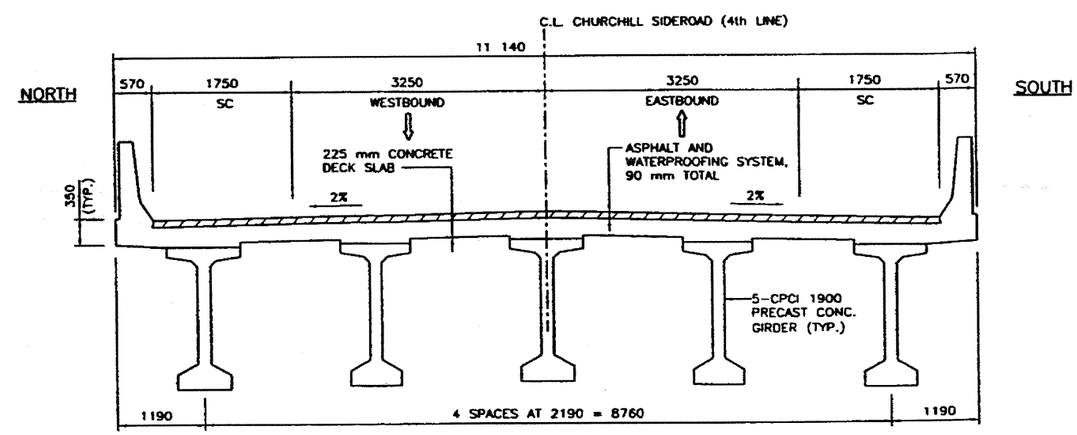

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Figure 3 - 6th Line (Innisfil Township Road) Overpass (Site 30-211)

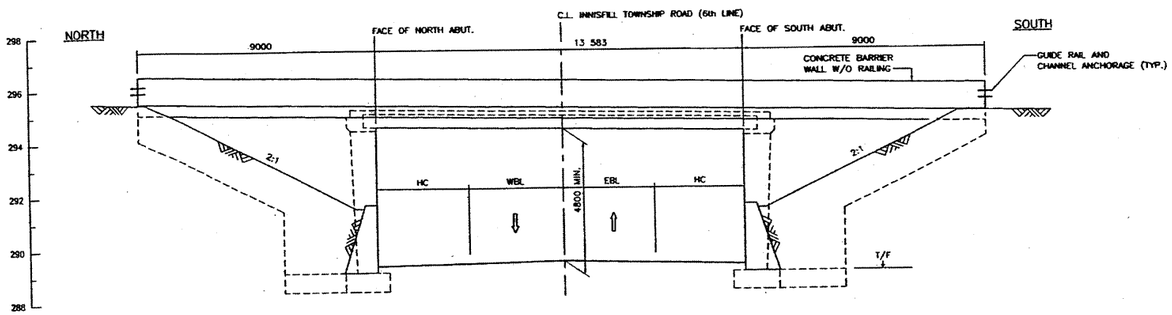
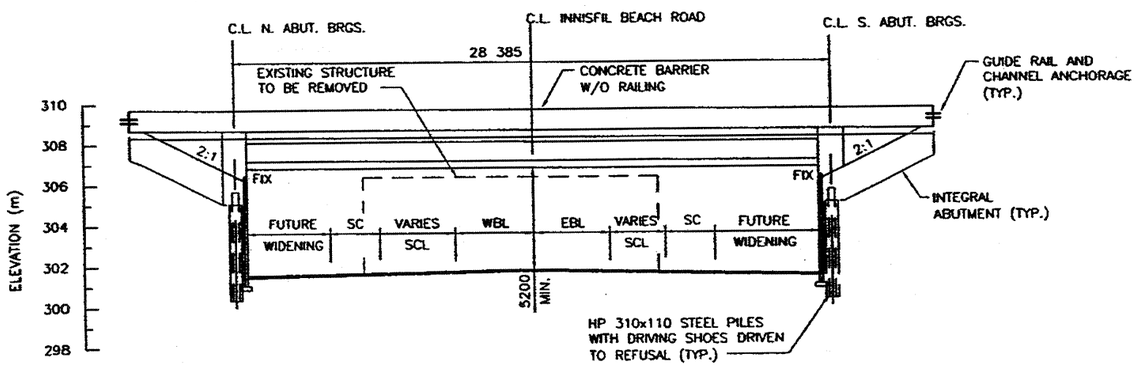


Figure 4 - Innisfil Beach Road Overpass (Site 30-210)



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LOCATION: From 1 km south of Highway 89 to the Junction at Highway 11

Figure 5 - Barrie/Collingwood Railway Overhead (Site 30-209)

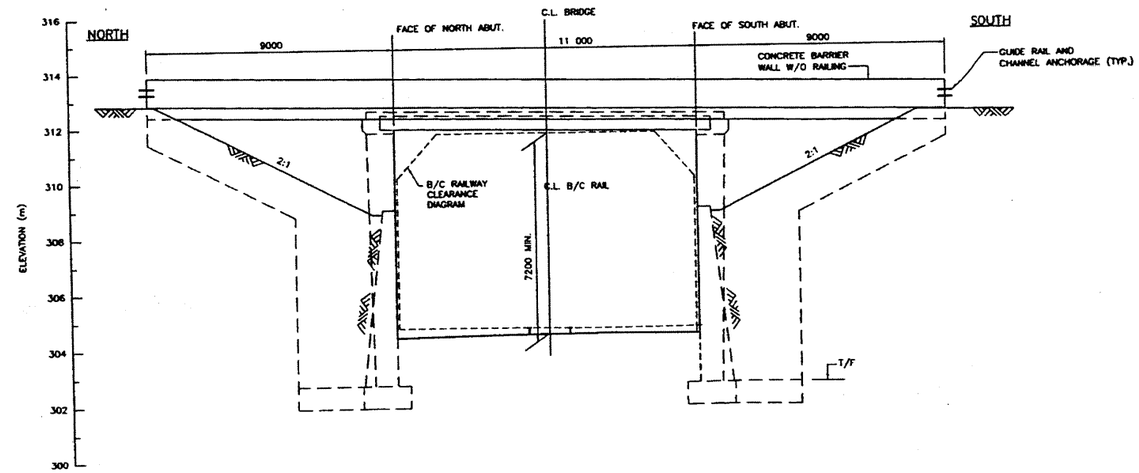
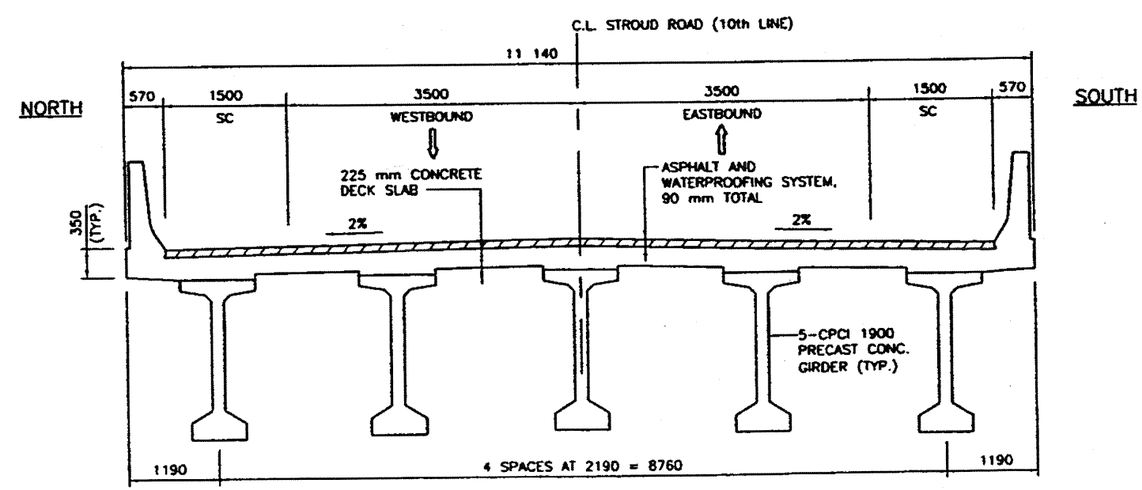


Figure 6 - 10th Line (Stroud Road) Underpass (Site 30-208)



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LOCATION: From 1 km south of Highway 89 to the Junction at Highway 11

Figure 7 – Molson Park Drive Overpass (Site 30-179)

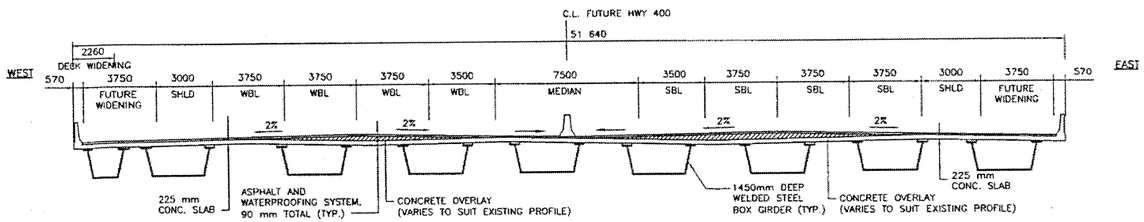


Figure 7 - Essa Road Overpass (Site 30-178)

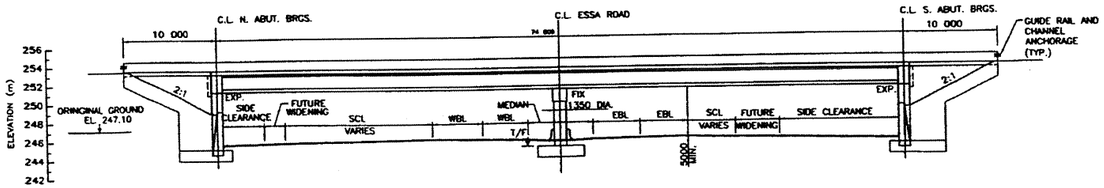
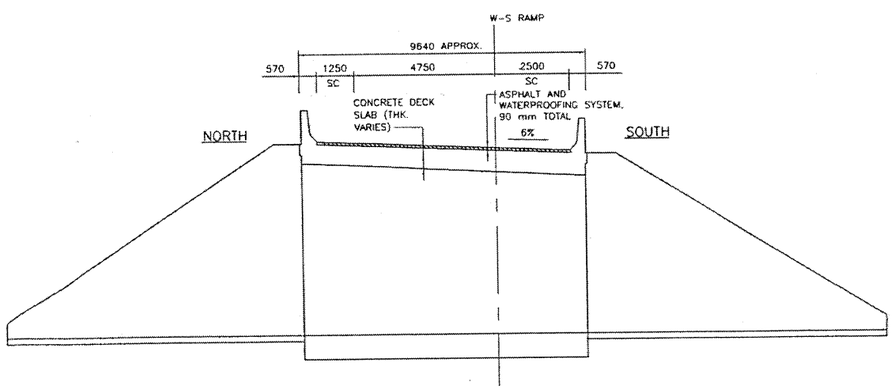


Figure 7a - Essa Road W-S Ramp Bridge



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TYPE OF PROJECT: Widening, Interchange, Structure Replacement, Structure Widening and Illumination **LENGTH:** 30 km

LOCATION: From 1 km south of Highway 89 to the Junction at Highway 11

Figure 8 - Barrie/Collingwood Railway Overhead Site (30-177)

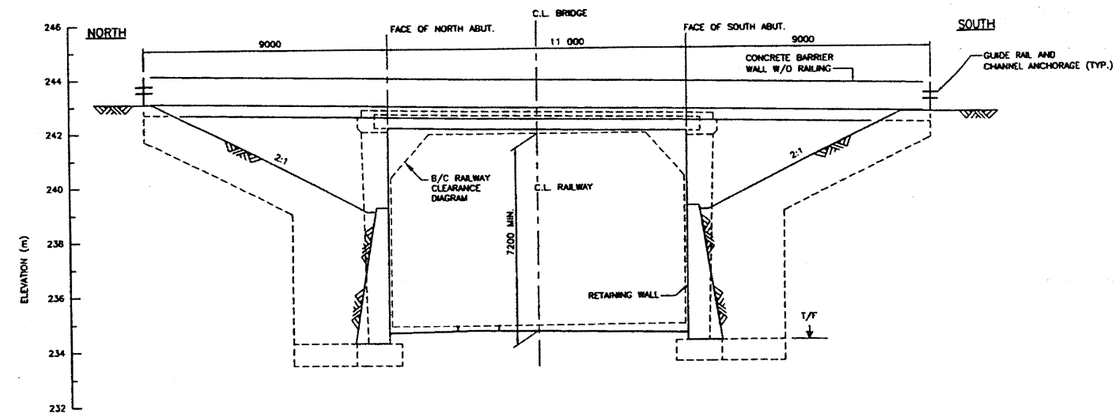


Figure 9 - Tiffin Street Overpass (Site 30-176)

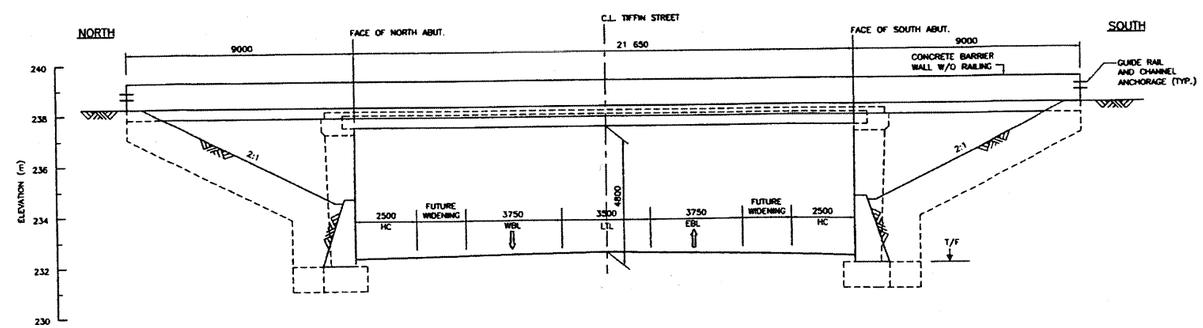
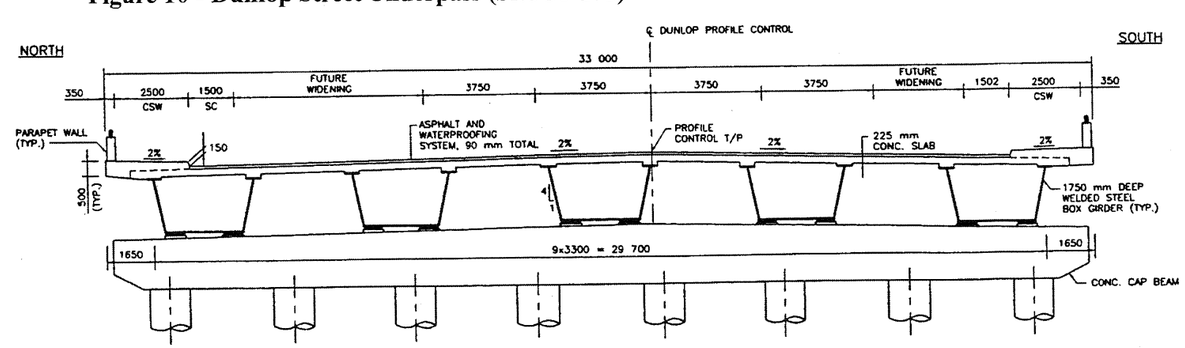


Figure 10 - Dunlop Street Underpass (Site 30-175)



WORK PROJECT NO.: 30-95-00 **OPERATIONS:** London **HWY. NO.:** 400
TYPE OF PROJECT: Widening, Interchange, Structure Replacement, Structure Widening and Illumination **LENGTH:** 30 km

LOCATION: From 1 km south of Highway 89 to the Junction at Highway 11

Figure 11 - Anne Street Underpass Site (30-347)

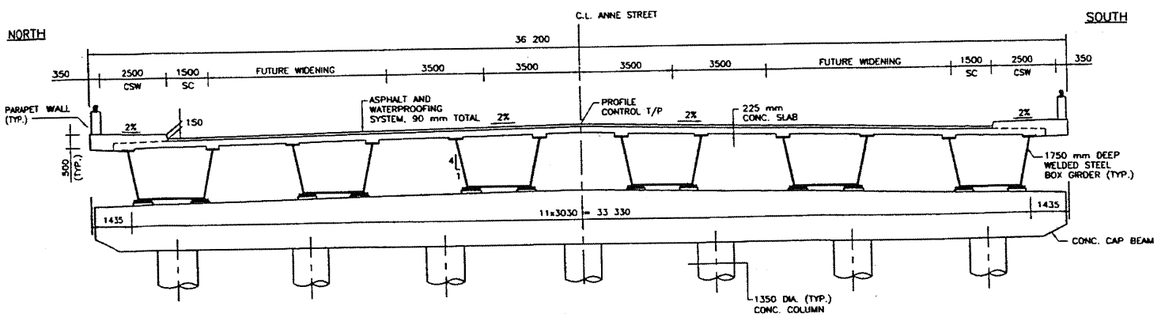


Figure 12 - Sunnidale Road Underpass (Site 30-173)

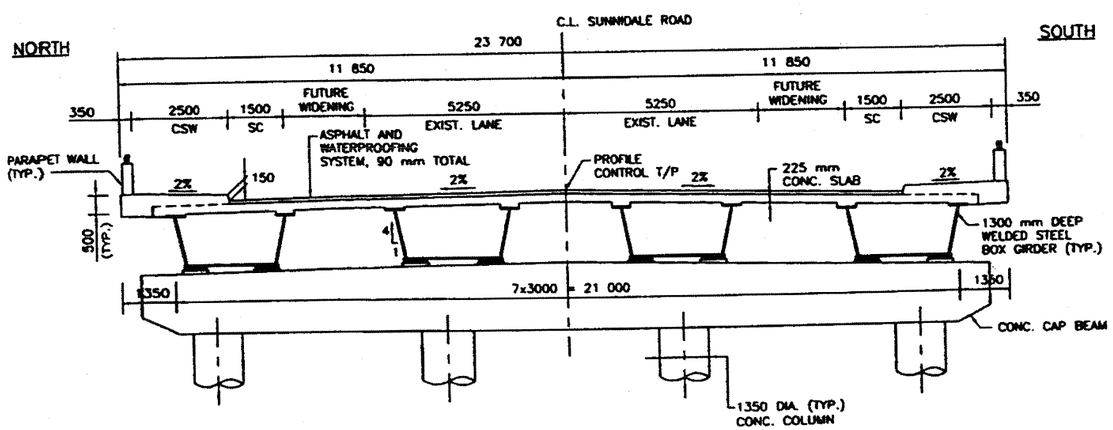
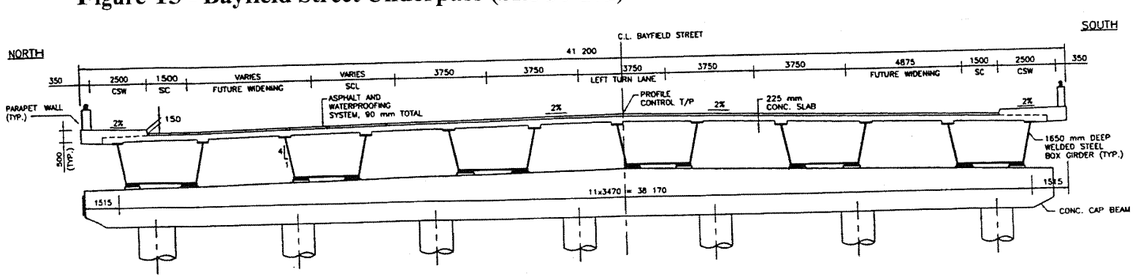


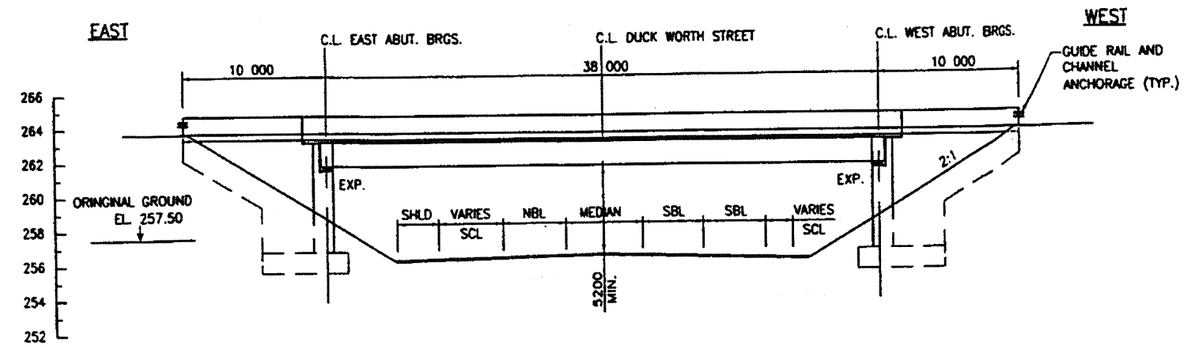
Figure 13 - Bayfield Street Underpass (Site 30-172)



WORK PROJECT NO.: 30-95-00 **OPERATIONS:** London **HWY. NO.:** 400
TYPE OF PROJECT: Widening, Interchange, Structure Replacement, Structure Widening and Illumination **LENGTH:** 30 km

LOCATION: From 1 km south of Highway 89 to the Junction at Highway 11

Figure 14 - Duckworth Street Overpass (Site 30-140)

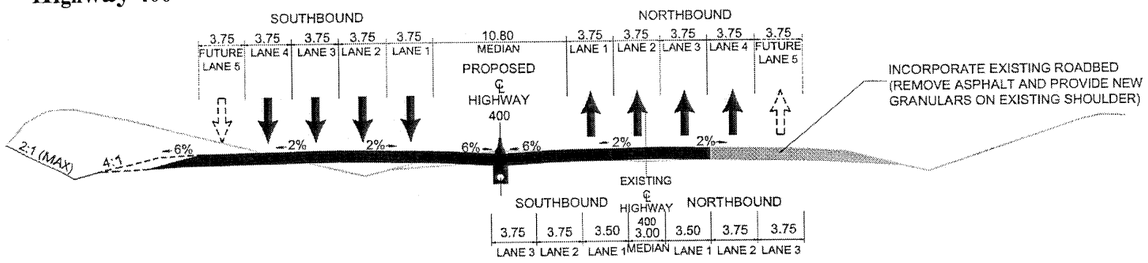


WORK PROJECT NO.: 30-95-00 **OPERATIONS:** London **HWY. NO.:** 400
TYPE OF PROJECT: Widening, Interchange, Structure Replacement, Structure Widening and Illumination **LENGTH:** 30 km

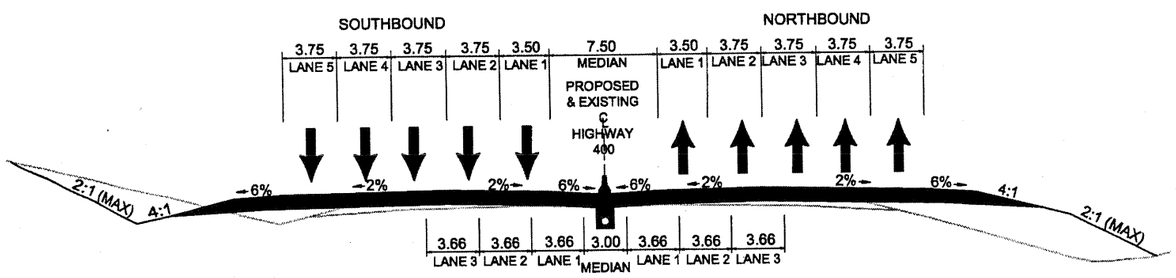
LOCATION: From 1 km south of Highway 89 to the Junction at Highway 11

TYPICAL ROADWAY CROSS SECTIONS (N.T.S.)

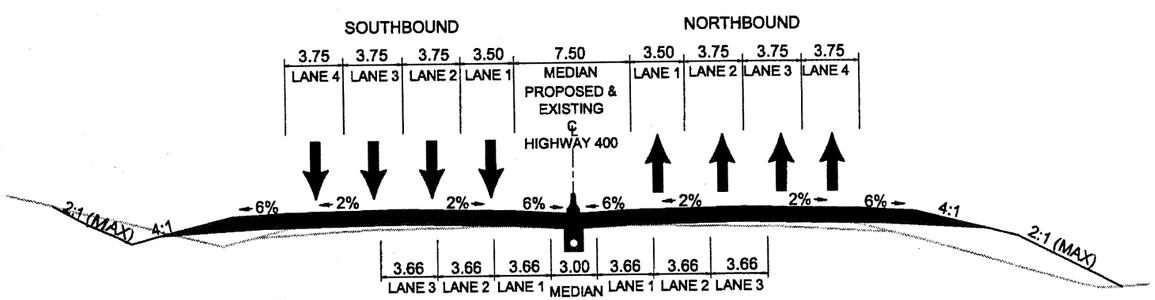
Highway 400



TECHNICALLY PREFERRED SECTION
1KM SOUTH OF HIGHWAY 89 TO ESSA ROAD (WITH PROVISION FOR HOV LANES)



TECHNICALLY PREFERRED SECTION
ESSA ROAD TO BAYFIELD STREET (NO PROVISION FOR HOV LANES)

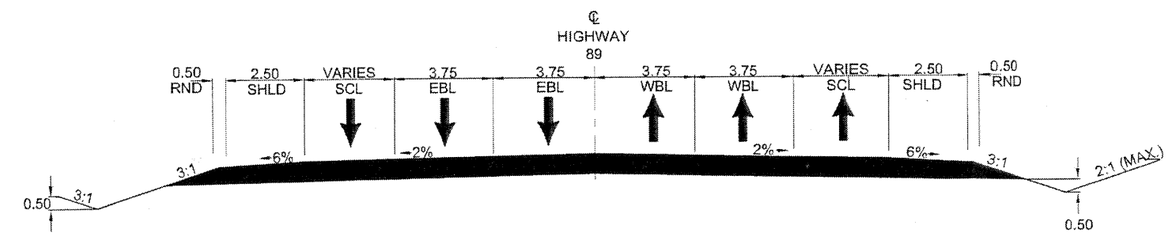


TECHNICALLY PREFERRED SECTION
BAYFIELD STREET TO HIGHWAY 11
(NO PROVISION FOR HOV LANES)

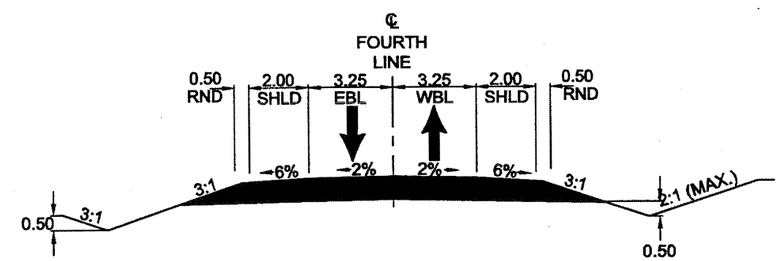
WORK PROJECT NO.: 30-95-00 **OPERATIONS:** London **HWY. NO.:** 400
TYPE OF PROJECT: Widening, Interchange, Structure Replacement, Structure Widening and Illumination **LENGTH:** 30 km

LOCATION: From 1 km south of Highway 89 to the Junction at Highway 11

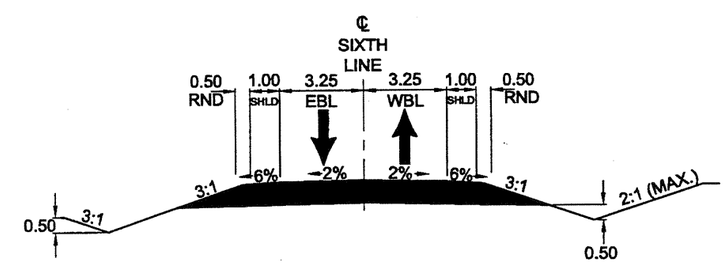
Highway 89



Fourth Line



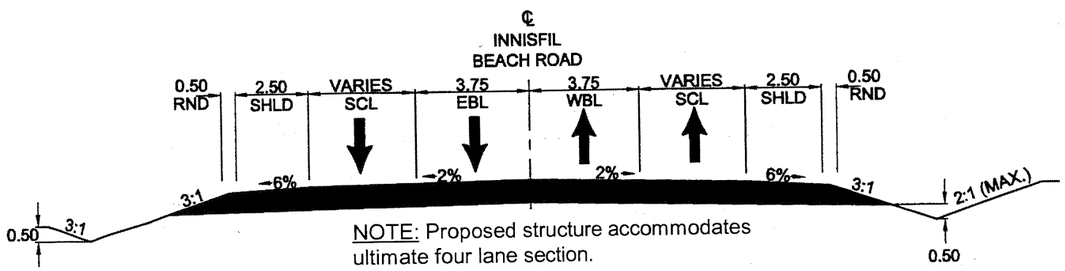
Sixth Line



WORK PROJECT NO.: 30-95-00 **OPERATIONS:** London **HWY. NO.:** 400
TYPE OF PROJECT: Widening, Interchange, Structure Replacement, Structure Widening and Illumination **LENGTH:** 30 km

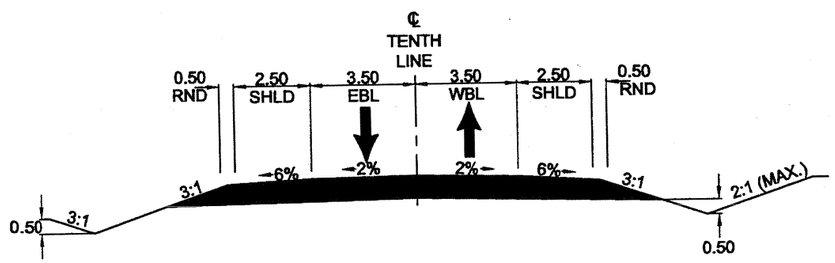
LOCATION: From 1 km south of Highway 89 to the Junction at Highway 11

Innisfil Beach Road

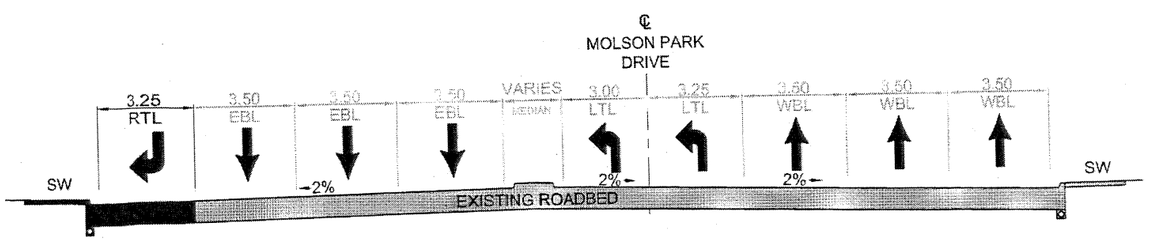


NOTE: Proposed structure accommodates ultimate four lane section.

Tenth Line



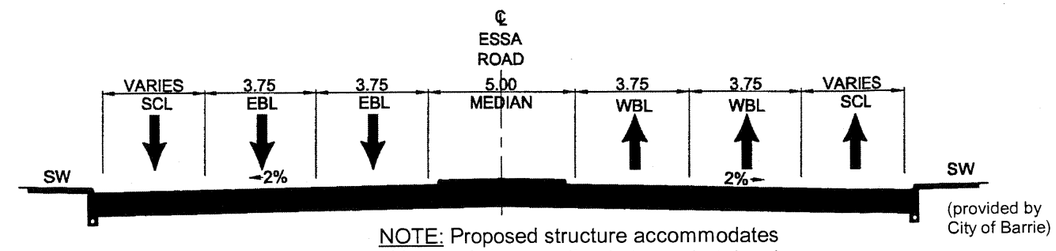
Molson Park Drive (No changes proposed except for right-turn-lanes)



WORK PROJECT NO.: 30-95-00 **OPERATIONS:** London **HWY. NO.:** 400
TYPE OF PROJECT: Widening, Interchange, Structure Replacement, Structure Widening and Illumination **LENGTH:** 30 km

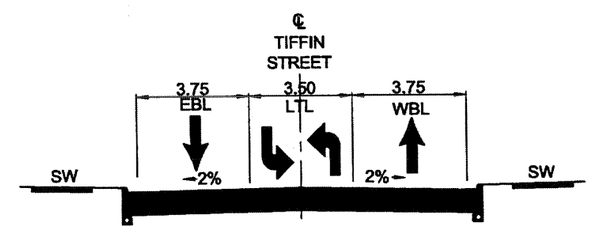
LOCATION: From 1 km south of Highway 89 to the Junction at Highway 11

Essa Road



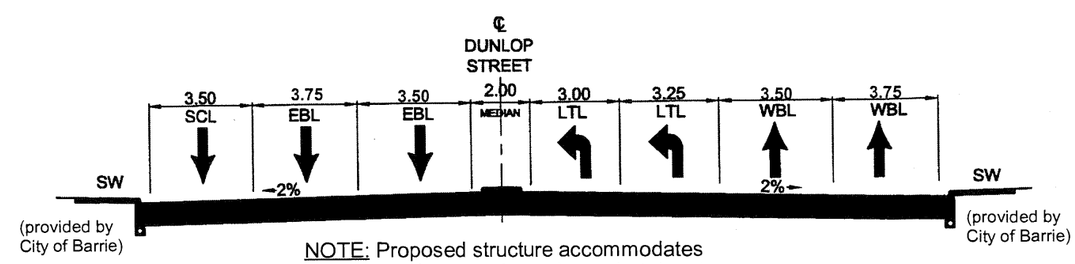
NOTE: Proposed structure accommodates ultimate six lane section to be undertaken by the municipality

Tiffin Street



NOTE: Proposed structure accommodates ultimate four lane section to be undertaken by the municipality.

Dunlop Street

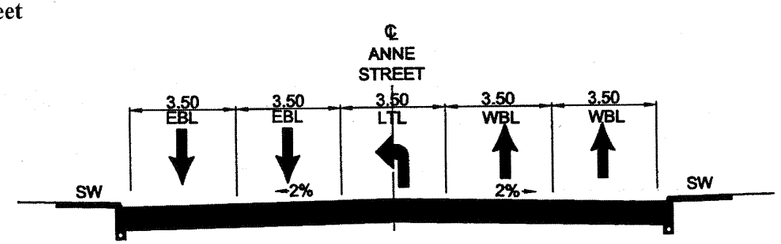


NOTE: Proposed structure accommodates ultimate six lane section to be undertaken by the municipality.

WORK PROJECT NO.: 30-95-00 **OPERATIONS:** London **HWY. NO.:** 400
TYPE OF PROJECT: Widening, Interchange, Structure Replacement, Structure Widening and Illumination **LENGTH:** 30 km

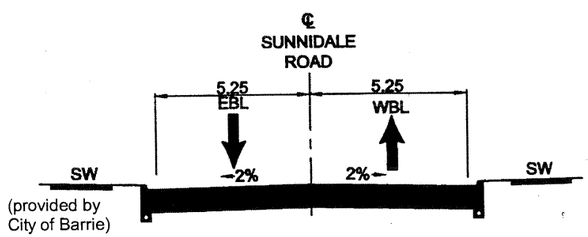
LOCATION: From 1 km south of Highway 89 to the Junction at Highway 11

Anne Street



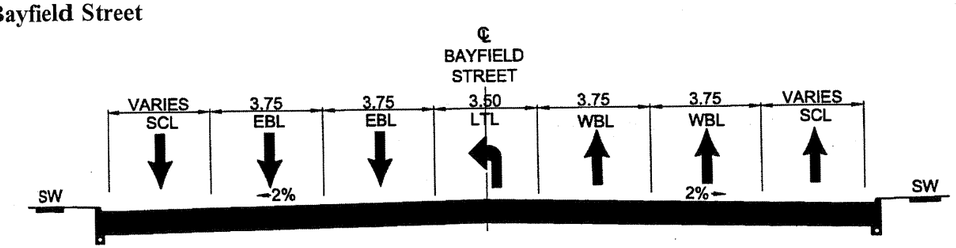
NOTE: Proposed structure accommodates ultimate six lane section to be undertaken by the municipality.

Sunnidale Road



NOTE: Proposed structure accommodates ultimate four lane section to be undertaken by the municipality.

Bayfield Street

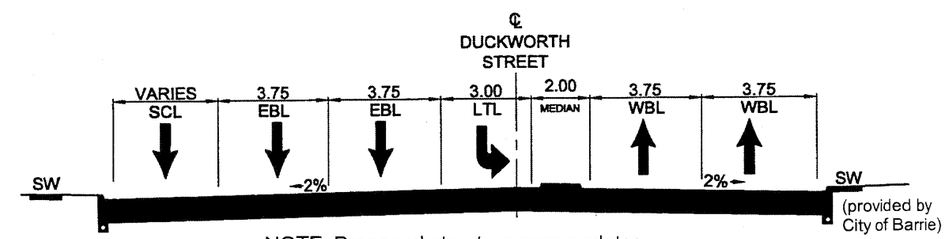


NOTE: Proposed structure accommodates ultimate six lane section.

WORK PROJECT NO.: 30-95-00 **OPERATIONS:** London **HWY. NO.:** 400
TYPE OF PROJECT: Widening, Interchange, Structure Replacement, Structure Widening and Illumination **LENGTH:** 30 km

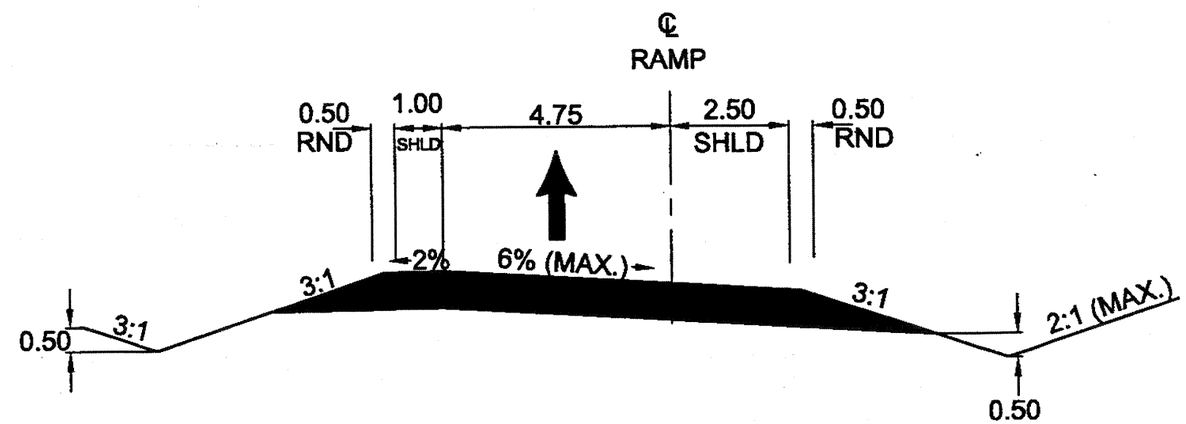
LOCATION: From 1 km south of Highway 89 to the Junction at Highway 11

Duckworth Street



NOTE: Proposed structure accomodates ultimate four lane section.

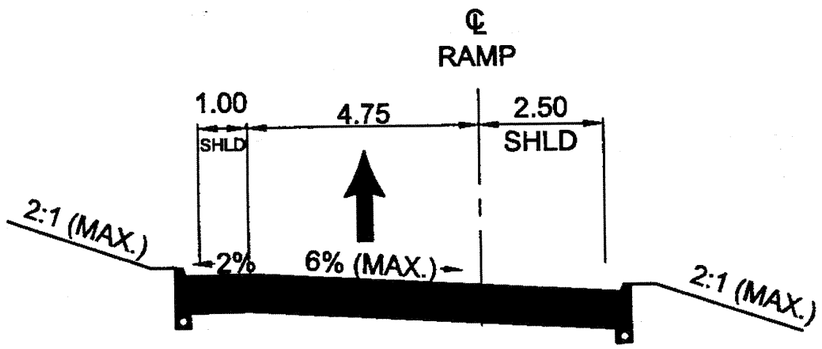
Interchange Ramps



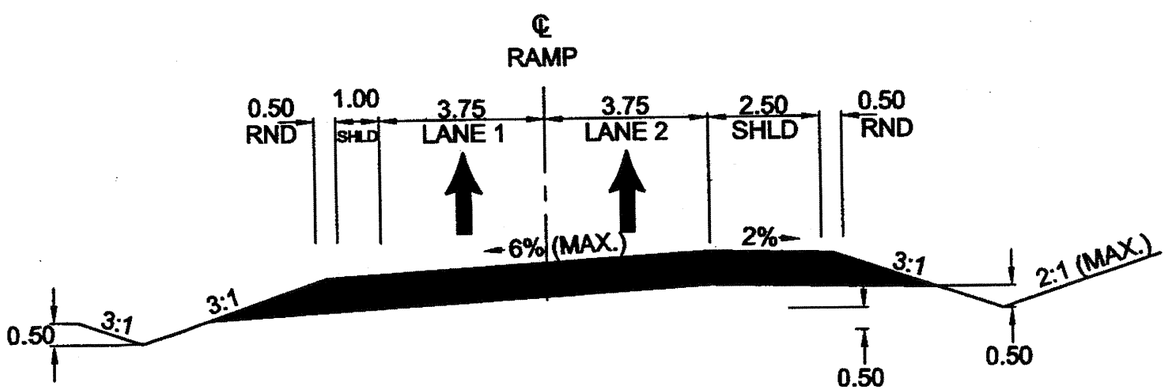
RURAL SINGLE LANE RAMP SECTION

WORK PROJECT NO.: 30-95-00 **OPERATIONS:** London **HWY. NO.:** 400
TYPE OF PROJECT: Widening, Interchange, Structure Replacement, Structure Widening and Illumination **LENGTH:** 30 km

LOCATION: From 1 km south of Highway 89 to the Junction at Highway 11



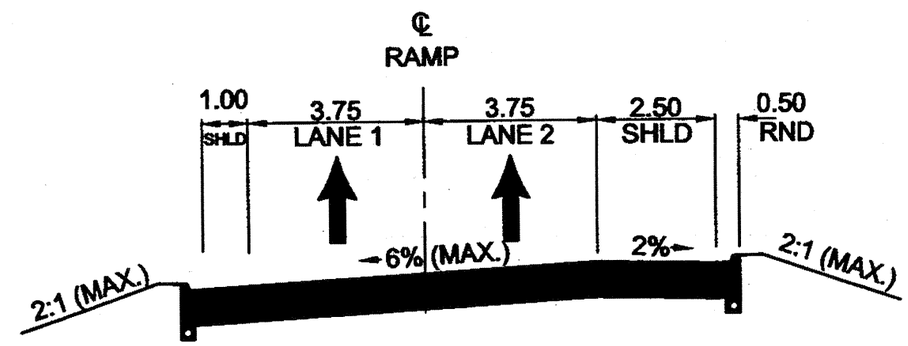
URBAN SINGLE LANE RAMP SECTION



RURAL 2-LANE RAMP

WORK PROJECT NO.: 30-95-00 **OPERATIONS:** London **HWY. NO.:** 400
TYPE OF PROJECT: Widening, Interchange, Structure Replacement, Structure Widening and Illumination **LENGTH:** 30 km

LOCATION: From 1 km south of Highway 89 to the Junction at Highway 11

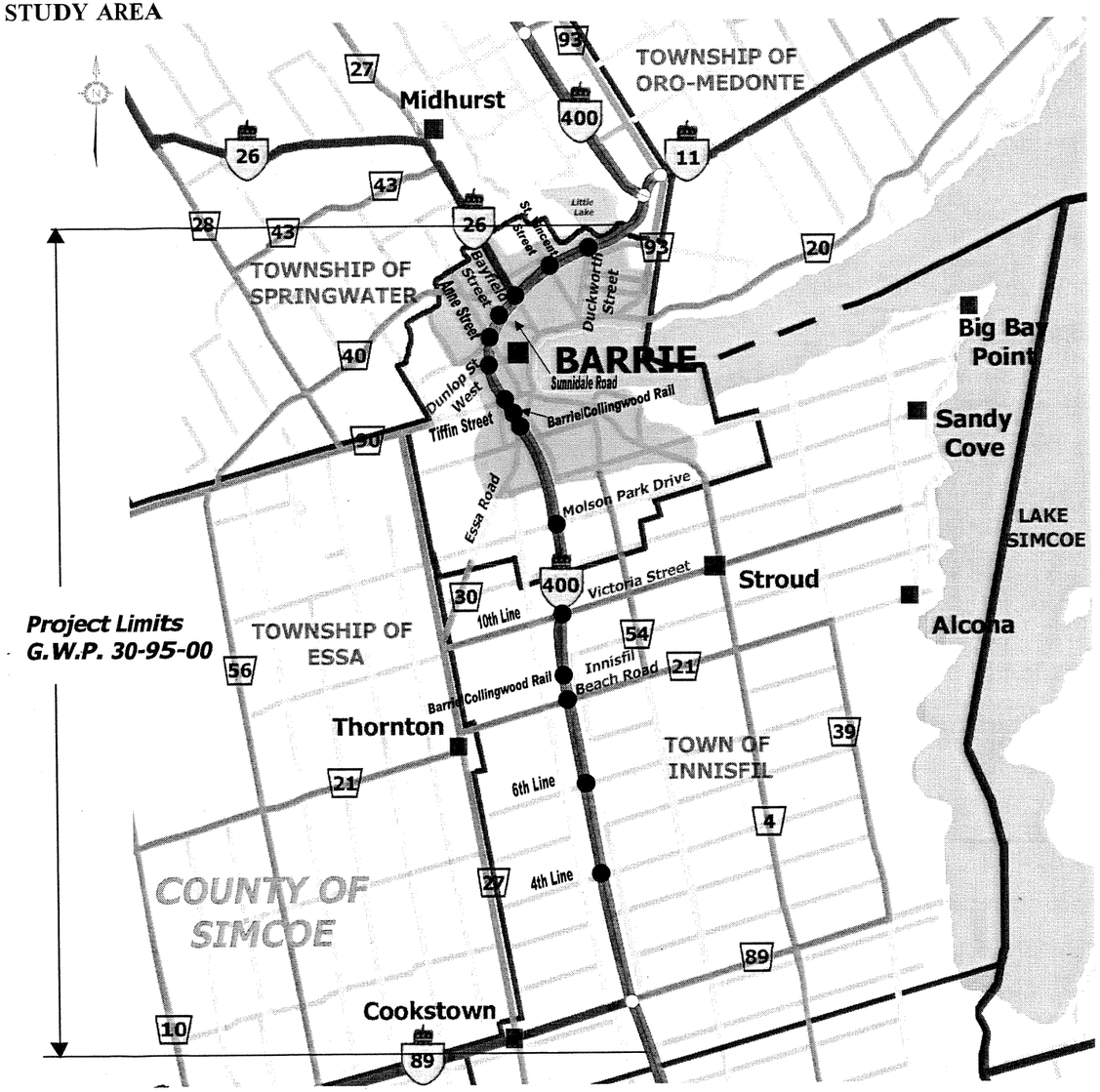


URBAN 2-LANE RAMP

WORK PROJECT NO.: 30-95-00 **OPERATIONS:** London **HWY. NO.:** 400
TYPE OF PROJECT: Widening, Interchange, Structure Replacement, Structure Widening and Illumination **LENGTH:** 30 km

LOCATION: From 1 km south of Highway 89 to the Junction at Highway 11

STUDY AREA



Ontario

Ministry of Transportation
Southwestern Region
Planning and Design Section

HIGHWAY 400 PLANNING AND PRELIMINARY DESIGN STUDY

FROM 1 KM SOUTH OF HIGHWAY 89 NORTHERLY 30 KM TO THE JUNCTION AT HIGHWAY 11

COUNTY OF SIMCOE
TOWN OF BRADFORD WEST GWILLIMBURY
TOWN OF INNISFIL
CITY OF BARRIE
TOWNSHIP OF SPRINGWATER

G.W.P. 30-95-00

APPENDIX M

CULTURAL RESOURCE IDENTIFICATION

HIGHWAY 400 PRELIMINARY DESIGN
HIGHWAY 89 TO HIGHWAY 11

CULTURAL RESOURCE INDENTIFICATION

October 5, 2000

Prepared for:
Cole, Sherman & Associates Ltd.

Prepared by:
Unterman McPhail Associates
Heritage Resource Management Consultants

HIGHWAY 400 PRELIMINARY DESIGN
HIGHWAY 89 TO HIGHWAY 11

CULTURAL RESOURCE INDENTIFICATION

The firm of Unterman McPhail Associates was retained to undertake a survey of cultural resources that may be impacted as part of the proposed widening of Highway 400 from Highway 89 to Highway 93. The field survey was undertaken by Richard Unterman, Project Manager and Jean Simonton, Associate, on 13 September 2000.

Based on the mapping provided, the following built heritage features (BHF) and cultural landscape units (CLU) were identified. The sites are numbered sequentially starting at the south end of the study area and progressing to the north. Unterman McPhail Associates reviewed BHF's and CLU's features located adjacent the Highway 400 ROW and on parallel service roads. The survey included the examination of local service roads, crossroads and interchanges within 500 metres of the ROW.

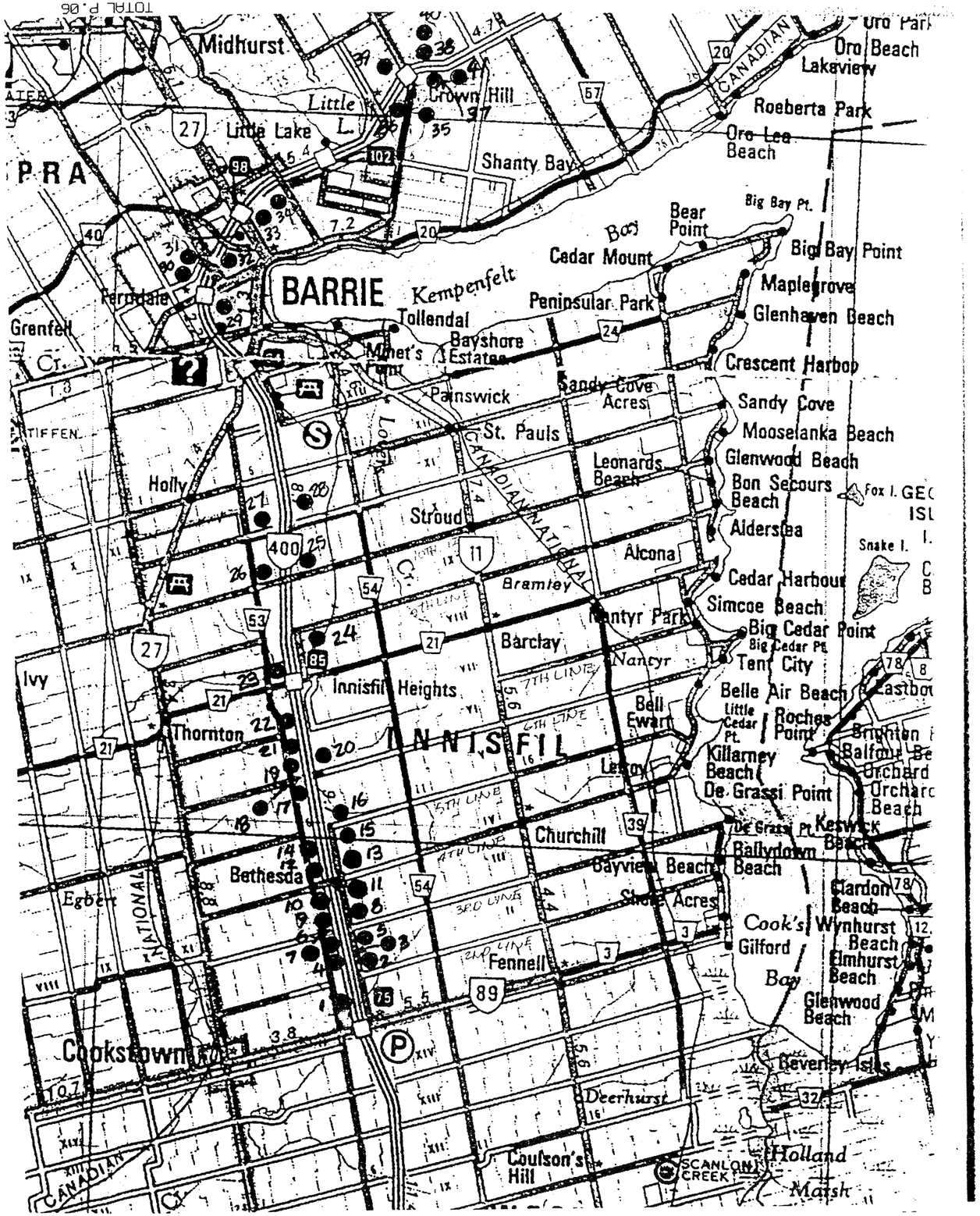
This initial report of findings will be included in the larger cultural resource report documenting the cultural environment associated with Highway 400 – Highway 89 to Highway 11. The purpose of this letter report is to assist in the generation of alternatives for the study area. Detailed evaluation and assignment of significance for built heritage features and cultural landscapes will follow.

TABLE 1: CULTURAL RESOURCES

Site #	Feature type	Feature category	Location
1	CLU	Farm complex	5665 5th Sideroad, west of Hwy 400, south of 2nd Line. Photos 1-1, 1-2.
2	CLU	Farm complex	Located on 2nd Line, north side, east of Hwy 400 at Reive Blvd. Photo 2-22.
3	CLU	Roadscape	Reive Blvd., paralleling Hwy 400, east side, 2nd Line north to 5th Line. Photo 1-3.
4	CLU	Farm complex (abandoned)	Located on 5th Sideroad, west of Hwy 400. North of 2nd Line. Photos 2-20, 2-21.
5	CLU	Farm complex	575 Reive Blvd., east of Hwy 400. Obscured by vegetation, no photo.
6	CLU (possibly BHF)	Farm complex (Farmhouse)	3rd Line, south side, west of Hwy 400. Confirm outbuildings. Photo 2-17.

Site #	Feature type	Feature category	Location
7	CLU	Roadscape	3rd Line, east and west of Hwy 400, road broken by highway. Photo 2-19
8	CLU	Farm complex	3428 3rd Line, north side, east of Hwy 400. Photo 2-18.
9	CLU	Farm complex	5th Sideroad, between 3rd and 4th Lines, west of Hwy 400. No photo.
10	CLU	Farm complex	3541 4th Line, south side, west of Hwy 400, "Bethesda Acres". Photos 1-6, 1-8.
11	BHF	Bridge	4th Line bridge over Hwy 400. Photos 1-9, 1-10.
12	CLU	Farm complex	3540 4th Line, north side, west of Hwy 400. Photos 1-5, 1-7.
13	CLU	Farm complex	3430 4th Line, north side, east of Hwy 400. Photo 1-4.
14	CLU	Farm complex	5th Sideroad, between 4th and 5th Lines, west of Hwy 400. Photos 1-11, 2-16.
15	CLU	Farm complex	3433 5th Line, south side, east of Hwy 400. Photo 1-12.
16	CLU	Farm complex	3434 5th Line, north side, east of Hwy 400. Photo 1-13.
17	CLU	Farm complex	3573 6th Line, south side, west of Hwy 400. Photo 1-16.
18	CLU	Roadscape	6th Line east and west of Hwy 400, road passes under highway. Photo 1-17.
19	CLU	Farm complex (no farmhouse)	6th Line, north side, west of Hwy 400, silo a landmark on 5th Sideroad. Photo 1-15.
20	CLU	Farm complex (abandoned)	6th Line, north side, east of Hwy 400. Photo 1-14.
21	CLU	Farm complex	6983 5th Sideroad, south of 7th Line, west of Hwy 400. Photo 2-15.
22	CLU	Farm complex	5th Sideroad, north of 7th Line, west of Hwy 400. No photo.
23	BHF	Farmhouse (former)	3576 Innisfil Beach Road, north side, west of Highway 400. Photo 1-18.

Site #	Feature type	Feature category	Location
24	BHF	Farmhouse (former)	3465 9th Line, south side, east of Hwy 400. Photo 1-19.
25	CLU	Farm Complex	3460 10th Line, north side, east of hwy 400, Photo 1-20.
26	CLU	Farm complex	Salem Road (11th Line), north side, west of Hwy 400. No photo.
27	BHF	Farmhouse (former)	3576 9th Line, north side, west of Hwy 400. No photo.
28	CLU	Farm complex (former)	Molson Park, Bayview Ave., north of Molson Park Drive, Barrie, east of Hwy 400. Photo 1-21, 1-22.
29	BHF	Bridge	Dunlop Street bridge over Hwy 400, Barrie. Photo 1-23.
30	BHF	House	105 Edgehill Drive, Barrie. Photo 2-14.
31	BHF	Bridge	Anne Street bridge over Hwy 400, Barrie. No photo.
32	BHF	Bridge	Sunnidale Road bridge over Hwy 400, Barrie. No photo.
33	BHF	Bridge	Bayfield Street bridge over Hwy 400, Barrie. No photo.
34	BHF	Public	O.P.P. Building, Rose St., Barrie, east side of Hwy 400. Photo 1-24.
35	CLU	Farm complex	2621 Gore Road, Crown Hill. Photos 2-11, 2-12.
36	CLU	Church and cemetery	572 Hwy 93, Crown Hill. St. James Cemetery, est. 1852 and frame church. Photos 2-8, 2-9.
37	BHF	Residence	6 Napoleon Road, Crown Hill. Photo 2-13.
38	BHF	Church	605 Hwy 93, Crown Hill. Crown Hill United Church, 1880. Photos 2-2-6, 2-7.
39	CLU	Farm complex	600 Hwy 93, Crown Hill, Caldwell Farm. Photos 2-4, 2-5.
40	CLU	Farm complex	Hwy 93, Crown Hill, Chappell Farm. Photos 2-1, 2-2, 2-3.
41	CLU	Hamlet	Crown Hill, junction Hwy 400 & Hwy 93.


Midhurst
Little L.
Little Lake
Crown Hill
Shanty Bay
Oro Beach
Lakeview
Roeberta Park
Oro Lea Beach
Big Bay Pt.
Bear Point
Big Bay Point
Cedar Mount
Maplegrove
BARRIE
Kempenfelt
Peninsular Park
Glenhaven Beach
Ferndale
Grenfell
Tollendal
Bayshore Estates
Crescent Harbor
Painswick
Sandy Cove Acres
Sandy Cove
St. Pauls
Mooselanka Beach
Glenwood Beach
Leonards Beach
Bon Secours Beach
Holly
Stroud
Aldersea
Alcona
Cedar Harbour
Bramley
Simcoe Beach
Big Cedar Point
Barclay
Nantyr
Tent City
Innisfil Heights
Belle Air Beach
Bell Ewart
Roches Point
Thornton
INNISFIL
Killarney Beach
Lefroy
De Grassi Point
Churchill
Ballydown Beach
Bayview Beach
Bethesda
Shore Acres
Cook's Bay
Wynhurst Beach
Gilford
Elmhurst Beach
Egbert
Fennell
Glenwood Beach
Cookstown
Beverley Isles
Deerhurst
Coulson's Hill
Holland Marsh
SCANLON CREEK
Fox I.
Snake I.
CANADIAN NATIONAL

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COUNTY OF SIMCOE
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TOWN OF INNISFIL
CITY OF BARRIE
TOWNSHIP OF SPRINGWATER

G.W.P. 30-95-00

APPENDIX N

ELECTRICAL REPORT

Electrical Report
Highway 400 Preliminary Design

From 1.0 km South of Highway 89 Northerly 30.0 km To The Junction of Highway 11
Town of Innisfil, City of Barrie, County of Simcoe, County of Springwater
(G.W.P. 30-95-00)

Prepared For:

Ontario

Ministry of Transportation
Southwest Region
659 Exeter Road
London, Ontario
N6E 1L3

Prepared By:

Cole, Sherman & Associates Limited
75 Commerce Valley Drive East
Thornhill, Ontario
L3T 7N9

URS
Cole Sherman

September 2003
CN 299 00147.00

1.0 Introduction

This *Electrical Report* is a component of *Total Project Management (TPM) Preliminary Design Study Highway Hwy400, From 1.0 km South of Highway 89 Northerly 30.0 km To The Junction of Highway 11, Town of Innisfil, City of Barrie, County of Simcoe, and County of Springwater (G.W.P. 30-95-00).* This report includes a review of traffic signal and illumination installations through the study area. Based on the site inspection undertaken for this study, operational deficiencies are identified and potential improvements are recommended.

This report includes the following:

- A review of existing conditions in the study area to identify operational deficiencies, problem locations, and issues;
- Recommended improvements that will address identified deficiencies.

1.1 *Existing Electrical Equipment in Study Area*

Highway 400 is a King's Controlled Access Freeway serving as the primary North-South access for local, inter-regional, commuter, and recreational traffic for southern and northern Ontario. The Highway 400 study area is located approximately west of Lake Simcoe and southwest of Georgian Bay, crossing through Simcoe County, Springwater County, City of Barrie, and Town of Innisfil. The study area for Highway 400 is defined from Highway 400/11 interchange approximately 30 km southerly to the Highway 89/SR89 interchange.

Highway 400 mainline consists of six-lane cross-section with posted speed of 100 km/h within the study limits. Available access points along the highway are provided by interchanges which are located at:

- Highway 89/Simcoe Road 89
- Innisfil Beach Road
- Molson Park Drive
- Essa Road
- Dunlop Street
- Highway 26/Bayfield Street
- Duckworth Street
- Highway Hwy400 Extension/Highway 11

Highway 89/Simcoe Road 89 (SR 89) is an east-west oriented two lane King's Highway. Highway 89 is under the jurisdiction of MTO while SR 89 is under the jurisdiction of Simcoe County. The speed limit on Highway 89/SR 89 is posted 60 km/h. The N-E/W ramp provides access to both Highway 89 and a service centre located in the northwest quadrant of the interchange. Direct accesses to the service centre are provided via Highway 400 southbound off-ramp as well as from Highway 89. A commuter parking lot with approximately 150 spaces is currently located in the southeast quadrant of the interchange accessible via S-E/W and E/W-N ramps. The Highway 89/SR 89 approaches to the Highway 400 structure were recently reconstructed to improve sight distances over the crossing bridge.

Existing Electrical Equipment:

a) Traffic Signals: Highway 400 intersection ramp terminals at Highway 89/SR 89 were upgraded to signalized controls during the time of the reconstruction.
b) MTO Lighting: Partial illumination consisting of steel poles on frangible bases upgraded 2001 is installed at all ramp terminals, entrance to the commuter parking lot and was not affected by reconstruction.
c) Municipal Lighting: no
d) Underpass: 50mm rigid PVC conduit on south fascia

Innisfil Beach Road/Simcoe Road 21 (SR 21) is a two-lane road running east-west and crossing beneath the Highway 400 structure. Innisfil Beach Road/SR 21 is under the jurisdiction of Simcoe County and has a posted speed of 60 km/h. Highway 400 ramp ramp terminals at Innisfil Beach Road/SR 21 are currently Two-Way-Stop-Controlled (TWSC). A commuter parking lot with approximately 150 spaces is currently located in the southwest quadrant of the interchange accessible via Highway Hwy400 E/W – S and N - E ramps.

Existing Electrical Equipment:

a) Traffic Signals: The interchange is anticipated to be reconstructed in 2003 and the intersection ramp terminals will be signalized after the interchange upgrade.
b) MTO Lighting: Existing commuter parking lot is anticipated to be relocated to the northeast quadrant of the interchange after the construction.
c) Municipal Lighting: no:

Molson Park Drive is an east-west multi-lane roadway with six (6) through-lanes crossing under the Highway 400. Molson Park Drive is under the jurisdiction of City of Barrie and has a posted speed of 60 km/h. Reconstruction of the interchange was completed in August of 2000. There are major retail/commercial developments are within close proximity of the interchange.

Existing Electrical Equipment:

a) Traffic Signals: Highway 400 intersection ramp terminals at Molson Park Drive are signalized and maintained by the City of Barrie.
b) MTO Lighting: There is existing partial illumination at the W-E/N-B Hwy 400 and S-B Hwy 400/W-E ramp terminals consisting of 2-year-old steel poles base mounted with frangible bases.
c) Municipal Lighting: yes
d) Overpass: Underpass illumination including 50mm rigid PVC conduits.

Essa Road is a southwest-northeast multi-lane roadway with four (4) lanes crossing under the Highway 400 structure. Essa Road is under the jurisdiction of City of Barrie and has a posted speed of 50 km/h. There are currently no provisions for pedestrians to cross Highway 400 structure at Essa Road.

Existing Electrical Equipment:

a) Traffic Signals: Highway 400 intersection ramp terminals at Essa Road are signalized.
b) MTO Lighting: Illumination of Essa Road was installed in approximately 1975. Old sectional steel poles are installed in locations not meeting requirements of the present MTO safe pole policy. S-B Hwy 400/W-E ramp consists of old sectional steel direct buried poles.
c) Municipal Lighting: yes

Dunlop Street West is an east-west multi-lane roadway, which narrows to two (2) through-lanes across the Highway 400 structure. Dunlop Road is under the jurisdiction of City of Barrie and has a posted speed of 50 km/h. Most ramps intersecting Dunlop Street West are stop/yield-controlled merging and diverging ramps.

Existing Electrical Equipment:

a) Traffic Signals:West ramps at Cedar Point intersection is signalized.
b) MTO Lighting: Partial illumination at the N-B Hwy 400/W-E off ramp consists of old sectional steel poles, direct buried.
c) Municipal Lighting: no

Highway 26/Bayfield Street is a north-south multi-lane roadway with four (4) lanes across the Highway 400 structure. Highway 26 is under the jurisdiction of MTO while Bayfield Street is under the jurisdiction of City of Barrie. The speed limit on Highway 26/Bayfield Street is posted at 50 km/h. Rose Street is currently connected to N/S – N ramp and forms a minor street stop-controlled intersection.

Existing Electrical Equipment:

a) Traffic Signals: Both Highway 400 intersection ramp terminals are signalized;
b) MTO Lighting: Illumination of off-ramps consists of direct buried sectional steel poles.
c) Municipal Lighting: yes
d) Underpass: Lighting pole on south fascia installed over lane 3 of Hwy 400 traffic

Duckworth Street is a north-south roadway with four (4) lanes east of Highway 400 structure and two (2) lanes west of Highway 400 structure. Duckworth Street is under the jurisdiction of City of Barrie and has a posted speed of 50 km/h.

Existing Electrical Equipment:

a) Traffic Signals: The ramp terminal intersection east of Highway Hwy400 structure is newly signalized while intersection west of Highway 400 structure is stop sign controlled.
b) MTO Lighting: Partial illumination consists of new steel poles base mounted at the northbound Hwy 400 off ramp. There is no illumination for at the Hwy 400 off ramp.
c) Municipal Lighting: yes.
d) Overpass: 50mm rigid PVC lighting conduit and underpass illumination installed on south and north abutment walls.

Highway 400 Extension/Highway 11 interchange is a major merging/diverging junction. Highway 11 is under jurisdiction of MTO and has a posted speed of 100 km/h. Highway 400 and Highway 11 both have 4-lane cross-sections north of the merging/diverging junction.
Existing Electrical Equipment:

a) Lighting: The existing illumination consists of sectional steel directed buried poles on Hwy 400 southbound ramp merge with Hwy 11 southbound ramp.
b) Miscellaneous: There is an existing signage relaying travel information on the northbound approach. The signs will be left with existing illumination.

Service Station (Hwy 400 Northbound) There is existing illumination on the N-B Hwy 400 off-ramp to the service station and the N-B Hwy 400 on-ramp from the service station. The installation is sub-standard concrete consisting of sectional steel and aluminum poles direct buried. Also, there is old 240V underground service and wiring. Recommendation is to remove existing illumination and replace with MTO standard illumination. The Ministry's jurisdiction ends at the entrance and exit of the service station. Replacement of the service station illumination will be up to the owner.

Service Station (Hwy 400 Southbound) Existing Illumination consists of sectional steel direct buried poles. Recommendation for replacing these poles will depend on the future of the service station, which has been slated, for closure.

Churchill Side Road Underpass, Structure # 30-212, Sta. 14+180: 75mm steel galvanized conduit along south fascia.

Anne Street Underpass, Structure # 30-347, Sta. 11+820: 100mm conduit attached to the top of west abutment wall and NW wing wall.

St. Vincent Street Underpass, Structure # 30-394, Sta. 14+940: Two light poles for illumination of St. Vincent Street on south fascia, 75mm lighting duct installed in south sidewalk and 6 – 90 mm Bell conduits in north sidewalk.

Willow Creek Bridge, Structure # 30-139, Sta. 18+290: One light pole base on west barrier without pole, 75mm rigid PVC duct installed in barrier wall.

1.2 Design Criteria

The photometric design criteria are as per Design Manual including the latest memo about obtrusive lighting:

E_{ave}= 8.5 lux	average illumination
U_1= E_{ave}/E_{min}=3:1	uniformity
U_2=E_{max}/E_{min}=6:1	uniformity
E_o=1lux	obtrusive illumination at property line.

1.3 Data Collection

The following data was reviewed in this study:

a) Photo log.
b) Findings of site inspection: The minutes with findings of the site inspection are attached in the appendix A.
c) Existing documentation. The following contracts served as a reference for preparation of this report:
Innisfill Beach Road (Contract 2003-3006)
Molson Park Drive (Contract 98-65)
Hwy 400/Hwy11 Interchange (Contract 77-112)
Bayfield Street (Contract 2000-0074)
Hwy 89 (Contract 2000-061)
d) Preliminary design and traffic operation report of this study
e) Structural Evaluation Report - selected information for structures with electrical equipment included in this report.
f)

2. *Illumination*

2.1 Summary of Warrants

Highway 400 mainline including all interchanges were analyzed in accordance with the methodology outlined in the new Directives B-5 and B-6. A summary of illumination warrants for existing conditions, and midterm planning horizon are summarized in table below.

	2001	2011
Interchanges		
Hwy400&89	84.7	80.1
Hwy400&Bayfield Street	95.1	95.5
Hwy400&Duckworth Street	105.1	100.5
Hwy400&Dunlop Street	101.2	98.1
Hwy400&Essa Street	99.4	92.5
Hwy400&Innisfill Beach Road	81.7	69.8
Hwy400&Molson Park Road	73.6	82.1
Warranting Condition	75	75
Mainline		
Hwy400	111.8	105.3
Warranting Condition	80	80

Note: the highlighted fields do not satisfy warranting conditions. All associated forms for this analysis are in the Appendix B.

2.2 Economic Evaluation

Based on the results of this analysis two alternatives – conventional and high mast for full interchange illumination were investigated.

The interchange of Hwy 400 and Hwy 89 served as a typical sample for the life cycle cost analysis with the following results:

	Conventional	HML
Basic Information:		
Time Frame for Analysis	40	40
Inflation Rate	5%	5%
Discount Rate	5%	5%
Relevant Information ($x1,000)		
Initial Construction Cost	608	1255
Other Construction Cost	15	45
Detour Lighting Cost	0	0
Replacement cost "A" (lamps)	79	8
Replacement cost "B" (luminaires)	131	106
Replacement cost "C" (poles)	274	0
Annual Energy Cost	11	14
Annual Maintenance Cost	28	1
Annual Incident Cost	2	0
Salvage Value	49	50
Present Value Lifecycle Costs	3,191	2,104

All forms related to the Lifecycle Cost Analysis are in appendix D.

2.3 MTO Illumination

2.3.1 Illumination of Interchanges and Mainline
The detail design will provide full high mast illumination for all interchanges. In appendix C are all interchange layouts for this analysis and high mast lighting preliminary lighting layouts for the following interchanges

- Hwy 400 & Hwy 89
- Hwy 400 & Bayfield Street
- Hwy400&Duckworth Street
- Hwy400&Dunlop Street
- Hwy400&Essa Street
- Hwy400&Innisfill Beach Road
- Hwy400&Molson Park Road
- Fill in illumination of mainline from Molson Park Drive to Duckworth Street

2.3.2 Illumination of Crossroads
The MTO conventional illumination will be added between all ramp terminals for each interchange:

- Hwy 400 & Hwy 89, 700m
- Hwy 400 & Bayfield Street, 300m
- Hwy400&Duckworth Street, 450m
- Hwy400&Dunlop Street, 700m
- Hwy400&Essa Street, 550m

- Hwy400&Innisfill Beach Road including commuter parking lot
- Hwy400&Molson Park Road including commuter parking lot

2.3.3 Temporary Illumination
The redesign method for all interchanges will require some sections of cross road built in stages that consequently will require installation of temporary illumination. We estimate that total length of temporary illumination will be required for one third length of cross roads, approximately 900m.

2.4 Municipal Illumination
The extensive redesign of all interchanges brings changes to long sections of municipal roads beyond the existing ROW. Changed alignment will affect location of the existing illumination and traffic signals equipment in the following locations:

- Essa Road from Ardagh Road to N-E/W ramp terminal affected by construction of new W-S ramp, approx. length of 250m.
- South leg of the N-E/W ramp terminal intersection beyond W-S ramp, approx. length 100m.
- Molson Park Drive East & E/W – S ramp, a new right turn lane is added on Molson Park Drive for a length of approximately 125m.
- Molson Park Drive West from E/W – N ramp to Barrie View Drive , a new right turn lane is added on Molson Park Drive for a length of approximately 225m.
- Dunlop Street West from Anne Street to E/W-N terminal for a length of approx. 100m.
- Dunlop Street, west of E/W-S ramp terminal for a length of approx. 200m.
- Ann Street from Wellington Street to Edgehill Drive, length approx. 400m.
- Sunnidale Road, length approx. 250m.
- Cedar Pointe Drive, approx. length of 150m.
- Bayfield Street from Grove Street to E/W-N ramp terminal, approx. length 200m.
- Bayfield Street from Ferris Lane to N-E/W ramp terminal , approx. length 250m.
- Rose Street cul-de-sac.
- Coulter Street , approx. length 150m
- Cookstown Outlet Mall Road realignment, approx. length 200m.

3. Underpass Illumination

3.1 MTO Underpass Illumination
All underpass lighting will be installed according to standard MTO practice. Underpass lighting will be installed for the following structures:

- Hwy 400 & Hwy 89
- Hwy 400 & Bayfield Street
- Hwy400&Duckworth Street
- Hwy400&Dunlop Street
- Hwy400&Essa Street
- Hwy400&Innisfill Beach Road
- Hwy400&Molson Park Road

3.2 Municipal Underpass Illumination.

The municipal underpass illumination will be reinstated to the municipal standards for the following structures:

- Molson Park Road
- Duckworth Street

4. Power Supply

The standard distribution assemblies will be installed in selected location at each interchange to provide power for lighting and 60kVA reserve for the FTMS equipment. The power connection for the municipal lighting will be reinstated as per existing conditions during detail design.

5. Traffic Counting Loops

The new loops should be placed outside of the ramp acceleration/deceleration area, on tangent roadway, and outside of the guide rail locations. The exact location of the replacement loops will be determined during the detailed design stage.

Traffic counting loops are located at:

- 1.6 km north of Highway 89 (STA 11+600)
- 2.0 km north of Innisfil Beach Road (STA 21+630)
- 1.0 km north of Molson Park Drive (STA 26+280)
- 0.7 km north of Essa Road (STA 29+180)
- 1.3 km north of Dunlop (STA 12+650)
- 0.6 km north of Bayfield Street (STA 14+080)
- 0.5 km north of Duckworth Street (STA 16+675)

The loops north of Duckworth Street are situated within the Duckworth N-N/S ramp roadway. This existing loops location will affect the accuracy of mainline vehicle counts. It is recommended that the existing loops be removed and replaced further north of the interchange.

6. Traffic Signals

6.1 Permanent Traffic Signals

Permanent traffic signals will be installed at the following ramp terminals:

- Essa Road & N-E/W ramp terminal
- Essa Road & S-E/W ramp terminal
- Essa Road & E-N ramp terminal
- Molson Park Drive & E/W – N ramp terminal (one quadrant affected by ramp realignment)
- Molson Park Drive & N – E/W ramp terminal (two quadrants affected by ramp realignment)
- Hwy 89 & Realigned Reive Road
- Hwy 89 & S-E/W ramp terminal
- Hwy 89 & N-E/W ramp terminal
- Simcoe Road 21 & S-E/W ramp terminal
- Duckworth Street & S-E/W ramp terminal (one quadrant and two median islands modified)
- Duckworth Street & N-E/W ramp terminal
- Bayfield Street & S-E/W ramp terminal
- Bayfield Street & N-E/W ramp terminal
- Dunlop Street & Hart Drive/ E/W-N ramp terminal
- Dunlop Street & N-E/W ramp terminal /Cedar Pointe Drive

6.2 Temporary Traffic Signals

The temporary traffic signals will be installed at the following locations

- Essa Road & N-E/W ramp terminal (conversion of 3 leg ramp terminal to full 4 leg intersection)
- Essa Road & S-E/W ramp terminal (complete shift of existing ramp terminal east)
- Essa Road & E-N ramp terminal (two quadrants affected by ramp realignment)
- Molson Park Drive & E/W – N ramp terminal (one quadrant affected by ramp realignment)
- Molson Park Drive & N – E/W ramp terminal (two quadrants affected by ramp realignment)
- Hwy 89 & Realigned Reive Road
- Hwy 89 & S-E/W ramp terminal (complete shift of existing ramp terminal north)
- Hwy 89 & N-E/W ramp terminal (complete shift of existing ramp terminal north)
- Simcoe Road 21 & S-E/W ramp terminal (complete shift of existing ramp terminal east)
- Duckworth Street & S-E/W ramp terminal (one quadrant and two median islands modified)
- Duckworth Street & N-E/W ramp terminal (three quadrants and two median islands modified)
- Bayfield Street & S-E/W ramp terminal (complete shift of existing ramp terminal north) Detail design should consider two stages during construction
- Bayfield Street & N-E/W ramp terminal (complete shift of existing ramp terminal north) with two construction stages
- Dunlop Street & Hart Drive/ E/W-N ramp terminal with two construction stages
- Dunlop Street & N-E/W ramp terminal /Cedar Pointe Drive with two construction stages

6.3 Municipal Traffic Signals

- Anne Street & Wellington Street

7. *Recommendations*

The review of warrants for illumination, results of Life Cycle Cost Analysis, preliminary design and traffic operation report and preferred roadway design alternative show that the following equipment will be installed:

- MTO full illumination for interchange of Hwy 400 & Hwy 89, Hwy 400 & Bayfield Street, Hwy400 & Duckworth Street, Hwy400 & Dunlop Street, Hwy 400 & Essa Street, Hwy 400 & Innisfill Beach Road, Hwy 400 & Molson Park Road
- MTO fill - in illumination for mainline from Essa Road to Duckworth Street
- MTO illumination of crossroads as per section 2.3.2
- MTO temporary illumination as per section 2.3.3
- Municipal illumination as per section 2.4
- MTO underpass illumination as per section 3.1
- Municipal illumination as per section 3.2
- Traffic counting loops as per section 5
- MTO traffic signals at following locations as per section 6.1 and 6.2
- municipal traffic signals affected by construction as per section 6.3
- removals

8. *Construction Cost Estimate*

For detailed construction cost estimate see Appendix E

A. Hwy 400 Illumination	10,770
B. Municipal Illumination	415
C. Traffic Signals Permanent	1,530
D. Traffic signals Temporary	1,025
E. Municipal Traffic Signals	160
F. Traffic Counting Loops	140
G. Distribution Assemblies	245
Total	14,405

APPENDICIES:

A. Minutes of Meeting
B. Form 1, Freeway Interchange – Full Illumination - Year 2001, Hwy400&Hwy89
Form 1, Freeway Interchange – Full Illumination –Year 2011, Hwy400&Hwy89
Form 1, Freeway Interchange – Full Illumination –Year 2001, Hwy400&Bayfield Street
Form 1, Freeway Interchange – Full Illumination –Year 2011, Hwy400&Bayfield Street
Form 1, Freeway Interchange – Full Illumination –Year 2001, Hwy400&Duckworth Street
Form 1, Freeway Interchange – Full Illumination –Year 2011, Hwy400&Duckworth Street
Form 1, Freeway Interchange – Full Illumination –Year 2001, Hwy400&Dunlop Street
Form 1, Freeway Interchange – Full Illumination –Year 2011, Hwy400&Dunlop Street
Form 1, Freeway Interchange – Full Illumination –Year 2001, Hwy400&Essa Street
Form 1, Freeway Interchange – Full Illumination –Year 2011, Hwy400&Essa Street
Form 1, Freeway Interchange – Full Illumination –Year 2001, Hwy400&Innisfill Beach Road
Form 1, Freeway Interchange – Full Illumination –Year 2011, Hwy400&Innisfill Beach Road
Form 1, Freeway Interchange – Full Illumination –Year 2001, Hwy400&Molson Park Road
Form 1, Freeway Interchange – Full Illumination –Year 2011, Hwy400&Molson Park Road
Form 2, Freeway – Continuous Illumination – Year 2001, Hwy 400
Form 2, Freeway – Continuous Illumination – Year 2011, Hwy 400
C. Lighting Layouts
Hwy400&89, High Mast Lighting Layout
Hwy400&Bayfield Street, High Mast Lighting Layout
Hwy400&Duckworth Street, High Mast Lighting Layout
Hwy400&Dunlop Street, High Mast Lighting Layout
Hwy400&Essa Street, High Mast Lighting Layout
Hwy400&Innisfill Beach Road, High Mast Lighting Layout
Hwy400&Molson Park Road, High Mast Lighting Layout
Hwy400&89, Conventional Lighting Layout
D. Life Cycle Cost Analysis, Conventional Lighting, Calculation Form
Life Cycle Cost Analysis, High Mast Lighting, Calculation Form
Life Cycle Cost Analysis, Conventional Lighting, Analysis Results
Life Cycle Cost Analysis, High Mast Lighting, Analysis Results
E. Construction Cost Estimate

APPENDIX A

URS COLE, SHERMAN
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MINUTES OF MEETING

PROJECT NAME:	Highway 400-PDR GWP 30-95-00	**MEETING No.**	1
PROJECT No.	CN29900147.00	**DATE:**	September 18, 2000
LOCATION:	Innisfil, Ontario	**TIME:**	10:00am
PRESENT:	Mr. B.T. Johnston – Electrical Co-ordinator		
	Thomas Christensen – Electrical Project Manager, Planning and Design Section		
	Norm W.J. Kelly – Traffic Analyst, Regional Traffic Section		
	Robert J. Henderson – Traffic Operations Analyst		
	Marcello Fossella – Electrical Design, URS Cole, Sherman		
PURPOSE:	**Review and recommendation of interchange illumination and traffic signals**		

Items Description

HIGHWAY 89/SIMCOE COUNTY ROAD 89

- There is existing partial illumination of the all ramp terminals. They consist of sectional steel poles, direct buried.
- Currently, the Highway 89 ramps and overpass are being reconstructed. Existing illumination after the reconstruction will remain.
- It is recommended that existing partial illumination be replaced with MTO standard partial illumination.
- In 2000, new traffic signals at both the S-B Hwy 400/W-E and N-B Hwy 400/W-E off-ramps will be installed.

SERVICE STATION (Hwy 400 Northbound)

- There is existing illumination on the N-B Hwy 400 off-ramp to the service station and the N-B Hwy 400 on-ramp from the service station. They consist of sub-standard concrete, sectional steel direct buried and aluminum poles. Also, there is old 220V underground service and wiring..
- Recommendation is to remove existing illumination and replace with MTO standard illumination.
- The Ministry's jurisdiction ends at the entrance and exit of the service station. Replacement of the service station illumination will be up to the owner.

SERVICE STATION (Hwy 400 Southbound)

- Existing Illumination consists of sectional steel direct buried poles.

PLEASE NOTE: If your records of this meeting do not agree with this document, or if there are any omissions, please advise the writer at once, otherwise the contents of this document shall be assumed accurate and correct.

Items Description

- Recommendation for replacing these poles will depend on the future of the service station, which has been slated, for closure.

INNISFIL BEACH ROAD (Simcoe County Road 21)

- It is recommended that partial illumination be added to the Innisfil Beach Road interchange to illuminate the bullnose.
- The existing commuter parking lot is to be reconstructed in a future contract.
- In 2001, there will be new traffic signals at the N-B Hwy 400/E-W and S/B Hwy 400/E-W off ramps.

MOLSON PARK ROAD (City of Barrie)

- There is existing partial illumination at the W-E/N-B Hwy 400 and S-B Hwy 400/W-E ramp terminals. They consist of 2-year-old steel poles with frangible bases.
- The traffic signals at the southbound and southbound terminals were installed 2 years ago.
- It is not recommended to replace the traffic signals or illumination.
- All traffic signals at ramp terminals within the City of Barrie are own by the Ministry of Transportation and maintained by the City of Barrie.

ESSA ROAD (City of Barrie)

- Illumination of Essa Road was installed in approximately 1975. Old sectional steel poles that are sub-standard to present MTO practice. E.g. offset of poles.
- Traffic signals at the N-B Hwy 400/W-E ramp were installed 2 years ago when Essa Road was widened. There is no recommendation to have these traffic signals replaced.
- Fairview Drive and E-W/N-B Hwy 400 on-ramp traffic signals were also installed 2 years ago. This intersection is not classified as a ramp terminal and is not considered as MTO jurisdiction.
- S-B Hwy 400/W-E ramp consists of old sectional steel direct buried poles. It is recommended to replace these poles with MTO standard poles.

DUNLOP STREET-Formerly Hwy 90 (City of Barrie)

- At the N-B Hwy 400/W-E off ramp, partial illumination consists of old sectional steel poles, direct buried.
- Recommendation is to replace existing partial illumination with MTO standard illumination.
- Power supply cabinets are recommended to be located along side of Highway 400 for MTO maintenance crew to have better access to them, opposed to the present condition of being located on the ramp terminals adjacent to municipal property.
- It is recommended to keep Highway 400 illumination separate from illumination that is not on the highway's Right of way. E.g., adjacent municipal roads.

Items Description

BAYFIELD STREET -Hwy 26 (City of Barrie)

- Illumination of N-B Hwy 400/N-S Bayfield Street off-ramp consists of direct buried sectional steel poles.
- The recommendation is to replace existing partial illumination with new MTO standard illumination.
- There are existing traffic signals at N-B Hwy 400 off ramp/ N-S Bayfield Street. It is recommended to replace the these traffic signals.

DUCKWORH ROAD (City of Barrie)

- There are new steel poles base mounted with 250W HPS luminaires at the northbound Hwy 400 off ramp.
- New traffic signals were installed at the northbound Hwy 400 off ramp.
- Southbound Hwy 400 off ramp has no present illumination and traffic signals.
- It is recommended to install partial illumination and traffic signals at the southbound Hwy 400 off ramp.
- It is recommended to review existing underpass lighting.

HIGHWAY 11 & 400 JUNCTION

- On the northbound approach, there is existing signage relaying travel information. The signs will be left with existing illumination. It should be determined if these signs will be replaced.
- On Hwy 400 Southbound ramp merge with Hwy 11 southbound ramp the existing Illumination consists of sectional steel directed buried poles.
- Recommendation may include removal of conventional lighting and replaced with highmast lighting.

Submitted by: Marcello Fossella

Distribution: Mr. B.T. Johnston – Electrical Co-ordinator

Thomas Christensen – Electrical Project Manager, Planning and Design Section

Norm W.J. Kelly – Traffic Analyst, Regional Traffic Section

Robert J. Henderson – Traffic Operations Analyst

Stefan Dzamba – URS Cole, Sherman

APPENDIX B

FORM 1 - Year 2001

FREEWAY INTERCHANGE - FULL ILLUMINATION

Highway: Interchange 400 & Highway 89 W.P. No.: G.W.P. 30-95-00

Limits: from: 1 km South of Highway 89 to: Junction at Highway 11 Name: Date: Mar-02

CLASSIFICATION FACTOR	RATING (I) 1	2	3	4	5	UNLIT WEIGHT (A)	LIGHTED WEIGHT (B)	DIFF. (A-B)	SCORE [RATINGx (A-B)]
Geometric Factors									
Ramp Types	Directional	Diamond	Cloverleaf Buttonhook	Parclo	Multilevel (iv)Trumpet	2.0	1.0	1.0	2.0
Cross-road Channelization (Raised Islands)	none		continuous		at interchange intersection	2.0	1.0	1.0	1.0
Service Roads	none		one-way		two-way	1.5	1.0	0.5	2.5
Freeway Lane Widths (m)	>3.75	3.75	3.66	3.5	<3.50	3.0	2.5	0.5	1.0
Freeway Median Widths (m)	>15 or barrier		10-15		<10	1.0	0.5	0.5	2.5
No of Freeway Lanes	4		6		8	1.0	0.8	0.2	0.6
Main Line Curves (m) deg	>1800 1	1,800-850 1-2	849-600 2.1-3	599-450 3.1-4	<450 >4	13.0	5.0	8.0	8.0
Grades (vertical)	>3	3.-3.9	4-4.9	506.9	>7	3.2	2.8	0.4	0.4
Sight Distance Cross Road Intersection (m)	>305	216-305	151-215	120-150	<120	2.0	1.8	0.2	0.2
						Geometric Total			18.2
Operational Factors									
Level of Service (ii) (any dark hour)	A	B	C	D	E	6.0	1.0	5.0	25.0
						Operational Total			25.0
Environmental Factors									
Development	none	1 quad	2 quad	3 quad	4 quad	2.0	0.3	1.7	8.5
Cross-road Approach Illumination	none		partial		continuous	3.0	2.0	1.0	3.0
Freeway Illumination	none		adjacent interchanges		continuous	5.0	3.0	2.0	6.0
						Environmental Total			17.5
Accidents									
% of Night-to Total Accidents (3 yr. Avg.)(iii)	<20%	20-30%	31-40%	41-50%	>50%	10.0	2.0	8.0	24.0
						Accidents Total			24.0
Benefit Cost Ratio B/C									

GEOMETRIC TOTAL	18.2	points
OPERATIONAL TOTAL	25.0	points
ENVIROMENTAL TOTAL	17.5	points
ACCIDENTS TOTAL	24.0	points
SUM	84.7	points
FULL ILLUMINATION WARRANTING CONDITION	75	points

i. A rating of between 1 and 5 shall be assingned for each factor in the FORM depending on the conditions that are encountered by motorists on the roadway. The higher the rating, the more critical the need for illumination with regard to that particular factor.

ii. Use LOS methodology approved by the MTO

iii For night- to- total accident ratio, accidents during darkness are used (including dask/dawn).

iv. The number of points for the warranting condition is based on 50% of the total points attainable, if all factors were rated 5.

Note: Worst case scenarions should be considered when assigning the ratings. For example, a section could have rush hour volumes during the hours of darkness in wintertime.

FORM 1 - Year 2011

FREEWAY INTERCHANGE - FULL ILLUMINATION

Highway: 400&89 W.P. No.: G.W.P. 30-95-00 PLANNING STUDY

Limits: from: 1km South of Hwy89 to: Junction at Hwy 11 Name: Date: Mar-02

CLASSIFICATION FACTOR	RATING (I) 1	2	3	4	5	UNLIT WEIGHT (A)	LIGHTED WEIGHT (B)	DIFF. (A-B)	SCORE [RATINGx (A-B)]
Geometric Factors									
Ramp Types	Directional	Diamond	Cloverleaf Buttonhook	Parclo A4	Multilevel (iv)Trumpet	2.0	1.0	1.0	4.0
Cross-road Channelization (Raised Islands)	none		continuous		at interchange intersection	2.0	1.0	1.0	1.0
Service Roads	none		one-way		two-way	1.5	1.0	0.5	2.5
Freeway Lane Widths (m)	>3.75	3.75	3.66	3.5	<3.50	3.0	2.5	0.5	1.0
Freeway Median Widths (m)	>15 or barrier		10-15		<10	1.0	0.5	0.5	0.5
No of Freeway Lanes	4		6		8 10	1.0	0.8	0.2	1.0
Main Line Curves (m) deg	>1800 1	1,800-850 1-2	849-600 2.1-3	599-450 3.1-4	<450 >4	13.0	5.0	8.0	8.0
Grades (vertical)	>3	3.-3.9	4-4.9	506.9	>7	3.2	2.8	0.4	0.4
Sight Distance Cross Road Intersection (m)	>305	216-305	151-215	120-150	<120	2.0	1.8	0.2	0.2
						Geometric Total			18.6
Operational Factors									
Level of Service (ii) (any dark hour)	A	B	C	D	E	6.0	1.0	5.0	20.0
						Operational Total			20.0
Environmental Factors									
Development	none	1 quad	2 quad	3 quad	4 quad	2.0	0.3	1.7	8.5
Cross-road Approach Illumination	none		partial		continuous	3.0	2.0	1.0	3.0
Freeway Illumination	none		adjacent interchanges		continuous	5.0	3.0	2.0	6.0
						Environmental Total			17.5
Accidents									
% of Night-to Total Accidents (3 yr. Avg.)(iii)	<20%	20-30%	31-40%	41-50%	>50%	10.0	2.0	8.0	24.0
						Accidents Total			24.0
Benefit Cost Ratio B/C									

GEOMETRIC TOTAL	18.6	points
OPERATIONAL TOTAL	20.0	points
ENVIROMENTAL TOTAL	17.5	points
ACCIDENTS TOTAL	24.0	points
SUM	80.1	points
FULL ILLUMINATION WARRANTING CONDITION	75	points

i. A rating of between 1 and 5 shall be assingned for each factor in the FORM depending on the conditions that are encountered by motorists on the roadway. The higher the rating, the more critical the need for illumination with regard to that particular factor.

ii. Use LOS methodology approved by the MTO

iii For night- to- total accident ratio, accidents during darkness are used (including dask/dawn).

iv. The number of points for the warranting condition is based on 50% of the total points attainable, if all factors were rated 5.

Note: Worst case scenarions should be considered when assigning the ratings. For example, a section could have rush hour volumes during the hours of darkness in wintertime.

FORM 1 - Year 2001

FREEWAY INTERCHANGE - FULL ILLUMINATION

Highway: Interchange 400 & Bayfield Street W.P. No.: G.W.P. 30-95-00

Limits: from: 1 km South of Highway 89 to: Junction at Highway 11 Name: Date: Mar-02

CLASSIFICATION FACTOR	RATING (I) 1	2	3	4	5	UNLIT WEIGHT (A)	LIGHTED WEIGHT (B)	DIFF. (A-B)	SCORE [RATINGx (A-B)]
Geometric Factors									
Ramp Types	Directional	Diamond	Cloverleaf Buttonhook	Parclo	Multilevel (iv)Trumpet	2.0	1.0	1.0	2.0
Cross-road Channelization (Raised Islands)	none		continuous		at interchange intersection	2.0	1.0	1.0	1.0
Service Roads	none		one-way		two-way	1.5	1.0	0.5	2.5
Freeway Lane Widths (m)	>3.75	3.75	3.66	3.5	<3.50	3.0	2.5	0.5	1.0
Freeway Median Widths (m)	>15 or barrier		10-15		<10	1.0	0.5	0.5	2.5
No of Freeway Lanes	4		6		8	1.0	0.8	0.2	0.6
Main Line Curves (m) deg	>1800 1	1,800-850 1-2	849-600 2.1-3	599-450 3.1-4	<450 >4	13.0	5.0	8.0	16.0
Grades (vertical)	>3	3.-3.9	4-4.9	506.9	>7	3.2	2.8	0.4	0.8
Sight Distance Cross Road Intersection (m)	>305	216-305	151-215	120-150	<120	2.0	1.8	0.2	0.2
						Geometric Total			26.6
Operational Factors									
Level of Service (ii) (any dark hour)	A	B	C	D	E	6.0	1.0	5.0	25.0
						Operational Total			25.0
Environmental Factors									
Development	none	1 quad	2 quad	3 quad	4 quad	2.0	0.3	1.7	8.5
Cross-road Approach Illumination	none		partial		continuous	3.0	2.0	1.0	5.0
Freeway Illumination	none		adjacent interchanges		continuous	5.0	3.0	2.0	6.0
						Environmental Total			19.5
Accidents									
% of Night-to Total Accidents (3 yr. Avg.)(iii)	<20%	20-30%	31-40%	41-50%	>50%	10.0	2.0	8.0	24.0
						Accidents Total			24.0
Benefit Cost Ratio B/C									

GEOMETRIC TOTAL	26.6	points
OPERATIONAL TOTAL	25.0	points
ENVIROMENTAL TOTAL	19.5	points
ACCIDENTS TOTAL	24.0	points
SUM	95.1	points

FULL ILLUMINATION WARRANTING CONDITION 75 points

I. A rating of between 1 and 5 shall be assingned for each factor in the FORM depending on the conditions that are encountered by motorists on the roadway. The higher the rating, the more critical the need for illumination with regard to that particular factor.

ii. Use LOS methodology approved by the MTO

iii For night- to- total accident ratio, accidents during darkness are used (including dask/dawn).

iv. The number of points for the warranting condition is based on 50% of the total points attainable, if all factors were rated 5.

Note: Worst case scenarios should be considered when assigning the ratings. For example, a section could have rush hour volumes during the hours of darkness in wintertime.

FORM 1 - Year 2011

FREEWAY INTERCHANGE - FULL ILLUMINATION

Highway: Interchange 400 & Bayfield Street W.P. No.: G.W.P. 30-95-00

Limits: from: 1 km South of Highway 89 to: Junction at Highway 11 Name: Date: Mar-02

CLASSIFICATION FACTOR	RATING (I) 1	2	3	4	5	UNLIT WEIGHT (A)	LIGHTED WEIGHT (B)	DIFF. (A-B)	SCORE [RATINGx (A-B)]
Geometric Factors									
Ramp Types	Directional	Diamond	Cloverleaf Buttonhook	Parclo B	Multilevel (iv)Trumpet	2.0	1.0	1.0	2.0
Cross-road Channelization (Raised Islands)	none		continuous		at interchange intersection	2.0	1.0	1.0	3.0
Service Roads	none		one-way		two-way	1.5	1.0	0.5	2.5
Freeway Lane Widths (m)	>3.75	3.75	3.66	3.5	<3.50	3.0	2.5	0.5	1.0
Freeway Median Widths (m)	>15 or barrier		10-15		<10	1.0	0.5	0.5	0.5
No of Freeway Lanes	4		6		8 10	1.0	0.8	0.2	1.0
Main Line Curves (m) deg	>1800 1	1,800-850 1-2	849-600 2.1-3	599-450 3.1-4	<450 >4	13.0	5.0	8.0	16.0
Grades (vertical)	>3	3.-3.9	4-4.9	506.9	>7	3.2	2.8	0.4	0.8
Sight Distance Cross Road Intersection (m)	>305	216-305	151-215	120-150	<120	2.0	1.8	0.2	0.2
						Geometric Total			27.0
Operational Factors									
Level of Service (ii) (any dark hour)	A	B	C	D	E	6.0	1.0	5.0	25.0
						Operational Total			25.0
Environmental Factors									
Development	none	1 quad	2 quad	3 quad	4 quad	2.0	0.3	1.7	8.5
Cross-road Approach Illumination	none		partial		continuous	3.0	2.0	1.0	5.0
Freeway Illumination	none		adjacent interchanges		continuous	5.0	3.0	2.0	6.0
						Environmental Total			19.5
Accidents									
% of Night-to Total Accidents (3 yr. Avg.)(iii)	<20%	20-30%	31-40%	41-50%	>50%	10.0	2.0	8.0	24.0
						Accidents Total			24.0
Benefit Cost Ratio B/C									

GEOMETRIC TOTAL	27.0	points
OPERATIONAL TOTAL	25.0	points
ENVIROMENTAL TOTAL	19.5	points
ACCIDENTS TOTAL	24.0	points
SUM	95.5	points

FULL ILLUMINATION WARRANTING CONDITION 75 points

I. A rating of between 1 and 5 shall be assingned for each factor in the FORM depending on the conditions that are encountered by motorists on the roadway. The higher the rating, the more critical the need for illumination with regard to that particular factor.

ii. Use LOS methodology approved by the MTO

iii For night- to- total accident ratio, accidents during darkness are used (including dask/dawn).

iv. The number of points for the warranting condition is based on 50% of the total points attainable, if all factors were rated 5.

Note: Worst case scenarios should be considered when assigning the ratings. For example, a section could have rush hour volumes during the hours of darkness in wintertime.

FORM 1 - Year 2001

FREEWAY INTERCHANGE - FULL ILLUMINATION

Highway: Interchange 400 & Duckworth Street W.P. No.: G.W.P. 30-95-00

Limits: from: 1 km South of Highway 89 to: Junction at Highway 11 Name: Date: Mar-02

CLASSIFICATION FACTOR	RATING (I) 1	2	3	4	5	UNLIT WEIGHT (A)	LIGHTED WEIGHT (B)	DIFF. (A-B)	SCORE [RATINGx (A-B)]
Geometric Factors									
Ramp Types	Directional	Diamond	Cloverleaf Buttonhook	Parclo	Multilevel (iv)Trumpet	2.0	1.0	1.0	4.0
Cross-road Channelization (Raised Islands)	none		continuous		at interchange intersection	2.0	1.0	1.0	1.0
Service Roads	none		one-way		two-way	1.5	1.0	0.5	2.5
Freeway Lane Widths (m)	>3.75	3.75	3.66	3.5	<3.50	3.0	2.5	0.5	1.0
Freeway Median Widths (m)	>15 or barrier		10-15		<10	1.0	0.5	0.5	2.5
No of Freeway Lanes	4		6		8	1.0	0.8	0.2	0.6
Main Line Curves (m) deg	>1800 1	1,800-850 1-2	849-600 2.1-3	599-450 3.1-4	<450 >4	13.0	5.0	8.0	24.0
Grades (vertical)	>3	3.-3.9	4-4.9	506.9	>7	3.2	2.8	0.4	0.4
Sight Distance Cross Road Intersection (m)	>305	216-305	151-215	120-150	<120	2.0	1.8	0.2	0.6
						Geometric Total			36.6
Operational Factors									
Level of Service (ii) (any dark hour)	A	B	C	D	E	6.0	1.0	5.0	25.0
						Operational Total			25.0
Environmental Factors									
Development	none	1 quad	2 quad	3 quad	4 quad	2.0	0.3	1.7	8.5
Cross-road Approach Illumination	none		partial		continuous	3.0	2.0	1.0	5.0
Freeway Illumination	none		adjacent interchanges		continuous	5.0	3.0	2.0	6.0
						Environmental Total			19.5
Accidents									
% of Night-to Total Accidents (3 yr. Avg.)(iii)	<20%	20-30%	31-40%	41-50%	>50%	10.0	2.0	8.0	24.0
						Accidents Total			24.0
Benefit Cost Ratio B/C									

GEOMETRIC TOTAL	36.6	points
OPERATIONAL TOTAL	25.0	points
ENVIROMENTAL TOTAL	19.5	points
ACCIDENTS TOTAL	24.0	points
SUM	105.1	points
FULL ILLUMINATION WARRANTING CONDITION	75	points

i. A rating of between 1 and 5 shall be assingned for each factor in the FORM depending on the conditions that are encountered by motorists on the roadway. The higher the rating, the more critical the need for illumination with regard to that particular factor.

ii. Use LOS methodology approved by the MTO

iii For night- to- total accident ratio, accidents during darkness are used (including dask/dawn).

iv. The number of points for the warranting condition is based on 50% of the total points attainable, if all factors were rated 5.

Note: Worst case scenarions should be considered when assigning the ratings. For example, a section could have rush hour volumes during the hours of darkness in wintertime.

FORM 1 - Year 2011

FREEWAY INTERCHANGE - FULL ILLUMINATION

Highway: Interchange 400 & Duckworth Street W.P. No.: G.W.P. 30-95-00

Limits: from: 1 km South of Highway 89 to: Junction at Highway 11 Name: Date: Mar-02

CLASSIFICATION FACTOR	RATING (I) 1	2	3	4	5	UNLIT WEIGHT (A)	LIGHTED WEIGHT (B)	DIFF. (A-B)	SCORE [RATINGx (A-B)]
Geometric Factors									
Ramp Types	Directional	Diamond	Cloverleaf Buttonhook	Parclo	Multilevel (iv)Trumpet	2.0	1.0	1.0	4.0
Cross-road Channelization (Raised Islands)	none		continuous		at interchange intersection	2.0	1.0	1.0	3.0
Service Roads	none		one-way		two-way	1.5	1.0	0.5	2.5
Freeway Lane Widths (m)	>3.75	3.75	3.66	3.5	<3.50	3.0	2.5	0.5	1.0
Freeway Median Widths (m)	>15 or barrier		10-15		<10	1.0	0.5	0.5	0.5
No of Freeway Lanes	4		6		8	1.0	0.8	0.2	1.0
Main Line Curves (m) deg	>1800 1	1,800-850 1-2	849-600 2.1-3	599-450 3.1-4	<450 >4	13.0	5.0	8.0	24.0
Grades (vertical)	>3	3.-3.9	4-4.9	506.9	>7	3.2	2.8	0.4	0.4
Sight Distance Cross Road Intersection (m)	>305	216-305	151-215	120-150	<120	2.0	1.8	0.2	0.6
						Geometric Total			37.0
Operational Factors									
Level of Service (ii) (any dark hour)	A	B	C	D	E	6.0	1.0	5.0	20.0
						Operational Total			20.0
Environmental Factors									
Development	none	1 quad	2 quad	3 quad	4 quad	2.0	0.3	1.7	8.5
Cross-road Approach Illumination	none		partial		continuous	3.0	2.0	1.0	5.0
Freeway Illumination	none		adjacent interchanges		continuous	5.0	3.0	2.0	6.0
						Environmental Total			19.5
Accidents									
% of Night-to Total Accidents (3 yr. Avg.)(iii)	<20%	20-30%	31-40%	41-50%	>50%	10.0	2.0	8.0	24.0
						Accidents Total			24.0
Benefit Cost Ratio B/C									

GEOMETRIC TOTAL	37.0	points
OPERATIONAL TOTAL	20.0	points
ENVIROMENTAL TOTAL	19.5	points
ACCIDENTS TOTAL	24.0	points
SUM	100.5	points
FULL ILLUMINATION WARRANTING CONDITION	75	points

i. A rating of between 1 and 5 shall be assingned for each factor in the FORM depending on the conditions that are encountered by motorists on the roadway. The higher the rating, the more critical the need for illumination with regard to that particular factor.

ii. Use LOS methodology approved by the MTO

iii For night- to- total accident ratio, accidents during darkness are used (including dask/dawn).

iv. The number of points for the warranting condition is based on 50% of the total points attainable, if all factors were rated 5.

Note: Worst case scenarions should be considered when assigning the ratings. For example, a section could have rush hour volumes during the hours of darkness in wintertime.

FORM 1 - Year 2001

FREEWAY INTERCHANGE - FULL ILLUMINATION

Highway: Interchange 400 & Dunlop Street W.P. No.: G.W.P. 30-95-00

Limits: from: 1 km South of Highway 89 to: Junction at Highway 11 Name: Date: Mar-02

CLASSIFICATION FACTOR	RATING (I) 1	2	3	4	5	UNLIT WEIGHT (A)	LIGHTED WEIGHT (B)	DIFF. (A-B)	SCORE [RATINGx (A-B)]
Geometric Factors									
Ramp Types	Directional	Diamond	Cloverleaf Buttonhook	Parclo	Multilevel (iv)Trumpet	2.0	1.0	1.0	4.0
Cross-road Channelization (Raised Islands)	none		continuous		at interchange intersection	2.0	1.0	1.0	1.0
Service Roads	none		one-way		two-way	1.5	1.0	0.5	1.0
Freeway Lane Widths (m)	>3.75	3.75	3.66	3.5	<3.50	3.0	2.5	0.5	1.0
Freeway Median Widths (m)	>15 or barrier		10-15		<10	1.0	0.5	0.5	2.5
No of Freeway Lanes	4		6		8	1.0	0.8	0.2	0.6
Main Line Curves (m) deg	>1800 1	1,800-850 1-2	849-600 2.1-3	599-450 3.1-4	<450 >4	13.0	5.0	8.0	24.0
Grades (vertical)	>3	3.-3.9	4-4.9	506.9	>7	3.2	2.8	0.4	0.4
Sight Distance Cross Road Intersection (m)	>305	216-305	151-215	120-150	<120	2.0	1.8	0.2	0.2
						Geometric Total			34.7
Operational Factors									
Level of Service (ii) (any dark hour)	A	B	C	D	E	6.0	1.0	5.0	25.0
						Operational Total			25.0
Environmental Factors									
Development	none	1 quad	2 quad	3 quad	4 quad	2.0	0.3	1.7	8.5
Cross-road Approach Illumination	none		partial		continuous	3.0	2.0	1.0	3.0
Freeway Illumination	none		adjacent interchanges		continuous	5.0	3.0	2.0	6.0
						Environmental Total			17.5
Accidents									
% of Night-to Total Accidents (3 yr. Avg.)(iii)	<20%	20-30%	31-40%	41-50%	>50%	10.0	2.0	8.0	24.0
						Accidents Total			24.0
Benefit Cost Ratio B/C									

GEOMETRIC TOTAL	34.7	points
OPERATIONAL TOTAL	25.0	points
ENVIROMENTAL TOTAL	17.5	points
ACCIDENTS TOTAL	24.0	points
SUM	101.2	points
FULL ILLUMINATION WARRANTING CONDITION	75	points

I. A rating of between 1 and 5 shall be assingned for each factor in the FORM depending on the conditions that are encountered by motorists on the roadway. The higher the rating, the more critical the need for illumination with regard to that particular factor.

ii. Use LOS methodology approved by the MTO

iii For night- to- total accident ratio, accidents during darkness are used (including dask/dawn).

iv. The number of points for the warranting condition is based on 50% of the total points attainable, if all factors were rated 5.

Note: Worst case scenarions should be considered when assigning the ratings. For example, a section could have rush hour volumes during the hours of darkness in wintertime.

FORM 1 - Year 2011

FREEWAY INTERCHANGE - FULL ILLUMINATION

Highway: Interchange 400 & Dunlop Street W.P. No.: G.W.P. 30-95-00

Limits: from: 1 km South of Highway 89 to: Junction at Highway 11 Name: Date: Mar-02

CLASSIFICATION FACTOR	RATING (I) 1	2	3	4	5	UNLIT WEIGHT (A)	LIGHTED WEIGHT (B)	DIFF. (A-B)	SCORE [RATINGx (A-B)]
Geometric Factors									
Ramp Types	Directional	Diamond	Cloverleaf Buttonhook	Parclo B	Multilevel (iv)Trumpet	2.0	1.0	1.0	4.0
Cross-road Channelization (Raised Islands)	none		continuous		at interchange intersection	2.0	1.0	1.0	3.0
Service Roads	none		one-way		two-way	1.5	1.0	0.5	2.5
Freeway Lane Widths (m)	>3.75	3.75	3.66	3.5	<3.50	3.0	2.5	0.5	1.0
Freeway Median Widths (m)	>15 or barrier		10-15		<10	1.0	0.5	0.5	0.5
No of Freeway Lanes	4		6		8 10	1.0	0.8	0.2	1.0
Main Line Curves (m) deg	>1800 1	1,800-850 1-2	849-600 2.1-3	599-450 3.1-4	<450 >4	13.0	5.0	8.0	24.0
Grades (vertical)	>3	3.-3.9	4-4.9	506.9	>7	3.2	2.8	0.4	0.4
Sight Distance Cross Road Intersection (m)	>305	216-305	151-215	120-150	<120	2.0	1.8	0.2	0.2
						Geometric Total			36.6
Operational Factors									
Level of Service (ii) (any dark hour)	A	B	C	D	E	6.0	1.0	5.0	20.0
						Operational Total			20.0
Environmental Factors									
Development	none	1 quad	2 quad	3 quad	4 quad	2.0	0.3	1.7	8.5
Cross-road Approach Illumination	none		partial		continuous	3.0	2.0	1.0	3.0
Freeway Illumination	none		adjacent interchanges		continuous	5.0	3.0	2.0	6.0
						Environmental Total			17.5
Accidents									
% of Night-to Total Accidents (3 yr. Avg.)(iii)	<20%	20-30%	31-40%	41-50%	>50%	10.0	2.0	8.0	24.0
						Accidents Total			24.0
Benefit Cost Ratio B/C									

GEOMETRIC TOTAL	36.6	points
OPERATIONAL TOTAL	20.0	points
ENVIROMENTAL TOTAL	17.5	points
ACCIDENTS TOTAL	24.0	points
SUM	98.1	points
FULL ILLUMINATION WARRANTING CONDITION	75	points

I. A rating of between 1 and 5 shall be assingned for each factor in the FORM depending on the conditions that are encountered by motorists on the roadway. The higher the rating, the more critical the need for illumination with regard to that particular factor.

ii. Use LOS methodology approved by the MTO

iii For night- to- total accident ratio, accidents during darkness are used (including dask/dawn).

iv. The number of points for the warranting condition is based on 50% of the total points attainable, if all factors were rated 5.

Note: Worst case scenarions should be considered when assigning the ratings. For example, a section could have rush hour volumes during the hours of darkness in wintertime.

FORM 1 - Year 2001

FREEWAY INTERCHANGE - FULL ILLUMINATION

Highway: Interchange 400 & Essa Road W.P. No.: G.W.P. 30-95-00

Limits: from: 1 km South of Highway 89 to: Junction at Highway 11 Name: Date: Mar-02

CLASSIFICATION FACTOR	RATING (I) 1	2	3	4	5	UNLIT WEIGHT (A)	LIGHTED WEIGHT (B)	DIFF. (A-B)	SCORE [RATINGx (A-B)]
Geometric Factors									
Ramp Types	Directional	Diamond	Cloverleaf Buttonhook	Parclo	Multilevel (iv)Trumpet	2.0	1.0	1.0	4.0
Cross-road Channelization (Raised Islands)	none		continuous		at interchange intersection	2.0	1.0	1.0	5.0
Service Roads	none		one-way		two-way	1.5	1.0	0.5	2.5
Freeway Lane Widths (m)	>3.75	3.75	3.66	3.5	<3.50	3.0	2.5	0.5	1.0
Freeway Median Widths (m)	>15 or barrier		10-15		<10	1.0	0.5	0.5	2.5
No of Freeway Lanes	4		6		8	1.0	0.8	0.2	0.6
Main Line Curves (m) deg	>1800 1	1,800-850 1-2	849-600 2.1-3	599-450 3.1-4	<450 >4	13.0	5.0	8.0	16.0
Grades (vertical)	>3	3.-3.9	4-4.9	506.9	>7	3.2	2.8	0.4	0.8
Sight Distance Cross Road Intersection (m)	>305	216-305	151-215	120-150	<120	2.0	1.8	0.2	0.2
						Geometric Total			32.6
Operational Factors									
Level of Service (ii) (any dark hour)	A	B	C	D	E	6.0	1.0	5.0	25.0
						Operational Total			25.0
Environmental Factors									
Development	none	1 quad	2 quad	3 quad	4 quad	2.0	0.3	1.7	6.8
Cross-road Approach Illumination	none		partial		continuous	3.0	2.0	1.0	5.0
Freeway Illumination	none		adjacent interchanges		continuous	5.0	3.0	2.0	6.0
						Environmental Total			17.8
Accidents									
% of Night-to Total Accidents (3 yr. Avg.)(iii)	<20%	20-30%	31-40%	41-50%	>50%	10.0	2.0	8.0	24.0
						Accidents Total			24.0
Benefit Cost Ratio B/C									

GEOMETRIC TOTAL	32.6	points
OPERATIONAL TOTAL	25.0	points
ENVIROMENTAL TOTAL	17.8	points
ACCIDENTS TOTAL	24.0	points
SUM	99.4	points

FULL ILLUMINATION WARRANTING CONDITION 75 points

I. A rating of between 1 and 5 shall be assingned for each factor in the FORM depending on the conditions that are encountered by motorists on the roadway. The higher the rating, the more critical the need for illumination with regard to that particular factor.

ii. Use LOS methodology approved by the MTO

iii For night- to- total accident ratio, accidents during darkness are used (including dask/dawn).

iv. The number of points for the warranting condition is based on 50% of the total points attainable, if all factors were rated 5.

Note: Worst case scenarios should be considered when assigning the ratings. For example, a section could have rush hour volumes during the hours of darkness in wintertime.

FORM 1 - Year 2011

FREEWAY INTERCHANGE - FULL ILLUMINATION

Highway: Interchange 400 & Essa Road W.P. No.: G.W.P. 30-95-00

Limits: from: 1 km South of Highway 89 to: Junction at Highway 11 Name: Date: Mar-02

CLASSIFICATION FACTOR	RATING (I) 1	2	3	4	5	UNLIT WEIGHT (A)	LIGHTED WEIGHT (B)	DIFF. (A-B)	SCORE [RATINGx (A-B)]
Geometric Factors									
Ramp Types	Directional	Diamond	Cloverleaf Buttonhook	Parclo A4	Multilevel (iv)Trumpet	2.0	1.0	1.0	4.0
Cross-road Channelization (Raised Islands)	none		continuous		at interchange intersection	2.0	1.0	1.0	3.0
Service Roads	none		one-way		two-way	1.5	1.0	0.5	2.5
Freeway Lane Widths (m)	>3.75	3.75	3.66	3.5	<3.50	3.0	2.5	0.5	1.0
Freeway Median Widths (m)	>15 or barrier		10-15		<10	1.0	0.5	0.5	0.5
No of Freeway Lanes	4		6		8 10	1.0	0.8	0.2	1.0
Main Line Curves (m) deg	>1800 1	1,800-850 1-2	849-600 2.1-3	599-450 3.1-4	<450 >4	13.0	5.0	8.0	16.0
Grades (vertical)	>3	3.-3.9	4-4.9	506.9	>7	3.2	2.8	0.4	0.8
Sight Distance Cross Road Intersection (m)	>305	216-305	151-215	120-150	<120	2.0	1.8	0.2	0.2
						Geometric Total			29.0
Operational Factors									
Level of Service (ii) (any dark hour)	A	B	C	D	E	6.0	1.0	5.0	20.0
						Operational Total			20.0
Environmental Factors									
Development	none	1 quad	2 quad	3 quad	4 quad	2.0	0.3	1.7	8.5
Cross-road Approach Illumination	none		partial		continuous	3.0	2.0	1.0	5.0
Freeway Illumination	none		adjacent interchanges		continuous	5.0	3.0	2.0	6.0
						Environmental Total			19.5
Accidents									
% of Night-to Total Accidents (3 yr. Avg.)(iii)	<20%	20-30%	31-40%	41-50%	>50%	10.0	2.0	8.0	24.0
						Accidents Total			24.0
Benefit Cost Ratio B/C									

GEOMETRIC TOTAL	29.0	points
OPERATIONAL TOTAL	20.0	points
ENVIROMENTAL TOTAL	19.5	points
ACCIDENTS TOTAL	24.0	points
SUM	92.5	points

FULL ILLUMINATION WARRANTING CONDITION 75 points

I. A rating of between 1 and 5 shall be assingned for each factor in the FORM depending on the conditions that are encountered by motorists on the roadway. The higher the rating, the more critical the need for illumination with regard to that particular factor.

ii. Use LOS methodology approved by the MTO

iii For night- to- total accident ratio, accidents during darkness are used (including dask/dawn).

iv. The number of points for the warranting condition is based on 50% of the total points attainable, if all factors were rated 5.

Note: Worst case scenarios should be considered when assigning the ratings. For example, a section could have rush hour volumes during the hours of darkness in wintertime.

FORM 1 - Year 2001

FREEWAY INTERCHANGE - FULL ILLUMINATION

Highway: Interchange 400 & Innisfil Beach Road W.P. No.: G.W.P. 30-95-00

Limits: from: 1 km South of Highway 89 to: Junction at Highway 11 Name: Date: Mar-02

CLASSIFICATION FACTOR	RATING (I) 1	2	3	4	5	UNLIT WEIGHT (A)	LIGHTED WEIGHT (B)	DIFF. (A-B)	SCORE [RATINGx (A-B)]
Geometric Factors									
Ramp Types	Directional	Diamond	Cloverleaf Buttonhook	Parclo	Multilevel (iv)Trumpet	2.0	1.0	1.0	4.0
Cross-road Channelization (Raised Islands)	none		continuous		at interchange intersection	2.0	1.0	1.0	1.0
Service Roads	none		one-way		two-way	1.5	1.0	0.5	2.5
Freeway Lane Widths (m)	>3.75	3.75	3.66	3.5	<3.50	3.0	2.5	0.5	1.0
Freeway Median Widths (m)	>15 or barrier		10-15		<10	1.0	0.5	0.5	2.5
No of Freeway Lanes	4		6		8	1.0	0.8	0.2	0.6
Main Line Curves (m) deg	>1800 1	1,800-850 1-2	849-600 2.1-3	599-450 3.1-4	<450 >4	13.0	5.0	8.0	8.0
Grades (vertical)	>3	3.-3.9	4-4.9	506.9	>7	3.2	2.8	0.4	0.8
Sight Distance Cross Road Intersection (m)	>305	216-305	151-215	120-150	<120	2.0	1.8	0.2	0.2
						Geometric Total			20.6
Operational Factors									
Level of Service (ii) (any dark hour)	A	B	C	D	E	6.0	1.0	5.0	25.0
						Operational Total			25.0
Environmental Factors									
Development	none	1 quad	2 quad	3 quad	4 quad	2.0	0.3	1.7	5.1
Cross-road Approach Illumination	none		partial		continuous	3.0	2.0	1.0	1.0
Freeway Illumination	none		adjacent interchanges		continuous	5.0	3.0	2.0	6.0
						Environmental Total			12.1
Accidents									
% of Night-to Total Accidents (3 yr. Avg.)(ii)	<20%	20-30%	31-40%	41-50%	>50%	10.0	2.0	8.0	24.0
						Accidents Total			24.0
Benefit Cost Ratio B/C									

GEOMETRIC TOTAL	20.6	points
OPERATIONAL TOTAL	25.0	points
ENVIROMENTAL TOTAL	12.1	points
ACCIDENTS TOTAL	24.0	points
SUM	81.7	points
FULL ILLUMINATION WARRANTING CONDITION	75	points

I. A rating of between 1 and 5 shall be assingned for each factor in the FORM depending on the conditions that are encountered by motorists on the roadway. The higher the rating, the more critical the need for illumination with regard to that particular factor.

ii. Use LOS methodology approved by the MTO

iii For night- to- total accident ratio, accidents during darkness are used (including dask/dawn).

iv. The number of points for the warranting condition is based on 50% of the total points attainable, if all factors were rated 5.

Note: Worst case scenarions should be considered when assigning the ratings. For example, a section could have rush hour volumes during the hours of darkness in wintertime.

FORM 1 - Year 2011

FREEWAY INTERCHANGE - FULL ILLUMINATION

Highway: Interchange 400 & Innisfil Beach Road W.P. No.: G.W.P. 30-95-00

Limits: from: 1 km South of Highway 89 to: Junction at Highway 11 Name: Date: Mar-02

CLASSIFICATION FACTOR	RATING (I) 1	2	3	4	5	UNLIT WEIGHT (A)	LIGHTED WEIGHT (B)	DIFF. (A-B)	SCORE [RATINGx (A-B)]
Geometric Factors									
Ramp Types	Directional	Diamond	Cloverleaf Buttonhook	Parclo A4	Multilevel (iv)Trumpet	2.0	1.0	1.0	4.0
Cross-road Channelization (Raised Islands)	none		continuous		at interchange intersection	2.0	1.0	1.0	1.0
Service Roads	none		one-way		two-way	1.5	1.0	0.5	2.5
Freeway Lane Widths (m)	>3.75	3.75	3.66	3.5	<3.50	3.0	2.5	0.5	1.0
Freeway Median Widths (m)	>15 or barrier		10-15		<10	1.0	0.5	0.5	0.5
No of Freeway Lanes	4		6		8 10	1.0	0.8	0.2	2.0
Main Line Curves (m) deg	>1800 1	1,800-850 1-2	849-600 2.1-3	599-450 3.1-4	<450 >4	13.0	5.0	*	
Grades (vertical)	>3	3.-3.9	4-4.9	506.9	>7	3.2	2.8	0.4	0.8
Sight Distance Cross Road Intersection (m)	>305	216-305	151-215	120-150	<120	2.0	1.8	0.2	0.2
						Geometric Total			12.0
Operational Factors									
Level of Service (ii) (any dark hour)	A	B	C	D	E	6.0	1.0	5.0	20.0
						Operational Total			20.0
Environmental Factors									
Development	none	1 quad	2 quad	3 quad	4 quad	2.0	0.3	1.7	6.8
Cross-road Approach Illumination	none		partial		continuous	3.0	2.0	1.0	1.0
Freeway Illumination	none		adjacent interchanges		continuous	5.0	3.0	2.0	6.0
						Environmental Total			13.8
Accidents									
% of Night-to Total Accidents (3 yr. Avg.)(ii)	<20%	20-30%	31-40%	41-50%	>50%	10.0	2.0	8.0	24.0
						Accidents Total			24.0
Benefit Cost Ratio B/C									

GEOMETRIC TOTAL	12.0	points
OPERATIONAL TOTAL	20.0	points
ENVIROMENTAL TOTAL	13.8	points
ACCIDENTS TOTAL	24.0	points
SUM	69.8	points
FULL ILLUMINATION WARRANTING CONDITION	75	points

I. A rating of between 1 and 5 shall be assingned for each factor in the FORM depending on the conditions that are encountered by motorists on the roadway. The higher the rating, the more critical the need for illumination with regard to that particular factor.

ii. Use LOS methodology approved by the MTO

iii For night- to- total accident ratio, accidents during darkness are used (including dask/dawn).

iv. The number of points for the warranting condition is based on 50% of the total points attainable, if all factors were rated 5.

Note: Worst case scenarions should be considered when assigning the ratings. For example, a section could have rush hour volumes during the hours of darkness in wintertime.

FORM 1 - Year 2001

FREEWAY INTERCHANGE - FULL ILLUMINATION

Highway: Interchange 400 & Molson Park Road W.P. No.: G.W.P. 30-95-00

Limits: from: 1 km South of Highway 89 to: Junction at Highway 11 Name: Date: Mar-02

CLASSIFICATION FACTOR	RATING (I) 1	2	3	4	5	UNLIT WEIGHT (A)	LIGHTED WEIGHT (B)	DIFF. (A-B)	SCORE [RATINGx (A-B)]
Geometric Factors									
Ramp Types	Directional	Diamond	Cloverleaf Buttonhook	Parclo	Multilevel (iv)Trumpet	2.0	1.0	1.0	2.0
Cross-road Channelization (Raised Islands)	none		continuous		at interchange intersection	2.0	1.0	1.0	3.0
Service Roads	none		one-way		two-way	1.5	1.0	0.5	2.5
Freeway Lane Widths (m)	>3.75	3.75	3.66	3.5	<3.50	3.0	2.5	0.5	1.0
Freeway Median Widths (m)	>15 or barrier		10-15		<10	1.0	0.5	0.5	2.5
No of Freeway Lanes	4		6		8	1.0	0.8	0.2	0.6
Main Line Curves (m) deg	>1800 1	1,800-850 1-2	849-600 2.1-3	599-450 3.1-4	<450 >4	13.0	5.0	8.0	8.0
Grades (vertical)	>3	3.-3.9	4-4.9	506.9	>7	3.2	2.8	0.4	0.4
Sight Distance Cross Road Intersection (m)	>305	216-305	151-215	120-150	<120	2.0	1.8	0.2	0.2
						Geometric Total			20.2
Operational Factors									
Level of Service (ii) (any dark hour)	A	B	C	D	E	6.0	1.0	5.0	15.0
						Operational Total			15.0
Environmental Factors									
Development	none	1 quad	2 quad	3 quad	4 quad	2.0	0.3	1.7	3.4
Cross-road Approach Illumination	none		partial		continuous	3.0	2.0	1.0	5.0
Freeway Illumination	none		adjacent interchanges		continuous	5.0	3.0	2.0	6.0
						Environmental Total			14.4
Accidents									
% of Night-to Total Accidents (3 yr. Avg.)(iii)	<20%	20-30%	31-40%	41-50%	>50%	10.0	2.0	8.0	24.0
						Accidents Total			24.0
Benefit Cost Ratio B/C									

GEOMETRIC TOTAL	20.2	points
OPERATIONAL TOTAL	15.0	points
ENVIROMENTAL TOTAL	14.4	points
ACCIDENTS TOTAL	24.0	points
SUM	73.6	points
FULL ILLUMINATION WARRANTING CONDITION	75	points

I. A rating of between 1 and 5 shall be assingned for each factor in the FORM depending on the conditions that are encountered by motorists on the roadway. The higher the rating, the more critical the need for illumination with regard to that particular factor.

ii. Use LOS methodology approved by the MTO

iii For night- to- total accident ratio, accidents during darkness are used (including dask/dawn).

iv. The number of points for the warranting condition is based on 50% of the total points attainable, if all factors were rated 5.

Note: Worst case scenarios should be considered when assigning the ratings. For example, a section could have rush hour volumes during the hours of darkness in wintertime.

FORM 1 - Year 2011

FREEWAY INTERCHANGE - FULL ILLUMINATION

Highway: Interchange 400 & Molson Park Road W.P. No.: G.W.P. 30-95-00

Limits: from: 1 km South of Highway 89 to: Junction at Highway 11 Name: Date: 2-Mar

CLASSIFICATION FACTOR	RATING (I) 1	2	3	4	5	UNLIT WEIGHT (A)	LIGHTED WEIGHT (B)	DIFF. (A-B)	SCORE [RATINGx (A-B)]
Geometric Factors									
Ramp Types	Directional	Diamond	Cloverleaf Buttonhook	Parclo	Multilevel (iv)Trumpet	2.0	1.0	1.0	2.0
Cross-road Channelization (Raised Islands)	none		continuous		at interchange intersection	2.0	1.0	1.0	3.0
Service Roads	none		one-way		two-way	1.5	1.0	0.5	2.5
Freeway Lane Widths (m)	>3.75	3.75	3.66	3.5	<3.50	3.0	2.5	0.5	1.0
Freeway Median Widths (m)	>15 or barrier		10-15		<10	1.0	0.5	0.5	0.5
No of Freeway Lanes	4		6		8 10	1.0	0.8	0.2	1.0
Main Line Curves (m) deg	>1800 1	1,800-850 1-2	849-600 2.1-3	599-450 3.1-4	<450 >4	13.0	5.0	8.0	8.0
Grades (vertical)	>3	3.-3.9	4-4.9	506.9	>7	3.2	2.8	0.4	0.4
Sight Distance Cross Road Intersection (m)	>305	216-305	151-215	120-150	<120	2.0	1.8	0.2	0.2
						Geometric Total			18.6
Operational Factors									
Level of Service (ii) (any dark hour)	A	B	C	D	E	6.0	1.0	5.0	20.0
						Operational Total			20.0
Environmental Factors									
Development	none	1 quad	2 quad	3 quad	4 quad	2.0	0.3	1.7	8.5
Cross-road Approach Illumination	none		partial		continuous	3.0	2.0	1.0	5.0
Freeway Illumination	none		adjacent interchanges		continuous	5.0	3.0	2.0	6.0
						Environmental Total			19.5
Accidents									
% of Night-to Total Accidents (3 yr. Avg.)(iii)	<20%	20-30%	31-40%	41-50%	>50%	10.0	2.0	8.0	24.0
						Accidents Total			24.0
Benefit Cost Ratio B/C									

GEOMETRIC TOTAL	18.6	points
OPERATIONAL TOTAL	20.0	points
ENVIROMENTAL TOTAL	19.5	points
ACCIDENTS TOTAL	24.0	points
SUM	82.1	points
FULL ILLUMINATION WARRANTING CONDITION	75	points

I. A rating of between 1 and 5 shall be assingned for each factor in the FORM depending on the conditions that are encountered by motorists on the roadway. The higher the rating, the more critical the need for illumination with regard to that particular factor.

ii. Use LOS methodology approved by the MTO

iii For night- to- total accident ratio, accidents during darkness are used (including dask/dawn).

iv. The number of points for the warranting condition is based on 50% of the total points attainable, if all factors were rated 5.

Note: Worst case scenarios should be considered when assigning the ratings. For example, a section could have rush hour volumes during the hours of darkness in wintertime.

FORM 2 - 2001

FREEWAY - CONTINUOUS ILLUMINATION

Highway: 400 W.P. No.: G.W.P. 30-95-00

Limits: from: 1 km South of Hwy89 to: Junction at Hwy 11 Name: Date: Mar-02

CLASSIFICATION FACTOR	RATING (I) 1	2	3	4	5	UNLIT WEIGHT (A)	LIGHTED WEIGHT (B)	DIFF. (A-B)	SCORE [RATINGx (A-B)]
Geometric Factors									
No. of Lanes (2 -way)	4		6		8	1	0.5	0.5	1.5
Lane Width (m)	>3.75	3.75	3.66	3.5	<3.5	3	2.5	0.5	1.0
Median Width (m)	>15 or barrier		10-15		<10	1	0.5	0.5	2.5
Shoulders (m)	3.5	3.25	3	2.75	2.5	1	0.5	0.5	1.5
Slopes (m)	7:1	6:1	5:1	4:1	<4:1	1	0.5	0.5	2.5
Critical Curves m & degrees	>3500 & (<1.2°)	3500-1800 & (2-1°)	1799-850 & (1.1-2°)	849-600 & (2.1-3°)	599-450 & (3.1-4°)	13	4.5	8.5	34.0
Grades (vertical)	<3%	3-3.9%	4-4.9%	5-6.9%	7%	3.2	2.8	0.4	0.8
Interchange Spacing (km)	>3	2.1-3	1.6-2	1-1.5	<1	4	1	3	6.0
						Geometric Total			49.8
Operational Factors									
Level of Service (ii) (any dark hour)	A	B	C	D	E	6	1	5	25.0
						Operational Total			25.0
Environmental Factors									
% Development	0%	25%	50%	75%	100%	3.5	0.5	3	9.0
Illumination Adjacent to Freeway	none	0-40%	41-60%	61-80%	essentially continuous	3	1	2	4.0
						Environmental Total			13.0
Accidents									
% of Night-to Total(ii) Accidents (3yravg.)	<20%	20-30%	31-40%	41-50%	>50%	10	2	8	24.0
						Accidents Total			24.0
Benefit Cost Ratio B/C									

GEOMETRIC TOTAL	49.8	points
OPERATIONAL TOTAL	25.0	points
ENVIROMENTAL TOTAL	13.0	points
ACCIDENTS TOTAL	24.0	points
SUM	111.8	points
CONTINUOUS ILLUMINATION WARRANTING CONDITION	80	points

I. A rating of between 1 and 5 shall be assingned for each factor in the FORM depending on the conditions that are encountered by motorists ont the roadway. The higher the rating, the more critical the need for illumination with regard to that particular factor.

ii. Use LOS methodology approve by the MTO

iii For night- to- total accident ratio, accidents during darkness are used (including dask/dawn).

iv. The number of points for the warranting condition is based on 50% of the total points attainable, if all factors were rated 5.

Note: Worst case scenarios should be considered when assigning the ratings. For example, a section could have rush hour volumes during the hours of darkness in wintertime.

FORM 2 - 2011

FREEWAY - CONTINUOUS ILLUMINATION

Highway: 400 W.P. No.: G.W.P. 30-95-00

Limits: from: 1 km South of Hwy89 to: Junction at Hwy 11 Name: Date: Mar-02

CLASSIFICATION FACTOR	RATING (I) 1	2	3	4	5	UNLIT WEIGHT (A)	LIGHTED WEIGHT (B)	DIFF. (A-B)	SCORE [RATINGx (A-B)]
Geometric Factors									
No. of Lanes (2 -way)	4		6		8 10	1	0.5	0.5	2.5
Lane Width (m)	>3.75	3.75	3.66	3.5	<3.5	3	2.5	0.5	1.0
Median Width (m)	>15 or barrier		10-15		<10	1	0.5	0.5	0.5
Shoulders (m)	3.5	3.25	3	2.75	2.5	1	0.5	0.5	1.5
Slopes (m)	7:1	6:1	5:1	4:1	<4:1	1	0.5	0.5	2.0
Critical Curves m & degrees	>3500 & (<1.2°)	3500-1800 & (2-1°)	1799-850 & (1.1-2°)	849-600 & (2.1-3°)	599-450 & (3.1-4°)	13	4.5	8.5	34.0
Grades (vertical)	<3%	3-3.9%	4-4.9%	5-6.9%	7%	3.2	2.8	0.4	0.8
Interchange Spacing (km)	>3	2.1-3	1.6-2	1-1.5	<1	4	1	3	6.0
						Geometric Total			48.3
Operational Factors									
Level of Service (ii) (any dark hour)	A	B	C	D	E	6	1	5	20.0
						Operational Total			20.0
Environmental Factors									
% Development	0%	25%	50%	75%	100%	3.5	0.5	3	9.0
Illumination Adjacent to Freeway	none	0-40%	41-60%	61-80%	essentially continuous	3	1	2	4.0
						Environmental Total			13.0
Accidents									
% of Night-to Total(ii) Accidents (3yravg.)	<20%	20-30%	31-40%	41-50%	>50%	10	2	8	24.0
						Accidents Total			24.0
Benefit Cost Ratio B/C									

GEOMETRIC TOTAL	48.3	points
OPERATIONAL TOTAL	20.0	points
ENVIROMENTAL TOTAL	13.0	points
ACCIDENTS TOTAL	24.0	points
SUM	105.3	points
CONTINUOUS ILLUMINATION WARRANTING CONDITION	80	points

I. A rating of between 1 and 5 shall be assingned for each factor in the FORM depending on the conditions that are encountered by motorists ont the roadway. The higher the rating, the more critical the need for illumination with regard to that particular factor.

ii. Use LOS methodology approve by the MTO

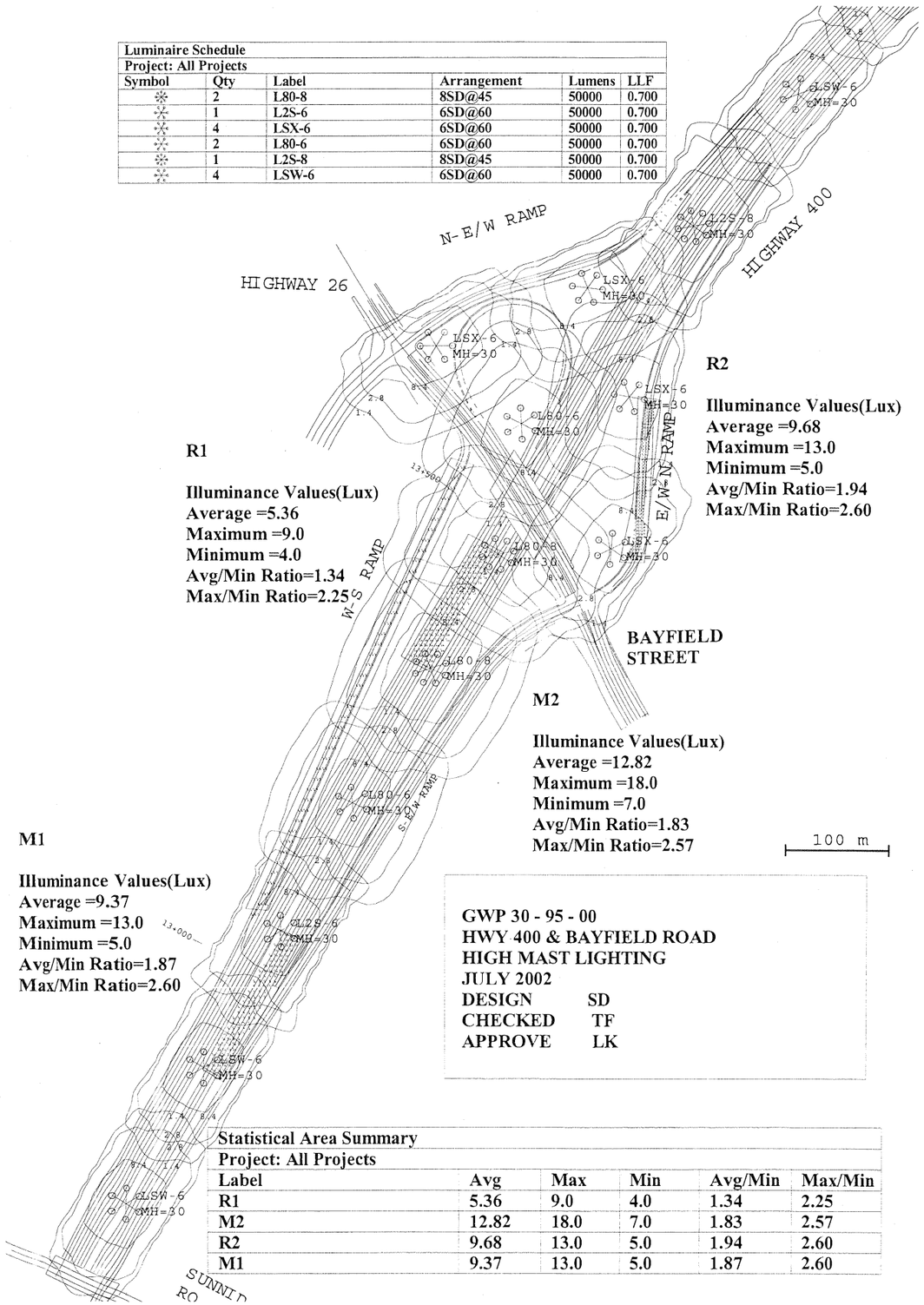
iii For night- to- total accident ratio, accidents during darkness are used (including dask/dawn).

iv. The number of points for the warranting condition is based on 50% of the total points attainable, if all factors were rated 5.

Note: Worst case scenarios should be considered when assigning the ratings. For example, a section could have rush hour volumes during the hours of darkness in wintertime.

APPENDIX C

Luminaire Schedule					
Project: All Projects					
Symbol	Qty	Label	Arrangement	Lumens	LLF
	2	L80-8	8SD@45	50000	0.700
	1	L2S-6	6SD@60	50000	0.700
	4	LSX-6	6SD@60	50000	0.700
	2	L80-6	6SD@60	50000	0.700
	1	L2S-8	8SD@45	50000	0.700
	4	LSW-6	6SD@60	50000	0.700



R1

Illuminance Values(Lux)
Average =5.36
Maximum =9.0
Minimum =4.0
Avg/Min Ratio=1.34
Max/Min Ratio=2.25

R2

Illuminance Values(Lux)
Average =9.68
Maximum =13.0
Minimum =5.0
Avg/Min Ratio=1.94
Max/Min Ratio=2.60

M2

Illuminance Values(Lux)
Average =12.82
Maximum =18.0
Minimum =7.0
Avg/Min Ratio=1.83
Max/Min Ratio=2.57

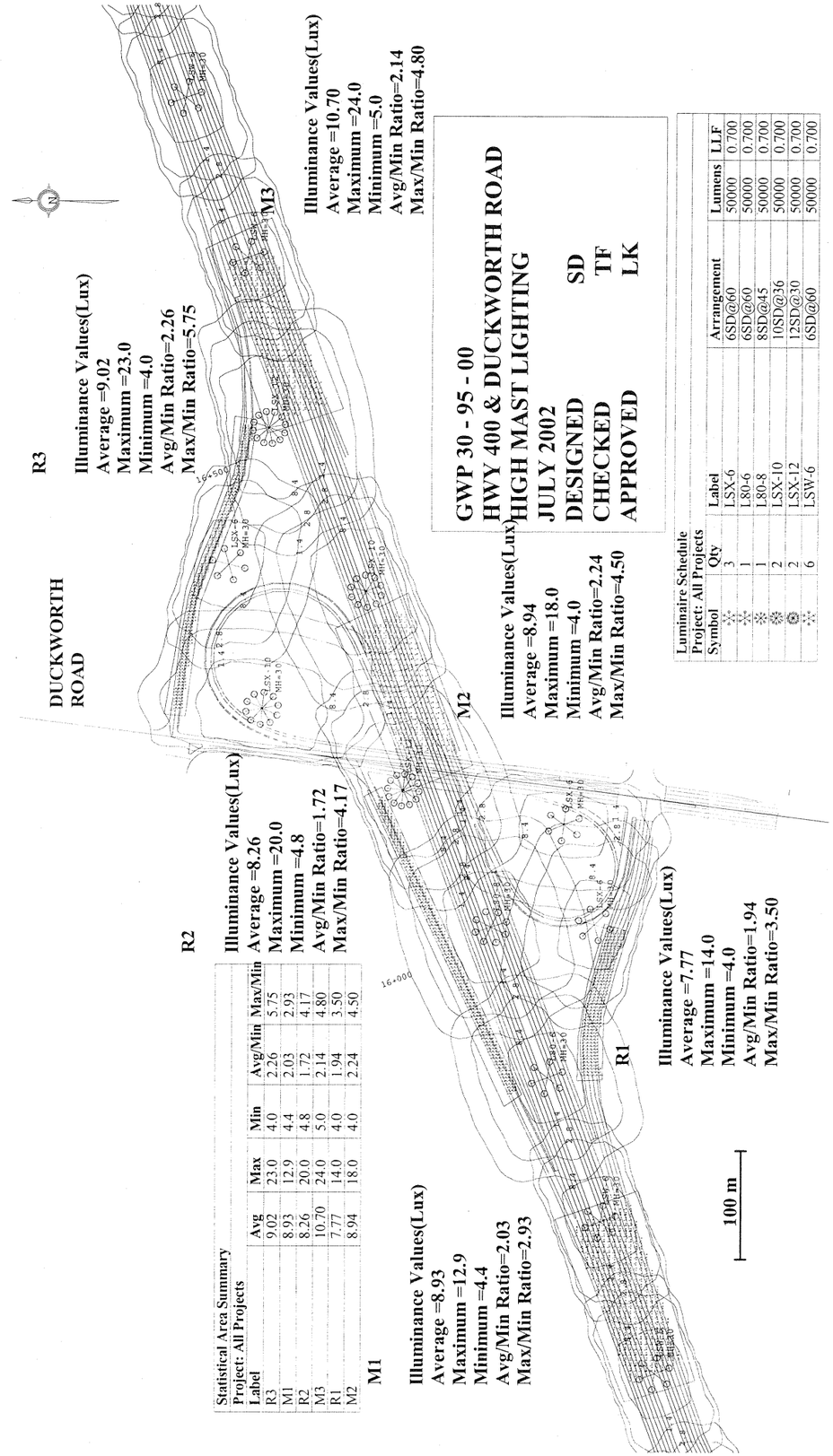
M1

Illuminance Values(Lux)
Average =9.37
Maximum =13.0
Minimum =5.0
Avg/Min Ratio=1.87
Max/Min Ratio=2.60

100 m

GWP 30 - 95 - 00
HWY 400 & BAYFIELD ROAD
HIGH MAST LIGHTING
JULY 2002
DESIGN SD
CHECKED TF
APPROVE LK

Statistical Area Summary					
Project: All Projects					
Label	Avg	Max	Min	Avg/Min	Max/Min
R1	5.36	9.0	4.0	1.34	2.25
M2	12.82	18.0	7.0	1.83	2.57
R2	9.68	13.0	5.0	1.94	2.60
M1	9.37	13.0	5.0	1.87	2.60



R3

Illuminance Values(Lux)
Average =9.02
Maximum =23.0
Minimum =4.0
Avg/Min Ratio=2.26
Max/Min Ratio=5.75

M3

Illuminance Values(Lux)
Average =10.70
Maximum =24.0
Minimum =5.0
Avg/Min Ratio=2.14
Max/Min Ratio=4.80

M2

Illuminance Values(Lux)
Average =8.94
Maximum =18.0
Minimum =4.0
Avg/Min Ratio=2.24
Max/Min Ratio=4.50

R2

Illuminance Values(Lux)
Average =8.26
Maximum =20.0
Minimum =4.8
Avg/Min Ratio=1.72
Max/Min Ratio=4.17

R1

Illuminance Values(Lux)
Average =7.77
Maximum =14.0
Minimum =4.0
Avg/Min Ratio=1.94
Max/Min Ratio=3.50

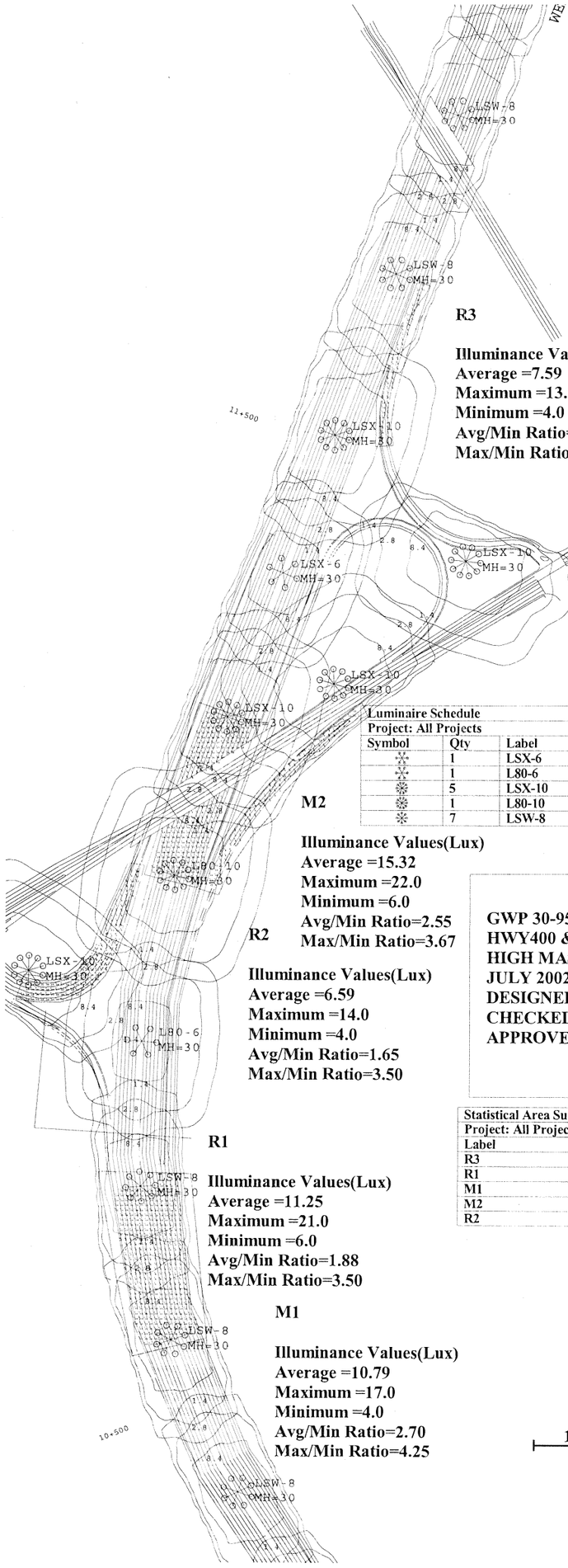
M1

Illuminance Values(Lux)
Average =8.93
Maximum =12.9
Minimum =4.4
Avg/Min Ratio=2.03
Max/Min Ratio=2.93

GWP 30 - 95 - 00
HWY 400 & DUCKWORTH ROAD
HIGH MAST LIGHTING
JULY 2002
DESIGNED SD
CHECKED TF
APPROVED LK

Statistical Area Summary					
Project: All Projects					
Label	Avg	Max	Min	Avg/Min	Max/Min
R3	9.02	23.0	4.0	2.26	5.75
M1	8.93	12.9	4.4	2.03	2.93
R2	8.26	20.0	4.8	1.72	4.17
M3	10.70	24.0	5.0	2.14	4.80
R1	7.77	14.0	4.0	1.94	3.50
M2	8.94	18.0	4.0	2.24	4.50

Luminaire Schedule					
Project: All Projects					
Symbol	Qty	Label	Arrangement	Lumens	LLF
	3	LSX-6	6SD@60	50000	0.700
	1	L80-6	6SD@60	50000	0.700
	1	L80-8	8SD@45	50000	0.700
	2	LSX-10	10SD@36	50000	0.700
	2	LSX-12	12SD@30	50000	0.700
	6	LSW-6	6SD@60	50000	0.700



R3

Illuminance Values(Lux)
Average =7.59
Maximum =13.0
Minimum =4.0
Avg/Min Ratio=1.90
Max/Min Ratio=3.25

DUNLOP STREET WEST

Luminaire Schedule					
Project: All Projects					
Symbol	Qty	Label	Arrangement	Lumens	LLF
	1	LSX-6	6SD@60	50000	0.700
	1	L80-6	6SD@60	50000	0.700
	5	LSX-10	10SD@36	50000	0.700
	1	L80-10	10SD@36	50000	0.700
	7	LSW-8	8SD@45	50000	0.700

M2

Illuminance Values(Lux)
Average =15.32
Maximum =22.0
Minimum =6.0
Avg/Min Ratio=2.55
Max/Min Ratio=3.67

R2

Illuminance Values(Lux)
Average =6.59
Maximum =14.0
Minimum =4.0
Avg/Min Ratio=1.65
Max/Min Ratio=3.50

GWP 30-95-00
HWY400 & DUNLOP STREET WEST
HIGH MAST LIGHTING
JULY 2002
DESIGNED SD
CHECKED TF
APPROVED LK

Statistical Area Summary					
Project: All Projects					
Label	Avg	Max	Min	Avg/Min	Max/Min
R3	7.59	13.0	4.0	1.90	3.25
R1	11.25	21.0	6.0	1.88	3.50
M1	10.79	17.0	4.0	2.70	4.25
M2	15.32	22.0	6.0	2.55	3.67
R2	6.59	14.0	4.0	1.65	3.50

R1

Illuminance Values(Lux)
Average =11.25
Maximum =21.0
Minimum =6.0
Avg/Min Ratio=1.88
Max/Min Ratio=3.50

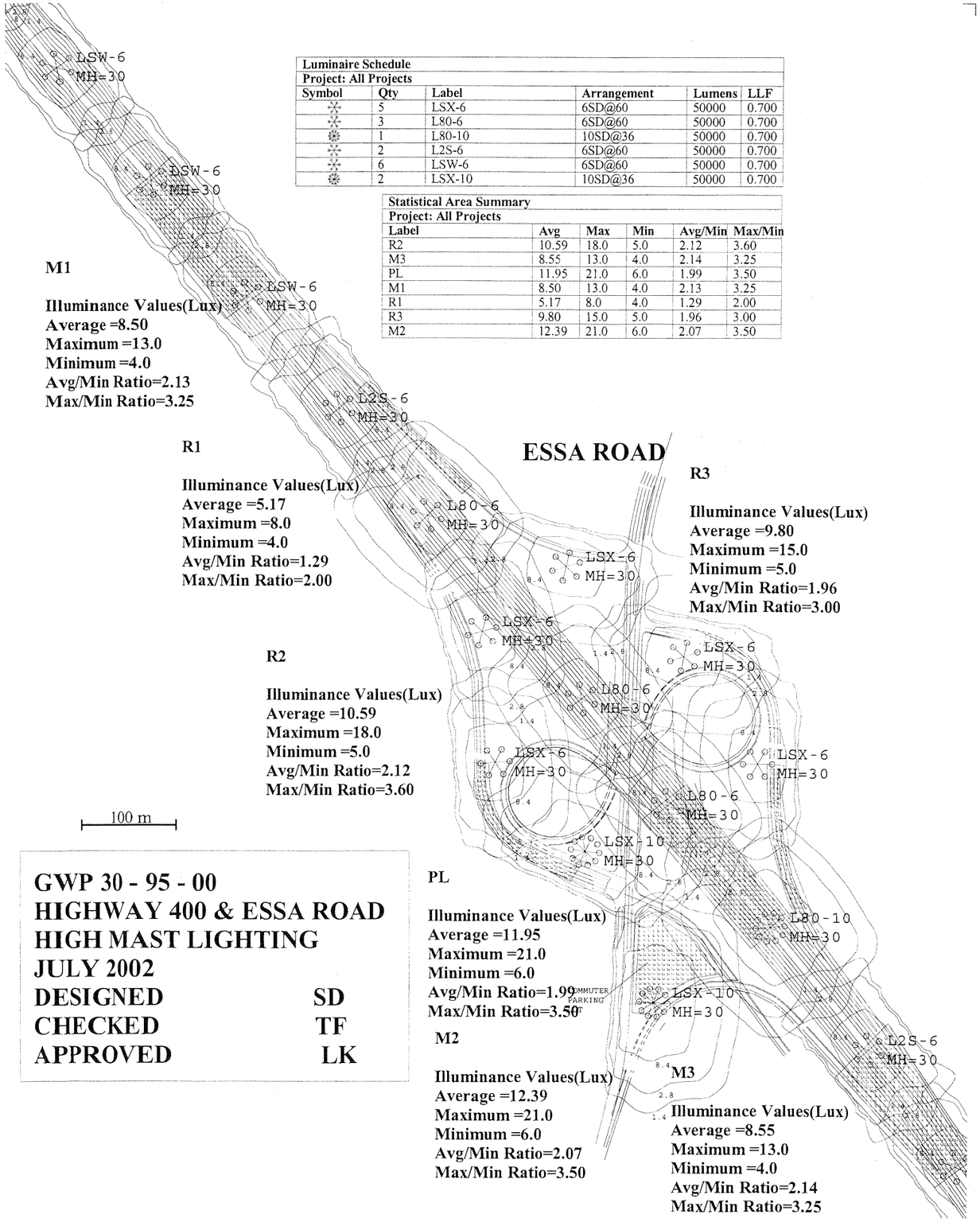
M1

Illuminance Values(Lux)
Average =10.79
Maximum =17.0
Minimum =4.0
Avg/Min Ratio=2.70
Max/Min Ratio=4.25

100 m

Luminaire Schedule					
Project: All Projects					
Symbol	Qty	Label	Arrangement	Lumens	LLF
	5	LSX-6	6SD@60	50000	0.700
	3	L80-6	6SD@60	50000	0.700
	1	L80-10	10SD@36	50000	0.700
	2	L2S-6	6SD@60	50000	0.700
	6	LSW-6	6SD@60	50000	0.700
	2	LSX-10	10SD@36	50000	0.700

Statistical Area Summary					
Project: All Projects					
Label	Avg	Max	Min	Avg/Min	Max/Min
R2	10.59	18.0	5.0	2.12	3.60
M3	8.55	13.0	4.0	2.14	3.25
PL	11.95	21.0	6.0	1.99	3.50
M1	8.50	13.0	4.0	2.13	3.25
R1	5.17	8.0	4.0	1.29	2.00
R3	9.80	15.0	5.0	1.96	3.00
M2	12.39	21.0	6.0	2.07	3.50



GWP 30 - 95 - 00
HIGHWAY 400 & ESSA ROAD
HIGH MAST LIGHTING
JULY 2002

DESIGNED	SD
CHECKED	TF
APPROVED	LK

Luminaire Schedule
Project: All Projects

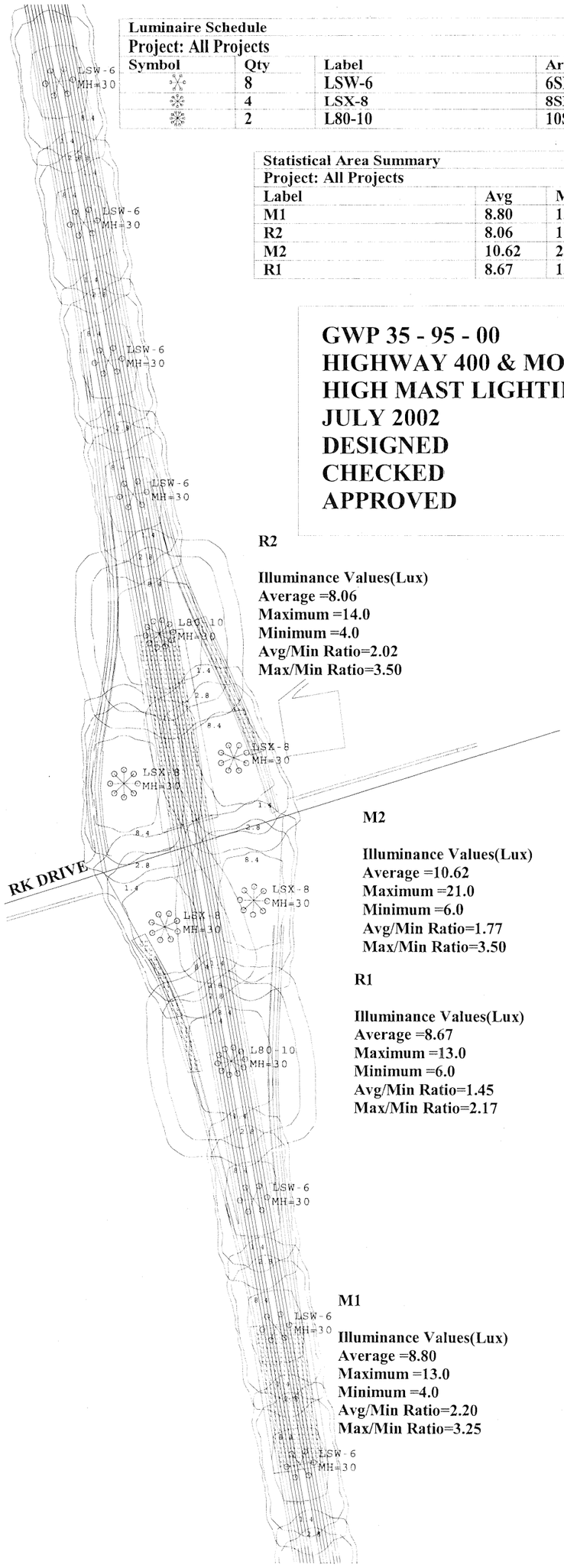
Symbol	Qty	Label	Arrangement	Lumens	LLF
	8	LSW-6	6SD@60	50000	0.700
	4	LSX-8	8SD@45	50000	0.700
	2	L80-10	10SD@36	50000	0.700

Statistical Area Summary
Project: All Projects

Label	Avg	Max	Min	Avg/Min	Max/Min
M1	8.80	13.0	4.0	2.20	3.25
R2	8.06	14.0	4.0	2.02	3.50
M2	10.62	21.0	6.0	1.77	3.50
R1	8.67	13.0	6.0	1.45	2.17

GWP 35 - 95 - 00
HIGHWAY 400 & MOLSON PARK DRIVE
HIGH MAST LIGHTING
JULY 2002

DESIGNED	SD
CHECKED	TF
APPROVED	LK



R2

Illuminance Values(Lux)
Average =8.06
Maximum =14.0
Minimum =4.0
Avg/Min Ratio=2.02
Max/Min Ratio=3.50

100 m

M2

Illuminance Values(Lux)
Average =10.62
Maximum =21.0
Minimum =6.0
Avg/Min Ratio=1.77
Max/Min Ratio=3.50

R1

Illuminance Values(Lux)
Average =8.67
Maximum =13.0
Minimum =6.0
Avg/Min Ratio=1.45
Max/Min Ratio=2.17

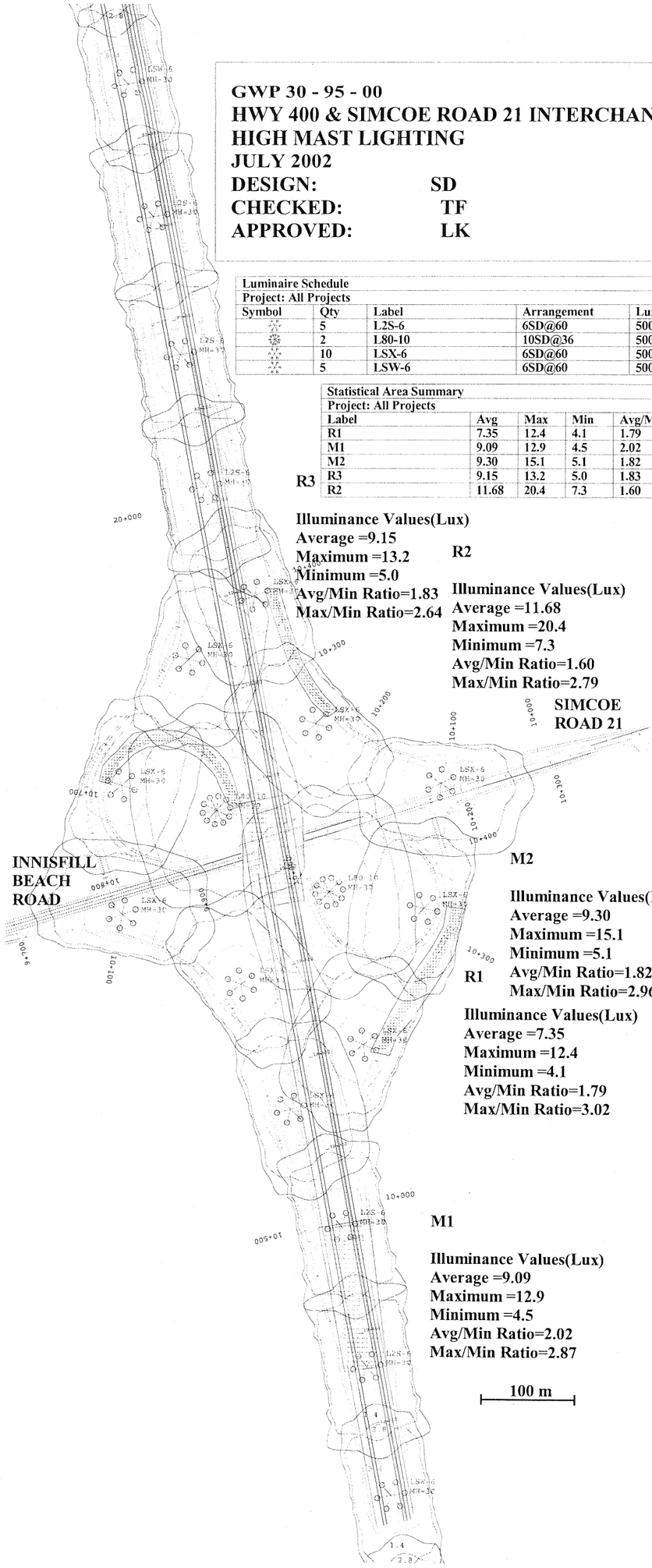
M1

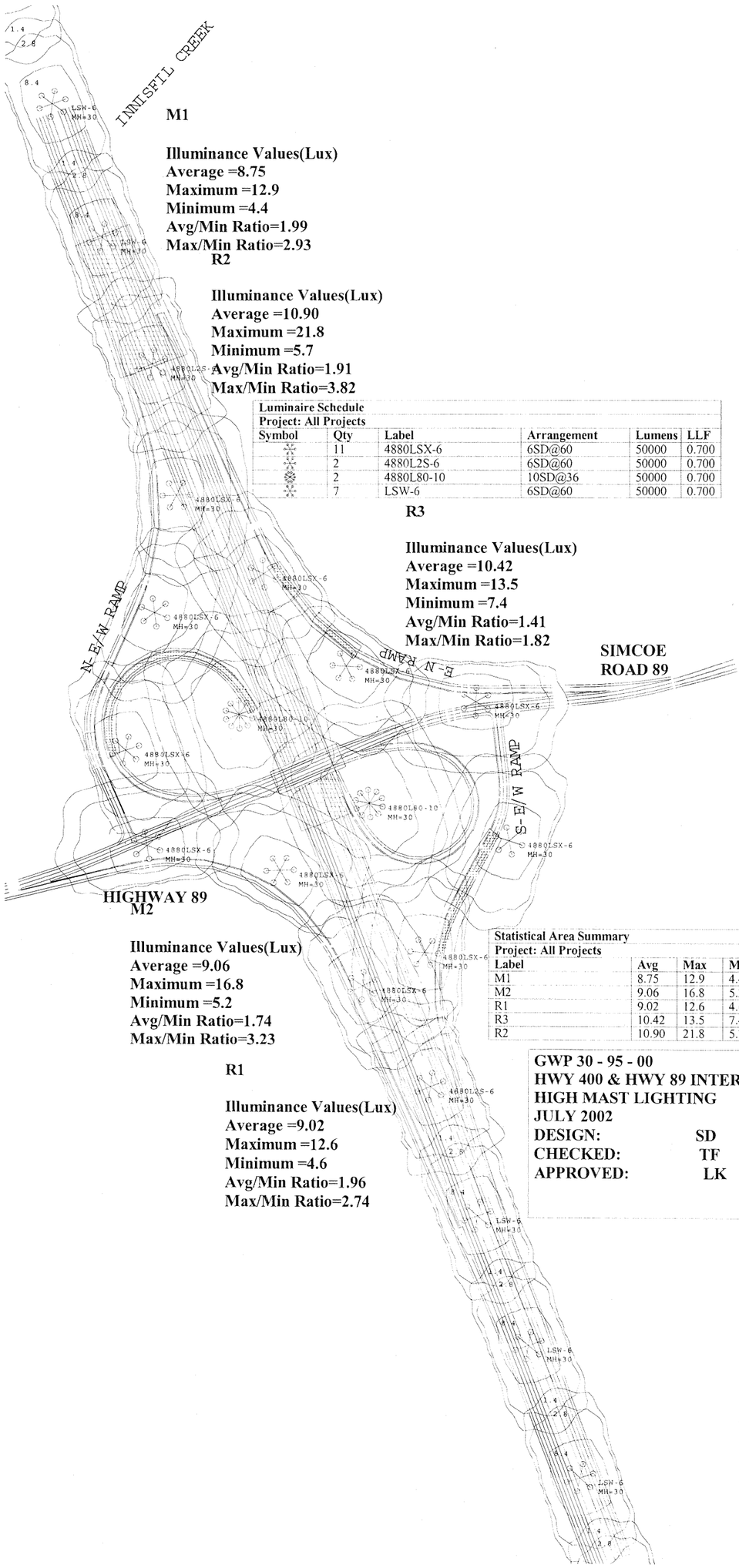
Illuminance Values(Lux)
Average =8.80
Maximum =13.0
Minimum =4.0
Avg/Min Ratio=2.20
Max/Min Ratio=3.25

GWP 30 - 95 - 00
HWY 400 & SIMCOE ROAD 21 INTERCHANGE
HIGH MAST LIGHTING
JULY 2002
DESIGN: SD
CHECKED: TF
APPROVED: LK

Luminaire Schedule					
Project: All Projects					
Symbol	Qty	Label	Arrangement	Lumens	LLF
	5	L2S-6	6SD@60	50000	0.700
	2	L80-10	10SD@36	50000	0.700
	10	LSX-6	6SD@60	50000	0.700
	5	LSW-6	6SD@60	50000	0.700

Statistical Area Summary					
Project: All Projects					
Label	Avg	Max	Min	Avg/Min	Max/Min
R1	7.35	12.4	4.1	1.79	3.02
M1	9.09	12.9	4.5	2.02	2.87
M2	9.30	15.1	5.1	1.82	2.96
R3	9.15	13.2	5.0	1.83	2.64
R2	11.68	20.4	7.3	1.60	2.79





M1

Illuminance Values(Lux)
Average =8.75
Maximum =12.9
Minimum =4.4
Avg/Min Ratio=1.99
Max/Min Ratio=2.93

R2

Illuminance Values(Lux)
Average =10.90
Maximum =21.8
Minimum =5.7
Avg/Min Ratio=1.91
Max/Min Ratio=3.82

Luminaire Schedule
Project: All Projects

Symbol	Qty	Label	Arrangement	Lumens	LLF
	11	4880LSX-6	6SD@60	50000	0.700
	2	4880L2S-6	6SD@60	50000	0.700
	2	4880L80-10	10SD@36	50000	0.700
	7	LSW-6	6SD@60	50000	0.700

R3

Illuminance Values(Lux)
Average =10.42
Maximum =13.5
Minimum =7.4
Avg/Min Ratio=1.41
Max/Min Ratio=1.82

M2

Illuminance Values(Lux)
Average =9.06
Maximum =16.8
Minimum =5.2
Avg/Min Ratio=1.74
Max/Min Ratio=3.23

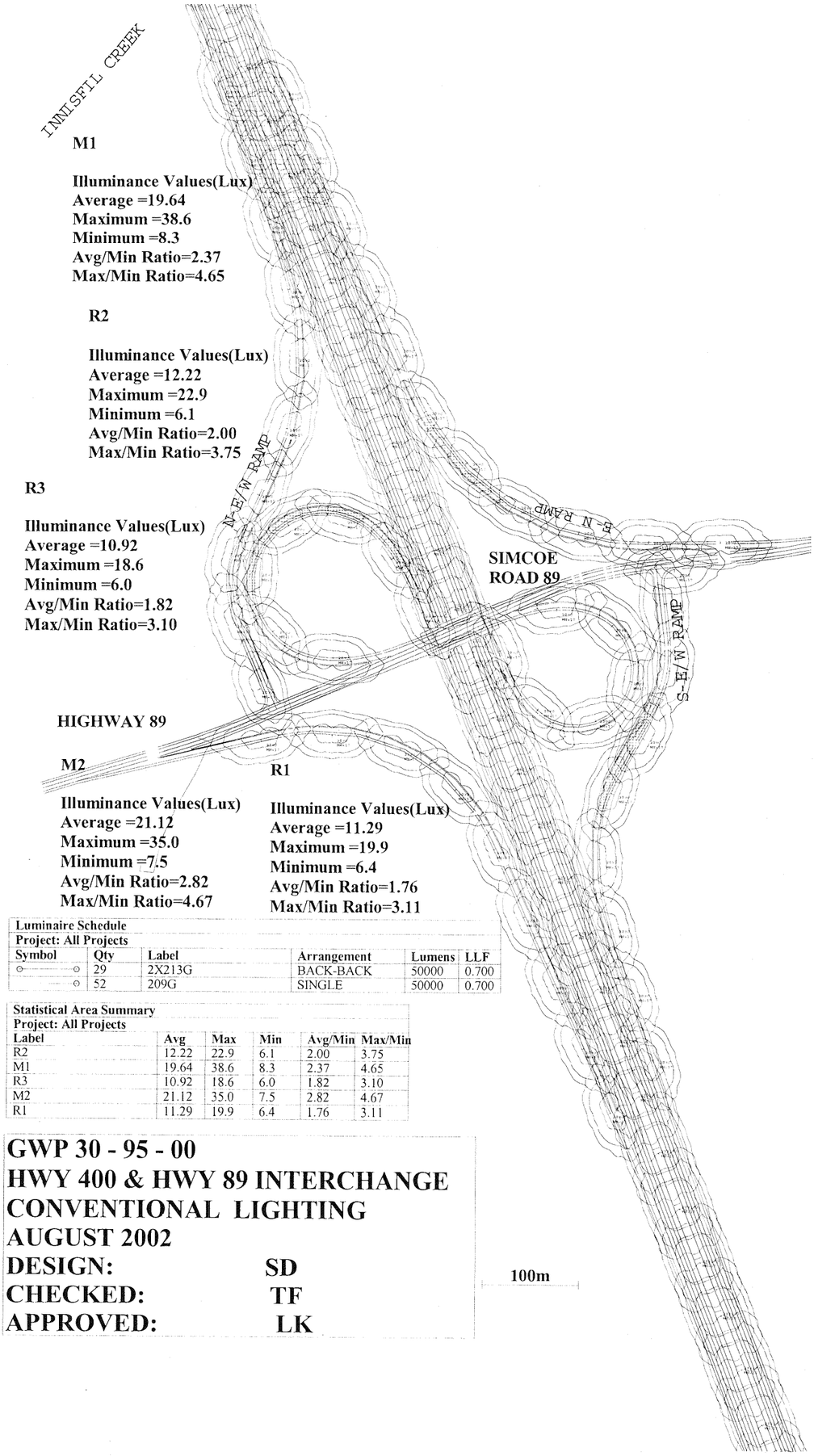
R1

Illuminance Values(Lux)
Average =9.02
Maximum =12.6
Minimum =4.6
Avg/Min Ratio=1.96
Max/Min Ratio=2.74

Statistical Area Summary
Project: All Projects

Label	Avg	Max	Min	Avg/Min	Max/Min
M1	8.75	12.9	4.4	1.99	2.93
M2	9.06	16.8	5.2	1.74	3.23
R1	9.02	12.6	4.6	1.96	2.74
R3	10.42	13.5	7.4	1.41	1.82
R2	10.90	21.8	5.7	1.91	3.82

GWP 30 - 95 - 00
HWY 400 & HWY 89 INTERCHANGE
HIGH MAST LIGHTING
JULY 2002
DESIGN: SD
CHECKED: TF
APPROVED: LK



Luminaire Schedule
Project: All Projects

Symbol	Qty	Label	Arrangement	Lumens	LLF
	29	2X213G	BACK-BACK	50000	0.700
	52	209G	SINGLE	50000	0.700

Statistical Area Summary
Project: All Projects

Label	Avg	Max	Min	Avg/Min	Max/Min
R2	12.22	22.9	6.1	2.00	3.75
M1	19.64	38.6	8.3	2.37	4.65
R3	10.92	18.6	6.0	1.82	3.10
M2	21.12	35.0	7.5	2.82	4.67
R1	11.29	19.9	6.4	1.76	3.11

GWP 30 - 95 - 00
HWY 400 & HWY 89 INTERCHANGE
CONVENTIONAL LIGHTING
AUGUST 2002
DESIGN: SD
CHECKED: TF
APPROVED: LK

APPENDIX D

HWY404 & SIMCOE ROAD 89 INTERCHANGE ***HIGH MAST LIGHTING*** AUG, 2002

LIFE CYCLE COST COMPONENT	DESCRIPTION	SYSTEM							TOTAL
Initial Construction Cost	No. of poles x cost/pole Cost/Pole=total cost of footing, luminaires, lamps raising and lowering device and estimated cost of wiring systemand power supply per assembly, including material and labour	22 x	$55,000 =					$1,210,000	
Other Construction Cost	Cost of additional construction associated with the system in median	9 x 0 x	$5,000 = $0 =					$45,000 $0	
Detour Lighting	Temporary overhead wiring	0 x	$0 x	0 =				$0	$1,255,000
Replacement Cost 'A' (Lamps)	Material, labour and equiment cost. Material= No. of lamps x lamp cost. Labour=No. of lamps x replacement time per lamp x no. of men x hourly rate Equipment= Total time (no. of poles x time per pole) x vehicles x equipment hourly rate	2 x 20 x 2 x 20 x 2 x 22 x	10 x 6 x 10 x 6 x 10 x 6 x	$35 = $35 = 0.5 x 0.5 x 0.5 x 0.5 x	1 x 1 x 1 x 1 x	$40 = $40 = $25 = $25 =		$700 $4,200 $400 $2,400 $250 $1,650	$7,950
Replacement Period 'A' (Lamps)	Time period in which all lamps are to be replaced. This calculation is based on lamp life (24,000 hrs for HPS) and estimated 4,000 burning hours/year						6years		
Replacement Cost 'B' (Luminaires)	Material, labour and equiment cost. Material= No. of luminaires x luminaire cost. Labour= No. of luminaires x replacement time / luminaire x no. of men x hourly rate Equipment= Total time (no. of poles x time per pole) x vehicles x equipment hourly rate	2 x 20 x 2 x 22 x 2 x 22 x	10 x 6 x 10 x 6 x 10 x 6 x	$700 = $700 = 0.5 x 0.5 x 0.5 x 0.5 x	2 x 2 x 1 x 1 x	$40 = $40 = $25 = $25 =		$14,000 $84,000 $800 $5,280 $250 $1,650	$105,980
Replacement Period 'B' (Luminaires)	Time period in which all luminaires are to be replaced (Ave. 20 years)						20 years		
Replacement Cost 'C' (Poles)	Material, labour and equiment cost. Material= No. of poles x cost/pole Labour= Replacement time / pole x no. of poles x no. of men x hourly rate Equipment= Total time x vehicles x equipment hourly rate	0 x 0 x 0 x	$22,000 = 4 x 4 x	6 x 3 x	$40 = $150 =			$0 $0 $0	$0
Replacement Period 'C' (Poles)	Time period (in years)in which all poles are to be replaced. Conventional concrete poles: 20 Conventional steel poles: 30 High mast poles: 40						40 years		
Annual Energy Cost	Watts/luminaire (incl. ballast loss) x total no. of luminaires x estimated burning hours per year x energy rate per kWh	2 x 20 x	10 x 6 x	0.485 x 0.485 x	4000 x 4000 x	0.05 = 0.05 =		$1,940 $11,640	$13,580
Annual Maintenance Cost	Labour and equiment cost. Labour= No. of poles x maintenance time / pole x no. of men x hourly rate Equipment= Total time (no. of poles x time per pole) x vehicles x equipment hourly rate	22 x 22 x	x x	1 x 1 x	1 x 1 x	$40 = $25 =		$880 $550	$1,430
Incident Cost	Conventional lighting: 2% of total no. of poles x pole replacement cost. Pole replacement includes material, labour and equipment cost.	0 of 0 x	2 = $22,000 =	0			1	$0	$0
Salvage Value	No. of units (lamps, luminaires, poles) x salvaged value per unit. Salvaged value per unit is calculated as percentage of remaining life. Salvaged value at the end of life: Lamp: 0 Luminaire: 0 Concrete pole: 0 Steel pole: $600 High mast pole: $2,500	0 x 20 x	$600 = $2,500 =					$0 $50,000	$50,000
Total									$1,433,940

HIGH MAST LIGHTING

HWY 404 SIMCOE ROAD INTERCHANGE

GWP 30-95-00 HWY 404 PDR

Lifecycle Cost Analysis
16-Sep-03

Basic Information			
Time Frame For Analysis	40	years	(Should be <= 50 years)
Inflation Rate	5.0%	(eg .05 For 5%)	
Discount Rate	5.0%	(eg .1 For 10%)	

Relevant Information	($ x 1,000)		Replacement Period in years (eg if every 5 years enter 5)
Inital Construction Cost	$1,255		
Other Construction Cost	$45		
Detour Lighting Cost	$0		
Replacement Cost "A"	$8		6
Replacement Cost "B"	$106		20
Replacement Cost "C"	$0		40
Annual Energy Cost	$14		
Annual Maintenance Cost	$1	*	********** ********** **********
Annual Incident Cost	$0		Present Value Lifecycle Costs $2,104
Salvage Value	$50	*	********** ********** **********

Analysis Of Alternative ($ x 1,000)	Time 0	Year 1	Year 2	Year 3	Year 4	Year 5	Year 6	Year 7	Year 8	Year 9	Year 10	Year 11
Inflation Index	1.000	1.050	1.103	1.158	1.216	1.276	1.340	1.407	1.477	1.551	1.629	1.710
Inital Construction Cost	1,255											
Other Construction Cost	45											
Detour Lighting Cost	0											
Replacement Cost "A"		0	0	0	0	0	11	0	0	0	0	0
Replacement Cost "B"		0	0	0	0	0	0	0	0	0	0	0
Replacement Cost "C"		0	0	0	0	0	0	0	0	0	0	0
Annual Energy Cost		15	15	16	17	18	19	20	21	22	23	24
Annual Maintenance Cost		1	1	1	1	1	1	1	1	2	2	2
Annual Incident Cost		0	0	0	0	0	0	0	0	0	0	0
Salvage Value		0	0	0	0	0	0	0	0	0	0	0
Annual Cash Flow	1,300	16	17	17	18	19	31	21	22	23	24	26
Discount Factors	1.0000	0.9524	0.9070	0.8638	0.8227	0.7835	0.7462	0.7107	0.6768	0.6446	0.6139	0.5847
Discounted Cash Flow	1,300	15	15	15	15	15	23	15	15	15	15	15
Present Value Lifecycle Costs		$2,104										

Highway Lighting
Economic Analysis
16-Sep-03

GWP 30-95-00 HWY 404 PDR HWY 404 SIMCOE ROAD INTERCHANGE HIGH MAST LIGHTING

Year 12	Year 13	Year 14	Year 15	Year 16	Year 17	Year 18	Year 19	Year 20	Year 21	Year 22	Year 23	Year 24	Year 25	Year 26	Year 27
1.796	1.886	1.980	2.079	2.183	2.292	2.407	2.527	2.653	2.786	2.925	3.072	3.225	3.386	3.556	3.733
14	0	0	0	0	0	19	0	0	0	0	0	26	0	0	0
0	0	0	0	0	0	0	0	281	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
25	26	28	29	31	32	34	35	37	39	41	43	45	47	50	52
2	2	2	2	2	2	2	3	3	3	3	3	3	3	4	4
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
41	28	30	31	33	34	55	38	321	42	44	46	74	51	53	56
0.5568	0.5303	0.5051	0.4810	0.4581	0.4363	0.4155	0.3957	0.3769	0.3589	0.3418	0.3256	0.3101	0.2953	0.2812	0.2678
23	15	15	15	15	15	23	15	121	15	15	15	23	15	15	15

Year 28	Year 29	Year 30	Year 31	Year 32	Year 33	Year 34	Year 35	Year 36	Year 37	Year 38	Year 39	Year 40	Year 41	Year 42	Year 43
3.920	4.116	4.322	4.538	4.765	5.003	5.253	5.516	5.792	6.081	6.385	6.705	7.040	7.392	7.762	8.150
0	0	35	0	0	0	0	0	46	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
55	58	61	64	67	70	74	77	81	85	89	94	99	0	0	0
4	4	4	5	5	5	5	6	6	6	6	7	7	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	352	0	0	0
59	62	99	68	71	75	79	83	133	91	96	101	458	0	0	0
0.2551	0.2429	0.2314	0.2204	0.2099	0.1999	0.1904	0.1813	0.1727	0.1644	0.1566	0.1491	0.1420	0.1353	0.1288	0.1227
15	15	23	15	15	15	15	15	23	15	15	15	65	0	0	0

GWP 30-95-00 HWY 404 PDR HWY 404 SIMCOE ROAD INTERCHANGE HIGH MAST LIGHTING

Year 44	Year 45	Year 46	Year 47	Year 48	Year 49	Year 50
8.557	8.985	9.434	9.906	10.401	10.921	11.467
0	0	0	0	0	0	0
0	0	0	0	0	0	0
0	0	0	0	0	0	0
0	0	0	0	0	0	0
0	0	0	0	0	0	0
0	0	0	0	0	0	0
0	0	0	0	0	0	0
0	0	0	0	0	0	0
0.1169	0.1113	0.1060	0.1009	0.0961	0.0916	0.0872
0	0	0	0	0	0	0

HWY 404 & SIMCOE ROAD 89 INTERCHANGE CONVENTIONAL LIGHTING AUG, 2002

LIFE CYCLE COST COMPONENT	DESCRIPTION	SYSTEM	TOTAL
Initial Construction Cost	No. of poles x cost/pole Cost/Pole=total cost of footing, luminaires, lamps raising and lowering device and estimated cost of wiring system and power supply per assembly, including material and labour	81 x $7,500 = $607,500	
Other Construction Cost	Cost of additional construction associated with the system. Extra cost for installation of pole bases in median	29 x $500 = $14,500	
Detour Lighting	No. of poles x cost/pole Cost of wiring	0 x $6,000 = $0 0 x $2,000 x 20 = $0	$622,000
Replacement Cost 'A' (Lamps)	Material, labour and equiment cost.	No. of lamps= (29x2)+52=110	
	Material= No. of lamps x lamp cost.	110 x $25 = $2,750	
	Labour= no. of lamps x replacement time per lamp x no. of men x hourly rate	110 x 1 x 6 x $40 = $26,400	
	Equipment= Total time (no. of lamps x replacement time/lamp) x no. of vehicles x equipment hourly rate	110 x 1 x 3 x $150 = $49,500	$78,650
Replacement Period 'A' (Lamps)	Time period in which all lamps are to be replaced. This calculation is based on lamp life (24,000 hrs for HPS) and estimated 4,000 burning hours/year	6years	
Replacement Cost 'B' (Luminaires)	Material, labour and equiment cost. Material= No. of luminaires x luminaire cost.	110 x $500 = $55,000	
	Labour= No. of luminaires x replacement time / luminaire x no. of men x hourly rate	110 x 1 x 6 x $40 = $26,400	
	Equipment= Total time (no. of luminaires x replacement time/luminaire) x no. of vehicles x equipment hourly rate	110 x 1 x 3 x $150 = $49,500	$130,900
Replacement Period 'B' (Luminaires)	Time period in which all luminaires are to be replaced (Ave. 20 years)	20 years	
Replacement Cost 'C' (Poles)	Material, labour and equiment cost. Material= No. of poles x cost/pole	81 x $2,000 = $162,000	
	Labour= No. of poles xreplacement time / pole x no. of men x hourly rate	81 x 2 x 6 x $40 = $38,880	
	Equipment= Total time (no. of poles x replacement time/pole) x no. of vehicles x equipment hourly rate	81 x 2 x 3 x $150 = $72,900	$273,780
Replacement Period 'C' (Poles)	Time period (in years)in which all poles are to be replaced. Conventional concrete poles: 20 Conventional steel poles: High mast poles: 30 40	30 years	
Annual Energy Cost	Watts/luminaire (incl. ballast loss) x total no. of luminaires x estimated burning hours per year x energy rate per kWh	110 x 0.485 x 4000 x 0.05 = $10,670	$10,670
Annual Maintenance Cost	Labour and equiment cost. Labour= no. of poles x maintenance time / pole x no. of men x hourly rate	81 x 0.5 x 6 x $40 = $9,720	
	Equipment= no. of poles x maintenance time / pole x vehicles x equipment hourly rate	81 x 0.5 x 3 x $150 = $18,225	$27,945
Incident Cost	Conventional lighting: 2% of total no. of poles x pole replacement cost. Pole replacement includes material, labour and equipment cost.	2 of 52 = 1 1 x $2,000 = $2,000	$2,000
Salvage Value	No. of units (lamps, luminaires, poles) x salvaged value per unit. Salvaged value per unit is calculated as percentage of remaining life. Salvaged value at the end of life: Lamp: 0 Luminaire: 0 Concrete pole: 0 Steel pole: High mast pole: $2,500 $600	81 x $600 = $48,600	$48,600
Total			$1,209,045

NOTES: Total: 81 poles, 30x2+16=76 luminaires

GWP 35-90-00 HWY 404 PDR HWY 404 SIMCOE ROAD INTERCHANGE CONVENTIONAL LIGHTING

Lifecycle Cost Analysis 16-Sep-03

Title

Basic Information
- Time Frame For Analysis 40 years (Should be <= 50 years)
- Inflation Rate 5.0% (eg .05 For 5%)
- Discount Rate 5.0% (eg .1 For 10%)

	($ x 1,000)	Replacement Period in years (eg if every 5 years enter 5)
Relevant Information		
Inital Construction Cost	$608	
Other Construction Cost	$15	
Detour Lighting Cost	$0	
Replacement Cost "A"	$79	6
Replacement Cost "B"	$131	20
Replacement Cost "C"	$274	30
Annual Energy Cost	$11	
Annual Maintenance Cost	$28	* ********** ********** **********
Annual Incident Cost	$2	Present Value
		Lifecycle Costs $3,191
Salvage Value	$49	* ********** ********** ********** **********

Analysis Of Alternative ($ x 1,000)	Time 0	Year 1	Year 2	Year 3	Year 4	Year 5	Year 6	Year 7	Year 8	Year 9	Year 10	Year 11
Inflation Index	1.000	1.050	1.103	1.158	1.216	1.276	1.340	1.407	1.477	1.551	1.629	1.710
Inital Construction Cost	608											
Other Construction Cost	15											
Detour Lighting Cost	0											
Replacement Cost "A"		0	0	0	0	0	106	0	0	0	0	0
Replacement Cost "B"		0	0	0	0	0	0	0	0	0	0	0
Replacement Cost "C"		0	0	0	0	0	0	0	0	0	0	0
Annual Energy Cost		12	12	13	13	14	15	15	16	17	18	19
Annual Maintenance Cost		29	31	32	34	36	38	39	41	43	46	48
Annual Incident Cost		2	2	2	2	3	3	3	3	3	3	3
Salvage Value		0	0	0	0	0	0	0	0	0	0	0
Annual Cash Flow	623	43	45	47	50	52	161	58	61	64	67	70
Discount Factors	1.0000	0.9524	0.9070	0.8638	0.8227	0.7835	0.7462	0.7107	0.6768	0.6446	0.6139	0.5847
Discounted Cash Flow	623	41	41	41	41	41	120	41	41	41	41	41
Present Value Lifecycle Costs		$3,191										

Highway Lighting
Economic Analysis
16-Sep-03

GWP 35-90-00 HWY 404 PDR HWY 404 SIMCOE ROAD INTERCHANGE CONVENTIONAL LIGHTING

Year 12	Year 13	Year 14	Year 15	Year 16	Year 17	Year 18	Year 19	Year 20	Year 21	Year 22	Year 23	Year 24	Year 25	Year 26	Year 27
1.796	1.886	1.980	2.079	2.183	2.292	2.407	2.527	2.653	2.786	2.925	3.072	3.225	3.386	3.556	3.733
142	0	0	0	0	0	190	0	0	0	0	0	255	0	0	0
0	0	0	0	0	0	0	0	348	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
20	21	22	23	24	25	26	28	29	31	32	34	35	37	39	41
50	53	55	58	61	64	67	71	74	78	82	86	90	95	100	105
4	4	4	4	4	5	5	5	5	6	6	6	6	7	7	7
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
216	77	81	85	89	94	289	104	456	114	120	126	387	139	146	153
0.5568	0.5303	0.5051	0.4810	0.4581	0.4363	0.4155	0.3957	0.3769	0.3589	0.3418	0.3256	0.3101	0.2953	0.2812	0.2678
120	41	41	41	41	41	120	41	172	41	41	41	120	41	41	41

Year 28	Year 29	Year 30	Year 31	Year 32	Year 33	Year 34	Year 35	Year 36	Year 37	Year 38	Year 39	Year 40	Year 41	Year 42	Year 43
3.920	4.116	4.322	4.538	4.765	5.003	5.253	5.516	5.792	6.081	6.385	6.705	7.040	7.392	7.762	8.150
0	0	341	0	0	0	0	0	458	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	1,184	0	0	0	0	0	0	0	0	0	0	0	0	0
43	45	48	50	52	55	58	61	64	67	70	74	77	0	0	0
110	115	121	127	133	140	147	154	162	170	179	188	197	0	0	0
8	8	9	9	10	10	11	11	12	12	13	13	14	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0	345	0	0	0
161	169	1,703	186	195	205	215	226	695	249	262	275	634	0	0	0
0.2551	0.2429	0.2314	0.2204	0.2099	0.1999	0.1904	0.1813	0.1727	0.1644	0.1566	0.1491	0.1420	0.1353	0.1288	0.1227
41	41	394	41	41	41	41	41	120	41	41	41	90	0	0	0

GWP 35-90-00 HWY 404 PDR HWY 404 SIMCOE ROAD INTERCHANGE CONVENTIONAL LIGHTING

Year 44	Year 45	Year 46	Year 47	Year 48	Year 49	Year 50
8.557	8.985	9.434	9.906	10.401	10.921	11.467
0	0	0	0	0	0	0
0	0	0	0	0	0	0
0	0	0	0	0	0	0
0	0	0	0	0	0	0
0	0	0	0	0	0	0
0	0	0	0	0	0	0
0	0	0	0	0	0	0
0	0	0	0	0	0	0
0.1169	0.1113	0.1060	0.1009	0.0961	0.0916	0.0872
0	0	0	0	0	0	0

APPENDIX E

Appendix E

CONSTRUCTION COST ESTIMATE

A. Illumination - Interchanges

	poles	fixtures		cost
Hwy400&89, HML	22	140		1,600
Hwy400&89, 800m Conv.	20			140
Hwy400&Bayfield Street, HML		14	90	1,025
Hwy400&Bayfield Street, 300m Conv.		7		50
Hwy400&Duckworth Street, HML		15	112	1,125
Hwy400&Duckworth Street, 450m Conv.		11		80
Hwy400&Dunlop Street, HML		15	128	1,175
Hwy400&Dunlop Street, 700 Conv.	17			120
Hwy400&Essa Street, HML	19	126		1,325
Hwy400&Essa Street, 550m Conv.		14		100
Hwy400&Innisfill Beach Road, HML		22	140	1,500
Hwy400&Innisfill Beach Road, 650m Conv.		16		115
Hwy400&Molson Park Road, HML	14	100		1,050
Hwy400&Molson Park Road, 200m Conv.		5		35
Hwy400&Simcoe Road 21 commuter parking lot	8			60
Hwy400&Molson Park Road commuter parking lot	8			60
Temporary Illumination	30			150

B. Illumination – Mainline

Molson Park Drive to Essa Road	10	600
Essa Road to Dunlop Street West	1	60
Dunlop Street West to Bayfield Street	3	180
Bayfield Street to Duckworth Street	7	420
HML Illumination Subtotal		10,770

C. Municipal Illumination

- Essa Road from Ardagh Road to N-E/W approx. length of 250m.	7	35
- South leg of the N-E/W ramp terminal approx. length 100m.	3	15
- Molson Park Drive East & E/W – S ramp approx. length 125m.	5	25
- Molson Park Drive West approx. length225m.	6	30
- Dunlop Street West approx. length 100m.	3	15
- Dunlop Street, approx. length 200m.	6	30
- Ann Street, length approx. 400m.	11	55
- Sunnidale Road, length approx. 250m.	7	35
- Cedar Pointe Drive, approx. length of 150m.	5	25
- Bayfield Street, approx. length 200m.	6	30
- Bayfield Street, approx. length 250m.	7	35
- Rose Street cul-de-sac.	3	15
- Coulter Street , approx. length 150m	5	25
- Cookstown Outlet Mall, approx. length 200m.	6*	45
Municipal Illumination Subtotal		415

* Decorative Luminaires

D. Underpass Illumination

- Hwy 400 & Hwy 89 Underpass	16
- Hwy 400 & Bayfield Street Underpass	20
- Hwy400&Duckworth Street Underpass	16
- Hwy400&Dunlop Street Underpass	16
- Hwy400&Essa Street Overpass	48
- Hwy400&Innisfill Beach Road Overpass	32
- Hwy400&Molson Park Road Overpass	32
Underpass Illumination Subtotal	180

E. Traffic Signals Permanent

- Essa Road & N-E/W ramp terminal	125
- Essa Road & S-E/W ramp terminal	100
- Essa Road & E-N ramp terminal	90
- Molson Park Drive & E/W – N ramp terminal (one quadrant affected by ramp realignment)	50
- Molson Park Drive & N – E/W ramp terminal (two quadrants affected by ramp realignment)	75
- Hwy 89 & Realigned Reive Road	90
- Hwy 89 & S-E/W ramp terminal	100
- Hwy 89 & N-E/W ramp terminal	100
- Simcoe Road 21 & S-E/W ramp terminal	90
- Innisfill Beach Road & N-E/W ramp terminal	90
- Duckworth Street & S-E/W ramp Terminal (one quadrant and two median islands modified)	75
- Duckworth Street & N-E/W ramp terminal	100
- Bayfield Street & S-E/W ramp terminal	110
- Bayfield Street & N-E/W ramp terminal	110
- Dunlop Street & Hart Drive/ E/W-N ramp terminal	100
- Dunlop Street & N-E/W ramp terminal /Cedar Pointe Drive	125
Traffic Signals Permanent Subtotal	1,530

F. Traffic signals Temporary

- Essa Road & N-E/W ramp terminal (conversion of 3 leg ramp terminal to full 4 leg intersection)	75
- Essa Road & S-E/W ramp terminal (complete shift of existing ramp terminal east)	65
- Essa Road & E-N ramp terminal (two quadrants affected by ramp realignment)	75
- Molson Park Drive & E/W – N ramp terminal (one quadrant affected by ramp realignment)	35
- Molson Park Drive & N – E/W ramp terminal (two quadrants affected by ramp realignment)	45
- Hwy 89 & Realigned Reive Road	
- Hwy 89 & S-E/W ramp terminal complete shift of existing ramp terminal north)	75
- Hwy 89 & N-E/W ramp terminal (complete shift of existing ramp terminal north)	75
- Simcoe Road 21 & S-E/W ramp Terminal (complete shift of existing ramp terminal east)	75
- Duckworth Street & S-E/W ramp terminal (one quadrant and two median islands modified)	50
- Duckworth Street & N-E/W ramp terminal (three quadrants and two median islands modified)	75
- Bayfield Street & S-E/W ramp terminal (complete shift of existing ramp terminal north) Detail design in two stages during construction	95
- Bayfield Street & N-E/W ramp terminal (complete shift of existing ramp terminal north), two construction stages	95
- Dunlop Street & Hart Drive/ E/W-N ramp terminal, two construction stages	95
- Dunlop Street & N-E/W ramp terminal /Cedar Pointe Drive, two construction stages	95
Traffic Signals Temporary Subtotal	1,025

G. Municipal Traffic Signals

- Anne Street & Wellingtion Street	75
- Simcoe Road & Industrial Park Road	85
Municipal Traffic Signals Subtotal	160

H. Traffic Counting Loops

1.6 km north of Highway 89 (STA 11+600), 16 loops	18
2.0 km north of Innisfil Beach Road (STA 21+630), 16 loops	18
1.0 km north of Molson Park Drive (STA 26+280), 16 loops	18
0.7 km north of Essa Road (STA 29+180), 22 loops	25
1.3 km north of Dunlop (STA 12+650), 20 loops	23
0.6 km north of Bayfield Street (STA 14+080), 18 loops	20
0.5 km north of Duckworth Street (STA 16+675), 16 loops	18
Traffic Counting Loops Subtotal	140

I. Distribution Assemblies

- Hwy 400 & Hwy 89	35
- Hwy 400 & Bayfield Street	35
- Hwy400&Duckworth Street	35
- Hwy400&Dunlop Street	35
- Hwy400&Essa Street	35
- Hwy400&Innisfill Beach Road	35
- Hwy400&Molson Park Road	35
Distribution Assemblies Subtotal	245

Ontario

Ministry of Transportation
Southwestern Region
Planning and Design Section

HIGHWAY 400 PLANNING AND PRELIMINARY DESIGN STUDY

FROM 1 KM SOUTH OF HIGHWAY 89 NORTHERLY 30 KM TO THE JUNCTION AT HIGHWAY 11

COUNTY OF SIMCOE
TOWN OF BRADFORD WEST GWILLIMBURY
TOWN OF INNISFIL
CITY OF BARRIE
TOWNSHIP OF SPRINGWATER

G.W.P. 30-95-00

APPENDIX O

PROPERTY WASTE & CONTAMINATION ASSESSMENT

Golder Associates Ltd.

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

DRAFT REPORT ON

PROPERTY WASTE AND CONTAMINATION ASSESSMENT
WASTE MATERIAL ASSESSMENT AND MANAGEMENT
HIGHWAY 400 FROM HIGHWAY 9
NORTHERLY TO HIGHWAY 11 (BARRIE)
G.W.P.30-95-00, AGREEMENT NUMBER: 3005-A-000074
AND
G.W.P.40-00-00, AGREEMENT NUMBER: 3005-A-000177

Submitted to:

Cole Sherman & Associates Ltd.
75 Commerce Valley Drive East
Thornhill, Ontario
L3T 7N9

Distribution of Report:

2 copies - Cole Sherman & Associates Ltd.
2 copies - Golder Associates Ltd.

March 2001

001-8393

50 BEST EMPLOYERS IN CANADA 2004

CANADA'S 50 BEST MANAGED COMPANIES

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder Associates), was retained by Cole Sherman & Associates Ltd. (Cole Sherman) to conduct a Waste, Contamination and Excess Material Assessment and Management investigation of the portion of Highway 400 extending from Highway 9 northerly to Highway 11 (i.e., Site) in the Townships of King, Bradford-West Gwillimbury, Innisfil and Springwater, Ontario. The objective of the work was to determine in a preliminary fashion, the relative potential for the presence of waste materials and/or subsurface contamination that could affect the proposed construction works to be present within the proposed Highway 400 right of way (ROW). Further, the purpose of the assessment was to provide recommendations relating to the management and disposal of identified waste and contaminated materials that may be generated during the proposed upgrading of Highway 400.

2.0 RECORDS REVIEW

2.1 Scope of Work

Golder Associates' assessment included a review of available references concerning site history and subsurface conditions, and a one-day field reconnaissance. Details of the information reviewed and field reconnaissance findings are provided in Sections 3.1 to 5.0 of this report. Section 6.0 outlines identified issues of potential environmental concern and Section 7.0 details the waste material assessment and management findings. Study limitations and appropriate uses for this report are given in Section 8.0. References are provided in Section 9.0.

The scope of this Waste, Contamination and Excess Material Assessment is sufficient only to identify environmental liabilities, which are obvious from visual examination of surface features or from available sources of information. No soil, water, liquid, gas, construction material, product or chemical sampling and testing on or in the vicinity of the subject site was conducted as part of this assessment. This assessment included a cursory overview of the neighbouring land uses and does not constitute a complete assessment of the adjacent facilities.

3.0 HISTORICAL INFORMATION REVIEW

In order to assess historical and current land uses and characteristics along the ROW, a preliminary review of information from the following sources was conducted:

- Ministry of Environment (MOE), Barrie District Office;
- Ministry of Environment (MOE), Owen Sound District Office;
- Ministry of Natural Resources, Midhurst District Office;
- The Corporation of the City of Barrie;
- The Township of King;
- The Township of Bradford-West Gwillimbury;
- The Township of Innisfil;
- The Township of Springwater;
- The County of Simcoe;
- The Regional Municipality of York;
- Canadian National ;
- Aerial Photographs obtained from the National Air Photo Library, Ottawa, Ontario;
- Various MOE databases relating to PCB storage sites, waste disposal sites, and coal gasification and related facilities;
- Published geological maps and reports;
- MOE water well records; and
- Field reconnaissance.

3.1 The Ministry of Environment, Barrie and Owen Sound District Offices

The Ministry of Environment (MOE), Barrie and Owen Sound District Offices were contacted by fax on December 15, 2000, and January 4, 2001, respectively, to determine if the MOE has any environmental information pertaining to the ROW. Specifically, the MOE was asked to respond to the following questions:

1. Has the MOE ever issued any approvals, permits or licences for the Site?
2. Has the MOE ever issued any control orders or violation notices with respect to the Site?
3. Does the MOE have any knowledge that the Site or adjacent properties have ever been used or are currently used as a waste disposal sites?
4. Does the MOE have any records of spills on or within a one-kilometre radius of the subject property?

At the time of issuing this report, Golder Associates had not received a response to these specific questions from the MOE Barrie or Owen Sound District Offices. Once a response is received it will be reviewed by Golder Associates and forwarded to Cole Sherman and Associates Limited and should be attached to Appendix A of this report.

Correspondence sent to Golder Associates from the MOE, including figures prepared by Clayton Environmental Consultants and dated January 1999 and from the City of Barrie, including a report prepared by International Water Consultants Ltd. and dated October 2000, provide further information on the extent of a TCE impacted groundwater plume in the vicinity of Tiffin Street. Based on the information contained in the International Water Consultants Ltd. report entitled *"The City of Barrie, Wood Street Well, Water Quality Investigation, Interim Report, 2000"*, the extent of the TCE impacted groundwater plume extends from east portion of the Canplas property on the west side of Highway 400 to at least 600 m east of Highway 400 approaching Anne Street. Groundwater flow is anticipated to be in a northeasterly direction. The report recommends the installation of four to six new wells in help determine the northeasterly extent of the plume. The extent and nature of the TCE impacted groundwater plume in the vicinity of Tiffin Street and Highway 400 should be specifically investigated during the design phase of the project, to address health and safety and the potential generation of soil and groundwater waste.

3.2 The Ministry of Natural Resources, Midhurst District Office

The Ministry of Natural Resources, Midhurst District Office (the MNR), was contacted by letter on December 15, 2000, to determine if the MNR has any environmental information pertaining to the site including sewer use discharges/violations, waste disposal, filling activities, spills, borehole logs, etc.

Golder Associates received a response from Ms. Brenda Robinson from the Ministry of Natural

Resources, Midhurst District Office dated January 3, 2001. The response indicated that the MNR has no records on file with respect to the Site.

3.3 The Corporation of the City of Barrie

The Corporation of the City of Barrie (the City) was contacted by letter on December 15, 2000, and March 13, 2001, to determine if the City has any environmental information pertaining to the subject property including sewer use discharges/violations, waste disposal, filling activities, spills, borehole logs, etc.

At the time of issuing this report, Golder Associates had not received a response from the City. Once a response is received it will be reviewed by Golder Associates and forwarded to Cole Sherman and Associates Limited and should be attached to Appendix A of this report.

3.4 The Townships of King, Bradford-West Gwillimbury, Innisfil

The Townships of King, Bradford West Gwillimbury, Innisfil and Springwater (the Townships) were contacted by letter on March 13, 2001, to determine if the Townships have any environmental information pertaining to the subject property including sewer use discharges/violations, waste disposal, filling activities, spills, borehole logs, etc.

At the time of issuing this report, Golder Associates had not received a response from the Townships. Once a response is received it will be reviewed by Golder Associates and forwarded to Cole Sherman and Associates Limited and should be attached to Appendix A of this report.

3.5 The Township of Springwater

The Township of Springwater (the Township) was contacted by letter on March 13, 2001, to determine if the Township has any environmental information pertaining to the subject property including sewer use discharges/violations, waste disposal, filling activities, spills, borehole logs, etc.

Mr. Donald H. Priest, Public Works Superintendent for the Township of Springwater issued a response dated March 29, 2001. The response indicated that the area is generally farmland and open space and therefore does not have any information pertaining to the Site

3.6 The County of Simcoe

The County of Simcoe (the County) was contacted by letter on December 15, 2000, to determine if the County has any environmental information pertaining to the subject property including sewer use discharges/violations, waste disposal, filling activities, spills, borehole logs, etc.

At the time of issuing this report, Golder Associates had not received a response from the County. Once a response is received it will be reviewed by Golder Associates and forwarded to Cole Sherman and Associates Limited and should be attached to Appendix A of this report.

3.7 The Regional Municipality of York

The Regional Municipality of York (the Region) was contacted by letter on March 13, 2001, to determine if the Region has any environmental information pertaining to the subject property including sewer use discharges/violations, waste disposal, filling activities, spills, borehole logs, etc.

At the time of issuing this report, Golder Associates had not received a response from the Region. Once a response is received it will be reviewed by Golder Associates and forwarded to Cole Sherman and Associates Limited and should be attached to Appendix A of this report.

3.8 Canadian National

Canadian National (CN) was contacted via facsimile transmission on January 3, 2001, to determine if CN has any record of incidences of possible environmental impact occurring in the vicinity of the project limits and specifically the CN rail line corridor across the ROW near Tiffin Street in the City of Barrie. Mr. Erwin Waldinsperger of CN issued a response dated January 17, 2001, indicating that information with respect to such incidences was not available for review.

3.9 MOE Databases

The MOE Publications "Ontario Inventory of PCB Storage Sites" (January, 1998), "Waste Disposal Site Inventory" (June, 1991), "Coal Gasification Plant Waste Sites in Ontario" (April, 1987), "Inventory of Industrial Sites Producing or Using Coal Tar and Related Tars in Ontario"

(November 1988), "MOE Waste Generator Database" (1996) and MOE water well records for the ROW were reviewed by Golder. These sources indicated that:

The following locations were identified as MOE registered PCB Storage Sites:

- Corah Ltd., 60 Bell Farm Road, located on the east side of ROW;
- Ontario Hydro-Barrie, 306 Tiffin Street;
- West Bend, 28 Currie Street, located on the east side of ROW; and
- 200 Brock Street, located on the east side of ROW.

The following locations, as shown on Figure 2, were identified as being a closed landfill sites:

- Vespra and Innisfill Streets (Landfill Sites No. 4102 and 4013);
- Anne Street North and Vespra Street (Landfill Site No. 4101);
- John and Innisfil Streets (Landfill Site No. 4106);
- Highway 400, approximately 1.7 km south of Highway 89 (Landfill Site No. 4163); and
- Approximately 1 km west of the ROW and 3.6 km north of Highway 9 (Landfill Site No. 4166)

An active landfill site, as shown on Figure 2, was identified on the east side of the ROW, approximately one kilometre south of Innisfil Heights (Landfill Site No. 252202).

- The Site and adjacent properties are not listed as a coal gasification plant waste site.
- No listings were found in the MOE Waste Generator Database for the Site and adjacent properties.

3.10 Aerial Photographs

Aerial photographs for the area in which the Site is located were reviewed in order to assess historical and current land uses and characteristics. Aerial photographs for the years 1975 and 1995 indicated the following:

<u>1975</u>

- Properties located along Highway 400 extending from the south project limits northerly to the City of Barrie Municipal Limits primarily consist of undeveloped/agricultural land uses with sparse residential developments.
- A sewage treatment plant is present on the southeast portion of Highway 400, approximately

500 m south of the Highway 400/88 junction.

- A possible gas station is present on the northwest corner of Highway 400 and Highway 89.
- Innisfil Heights is present on the southeast side of the ROW, approximately 600 m south of the CN rail line and Highway 400 junction
- An industrial structure is present on the east side of the ROW, approximately 450 m northeast of the CN rail line and Highway 400 junction.
- A brewery and filtration plant exists on the east side of ROW, approximately two kilometers south of Highway 27.
- Commercial/retail, sparse recreational and industrial developments are located on the immediate east and west portions of the ROW. Dense residential development extends further east and west of the ROW.

<u>1995</u>

- Further commercial/industrial development exists on the east side of the ROW, on the north and south portions of the CN rail line and Highway 400 junction.
- Commercial/retail and residential development has extended south to the Municipal Limits of the City of Barrie and Innisfil Township.
- An abundance of commercial/retail, industrial and residential developments occupy the adjacent properties to the east and west of the ROW within the City of Barrie Limits.

3.11 Previously Report

Cole Sherman provided Golder Associates with a report entitled *"Phase I Environmental Site Assessment, WP 295-99-00, Highway 400 / Innisfil Beach Road, Interchange Improvements, Town of Innisfil, County of Simcoe, District 33 - Area North, November 2000"* completed by Peto MacCallum Ltd. The Phase I ESA was carried out on a vacant strip of land located on the southwest corner of the Innisfil Beach Road (County Road 21) and Highway 400 in the Town of Innisfil, Ontario. Based on the findings of the Phase I ESA, no actual sources of contamination were identified for the subject property. However, potential sources of contamination were identified at the time of the Phase I ESA, including agricultural residues, localized fill materials, road salts and petroleum hydrocarbons associated with a previous owner and related to the adjacent road/highway/parking lot.

Cole Sherman provided Golder Associates with a report entitled *"Phase II Environmental Site*

Assessment, WP 295-99-00, Highway 400 / Innisfil Beach Road, Interchange Improvements, Town of Innisfil, County of Simcoe, District 33 - Area North, January 2000" completed by Peto MacCallum Ltd. The Phase II ESA was carried out on a vacant strip of land located on the southwest corner of the Innisfil Beach Road (County Road 21) and Highway 400 in the Town of Innisfil, Ontario. The Phase II ESA was conducted following the completion of the Phase I ESA, to investigate the subsurface conditions at the subject property for evidence of contamination, including petroleum hydrocarbons related to a suspected petroleum-related facility that may have historically occupied the subject property. Additionally, the chemical quality of the existing topsoil and localized fill was analyzed in order to determine appropriate handling options during construction of the interchange improvements. Based on the findings of the Phase II ESA, there was no evidence to suggest that a petroleum-related facility occupied the subject property. The results of the chemical testing program indicated that no petroleum or volatile organic compounds (VOCs) were present in the soils. The topsoil was found to contain elevated concentrations of select pesticides, including DDE, DDT and aldrin, in addition to dieldrin concentrations that exceeded the provincial guideline. Peto MacCallum Ltd. recommended further sampling and analyses to determine the lateral and vertical extent of dieldrin levels in the topsoil and appropriate handling options.

Cole Sherman provided Golder Associates with a report entitled *"Supplemental Phase II Environmental Site Assessment, WP 295-99-00, Highway 400 / Innisfil Beach Road, Interchange Improvements, Town of Innisfil, County of Simcoe, District 33 - Area North, February 2000"* completed by Peto MacCallum Ltd. The Supplemental Phase II ESA was carried out on a vacant strip of land located on the southwest corner of the Innisfil Beach Road (County Road 21) and Highway 400 in the Town of Innisfil, Ontario. The Supplemental Phase II ESA was conducted to investigate the lateral and vertical extent of dieldrin levels found in the on-site topsoil during the initial Phase II Investigation and to provide appropriate handling options. Based on the findings of the Supplemental Phase II ESA, approximately 370 m^3 (750 tonnes) of dieldrin-impacted topsoil was determined to be present on-site. Further, dieldrin levels in the underlying native sandy soils were found to meet the provincial criterion. Peto MacCallum Ltd. recommended that the topsoil be remediated to meet the provincial criteria. However, an option to manage the contaminated soil through a risk assessment and monitoring program and an option to have the current land owner appropriately remove and recycle the topsoil prior to selling the property, was presented.

4.0 PHYSICAL SETTING

4.1 Regional Geological Setting

Site specific groundwater and subsurface conditions were not directly investigated by Golder Associates as part of this report. Based on the information provided on the Surveys and Mapping Branch, Department of Energy, Mines and Resources topographic and 31 D/4, edition 7, Alliston, the southernmost limit of the subject property is located approximately 1.2 km south of the Holland River, which flows northeast towards Lake Simcoe. Several unidentified tributaries, which generally flow northeast towards Lake Simcoe, cross the ROW in various locations throughout the project limits. Innisfil Creek, flowing northeast towards Lake Simcoe, underlies the ROW approximately 1 km north of the Highway 89/400 junction. According to the Surveys and Mapping Branch, Department of Energy, Mines and Resources, topographic map 31 D/5, edition 6, Barrie, groundwater in the vicinity of the City of Barrie is generally expected to flow east towards Kempenfelt Bay. The northernmost limit of the subject property is located approximately 0.3 km north of an underground reservoir, which flows west towards Little Lake.

According to Ontario Geological Survey, Ministry of Northern Development and Mines, Map 2544, Bedrock Geology of Ontario, Southern Sheet, bedrock underlying the entire length of the project limits consists of limestone, dolostone, shale, arkose and sandstone of the Ottawa and Simcoe Groups and the Shadow Lake Formation.

According to the Ontario Geological Survey, Ministry of Natural Resources Map 2175, Physiography of Southern Ontario, 3rd Edition, a region of peat and muck (Holland Marsh) extends from the south project limits northerly for approximately 4 km. A series of drumlins span across a region of clay plain, drumlinized till plain, kame moraine and sand plan, from the northern tip of the Holland Marsh to approximately 5 km north of Highway 89. A till plain exists from Highway 89 through the Highway 27/400 junction. The portion of the ROW located immediately west of Kempenfelt Bay in the City of Barrie, predominately consists of a sand plain of raised beaches and shorecliffs. Drumlinized till plain extends from the City of Barrie Municipal Limits to the north project limits.

According to the Ontario Geological Survey, Ministry of Northern Development and Mines, Map 2556, Quaternary Geology of Ontario, Southern Sheet, Glaciolacustrine deposits lie within the

ROW from the southernmost project limits northerly through Highway 88. A combination of sand and silt to silt matrix of Newmarket Till, Glaciolacustrine deposits of gravel and sand, sand and gravelly sand, and Glaciofluvial ice-contact deposits of gravel and sand, minor till including eskers, kames and end moraine extend from Highway 88 northerly to the City of Barrie. A till of sandy silt to silt matrix exists from the City of Barrie to the northernmost project limits.

5.0 FIELD SURVEY

A visual site reconnaissance of the length of Highway 400 under investigation was conducted, in which the road allowance and adjacent properties were visually inspected for the potential presence of waste materials and/or subsurface contamination. The site reconnaissance was conducted on December 15, 2000 and March 12, 2001, by an experienced environmental specialist from the Golder Associates' Whitby office.

Land uses in the vicinity of the southern portion of the site were observed to be primarily residential and agricultural with some isolated areas of commercial usage. Land uses in the vicinity of the northern portion of the site, through the City of Barrie, were observed to be primarily commercial and industrial. No areas of significant staining or vegetative die back, which might indicate the presence of subsurface impact, were observed along the project limits. It should be noted that the ground surface was snow-covered at the time of the Site visit.

Numerous active farming operations are present along the ROW. No specific issues of potential environmental concern were typically noted at the farm properties, however, farming operations typically include vehicle maintenance and re-fuelling, historical waste disposal and fertilizer, herbicide and pesticide storage. Where farm properties are to be acquired that include the farm buildings, (as opposed to fields) then the properties should be assessed on a case by case basis.

A summary of the significant observations of the visual reconnaissance is provided below:

East Side of ROW

- Commercial farming operations are located immediately north of the Highway 9 intersection, where the Holland River crosses Highway 400.
- A Husky Service Centre is located southeast of the Highway 400 and Simcoe County Road 88 interchange. Three pump islands with 5 pumps (diesel) were located on the north portion of the property and two pump islands with 4 pumps (gasoline) were located on the west portion of the property. One propane aboveground storage tank (AST) was located on the south portion of the property. Because of the presence of gasoline and diesel pumps and

associated underground storage tanks (USTs), there is the potential for subsurface impacts.

- A lumberyard is located on the corner of Highway 400 and the 9th Line overpass. Outside storage of treated lumber was observed and the potential exists for subsurface impacts related to the use of wood preserving chemicals.
- Cookstown outlet mall is located on the southeast corner of Highway 400 and Highway 89. Numerous commercial stores including Nike, Sony and Harvey's were observed to tenants in the Cookstown outlet mall.
- RV Warehouse sales office and yard area is located on the northeast corner of Highway 400 and Highway 89.
- A golf school, with playing facilities is located approximately 1 km north of the RV Warehouse property.
- An industrial complex, labeled as a chemical plant on the NTS map 31 D/5, Edition 6 is located approximately 500 m north of Innisfil Beach Road and there is the potential for subsurface impact related to the activities associated with the chemical plant.
- Molson Park is located at the Molson Park Drive exit from Highway 400. The former Molson Brewery is located north of Molson Park Drive and includes a filtration plant and sewage treatment lagoon. The potential exists for subsurface impact related to the activities associated with the former Molson Brewery.
- A Petro Canada service centre is located approximately 500 m north of the former Molson Brewery. Two pump islands with 4 pumps (gasoline) were located on the central portion of the property and one pump island with 1 pump (diesel) was located on the north portion of the property. Because of the presence of gasoline and diesel pumps and associated USTs, there is the potential for subsurface impacts.
- A Petro King gas station is located approximately 1 km north of the Petro Canada. Two pump islands with 4 pumps (gasoline and diesel) were located on the central portion of the property. Because of the presence of gasoline and diesel pumps and associated USTs, there is the potential for subsurface impacts.
- The Barrie Fairground is located approximately 750 m northwest of the Petro King. Livestock, likely horses appear to be boarded on the fairground property.
- Barrie Metals Ltd. is located approximately 1 km north of the Barrie Fairground. The potential exists for subsurface impact related to the activities associated with Barrie Metals Ltd.
- A lumberyard is located immediately north of Barrie Metals Ltd. Storage of lumber was observed and the potential exists for subsurface impacts related to the use of wood preserving chemicals.
- A Pioneer service centre is located approximately 500 m north of the lumberyard. Two pump islands with 4 pumps (gasoline and diesel) were located on the central portion of the property. Because of the presence of gasoline and diesel pumps and associated USTs, there is the potential for subsurface impacts.
- An Ontario Provincial Police (OPP) detachment is located approximately 1 km north of the Pioneer service centre.
- Flags Unlimited is located approximately 1 km north of the OPP detachment.

- Georgian College and Barrie General Hospital are located approximately 1 km to the northeast of Flags Unlimited.

West Side of ROW

- Commercial farming operations are located immediately north of the Highway 9 intersection, where the Holland River crosses Highway 400.
- Yogi's Camp, a camp ground, is located on the northwest corner of Highway 400 and Simcoe County Road 88.
- A Petro Canada service station is located on the northwest corner of Highway 400 and Highway 89. Three pump islands, each with two pumps, one pump island with one pump and fill and vent pipes were observed along the north side of the building and consequently there is the potential for subsurface impacts.
- Agricultural or vacant land is present from north of the Petro Canada to Molson Park Drive, located just south of Barrie.
- A Ministry of Transportation yard, with three salt domes is located approximately 500 m west of Highway 400 at Essa Street in Barrie.
- Canplas Industries Ltd. (Canplas) is located approximately 100 m south of Tiffin Street. Three ASTs were noted along the south side of the main building. Based on documentation provided to Golder Associates by the Ministry of the Environment (MOE) and the City of Barrie, it is understood that groundwater in the vicinity of the Canplas facility underneath Highway 400 is impacted with the chlorinated solvent trichloroethylene (TCE).
- A former Ontario Hydro yard is located approximately 750 m north of Canplas. Metal drums, transformers and hydro poles were observed in the yard and consequently there is the potential for subsurface impacts.
- A concrete plant is located to the north of the Ontario Hydro Yard. One AST was observed along the south side of the plant building.
- A Chrysler car dealership is located to the north of the cement plant. Servicing of automobiles is often conducted at automotive dealerships and consequently there is the potential for subsurface impacts.
- A Barrie Police Department detachment is located approximately 2 km north of the Chrysler dealership.
- Land adjacent to the ROW north of the Barrie Police Department detachment is primarily vacant or residential to the end of the study area at Highway 11.

6.0 ISSUES OF POTENTIAL ENVIRONMENTAL CONCERN

Based on the Waste, Contamination and Excess Material Assessment conducted on Highway 400 from Highway 9 northerly to Highway 11 in the Townships of King, Bradford-West Gwillimbury, Innisfil and Springwater, the primary properties of concern consist of active gas stations and industrial activity. No specific indications of issues of potential environmental concern were observed within the ROW, with the exception of the known TCE impacted groundwater plume under the Highway 400 ROW, in the vicinity of the Tiffin Street overpass in the City of Barrie.

Numerous active farming operations are present along the ROW. No specific issues of potential environmental concern were typically noted at the farm properties, however, farming operations typically include vehicle maintenance and re-fuelling, historical waste disposal and fertilizer, herbicide and pesticide storage. Where farm properties are to be acquired that include the farm buildings, (as opposed to fields) then the properties should be assessed on a case by case basis.

The following properties along the right of way are highly likely to have waste materials/contamination present:

East Side of ROW

- Commercial farming operations are located immediately north of the Highway 9 intersection, where the Holland River crosses Highway 400.
- A Husky Service Centre is located on the southeast corner of the Highway 400 and Simcoe County Road 88 intersection. Three pump islands with 5 pumps (diesel) were located on the north portion of the property and two pump islands with 4 pumps (gasoline) were located on the west portion of the property. One propane AST was located on the south portion of the property. Because of the presence of gasoline and diesel pumps and associated USTs, there is the potential for subsurface impacts.
- A lumberyard is located on the corner of Highway 400 and the 9th Line overpass. Storage of lumber was observed and the potential exists for subsurface impacts related to the use of wood preserving chemicals.
- An industrial complex, labeled as a chemical plant on the NTS map 31 D/5, Edition 6 is located approximately 500 m north of Innisfil Beach Road and there is the potential for subsurface impact related to the activities associated with the chemical plant.
- Molson Park is located at the Molson Park Drive exit from Highway 400. The former Molson Brewery is located north of Molson Park Drive and includes a filtration plant and sewage treatment lagoon. The potential exists for subsurface impact related to the activities associated with the former Molson Brewery.
- A Petro Canada service centre is located approximately 500 m north of the former Molson

Brewery. Two pump islands with 4 pumps (gasoline) were located on the central portion of the property and one pump island with 1 pump (diesel) was located on the north portion of the property. Because of the presence of gasoline and diesel pumps and associated USTs, there is the potential for subsurface impacts.

- A Petro King service centre is located approximately 1 km north of the Petro Canada. Two pump islands with 4 pumps (gasoline and diesel) were located on the central portion of the property. Because of the presence of gasoline and diesel pumps and associated USTs, there is the potential for subsurface impacts.
- The Barrie Fairground is located approximately 750 m northwest of the Petro King. Livestock, likely horses appear to be boarded on the fairground property.
- Barrie Metals Ltd. is located approximately 1 km north of the Barrie. The potential exists for subsurface impact related to the activities associated with Barrie Metals Ltd.
- A lumberyard is located immediately north of Barrie Metals Ltd. Storage of lumber was observed and the potential exists for subsurface impacts related to the use of wood preserving chemicals.
- A Pioneer service centre is located approximately 500 m north of the lumberyard. Two pump islands with 4 pumps (gasoline and diesel) were located on the central portion of the property. Because of the presence of gasoline and diesel pumps and associated USTs, there is the potential for subsurface impacts.

West Side of ROW

- Commercial farming operations are located immediately north of the Highway 9 intersection, where the Holland River crosses Highway 400.
- A Petro Canada service station is located on the northwest corner of Highway 400 and Highway 89. Three pump islands, each with two pumps, one pump island with one pump and fill and vent pipes were observed along the north side of the building and consequently there is the potential for subsurface impacts.
- Agricultural or vacant land is present from north of the Petro Canada to Molson Park Drive, located just south of Barrie.
- A Ministry of Transportation yard, with three salt domes is located approximately 500 m west of Highway 400 at Essa Street in Barrie.
- Canplas Industries Ltd. (Canplas) is located approximately 100 m south of Tiffin Street. Three ASTs were noted along the south side of the main building. Based on documentation provided to Golder Associates by the Ministry of the Environment (MOE) and the City of Barrie, it is understood that groundwater east of the Canplas facility underneath Highway 400 is impacted with the chlorinated solvent TCE.
- A former Ontario Hydro yard is located approximately 750 m north of Canplas. Metal drums, transformers and hydro poles were observed in the yard and consequently there is the potential for subsurface impacts.
- A cement plant is located to the north of the Ontario Hydro Yard. One AST was observed along the south side of the cement plant building.

- A Chrysler car dealership is located to the north of the cement plant. Servicing of automobiles is often conducted at automotive dealerships and consequently there is the potential for subsurface impacts.

7.0 WASTE MATERIAL ASSESSMENT AND MANAGEMENT

As per MTO objectives, to the extent possible, this project should strive for zero waste generation. Where recommendations for re-use of materials are made below, it should be noted that these recommendations are made from an environmental perspective only. The geotechnical and structural implications of the re-use of materials should be reviewed by individuals with the appropriate expertise in those areas. All materials should be managed in accordance with OPSS 180.

Based on the results of the site reconnaissance it is expected that wastes which will be generated by the project would generally consist of:

- RAP from milling of existing asphalt surfaces;
- Concrete, likely reinforced, from the removal of box culvert and bridge structures;
- Swamp material from excavation in wetland areas;
- Manufactures wood waste from guard rail and the like removals;
- Natural wood waste from clearing/grubbing operations;
- Scrap metal such as wire, corrugated steel pipe, and bridge guardrails.

RAP can be re-used on site for a variety of purposes on site as part of the construction activities, including, but not necessarily limited to, shoulder treatments, general fill and sub-grade fill. Non-reinforced concrete can be broken up and re-used within the project limits. Manufactured wood waste will require off-site disposal at licensed receiving facilities. Natural wood waste may be left on site within the right of way. Scrap metal should be collected for re-cycling at an off-site receiving facility.

Depending on the design selected for the highway upgrade it is possible that some contaminated materials will be encountered during construction, which will result in the generation of waste. Materials including soil and groundwater, which are chemically impacted above the applicable MOE *"Guideline for Use at Contaminated Sites in Ontario"* criteria and which are encountered in the course of the construction activities should be managed off-site in accordance with current Ontario waste management regulations (i.e. O.Reg 558 or equivalent). Appropriate approvals should be sought for known contaminated materials, which are to be left or managed within the ROW.

As per MTO's terms of reference, the following waste types were specifically considered during the site reconnaissance portion of the assessment:

- Asbestos waste (generated from the removal of asbestos containing electrical conduit insulation in concrete bridge decks);
- Spent abrasive blast cleaning material from the removal of concrete, bridge structure paint and other coating materials;
- Spent liquid and sludge material from hydraulic blast cleaning operations;
- Slag or asbestos containing asphalt pavement; and,
- PCBs (e.g., in electrical equipment to be removed such as transformers, ballasts and capacitors).

Extensive demolition and rehabilitation of structures is not expected for this project, with the potential exception of the numerous bridge structure over the Highway 400 right of way and some concrete box and steel culverts. The bridge structures are essentially a concrete structure with a painted steel railing. Depending on the extent of rehabilitation or demolition, the following wastes could be generated:

- Asphalt;
- Reinforced concrete;
- Paint and concrete residues from hydraulic demotion or abrasive cleaning operations; and,
- Scrap metal.

The wastes to be generated should be reviewed in detail, once the scope of work relating to the bridges is finalized. As a minimum, the painted metal surfaces should be tested for lead content so that appropriate handling, health and safety and disposal protocols can be followed. Paint and other residues from any abrasive cleaning of the steel railings should be contained and collected for off-site disposal as waste, regardless of lead content.

No evidence of slag or asbestos containing asphalt pavement was observed within the project limits.

8.0 LIMITATIONS AND USE OF REPORT

This report was prepared for the exclusive use of the Ontario Ministry of Transportation. This report is based on data and information collected during the Waste and Contamination Assessment and Secondary Source Groundwater Investigation of the site conducted by Golder Associates Ltd. and is based solely on site conditions encountered at the time of the site visit in December 2000 and March 2001 supplemented by historical information and geological data obtained by Golder Associates, as described in this report.

In evaluating the site, Golder Associates Ltd. has relied in good faith on information provided by others. We accept no responsibility for any deficiency, misstatements or inaccuracies contained in this report as a result of omission, misinterpretations or fraudulent acts of the persons interviewed. Golder Associates Ltd. accepts no responsibility for any reduction in property value, either real or perceived, as a result of the reporting of factual information herein.

It should be noted that the results of an investigation of this nature should, in no way, be construed as a warranty that the site is free from any and all contamination from past or current practices. This assessment was carried out using existing historical information as available from various agencies and no assurance is made regarding the accuracy or completeness of this information. No sampling of soil, groundwater, air emissions or gas was conducted.

This investigation has included a visual assessment of adjacent properties from public areas within the Highway 400 ROW. Based on this visual assessment Golder Associates has indicated its opinion with respect to the relative potential for contamination or waste materials to be present at adjacent properties. This opinion should not be interpreted to represent a warranty or guarantee that contamination or waste materials are either present or absent at any of the adjacent properties.

If new information is discovered during future work, including excavations, borings or other studies, Golder Associates Ltd. should be requested to re-evaluate the conclusions presented in this report and to provide amendments as required.

Yours truly,

GOLDER ASSOCIATES LTD.

DRAFT

S. D. Lytle, B.Sc.
Associate, Environmental Division

DRAFT

T. A. McIelwain, P.Eng.
Principal, Environmental Division

MLC/TAM/SDL: dla/mlc

9.0 REFERENCES

Chapman, L.J., and Putnam, D.F., 1984, The Physiography of Southern Ontario, Ontario Geological Survey, Special Volume 2, p. 270. Accompanied by Map P.2715, scale 1:600,000.

Ontario Ministry of Environment, 1999, Ontario Inventory of PCB Storage Sites.

Ontario Ministry of Environment, June 1991, Waste Disposal Site Inventory.

Ontario Ministry of Environment, April 1987, Inventory of Coal Gasification Plant Waste Sites in Ontario, Volumes I & II.

Ontario Ministry of Environment, November 1988, Inventory of Industrial Sites Producing or using Coal Tar and Related Tars in Ontario.

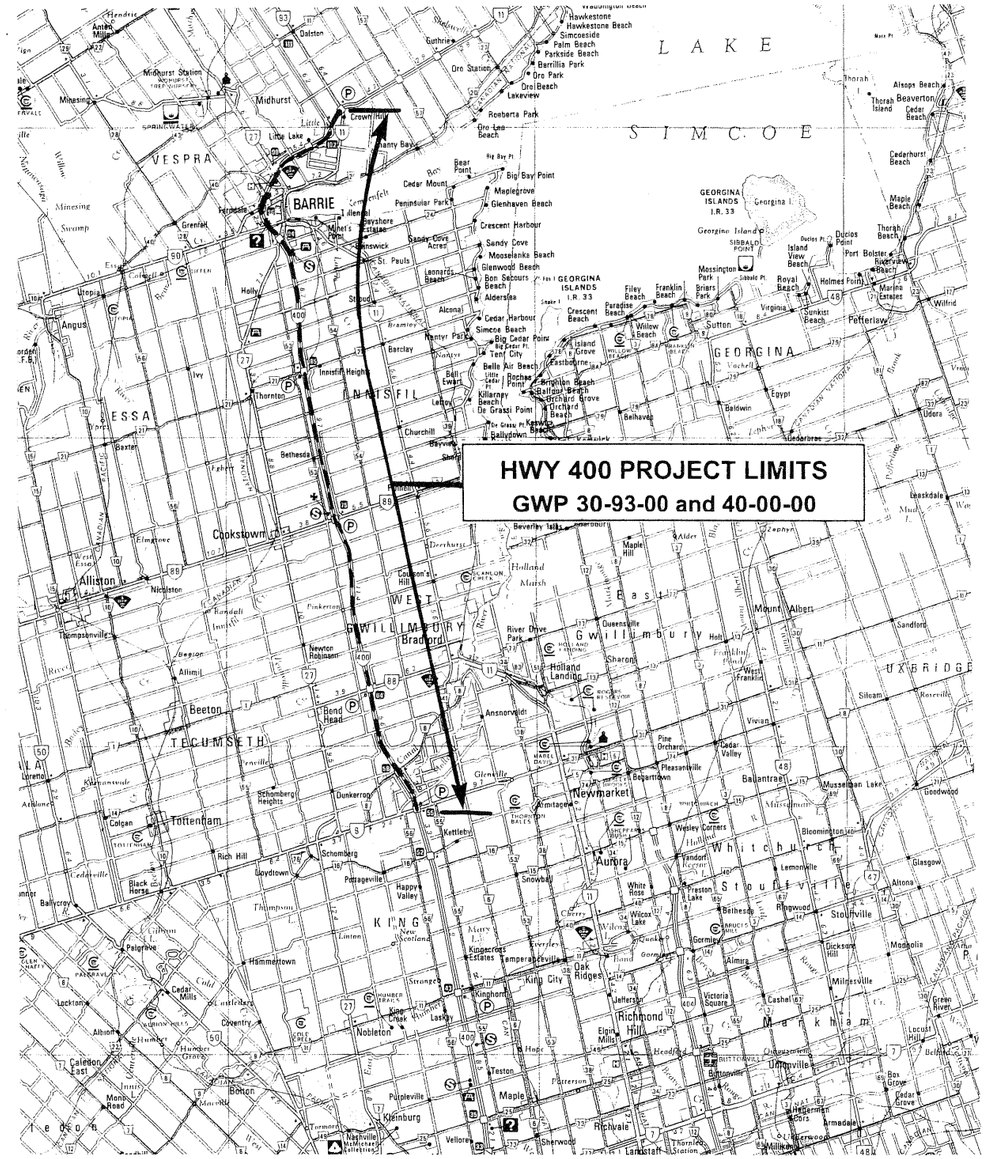
Ministry of Northern Development and Mines, Map 2556, Quaternary Geology of Ontario, Southern Sheet, 1991 (Scale 1:1,000,000).

Ministry of Northern Development and Mines, Map 2544, Bedrock Geology of Ontario, Southern Sheet, 1991 (Scale 1:1,000,000).

KEY PLAN

Highway 400 ROW from Highway 9 Northerly to Highway 11

FIGURE 1



Reference: Ontario Transportation Map Series

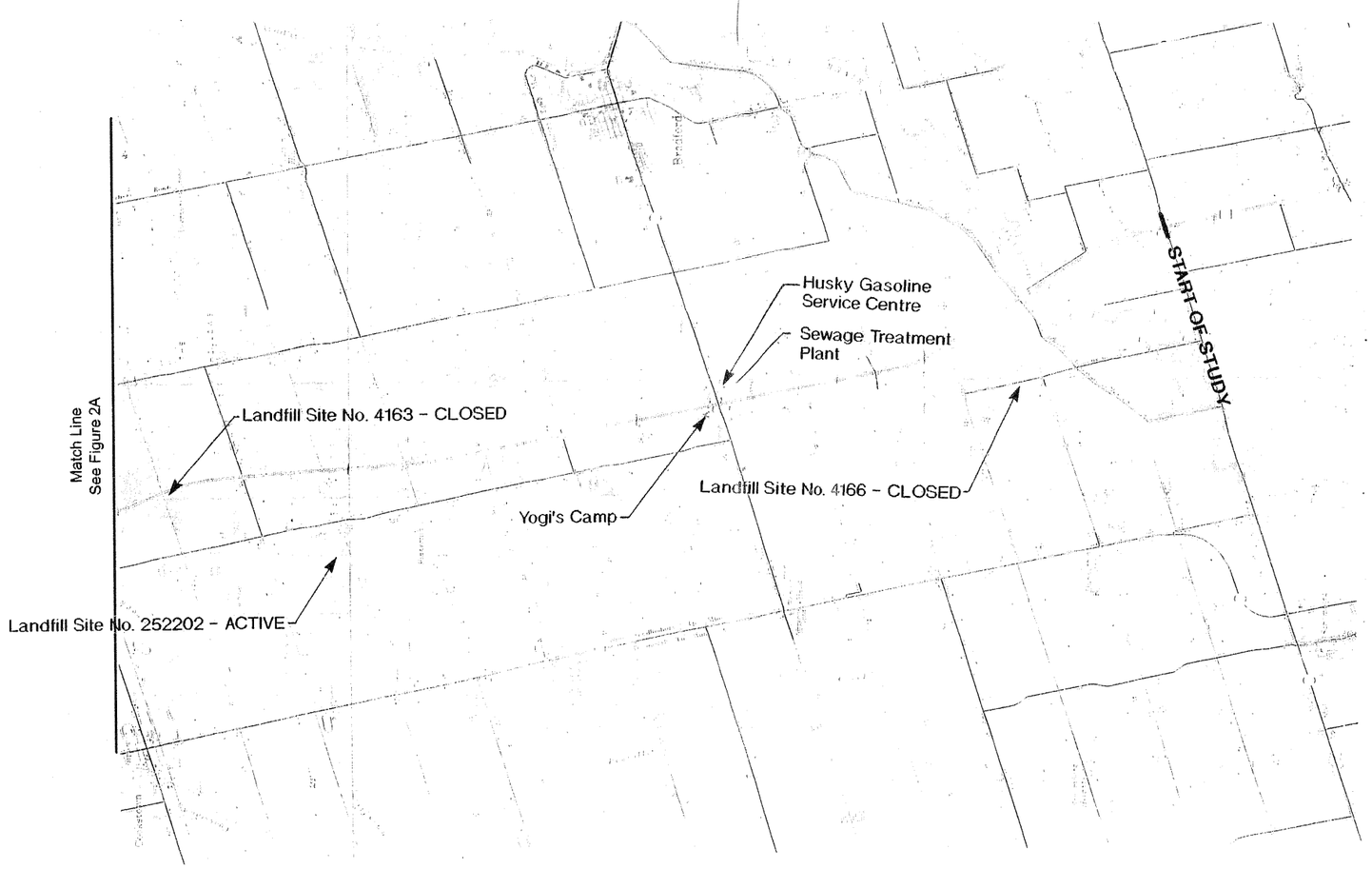
Date March 2001 Drawn ph

Project 001-8393 **Golder Associates** Checked

SITE LOCATION PLAN

Highway 400 ROW from Highway 9 Northerly to Highway 11

FIGURE 2A



REFERENCE

Energy, Mines and Resources Canada, Surveys and Mapping Branch, "Alliston, 31 D/4" Edition 7, and "Barrie 31 D/5" Edition 6, 1986 topographic maps

SCALE

1 : 100,000

ALL LOCATIONS ARE APPROXIMATE

Date March 2001

Project 001-8393

Golder Associates

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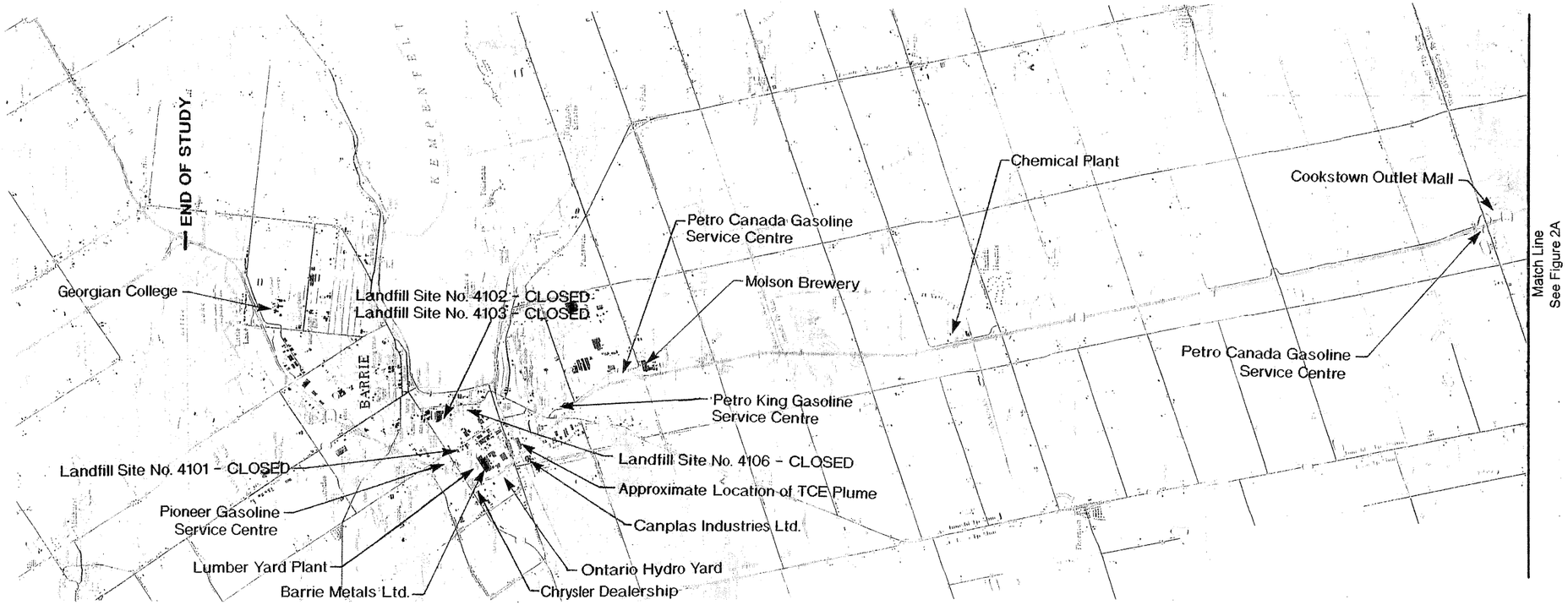
SITE LOCATION PLAN

Highway 400 ROW from Highway 9 Northerly to Highway 11

FIGURE 2B



REFERENCE

Energy, Mines and Resources Canada, Surveys and Mapping Branch, "Alliston, 31 D/4" Edition 7, and "Barrie 31 D/5" Edition 6, 1986 topographic maps

SCALE

1 : 100,000

ALL LOCATIONS ARE APPROXIMATE

Date March 2001

Project 001-8393

Golder Associates

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APPENDIX A

REGULATORY RESPONSES

TOWNSHIP OF SPRINGWATER

Simcoe County Administration Centre, Midhurst, Ontario L0L 1X0
Phone No. (705) 728-4784 Fax No. (705) 728-6957
Website: www.twp.springwater.on.ca E-mail: springwater@simcoe.net

March 29, 2001

Diana Addley
Golder Associates Ltd.
100 Scotia Court
Whitby, On
L1N 8Y6

GOLDER ASSOCIATES
APR - 2 2001
RECEIVED - WHITBY

Dear Ms Addley:

RE: SPRINGWATER TOWNSHIP ENVIRONMENTAL INFORMATION HIGHWAY 400

This is in response to your request for environmental information pertaining to lands abutting Highway 400 in our Township. The area is farm land and open space, and we have no files with information that would help you out. I have been with the Township for 22 years and to my knowledge there has been no filling or waste disposal in the area. We have no discharge of storm sewers or municipal drains in the area in question.

If I can be of any further help, please do not hesitate to contact my office.

Yours truly,

Don Priest

Donald H. Priest
Public Works Superintendent

DHP/jl

CITY HALL
70 COLLIER STREET
P.O. Box 400
Barrie, Ontario
L4M 4T5

THE PEOPLE ARE THE CITY

THE CORPORATION OF THE CITY OF BARRIE
City Clerk's Office
"Committed to Total Service Excellence"

DIRECT LINE (705) 739-4204
TEL. (705) 726-4242
FAX. (705) 739-4243

February 9, 2001

File: A17-2000/63

Ms. Diana Addley
Golder Associates Ltd.
100 Scotia court
Whitby, Ontario
L1N 8Y6

Freedom of Information Access Request re: Highway 400 – E.S.A.

Dear Ms. Addley:

Thank you for your request under the *Municipal Freedom of Information and Protection of Privacy Ac.* Your complete request was received in the City Clerk's Office on January 11, 2001.

As a result of our preliminary review of the requested records it is expected that your request will take 9.25 hours of search/review/consultation time* and produce approximately 525 pages of information resulting in a fee of $390.00.

Under *the Municipal Freedom of Information and Protection of Privacy Act*, the Corporation is allowed to request a deposit of 50% in advance of the work being commenced.

Accordingly, we request $195.00 to be submitted prior to our proceeding with your request. Payment can be made at City Hall, Clerk's Department, 70 Collier Street, Barrie, Ontario from 8:30 a.m. to 4:30 p.m., Monday to Friday. Please note that the legislated 30-day time limit for response to your request is placed on hold until we receive the requested deposit fee.

In accordance with Section 22, Subsection 1, Paragraph b, Sub-paragraph (iv) and Section 45, Subsection (5), of the Act, you may request that this fee be reviewed by the Information and Privacy Commissioner of Ontario at: 80 Bloor Street West, Suite 1700, Toronto, Ontario, M5S 2V1, 1-800-387-0073.

If you decide to launch an appeal to the Information and Privacy Commissioner, please provide the Commissioner's office with the following: the request number which is 2000/63; a copy of this fee estimate letter; a copy of your original request for information and an appeal fee (a certified cheque or a money order, made payable to the Minister of Finance). Your request is for general information, so the appeal fee is $25.00.

Please note that you have thirty (30) days from the receipt of this letter to request a review by the Information and Privacy Commissioner.

Sincerely,

John Craig
City Clerk

*Based on $7.50 per 15 minutes and $20. per copy as per Ontario Regulation 517/90

Kan 6.

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INTERNATIONAL WATER CONSULTANTS LTD.
Ground Water Engineering
BARRIE BELOEIL, P.Q. SASKATOON
342 Bayview Dr., Post Office Box 310
Barrie, Ontario, Canada L4M 4T5
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October 11, 2000

The City of Barrie
P.O. Box 400
249 Bradford Street
Barrie, Ontario
L4M 4T5

Attention: Ronald Gemmell, C.E.T.
Manager Environmental Services

Reference: **Interim Report**
Wood Street Well
Water Quality Investigation

Please find enclosed 3 copies of our Interim Report on Wood Street Well Water Quality Investigation.

If you have any questions, please do not hesitate to call.

Yours truly,

Gary A. Kuehl, P. Geo. (Sask)
GAK/lw

cc: Keith Wice, City of Barrie

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THE CITY OF BARRIE
WOOD STREET WELL
WATER QUALITY INVESTIGATION
INTERIM REPORT
2000

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THE CITY OF BARRIE
WOOD STREET WELL
WATER QUALITY INVESTIGATION
INTERIM REPORT

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THE CITY OF BARRIE
WOOD STREET WELL
WATER QUALITY INVESTIGATION
INTERIM REPORT

INTRODUCTION

This report documents the findings to date of a Phase 1 Investigation aimed at assessing the possibility of VOC contaminant sources in the area of the Wood Street Well, other than the known plume identified in the Clayton Environmental Report of January 1998. This latter report identified a plume of mainly Trichloroethylene (TCE) in an upper fine sand aquifer, extending in an easterly direction from Highway 400 as shown on Drawing B-00217. It is noted that the extent of the plume was largely undetermined.

PROCEDURES

The Phase 1 program included drilling, soil sampling, installation of multi-level monitoring wells within the upper fine sand aquifer zone at 6 locations, followed by purging, sampling and analysis for selected Volatile Organic Components (VOC) The monitoring well locations were selected to assess the water quality in directions away from the known plume. The monitoring well locations are shown on Drawing B-00217 and the monitoring well logs and construction details are contained in Appendix A. All monitoring wells consist of nominal 50 mm diameter PVC casing with slotted screen. The annular spaces around the casings were sealed using bentonite grout and pellets placed from near the top of screens to surface. The shallower screen levels were installed in separate holes and protective steel casings installed over each monitoring well.

Following monitoring well construction, each well was developed and purged using dedicated Waterra samplers. Samples were then collected and delivered to the City for submission to Maxxam Laboratory for analysis. Two sets of samples were collected on August 8 and 23, 2000. Sampling included TW 1/66 within the lower aquifer at the Wood Street Well and TW 1/65 on Morrow Road.

RESULTS TO DATE

The results of water analyses are contained in Appendix B. The water analyses show non-detect for the VOC scan at most locations except Monitoring Wells (MW's) 1 and 2 where low to high levels of TCE were detected and MW 3 where a trace level of Dichloromethane and Chloroform was detected. MW's 1 and 2 are located north of the Wood Street Well along the railway spur as shown on Drawing B-00217. In addition to the TCE detected at these locations, trace levels of Tetrachloroethylene, and low levels of cis-1, 2 Dichloroethylene were detected from the lower screen setting. The highest TCE level detected was 4400 ug/L from the lower screen setting of MW 2.

The TCE levels detected from the lower screen settings at the 5 monitoring wells recently installed, together with Clayton Environmental results from 1998 are shown on Drawing B-00217.

DISCUSSION

The known plume identified by Clayton Environmental consists mainly of TCE, which is a Dense Nonaqueous Phase Liquid (DNAPL). Migration of such contaminants within a groundwater system can involve a dissolved phase, which migrates with the groundwater flow system and a nonaqueouse phase, which sinks vertically to a confining layer and then migrates down slope along the surface of the confining layer. Migration of this component can therefore be unrelated to the groundwater flow direction. The investigations to date have identified a dissolved phase plume migrating within the flow system of an upper fine sand aquifer.

The TCE concentrations shown on Drawing B-00217, demonstrate that there is no evidence of TCE contamination to the south or east of the Wood Street Well. All monitoring wells in these directions show non-detect for the VOC scan except for a trace level of chloroform and dichloromethane at MW 3. However, MW's 1 and 2 to the north show TCE contaminant

The level of TCE detected at MW's 1 and 2, the type of associated compounds identified, and the implied groundwater flow direction shown on Drawing B-00218 suggest that the plume identified by Clayton Environmental extends from Highway 400 to MW 2 and beyond. However, additional drilling and monitoring well installation are recommended to show continuity of the TCE plume and provide further support that there is not a secondary contaminate source contributing to the MW 2 location. At this stage, we suggest installation of an additional 4 to 6 monitoring wells (2 levels), and surveying of all monitoring wells to accurately determine well locations and measuring point elevations in order to confirm the groundwater flow system. It would also be helpful to collect and analyze samples from the MW's installed and sampled in 1998 by Clayton Environmental to establish current VOC levels at these locations.

Gary A. Kuehl, P. Geo. (Sask)
GAK/lw

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DRAWINGS

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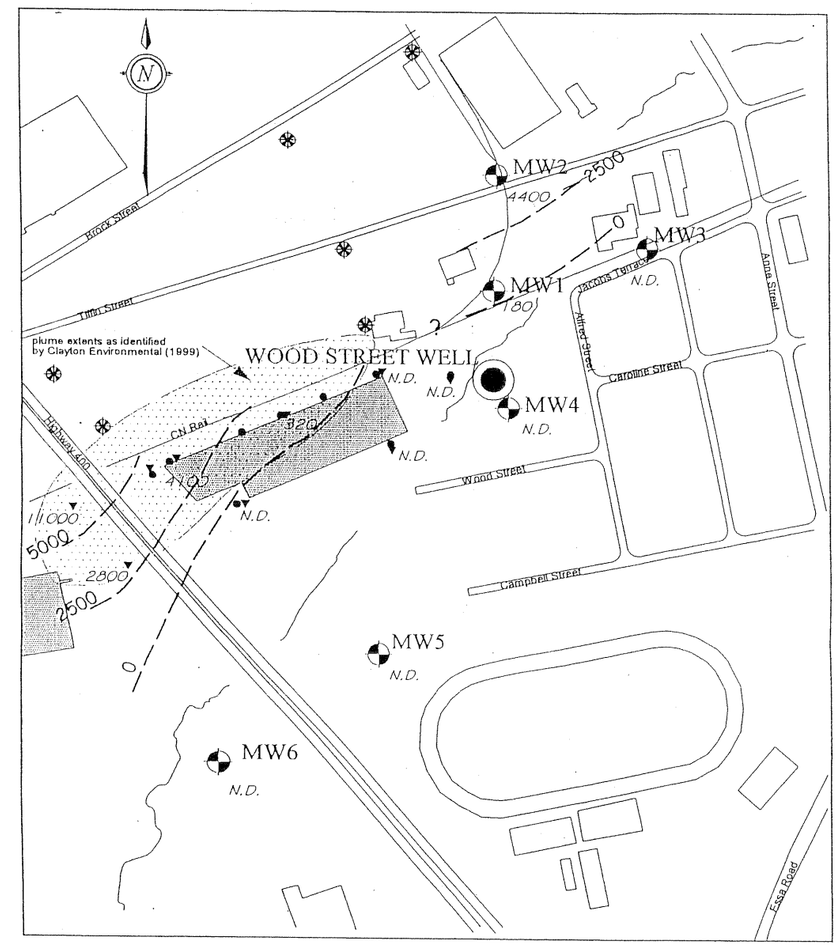

2500 Implied TCE Concentrations (ug/L) - Upper Aquifer

N.D. Monitoring Well in Upper Aquifer with TCE results in ug/L (August 8, 2000)

Proposed Monitoring Well Location

Installations by Clayton Environmental
- Shallow (< 5m)
- Deep (<14m)

0 100m 200m

NO.	WAS	BY	DATE

CITY OF BARRIE
WOOD STREET WELL No. 6
TCE CONCENTRATIONS - UPPER AQUIFER

International Water Consultants Ltd.
MONTREAL - BARRIE - SASKATOON

DRN: T. Brown
CKD: G. Kuehl

DRAWING NO. B00217

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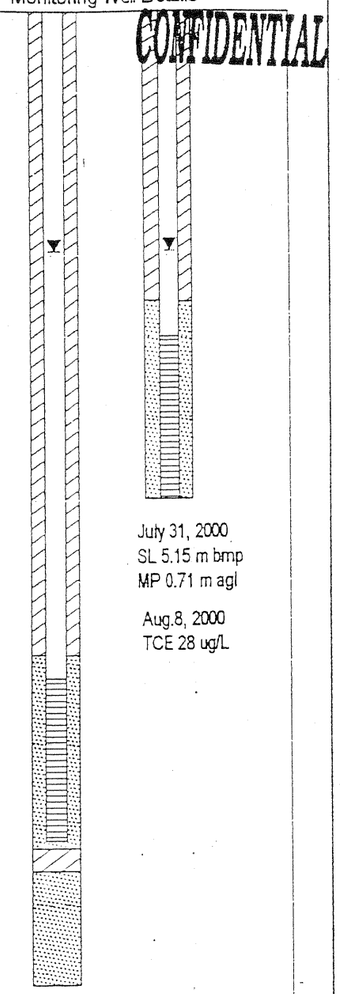
Lithology		Field Comments
Sand fine to medium red		Wells constructed with 2" sch 40 PVC flush joint casing and screen
	6	
Sand fine some medium light brown		Screened interval stabilized with silica sand
	14	
Sand fine brown		Casing sealed in place with bentonite grout and pellets
	16	
Sand fine to very fine		
	18	
Sand fine to very fine silty		
	20	
Sand fine to medium		
	22	
Sand fine		
	30	
Sand fine to very fine minor silt grey		
	36	
Sand fine		
	52	
Silt some sand very fine and clay		
	60	

Feet: 0, 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, 70

Metres: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22

Monitoring Well Details



July 31, 2000
SL 5.15 m bmp
MP 0.71 m agl

Aug.8, 2000
TCE 28 ug/L

July 31, 2000
SL 5.27 m bmp
MP 0.79 m agl

Aug. 8, 2000
TCE 180 ug/L

CONFIDENTIAL

DRILLING METHOD: Hollow Stem Auger
BOREHOLE DIAMETER: 8 inch
DRILLER: Lantech Drilling Services Inc.
DATE: July 26, 2000

CITY OF BARRIE
WOOD STREET UPPER AQUIFER MONITORING WELLS
MW 1

IWC INTERNATIONAL WATER CONSULTANTS LTD.
MONTREAL BARRIE SASKATOON

DRN:
CKD: G. Kuehl
DRAWING No. A00209

Lithology		Field Comments
Fill sand		Wells constructed with 2" sch 40 PVC flush joint casing and screen
Sand fine brown	4	
Sand fine to medium with 2" streak of sand fine silty @ 11'	10	Screened interval stabilized with silica sand
Sand fine with 4" streak of clayeysilt @ 14'	14	
Sand fine to medium brown	18	Casing sealed in place with bentonite grout and pellets
Sand fine brown	27	
Silt some very sand and minor clay grey	29	
Sand fine some medium grey	31	
Sand very fine to fine some silt grey	45	
Silt some clay grey	50	
Clay and silt grey odd pebble	51	
	60	

Feet: 0, 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, 70

Metres: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22

Monitoring Well Details

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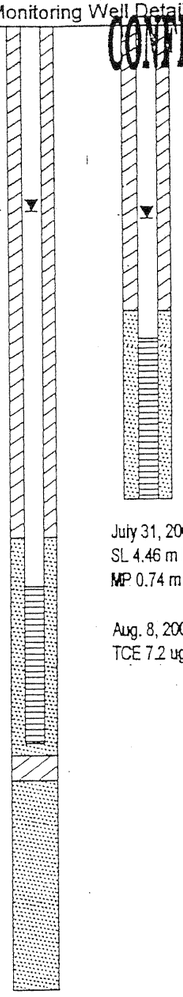

July 31, 2000
SL 4.46 m bmp
MP 0.74 m agl

Aug. 8, 2000
TCE 7.2 ug/L

July 31, 2000
SL 3.90 m bmp
MP 0.36 m agl

Aug. 8, 2000
TCE 4400 ug/L

DRILLING METHOD: Hollow Stem Auger
BOREHOLE DIAMETER: 8 inch
DRILLER: Lantech Drilling Services Inc.
DATE: July 26, 2000

CITY OF BARRIE
WOOD STREET UPPER AQUIFER MONITORING WELLS
MW2

INTERNATIONAL WATER CONSULTANTS LTD.
MONTREAL BARRIE SASKATOON

DRN:
CKD: G. Kuehl
DRAWING No. A00210

Lithology		Field Comments
Sand fine brown		Wells constructed with 2" sch 40 PVC flush joint casing and screen
Sand fine some medium brown	14	Screened interval stabilized with silica sand
Sand fine to very fine brown	18	Casing sealed in place with bentonite grout and pellets
Sand fine some medium brown	24	
Sand fine minor silt brown	36	
Sand fine some medium brown	42	
Sand very fine minor silt grey	69	
	70	

Feet: 0, 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, 70

Metres: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22

Monitoring Well Details

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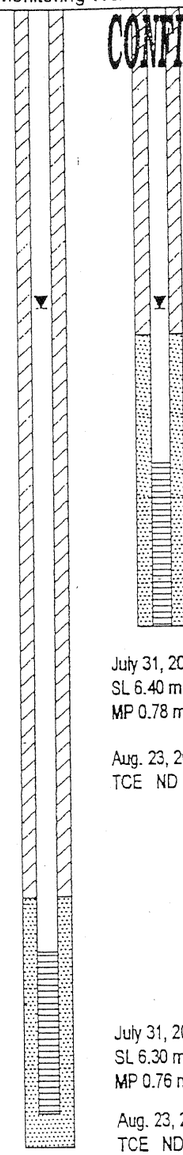

July 31, 2000
SL 6.40 m bmp
MP 0.78 m agl

Aug. 23, 2000
TCE ND

July 31, 2000
SL 6.30 m bmp
MP 0.76 m agl

Aug. 23, 2000
TCE ND

DRILLING METHOD: Hollow Stem Auger
BOREHOLE DIAMETER: 8 inch
DRILLER: Lantech Drilling Services Inc.
DATE: July 26, 2000

CITY OF BARRIE
WOOD STREET UPPER AQUIFER MONITORING WELLS
MW 3

INTERNATIONAL WATER CONSULTANTS LTD.
MONTREAL BARRIE SASKATOON

DRN:
CKD: G. Kuehl
DRAWING No. A00211

Lithology		Field Comments
Sand fine brown		Wells constructed with 2" sch 40 PVC flush joint casing and screen
Sand fine to medium brown	5	
odd stone	6.5	Screened interval stabilized with silica sand
Sand fine light brown		
Sand fine to medium brown	11	
Sand fine light brown odd stone	12	Casing sealed in place with bentonite grout and pellets
Sand fine some silt grey brown	15	
Silt and clay odd stone grey	22	
Sand fine some silt grey	27	
Silt some fine sand and clay grey	37	
Clay and silt minor fine sand	42	
Sand fine to medium grey	47	
Sand fine and silt grey	55	
Sand fine some medium grey	56	
	61	

Feet: 0, 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, 70

Metres: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22

Monitoring Well Details

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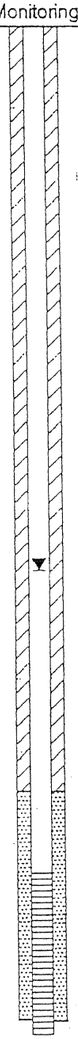

July 31. 2000
SL 5.06 m bmp
MP 0.68 m agl

Aug. 8, 2000
TCE ND

July 31. 2000
SL 10.95 m bmp
MP 0.73 m agl

Aug. 8, 2000
TCE ND

DRILLING METHOD: Hollow Stem Auger
BOREHOLE DIAMETER: 8 inch
DRILLER: Lantech Drilling Services Inc.
DATE: July 26, 2000

CITY OF BARRIE
WOOD STREET UPPER AQUIFER MONITORING WELLS
MW 4

INTERNATIONAL WATER CONSULTANTS LTD.
MONTREAL BARRIE SASKATOON

DRN:
CKD: G. Kuehl
DRAWING No. A00212

Lithology		Field Comments
Topsoil		Wells constructed with 2" sch 40 PVC flush joint casing and screen
Sand fine brown		
Sand fine silty brown	10	Screened interval stabilized with silica sand
Sand fine to medium brown	12	
Sand fine grey	33	Casing sealed in place with bentonite grout and pellets
Sand fine some silt grey	38	
Sand fine silty grey	43	
	65	

Feet: 0, 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, 70

Metres: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22

Monitoring Well Details

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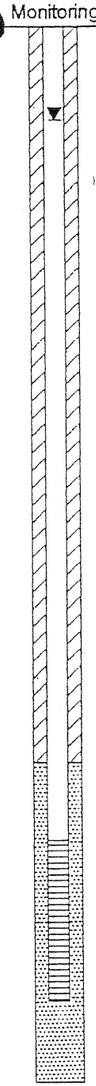

July 31, 2000
SL 3.32 m bmp
MP 0.79 m agl

Aug. 8, 2000
TCE ND

July 31, 2000
SL 3.50 m bmp
MP 0.76 m agl

Aug. 8, 2000
TCE ND

DRILLING METHOD: Hollow Stem Auger
BOREHOLE DIAMETER: 8 inch
DRILLER: Lantech Drilling Services Inc.
DATE: July 26, 2000

CITY OF BARRIE
WOOD STREET UPPER AQUIFER MONITORING WELLS
MW 5

INTERNATIONAL WATER CONSULTANTS LTD.
MONTREAL BARRIE SASKATOON

DRN:
CKD: G..Kuehl
DRAWING No. A00213

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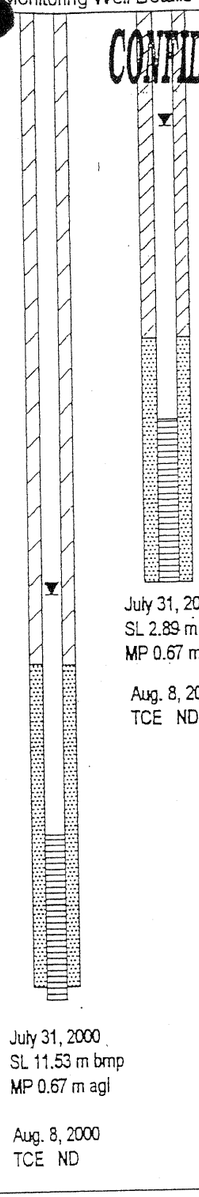
APPENDIX B

Lithology		Field Comments
Topsoil	1	Wells constructed with 2" sch 40 PVC flush joint casing and screen
Sand very fine silty brown	5	
Sand fine silty	14.5	Screened interval stabilized with silica sand
Sand fine silty minor clay	16	Casing sealed in place with bentonite grout and pellets
Sand fine silty grey	24	
Sand fine silty minor clay grey	28	
Clay silty grey	32	
Clay grey	36	
Clay minor silt grey	40	
Clay grey	46	
Sand fine silty grey	60	

Feet: 0, 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, 70

Metres: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22

Monitoring Well Details



July 31, 2000
SL 2.89 m bmp
MP 0.67 m agl

Aug. 8, 2000
TCE ND

July 31, 2000
SL 11.53 m bmp
MP 0.67 m agl

Aug. 8, 2000
TCE ND

DRILLING METHOD: Hollow Stem Auger
BOREHOLE DIAMETER: 8 inch
DRILLER: Lantech Drilling Services Inc.
DATE: July 26, 2000

CITY OF BARRIE
WOOD STREET UPPER AQUIFER MONITORING WELLS
MW 6

INTERNATIONAL WATER CONSULTANTS LTD.
MONTREAL BARRIE SASKATOON

DRN:
CKD: G. Kuehl
DRAWING No. A00214

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Maxxam Analytics Inc

REPORT DATE: 2000/08/17

MAXXAM JOB #: A015229

VOLATILE ORGANICS (SELECTED) COMPOUNDS IN LIQUID

Maxxam ID		428956		428957	428958	428959	
COC Number		25528		25528	25528	25528	
Sampling Date		2000/08/08		2000/08/08	2000/08/08	2000/08/08	
Parameter	Units	MW 2 D	MDL	MW 3 S	MW 3 D	MW 4 S	MDL
1,2-Dichlorobenzene	ug/L	ND	0.5	ND	ND	ND	0.5
1,2-Dichloroethane	ug/L	ND	0.5	ND	ND	ND	0.5
cis-1,2-Dichloroethylene	ug/L	34	1	ND	ND	ND	1
1,4-Dichlorobenzene	ug/L	ND	0.5	ND	ND	ND	0.5
Benzene	ug/L	ND	0.5	ND	ND	ND	0.5
Bromodichloromethane	ug/L	ND	0.2	ND	ND	ND	0.2
Bromoform	ug/L	ND	0.2	ND	ND	ND	0.2
Carbon Tetrachloride	ug/L	ND	1	ND	ND	ND	1
Chlorobenzene	ug/L	ND	0.5	ND	ND	ND	0.5
Chloroform	ug/L	ND	0.2	0.9	ND	ND	0.2
Dibromochloromethane	ug/L	ND	0.2	ND	ND	ND	0.2
Dichloromethane(Methylene Chloride)	ug/L	ND	1	ND	ND	ND	1
Ethylbenzene	ug/L	ND	0.5	ND	ND	ND	0.5
Tetrachloroethylene	ug/L	0.7	0.5	ND	ND	ND	0.5
Toluene	ug/L	ND	0.5	ND	ND	ND	0.5
Trichloroethylene	ug/L	4400	50	ND	ND	ND	0.5
Vinyl Chloride	ug/L	6.5	0.5	ND	ND	ND	0.5
o-Xylene	ug/L	ND	0.5	ND	ND	ND	0.5
p+m-Xylene	ug/L	ND	0.5	ND	ND	ND	0.5

Recovery of Surrogates
(%)

Parameter	MW 2 D	MW 3 S	MW 3 D	MW 4 S
4-Bromofluorobenzene	91	88	92	92
D4-1,2-Dichloroethane	109	86	93	105
D8-Toluene	99	94	99	95

ND = Not detected
MDL = METHOD DETECTION LIMIT

ANKE HOLTKAMP

CONFIDENTIAL

Maxxam Analytics Inc

REPORT DATE: 2000/08/17

MAXXAM JOB #: A015229

VOLATILE ORGANICS (SELECTED) COMPOUNDS IN LIQUID

Maxxam ID		428960	428961	428962	428963	428964	
COC Number		25528	25528	25528	25528	25528	
Sampling Date		2000/08/08	2000/08/08	2000/08/08	2000/08/08	2000/08/08	
Parameter	Units	MW 4 D	MW 5 S	MW 5 D	MW 6 S	MW 6 D	MDL
1,2-Dichlorobenzene	ug/L	ND	ND	ND	ND	ND	0.5
1,2-Dichloroethane	ug/L	ND	ND	ND	ND	ND	0.5
cis-1,2-Dichloroethylene	ug/L	ND	ND	ND	ND	ND	1
1,4-Dichlorobenzene	ug/L	ND	ND	ND	ND	ND	0.5
Benzene	ug/L	ND	ND	ND	ND	ND	0.5
Bromodichloromethane	ug/L	ND	ND	ND	ND	ND	0.2
Bromoform	ug/L	ND	ND	ND	ND	ND	0.2
Carbon Tetrachloride	ug/L	ND	ND	ND	ND	ND	1
Chlorobenzene	ug/L	ND	ND	ND	ND	ND	0.5
Chloroform	ug/L	ND	ND	ND	ND	ND	0.2
Dibromochloromethane	ug/L	ND	ND	ND	ND	ND	0.2
Dichloromethane(Methylene Chloride)	ug/L	ND	ND	ND	ND	ND	1
Ethylbenzene	ug/L	ND	ND	ND	ND	ND	0.5
Tetrachloroethylene	ug/L	ND	ND	ND	ND	ND	0.5
Toluene	ug/L	ND	ND	ND	ND	ND	0.5
Trichloroethylene	ug/L	ND	ND	ND	ND	ND	0.5
Vinyl Chloride	ug/L	ND	ND	ND	ND	ND	0.5
o-Xylene	ug/L	ND	ND	ND	ND	ND	0.5
p+m-Xylene	ug/L	ND	ND	ND	ND	ND	0.5

Recovery of Surrogates
(%)

Parameter	MW 4 D	MW 5 S	MW 5 D	MW 6 S	MW 6 D
4-Bromofluorobenzene	89	90	95	93	90
D4-1,2-Dichloroethane	102	94	97	93	96
D8-Toluene	98	96	97	97	97

ND = Not detected
MDL = METHOD DETECTION LIMIT

ANKE HOLTKAMP

Maxxam Analytics Inc

CONFIDENTIAL

REPORT DATE: 2000/08/17

MAXXAM JOB #: A015229

VOLATILE ORGANICS (SELECTED) COMPOUNDS IN LIQUID

Maxxam ID		428965	428966		
COC Number		25528	25528		
Sampling Date		2000/08/08	2000/08/08		
Parameter	Units	TW 1/65	WOOD ST TW	METHOD BLANK	MDL
1,2-Dichlorobenzene	ug/L	ND	ND	ND	0.5
1,2-Dichloroethane	ug/L	ND	ND	ND	0.5
cis-1,2-Dichloroethylene	ug/L	ND	ND	ND	1
1,4-Dichlorobenzene	ug/L	ND	ND	ND	0.5
Benzene	ug/L	ND	ND	ND	0.5
Bromodichloromethane	ug/L	ND	ND	ND	0.2
Bromoform	ug/L	ND	ND	ND	0.2
Carbon Tetrachloride	ug/L	ND	ND	ND	1
Chlorobenzene	ug/L	ND	ND	ND	0.5
Chloroform	ug/L	ND	ND	ND	0.2
Dibromochloromethane	ug/L	ND	ND	ND	0.2
Dichloromethane(Methylene Chloride)	ug/L	ND	ND	ND	1
Ethylbenzene	ug/L	ND	ND	ND	0.5
Tetrachloroethylene	ug/L	ND	ND	ND	0.5
Toluene	ug/L	ND	ND	ND	0.5
Trichloroethylene	ug/L	ND	7.7	ND	0.5
Vinyl Chloride	ug/L	ND	ND	ND	0.5
o-Xylene	ug/L	ND	ND	ND	0.5
p+m-Xylene	ug/L	ND	ND	ND	0.5

Recovery of Surrogates
(%)

Parameter	TW 1/65	WOOD ST TW	METHOD BLANK
4-Bromofluorobenzene	89	95	86
D4-1,2-Dichloroethane	94	97	107
D8-Toluene	98	107	97

ND = Not detected
MDL = METHOD DETECTION LIMIT

ANKE HOLTKAMP

08/18/00 08:16 ☎705 739 4255
08/17/00 18:41 FAX 905 6900570

Maxxam Analytics Inc

CONFIDENTIAL

REPORT DATE: 2000/08/17

MAXXAM JOB #: A015229

VOLATILE ORGANICS (SELECTED) COMPOUNDS IN LIQUID

Maxxam ID		428953	428953	428954	428955	
COC Number		25528	25528	25528	25528	
Sampling Date		2000/08/08	2000/08/08	2000/08/08	2000/08/08	
Parameter	Units	MW 1 S	MW 1 S DUP	MW 1 D	MW 2 S	MDL
1,2-Dichlorobenzene	ug/L	ND	ND	ND	ND	0.5
1,2-Dichloroethane	ug/L	ND	ND	ND	ND	0.5
cis-1,2-Dichloroethylene	ug/L	2.0	1.8	2.2	ND	1
1,4-Dichlorobenzene	ug/L	ND	ND	ND	ND	0.5
Benzene	ug/L	ND	ND	ND	ND	0.5
Bromodichloromethane	ug/L	ND	ND	ND	ND	0.2
Bromoform	ug/L	ND	ND	ND	ND	0.2
Carbon Tetrachloride	ug/L	ND	ND	ND	ND	1
Chlorobenzene	ug/L	ND	ND	ND	ND	0.5
Chloroform	ug/L	0.4	0.5	0.7	ND	0.2
Dibromochloromethane	ug/L	ND	ND	ND	ND	0.2
Dichloromethane(Methylene Chloride)	ug/L	ND	ND	ND	ND	1
Ethylbenzene	ug/L	ND	ND	ND	ND	0.5
Tetrachloroethylene	ug/L	0.7	0.7	ND	ND	0.5
Toluene	ug/L	ND	ND	ND	ND	0.5
Trichloroethylene	ug/L	28	28	180	7.2	0.5
Vinyl Chloride	ug/L	ND	ND	ND	ND	0.5
o-Xylene	ug/L	ND	ND	ND	ND	0.5
p+m-Xylene	ug/L	ND	ND	ND	ND	0.5

Recovery of Surrogates
(%)

Parameter	MW 1 S	MW 1 S DUP	MW 1 D	MW 2 S
4-Bromofluorobenzene	91	91	96	90
D4-1,2-Dichloroethane	103	103	96	95
D8-Toluene	95	97	99	94

ND = Not detected
MDL = METHOD DETECTION LIMIT

ANKE HOLTKAMP

CONFIDENTIAL

Maxxam Analytics Inc

REPORT DATE: 2000/08/31

MAXXAM JOB #: A016577

VOLATILE ORGANICS (SELECTED) COMPOUNDS IN LIQUID

Maxxam ID		436674	436675		
COC Number		25529	25529		
Sampling Date		2000/08/23	2000/08/23		
Parameter	Units	WOOD ST. TW.	TW1/65	METHOD BLANK	MDL
1,2-Dichlorobenzene	ug/L	ND	ND	ND	0.5
1,2-Dichloroethane	ug/L	ND	ND	ND	0.5
cis-1,2-Dichloroethylene	ug/L	ND	ND	ND	1
1,4-Dichlorobenzene	ug/L	ND	ND	ND	0.5
Benzene	ug/L	ND	ND	ND	0.5
Bromodichloromethane	ug/L	ND	ND	ND	0.2
Bromoform	ug/L	ND	ND	ND	0.2
Carbon Tetrachloride	ug/L	ND	ND	ND	1
Chlorobenzene	ug/L	ND	ND	ND	0.5
Chloroform	ug/L	ND	ND	ND	0.2
Dibromochloromethane	ug/L	ND	ND	ND	0.2
Dichloromethane(Methylene Chloride)	ug/L	ND	ND	ND	1
Ethylbenzene	ug/L	ND	ND	ND	0.5
Tetrachloroethylene	ug/L	ND	ND	ND	0.5
Toluene	ug/L	ND	ND	ND	0.5
Trichloroethylene	ug/L	6.8	ND	ND	0.5
Vinyl Chloride	ug/L	ND	ND	ND	0.5
o-Xylene	ug/L	ND	ND	ND	0.5
p+m-Xylene	ug/L	ND	ND	ND	0.5

Recovery of Surrogates
(%)

Parameter	WOOD ST. TW.	TW1/65	METHOD BLANK
4-Bromofluorobenzene	100	100	100
D4-1,2-Dichloroethane	110	110	108
D8-Toluene	100	100	103

ND = Not detected
MDL = METHOD DETECTION LIMIT

CHARTERED ASSOCIATION OF THE CHEMICAL PROFESSION OF ONTARIO
Taras W. Obal
CHEMIST

TERRY OBAL, Ph.D., C. Chem

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Maxxam Analytics Inc

REPORT DATE: 2000/08/31

MAXXAM JOB #: A016577

VOLATILE ORGANICS (SELECTED) COMPOUNDS IN LIQUID

Maxxam ID		436669	436670	436671	436672	436673	
COC Number		25529	25529	25529	25529	25529	
Sampling Date		2000/08/23	2000/08/23	2000/08/23	2000/08/23	2000/08/23	
Parameter	Units	MW4 D	MW5 S	MW5 D	MW6 S	MW6 D	MDL
1,2-Dichlorobenzene	ug/L	ND	ND	ND	ND	ND	0.5
1,2-Dichloroethane	ug/L	ND	ND	ND	ND	ND	0.5
cis-1,2-Dichloroethylene	ug/L	ND	ND	ND	ND	ND	1
1,4-Dichlorobenzene	ug/L	ND	ND	ND	ND	ND	0.5
Benzene	ug/L	ND	ND	ND	ND	ND	0.5
Bromodichloromethane	ug/L	ND	ND	ND	ND	ND	0.2
Bromoform	ug/L	ND	ND	ND	ND	ND	0.2
Carbon Tetrachloride	ug/L	ND	ND	ND	ND	ND	1
Chlorobenzene	ug/L	ND	ND	ND	ND	ND	0.5
Chloroform	ug/L	ND	ND	ND	ND	ND	0.2
Dibromochloromethane	ug/L	ND	ND	ND	ND	ND	0.2
Dichloromethane(Methylene Chloride)	ug/L	ND	ND	ND	ND	ND	1
Ethylbenzene	ug/L	ND	ND	ND	ND	ND	0.5
Tetrachloroethylene	ug/L	ND	ND	ND	ND	ND	0.5
Toluene	ug/L	ND	ND	ND	ND	ND	0.5
Trichloroethylene	ug/L	ND	ND	ND	ND	ND	0.5
Vinyl Chloride	ug/L	ND	ND	ND	ND	ND	0.5
o-Xylene	ug/L	ND	ND	ND	ND	ND	0.5
p+m-Xylene	ug/L	ND	ND	ND	ND	ND	0.5

Recovery of Surrogates
(%)

Parameter	MW4 D	MW5 S	MW5 D	MW6 S	MW6 D
4-Bromofluorobenzene	95	100	100	100	100
D4-1,2-Dichloroethane	111	110	110	110	110
D8-Toluene	99	100	100	100	100

ND = Not detected
MDL = METHOD DETECTION LIMIT

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CHEMIST

TERRY OBAL, Ph.D., C. Chem

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Maxxam Analytics Inc

REPORT DATE: 2000/08/31

MAXXAM JOB #: A016577

VOLATILE ORGANICS (SELECTED) COMPOUNDS IN LIQUID

Maxxam ID		436665		436666	436667	436668	
COC Number		25529		25529	25529	25529	
Sampling Date		2000/08/23		2000/08/23	2000/08/23	2000/08/23	
Parameter	Units	MW2 D	MDL	MW3 S	MW3 D	MW4 S	MDL
1,2-Dichlorobenzene	ug/L	ND	0.5	ND	ND	ND	0.5
1,2-Dichloroethane	ug/L	ND	0.5	ND	ND	ND	0.5
cis-1,2-Dichloroethylene	ug/L	36	1	ND	ND	ND	1
1,4-Dichlorobenzene	ug/L	ND	0.5	ND	ND	ND	0.5
Benzene	ug/L	ND	0.5	ND	ND	ND	0.5
Bromodichloromethane	ug/L	ND	0.2	ND	ND	ND	0.2
Bromoform	ug/L	ND	0.2	ND	ND	ND	0.2
Carbon Tetrachloride	ug/L	ND	1	ND	ND	ND	1
Chlorobenzene	ug/L	ND	0.5	ND	ND	ND	0.5
Chloroform	ug/L	ND	0.2	1.0	ND	ND	0.2
Dibromochloromethane	ug/L	ND	0.2	ND	ND	ND	0.2
Dichloromethane(Methylene Chloride)	ug/L	ND	1	1.2	ND	ND	1
Ethylbenzene	ug/L	ND	0.5	ND	ND	ND	0.5
Tetrachloroethylene	ug/L	ND	0.5	ND	ND	ND	0.5
Toluene	ug/L	ND	0.5	ND	ND	ND	0.5
Trichloroethylene	ug/L	3000	18	ND	ND	ND	0.5
Vinyl Chloride	ug/L	6.3	0.5	ND	ND	ND	0.5
o-Xylene	ug/L	ND	0.5	ND	ND	ND	0.5
p+m-Xylene	ug/L	ND	0.5	ND	ND	ND	0.5

Recovery of Surrogates
(%)

Parameter	MW2 D	MW3 S	MW3 D	MW4 S
4-Bromofluorobenzene	104	97	97	101
D4-1,2-Dichloroethane	106	98	102	106
D8-Toluene	101	102	102	101

ND = Not detected
MDL = METHOD DETECTION LIMIT

Sample 436665: The sample was diluted due to high concentrations of target compounds. The MDLs were adjusted accordingly.

TERRY OBAL, Ph.D., C.Chem

ASSOCIATION OF THE CHEMICAL PROFESSION OF ONTARIO
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Maxxam Analytics Inc

REPORT DATE: 2000/08/31

MAXXAM JOB #: A016577

VOLATILE ORGANICS (SELECTED) COMPOUNDS IN LIQUID

Maxxam ID		436662	436662	436663	436664	
COC Number		25529	25529	25529	25529	
Sampling Date		2000/08/23	2000/08/23	2000/08/23	2000/08/23	
Parameter	Units	MW1 S	MW1 S DUP	MW 1D	MW2 S	MDL
1,2-Dichlorobenzene	ug/L	ND	ND	ND	ND	0.5
1,2-Dichloroethane	ug/L	ND	ND	ND	ND	0.5
cis-1,2-Dichloroethylene	ug/L	2.0	2.1	2.2	ND	1
1,4-Dichlorobenzene	ug/L	ND	ND	ND	ND	0.5
Benzene	ug/L	ND	ND	ND	ND	0.5
Bromodichloromethane	ug/L	ND	ND	ND	ND	0.2
Bromoform	ug/L	ND	ND	ND	ND	0.2
Carbon Tetrachloride	ug/L	ND	ND	ND	ND	1
Chlorobenzene	ug/L	ND	ND	ND	ND	0.5
Chloroform	ug/L	ND	ND	0.5	ND	0.2
Dibromochloromethane	ug/L	ND	ND	ND	ND	0.2
Dichloromethane(Methylene Chloride)	ug/L	1.7	1.7	1.8	ND	1
Ethylbenzene	ug/L	ND	ND	ND	ND	0.5
Tetrachloroethylene	ug/L	0.6	0.8	ND	ND	0.5
Toluene	ug/L	ND	ND	ND	ND	0.5
Trichloroethylene	ug/L	24	25	130	66	0.5
Vinyl Chloride	ug/L	ND	ND	ND	ND	0.5
o-Xylene	ug/L	ND	ND	ND	ND	0.5
p+m-Xylene	ug/L	ND	ND	ND	ND	0.5

Recovery of Surrogates
(%)

Parameter	MW1 S	MW1 S DUP	MW 1D	MW2 S
4-Bromofluorobenzene	97	97	103	99
D4-1,2-Dichloroethane	102	102	104	101
D8-Toluene	102	102	101	105

ND = Not detected
MDL = METHOD DETECTION LIMIT

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APPENDIX B

SITE PHOTOGRAPHS

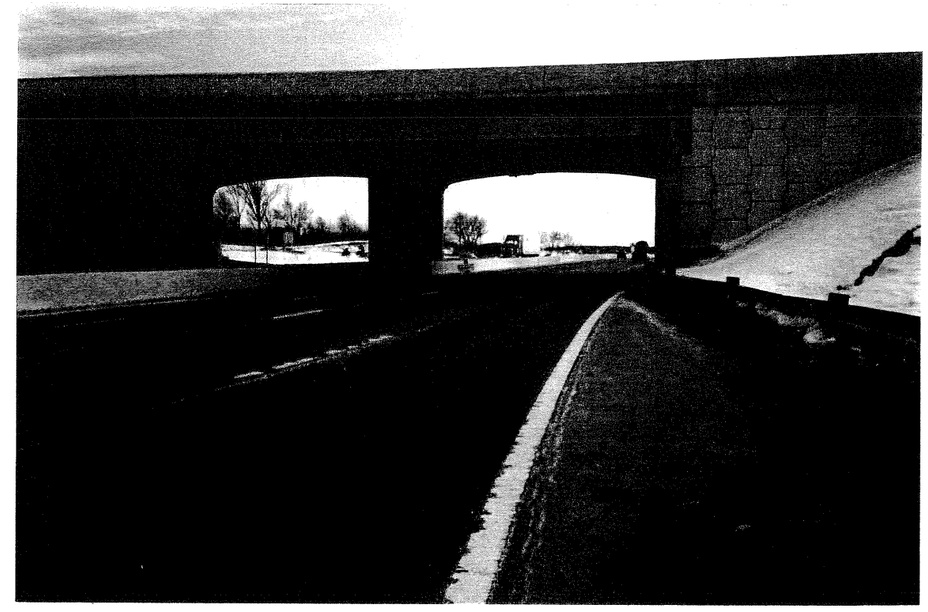


Photo 1: Highway 9 overpass, southern limit of Highway 400 ROW study area, looking south.

Photo 2: Concrete median along Highway 400 at Highway 9, looking north.

Photo 3: Husky gasoline service centre, southeast corner of Highway 400 ROW at Simcoe County Road 88.

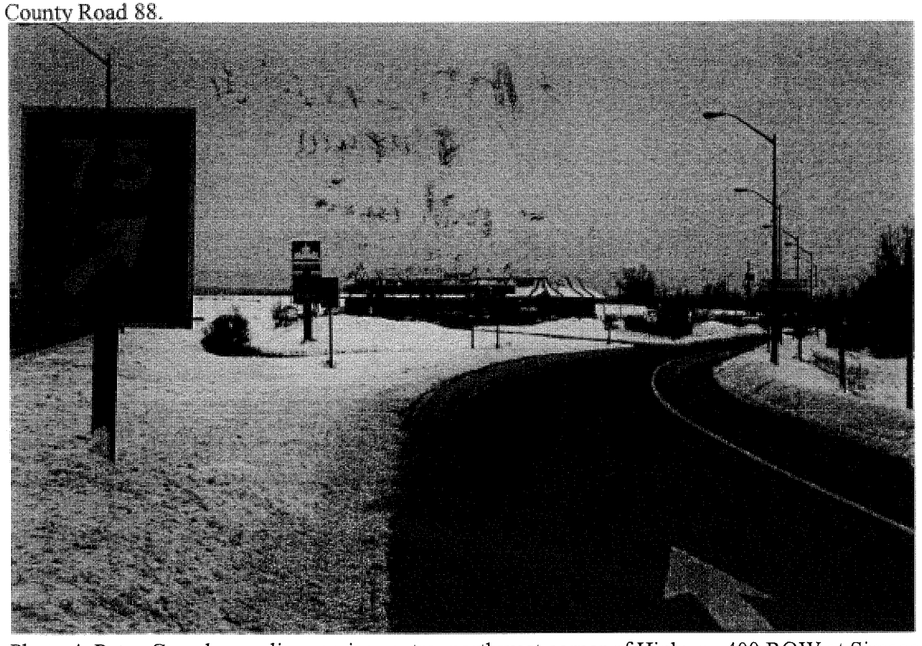


Photo 4: Petro Canada gasoline service centre, northwest corner of Highway 400 ROW at Simcoe County Road 89.

Photo 5: Lumberyard, southeast corner of Highway 400 ROW at 9th Line overpass.

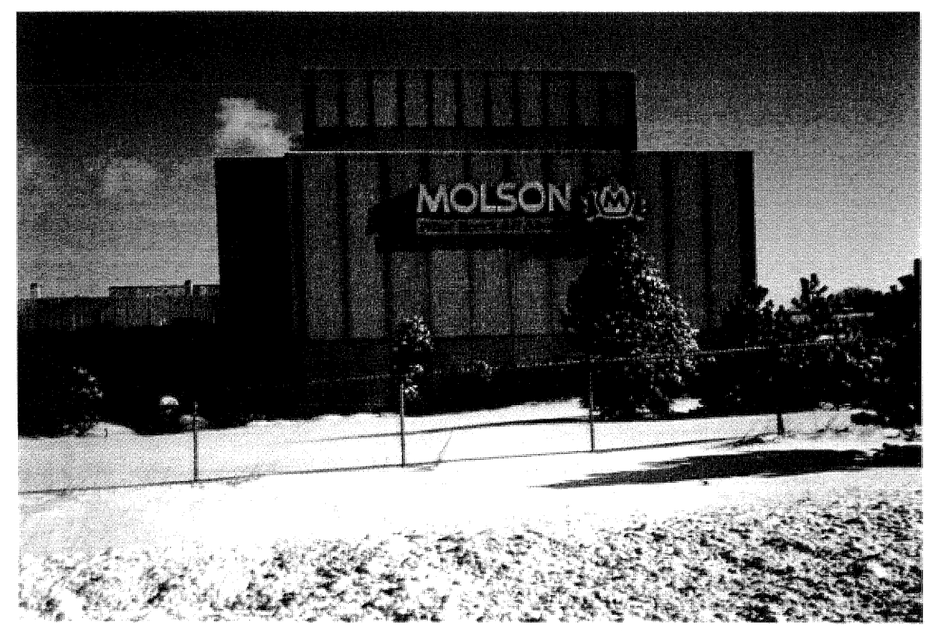


Photo 6: Former Molson Brewery located along east side of Highway 400 ROW at Molson Park Drive, Barrie.

Photo 7: Barrie Metals Ltd. located along east side of Highway 400 ROW, Barrie.

Photo 8: Canplas Industries Ltd. located along west side of Highway 400 ROW, Barrie. Note monitoring wells in foreground.

Photo 9: Ontario Hydro yard located along west side of Highway 400 ROW, Barrie.

Photo 10: Pioneer gasoline service centre located along east side of Highway 400 ROW, Barrie.

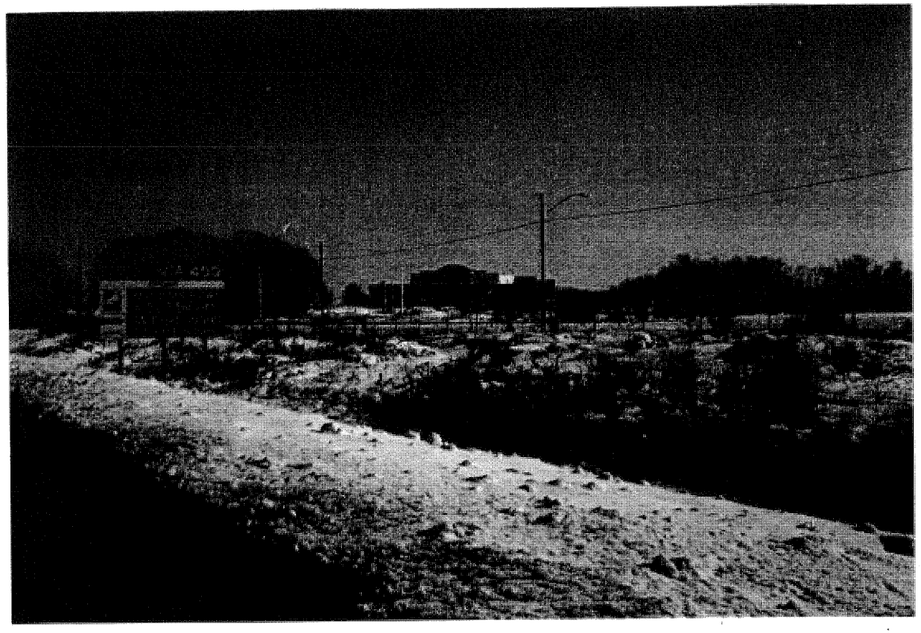


Photo 11: Barrie General Hospital located along east side of Highway 400 ROW, Barrie.

Photo 12: Typical steel rail and wood post guardrails along section of Highway 400 ROW.

Photo 13: Typical metal post and rail centre median along section of Highway 400 ROW.

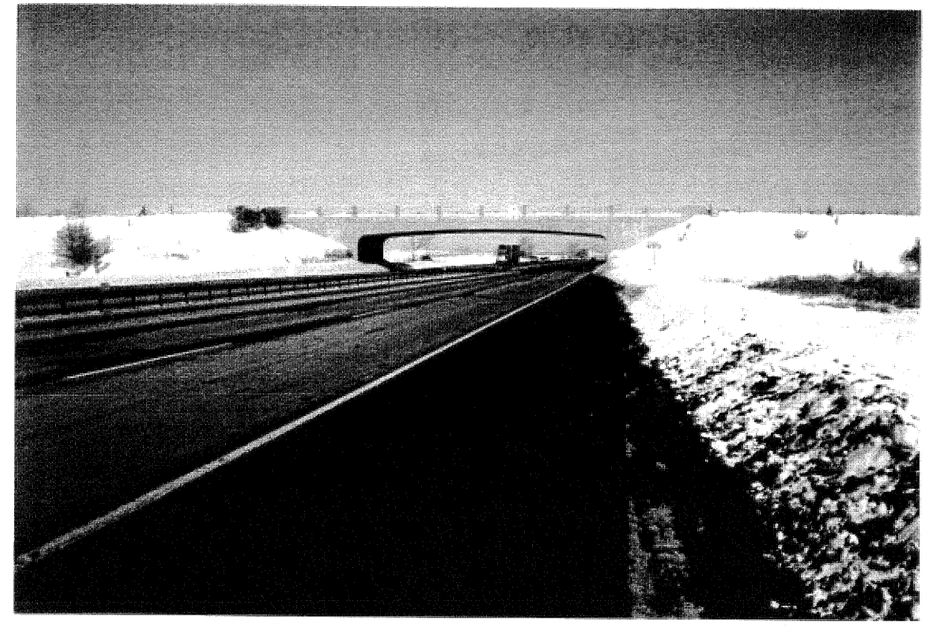


Photo 14: Typical concrete overpass, Highway 400 ROW.

Ontario

Ministry of Transportation
Southwestern Region
Planning and Design Section

HIGHWAY 400 PLANNING AND PRELIMINARY DESIGN STUDY

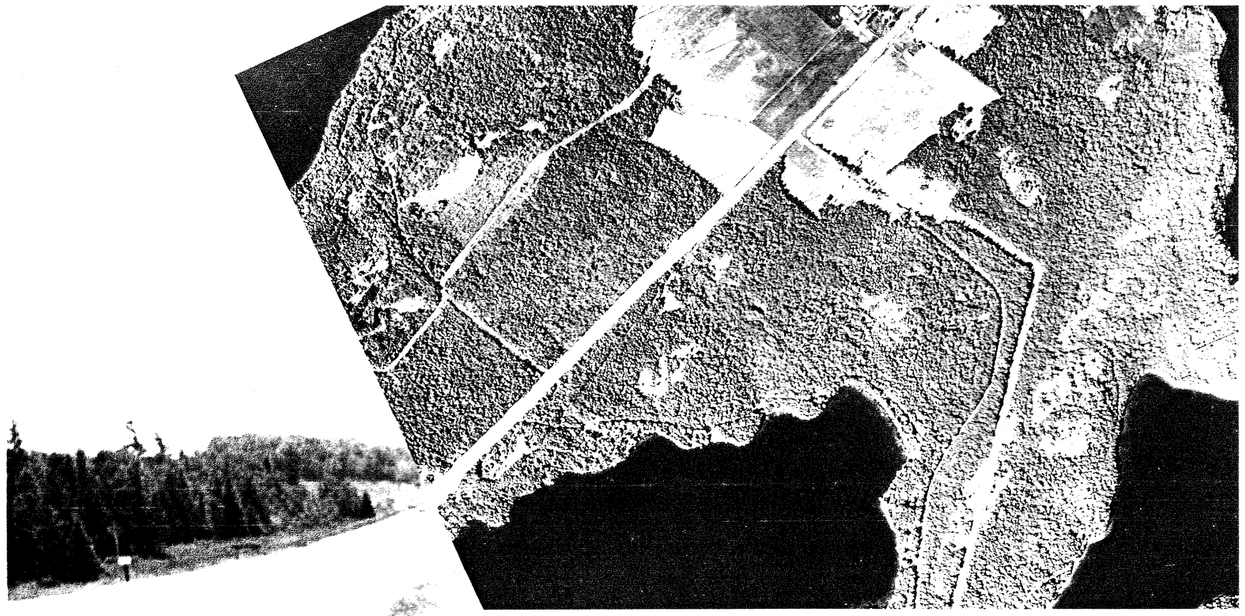
FROM 1 KM SOUTH OF HIGHWAY 89 NORTHERLY 30 KM TO THE JUNCTION AT HIGHWAY 11

COUNTY OF SIMCOE
TOWN OF BRADFORD WEST GWILLIMBURY
TOWN OF INNISFIL
CITY OF BARRIE
TOWNSHIP OF SPRINGWATER

G.W.P. 30-95-00

APPENDIX P

NATURAL SCIENCES TECHNICAL REPORT



Natural Sciences
Technical Report

Planning Study
Highway 400 1.0km
South of Highway 89 Northerly
30.0km to Junction of Highway 11

MTO Agreement No: 3005-A-000074
G.W.P. 30-95-00

May 2002

FRi
ECOLOGICAL SERVICES

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1.0 Introduction

The purpose of this report is to describe the existing natural environment conditions and features within the study area. The description of the existing conditions and features of the area will aid in the development of the preliminary design of the project, future work and agency consultation requirements. The results contained in this report include secondary source information supplemented with original field investigations and mapping. The report and all environmental investigations meet or exceed the MTO Provincial Highways Class Environmental Assessment for Provincial Transportation Facilities (as amended 2000) for a Group "B" undertaking. All fisheries investigations meet or exceed the Environmental Manual - Fisheries (Volume 1, 1994) and the MTO/MNR Fisheries Protocol.

This report provides information on existing environmental conditions, expected impacts, and proposed mitigation and enhancement strategies. The results of this study are the product of pre-field consultation with relevant agencies and interested parties, review of existing information and field investigations. Correspondence and contact records are contained in Appendix A.

2.0 Methodology

The preparation of this report included a review of secondary source information from a wide range of external agencies including the following:

- Nottawasaga Valley Conservation Authority
- Simcoe Region Conservation
- Natural Heritage Information Centre
- Ontario Ministry of Natural Resources (Midhurst)
- Ontario Ministry of Agriculture, Food and Rural Affairs
- Ontario Ministry of Environment
- City of Barrie
- Department of Fisheries and Oceans

Aerial photographs, Forest Resource Inventory maps, Natural Resource Values Inventory maps, Wildlife Values maps and Spawning Resource maps were consulted to plan the field investigations, identify gaps and required updates to the existing information. Distinct vegetation communities were identified prior to field visits and confirmed through original investigations. Terrestrial and preliminary fisheries investigations took place from September 11 to September 13, 2000. Detailed fisheries investigations were completed on September 26 and 27, 2000.

Natural environment features and functions were identified within 200m on either side of the existing centreline. Aquatic habitats were investigated as per the fisheries protocol outlined in the Environmental Manual Fisheries-Volume 1(1994).

Fish habitat was classified as Type 1, 2 or 3 and either seasonal or permanent as described in section 3.4 of the Fish Habitat Guidelines for Developing Areas (OMNR, 1994). All crossings were further characterized as warmwater or coldwater crossings. Type 1 habitats are those which, "in the best judgement of the MNR (significantly contribute to) the overall productive capacity" and require a high level of protection. Type 2 habitat "is important to the fish population, but, in the best judgement of MNR, is not a limiting factor for the habitat productive capacity". These habitats require a moderate level of protection. Type 3 habitats "have low productive capacity and, in the best judgement of the MNR, do not contribute significantly, directly or indirectly, to fish production" and require a minimum level of protection. These types of habitats may be utilised by fish on a seasonal (i.e. migration/passage) or a permanent (i.e. life cycle requirements) basis. All natural environment features and functions identified within 200m of the ROW from secondary source information and original field work are displayed in Figures 2 to 11.

3.0 Setting

The project study limits include the Highway 400 corridor 1.0km south of Highway 89 north 30km to the junction of Highway 11. The study area stretches from West Gwillimbury Township near the Town of Cookstown through the Townships of Innisfil and Vespra just north of the City of Barrie (Figure 1). A summary of the values can be found Table 18.

3.1. Ecological Setting

The study area occurs within the Lakes Simcoe-Rideau Site Region known as 6E. This region extends from the Ottawa River in the east to Lake Huron in the west and includes 12 site districts. The forest climate type is mid-humid, Great Lakes hardwoods. The regional landform is gently undulating to rolling terrain of ice-laid materials deeply covering the Paleozoic bedrock over eighty percent of the area. Local plains of smoother lacustrine deposits occur; also, limestone plains occur very shallowly covered with drift. The regional soil type is of limy soil materials that support tolerant hardwood-conifer forests in a grey-brown podsolic except in areas of stony limestone till where limestone gravel is near the surface. In these latter areas, brown forest soils are common. Soils and landforms of this site region support beech, hard maple and hemlock on normal fresh sites; oak and hickory on warmer fresh sites; oak and ash on hot dry sites; hard maple, oak and ash on normal dry sites; white pine, elm and ash on cold dry sites; and, hemlock, yellow birch or spruce and white cedar on wet sites. Site district 6E-8 (Peterborough) is an area of very high lime loam (drumlinized till) with local areas of high to moderately low lime, water-laid silt and clay.

Information Sources

- OMNR. 1992. Report on the Status of Provincial Parks in the Site Regions and Districts of Ontario. Draft Report
- Hills, G.A. 1959. A Ready Reference to the Description of the Land of Ontario and its Productivity. Ontario Department of Lands and Forests, Division of Research, Maple, Ontario.

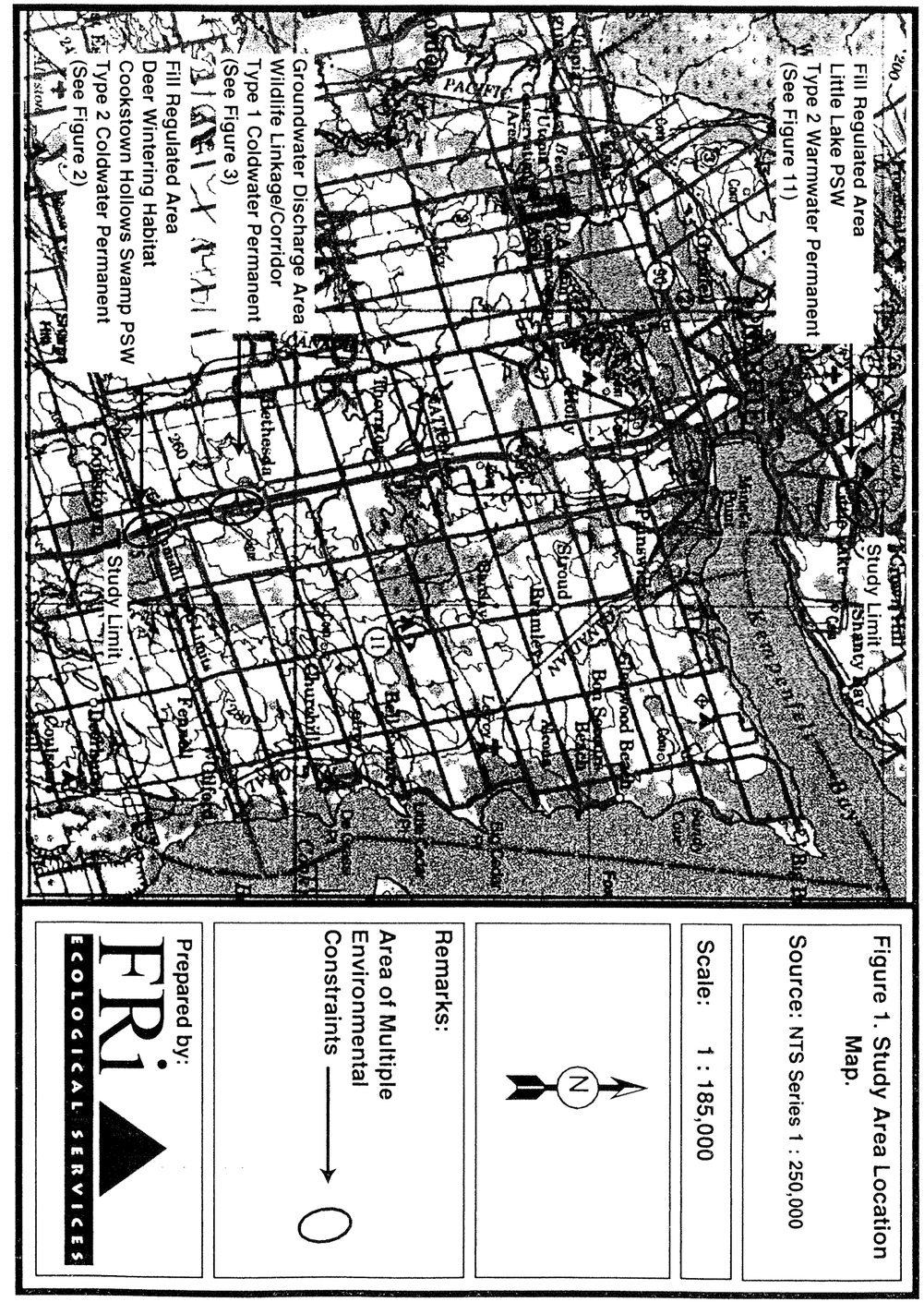
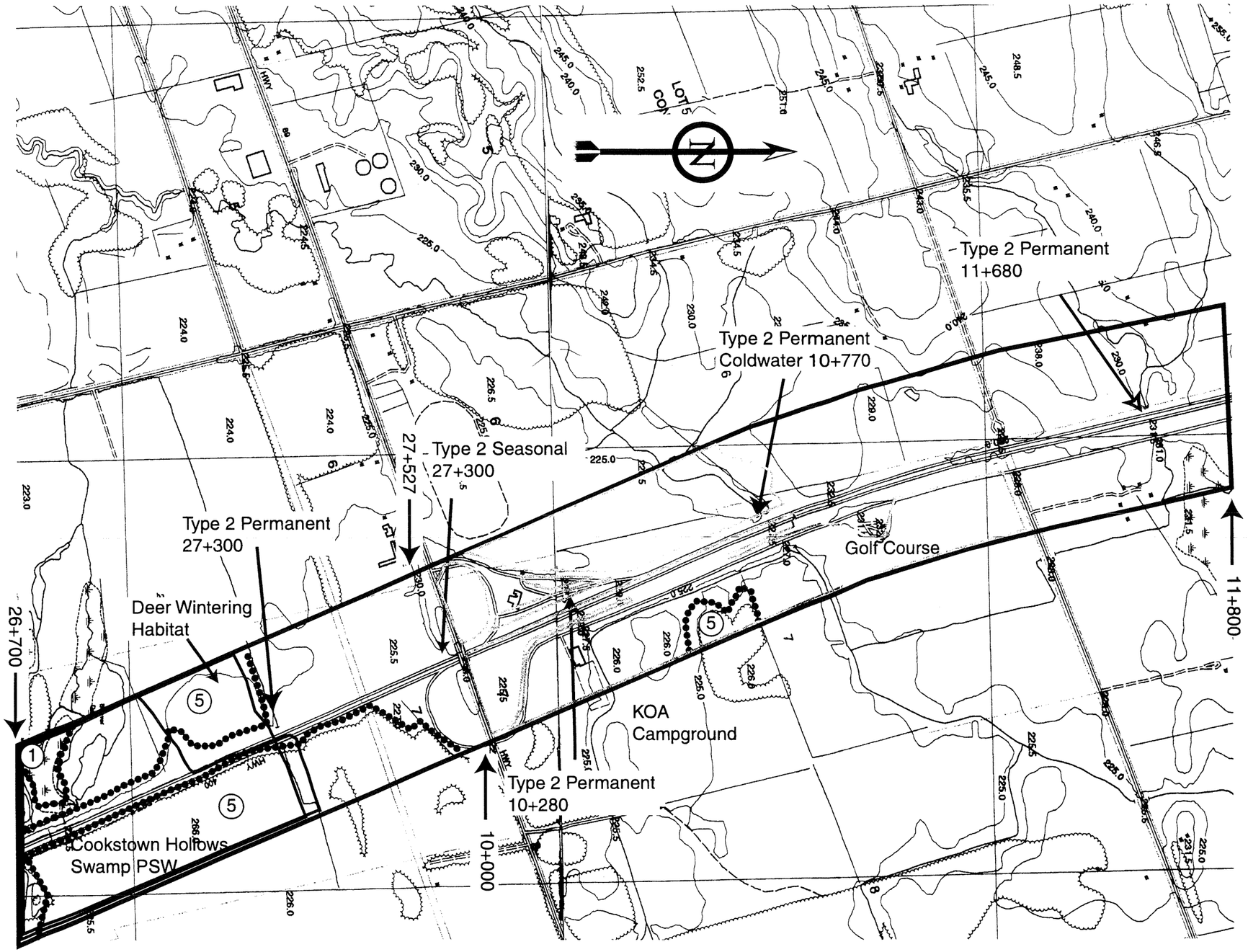


Figure 1. Study Area Location Map.
Source: NTS Series 1 : 250,000



LEGEND

- Groundwater Recharge Area
- Groundwater Discharge Area
- Fill Regulated Area
- Deer Wintering Habitat
- ANSI
- Provincially Significant Wetland
- Unevaluated Wetland
- Forest Community
- Pr — Red Pine Plantation
- Ps — Scots Pine Plantation
- Study Corridor
- Wildlife Linkage/Corridor

250 0 250 500

Scale 1 : 10,000

URS Cole, Sherman & Associates

Figure 2. Highway 400 Natural Heritage Features From 26+700 to 27+545 West Gwillimbury Twp. and 10+000 to 11+800 Innisfil Twp.

Drawn By: RSB	FRi Project #: 00-74
Checked By: CSH	Agreement #:3005-A-000027
Date: January 10, 2001	G.W.P. #: 30-95-00
CSA#: CN29900147	Filename: 30-95-00-02.ai

Prepared by:

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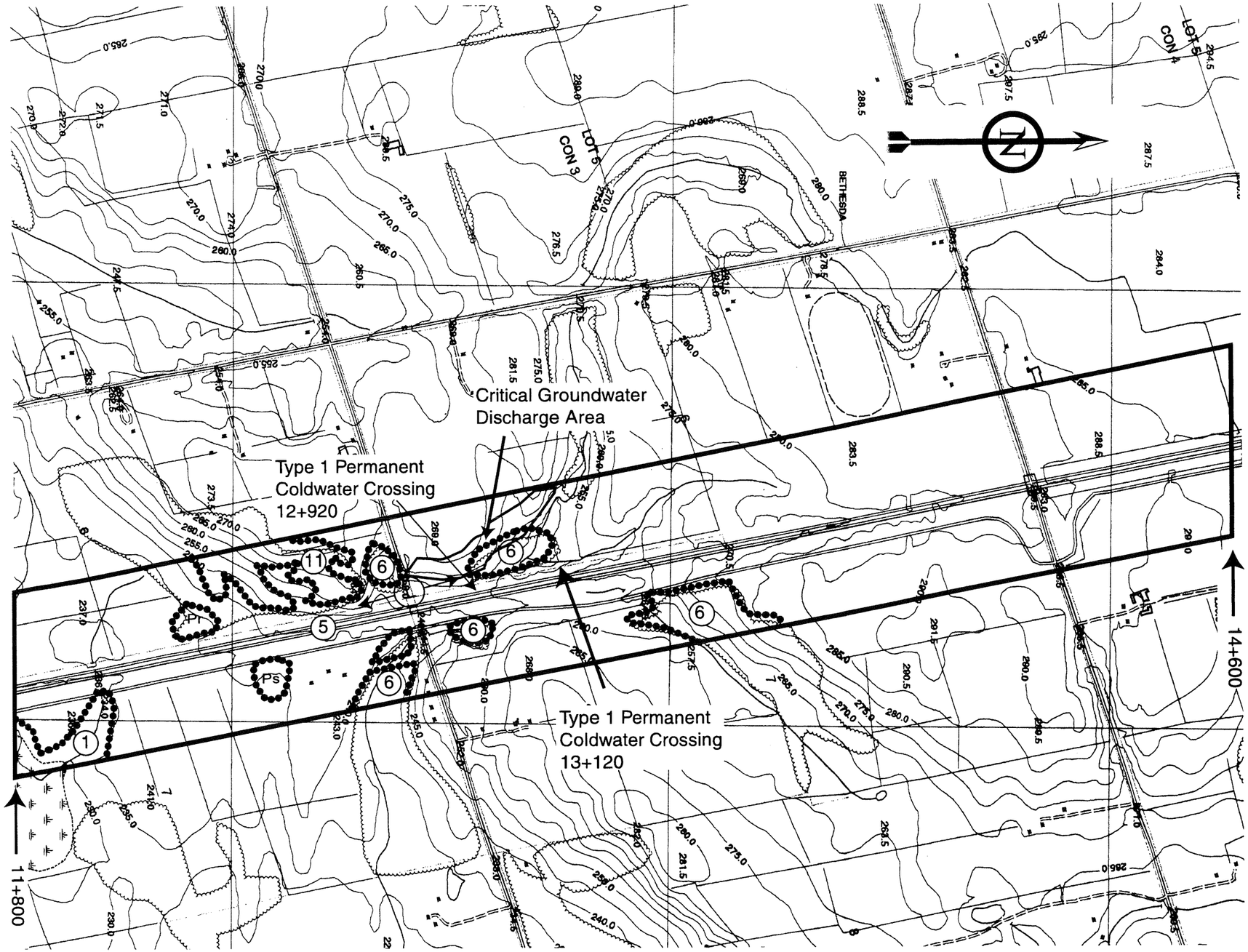

LEGEND

- Groundwater Recharge Area
- Groundwater Discharge Area
- Fill Regulated Area
- Deer Wintering Habitat
- ANSI
- Provincially Significant Wetland
- Unevaluated Wetland
- (8) Forest Community
- Pr Red Pine Plantation
- Ps Scots Pine Plantation
- Study Corridor
- (L) Wildlife Linkage/Corridor

250 0 250 500

Scale 1 : 10,000

URS Cole, Sherman & Associates

Figure 3. Highway 400 Natural Heritage Features From 11+800 to 14+600 Innisfil Twp.

Drawn By: RSB	FRi Project #: 00-74
Checked By: CSH	Agreement #:3005-A-000027
Date: January 10, 2001	G.W.P. #: 30-95-00
CSA#: CN29900147	Filename: 30-95-00-03.ai

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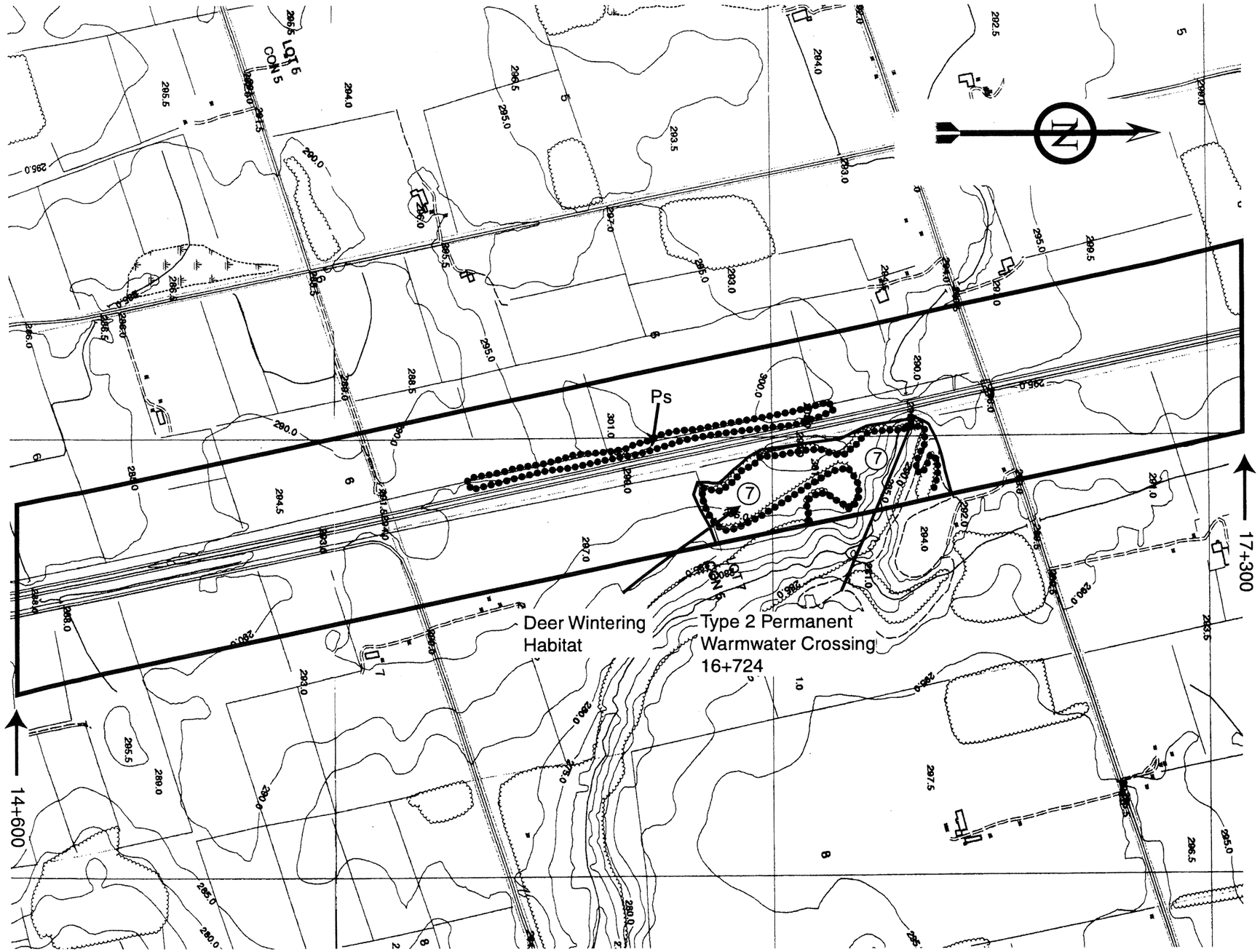

LEGEND

- Groundwater Recharge Area
- Groundwater Discharge Area
- Fill Regulated Area
- Deer Wintering Habitat
- ANSI
- Provincially Significant Wetland
- Unevaluated Wetland
- (8) Forest Community
- Pr Red Pine Plantation
- Ps Scots Pine Plantation
- Study Corridor
- (L) Wildlife Linkage/Corridor

250 0 250 500

Scale 1 : 10,000

URS Cole, Sherman & Associates

Figure 4. Highway 400 Natural Heritage Features From 14+600 to 17+300 Innisfil Twp.

Drawn By: RSB	FRi Project #: 00-74
Checked By: CSH	Agreement #:3005-A-000027
Date: January 10, 2001	G.W.P. #: 30-95-00
CSA#: CN29900147	Filename: 30-95-00-04.ai

Prepared by:

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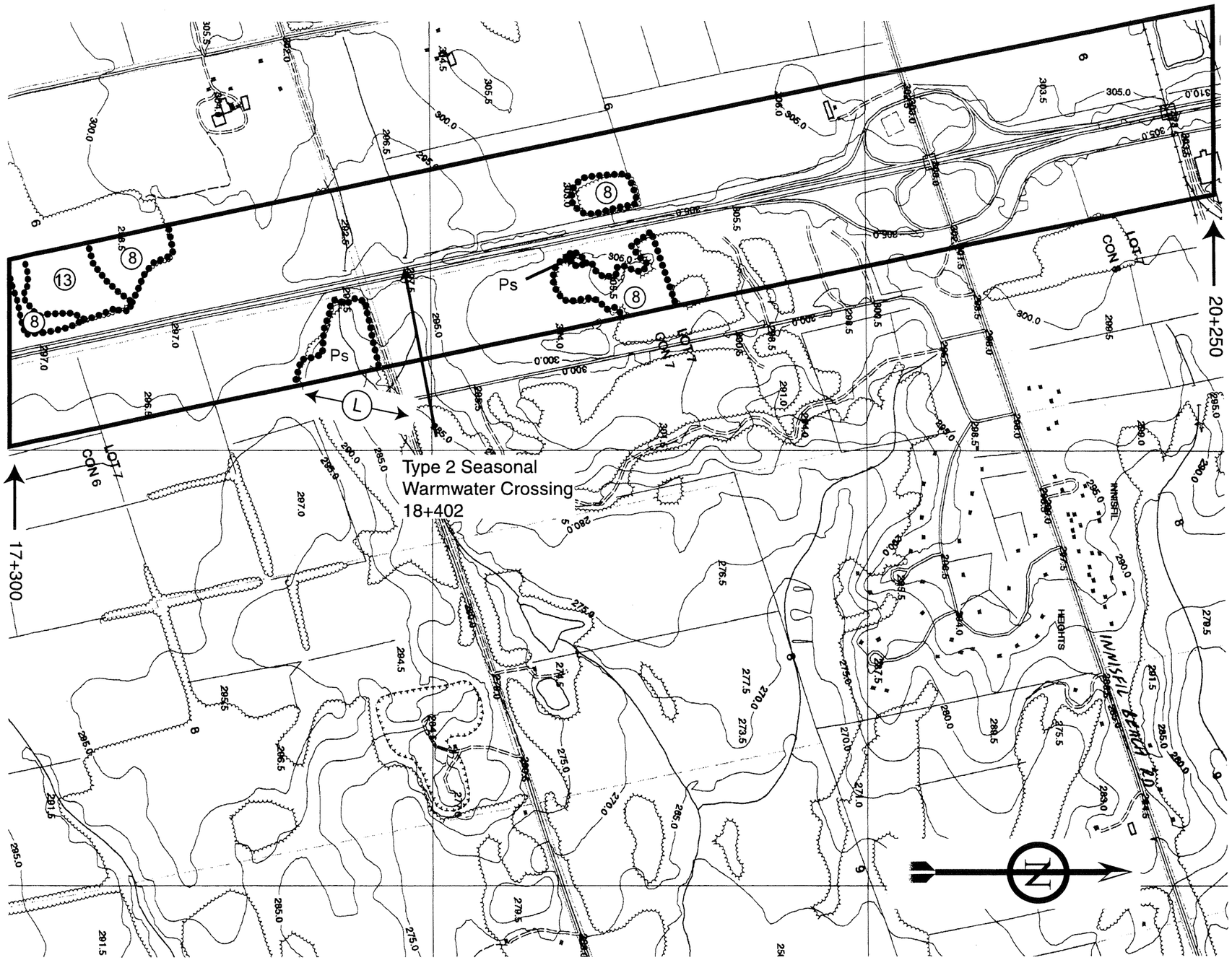

LEGEND

- Groundwater Recharge Area
- Groundwater Discharge Area
- Fill Regulated Area
- Deer Wintering Habitat
- ANSI
- Provincially Significant Wetland
- Unevaluated Wetland
- (8) Forest Community
- Pr Red Pine Plantation
- Ps Scots Pine Plantation
- Study Corridor
- (L) Wildlife Linkage/Corridor

250 0 250 500

Scale 1 : 10,000

URS Cole, Sherman & Associates

Figure 5. Highway 400 Natural Heritage Features From 17+300 to 20+250 Innisfil Twp.

Drawn By: RSB	FRi Project #: 00-74
Checked By: CSH	Agreement #:3005-A-000027
Date: January 10, 2001	G.W.P. #: 30-95-00
CSA#: CN29900147	Filename: 30-95-00-05.ai

Prepared by:

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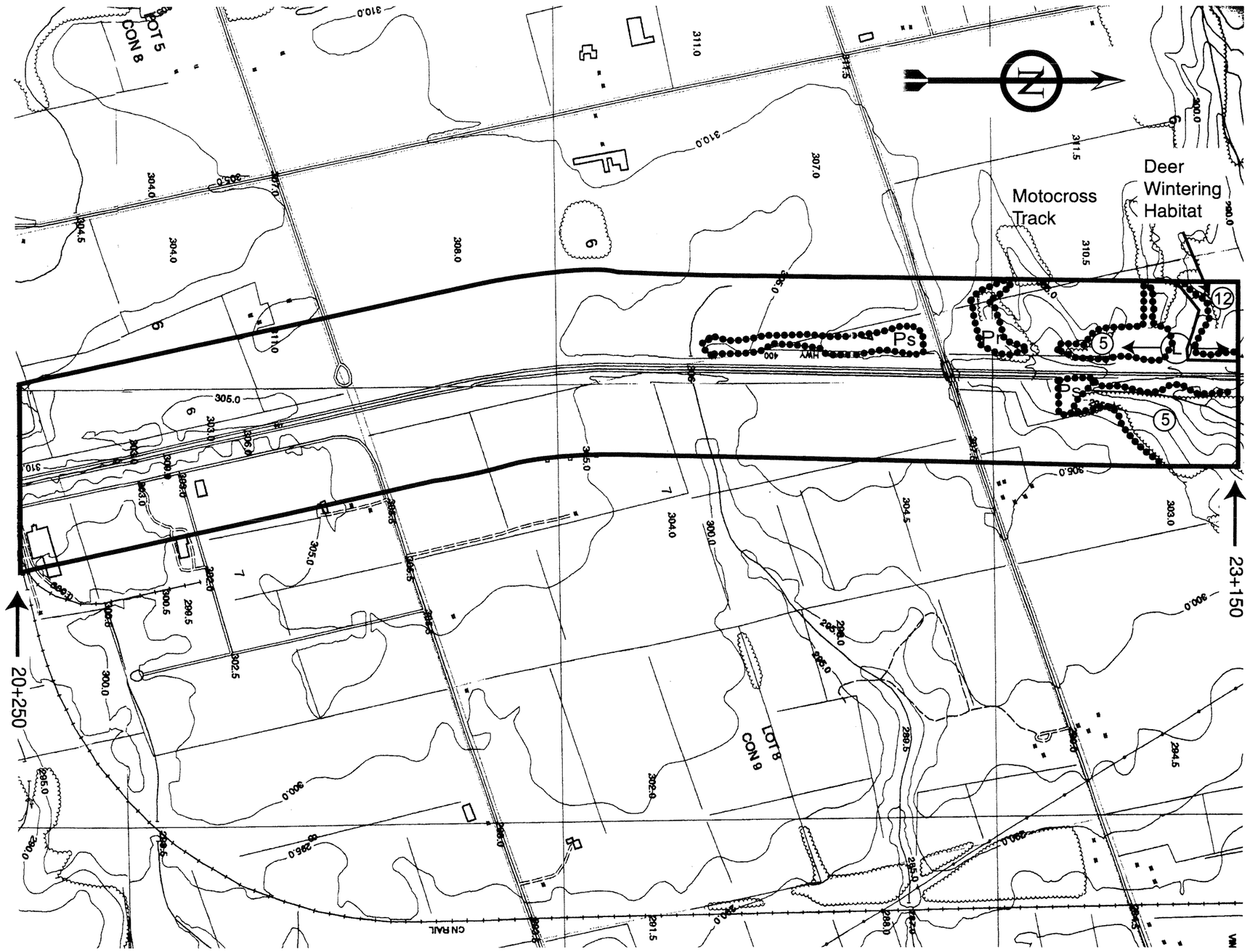


LEGEND

- Groundwater Recharge Area
- Groundwater Discharge Area
- Fill Regulated Area
- Deer Wintering Habitat
- ANSI
- Provincially Significant Wetland
- Unevaluated Wetland
- (8) Forest Community
- Pr Red Pine Plantation
- Ps Scots Pine Plantation
- Study Corridor
- (L) Wildlife Linkage/Corridor

250 0 250 500

Scale 1 : 10,000

URS Cole, Sherman & Associates

Figure 6. Highway 400 Natural Heritage Features From 20+250 to 23+150 Innisfil Twp.

Drawn By: RSB	FRi Project #: 00-74
Checked By: CSH	Agreement #:3005-A-000027
Date: January 10, 2001	G.W.P. #: 30-95-00
CSA#: CN29900147	Filename: 30-95-00-06.ai

Prepared by:

FRi ECOLOGICAL SERVICES

LEGEND

- Groundwater Recharge Area
- Groundwater Discharge Area
- Fill Regulated Area
- Deer Wintering Habitat
- ANSI
- Provincially Significant Wetland
- Unevaluated Wetland
- (8) Forest Community
- Pr Red Pine Plantation
- Ps Scots Pine Plantation
- Study Corridor
- (L) Wildlife Linkage/Corridor

250 0 250 500

Scale 1 : 10,000

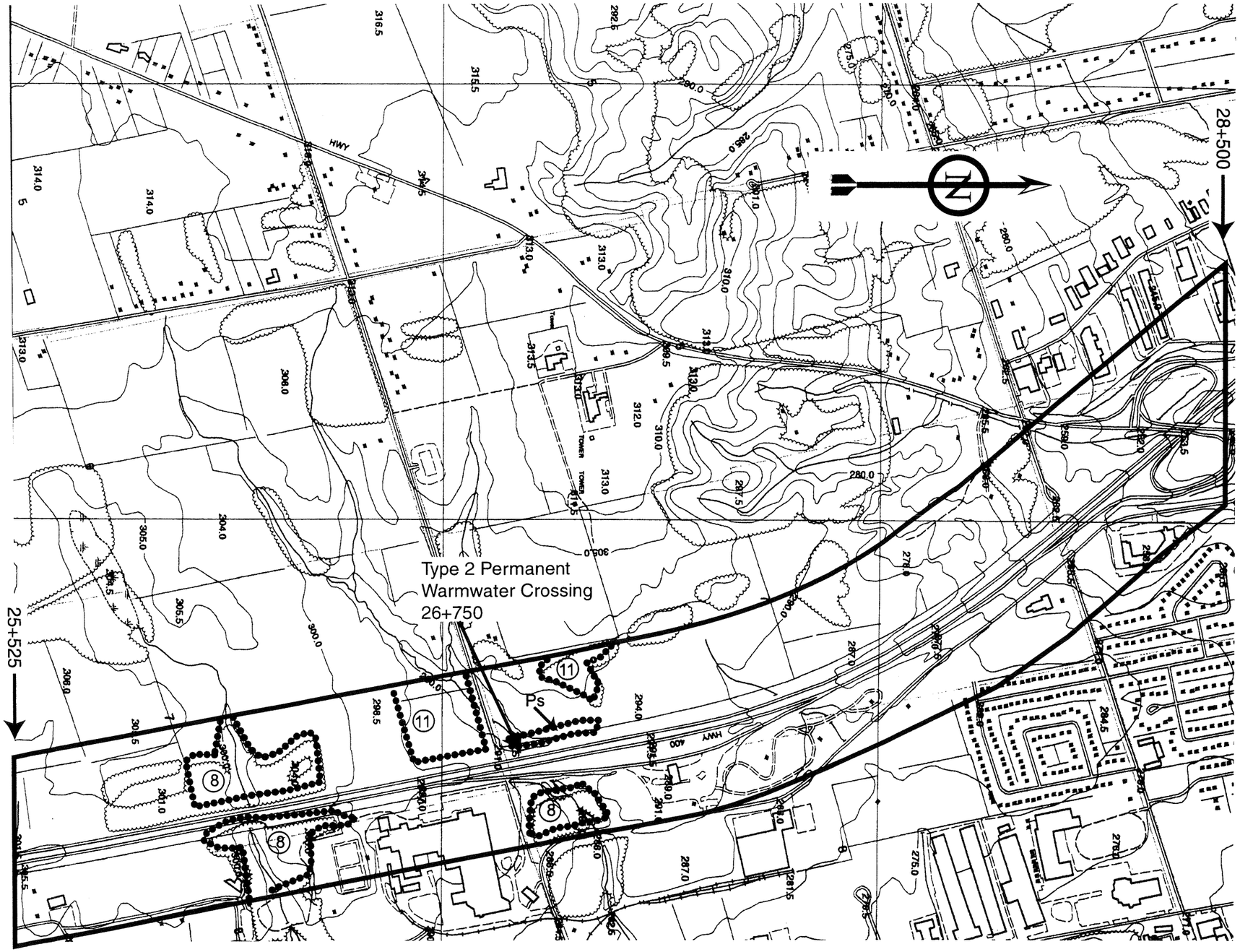
URS Cole, Sherman & Associates

Figure 7. Highway 400 Natural Heritage Features From 23+150 to 25+525 Innisfil Twp.

Drawn By: RSB	FRi Project #: 00-74
Checked By: CSH	Agreement #:3005-A-000027
Date: January 10, 2001	G.W.P. #: 30-95-00
CSA#: CN29900147	Filename: 30-95-00-07.ai

Prepared by:

FRi ECOLOGICAL SERVICES



LEGEND

- Groundwater Recharge Area
- Groundwater Discharge Area
- Fill Regulated Area
- Deer Wintering Habitat
- ANSI
- Provincially Significant Wetland
- Unevaluated Wetland
- (8) Forest Community
- Pr Red Pine Plantation
- Ps Scots Pine Plantation
- Study Corridor
- (L) Wildlife Linkage/Corridor

250 0 250 500

Scale 1 : 10,000

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Figure 8. Highway 400 Natural Heritage Features From 25+525 to 28+500 Innisfil Twp.

Drawn By: RSB	FRi Project #: 00-74
Checked By: CSH	Agreement #:3005-A-000027
Date: January 10, 2001	G.W.P. #: 30-95-00
CSA#: CN29900147	Filename: 30-95-00-08.ai

Prepared by:

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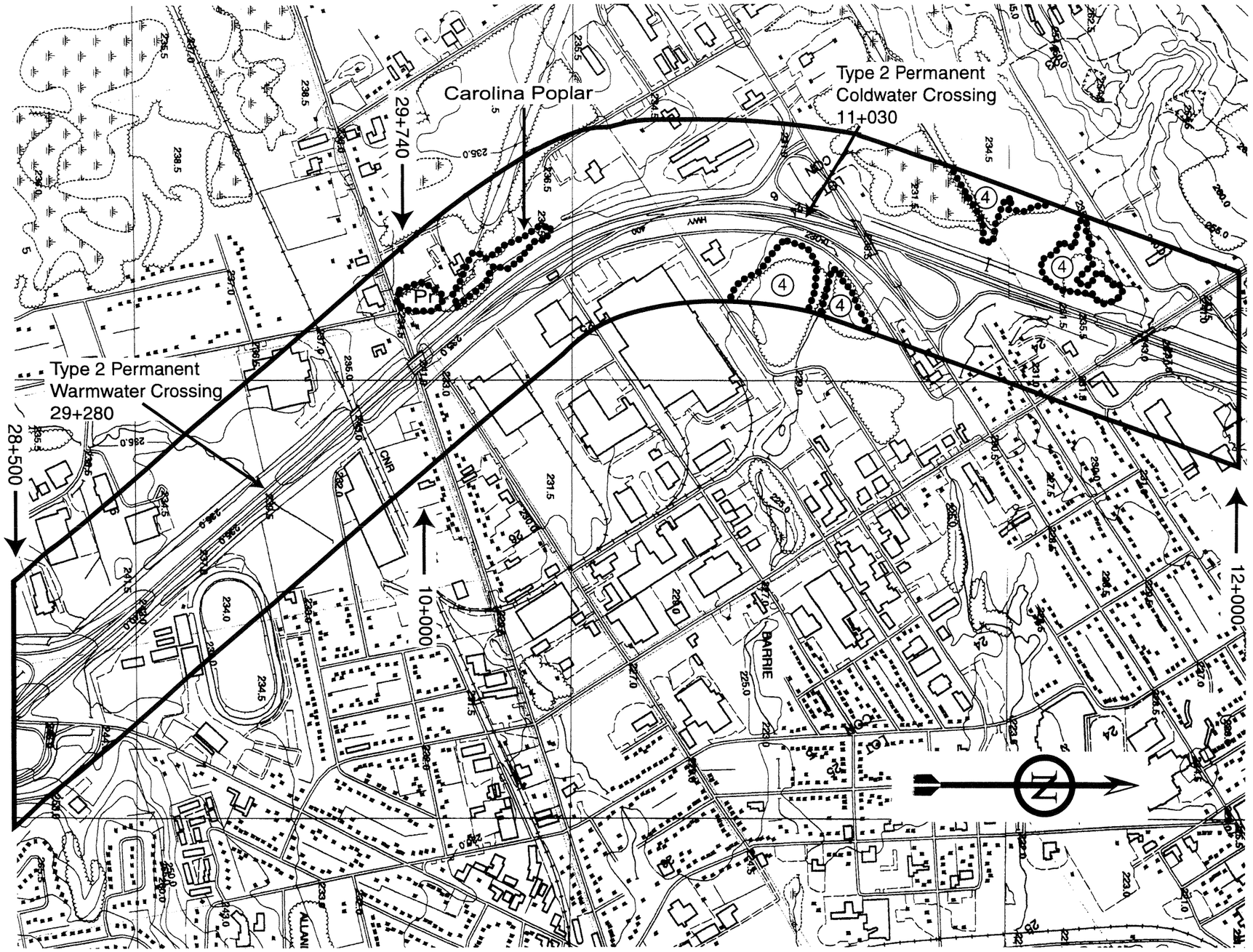

LEGEND

- Groundwater Recharge Area
- Groundwater Discharge Area
- Fill Regulated Area
- Deer Wintering Habitat
- ANSI
- Provincially Significant Wetland
- Unevaluated Wetland
- (8) Forest Community
- Pr Red Pine Plantation
- Ps Scots Pine Plantation
- Study Corridor
- (L) Wildlife Linkage/Corridor

250 0 250 500

Scale 1 : 10,000

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Figure 9. Highway 400 Natural Heritage Features From 28+500 to 29+725 Innisfil Twp. and 10+000 to 12+000 Vespra Twp.

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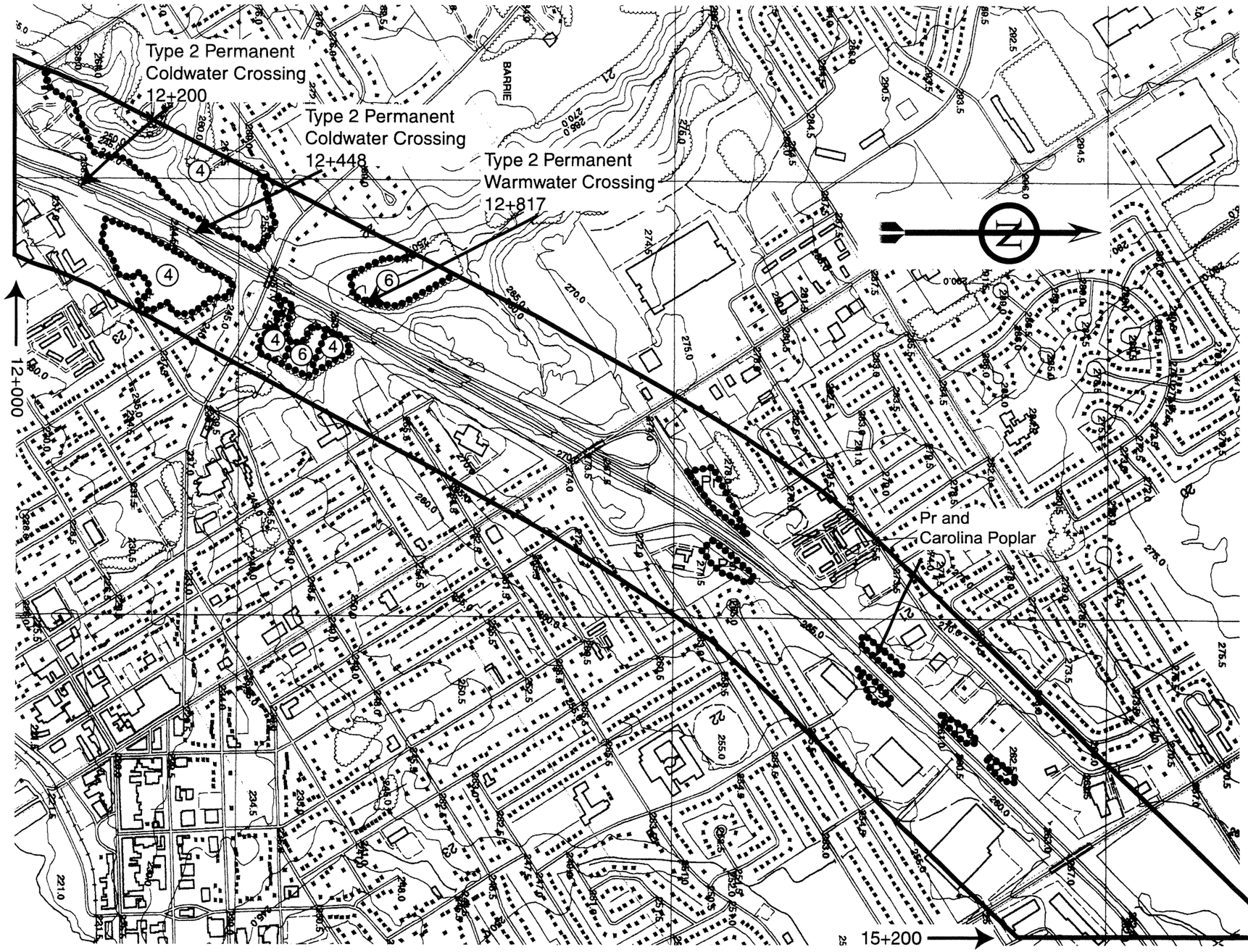
Checked By: CSH	Agreement #:3005-A-000027
Date: January 10, 2001	G.W.P. #: 30-95-00
CSA#: CN29900147	Filename: 30-95-00-09.ai

Prepared by:

FRi ECOLOGICAL SERVICES



LEGEND

- Groundwater Recharge Area
- Groundwater Discharge Area
- Fill Regulated Area
- Deer Wintering Habitat
- ANSI
- Provincially Significant Wetland
- Unevaluated Wetland
- Forest Community
- Pr — Red Pine Plantation
- Ps — Scots Pine Plantation
- Study Corridor
- Wildlife Linkage/Corridor

250 0 250 500

Scale 1 : 10,000

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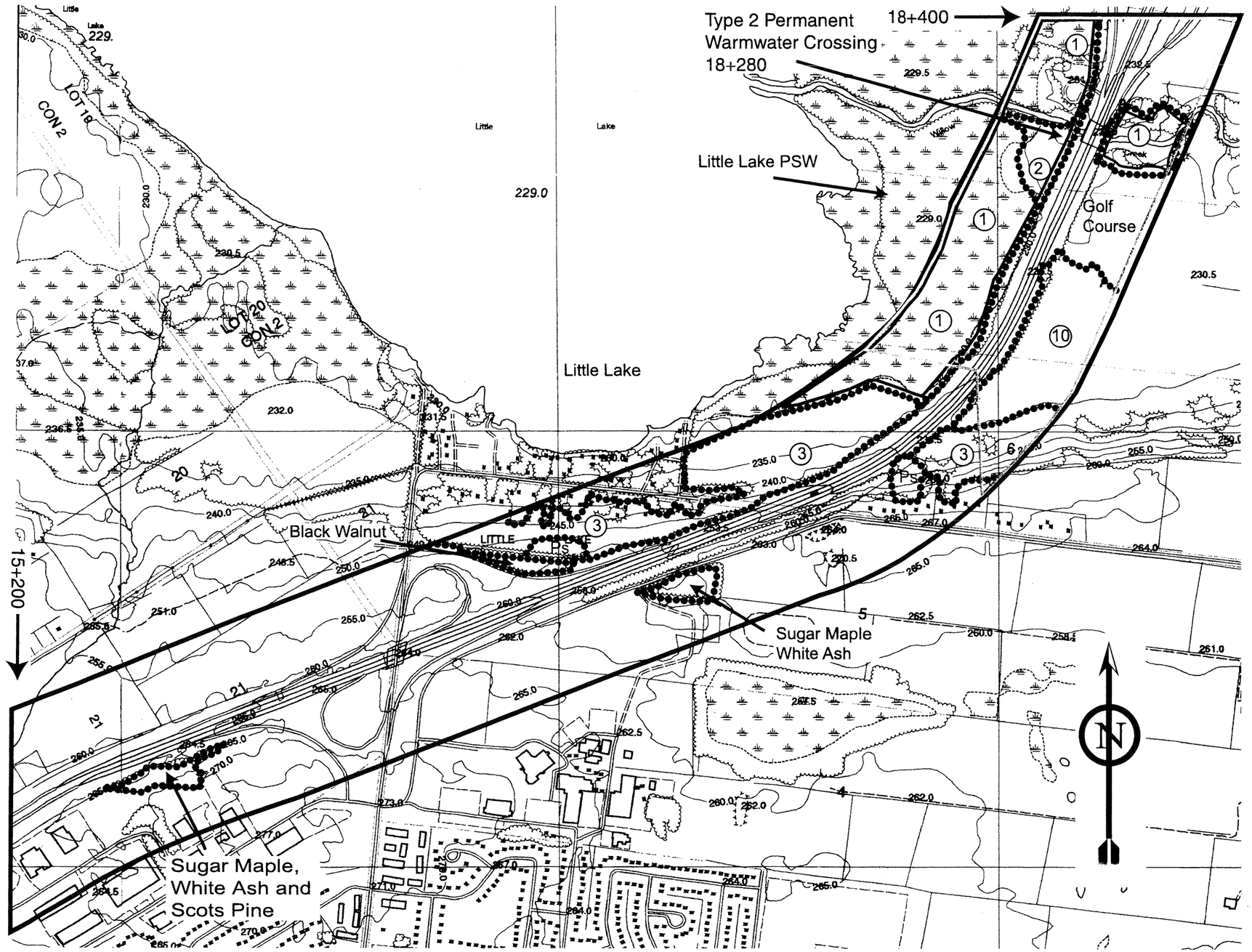
Figure 10. Highway 400 Natural Heritage Features From 12+000 to 15+200 Vespra Twp.

Drawn By: RSB	FRi Project #: 00-74
Checked By: CSH	Agreement #:3005-A-000027
Date: January 10, 2001	G.W.P. #: 30-95-00
CSA#: CN29900147	Filename: 30-95-00-10.ai

Prepared by:

FRi ECOLOGICAL SERVICES



LEGEND

- Groundwater Recharge Area
- Groundwater Discharge Area
- Fill Regulated Area
- Deer Wintering Habitat
- ANSI
- Provincially Significant Wetland
- Unevaluated Wetland
- (8) Forest Community
- Pr Red Pine Plantation
- Ps Scots Pine Plantation
- Study Corridor
- (L) Wildlife Linkage/Corridor

250 0 250 500

Scale 1 : 10,000

URS Cole, Sherman & Associates

Figure 11. Highway 400 Natural Heritage Features From 15+200 to 18+400 Vespra Twp.

Drawn By: RSB	FRi Project #: 00-74
Checked By: CSH	Agreement #:3005-A-000027
Date: January 10, 2001	G.W.P. #: 30-95-00
CSA#: CN29900147	Filename: 30-95-00-11.ai

Prepared by:

FRi ECOLOGICAL SERVICES

3.2. Geological Setting

The bedrock of the study area is situated in the Grenville Province lying in the Central Metasedimentary Belt. Limestone, dolostone, shale, arkose and sandstone from the Middle Ordovician period are typical of the area. There are no geological faults within or near the study area.

Information Sources

- Ministry of Northern Development and Mines. 1991. Bedrock Geology of Ontario - Southern Sheet. Map 2544.

3.3. Physiography

The study area is situated in three distinct physiographic divisions. The southerly project limit begins in the Peterborough Drumlin Field then continues through the Simcoe Uplands and the City of Barrie is situated in the Simcoe Lowlands.

The Peterborough Drumlin Field is a distinctive physiographic division within the study area. The individual drumlin features provide important functions related to groundwater recharge and discharge. Only two drumlin features actually occur within the study area corridor. Both of these features abut the highway near chainage 13+000 in Innisfil Township.

The Simcoe Uplands are characterized by curved ridges and steep-sided, flat-floored valleys. The southerly portion, Innisfil Uplands, is a flat-topped till plain approximately 60m higher than the surrounding lake plains. This upland area and the portion north of Kempenfelt Bay is bisected by another physiographic region; the Simcoe Lowlands.

This lowland unit is composed of the Nottawasaga and Simcoe basins that are connected at Barrie at the end of Kempenfelt Bay. The most striking feature within the general study area is the extensive wetland at the headwaters of Innisfil and Lover's Creek. The Simcoe basin is relatively poorly drained compared to the Nottawasaga basin.

Information Sources

- Chapman, L.J. and D.F. Putnam. 1966. The Physiography of Southern Ontario. Ontario Research Foundation & The University of Toronto Press. 386p.

4.0 Soils

Soils were classified according to their agricultural capability classes. There is good representation of nearly all of the soil capability classes. The majority of the soils within the study area fall into either the Class 1 or Class 3 categories (Appendix B).

Class 1 soils are defined to have no significant limitations in use for crops. They are level or have very gentle slopes, they are deep, well to imperfectly drained and have a good water-holding capacity. They are easily maintained in good tilth and productivity, and damage from erosion is slight. Class 3 soils are defined to have moderately severe limitations that restrict the range of crops or require special conservation practices. The most commonly encountered limitations to soil capability are excess water, low fertility and topography. Most of these limitations are directly related to the physiographic character of the study area. The drumlin and upland ridge features typically have moderate to high relief with stony or sandy infertile soils. There are deposits of clay and silt on the plains between the drumlin and ridge features. Organic soils are also linked to the physiographic characteristics of the study area. They occur only in wetland areas near Little Lake. Organic soils are defined as soils that contain 30 % or more of organic matter and have a depth of 12 inches or more of consolidated organic material.

Our field investigations did not identify any areas where soil erosion was significant. In general the upland areas were composed of porous sandy tills and loams with a thin veneer of coarse outwash material. The organic soils in the depressional areas are predominantly humisols and fine textured silt loam to clay. The following table documents the occurrence of the various soil classes.

Information Sources

- Environment Canada. 1967. Canada Land Inventory-Soil Capability For Agriculture. Lake Simcoe Map 31D. Surveys and Mapping Branch.

Table 1. Classes of Soil Capability for Agriculture of the Study Area Corridor.

Soil Capability Class	Occurrence in the Study Area Corridor
Class 1	27%
Class 2	7%
Class 3	25%
Class 4	12%
Class 5	16%
Class 6	0
Class 7	10%
Organic	3%

5.0 Surface Drainage

Surface drainage within the study area occurs primarily in two tertiary watershed divisions (Appendix C). The study area straddles the Nottawasaga River watershed (2ED) to the west and the Severn River watershed (2EC) to the east.

The highway crosses three main drainage systems within the study area. The first system to the south is Innisfil Creek and its tributaries. The second major system, Lover's Creek, is a significant coldwater system. The highway crosses a number of tributaries within this system.

The final drainage system is Willow Creek that drains into Little Lake and eventually empties into the Nottawasaga River.

Information Sources

- Dodge, D.P., G.A. Goodchild, J.C. Tilt and D.G. Waldriff. 1983. OMNR Manual of Instructions - Aquatic Habitat Inventory Surveys. Fisheries Branch. 168pp.

6.0 Groundwater

The groundwater component was investigated under two categories: recharge areas and discharge areas. Appendix D is a listing of all the water well records within the study area including domestic, commercial, municipal, irrigation and industrial wells. The following table identifies wells that are within the study area corridor. A detailed hydrogeological report was prepared by Golder & Associates that documents potential impacts to water wells and groundwater resources.

Table 2. Water Wells within the Study Area Corridor

Well Type	Deep Well (>50 ft.)	Shallow Well (<50 ft.)	Unreported Depth	Total
Domestic	19	18	0	37
Commercial	5	2	0	7
Irrigation	2	0	0	2
Industrial	2	1	0	3
Public Supply	5	1	1	7
Municipal	1	1	0	2
Not Used	4	1	3	8
Unknown	0	1	15	16
TOTALS	38	25	19	82

Information Sources

- **Water Well Data System, Groundwater Bulletin Reports, MOE**

6.1. Groundwater Recharge Areas

The study area does not have well-defined drumlins as does other areas in the Peterborough Drumlin Fields that function as important groundwater recharge areas. The Lover's Creek Infiltration area is an environmentally significant area and earth science site just east of the study area. The upland area that Highway 400 occupies in the northern portion of Innisfil Township is a non-specific groundwater recharge area and helps maintain baseflows in Lover's Creek. Community 9 between chainage 23+300 and 23+850 in Innisfil Township collects and holds considerable surface water in an area of porous soils and provides groundwater recharge functions.

Information Sources

- Natural Resource Values Information System Mapping- Ontario Ministry of Natural Resources

6.2. Groundwater Discharge Areas

Two significant groundwater discharge areas were identified within the study area. These areas were identified as having either a sub-grade source and/or temperatures consistent with groundwater sources for the area. Perhaps the most significant groundwater discharge area is between chainage 12+925 and 13+080 in Innisfil Township on the west side of the highway. There are numerous groundwater seepages at the base of the stream valley and likely groundwater upwellings in the stream channel itself. This particular location is intermediate between the two drumlin features within the study area. The other groundwater discharge area is in the Lowland Cedar Thicket vegetation community. There is abundant evidence of groundwater seepage and a rich diverse herb community consistent with groundwater influenced vegetation.

6.3. Impacts

- Approximately 0.35ha of groundwater recharge area in Community 9 may be impacted near 23+400 in Innisfil Township
- Alterations to groundwater discharges between chainage 12+925 and 13+080 in Innisfil Township on the west side of the highway

6.4. Mitigation

- The Stormwater Management and Drainage Plan will involve the creation of ponds and ditches designed to promote infiltration that will address the minor impacts to groundwater recharge functions in Community 9 at chainage 23+400.
- Realigned stream channel sections between 12+925 and 13+080 in Innisfil Township will be constructed at the base of the drumlin features matching as closely as possible the existing elevations to maintain groundwater discharges to the stream system

7.0 Terrestrial Habitat

A total of 10 distinct terrestrial forested communities were encountered within the study area composing approximately 18 percent of the study area corridor. There were also two forested wetland communities and one non-forested wetland community. Community #5 – Mixed Wood was the most common community comprising 18 percent of all the vegetation communities.

Table 3. Vegetation Community Composition of the Study Area Corridor.

Vegetation Community	Occurrence in the Study Area Corridor
#1-Hardwood Swamp	6%
#2-Mixed Wood Swamp	<1%
#3-Upland Tolerant Hardwood	15%
#4-Upland Mixed Wood	15%
#5-Mixed Wood	18%
#6-Riparian Hardwoods	8%
#7-Upland Intolerant Hardwood Thicket	5%
#8-Upland Maple	14%
#9-Narrow Emergent Marsh	6%
#10-Lowland Conifer	3%
#11-Immature Aspen Thicket	5%
#12-Lowland Cedar Thicket	3%
#13-Lowland Hardwoods	2%

7.1. Vegetation Communities

7.1.1. Community #1 – Hardwood Swamp

Plate 1. Community #1 – Hardwood Swamp (September 15, 2000)

This community is in good condition and covers a distance of approximately 640 linear metres of the study area corridor. It has a normal microclimate and a moist moisture regime. Black Ash, White Elm, Red Maple, and Silver Maple comprise the overstory layer. The shrub layer consists of White Elm, Eastern White Cedar, Black Ash, Sandbar Willow and Red-Osier Dogwood as the dominant species. Canada Golden Rod, New England Aster and Canada Anemone are common in the herbaceous layer. Fringed Brome Grass and Bushy Pasture Grass are also representative of the community. This is a wetland community and is part of the Little Lake Provincially Significant Wetland.

Table 4. Community #1- Hardwood Swamp

Common Name	Scientific Name
Trees	
Black Ash	*Fraxinus nigra*
White Elm	*Ulmus americana*
Red Maple	*Acer rubrum*
Silver Maple	*Acer saccharinum*
Shrubs	
White Elm	*Ulmus americana*
Eastern White Cedar	*Thuja occidentalis*
Black Ash	*Fraxinus nigra*
Sandbar Willow	*Salix exigua*
Red-Osier Dogwood	*Cornus stolonifera*
Herbs	
Canada Goldenrod	*Solidago canadensis*
New England Aster	*Aster novae-angliae*
Milkweed	*Asclepiadaceae spp.*
Slender White Aster	*Aster borealis*
Canada Anemone	*Anemone canadensis*
Grasses & Sedges	
Fringed Brome Grass	*Bromus ciliatus*
Bushy Pasture Spear Grass	*Poa saltuensis*

7.1.2. Community #2 – Mixed Wood Swamp

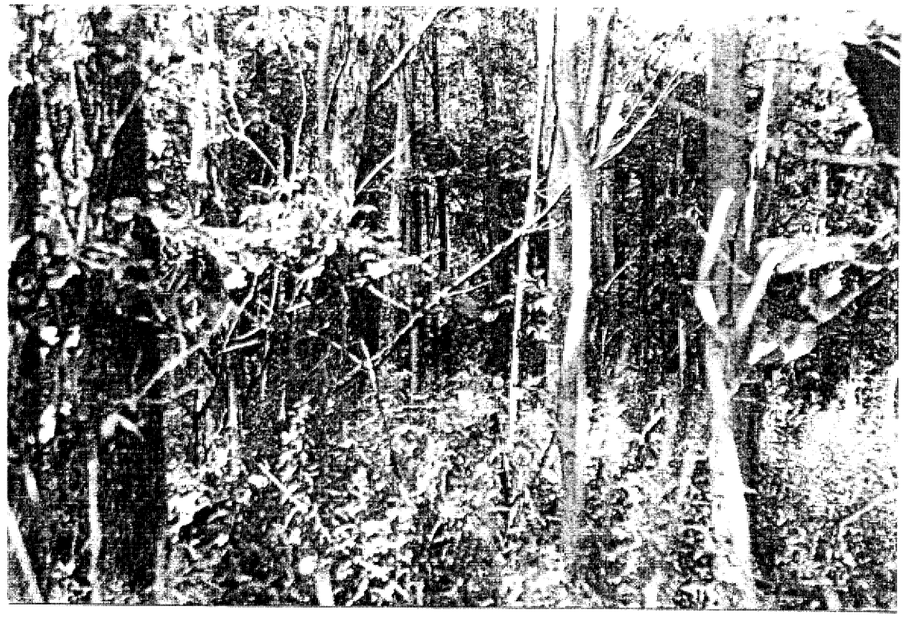


Plate 2. Community #2 - Mixed Wood Swamp (September 15, 2000)

This community is in good condition and covers a distance of only 40 linear metres of the study area corridor. It has a normal microclimate and a moist moisture regime. Black Ash, White Elm, Red Maple, and Silver Maple, Basswood and Eastern White Cedar comprise the overstory layer. The shrub layer consists of Basswood and Riverbank Grape as the dominant species. Canada Golden Rod, New England Aster and Narrow-Leaved Cattail are common in the herbaceous layer. This is a wetland community and is part of the Little Lake Provincially Significant Wetland.

Table 5. Community #2- Mixed Wood Swamp

Common Name	Scientific Name
Trees	
Black Ash	*Fraxinus nigra*
White Elm	*Ulmus americana*
Red Maple	*Acer rubrum*
Silver Maple	*Acer saccharinum*
Eastern White Cedar	*Thuja occidentalils*
Basswood	*Tilia americana*
Shrubs	
Basswood	*Tilia americana*
Riverbank Grape	*Vitis riparia*
Herbs	
Canada Goldenrod	*Solidago canadensis*
New England Aster	*Aster novae-angliae*
Milkweed	*Asclepiadaceae spp.*
Slender White Aster	*Aster borealis*
Narrow-leaved Cattail	*Typha angustifolia*
Canada Anenome	*Anenome canadensis*

7.1.3. Community #3 – Upland Tolerant Hardwood

Plate 3. Community #3 – Upland Tolerant Hardwood (September 15, 2000)

This community is in fair condition and covers a distance of 1440 linear metres of the study area corridor. It has a normal microclimate and a fresh to moderately fresh moisture regime. Hard Maple, White Elm and Buckthorn dominate the overstory. The shrub layer consists of Hard Maple, American Mountain Ash and Buckthorn. Wild Sarsaparilla, Rose-Twisted Stalk and Slender White Aster are representative of the herbaceous layer. A small patch of Black Walnut exists at chainage 16+600 in Vespra Township.

Table 6. Community #3 – Upland Tolerant Hardwood

Common Name	Scientific Name
Trees	
Hard Maple	*Acer saccharum*
White Elm	*Ulmus americana*
Buckthorn	*Rhamnus spp.*
Black Walnut	*Juglans nigra*
Shrubs	
Hard Maple	*Acer saccharum*
American Mountain Ash	*Sorbus americana*
Buckthorn	*Rhamnus spp.*
Herbs	
Wild Sarsaparilla	*Aralia Nudicaulis*
Rose-twisted Stalk	*Streptopus roseus*
Slender White Aster	*Aster borealis*

7.1.4. Community #4 – Upland Mixed Wood

Plate 4. Community #4 – Upland Mixed Wood (September 15, 2000)

This community is in good condition and covers a distance of 1480 linear metres of the study area corridor. It has a normal microclimate and a fresh moisture regime. Trees in this community include White Birch, Trembling Aspen, Largetooth Aspen, White Ash and Eastern White Cedar. The shrub layer consists of Buckthorn, American Mountain Ash, Riverbank Grape and Eastern White Cedar. There are some isolated stems of Bur Oak in areas that hold more moisture.

Table 7. Community #4 – Upland Mixed Wood

Common Name	Scientific Name
Trees	
White Birch	*Betula papyifera*
Scots Pine	*Pinus sylvestris*
Trembling Aspen	*Populus tremuloides*
Largetooth Aspen	*Populus grandidentata*
White Ash	*Fraxinus americana*
Bur Oak	*Quercus macrocarpa*
Red Maple	*Acer rubrum*
Eastern White Cedar	*Thuja occidentalils*
Eastern White Pine	*Pinus stobus*
Shrubs	
Buckthorn	*Rhamnus spp.*
Bittersweet Nightshade	*Solanum dulcamara*
Riverbank Grape	*Vitis riparia*
American Mountain Ash	*Sorbus americana*
Eastern White Cedar	*Thuja occidentalils*

7.1.5. Community # 5 – Mixed Wood

Plate 5. Community #5 – Mixed Wood (September 12, 2000)

This community is in good condition and covers a distance of 1820 linear metres of the study area corridor. It has a normal microclimate and a very fresh to moist moisture regime. Trees in this community include White Birch, Trembling Aspen, Largetooth Aspen, Black Ash, Bur Oak, Balsam Fir and Eastern White Cedar. The shrub layer consists of Red-Osier Dogwood, Nannyberry, Choke Cherry, Highbush Cranberry, Hawthorn, Riverbank Grape and Eastern White Cedar. Common Strawberry, Canada Goldenrod, Sweet Coltsfoot and Lesser Pyrola are common herbs along with Bracken Fern and Ostrich Fern.

Table 8. Community #5 – Mixed Wood

Common Name	Scientific Name
Trees	
Trembling Aspen	*Populus tremuloides*
Black Ash	*Fraxinus nigra*
Largetooth Aspen	*Populus grandidentata*
White Birch	*Betula papyifera*
Bur Oak	*Quercus macrocarpa*
Eastern White Cedar	*Thuja occidentalils*
Balsam Fir	*Abies balsamia*
Shrubs	
Red-Osier Dogwood	*Cornus stolonifera*
Nannyberry	*Viburnum lentago*
Bur Oak	*Quercus macrocarpa*
Choke Cherry	*Prunus virginiana*
Riverbank Grape	*Vitis riparia*
Eastern White Cedar	*Thuja occidentalils*
High-bush Cranberry	*Viburnum trilobum*
Hawthorn	*Crataegus spp.*
Herbs	
Common Strawberry	*Fragaria virginiana*
Canada Goldenrod	*Solidago canadensis*
Sweet Coltsfoot	*Petasites frigidus*
Lesser Pyrola	*Pyrole mineure*
White Aster	*Aster vimineus*
Ferns and Allies	
Bracken Fern	*Pteridium aulinum*
Ostrich Fern	*Matteuccia struthiopteris*
Mosses, Lichens and Liverworts	
Sickle Moss	*Sanionia uncinata*

7.1.6. Community # 6 – Riparian Hardwoods

Plate 6. Community #6 – Riparian Hardwoods (September 12, 2000)

This community is in good condition and covers a distance of 780 linear metres of the study area corridor. It has a cooler than normal microclimate and a moist moisture regime. Trees in this community include Black Willow, Manitoba Maple, White Elm, Basswood and Eastern White Cedar. The shrub layer consists of Black Ash, Choke Cherry, Highbush Cranberry and Mountain Holly. Ground cover includes Bittersweet Nightshade, Sensitive Fern and Green-Tongue Liverwort. Many locations of this community receive heavy use by white-tailed deer since the waterways provide travel corridors and the cooler than normal microclimate may provide summer thermal cover.

Table 9. Community #6 - Riparian Hardwoods

Common Name	Scientific Name
Trees	
Black Willow	*Salix nigra*
Manitoba Maple	*Acer negundo*
Hawthorne	*Crataegus spp.*
White Elm	*Ulmus americana*
Basswood	*Tilia americana*
Black Ash	*Fraxinus nigra*
Eastern White Cedar	*Thuja occidentalils*
White Ash	*Fraxinus americana*
Shrubs	
Staghorn Sumac	*Rhus typhina*
Black Ash	*Fraxinus nigra*
Mountain Holly	*Nemopanthus mucronata*
Choke Cherry	*Prunus virginiana*
High-bush Cranberry	*Viburnum trilobum*
Herbs	
Bittersweet Nightshade	*Solanum dulcamara*
Grasses & Sedges	
Canada Blue Joint	*Calamagrostis canadensis*
Ferns and Allies	
Sensitive Fern	*Onoclea sensibilis*
Mosses, Lichens and Liverworts	
Green-Tongue Liverwort	*Marchantia polymorpha*

7.1.7. Community #7 – Upland Intolerant Hardwood Thicket

Plate 7. Community #7 – Intolerant Hardwood Thicket (September 12, 2000)

This community is in poor condition and covers a distance of 520 linear metres of the study area corridor. It has a normal to warmer than normal microclimate and a moderately fresh to dry moisture regime. Trees do not dominate this community that is largely a shrub thicket. The scattered stems include White Birch, White Ash, White Elm and White Spruce. A non-native species, Amur Maple is by far the most common shrub in this community followed by Staghorn Sumac. Ground cover is very patchy since the shrub layer is extremely thick. Bracken Ferns occupy any openings in the shrub layer. Many of the White Birch in this community display signs of die-back.

Table 10. Community #7 - Upland Intolerant Hardwood Thicket

Common Name	Scientific Name
Trees	
White Birch	*Betula Papyifera*
White Ash	*Fraxinus americana*
White Elm	*Ulmus americana*
White Spruce	*Picea glauca*
Basswood	*Tilia americana*
Shrubs	
Staghorn Sumac	*Rhus typhina*
Amur Maple	*Acer ginnala maxim*
Meadowsweet	*Spiraea spp.*
Herbs	
Canada Goldenrod	*Solidago canadensis*
Milkweed	*Aslepias spp.*
Grasses & Sedges	
Fringed Brome Grass	*Bromus ciliatus*
Ferns and Allies	
Bracken Fern	*Pteridium aulinum*

7.1.8. Community #8 – Upland Maple

Plate 8. Community #8 – Upland Maple (September 12, 2000)

This community is in excellent condition and covers a distance of 1380 linear metres of the study corridor. It has a normal microclimate and a very fresh moisture regime. Trees in this community include Hard Maple, Red Oak and White Ash. The canopy is well developed and in excellent health. The ground cover consists of Sweet Cicely, Fringed Brome Grass and Plume Moss.

Table 11. Community #8 - Upland Maple

Common Name	Scientific Name
Trees	
Hard Maple	*Acer saccharum*
White Ash	*Fraxinus americana*
Red Oak	*Quercus rubra*
Shrubs	
Hard Maple	*Acer saccharum*
White Ash	*Fraxinus americana*
Herbs	
Sweet Cicely	*Osmorhiza claytonii*
Grasses and Sedges	
Fringed Brome Grass	*Bromus ciliatus*
Mosses, Lichens and Liverworts	
Plume Moss	*Ptilium crista-castrensis*

7.1.9. Community #9 – Narrow Emergent Marsh

Plate 9. Community # 9 – Narrow Emergent Marsh (September 15, 2000)

This community is in good condition and covers a distance of 550 linear metres of the study area corridor. It has a normal microclimate and a wet moisture regime. Dead trees are scattered throughout this community and include Black Ash, Eastern White Cedar and White Spruce. This marsh is dominated by Common Cattail and forms a patchwork of wetland and upland in conjunction with Community #5, Mixed Wood. The irregular boundaries of this community provide habitat for wildlife species that require edge habitats. It is situated at a relatively high elevation. It provides groundwater recharge functions and baseflow maintenance to the Lover's Creek system.

Table 12. Community #9 – Narrow Emergent Marsh

Common Name	Scientific Name
Trees	
Eastern White Cedar	*Thuja occidentalils*
Black Ash	*Fraxinus nigra*
White Spruce (Dead)	*Picea glauca*
Herbs	
Common Cattail	*Typha latifolia*

7.1.10. Community #10 – Lowland Conifer

Plate 10. Community #10 – Lowland Conifer (September 13, 2000)

This community is in fair condition and covers a distance of 300 linear metres of the study area corridor. It has a cooler than normal microclimate and a very moist moisture regime. The primary tree species are Eastern White Cedar and White Spruce with scattered Eastern White Pine. Shrubs include Buckthorn, Balsam Fir, Virginia Creeper, Round–Leaved Dogwood, Labrador Tea and Canada Yew. There is a relatively rich herb layer consisting of Wild Sarsaparilla, Jewelweed, Yellow Clintonia and Spinulose Wood Fern. There are a wide variety of mosses also. These include Sickle Moss, Common Brown Peat Moss, Spiky Dicranum and Schreber's Moss. The underlying soils are predominantly organic. This community has been compromised somewhat by the construction of a golf course adjacent to it.

Table 13. Community #10 – Lowland Conifer

Common Name	Scientific Name
Trees	
Eastern White Cedar	*Thuja occidentalils*
Eastern White Pine	*Pinus strobus*
White Spruce	*Picea glauca*
Shrubs	
Buckthorn	*Rhamnus spp.*
Balsam Fir	*Abies balsamia*
Round-leaved Dogwood	*Cornus rugosa*
Canada Yew	*Taxus canadensis*
Bittersweet Nightshade	*Solanum dulcamara*
Virginia Creeper	*Parthenocissus vitacea*
Herbs	
Wild Sarsaparilla	*Aralia nudicaulis*
Yellow Clitonia	*Clintonia borealis*
Jewelweed	*Impatiens capensis*
Ferns and Allies	
Spinulose Woodfern	*Dryopteris carthusiana*
Mosses, Lichens and Liverworts	
Sickle Moss	*Sanionia uncinata*
Screber's Moss	*Pleurozium schreberi*
Spiky Dicranum	*Dicranum flagellare*
Common Brown Peat Moss	*Sphagnum fuscum*

7.1.11. Community #11 - Immature Aspen Thicket

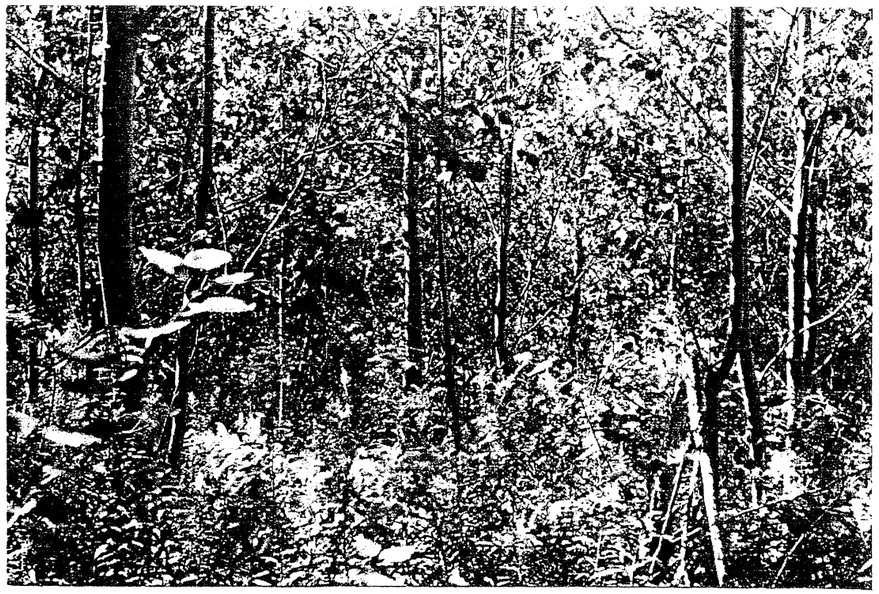


Plate 11. Community #11 – Immature Aspen Thicket (September 13, 2000)

This community is in good condition and covers a distance of 480 linear metres of the study area corridor. It has a normal microclimate and a dry to moderately fresh moisture regime. It is a monoculture of young Trembling Aspen reclaiming former open pasture. Shrubs include Staghorn Sumac and Riverbank Grape.

Table 14. Community #11 – Immature Aspen Thicket

Common Name	Scientific Name
Trees	
Trembling Aspen	*Populus tremuloides*
Shrubs	
Staghorn Sumac	*Rhus typhina*
RiverbankGrape	*Vitis riparia*
Herbs	
Canada Goldenrod	*Solidago canadensis*
Grasses & Sedges	
Timothy Grass	*Phleum pratense*

7.1.12. Community #12 – Lowland Cedar Thicket

Plate 12. Community #12 – Lowland Cedar Thicket (September 13, 2000)

This community is in good condition and covers a distance of 260 linear metres of the study area corridor. It has a cooler than normal microclimate and a moist moisture regime. It is a rich community composed of Eastern White Cedar, Black Ash, Yellow Birch, White Elm and Balsam Fir. It has a simple shrub layer consisting of Black Ash, Choke Cherry and a thick groundcover of Canada Mayflower, Jewelweed, Sensitive Fern, Oak Fern, Royal Fern, Lady Fern and Dwarf Scouring Rush. A wide variety of mosses are present including Sickle Moss, Schreber's Moss, Spiky Dicranum and Common Brown Peat Moss. The vegetation in this community is characteristic of one where the groundwater table is at or near the surface and there was evidence of groundwater seepage in several locations. There was considerable sign of white-tailed deer and a heavily used travel route parallel to the highway ROW.

Table 15. Community #12 - Lowland Cedar Thicket

Common Name	Scientific Name
Trees	
Eastern White Cedar	*Thuja occidentalils*
Black Ash	*Fraxinus nigra*
Yellow Birch	*Betula alleghaniensis*
White Elm	*Ulmus americana*
White Birch	*Betula papyifera*
Balsam Fir	*Abies balsamia*
Shrubs	
Black Ash	*Fraxinus nigra*
Choke Cherry	*Prunus vitginiana*
Herbs	
Canada Mayflower	*Maianthemum canadense*
Jewelweed	*Impatiens capensis*
Starflower	*Trientalis borealis*
Grasses & Sedges	
Fowl Manna Grass	*Glyceria striata*
Ferns and Allies	
Sensitive Fern	*Onoclea sensibilis*
Meadow Horsetail	*Equisetum pratense*
Royal Fern	*Osmunda regalis*
Dwarf Scouring Rush	*Equisetum scirpoides*
Lady Fern	*Athyrium filix-femina ssp. angustum*
Oak Fern	*Gymnocarpium dryopteris*
Spinulose Woodfern	*Dryopteris carthusiana*
Mosses, Lichens and Liverworts	
Sickle Moss	*Sanionia uncinata*
Screber's Moss	*Pleurozium schreberi*
Spiky Dicranum	*Dicranum flagellare*
Common Brown Peat Moss	*Sphagnum fuscum*

7.1.13. Community #13 – Lowland Hardwoods

Plate 13. Community #13 – Lowland Hardwoods (September 13, 2000)

This community is in good condition and covers a distance of 200 linear metres of the study area corridor. It has a normal microclimate and a moist moisture regime. It is also a rich community composed of Silver Maple, Black Ash, Basswood, White Elm and Hard Maple. It has a simple shrub layer consisting of Black Ash and White Elm. A thick groundcover of Poison Ivy, Jack-in-the-Pulpit, Sweet Coltsfoot and Clearweed is present. Ostrich Fern, Sensitive Fern, Spinulose Wood Fern, Meadow Horsetail and Plume Moss complete the groundcover.

Table 16. Community #13 - Lowland Hardwoods

Common Name	Scientific Name
Trees	
Silver Maple	*Acer saccharinum*
Black Ash	*Fraxinus nigra*
Basswood	*Tilia americana*
Hard Maple	*Acer saccharum*
White Elm	*Ulmus americana*
Shrubs	
Poison Ivy	*Toxicodendron radicans*
White Elm	*Ulmus americana*
Herbs	
Small Jack-in-the-pulpit	*Arisaema triphyllum*
Clearweed	*Pilia pumila*
Sweet Coltsfoot	*Petasites frigidus*
Grasses & Sedges	
Ferns and Allies	
Sensitive Fern	*Onoclea sensibilis*
Meadow Horsetail	*Equisetum pratense*
Ostrich Fern	*Matteuccia struthiopteris*
Spinulose Wood Fern	*Dryopteris carthusiana*
Mosses, Lichens and Liverworts	
Plume Moss	*Ptilium crista-castrensis*

In summary, vegetation communities 1, 2, 6, 8, 9, 10, 12 and 13 are considered significant. Communities 1 and 2 are significant because they are uncommon within the study area and they form part of the Little Lake PSW. Community 10 is significant since it is part of the adjacent lands of Little Lake PSW and it is a relatively uncommon community within the study area. The remaining significant communities are in excellent health, sensitive to disturbance and/or are very diverse. There is an isolated patch of Black Walnut near the Duckworth interchange that would be considered significant since they are relatively uncommon here at the northern extent of their range.

Information Sources

- Aerial photographs
- OMNR Forest Resource Inventory Maps
- OMNR Natural Resource Values Maps
- Tamarack Forest Service 1998. Honour Role of Ontario Trees

7.2. Impacts

- Approximately 0.47ha of Community 1 may be impacted in the vicinity of 18+000 and 18+350 Vespra Township
- Approximately 0.14ha of Community 2 may be impacted near 18+200 in Vespra Township

- Approximately 1.4ha of Community 6 may be impacted at two locations, 13+010 in Innisfil Township and 12+800 in Vespra Township
- Approximately 0.79ha of Community 8 may be impacted in the vicinity of 26+200 in Innisfil Township
- Approximately 0.35ha of Community 9 may be impacted near 23+400 in Innisfil Township
- Approximately 0.10ha of Community 12 may be impacted at chainage 23+100 in Innisfil Township
- Approximately 0.09ha of Black Walnut may be impacted near 16+450 in Vespra Township

7.3. Mitigation

- Use a selective vegetation removal strategy to ensure that the disturbed area is only as large as required to complete the platform widening
- Any trees that must be cut that are suitable for fish habitat compensation measures will be stockpiled and used for such works
- Individual trees of significant value (i.e. Black Walnut) shall be marked for retention wherever possible
- All exposed soils will be revegetated immediately following the completion of the required work and the final grade has been established. All seeded areas shall be inspected at 30, 60 and 90 days after seeding to ensure success of the seed mix and cover application as per Seeding and Cover Quality Assurance Visual Inspection Field Guide (MTO 1999)
- Those areas not successfully meeting the performance measures will be re-seeded and subject to a further 90-day inspection cycle

7.4. Potential Enhancements

- Plant native trees and shrubs where possible to contribute to slope stabilization, naturalize ROW and provide linkage between isolated vegetation communities
- Plant replacement Black Walnut trees to compensate for those lost

8.0 Vulnerable Threatened or Endangered (VTE) Species

The Natural Heritage Information Centre databases along with OMNR values mapping were consulted to determine if there were any documented observations for any VTE species. No observations or significant habitat for VTE species was documented within or near the study area. A list of known VTE species for Simcoe County was consulted prior to conducting field investigations to scope the range of potential VTE species (Appendix E).

Information Sources

- NHIC Database
- Natural Resource Values Maps

8.1. Impacts

- None expected

8.2. Mitigation

- None required

9.0 Significant Wetlands

The study area crosses or encroaches on two Provincially Significant Wetlands (PSW). The Little Lake PSW, near the north end of the study limits provides significant social and recreation functions given its proximity to a relatively large urban centre (Appendix F). The wetland communities within the study area provide breeding habitat for at least one provincially significant species (Caspian Tern). Willow Creek provides habitat for a number of warm and coolwater species including walleye, northern pike, yellow perch and white suckers. There is a large portion of this wetland within the study area that is fill-regulated by Nottawasaga Valley Conservation.

The second PSW is at the extreme south end of the study limits. Cookstown Hollows Swamp PSW provides significant wildlife habitat (deer wintering) and contiguous forest cover for a number of species. It also provides breeding habitat for amphibians. Many of the impacts on the features and functions of these wetlands has been dealt with under sections covering significant vegetation, fish and aquatic habitat and significant wildlife habitat

Information Sources

- NHIC Database
- Natural Resource Values Maps
- Wetland Evaluations
- Natural Area Reports

9.1. Impacts

- Approximately 1.03ha of the Cookstown Hollows Swamp PSW near 26+250 in West Gwillimbury Township will be affected by vegetation removal and filling activities
- Approximately 0.60ha of the Little Lake PSW near 18+200 in Vespra Township will be affected by vegetation removal and filling activities
- Loss of flood storage and attenuation areas

9.2. Mitigation

- A Stormwater Management and Drainage Plan will address impacts related to lost flood storage area, changes in drainage and impacts to fill-regulated areas by implementing stormwater management practices (SWMP's) such as wet ponds, dry ponds, constructed wetlands and underground storage tanks; infiltration SWMP's such as infiltration basins, infiltration trenches, sand filters and porous pavement; vegetative SWMP's such as buffer strips, grassed swales and filter strips.

10.0 Fish and Aquatic Habitat

There was very little existing information for any of the streams within the study area. Graham Findlay from OMNR Midhurst District, indicated that the Willow Creek was a typical warmwater stream containing yellow perch, northern pike and migrating walleye but was not able to supply any documented fish capture records for the area. He also indicated that recent electrofishing efforts by the Department of Fisheries and Oceans at Innisfil Creek in the vicinity of Highway 400 captured rainbow trout and sculpins indicative of a coldwater stream.

In total there are 17 stream crossings that are considered fish habitat within the study area (Appendix G). These findings were based on our field investigations and supported with secondary source information. Fish communities were sampled with a variety of gear including backpack electrofisher, mini trapnets, minnow traps and seines (Appendix H). The most significant fish habitat encountered are the two Type 1 permanent coldwater crossings at 12+879 and 13+100 in Innisfil Township. Both of these crossings are part of the same tributary of Innisfil Creek. A. total of 21 brook trout and four rainbow trout were captured at the two crossings. The catch represented a wide cross section of sizes ranging from 86mm to 221mm fork length. At least six of the captured brook trout were ripe males and one was a ripe female. The habitat was ideal for brook trout spawning with an abundance of groundwater seepages and upwellings along with suitable substrate. We suspect that the presence of ripe fish and excellent habitat indicates that this area is being used as a spawning area. The composition of the cover and available small seepage streams would also provide nursery and juvenile habitat for brook trout as well. The range of sizes of the captured brook trout supports the existence of nursery and juvenile habitat. It is interesting to note that this location is positioned between two drumlin features.

There are three Type 2 permanent coldwater crossings as 11+025, 12+200 and 12+450 in Vespra Township. We did not capture any species that would confirm the coldwater status but the temperature and origin of the streams is consistent with coldwater streams. There was also an abundance of water cress which is typical of many coldwater streams. These streams contribute baseflow downstream and likely supported limited fisheries in the past.

The remaining crossings are Type 2 seasonal or permanent habitats. They are typical warmwater communities consisting mainly of baitfish species (Appendix I). The following chart lists all of the watercrossings encountered.

10.1. Expected Impacts

In general, the fish habitat impacts are related to platform widening. All of the crossings will require culvert extensions to accommodate the additional lanes. Culvert extensions will cause a minor reduction in primary productivity as a result of shading. Many of the culverts will be in the range of 70 to 100m long. The natural meander width for many of these smaller streams would be in the order of 10m. The extended culverts effectively straighten and shorten the natural stream length, which in turn increases velocities. This changes the hydraulic properties of the reach and can increase the erosive potential. Changes in velocities that affect the erosive potential in the channel can also negatively impact benthic invertebrate distribution. The cumulative impact of culvert extensions and increase in storm water runoff in small watersheds as highways expand from two-lane to four-lane and then multi-lane can be significant. Stream channel realignments will likely be required at four of the crossings where they run parallel to the existing ROW.

Stream realignments will be necessary since the stream parallels the ROW from approximately 12+600 to 13+200.

Temporary impacts can be expected during construction including brief interruptions to fish passage, vegetation disturbance and the potential for minor sediment transport.

10.2. Typical Mitigation

Typical mitigation strategies will be employed for both the coldwater and warmwater crossings. They will include:

- No inwater work from March 15 to June 30 (warmwater)
- No inwater work from September 1 to June 1 (coldwater)
- Implement a sediment and erosion control plan with typical mitigation measures including sediment fences, check dams and/or straw bales in affected drainages and revegetation of exposed soils within 45 days of disruption
- Stockpiles will be located and isolated to ensure material will not enter any watercourse
- Culvert extensions will be done in the dry
- All pump outlets must discharge sediment laden water into a temporary sediment control basin or dense vegetation utilizing a diffusion fan and or diffusion mat before re-entering the watercourse

10.3. General Compensation Strategies

In general, the compensation strategies are intended to address the common negative conditions at most of the crossings and utilize the suggested enhancement opportunities identified during the field investigations. Invasive vegetation (cattails), at many of the crossings, chokes the stream channel to the extent that open water is limited and fish passage is difficult. The other common negative condition is periods of drought or no flow. To enhance the fish habitat on site we have recommended for many of the crossings the construction of refuge pools and the addition of hard substrate to decrease the occurrence of invasive cattails. This mitigation measure is complimentary to the recommendations in the Stormwater Management and Drainage Plan by providing energy dissipation during storm events. Table 17 summarizes the following information.

Table 17. Fisheries Values for Centreline Watercourse Crossings.

Station	Physical Description	Fish Habitat Assessment	Potential Improvements	Fish Habitat Potentially Impacted	Proposed Mitigation Measures
27+300	• Permanent warmwater tributary of Innisfil Creek • 1.20m X 0.90m X 50m Concrete Box Culvert with a 1.20m X 1.20m Corrugated Steel Pipe extension on upstream side • Stream flows through a modified drainage channel and continues west under highway. It is vegetated with cattails, sedges and rushes with very little erosion • Depth varies but averages 0.075m, width averages 0.6m and the substrate is composed of sand silt and muck	Provides unspecialized spawning, nursery, feeding, migration and cover habitat for warmwater bantfish. Habitat is degraded from straight channel ditching and roadside impacts	Culvert will be extended 28.0 on west side and 8.5m on east side to accommodate platform widening and/or improved alignment.	Some harmful alteration of fish habitat caused by proposed culvert extensions. Temporary disruption of habitat to complete work in the dry.	Generic warmwater mitigation Impose inwater timing restrictions between March 15-June 30.
27+327	• Permanent warmwater tributary of Innisfil Creek • 2.45m X 1.50m X 51.8m Concrete Box Culvert • Stream flows west under the highway through dense cattails and reed canary grass with no erosion • Depth averages 0.20m, width averages 0.5m and the substrate is composed of gravel, sand, silt and clay	Provides unspecialized spawning, nursery, feeding, migration and cover habitat for warmwater bantfish. Habitat is heavily modified and degraded from interchange ramps, drainage and vegetation management	Culvert will be extended 12.0 on the west side and 18.5m on the east side to accommodate platform widening and/or improved alignment	Some harmful alteration of fish habitat caused by proposed culvert extensions. Temporary disruption of habitat to complete work in the dry.	Generic warmwater mitigation Impose inwater timing restrictions between March 15-June 30
10+282	• Permanent warmwater tributary of Innisfil Creek • 4.30m X 1.50m X 29.5m Concrete Box Culvert • Stream flows east under the highway through grasses, shrubs and herbs • High degree of pollution and degraded water quality • Depth averages 0.5m, width averages 0.5m and the substrate is composed of muck	Provides unspecialized spawning, nursery, feeding, migration and cover habitat for warmwater bantfish. Water quality is severely degraded from nutrient enrichment and excessive algae growth.	Culvert will be extended 11.5m on the west side and 37.5m on the east side to accommodate platform widening and/or improved alignment.	Some harmful alteration of fish habitat caused by proposed culvert extensions. Temporary disruption of habitat to complete work in the dry.	Generic warmwater mitigation Impose inwater timing restrictions between March 15-June 30.
10+760	• Permanent coldwater - Innisfil Creek • 2 (5030mm X 3050mm X 43.00m) Corrugated Steel Pipe Arch & 1(2300mm X 61.00m) Corrugated Steel Pipe • Stream flows west under the highway. The banks are vegetated with grasses and sedges with some erosion, culvert perching and distortion • Depth varies but averages 0.5m, channels are ponded near the culvert ends. The substrate includes sand, clay and muck.	Provides unspecialized spawning, nursery, feeding, migration and cover habitat for bantfish and juvenile habitat for rainbow trout. Habitat has been severely degraded on the upstream side from the Innisfil Creek Golf Course.	CSPA Culverts will be extended approx. 16.5m on the west side and 15m on the east side. The CSP culvert will be extended 17m on the west side and 15m on the east side in order to accommodate platform widening and/or improved alignment.	Some harmful alteration of fish habitat caused by proposed culvert extensions. Temporary disruption of habitat to complete work in the dry.	Generic coldwater mitigation Impose inwater timing restrictions between September 15 - July 7 (Rainbow Trout)
11+688	• Permanent warmwater tributary of Innisfil Creek • 5.0m X 3.0m X 42.70m Corrugated Steel Pipe Arch • Stream flows east under the highway. The banks are vegetated with grasses and herbs. The channel is choked with invasive vegetation. • Average depth is 0.3m with the width being 0.75m.	Provides unspecialized spawning, nursery, feeding, migration and cover habitat for warmwater bantfish.	Culvert will be extended 23.5m on the west side to accommodate platform widening and/or improved alignment	Some harmful alteration of fish habitat caused by proposed culvert extensions. Temporary disruption of habitat to complete work in the dry.	Generic warmwater mitigation Impose inwater timing restrictions between March 15-June 30.

Table 17. Fisheries Values for Centreline Watercourse Crossings. (continued)

Crossing # and Chainage	Physical Description	Fish Habitat Assessment	Proposed Improvements	Fish Habitat Potentially Impacted	Proposed Mitigation Measures
12+879 & 13+100	• Permanent coldwater tributary of Innisfil Creek • 12+879 - 4.25m X 1.50m X 105m Concrete Box Culvert 13+100 - 4.25m X 1.50m X 95m Concrete Box Culvert • Stream flows west under the highway at 13+100 and continues east under the hagbway at 13+100. The banks and channel are well vegetated with riparian hardwoods and cedar. • Average depth is 0.20m and average width is 2.0m. Substrate is composed of gravel, rubble, detritus, sand, marl, clay, silt and boulders • Significant groundwater discharges occur in the vicinity of these crossings • Generally, the most significant habitat occurs on the west side of the highway	Provides specialized spawning, nursery, feeding, migration and cover habitat for coldwater species including brook and rainbow trout. Habitat is somewhat degraded in the ROW from vegetation clearing.	Culvert at 12+879 will be extended 26.5m on the west side and the culvert at 13+100 will be extended 31m on the west side.	Harmful alteration of fish habitat caused by proposed culvert extensions. Possible +/- 200m stream realignment on west side. Significant removal of riparian vegetation and potential interference with groundwater discharges. Temporary disruption of habitat to complete work in the dry.	Generic coldwater mitigation Impose inwater timing restrictions between September 15 - July 7 (Rainbow & Brook Trout)
16+722	• Permanent warmwater tributary of Innisfil Creek • 7.30m X 4.00m X 60m Concrete Arch Culvert • Stream flows east under the highway. The banks and channel are vegetated with grasses, sedge and marigold with little bank erosion limited to minor undercutting • Average depth is 0.20m and average width is 1.0m. The substrate is primarily gravel with some silt ,sand, boulder, detritus, clay and rubble	Provides unspecialized spawning, nursery, feeding, migration and cover habitat for warmwater bantfish. Culvert is perched on the downstream end and prevents fish passage.	Culvert will be extended 24.0m on the west side to accommodate platform widening and/or improved alignment. May also require some channel realignment (+/- 20m) at upstream end.	Some harmful alteration of fish habitat caused by proposed culvert extensions and stream alignment. Temporary disruption habitat to complete work in the dry.	Generic warmwater mitigation Impose inwater timing restrictions between March 15-June 30.
18+400	• Seasonal warmwater tributary of Lovers Creek • 1.50m X 1.20m X 52.0m Concrete Culvert • Stream flows east under highway. The banks and channel are heavily vegetated with cattail, sedges, reed canary grass and shrubs • Average depth is 0.15m and average width is 0.6m. The substrate includes gravel, sand, silt and clay	Provides unspecialized spawning, nursery, feeding, migration and cover habitat for warmwater bantfish. Fish passage is limited by low flows and terrestrial vegetation in the channel	Culvert will be extended 24m on the west side to accommodate platform widening and/or improved alignment.	Some harmful alterations of fish habitat caused by proposed culvert extensions and new ramp crossings. Temporary disruption of habitat to complete work in the dry.	Generic warmwater mitigation Impose inwater timing restrictions between March 15-June 30.
23+273	• Permanent warmwater tributary of Lovers Creek • 1.20m X 0.90m X 50.9m Open Concrete Box Culvert • Stream flows east from a lowland cedar thicket. The channel is heavily vegetated with cattails, grasses and various herbs. • Average depth is 0.75m and average width is 2.0m. Substrate is muck and detritus	Provides unspecialized spawning, nursery, feeding, migration and cover habitat for warmwater bantfish. Some evidence of nutrient enrichment and excessive algae growth.	Culvert will be extended 23.5m on the west side to accommodate platform widening and/or improved alignment.	Some harmful alterations of fish habitat caused by proposed culvert extensions. Temporary disruption of habitat to complete work in the dry.	Generic warmwater mitigation Impose inwater timing restrictions between March 15-June 30.
26+750	• Permanent warmwater tributary of Kempenfelt Bay • 1.22m X 0.90m (east) & 1.22m X 1.22m X 89.4m (west) Concrete Box Culvert • Stream flows east under the highway. The channel is heavily vegetated with cattails and sedges, staghorn sumach, jewelweed and Canada goldenrod with no apparent erosion • Average depth is 0.2m and average width is 0.5m, substrate is sand, gravel, silt and rubble	Provides unspecialized spawning, nursery, feeding, migration and cover habitat for warmwater bantfish.	Culvert will be extended 23.0m on the west side to accommodate platform widening and/or improved alignment.	Some harmful alterations of fish habitat caused by proposed culvert extensions. Temporary disruption of habitat to complete work in the dry.	Generic warmwater mitigation Impose inwater timing restrictions between March 15-June 30.

Table 17. Fisheries Values for Centreline Watercourse Crossings. (continued)

Crossing # and Chainage	Physical Description	Fish Habitat Assessment	Proposed Improvements	Fish Habitat Potentially Impacted	Proposed Mitigation Measures
29+282	• Seasonal warmwater tributary of Kempenfelt Bay • 1830mm X 76m Corrugated Steel Pipe • Stream flows east under the highway. Bank vegetation consists of grasses and herbs • Average depth is 0.2m and average width is 0.76m, substrate is, gravel, sand and silt	Provides unspecialized spawning, nursery, feeding, migration and cover habitat for warmwater baitfish	Culvert will be extended 13.5m on the west side and 13.0m on the east side to accommodate platform widening and/or improved alignment	Some harmful alterations of fish habitat caused by proposed culvert extensions. Temporary disruption of habitat to complete work in the dry	Generic warmwater mitigation Impose inwater timing restrictions between March 15 June 30
11+025	• Permanent coldwater tributary of Kempenfelt Bay • 1.85m X 1.20m X 83.0m Concrete Box Culvert • Stream flows east through the interchange at Dunlop Street and under the highway. Riparian vegetation includes Canada rush, water cress, purple loosestrife, grasses, horsetail and sedges with a relatively stable bank • Average depth is 0.2m and average width is 0.75m, substrate is boulders, rubble, gravel, sand, silt and clay	Provides unspecialized spawning, nursery, feeding, migration and cover habitat for resident species. Also provides cooler water providing temperature moderation	Culvert will be extended 34m on the west side and 5m on the east side to accommodate platform widening and/or improved alignment	Some harmful alterations of fish habitat caused by proposed culvert extensions and stream realignments. Temporary disruption of habitat to complete work in the dry	Generic coldwater mitigation Impose inwater timing restrictions between September 1 July 1
12+200	• Permanent coldwater tributary of Kempenfelt Bay • 1.20m X 1.20m X 52.0 Concrete Box Culvert • Stream flows east under highway. Vegetation consists of dense grasses, water cress and herbs • Average depth is 0.15m and average width is 0.5m, substrate is rubble, gravel, sand and clay	Provides unspecialized spawning, nursery, feeding, migration and cover habitat for resident species. Also provides cooler water providing temperature moderation	Culvert will be extended 14.5m on the west side and 5.5m on the east side to accommodate platform widening and/or improved alignment	Some harmful alterations of fish habitat caused by proposed culvert extensions and stream realignments. Temporary disruption of habitat to complete work in the dry	Generic coldwater mitigation Impose inwater timing restrictions between September 1 July 1
12+450	• Permanent coldwater tributary of Kempenfelt Bay • 1.20m X 0.91m X 56.0 Concrete Box Culvert • Stream flows east under highway. Vegetation consists of dense grasses, Canada goldenrod and herbs. • Average depth is 0.10m and average width is 0.4m, substrate is rubble, gravel, sand, silt and muck	Provides unspecialized spawning, nursery, feeding, migration and cover habitat for resident species. Also provides cooler water providing temperature moderation	Culvert will be extended 10.5m on the west side and 10.5m on the east side to accommodate platform widening and/or improved alignment	Some harmful alterations of fish habitat caused by proposed culvert extensions and stream realignments. Temporary disruption of habitat to complete work in the dry	Generic coldwater mitigation Impose inwater timing restrictions between September 1 July 1

Table 17. Fisheries Values for Centreline Watercourse Crossings. (continued)

Crossing # and Chainage	Physical Description	Fish Habitat Assessment	Proposed Improvements	Fish Habitat Potentially Impacted	Proposed Mitigation Measures
12+800	• Permanent warmwater tributary of Kempenfelt Bay • 2.40m X 1.80m X 82.0m Concrete Box Culvert • Upstream treatments around culvert include Gabion Baskets and concrete slope stabilization at outlet from stormwater management basin • Stream flows east under highway. Vegetation consists of silver poplar, grasses and herbs • Average depth is 0.10m and average width is 0.8m, substrate is rubble, gravel, sand, silt and muck	Provides unspecialized spawning, nursery, feeding, migration and cover habitat for warmwater baitfish. Stormwater management basin prevents upstream movement of fish	Culvert will be extended 13.5m on the west side and 9.0m on the east side to accommodate platform widening and/or improved alignment	Some harmful alterations of fish habitat caused by proposed culvert extensions and stream realignments. Temporary disruption of habitat to complete work in the dry	Generic warmwater mitigation Impose inwater timing restrictions between March 15 June 30
18+280	• Permanent warmwater - Willow Creek • Three span steel beams bridge (site 30 139) • Associated with Little Lake PSW • Stream flows west under highway. Vegetation consists of grasses, sedges herbs and shrubs. • Average depth is 0.80m and average width is 8.4m, substrate is sand, silt, detritus and muck	Provides unspecialized spawning, nursery, feeding, migration and cover habitat for warmwater baitfish, yellow perch, northern pike and provides a migration route for walleye	Bridge will require rehabilitation to accommodate platform widening	Some harmful alterations of fish habitat caused by proposed bridge rehabilitation works. Temporary disruption of habitat during construction	Generic warmwater mitigation Impose inwater timing restrictions between March 15 June 30

10.3.1. Station 27+300 West Gwillimbury Township

10.3.1.1. Impacts

This crossing is a tributary of Innisfil Creek and supports a warmwater baitfish community. The existing structure is a concrete box culvert. The widening of the highway platform will require an extension to the existing structure.

- The culvert will be extended ~28.0m on the west (downstream) side
- The culvert will be extended ~8.5m on the east (upstream) side
- There will be some minor loss of primary productivity resulting in a HADD

10.3.1.2. Compensation Strategy

- Construct a small pool in the stream channel on the downstream end of the culvert to dissipate flow energy and provide refuge to resident fish during low flows.
- Add hard substrate (gravel 80%, cobble 20%) to the stream bottom within the ROW to diversify habitat and reduce vegetation barriers within the channel

10.3.2. Station 27+527 West Gwillimbury Township

10.3.2.1. Impacts

This crossing is a tributary of Innisfil Creek and supports a warmwater baitfish community. The existing structure is a concrete box culvert. The widening of the highway platform will require an extension to the existing structure.

- The culvert will be extended ~12.0m on the west (downstream) side
- The culvert will be extended ~18.5m on the east (upstream) side
- A new culvert will be installed at approximately 10+140 on the W-S ramp
- New culverts will be installed at approximately 10+345 and 10+080 on the W-N ramp
- There will be some minor loss of primary productivity resulting in a HADD

10.3.2.2. Compensation Strategy

- Plant native shrub species in the riparian zone within the ROW to improve shading, bank stability and buffering qualities

10.3.3. Station 10+282 Innisfil Township

10.3.3.1. Impacts

This crossing is a tributary of Innisfil Creek and supports a limited warmwater baitfish community. The existing structure is a concrete box culvert. The widening of the highway platform will require an extension to the existing structure.

- The culvert will be extended ~11.5m on the west (upstream) side
- The culvert will be extended ~37.5m on the east (downstream) side
- There will be some minor loss of primary productivity resulting in a HADD

10.3.3.2. Compensation Strategy

- Plant native shrub species in the riparian zone within the ROW to improve shading, bank stability and buffering qualities

10.3.4. Station 10+760 Innisfil Township

10.3.4.1. Impacts

This crossing is Innisfil Creek and supports a coldwater and baitfish community. The existing structures include two corrugated steel pipe arches and a corrugated steel pipe. The widening of the highway platform will require an extension to the existing structure.

- The CSPA's will be extended ~11.5m on the west (downstream) side
- The CSPA's will be extended ~15.0m on the east (upstream) side
- The CSP will be extended ~17.0m on the west (downstream) side
- The CSP will be extended ~15.0m on the east (upstream) side
- There will be some minor loss of primary productivity resulting in a HADD

10.3.4.2. Compensation Strategy

- Repair perched culverts on downstream side to allow fish passage
- Plant native shrub species in the riparian zone within the ROW to improve shading, bank stability and buffering qualities
- Add woody debris (i.e. root wads and/or tree tops) anchored into the bank to improve fish cover habitat

10.3.5. Station 11+688 Innisfil Township

10.3.5.1. Impacts

This crossing is a tributary of Innisfil Creek and supports a warmwater baitfish community. The existing structure is a corrugated steel pipe arch. The widening of the highway platform will require an extension to the existing structure.

- The culvert will be extended ~23.5m on the west (upstream) side
- There will be some minor loss of primary productivity resulting in a HADD

10.3.5.2. Compensation Strategy

- Plant native shrub species in the riparian zone within the ROW to improve shading, bank stability and buffering qualities
- Add gravel (<5cm diametre) at the outlet of pool on downstream side to improve spawning facilities for creek chub

10.3.6. Station 12+879 & 13+100 Innisfil Township

10.3.6.1. Impacts

This crossing is a tributary of Innisfil Creek and supports a high quality coldwater community. These two crossings were sampled together since the normal stations overlap. The sampling station included 20m upstream of the 13+100 culvert, continues downstream for 220m to the 12+879 culvert and continues 100m downstream.

- The culvert at 12+879 will be extended ~6.5m on the west (upstream) side
- The culvert at 13+100 will be extended ~31.0m on the west (downstream) side
- Approximately 220m of specialized coldwater spawning habitat will be lost on the west side of the highway from approximately 12+920 to 13+080 due to platform widening
- There will be some minor loss of primary productivity resulting in a HADD

10.3.6.2. Compensation Strategy

- Plant native shrub species in the riparian zone within the ROW on the east side of the highway downstream of culvert 12+879 to improve shading, bank stability and buffering qualities
- Plant native shrub species in the riparian zone within the ROW on the east side of the highway upstream of culvert 13+100 to improve shading, bank stability and buffering qualities

- Construct a new, natural channel system on the west side of the highway to replace the existing displaced channel
- Addition of woody debris in riffle margins to improve nursery habitat for coldwater species

10.3.7. Station 16+722 Innisfil Township

10.3.7.1. Impacts

This crossing is a tributary of Innisfil Creek and supports a warmwater baitfish community. The existing structure is a concrete arch culvert. The widening of the highway platform will require an extension to the existing structure.

- The culvert will be extended ~24.0m on the west (upstream) side
- Approximately 20m of stream channel on the upstream side may be realigned to connect to the extended culvert
- There will be some minor loss of primary productivity resulting in a HADD

10.3.7.2. Compensation Strategy

- Create a low flow channel within the existing culvert
- Adjust streambed elevation on downstream end of culvert to remove fish passage barrier created by perched culvert

10.3.8. Station 18+400 Innisfil Township

10.3.8.1. Impacts

This crossing is a seasonal tributary of Lovers Creek and supports a warmwater baitfish community downstream of the ROW. The existing structure is a concrete box culvert. The widening of the highway platform will require an extension to the existing structure.

- The culvert will be extended ~24.0m on the west (upstream) side

10.3.8.2. Compensation Strategy

- None required. The culvert extension and abundance of vegetation within the channel will shade and buffer the water that contributes to fish habitat downstream of the study area.

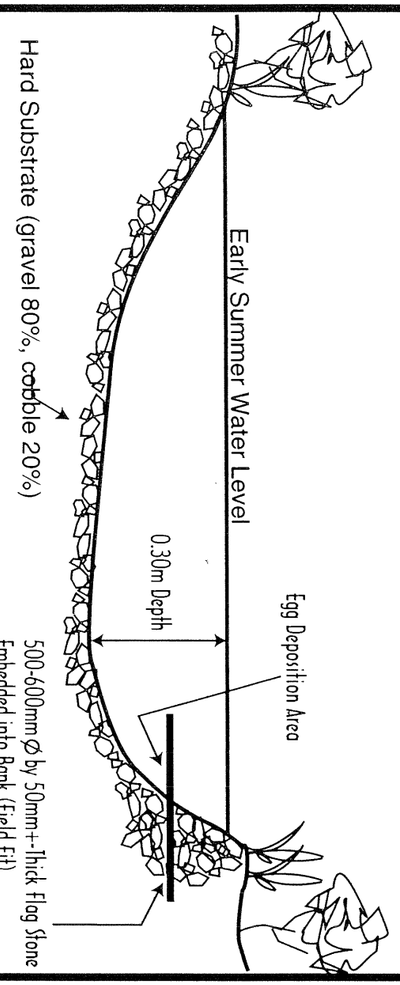


Figure 12. Conceptual Drawing of Fathead and Bluntnose Minnow Spawning Structure

Scale: not to scale

Remarks:

Two to three flagstone structures could be added to the pool to create or enhance spawning habitat for fathead and bluntnose minnows. The refuge pool will help ensure that suitable water levels are available during the incubation period.

Prepared by:

FRi ECOLOGICAL SERVICES

- There will be some minor loss of primary productivity resulting in a HADD

10.3.11.2. Compensation Strategy

- Construct a small pool in the stream channel on the downstream end of the culvert to dissipate flow energy and provide refuge to resident fish during low flows.
- Plant native shrub species in the riparian zone within the ROW to improve shading, bank stability and buffering qualities

10.3.12.Station 11+025 Vespra Township

10.3.12.1. Impacts

This crossing is a permanent coldwater tributary of Kempenfelt Bay. The existing structure is a concrete box culvert. The widening of the highway platform will require an extension to the existing structure.

- The culvert will be extended ~34.0 on the west (upstream) side
- The culvert will be extended ~5.0m on the east (downstream) side
- There will be some minor loss of primary productivity resulting in a HADD

10.3.12.2. Compensation Strategy

- Plant native shrub species in the riparian zone within the ROW to improve shading, bank stability and buffering qualities

10.3.13.Station 12+200 Vespra Township

10.3.13.1. Impacts

This crossing is a permanent coldwater tributary of Kempenfelt Bay. The existing structure is a concrete box culvert. The widening of the highway platform will require an extension to the existing structure.

- The culvert will be extended ~14.5m on the west (upstream) side
- The culvert will be extended ~5.5m on the east (downstream) side
- There will be some minor loss of primary productivity resulting in a HADD

10.3.13.2. Compensation Strategy

- Plant native shrub species in the riparian zone within the ROW to improve shading, bank stability and buffering qualities

10.3.14.Station 12+450 Vespra Township

10.3.14.1. Impacts

This crossing is a permanent coldwater tributary of Kempenfelt Bay. The existing structure is a concrete box culvert. The widening of the highway platform will require an extension to the existing structure.

- The culvert will be extended ~10.5 on the west (upstream) side
- The culvert will be extended ~10.5m on the east (downstream) side
- There will be some minor loss of primary productivity resulting in a HADD

10.3.14.2. Compensation Strategy

- Plant native shrub species in the riparian zone within the ROW to improve shading, bank stability and buffering qualities

10.3.15.Station 12+800 Vespra Township

10.3.15.1. Impacts

This crossing is a permanent warmwater tributary of Kempenfelt Bay and supports a warmwater baitfish community. The existing structure is a concrete box culvert. A stormwater basin creates an upstream barrier to fish passage. The widening of the highway platform will require an extension to the existing structure.

- The culvert will be extended ~13.5m on the west (upstream) side
- The culvert will be extended ~9.0 on the east (downstream) side
- There will be some minor loss of primary productivity resulting in a HADD

10.3.15.2. Compensation Strategy

- Plant native shrub species in the riparian zone within the ROW on downstream side to improve shading, bank stability and buffering qualities

10.3.16.Station 18+280 Vespra Township

10.3.16.1. Impacts

This crossing is Willow Creek; a permanent warmwater community supporting baitfish and sportfish. The existing structure is a three span, steel beam bridge. The widening of the highway

platform will require rehabilitation to the existing structure to facilitate taper to four-lane highway.

- None expected

10.3.16.2. Compensation Strategy

- None required

Information Sources

- MNR Fishery Files for Willow and Innisfil Creeks
- Natural Resource Values Maps

11.0 Significant Wildlife Habitat

Secondary source information indicated three areas of significant deer wintering habitat as indicated on the natural heritage mapping. Contiguous conifer cover near a suitable food source is not in great supply within the study area. The identified areas are fairly specialized habitat and tend to have lowland conifer cover with areas of thick shrub growth. The large wetland complex east of Highway 400 is one of the few examples near the study area that has reasonably good contiguous forest cover. The combination of lowland conifers and groundwater sources provide excellent summer thermal cover for ungulates. In general, the wetland habitats provide virtually the only locations suitable for a wide range of amphibian breeding and are significant for this reason.

The combination of lowland conifers and groundwater sources provides excellent summer thermal cover for ungulates. This type of habitat is available in Community #12 described in Section 7.1.12 (Figure 6).

The Canada Land Inventory Mapping for land capability for waterfowl production indicates that Little Lake and its associated wetlands are represented as Class 4, moderate limitations to the production of waterfowl. The Innisfil Creek/Lovers Creek wetland complex is identified as Class 3S, slight limitations to the production of waterfowl and serve as important migration stops. The remainder of the study area is identified as class 7, severe limitations to the production of waterfowl.

Secondary Sources Consulted

- CLI Waterfowl Production Capability Maps
- Natural Resource Values Maps
- NHIC DataBase

- Wildlife Values Maps

11.1. Impacts

- Loss of 1.02ha of deer wintering area

11.2. Mitigation

- Use a selective vegetation removal strategy to ensure that the disturbed area is only as large as required to complete the platform widening
- Any trees that must be cut that are suitable for fish habitat compensation measures will be stockpiled and used for such works

11.3. Potential Enhancements

- Plant native conifers within ROW where possible to provide linkage between isolated forested deer wintering areas

12.0 Wildlife Linkage/Corridor

Our field investigations focussed on lateral linkages that may be affected by the expansion of the highway footprint. Cross-linkages are already negatively impacted by the present footprint and would experience an incremental cumulative impact by increasing the width of the highway corridor. We located three important linkages that connect distinct habitat types. The most evident use was by white-tailed deer but a wide range of wildlife use wetlands and stream valleys as travel corridors. One area in Community #6 at chainage 12+920 in Innisfil Township provides a significant linkage between riparian hardwoods and abandoned pasture areas important for feeding (Figure 3). This vegetation community offers heavy cover and acts as a travel corridor between feeding and resting areas for deer. Another important linkage occurs at chainage 18+402 in Innisfil Township on the east side of the highway (Figure 5). This corridor is mainly used by white-tailed deer travelling along the edge of the large Innisfil Creek/Lovers Creek wetland complex. This linkage area is nearly 200m to the east of the existing ROW. The final key corridor is at chainage 23+130 in Innisfil Township that links a mixed wood community to open pasture and a lowland conifer community (Figure 6). This is also primarily used by white-tailed deer but is used by other species as well (i.e. racoon, red fox).

Information Sources

- Aerial Photographs
- Natural Resource Values Maps

12.1. Impacts

- Loss of linkage associated with communities 11 and 6 at 12+920 due to platform widening

12.2. Mitigation

- Plant native conifers within ROW where possible to provide cover and linkage between isolated vegetation communities

13.0 Significant Areas of Natural and Scientific Interest (ANSI)

There are no areas of natural and scientific interest within the study area corridor.

Information Sources

- NHIC Database
- Natural Resource Values Maps

Table 18. Summary Table of Environmental Conditions.

Natural Environment Factor	Description	Quantity Within the Study Area Corridor
Soils	Soil capability class 1	27%
	Soil capability class 2	7%
	Soil capability class 3	25%
	Soil capability class 4	12%
	Soil capability class 5	16%
	Soil capability class 6	0%
	Soil capability class 7	10%
	Organic soils	3%
Groundwater Recharge	Groundwater recharge areas include communities #1, 2 and 9.	12,230 linear metres (l.m.)
Groundwater Discharge	12+925-13+080 Innisfil Township 23+150-23+272 Innisfil Township	155 l.m. 122 l.m.

Table 18. Summary Table of Environmental Conditions (continued).

Natural Environment Factor	Description	Quantity Within the Study Area Corridor
Area of Forested Land	12 Natural forest communities were documented	16%
Distinct Vegetation Communities	Community #1 – Hardwood Swamp	1%
	Community #2 - Mixed Wood Swamp	<1%
	Community #3 – Upland Tolerant Hardwood	2%
	Community #4 – Upland Mixed Wood	2%
	Community #5 – Mixed Wood	3%
	Community #6 – Riparian Hardwood	1%
	Community #7 – Upland Intolerant Hardwood Thicket	1%
	Community #8 – Upland Maple	1%
	Community #9 – Narrow Emergent Marsh	1%
	Community #10 – Lowland Conifer	1%
	Community #11 – Immature Aspen Thicket	1%
	Community #12 – Lowland Cedar Thicket	<1%
	Community #13 – Lowland Hardwoods	<1%
Area of Significant Habitat of VTE Species	None were documented.	0
Provincially Significant Wetlands	Little Lake PSW	600 l.m.
	Cookstown Hollows Swamp PSW.	650 l.m.
Area of Wetland Communities	3 wetland vegetation communities were documented (all are represented as distinct vegetation communities)	3%
Distinct Wetland Communities	Community #1 – Hardwood Swamp	1%
	Community #2 – Mixed Wood Swamp	<1%
	Community #9 – Narrow Emergent Marsh	1%
Fish Habitat Crossings	Centreline crossings of streams with confirmed fish habitat.	17

Table 18. Summary Table of Environmental Conditions (continued).

Natural Environment Factor	Description	Quantity Within the Study Area Corridor
Type 1 Coldwater Permanent Fish Habitat Encroachments	12+650-12+880 Innisfil Township 12+920-13+080 Innisfil Township 13+140-13+300 Innisfil Township	3
Type 1 Coldwater Permanent	12+920 Innisfil Township 13+120 Innisfil Township	2
Type 2 Coldwater Permanent	10+770 Innisfil Township (Innisfil Creek) 11+030 Vespra Township 12+200 Vespra Township 12+448 Vespra Township	4
Type 2 Warmwater Permanent	27+300 West Gwillimbury Township 27+527 West Gwillimbury Township 10+280 Innisfil Township 11+680 Innisfil Township 16+724 Innisfil Township 23+272 Innisfil Township 26+750 Innisfil Township 29+280 Innisfil Township 12+817 Vespra Township 18+820 Vespra Township	10
Type 2 Warmwater Seasonal	18+820 Vespra Township	1
Significant Wildlife Habitat	Deer wintering habitat. 26+000-26+690 West Gwillimbury 10+280-10+650 Innisfil Township 16+550-16+750 Innisfil Township	 690 l.m. 370 l.m. 200 l.m.
Wildlife Linkage/Corridor	Significant linkages or travel corridors between critical habitats especially for deer.	3
ANSI's	None were documented	0
Fill Regulated Lands	Areas regulated by Simcoe Region Conservation, Nottawasaga Valley Conservation and the City of Barrie	9,500 linear metres

APPENDIX A
Correspondence

November 20, 2000

Ministry of the Environment
Water Well Record Department
125 Resources Rd., West Wing Upper Level
Etobicoke, ON M9P 3V6

Dear Ms. Yee:

Subject: **Water Well Records**

I am submitting this request in reference to our interest in acquiring Water Well Records for a section of land along Highway 400 from Highway 89 north to the junction at Highway 11. The area of interest includes a 30km section of Highway with a width of approximately 200m on either side of the highway (400 m in total).
The records requested are as follows:
The County of Simcoe, Township of Innisfil, Lots 6,7 for Concessions 1-9 and 14.
The County of Simcoe, Township of Innisfil, Lots 7,8 for Concessions 10-13.
The City of Barrie, Township of Vespra, Lots 23,24,25, Concession 6.
The City of Barrie, Township of Vespra, Lot 23, Concession 5.
The City of Barrie, Township of Vespra, Lot 21, Concession 4.
The City of Barrie, Township of Vespra, Lots 21,22, Concession 3.
The City of Barrie, Township of Vespra, Lots 4-10, 1WPR.

If you have any further questions, feel free to contact us. Thank you for your time.

Yours truly,
Nena Hulse
FRi Ecological Services

CONTACT MEMORANDUM

Contact Name: Graham Findlay Agency/Organization: MNR - Midhurst Address: Midhurst ON L0L 1X0	FRi Staff: RSB Date: Sept. 13/2000 Phone: 705-725-7530 Fax:
Project Name: Hwy 400	Project/File No.: CN29900147 GWP #30-95-00

Re: Fisheries Information

- I contacted Graham to obtain information on any of the 42 water crossings within the study limits
- He could only offer very general information on Willow + Innisfil CKs.
- Willow CK is a warmwater creek with yellow perch, pike + migrating walleye
- Innisfil CK is a coldwater system + recent electrofishing has turned up rainbow trout + sculpins.

Action:

Distribution List: - to file
- include in report

CONTACT MEMORANDUM

Contact Name: John Struick Agency/Organization: City of Barrie. Address: 70 Collier St.	FRi Staff: RSB Date: Sept. 13/2000 Phone: Fax:
Project Name: Hwy 400	Project/File No.: CN29900147 GWP #30-95-00

Re: Fill Regulation Mapping.

- Met with John Struick and reviewed existing fill regulation mapping adjacent to Hwy 400 corridor
- Transferred information on to ETR Plates

Action:

- John said he could supply copies of the maps if they are required at a later date.

Distribution List: - to file
- include in report

CONTACT MEMORANDUM

Contact Name: Dave Burrith Agency/Organization: N.V.C.A. Address: Argus ON L0M 1B0 RR#1	FRi Staff: RSB. Date: Sept. 12/2000 Phone: 705-424-1479 Fax: 705-424-2115
Project Name: Hwy 400	Project/File No.: CN29900147 GWP # 30-95-00

Re: Fill Regulation Mapping.

- Met with Dave, he provided Fill reg. mapping for Innisfil CK + Willow CK.
- He suggested I contact the City of Barrie to obtain Flood + Fill mapping within the city limits

Action:
- Make appointment with John Struick from the City of Barrie on Sept. 13/2000

Distribution List: - to File
- include in report

CONTACT MEMORANDUM

Contact Name: Paul Jurjans / Brad Allan Agency/Organization: MNR - Midhurst Address: Midhurst ON L0L 1X0	FRi Staff: RSB Date: Sept. 11/2000 Phone: 705-725-7500 Fax:
Project Name: Hwy. 400	Project/File No.: CN29900147 GWP # 30-95-00

Re: G.I.S. Mapping + Fill Regulation Mapping.

- Met with P. Jurjans + B. Allan
- Paul provided G.I.S. mapping for ANSI, wetlands, wildlife habitat, fish habitat
- B. Allan suggested I contact Graham Findlay to access any relevant fisheries files
- Gary Allan indicated that they did not have any Fill regulation mapping and to contact Dave Burrith at N.V.C.A.

Action:
- Make appointment with Graham Findlay.
- Meet with Dave Burritt at N.V.C.A.

Distribution List: - to file
- include in report.

CONTACT MEMORANDUM

Contact Name: Tim Salkeld Agency/Organization: Nottawasaga Valley CA. Address: R.R #1 Angus ON L0M 1B1	FRi Staff: RSB Date: Sept. 5/2000 Phone: 705-424-1479 Fax: 705-424-2115
Project Name: Hwy. 400	Project/File No.: CN29900147 GWP #30-95-00

Re: Fill Regulation Mapping.

- Tim suggested that I contact MNR-Midhurst to obtain mapping

Action:

- Meet with Gary Allan on Sept. 11/2000

Distribution List: - to file
- include in report

CONTACT MEMORANDUM

Contact Name: Brad Allan Agency/Organization: MNR - Midhurst. Address: Midhurst ON L0L 1X0	FRi Staff: RSB Date: Sept. 5/2000 Phone: 705-725-7519 Fax:
Project Name: Hwy. 400	Project/File No.: CN29900147 GWP #30-95-00

Re: Fisheries Files

- requested appointment to review relevant Fisheries files including past scientific collectors permits
- Brad suggested I meet with Graham Findlay from their office to review files.

Action:

- Meet with Graham + Brad in the a.m. on Sept. 11/2000

Distribution List: - to file
- include in report

CONTACT MEMORANDUM

Contact Name: Paul Jurgens Agency/Organization: MNR - Midhurst Address: Midhurst ON L0L 1X0	FRi Staff: RSB Date: Sept. 5/2000 Phone: 705-725-7500 Fax:
Project Name: Hwy 400	Project/File No.: CN29900147 GWP # 30.95-00

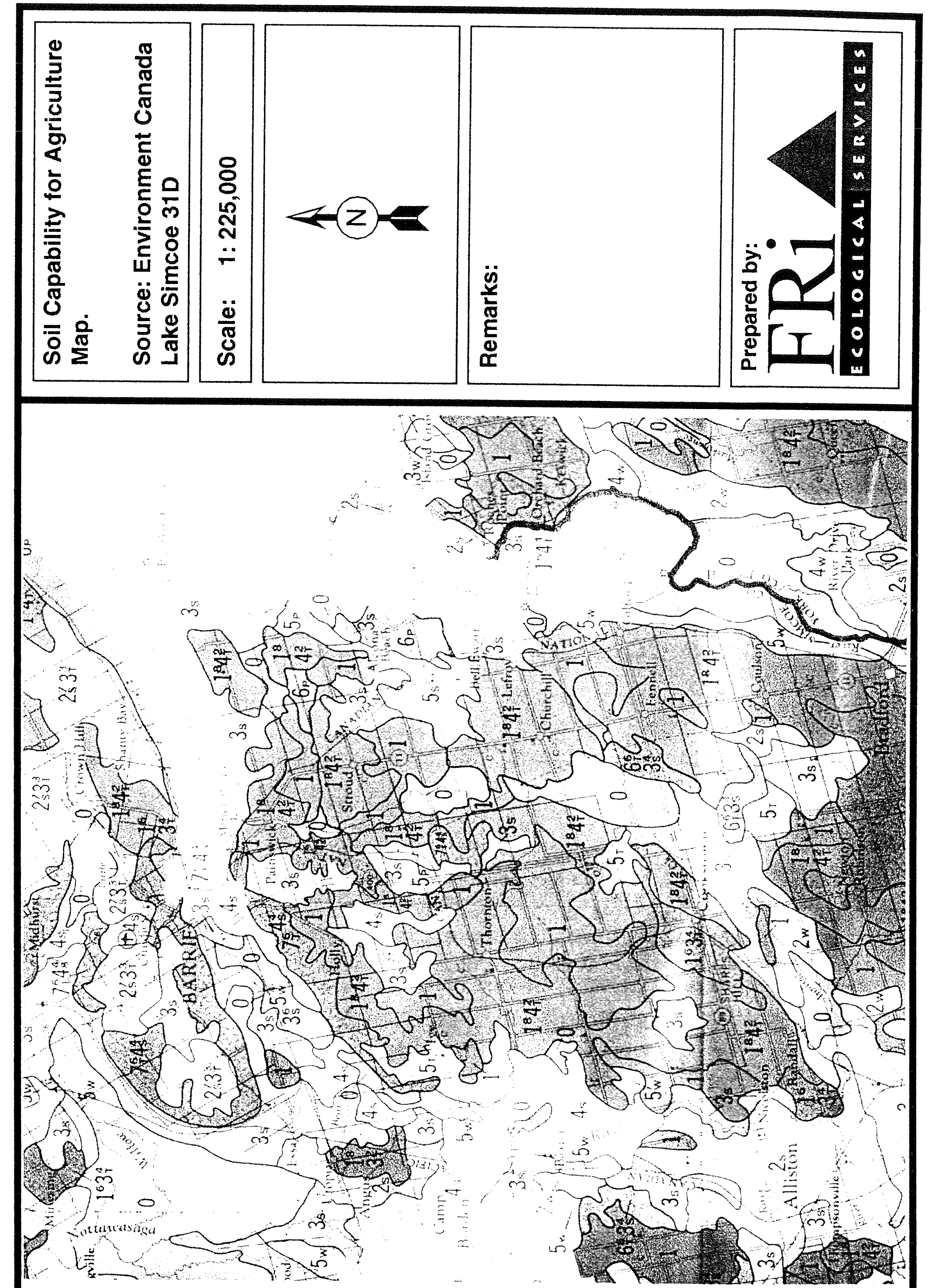
Re: GIS Mapping

- requested mapping of Hwy 400 corridor from 89 to 11/400 Interchange
- specifically interested in fish habitat, wetlands, ANSI, OLL areas, significant wildlife habitat.
- Paul will have mapping prepared by Sept. 8/2000

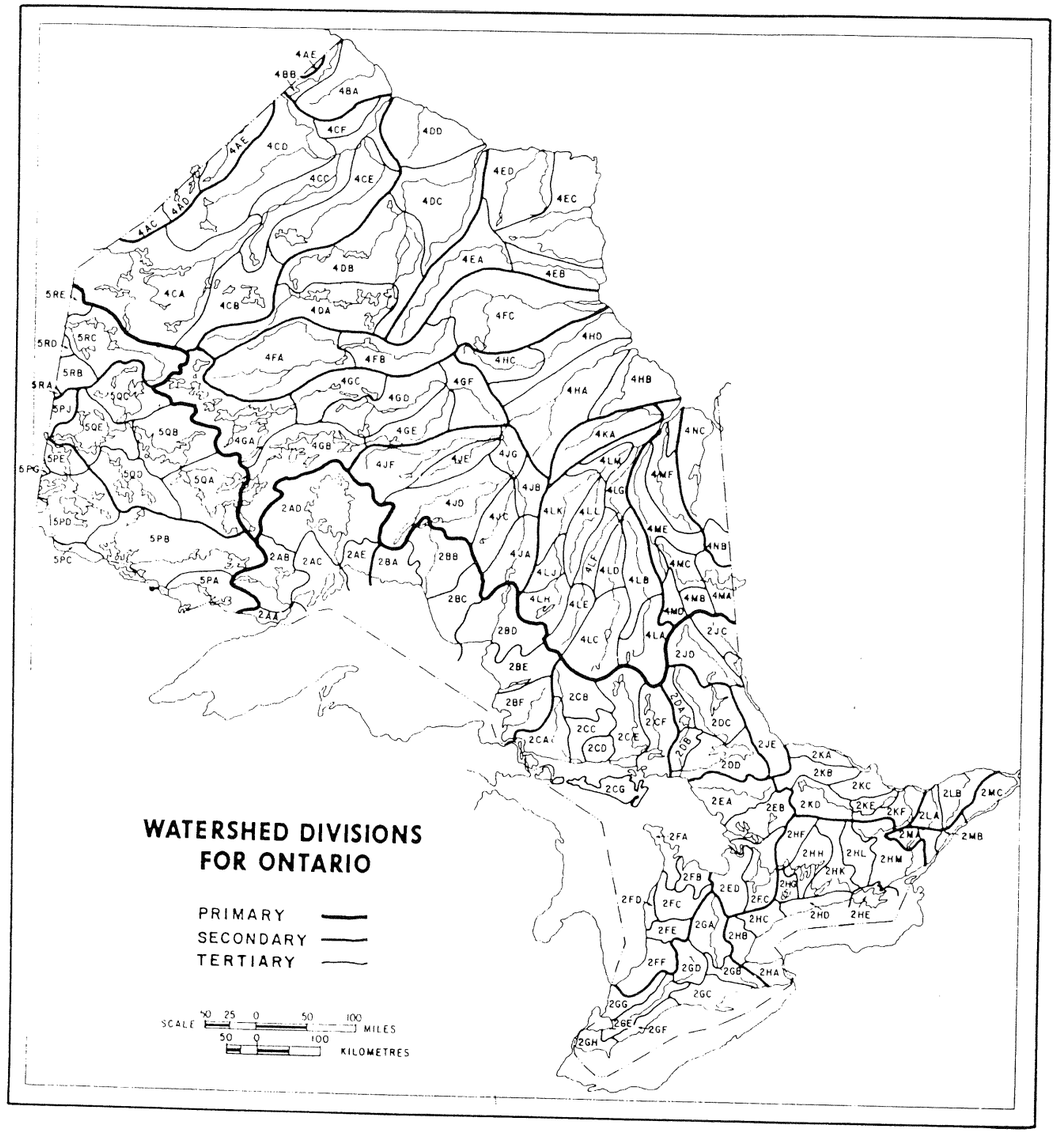
Action:
- Pick up mapping from the Midhurst Office at 8:30am Sept. 11/2000

Distribution List:
- to file
- include in report.

APPENDIX B
Soils Capability Map



APPENDIX C
Watershed Key Map



Reprinted from :- Counts and Measurements of Ontario Lakes 1978 by :- E.T. Cox

APPENDIX D
Water Well Records

WATER WELL DATA SYSTEM Dec 04 2000 PAGE: 1 COUNTY: SIMCOE GROUND WATER BULLETIN REPORT

MUNICIPALITY CONCESSION ETC	LOT	WELL NO	UTM EASTING NORTHING	ELEV FEET	DATE	DRILLER	CSG DIA INS	KIND OF WATER	WATER FOUND FEET	STAT LVL FEET	PUMP LVL FEET	TEST RATE GPM	TEST TIME HR:MN	WATER USE	SCREEN DEPTH FEET	SCREEN LENGTH FEET	OWNER DEPTHS IN FEET TO WHICH FORMATIONS EXTEND
INNISFIL TOWNSHIP																	
	02	006	57-21419	606570 4896631		1986/08	4919	30 UK	0050	34	50		0 :30	DO			COATS BRWN LOAM HARD 0001 BRWN CLAY 0050 BRWN GRVL LOOS 0054
	07	007	57-21728	605776 4903595		1986/11	4919	30 UK	0010	10	28		0 :30	DO			M., SANDERSEN BRWN LOAM HARD 0001 BRWN CLAY SAND PCKD 0020 GREY CLAY HARD 0030
	08	006	57-22115	604928 4904760		1985/08	3135	06 UK	0068	32	45	20	2 :0	CO	0073	09	INISVALE, CEMETARY LOAM 0001 CLAY SAND GRVL 0068 SAND WBRG 0083
CON	01	006	57-00867	607321 4894917	745	1961/04	3108	06 FR	0115	10	100	8	8 :0	CO	0119	11	BRITISH AMERICAN OIL FILL 0007 GRVL CLAY 0110 FSND 0115 CSND 0130
CON	01	006	57-25373	606857 4895280		1989/08	4645	06 FR	0027	4	19	3	12:0	DO	0023	04	WALLACE, JEAN BRWN SAND SILT 0005 GREY MSND SILT 0027 GREY CLAY SILT SAND 0100 GREY CLAY SILT HARD 0140
CON	01	006	57-26358	606857 4895280		1990/01	4645	06 FR	0022	10	18	4	3 :0	DO	0008	04	MOIR, GROREE BLCK LOAM 0001 BRWN SAND SILT 0005 GREY MSND SILT 0026 GREY CLAY SOFT 0026
CON	01	006	57-16622	607100 4894700	775	1979/08	3135	05 FR	0242	FLW			:	DO	0251	04	DAVIS B BRWN SAND 0022 GREY CLAY SOFT 0108 CLAY SILT LYRD 0210 CLAY 0242 SAND GRVL 0255
CON	01	006	57-25374	606857 4895280		1989/08	4645	06 FR	0027	4	19	3	1 :0	DO	0023	04	MOIR, GEORGE BLCK LOAM 0001 BRWN SAND SILT 0005 GREY MSND SILT 0027 GREY SILT CLAY 0030
CON	01	006	57-00865	607316 4894857	745	1957/09	2318	06 FR	0032	8	76	25	24:0	CO	0121	10	BRITISH AMERICAN OIL FILL 0008 BLUE CLAY 0032 BLUE FSND 0110 BLUE MSND 0132
CON	01	006	57-33385	606857 4895280		1997/09	3406	10 GS	0296	12	109	120	1 :0	IR	0218	21	LONEHO GOLF COURSE LOAM 0002 BRWN CLAY SLTY 0007 GREY CLAY SNDY 0016 GREY CLAY SAND STNS 0038 GREY CLAY SOFT STKY 0089 GREY SILT CLAY 0099 GREY CLAY STKY 0111 GREY SILT CLAY 0149 GREY CLAY 0184 GREY SILT CLAY STNS 0206 GREY SAND MGRD 0229 GREY SAND SLTY 0244 GREY CLAY SOFT STKY 0296 GREY CLAY SAND DRTY 0304 SHLE 0308
CON	01	007	57-00868	607315 4895177	745	1954/05	4623	08 FR	0320	FLW			:	CO			BRITISH AMERICAN OIL LOAM 0004 QSND 0150 HPAN CLAY 0215 GRVL CLAY 0220 HPAN CLAY 0297 LMSN SHLE 0340
CON	01	007	57-00870	607471 4895002	745	1964/08	3414	04 UK	0120	12	44	10	8 :0	IN	0116	04	CHOMOLOCK S LOAM 0001 BRWN FSND 0012 SILT CLAY 0117 MSND 0120
CON	01	007	57-00869	607321 4894917	745	1960/12	1308	30 FR	0003	3		4	8 :0	IN			CHOMOLOK S BRWN CLAY MSND 0003 BRWN MSND 0008 BLUE MSND 0016
CON	01	007	57-31250	607445 4895483		1994/10	2514	06 FR	0214	FLW	122	38	4 :0	PS	0214	10	KOA CAMP GROUNDS BRWN CLAY SNDY 0020 BRWN SAND 0024 GREY CLAY 0055 GREY SAND CLAY 0060 GREY CLAY 0202 GREY SAND 0210 GREY CLAY 0214 GREY SAND 0224
CON	02	006	57-06141	606340 4996950	875	1969/01	4608	30 FR	0050	32			:	DO			PHILLIP ROBERT BRWN CLAY 0015 MSND STNS 0050
CON	02	006	57-18317	606300 4896700	875	1982/10	4919	30 UK	0030	20	40		0 :30	DO			SMITH BRWN LOAM HARD 0001 BRWN CLAY GRVL SAND 0042
CON	02	006	57-18315	606300 4896750	875	1982/10	4919	30 UK	0050	45	59		0 :30	DO			LEON B BRWN LOAM HARD 0001 BRWN CLAY HARD 0050 BRWN SAND LOOS 0060

MUNICIPALITY CONCESSION ETC	LOT	WELL NO	UTM EASTING NORTHING	ELEV FEET	DATE	DRILLER	CSG DIA INS	KIND OF WATER	WATER FOUND FEET	STAT LVL FEET	PUMP LVL FEET	TEST RATE GPM	TEST TIME HR:MN	WATER USE	SCREEN DEPTH FEET	LENGTH FEET	OWNER DEPTHS IN FEET TO WHICH FORMATIONS EXTEND
CONTINUING... INNISFIL TOWNSHIP																	
CON	02 007	57-34396	607153 4896822		1999/05	4645	64	FR	0250	10	110	30	6 :0	DO	0246	04	ARMSTRONG,JIM BLCK LOAM SOFT 0001 BRWN CLAY STNS HARD 0017 GREY CLAY DNSE 0020 GREY SAND LOOS 0025 GREY CLAY SILT LYRD 0173 GREY CLAY DNSE 0237 GREY SAND SILT LOOS 0245 GREY MSND LOOS 0250
CON	02 007	57-18316	606850 4897400	800	1982/06	4919	30	UK	0044	25	48		0 :30	DO			RAINEY J BRWN LOAM HARD 0001 BRWN CLAY STNS HARD 0045 GREY CLAY STNS HARD 0052
CON	02 007	57-24924	607153 4896822		1989/04	4919	30	UK	0028	28	38	5	1 :0	DO			RANDY BRWN SAND PCKD 0040
CON	03 006	57-34611	606288 4898022		1999/09	3602	06	FR	0100	20	80	7	1 :30	DO	0101	03	RAINEY,JOHN BRWN CLAY 0008 BRWN GRVL SAND CLAY 0011 BRWN CLAY SAND 0020 BRWN CLAY SAND GRVL 0075 GREY CLAY 0100 BRWN SAND 0104
CON	03 006	57-06454	605970 4898160	920	1969/07	4608	30	FR	0012	6	9		1 :0	DO			MATTAR L GREY MSND STNS 0012
CON	03 006	57-32558	606288 4898022		1996/09	6782	08	FR	0071	5	32	7	2 :30	DO	0077	03	BRANKSTON LOAM 0003 BRWN SAND CLAY STNS 0071 BRWN MSND 0080
CON	03 006	57-32684	606288 4898022		1996/09	4645	06	FR	0123	23	50	10	12:0	DO		04	RAINY, ERIC BLCK LOAM SOFT 0002 BRWN SAND GRVL LOOS 0008 BRWN SAND SILT 0015 BRWN CLAY STNS HARD 0019 BRWN GRVL SAND LOOS 0025 BRWN CLAY GRVL HARD 0033 BRWN SAND GRVL LOOS 0041 BRWN CLAY HARD 0059 BRWN CLAY SILT SOFT 0067 BRWN CLAY STNS LOOS 0120 BRWN SAND GRVL LOOS 0130
CON	03 006	57-07257	606000 4898080	930	1970/05	3203	05	FR	0040	22	40	2	1 :0	DO	0045	03	CAMILLO P BLCK LOAM 0001 BRWN CLAY STNS 0025 BRWN MSND CLAY 0033 BRWN MSND CLAY 0040 BRWN MSND 0048
CON	03 007	57-24603	606876 4898212		1988/08	1663	06	FR	0077	0	85	10	1 :30	DO	0087	03	KEEVES, J. BRWN FILL 0004 BRWN SAND CLAY GRVL 0018 BLUE CLAY GRVL 0048 BLUE CLAY GRVL SAND 0074 BRWN CSND GRVL CLAY 0087 BRWN SAND GRVL 0095
CON	03 007	57-00910	606993 4897606	865	1964/12	4608	30	FR	0050	38		2	:	ST DO			BAILEY ED BRWN LOAM MSND 0003 BRWN CLAY MSND 0050
CON	04 006	57-16679	606150 4898750	950	1979/11	3742	36	FR	0054	22	42	3	6 :0	DO			GIBBINS W LOAM 0002 BRWN CLAY 0027 BLUE CLAY 0050 CGVL 0058
CON	04 006	57-00938	605891 4899632	950	1964/11	4608	30	FR	0020	20		2	:	ST DO			PEARSON H CLAY 0032
CON	04 006	57-33295	606035 4899383		1997/10	3413	30	FR	0032	7	40	30	2 :0	DO			GRIFFIN CONSTRUCTION BLCK LOAM 0004 BRWN SAND 0008 GREY CLAY 0032 GREY SAND 0037 GREY CLAY 0060
CON	04 006	57-31752	606035 4899383		1995/07	3660	06	FR	0067	4	75	3	2 :0	DO	0075	05	TOMASCHEK BLCK LOAM 0002 BRWN CLAY 0006 GREY CLAY 0025 BLUE CLAY SOFT 0064 GREY CLAY SILT 0067 GREY FSND WBRG 0080
CON	04 006	57-33825	606035 4899383		1998/07	7061	06	FR	0060	25	60	5	1 :30	DO	0061	04	SHANUGHNESSY, PERCY CLAY GRVL 0015 SAND CLAY 0035 CLAY 0060 SAND 0065
CON	04 007	57-00941	606694 4899028	955	1965/05	4608	30	FR	0065	55		3	:	ST DO			SHEAN W LOAM 0002 CLAY MSND 0065
CON	04 007	57-33296	606615 4899576		1997/01	3413	30	FR	0032	7	40	30	2 :0	DO			GRIFFIN CONSTRUCTION BLCK LOAM 0004 BRWN SAND 0008 GREY CLAY 0032 GREY SAND 0037 GREY CLAY 0060

MUNICIPALITY CONCESSION ETC	LOT	WELL NO	UTM EASTING NORTHING	ELEV FEET	DATE	DRILLER	CSG DIA INS	KIND OF WATER	WATER FOUND FEET	STAT LVL FEET	PUMP LVL FEET	TEST RATE GPM	TEST TIME HR:MN	WATER USE	SCREEN DEPTH FEET	LENGTH FEET	OWNER DEPTHS IN FEET TO WHICH FORMATIONS EXTEND
CONTINUING... INNISFIL TOWNSHIP																	
CON	04 007	57-25330	606615 4899576		1989/07	3108	06	FR	0102	48	90	10	2 :0	DO	0102	07	WILLIAMSON, DON BLCK LOAM 0001 BRWN CLAY 0003 BRWN CLAY SAND GRVL 0052 BLUE CLAY GRVL STNS 0071 BRWN SAND 0110 GREY SAND CLAY 0120 BLUE CLAY GRVL 0140
CON	04 007	57-34159	999999 9999999		1999/04	1413	06	FR	0132	87	125	10	1 :30	DO	0127	05	AC SOUND INVESTMENTS BRWN CLAY SAND HARD 0117 BRWN SAND FSND 0132
CON	04 007	57-27556	606615 4899576		1990/12	3660	06	FR	0068	40	79	4	2 :0	DO	0086	03	GRILLO, FRANK BLCK LOAM 0002 GREY CLAY GRVL 0068 GREY GRVL SAND CLAY 0090 GREY CLAY HARD 0095 GREY CLAY HARD 0095
CON	05 006	57-11926	605550 4901350	955	1974/08	3109	30	FR	0035	16			:	DO			STRUGESS C LOAM 0002 BRWN CLAY SNDY 0034 FSND 0037
CON	05 007	57-30867	606346 4900916		1994/07	3602	06	FR	0130	7	135	60	1 :0	DO	0137	05	SUNNIDALE HOMES. BRWN SAND 0010 GREY CLAY GRVL LYRD 0071 GREY SAND SNDY CLAY 0074 GREY CLAY GRVL 0090 GREY CLAY SNDS 0130 BRWN SAND CLN WBRG 0141
CON	05 007	57-34464	606346 4900916		1999/08	1413	06	FR	0148	57	140	80	1 :0	DO	0145	03	KLEINVEL FARMS BRWN SAND GRVL PCKD 0037 BRWN SAND CLAY CMTD 0137 BRWN FSND 0148
CON	05 007	57-00996	606447 4900356	965	1962/12	1308		FR	0035	30		1	1 :0	ST DO			MAYES J C PRDG 0034 BRWN CLAY MSND 0062
CON	05 007	57-33843	606346 4900916		1998/10	1467	05	FR	0026			10	2 :0	DO		03	GRIFFIN CONSTRUCTION BLCK LOAM 0001 GREY STNS CLAY GRVL 0009 GREY SAND GRVL 0026
CON	06 006	57-25988	605483 4902080		1989/10	3903	06	FR	0230	70	180	40	4 :0	DO ST	0226	04	HENRY, ANGUS BRWN CLAY SAND LYRD 0006 GREY CLAY SAND SILT 0044 BRWN FSND CLAY LYRD 0058 GREY CLAY STNS LYRD 0072 GREY SILT CLAY LYRD 0215 BRWN CSND LOOS 0230
CON	06 006	57-05418	605150 4902550	1000	1968/04	4608	30	FR	0025	12			:	DO			HENRY ANGUS LOAM 0002 BRWN CLAY MSND STNS 0015 GREY CLAY MSND STNS 0045
CON	06 007	57-01049	606256 4901938	965	1965/06	4608	30	FR	0038	22		3	:	ST DO			HICKEY W E CLAY MSND 0038
CON	07 006	57-28424	605196 4903407		1991/08	3413	30	FR	0055	15	36		:	DO			A, MILLIGAN BRWN CLAY HARD 0020 GREY CLAY BLDR HARD 0030 GREY MSND 0042 GREY CLAY 0055 GREY FSND 0056
CON	07 006	57-10050	605055 4903084	995	1973/05	4816		DRY					:				MARCELLUS M CLAY GRVL 0067 FSND 0093 SILT GRVL 0149 CLAY GRVL 0165 CLAY 0220 FSND 0245 CSND 0273 GRVL 0287 CLAY SILT 0300
CON	07 006	57-01072	605140 4903971	1000	1965/03	4608	30	FR	0030	30		2	:	ST DO			SZERPS W S LOAM 0002 GREY CLAY 0024 FSND 0045
CON	07 006	57-05630	605050 4904020	1000	1968/03	3108	07	UK	0085	12	104	25	5 :0	CO			SHELL CANADA LTD LOAM 0002 YLLW CLAY STNS 0014 YLLW MSND 0018 BRWN CLAY MSND 0085 BRWN FSND 0104 MSND 0110
CON	07 006	57-10894	604976 4903291	995	1973/05	4816	06	FR	0208	88		20	5 :0	CO	0207	06	MARCELLUS M CLAY SAND 0063 FSND 0089 GREY CLAY 0091 FSND 0098 CLAY SILT 0151 CLAY 0208 MSND 0213 FSND 0230 SAND SILT 0285 GREY CLAY 0290
CON	07 007	57-08737	605670 4904190	975	1971/09	4816	06	UK	0194				:	NU			NORRICK HOLDINGS LTD MSND 0007 CLAY MSND 0021 MSND CLAY 0073 MSND 0132 CLAY 0194 CSND MSND 0198 CLAY SILT 0310

MUNICIPALITY CONCESSION ETC	LOT	WELL NO	UTM EASTING NORTHING	ELEV FEET	DATE	DRILLER	CSG DIA INS	KIND OF WATER	WATER FOUND FEET	STAT LVL FEET	PUMP LVL FEET	TEST RATE GPM	TEST TIME HR:MN	WATER USE	SCREEN DEPTH FEET	SCREEN LENGTH FEET	OWNER DEPTHS IN FEET TO WHICH FORMATIONS EXTEND
CONTINUING... INNISFIL TOWNSHIP																	
CON	07	007	57- 08738	605850 4903450	1000	1971/09	4816	06			106	109	11	1 :0	NU	0214 09 0223 03	NORRICK HOLDINGS LTD MSND 0064 CLAY SILT 0168 GREY CLAY 0214 MSND 0220 MSND SILT 0229 CLAY SILT 0245
CON	07	007	57- 07421	605870 4904170	940	1970/08	4816	06	FR	0193	78		60	7 :0	CO	0190 04	NORRICK HOLDINGS LTD CLAY GRVL 0038 MSND GRVL 0077 GREY CLAY 0098 SILT 0134 GREY CLAY 0181 SILT 0190 GREY CLAY 0193 MSND GRVL 0216 SILT 0278 CLAY SILT 0320
CON	07	007	57- 08736	605680 4903870	985	1971/09	4816	06	UK	0209				:	NU		NORRICK HOLDINGS LTD MSND 0024 CLAY 0029 MSND CLAY 0063 CLAY 0110 FSND 0138 CLAY SILT 0172 CLAY 0209 CSND MSND 0218 SILT CLAY 0310
CON	07	007	57- 08735	605750 4903900	985	1971/09	4816	06						:		0202 08 0210 04	NORRICK HOLDINGS LTD MSND GRVL 0016 GREY CLAY 0022 MSND 0104 CLAY MSND SILT 0182 GREY CLAY 0199 MSND FSND 0209 CSND SILT 0215 CLAY SILT 0245
CON	08	006	57- 01127	604639 4904684	995	1966/11	4608	30	FR	0024	21		2	:	DO		STURGEN J BRWN CLAY 0053
CON	08	006	57- 01126	604774 4904008	1000	1965/10	4608	30	FR	0034	25		2	:	DO		POLAND P J GREY CLAY 0018 MSND 0045
CON	08	006	57- 09006	605100 4904220	1000	1972/07	3518	07	FR	0240	88		60	42:30	CO	0244 10 0254 04	B P OIL LTD BLCK LOAM 0003 BRWN CLAY STNS 0015 BLUE CLAY STNS 0060 GREY QSND CLAY 0150 GREY CLAY STNS 0225 GREY FSND 0240 GREY MSND CSND 0258
CON	08	006	57- 09007	605140 4904230	1000	1972/07	3518	07						:	NU		B P OIL LTD BLCK LOAM 0003 BRWN CLAY STNS 0015 BLUE CLAY STNS 0060 GREY QSND CLAY 0150 GREY CLAY STNS 0225 GREY FSND 0240 GREY MSND FSND 0258
CON	08	006	57- 10594	604623 4904750	995	1973/11	1204	05	FR	0214	92	110	6	3 :0	DO		STORGESS E GREY CLAY 0020 GREY CLAY SAND GRVL 0060 GREY SILT 0070 GREY SAND CLAY 0085 GREY CLAY SAND 0214
CON	08	007	57- 08108	605480 4904650	990	1971/07	4816	06	FR	0240	80	116	203	24:0	CO	0233 12	NORRICK HOLDINGS LTD MSND GRVL 0029 GREY CLAY 0041 FSND 0124 CLAY MSND 0157 FSND 0194 MSND CSND 0247 MSND 0304 FSND 0346 CLAY SILT 0390
CON	08	007	57- 20029	605700 4905100	975	1985/07	4816	06			82		100	4 :0	CO	0280 07	THE WATERMAN BRWN CLAY STNS 0038 BRWN SAND 0053 CLAY 0055 FSND SILT 0110 GREY CLAY SILT 0188 CSND 0215 GREY MSND 0288 BRWN CSND STNS CLAY 0297
CON	08	007	57- 08107	605480 4904720	990	1971/07	4816	02			72			:		0240 20	NORRICK HOLDINGS LTD MSND GRVL 0028 GREY CLAY MSND 0047 GREY CLAY 0074 MSND CLAY 0163 CLAY SILT 0198 MSND 0246 MSND 0260
CON	08	007	57- 11853	605600 4904900	980	1974/08	4816	10	FR	0190	84	177	510	24:0	MN	0224 30	ABASCO MANAGEMENT CLAY SNDY 0019 SAND GRVL 0033 CLAY 0037 FSND 0074 SAND SILT 0124 SILT CLAY STNS 0190 FSND MSND 0270 FSND 0289 CLAY SILT 0300
CON	08	007	57- 27320	605511 4904953		1990/08	2801	24			120	154	500	72:0	PS	0201 25	TOWNSHIP OF INNISFIL BRWN CLAY 0025 CLAY SNDY 0030 SAND GRVL PCKD 0037 CLAY GRVL SNDY 0103 SAND GRVL CLAY 0131 GREY CLAY GRVL 0194 GRVL SAND CLAY 0229
CON	08	007	57- 21189	605600 4904950	975	1985/07	4816	06					100	4 :0	MN	0280 07	ANDERSON I BRWN CLAY STNS 0038 BRWN SAND 0053 CLAY 0055 FSND SILT 0110 GREY CLAY SILT 0188 CSND 0215 GREY MSND 0288 BRWN CSND STNS CLAY 0297

MUNICIPALITY CONCESSION ETC	LOT	WELL NO	UTM EASTING NORTHING	ELEV FEET	DATE	DRILLER	CSG DIA INS	KIND OF WATER	WATER FOUND FEET	STAT LVL FEET	PUMP LVL FEET	TEST RATE GPM	TEST TIME HR:MN	WATER USE	SCREEN DEPTH FEET	SCREEN LENGTH FEET	OWNER DEPTHS IN FEET TO WHICH FORMATIONS EXTEND
CONTINUING... INNISFIL TOWNSHIP																	
CON	08	007	57- 08739	605500 4904825	995	1971/09	4816	02	UK	0190	80			:	NU	0240 20	NORRICK HOLDINGS LTD CLAY SAND 0016 SAND GRVL 0034 CLAY 0036 SAND 0075 SAND SILT 0123 GREY CLAY 0181 SAND 0190 MSND FSND 0285 CLAY 0290

TABLE

WATER WELL RECORDS

TOTAL WELLS DRILLED	WELLS ENDING IN: OVER-BURDEN	BED-ROCK	KIND OF WATER: FRESH	SALT	SULPH	MIN-ERAL	DRY HOLE	WATER USE, ETC.: DOM. OR STOCK	IRRIG-ATION	INDUS-TRIAL	COMM-ERCIAL	MUNI-CIPAL	PUBLIC SUPPLY	COOL/ AIR COND	NOT USED	TEST HOLE	ABAN-DONED
67	64	3	46	0	0	0	1	42	1	2	10	2	2	0	5	6	1

WATER WELL DATA SYSTEM Dec 04 2000 PAGE: 1 COUNTY: SIMCOE GROUND WATER BULLETIN REPORT

MUNICIPALITY CONCESSION ETC	LOT	WELL NO	UTM EASTING NORTHING	ELEV FEET	DATE	DRILLER	CSG DIA INS	KIND OF WATER	WATER FOUND FEET	STAT LVL FEET	PUMP LVL FEET	TEST RATE GPM	TEST TIME HR:MN	WATER USE	SCREEN DEPTH FEET	SCREEN LENGTH FEET	OWNER DEPTHS IN FEET TO WHICH FORMATIONS EXTEND
INNISFIL TOWNSHIP																	
14	006	57-29775	603298 4912961		1992/10	6386	06	FR	0092	FLW	51	10	2 :30	DO	0102	04	CORMER, ANNE, CECIK BRWN SAND LOOS 0001 BRWN GRVL HPAN LOOS 0013 BLUE BLDR GRVL DNSE 0017 GREY CLAY SAND DNSE 0033 GREY CLAY DNSE 0092 GREY SAND SILT LOOS 0105 GREY SAND GRVL LOOS 0106
14	006	57-23726	603298 4912961		1988/08	2801	02			46	59	24	4 :0	NU			BARRIE, P.U.C SAND STNS 0006 BRWN CLAY SLTY 0012 GREY CLAY GRVL 0018 GREY CLAY DLMT SLTY 0052 GREY CLAY CLAY SLTY 0076 GREY SAND CLAY SLTY 0080 FSND VERY MGRD 0092 SAND CLAY SLTY 0105 FSND MGVL VERY 0118 GREY CLAY GRVL SLTY 0130 GREY SAND CLAY SLTY 0162 SAND CLAY SLTY 0187 GREY CLAY SAND SLTY 0205 SAND CLAY LYRD 0218 MSND FGVL 0225 SAND CLAY SLTY 0228 GREY CLAY SAND PCKD 0242 GRVL LYRD 0242
CON 14	006	57-32730	603298 4912961		1997/03	3660	06	FR	0118	34	120	3	2 :0	DO	0121	04	INVESTORS GROUP BLCK LOAM 0001 BRWN SAND GRVL 0040 BLUE CLAY GRVL 0095 GREY CLAY SILT HARD 0118 GREY FSND SILT WBRG 0125
CON 14	007	57-31933	603888 4913158		1995/10	1467	05	FR	0054	23	41	15	2 :0	DO	0055	08	KERBEL, HOWARD GREY SAND CLAY 0026 BRWN SAND 0037 BRWN SAND CLAY 0043 GREY CLAY 0054 GREY SAND 0063 GREY CLAY 0063

TABLE

WATER WELL RECORDS

TOTAL WELLS DRILLED	WELLS ENDING IN: OVER-BURDEN	BED-ROCK	KIND OF WATER: FRESH	SALT	SULPH	MIN-ERAL	DRY HOLE	DOM. OR STOCK	WATER USE, ETC.: IRRIG-ATION	INDUS-TRIAL	COMM-ERCIAL	MUNI-CIPAL	PUBLIC SUPPLY	COOL/ AIR COND	NOT USED	TEST HOLE	ABAN-DONED
4	3	1	3	0	0	0	0	3	0	0	0	0	0	0	1	0	0

WATER WELL DATA SYSTEM Dec 04 2000 PAGE: 1 COUNTY: SIMCOE GROUND WATER BULLETIN REPORT

MUNICIPALITY CONCESSION ETC	LOT	WELL NO	UTM EASTING NORTHING	ELEV FEET	DATE	DRILLER	CSG DIA INS	KIND OF WATER	WATER FOUND FEET	STAT LVL FEET	PUMP LVL FEET	TEST RATE GPM	TEST TIME HR:MN	WATER USE	SCREEN DEPTH FEET	SCREEN LENGTH FEET	OWNER DEPTHS IN FEET TO WHICH FORMATIONS EXTEND
INNISFIL TOWNSHIP																	
	10	008	57- 23083	605572 4907837		1987/09	3135	05 FR	0214		38	6	24:0	DO			SILVER, R. CLAY 0015 CLAY SAND 0080 CLAY 0085 CLAY SILT 0120 CLAY SOFT 0201 GRVL SAND CLAY 0213 GRVL SAND 0214
	12	007	57- 20702	604452 4910366		1985/12	4919	30 UK	0034	34	48		0 :30	DO			LM'RFEE BRWN LOAM HARD 0001 BRWN SAND PCKD 0054
	13	007	57- 14928	604175 4911741			4608	30 FR	0012	13			:	DO			WALLIIN BRWN CLAY STNS 0012 GRVL 0015 SAND 0025
	13	008	57- 24267	604761 4911932		1988/11	2514	06 FR	0127	2	60	15	2 :0	DO	0127	03	MURDOCK, B BLCK LOAM 0001 BRWN CLAY STNS 0008 GREY CLAY 0127 GREY SAND 0130
CON	10	007	57- 01256	605240 4907058	1000	1966/07	4715	04 FR	0070	52	55	6	2 :0	DO	0084	04	LEARY G PRDG 0040 CLAY MSND GRVL 0070 FSND 0088 CLAY SILT MSND 0090
CON	10	008	57- 31646	605572 4907837		1995/04	2514	06 FR	0123	86		15	2 :0	DO	0124	03	WILSON, MCAROL BRWN LOAM 0001 BRWN SAND CLAY 0050 GREY CLAY SAND GRVL 0123 BRWN SAND 0127
CON	10	008	57- 20818	605572 4907837		1986/06	3030	36 FR	0030	15			:	DO			MORRIELLO CONST. BRWN SAND 0004 BRWN CLAY STNS 0015 GREY SAND SILT STNS 0030 BRWN SAND 0035
CON	11	007	57- 23735	604722 4909008		1988/08	1456	05 FR	0072	19	50	10	2 :0	DO	0066	06	P. SEGRS
CON	11	008	57- 01394	605197 4909744	950	1965/05	2514	FR	0109	64	110	2	2 :0	ST DO	0109	03	PETERSON M PRDG 0032 CSND 0090 MSND CLAY 0107 FSND 0112
CON	12	007	57- 01447	604620 4909888	985	1966/10	4608	30 FR	0006	6		2	:	ST DO			MAXWELL E MSND 0018 CLAY 0020
CON	12	008	57- 15534	604900 4910900	960	1978/07	2801	02 FR	0379	227	237		:	NU	0380	10	MOLSONS GREY CLAY SAND LYRD 0007 SAND GRVL 0070 FSND FGVL CLAY 0138 CLAY SAND SLTY 0190 CLAY SLTY 0270 CLAY SILT SOFT 0376 SILT CLAY SAND 0420 SILT FSND GRVL 0446 GREY CLAY HARD 0450
CON	12	008	57- 01448	605079 4911239	930	1957/04	5510	04 FR	0090	43	48	4	3 :0	ST DO		10	SKY LINE HATCHERY LOAM MSND 0001 FSND 0076 BLUE CLAY 0079 MSND 0090
CON	12	008	57- 15533	604950 4910900	960	1978/07	2801						:				MOLSONS SAND CLAY DRTY 0007 SAND GRVL 0065 GREY CLAY FSND LYRD 0082 FSND MSND GRVL 0127 FSND CLAY 0143 GREY CLAY SILT SAND 0170 CLAY SLTY PCKD 0269 SILT CLAY PCKD 0310 CLAY SILT PCKD 0374 SILT FSND PCKD 0431 SILT FSND GRVL 0444 GREY CLAY HARD 0460
CON	12	008	57- 15535	604950 4910900	950	1978/08	2801	07 MN	0417	222	286	50	9 :0				MOLSONS SAND CLAY DRTY 0007 SAND GRVL 0050 SAND CLAY 0080 CLAY SAND 0145 CLAY SAND SLTY 0193 CLAY 0274 CLAY SLTY SOFT 0360 SILT FSND PCKD 0383 SILT FSND CLAY 0400 CLAY SLTY PCKD 0407 CLAY SILT SNDY 0417 FSND SILT CLAY 0437
CON	12	008	57- 15536	605050 4909950	975	1978/10	2801						:				MOLSON SAND CLAY BLDR 0012 CLAY 0027 SAND GRVL CLAY 0111 CLAY SAND LYRD 0150 SILT CLAY SAND 0203 GREY CLAY STKY 0241 GREY CLAY HARD 0285 CLAY STKY 0310 CLAY SLTY SOFT 0371 CLAY SILT FGVL 0445 GREY CLAY HARD 0465

MUNICIPALITY CONCESSION ETC	LOT	WELL NO	UTM EASTING NORTHING	ELEV FEET	DATE	DRILLER	CSG DIA INS	KIND OF WATER	WATER FOUND FEET	STAT LVL FEET	PUMP LVL FEET	TEST RATE GPM	TEST TIME HR:MN	WATER USE	SCREEN DEPTH FEET	SCREEN LENGTH FEET	OWNER DEPTHS IN FEET TO WHICH FORMATIONS EXTEND
CONTINUING... INNISFIL TOWNSHIP																	
CON	12	008	57-16625	604900 4910900	975	1979/06	2801	54					:	NU			MOLSON'S BREWERY BRWN SAND GRVL CLAY 0198 GREY CLAY PCKD 0270 BRWN SAND CLAY LOOS 0296 GREY CLAY PCKD 0372 BRWN SAND CLAY LOOS 0394 GREY CLAY PCKD 0401 BRWN SAND GRVL CLAY 0423 GREY CLAY HARD 0465 GREY CLAY GRVL HARD 0503
CON	12	008	57-16626	604900 4910850	975	1979/07	2801	54					:	NU			MOLSON'S BREWERY BRWN SAND GRVL LOOS 0074 GREY SAND CLAY PCKD 0189 GREY CLAY PCKD 0266 GREY SAND CLAY LYRD 0299 GREY CLAY SAND LYRD 0418 GREY CLAY GRVL LYRD 0441 GREY CLAY HARD 0462 GREY CLAY GRVL HARD 0498
CON	12	008	57-15532	604750 4910900	960	1978/07	2801						:				MOLSONS SAND CLAY DRTY 0010 BRWN SILT CLAY 0022 SAND GRVL 0091 SAND CLAY LYRD 0104 SAND CLAY PCKD 0170 GREY SAND CLAY SILT 0229 GREY CLAY SAND SLTY 0240 GREY CLAY SLTY 0300 GREY CLAY SLTY SOFT 0360 GREY CLAY SAND SLTY 0380 GREY CLAY SLTY 0473 GREY CLAY HARD 0500
CON	13	007	57-01538	604297 4912456	925	1960/12	3108	04 FR	0056	44	45	24	2 :0	PS	0066 10		D H O MSND FILL 0001 YLLW LOAM MSND 0010 GRVL MSND 0012 YLLW FSND 0076
CON	13	007	57-01537	604559 4911970	920	1960/06	3108	04 FR	0095	66	69	16	2 :0	DO	0104 07		JOHNSON W E BRWN CLAY 0002 FSND GRVL 0095 CSND 0111
CON	13	007	57-01536	604544 4911568	960	1960/06	3108	08 FR	0115	60	90	60	2 :0	PS	0126 20		DHO LOAM 0002 YLLW MSND GRVL CLAY 0110 FSND 0115 CSND 0125 FSND 0132 MSND 0137 FSND 0146
CON	13	007	57-01535	604556 4911620	960	1959/10	3108	04 FR	0125	60	140	40	12:0	IR	0131 06 0137 06		D H O LOAM 0002 YLLW MSND CLAY 0110 FSND 0115 CSND 0125 FSND 0132 MSND 0138 CSND 0143
CON	13	007	57-01534	604496 4911434	960	1959/09	3108	04 FR	0100	85	105	10	2 :0	IR	0104 03 0107 03		D H O LOAM 0002 YLLW MSND CLAY STNS 0095 BRWN MSND 0100 YLLW FSND 0104 YLLW FSND 0110 FSND 0111
CON	13	007	57-01533	603715 4912243	860	1956/07	2514	06 FR	0224	150	170	8	2 :0	CO			FINA PETROLEUM COMP LOAM 0002 BLUE CLAY 0040 BLUE MSND 0090 MSND GRVL 0105 MSND 0207 CLAY 0210 MSND GRVL 0224
CON	13	008	57-33745	604761 4911932		1998/06	3660	06 FR	0043	3	35	8	1 :0	DO	0043 05		WALMAN,BERNIC BRWN SAND 0025 GREY GRVL CLAY HARD 0043 GREY FSND 0048

TABLE

WATER WELL RECORDS

WELLS DRILLED	WELLS ENDING IN: OVER-BURDEN	BED-ROCK	KIND OF WATER: FRESH	SALT	SULPH	MIN-ERAL	DRY HOLE	DOM. OR STOCK	IRRIG-ATION	WATER USE, ETC.: INDUS-TRIAL	COMM-ERCIAL	MUNI-CIPAL	PUBLIC SUPPLY	COOL/ AIR COND	NOT USED	TEST HOLE	ABAN-DONED
25	24	0	18	0	0	1	0	13	2	0	1	0	2	0	3	3	2

MUNICIPALITY CONCESSION ETC	LOT	WELL NO	UTM EASTING NORTHING	ELEV FEET	DATE	DRILLER	CSG DIA INS	KIND OF WATER	WATER FOUND FEET	STAT LVL FEET	PUMP LVL FEET	TEST RATE GPM	TEST TIME HR:MN	WATER USE	SCREEN DEPTH FEET	LENGTH FEET	OWNER DEPTHS IN FEET TO WHICH FORMATIONS EXTEND
BARRIE CITY (VESPRA)																	
	06 024	57-23010	602570 4914530		1988/02	2801	02	FR	0138	29	30	40	4 :0	NU	0152	20	BARRIE, P.U.C. RED SAND LOOS 0023 BRWN SAND SAND LOOS 0051 GREY CLAY SLTY SOFT 0130 SAND GRVL LYRD 0175 GREY CLAY HARD 0192 CLAY SAND SNDY 0228 SAND GRVL LYRD 0244 CLAY SNDY HARD 0267 GREY CLAY VERY HARD 0289 CLAY GRVL SNDY 0306
	06 024	57-22879	602570 4914530		1988/01	2801	42	FR	0288	24	90	750	6 :0	IN	AC0292	35	GENERAL, TIRE LOAM 0001 BRWN SAND 0004 MSND CSND FGVL 0029 MGVL CGVL SAND 0039 GREY CLAY STKY 0074 FSND CSND FGVL 0132 GREY CLAY STKY 0146 FSND MSND 0158 FGVL CGVL BLDR 0202 GREY CLAY FGVL SNDY 0223 CLAY GRVL PCKD 0243 FGVL CLAY BLDR 0263 CLAY GRVL SNDY 0271 FSND PCKD 0288 FSND FGVL 0292 FGVL CGVL CLAY 0299 FGVL CGVL PCKD 0309 CLAY PCKD 0313 GRVL CLAY CMTD 0316 FGVL CGVL 0319 GREY CLAY SNDY 0323 FGVL MGVL CLAY 0338 FGVL CGVL PCKD 0341 GRVL CMTD 0344
CON	06 023	57-04795	602559 4915179	776	1959/06	2801	05						:				BARRIE PUC BLCK MUCK 0004 MSND SILT BLDR 0013 BRWN CLAY 0033 CLAY MSND 0045 BLUE CLAY 0162 CLAY MSND SILT 0216 BLUE CLAY 0233 SILT MSND CLAY 0265 BLUE CLAY 0276 BLUE CLAY GRVL 0289
CON	06 023	57-04794	602684 4915176	770	1959/06	2801	05	UK	0047				:	NU			BARRIE PUC LOAM 0001 MSND SILT 0033 BLUE CLAY 0047 GRVL 0050 BLUE CLAY 0162 BLUE CLAY GRVL 0205 BLUE CLAY 0221 CLAY MSND GRVL 0256 BLUE CLAY 0271 BLUE CLAY BLDR 0275 BLUE CLAY 0318
CON	06 023	57-04793	602621 4915302	790	1955/07	2529	06	FR	0084	28	50	50	12:0	MN			OSBORNE R MSND 0030 CLAY MSND 0060 CSND 0090 GRVL MSND 0096
CON	06 024	57-04799	601965 4914352	765	1965/04	1614	06	FR	0101	18	45	36	2 :0	CO			PEACOCK B YLLW MSND MUCK 0018 GREY CLAY STNS 0096 MSND 0100 GRVL 0101
CON	06 024	57-04798	601992 4914244	765	1954/09	1637	02	FR	0085	11	16	6	3 :0	CO	0088	04	JACKSON MOTORS MSND MUCK 0014 STNS 0030 BLUE CLAY MSND 0085 CSND 0092
CON	06 024	57-04797	602205 4914272	765	1953/10	1637	02	FR	0075	9	18	5	1 :0	DO	0098	04	HILL T GRVL 0016 CLAY 0034 CLAY MSND 0070 HPAN 0075 FSND STNS 0092 MSND 0102
CON	06 024	57-04796	602792 4915271	770	1953/05	5510	04	FR	0046	8	36	3	1 :0	DO	0046	09	FEE E LOAM 0001 MSND STNS 0045 MSND 0055
CON	06 025	57-04805	602529 4913608	765	1954/09	1637	02	FR	0092	10	18	6	2 :0	DO	0099	04	GRAY L YLLW MUCK MSND 0007 CLAY 0014 GRVL STNS 0030 BLUE CLAY 0065 CLAY FSND 0092 FSND 0097 CSND 0103
CON	06 025	57-04804	602499 4913603	765	1954/08	1637	02	FR		10	18	8	2 :0	DO	0112	04	HEPC ONT YLLW MUCK MSND 0014 BLUE CLAY 0062 FSND CLAY 0068 CLAY MSND 0101 STNS 0103 CLAY MSND 0110 MSND GRVL 0116
CON	06 025	57-04803	602519 4913606	765	1953/09	1637	02	FR	0052	9	16	5	1 :0	DO	0064	04	PAYNE J MSND MUCK 0016 GRVL 0022 BLUE CLAY MSND 0052 FSND 0062 CSND 0068
CON	06 025	57-04802	602452 4913578	765	1952/08	1637	02	FR	0060	8	22	8	2 :0	DO	0061	04	ABLEWHYTE W MUCK 0008 MSND GRVL 0028 BLUE CLAY 0055 MSND 0065

MUNICIPALITY CONCESSION ETC	LOT	WELL NO	UTM EASTING NORTHING	ELEV FEET	DATE	DRILLER	CSG DIA INS	KIND OF WATER	WATER FOUND FEET	STAT LVL FEET	PUMP LVL FEET	TEST RATE GPM	TEST TIME HR:MN	WATER USE	SCREEN DEPTH FEET	LENGTH FEET	OWNER DEPTHS IN FEET TO WHICH FORMATIONS EXTEND
CONTINUING... BARRIE CITY (VESPRA)																	
CON	06 025	57-04801	602472 4913626	765	1952/07	1637	01	FR	0020	4	4	5	2 :0	DO	0020	04	ALBEWHITE W GREY CLAY STNS 0010 FSND CLAY 0018 MSND 0024
CON	06 025	57-04800	601935 4914332	765	1952/05	1637	01	FR	0020	4	4	5	2 :0	DO	0020	04	KUZAK F GREY CLAY STNS 0010 FSND CLAY 0018 MSND 0024

TABLE

WATER WELL RECORDS

TOTAL WELLS DRILLED	WELLS ENDING IN OVER-BURDEN	WELLS ENDING IN BED-ROCK	KIND OF WATER FRESH	KIND OF WATER SALT	KIND OF WATER SULPH	WATER USE, ETC. MIN-ERAL	WATER USE, ETC. DRY HOLE	WATER USE, ETC. DOM. OR STOCK	WATER USE, ETC. IRRIG-ATION	WATER USE, ETC. INDUS-TRIAL	WATER USE, ETC. COMM-ERCIAL	WATER USE, ETC. MUNI-CIPAL	WATER USE, ETC. PUBLIC SUPPLY	WATER USE, ETC. COOL/ AIR COND	WATER USE, ETC. NOT USED	WATER USE, ETC. TEST HOLE	WATER USE, ETC. ABAN-DONED
15	15	0	13	0	0	0	0	8	0	1	2	1	0	1	2	2	0

WATER WELL DATA SYSTEM Dec 04 2000 PAGE: 1 COUNTY: SIMCOE GROUND WATER BULLETIN REPORT

MUNICIPALITY CONCESSION ETC	LOT	WELL NO	UTM EASTING NORTHING	ELEV FEET	DATE	DRILLER	CSG DIA INS	KIND OF WATER	WATER FOUND FEET	STAT LVL FEET	PUMP LVL FEET	TEST RATE GPM	TEST TIME HR:MN	WATER USE	SCREEN DEPTH FEET	SCREEN LENGTH FEET	OWNER DEPTHS IN FEET TO WHICH FORMATIONS EXTEND
BARRIE CITY (VESPRA)																	
CON 04	021	57-04730	603873 4917846	915	1966/06	1510	04	FR	0175	110	120	10	2 :0	DO	0171	04	MERIDITH J LOAM 0003 BRWN CLAY BLDR 0080 HPAN STNS 0140 FSND 0160 HPAN 0168 CSND 0175
CON 04	021	57-04729	603923 4917894	915	1962/03	1614	04	FR	0125	89	118	8	3 :0	DO	0120	05	WATSON J LOAM MSND 0010 GRVL STNS 0115 GRVL MSND 0125
CON 04	021	57-04728	603394 4917287	920	1958/07	2514	06	FR	0139	65	126	15	3 :30	CO	0136	03	CITIES SERVICE OIL MSND 0006 BRWN CLAY 0013 MSND GRVL 0139
CON 04	021	57-04727	603562 4917004	900	1956/07	5510	04	FR	0120	72	76	4	2 :0	DO	0106	14	GRAUES BROS CONST LOAM 0001 MSND STNS 0005 HPAN 0084 MSND 0087 HPAN 0092 GRVL 0096 HPAN 0101 MSND 0120
CON 04	021	57-04726	603140 4917790	955	1953/10	5510	04	FR	0115	82	87	5	1 :30	DO	0115	12	MACCEAN I LOAM 0001 GRVL 0003 MSND CLAY BLDR 0118 CSND 0127

TABLE

WATER WELL RECORDS

TOTAL WELLS DRILLED	WELLS ENDING IN: OVER-BURDEN	BED-ROCK	KIND OF WATER: FRESH	SALT	SULPH	WATER USE, ETC.: MIN-ERAL	DRY HOLE	DOM. OR STOCK	IRRIG-ATION	INDUS-TRIAL	COMM-ERCIAL	MUNI-CIPAL	PUBLIC SUPPLY	COOL/ AIR COND	NOT USED	TEST HOLE	ABAN-DONED
5	5	0	5	0	0	0	0	4	0	0	1	0	0	0	0	0	0

WATER WELL DATA SYSTEM Dec 04 2000 PAGE: 1 COUNTY: SIMCOE GROUND WATER BULLETIN REPORT

MUNICIPALITY CONCESSION ETC	LOT	WELL NO	UTM EASTING NORTHING	ELEV FEET	DATE	DRILLER	CSG DIA INS	KIND OF WATER	WATER FOUND FEET	STAT LVL FEET	PUMP LVL FEET	TEST RATE GPM	TEST TIME HR:MN	WATER USE	SCREEN DEPTH FEET	LENGTH FEET	OWNER DEPTHS IN FEET TO WHICH FORMATIONS EXTEND
BARRIE CITY (VESPRA)																	
CON 03	022	57-04614	605351 4918219	895	1957/04	1637	04	FR	0060	45	65	4	2 :0	ST DO	06		CUMMINGS A HPAN 0060 FSND 0085 MSND FSND 0090

TABLE

WATER WELL RECORDS

TOTAL WELLS DRILLED	WELLS ENDING IN OVER-BURDEN	BED-ROCK	KIND OF WATER FRESH	SALT	SULPH	MIN-ERAL	DRY HOLE	DOM. OR STOCK	WATER USE, ETC. IRRIG-ATION	INDUS-TRIAL	COMM-ERCIAL	MUNI-CIPAL	PUBLIC SUPPLY	COOL/ AIR COND	NOT USED	TEST HOLE	ABAN-DONED
1	1	0	1	0	0	0	0	1	0	0	0	0	0	0	0	0	0

WATER WELL DATA SYSTEM Dec 04 2000 PAGE: 1 COUNTY: SIMCOE GROUND WATER BULLETIN REPORT

MUNICIPALITY CONCESSION ETC	LOT	WELL NO	UTM EASTING NORTHING	ELEV FEET	DATE	DRILLER	CSG DIA INS	KIND OF WATER	WATER FOUND FEET	STAT LVL FEET	PUMP LVL FEET	TEST RATE GPM	TEST TIME HR:MN	WATER USE	SCREEN DEPTH FEET	LENGTH FEET	OWNER DEPTHS IN FEET TO WHICH FORMATIONS EXTEND
VESPRA TOWNSHIP																	
PR W	01	004	57-24200 999999 999999		1988/10	1583	06	FR	0189	95	120	7	1 :30	DO	0186	03	RALPH, HODGSON LOAM 0004 CLAY SAND 0018 GREY CLAY FSND 0146 GREY CLAY 0184 BRWN MSND 0190 CLAY 0192
PR W	01	004	57-04918 607633 4918032	875	1967/06	3203	05	FR	0097	95	110	8	2 :0	IN	0142	03	SIMCOE GLASS LOAM 0002 CLAY MSND STNS 0046 CLAY MSND 0097 FSND 0111 FSND MSND 0138 MSND CSND 0145
PR W	01	004	57-04919 607582 4917937	875	1967/08	2514	06	FR	0128	70	118	8	1 :30	DO	0132	03	CLAVERY J FILL 0001 BRWN CLAY MSND 0057 YLLW MSND SILT 0128 YLLW MSND 0135
PR W	01	004	57-09618 607600 4918010	855	1972/12	4608		DRY					:				PARTRICH FARM EQUIP GREY CLAY 0068
PR W	01	005	57-04922 605950 4918787	800	1966/06	4608	30	FR	0007	7		2	:	DO			GLENN B E LOAM 0002 MSND 0007 GREY CLAY 0023
PR W	01	005	57-09039 606250 4918860	790	1972/07	4608	30	FR	0015	15	20	4	1 :0	DO			RUNCE R BRWN CLAY 0015 BRWN GRVL SAND 0028
PR W	01	005	57-08875 606400 4918500	860	1972/07	4608	30	FR	0078	7	81	2	1 :30	DO			GEORGIAN COLLEGE GREY CLAY 0075 GREY SAND CLAY 0085
PR W	01	005	57-10036 606265 4918850	795	1973/05	4608	30	FR	0010	11	18	4	1 :0	DO			DALLMORE BRWN CLAY 0007 BLUE CLAY GRVL 0023
PR W	01	005	57-03533 607590 4918394	850	1965/07	1637	06	FR	0139	80	115	15	3 :0	ST	D00147	04	DUNSMORE G CLAY STNS 0032 MSND MUCK CLAY 0120 HPAN 0139 CSND 0153
PR W	01	005	57-04924 606223 4918854	785	1967/02	4608	30	FR	0042	42		2	:	DO			COTTRELL A BRWN CLAY 0012 GREY CLAY 0066
PR W	01	005	57-09041 606260 4918850	790	1972/07	4608	30	FR	0018	18	24	4	1 :0	DO			RUNCE R BRWN CLAY GRVL 0018 BRWN SAND GRVL 0030
PR W	01	005	57-04923 605978 4918829	785	1966/08	4608	30	UK	0011	11		2	:	DO			WILLISON J BRWN CLAY MSND 0006 GREY CLAY STNS 0022
PR W	01	005	57-10838 606250 4918850	795	1974/01	2514	06	FR	0070	24	61	20	2 :0	DO	0070	03	BARR D BRWN CLAY STNS 0024 BRWN GRVL SILT CLAY 0044 GREY CLAY SAND GRVL 0070 BRWN SAND SILT CLAY 0085
PR W	01	005	57-15599 606100 4918900	775	1978/09	4816	06	FR	0077	14		12	1 :30	DO	0079	03	TONGE G SAND 0008 CLAY 0070 SAND CLAY 0077 FSND 0087
PR W	01	005	57-04921 605998 4918859	775	1965/08	4608	30	FR	0008	8		3	:	DO			VANDERVEEN J CLAY MSND 0018
PR W	01	005	57-04920 605965 4918859	775	1965/08	4608	30	FR	0006	6		2	:	MN			MYERS M BRWN CLAY 0006 GREY CLAY MSND 0017
PR W	01	005	57-16862 606050 4918900	775	1980/04	4816	06	FR	0047	12	43	7	1 :0	DO	0053	03	TONGE K PRDG 0017 SAND GRVL CLAY 0022 FSND 0050 UNKN 0053
PR W	01	005	57-16923 606350 4918800	775	1980/06	1467	05	FR	0075	28	70	5	2 :0	DO	0076	04	GALLANT L BRWN GRVL STNS 0031 GREY GRVL SAND CMTD 0075 BRWN FSND 0081 GREY CLAY SAND 0081
PR W	01	005	57-06645 607680 4918580	857	1969/09	2514	06	FR	0114	85	110	10	3 :0	DO			MURPHY J E LOAM 0002 GREY CLAY BLDR 0009 CLAY GRVL BLDR 0027 BRWN CLAY 0036 MSND 0043 MSND GRVL 0114 GRVL 0115

MUNICIPALITY CONCESSION ETC	LOT	WELL NO	UTM EASTING NORTHING	ELEV FEET	DATE	DRILLER	CSG DIA INS	KIND OF WATER	WATER FOUND FEET	STAT LVL FEET	PUMP LVL FEET	TEST RATE GPM	TEST TIME HR:MN	WATER USE	SCREEN DEPTH FEET	LENGTH FEET	OWNER DEPTHS IN FEET TO WHICH FORMATIONS EXTEND
CONTINUING... VESPRA TOWNSHIP																	
PR W	01 005	57-25043	999999 9999999		1989/06	2801	02			82	93	30	3 :0	NU	0002	05	R. VICTORIA HOSPITAL BRWN CLAY GRVL SNDY 0025 BRWN CLAY STKY 0032 GREY CLAY STKY 0057 GRVL SAND BLDR 0082 FGVL SAND FCRD 0104 WHIT GRVL CLAY MARL 0117 GREY GRVL CLAY 0127 GRVL CLAY SNDY 0140 BRWN CLAY GRVL 0151 GREY CLAY GRVL 0202 BRWN CLAY SNDY GRVL 0210 GREY CLAY GRVL 0241 FSND CLAY LYRD 0250 FSND SILT 0273 GREY CLAY GRVL 0310 FGVL CLAY 0329 GREY CLAY GRVL 0358 GREY CLAY GRVL HARD 0408 ROCK 0410
PR W	01 005	57-13399	606190 4918840	850	1975/12	3203	05	FR	0070	70	80	5	1 :0	DO	0095	03	TREMBLY BRWN CLAY STNS 0015 GREY CLAY STNS 0070 BRWN SAND 0098
PR W	01 006	57-05941	606850 4918860	850	1968/07	1510	04	FR	0125	90	100	10	2 :0	DO	0131	04	SHOEMAKER NORM LOAM 0002 BLUE CLAY 0090 CLAY 0125 CSND 0135
PR W	01 006	57-05905	607150 4918850	860	1968/07	1510	04	FR	0090	82	100	10	2 :0	DO	0111	04	TASCONA FRANK LOAM 0002 GRVL MSND CLAY 0015 RED MSND 0080 MSND 0115
PR W	01 006	57-05646	607050 4918850	860	1968/05	1510	04	FR	0178	120	125	10	2 :0	DO	0174	04	GARDNER GEORGE LOAM 0002 GRVL HPAN BLDR 0090 BLUE CLAY 0100 QSND 0160 CLAY 0168 CSND 0178
PR W	01 006	57-04991	605863 4918974	765	1955/11	5510	04	FR	0032	FLW			:	DO			KUNCE P LOAM 0001 BLUE CLAY MUCK MSND 0032
PR W	01 006	57-04928	606913 4918890	852	1967/03	1510	04	FR	0115	90	100	10	2 :0	DO	0111	04	MORTEL D LOAM 0002 BRWN CLAY 0030 CSND 0065 CLAY MSND 0095 MSND 0115
PR W	01 006	57-04927	607724 4918769	785	1966/11	1510	04	FR	0125	90	90	8	1 :0	DO	0121	04	KENNY E E LOAM CLAY 0003 BRWN CLAY STNS 0030 HPAN CLAY 0115 MSND 0125
PR W	01 006	57-32130	999999 9999999		1996/01	1467	05	FR	0134	103	126	5	2 :0	DO	0139	07	GOW, GORDON BRWN LOAM 0001 BRWN SAND 0029 GREY SAND CLAY 0038 BRWN SAND GRVL 0052 GREY SAND CLAY 0134 GREY FSND 0147 GREY CLAY SAND 0147
PR W	01 006	57-04925	605770 4918994	765	1952/06	5510	04	FR	0031	FLW			:	CO			KUNCE P LOAM 0001 CLAY 0031
PR W	01 006	57-06221	607150 4919820	860	1969/01	1510	04	FR	0120	85	95	10	1 :0	DO	0116	04	WARNER PAUL LOAM 0002 BRWN CLAY STNS 0022 HPAN STNS 0060 FSND 0100 MSND 0120
PR W	01 006	57-04584	606908 4918868	860	1967/10	1510	04	FR	0120	90	110	5	2 :0	DO	0116	04	GOW G LOAM 0002 CLAY MSND LOAM 0012 RED MSND CLAY 0028 RED CLAY 0032 WHIT HPAN CLAY 0080 FSND 0110 MSND 0120
PR W	01 006	57-04926	606378 4918909	775	1966/07	4608	30	FR	0005	5		2	:	DO			SMITH G MSND GRVL STNS 0013
PR W	01 006	57-16589	605900 4919000	775	1979/08	4608	30	FR	0016	6			:	DO			HITCH W FILL 0003 GREY CLAY STNY 0022
PR W	01 006	57-05906	607100 4918830	860	1968/07	1510	04	FR	0090	27	80	3	3 :0	DO	0084	06	JOHNSON J B LOAM 0002 CLAY STNS 0030 HPAN STNS 0080 FSND 0090
PR W	01 006	57-09525	606930 4918850	860	1972/12	3203	05	UK	0147	100	115	8	1 :0	DO	0144	03	SIMNS B BRWN SAND GRVL 0031 BRWN CLAY 0040 GREY CLAY 0084 BRWN CLAY 0100 BRWN SAND 0147
PR W	01 006	57-06549	606750 4918880	825	1969/08	1510	04	FR	0150	90	110	8	2 :0	DO	0145	05	GOODERHAM BUD LOAM 0002 BRWN CLAY STNS 0020 BLUE CLAY MSND 0130 FSND 0150

MUNICIPALITY CONCESSION ETC	LOT	WELL NO	UTM EASTING NORTHING	ELEV FEET	DATE	DRILLER	CSG DIA INS	KIND OF WATER	WATER FOUND FEET	STAT LVL FEET	PUMP LVL FEET	TEST RATE GPM	TEST TIME HR:MN	WATER USE	SCREEN DEPTH FEET	LENGTH FEET	OWNER DEPTHS IN FEET TO WHICH FORMATIONS EXTEND
CONTINUING... VESPRA TOWNSHIP																	
PR W	01 006	57-10077	606900 4918860	952	1973/07	1204	05	FR	0105	100	106	5	2 :30	DO	0115	03	LOM T BRWN CLAY STNS 0003 GREY CLAY SAND 0015 GREY CLAY 0030 GREY SAND CLAY 0048 GREY SILT FSND 0121
PR W	01 006	57-10822	607150 4918800	860	1973/12	3203	05	FR	0110	100	110	7	1 :20	DO	0130	06	BERTRUM BILL PRDG 0005 BRWN SAND LOOS 0021 BRWN SAND GRVL PCKD 0046 GREY CLAY 0100 BRWN SAND CLAY 0110 BRWN SAND WBRG 0136
PR W	01 006	57-13400	606750 4918860	850	1975/12	3203	05	FR	0105	95	108	7	2 :0	DO	0135	03	NIA W BRWN SAND CLAY 0018 GREY CLAY 0025 BRWN SAND CLAY 0060 GREY CLAY 0105 BRWN SAND 0132 GREY SAND 0138
PR W	01 006	57-18713	607200 4918800	875	1983/09	1467	05	FR	0129	92	120	8	2 :0	DO	0132	03	LYNN J GREY CGVL STNS 0031 GREY GRVL CMTD 0040 GREY FSND 0081 GREY CLAY 0129 BRWN MSND 0135 GREY CLAY 0135
PR W	01 006	57-09364	607010 4918840	860	1972/10	4715	04	FR	0154	99	110	10	2 :0	DO	0184	03	WARNAAR A FILL 0003 CLAY SAND GRVL 0028 GREY CLAY 0154 FSND 0180 SAND 0187 SAND CLAY 0193
PR W	01 006	57-08445	606970 4918840	860	1971/10	1204	05	FR	0178	96	150	5	1 :0	DO	0178	03	BRISTOW H GREY GRVL SAND 0002 BRWN SAND 0015 GREY CLAY SAND STNS 0058 GREY CLAY BLDR 0162 BRWN SILT SAND 0178 BRWN FSND 0182
PR W	01 006	57-07297	606220 4918970	775	1970/06	4608	30	FR	0015	10	13	5	1 :0	DO			KOMASZKO V GREY MSND 0015
PR W	01 006	57-07253	606710 4918860	850	1970/04	3203	06	FR	0090	90	105	10	2 :0	DO	0121	04	LEWIS JACK FILL 0002 BRWN MSND CLAY 0018 GREY CLAY MSND 0032 GREY MSND 0090 GREY MSND 0125 GREY MSND CLAY 0126
PR W	01 006	57-06807	606270 4918940	775	1969/10	4608	30	FR	0005	1	5		1 :0	DO			SHEPHARD L G BLCK LOAM 0001 GREY CSND 0012
PR W	01 007	57-18062	607800 4919250	775	1981/11	3205	05	FR	0050	5	38	5	1 :0	DO	0055	03	SUTTON BRWN SAND CLAY STNS 0024 BRWN TILL CLAY STNS 0039 BRWN SAND CLAY 0050 GREY SAND LOOS 0058
PR W	01 007	57-20391	999999 9999999		1985/05	3203	05	FR	0032	9	40	10	1 :0	DO	0056	03	ZIMMERMANN, H. PRDG 0018 BRWN SAND CLAY 0032 BRWN SAND 0053 GREY SAND 0059 BRWN SAND SOFT 0059
PR W	01 007	57-12307	607800 4919150	795	1975/06	1204	05	FR	0093	32	47	10	1 :15	DO	0095	03	REID LARRY GREY CLAY SAND 0010 GREY FSND 0043 BRWN SILT CLAY 0078 BRWN FSND SILT CLAY 0084 GREY SILT CLAY 0093 GREY MSND 0100
PR W	01 007	57-09613	607800 4919400	770	1972/11	4608	30	FR	0006	0	9	5	1 :0	DO			ORO MILL SUPPLY GREY SAND 0012 BRWN CLAY 0018
PR W	01 007	57-14222	607800 4919300	775	1976/11	3203	05	FR	0046	12	40	5	1 :30	DO	0049	05	KENWELL F BRWN GRVL SAND STNS 0018 BRWN SAND 0036 GREY CLAY SAND 0046 BRWN SAND 0054 BRWN SAND CLAY 0054
PR W	01 008	57-04933	607252 4919871	765	1952/02	2801	06						:				DND CLAY MSND 0005 MSND SILT CLAY 0025 CLAY 0031 MSND SILT CLAY 0085
PR W	01 008	57-04929	607247 4919838	765	1951/07	1656	12	SU	0106	FLW	50	2	72:0	PS			DND BRWN LOAM MSND 0001 BRWN MSND SILT 0004 BRWN MSND 0010 GREY MSND SILT 0011 GREY CLAY SILT 0019 GREY MSND GRVL 0020 BRWN MSND GRVL 0023 GREY MSND 0036 CLAY FSND CLAY 0042 GREY MSND SILT 0051 GREY CLAY 0099 GREY CLAY CSND GRVL 0106

MUNICIPALITY CONCESSION ETC	LOT	WELL NO	UTM EASTING NORTHING	ELEV FEET	DATE	DRILLER	CSG DIA INS	KIND OF WATER	WATER FOUND FEET	STAT LVL FEET	PUMP LVL FEET	TEST RATE GPM	TEST TIME HR:MN	WATER USE	SCREEN DEPTH FEET	LENGTH FEET	OWNER DEPTHS IN FEET TO WHICH FORMATIONS EXTEND
CONTINUING... VESPRA TOWNSHIP																	
PR W	01 008	57-04930	607245 4919866	765	1952/06	2801	06						:				DND BLCK MUCK 0003 BRWN CLAY 0004 SILT MSND 0013 BLUE CLAY MSND 0015 MSND 0027 CLAY MSND 0031 MSND SILT 0046 CSND 0051 CLAY MSND 0057 FSND 0083 BLUE CLAY SILT 0098 CLAY GRVL BLDR 0121 CLAY GRVL 0155 CLAY 0179 CLAY GRVL 0228 CLAY 0236 LMSN 0237
PR W	01 008	57-04932	607207 4919823	765	1952/01	2801	06						:				DND YLLW MSND 0002 GREY MSND 0010 CLAY 0012 MSND CLAY 0035 MSND CLAY 0083 BLUE CLAY MSND 0101 HPAN 0102 CLAY GRVL 0122 CLAY 0130
PR W	01 008	57-04934	607109 4919681	765	1952/02	2801	02	FR					:	PS	0218	10	DND LOAM 0001 BRWN CLAY MSND 0003 GREY MSND 0014 BLUE CLAY MSND 0035 MSND CLAY 0062 BLUE CLAY 0083 CLAY GRVL 0094 CLAY GRVL 0120 CLAY BLDR 0122 CLAY 0139 BLUE CLAY MSND 0159 BLUE CLAY FSND 0173 BLUE CLAY 0192 MSND GRVL 0228 CLAY 0232
PR W	01 008	57-04935	607134 4919673	765	1952/03	2801	06						:				DND BLCK MUCK 0005 SILT MSND 0015 MSND CLAY 0026 CLAY MSND GRVL 0052 CLAY SILT BLDR 0060 GRVL SILT MSND 0080 CLAY GRVL SILT 0105 BLUE CLAY 0143 MSND CLAY 0174 BLUE CLAY 0188 MSND CLAY GRVL 0234 BLUE CLAY 0241
PR W	01 008	57-04936	607140 4919786	765	1952/03	2801	06						:				DND LOAM 0001 CLAY 0002 GREY MSND 0010 BLUE CLAY MSND 0046 SILT MSND CLAY 0067 CLAY 0080 MSND BLDR CLAY 0089 GRVL CLAY 0101 BLUE CLAY GRVL 0114 BLUE CLAY BLDR 0122 BLUE CLAY 0144 MSND CLAY 0169 BLUE CLAY GRVL 0194 BLUE CLAY MSND 0204 FSND GRVL 0230 LMSN 0231
PR W	01 008	57-04937	607127 4919806	765	1952/03	2801	06						:				DND LOAM 0001 MSND 0012 MSND SILT 0014 MSND BLDR CLAY 0034 MSND SILT 0041 CLAY MSND 0047 SILT MSND CLAY 0080 CLAY MSND BLDR 0095 MSND CLAY 0106 BLUE CLAY 0111 CLAY SILT MSND 0123 BLDR 0124 CLAY GRVL 0132 CLAY GRVL BLDR 0144 CLAY GRVL 0159 CLAY GRVL BLDR 0169 CLAY GRVL 0183 BLUE CLAY 0188 CLAY MSND GRVL 0203 MSND CLAY 0215 CLAY 0240
PR W	01 008	57-04938	607069 4919621	765	1952/03	2801	06						:				DND MSND CLAY 0017 BLUE CLAY MSND 0048 SILT MSND CLAY 0084 BLUE CLAY BLDR 0095 CLAY GRVL 0135 HPAN 0136 CLAY GRVL 0146 HPAN 0151 CLAY 0158 MSND CLAY GRVL 0173 CLAY 0191 FSND GRVL 0222 FSND GRVL CLAY 0234 BLUE CLAY 0240
PR W	01 008	57-04939	607245 4919871	765	1952/04	2801	06						:				DND LOAM 0001 FSND 0005 BRWN CLAY 0016 BLUE CLAY MSND 0045 MSND SILT CLAY 0084 BLUE CLAY 0095 CLAY GRVL 0126 CLAY 0133 CLAY BLDR GRVL 0138 HPAN 0141 CLAY GRVL 0153 BLUE CLAY 0166 SILT MSND 0172 CLAY 0190
PR W	01 008	57-04940	607109 4919576	765	1952/08	2801	08	FR		FLW	2	244	36:0	PS	0207	15	DND MSND CLAY 0017 BLUE CLAY MSND 0048 SILT CLAY MSND 0084 BLUE CLAY BLDR 0095 CLAY GRVL 0135 HPAN 0136 CLAY GRVL 0146 HPAN 0151 CLAY 0158 MSND CLAY GRVL 0173 CLAY 0191 FSND GRVL 0222

MUNICIPALITY CONCESSION ETC	LOT	WELL NO	UTM EASTING NORTHING	ELEV FEET	DATE	DRILLER	CSG DIA INS	KIND OF WATER	WATER FOUND FEET	STAT LVL FEET	PUMP LVL FEET	TEST RATE GPM	TEST TIME HR:MN	WATER USE	SCREEN DEPTH FEET	LENGTH FEET	OWNER DEPTHS IN FEET TO WHICH FORMATIONS EXTEND
CONTINUING... VESPRA TOWNSHIP																	
PR W	01 008	57-04941	607079 4919551	765	1952/10	2801	08	FR		FLW	3	158	36:0	PS	0214	15	DND LOAM 0001 BRWN CLAY MSND 0003 GREY MSND 0014 BLUE CLAY MSND 0035 MSND CLAY 0062 BLUE CLAY 0083 CLAY GRVL 0094 CLAY GRVL 0120 CLAY BLDR 0122 CLAY 0139 BLUE CLAY MSND 0159 BLUE CLAY FSND 0173 BLUE CLAY 0192 CLAY 0229
PR W	01 008	57-04931	607227 4919836	765	1952/01	2801	06						:				DND BLCK MUCK 0002 BRWN CLAY MSND 0010 BLUE CLAY MSND 0016 MSND 0035 CLAY 0037 MSND SILT CLAY 0065 CLAY MSND 0080 CLAY 0098 CLAY GRVL SILT 0109 SILT MSND 0110
PR W	01 010	57-16108	607700 4920550	825	1978/12	3203	05	FR	0089	56	80	8	2 :0	DO			SIMCOE A BRWN CLAY 0022 BRWN SAND 0048 GREY CLAY 0062 BRWN SAND CLAY 0079 GREY CLAY 0089 BRWN SAND 0106
PR W	01 010	57-04942	607906 4920530	790	1952/02	2801	05	FR		FLW	10	11	:	NU			DND LOAM 0001 BRWN CLAY BLDR 0005 MSND CLAY 0022 MSND 0028 MSND SILT 0034 GRVL 0052 BRWN CLAY MSND 0060 BLUE CLAY GRVL 0074 CLAY MSND 0097
PR W	01 010	57-04943	607893 4920538	790	1952/03	2801	06						:				DND LOAM 0001 CLAY GRVL 0006 MSND CLAY BLDR 0010 MSND GRVL CLAY 0028 MSND GRVL 0032 MSND GRVL BLDR 0039 BRWN MSND 0053 CLAY MSND 0062 BLUE CLAY 0074 BLUE CLAY 0100 GRVL 0102 BLUE CLAY 0145 MSND SILT CLAY 0214 MSND SILT CLAY 0235 MSND CLAY GRVL 0254

TABLE

WATER WELL RECORDS

TOTAL WELLS DRILLED	WELLS ENDING IN OVER-BURDEN	BED-ROCK	KIND OF WATER FRESH	SALT	SULPH	MIN-ERAL	DRY HOLE	WATER USE, ETC. DOM. OR STOCK	IRRIG-ATION	INDUS-TRIAL	COMM-ERCIAL	MUNI-CIPAL	PUBLIC SUPPLY	COOL/ AIR COND	NOT USED	TEST HOLE	ABAN-DONED
66	63	3	51	0	1	0	1	46	0	1	1	1	4	0	2	12	2

WATER WELL DATA SYSTEM Dec 04 2000 PAGE: 1 COUNTY: SIMCOE GROUND WATER BULLETIN REPORT

MUNICIPALITY CONCESSION ETC	LOT	WELL NO	UTM EASTING NORTHING	ELEV FEET	DATE	DRILLER	CSG DIA INS	KIND OF WATER	WATER FOUND FEET	STAT LVL FEET	PUMP LVL FEET	TEST RATE GPM	TEST TIME HR:MN	WATER USE	SCREEN DEPTH FEET	SCREEN LENGTH FEET	OWNER DEPTHS IN FEET TO WHICH FORMATIONS EXTEND
VESPRA TOWNSHIP																	
CON 03	021	57-04613	604528 4918399	870	1964/12	4608	30	FR	0032	32		2	:				PARRIT M LOAM 0002 BRWN LOAM MSND 0046
CON 03	021	57-04612	604697 4918520	860	1964/02	2514	06	DRY					:				DAVIES W E PRDG 0015 BRWN CLAY 0017 YLLW MSND 0033 BRWN CLAY MSND GRVL 0058 BLUE CLAY 0090 GREY MSND 0158
CON 03	021	57-04611	604712 4918525	860	1964/01	2514	06	DRY					:				DAVIES W E LOAM 0001 BRWN CLAY 0017 YLLW MSND 0033 BRWN CLAY MSND GRVL 0058 BLUE CLAY 0090 GREY MSND CLAY 0275 MSND 0285
CON 03	022	57-19821	605158 4917720		1984/07	2514	06	FR	0100	57	90	15	1 :30	DO	0102	03	BELL, C. BRWN CLAY SAND 0090 YLLW SAND 0105

TABLE

WATER WELL RECORDS

WELLS				KIND OF WATER							WATER USE, ETC.						
TOTAL WELLS DRILLED	ENDING IN OVER-BURDEN	ENDING IN BED-ROCK	FRESH	SALT	SULPH	MIN-ERAL	DRY HOLE	DOM. OR STOCK	IRRIG-ATION	INDUS-TRIAL	COMM-ERCIAL	MUNI-CIPAL	PUBLIC SUPPLY	COOL/ AIR COND	NOT USED	TEST HOLE	ABAN-DONED
4	4	0	2	0	0	0	2	1	0	0	0	0	0	0	0	0	2

APPENDIX E
Rare Species for Simcoe County

Known Rare Species (71) in SIMCOE County

(Based on current and historical Element Occurrence records in the NHIC databases)

Scientific Name	Common Name	GRANK	SRANK	MNR	COSEWIC
Ixobrychus exilis	Least Bittern	G5	S3B,SZN	VUL	SC
Casmerodius albus	Great Egret	G5	S2B,SZN		
Nycticorax nycticorax	Black-crowned Night-heron	G5	S3B,SZN		
Buteo lineatus	Red-shouldered Hawk	G5	S4B,SZN	VUL	SC
Colinus virginianus	Northern Bobwhite	G5	S1S2		END
Rallus elegans	King Rail	G4G5	S2B,SZN	END	END
Chlidonias niger	Black Tern	G4	S3B,SZN	VUL	NAR
Baeolophus bicolor	Tufted Titmouse	G5	S2S3		
Lanius ludovicianus	Loggerhead Shrike	G5	S2B,SZN	END	END
Dendroica discolor	Prairie Warbler	G5	S3S4B,SZN	NIAC	NAR
Dendroica cerulea	Cerulean Warbler	G4	S3B,SZN	VUL	SC
Seiurus motacilla	Louisiana Waterthrush	G5	S3B,SZN	VUL	SC
Ammodramus henslowii	Henslow's Sparrow	G4	S1B,SZN	END	END
Ichthyomyzon fossor	Northern Brook Lamprey	G4	S3	VUL	SC
Myoxocephalus thompsoni	Deepwater Sculpin	G5	S4	NIAC	THR
Acipenser fulvescens	Lake Sturgeon	G3	S3		NAR
Coregonus reighardi	Shortnose Cisco	G1	SX	EXP	THR
Clemmys guttata	Spotted Turtle	G5	S3	VUL	SC
Eumeces fasciatus	Five-lined Skink	G5	S3	VUL	SC
Elaphe gloydi	Eastern Fox Snake	G3	S3	THR	THR
Heterodon platirhinos	Eastern Hognose Snake	G5	S3	VUL	SC
Sistrurus catenatus catenatus	Eastern Massasauga	G3G4T3T4	S3	THR	THR
Amblyscirtes hegon	Pepper and Salt Skipper	G5	S3?		

Euchloe olympia	Olympia Marble	G4G5	S3?	
Asterocampa celtis	Hackberry Emperor	G5	S2	
Tortula cainii	A Moss	G1	S1	
Cirsium hillii	Hill's Thistle	G3	S3	
Cacalia plantaginea	Indian-plantain	G4G5	S3	SC
Triadenum virginicum	Marsh St. John's-wort	G5	S3	
Bartonia virginica	Yellow Screwstem	G5	S2	
Linum medium var. medium	Stiff Yellow Flax	G5T?	S3	
Linum striatum	Ridged Yellow Flax	G5	S1	
Utricularia geminiscapa	Hidden-fruited Bladderwort	G4G5	S3	
Rhexia virginica	Virginia Meadow-beauty	G5	S3S4	
Pterospora andromedea	Giant Pinedrops	G5	S2	
Oenothera pilosella	Evening Primrose	G5	S2	
Polygonum arifolium	Halberd-leaved Tear-thumb	G5	S3	
Polygonum careyi	Carey's Smartweed	G4	S3S4	
Chimaphila maculata	Spotted Wintergreen	G5	S1	END
Hedyotis caerulea	Bluets	G5	SH	
Collinsia parviflora	Small-flower Blue-eyed Mary	G5	S2S3	
Valeriana sitchensis	Marsh Valerian	G4G5	S2	
Sagittaria graminea var. cristata	Crested Arrowhead	G4?	S3	
Peltandra virginica	Arrow-arum	G5	S2	
Bulbostylis capillaris	Bulbostylis	G5	S3?	
Carex conoidea	Field Sedge	G5	S3	
Carex folliculata	Long Sedge	G4G5	S3	
Carex haydenii	Cloud Sedge	G5	S2	
Cyperus houghtonii	Houghton's Umbrella-sedge	G4?	S3?	

Eleocharis rostellata	Beaked Spike-rush	G5	S3	
Scirpus heterochaetus	Slender Bulrush	G5	S2	
Scirpus smithii	Smith's Bulrush	G5?	S2?	
Scleria verticillata	Low Nutrush	G5	S3	
Juncus acuminatus	Sharp-fruit Rush	G5	S3	
Juncus secundus	Secund Rush	G5?	S2	
Najas gracillima	Thread-like Naiad	G5?	S2	
Aplectrum hyemale	Puttyroot	G5	S2	
Cypripedium arietinum	Ram's-head Lady's-slipper	G3	S3	
Listera australis	Southern Twayblade	G4	S1	
Platanthera blephariglottis	White-fringed Orchid	G4G5	S3S4	
Platanthera leucophaea	Eastern Prairie White-fringed Orchid	G2	S2	SC
Platanthera macrophylla	Goldie's Round-leaved Orchid	G5?T4	S2	
Aristida basiramea	Forked Three-awned Grass	G5	S1	
Panicum sphaerocarpon var. sphaerocarpon	Panic Grass	G5T5	S3	
Panicum spretum	Panic Grass	G5	S2	
Panicum rigidulum	Redtop Panic Grass	G5	S2S3	
Sporobolus asper	Longleaf Dropseed	G5	S1S2	
Potamogeton bicupulatus	Snail-seed Pondweed	G4?	S3S4	
Xyris difformis	Carolina Yellow-eyed-grass	G5	S3?	
Asplenium scolopendrium var. americanum	Hart's-tongue Fern	G4T3	S3	SC
Botrychium lanceolatum	Lance-leaved Grapefern	G5	S3	

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APPENDIX F
Little Lake PSW Wetland Evaluation and Cookstown Hollows Swamp PSW Natural Areas Report

WETLAND DATA RECORD (Little)

(i). WETLAND NAME AND/OR NUMBER 040 205 (Willow creek) Little Lake

(ii). ADMINISTRATIVE REGION Central, AND DISTRICT Huronia OF ONTARIO MINISTRY OF NATURAL RESOURCES

(iii). CONSERVATION AUTHORITY JURISDICTION Nottawasaga Valley

If not within a designated Conservation Authority, check here ___

(iv). COUNTY OR REGIONAL MUNICIPALITY Simcoe County

(v). TOWNSHIP Vespra twp.

(vi). LOTS AND CONCESSIONS Cons I lot 5-14 Cons II Lots 5-15 Conc. III lots 13-18

(vii). MAP AND AIR PHOTO REFERENCES

(a) Longitude and Latitude 79° 41' 44° 26

(b) U.T.M. Grid Reference Zone: 17T; Grid: 040 205

(c) National Topographic Series Scale and Map Number(s) & Name Barrie Cat. 1:50,000 31D/5

(d) Air Photos

(1) Date photo taken 1978

(2) Scale of air photos 1:10,000

(3) Flight and plate numbers 444793 4430 # 26-30

(viii). WETLAND SIZE AND BOUNDARIES

(a) Single contiguous wetland area: 285 hectares

OR

(b) "Wetland Complex" comprised of _____ individual wetlands as follows:.

Wetland Number (for reference purposes)	Size of each wetland in the complex
Wetland No. 1	_____ hectares
Wetland No. 2	_____ "
Wetland No. 3	_____ "
Wetland No. 4	_____ "
Wetland No. 5	_____ "
Wetland No. 6	_____ "
Total size of wetland complex:	_____ "

1.1.5. Nutrient Status of Surface Water

(a) Write conductivity bridge reading and calculate T.D.S. at 25°C as per tables in Appendix VIII.

Location Sampled (ie. inflow, outflow, etc.)	Initial Specific Conductance (umhos/cm)	Temperature (°C)		Total Dissolved Solids (T.D.S.) (mg/l)
outflow	[illegible]	18°	=	[illegible]
inflow	[illegible]	[illegible]	=	[illegible]
in lake	[illegible]	[illegible]	=	[illegible]
			=	

Average T.D.S. = 258.9

(b) Check appropriate category (from (a))

Average T.D.S. (mg/l)	
<100	
100-500	✓
501-1500	
>1500	
NO READING	

1.2. DIVERSITY VALUES

1.2.1. Number of Wetland Types

(check one)

- ___ One
- ___ Two
- ✓ Three
- ___ Four

1.2.2. Vegetation Communities

(enter form and map code if available, or enter dominant species if known, and appropriate code/symbol)

a) One form

Code		Dominant Form	% Wetland
		re	46
		h	37
		c	10
		ch	4
		ls	2
		ne	1

1.2.3. Diversity of Surrounding Habitat

(check all appropriate items)

- ✓ row crops
- ✓ pasture
- ✓ abandoned agricultural land
- ✓ deciduous forest
- ✓ coniferous forest
- ✓ urban or cottage development
- ___ pits, quarries or mining waste disposal
- ✓ open lake or deep river
- ✓ fence rows with cover, or shelterbelts
- ✓ terraine undulating or hilly with ravines
- ✓ creek(s)

Enter Total = 10

1.2.4. Proximity to Other Wetlands

(check first appropriate category)

i) Hydrologically connected by surface water to other wetlands (different dominant type) or open water within 1.5 km. ✓

ii) Hydrologically connected by surface water to other wetlands (same dominant type) within 0.5 km. ___

iii) Hydrologically connected by surface water to other wetlands (different dominant type) or open water body from 1.5 to 4 km away. ___

iv) Hydrologically connected by surface water to other wetlands (same dominant type) from 0.5 to 1.5 km away. ___

v) Within 0.75 km of other wetlands (different dominant type) or open water body, but not hydrologically connected by surface water. ___

vi) Within 1 km of other wetlands, but not hydrologically connected by surface water. ___

vii) No wetland within 1.5 km. ___

1.2.5. Interspersion

(check one)

- Type 1 ___
- Type 2 ✓
- Type 3 ___
- Type 4 ___

2.1.5. Snapping Turtles
(1) ✓ Present
(2) ____ Absent
Source of Information: Willow Creek study 1975

2.1.6. Furbearers
(check if present)
✓ muskrat
✓ raccoon
✓ beaver
✓ mink
✓ other fox, + coyote.
Source of Information: R. Toth (M.NR)

2.2. RECREATIONAL ACTIVITIES
(check appropriate spaces)

Intensity of Use	Type of Wetland Associated Use: Hunting	Nature Appreciation or Study	Fishing	Canoeing/Boating
High				
Moderate	✓	✓	✓	
Low				✓
None Known				
Not Possible				

Source of Information R. Toth

2.3. AESTHETICS

2.3.1. Landscape Distinctness
(1) ✓ Clearly distinct
(2) ____ Indistinct

2.4.3. Research and Studies
(check one)
(1) ____ One or more wetland-related scientific research papers published in a scientific journal;
(2) ✓ One or more reports written outlining some aspect of the wetland's natural resources;
(3) ____ No reports or papers.

List scientific papers, reports, etc.

Preservation & Rehabilitation of Willow Creek
Willow Creek study 1975
Small Lakes Creel Census, 1978, MNR Lake Survey, 1968

2.5. PROXIMITY TO URBAN AREAS
(check one)
(1) ✓ In an urban or suburan area
(2) ____ <10 km from a population center greater than 10,000
(3) ____ 10 to 60 km from a population center greater than 10,000
(4) ____ Isolated or relatively remote

2.6. OWNERSHIP/ACCESSIBILITY

Estimate % of area and enter in the appropriate space(s)

ACCESSIBILITY	OWNERSHIP: Public, unrestricted activities	Public, restricted activities	Private, open to public for limited activities	Private Club, closed to public	Private or Private and posted
1) Easy at most times by road/waterway	62				38
2) Easy only at certain times of the year					
3) Limited, moderate effort required					
4) Difficult*					

* Requires extended effort due to distance from roads, navigable waterways or isolated geographical position.

Source of information Vespra Township Assessment

3.2.1.4. Size of Adjoining River (Riverine wetlands only)
(not assessed)

3.2.1.5. Location and Size of Detention Areas (Lakes, Reservoirs and Wetlands) within 30 km above and below the wetland

(NOTE: 1 sq. km = 100 ha)

(a) Detention areas above the wetland (within 30 km)

Name and/or Number of Detention Area	Distance upstream from wetland (in km)	Size (hectares)	For Scoring Use
Dalston Wetland Complex	5	561.5	-14

(b) Detention areas below the wetland (within 30 km)

Name and/or Number of Detention Area	Distance downstream from wetland (in km)	Size (hectares)	For Scoring Use
[illegible]	16.1	6349.2	-20
			34

3.3.1.2. Actual Wetland Area Dominated by Robust Emergents and Submergents
(check one)

- ____ <5
- ____ 5 - 50
- ____ 51 - 100
- ✓ 101 - 250
- ____ 251 - 500
- ____ 501 - 1000
- ____ >1000 hectares

3.3.1.3. Land Use in Catchment Basin
(check one)

(1) ✓ Mainly agriculture and/or urban
(2) ____ Roughly 40-60% agriculture; remainder forested or abandoned agriculture
(3) ____ Mainly forested and/or less than 40% agriculture

3.2.2. Long Term Nutrient Trap
(check one)

(1) ____ Wetland located on an active delta
(2) ____ Wetland rivermouth but without obvious delta
(3) ✓ Wetland with organic soils occupying 50% or more of the area
(4) ____ Wetland with organic soils occupying less than 50% of the area (i.e. mainly mineral or undesignated soils)

3.4. EROSION CONTROL

3.4.1. Erosion Buffer (Lacustrine and Riverine wetlands only)

NOTE: Assess for the dominant site type (see 3.3.1.1)

3.4.1.1. Riverine Wetlands (shoreland and flood plain)
(check principal vegetation form)

(1) ____ Trees or Shrubs
(2) ____ Emergents
(3) ____ Non-vegetated or nearly so

3.4.1.2. Lacustrine Wetlands (with or without barrier beach)
(check principal vegetation form)

(1) ✓ Trees or Shrubs
(2) ____ Emergents
(3) ____ Submergents and Floating
(4) ____ Non-vegetated or nearly so

4.0. SPECIAL FEATURES COMPONENT

4.1. RARITY AND/OR SCARCITY

4.1.1. Individual Wetlands

Name of Physiographic Unit: Simcoe Lowlands

Unit Number: 8

4.1.2. Wetland Type Representation (minimum size 0.5 ha)

(check one or more)

- ✓ Marsh
- ✓ Swamp
- ✓ Fen
- ___ Bog

4.1.3. Individual Species

4.1.3.1. Breeding Habitat for an Endangered Animal or Plant Species

	Name of Species	Source of Information
(1)		
(2)		

4.1.3.2. Traditional Migration or Feeding Habitat for an Endangered Animal Species

	Name of Species	Source of Information
(1)		
(2)		

4.1.3.3. Breeding or Feeding Habitat for a Provincially Significant Animal Species

	Name of Species	Source of Information
(1)	Caspian tern	observation
(2)		

4.1.3.4. Provincially Significant Plant Species

	Name of Species	Source of Information
(1)		
(2)		

4.2.5. Migratory Passerine and/or Shorebird Stopover Area

(check one)

(1) ___ High significance

(2) ✓ No significance

Source of Information: observation MNR District Biologist Robin Craig

4.2.6. Significance for Fish Spawning and Rearing

(check one)

(1) ___ Regional significance

(2) ✓ Present

(3) ___ Unknown

(4) ___ Not possible

Species and Source of Information: Walleye, Pike, LM Bass, forage fish. Lake survey, 1968 (R Toth) MNR

4.2.7. Unusual Geological or other Surficial Features

Feature	Source of Information
(1) —	
(2) —	

4.3. ECOLOGICAL AGE

Type of Wetland	Enter % of Area
___ Bog	
✓ Fen	1%
✓ Swamp	52%
✓ Marsh	47%

INVESTIGATORS

[illegible]

Gail [illegible]

AFFILIATION

MNR

DATE

July [illegible]/84

ESTIMATED TIME DEVOTED TO COMPLETING THE FIELD SURVEY IN "PERSON HOURS"

WEATHER CONDITIONS

(i) at time of field work:

(ii) summer conditions in general: Normal

WETLAND EVALUATION RECORD

WETLAND NAME AND/OR NUMBER Willow Creek / Little Lake

1.0 BIOLOGICAL COMPONENT

1.1. PRODUCTIVITY VALUES

1.1.1.	Growing Degree-Days	14
1.1.2.	Soils	6
1.1.3.	Type of Wetland	16
1.1.4.	Site	4
1.1.5.	Nutrient Status of Surface Water	20
	TOTAL for Productivity Values	60

1.2. DIVERSITY VALUES

1.2.1.	Number of Wetland Types	9
1.2.2.	Vegetation Communities (not to exceed 30)	30
1.2.3.	Diversity of Surrounding Habitat	10
1.2.4.	Proximity to Other Wetlands	10
1.2.5.	Interspersion	12
1.2.6.	Open Water Types	8
	TOTAL for Diversity Values	79

1.3. SIZE (Biological Component) 50

TOTAL FOR BIOLOGICAL COMPONENT (not to exceed 250) 189

2.0 SOCIAL COMPONENT

2.1. RESOURCE PRODUCTS WITH CASH VALUE

2.1.1.	Timber (lumber and firewood)	10
2.1.2.	Wild Rice	10
2.1.3.	Commercial Fish (Bait Fish and/or Coarse Fish)	30
2.1.4.	Bullfrogs	0
2.1.5.	Snapping Turtles	2
2.1.6.	Furbearers	15
	TOTAL for Resource Products with Cash Value (not to exceed 60)	60

2.2. RECREATIONAL ACTIVITIES (not to exceed 70) 70

2.3. AESTHETICS

2.3.1.	Landscape Distinctness	5
2.3.2.	Absence of Human Disturbances	15
	TOTAL for Aesthetics	20

2.4. EDUCATION AND PUBLIC AWARENESS

2.4.1.	Educational Uses	0
2.4.2.	Facilities and Programs	0
2.4.3.	Research and Studies	3
	TOTAL for Education and Public Awareness	3

2.5. PROXIMITY TO URBAN AREAS 20

2.6. OWNERSHIP/ACCESSIBILITY 14

2.7. SIZE (Social Component) 20

TOTAL FOR SOCIAL COMPONENT (not to exceed 250) 207

3.0. HYDROLOGICAL COMPONENT

3.1. EFFECT OF ADJOINING LARGE WATER BODY

3.2. FLOW STABILIZATION

3.2.1. Detention Due to Surface Area					
3.2.1.1. and					
3.2.1.2. FIRST step (from table)				110 90	
3.2.1.3. SECOND step	minus	1	=	109	
3.2.1.5. THIRD step	minus	34	=	75	
3.2.1.6. FOURTH step	minus	0	=	75	←(minimum allowable = 0)
3.2.1.7. FIFTH step	plus	34	=	109	
TOTAL for Detention Due to Surface Area				109	
3.2.2. Flow Augmentation (from table)				0	
TOTAL for Flow Stabilization					109 97

3.3. WATER QUALITY IMPROVEMENT

3.3.1. Short Term Removal of Nutrients from Surface Water		
3.3.1.1. Site Type	2	
3.3.1.2. Actual Wetland Area Dominated by Robust Emergents and Submergents	6	
3.3.1.3. Land Use in Catchment Basin	10	
TOTAL for Short Term Removal of Nutrients from Surface Water		18
3.3.2. Long Term Nutrient Trap		6
TOTAL for Water Quality Improvement		24

3.4. EROSION CONTROL

3.4.1. Erosion Buffer		
3.4.1.1. Riverine Wetlands	0	
3.4.1.2. Lacustrine Wetlands	15	
3.4.1.3. Fetch	1	
TOTAL for Erosion Buffer		16
3.4.2. Sheet Erosion		0
TOTAL for Erosion Control		16

TOTAL FOR HYDROLOGICAL COMPONENT (not to exceed 250) 149

4.0 SPECIAL FEATURES COMPONENT

4.1. RARITY AND/OR SCARCITY

4.1.1. Individual Wetlands	20
4.1.2. Wetland Type Representation	20
4.1.3. Individual Species	
4.1.3.1. Breeding Habitat for an Endangered Animal or Plant Species	0
4.1.3.2. Traditional Migration or Feeding Habitat for an Endangered Animal Species	0
4.1.3.3. Breeding or Feeding Habitat for a Provincially Significant Animal Species	100
4.1.3.4. Provincially Significant Plant Species	0
4.1.3.5. Regionally Significant Species	0
TOTAL for Individual Species (not to exceed 250)	100
TOTAL FOR RARITY AND/OR SCARCITY (not to exceed 250)	140

4.2. SIGNIFICANT FEATURES AND/OR FISH AND WILDLIFE HABITAT

4.2.1. Nesting of Colonial Waterbirds	3
4.2.2. Winter Cover for Wildlife	10
4.2.3. Waterfowl Staging	0
4.2.4. Waterfowl Production	5
4.2.5. Migratory Passerine and/or Shorebird Stopover Area	0
4.2.6. Significance for Fish Spawning and Rearing	15
4.2.7. Unusual Geological or other Surficial Features	0
TOTAL FOR SIGNIFICANT FEATURES AND/OR FISH AND WILDLIFE HABITAT (not to exceed 250)	33

4.3. ECOLOGICAL AGE

2

TOTAL FOR SPECIAL FEATURES COMPONENT (not to exceed 250) 175

SUMMARY OF EVALUATION RESULTS

FOR THE Willow Creek (Little Lake) WETLAND
(name or number)

TOTAL FOR 1.0, BIOLOGICAL COMPONENT	189
TOTAL FOR 2.0, SOCIAL COMPONENT	207
TOTAL FOR 3.0, HYDROLOGICAL COMPONENT	~~149~~ 129
TOTAL FOR 4.0, SPECIAL FEATURES COMPONENT	175
	720

M.S.

Cl. I

INVESTIGATORS

MARK FERGUSON (Revised).

AFFILIATION

M.N.R - HURONIA District

DATE

Dec. 5/86

Number of natural areas selected: 1

COOKSTOWN HOLLOWS SWAMP

AREA_ID: 5643

Significance	Area Type	Size	Centroid UTM	Map #
Provincial	Wetland	656.7 ha	17,607900,4893600	31D/4

Description
A Provincially significant wetland, composed of two wetland types (99% swamp and 1% marsh) (Sheridan and Coulter, 1985);

Vegetation
Dominant Vegetation Forms (Sheridan and Coulter, 1985):
73% deciduous trees, 20% coniferous trees, 4% tall shrubs, 3% low shrubs, robust emergents, narrow-leaved emergents, and submergents;

Vegetation Communities (Sheridan and Coulter, 1985);
One form
M1: robust emergents;
M2: narrow-leaved emergents;
W1: submergents;

Two forms
S1: coniferous trees; herbs;
S2: deciduous trees; free-floating plants;
W2: submergents; narrow-leaved emergents;
W3: narrow-leaved emergents; robust emergents;
W4: submergents; robust emergents;

Three forms
S3: tall shrubs; low shrubs; narrow-leaved emergents;
S4: deciduous trees; low shrubs; herbs;
S5: low shrubs; tall shrubs; narrow-leaved emergents;
S6: coniferous trees; deciduous trees; herbs;
S7: deciduous trees; narrow-leaved emergents; free-floating plants;
W5: submergents; floating plants; free-floating plants;

Four forms
S8: deciduous trees; dead deciduous trees; tall shrubs; free-floating plants;

Five forms
S9: deciduous trees; tall shrubs; narrow-leaved emergents; submergents; free-floating plants;

Representation

Landform

Soils (Sheridan and Coulter. 1985): 75% clays, loams or silts; 25% organic;
Site Type (Sheridan and Coulter. 1985): 90% palustrine (permanent or intermittent outflow), 10% riverine;

References

Sheridan. E. and S. Coulter. 1985. Wetland Data Record and Evaluation- Cookstown Hollows Swamp. Second Edition. June 5-7, 1985. Nottawasaga Valley Conservation Authority. Manuscript. 22 pp + 1 map.

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Produced by the Natural Heritage Information Centre

APPENDIX G
Fish Habitat Crossing Photos

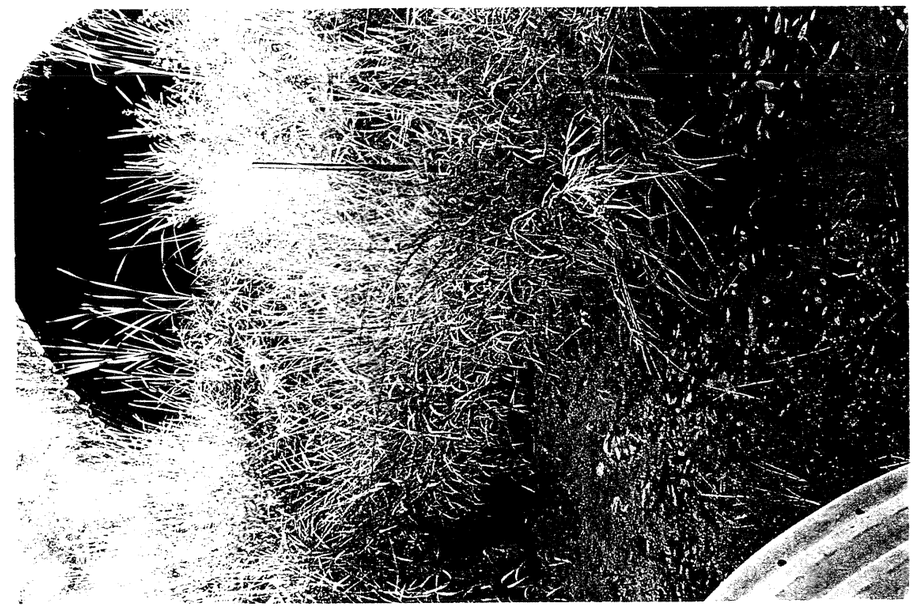

Plate 1. 10+280 Innisfil Township-Upstream

Plate 2. 10+280 Innisfil Township-Downstream

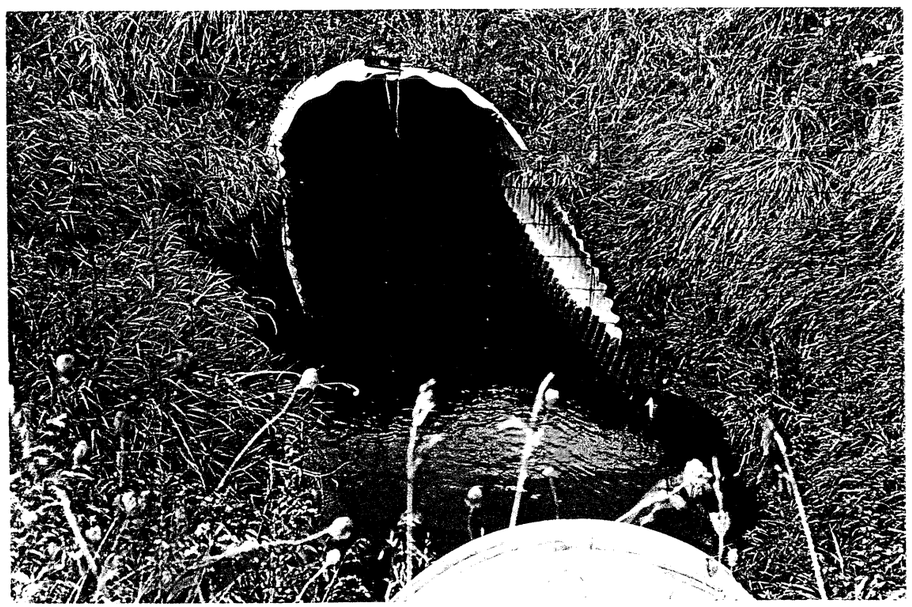

Plate 3. 10+770 Innisfil Township-Upstream

Plate 4. 10+770 Innisfil Township-Downstream

Plate 5. 11+680 Innisfil Township-Upstream

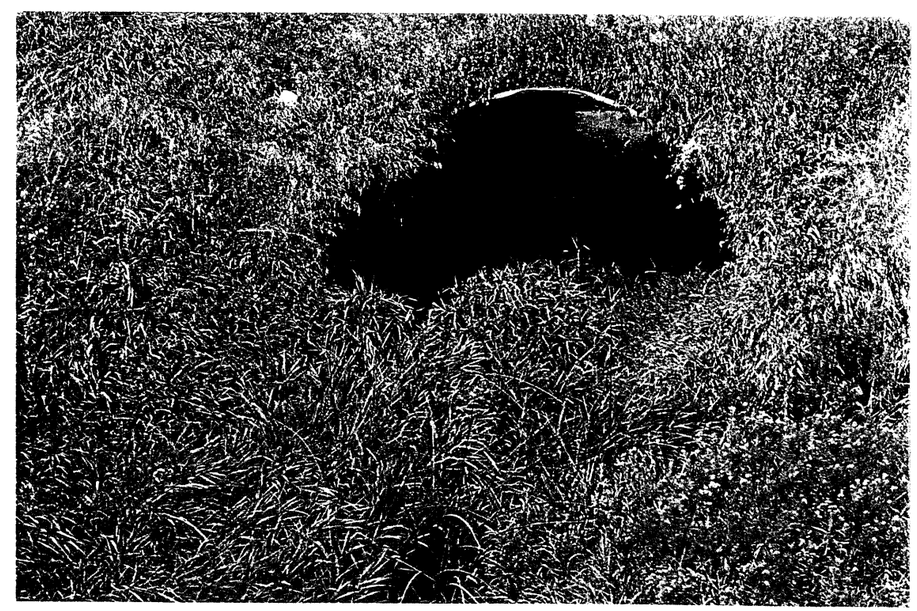

Plate 6. 11+680 Innisfil Township-Downstream

Plate 7. 12+920 Innisfil Township-Upstream

Plate 8. 12+920 Innisfil Township-Downstream

Plate 9. 13+120 Innisfil Township-Upstream

Plate 10. 13+120 Innisfil Township-Downstream

Plate 11. 16+724 Innisfil Township-Upstream

Plate 12. 16+724 Innisfil Township-Downstream

Plate 13. 18+402 Innisfil Township-Upstream

Plate 14. 18+402 Innisfil Township-Downstream

Plate 15. 23+272 Innisfil Township-Upstream

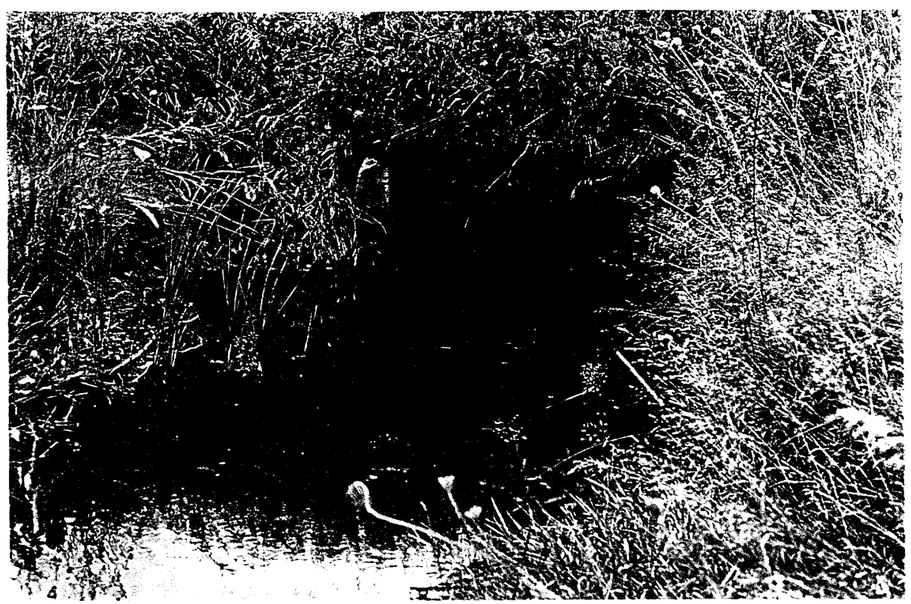

Plate 16. 23+272 Innisfil Township-Downstream

Plate 17. 26+750 Innisfil Township-Upstream

Plate 18. 26+750 Innisfil Township-Downstream

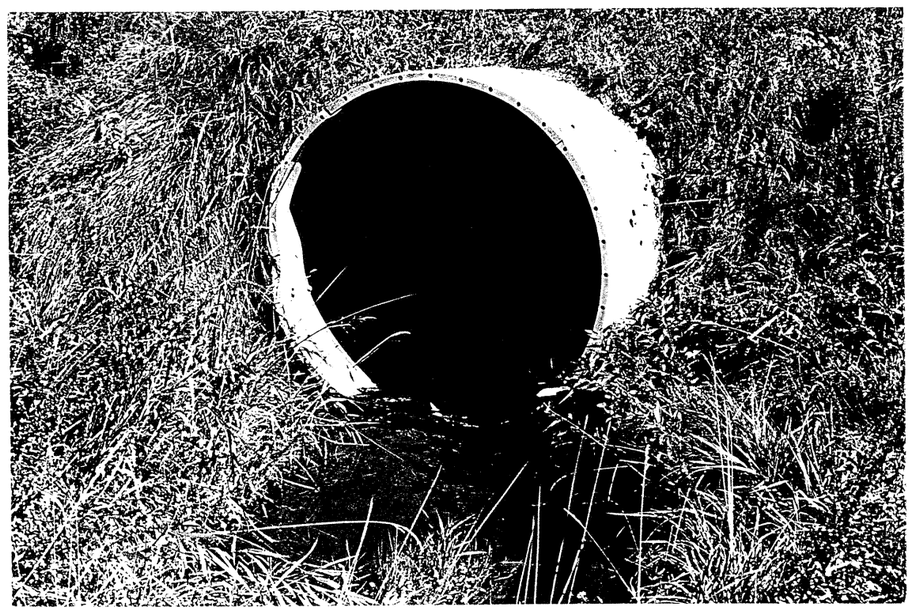

Plate 19. 29+280 Innisfil Township-Upstream

Plate 20. 29+280 Innisfil Township-Downstream

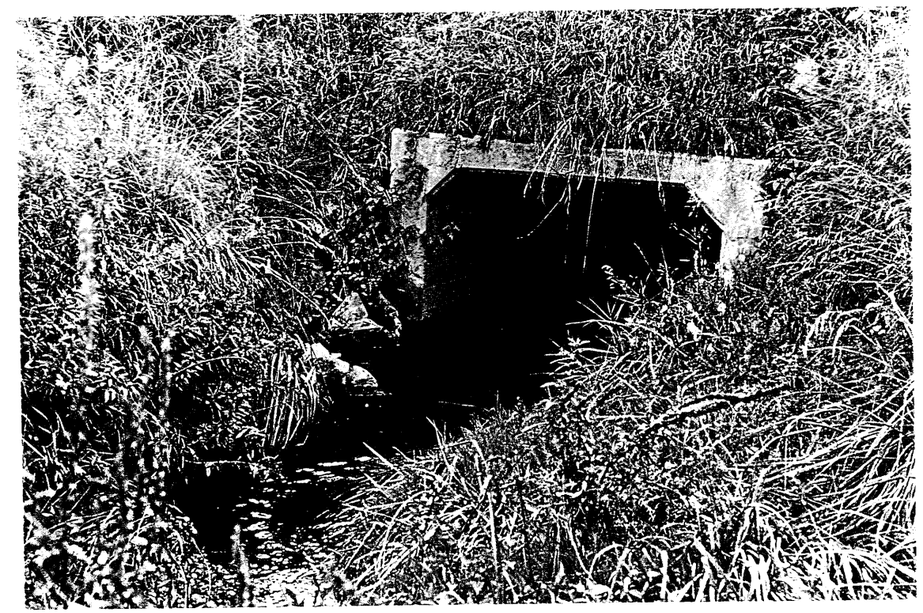

Plate 21. 11+030 Vespra Township-Downstream

Plate 22. 12+200 Vespra Township-Upstream

Plate 23. 12+200 Vespra Township-Downstream

Plate 24. 12+448 Vespra Township-Upstream

Plate 25. 12+448 Vespra Township-Downstream

Plate 26. 12+817 Vespra Township-Upstream

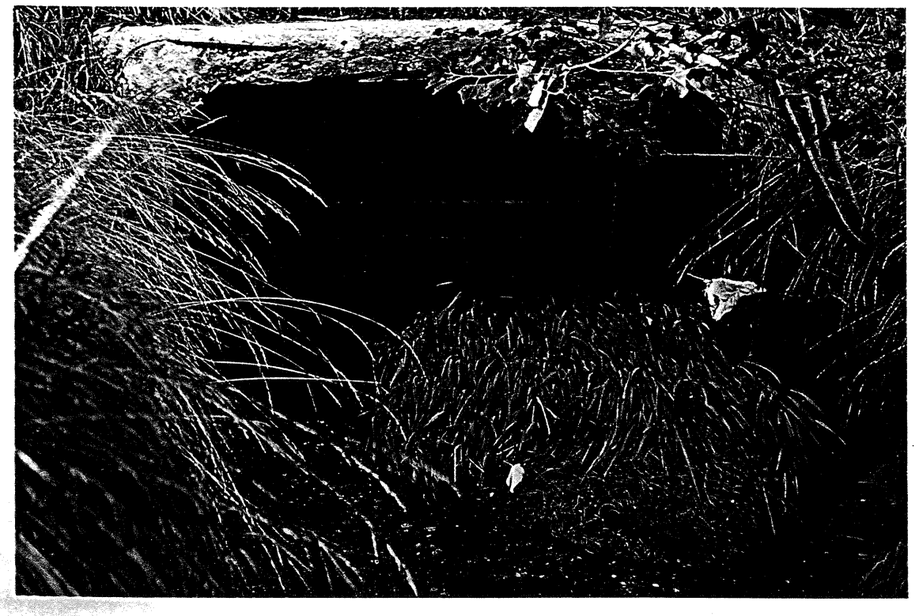

Plate 27. 12+817 Vespra Township-Downstream

Plate 28. 18+280 Vespra Township-Upstre n

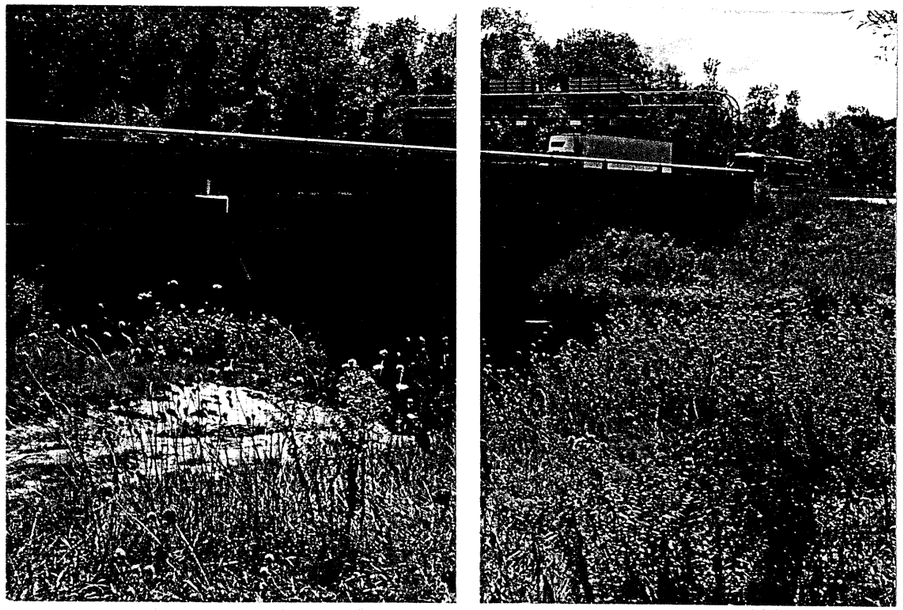

Plate 29. 18+280 Vespra Township-Down: eam

APPENDIX H
Scientific Collectors Permit

Ontario

Ministry of Natural Resources
Ministère des Richesses naturelles

Licence to Collect Fish for Scientific Purposes
Permis pour faire la collecte de poissons à des fins scientifiques

Licence No. / N° de permis: 2000 - 018
District ID # / N° d'ident. du district: 71

This licence is issued under Part I of the Fish Licensing Regulation made under the Fish and Wildlife Conservation Act, 1997 to:
Ce permis est délivré en vertu de la Partie I du règlement sur la délivrance de permis de pêche formulé conformément à la *Loi sur la protection du poisson et de la faune* de 1997 à :

Name of Licencee / Nom du titulaire	Last Name / Nom de famille: BILZ	First Name / Prénom: Rod	Middle Name / Second prénom:
	Name of Business/Organization/Affiliation (if applicable) / Nom de l'entreprise/de l'organisme/de l'affiliation (le cas échéant): FRi Ecological Services		
Mailing address of Licencee / Adresse postale du titulaire	Street Name & No./PO Box/RR#/Gen. Del. / N°, rue/C.P./R.R./poste restante: 1875 SEymour Street		
	City/Town/Municipality / Ville/village/municipalité: North Bay	Province/State / Province/Etat: Ontario	Postal Code/Zip Code / Code postal/Zip: P1B 8G4

to collect the species, size and quantities of fish from the waters as set out below.
pour faire la collecte des espèces suivantes (stade et nombre indiqués ci-dessous) :

(Attach list, if insufficient space) (Joignez une liste si vous manquez d'espace)

Species / Espèce	Eggs / Oeuf √	Juvenile / Fretin √	Adults / Adulte √	Numbers / Nombre	Name of Waterbody / Nom de l'étendue d'eau
Various Species Present	--	--	--	0	Watercourses crossed by the Hwy 400 corridor between the Holland River north to the Hwy 400/Hwy interchange

Purpose of collection / But le la collecte: To provide for sampling, inventory and live release of all fish collected by eletrofishing, trapnet, minnow trap ~~and other~~ GF equipment.

Licence conditions / Conditions du permis: Yes/Oui [X] No/Non [ ] Conditions attached / Conditions ci-jointes

Licence Dates / Dates du permis: Effective Date / Date d'entrée en vigueur: 2000 09 26 — Expiry Date / Date d'expiration: 2000 10 31

Issued by (please print) / Délivré par (veuillez écrire en caractères d'imprimerie): John T. Kus, Area Supervisor
Signature of Issuer / Signature du délivreur:
Date of Issue / Date de délivrance: 2000 09 26

Signature of Licencee / Signature du titulaire du permis:
Date: 2000 09 26

Personal information contained on this form is collected under the authority of the *Fish and Wildlife Conservation Act, 1997* and will be used for the purpose of licencing, identification, enforcement, resource management and customer service surveys. Please direct further enquiries to the District Manager of the MNR issuing district.

Les renseignements personnels dans ce formulaire sont recueillis conformement à la *Loi sur la protection du poisson et de la faune*, 1997, et ils seront utilisés aux fins de delivrance de permis, d'identification, d'application des reglements, de gestion des ressources et de sondage sur les services à la clientèle. Veuillez communiquer avec le chef du district du MRN qui delivre le permis si vous avez des questions.

FW0032 (02/99)

LICENCE TO COLLECT FISH FOR SCIENTIFIC PURPOSES CONDITIONS

Licence Number	Licence Holder	Date
2000-018	Rod Bilz	September 26, 2000

This licence is subject to the conditions listed below:

1. This Licence is valid only for the persons, species, numbers, areas and calendar year indicated. A written report covering the operation of the preceding year must be submitted to the licence issuer within 30 days of the termination date, but in no case later than January 31 next following the year of issue. The report shall contain a statement outlining the objectives of the operations, the methods used, the number and species of fish caught and their fate as well as a map indicating where the collections took place. An analysis is not required. The submission of a satisfactory report is a prerequisite to any subsequent renewals.

2. Before carrying out any operation under the licence in any area the licenced person shall inform the Area Supervisor or Lake Manager of his or her intentions at least a week before commencing work and include information as to the type of operation, location, duration, and the name or names of personnel involved.

3. A copy of the original licence must be carried by the licenced person when working at the designated sites. An assistant of the licenced person who is carrying out activities under this licence during the absence of the licenced person shall carry a copy of the licence on his or her person.

4. All collection gear shall be clearly marked with the licenced person's and the organization's name.

5. This licence is not valid in Provincial Parks, park reserves, or National Parks without the written permission from the authorized person in charge of the area concerned.

6. Capture gear shall be inspected regularly and live holding traps must be inspected at least once daily.

7. This licence does not allow access to any property without permission of the landowner.

8. All fish are to be released alive back to the watercourse from which they were collected. Exception, individual fish may be retained only to obtain positive identification. Advise the issuing office at the time of collection if fish are retained.

APPENDIX I
Fish Collection Records

FIELD COLLECTION RECORD

Project #	Project Description	Day/Month/Year
GWP 30-95-00 ?	Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	10/10/2000

Name of Waterbody	Drainage System	Station #	Collection #
tributary of Innisfil Creek	2ED	27+300	1

Location of Crossing	Structure Type
South of Highway 89 on Highway 400	☐ Bridge ☒ Box ☐ CSP ☐ Other

Latitude	Longitude
44°11'47"	79°39'09"

Township	MNR District / Area
West Gwillimbury	Midhurst

Collectors
Bilz, Hulse

Weather Conditions
Overcast, cool

Station	Length (m)	Time Started	Time Finished	Sampling Duration
Upstream	20	1005	1020	15
Downstream	100	1030	1055	25

Water Type		Permanent	Associated Wetland
☒ Stream/River	☐ Lake	☒ Permanent	Cookstown Hollows Swamp PSW
☐ Pond	☐ Canal	☐ Intermittent	

Channel Dimensions	Mean Width (m)	Mean Depth (m)	Bankfull Width (m)	Bankfull Depth (m)	W/D Ratio
Upstream	0.7	0.05	1.3		14.0
Downstream	0.5	0.08	1.1		6.25

	Stream Gradient (m/100m)	Current Velocity (m/s)	Stream realignment required
Upstream			☐ Yes ☒ No
Downstream			☐ Yes ☒ No

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Conductivity
Upstream			12	9	
Downstream			12	9	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
Upstream				100	
Downstream				100	

Water Colour	Colourless	Yellow/Brown	Blue/Green	Turbid	Other
Upstream	☒	☐	☐	☐	☐
Downstream	☒	☐	☐	☐	☐

Substrate (%Upstream Downstream)	Bedrock	Boulder	Rubble	Gravel	Sand	Silt	Clay	Muck	Marl	Detritus
Upstream					30	30		40		
Downstream					20	40		40		

Instream Cover (% surface area)	Undercut Banks	Boulders	Logs/Trees	Organic Debris	VascularPlants	None
Upstream				10	15	75
Downstream				8	15	77

FIELD COLLECTION RECORD – Page 2

Shore Cover (% of stream shaded)	90-100%	60-90%	30-60%	1-30%	None
Upstream	☐	☐	☒	☐	☐
Downstream	☐	☐	☐	☒	☐

Vegetation Type (% of surface area)	Submergent	Floating	Emergent	None
Upstream			15	85
Downstream			15	85

Banks	Stable (%)	Unstable (%)	Surrounding Land Use
Upstream	100		Forested
Downstream	90	10	

Migratory Obstructions	None	Seasonal	Permanent	Sources of Pollution
Upstream	☒			
Downstream	☒			

Potential Critical Habitat	Spawning	Upwellings	Other
Upstream	☐	☐	
Downstream	☐	☐	

Potential Enhancement Opportunities
Refuge pool on downstream side

Comments

Sinuosity	D50	Bankfull Return Period	Erodible Soils
		☐ >1.2-1.5yrs ☐ <1.2-1.5yrs	☐ High ☐ Moderate ☒ Low

Stream Classifications

MNR		DFO	Rosgen
☐ Coldwater	☒ Warmwater Habitat	☐ 1	
☐ Potential Coldwater	☐ Warmwater Urban	☐ 2	
☐ Warmwater Sportsfish	☐ Migratory	☐ 3	

Additional Notes

FIELD COLLECTION RECORD

Project Name	Project #	Date: Day/Month/Year
Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	GWP 30-95-00	10/10/2000

Name of Waterbody	Drainage System	Station No.	Collection No.
tributary of Innisfil Creek	2ED	27+300	1

County or Judicial District	Township
Simcoe	West Gwillimbury

UTM Coordinates	Easting	Northing
	607690	4894330

Data Collectors
Bilz, Hulse

Weather Conditions
Overcast, cool

Water Type
☐ Spring ☐ Canal ☒ Stream/River ☐ River/Lake Junction ☐ Flooded Area ☐ Pool
☐ Pond ☐ Lake ☐ Muskeg/Bog ☐ Reservoir ☐ Other

Dimensions	Length(m)	Mean Wetted (m)	Mean Channel Width (m)	Mean Depth(m)	Current Velocity
	120	0.6	1.2	0.075	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
				100	

Plant Type (%)	Submergent	Floating	Emergent	None
	Eg.	Eg.	15 Eg. cattails sedge reed canary grass	

Bottom Type (%)	Bedrock	Boulder	Rubble	Gravel	Sand	Other
	Clay	Muck 40	Marl	Detritus	25 Silt 35	

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Water Color
			12	9	☒ Colorless ☐ Blue/Green ☐ Yellow/Brown ☐ Other ☐ Turbid

Cover (instream) % of total surface area	Undercut Banks	Boulders	Logs/Trees	Organic Debris	Vascular Macrophytes	Combination	No Cover
				9	15		76

Cover/Canopy (Shore) % stream shaded	Full Canopy (100%)	Fair Canopy (40-60%)	Bank Characteristics (stability)	Surrounding Land Use
	Good Canopy (70-90%) ☐ No Canopy ☐	Poor Canopy (10-30%) ☒	stable	Forested

Gear	Electrofisher	Seine	Gill Net	Angled	Dip Net	Surber
	☒ Length 120m Electrosec 108 Hoop Net ☐	☐ # Hauls Trap Net ☐	☐ Minnow Trap ☐	☐ Trawl ☐	☐ Piscicide ☐	☐ Other ☐

Size of Net (Gill or Seine)	Size of net or mouth (Trap,Hoop,Trawl)	Mesh Size	Depth of Capture
Length m	m	Smallest cm Largest cm	Min 0m Max 0.15m

Fish Kept?	No. of Bags	Preservative
Yes ☐ No ☒		Formalin ☐ Kahle's Solution ☐ Alcohol ☐ Frozen ☐ Other ☐

Additional Data: Pollution,Migratory Obstructions, Parasites,Color & Condition of Fish etc.	Habitat Map Included
	Yes ☐ No ☒

FIELD COLLECTION RECORD

Project No. GWP 30-95-00	Station No. 27+300	Collection No. 1

No.	Scientific Name	Total Length Range (cm)	Weight Range (gm)	Physical Condition	
				# fish with blackspot	(lesions, tumors, maturity etc.)
1	Central Mudminnow Umbra limi	5.4			
2	Brook Stickleback Culaea inconstans	2.4-4.1			

Identified by:	Date	Day / Month / Year
R. Bilz		10/10/2000

Comments:

STATION DIAGRAM

FIELD COLLECTION RECORD

Project #	Project Description	Day/Month/Year
GWP 30-95-00	Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	10/10/2000

Name of Waterbody	Drainage System	Station #	Collection #
tributary of Innisfil Creek	2ED	27+527	1

Location of Crossing	Structure Type
South of Highway 89 on Highway 400	☐ Bridge ☒ Box ☐ CSP ☐ Other

Latitude	Longitude
44°12'00"	79°39'17"

Township	MNR District / Area
West Gwillimbury	Midhurst

Collectors
Bilz, Hulse

Weather Conditions
Overcast, cool

Station	Length (m)	Time Started	Time Finished	Sampling Duration
Upstream	20	1105	1110	5
Downstream	100	1125	1200	35

Water Type		Permanent	Associated Wetland
☒ Stream/River	☐ Lake	☒ Permanent	
☐ Pond	☐ Canal	☐ Intermittent	

Channel Dimensions	Mean Width (m)	Mean Depth (m)	Bankfull Width (m)	Bankfull Depth (m)	W/D Ratio
Upstream	0.55	0.25	0.9		2.2
Downstream	0.45	0.18	1.1		2.5

	Stream Gradient (m/100m)	Current Velocity (m/s)	Stream realignment required
Upstream			☐ Yes ☒ No
Downstream			☐ Yes ☒ No

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Conductivity
Upstream			13	9	
Downstream			13	9	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
Upstream				100	
Downstream			20	80	

Water Colour	Colourless	Yellow/Brown	Blue/Green	Turbid	Other
Upstream	☒	☐	☐	☐	☐
Downstream	☒	☐	☐	☐	☐

Substrate (%Upstream Downstream)	Bedrock	Boulder	Rubble	Gravel	Sand	Silt	Clay	Muck	Marl	Detritus
Upstream				10	30	30	30			
Downstream				10	30	40	20			

Instream Cover (% surface area)	Undercut Banks	Boulders	Logs/Trees	Organic Debris	VascularPlants	None
Upstream				10	25	65
Downstream				5	25	70

FIELD COLLECTION RECORD – Page 2

Shore Cover (% of stream shaded)	90-100%	60-90%	30-60%	1-30%	None
Upstream	☐	☐	☐	☐	☒
Downstream	☐	☐	☐	☐	☒

Vegetation Type (% of surface area)	Submergent	Floating	Emergent	None
Upstream			25	75
Downstream			25	75

Banks	Stable (%)	Unstable (%)	Surrounding Land Use
Upstream	100		Commercial
Downstream	100		Highway Interchange

Migratory Obstructions	None	Seasonal	Permanent	Sources of Pollution
Upstream	☒			
Downstream	☒			

Potential Critical Habitat	Spawning	Upwellings	Other
Upstream	☐	☐	
Downstream	☐	☐	

Potential Enhancement Opportunities
Plant shrubs within ROW

Comments

Sinuosity	D50	Bankfull Return Period	Erodible Soils
		☐ >1.2-1.5yrs ☐ <1.2-1.5yrs	☐ High ☐ Moderate ☒ Low

Stream Classifications
MNR
☐ Coldwater
☐ Potential Coldwater
☐ Warmwater Sportsfish
☒ Warmwater Habitat
☐ Warmwater Urban
☐ Migratory

DFO
☐ 1
☐ 2
☐ 3

Rosgen

Additional Notes

FIELD COLLECTION RECORD

Project Name	Project #	Date: Day/Month/Year
Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	GWP 30-95-00	10/10/2000

Name of Waterbody	Drainage System	Station No.	Collection No.
tributary of Innisfil Creek	2ED	27+527	1

County or Judicial District	Township
Simcoe	West Gwillimbury

UTM Coordinates	Easting	Northing
	607490	4894760

Data Collectors
Bilz, Hulse

Weather Conditions
Overcast, cool

Water Type
☐ Spring ☐ Canal ☒ Stream/River ☐ River/Lake Junction ☐ Flooded Area ☐ Pool
☐ Pond ☐ Lake ☐ Muskeg/Bog ☐ Reservoir ☐ Other

Dimensions	Length(m)	Mean Wetted (m)	Mean Channel Width (m)	Mean Depth(m)	Current Velocity
	120	0.5	0.9	0.2	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
			20	80	

Plant Type (%)	Submergent	Floating	Emergent	None
	Eg.	Eg.	25 Eg. cattails reed canary grass	

Bottom Type (%)	Bedrock	Boulder	Rubble	Gravel	Sand	Other
	Clay 25	Muck	Marl	10 Detritus	30 Silt 35	

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Water Color
			13	9	☒ Colorless ☐ Yellow/Brown ☐ Turbid ☐ Blue/Green ☐ Other

Cover (instream) % of total surface area	Undercut Banks	Boulders	Logs/Trees	Organic Debris	Vascular Macrophytes	Combination	No Cover
				8	25		67

Cover/Canopy (Shore) % stream shaded	Full Canopy (100%)	Fair Canopy (40-60%)	Bank Characteristics (stability)	Surrounding Land Use
	Good Canopy (70-90%) ☐ No Canopy ☒	Poor Canopy (10-30%) ☐	stable	Commercial Highway Interchange

Gear	Electrofisher	Seine	Gill Net	Angled	Dip Net	Surber
	☒ Length 120m Electrosec 68 Hoop Net ☐	☐ # Hauls Trap Net ☐	☐ Minnow Trap ☐	☐ Trawl ☐	☐ Piscicide ☐	☐ Other ☐

Size of Net (Gill or Seine)	Size of net or mouth (Trap,Hoop,Trawl)	Mesh Size	Depth of Capture
Length m	m	Smallest cm Largest cm	Min 0m Max 0.15m

Fish Kept?	No. of Bags	Preservative
Yes ☐ No ☒		Formalin ☐ Kahle's Solution ☐ Alcohol ☐ Frozen ☐ Other ☐

Additional Data: Pollution,Migratory Obstructions, Parasites,Color & Condition of Fish etc.	Habitat Map Included
	Yes ☐ No ☒

FIELD COLLECTION RECORD

Project No. GWP 30-95-00	Station No. 27+527	Collection No. 1

No.	Scientific Name	Total Length Range (cm)	Weight Range (gm)	Physical Condition	
				# fish with blackspot	(lesions, tumors, maturity etc.)
4	Brook Stickleback Culaea inconstans	2.6-4.2			

Identified by:	Date	Day / Month / Year
R. Bilz		10/10/2000

Comments:

STATION DIAGRAM

FIELD COLLECTION RECORD

Project #	Project Description	Day/Month/Year
GWP 30-95-00	Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	26/09/2000

Name of Waterbody	Drainage System	Station #	Collection #
tributary of Innisfil Creek	2ED	10+282	1

Location of Crossing	Structure Type
North of Highway 89 on Highway 400	☐ Bridge ☒ Box ☐ CSP ☐ Other
Latitude: 44°12'11"	Longitude: 79°39'22"

Township	MNR District / Area
Innisfil	Midhurst

Collectors: Bilz, Hulse

Weather Conditions: Sunny, Clear

Station	Length (m)	Time Started	Time Finished	Sampling Duration
Upstream	25	1600	1605	5
Downstream	100	1605	1615	10

Water Type		Permanent	Associated Wetland
☒ Stream/River	☐ Lake	☒ Permanent	
☐ Pond	☐ Canal	☐ Intermittent	

Channel Dimensions	Mean Width (m)	Mean Depth (m)	Bankfull Width (m)	Bankfull Depth (m)	W/D Ratio
Upstream	0.4	0.22	0.8		1.8
Downstream	0.6	0.61	1.0		1.0

	Stream Gradient (m/100m)	Current Velocity (m/s)	Stream realignment required
Upstream			☐ Yes ☒ No
Downstream			☐ Yes ☒ No

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Conductivity
Upstream			13	15	
Downstream			13	15	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
Upstream			20	80	
Downstream			20	80	

Water Colour	Colourless	Yellow/Brown	Blue/Green	Turbid	Other
Upstream	☒	☐	☐	☐	☐
Downstream	☒	☐	☐	☐	☐

Substrate (%Upstream Downstream)	Bedrock	Boulder	Rubble	Gravel	Sand	Silt	Clay	Muck	Marl	Detritus
Upstream								100		
Downstream								100		

Instream Cover (% surface area)	Undercut Banks	Boulders	Logs/Trees	Organic Debris	VascularPlants	None
Upstream					15	85
Downstream					15	85

FIELD COLLECTION RECORD —Page 2

Shore Cover (% of stream shaded)	90-100%	60-90%	30-60%	1-30%	None
Upstream	☐	☐	☐	☐	☒
Downstream	☐	☐	☐	☐	☒

Vegetation Type (% of surface area)	Submergent	Floating	Emergent	None
Upstream		25	15	65
Downstream		35	10	55

Banks	Stable (%)	Unstable (%)	Surrounding Land Use
Upstream	100		Commercial/Agricultural
Downstream	100		Highway Interchange

Migratory Obstructions	None	Seasonal	Permanent	Sources of Pollution
Upstream	☒			
Downstream	☒			

Potential Critical Habitat	Spawning	Upwellings	Other
Upstream	☐	☐	
Downstream	☐	☐	

Potential Enhancement Opportunities
Plant shrubs within ROW

Comments
Strong Hydrogen Sulphide odour
Evidence of nutrient enrichment

Sinuosity	D50	Bankfull Return Period	Erodible Soils
		☐ >1.2-1.5yrs ☐ <1.2-1.5yrs	☐ High ☐ Moderate ☒ Low

Stream Classifications
MNR
☐ Coldwater
☐ Potential Coldwater
☐ Warmwater Sportsfish
☒ Warmwater Habitat
☐ Warmwater Urban
☐ Migratory

DFO
☐ 1
☐ 2
☐ 3

Rosgen

Additional Notes

FIELD COLLECTION RECORD

Project Name	Project #	Date: Day/Month/Year
Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	GWP 30-95-00	26/09/2000

Name of Waterbody	Drainage System	Station No.	Collection No.
tributary of Innisfil Creek	2ED	10+282	1

County or Judicial District	Township
Simcoe	Innisfil

UTM Coordinates	Easting	Northing
	607370	4895080

Data Collectors
Bilz, Hulse

Weather Conditions
Sunny Clear

Water Type
☐ Spring ☐ Canal ☒ Stream/River ☐ River/Lake Junction ☐ Flooded Area ☐ Pool
☐ Pond ☐ Lake ☐ Muskeg/Bog ☐ Reservoir ☐ Other

Dimensions	Length(m)	Mean Wetted (m)	Mean Channel Width (m)	Mean Depth(m)	Current Velocity
	125	0.5	0.9	0.5	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
			20	80	

Plant Type (%)	Submergent	Floating	Emergent	None
	Eg.	30 Eg. duckweed	13 Eg. cattails	

Bottom Type (%)	Bedrock	Boulder	Rubble	Gravel	Sand	Other
	Clay	Muck 100	Marl	Detritus	Silt	

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Water Color
			13	15	☒ Colorless ☐ Yellow/Brown ☐ Turbid ☐ Blue/Green ☐ Other

Cover (instream) % of total surface area	Undercut Banks	Boulders	Logs/Trees	Organic Debris	Vascular Macrophytes	Combination	No Cover
					15		85

Cover/Canopy (Shore) % stream shaded			Bank Characteristics (stability)	Surrounding Land Use
	Full Canopy (100%) ☐ Good Canopy (70-90%) ☐ No Canopy ☒	Fair Canopy (40-60%) ☐ Poor Canopy (10-30%) ☐	stable	Commercial/Agricultural Highway Interchange

Gear	Electrofisher	Seine	Gill Net	Angled	Dip Net	Surber
	☒ Length 125 Electrosec 77 Hoop Net ☐	☐ # Hauls Trap Net ☐	☐ Minnow Trap ☐	☐ Trawl ☐	☐ Piscicide ☐	☐ Other ☐

Size of Net (Gill or Seine)	Size of net or mouth (Trap,Hoop,Trawl)	Mesh Size	Depth of Capture
Length m	m	Smallest cm Largest cm	Min 0m Max 0.7m

Fish Kept?	No. of Bags	Preservative
Yes ☐ No ☒		Formalin ☐ Kahle's Solution ☐ Alcohol ☐ Frozen ☐ Other ☐

Additional Data: Pollution,Migratory Obstructions, Parasites,Color & Condition of Fish etc.	Habitat Map Included
	Yes ☐ No ☒

FIELD COLLECTION RECORD

Project No. GWP 30-95-00	Station No. 10+282	Collection No. 1

No.	Scientific Name	Total Length Range (cm)	Weight Range (gm)	Physical Condition	
				# fish with blackspot	(lesions, tumors, maturity etc.)
4	Brook Stickleback Culaea inconstans	2.1-4.2			
2	Creek Chub Semotilus atromaculatus	2.4-3.5			

Identified by:	Date Day / Month / Year
R. Bilz	26/09/2000

Comments:

STATION DIAGRAM

FIELD COLLECTION RECORD

Project #	Project Description	Day/Month/Year
GWP 30-95-00	Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	26/09/2000

Name of Waterbody	Drainage System	Station #	Collection #
Innisfil Creek	2ED	10+760	1

Location of Crossing	Structure Type
North of Highway 89 on Highway 400	☐ Bridge ☒ Box ☒ CSP ☐ Other

Latitude	Longitude
44°12'25"	79°39'30"

Township	MNR District / Area
Innisfil	Midhurst

Collectors: Bilz, Hulse

Weather Conditions: Sunny, Clear

Station	Length (m)	Time Started	Time Finished	Sampling Duration
Upstream	20			
Downstream	100			

Water Type			Associated Wetland
☒ Stream/River	☐ Lake	☒ Permanent	
☐ Pond	☐ Canal	☐ Intermittent	

Channel Dimensions	Mean Width (m)	Mean Depth (m)	Bankfull Width (m)	Bankfull Depth (m)	W/D Ratio
Upstream	8.5	0.5	10.5		17.0
Downstream	9.6	0.8	10.5		12.0

	Stream Gradient (m/100m)	Current Velocity (m/s)	Stream realignment required
Upstream			☐ Yes ☒ No
Downstream			☐ Yes ☒ No

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Conductivity
Upstream			13	15	
Downstream			13	15	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
Upstream	10		55	35	
Downstream	10		80	10	

Water Colour	Colourless	Yellow/Brown	Blue/Green	Turbid	Other
Upstream	☒	☐	☐	☐	☐
Downstream	☒	☐	☐	☐	☐

Substrate (%Upstream Downstream)	Bedrock	Boulder	Rubble	Gravel	Sand	Silt	Clay	Muck	Marl	Detritus
Upstream			15	25	15	25		10		10
Downstream			10	10	20	25		20		15

Instream Cover (% surface area)	Undercut Banks	Boulders	Logs/Trees	Organic Debris	VascularPlants	None
Upstream					5	95
Downstream	20				5	75

FIELD COLLECTION RECORD – Page 2

Shore Cover (% of stream shaded)	90-100%	60-90%	30-60%	1-30%	None
Upstream	☐	☐	☐	☐	☒
Downstream	☐	☐	☐	☐	☒

Vegetation Type (% of surface area)	Submergent	Floating	Emergent	None
Upstream			5	95
Downstream			5	95

Banks	Stable (%)	Unstable (%)	Surrounding Land Use
Upstream	90	10	Agricultural
Downstream	80	20	Golf Course

Migratory Obstructions	None	Seasonal	Permanent	Sources of Pollution
Upstream	☒			Golf Course
Downstream	☐		perched culvert	

Potential Critical Habitat	Spawning	Upwellings	Other
Upstream	☐	☐	
Downstream	☐	☐	

Potential Enhancement Opportunities
Plant shrubs within ROW
Repair perched culvert
Add woody debris

Comments

Sinuosity	D50	Bankfull Return Period	Erodible Soils
		☐ >1.2-1.5yrs ☐ <1.2-1.5yrs	☐ High ☐ Moderate ☒ Low

Stream Classifications

MNR
- ☒ Coldwater
- ☐ Potential Coldwater
- ☐ Warmwater Sportsfish
- ☐ Warmwater Habitat
- ☐ Warmwater Urban
- ☐ Migratory

DFO
- ☐ 1
- ☐ 2
- ☐ 3

Rosgen

Additional Notes

FIELD COLLECTION RECORD

Project Name	Project #	Date: Day/Month/Year
Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	GWP 30-95-00	26/09/2000

Name of Waterbody	Drainage System	Station No.	Collection No.
Innisfil Creek	2ED	10+760	1

County or Judicial District	Township
Simcoe	Innisfil

UTM Coordinates	Easting	Northing
	607200	4895510

Data Collectors
Bilz, Hulse

Weather Conditions
Sunny Clear

Water Type
☐ Spring ☐ Canal ☒ Stream/River ☐ River/Lake Junction ☐ Flooded Area ☐ Pool
☐ Pond ☐ Lake ☐ Muskeg/Bog ☐ Reservoir ☐ Other

Dimensions	Length(m)	Mean Wetted (m)	Mean Channel Width (m)	Mean Depth(m)	Current Velocity
	120	9.0	10.5	0.5	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
	10		70	20	

Plant Type (%)	Submergent	Floating	Emergent	None
	Eg.	Eg.	5 Eg. nodding bur marigold	

Bottom Type (%)	Bedrock	Boulder	Rubble	Gravel	Sand	Other
	Clay	Muck 15	Marl 10	Detritus 15	Silt 25	

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Water Color
			13	15	☒ Colorless ☐ Blue/Green ☐ Yellow/Brown ☐ Other ☐ Turbid

Cover (instream) % of total surface area	Undercut Banks	Boulders	Logs/Trees	Organic Debris	Vascular Macrophytes	Combination	No Cover
	10				5		85

Cover/Canopy (Shore) % stream shaded			Bank Characteristics (stability)	Surrounding Land Use
	Full Canopy (100%) ☐ Good Canopy (70-90%) ☐ No Canopy ☒	Fair Canopy (40-60%) ☐ Poor Canopy (10-30%) ☐	Slightly unstable	Golf Course Agricultural

Gear						
	Electrofisher ☐ Length Electrosec Hoop Net ☐	Seine ☐ # Hauls Trap Net ☐	Gill Net ☐ Minnow Trap ☐	Angled ☐ Trawl ☐	Dip Net ☐ Piscicide ☐	Surber ☐ Other ☐

Size of Net (Gill or Seine)	Size of net or mouth (Trap,Hoop,Trawl)	Mesh Size	Depth of Capture
Length m	m	Smallest cm Largest cm	Min 0m Max 0.7m

Fish Kept?	No. of Bags	Preservative
Yes ☐ No ☒		Formalin ☐ Kahle's Solution ☐ Alcohol ☐ Frozen ☐ Other ☐

Additional Data: Pollution,Migratory Obstructions, Parasites,Color & Condition of Fish etc.	Habitat Map Included
	Yes ☐ No ☒

FIELD COLLECTION RECORD

Project No. GWP 30-95-00	Station No. 10+760	Collection No. 1

No.	Scientific Name	Total Length Range (cm)	Weight Range (gm)	Physical Condition	
				# fish with blackspot	(lesions, tumors, maturity etc.)

Identified by:	Date	Day / Month / Year
		26/09/2000

Comments:
No fish sampling was completed since there was recent existing information from Department of Fisheries and Oceans documenting coldwater habitat including rainbow trout and mottled sculpins at this location.

STATION DIAGRAM

FIELD COLLECTION RECORD

Project #	Project Description	Day/Month/Year
GWP 30-95-00	Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	26/09/2000

Name of Waterbody	Drainage System	Station #	Collection #
Tributary of Innisfil Creek	2ED	11+688	1

Location of Crossing	Structure Type
Concession 2, Lots 6 & 7	☐ Bridge ☐ Box ☒ CSP ☐ Other

Latitude	Longitude
44°12'53"	79°39'39"

Township	MNR District / Area
Innisfil	Midhurst

Collectors
Bilz, Hulse

Weather Conditions
Sunny, Clear

Station	Length (m)	Time Started	Time Finished	Sampling Duration
Upstream	20	1625	1630	5
Downstream	130	1630	1640	10

Water Type		Associated Wetland
☒ Stream/River ☐ Lake ☐ Pond ☐ Canal	☒ Permanent ☐ Intermittent	

Channel Dimensions	Mean Width (m)	Mean Depth (m)	Bankfull Width (m)	Bankfull Depth (m)	W/D Ratio
Upstream	0.65	0.35	1.85		1.9
Downstream	0.84	0.28	2.24		3.0

	Stream Gradient (m/100m)	Current Velocity (m/s)	Stream realignment required
Upstream			☐ Yes ☒ No
Downstream			☐ Yes ☒ No

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Conductivity
Upstream			13	15	
Downstream			13	15	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
Upstream	10	25		65	
Downstream	10	25		65	

Water Colour	Colourless	Yellow/Brown	Blue/Green	Turbid	Other
Upstream	☒	☐	☐	☐	☐
Downstream	☒	☐	☐	☐	☐

Substrate (%Upstream Downstream)	Bedrock	Boulder	Rubble	Gravel	Sand	Silt	Clay	Muck	Marl	Detritus
Upstream			20	20	15	25		10		10
Downstream				20	35	25		10		10

Instream Cover (% surface area)	Undercut Banks	Boulders	Logs/Trees	Organic Debris	VascularPlants	None
Upstream					5	95
Downstream	20				5	75

FIELD COLLECTION RECORD – Page 2

Shore Cover (% of stream shaded)	90-100%	60-90%	30-60%	1-30%	None
Upstream	☐	☐	☐	☐	☒
Downstream	☐	☐	☐	☐	☒

Vegetation Type (% of surface area)	Submergent	Floating	Emergent	None
Upstream			5	95
Downstream			5	95

Banks	Stable (%)	Unstable (%)	Surrounding Land Use
Upstream	100		Agricultural
Downstream	100		

Migratory Obstructions	None	Seasonal	Permanent	Sources of Pollution
Upstream	☒			
Downstream	☒			

Potential Critical Habitat	Spawning	Upwellings	Other
Upstream	☐	☐	
Downstream	☐	☐	

Potential Enhancement Opportunities
Plant shrubs within ROW
Add hard granular substrate

Comments

Sinuosity	D50	Bankfull Return Period	Erodible Soils
		☐ >1.2-1.5yrs ☐ <1.2-1.5yrs	☐ High ☐ Moderate ☒ Low

Stream Classifications
MNR
☐ Coldwater
☐ Potential Coldwater
☐ Warmwater Sportsfish
☒ Warmwater Habitat
☐ Warmwater Urban
☐ Migratory

DFO
☐ 1
☐ 2
☐ 3

Rosgen

Additional Notes

FIELD COLLECTION RECORD

Project Name	Project #	Date: Day/Month/Year
Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	GWP 30-95-00	26/09/2000

Name of Waterbody	Drainage System	Station No.	Collection No.
tributary of Innisfil Creek	2ED	11+688	1

County or Judicial District	Township
Simcoe	Innisfil

UTM Coordinates	Easting	Northing
	606960	4896370

Data Collectors
Bilz, Hulse

Weather Conditions
Sunny Clear

Water Type
☐ Spring ☐ Canal ☒ Stream/River ☐ River/Lake Junction ☐ Flooded Area ☐ Pool
☐ Pond ☐ Lake ☐ Muskeg/Bog ☐ Reservoir ☐ Other

Dimensions	Length(m)	Mean Wetted (m)	Mean Channel Width (m)	Mean Depth(m)	Current Velocity
	150	0.75	2.0	0.5	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
	10	25		65	

Plant Type (%)	Submergent	Floating	Emergent	None
	Eg.	Eg.	5 Eg. nodding bur marigold Canada rush	

Bottom Type (%)	Bedrock	Boulder	Rubble	Gravel	Sand	Other
	Clay	Muck 10	Marl	Detritus 10	Silt 25	

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Water Color
			13	15	☒ Colorless ☐ Yellow/Brown ☐ Turbid ☐ Blue/Green ☐ Other

Cover (instream) % of total surface area	Undercut Banks	Boulders	Logs/Trees	Organic Debris	Vascular Macrophytes	Combination	No Cover
	10				5		85

Cover/Canopy (Shore) % stream shaded	Bank Characteristics (stability)	Surrounding Land Use
Full Canopy (100%) ☐ Fair Canopy (40-60%) ☐ Good Canopy (70-90%) ☐ Poor Canopy (10-30%) ☐ No Canopy ☒	stable	Golf Course Agricultural

Gear	Electrofisher	Seine	Gill Net	Angled	Dip Net	Surber
	☒ Length 150 Electrosec 102 Hoop Net ☐	☐ # Hauls Trap Net ☐	☐ Minnow Trap ☐	☐ Trawl ☐	☐ Piscicide ☐	☐ Other ☐

Size of Net (Gill or Seine)	Size of net or mouth (Trap,Hoop,Trawl)	Mesh Size	Depth of Capture
Length m	m	Smallest cm Largest cm	Min 0m Max 0.7m

Fish Kept?	No. of Bags	Preservative
Yes ☐ No ☒		Formalin ☐ Kahle's Solution ☐ Alcohol ☐ Frozen ☐ Other ☐

Additional Data: Pollution,Migratory Obstructions, Parasites,Color & Condition of Fish etc.	Habitat Map Included
	Yes ☐ No ☒

FIELD COLLECTION RECORD

Project No. GWP 30-95-00	Station No. 11+688	Collection No. 1

No.	Scientific Name	Total Length Range (cm)	Weight Range (gm)	Physical Condition	
				# fish with blackspot	(lesions, tumors, maturity etc.)
1	Brook Stickleback Culaea inconstans	5.3			
1	Creek Chub Semotilus atromaculatus	6.4			

Identified by:	Date	Day / Month / Year
R. Bilz		26/09/2000

Comments:

STATION DIAGRAM

FIELD COLLECTION RECORD

Project #	Project Description	Day/Month/Year
GWP 30-95-00	Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	26/09/2000

Name of Waterbody	Drainage System	Station #	Collection #
Tributary of Innisfil Creek	2ED	12+879	1

Location of Crossing	Structure Type
Concession 3, Lots 6 &7	☐ Bridge ☒ Box ☐ CSP ☐ Other

Latitude	Longitude
44°13'35"	79°39'50"

Township	MNR District / Area
Innisfil	Midhurst

Collectors: Bilz, Hulse

Weather Conditions: Sunny, Clear

Station	Length (m)	Time Started	Time Finished	Sampling Duration
Upstream	20	1645	1655	10
Downstream	100	1655	1725	30

Water Type		Permanent	Associated Wetland
☒ Stream/River	☐ Lake	☒ Permanent	
☐ Pond	☐ Canal	☐ Intermittent	

Channel Dimensions	Mean Width (m)	Mean Depth (m)	Bankfull Width (m)	Bankfull Depth (m)	W/D Ratio
Upstream	1.5	0.55	2.8		2.7
Downstream	1.6	0.45	3.0		3.6

	Stream Gradient (m/100m)	Current Velocity (m/s)	Stream realignment required
Upstream			☒ Yes ☐ No
Downstream			☒ Yes ☐ No

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Conductivity
Upstream			9	10	
Downstream			9	10	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
Upstream	40	20	30	10	
Downstream	40	20	30	10	

Water Colour	Colourless	Yellow/Brown	Blue/Green	Turbid	Other
Upstream	☒	☐	☐	☐	☐
Downstream	☒	☐	☐	☐	☐

Substrate (%	Bedrock	Boulder	Rubble	Gravel	Sand	Silt	Clay	Muck	Marl	Detritus
Upstream		10	35	25	10		5		5	10
Downstream)		15	45	25	15		5			5

Instream Cover (% surface area)	Undercut Banks	Boulders	Logs/Trees	Organic Debris	VascularPlants	None
Upstream	5	5	2	2		91
Downstream	5	5	2	2		91

FIELD COLLECTION RECORD – Page 2

Shore Cover (% of stream shaded)	90-100%	60-90%	30-60%	1-30%	None
Upstream	☐	☒	☐	☐	☐
Downstream	☐	☒	☐	☐	☐

Vegetation Type (% of surface area)	Submergent	Floating	Emergent	None
Upstream				100
Downstream				100

Banks	Stable (%)	Unstable (%)	Surrounding Land Use
Upstream	100		Agricultural
Downstream	100		Forested

Migratory Obstructions	None	Seasonal	Permanent	Sources of Pollution
Upstream	☒			
Downstream	☒			

Potential Critical Habitat	Spawning	Upwellings	Other
Upstream	☒	☒	
Downstream	☐	☐	Nursery

Potential Enhancement Opportunities
Plant shrubs within ROW
Add woody debris in fringe riffle areas to improve nursery habitat

Comments
Upstream realignment will be very difficult because of deeply incised banks and presence of groundwater discharges

Sinuosity	D50	Bankfull Return Period	Erodible Soils
		☐ >1.2-1.5yrs ☐ <1.2-1.5yrs	☐ High ☐ Moderate ☒ Low

Stream Classifications MNR		DFO	Rosgen
☒ Coldwater	☐ Warmwater Habitat	☐ 1	
☐ Potential Coldwater	☐ Warmwater Urban	☐ 2	
☐ Warmwater Sportsfish	☐ Migratory	☐ 3	

Additional Notes
Excellent habitat with high quality spawning and nursery habitat for coldwater species

FIELD COLLECTION RECORD

Project Name	Project #	Date: Day/Month/Year
Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	GWP 30-95-00	26/09/2000

Name of Waterbody	Drainage System	Station No.	Collection No.
tributary of Innisfil Creek	2ED	12+879	1

County or Judicial District	Township
Simcoe	Innisfil

UTM Coordinates	Easting	Northing
	606690	4897660

Data Collectors
Bilz, Hulse

Weather Conditions
Sunny Clear

Water Type
☐ Spring ☐ Canal ☒ Stream/River ☐ River/Lake Junction ☐ Flooded Area ☐ Pool
☐ Pond ☐ Lake ☐ Muskeg/Bog ☐ Reservoir ☐ Other

Dimensions	Length(m)	Mean Wetted (m)	Mean Channel Width (m)	Mean Depth(m)	Current Velocity
	120	1.6	2.9	0.5	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
	40	20	30	10	

Plant Type (%)	Submergent	Floating	Emergent	None
	Eg.	Eg.	Eg.	

Bottom Type (%)	Bedrock	Boulder	Rubble	Gravel	Sand	Other
	Clay 5	12 Muck	40 Marl 3	25 Detritus 5	10 Silt	

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Water Color
			9	10	☒ Colorless ☐ Blue/Green ☐ Yellow/Brown ☐ Other ☐ Turbid

Cover (instream) % of total surface area	Undercut Banks	Boulders	Logs/Trees	Organic Debris	Vascular Macrophytes	Combination	No Cover
	5	5	2	2			91

Cover/Canopy (Shore) % stream shaded			Bank Characteristics (stability)	Surrounding Land Use
	Full Canopy (100%) ☐ Good Canopy (70-90%) ☒ No Canopy ☐	Fair Canopy (40-60%) ☐ Poor Canopy (10-30%) ☐	stable	Forested Agricultural

Gear						
Electrofisher ☒ Length 120 Electrosec 417 Hoop Net ☐	Seine ☐ # Hauls Trap Net ☐	Gill Net ☐ Minnow Trap ☐	Angled ☐ Trawl ☐	Dip Net ☐ Piscicide ☐	Surber ☐ Other ☐	

Size of Net (Gill or Seine)	Size of net or mouth (Trap,Hoop,Trawl)	Mesh Size	Depth of Capture
Length m	m	Smallest cm Largest cm	Min 0m Max 0.7m

Fish Kept?	No. of Bags	Preservative
Yes ☐ No ☒		Formalin ☐ Kahle's Solution ☐ Alcohol ☐ Frozen ☐ Other ☐

Additional Data: Pollution,Migratory Obstructions, Parasites,Color & Condition of Fish etc.	Habitat Map Included
	Yes ☐ No ☒

FIELD COLLECTION RECORD

Project No. GWP 30-95-00	Station No. 12+879	Collection No. 1

No.	Scientific Name	Total Length Range (cm)	Weight Range (gm)	Physical Condition	
				# fish with blackspot	(lesions, tumors, maturity etc.)
4	Brook Trout Salvelinus fontinalis	17.0-22.1			2 ripe males
3	Rainbow Trout Onchorynchus mykiss	9.2-14.0			
9	Creek Chub Semotilus atromaculatus	7.1-16.2			
6	Blacknose Dace Rhinichthys atratulus	7.7-8.9			

Identified by:	Date	Day / Month / Year
R. Bilz		26/09/2000

Comments:

STATION DIAGRAM

FIELD COLLECTION RECORD

Project #	Project Description	Day/Month/Year
GWP 30-95-00	Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	26/09/2000

Name of Waterbody	Drainage System	Station #	Collection #
Tributary of Innisfil Creek	2ED	13+100	1

Location of Crossing	Structure Type
Concession 3, Lots 6 &7	☐ Bridge ☒ Box ☐ CSP ☐ Other

Latitude	Longitude
44°13'43"	79°39'52"

Township	MNR District / Area
Innisfil	Midhurst

Collectors
Bilz, Hulse

Weather Conditions
Sunny, Clear

Station	Length (m)	Time Started	Time Finished	Sampling Duration
Upstream	20	1425	1435	10
Downstream	200	1435	1550	75

Water Type		Permanent	Associated Wetland
☒ Stream/River	☐ Lake	☒ Permanent	
☐ Pond	☐ Canal	☐ Intermittent	

Channel Dimensions	Mean Width (m)	Mean Depth (m)	Bankfull Width (m)	Bankfull Depth (m)	W/D Ratio
Upstream	1.8	0.25	2.6		7.2
Downstream	2.0	0.20	3.0		10.0

	Stream Gradient (m/100m)	Current Velocity (m/s)	Stream realignment required
Upstream			☒ Yes ☐ No
Downstream			☒ Yes ☐ No

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Conductivity
Upstream			9	15	
Downstream			9	15	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
Upstream	55		10	35	
Downstream	55		25	20	

Water Colour	Colourless	Yellow/Brown	Blue/Green	Turbid	Other
Upstream	☒	☐	☐	☐	☐
Downstream	☒	☐	☐	☐	☐

Substrate (%Upstream Downstream)	Bedrock	Boulder	Rubble	Gravel	Sand	Silt	Clay	Muck	Marl	Detritus
Upstream		5	20	25	10	5	15		5	15
Downstream		5	25	30	5	0	10		5	20

Instream Cover (% surface area)	Undercut Banks	Boulders	Logs/Trees	Organic Debris	VascularPlants	None
Upstream		1	5	5		89
Downstream	3	2	12	10		83

FIELD COLLECTION RECORD – Page 2

Shore Cover (% of stream shaded)	90-100%	60-90%	30-60%	1-30%	None
Upstream	☐	☐	☒	☐	☐
Downstream	☐	☒	☐	☐	☐

Vegetation Type (% of surface area)	Submergent	Floating	Emergent	None
Upstream				100
Downstream				100

Banks	Stable (%)	Unstable (%)	Surrounding Land Use
Upstream	100		Agricultural
Downstream	100		Forested

Migratory Obstructions	None	Seasonal	Permanent	Sources of Pollution
Upstream	☒			
Downstream	☒			

Potential Critical Habitat	Spawning	Upwellings	Other
Upstream	☐	☐	Nursery
Downstream	☒	☒	Nursery

Potential Enhancement Opportunities
Plant shrubs within ROW
Add woody debris in fringe riffle areas to improve nursery habitat

Comments
Downstream realignment will be very difficult because of deeply incised banks and presence of groundwater discharges

Sinuosity	D50	Bankfull Return Period	Erodible Soils
		☐ >1.2-1.5yrs ☐ <1.2-1.5yrs	☐ High ☐ Moderate ☒ Low

Stream Classifications

MNR		DFO	Rosgen
☒ Coldwater	☐ Warmwater Habitat	☐ 1	
☐ Potential Coldwater	☐ Warmwater Urban	☐ 2	
☐ Warmwater Sportsfish	☐ Migratory	☐ 3	

Additional Notes
Excellent habitat with high quality spawning and nursery habitat for coldwater species

FIELD COLLECTION RECORD

Project Name	Project #	Date: Day/Month/Year
Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	GWP 30-95-00	26/09/2000

Name of Waterbody	Drainage System	Station No.	Collection No.
tributary of Innisfil Creek	2ED	13+100	1

County or Judicial District	Township
Simcoe	Innisfil

UTM Coordinates	Easting	Northing
	606670	4897890

Data Collectors
Bilz, Hulse

Weather Conditions
Sunny Clear

Water Type
☐ Spring ☐ Canal ☒ Stream/River ☐ River/Lake Junction ☐ Flooded Area ☐ Pool
☐ Pond ☐ Lake ☐ Muskeg/Bog ☐ Reservoir ☐ Other

Dimensions	Length(m)	Mean Wetted (m)	Mean Channel Width (m)	Mean Depth(m)	Current Velocity
	240	1.9	2.8	0.22	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
	55		20	25	

Plant Type (%)	Submergent	Floating	Emergent	None
	Eg.	Eg.	Eg.	

Bottom Type (%)	Bedrock	Boulder	Rubble	Gravel	Sand	Other
	Clay 12	Muck	Marl 5	Detritus 18	Silt 3	

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Water Color
			9	15	☒ Colorless ☐ Blue/Green ☐ Yellow/Brown ☐ Other ☐ Turbid

Cover (instream) % of total surface area	Undercut Banks	Boulders	Logs/Trees	Organic Debris	Vascular Macrophytes	Combination	No Cover
	3	2	10	8			87

Cover/Canopy (Shore) % stream shaded			Bank Characteristics (stability)	Surrounding Land Use
	Full Canopy (100%) ☐	Fair Canopy (40-60%) ☐	stable	Forested
	Good Canopy (70-90%) ☒	Poor Canopy (10-30%) ☐		Agricultural
	No Canopy ☐			

Gear						
	Electrofisher ☒	Seine ☐	Gill Net ☐	Angled ☐	Dip Net ☐	Surber ☐
	Length 220	# Hauls	Minnow Trap ☐	Trawl ☐	Piscicide ☐	Other ☐
	Electrosec 617	Trap Net ☐				
	Hoop Net ☐					

Size of Net (Gill or Seine)	Size of net or mouth (Trap,Hoop,Trawl)	Mesh Size	Depth of Capture
Length m	m	Smallest cm Largest cm	Min 0m Max 0.3m

Fish Kept?	No. of Bags	Preservative
Yes ☐ No ☒		Formalin ☐ Kahle's Solution ☐ Alcohol ☐ Frozen ☐ Other ☐

Additional Data: Pollution,Migratory Obstructions, Parasites,Color & Condition of Fish etc.	Habitat Map Included
	Yes ☐ No ☒

FIELD COLLECTION RECORD

Project No. GWP 30-95-00	Station No. 13+100	Collection No. 1

No.	Scientific Name	Total Length Range (cm)	Weight Range (gm)	Physical Condition	
				# fish with blackspot	(lesions, tumors, maturity etc.)
17	Brook Trout Salvelinus fontinalis	8.6-19.5			4 ripe males 1 ripe female
1	Rainbow Trout Onchorynchus mykiss	10.6			
4	Creek Chub Semotilus atromaculatus	3.6-13.5			
8	Blacknose Dace Rhinichthys atratulus	6.6-10.9			
1	Brook Stickleback Culaea inconstans	5.9			

Identified by:	Date	Day / Month / Year
R. Bilz		26/09/2000

Comments:

STATION DIAGRAM

FIELD COLLECTION RECORD

Project #	Project Description	Day/Month/Year
GWP 30-95-00	Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	26/09/2000

Name of Waterbody	Drainage System	Station #	Collection #
Tributary of Innisfil Creek	2ED	16+722	1

Location of Crossing	Structure Type
Concession 5, Lots 6 & 7	☐ Bridge ☐ Box ☐ CSP ☒ Other

Latitude	Longitude
44°15'33"	79°40'22"

Township	MNR District / Area
Innisfil	Midhurst

Collectors: Bilz, Hulse

Weather Conditions: Sunny, Clear

Station	Length (m)	Time Started	Time Finished	Sampling Duration
Upstream	40	1330	1340	10
Downstream	100	1340	1420	40

Water Type			Associated Wetland
☒ Stream/River	☐ Lake	☒ Permanent	
☐ Pond	☐ Canal	☐ Intermittent	

Channel Dimensions	Mean Width (m)	Mean Depth (m)	Bankfull Width (m)	Bankfull Depth (m)	W/D Ratio
Upstream	0.9	0.23	1.3		3.9
Downstream	1.0	0.20	1.7		5.0

	Stream Gradient (m/100m)	Current Velocity (m/s)	Stream realignment required
Upstream			☒ Yes ☐ No
Downstream			☐ Yes ☒ No

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Conductivity
Upstream			12.5	15	
Downstream			12.5	15	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
Upstream	30	30	20	20	
Downstream	30	30	20	20	

Water Colour	Colourless	Yellow/Brown	Blue/Green	Turbid	Other
Upstream	☒	☐	☐	☐	☐
Downstream	☒	☐	☐	☐	☐

Substrate (%Upstream Downstream)	Bedrock	Boulder	Rubble	Gravel	Sand	Silt	Clay	Muck	Marl	Detritus
Upstream		5	25	30	10	5	5			20
Downstream		10	25	30	15	5	5			10

Instream Cover (% surface area)	Undercut Banks	Boulders	Logs/Trees	Organic Debris	VascularPlants	None
Upstream		5	5	5	1	84
Downstream	1	5	15	5	1	73

FIELD COLLECTION RECORD – Page 2

Shore Cover (% of stream shaded)	90-100%	60-90%	30-60%	1-30%	None
Upstream	☐	☒	☐	☐	☐
Downstream	☐	☒	☐	☐	☐

Vegetation Type (% of surface area)	Submergent	Floating	Emergent	None
Upstream	1		1	98
Downstream	1		1	98

Banks	Stable (%)	Unstable (%)	Surrounding Land Use
Upstream	95	5	Agricultural
Downstream	95	5	Forested

Migratory Obstructions	None	Seasonal	Permanent	Sources of Pollution
Upstream	☒			
Downstream	☐		perched culvert	

Potential Critical Habitat	Spawning	Upwellings	Other
Upstream	☐	☐	
Downstream	☐	☐	

Potential Enhancement Opportunities
Repair downstream perched culvert
Create low flow channel inside culvert

Comments

Sinuosity	D50	Bankfull Return Period	Erodible Soils
		☐ >1.2-1.5yrs ☐ <1.2-1.5yrs	☐ High ☐ Moderate ☒ Low

Stream Classifications

MNR		DFO	Rosgen
☐ Coldwater	☒ Warmwater Habitat	☐ 1	
☐ Potential Coldwater	☐ Warmwater Urban	☐ 2	
☐ Warmwater Sportsfish	☐ Migratory	☐ 3	

Additional Notes

FIELD COLLECTION RECORD

Project Name	Project #	Date: Day/Month/Year
Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	GWP 30-95-00	26/09/2000

Name of Waterbody	Drainage System	Station No.	Collection No.
tributary of Innisfil Creek	2ED	16+722	1

County or Judicial District	Township
Simcoe	Innisfil

UTM Coordinates	Easting	Northing
	605940	4901300

Data Collectors
Bilz, Hulse

Weather Conditions
Sunny Clear

Water Type
☐ Spring ☐ Canal ☒ Stream/River ☐ River/Lake Junction ☐ Flooded Area ☐ Pool
☐ Pond ☐ Lake ☐ Muskeg/Bog ☐ Reservoir ☐ Other

Dimensions	Length(m)	Mean Wetted (m)	Mean Channel Width (m)	Mean Depth(m)	Current Velocity
	140	1.0	1.5	0.20	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
	30	30	20	20	

Plant Type (%)	Submergent 1	Floating	Emergent 1	None
	Eg. Algae	Eg.	Eg. 3-way Sedge Water Cress nodding bur marigold	

Bottom Type (%)	Bedrock	Boulder 10	Rubble 25	Gravel 30	Sand 15	Other
	Clay 5	Muck	Marl	Detritus 10	Silt 5	

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Water Color
			12.5	15	☒ Colorless ☐ Yellow/Brown ☐ Turbid ☐ Blue/Green ☐ Other

Cover (instream) % of total surface area	Undercut Banks	Boulders	Logs/Trees	Organic Debris	Vascular Macrophytes	Combination	No Cover
	1	5	15	5	1		73

Cover/Canopy (Shore) % stream shaded			Bank Characteristics (stability)	Surrounding Land Use
	Full Canopy (100%) ☐ Good Canopy (70-90%) ☒ No Canopy ☐	Fair Canopy (40-60%) ☐ Poor Canopy (10-30%) ☐	stable	Forested Agricultural

Gear						
	Electrofisher ☒ Length 140 Electrosec 247 Hoop Net ☐	Seine ☐ # Hauls Trap Net ☐	Gill Net ☐ Minnow Trap ☐	Angled ☐ Trawl ☐	Dip Net ☐ Piscicide ☐	Surber ☐ Other ☐

Size of Net (Gill or Seine)	Size of net or mouth (Trap,Hoop,Trawl)	Mesh Size	Depth of Capture
Length m	m	Smallest cm Largest cm	Min 0m Max 0.3m

Fish Kept?	No. of Bags	Preservative
Yes ☐ No ☒		Formalin ☐ Kahle's Solution ☐ Alcohol ☐ Frozen ☐ Other ☐

Additional Data: Pollution,Migratory Obstructions, Parasites,Color & Condition of Fish etc.

Habitat Map Included Yes ☐ No ☒

FIELD COLLECTION RECORD

Project No. GWP 30-95-00	Station No. 16+722	Collection No. 1

No.	Scientific Name	Total Length Range (cm)	Weight Range (gm)	Physical Condition	
				# fish with blackspot	(lesions, tumors, maturity etc.)
1	Brassy Minnow Hybognathus hankinsonii	7.8			
1	Fathead Minnow Pimephales promelas	4.5			
46	Creek Chub Semotilus atromaculatus	2.6-10.5			
3	Blacknose Dace Rhinichthys atratulus	3.6-3.8			
4	Brook Stickleback Culaea inconstans	3.6-4.1			

Identified by:	Date Day / Month / Year
R. Bilz	26/09/2000

Comments:

STATION DIAGRAM

FIELD COLLECTION RECORD

Project #	Project Description	Day/Month/Year
GWP 30-95-00	Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	10/10/2000

Name of Waterbody	Drainage System	Station #	Collection #
Tributary of Lovers Creek	2EC	18+400	1

Location of Crossing	Structure Type
Concession 7, Lots 6 & 7	☐ Bridge ☒ Box ☐ CSP ☐ Other

Latitude	Longitude
44°16'28"	79°40'36"

Township	MNR District / Area
Innisfil	Midhurst

Collectors
Bilz, Hulse

Weather Conditions
Overcast, cool

Station	Length (m)	Time Started	Time Finished	Sampling Duration
Upstream	20	1300	1315	15
Downstream	100	1315	1340	25

Water Type		Permanent	Associated Wetland
☒ Stream/River	☐ Lake	☐ Permanent	
☐ Pond	☐ Canal	☒ Intermittent	

Channel Dimensions	Mean Width (m)	Mean Depth (m)	Bankfull Width (m)	Bankfull Depth (m)	W/D Ratio
Upstream	.8	0.10	1.3		8
Downstream	.57	0.19	1.1		3

	Stream Gradient (m/100m)	Current Velocity (m/s)	Stream realignment required
Upstream			☐ Yes ☒ No
Downstream			☐ Yes ☒ No

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Conductivity
Upstream			13	9	
Downstream			13	9	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
Upstream	0	0	20	80	
Downstream	0	0	20	80	

Water Colour	Colourless	Yellow/Brown	Blue/Green	Turbid	Other
Upstream	☒	☐	☐	☐	☐
Downstream	☒	☐	☐	☐	☐

Substrate (%Upstream Downstream)	Bedrock	Boulder	Rubble	Gravel	Sand	Silt	Clay	Muck	Marl	Detritus
Upstream				30	40	5	5			20
Downstream				20	45	10	10			15

Instream Cover (% surface area)	Undercut Banks	Boulders	Logs/Trees	Organic Debris	VascularPlants	None
Upstream				10	25	65
Downstream				10	25	65

FIELD COLLECTION RECORD – Page 2

Shore Cover (% of stream shaded)	90-100%	60-90%	30-60%	1-30%	None
Upstream	☐	☐	☐	☐	☒
Downstream	☐	☐	☐	☐	☒

Vegetation Type (% of surface area)	Submergent	Floating	Emergent	None
Upstream			25	75
Downstream			25	75

Banks	Stable (%)	Unstable (%)	Surrounding Land Use
Upstream	100		Agricultural
Downstream	100		Commercial

Migratory Obstructions	None	Seasonal	Permanent	Sources of Pollution
Upstream	☒			
Downstream	☐	terrestrial vegetation		

Potential Critical Habitat	Spawning	Upwellings	Other
Upstream	☐	☐	
Downstream	☐	☐	

Potential Enhancement Opportunities

Comments

Sinuosity	D50	Bankfull Return Period	Erodible Soils
		☐ >1.2-1.5yrs ☐ <1.2-1.5yrs	☐ High ☐ Moderate ☒ Low

Stream Classifications

MNR

☐ Coldwater
☐ Potential Coldwater
☐ Warmwater Sportsfish
☒ Warmwater Habitat
☐ Warmwater Urban
☐ Migratory

DFO
☐ 1
☐ 2
☐ 3

Rosgen

Additional Notes

FIELD COLLECTION RECORD

Project Name	Project #	Date: Day/Month/Year
Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	GWP 30-95-00	10/10/2000

Name of Waterbody	Drainage System	Station No.	Collection No.
tributary of Lovers Creek	2EC	18+400	1

County or Judicial District	Township
Simcoe	Innisfil

UTM Coordinates	Easting	Northing
	605590	4902990

Data Collectors
Bilz, Hulse

Weather Conditions
Overcast, cool

Water Type
☐ Spring ☐ Canal ☒ Stream/River ☐ River/Lake Junction ☐ Flooded Area ☐ Pool
☐ Pond ☐ Lake ☐ Muskeg/Bog ☐ Reservoir ☐ Other

Dimensions	Length(m)	Mean Wetted (m)	Mean Channel Width (m)	Mean Depth(m)	Current Velocity
	120	0.6	1.2	0.15	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
			20	80	

Plant Type (%)	Submergent	Floating	Emergent 25	None
	Eg.	Eg.	Eg. cattails reed canary grass sedges	

Bottom Type (%)	Bedrock	Boulder	Rubble	Gravel 25	Sand 45	Other
	Clay 5	Muck	Marl	Detritus 20	Silt 10	

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Water Color
	8		13	9	☒ Colorless ☐ Yellow/Brown ☐ Turbid ☐ Blue/Green ☐ Other

Cover (instream) % of total surface area	Undercut Banks	Boulders	Logs/Trees	Organic Debris	Vascular Macrophytes	Combination	No Cover
				10	25		65

Cover/Canopy (Shore) % stream shaded	Full Canopy (100%)	Fair Canopy (40-60%)	Bank Characteristics (stability)	Surrounding Land Use
	Good Canopy (70-90%) ☐ No Canopy ☒	Poor Canopy (10-30%) ☐	stable	Commercial Agricultural

Gear	Electrofisher	Seine	Gill Net	Angled	Dip Net	Surber
	☒	☐	☐	☐	☐	☐
	Length 120	# Hauls	Minnow Trap ☐	Trawl ☐	Piscicide ☐	Other ☐
	Electrosec 211	Trap Net ☐				
	Hoop Net ☐					

Size of Net (Gill or Seine)	Size of net or mouth (Trap,Hoop,Trawl)	Mesh Size	Depth of Capture
Length m	m	Smallest cm Largest cm	Min 0m Max 0.3m

Fish Kept?	No. of Bags	Preservative
Yes ☐ No ☒		Formalin ☐ Kahle's Solution ☐ Alcohol ☐ Frozen ☐ Other ☐

Additional Data: Pollution,Migratory Obstructions, Parasites,Color & Condition of Fish etc.	Habitat Map Included
	Yes ☐ No ☒

FIELD COLLECTION RECORD

Project No. GWP 30-95-00	Station No. 18+400	Collection No. 1

No.	Scientific Name	Total Length Range (cm)	Weight Range (gm)	Physical Condition	
				# fish with blackspot	(lesions, tumors, maturity etc.)

Identified by:	Date	Day / Month / Year
R. Bilz		10/10/2000

Comments: No catch within study area. Brook sticklebacks observed in the downstream channel adjacent to a wetland approximately 225m downstream of the highway.

STATION DIAGRAM

FIELD COLLECTION RECORD

Project #	Project Description	Day/Month/Year
GWP 30-95-00	Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	26/09/2000

Name of Waterbody	Drainage System	Station #	Collection #
Tributary of Lovers Creek	2EC	23+273	1

Location of Crossing	Structure Type
Concession 10, Lot 7	☐ Bridge ☒ Box ☐ CSP ☐ Other

Latitude	Longitude
44°19'04"	79°41'00"

Township	MNR District / Area
Innisfil	Midhurst

Collectors: Bilz, Hulse

Weather Conditions: Sunny, Clear

Station	Length (m)	Time Started	Time Finished	Sampling Duration
Upstream	20	1245	1255	15
Downstream	120	1255	1315	20

Water Type			Associated Wetland
☒ Stream/River	☐ Lake	☒ Permanent	
☐ Pond	☐ Canal	☐ Intermittent	

Channel Dimensions	Mean Width (m)	Mean Depth (m)	Bankfull Width (m)	Bankfull Depth (m)	W/D Ratio
Upstream	4.2	0.75	7.0		5.6
Downstream	1.8	0.75	5.2		2.4

	Stream Gradient (m/100m)	Current Velocity (m/s)	Stream realignment required
Upstream			☐ Yes ☒ No
Downstream			☐ Yes ☒ No

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Conductivity
Upstream			13	15	
Downstream			13	15	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
Upstream	0	0	100	0	
Downstream	0	0	20	80	

Water Colour	Colourless	Yellow/Brown	Blue/Green	Turbid	Other
Upstream	☒	☐	☐	☐	☐
Downstream	☒	☐	☐	☐	☐

Substrate (%Upstream Downstream)	Bedrock	Boulder	Rubble	Gravel	Sand	Silt	Clay	Muck	Marl	Detritus
Upstream				5				50		45
Downstream				5	10	10		50		25

Instream Cover (% surface area)	Undercut Banks	Boulders	Logs/Trees	Organic Debris	VascularPlants	None
Upstream					35	65
Downstream					35	65

FIELD COLLECTION RECORD – Page 2

Shore Cover (% of stream shaded)	90-100%	60-90%	30-60%	1-30%	None
Upstream	☐	☒	☐	☐	☐
Downstream	☐	☒	☐	☐	☐

Vegetation Type (% of surface area)	Submergent	Floating	Emergent	None
Upstream	15	10	10	65
Downstream	15	10	10	65

Banks	Stable (%)	Unstable (%)	Surrounding Land Use
Upstream	100		Agricultural
Downstream	100		Forested

Migratory Obstructions	None	Seasonal	Permanent	Sources of Pollution
Upstream	☒			
Downstream	☒			

Potential Critical Habitat	Spawning	Upwellings	Other
Upstream	☐	☐	
Downstream	☐	☐	

Potential Enhancement Opportunities
Plant native shrubs within ROW

Comments

Sinuosity	D50	Bankfull Return Period	Erodible Soils
		☐ >1.2-1.5yrs ☐ <1.2-1.5yrs	☐ High ☐ Moderate ☒ Low

Stream Classifications
MNR
☐ Coldwater
☐ Potential Coldwater
☐ Warmwater Sportsfish
☒ Warmwater Habitat
☐ Warmwater Urban
☐ Migratory

DFO
☐ 1
☐ 2
☐ 3

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Additional Notes

FIELD COLLECTION RECORD

Project Name	Project #	Date: Day/Month/Year
Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	GWP 30-95-00	26/09/2000

Name of Waterbody	Drainage System	Station No.	Collection No.
tributary of Lovers Creek	2EC	23+273	1

County or Judicial District	Township
Simcoe	Innisfil

UTM Coordinates	Easting	Northing
	604980	4907770

Data Collectors
Bilz, Hulse

Weather Conditions
Sunny, Clear

Water Type
☐ Spring ☐ Canal ☒ Stream/River ☐ River/Lake Junction ☐ Flooded Area ☐ Pool
☐ Pond ☐ Lake ☐ Muskeg/Bog ☐ Reservoir ☐ Other

Dimensions	Length(m)	Mean Wetted (m)	Mean Channel Width (m)	Mean Depth(m)	Current Velocity
	140	2.0	6.0	0.75	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
			20	80	

Plant Type (%)	Submergent 15	Floating 15	Emergent 15	None
	Eg. algae	Eg. duckweed	Eg. cattails Canada rush nodding bur marigold	

Bottom Type (%)	Bedrock	Boulder	Rubble	Gravel 5	Sand 10	Other
	Clay	Muck 40	Marl	Detritus 35	Silt 10	

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Water Color
			13	15	☒ Colorless ☐ Blue/Green ☐ Yellow/Brown ☐ Other ☐ Turbid

Cover (instream) % of total surface area	Undercut Banks	Boulders	Logs/Trees	Organic Debris	Vascular Macrophytes	Combination	No Cover
					35		65

Cover/Canopy (Shore) % stream shaded	Full Canopy (100%)	Fair Canopy (40-60%)	Bank Characteristics (stability)	Surrounding Land Use
	Good Canopy (70-90%) ☒ No Canopy ☐	Poor Canopy (10-30%) ☐	stable	Forested Agricultural

Gear	Electrofisher ☒	Seine ☐	Gill Net ☐	Angled ☐	Dip Net ☐	Surber ☐
	Length 140 Electrosec 147 Hoop Net ☐	# Hauls Trap Net ☐	Minnow Trap ☐	Trawl ☐	Piscicide ☐	Other ☐

Size of Net (Gill or Seine)	Size of net or mouth (Trap,Hoop,Trawl)	Mesh Size	Depth of Capture
Length m	m	Smallest cm Largest cm	Min 0m Max 0.75m

Fish Kept?	No. of Bags	Preservative
Yes ☐ No ☒		Formalin ☐ Kahle's Solution ☐ Alcohol ☐ Frozen ☐ Other ☐

Additional Data: Pollution,Migratory Obstructions, Parasites,Color & Condition of Fish etc.

Habitat Map Included
Yes ☐ No ☒

FIELD COLLECTION RECORD

Project No. GWP 30-95-00	Station No. 23+273	Collection No. 1

No.	Scientific Name	Total Length Range (cm)	Weight Range (gm)	Physical Condition # fish with blackspot	(lesions, tumors, maturity etc.)
2	Creek Chub Semotilus atromaculatus	7.2-10.3			

Identified by:	Date	Day / Month / Year
R. Bilz		26/09/2000

Comments:

STATION DIAGRAM

FIELD COLLECTION RECORD

Project #	Project Description	Day/Month/Year
GWP 30-95-00	Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	26/09/2000

Name of Waterbody	Drainage System	Station #	Collection #
Tributary of Kempenfelt Bay	2EC	26+750	1

Location of Crossing	Structure Type
Concession 13, Lots 7 & 8	☐ Bridge ☒ Box ☐ CSP ☐ Other

Latitude	Longitude
44°20'55"	79°41'17"

Township	MNR District / Area
Innisfil	Midhurst

Collectors: Bilz, Hulse

Weather Conditions: Sunny, Clear

Station	Length (m)	Time Started	Time Finished	Sampling Duration
Upstream	20	1120	1125	5
Downstream	100	1125	1135	10

Water Type			Associated Wetland
☒ Stream/River	☐ Lake	☒ Permanent	
☐ Pond	☐ Canal	☐ Intermittent	

Channel Dimensions	Mean Width (m)	Mean Depth (m)	Bankfull Width (m)	Bankfull Depth (m)	W/D Ratio
Upstream	0.45	0.18	0.7		2.5
Downstream	0.5	0.20	0.8		2.5

	Stream Gradient (m/100m)	Current Velocity (m/s)	Stream realignment required
Upstream			☐ Yes ☒ No
Downstream			☐ Yes ☒ No

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Conductivity
Upstream			11	15	
Downstream			11	15	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
Upstream	5	25		70	
Downstream	10	20		70	

Water Colour	Colourless	Yellow/Brown	Blue/Green	Turbid	Other
Upstream	☒	☐	☐	☐	☐
Downstream	☒	☐	☐	☐	☐

Substrate (%Upstream Downstream)	Bedrock	Boulder	Rubble	Gravel	Sand	Silt	Clay	Muck	Marl	Detritus
Upstream			20	20	50	10				
Downstream			20	20	50	10				

Instream Cover (% surface area)	Undercut Banks	Boulders	Logs/Trees	Organic Debris	VascularPlants	None
Upstream	15		2		10	73
Downstream	20		2			78

FIELD COLLECTION RECORD – Page 2

Shore Cover (% of stream shaded)	90-100%	60-90%	30-60%	1-30%	None
Upstream	☐	☐	☒	☐	☐
Downstream	☐	☐	☒	☐	☐

Vegetation Type (% of surface area)	Submergent	Floating	Emergent	None
Upstream			10	90
Downstream				100

Banks	Stable (%)	Unstable (%)	Surrounding Land Use
Upstream	100		Agricultural
Downstream	100		Forested

Migratory Obstructions	None	Seasonal	Permanent	Sources of Pollution
Upstream	☒			
Downstream	☒			

Potential Critical Habitat	Spawning	Upwellings	Other
Upstream	☐	☐	
Downstream	☐	☐	

Potential Enhancement Opportunities
Construct refuge pools with spawning facilities for fathead minnows

Comments

Sinuosity	D50	Bankfull Return Period	Erodible Soils
		☐ >1.2-1.5yrs ☐ <1.2-1.5yrs	☐ High ☐ Moderate ☒ Low

Stream Classifications
MNR
☐ Coldwater
☐ Potential Coldwater
☐ Warmwater Sportsfish
☒ Warmwater Habitat
☐ Warmwater Urban
☐ Migratory

DFO
☐ 1
☐ 2
☐ 3

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Additional Notes

FIELD COLLECTION RECORD

Project Name	Project #	Date: Day/Month/Year
Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	GWP 30-95-00	26/09/2000

Name of Waterbody	Drainage System	Station No.	Collection No.
tributary of Kempenfelt Bay	2EC	26+750	1

County or Judicial District	Township
Simcoe	Innisfil

UTM Coordinates	Easting	Northing
	604550	4911230

Data Collectors
Bilz, Hulse

Weather Conditions
Sunny, Clear

Water Type
☐ Spring ☐ Canal ☒ Stream/River ☐ River/Lake Junction ☐ Flooded Area ☐ Pool
☐ Pond ☐ Lake ☐ Muskeg/Bog ☐ Reservoir ☐ Other

Dimensions	Length(m)	Mean Wetted (m)	Mean Channel Width (m)	Mean Depth(m)	Current Velocity
	120	0.5	0.75	0.2	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
	10	20		70	

Plant Type (%)	Submergent	Floating	Emergent	None
	Eg.	Eg.	10 Eg. cattails sedges	

Bottom Type (%)	Bedrock	Boulder	Rubble	Gravel	Sand	Other
	Clay	Muck	20 Marl	20 Detritus	50 Silt 10	

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Water Color
			11	15	☒ Colorless ☐ Yellow/Brown ☐ Turbid ☐ Blue/Green ☐ Other

Cover (instream) % of total surface area	Undercut Banks	Boulders	Logs/Trees	Organic Debris	Vascular Macrophytes	Combination	No Cover
	20		2		10		68

Cover/Canopy (Shore) % stream shaded	Full Canopy (100%)	Fair Canopy (40-60%)	Bank Characteristics (stability)	Surrounding Land Use
	Good Canopy (70-90%) ☐ No Canopy ☐	Poor Canopy (10-30%) ☐	stable	Forested Agricultural

Gear	Electrofisher	Seine	Gill Net	Angled	Dip Net	Surber
	☒ Length 120 Electrosec 77 Hoop Net ☐	☐ # Hauls Trap Net ☐	☐ Minnow Trap ☐	☐ Trawl ☐	☐ Piscicide ☐	☐ Other ☐

Size of Net (Gill or Seine)	Size of net or mouth (Trap,Hoop,Trawl)	Mesh Size	Depth of Capture
Length m	m	Smallest cm Largest cm	Min 0m Max 0.25m

Fish Kept?	No. of Bags	Preservative
Yes ☐ No ☒		Formalin ☐ Kahle's Solution ☐ Alcohol ☐ Frozen ☐ Other ☐

Additional Data: Pollution,Migratory Obstructions, Parasites,Color & Condition of Fish etc.	Habitat Map Included
	Yes ☐ No ☒

FIELD COLLECTION RECORD

Project No. GWP 30-95-00	Station No. 26+750	Collection No. 1

No.	Scientific Name	Total Length Range (cm)	Weight Range (gm)	Physical Condition	
				# fish with blackspot	(lesions, tumors, maturity etc.)
2	Fathead Minnow Pimephales promelas	3.0-3.5			

Identified by:	Date	Day / Month / Year
R. Bilz		26/09/2000

Comments:

STATION DIAGRAM

FIELD COLLECTION RECORD

Project #	Project Description	Day/Month/Year
GWP 30-95-00	Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	10/10/2000

Name of Waterbody	Drainage System	Station #	Collection #
Tributary of Kempenfelt Bay	2EC	29+282	1

Location of Crossing	Structure Type
Concession 14, Lot 6	☐ Bridge ☐ Box ☒ CSP ☐ Other

Latitude	Longitude
44°22'04"	79°42'13"

Township	MNR District / Area
Innisfil	Midhurst

Collectors: Bilz, Hulse

Weather Conditions: Overcast, cool

Station	Length (m)	Time Started	Time Finished	Sampling Duration
Upstream	20	1405	1415	10
Downstream	100	1415	1435	20

Water Type			Associated Wetland
☒ Stream/River	☐ Lake	☐ Permanent	
☐ Pond	☐ Canal	☒ Intermittent	

Channel Dimensions	Mean Width (m)	Mean Depth (m)	Bankfull Width (m)	Bankfull Depth (m)	W/D Ratio
Upstream	.8	0.18	1.3		4.4
Downstream	0.6	0.24	1.2		2.5

	Stream Gradient (m/100m)	Current Velocity (m/s)	Stream realignment required
Upstream			☐ Yes ☒ No
Downstream			☐ Yes ☒ No

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Conductivity
Upstream			13	9	
Downstream			13	9	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
Upstream	0	0	0	100	
Downstream	10	0	0	90	

Water Colour	Colourless	Yellow/Brown	Blue/Green	Turbid	Other
Upstream	☒	☐	☐	☐	☐
Downstream	☒	☐	☐	☐	☐

Substrate (%Upstream Downstream)	Bedrock	Boulder	Rubble	Gravel	Sand	Silt	Clay	Muck	Marl	Detritus
Upstream				30	40	30				
Downstream				20	50	30				

Instream Cover (% surface area)	Undercut Banks	Boulders	Logs/Trees	Organic Debris	VascularPlants	None
Upstream					10	90
Downstream					10	90

FIELD COLLECTION RECORD – Page 2

Shore Cover (% of stream shaded)	90-100%	60-90%	30-60%	1-30%	None
Upstream	☐	☐	☐	☐	☒
Downstream	☐	☐	☐	☐	☒

Vegetation Type (% of surface area)	Submergent	Floating	Emergent	None
Upstream				100
Downstream				100

Banks	Stable (%)	Unstable (%)	Surrounding Land Use
Upstream	100		Urban
Downstream	100		Commercial

Migratory Obstructions	None	Seasonal	Permanent	Sources of Pollution
Upstream	☒			
Downstream	☒			

Potential Critical Habitat	Spawning	Upwellings	Other
Upstream	☐	☐	
Downstream	☐	☐	

Potential Enhancement Opportunities

Comments

Sinuosity	D50	Bankfull Return Period	Erodible Soils
		☐ >1.2-1.5yrs ☐ <1.2-1.5yrs	☐ High ☐ Moderate ☒ Low

Stream Classifications

MNR

- ☐ Coldwater
- ☐ Potential Coldwater
- ☐ Warmwater Sportsfish
- ☒ Warmwater Habitat
- ☐ Warmwater Urban
- ☐ Migratory

DFO

- ☐ 1
- ☐ 2
- ☐ 3

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Additional Notes

FIELD COLLECTION RECORD

Project Name	Project #	Date: Day/Month/Year
Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	GWP 30-95-00	10/10/2000

Name of Waterbody	Drainage System	Station No.	Collection No.
tributary of Kempenfelt Bay	2EC	29+282	1

County or Judicial District	Township
Simcoe	Innisfil

UTM Coordinates	Easting	Northing
	603270	4913310

Data Collectors
Bilz, Hulse

Weather Conditions
Overcast, cool

Water Type

☐ Spring ☐ Canal ☒ Stream/River ☐ River/Lake Junction ☐ Flooded Area ☐ Pool
☐ Pond ☐ Lake ☐ Muskeg/Bog ☐ Reservoir ☐ Other

Dimensions	Length(m)	Mean Wetted (m)	Mean Channel Width (m)	Mean Depth(m)	Current Velocity
	120	0.7	1.2	0.15	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
	10			90	

Plant Type (%)	Submergent	Floating	Emergent	None
	Eg.	Eg.	Eg.	

Bottom Type (%)	Bedrock	Boulder	Rubble	Gravel	Sand	Other
	Clay 5	Muck	Marl	Detritus 25	Silt 30	45

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Water Color
			13	9	☒ Colorless ☐ Blue/Green ☐ Yellow/Brown ☐ Other ☐ Turbid

Cover (instream) % of total surface area	Undercut Banks	Boulders	Logs/Trees	Organic Debris	Vascular Macrophytes	Combination	No Cover
					10		90

Cover/Canopy (Shore) % stream shaded	Full Canopy (100%)	Fair Canopy (40-60%)	Bank Characteristics (stability)	Surrounding Land Use
	Good Canopy (70-90%) ☐ No Canopy ☒	Poor Canopy (10-30%) ☐	stable	Commercial Urban

Gear	Electrofisher	Seine	Gill Net	Angled	Dip Net	Surber
	☒ Length 120 Electrosec 85 Hoop Net ☐	☐ # Hauls Trap Net ☐	☐ Minnow Trap ☐	☐ Trawl ☐	☐ Piscicide ☐	☐ Other ☐

Size of Net (Gill or Seine)	Size of net or mouth (Trap,Hoop,Trawl)	Mesh Size	Depth of Capture
Length m	m	Smallest cm Largest cm	Min 0m Max 0.2m

Fish Kept?	No. of Bags	Preservative
Yes ☐ No ☒		Formalin ☐ Kahle's Solution ☐ Alcohol ☐ Frozen ☐ Other ☐

Additional Data: Pollution,Migratory Obstructions, Parasites,Color & Condition of Fish etc.

Habitat Map Included
Yes ☐ No ☒

FIELD COLLECTION RECORD

Project No.	Station No.	Collection No.
GWP 30-95-00	29+282	1

No.	Scientific Name	Total Length Range (cm)	Weight Range (gm)	Physical Condition	
				# fish with blackspot	(lesions, tumors, maturity etc.)
1	Central Mudminnow Umbra limi	5.2			

Identified by:	Date	Day / Month / Year
R. Bilz		10/10/2000

Comments:

STATION DIAGRAM

FIELD COLLECTION RECORD

Project #	Project Description	Day/Month/Year
GWP 30-95-00	Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	27/09/2000

Name of Waterbody	Drainage System	Station #	Collection #
Tributary of Kempenfelt Bay	2EC	11+025	1

Location of Crossing	Structure Type
Concession 6, Lot 24	☐ Bridge ☒ Box ☐ CSP ☐ Other

Latitude	Longitude
44°22'46"	79°42'41"

Township	MNR District / Area
Vespra	Midhurst

Collectors: Bilz, Hulse

Weather Conditions: Partly Sunny

Station	Length (m)	Time Started	Time Finished	Sampling Duration
Upstream	0			
Downstream	100	0745	0810	25

Water Type		Associated Wetland
☒ Stream/River ☐ Lake ☐ Pond ☐ Canal	☒ Permanent ☐ Intermittent	

Channel Dimensions	Mean Width (m)	Mean Depth (m)	Bankfull Width (m)	Bankfull Depth (m)	W/D Ratio
Upstream					
Downstream	0.75	0.20	1.0		3.8

	Stream Gradient (m/100m)	Current Velocity (m/s)	Stream realignment required
Upstream			☐ Yes ☐ No
Downstream			☐ Yes ☒ No

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Conductivity
Upstream					
Downstream			10.5	10	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
Upstream					
Downstream	10	50	20	20	

Water Colour	Colourless	Yellow/Brown	Blue/Green	Turbid	Other
Upstream	☒	☐	☐	☐	☐
Downstream	☒	☐	☐	☐	☐

Substrate (%Upstream Downstream)	Bedrock	Boulder	Rubble	Gravel	Sand	Silt	Clay	Muck	Marl	Detritus
		2	13	30	20	15	20			

Instream Cover (% surface area)	Undercut Banks	Boulders	Logs/Trees	Organic Debris	VascularPlants	None
Upstream						
Downstream	5	1				94

FIELD COLLECTION RECORD – Page 2

Shore Cover (% of stream shaded)	90-100%	60-90%	30-60%	1-30%	None
Upstream	☐	☐	☐	☐	☐
Downstream	☐	☐	☐	☒	☐

Vegetation Type (% of surface area)	Submergent	Floating	Emergent	None
Upstream				
Downstream	2		15	83

Banks	Stable (%)	Unstable (%)	Surrounding Land Use
Upstream	100		Urban
Downstream	100		Highway interchange

Migratory Obstructions	None	Seasonal	Permanent	Sources of Pollution
Upstream	☐		Stream goes underground	
Downstream	☒			

Potential Critical Habitat	Spawning	Upwellings	Other
Upstream	☐	☐	
Downstream	☐	☐	

Potential Enhancement Opportunities
Plant native shrubs within ROW

Comments

Sinuosity	D50	Bankfull Return Period	Erodible Soils
		☐ >1.2-1.5yrs ☐ <1.2-1.5yrs	☐ High ☐ Moderate ☒ Low

Stream Classifications
MNR
☒ Coldwater
☐ Potential Coldwater
☐ Warmwater Sportsfish
☐ Warmwater Habitat
☐ Warmwater Urban
☐ Migratory

DFO	Rosgen
☐ 1 ☐ 2 ☐ 3	

Additional Notes

FIELD COLLECTION RECORD

Project Name	Project #	Date: Day/Month/Year
Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	GWP 30-95-00	26/09/2000

Name of Waterbody	Drainage System	Station No.	Collection No.
tributary of Kempenfelt Bay	2EC	11+025	1

County or Judicial District	Township
Simcoe	Vespra

UTM Coordinates	Easting	Northing
	6602660	4915630

Data Collectors
Bilz, Hulse

Weather Conditions
Sunny, Clear

Water Type
☐ Spring ☐ Canal ☒ Stream/River ☐ River/Lake Junction ☐ Flooded Area ☐ Pool
☐ Pond ☐ Lake ☐ Muskeg/Bog ☐ Reservoir ☐ Other

Dimensions	Length(m)	Mean Wetted (m)	Mean Channel Width (m)	Mean Depth(m)	Current Velocity
	100	0.75	1.0	0.2	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
	10	50	20	20	

Plant Type (%)	Submergent 2	Floating	Emergent 15	None
	Eg. algae	Eg.	Eg. water cress Canada rush Horsetail	

Bottom Type (%)	Bedrock	Boulder	Rubble	Gravel	Sand	Other
	Clay 20	2 Muck	13 Marl	30 Detritus	20 Silt 15	

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Water Color
			10.5	10	☒ Colorless ☐ Blue/Green ☐ Yellow/Brown ☐ Other ☐ Turbid

Cover (instream) % of total surface area	Undercut Banks	Boulders	Logs/Trees	Organic Debris	Vascular Macrophytes	Combination	No Cover
	5	1					94

Cover/Canopy (Shore) % stream shaded			Bank Characteristics (stability)	Surrounding Land Use
	Full Canopy (100%) ☐ Good Canopy (70-90%) ☐ No Canopy ☐	Fair Canopy (40-60%) ☐ Poor Canopy (10-30%) ☒	stable	Urban Highway Interchange

Gear	Electrofisher	Seine	Gill Net	Angled	Dip Net	Surber
	☒ Length 100 Electrosec 226 Hoop Net ☐	☐ # Hauls Trap Net ☐	☐ Minnow Trap ☐	☐ Trawl ☐	☐ Piscicide ☐	☐ Other ☐

Size of Net (Gill or Seine)	Size of net or mouth (Trap,Hoop,Trawl)	Mesh Size	Depth of Capture
Length m	m	Smallest cm Largest cm	Min 0m Max 0.2m

Fish Kept?	No. of Bags	Preservative
Yes ☐ No ☒		Formalin ☐ Kahle's Solution ☐ Alcohol ☐ Frozen ☐ Other ☐

Additional Data: Pollution,Migratory Obstructions, Parasites,Color & Condition of Fish etc.	Habitat Map Included
	Yes ☐ No ☒

FIELD COLLECTION RECORD

Project No. GWP 30-95-00	Station No. 11+025	Collection No. 1

No.	Scientific Name	Total Length Range (cm)	Weight Range (gm)	Physical Condition	
				# fish with blackspot	(lesions, tumors, maturity etc.)
7	Creek Chub Semotilus atromaculatus	6.0-13.7			
1	White Sucker Catostomus commersoni	6.7			
4	Brook Stickleback Culaea inconstans	4.4-5.2			
3	Central Mudminnow Umbra limi	3.3-3.9			

Identified by:	Date Day / Month / Year
R. Bilz	26/09/2000

Comments:

STATION DIAGRAM

FIELD COLLECTION RECORD

Project #	Project Description	Day/Month/Year
GWP 30-95-00	Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	26/09/2000

Name of Waterbody	Drainage System	Station #	Collection #
Tributary of Kempenfelt Bay	2EC	12+200	1

Location of Crossing	Structure Type
Concession 5, Lot 23	☐ Bridge ☒ Box ☐ CSP ☐ Other

Latitude	Longitude
44°23'19"	79°42'23"

Township	MNR District / Area
Vespra	Midhurst

Collectors
Bilz, Hulse

Weather Conditions
Sunny, Clear

Station	Length (m)	Time Started	Time Finished	Sampling Duration
Upstream	20	1015	1020	5
Downstream	100	1020	1030	10

Water Type			Associated Wetland
☒ Stream/River	☐ Lake	☒ Permanent	
☐ Pond	☐ Canal	☐ Intermittent	

Channel Dimensions	Mean Width (m)	Mean Depth (m)	Bankfull Width (m)	Bankfull Depth (m)	W/D Ratio
Upstream	0.5	0.12	0.8		4.2
Downstream	0.5	0.15	0.65		3.3

	Stream Gradient (m/100m)	Current Velocity (m/s)	Stream realignment required
Upstream			☐ Yes ☒ No
Downstream			☐ Yes ☒ No

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Conductivity
Upstream			10	10	
Downstream			10	10	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
Upstream	100				
Downstream	75		25		

Water Colour	Colourless	Yellow/Brown	Blue/Green	Turbid	Other
Upstream	☒	☐	☐	☐	☐
Downstream	☒	☐	☐	☐	☐

Substrate (%Upstream Downstream)	Bedrock	Boulder	Rubble	Gravel	Sand	Silt	Clay	Muck	Marl	Detritus
Upstream			60	20	10		10			
Downstream			60	20	10		10			

Instream Cover (% surface area)	Undercut Banks	Boulders	Logs/Trees	Organic Debris	VascularPlants	None
Upstream					5	95
Downstream	5				5	94

FIELD COLLECTION RECORD – Page 2

Shore Cover (% of stream shaded)	90-100%	60-90%	30-60%	1-30%	None
Upstream	☐	☐	☐	☐	☒
Downstream	☐	☐	☐	☐	☒

Vegetation Type (% of surface area)	Submergent	Floating	Emergent	None
Upstream			5	95
Downstream			5	95

Banks	Stable (%)	Unstable (%)	Surrounding Land Use
Upstream	100		Urban
Downstream	100		

Migratory Obstructions	None	Seasonal	Permanent	Sources of Pollution
Upstream	☒			
Downstream	☒			

Potential Critical Habitat	Spawning	Upwellings	Other
Upstream	☐	☐	
Downstream	☐	☐	

Potential Enhancement Opportunities
Plant native shrubs within ROW

Comments

Sinuosity	D50	Bankfull Return Period	Erodible Soils
		☐ >1.2-1.5yrs ☐ <1.2-1.5yrs	☐ High ☐ Moderate ☒ Low

Stream Classifications
MNR
☒ Coldwater
☐ Potential Coldwater
☐ Warmwater Sportsfish
☐ Warmwater Habitat
☐ Warmwater Urban
☐ Migratory

DFO
☐ 1
☐ 2
☐ 3

Rosgen

Additional Notes

FIELD COLLECTION RECORD

Project Name	Project #	Date: Day/Month/Year
Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	GWP 30-95-00	26/09/2000

Name of Waterbody	Drainage System	Station No.	Collection No.
tributary of Kempenfelt Bay	2EC	12+200	1

County or Judicial District	Township
Simcoe	Vespra

UTM Coordinates	Easting	Northing
	603020	4915630

Data Collectors
Bilz, Hulse

Weather Conditions
Sunny, Clear

Water Type
☐ Spring ☐ Canal ☒ Stream/River ☐ River/Lake Junction ☐ Flooded Area ☐ Pool
☐ Pond ☐ Lake ☐ Muskeg/Bog ☐ Reservoir ☐ Other

Dimensions	Length(m)	Mean Wetted (m)	Mean Channel Width (m)	Mean Depth(m)	Current Velocity
	120	0.5	0.75	0.15	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
	75		25		

Plant Type (%)	Submergent	Floating	Emergent	None
	Eg.	Eg.	5 Eg. water cress	

Bottom Type (%)	Bedrock	Boulder	Rubble	Gravel	Sand	Other
	Clay 10	Muck	60 Marl	20 Detritus	10 Silt	

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Water Color
			10	10	☒ Colorless ☐ Yellow/Brown ☐ Turbid ☐ Blue/Green ☐ Other

Cover (instream) % of total surface area	Undercut Banks	Boulders	Logs/Trees	Organic Debris	Vascular Macrophytes	Combination	No Cover
	5				5		94

Cover/Canopy (Shore) % stream shaded	Full Canopy (100%)	Fair Canopy (40-60%)	Bank Characteristics (stability)	Surrounding Land Use
	Good Canopy (70-90%) ☐ No Canopy ☒	Poor Canopy (10-30%) ☐	stable	Urban

Gear	Electrofisher	Seine	Gill Net	Angled	Dip Net	Surber
	☒ Length 120 Electrosec 180 Hoop Net ☐	☐ # Hauls Trap Net ☐	☐ Minnow Trap ☐	☐ Trawl ☐	☐ Piscicide ☐	☐ Other ☐

Size of Net (Gill or Seine)	Size of net or mouth (Trap,Hoop,Trawl)	Mesh Size	Depth of Capture
Length m	m	Smallest cm Largest cm	Min 0m Max 0.25m

Fish Kept?	No. of Bags	Preservative
Yes ☐ No ☒		Formalin ☐ Kahle's Solution ☐ Alcohol ☐ Frozen ☐ Other ☐

Additional Data: Pollution,Migratory Obstructions, Parasites,Color & Condition of Fish etc.	Habitat Map Included
	Yes ☐ No ☒

FIELD COLLECTION RECORD

Project No. GWP 30-95-00	Station No. 12+200	Collection No. 1

No.	Scientific Name	Total Length Range (cm)	Weight Range (gm)	Physical Condition	
				# fish with blackspot	(lesions, tumors, maturity etc.)

Identified by:	Date	Day / Month / Year
R. Bilz		26/09/2000

Comments: No fish were captured but young of the year minnows were observed within the study area.

STATION DIAGRAM

FIELD COLLECTION RECORD

Project #	Project Description	Day/Month/Year
GWP 30-95-00	Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	27/09/2000

Name of Waterbody	Drainage System	Station #	Collection #
Tributary of Kempenfelt Bay	2EC	12+450	1

Location of Crossing	Structure Type
Concession 5, Lot 23	☐ Bridge ☒ Box ☐ CSP ☐ Other

Latitude	Longitude
44°23'29"	79°42'10"

Township	MNR District / Area
Vespra	Midhurst

Collectors
Bilz, Hulse

Weather Conditions
Sunny, Clear

Station	Length (m)	Time Started	Time Finished	Sampling Duration
Upstream	20	0820	0825	5
Downstream	100	0825	0835	10

Water Type			Associated Wetland
☒ Stream/River ☐ Pond	☐ Lake ☐ Canal	☒ Permanent ☐ Intermittent	

Channel Dimensions	Mean Width (m)	Mean Depth (m)	Bankfull Width (m)	Bankfull Depth (m)	W/D Ratio
Upstream	0.4	0.12	0.5		3.3
Downstream	0.4	0.09	0.5		4.4

	Stream Gradient (m/100m)	Current Velocity (m/s)	Stream realignment required
Upstream			☐ Yes ☒ No
Downstream			☐ Yes ☒ No

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Conductivity
Upstream			10.5	10	
Downstream			10.5	10	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
Upstream	100				
Downstream	15			85	

Water Colour	Colourless	Yellow/Brown	Blue/Green	Turbid	Other
Upstream	☒	☐	☐	☐	☐
Downstream	☒	☐	☐	☐	☐

Substrate (%Upstream Downstream)	Bedrock	Boulder	Rubble	Gravel	Sand	Silt	Clay	Muck	Marl	Detritus
Upstream			5	40	40	5		10		
Downstream			5	40	40	5		10		

Instream Cover (% surface area)	Undercut Banks	Boulders	Logs/Trees	Organic Debris	VascularPlants	None
Upstream					40	60
Downstream					40	60

FIELD COLLECTION RECORD – Page 2

Shore Cover (% of stream shaded)	90-100%	60-90%	30-60%	1-30%	None
Upstream	☐	☐	☐	☒	☐
Downstream	☐	☐	☐	☒	☐

Vegetation Type (% of surface area)	Submergent	Floating	Emergent	None
Upstream	5		35	60
Downstream	5		35	60

Banks	Stable (%)	Unstable (%)	Surrounding Land Use
Upstream	100		Urban
Downstream	100		Forested

Migratory Obstructions	None	Seasonal	Permanent	Sources of Pollution
Upstream	☒			
Downstream	☒			

Potential Critical Habitat	Spawning	Upwellings	Other
Upstream	☐	☐	
Downstream	☐	☐	

Potential Enhancement Opportunities
Plant native shrubs within ROW

Comments

Sinuosity	D50	Bankfull Return Period	Erodible Soils
		☐ >1.2-1.5yrs ☐ <1.2-1.5yrs	☐ High ☐ Moderate ☒ Low

Stream Classifications
MNR
☒ Coldwater
☐ Potential Coldwater
☐ Warmwater Sportsfish
☐ Warmwater Habitat
☐ Warmwater Urban
☐ Migratory

DFO
☐ 1
☐ 2
☐ 3

Rosgen

Additional Notes

FIELD COLLECTION RECORD

Project Name	Project #	Date: Day/Month/Year
Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	GWP 30-95-00	27/09/2000

Name of Waterbody	Drainage System	Station No.	Collection No.
tributary of Kempenfelt Bay	2EC	12+450	1

County or Judicial District	Township
Simcoe	Vespra

UTM Coordinates	Easting	Northing
	603150	4915930

Data Collectors
Bilz, Hulse

Weather Conditions
Sunny, Clear

Water Type
☐ Spring ☐ Canal ☒ Stream/River ☐ River/Lake Junction ☐ Flooded Area ☐ Pool
☐ Pond ☐ Lake ☐ Muskeg/Bog ☐ Reservoir ☐ Other

Dimensions	Length(m)	Mean Wetted (m)	Mean Channel Width (m)	Mean Depth(m)	Current Velocity
	120	0.4	0.5	0.1	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
	15			85	

Plant Type (%)	Submergent 5	Floating	Emergent 35	None
	Eg. algae	Eg.	Eg. water cress Canada rush	60

Bottom Type (%)	Bedrock	Boulder	Rubble 5	Gravel 40	Sand 40	Other
	Clay	Muck 10	Marl	Detritus	Silt 5	

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Water Color
			10.5	10	☒ Colorless ☐ Yellow/Brown ☐ Turbid ☐ Blue/Green ☐ Other

Cover (instream) % of total surface area	Undercut Banks	Boulders	Logs/Trees	Organic Debris	Vascular Macrophytes	Combination	No Cover
					40		60

Cover/Canopy (Shore) % stream shaded	Full Canopy (100%)	Fair Canopy (40-60%)	Bank Characteristics (stability)	Surrounding Land Use
	Good Canopy (70-90%) ☐	Poor Canopy (10-30%) ☒	stable	Urban
	No Canopy ☐			Forested

Gear	Electrofisher	Seine	Gill Net	Angled	Dip Net	Surber
	☒	☐	☐	☐	☐	☐
	Length 120	# Hauls	Minnow Trap ☐	Trawl ☐	Piscicide ☐	Other ☐
	Electrosec 204	Trap Net ☐				
	Hoop Net ☐					

Size of Net (Gill or Seine)	Size of net or mouth (Trap,Hoop,Trawl)	Mesh Size	Depth of Capture
Length m	m	Smallest cm Largest cm	Min 0m Max 0.15m

Fish Kept?	No. of Bags	Preservative
Yes ☐ No ☒		Formalin ☐ Kahle's Solution ☐ Alcohol ☐ Frozen ☐ Other ☐

Additional Data: Pollution,Migratory Obstructions, Parasites,Color & Condition of Fish etc.	Habitat Map Included
	Yes ☐ No ☒

FIELD COLLECTION RECORD

Project No. GWP 30-95-00	Station No. 12+450	Collection No. 1

No.	Scientific Name	Total Length Range (cm)	Weight Range (gm)	Physical Condition	
				# fish with blackspot	(lesions, tumors, maturity etc.)

Identified by:	Date Day / Month / Year
R. Bilz	26/09/2000

Comments: No fish were captured but brook sticklebacks were observed

STATION DIAGRAM

FIELD COLLECTION RECORD

Project #	Project Description	Day/Month/Year
GWP 30-95-00	Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	10/10/2000

Name of Waterbody	Drainage System	Station #	Collection #
Tributary of Kempenfelt Bay	2EC	12+800	1

Location of Crossing	Structure Type
Concession 5, Lot 22	☐ Bridge ☒ Box ☐ CSP ☐ Other

Latitude	Longitude
44°23'38"	79°42'10"

Township	MNR District / Area
Vespra	Midhurst

Collectors
Bilz, Hulse

Weather Conditions
Overcast, Cool

Station	Length (m)	Time Started	Time Finished	Sampling Duration
Upstream	10	1505	1515	10
Downstream	100	1515	1535	20

Water Type			Associated Wetland
☒ Stream/River ☐ Pond	☐ Lake ☐ Canal	☒ Permanent ☐ Intermittent	

Channel Dimensions	Mean Width (m)	Mean Depth (m)	Bankfull Width (m)	Bankfull Depth (m)	W/D Ratio
Upstream	0.8	0.10	1.5		8.0
Downstream	0.8	0.10	1.5		8.0

	Stream Gradient (m/100m)	Current Velocity (m/s)	Stream realignment required
Upstream			☐ Yes ☒ No
Downstream			☐ Yes ☒ No

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Conductivity
Upstream			13	11	
Downstream			13	11	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
Upstream				100	
Downstream	40		20	40	

Water Colour	Colourless	Yellow/Brown	Blue/Green	Turbid	Other
Upstream	☒	☐	☐	☐	☐
Downstream	☒	☐	☐	☐	☐

Substrate (%Upstream Downstream)	Bedrock	Boulder	Rubble	Gravel	Sand	Silt	Clay	Muck	Marl	Detritus
Upstream			5	40	40	5	5			5
Downstream			5	40	40	5	5			5

Instream Cover (% surface area)	Undercut Banks	Boulders	Logs/Trees	Organic Debris	VascularPlants	None
Upstream						100
Downstream	5			10	10	75

FIELD COLLECTION RECORD – Page 2

Shore Cover (% of stream shaded)	90-100%	60-90%	30-60%	1-30%	None
Upstream	☐	☐	☐	☐	☒
Downstream	☐	☒	☐	☐	☐

Vegetation Type (% of surface area)	Submergent	Floating	Emergent	None
Upstream				100
Downstream			10	90

Banks	Stable (%)	Unstable (%)	Surrounding Land Use
Upstream	100		Urban
Downstream	100		Forested

Migratory Obstructions	None	Seasonal	Permanent	Sources of Pollution
Upstream	☐		Stormwater Basin	
Downstream	☒			

Potential Critical Habitat	Spawning	Upwellings	Other
Upstream	☐	☐	
Downstream	☐	☐	

Potential Enhancement Opportunities
Plant native shrubs within ROW

Comments

Sinuosity	D50	Bankfull Return Period	Erodible Soils
		☐ >1.2-1.5yrs ☐ <1.2-1.5yrs	☐ High ☐ Moderate ☒ Low

Stream Classifications
MNR
☐ Coldwater
☐ Potential Coldwater
☐ Warmwater Sportsfish
☒ Warmwater Habitat
☐ Warmwater Urban
☐ Migratory

DFO
☐ 1
☐ 2
☐ 3

Rosgen

Additional Notes

FIELD COLLECTION RECORD

Project Name	Project #	Date: Day/Month/Year
Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	GWP 30-95-00	10/10/2000

Name of Waterbody	Drainage System	Station No.	Collection No.
tributary of Kempenfelt Bay	2EC	12+800	1

County or Judicial District	Township
Simcoe	Vespra

UTM Coordinates	Easting	Northing
	603310	4916220

Data Collectors
Bilz, Hulse

Weather Conditions
Overcast, Cool

Water Type
☐ Spring ☐ Canal ☒ Stream/River ☐ River/Lake Junction ☐ Flooded Area ☐ Pool
☐ Pond ☐ Lake ☐ Muskeg/Bog ☐ Reservoir ☐ Other

Dimensions	Length(m)	Mean Wetted (m)	Mean Channel Width (m)	Mean Depth(m)	Current Velocity
	110	0.8	1.5	0.1	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
	40		20	40	

Plant Type (%)	Submergent	Floating	Emergent	None
	Eg.	Eg.	10 Eg. sedges Canada rush	90

Bottom Type (%)	Bedrock	Boulder	Rubble	Gravel	Sand	Other
	Clay 5	Muck	Marl	40 Detritus 5	40 Silt 5	

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Water Color
			13	11	☒ Colorless ☐ Yellow/Brown ☐ Turbid ☐ Blue/Green ☐ Other

Cover (instream) % of total surface area	Undercut Banks	Boulders	Logs/Trees	Organic Debris	Vascular Macrophytes	Combination	No Cover
	5			10	10		75

Cover/Canopy (Shore) % stream shaded	Full Canopy (100%)	Fair Canopy (40-60%)	Bank Characteristics (stability)	Surrounding Land Use
	Good Canopy (70-90%) ☒ No Canopy ☐	Poor Canopy (10-30%) ☐	stable	Urban Forested

Gear	Electrofisher	Seine	Gill Net	Angled	Dip Net	Surber
	☒ Length 110 Electrosec 231 Hoop Net ☐	☐ # Hauls Trap Net ☐	☐ Minnow Trap ☐	☐ Trawl ☐	☐ Piscicide ☐	☐ Other ☐

Size of Net (Gill or Seine)	Size of net or mouth (Trap,Hoop,Trawl)	Mesh Size	Depth of Capture
Length m	m	Smallest cm Largest cm	Min 0m Max 0.2m

Fish Kept?	No. of Bags	Preservative
Yes ☐ No ☒		Formalin ☐ Kahle's Solution ☐ Alcohol ☐ Frozen ☐ Other ☐

Additional Data: Pollution,Migratory Obstructions, Parasites,Color & Condition of Fish etc.	Habitat Map Included
	Yes ☐ No ☒

FIELD COLLECTION RECORD

Project No. GWP 30-95-00			Station No. 12+800	Collection No. 1	
No.	Scientific Name	Total Length Range (cm)	Weight Range (gm)	Physical Condition	
				# fish with blackspot	(lesions, tumors, maturity etc.)
4	Fathead Minnow Pimephales promelas	4.2-5.8			
3	Brook Stickleback Culaea inconstans	2.9-4.1			

Identified by:	Date Day / Month / Year
R. Bilz	10/10/2000

Comments:

STATION DIAGRAM

FIELD COLLECTION RECORD

Project #	Project Description	Day/Month/Year
GWP 30-95-00	Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	10/10/2000

Name of Waterbody	Drainage System	Station #	Collection #
Willow Creek	2EC	18+280	1

Location of Crossing	Structure Type
Concession 1, Lot 8	☒ Bridge ☐ Box ☐ CSP ☐ Other

Latitude	Longitude
44°25'30"	79°39'10"

Township	MNR District / Area
Vespra	Midhurst

Collectors: Bilz, Hulse

Weather Conditions: Overcast, Cool

Station	Length (m)	Time Started	Time Finished	Sampling Duration
Upstream	20	1620	1635	15
Downstream	100	1635	1700	25

Water Type		Associated Wetland
☒ Stream/River ☐ Lake ☐ Pond ☐ Canal	☒ Permanent ☐ Intermittent	Little Lake PSW

Channel Dimensions	Mean Width (m)	Mean Depth (m)	Bankfull Width (m)	Bankfull Depth (m)	W/D Ratio
Upstream	8.9	0.75	14.5		11.9
Downstream	8.1	1.0	15.6		8.1

	Stream Gradient (m/100m)	Current Velocity (m/s)	Stream realignment required
Upstream			☐ Yes ☒ No
Downstream			☐ Yes ☒ No

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Conductivity
Upstream			15	11	
Downstream			15	11	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
Upstream		50	25	25	
Downstream		40	25	35	

Water Colour	Colourless	Yellow/Brown	Blue/Green	Turbid	Other
Upstream	☐	☐	☐	☒	☐
Downstream	☐	☐	☐	☒	☐

Substrate (%Upstream Downstream)	Bedrock	Boulder	Rubble	Gravel	Sand	Silt	Clay	Muck	Marl	Detritus
Upstream			5	20	30	20	10	10		5
Downstream			5	20	30	20	10	10		5

Instream Cover (% surface area)	Undercut Banks	Boulders	Logs/Trees	Organic Debris	VascularPlants	None
Upstream	5			10	10	75
Downstream	5		5	10		70

FIELD COLLECTION RECORD – Page 2

Shore Cover (% of stream shaded)	90-100%	60-90%	30-60%	1-30%	None
Upstream	☐	☒	☐	☐	☐
Downstream	☐	☒	☐	☐	☐

Vegetation Type (% of surface area)	Submergent	Floating	Emergent	None
Upstream	3	3	4	90
Downstream	3	3	4	90

Banks	Stable (%)	Unstable (%)	Surrounding Land Use
Upstream	100		Golf Course
Downstream	100		Forested

Migratory Obstructions	None	Seasonal	Permanent	Sources of Pollution
Upstream	☒			
Downstream	☒			

Potential Critical Habitat	Spawning	Upwellings	Other
Upstream	☐	☐	
Downstream	☐	☐	

Potential Enhancement Opportunities

Comments

Sinuosity	D50	Bankfull Return Period	Erodible Soils
		☐ >1.2-1.5yrs ☐ <1.2-1.5yrs	☐ High ☐ Moderate ☒ Low

Stream Classifications

MNR

- ☐ Coldwater
- ☐ Potential Coldwater
- ☒ Warmwater Sportsfish
- ☒ Warmwater Habitat
- ☐ Warmwater Urban
- ☐ Migratory

DFO

- ☐ 1
- ☐ 2
- ☐ 3

Rosgen

Additional Notes

FIELD COLLECTION RECORD

Project Name	Project #	Date: Day/Month/Year
Highway 400 1.0km South of Highway 89 northerly 30km to Junction of Highway 11	GWP 30-95-00	10/10/2000

Name of Waterbody	Drainage System	Station No.	Collection No.
Willow Creek	2EC	18+280	1

County or Judicial District	Township
Simcoe	Vespra

UTM Coordinates	Easting	Northing
	607220	4919720

Data Collectors: Bilz, Hulse

Weather Conditions: Overcast, Cool

Water Type: ☐ Spring ☐ Canal ☒ Stream/River ☐ River/Lake Junction ☐ Flooded Area ☐ Pool ☐ Pond ☐ Lake ☐ Muskeg/Bog ☐ Reservoir ☐ Other

Dimensions	Length(m)	Mean Wetted (m)	Mean Channel Width (m)	Mean Depth(m)	Current Velocity
	120	8.4	15.1	0.8	

Stream Morphology (# of or %)	Riffles	Runs	Pools	Flats	Other
		45	25	30	

Plant Type (%)	Submergent	Floating	Emergent	None
	3 Eg. algae	3 Eg. pondweed	4 Eg. sedges Canada rush	90

Bottom Type (%)	Bedrock	Boulder	Rubble	Gravel	Sand	Other
	Clay 10	Muck 10	Marl 5	Detritus 5	Silt 20	

Water Chemistry	pH	D.O (mg/l)	Water Temp.	Air Temp.	Water Color
			15	11	☐ Colorless ☐ Blue/Green ☐ Yellow/Brown ☐ Other ☒ Turbid

Cover (instream) % of total surface area	Undercut Banks	Boulders	Logs/Trees	Organic Debris	Vascular Macrophytes	Combination	No Cover
	5		5	10	10		70

Cover/Canopy (Shore) % stream shaded			Bank Characteristics (stability)	Surrounding Land Use
	Full Canopy (100%) ☐ Good Canopy (70-90%) ☒ No Canopy ☐	Fair Canopy (40-60%) ☐ Poor Canopy (10-30%) ☐	stable	Golf Course Forested

Gear						
	Electrofisher ☐ Length Electrosec Hoop Net ☐	Seine ☐ # Hauls Trap Net ☐	Gill Net ☐ Minnow Trap ☐	Angled ☐ Trawl ☐	Dip Net ☐ Piscicide ☐	Surber ☐ Other ☐

Size of Net (Gill or Seine)	Size of net or mouth (Trap,Hoop,Trawl)	Mesh Size	Depth of Capture
Length m	m	Smallest cm Largest cm	Min m Max m

Fish Kept?	No. of Bags	Preservative
Yes ☐ No ☒		Formalin ☐ Kahle's Solution ☐ Alcohol ☐ Frozen ☐ Other ☐

Additional Data: Pollution,Migratory Obstructions, Parasites,Color & Condition of Fish etc.	Habitat Map Included
	Yes ☐ No ☒

FIELD COLLECTION RECORD

Project No. GWP 30-95-00			Station No. 18+280	Collection No. 1	
No.	Scientific Name	Total Length Range (cm)	Weight Range (gm)	Physical Condition	
				# fish with blackspot	(lesions, tumors, maturity etc.)

Identified by:	Date Day / Month / Year
R. Bilz	10/10/2000

Comments: No fish sampling was completed since there was MNR records documenting a warmwater fishery which includes yellow perch, northern pike and migrating walleye.

STATION DIAGRAM

Ontario

Ministry of Transportation
Southwestern Region
Planning and Design Section

HIGHWAY 400 PLANNING AND PRELIMINARY DESIGN STUDY

FROM 1 KM SOUTH OF HIGHWAY 89 NORTHERLY 30 KM
TO THE JUNCTION AT HIGHWAY 11

COUNTY OF SIMCOE
TOWN OF BRADFORD WEST GWILLIMBURY
TOWN OF INNISFIL
CITY OF BARRIE
TOWNSHIP OF SPRINGWATER

G.W.P. 30-95-00

APPENDIX Q

ENVIRONMENTAL NOISE IMPACT ASSESSMENT REPORT

FINAL REPORT
ENVIRONMENTAL NOISE IMPACT ASSESSMENT
HIGHWAY 400 (HWY 89 to HWY 11)
BARRIE, ONTARIO
GWP 30-95-00

Project Number: 01-133

Date: September 22, 2003

Submitted By: Rowan Williams Davies & Irwin Inc.

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APPENDICES

Appendix A - RWDI DRAFT Preliminary Report - Highway 400 (Hwy 89 to HWY 11)

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Appendix C - Modelling Results for Sample Mainline Receptors

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1. INTRODUCTION

Rowan Williams Davies and Irwin Inc. (RWDI) was retained by URS Cole, Sherman and Associates Limited to perform a noise impact assessment of the preferred widening and interchange improvements for Highway 400, extending from Highway 89 to Highway 11, north of Barrie. Eight lanes are proposed from just south of Highway 89 to just south of Essa Road. The widening in this section will be constructed to the west. Widening to ten lanes will take place through most of the City of Barrie, decreasing to eight lanes again just north of Bayfield Street to the Hwy 400/Hwy 11 interchange. Road widening in this section will, for the most part, occur along the centre line; however, a westerly shift of 5m's between Essa Road and Anne Street is proposed. Significant changes, (e.g., several new ramp structures), are proposed for every major interchange through the City of Barrie.

A key map, showing the extent of the study area, and the location of key features of the work, is shown in **Figure 1**.

Future traffic noise impacts from highway operation and temporary construction noise impacts associated with the proposed changes were assessed. Noise impacts associated with possible pile driving, required at the overpasses and underpasses, were also evaluated. This report provides the findings due to the proposed widening and interchange improvements at representative worst-case residences in the study area. A report outlining the preliminary findings was sent to URS Cole, Sherman for their review. A review of the potential noise impacts of the various route alternatives was also completed. A copy of these reports can be found in **Appendix A**.

2. NOISE ASSESSMENT METHODOLOGY

A glossary of environmental noise terms used in this report can be found in **Appendix B**.

2.1 Noise Model

The potential noise impacts associated with the proposed widening and interchange improvements were assessed using the traffic noise prediction model, STAMSON 5.03, the computerized noise model advocated by the Ministry of the Environment (MOE) [1]. The following factors were taken into account:

a) Horizontal and vertical road-receiver geometry;
b) Ground absorption;
c) Barrier attenuation provided by rows of houses, industrial/commercial buildings, existing road structures, and topography;
d) Attenuation provided by treed areas;
e) Road grade;
f) Traffic volume and percentage of heavy and medium trucks; and
g) Vehicle speed.

The modelled vehicle speed was the posted speed, in accordance with the additional assessment guidelines set out in the MTO *Environmental Office Manual: Technical Areas - Noise [2]*. The traffic on the mainline, therefore, was modelled at a speed of 100 km/h. Since traffic speed on interchange ramps was not provided, conservative estimates were assumed.

2.2 Applicable Guidelines

The impact assessment, in accordance with the procedural guidelines set out in the MOE/MTO Joint Protocol and MTO Directive QST-A1, is chiefly based on the change in the average 24-hour noise level ($L_{eq}(24)$) from the future "no-build" to the future "build" condition [3,4]. The future "no-build" condition is based on the future traffic volume (at least 10 years from the date

of completion), assuming that no roadway changes take place. The future "build" condition is also based on the future traffic volume, but assumes that the undertaking is in place. In this assessment, the future "no-build" traffic volume was taken to be equal to the future "build" traffic volume.

The noise impact was evaluated outdoors at Noise Sensitive Areas (NSA's) in the study area. NSA's are defined by the MTO Directive and include private homes such as single family residences, townhouses, apartments, hospitals and nursing homes. NSA's must have Outdoor Living Areas (OLA's) associated with the residential unit.

The investigation of noise mitigation by MTO is not required for increases up to 5 dB above the future "no-build" ambient level. For increases of more than 5 dB, an investigation for implementing mitigation within the road right-of-way is required. In addition to the change in sound exposure, the guidelines indicate an outdoor objective of either 55 dBA or the pre-construction ambient, whichever is higher.

In addition to the above-mentioned Joint Protocol and MTO Directive, the MTO has also developed a Retrofit Noise Barrier Program to alleviate noise impacts on existing noise sensitive areas adjacent to existing freeways. The retrofit policy is appended to the MTO QST A-1 Directive and is based on the principle that existing NSA's exposed to high noise levels due to their proximity to a freeway should receive some consideration. Where NSA's receive 24-hour average noise levels ($L_{eq}(24)$) in excess of 60 dBA, they may qualify for inclusion on the Candidates for Noise Barrier Retrofit List. However, it is not the intent of the retrofit program to provide noise barriers at all such sites. Mitigation at some sites may not be constructed for a number of reasons, such as the inability to achieve perceptible attenuation, excessive costs to provide mitigation for few homes, or physical limitations. Budget allocations and prioritization of candidate sites determine the implementation of this policy.

2.3 Traffic Data

Existing (2001) and projected (2011) traffic volumes for the study area were supplied by URS Cole, Sherman. The volumes for the mainline (Highway 400) were supplied in the form of

Summer Average Daily Traffic (SADT). Traffic volumes for the interchange ramps were supplied in the form of Annual Average Daily Traffic (AADT), as this was the best available data. The mainline is the dominant source of noise in comparison to the interchange ramps.

Year 2011 volumes were used for both the future "no build" and "build" conditions, as per the direction of URS Cole, Sherman. However, for the "no-build" case, the level of service would be low, and the traffic speed would likely be decreased. Consequently, the noise model for the "no-build" scenario may over predict noise levels. For the purpose of comparison, existing conditions were also assessed using 2001 SADT data for the mainline and 2001 AADT data for the interchange ramps.

The percentage of total commercial vehicles on the study section of Highway 400 was supplied by URS Cole, Sherman. The percentage of commercial vehicles on the ramps was assumed to be equal to those on the applicable mainline link. The breakdown of total commercial vehicles into heavy trucks and medium trucks, for the purpose of this analysis, was assumed to be proportional to the default truck percentages specified in the MTO Environmental Office Manual for Noise [2]. In the interest of conservatively evaluating the impact of the interchange ramps, the same percentages of heavy and medium trucks were used for the interchange ramps as for Highway 400. It was assumed that the percentage of commercial vehicles remains the same for the existing, future "no-build", and future "build" conditions. The modeled vehicle speed on the interchange ramps was an estimated average ramp speed. Traffic data used in the analysis of the mainline and the interchange ramps (ramps that were not modeled, are not included in Table 2) are summarized in **Tables 1 and 2**.

Traffic on local roads and the interchanges were not included in this noise impact assessment. Therefore, the predicted noise impact due to the Highway 400 widening is provided as a conservative estimate.

2.4 Noise Sensitive Areas

Numerous residential Noise Sensitive Areas (NSA's) were identified in the study area. They were identified during a field visit by RWDI staff, with the aid of Ontario Base Maps (OBM's) and the Preliminary Engineering Track Drawings (ETR's) supplied by URS Cole, Sherman, issued on February 20, 2002. Noise levels were predicted at specific Outdoor Living Area (OLA) locations of thirty-five selected representative NSA's. These thirty-five discrete NSA locations were selected jointly by RWDI and Mr. Chris Blaney, MTO Senior Environmental Planner - Acoustics[1]. The thirty-five residential NSA's included in the noise model are summarized in **Table 3**. Modelled NSA locations along the route are shown in **Figures 3a to 3p** and are contained in **Appendix F**. The modelled NSA's represent the worst-case noise impacts in the study area due to the project.

3. RESULTS

Predicted 24-hour average sound levels ($L_{eq}(24)$) at the modelled NSA's are summarized in **Tables 4** and **5**. Modelling results for the existing and future "build" scenarios are included in **Appendix C** for two sample receptors (DNH1 and BAH2). The model runs include the ramps and the freeway links, which were run separately.

The modeled NSA's represent locations of worst-case predicted noise impact, and therefore, any other NSA's in the study area would be expected to receive lower overall noise levels and smaller changes than those presented above. The increase from future "no-build" sound levels to future "build" sound levels at most NSA's was predicted to be approximately 0 to 2 dB. Generally, in terms of human perception of changes in noise levels, a change from 1 to 3 dB is considered to be imperceptible, and a change of 4 to 5 dB is considered to be just noticeable. This change would occur over a period of many years as construction progresses and the traffic increases.

The worst-case increase in operational traffic noise of 4 dB in **Table 5** was predicted at receptor BAH2. The primary cause of this increase is the loss of an existing house, resulting in a loss of noise attenuation. The predicted change from existing conditions at this receptor, as shown in

[1] Telephone conversation between Mr. Chris Blaney, MTO, and Mr. Peter VanDelden, RWDI, March 8, 2002.

Table 4, was slightly greater than 5 dB. This is consistent with the "build" versus "no-build" change, and highlights the effect of the house removal.

Mitigation requirements are based on comparison of future "no-build" and "build" sound levels. As shown in **Table 5**, predicted increases in sound exposure from future "no-build" to "build" at worst-case NSA's are very slight. According to the MTO Directive QST A-1, noise mitigation is not required for increases of less than 5 dB. However, as per the Directive's policy on Retrofit of Existing Freeways, NSA's built before 1977 that are exposed to existing noise levels in excess of 60 dBA, may receive some consideration by the MTO for noise barrier retrofit. These areas may be considered for noise barrier retrofit if an insertion loss of 5 dBA could be achieved, and if the work is considered to be cost-efficient and administratively feasible. This is consistent with the MTO objective to reduce sound levels to the Provincial Objective of 55 dBA. Existing sound levels at several NSA's within the study area were predicted to exceed 60 dBA. At this time, information is not available as to whether or not any noise sensitive areas identified within the study area are on the MTO's Retrofit Barrier Candidates List.

Under MTO Directive QST A-1, investigation of mitigation at receptor BAH2 would not be required. This site is representative of 3 residences. Considering that the change from existing conditions is more than 5 dB (5.5 dB when rounding to one decimal point) in this area, the MTO may wish to consider noise barriers in this area regardless of the fact that it is not strictly required.

4. CONSTRUCTION NOISE ASSESSMENT

4.1 Predicted Sound Levels During Construction

Noise levels at representative worst-case NSA's due to expected typical construction activity were predicted by noise modelling. The noise propagation model took into account construction duration and distance attenuation. The noise model generated 1-hour average sound levels ($L_{eq}(1)$) received at the NSA's. The equipment was modelled as being located within the proposed right-of-way and in the immediate vicinity of each NSA, to present worst-case construction noise levels. Sound levels were predicted using reference sound data for various construction equipment, as

summarized in **Table 6**. Sample calculations of construction noise levels are included in **Appendix D** for a number of representative receptors.

The predicted sound levels due to Highway 400 construction noise was 81 dBA for the worst-case receptor (DKH2). The predicted sound levels due to potential pile driving activities at overpasses and underpasses was 88 dBAI for the worst-case receptor (BAH4).

Construction noise, particularly noise associated with potential pile driving activities at overpasses and underpasses, will be noticeable at times and significant at several residences. Impulsive noise levels associated with pile driving are expected to reach as high as 88 dBAI at receptor BAH4 as it is in close proximity to an overpass or underpass. A number of homes, including BAH4, are similarly close to potential pile driving locations, and therefore, are predicted to receive sound levels in the same order of magnitude as those predicted for BAH4. Noise levels in this order are considered to be significant. In order to mitigate these noise impacts, it is recommended that pile driving be restricted to daytime hours, wherever possible.

Construction noise levels do not include the additional contribution from ambient noise associated with road traffic. The predicted existing traffic noise levels can be compared to the sound levels associated with construction activities. Although averaged over different time periods (1 hour versus 24 hours), these levels may be compared due to relatively constant or steady noise characteristics of highways over a 24-hour period.

The worst-case change in construction noise levels associated with piling driving was predicted to be 19 dB (88 dBAI compared to 69 dBA) louder at receptor BAH4 than existing ambient noise, due to summer traffic. The worst-case change in construction noise associated with normal construction activities are predicted to be 10 dB (81 dBA compared to 71 dBA) louder at receptor DKH2 than existing ambient noise, due to summer traffic. An increase in ambient sound levels equal to and greater than 10 dBA are significant (perceived as a doubling of sound exposure). For this reason, it is recommended that construction be limited to daytime hours, wherever possible. Construction noise impact can also be minimized by the following recommendations detailed in Sections 4.2, 4.3 and 4.4.

4.2 Local Noise Control By-Laws

The length of Highway 400 covered in this application lies entirely within the following jurisdictions:

- Town of Innisfil; and
- City of Barrie.

By-law requirements are summarized in **Table 7** and detailed in **Appendix E** [5, 6].

4.3 MOE Model Municipal Noise Control By-Law

Construction noise impacts are temporary in nature, and largely unavoidable. It is for this reason that applicable MOE guidelines stipulate limits on individual pieces of equipment as opposed to a site limit. With adequate controls, impacts can be minimized. However, for some periods of time and types of work, construction noise will be noticeable. In the presence of persistent noise complaints, the following sound emission standards for various types of construction equipment are specified in MOE Publication NPC-115 – "Construction Equipment" and shown in **Table 8** [7].

4.4 Construction Code of Practice

To minimize the potential for construction noise impacts, it is recommended that a Construction Code of Practice be written into the contract documentation for the contractor, as outlined below:

- Construction should be limited to the time periods allowed by local bylaws, as outlined in Section 4.2. If construction activities are required outside of these hours, exemptions from the relevant local noise by-laws should be sought in advance by the contractor directly from the municipality in question.

- There should be explicit indication that MTO contractors are expected to comply with all applicable requirements of the contract and local noise by-laws. Enforcement of noise

control by-laws is the responsibility of the municipality for all work done by MTO contractors.

- All equipment should be properly maintained to limit noise emissions. As such, all construction equipment should be operated with effective muffling devices that are in good working order and constant operation.

- The contract documents should contain a provision that any initial noise complaint will trigger verification that the general noise control measures agreed to are in effect.

- In the presence of persistent noise complaints, all construction equipment should comply with MOE NPC-115 guidelines, as outlined in Table 8.

- In the presence of persistent complaints and subject to the results of a field investigation, the MTO may require alternative noise control measures, where they are reasonably available. In selecting appropriate noise control and mitigation measures, MTO will give consideration to the technical, administrative and economic feasibility of the various alternatives.

5. CONCLUSIONS

Results of the noise modelling indicated that the predicted noise impacts associated with the Highway 400 widening and interchange improvements were no more than 4 dB above the future "no-build" noise levels at worst-case residential Noise Sensitive Areas (NSA's). No appreciable or perceptible increase in sound level was predicted to occur between the future "no-build" and "build" scenarios. Noise mitigation is not required under applicable MTO/MOE guidelines.

The increase at BAH2 was predicted to be 4 dB between the future "no-build" and future "build" case. The primary cause of this increase is the loss of an existing house, resulting in a loss of noise attenuation. Under MTO/MOE guidelines, investigation of noise mitigation is not required for receptor BAH2. This site is representative of three residences. Considering that the change from

existing conditions is slightly more than 5 dB (5.5 dB) in this area, the MTO may wish to consider noise barriers in this area regardless of the fact that it is not strictly required.

Construction noise impacts are temporary in nature but will be noticeable at times at the residential NSA's. Noise associated with pile driving at overpasses and underpasses will be significant at several receptors, and it is recommended that pile driving be restricted to daytime hours. Methods to minimize construction noise impacts should be included in the Construction Code of Practice, such as adhering to local municipal noise by-laws, and ensuring that construction equipment is in good working order with effective muffling devices.

6. REFERENCES

1. Ontario Ministry of the Environment. STAMSON v 5.03. Noise Assessment and Systems Support Unit. c. 1996.

2. Ontario Ministry of Transportation. *Environmental Office Manual: Technical Areas - Noise* (EO-V-1000-00). Quality & Standards Division, Environmental Office. May 15, 1992.

3. Ontario Ministry of Environment and Ontario Ministry of Transportation. *A Protocol for Dealing with Noise Concerns during Preparation, Review and Evaluation of Provincial Highway's Environmental Assessments.* February 1986.

4. Ontario Ministry of Transportation. *Ministry Directive QST A-1: Noise Policy and Acoustic Standards for Provincial Highways.* February 21, 1992.

5. The Corporation of the Town of Innsifil. By-Law No. 72-91, *A by-law for prohibiting or regulating within the Municipality of the Town of Innisfil, or within any defined area or areas thereof, the making of noises, or public nuisances, likely to disturb the inhabitants of the said Town.* August 1991.

6. The Corporation of the City of Barrie. *By-Law 76-140, Noise Control By-Law.*

7. Ontario Ministry of the Environment. *Model Municipal Noise Control By-Law - Final Report.* August 1978.

Table 1: Two-Way Mainline Traffic Data Used in Noise Prediction Model

Segment of Highway 400	SADT[1]		Percentage of Commercial Vehicles[2]
	2001	2011	
CR 88 to Highway 89	87,300	121,800	12
Highway 89 to Innisfil Beach Rd.	88,900	126,800	12
Innisfil Beach Rd. to Molson Park Dr.	95,700	136,900	12
Molson Park Dr. to Essa Rd.	94,400	135,200	12
Essa Rd. to Dunlop St.	113,600	161,000	12
Dunlop St. to Bayfield St.	115,700	169,000	9.5
Bayfield St. to Duckworth St.	83,700	111,400	12
Duckworth St. to Highway 11	78,300	107,200	12

Notes: 1 SADT - Summer Average Daily Traffic volumes, provided by URS Cole, Sherman.
2 Percentages of heavy vehicles provided by URS Cole, Sherman. The ratio of medium to heavy trucks for each segment is expected to match the default MTO ratio of 25 percent medium and 75 percent heavy trucks.

Table 2: Interchange Traffic Data Used in Noise Prediction Model

Interchange	Ramp	AADT[1]		Percentage of Commercial Vehicles[2]	Posted Speed (km/h)[3]
		2001	2011		
Essa Road	ESSE/W1 to ESSE/W2	5,100	6,800	12	60
Dunlop Street	DNS-W1	9,100	12,600	9.5	60
	DNS-W2 to DNS-W3	9,100	12,600	9.5	50
	DNE/W-N1	7,800	10,800	9.5	50
	DNE/W-N2	7,800	10,800	9.5	90
	DNS-W1055	9,100	12,600	9.5	80
	DNS-W1067	9,100	12,600	9.5	60
	DNS-W1072	9,100	12,600	9.5	60
	DNS-W1077	9,100	12,600	9.5	60
	DNS-W1082	9,100	12,600	9.5	60
	DNE/WN101 to DNE/WN102	7,800	10,800	9.5	60
	DNE/WN103	7,800	10,800	9.5	80
Bayfield Street	BAS-E/W	13,400	14,600	9.5	60
	BAS-E/W1	13,400	14,600	9.5	80
	BAS-E/W2	13,400	14,600	9.5	60
	BAS-E/W3	13,400	14,600	9.5	40
	BAE/W-N1	4,400	4,800	12	60
	BAE/W-N2	4,400	4,800	12	60
	BAE/W-N3	4,400	4,800	12	80
	BAN-E/W to BAN-E/W1	4,500	4,900	12	70
	BAN-E/W2	4,500	4,900	12	90
Duckworth Street	DKS-E/W1	9,500	14,800	12	90
	DKS-E/W2	9,500	14,800	12	70

Notes:
1 AADT - Average Annual Daily Traffic volumes, provided by URS Cole, Sherman.
2 Percentages of heavy vehicles provided by URS Cole, Sherman. The ratio of medium to heavy trucks for each segment is expected to match the default MTO ratio of 25 percent medium and 75 percent heavy trucks.
3 Assumed to be conservative values. Actual posted limits are not known.

Table 3: Locations of Representative Mainline NSA's

Receptor Identifier	Nearest Interchange	Location Description	Distance from Existing Highway 400 Centerline (m)
89H3	N of Hwy 89	last house before 3rd Line on Reive Road	106
INH10	N of Innisfil Beach Road	first house E of Hwy 400, N of 10th Line	233
ESH2	Essa Road	house parallel to Hwy 400 at station 27+800, at the elbow of Chieftain Crescent; east side of the street	53
ESH3	Essa Road	house near station 27+800, at the elbow of Chieftain Cres.; west side of the street	128
ESH4	Essa Road	fourth house from Chieftain Crescent on W side of Marshall Street	154
ESH5	Essa Road	house on Jane Crescent with pool that overlooks the Essa Road interchange	190
ESH6	Essa Road	house on Jane Crescent directly E of house with pool (ESH5)	245
ESH7	N of Essa Road	first house E of Hwy 400 on N side of Tiffin Street	86
ESH8	N of Essa Road	fourth house E of Hwy 400 on N side of Tiffin Street	142
DNA2	Dunlop St	apartment building on Wellington Street at station 12+500	84.5
DNH1	Dunlop St	house on N side of Henry Street at the W end	86
DNH2	Dunlop St	house on N side of Henry Street, 3rd from the W end	124
DNH3	Dunlop St	house on N side of Henry Street, 5th from the W end	167
DNH4	Dunlop St	house on the SW corner of Anne Street intersection with Edgehill Drive	71.5
DNH5	N of Dunlop St	last house on the E end of Edgehill Drive	75
BAH1	Bayfield Road	first house NE of Sunnidale Road SE of Hwy 400	53
BAH2	Bayfield Road	second house NE of Sunnidale Road NE of Hwy 400	77
BAH3	Bayfield Road	last house on W side at the N end of Toronto Street	68
BAH4	Bayfield Road	third house S of Hwy 400 on W side of Bayfield Road	101.5
BAH5	Bayfield Road	house on Patricia Avenue closest to N-E/W ramp	125
BAH6	Bayfield Road	third house from elbow on NW side of Patricia Avenue	137.5
BAH7	Bayfield Road	house closest to Hwy 400 at elbow in Patricia Avenue	43
BAH8	Bayfield Road	S end of townhouse row on Ferris Lane which is located next to open area	67.5
BAH9	Bayfield Road	house W of the cul de sac at the end of Ottaway Avenue	75
BAH10	Bayfield Road	second house W of open space on N side of Ottaway Avenue (house with kidney shaped pool)	106

Receptor Identifier	Nearest Interchange	Location Description	Distance from Existing Highway 400 Centerline (m)
BAH11	Bayfield Road	house directly N of Lount Street at intersection with Agnes Street	125.5
BAH12	Bayfield Road	house NW of Ferris Lane at station 14+675	198
BAH13	Bayfield Road	house across Rose Street from OPP office driveway	178.5
BAH14	Bayfield Road	third house E of Toronto Street on N side of Grove Street	125
DKA1	Duckworth St	large NE building of the Georgian College Residence apartment buildings	66
DKH1	Duckworth St	NW corner of Cundles Road intersection with Street B on draft plan approved subdivision	320
DKH2	N of Duckworth St	house on S side of Little Lake Road at station 16+950	51
DKH3	N of Duckworth St	house on N side of Little Lake Road at station 16+950	128
DKH4	N of Duckworth St	first house W of Royal Victoria Hospital on the N side of Georgian Drive	62
DKH5	N of Duckworth St	fourth house W of Royal Victoria Hospital on the N side of Georgian Drive	98.5

Table 4: Predicted Existing and Future "Build" Sound Levels L_{eq} (24) at Representative Mainline NSA's

Receptor Location	Predicted Sound Level[1] (dBA)		Change in Sound Level (dB)[1]
	Future "Build"	Existing	
89H3	67	66	1
INH10	62	61	1
ESH2	73	71	2
ESH3	62	61	1
ESH4	61	59	2
ESH5	64	63	1
ESH6	61	59	2
ESH7	72	71	1
ESH8	68	67	1
DNA2	71	69	2
DNH1	70	67	3
DNH2	66	64	2
DNH3	64	62	2
DNH4	72	68	4
DNH5	68	67	1
BAH1	75	72	3
BAH2	71	66	5
BAH3	71	69	2
BAH4	71	69	2
BAH5	68	67	1
BAH6	59	58	1
BAH7	72	71	1
BAH8	72	70	2
BAH9	71	69	2
BAH10	69	68	1
BAH11	67	66	1
BAH12	60	59	1
BAH13	60	59	1
BAH14	62	60	2
DKA1	71	70	1
DKH1	59	58	1
DKH2	73	71	2
DKH3	65	63	2
DKH4	75	73	2
DKH5	73	71	2

Notes: 1 Change in predicted sound level from future "build" less the existing scenario.

Table 5: Predicted Future "No-Build" and Future "Build" Sound Levels L_{eq} (24) at Representative Mainline NSA's

Receptor Location	Predicted Sound Level [1] (dBA)		Change in Sound Level (dB) [1]
	Future "No-Build"	Future "Build"	
89H3	68	67	-1*
INH10	62	62	0
ESH2	73	73	0
ESH3	62	62	0
ESH4	61	61	0
ESH5	64	64	0
ESH6	61	61	0
ESH7	72	72	0
ESH8	68	68	0
DNA2	71	71	0
DNH1	69	70	+1
DNH2	66	66	0
DNH3	64	64	0
DNH4	70	72	+2
DNH5	68	68	0
BAH1	74	75	+1
BAH2	67	71	+4
BAH3	70	71	+1
BAH4	70	71	+1
BAH5	68	68	0
BAH6	59	59	0
BAH7	72	72	0
BAH8	71	72	+1
BAH9	70	71	+1
BAH10	69	69	0
BAH11	67	67	0
BAH12	60	60	0
BAH13	60	60	0
BAH14	61	62	+1
DKA1	71	71	0
DKH1	59	59	0
DKH2	73	73	0
DKH3	65	65	0
DKH4	75	75	0
DKH5	73	73	0

Notes: 1 Change in predicted sound level from future "build" less "no-build" scenario.
* For receptor 89H3 the modelled future "no-build" sound levels are higher than the future "build" sound levels. This is due to rounding effects.

Table 6: Reference Sound Levels for Heavy Equipment Included in Construction Noise Model

Heavy Equipment	Quantity Used in Model	Reference Sound Level [1,2,3] (dBA)	Reference Distance (m)
Grader	1	85	15
Bulldozer	1	85	15
Moving Dump Truck	1	88	15
Idling Dump Truck	3	65	15
Roller	1	74	15
Compressor	2	81	15

Notes:
1 The Reference Sound Level is the sound level received at the Reference Distance, not the distance between the NSA and the modelled noise source.
2 All equipment was modelled as being continuously operating (60 minutes of the hour), except for moving dump trucks, which were assumed to operate for half of any hour.
3 Reference sound levels for all equipment except bulldozers were obtained from United States Department of Transportation, *Transit Noise and Vibration Impacts Assessment*, Chapter 12: Noise and Vibration During Construction, Table 12-1, April 1995. The reference sound level for bulldozers was obtained from MOE Publication NPC-115, contained in the MOE *Model Municipal Noise Control By-Law* [7].

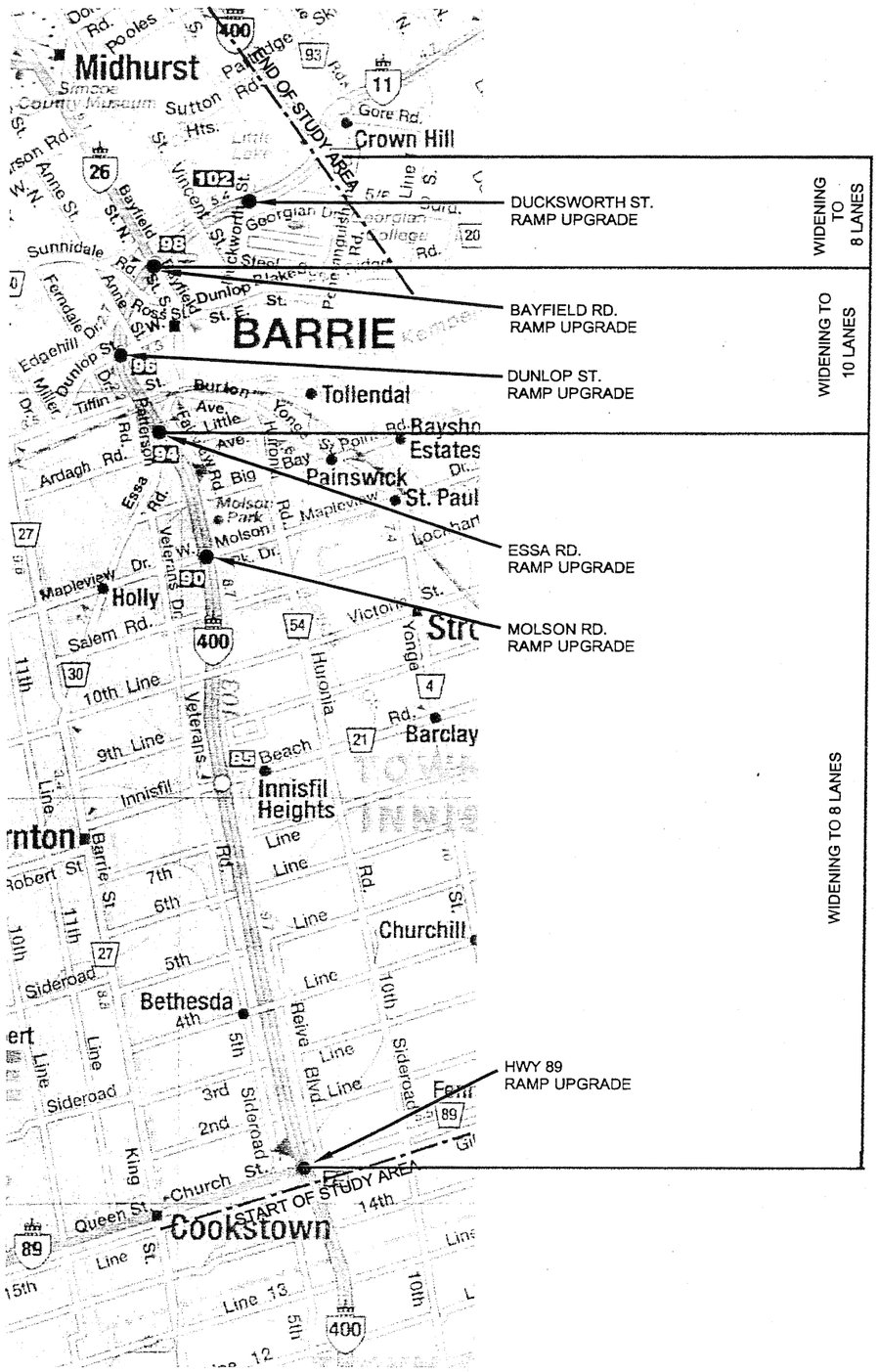
Table 7: Applicable Local Noise Control By-laws

Town	By-law No.	Applicable By-Law Provision
Town of Innisfil	72-91	"No person shall create or suffer to be created any noise within the limits of the Town of Innisfil which is likely to disturb any inhabitant thereof", including "any unreasonable or unnecessary noise or sound arising from any excavation or construction work whatsoever, ..." [5]
City of Barrie	76-140	"Construction equipment may not be used from 1900 hours one day to 0700 hours the next, and all day Sunday and statutory holidays." [6]

Table 8: NPC-115 Maximum Noise Emission Levels for Construction Equipment

Type of Unit	Maximum Sound Level[1] (dBA)	Distance (m)	Power Rating (kW)
Excavation Equipment [2]	83	15	Less than 75 kW
	85	15	75 kW or Greater
Pneumatic Equipment [3]	85	7	–
Portable Compressors	76	7	–

Notes:
1 Maximum permissible sound levels presented here are for equipment manufactured after January 1, 1981.
2 Excavation equipment includes bulldozers, backhoes, front end loaders, graders, excavators, steam rollers and other equipment capable of being used for similar applications.
3 Pneumatic equipment includes pavement breakers.



Key Plan Showing Study Area and Key Features

Environmental Noise Impact Assessment
Highway 400 (89 to 11) - Barrie, Ontario
GWP 30-95-00

Project #01-133

True North

Drawn by: SWJ	Figure: 1
Approx. Scale:	NTS
Date Revised:	May 23, 2002

RWDI

APPENDIX A

To:	Mike Bricks	**From:**	Scott Shayko
Company:	URS Cole, Sherman & Associates	**Date:**	April 3, 2002
Fax #:	905-882-4399	**RWDI Reference #:**	01-133-20
Re:	Preliminary Noise Impact Assessment of Preferred Hwy 400 Widening & Interchange Alternatives	**Pages:**	8 (Including cover sheet)

Mike,

This memo presents the preliminary results of the noise impact assessment of the preferred Highway 400 widening and interchange alternatives. This follows our June 2001 preliminary noise impact assessment of the two proposed widening alternatives and a number of interchange alternatives. This assessment presents the predicted noise impact of the preferred widening alternative and the preferred interchange improvements.

PROPOSED CHANGES UNDER REVIEW

Eight lanes are proposed from just south of Highway 89 to just south of Essa Road. This widening will be constructed to the west. Widening to ten lanes will take place through most of the City of Barrie, decreasing to 8 lanes again just north of Bayfield Street to the Hwy 400/Hwy 11 interchange. This widening for the most part will occur along the centre line (there is a westerly shift of 5 metres between Essa Road and Anne Street). Significant changes, e.g., several new ramp structures, are proposed for every major interchange through the City of Barrie. Predicted noise impacts associated with the relevant overpass, ramp and mainline reconfigurations have been included in this assessment.

APPLICABLE NOISE GUIDELINES and IMPACT ASSESSMENT

The applicable noise guideline used in this study is the Ontario Ministry of Transportation (MTO) QST A-1 Directive, *Noise Policy and Acoustic Standards for Provincial Highways*. Also referenced was the *Protocol for Dealing with Noise Concerns during Preparation, Review and Evaluation of Provincial Highway's Environmental Assessments*, developed jointly by the Ministry of Environment and the MTO.

The impact assessment, in accordance with the MTO QST A-1 Directive, is chiefly based on the change in the average 24-hour noise level ($L_{eq}(24)$) from the future "no-build" to the future "build" condition, evaluated at the location of a Noise Sensitive Area (NSA). The future "no-build" condition assumes that no roadway changes take place, while the future "build" condition assumes that the undertaking proceeds. Both scenarios are based on future traffic data, projected at least 10 years from the date of completion.

Where the future "no-build" noise level is predicted to increase by 5 dB or more, investigation of noise mitigation is required. Predicted sound levels for existing conditions were also generated for comparison purposes and are presented in this memo.

Noise Sensitive Areas (NSA's), defined by the QST A-1 Directive, are land uses within the study area that have Outdoor Living Areas (OLA's) associated with them, and include the following:

- Private homes such as single family residences;
- Townhouses;
- Multiple unit buildings, such as apartments with OLA's for use by all occupants;
- Hospitals and nursing homes where there are OLA's for the patients; and
- Dormitories with OLA's.

NOISE MODEL

Predicted noise levels associated with the existing, future "no-build", and future "build" scenarios were generated through analysis using STAMSON v.5.03, the traffic noise prediction model advocated by the MOE. The following factors were taken into account:

a) Horizontal and vertical road-receiver geometry;
b) Ground absorption;
c) Barrier attenuation provided by rows of houses, industrial/commercial buildings, existing road structures, and topography;
d) Attenuation provided by treed areas;
e) Road grade;
f) Traffic volume and percentage of heavy and medium trucks; and
g) Vehicle speed.

The modelled vehicle speed was the posted speed, in accordance with the additional assessment guidelines set out in the MTO *Environmental Office Manual: Technical Areas - Noise*. The traffic on the mainline, therefore, was modelled at a speed of 100 km/h for all three scenarios. Since traffic speed on interchange ramps was not provided, conservative estimates were assumed.

Noise levels were predicted at specific Outdoor Living Area (OLA) locations of 34 (thirty-four) selected representative NSA's. These 34 discrete NSA locations were selected jointly by RWDI and Mr. Chris Blaney, MTO Senior Environmental Planner - Acoustics[1].

TRAFFIC VOLUMES

Existing (2001) and future (2011) traffic volumes in the form of Summer Average Daily Traffic (SADT), as well as the percentage of commercial vehicles, used in the noise prediction modelling were supplied by URS Cole, Sherman. As directed by URS Cole, Sherman, the future "no build" traffic volumes were taken to be equal to the future "build" volumes.

[1] Telephone conversation between Mr. Chris Blaney, MTO, and Mr. Peter VanDelden, RWDI, March 8, 2002.

Traffic data used in the mainline analysis are summarized in **Table 1**, below.

Table 1: Two Directional Mainline Traffic Data Used in Noise Prediction Model

Segment of Highway 400	SADT		Percentage of Commercial Vehicles
	2001	2011	
CR 88 to Highway 89	87,300	121,800	12
Highway 89 to Innisfil Beach Rd.	88,900	126,800	12
Innisfil Beach Rd. to Molson Park Dr.	95,700	136,900	12
Molson Park Dr. to Essa Rd.	94,400	135,200	12
Essa Rd. to Dunlop St.	113,600	161,000	12
Dunlop St. to Bayfield St.	115,700	169,000	9.5
Bayfield St. to Duckworth St.	83,700	111,400	12
Duckworth St. to Highway 11	78,300	107,200	12

Traffic data for the interchange ramps was supplied in the form of Annual Average Daily Traffic (AADT) volumes, as this was the best available data. Therefore, AADT volumes were used in modelling the interchange ramps. **Table 2** summarizes the AADT volumes used in the analysis (ramps that were not modelled are not included in **Table 2**). The percentage of commercial vehicles on the ramps was assumed to be equal to those on the applicable mainline link.

Table 2: Interchange Traffic Data Used in Noise Prediction Model

Interchange	Ramp	AADT		Percentage of Commercial Vehicles
		2001	2011	
Essa Road	ESSE/W1 to ESSE/W2	5,100	6,800	12
Dunlop Street	DNS-W1 to DNS-W3	9,100	12,600	9.5
	DNE/W-N1 to DNE/W-N2	7,800	10,800	9.5
	DNS-W1055	9,100	12,600	9.5
	DNS-W1067	9,100	12,600	9.5
	DN-/W1072	9,100	12,600	9.5
	DNS-W1077	9,100	12,600	9.5
	DNS-W1082	9,100	12,600	9.5
	DNE/WN101 to DNE/WN103	7,800	10,800	9.5
Bayfield Street	BAS-E/W1 to BAS-E/W3	13,400	14,600	9.5
	BAE/W-N1 to BAE/W-N3	4,400	4,800	12
	BAN-E/W to BAN-E/W2	4,500	4,900	12
Duckworth Street	DKS-E/W1 to DKS-E/W2	9,500	14,800	12

Traffic on local roads and the interchanges were not included in this noise impact assessment. Therefore, a conservative assumption that highlights the noise impact due to the Highway 400 widening is provided.

MODELLED NOISE SENSITIVE AREAS

Receptors representing NSA's along the Highway 400 mainline were selected as shown in **Table 3**.

Table 3: Locations of Representative Mainline NSA's

Receptor Identifier	Nearest Interchange	Location Description
89H3	N of Hwy 89	last house before 3rd Line on Reive Road
INH10	N of Innisfil Beach Road	first house E of Hwy 400, N of 10th Line
ESH2	Essa Road	house parallel to Hwy 400 at station 27+800, at the elbow of Chieftain Crescent; east side of the street
ESH3	Essa Road	house near station 27+800, at the elbow of Chieftain Cres.; west side of the street
ESH4	Essa Road	fourth house from Chieftain Crescent on W side of Marshall Street
ESH5	Essa Road	house on Jane Crescent with pool that overlooks the Essa Road interchange
ESH6	Essa Road	house on Jane Crescent directly E of house with pool (ESH5)
ESH7	N of Essa Road	first house E of Hwy 400 on N side of Tiffin Street
ESH8	N of Essa Road	fourth house E of Hwy 400 on N side of Tiffin Steet
DNA2	Dunlop St	apartment building on Wellington Street at station 12+500
DNH1	Dunlop St	house on N side of Henry Street at the W end
DNH2	Dunlop St	house on N side of Henry Street, 3rd from the W end
DNH3	Dunlop St	house on N side of Henry Street, 5th from the W end
DNH4	Dunlop St	house on the SW corner of Anne Street intersection with Edgehill Drive
DNH5	N of Dunlop St	last house on the E end of Edgehill Drive
BAH1	Bayfield Road	first house NE of Sunnidale Road SE of Hwy 400
BAH2	Bayfield Road	second house NE of Sunnidale Road NE of Hwy 400
BAH3	Bayfield Road	last house on W side at the N end of Toronto Street
BAH4	Bayfield Road	third house S of Hwy 400 on W side of Bayfield Road
BAH5	Bayfield Road	house on Patricia Avenue closest to N-E/W ramp
BAH6	Bayfield Road	third house from elbow on NW side of Patricia Avenue
BAH7	Bayfield Road	house closest to Hwy 400 at elbow in Patrica Avenue
BAH8	Bayfield Road	S end of townhouse row on Ferris Lane which is located next to open area
BAH9	Bayfield Road	house W of the cul de sac at the end of Ottaway Avenue
BAH10	Bayfield Road	second house W of open space on N side of Ottaway Avenue (house with kidney shaped pool)
BAH11	Bayfield Road	house directly N of Lount Street at intersection with Agnes Street
BAH12	Bayfield Road	house NW of Ferris Lane at station 14+675
BAH13	Bayfield Road	house across Rose Street from OPP office driveway
BAH14	Bayfield Road	third house E of Toronto Street on N side of Grove Street
DKA1	Duckworth St	large NE building of the Georgian College Residence apartment buildings
DKH1	Duckworth St	NW corner of Cundles Road intersection with Street B on draft plan approved subdivision
DKH2	N of Duckworth St	house on S side of Little Lake Road at station 16+950
DKH3	N of Duckworth St	house on N side of Little Lake Road at station 16+950
DKH4	N of Duckworth St	first house W of Royal Victoria Hospital on the N side of Georgian Drive
DKH5	N of Duckworth St	fourth house W of Royal Victoria Hospital on the N side of Georgian Drive

RESULTS AND IMPACT ASSESSMENT

As directed by URS Cole, Sherman, the future "no build" traffic volumes were taken to be equal to the future "build" volumes. Any predicted increase in sound level from "no-build" to "build" was therefore due to the widening of the

highway. The future "no-build scenario" will likely not have the traffic volume capacity to carry the predicted (2011) traffic volumes resulting in lower traffic speeds than the posted speed. Therefore, the predicted sound levels for the future "no-build" scenario may be over-predicted. For this reason, the change in sound levels from the existing conditions to the future "build" scenario were also predicted to provide an additional conservative estimate of the change in sound levels due to the proposed widening. Predicted 24-hour average sound levels ($L_{eq}(24)$) at the modelled NSA's are summarized in Tables 4 and 5.

Table 4: Predicted Existing and Future "Build" Sound Levels L_{eq} (24) at Representative Mainline NSA's

Receptor Location	Predicted Sound Level (1) (dBA)		Change in Sound Level (dB)*
	Future "Build"	Existing	
89H3	67	66	1
INH10	62	61	1
ESH2	73	71	2
ESH3	62	61	1
ESH4	61	59	2
ESH5	64	63	1
ESH6	61	59	2
ESH7	72	71	1
ESH8	68	67	1
DNA2	71	69	2
DNH1	71	67	4
DNH2	66	64	2
DNH3	64	62	2
DNH4	72	68	4
DNH5	68	67	1
BAH1	75	72	3
BAH2	71	66	5
BAH3	71	69	2
BAH4	71	69	2
BAH5	68	67	1
BAH6	59	58	1
BAH7	72	71	1
BAH8	72	70	2
BAH9	71	69	2
BAH10	69	68	1
BAH11	67	66	1
BAH12	60	59	1
BAH13	60	59	1
BAH14	62	60	2
DKA1	71	70	1
DKH1	59	58	1
DKH2	73	71	2
DKH3	65	63	2
DKH4	75	73	2
DKH5	73	71	2

*Predicted sound level from future "build" verus existing scenario.

Table 5: Predicted "No-Build" and "Build" Sound Levels L_{eq} (24) at Representative Mainline NSA's

Receptor Location	Predicted Sound Level (1) (dBA)		Change in Sound Level (dB)[1]
	Future "No-Build"	Future "Build"	
89H3	68	67	-1*
INH10	62	62	0
ESH2	73	73	0
ESH3	62	62	0
ESH4	61	61	0
ESH5	64	64	0
ESH6	61	61	0
ESH7	72	72	0
ESH8	68	68	0
DNA2	71	71	0
DNH1	69	71	+2
DNH2	66	66	0
DNH3	64	64	0
DNH4	70	72	+2
DNH5	68	68	0
BAH1	74	75	+1
BAH2	67	71	+4
BAH3	70	71	+1
BAH4	70	71	+1
BAH5	68	68	0
BAH6	59	59	0
BAH7	72	72	0
BAH8	71	72	+1
BAH9	70	71	+1
BAH10	69	69	0
BAH11	67	67	0
BAH12	60	60	0
BAH13	60	60	0
BAH14	61	62	+1
DKA1	71	71	0
DKH1	59	59	0
DKH2	73	73	0
DKH3	65	65	0
DKH4	75	75	0
DKH5	73	73	0

[1] Predicted sound level from future "build" verus "no-build" scenario.

* For receptor 89H3 the modelled future "no-build" sound levels are higher than the future "build" sound levels. This is due to rounding effects.

The modelled NSA's represent locations of worst-case predicted noise impact, and therefore, any other NSA's in the study area would be expected to receive lower overall noise levels and smaller changes than those predicted above.

The increase from future "no-build" sound levels to future "build" sound levels at most NSA's was predicted to be approximately 0 to 2 dB. Generally a change from 1 to 3 dB is considered to be imperceptible, and a change of 4 to 5 dB is considered to be just noticeable.

The increase at BAH2 was predicted to be 4 dB. The primary cause of this increase is the loss of the existing plaza due to the construction of the interchange ramp, resulting in a loss of noise attenuation. The predicted change from existing conditions at this receptor was slightly greater than 5 dB. This is consistent with the "build" versus "no-build" change, and highlights the effect of the plaza removal.

According to the MTO Directive QST A-1, noise mitigation is required for increases in sound exposures (future "build" versus future "no-build") of more than 5 dB. Therefore, under QST-A1 guidelines, investigation of mitigation at receptor BAH2 would not be required. This site is representative of 3 residences. Considering that the change from existing conditions is more than 5 dB (5.5 dB) in this area, the MTO may wish to consider noise barriers in this area regardless of the fact that it is not strictly required.

CONCLUSIONS

The preliminary results of the noise modelling indicate that predicted noise impacts associated with the preferred Highway 400 widening and interchange alternatives are no more than 4 dB above the future "no-build" noise levels at worst-case NSA's. Therefore, under MTO/MOE guidelines, investigation of noise mitigation is not required. Regardless, the MTO may wish to consider some noise mitigation near receptor BAH2.

We hope you find these results useful. Should you have any questions or concerns, please do not hesitate to contact us.

Yours very truly,

ROWAN WILLIAMS DAVIES & IRWIN Inc.

Scott Shayko, Hon. B.Comm., B.Sc., d.E.T.
Project Manager

SCS/smd

fax memo

RWDI

To:	Mike Bricks	**Date:**	June 22, 2001
Company:	Cole, Sherman & Associates	**RWDI Reference:**	01-133
Fax #:	905-882-4399		
Pages:	10 **(Including cover sheet)**		
From:	Sarah McLean		
Re:	Draft Preliminary Environmental Noise Impact Assessment Highway 400 (89 to 11)		

SENT VIA:
JUN 22 2001
ORIG To SAM | XC To SCS

Message:

Mr. Bricks,

Please find attached our Draft Preliminary Noise Impact Assessment for the proposed widening and interchange improvements to Highway 400 from Highway 89 to Highway 11.

Should you have any questions or would like to discuss, please do not hesitate to call me.

Regards,

Sarah McLean, B.Eng.
Project Coordinator

Rowan Williams Davies & Irwin Inc.
650 Woodlawn Road West
Guelph, Ontario
N1K 1B8
Tel: (519) 823-1311
Fax: (519) 823-1316
Email: sam@rwdi.com

1. INTRODUCTION

Rowan Williams Davies and Irwin Inc. (RWDI) was retained by URS Cole, Sherman and Associates Limited to perform a noise impact assessment for the proposed widening and interchange improvements of Highway 400 from Highway 89 to Highway 11, north of Barrie. Two widening alternatives are proposed for the mainline, and alternatives are proposed for each of the major interchanges through Barrie as well. This report presents our preliminary findings of the relative noise impact of the mainline and interchange alternatives. The purpose of this preliminary study is to aid in the evaluation of the proposed alternatives and the ultimate selection of the preferred alternatives.

The two widening alternatives for the mainline are as follows:

Alternative 1: A widening of 8 to 10 lanes about the centerline. Eight lanes are proposed from just south of Highway 89 to approximately 2 km north of Molson Park Drive in Barrie. Widening to ten lanes will take place through most of the city of Barrie, decreasing to 8 lanes again just north of Bayfield Street to the Hwy 400/Hwy 11 interchange.

Alternative 2: A 12-lane express/collector system starting just north of Molson Park Drive to just north of Vincent St. in Barrie. There will be six express lanes and six collector lanes.

2. NOISE ASSESSMENT METHODOLOGY

2.1 Noise Model

The relative impact of the mainline alternatives were assessed by analysis using a traffic noise prediction model. The analysis was performed using STAMSON v5.03, the computerized noise model advocated by the Ministry of the Environment (MOE). The following factors were taken into account:

a) Horizontal road-receiver geometry;
b) Ground absorption;
c) Traffic volume and percentage of trucks; and
d) Vehicle speed.

The analysis was conservative as it did not take into account screening of road noise provided by vegetation, significant changes in topography, or nearby structures.

The interchange alternatives were assessed with a qualitative review based on the drawings provided.

2.2 Applicable Guidelines

The impact assessment, in accordance with the procedural guidelines set out in the MOE/MTO Joint Protocol and MTO Directive QST-A1, is chiefly based on the change in the average 24-hour noise level ($L_{eq}(24)$) from the future "no-build" to the future "build" condition. The future "no-build" condition is based on the future traffic volume (at least 10 years from the date of completion), assuming that no roadway changes take place. The future "build" condition, is also based on the future traffic volume, but assumes that the undertaking is in place. Where future "no-build" traffic volumes are not available, it is a typical conservative assumption that future "no-build" traffic volume is equal to the existing traffic volume.

Low impact is defined as an increase of less than 5 dB above existing sound levels. Moderate impact is defined as an increase of 5 to 10 dB, and high impact is defined as an increase of more than 10 dB.

2.3 Traffic Data

Existing (2001) and future (2011) traffic volumes in the form of Summer Average Daily Traffic (SADT), as well as the percentage of commercial vehicles, were supplied by Cole, Sherman. In this assessment it has been conservatively assumed that future "no-build" traffic

volumes are equal to existing volumes. Traffic volumes used in the analysis were in the form of Summer Average Daily Traffic (SADT) and were provided for several segments of the road. The two mainline alternatives use the same future traffic volume. Traffic data used in the analysis are summarized in Table 1, below.

Table 1: Traffic Data Used in Noise Prediction Model

Segment of Hwy 400	CSA SADT		% Commercial Vehicles
	2001	2011	
CR 88 to Highway 89	86,600	117,900	12
Highway 89 to Innisfil Beach Rd.	89,100	127,600	12
Innisfil Beach Rd. to Molson Park Dr.	96,200	140,600	12
Molson Park Dr. to Essa Rd.	94,100	133,800	12
Essa Rd. to Dunlop St.	114,000	163,000	12
Dunlop St. to Bayfield St.	115,500	168,300	9.5
Bayfield St. to Duckworth St.	87,300	132,900	12
Duckworth St. to Highway 11	78,000	105,600	12

2.3 Receptors of Concern

Numerous worst-case residential areas (single-family homes, townhouses, and apartment buildings) were identified along the Highway 400 corridor within the study limits, particularly through the city of Barrie where there are areas of fairly dense residential housing . Other noise-sensitive areas were identified in the study area but were not considered "worst-case" due to their significant distance from the highway. The existing noise environment at most noise-sensitive areas in the study area is dominated by traffic noise from Highway 400.

Due to the great number of noise-sensitive areas in the study area, the ambient noise environment was modelled at three setback distances from the road rather than at discrete receptor locations. This allowed for virtually all worst-case receptors to be captured in the modelled setback envelopes. Sound levels were predicted at 50 m, 100 m, and 150 m from the existing centerline of the highway. The vast majority of the receptors in the study limits are beyond the 100 m setback, and none have been identified within 50 m of the existing centerline. The Barrie hospital is located approximately 150 m from the existing centerline.

In general, the sound exposure experienced by a receptor between 50 m and 100 m from the centerline of the highway would lie between the sound levels predicted at those two setback distances. However, it should be mentioned that this is not strictly the case, as significant changes in topography such as ravines or escarpments, as well as screening provided by vegetation and structures, can result in noise contour lines that are not parallel to the road. This assessment is preliminary and has not taken into account topographical effects and screening, and therefore assumes generally parallel noise contours. Nonetheless, observations of the study area were made by RWDI staff and it is expected that these factors, if taken into account, would not appreciably affect the predicted results presented in this report.

3. RESULTS AND EVALUATION

3.1 Mainline Alternatives

The results of the preliminary noise modelling of Alternative 1 indicate that worst-case receptors between setbacks of 50 m and 100 m from the existing highway centerline are expected to experience an increase of no more than 2 dB above existing summer ambient noise levels. Alternative 2 would cause an increase at those receptors of no more than 4 dB above existing noise levels. Beyond 100 m from the centerline, the two alternatives have essentially the same noise impact on noise-sensitive receptors. The majority of receptors within the project limits are located beyond 100 m of the existing centerline, and the maximum expected increase associated with either widening alternative is approximately 2 dB over existing noise levels at those receptors. The noise impact at the Barrie Hospital will be an increase of less than 2 dB due to either widening alternative.

Existing and future sound levels are greater than the 60 dBA guideline used in assessing the need for retrofit noise barriers. Therefore, some consideration could be given to placing these receptors on the MTO Retrofit Barrier Candidate List.

The predicted noise impacts associated with the widening alternatives are summarized in Table 2. The evaluation of each alternative is shown in the Table 3.

Table 2: Predicted Unmitigated Sound Levels at Worst-Case Receptors due to Widening

Widening Alternative	Receptor Locations [3]	Maximum Predicted Sound Exposure Level $L_{eq}(24)$ (dBA)		Maximum Increase Above Existing Sound Level (dB)
		Existing (2001)	Future (2011)	
Alternative 1 [1]	Between 50 and 100 m	73 to 68	75 to 69	2
	Between 100 and 150 m	68 to 65	69 to 66	2
Alternative 2 [2]	Between 50 and 100 m	73 to 68	76 to 70	4 to 2
	Between 100 and 150 m	68 to 65	70 to 66	2

Notes:
1. 8 to 10 lane widening throughout the length of the project.
2. 12-lane core/collector system through the City of Barrie.
3. Measured either side of the highway from the existing highway centerline.

Table 3: Noise Impact Assessment of Mainline Alternatives

Measure	Alternative 1 [1]	Alternative 2 [2]
Analysis	Low impact to all receptors within 150 m either side of existing Highway 400 centerline. Increases of no more than 2 dB above existing ambient sound levels are predicted within 150 m. Impacts will be less than 2 dB beyond 150 m.	Low impact to all receptors within 150 m either side of existing Highway 400 centerline. Increases of no more than 4 dB above existing noise levels at receptors within 100 m, no more than 2 dB increase at receptors between 100 m and 150 m. Impacts will be less than 2 dB increase beyond 150 m.
Evaluation Ranking	1	2
Comment	Both alternatives result in similar low noise impacts due to similar changes in lane distances from receptors, identical increase in traffic volume for each future alternative, and the location of the majority of receptors in the study area. Noise impacts associated with Alternative 2 are only slightly greater than those associated with Alternative 1, and affect only a small minority of the receptors in the study area.	

Notes:
1. 8 to 10 lane widening throughout the length of the project.
2. 12-lane core/collector system through the City of Barrie.

3.2 Interchange Alternatives

The interchange alternatives were assessed by a qualitative review and are summarized in Tables 4 through 10. The expected noise impact was based only on ramp and road realignments, not on changes in traffic volumes.

Table 4: Noise Impact Assessment of Highway 89 Interchange Alternatives

Measure	Alternative 1	Alternative 2
Qualitative Review	Expected low impact on 2 homes and low to moderate impact on 1 campground.	Expected low impact on 2 homes and low to moderate impact on 1 campground.
Evaluation Ranking	1	1
Comment	Realignment of Reive Rd. (identical for both alternatives) will have an expected low to possibly moderate noise impact on the nearby campground. Alternative 2 involves ramps that are more unlike existing conditions but are expected to cause noise impacts only marginally higher than Alternative 1. Therefore, from a noise impact perspective, the alternatives are essentially equivalent.	

Table 5: Noise Impact Assessment of Innisfil Beach Road Interchange Alternatives

Measure	Alternative 1 [1]	Alternative 2 [1]	Alternative 3 [1]
Qualitative Review	Expected low impact on 5 homes.	Expected low impact on 5 homes.	Expected low impact on 5 homes.
Evaluation Ranking	1	1	1
Comment	Alternative 1 is most similar to existing conditions and is expected to cause a low impact. Alternatives 2 involves ramp structures that are not similar to existing conditions but low impacts are expected due to the significant distance of noise-sensitive receptors from the interchange. Alternative 3 involves minimal changes and similarly low impacts are expected. Therefore, from a noise impact perspective, all three alternatives are essentially equivalent.		

Table 6: Noise Impact Assessment of Molson Park Drive Interchange Alternatives

Measure	Alternative 1	Alternative 2
Qualitative Review	No expected noise impacts to nearby homes due to interchange improvements	Possible moderate impact to 1 home.
Evaluation Ranking	1	2
Comment	Alternative 1 involves no changes that will cause a noise impact. Alternative 2 involves a new ramp in close proximity to an existing home.	

Table 7: Noise Impact Assessment of Essa Road Interchange Alternatives

Measure	Alternative 1	Alternative 2
Qualitative Review	No expected noise impacts to nearby homes due to interchange improvements.	No expected noise impacts to nearby homes due to interchange improvements.
Evaluation Ranking	1	1
Comment	No noise impacts associated with either interchange are expected. Surrounding land use is chiefly commercial. The nearest residential receptors are located on Chieftan Crescent, off Little Avenue, and are set back far enough from the interchange that the proposed Alternatives will have minimal effect.	

Table 8: Noise Impact Assessment of Dunlop Street Interchange Alternatives

Measure	Alternative 1	Alternative 2
Qualitative Review	Expected low to moderate impacts to approximately 8 homes on Henry Street.	Very low noise impacts to homes on Henry Street.
Evaluation Ranking	2	1
Comment	Surrounding land use is chiefly commercial, save residential receptors on Henry St. Approximately 8 homes on Henry Street would experience a low to moderate increase in noise level due to the increased proximity of the new E-N ramps and the necessary displacement of "Dunlop Village Place", which currently provides screening from the highway noise. Receptors on Anne St. and Edgehill Dr. are significantly set back from the interchange and are not expected to experience impacts associated with either Alternative.	

Table 9: Noise Impact Assessment of Bayfield Street Interchange Alternatives

Measure	Alternative 1 (1)	Alternative 2 (1)	Alternative 3 (1)
Qualitative Review	Low impact to homes on approximately 10 homes on Patricia Ave.	No expected noise impact to nearby homes due to interchange improvements.	Low impact to 4 homes on south end of Patricia Ave. Low impact to homes on Toronto St. and Grove St. W.
Evaluation Ranking	2	1	3
Comment	Alternative 2 involves no changes that will appreciably affect noise levels at nearby homes. Alternative 1 involves a N-W ramp that could cause low impacts to homes on Patricia Ave. Alternative 3 involves possible low impact to several homes on Toronto St. and Grove St. W. due to realigned E-S ramp and probable relocation of YMCA building.		

Table 10: Noise Impact Assessment of Duckworth Street Interchange Alternatives

Measure	Alternative 1	Alternative 2	Alternative 3	Alternative 4
Qualitative Review	No expected impact to nearby homes or residences due to interchange improvements.			
Evaluation Ranking	1	1	1	1
Comment	Alternatives 1 and 2 involve ramp structures similar to existing conditions and are not expected to cause noise impacts at nearby residences. Alternatives 3 and 4 involve more significant changes but noise-sensitive receptors are far removed from those changes.			

4. CONSTRUCTION NOISE ASSESSMENT

Construction noise impacts are temporary in nature, and largely unavoidable. It is for this reason that applicable Ministry of Environment (MOE) guidelines stipulate limits on individual pieces of equipment as opposed to a site limit. With adequate controls, impacts can be minimized. However, for some periods of time and types of work, construction noise will be noticeable. In the presence of persistent noise complaints, the sound emission standards in Table 11 should be verified for the equipment used on site.

Table 11: MOE Sound Emission Standards for Construction Equipment (1)

Type of Equipment	Maximum Sound Level (dBA) (2)	Noise Measurement Distance	Power Rating
Excavation Equipment (3)	83	15 m	Less than 75 kW
	85	15 m	75 kW or greater
Pneumatic Equipment (4)	85	7 m	none specified
Portable Compressors	76	7 m	none specified

Notes:
(1) Documented in MOE Publication NPC-115.
(2) Maximum permissible sound levels presented here are for equipment manufactured after January 1, 1981.
(3) Excavation equipment includes bulldozers, loaders, backhoes, and other equipment capable of being used for similar application.
(4) Pneumatic equipment includes pavement breakers.

The noise by-laws for the Town of Innsifil and the City of Barrie have been obtained and reviewed. The Town of Innsifil noise by-law prohibits "any unreasonable or unnecessary noise or sound arising from any excavation or construction work whatsoever...". There is no reference in the by-law to any specific windows of time in which construction noise is prohibited, and it is assumed that the by-law is in effect 24-hours a day and driven by complaints. The noise by-law in use by the City of Barrie states that construction equipment may not be used from 1900 hours

one day to 0700 hours the next, and all day Sunday and statutory holidays. If the need for nighttime work is anticipated, it is recommended that exemptions from the noise by-laws be sought from the Town of Innisfil and the City of Barrie.

5. CONCLUSIONS

The noise impacts in the study area associated with two widening alternatives, an eight- to ten-lane widening of the highway in the project limits and a twelve-lane core/collector system through the City of Barrie, were assessed. The noise impacts associated with both alternatives – determined in terms of the expected increase above existing summer noise levels at worst-case residential receptors – are essentially equivalent, relative to the number of homes affected. Both alternatives were predicted to cause low impact (less than a 5 dB increase above existing noise levels) at residences of concern in the study area. Noise impacts associated Alternative 2, the core/collector system, are slightly higher for a relatively small number of residences, however the vast majority of homes in the study area are expected to experience similar impacts from either widening alternative.

The interchange alternatives were assessed qualitatively and their evaluations are presented in Section 3.2 of this report.

APPENDIX B

APPENDIX B

ENVIRONMENTAL NOISE TERMINOLOGY

Ambient or Background Noise: The ambient noise from all sources other than the sound of interest (i.e., sound other than that being measured). Under most MOE guidelines, aircraft overflights and train noise, due to their transient nature, are normally excluded from measurements of background noise.

Day-Night Average Sound Level (L_{dn} or DNL): An energy-average A-weighted sound level (see "Energy Equivalent Sound Level" and "dBA") over a one-day period, where the noise level occurring at night (2200 to 0700h) is penalized by 10 dB. This penalty is applied to account for people's tendency to be more sensitive to noise at night than during the day. The L_{dn} is used chiefly in the United States and is a good descriptor of the general annoyance of noise.

dB - Decibel: The logarithmic units associated with sound pressure level, sound power level, or acceleration level. See sound pressure level, for example.

dBA - Decibel, A-Weighted: The logarithmic unit associated with a sound pressure level, where the sound pressure signal has been filtered using the A-weighted filter network. A-weighted sound pressure levels are denoted by the suffix 'A' (i.e., dBA), and the term "pressure" is normally omitted from the description (e.g., sound level or noise level). The A-weighting adjusts the sound pressure signal such that the resultant signal mimics the subjective response of the human ear. The A-weighted level is therefore more representative of the level perceived by humans. It should be understood that "dBA" is simply a notation that indicates that the signal has been modified by the A-weighting; it is not a different unit than the dB. For example, the *change* from an existing level of 45 dBA to a future level of 50 dBA is an increase of 5 dB (not 5 dBA).

Effective Perceived Noise Level (EPNdB): A complex measure of perceived noisiness derived by making adjustments to the magnitude of measured sound levels in narrow frequency bands (1/3 octaves) for tonality and rise time of the noise. EPNdB values are the base measure of an individual overflight noise exposure from aircraft under the NEF metric, analogous to the manner in which SEL is used for computing $L_{eq}(24)$.

Energy Equivalent Sound Level (L_{eq}): An energy-average sound level taken over a specified period of time. It represents the average sound pressure encountered for the period. The time period is often added as a suffix to the label (i.e., $L_{eq}(24)$ for the 24-hour equivalent sound level). L_{eq} is usually A-weighted. An L_{eq} value expressed in dBA is a good, single value descriptor of the annoyance of noise.

Human Perception of Sound: The human perception of noise impact is an important consideration in qualifying the noise effects caused by projects. The following table presents a general guideline.

Increase in Noise Level (dBA)	Perception
1 to 3	insignificant due to imperceptibility
4 to 5	just-noticeable difference
6 to 9	marginally significant
10 or more	significant, perceived as a doubling of sound exposure

Impulsive Noise: Noise composed of a series of short, discrete events, characterized by rapid rise times and at least 0.5 seconds between events.

Exceedance Noise Level (L_N): The noise level exceeded N% of the time. It is a statistical measure of the noise level. For highly varying sounds, the L_{90} represents the background noise level, L_{50} represents the median or typical noise level, and L_{10} represents the short term peak noise levels, such as those due to occasional traffic or a barking dog.

Noise: Unwanted sound.

Noise Level: Analogous to Sound Level, except applied to unwanted sounds.

Noise Exposure Forecast (NEF): A calculated measure of aircraft noise based on the type of aircraft in use, the take-off and landing patterns of the aircraft, and times of operation. It represents the noise exposure over a typical 24 hour period. A penalty is applied to nighttime operation (as in the calculation of the Day-Night Average Sound Level).

Peak Sound Pressure Level: Same as Sound Pressure Level except that peak (not peak-to-peak) sound pressure values are used in place of RMS pressures.

Quasi-Steady Impulsive Noise: Noise composed of a series of short, discrete events, characterized by rapid rise times, but with less than 0.5 seconds elapsing between events.

Sound: a dynamic (fluctuating) pressure.

RMS Sound Pressure: The square-root of the mean squared pressure of a sound. It is usually the result of an RMS detector on a microphone signal.

Sound Exposure Level or Single Event Level (SEL): An L_{eq} (see Energy Equivalent Sound Level) referenced to a one-second duration. It provides a measure of the accumulation of sound energy over the duration of the event.

Sound Level (SL): The A-weighted Sound Pressure Level in dBA.

Sound Pressure Level (SPL): The logarithmic ratio of the RMS sound pressure to the sound pressure at the threshold of hearing. The sound pressure level is defined by equation (1) where P is the RMS pressure due to a sound and P_0 is the reference pressure. P_0 is usually taken as 2.0×10^{-5} Pascals.

$$(1) \quad SPL\ (dB) = 20 \log(P_{RMS}/P_0)$$

This logarithmic measure of sound is used simply to create a unit that can represent the audible range of sound in much more convenient format. Otherwise, audible sound pressure, expressed in Pascals, would range from 2.0×10^{-5} Pa to 200 Pa, a span of 10^7 Pa.

Sound Power Level (PWL): The logarithmic ratio of the instantaneous sound power (energy) of a noise source to that of an international standard reference power. The sound power level is defined by equation (2) where W is the sound power of the source in watts, and W_0 is the reference power of 10^{-12} watts.

$$\textbf{(2)} \quad PWL\ (dB) = 10 \log(W/W_0)$$

Interrelationships between sound pressure level (SPL) and sound power level (PWL) depend on the location and type of source.

APPENDIX C

```
STAMSON 5.0        NORMAL REPORT        Date: 02-04-2002 12:24:41
MINISTRY OF ENVIRONMENT AND ENERGY / NOISE ASSESSMENT

Filename: DNH1r.te             Time Period: 24 hours
Description:    Existing


Road data, segment # 1: DNS/W1
------------------------------
Car traffic volume  :  8236 veh/TimePeriod
Medium truck volume :   216 veh/TimePeriod
Heavy truck volume  :   649 veh/TimePeriod
Posted speed limit  :    60 km/h
Road gradient       :     0 %
Road pavement       :     1 (Typical asphalt or concrete)

Data for Segment # 1: DNS/W1
----------------------------
Angle1   Angle2           :  58.00 deg   68.00 deg
Wood depth                :      0       (No woods.)
No of house rows          :      0
Surface                   :      1       (Absorptive ground surface)
Receiver source distance  :  58.00 m
Receiver height           :   1.20 m
Topography                :      2       (Flat/gentle slope; with barrier)
Barrier angle1            :  58.00 deg   Angle2 : 68.00 deg
Barrier height            :   4.00 m
Barrier receiver distance :  15.00 m
Source elevation          : 232.50 m
Receiver elevation        : 231.00 m
Barrier elevation         : 231.00 m
Reference angle           :   0.00


Road data, segment # 2: DNS/W2
------------------------------
Car traffic volume  :  8236 veh/TimePeriod
Medium truck volume :   216 veh/TimePeriod
Heavy truck volume  :   649 veh/TimePeriod
Posted speed limit  :    50 km/h
Road gradient       :     0 %
Road pavement       :     1 (Typical asphalt or concrete)

Data for Segment # 2: DNS/W2
----------------------------
Angle1   Angle2           : -27.00 deg   0.00 deg
Wood depth                :      0       (No woods.)
No of house rows          :      0
Surface                   :      1       (Absorptive ground surface)
Receiver source distance  : 113.00 m
Receiver height           :   1.20 m
Topography                :      2       (Flat/gentle slope; with barrier)
Barrier angle1            : -27.00 deg   Angle2 : 0.00 deg
Barrier height            :   4.00 m
Barrier receiver distance :  62.00 m
Source elevation          : 233.00 m
Receiver elevation        : 231.00 m
Barrier elevation         : 231.00 m
Reference angle           :   0.00


Road data, segment # 3: DNS/W3
------------------------------
Car traffic volume  :  8236 veh/TimePeriod
Medium truck volume :   216 veh/TimePeriod
Heavy truck volume  :   649 veh/TimePeriod
Posted speed limit  :    50 km/h
Road gradient       :     0 %
Road pavement       :     1 (Typical asphalt or concrete)

Data for Segment # 3: DNS/W3
----------------------------
Angle1   Angle2           : -85.00 deg   -82.00 deg
Wood depth                :      0       (No woods.)
No of house rows          :      0
Surface                   :      1       (Absorptive ground surface)
Receiver source distance  :  26.00 m
Receiver height           :   1.20 m
Topography                :      2       (Flat/gentle slope; with barrier)
Barrier angle1            : -85.00 deg   Angle2 : -82.00 deg
Barrier height            :   4.00 m
Barrier receiver distance :   8.00 m
Source elevation          : 234.00 m
Receiver elevation        : 231.00 m
Barrier elevation         : 231.00 m
Reference angle           :   0.00


Road data, segment # 4: DNE/W-N1
--------------------------------
Car traffic volume  :  7059 veh/TimePeriod
Medium truck volume :   185 veh/TimePeriod
Heavy truck volume  :   556 veh/TimePeriod
Posted speed limit  :    50 km/h
Road gradient       :     0 %
Road pavement       :     1 (Typical asphalt or concrete)

Data for Segment # 4: DNE/W-N1
------------------------------
Angle1   Angle2           : -52.00 deg   12.00 deg
Wood depth                :      0       (No woods.)
No of house rows          :      0
Surface                   :      1       (Absorptive ground surface)
Receiver source distance  :  98.00 m
Receiver height           :   1.20 m
Topography                :      2       (Flat/gentle slope; with barrier)
Barrier angle1            : -52.00 deg   Angle2 : 12.00 deg
Barrier height            :   4.00 m
Barrier receiver distance :  67.00 m
Source elevation          : 231.50 m
Receiver elevation        : 231.00 m
Barrier elevation         : 231.00 m
Reference angle           :   0.00


Road data, segment # 5: DNE/W-N2
--------------------------------
Car traffic volume  :  7059 veh/TimePeriod
Medium truck volume :   185 veh/TimePeriod
Heavy truck volume  :   556 veh/TimePeriod
Posted speed limit  :    90 km/h
Road gradient       :     0 %
Road pavement       :     1 (Typical asphalt or concrete)

Data for Segment # 5: DNE/W-N2
------------------------------
Angle1   Angle2           : -43.00 deg   -24.00 deg
Wood depth                :      0       (No woods.)
No of house rows          :      0
Surface                   :      1       (Absorptive ground surface)
```

```
Receiver source distance  :  73.00 m
Receiver height           :   1.20 m
Topography                :      2       (Flat/gentle slope; with barrier)
Barrier angle1            : -43.00 deg   Angle2 : -24.00 deg
Barrier height            :   4.00 m
Barrier receiver distance :  39.00 m
Source elevation          : 231.50 m
Receiver elevation        : 231.00 m
Barrier elevation         : 231.00 m
Reference angle           :   0.00
```

Results segment # 1: DNS/W1

Source height = 1.63 m

Barrier height for grazing incidence

Source Height (m)	Receiver Height (m)	Barrier Height (m)	Elevation of Barrier Top (m)
1.63	1.20	1.70	232.70

ROAD (0.00 + 35.50 + 0.00) = 35.50 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
58	68	0.42	66.85	0.00	-8.37	-14.02	0.00	0.00	-8.96	35.50

Segment Leq : 35.50 dBA

Results segment # 2: DNS/W2

Source height = 1.63 m

Barrier height for grazing incidence

Source Height (m)	Receiver Height (m)	Barrier Height (m)	Elevation of Barrier Top (m)
1.63	1.20	2.54	233.54

ROAD (0.00 + 37.96 + 0.00) = 37.96 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
-27	0	0.42	65.46	0.00	-12.50	-8.31	0.00	0.00	-6.70	37.96

Segment Leq : 37.96 dBA

Results segment # 3: DNS/W3

Source height = 1.63 m

Barrier height for grazing incidence

Source Height (m)	Receiver Height (m)	Barrier Height (m)	Elevation of Barrier Top (m)
1.63	1.20	2.26	233.26

ROAD (0.00 + 33.85 + 0.00) = 33.85 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
-85	-82	0.42	65.46	0.00	-3.40	-21.81	0.00	0.00	-6.39	33.85

Segment Leq : 33.85 dBA

Results segment # 4: DNE/W-N1

Source height = 1.63 m

Barrier height for grazing incidence

Source Height (m)	Receiver Height (m)	Barrier Height (m)	Elevation of Barrier Top (m)
1.63	1.20	1.84	232.84

ROAD (0.00 + 39.75 + 0.00) = 39.75 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
-52	12	0.42	64.79	0.00	-11.62	-4.71	0.00	0.00	-8.71	39.75

Segment Leq : 39.75 dBA

Results segment # 5: DNE/W-N2

Source height = 1.63 m

Barrier height for grazing incidence

Source Height (m)	Receiver Height (m)	Barrier Height (m)	Elevation of Barrier Top (m)
1.63	1.20	1.70	232.70

ROAD (0.00 + 40.20 + 0.00) = 40.20 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
-43	-24	0.42	69.44	0.00	-9.79	-10.11	0.00	0.00	-9.33	40.20

Segment Leq : 40.20 dBA

Total Leq All Segments: 45.07 dBA

TOTAL Leq FROM ALL SOURCES: 45.07

```
STAMSON 5.0        NORMAL REPORT        Date: 02-04-2002 12:27:53
MINISTRY OF ENVIRONMENT AND ENERGY / NOISE ASSESSMENT

Filename: DNH1.te              Time Period: 24 hours
Description:   Existing


Road data, segment # 1: 113N
----------------------------
Car traffic volume  : 52354 veh/TimePeriod
Medium truck volume :  1371 veh/TimePeriod
Heavy truck volume  :  4125 veh/TimePeriod
Posted speed limit  :   100 km/h
Road gradient       :     0 %
Road pavement       :     1 (Typical asphalt or concrete)

Data for Segment # 1: 113N
--------------------------
Angle1   Angle2           : -76.00 deg   -35.00 deg
Wood depth                :      0       (No woods.)
No of house rows          :      0
Surface                   :      1       (Absorptive ground surface)
Receiver source distance  :  79.00 m
Receiver height           :   1.20 m
Topography                :      2       (Flat/gentle slope; with barrier)
Barrier angle1            : -76.00 deg   Angle2 : -35.00 deg
Barrier height            :   4.00 m
Barrier receiver distance :  28.00 m
Source elevation          : 232.00 m
Receiver elevation        : 231.00 m
Barrier elevation         : 231.00 m
Reference angle           :   0.00


Road data, segment # 2: 115N
----------------------------
Car traffic volume  : 52354 veh/TimePeriod
Medium truck volume :  1371 veh/TimePeriod
Heavy truck volume  :  4125 veh/TimePeriod
Posted speed limit  :   100 km/h
Road gradient       :     0 %
Road pavement       :     1 (Typical asphalt or concrete)

Data for Segment # 2: 115N
--------------------------
Angle1   Angle2           : -35.00 deg   41.00 deg
Wood depth                :      0       (No woods.)
No of house rows          :      0
Surface                   :      1       (Absorptive ground surface)
Receiver source distance  :  79.00 m
Receiver height           :   1.20 m
Topography                :      1       (Flat/gentle slope; no barrier)
Reference angle           :   0.00


Road data, segment # 3: 117N
----------------------------
Car traffic volume  : 52354 veh/TimePeriod
Medium truck volume :  1371 veh/TimePeriod
Heavy truck volume  :  4125 veh/TimePeriod
Posted speed limit  :   100 km/h
Road gradient       :     0 %
Road pavement       :     1 (Typical asphalt or concrete)

Data for Segment # 3: 117N
--------------------------
Angle1   Angle2           :  41.00 deg   75.00 deg
Wood depth                :      0       (No woods.)
No of house rows          :      0
Surface                   :      1       (Absorptive ground surface)
Receiver source distance  :  79.00 m
Receiver height           :   1.20 m
Topography                :      1       (Flat/gentle slope; no barrier)
Reference angle           :   0.00


Road data, segment # 4: 113S
----------------------------
Car traffic volume  : 52354 veh/TimePeriod
Medium truck volume :  1371 veh/TimePeriod
Heavy truck volume  :  4125 veh/TimePeriod
Posted speed limit  :   100 km/h
Road gradient       :     0 %
Road pavement       :     1 (Typical asphalt or concrete)

Data for Segment # 4: 113S
--------------------------
Angle1   Angle2           : -76.00 deg   -35.00 deg
Wood depth                :      0       (No woods.)
No of house rows          :      0
Surface                   :      1       (Absorptive ground surface)
Receiver source distance  :  93.00 m
Receiver height           :   1.20 m
Topography                :      2       (Flat/gentle slope; with barrier)
Barrier angle1            : -76.00 deg   Angle2 : -35.00 deg
Barrier height            :   4.00 m
Barrier receiver distance :  28.00 m
Source elevation          : 232.00 m
Receiver elevation        : 231.00 m
Barrier elevation         : 231.00 m
Reference angle           :   0.00


Road data, segment # 5: 115S
----------------------------
Car traffic volume  : 52354 veh/TimePeriod
Medium truck volume :  1371 veh/TimePeriod
Heavy truck volume  :  4125 veh/TimePeriod
Posted speed limit  :   100 km/h
Road gradient       :     0 %
Road pavement       :     1 (Typical asphalt or concrete)

Data for Segment # 5: 115S
--------------------------
Angle1   Angle2           : -35.00 deg   41.00 deg
Wood depth                :      0       (No woods.)
No of house rows          :      0
Surface                   :      1       (Absorptive ground surface)
Receiver source distance  :  93.00 m
Receiver height           :   1.20 m
Topography                :      1       (Flat/gentle slope; no barrier)
Reference angle           :   0.00

Road data, segment # 6: 117S
----------------------------
Car traffic volume  : 52354 veh/TimePeriod
Medium truck volume :  1371 veh/TimePeriod
Heavy truck volume  :  4125 veh/TimePeriod
Posted speed limit  :   100 km/h
Road gradient       :     1 %
```

Road pavement : 1 (Typical asphalt or concrete)

Data for Segment # 6: 117S

Angle1 Angle2	:	41.00 deg	75.00 deg
Wood depth	:	0	(No woods.)
No of house rows	:	0	
Surface	:	1	(Absorptive ground surface)
Receiver source distance	:	93.00 m	
Receiver height	:	1.20 m	
Topography	:	1	(Flat/gentle slope; no barrier)
Reference angle	:	0.00	

Results segment # 1: 113N

Source height = 1.63 m

Barrier height for grazing incidence

Source Height (m)	Receiver Height (m)	Barrier Height (m)	Elevation of Barrier Top (m)
1.63	1.20	1.71	232.71

ROAD (0.00 + 53.08 + 0.00) = 53.08 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
-76	-35	0.42	79.03	0.00	-10.28	-7.57	0.00	0.00	-8.10	53.08

Segment Leq : 53.08 dBA

Results segment # 2: 115N

Source height = 1.63 m

ROAD (0.00 + 63.09 + 0.00) = 63.09 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
-35	41	0.66	79.03	0.00	-11.98	-3.96	0.00	0.00	0.00	63.09

Segment Leq : 63.09 dBA

Results segment # 3: 117N

Source height = 1.63 m

ROAD (0.00 + 57.91 + 0.00) = 57.91 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
41	75	0.66	79.03	0.00	-11.98	-9.14	0.00	0.00	0.00	57.91

Segment Leq : 57.91 dBA

Results segment # 4: 113S

Source height = 1.63 m

Barrier height for grazing incidence

Source Height (m)	Receiver Height (m)	Barrier Height (m)	Elevation of Barrier Top (m)
1.63	1.20	1.63	232.63

ROAD (0.00 + 52.10 + 0.00) = 52.10 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
-76	-35	0.42	79.03	0.00	-11.29	-7.57	0.00	0.00	-8.06	52.10

Segment Leq : 52.10 dBA

Results segment # 5: 115S

Source height = 1.63 m

ROAD (0.00 + 61.91 + 0.00) = 61.91 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
-35	41	0.66	79.03	0.00	-13.15	-3.96	0.00	0.00	0.00	61.91

Segment Leq : 61.91 dBA

Results segment # 6: 117S

Source height = 1.63 m

ROAD (0.00 + 56.73 + 0.00) = 56.73 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
41	75	0.66	79.03	0.00	-13.15	-9.14	0.00	0.00	0.00	56.73

Segment Leq : 56.73 dBA

Total Leq All Segments: 67.03 dBA

TOTAL Leq FROM ALL SOURCES: 67.03

DNH1r.te	TOTAL Leq FROM ALL SOURCES:	45.07
DNH1.te	TOTAL Leq FROM ALL SOURCES:	67.03

TOTAL Leq FOR RECEPTOR DNH1 (Existing) = 67.06

STAMSON 5.0 NORMAL REPORT Date: 02-04-2002 12:12:04
MINISTRY OF ENVIRONMENT AND ENERGY / NOISE ASSESSMENT

Filename: BAH2.te Time Period: 24 hours
Description: Existing

Road data, segment # 1: 125N

```
Car traffic volume  : 52354 veh/TimePeriod
Medium truck volume :  1371 veh/TimePeriod
Heavy truck volume  :  4125 veh/TimePeriod
Posted speed limit  :   100 km/h
Road gradient       :     2 %
Road pavement       :     1 (Typical asphalt or concrete)
```

Data for Segment # 1: 125N

```
Angle1   Angle2            : -90.00 deg   90.00 deg
Wood depth                 :      0       (No woods.)
No of house rows           :      0
Surface                    :      1       (Absorptive ground surface)
Receiver source distance   :  84.00 m
Receiver height            :   1.20 m
Topography                 :      2       (Flat/gentle slope; with barrier)
Barrier angle1             : -50.00 deg   Angle2 : 34.00 deg
Barrier height             :   4.00 m
Barrier receiver distance  :  12.00 m
Source elevation           : 249.00 m
Receiver elevation         : 255.00 m
Barrier elevation          : 255.00 m
Reference angle            :   0.00
```

Road data, segment # 2: 125S

```
Car traffic volume  : 52354 veh/TimePeriod
Medium truck volume :  1371 veh/TimePeriod
Heavy truck volume  :  4125 veh/TimePeriod
Posted speed limit  :   100 km/h
Road gradient       :     0 %
Road pavement       :     1 (Typical asphalt or concrete)
```

Data for Segment # 2: 125S

```
Angle1   Angle2            : -90.00 deg   90.00 deg
Wood depth                 :      0       (No woods.)
No of house rows           :      0
Surface                    :      1       (Absorptive ground surface)
Receiver source distance   :  70.00 m
Receiver height            :   1.20 m
Topography                 :      2       (Flat/gentle slope; with barrier)
Barrier angle1             : -50.00 deg   Angle2 : 34.00 deg
Barrier height             :   4.00 m
Barrier receiver distance  :  12.00 m
Source elevation           : 249.00 m
Receiver elevation         : 255.00 m
Barrier elevation          : 255.00 m
Reference angle            :   0.00
```

Results segment # 1: 125N

Source height = 1.63 m

Barrier height for grazing incidence

Source Height (m)	Receiver Height (m)	Barrier Height (m)	Elevation of Barrier Top (m)
1.63	1.20	0.40	255.40

ROAD (57.88 + 50.90 + 60.20) = 62.51 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
-90	-50	0.66	80.17	0.00	-12.42	-9.87	0.00	0.00	0.00	57.88
-50	34	0.42	80.17	0.00	-10.66	-3.50	0.00	0.00	-15.11	50.90
34	90	0.66	80.17	0.00	-12.42	-7.55	0.00	0.00	0.00	60.20

Segment Leq : 62.51 dBA

Results segment # 2: 125S

Source height = 1.63 m

Barrier height for grazing incidence

Source Height (m)	Receiver Height (m)	Barrier Height (m)	Elevation of Barrier Top (m)
1.63	1.20	0.25	255.25

ROAD (58.05 + 50.37 + 60.38) = 62.64 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
-90	-50	0.66	79.03	0.00	-11.11	-9.87	0.00	0.00	0.00	58.05
-50	34	0.42	79.03	0.00	-9.53	-3.50	0.00	0.00	-15.62	50.37
34	90	0.66	79.03	0.00	-11.11	-7.55	0.00	0.00	0.00	60.38

Segment Leq : 62.64 dBA

Total Leq All Segments: 65.59 dBA

TOTAL Leq FROM ALL SOURCES: 65.59

```
STAMSON 5.0        NORMAL REPORT        Date: 02-04-2002 14:10:00
MINISTRY OF ENVIRONMENT AND ENERGY / NOISE ASSESSMENT

Filename: DNH1.te              Time Period: 24 hours
Description: Future "Build"


Road data, segment # 1: 113N-3
------------------------------
Car traffic volume  : 45884 veh/TimePeriod
Medium truck volume :  1202 veh/TimePeriod
Heavy truck volume  :  3615 veh/TimePeriod
Posted speed limit  :   100 km/h
Road gradient       :     0 %
Road pavement       :     1 (Typical asphalt or concrete)

Data for Segment # 1: 113N-3
----------------------------
Angle1   Angle2           : -76.00 deg   -35.00 deg
Wood depth                :      0       (No woods.)
No of house rows          :      0
Surface                   :      1       (Absorptive ground surface)
Receiver source distance  :  80.00 m
Receiver height           :   1.20 m
Topography                :      1       (Flat/gentle slope; no barrier)
Reference angle           :   0.00


Road data, segment # 2: 113N-2
------------------------------
Car traffic volume  : 30589 veh/TimePeriod
Medium truck volume :   801 veh/TimePeriod
Heavy truck volume  :  2410 veh/TimePeriod
Posted speed limit  :   100 km/h
Road gradient       :     0 %
Road pavement       :     1 (Typical asphalt or concrete)

Data for Segment # 2: 113N-2
----------------------------
Angle1   Angle2           : -76.00 deg   -35.00 deg
Wood depth                :      0       (No woods.)
No of house rows          :      0
Surface                   :      1       (Absorptive ground surface)
Receiver source distance  :  72.00 m
Receiver height           :   1.20 m
Topography                :      1       (Flat/gentle slope; no barrier)
Reference angle           :   0.00

Road data, segment # 3: 115N-3
------------------------------
Car traffic volume  : 45884 veh/TimePeriod
Medium truck volume :  1202 veh/TimePeriod
Heavy truck volume  :  3615 veh/TimePeriod
Posted speed limit  :   100 km/h
Road gradient       :     0 %
Road pavement       :     1 (Typical asphalt or concrete)

Data for Segment # 3: 115N-3
----------------------------
Angle1   Angle2           : -35.00 deg   41.00 deg
Wood depth                :      0       (No woods.)
No of house rows          :      0
Surface                   :      1       (Absorptive ground surface)
Receiver source distance  :  80.00 m
Receiver height           :   1.20 m
Topography                :      1       (Flat/gentle slope; no barrier)
Reference angle           :   0.00


Road data, segment # 4: 115N-2
------------------------------
Car traffic volume  : 30589 veh/TimePeriod
Medium truck volume :   801 veh/TimePeriod
Heavy truck volume  :  2410 veh/TimePeriod
Posted speed limit  :   100 km/h
Road gradient       :     0 %
Road pavement       :     1 (Typical asphalt or concrete)

Data for Segment # 4: 115N-2
----------------------------
Angle1   Angle2           : -35.00 deg   41.00 deg
Wood depth                :      0       (No woods.)
No of house rows          :      0
Surface                   :      1       (Absorptive ground surface)
Receiver source distance  :  72.00 m
Receiver height           :   1.20 m
Topography                :      1       (Flat/gentle slope; no barrier)
Reference angle           :   0.00


Road data, segment # 5: 117N-3
------------------------------
Car traffic volume  : 45884 veh/TimePeriod
Medium truck volume :  1202 veh/TimePeriod
Heavy truck volume  :  3615 veh/TimePeriod
Posted speed limit  :   100 km/h
Road gradient       :     0 %
Road pavement       :     1 (Typical asphalt or concrete)

Data for Segment # 5: 117N-3
----------------------------
Angle1   Angle2           :  41.00 deg   75.00 deg
Wood depth                :      0       (No woods.)
No of house rows          :      0
Surface                   :      1       (Absorptive ground surface)
Receiver source distance  :  80.00 m
Receiver height           :   1.20 m
Topography                :      1       (Flat/gentle slope; no barrier)
Reference angle           :   0.00


Road data, segment # 6: 117N-2
------------------------------
Car traffic volume  : 30589 veh/TimePeriod
Medium truck volume :   801 veh/TimePeriod
Heavy truck volume  :  2410 veh/TimePeriod
Posted speed limit  :   100 km/h
Road gradient       :     0 %
Road pavement       :     1 (Typical asphalt or concrete)

Data for Segment # 6: 117N-2
----------------------------
Angle1   Angle2           :  41.00 deg   75.00 deg
Wood depth                :      0       (No woods.)
No of house rows          :      0
Surface                   :      1       (Absorptive ground surface)
Receiver source distance  :  72.00 m
Receiver height           :   1.20 m
Topography                :      1       (Flat/gentle slope; no barrier)
Reference angle           :   0.00
```

Road data, segment # 7: 113S-3

Car traffic volume : 45884 veh/TimePeriod
Medium truck volume : 1202 veh/TimePeriod
Heavy truck volume : 3615 veh/TimePeriod
Posted speed limit : 100 km/h
Road gradient : 0 %
Road pavement : 1 (Typical asphalt or concrete)

Data for Segment # 7: 113S-3

Angle1 Angle2 : -76.00 deg -35.00 deg
Wood depth : 0 (No woods.)
No of house rows : 0
Surface : 1 (Absorptive ground surface)
Receiver source distance : 102.00 m
Receiver height : 1.20 m
Topography : 1 (Flat/gentle slope; no barrier)
Reference angle : 0.00

Road data, segment # 8: 113S-2

Car traffic volume : 30589 veh/TimePeriod
Medium truck volume : 801 veh/TimePeriod
Heavy truck volume : 2410 veh/TimePeriod
Posted speed limit : 100 km/h
Road gradient : 0 %
Road pavement : 1 (Typical asphalt or concrete)

Data for Segment # 8: 113S-2

Angle1 Angle2 : -76.00 deg -35.00 deg
Wood depth : 0 (No woods.)
No of house rows : 0
Surface : 1 (Absorptive ground surface)
Receiver source distance : 110.00 m
Receiver height : 1.20 m
Topography : 1 (Flat/gentle slope; no barrier)
Reference angle : 0.00

Results segment # 1: 113N-3

Source height = 1.63 m

ROAD (0.00 + 58.22 + 0.00) = 58.22 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
-76	-35	0.66	78.46	0.00	-12.07	-8.16	0.00	0.00	0.00	58.22

Segment Leq : 58.22 dBA

Results segment # 2: 113N-2

Source height = 1.63 m

ROAD (0.00 + 57.22 + 0.00) = 57.22 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
-76	-35	0.66	76.69	0.00	-11.31	-8.16	0.00	0.00	0.00	57.22

Segment Leq : 57.22 dBA

Results segment # 3: 115N-3

Source height = 1.63 m

ROAD (0.00 + 62.42 + 0.00) = 62.42 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
-35	41	0.66	78.46	0.00	-12.07	-3.96	0.00	0.00	0.00	62.42

Segment Leq : 62.42 dBA

Results segment # 4: 115N-2

Source height = 1.63 m

ROAD (0.00 + 61.42 + 0.00) = 61.42 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
-35	41	0.66	76.69	0.00	-11.31	-3.96	0.00	0.00	0.00	61.42

Segment Leq : 61.42 dBA

Results segment # 5: 117N-3

Source height = 1.63 m

ROAD (0.00 + 57.25 + 0.00) = 57.25 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
41	75	0.66	78.46	0.00	-12.07	-9.14	0.00	0.00	0.00	57.25

Segment Leq : 57.25 dBA

Results segment # 6: 117N-2

Source height = 1.63 m

ROAD (0.00 + 56.25 + 0.00) = 56.25 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
41	75	0.66	76.69	0.00	-11.31	-9.14	0.00	0.00	0.00	56.25

Segment Leq : 56.25 dBA

Results segment # 7: 113S-3

Source height = 1.63 m

ROAD (0.00 + 56.47 + 0.00) = 56.47 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
-76	-35	0.66	78.46	0.00	-13.82	-8.16	0.00	0.00	0.00	56.47

Segment Leq : 56.47 dBA

Results segment # 8: 113S-2

Source height = 1.63 m

ROAD (0.00 + 54.17 + 0.00) = 54.17 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
-76	-35	0.66	76.69	0.00	-14.36	-8.16	0.00	0.00	0.00	54.17

Segment Leq : 54.17 dBA

Total Leq All Segments: 67.77 dBA

TOTAL Leq FROM ALL SOURCES: 67.77

STAMSON 5.0 NORMAL REPORT Date: 02-04-2002 14:10:01
MINISTRY OF ENVIRONMENT AND ENERGY / NOISE ASSESSMENT

Filename: DNH1B.te Time Period: 24 hours
Description: Future "Build"

Road data, segment # 1: DNS/W1055

Car traffic volume : 11403 veh/TimePeriod
Medium truck volume : 299 veh/TimePeriod
Heavy truck volume : 898 veh/TimePeriod
Posted speed limit : 80 km/h
Road gradient : 0 %
Road pavement : 1 (Typical asphalt or concrete)

Data for Segment # 1: DNS/W1055

Angle1 Angle2 : -82.00 deg -62.00 deg
Wood depth : 0 (No woods.)
No of house rows : 0
Surface : 1 (Absorptive ground surface)
Receiver source distance : 53.00 m
Receiver height : 1.20 m
Topography : 1 (Flat/gentle slope; no barrier)
Reference angle : 0.00

Road data, segment # 2: DNS/W1067

Car traffic volume : 11403 veh/TimePeriod
Medium truck volume : 299 veh/TimePeriod
Heavy truck volume : 898 veh/TimePeriod
Posted speed limit : 60 km/h
Road gradient : 0 %
Road pavement : 1 (Typical asphalt or concrete)

Data for Segment # 2: DNS/W1067

Angle1 Angle2 : 63.00 deg 73.00 deg
Wood depth : 0 (No woods.)
No of house rows : 0
Surface : 1 (Absorptive ground surface)
Receiver source distance : 29.00 m
Receiver height : 1.20 m
Topography : 1 (Flat/gentle slope; no barrier)
Reference angle : 0.00

Road data, segment # 3: DNS/W1072

Car traffic volume : 11403 veh/TimePeriod
Medium truck volume : 299 veh/TimePeriod
Heavy truck volume : 898 veh/TimePeriod
Posted speed limit : 60 km/h
Road gradient : 0 %
Road pavement : 1 (Typical asphalt or concrete)

Data for Segment # 3: DNS/W1072

Angle1 Angle2 : -13.00 deg 29.00 deg
Wood depth : 0 (No woods.)
No of house rows : 0
Surface : 1 (Absorptive ground surface)
Receiver source distance : 58.00 m

```
Receiver height           :   1.20 m
Topography                :      1       (Flat/gentle slope; no barrier)
Reference angle           :   0.00


Road data, segment # 4: DNS/W1077
---------------------------------
Car traffic volume  : 11403 veh/TimePeriod
Medium truck volume :   299 veh/TimePeriod
Heavy truck volume  :   898 veh/TimePeriod
Posted speed limit  :    60 km/h
Road gradient       :     0 %
Road pavement       :     1 (Typical asphalt or concrete)

Data for Segment # 4: DNS/W1077
-------------------------------
Angle1   Angle2           : -71.00 deg   -55.00 deg
Wood depth                :      0       (No woods.)
No of house rows          :      0
Surface                   :      1       (Absorptive ground surface)
Receiver source distance  :  35.00 m
Receiver height           :   1.20 m
Topography                :      1       (Flat/gentle slope; no barrier)
Reference angle           :   0.00


Road data, segment # 5: DNS/W1082
---------------------------------
Car traffic volume  : 11403 veh/TimePeriod
Medium truck volume :   299 veh/TimePeriod
Heavy truck volume  :   898 veh/TimePeriod
Posted speed limit  :    60 km/h
Road gradient       :     0 %
Road pavement       :     1 (Typical asphalt or concrete)

Data for Segment # 5: DNS/W1082
-------------------------------
Angle1   Angle2           :  64.00 deg   75.00 deg
Wood depth                :      0       (No woods.)
No of house rows          :      0
Surface                   :      1       (Absorptive ground surface)
Receiver source distance  :  46.00 m
Receiver height           :   1.20 m
Topography                :      1       (Flat/gentle slope; no barrier)
Reference angle           :   0.00


Road data, segment # 6: DNE/W-N101
----------------------------------
Car traffic volume  :  9774 veh/TimePeriod  *
Medium truck volume :   256 veh/TimePeriod  *
Heavy truck volume  :   770 veh/TimePeriod  *
Posted speed limit  :    60 km/h
Road gradient       :     0 %
Road pavement       :     1 (Typical asphalt or concrete)

Data for Segment # 6: DNE/W-N101
--------------------------------
Angle1   Angle2           : -63.00 deg   -14.00 deg
Wood depth                :      0       (No woods.)
No of house rows          :      0
Surface                   :      1       (Absorptive ground surface)
Receiver source distance  :  68.00 m
Receiver height           :   1.20 m
Topography                :      2       (Flat/gentle slope; with barrier)
Barrier angle1            : -63.00 deg   Angle2 : -14.00 deg
Barrier height            :   4.00 m
Barrier receiver distance :   4.00 m
Source elevation          : 230.40 m
Receiver elevation        : 231.00 m
Barrier elevation         : 231.00 m
Reference angle           :   0.00


Road data, segment # 7: DNE/W-N102
----------------------------------
Car traffic volume  :  9774 veh/TimePeriod  *
Medium truck volume :   256 veh/TimePeriod  *
Heavy truck volume  :   770 veh/TimePeriod  *
Posted speed limit  :    60 km/h
Road gradient       :     3 %
Road pavement       :     1 (Typical asphalt or concrete)

Data for Segment # 7: DNE/W-N102
--------------------------------
Angle1   Angle2           : -82.00 deg   45.00 deg
Wood depth                :      0       (No woods.)
No of house rows          :      0
Surface                   :      1       (Absorptive ground surface)
Receiver source distance  :  28.00 m
Receiver height           :   1.20 m
Topography                :      1       (Flat/gentle slope; no barrier)
Reference angle           :   0.00


Road data, segment # 8: DNE/W-N103
----------------------------------
Car traffic volume  :  9774 veh/TimePeriod  *
Medium truck volume :   256 veh/TimePeriod  *
Heavy truck volume  :   770 veh/TimePeriod  *
Posted speed limit  :    80 km/h
Road gradient       :     1 %
Road pavement       :     1 (Typical asphalt or concrete)

Data for Segment # 8: DNE/W-N103
--------------------------------
Angle1   Angle2           :   3.00 deg   73.00 deg
Wood depth                :      0       (No woods.)
No of house rows          :      0
Surface                   :      1       (Absorptive ground surface)
Receiver source distance  :  42.00 m
Receiver height           :   1.20 m
Topography                :      1       (Flat/gentle slope; no barrier)
Reference angle           :   0.00


Results segment # 1: DNS/W1055
------------------------------

Source height = 1.63 m

ROAD (0.00 + 48.48 + 0.00) = 48.48 dBA
Angle1 Angle2  Alpha RefLeq  P.Adj  D.Adj  F.Adj  W.Adj  H.Adj  B.Adj SubLeq
--------------------------------------------------------------------------
   -82    -62   0.66  70.55   0.00  -9.10 -12.97   0.00   0.00   0.00  48.48
--------------------------------------------------------------------------

Segment Leq : 48.48 dBA
```

Results segment # 2: DNS/W1067

Source height = 1.63 m

ROAD (0.00 + 48.13 + 0.00) = 48.13 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
63	73	0.66	68.26	0.00	-4.75	-15.38	0.00	0.00	0.00	48.13

Segment Leq : 48.13 dBA

Results segment # 3: DNS/W1072

Source height = 1.63 m

ROAD (0.00 + 52.10 + 0.00) = 52.10 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
-13	29	0.66	68.26	0.00	-9.75	-6.41	0.00	0.00	0.00	52.10

Segment Leq : 52.10 dBA

Results segment # 4: DNS/W1077

Source height = 1.63 m

ROAD (0.00 + 49.36 + 0.00) = 49.36 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
-71	-55	0.66	68.26	0.00	-6.11	-12.80	0.00	0.00	0.00	49.36

Segment Leq : 49.36 dBA

Results segment # 5: DNS/W1082

Source height = 1.63 m

ROAD (0.00 + 45.02 + 0.00) = 45.02 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
64	75	0.66	68.26	0.00	-8.08	-15.16	0.00	0.00	0.00	45.02

Segment Leq : 45.02 dBA

Results segment # 6: DNE/W-N101

Source height = 1.63 m

Barrier height for grazing incidence

Source Height (m)	Receiver Height (m)	Barrier Height (m)	Elevation of Barrier Top (m)
1.63	1.20	1.19	232.19

ROAD (0.00 + 36.02 + 0.00) = 36.02 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
-63	-14	0.42	67.59	0.00	-9.35	-6.18	0.00	0.00	-16.03	36.02

Segment Leq : 36.02 dBA

Results segment # 7: DNE/W-N102

Source height = 1.63 m

ROAD (0.00 + 62.12 + 0.00) = 62.12 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
-82	45	0.66	68.96	0.00	-4.50	-2.34	0.00	0.00	0.00	62.12

Segment Leq : 62.12 dBA

Results segment # 8: DNE/W-N103

Source height = 1.63 m

ROAD (0.00 + 57.45 + 0.00) = 57.45 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
3	73	0.66	69.88	0.00	-7.42	-5.01	0.00	0.00	0.00	57.45

Segment Leq : 57.45 dBA

Total Leq All Segments: 64.16 dBA

TOTAL Leq FROM ALL SOURCES: 64.16

```
STAMSON 5.0        NORMAL REPORT        Date: 02-04-2002 13:49:05
MINISTRY OF ENVIRONMENT AND ENERGY / NOISE ASSESSMENT

Filename: DNH1C.te              Time Period: 24 hours
Description:  Future "Build"


Road data, segment # 1: 115S-3
------------------------------
Car traffic volume  : 45884 veh/TimePeriod
Medium truck volume :  1202 veh/TimePeriod
Heavy truck volume  :  3615 veh/TimePeriod
Posted speed limit  :   100 km/h
Road gradient       :     0 %
Road pavement       :     1 (Typical asphalt or concrete)

Data for Segment # 1: 115S-3
----------------------------
Angle1   Angle2           : -35.00 deg   45.00 deg
Wood depth                :      0       (No woods.)
No of house rows          :      0
Surface                   :      1       (Absorptive ground surface)
Receiver source distance  : 102.00 m
Receiver height           :   1.20 m
Topography                :      1       (Flat/gentle slope; no barrier)
Reference angle           :   0.00


Road data, segment # 2: 115S-2
------------------------------
Car traffic volume  : 30589 veh/TimePeriod
Medium truck volume :   801 veh/TimePeriod
Heavy truck volume  :  2410 veh/TimePeriod
Posted speed limit  :   100 km/h
Road gradient       :     0 %
Road pavement       :     1 (Typical asphalt or concrete)

Data for Segment # 2: 115S-2
----------------------------
Angle1   Angle2           : -35.00 deg   45.00 deg
Wood depth                :      0       (No woods.)
No of house rows          :      0
Surface                   :      1       (Absorptive ground surface)
Receiver source distance  : 110.00 m
Receiver height           :   1.20 m
Topography                :      1       (Flat/gentle slope; no barrier)
Reference angle           :   0.00


Road data, segment # 3: 117S-3
------------------------------
Car traffic volume  : 45884 veh/TimePeriod
Medium truck volume :  1202 veh/TimePeriod
Heavy truck volume  :  3615 veh/TimePeriod
Posted speed limit  :   100 km/h
Road gradient       :     1 %
Road pavement       :     1 (Typical asphalt or concrete)

Data for Segment # 3: 117S-3
----------------------------
Angle1   Angle2           :  41.00 deg   75.00 deg
Wood depth                :      0       (No woods.)
No of house rows          :      0
Surface                   :      1       (Absorptive ground surface)
Receiver source distance  : 102.00 m
Receiver height           :   1.20 m
Topography                :      1       (Flat/gentle slope; no barrier)
Reference angle           :   0.00


Road data, segment # 4: 117S-2
------------------------------
Car traffic volume  : 30589 veh/TimePeriod
Medium truck volume :   801 veh/TimePeriod
Heavy truck volume  :  2410 veh/TimePeriod
Posted speed limit  :   100 km/h
Road gradient       :     1 %
Road pavement       :     1 (Typical asphalt or concrete)

Data for Segment # 4: 117S-2
----------------------------
Angle1   Angle2           :  41.00 deg   75.00 deg
Wood depth                :      0       (No woods.)
No of house rows          :      0
Surface                   :      1       (Absorptive ground surface)
Receiver source distance  : 110.00 m
Receiver height           :   1.20 m
Topography                :      1       (Flat/gentle slope; no barrier)
Reference angle           :   0.00


Results segment # 1: 115S-3
---------------------------

Source height = 1.63 m

ROAD (0.00 + 60.86 + 0.00) = 60.86 dBA
Angle1 Angle2  Alpha RefLeq  P.Adj  D.Adj  F.Adj  W.Adj  H.Adj  B.Adj SubLeq
--------------------------------------------------------------------------------
   -35     45   0.66  78.46   0.00 -13.82  -3.77   0.00   0.00   0.00  60.86
--------------------------------------------------------------------------------

Segment Leq : 60.86 dBA


Results segment # 2: 115S-2
---------------------------

Source height = 1.63 m

ROAD (0.00 + 58.56 + 0.00) = 58.56 dBA
Angle1 Angle2  Alpha RefLeq  P.Adj  D.Adj  F.Adj  W.Adj  H.Adj  B.Adj SubLeq
--------------------------------------------------------------------------------
   -35     45   0.66  76.69   0.00 -14.36  -3.77   0.00   0.00   0.00  58.56
--------------------------------------------------------------------------------

Segment Leq : 58.56 dBA


Results segment # 3: 117S-3
---------------------------

Source height = 1.63 m

ROAD (0.00 + 55.50 + 0.00) = 55.50 dBA
Angle1 Angle2  Alpha RefLeq  P.Adj  D.Adj  F.Adj  W.Adj  H.Adj  B.Adj SubLeq
--------------------------------------------------------------------------------
    41     75   0.66  78.46   0.00 -13.82  -9.14   0.00   0.00   0.00  55.50
--------------------------------------------------------------------------------
```

Segment Leq : 55.50 dBA

Results segment # 4: 117S-2

Source height = 1.63 m

ROAD (0.00 + 53.19 + 0.00) = 53.19 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
41	75	0.66	76.69	0.00	-14.36	-9.14	0.00	0.00	0.00	53.19

Segment Leq : 53.19 dBA

Total Leq All Segments: 63.98 dBA

TOTAL Leq FROM ALL SOURCES: 63.98

DNH1.te	TOTAL Leq FROM ALL SOURCES:	67.77
DNH1B.te	TOTAL Leq FROM ALL SOURCES:	64.16
DNH1C.te	TOTAL Leq FROM ALL SOURCES:	63.98

TOTAL Leq FOR RECEPTOR DNH1 (Future Build) = 70.45

STAMSON 5.0 NORMAL REPORT Date: 02-04-2002 13:51:44
MINISTRY OF ENVIRONMENT AND ENERGY / NOISE ASSESSMENT

Filename: BAH2.te Time Period: 24 hours
Description: Future "Build"

Road data, segment # 1: 125N-3

Car traffic volume : 45884 veh/TimePeriod
Medium truck volume : 1202 veh/TimePeriod
Heavy truck volume : 3615 veh/TimePeriod
Posted speed limit : 100 km/h
Road gradient : 2 %
Road pavement : 1 (Typical asphalt or concrete)

Data for Segment # 1: 125N-3

Angle1 Angle2 : -90.00 deg 90.00 deg
Wood depth : 0 (No woods.)
No of house rows : 0
Surface : 1 (Absorptive ground surface)
Receiver source distance : 86.00 m
Receiver height : 1.20 m
Topography : 1 (Flat/gentle slope; no barrier)
Reference angle : 0.00

Road data, segment # 2: 125N-2

Car traffic volume : 30589 veh/TimePeriod
Medium truck volume : 801 veh/TimePeriod
Heavy truck volume : 2410 veh/TimePeriod
Posted speed limit : 100 km/h
Road gradient : 2 %
Road pavement : 1 (Typical asphalt or concrete)

Data for Segment # 2: 125N-2

Angle1 Angle2 : -90.00 deg 90.00 deg
Wood depth : 0 (No woods.)
No of house rows : 0
Surface : 1 (Absorptive ground surface)
Receiver source distance : 96.00 m
Receiver height : 1.20 m
Topography : 1 (Flat/gentle slope; no barrier)
Reference angle : 0.00

Road data, segment # 3: 125S-3

Car traffic volume : 45884 veh/TimePeriod
Medium truck volume : 1202 veh/TimePeriod
Heavy truck volume : 3615 veh/TimePeriod
Posted speed limit : 100 km/h
Road gradient : 0 %
Road pavement : 1 (Typical asphalt or concrete)

Data for Segment # 3: 125S-3

Angle1 Angle2 : -90.00 deg 90.00 deg
Wood depth : 0 (No woods.)
No of house rows : 0
Surface : 1 (Absorptive ground surface)
Receiver source distance : 68.00 m

```
Receiver height           :   1.20 m
Topography                :      1       (Flat/gentle slope; no barrier)
Reference angle           :   0.00


Road data, segment # 4: 125S-2
------------------------------
Car traffic volume  : 30589 veh/TimePeriod
Medium truck volume :   801 veh/TimePeriod
Heavy truck volume  :  2410 veh/TimePeriod
Posted speed limit  :   100 km/h
Road gradient       :     0 %
Road pavement       :     1 (Typical asphalt or concrete)

Data for Segment # 4: 125S-2
----------------------------
Angle1   Angle2           : -90.00 deg   90.00 deg
Wood depth                :      0       (No woods.)
No of house rows          :      0
Surface                   :      1       (Absorptive ground surface)
Receiver source distance  :  58.00 m
Receiver height           :   1.20 m
Topography                :      1       (Flat/gentle slope; no barrier)
Reference angle           :   0.00


Results segment # 1: 125N-3
---------------------------

Source height = 1.63 m

ROAD (0.00 + 65.55 + 0.00) = 65.55 dBA
Angle1 Angle2  Alpha RefLeq  P.Adj  D.Adj  F.Adj  W.Adj  H.Adj  B.Adj SubLeq
--------------------------------------------------------------------------
   -90     90   0.66  79.60   0.00 -12.59  -1.46   0.00   0.00   0.00  65.55
--------------------------------------------------------------------------

Segment Leq : 65.55 dBA


Results segment # 2: 125N-2
---------------------------

Source height = 1.63 m

ROAD (0.00 + 63.00 + 0.00) = 63.00 dBA
Angle1 Angle2  Alpha RefLeq  P.Adj  D.Adj  F.Adj  W.Adj  H.Adj  B.Adj SubLeq
--------------------------------------------------------------------------
   -90     90   0.66  77.83   0.00 -13.38  -1.46   0.00   0.00   0.00  63.00
--------------------------------------------------------------------------

Segment Leq : 63.00 dBA


Results segment # 3: 125S-3
---------------------------

Source height = 1.63 m

ROAD (0.00 + 66.10 + 0.00) = 66.10 dBA
Angle1 Angle2  Alpha RefLeq  P.Adj  D.Adj  F.Adj  W.Adj  H.Adj  B.Adj SubLeq
--------------------------------------------------------------------------
   -90     90   0.66  78.46   0.00 -10.90  -1.46   0.00   0.00   0.00  66.10
--------------------------------------------------------------------------

Segment Leq : 66.10 dBA


Results segment # 4: 125S-2
---------------------------

Source height = 1.63 m

ROAD (0.00 + 65.49 + 0.00) = 65.49 dBA
Angle1 Angle2  Alpha RefLeq  P.Adj  D.Adj  F.Adj  W.Adj  H.Adj  B.Adj SubLeq
--------------------------------------------------------------------------
   -90     90   0.66  76.69   0.00  -9.75  -1.46   0.00   0.00   0.00  65.49
--------------------------------------------------------------------------

Segment Leq : 65.49 dBA

Total Leq All Segments: 71.21 dBA


TOTAL Leq FROM ALL SOURCES:       71.21
```

APPENDIX D

Hwy 400 (89 to 11)
04/25/02
11:38:25 AM

PEAK HR DATA

Receptor **DKH2**

Scenario Construction Noise on Highway 400

Receiver Elevation 245.00
Receiver Ht. 1.20

SOURCE SOURCES	Ref. SEL (dBA)	SEL SPL LEQ PWL[Q]	Ref Dis. (m)	No. of Events (#/hr)	Event Duration (min/hr)	Receptor S-R (m)	Dis Adj (dBA)	Event Adj (dBA)	Time Adj (dBA)	SOURCE Ht.	SOURCE Elev.	BARRIER Ht	BARRIER Elev	B-R Dis. (m)	SCREENING Barrier (dBA)	SCREENING Other (dBA)	Use Source (Y/N)	Leq(1) (dBA)
Roadway Construction - Typical Noise Sources						51.0												
Grader	85.0	SPL	15	1	60	36.0	-7.6	0.0	0.0	3.0	230.0	0.0	0.0		0.0		Y	77.4
Bulldozer	85.0	SPL	15	1	60	50.6	-10.6	0.0	0.0	3.0	230.0	0.0	0.0		0.0		Y	74.4
Moving Dump Truck	88.0	SPL	15	1	30	52.4	-10.9	0.0	-3.0	3.0	230.0	0.0	0.0		0.0		Y	74.1
Idling Dump Truck 1	65.0	SPL	15	1	60	66.6	-12.9	0.0	0.0	3.0	230.0	0.0	0.0		0.0		Y	52.1
Idling Dump Truck 2	65.0	SPL	15	1	60	67.9	-13.1	0.0	0.0	3.0	230.0	0.0	0.0		0.0		Y	51.9
Idling Dump Truck 3	65.0	SPL	15	1	60	87.0	-15.3	0.0	0.0	3.0	230.0	0.0	0.0		0.0		Y	49.7
Roller	74.0	SPL	15	1	60	83.2	-14.9	0.0	0.0	3.0	230.0	0.0	0.0		0.0		Y	59.1
Compressor 1	81.0	SPL	15	1	60	109.3	-17.2	0.0	0.0	2.5	230.0	0.0	0.0		0.0		Y	63.8
Compressor 2	81.0	SPL	15	1	60	110.1	-17.3	0.0	0.0	2.5	230.0	0.0	0.0		0.0		Y	63.7

TOTAL STATNRY LEQ(1) **81**

PEAK HR DATA

Receptor BAH4

Scenario Construction Noise on Highway 400 - Pile driving only

Receiver Elevation 271.00
Receiver Ht. 1.20

SOURCE SOURCES	Ref. SEL (dBA)	SEL SPL LEQ PWL[Q]	Ref Dis. (m)	No. of Events (#/hr)	Event Duration (min/hr)	Receptor S-R (m)	Dis Adj (dBA)	Event Adj (dBA)	Time Adj (dBA)	SOURCE Ht.	SOURCE Elev.	BARRIER Ht	BARRIER Elev	B-R Dis. (m)	SCREENING Barrier (dBA)	SCREENING Other (dBA)	Use Source (Y/N)	Leq(1) (dBA)
Roadway Construction - Typical Noise Sources						101.5												
Grader	85.0	SPL	15.0	1.0	60	86.5	-15.2	0.0	0.0	3.0	259.0				0.0		N	0.0
Bulldozer	85.0	SPL	15.0	1.0	60	97.8	-16.3	0.0	0.0	3.0	259.0				0.0		N	0.0
Moving Dump Truck	88.0	SPL	15.0	1.0	30	99.7	-16.5	0.0	-3.0	3.0	259.0				0.0		N	0.0
Idling Dump Truck 1	65.0	SPL	15.0	1.0	60	106.9	-17.1	0.0	0.0	3.0	259.0				0.0		N	0.0
Idling Dump Truck 2	65.0	SPL	15.0	1.0	60	108.7	-17.2	0.0	0.0	3.0	259.0				0.0		N	0.0
Idling Dump Truck 3	65.0	SPL	15.0	1.0	60	120.6	-18.1	0.0	0.0	3.0	259.0				0.0		N	0.0
Roller	74.0	SPL	15.0	1.0	60	114.5	-17.7	0.0	0.0	3.0	259.0				0.0		N	0.0
Compressor 1	81.0	SPL	15.0	1.0	60	137.6	-19.2	0.0	0.0	2.5	259.0				0.0		N	0.0
Compressor 2	81.0	SPL	15.0	1.0	60	139.0	-19.3	0.0	0.0	2.5	259.0				0.0		N	0.0
Pile driving	101.0	SPL	15.0	1.0	60	66.0	-12.9	0.0	0.0	6.0	259.0				0.0		Y	88.1

TOTAL STATNRY LEQ(1) **88**

APPENDIX E

The following summary is based on a telephone conversation between Sarah McLean and an officer in the by-law department at the City of Barrie (705)726-4242.

City of Barrie - Noise By-law

For all construction equipment associated with construction:

- In a Quiet Zone: not from 1700 hrs to 0700 hrs, and all day Sunday & Stat holidays

"Quiet zone" means hospital area - but hospital is no longer located in the downtown core and therefore the "quiet zone" regulation doesn't have any meaning anymore.

- In a Residential Zone: not from 1900 hrs one day to 0700 hrs the next, and all day Sunday & Stat holidays

By-law #76-140
City of Barrie Noise Control By-law
Schedule 3 - pertains to the restricted times

A company can apply for an exemption to work during the nighttime. The last time an exemption was granted was in 1996 to the MTO to work on Hwy 400 from 1900 hrs to 0700 hrs including Sundays and stat holidays.

THE CORPORATION OF THE TOWN OF INNISFIL

BY-LAW NO. 72-91

A by-law for prohibiting or regulating within the Municipality of the Town of Innisfil, or within any defined area or areas thereof, the making of noises, or public nuisances, likely to disturb the inhabitants of the said Town.

NOW THEREFORE the Council of the Corporation of the Town of Innisfil enacts as follows:

TITLE AND SCOPE

1. This by-law shall be known as a by-law for the regulating of noise within the Municipality of the Town of Innisfil and may be cited by its short title, The Noise By-law.

1.1 Repeal Existing By-laws

Any by-law or part of any by-law passed by the former municipalities of the Township of Innisfil, Village of Cookstown, the Township of West Gwillimbury and the Township of Tecumseth which are in conflict with the provisions of this by-law, are hereby repealed.

1.2 Penalties

Any person who contravenes any of the provisions of this by-law unless otherwise provided herein, is guilty of an offence and, upon conviction is liable to a fine of not less than $100.00 and not more than $5,000.00.

1.2.1 Each day that a breach of this by-law continues shall constitute a separate offence.

2. Definitions

For the purpose of this by-law, and without limiting the generality hereof, the following definitions shall apply:

2.1 "Corporation" means the Corporation of the Town of Innisfil

2.2 "Noise(s)" means ringing of bells, blowing of horns, shouting, singing, swearing, or any unusual noises, or noises likely to disturb the inhabitants.

2.3 "Person(s)" means any human being, association, firm, partnership, incorporated company, corporation, agent or trustee, and the heirs, executors or other legal representatives of a person to whom the context can apply, according to law.

2.4 "Public Nuisance" means the commission of any act or deed that is likely to disturb the normal peace and tranquility of any person.

2.5 "Town" means the municipality of the Town of Innisfil.

2.6 "Unusual Noise(s)" means:

2.6.1 the sounding of any bell, horn, siren or other signal device on any vehicle or whatsoever kind except when required by law;

2.6.2 the sounding of any such bell, horn, siren or signal device for an unnecessary or unreasonable period of time;

2.6.3 the sound or noise from or created by any radio, phonograph, television, public address system, sound equipment, loudspeaker, or similar device or devices, or any musical or sound producing instrument of whatever kind when such device or instrument is played or operated in such volume as to annoy or disturb the peace, quiet, comfort or repose of any individual in the neighbourhood;

2.6.4 any sound made by any pet animal or pet bird which disturbs the peace, quiet, comfort or repose of any individual in the neighbourhood;

2.6.5 the grating, grinding or rattling noise or sound caused by a condition of disrepair or maladjustment of any vehicle, or part or accessory thereof;

2.6.6 the blowing of any steam or air whistle attached to or used in connection with any stationary boiler or other machine or mechanism, except for the purpose of giving notice to workmen of the time to commence or cease work, or as a warning of danger;

2.6.7 the discharge into the air of the exhaust of any steam engine, stationary internal combustion engine, or vehicle, except through a muffler or other device which effectively prevents loud or explosive noises;

2.6.8 any unreasonable or unnecessary noise or sound arising from any excavation or construction work whatsoever, including the erection, demolition, alteration or repair of any building;

2.6.9 any unreasonable or unnecessary noise in the vicinity of any school or court while the same is in session, or in the vicinity of any hospital, convalescent home or rest home provided conspicuous signs are displayed in or upon the streets adjoining any such school, court, hospital or home indicating that such noises are prohibited in the vicinity;

2.6.10 the noise created by driving any vehicle bearing material, articles or things which are loaded upon such vehicle in such manner as to create such noise;

2.6.11 the noise or sound created by the use or operation of any drum, horn, bell, radio or mechanical loudspeaker, or other instrument or device, or sound producing, sound reproducing, or sound transmitting instrument or apparatus for the purpose of advertising or for attracting attention to any performance, show or sale or display of goods, wares or merchandise, or which projects noise or sound into any street or other public place;

2.6.12 the noise or sound created by the use or operation of any radio or mechanical loudspeaker or amplifier or other instrument or device or sound producing, sound reproducing, or sound transmitting instrument or apparatus in or upon any vehicle;

2.6.13 the noise of sound caused by the racing, idling or loud operation of a motor or a standing or parked vehicle in excess of that necessary for the reasonable operation of the same;

2.6.14 the noise or sound created by the repairing, wrecking, dismantling or modifying of any vehicle, motor, machine or any part or parts thereof;

2.6.15 any unreasonable or unnecessary noise created by the installation and operation of any air conditioning units, refrigeration units, generators, which are externally mounted or installed.

3. None of the provisions of this by-law shall apply to:

3.1 the use in a reasonable manner of any apparatus or mechanism for the amplification of the human voice or of music in a public park or any other commodious space in connection with any public election meeting, public celebration or other reasonable gathering, provided written permission of the Council of the Town has first been obtained;

3.2 any military or other band or any parade operating under written permission obtained from the Council of the said Town;

3.3 any newsboy, pedlar, hawker, or tradesman plying his calling legitimately and moderately;

3.4 any vehicle of the Police or Fire Department or any ambulance or any public service of emergency vehicle while answering a call;

3.5 the sound from any private radio in a motor vehicle installed for the sole benefit or entertainment of the operator and occupants of such vehicle when same is not audible at a distance of 25 feet from such vehicle;

3.6 any sound arising from the operation of any railway which operates under the Railway Act of Canada;

3.7 any case of public convenience or necessity where written permission has been obtained from the Council of the said Town;

3.8 any farming operation where weather or other unusual circumstances make it necessary to complete the planting, cultivating or harvesting of crops.

4. No person shall create or suffer to be created, any noise within the limits of the Town of Innisfil which is likely to disturb any inhabitant thereof.

5. No person shall commit or suffer to be committed, any act within the limits of the Town of Innisfil, which may be deemed to be a public nuisance and which is likely to disturb any inhabitant thereof.

6. Should any section of this by-law or part thereof be declared by a court of competent jurisdiction to be ultra vires or illegal for any reason, the remaining sections, or parts thereof, shall, nevertheless, remain valid and binding, and shall be read as if the offending section, or part thereof, had been struck out.

7. This by-law shall come into force and take effect on the final passing thereof.

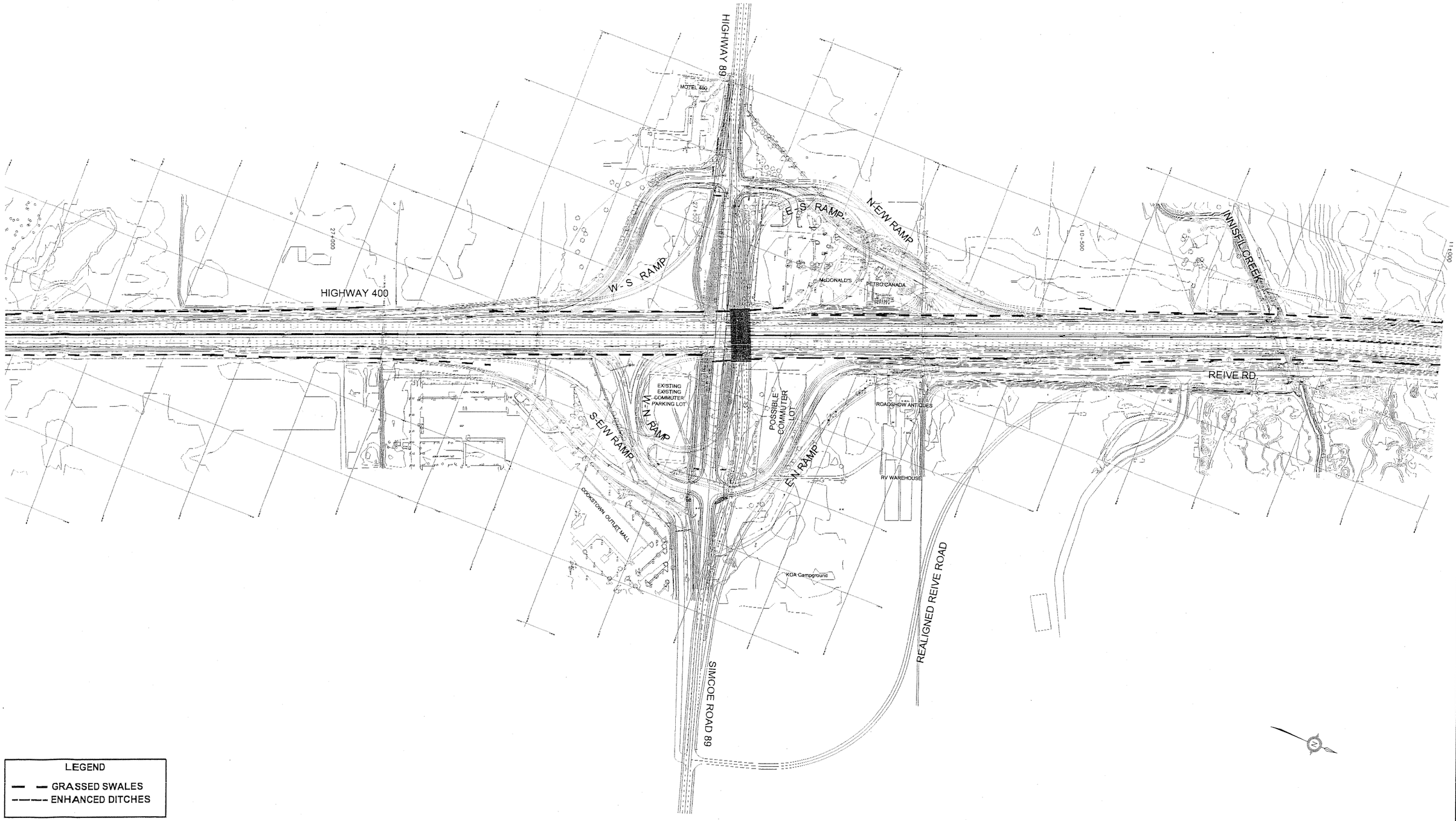
READ A FIRST AND SECOND TIME TIME this 14th day of August 1991.

READ A THIRD TIME AND FINALLY PASSED THIS 14th day of August 1991.

G. N. Andrade, Mayor

R. P. Groh, CAO/Clerk

APPENDIX F



Ontario
Ministry of Transportation

URS
Cole Sherman

HIGHWAY 400 PLANNING STUDY

1km South of Highway 89 to Junction at Highway 11

G.W.P. 30-95-00

KEY PLAN

TO ESSA ROAD

SCALE

25m 0 50m

HORIZONTAL

HIGHWAY 400 PLAN

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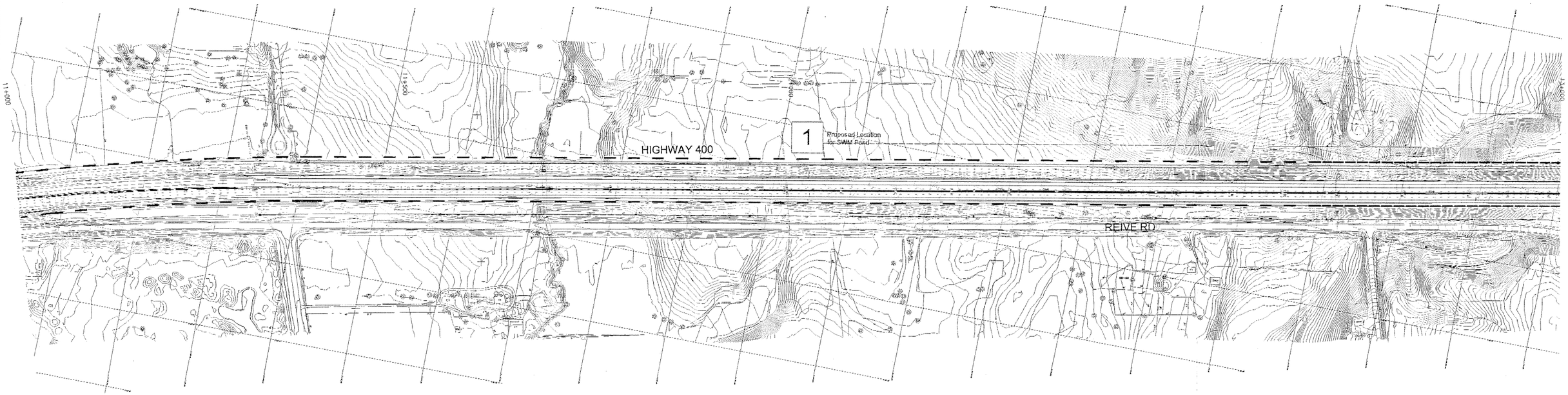

LEGEND

— — GRASSED SWALES

- - - - ENHANCED DITCHES

Ontario
Ministry of Transportation

URS
Cole Sherman

HIGHWAY 400 PLANNING STUDY

1km South of Highway 89 to Junction at Highway 11

G.W.P. 30-95-00

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HORIZONTAL

HIGHWAY 400 PLAN

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FIGURE

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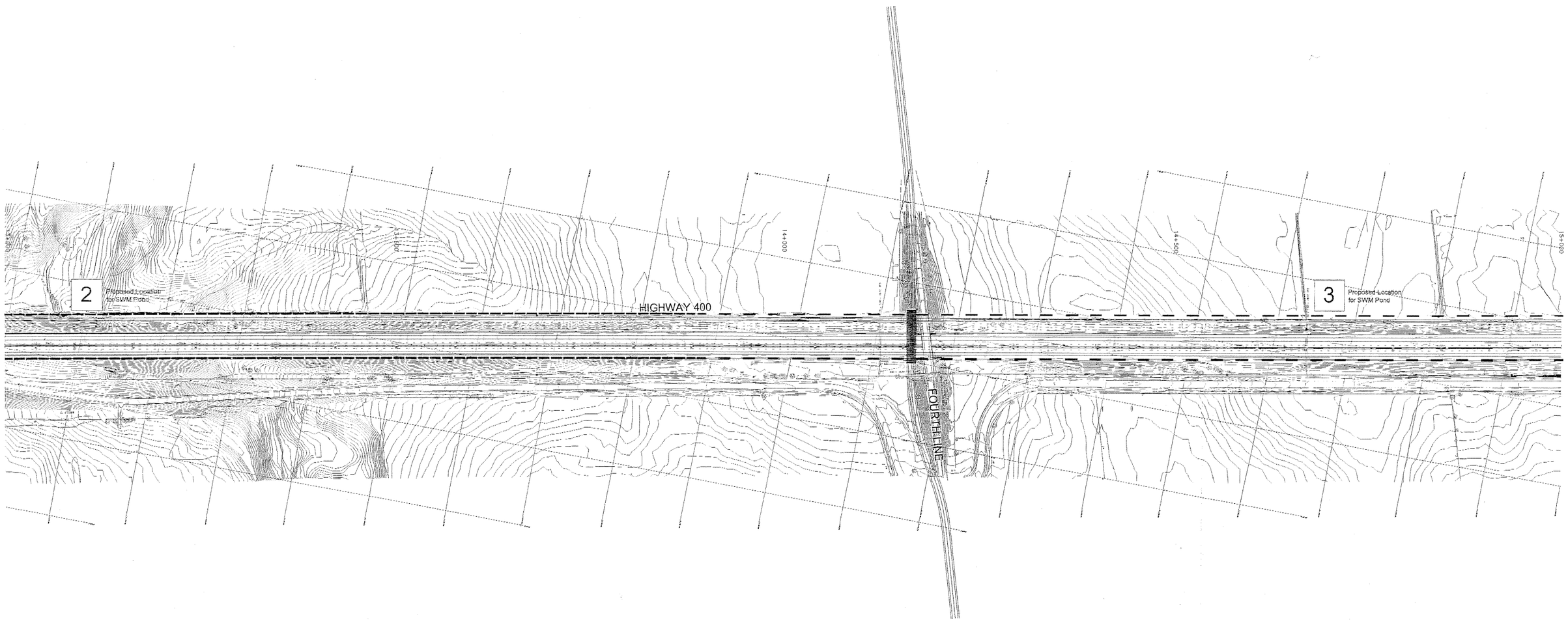

LEGEND

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Ontario
Ministry of Transportation

URS
Cole Sherman

HIGHWAY 400 PLANNING STUDY

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HIGHWAY 400 PLAN

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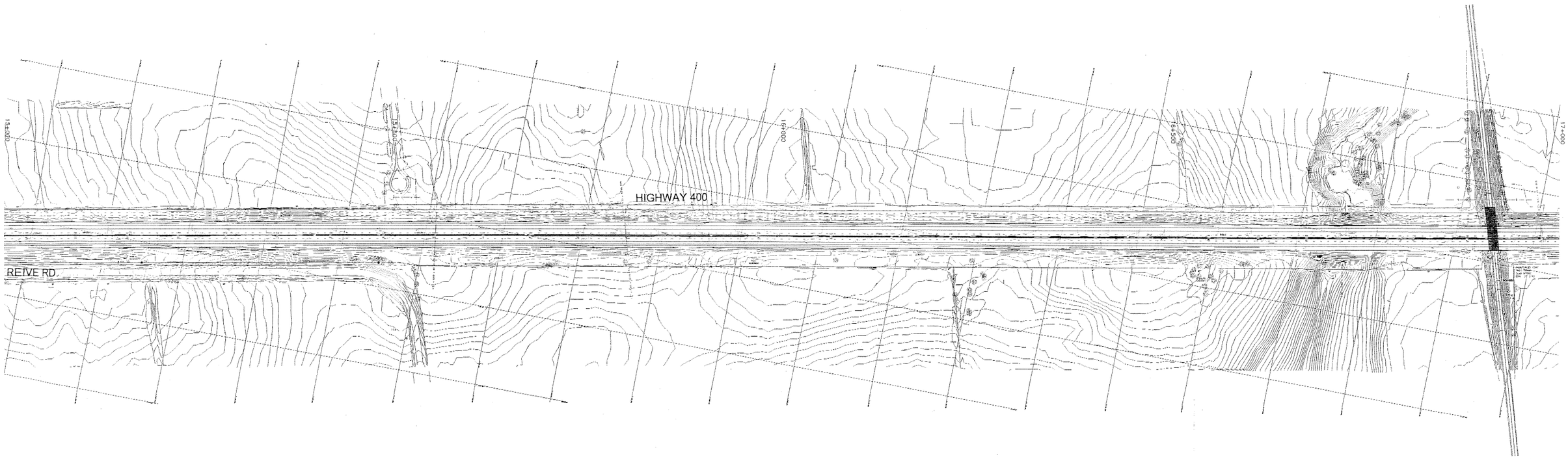

LEGEND

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-·-·- ENHANCED DITCHES

Ontario
Ministry of Transportation

URS
Cole Sherman

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HIGHWAY 400 PLAN

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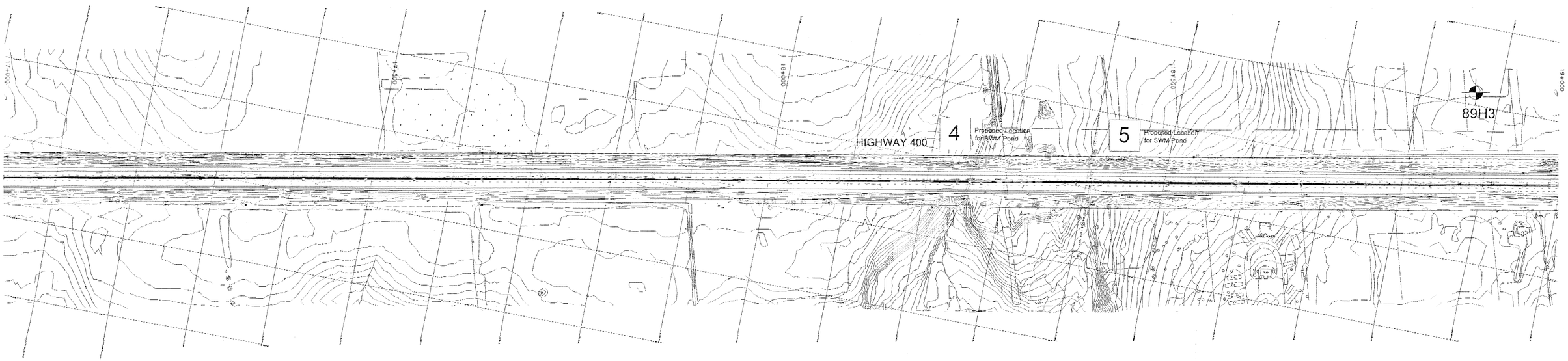

RWDI LEGEND:

MODELLED NOISE SENSITIVE AREA

LEGEND

— — GRASSED SWALES

– – – – ENHANCED DITCHES

Ontario
Ministry of Transportation

URS
Cole Sherman

HIGHWAY 400 PLANNING STUDY

1km South of Highway 89 to Junction at Highway 11

G.W.P. 30-95-00

KEY PLAN

← TO ESSA ROAD

SCALE

25m 0 50m

HORIZONTAL

HIGHWAY 400 PLAN

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DATE	MAY02						

FIGURE

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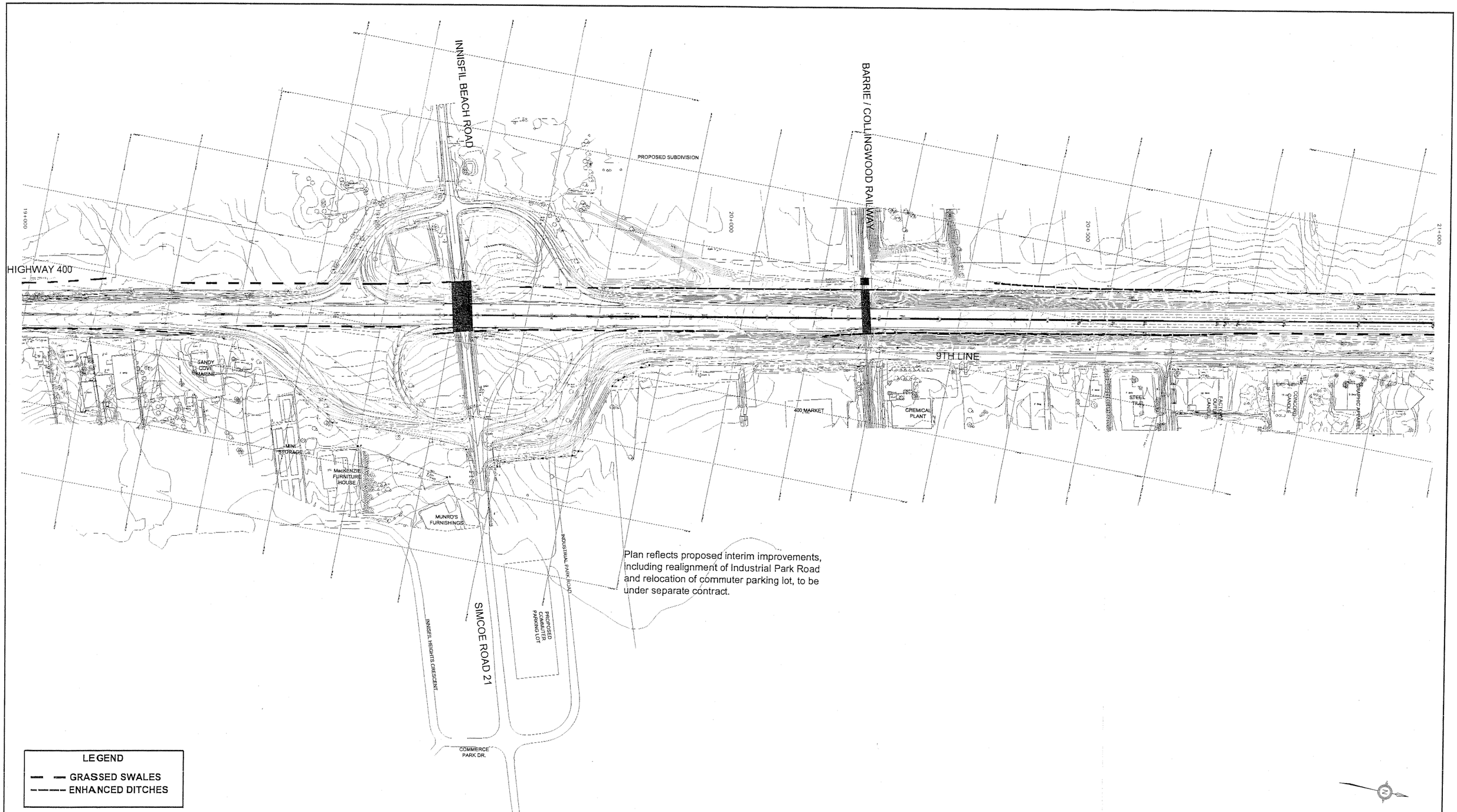

Ontario
Ministry of Transportation

URS
Cole Sherman

HIGHWAY 400 PLANNING STUDY

1km South of Highway 89 to Junction at Highway 11

G.W.P. 30-95-00

KEY PLAN

← TO ESSA ROAD

SCALE

25m 0 50m

HORIZONTAL

HIGHWAY 400 PLAN

DESIGN		DRAWN	CW	CHECK	PROJECT 00147
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DATE	MAY02				

FIGURE

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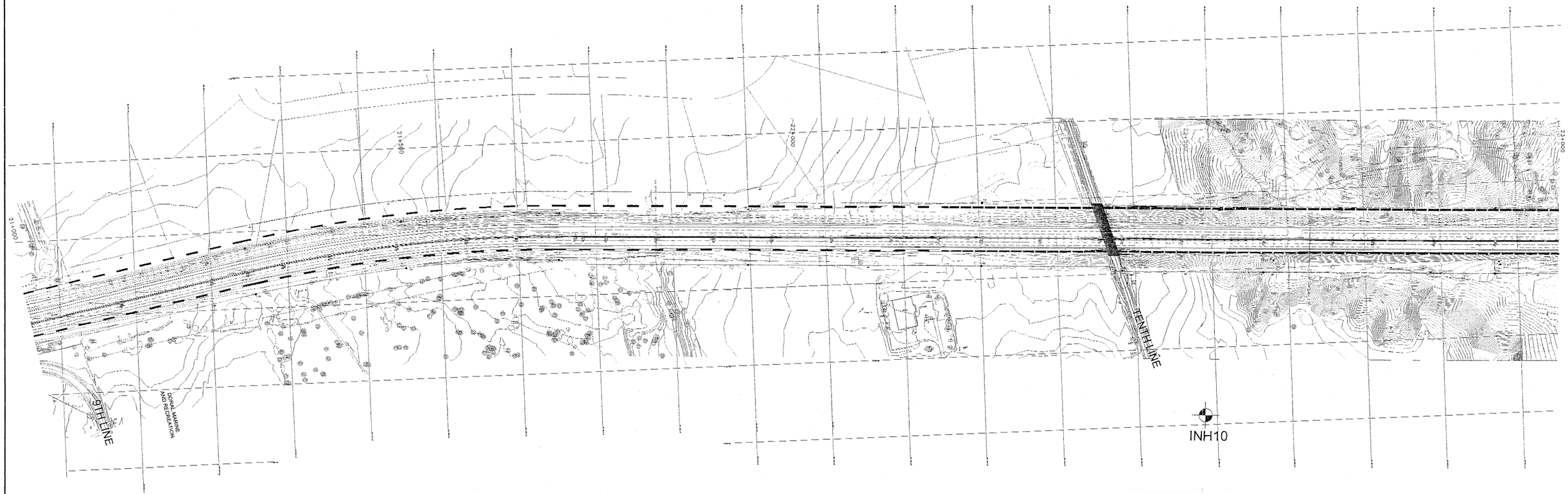

RWDI LEGEND:

MODELLED NOISE SENSITIVE AREA

LEGEND

— — GRASSED SWALES

-·-·- ENHANCED DITCHES

Ontario
Ministry of Transportation

URS
Cole Sherman

HIGHWAY 400 PLANNING STUDY

1km South of Highway 89 to Junction at Highway 11

G.W.P. 30-95-00

KEY PLAN

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SCALE

25m 0 50m

HORIZONTAL

HIGHWAY 400 PLAN

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DATE	MAY02						

FIGURE

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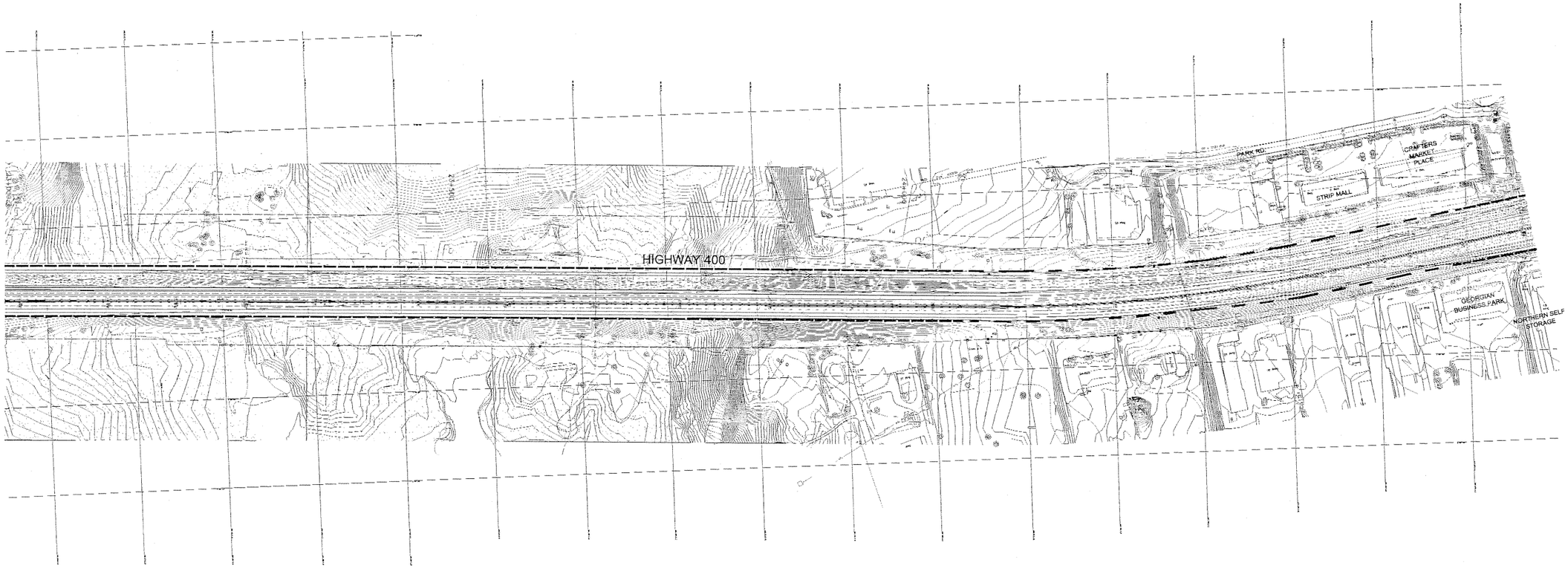

LEGEND

— — GRASSED SWALES

-- -- ENHANCED DITCHES

Ontario
Ministry of Transportation

URS
Cole Sherman

HIGHWAY 400 PLANNING STUDY

1km South of Highway 89 to Junction at Highway 11

G.W.P. 30-95-00

KEY PLAN

TO ESSA ROAD

SCALE

25m 0 50m

HORIZONTAL

HIGHWAY 400 PLAN

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DATE	MAY02				

FIGURE

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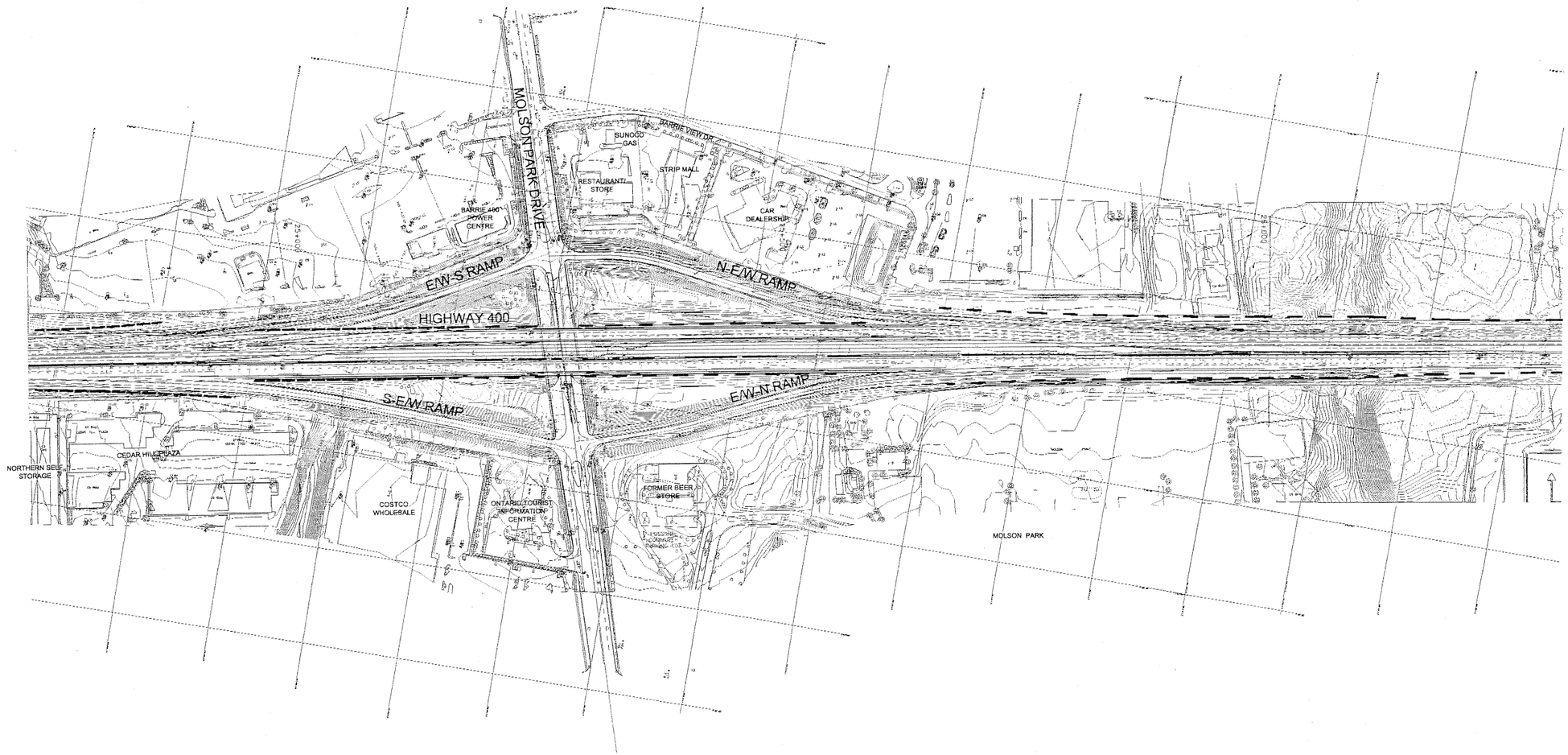

LEGEND

— — GRASSED SWALES

–––– ENHANCED DITCHES

Ontario
Ministry of Transportation

URS
Cole Sherman

HIGHWAY 400 PLANNING STUDY

1km South of Highway 89 to Junction at Highway 11

G.W.P. 30-95-00

KEY PLAN

TO ESSA ROAD

SCALE

25m 0 50m

HORIZONTAL

HIGHWAY 400 PLAN

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DATE	MAY02						

FIGURE

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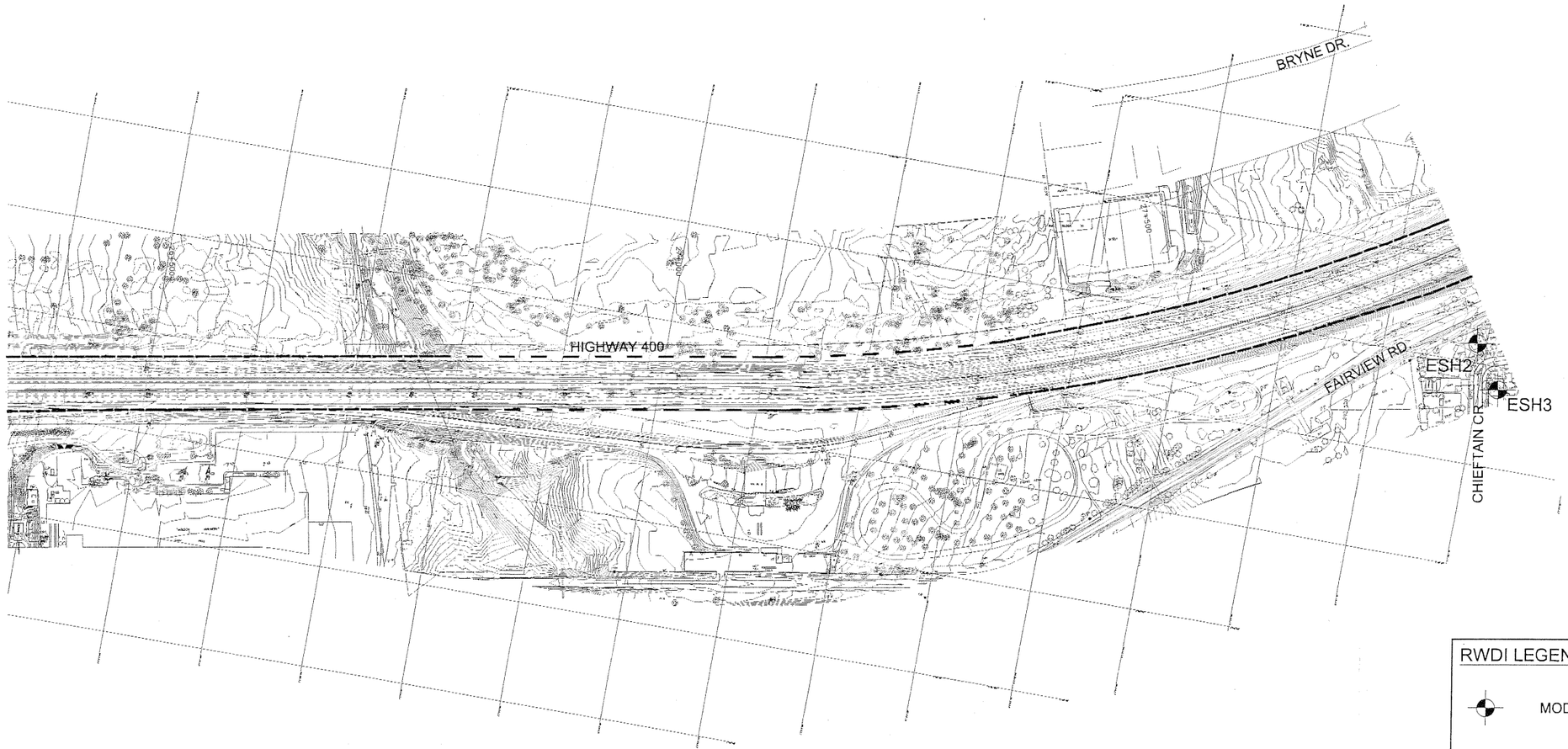

RWDI LEGEND:

MODELLED NOISE SENSITIVE AREA

LEGEND

— — GRASSED SWALES

–––– ENHANCED DITCHES

Ontario
Ministry of Transportation

URS
Cole Sherman

HIGHWAY 400 PLANNING STUDY

1km South of Highway 89 to Junction at Highway 11

G.W.P. 30-95-00

KEY PLAN

← TO ESSA ROAD

SCALE

25m 0 50m

HORIZONTAL

HIGHWAY 400 PLAN

DESIGN		DRAWN	CW	CHECK		PROJECT	00147
SCALES	1:5000	ACAD FILE NAME		DRAWING NUMBER			
DATE	MAY02						

FIGURE

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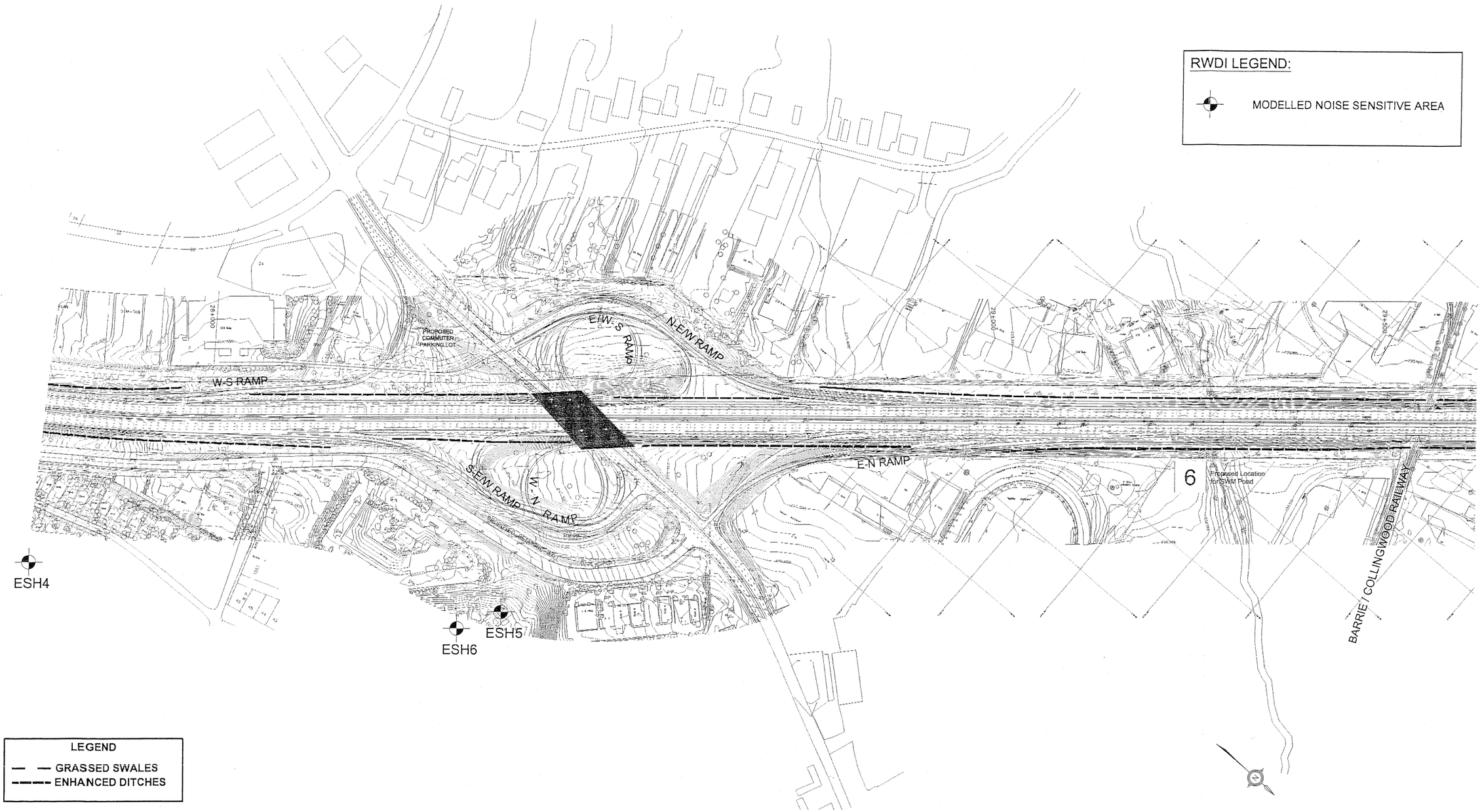

Ontario
Ministry of Transportation

URS
Cole Sherman

HIGHWAY 400 PLANNING STUDY

1km South of Highway 89 to Junction at Highway 11

G.W.P. 30-95-00

KEY PLAN

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HORIZONTAL

HIGHWAY 400 PLAN

DESIGN		DRAWN	CW	CHECK		PROJECT	00147
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DATE	MAY02						

FIGURE

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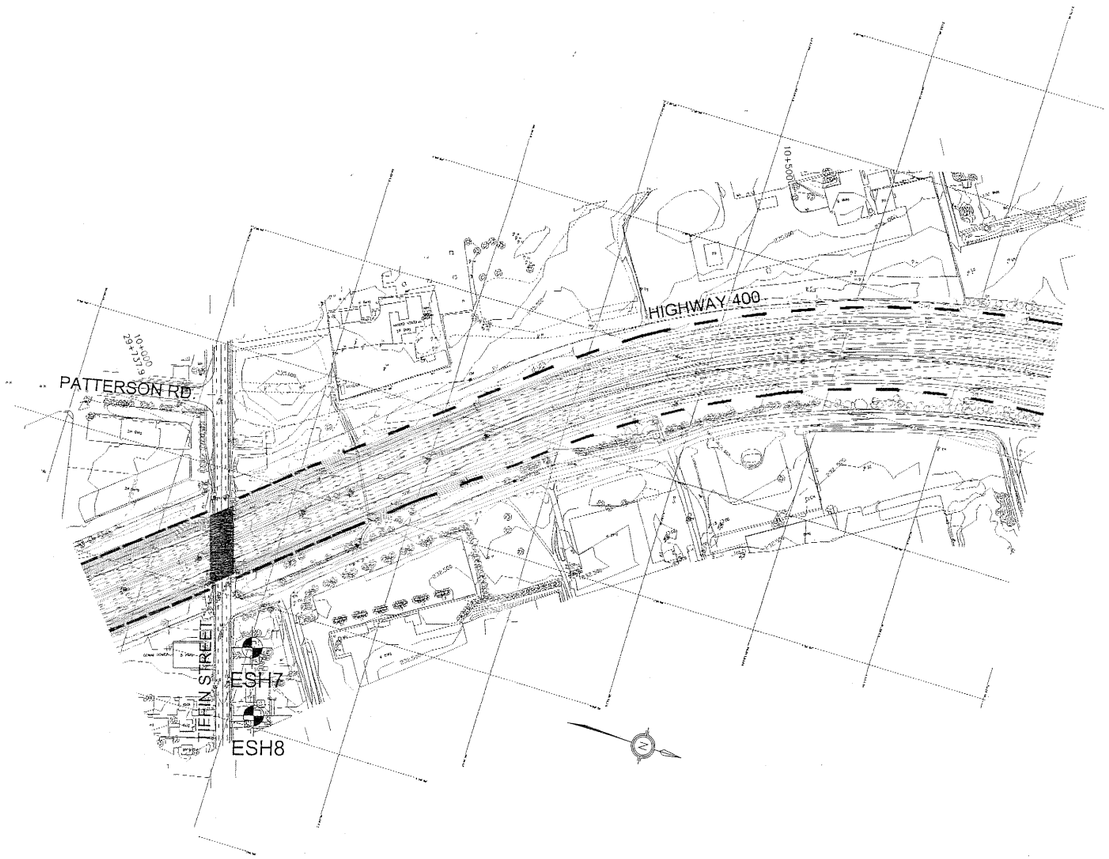

RWDI LEGEND:

MODELLED NOISE SENSITIVE AREA

LEGEND

GRASSED SWALES

ENHANCED DITCHES

Ontario
Ministry of Transportation

URS
Cole Sherman

HIGHWAY 400 PLANNING STUDY

1km South of Highway 89 to Junction at Highway 11

G.W.P. 30-95-00

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HIGHWAY 400 PLAN

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DATE	MAY02				

FIGURE

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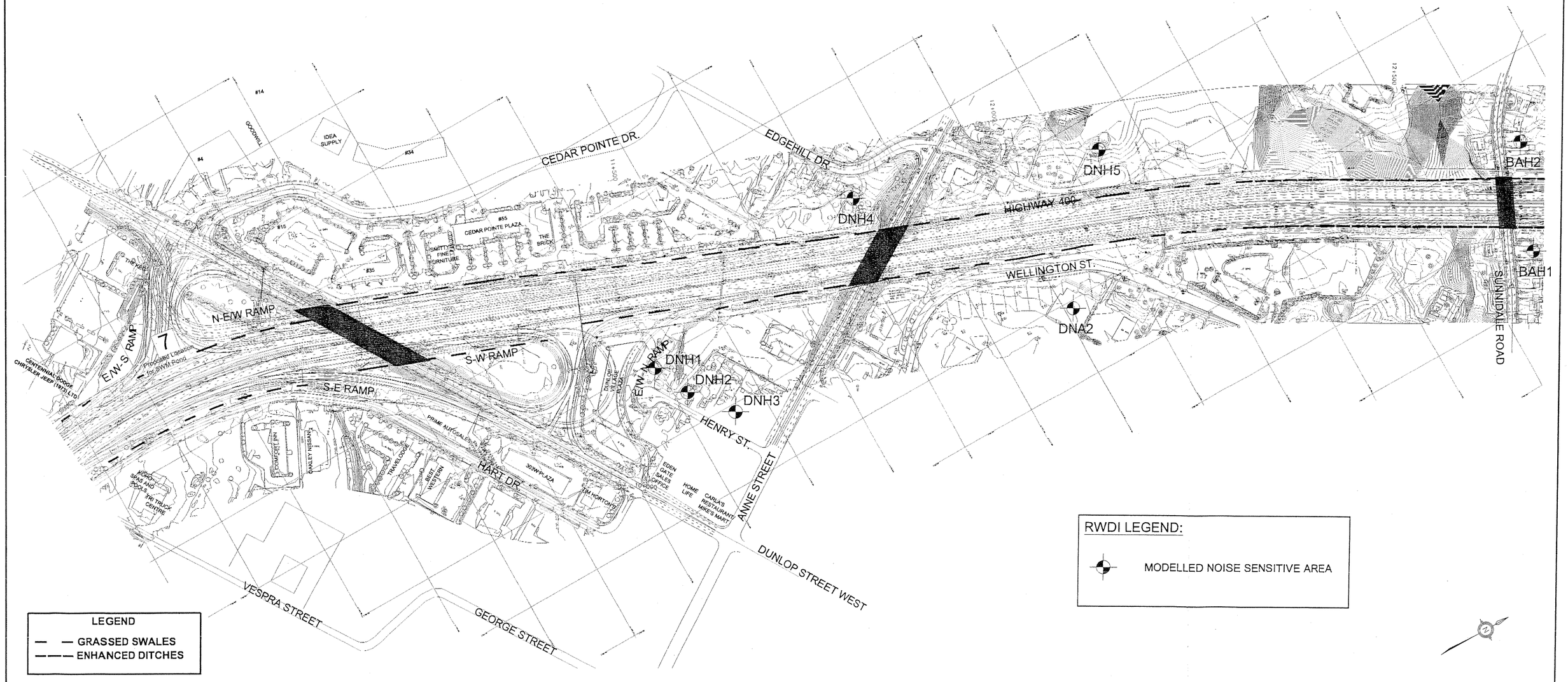

RWDI LEGEND:

MODELLED NOISE SENSITIVE AREA

LEGEND

— — GRASSED SWALES

— — — ENHANCED DITCHES

Ontario
Ministry of Transportation

URS
Cole Sherman

HIGHWAY 400 PLANNING STUDY

1km South of Highway 89 to Junction at Highway 11

G.W.P. 30-95-00

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HIGHWAY 400 PLAN

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FIGURE

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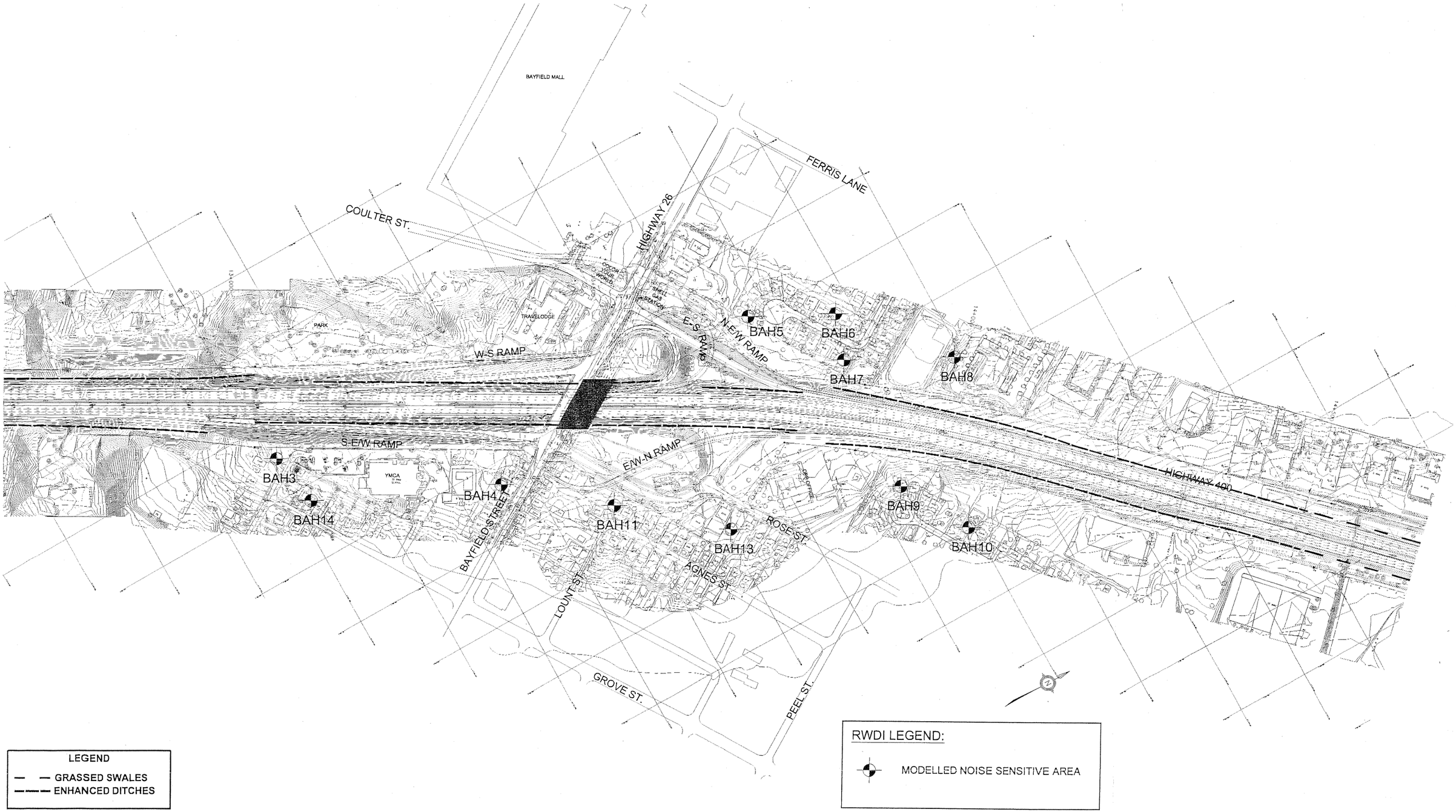

Ontario
Ministry of Transportation

URS
Cole Sherman

HIGHWAY 400 PLANNING STUDY

1km South of Highway 89 to Junction at
Highway 11
G.W.P. 30-95-00

KEY PLAN

← TO ESSA ROAD

SCALE

25m 0 50m
HORIZONTAL

HIGHWAY 400 PLAN

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DATE	MAY02						

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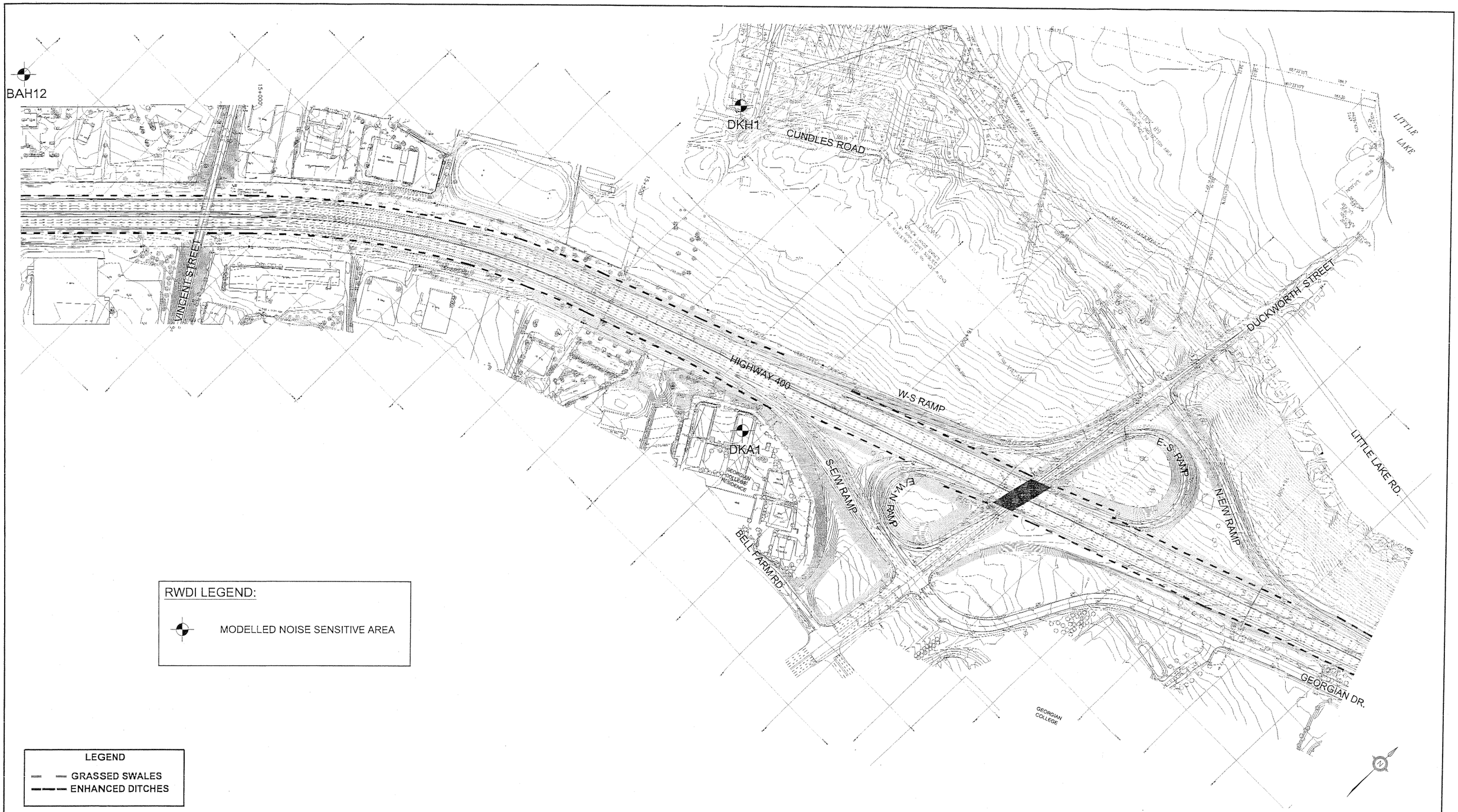

Ontario
Ministry of Transportation

URS
Cole Sherman

HIGHWAY 400 PLANNING STUDY

1km South of Highway 89 to Junction at
Highway 11
G.W.P. 30-95-00

KEY PLAN

TO ESSA ROAD

SCALE

25m 0 50m
HORIZONTAL

HIGHWAY 400 PLAN

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DATE	MAY02				

FIGURE

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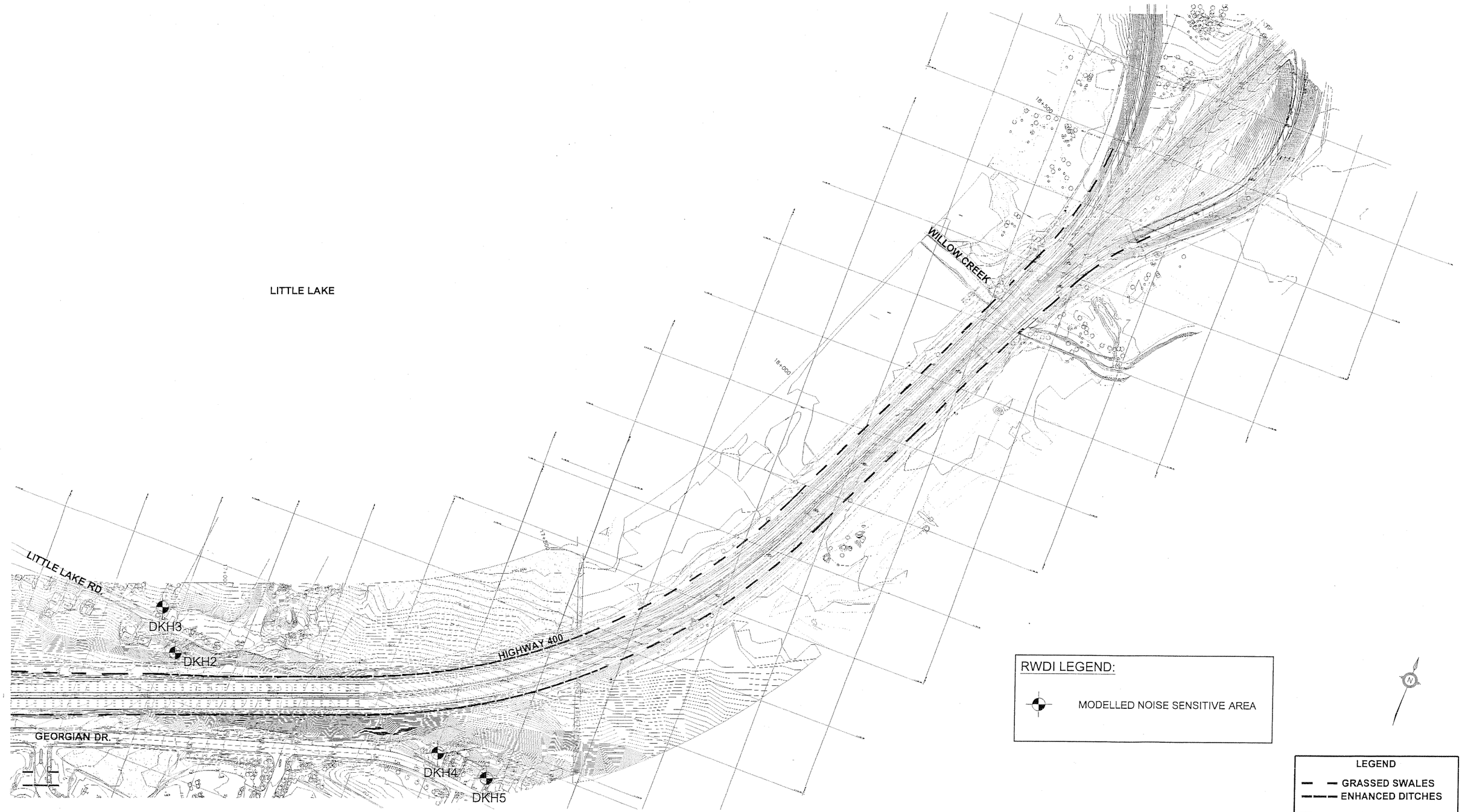

Ontario
Ministry of Transportation

URS
Cole Sherman

HIGHWAY 400 PLANNING STUDY

1km South of Highway 89 to Junction at
Highway 11
G.W.P. 30-95-00

KEY PLAN

TO ESSA ROAD

SCALE

25m 0 50m
HORIZONTAL

HIGHWAY 400 PLAN

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FIGURE

3p

Ontario

Ministry of Transportation
Southwestern Region
Planning and Design Section

HIGHWAY 400 PLANNING AND PRELIMINARY DESIGN STUDY

FROM 1 KM SOUTH OF HIGHWAY 89 NORTHERLY 30 KM TO THE JUNCTION AT HIGHWAY 11

COUNTY OF SIMCOE
TOWN OF BRADFORD WEST GWILLIMBURY
TOWN OF INNISFIL
CITY OF BARRIE
TOWNSHIP OF SPRINGWATER

G.W.P. 30-95-00

APPENDIX R

ARCHAEOLOGICAL ASSESSMENT REPORT

Stage 1 Archaeological Assessment of Highway 400
Planning Study from one kilometre south of Highway 89,
Northerly to the Penetanguishene Road (Highway 93)
At Highway 11, GWP 30-95-00

Submitted to:
URS Cole, Sherman and Associates
75 Commerce Valley Drive East
Thornhill, Ontario
L3T 7N9
Tel.: 905-882-4401

Prepared by:

ARCHAEOLOGICAL SERVICES INC.
528 Bathurst Street
Toronto, Ontario M5S 2P9
Tel: (416) 966-1069
Fax: (416) 966-9723
Email: archaeology@sympatico.ca
Website: www.archaeologicalservices.on.ca

License# 2001-020
CIF# 2001-020-021
Continuation of CIF# 2000-116-009
ASI File# 00MT-24

November 1, 2001

PROJECT PERSONNEL

Project Director:	Mr. Martin Cooper
Report Preparation:	Mr. TJ Hall

Stage 1 Archaeological Assessment of Highway 400 Planning Study from one kilometre south of Highway 89, northerly to the Penetanguishene Road (Highway 93) at Highway 11, GWP 30-95-00

1.0 INTRODUCTION

Archaeological Services Inc. was contracted by Cole, Sherman and Associates Limited to conduct a Stage 1 archaeological assessment of Highway 400 from one kilometre south of Highway 89 northerly to the Penetanguishene Road (Highway 93) at Highway 11, GWP 30-95-00 (Figure 1). The study area extends approximately 30 kilometres along the right-of-way.

The assessment was conducted under the archaeological consulting licence 2001-020, held by Mr. Martin Cooper of Archaeological Services Inc and in accordance with the Ontario Heritage Act (1990).

2.0 BACKGROUND RESEARCH

2.1 Previous Archaeological Research

In order that an inventory of archaeological resources could be compiled for the study area, three sources of information were consulted: the site record forms for registered sites housed at the Ministry of Tourism, Culture and Recreation (MTCR), published and unpublished documentary sources, and the files of Archaeological Services Inc.

In Ontario, information concerning archaeological sites is stored in the Ontario Archaeological Sites Database (OASD), a database maintained by the MTCR. This database contains archaeological sites registered within the Borden system. The Borden system was first proposed by Dr. Charles E. Borden, and is based on a block of latitude and longitude. A Borden block is approximately 13 kilometres east west by 18.5 kilometres north south. Sites within each block are numbered sequentially as they are found. The study area under review is located within the BcGv, BcGw, BbGv and BbGw Borden Blocks.

While no archaeological sites have been registered within the study area, 18 sites have been registered within a radius of two kilometres of the study area limits. Particulars concerning these sites are summarized in Table 1.

2.2 Physiography and Assessment of Prehistoric Potential

The study area is located within three physiographic regions. The southern portion of the subject property is located within the Peterborough Drumlin Field physiographic zone. This zone is characterized by approximately 3,000 distinct drumlins and many other drumlinoid formations (Chapman & Putnam 1984: 169). Generally, the rock underlying the region is limestone that is highly fossiliferous and disintegrates easily. The deep valleys typical of this area drain provide excellent drainage.

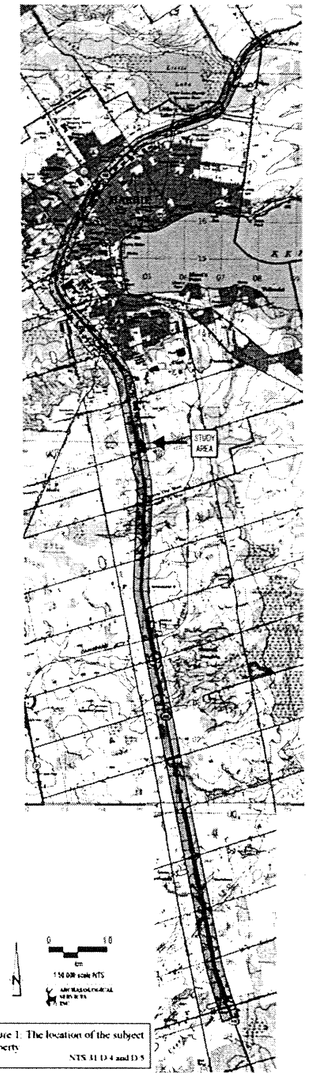

Figure 1. The location of the subject property. NTS 41 D 4 and D 5

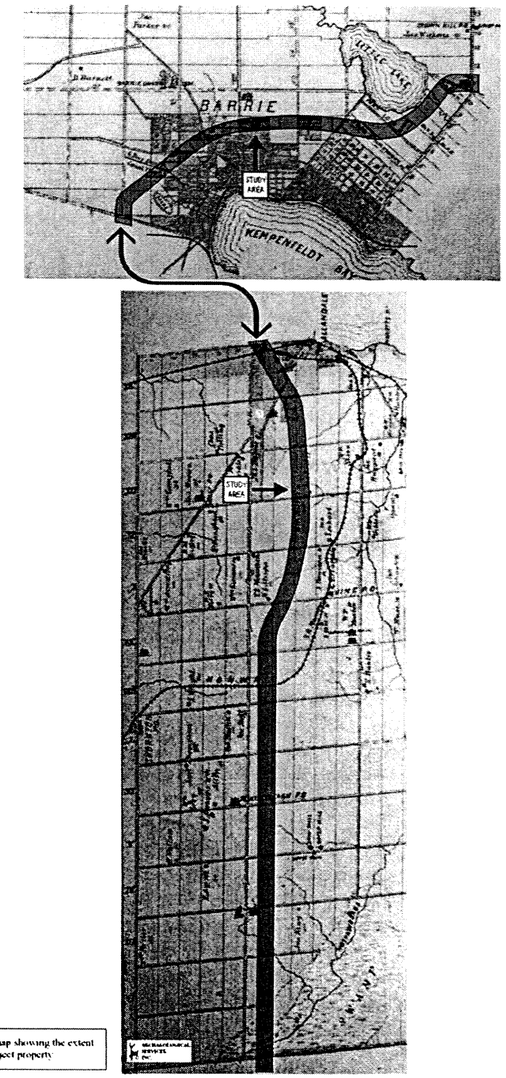

Figure 2. Historic map showing the extent of the subject property

The next zone is the Simcoe Lowlands that borders Georgian Bay in the west and Lake Simcoe in the east. Highway 400 passes through a small portion of this zone that is characterized by Tioga sand or sandy loam (Chapman & Putnam 1984: 179). The zone is well drained, with generally poor and droughty soil conditions.

The northernmost zone affecting the study area is within the Simcoe Uplands physiographic zone. The portion of Highway 400 under review extends only a short way into this zone. Typically the Simcoe uplands are broad, rolling till plains divided by steep-sided planar valleys (Chapman & Putnam 1984: 182). This zone is well drained and highly suited for mixed farming and features paleo-beaches and channels formed by the glacial Lake Algonquin system.

In terms of archaeological potential, potable water is arguably the single most important resource necessary for any extended human occupation or settlement. Since water sources have remained relatively stable in southern Ontario after the Pleistocene era, proximity to water can be regarded as a useful index for the evaluation of archaeological site potential. Indeed, distance from water has been one of the most commonly used variables for predictive modelling of site location.

The Ministry of Tourism, Culture and Recreation Primer on Archaeology, Land Use Planning and Development in Ontario (1997:12-13) stipulates that undisturbed lands within 300 metres of a primary water source, and undisturbed lands within 200 metres of a secondary water source, are considered to exhibit archaeological potential.

The physiography of the study area and the close proximity of the above noted sites attest to the significant potential for the recovery of archaeological remains from undisturbed lands adjacent to the portion of Highway 400 under review.

2.3 Summary of Land Use History

The *1881 Illustrated Atlas of the Dominion of Canada, Simcoe County Supplement* was reviewed to determine the potential for the presence of structures or other historic remains (Figure 2). According to the *Atlas* there are a number of structures including homesteads and farm-related structures on or near the study area. As well, the study area passes through the historic town of Barrie

An unknown employee of the Hudson's Bay Company (HBC) discovered the benefits of the area that is now Barrie in the early 1800s. Later in 1812, the HBC built a storehouse to facilitate local traders. The first dwelling was constructed in 1815 and Nine Mile Portage, located three miles east of Barrie, facilitated a military supply route between Lake Simcoe and Lake Huron.

The area was surveyed and given the name of Barrie in 1830, soon after which settlers began moving into the region. Throughout the mid and late 1800s services such as schools, churches and a post office were constructed and upon the construction of the railway, Barrie became a prosperous town to live in or near. Travel routes from Toronto and other southern Ontario towns and cities prompted settlers to acquire land and begin farming throughout the region.

It should be noted that not every feature of interest would have been illustrated on the 1881 *Atlas* maps and that no map should be considered definitive. However, based on the extent of settlement in the vicinity of the study area, it may be concluded that there is potential for the identification of historic archaeological material within the subject property.

The intensity of historic land use, the close proximity of archaeological sites and the nearness of the subject property to Lake Simcoe and numerous creeks and rivers prove that the region maintains potential for archaeological resources and that further investigation will offer additional information valuable to the history of the region.

TABLE 1: REGISTERED ARCHAEOLOGICAL SITES WITHIN 2 KM RADIUS OF THE SUBJECT PROPERTY

Borden	Site	Cultural/temporal Affiliation	Site Type	Researcher
BbGw-9	Hubbert	Late Iroquoian	Village	Hunter 1976, Warrick 1985, ASI 1990
BbGw-10	Cleary	Late Iroquoian	Village	Warrick 1985
BbGw-11	Roof	Late Iroquoian	Village	Warrick 1985
BbGw-14	Paisley	Late Iroquoian	Village	Warrick 1986
BbGw-19	Sun Valley	Middle to Late Woodland	Isolated find	ASI 1990
BbGw-20	Zachary	Iroquoian	Campsite	ASI 1990
BcGw-2	Bell	Indeterminate Precontact	Campsite	Hunter 1963
BcGw-11	Cundles	Iroquoian, Middleport	Campsite	Ridley 1970
BcGw-15	Little	Iroquoian, Middleport	Village	Hunter 1976, Lennox 1985, Warrick 1985
BcGw-20	Cundles Creek 2	Historic	Earthwork	Hunter 1977
BcGw-21	Cundles Creek 1	Iroquoian, Middleport	Hamlet	Hunter 1977
BcGw-22	Sunnidale Park	Iroquoian, Middleport	Hamlet	Hunter 1977
BcGw-26	Waicek	Iroquoian, Middleport	Village	Lennox 1983, ASI 1990
BcGw-27	Molson	Late Woodland	Village	Warrick 1985
BcGw-28	Little 2	Late Woodland	Village	Warrick 1985
BcGw-31	IF 7	Indeterminate Precontact	Isolated find	Warrick 1985
BcGw-33	Gnarly Man	Indeterminate Precontact	Isolated find	Warrick 1985
BcGv-25	Unassigned	Historic	Homestead	Poulton 1996

3.0 CONCLUSIONS AND RECOMMENDATIONS

During the Stage 1 Archaeological Assessment of the Highway 400 Planning Study from one kilometre south of Highway 89, northerly to the Penetanguishene Road (Highway 93) at Highway 11, GWP 30-95-00, it was determined that no archaeological sites are extant in the limits of the study area, but the nature of the landscape, the presence of 18 archaeological sites within a two kilometre radius and the general history of the region indicate that there is historic and precontact archaeological potential within the study area limits.

In light of these results it is recommended that:

1. Prior to any land disturbance, all lands currently under or formerly under agricultural production should be subjected to a Stage 2 archaeological assessment in accordance with the Stage 1-3 archaeological assessment technical guidelines provided by the MTCR. All land should be ploughed and allowed to weather through at least one significant rainfall, then pedestrian surveyed at five metre intervals to facilitate the recovery of archaeological material;

   Woodlots or areas containing scrub brush should be test pitted at five metre intervals. All test pits must be excavated to sub-soil and the material screened through six-metres mesh to facilitate the recovery of small artifacts.

2. Should deeply buried archaeological remains be found on the property during construction activities, the Heritage Operations Unit of the Ministry of Tourism, Culture and Recreation (MTCR) should be notified immediately;

3. In the event that human remains are encountered during construction, the proponent should immediately contact both MTCR, and the Registrar or Deputy Registrar of the Cemeteries Regulation Unit of the Ministry of Consumer and Commercial Relations, (416) 326-8392.

The documentation related to the archaeological assessment of the study area shall be curated by Archaeological Services Inc. until such a time that arrangements for their ultimate transfer to Her Majesty the Queen in right of Ontario, or other public institution, can be made to the satisfaction of the landowner, the Ministry of Tourism, Culture and Recreation, and any other legitimate interest groups.

4.0 REFERENCES CITED

Chapman, L.J. and F. Putnam
1984 *The Physiography of Southern Ontario. Ontario Geological Survey, Special Volume 2.*

Illustrated Atlas of the Dominion of Canada, Simcoe County Supplement
1881 Toronto: H. Belden.

Ministry of Tourism, Culture and Recreation
1997 *Conserving a Future for Our Past: Archaeology, Land Use Planning & Development in Ontario.* Cultural Programs Branch, Archaeology & Heritage Planning Unit, Toronto

Ontario

Ministry of Transportation
Southwestern Region
Planning and Design Section

HIGHWAY 400 PLANNING AND PRELIMINARY DESIGN STUDY

FROM 1 KM SOUTH OF HIGHWAY 89 NORTHERLY 30 KM TO THE JUNCTION AT HIGHWAY 11

COUNTY OF SIMCOE
TOWN OF BRADFORD WEST GWILLIMBURY
TOWN OF INNISFIL
CITY OF BARRIE
TOWNSHIP OF SPRINGWATER

G.W.P. 30-95-00

APPENDIX S

HYDROGEOLOGICAL ASSESSMENT

Golder Associates Ltd.

500 Nottinghill Road
London, Ontario, Canada N6K 3P1
Telephone (519) 471-9600
Fax (519) 471-4707

Golder Associates

May 17, 2001

001-3203

COPY

Cole, Sherman & Associates
75 Commerce Valley Drive East
Thornhill, Ontario
L3T 7N9

Attention: Mr. M. Bricks

**RE: HYDROGEOLOGICAL ASSESSMENT
PRELIMINARY GEOTECHNICAL
ENGINEERING SERVICES
PROPOSED HIGHWAY 400 WIDENING
GWP 30-95-00**

Dear Sirs:

As requested, the attached draft report provides the preliminary hydrogeological information for the above-noted project. As discussed with Mr. T. Sorochinsky, P. Eng. of Cole, Sherman & Associates, we are awaiting a decision from the Ministry of Transportation, Ontario on which widening option is to be selected prior to completing the draft pavement design report. As a result, additional background data from the report was assembled and included to provide as "complete" as possible package for your immediate needs. We understand that the due date for the draft Pavement Design Report GWP 30-95-00 has been delayed and, as a result, we are not in a position to issue the hydrogeologic component of the work in final format at this time.

We trust that the attached is sufficient for your present needs. If any point requires further clarification or if we can be of additional assistance, please contact this office.

Yours truly,

GOLDER ASSOCIATES LTD.

Philip R. Bedell, P. Eng.

Attachments

cc: Ms. A.S. Poschmann, P. Eng. (w/o attachment)
Mr. M.L.J. Maher, P. Eng. (w/o attachment)
Mr. K. Hadipour, P. Eng. (w/o attachment)
Golder Associates Ltd.

Mr. T. Sorochinsky, P. Eng. (w/o attachment)
Mr. L.V.S. Kozachuk, P. Eng. (w/o attachment)
Cole, Sherman & Associates

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BEST MANAGED PRIVATE COMPANIES

1.0 INTRODUCTION

As part of the Highway 400 Planning Study, Golder Associates Ltd. (Golder Associates) was retained by Cole, Sherman & Associates (Cole, Sherman) to conduct, in a preliminary fashion, a hydrogeological review of the section of Highway 400 described as GWP 30-95-00. The location of the site is shown on the Key Plan, Figure 1. The project extends from 1 kilometre south of Highway 89 (Station 26+550, Township of Bradford, West Gwillimbury) northerly some 30 kilometres to the Highway 400/Highway 11 split (Station 18+400, Township of Vespra).

The purpose of the hydrogeological review was to characterize the existing groundwater resources and hydrogeology in the vicinity of the right-of-way and to assess potential impact(s) on groundwater resources which may arise as a result of the proposed widening. Where applicable, possible mitigative measures to minimize the effect of the potential impacts to groundwater have been provided.

2.0 RECORDS REVIEW

2.1 Scope of Work

Golder Associates' assessment included a review of readily available references concerning soil and groundwater conditions at the site. Details are provided in the subsequent sections.

The references consulted included:

- Ontario Ministry of Environment (MOE) water well records;
- Quaternary Geology of the Alliston Area mapping;
- Soil Survey of Simcoe County mapping;
- Groundwater Susceptibility mapping; and
- Physiography of Southern Ontario mapping.

3.0 PHYSICAL SETTING

3.1 Physiography (Chapman & Putnam, 1984)

The major physiographic regions within GWP 30-95-00 are shown on Figure 2. From the project south limit to about Essa Road, Highway 400 crosses the Peterborough drumlin field. The underlying bedrock is reported to be limestone of the Lindsay and Verulam formations and is described as highly fossiliferous and easily disintegrated. The general orientation of the drumlin axes is from northeast to southwest. The drumlins are reportedly composed of highly calcareous till of a sandy nature.

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As Highway 400 rounds the western point of Kempenfelt Bay, for a distance of about 3.5 kilometres, it traverses the Simcoe lowlands, specifically an area within the Nottawasaga Basin known as the Camp Borden sand plain. The soils are reportedly comprised of the Tioga sand or sandy loam which is described as loose, coarse textured and well drained. Highway 11 also crosses the Camp Borden sand plain for some 1.0 kilometres just south of the junction with Simcoe Road 93.

The majority of the northern portion of the project lies within the Simcoe Uplands. The area is described as broad, rolling till plains separated by steep-sided, flat-floored valleys. The till is reportedly derived from Precambrian rock with a gritty loam texture.

3.2 Quaternary Geology

Quaternary geology mapping is available for the southern 5.8 kilometres of the site. The predominant quaternary geologic regions in this are of the site are outlined on Figure 3.

Based on the available quaternary geology mapping, the southern 0.9 kilometres of the project lies within glaciolacustrine shallow water deposits described as Lake Algonquin deposits.

From 0.8 kilometres to 5.7 kilometres north of Highway 89, the subsurface soils consist predominantly of Newmarket Till. The Newmarket Till is described as a sand to silty sand till.

From 5.7 kilometres north of Highway 89, bedrock topography and drift thickness mapping is available; however, quaternary geology mapping is not available.

3.3 Soil Survey of Simcoe County

The surficial soils, according to the Soil Survey of Simcoe County, are outlined on Figure 4. In general, the surficial soils consist of sandy loams. Details of the various soils series and soil types are provided on the Figure and are summarized below:

SERIES	SYMBOL	DESCRIPTION	NATURAL DRAINAGE
Alliston sandy loam	Ans	grey calcareous outwash sand	imperfect
Tioga loamy sand	Tis		good
Bondhead loam	Bl	light grey calcareous loam and sandy loam till	good
Bondhead sandy loam - steep phase	Bs.s		good
Muck	M	well decomposed organic material over 0.3 metres thick	very poor
Schomberg clay loam	Shsc	calcareous lacustrine varved silt loam and clay	good
Smithfield silty clay loam	Smsc		imperfect

SERIES	SYMBOL	DESCRIPTION	NATURAL DRAINAGE
Sargent gravelly sandy loam	Stsl	pale brown calcareous outwash gravel	good

Reference - Soil Map of Simcoe County, South Sheet, Soil Survey Report No. 29.

The sandy loam materials are predominant throughout the project. The Schomberg clay loam and Smithfield silty clay loam are reportedly located to the northeast of Barrie and about 3 to 5 kilometres north of Highway 89. Muck deposits are reportedly located from about 1 kilometre south to 0.2 kilometres north of Willow Creek.

4.0 GROUNDWATER INVESTIGATION

A preliminary groundwater investigation was conducted for the project. The purpose of the investigation was to identify areas of high water table and upwelling, groundwater recharge and discharge and high overburden permeability and the locations and usage of high volume producing water wells.

The tasks completed as part of this investigation included a review of topographic mapping, geological mapping, available MOE water well records, and any readily available borehole information from the Ministry of Transportation, Ontario (MTO) and Golder Associates' files.

This groundwater investigation presents a preliminary, generalized interpretation of hydrogeologic conditions and has been based on limited available background information. Actual hydrogeological conditions within the project limits will vary and are subject to confirmation with actual site specific investigations including boreholes, test pits, groundwater hydraulic testing, chemical analyses and the like.

4.1 Site Hydrogeology

The majority of the site is anticipated to represent an area of groundwater recharge. The most significant recharge zone is expected to be in the area of the Camp Borden sand plain at the western point of Kempenfelt Bay. Groundwater discharge is expected to be confined to stream channels and floodplains and wetland areas within the project limits. Based on the MOE water well records and a cursory site reconnaissance, the Cookston Hollow forest swamp at the southern project limit and the Little Lake marsh area at the northern limit are anticipated to be groundwater discharge zones.

Based on the information provided on the Surveys and Mapping Branch, Dept. of Energy, Mines and Resources, Alliston Topographic Map 31 D/4 and Barrie Topographic Map 310/5, surface flows from the Innisfil Township 5th Concession south generally flow to the southeast north of Highway 89 and northwest from south of Highway 89 to Innisfil Creek.

Between about the 5th Concession and Essa Road, surface flows generally flow east and ultimately report to Lover's Creek.

Through Barrie, surface water generally flows east to Kempenfelt Bay.

North of Highway 27, surface flows report to the Little Lake marsh area.

Also noted on the mapping, a sewage lagoon is present west of Highway 400, just north of the 4th Concession, Innisfil Township and another sewage lagoon is located in the southeast corner of Innisfil Beach Road and Highway 400. A junk yard is located west of Highway 400 and just south of the 14th Concession, West Gwillimbury Township. A pumping station is located in the northwest corner of Highway 400 and Highway 89. In the southeast corner of Highway 400 and Big Bay Point Road just south of Barrie, a junk yard, a filtration plant and poultry farm are present. Another junk yard is located at the southwest corner of Highway 400 and the Innisfil/Vespra Township boundary. An underground reservoir is located northwest of Highway 400 between Dunlop Street and Cundles Road.

During our recent cursory field reconnaissance, service centres with refuelling stations were noted at the northwest corner of Highway 400 and Highway 89 and on the east side of Highway 400 just north of Molson Park Drive.

Adjacent to the Highway 400 right-of-way throughout Barrie, several commercial and industrial properties are present. It is understood that a TCE (trichloroethylene) plume is present at the intersection of Tiffin Street and Highway 400. Investigation and implications of the TCE plume will be addressed under separate cover.

Areas of groundwater upwelling or high water table (i.e. less than 1 metre below ground surface) have been identified in the southern, central and northern portions of the project based on the topographic mapping and the MOE water well records examined. Interpreted areas of shallow water table are provided on Figure 5.

4.2 Groundwater Usage

Existing groundwater uses in the vicinity of the site consist of private domestic and commercial water wells associated with private residences, farms and commercial properties along the right-of-way. A plot of the locations of water wells within adjacent concessions along the Highway 400 right-of-way, based on MOE water well records, is shown on Figure 5. It should be noted that the MOE water well records are based on records current as of 1977. Additional wells which have been drilled subsequent to the compilation of the well records in 1977 or wells for which records were never filed will be present along the right-of-way. Water well records for large

diameter, shallow bored/dug wells commonly do not exist. A summary of the water well records reviewed is provided in Table I.

Existing well uses in the area of the site generally consist of drilled wells which utilize relatively shallow overburden aquifers. Of the 257 water well records reviewed, 70 of the wells utilize shallow aquifers (less than 15 metres deep) and a further 3 wells utilize bedrock aquifers. Forty-two wells utilize deep overburden aquifers greater than 15 metres below the ground surface. Thirteen well records indicated flowing conditions and 13 were dry to the depth drilled. Thirteen water well records offered no water level information.

A total of 14 large volume producing water wells (50 Imperial gallons per minute (Igpm) or 190 litres per minutes (L/min)) municipal or public supply water wells were identified from the MOE water well records in the vicinity of the project:

MOE WELL NUMBER	UTM Easting	UTM Northing	TESTED PRODUCTION RATE (L/min)	USAGE
57-1177	604158	4906189	189	domestic
57-1517	603235	4910774	27	public supply
57-1536	604544	4911568	227	public supply
57-1538	604297	4912456	91	public supply
57-1720	602800	4912894	15	public supply
57-4920	605965	4918859	8	municipal
57-4929	607247	4919838	8	public supply
57-4934	607109	4919681	-	public supply
57-4940	607109	4919576	924	public supply
57-4941	607079	4919551	598	public supply
57-8108	605480	4904650	768	commercial
57-9006	605100	4904220	227	commercial
57-11777	606217	4894427	76	municipal
57-11853	605600	4904900	1930	municipal

4.3 Groundwater Impact Assessment

Based on the above information, a groundwater impact assessment has been carried out. The purpose of the assessment has been to determine, to the extent possible based on the accessible information, the following:

- Expected areas of interception, draw-down, and impoundment of groundwater;
- Expected areas of obstruction to groundwater recharge and discharge;
- Likelihood and significance of detrimental effects of interception, draw-down, impoundment and obstruction to recharge and discharge of groundwater;
- Likelihood and significance of release of contaminants to groundwater;
- Likelihood and significance of interference with private/municipal wells;
- Impacts of areas of high groundwater table on the project; and
- Temporary and permanent construction impacts.

The project involves the widening of the existing six lane divided highway to an eight lane divided highway resulting in expanded paved surfaces. The specific geometry of the proposed widening has not yet been decided.

Subsurface works expected in conjunction with the widening are anticipated to consist of topsoil stripping, cutting and filling earthwork, culvert replacements, and ditch excavation. No significant excavations are anticipated. In general, it is expected that construction impacts are likely to stem from reduced recharge of groundwater under asphalt pavement, quality impacts from road runoff, including automotive fluids and road salt, and construction impacts, either temporary or long-term, from grade changes.

4.3.1 Impact on Groundwater Recharge and Discharge

A reduction of groundwater recharge to the subsurface will occur as a result of the construction of relatively impermeable asphalt pavements. The quantum of this reduction is expected to be variable throughout the project limits, but will be highest in areas of most significant recharge, namely within the Camp Borden sand plain area of the site between about Station 29+000, Township of Innisfil and Station 13+000, Township of Vespra.

In general, the relatively impermeable area covered by the asphalt pavement surface, in the context of the regional recharge in the vicinity of the project, is not considered to be significant. Recharge lost to impermeable surfaces will, in part, be mitigated by direction of runoff to ditches where some additional recharge above what is currently occurring can be anticipated.

Based on a relatively large area from which the deep overburden water wells and streams derive groundwater recharge, this reduction in overall groundwater recharge is not expected to be significant. The potential reduction in recharge is not anticipated to produce a measurable effect on groundwater recharge and discharge functions including base flow in streams and water well supply quantity.

The construction of enhanced infiltration structures along the road allowance could further mitigate the potential groundwater recharge reduction. In general, these structures function best in areas of permeable soils and deep water table conditions. Such conditions typically occur in the Camp Borden sand plain area of the project, namely within the City of Barrie area.

4.3.2 Water Well Interference

The most susceptible wells to either quantity or quality interference related to the proposed highway construction are the shallow overburden wells. As shown, these wells are distributed along much of the project area. In general, deep overburden and bedrock wells should not be

susceptible to short or long-term quality or quantity interference effects related to the construction.

The nature of the construction expected (as outlined above) is such that no extensive excavation below the water table is expected outside of identified areas of high water table. As such, no significant lowering of the water table is anticipated and no corresponding reduction in available water supply in shallow overburden wells should be experienced. If deep excavations or permanent service installations below the water table were to be carried out, then some long-term lowering of the water table in the vicinity of these installations may result. This may result in a corresponding reduction in groundwater supplies in shallow wells, but only generally in those wells located within approximately 50 metres of the installation. If such excavations or installations are anticipated, the actual subsurface soil and groundwater conditions in those areas should be assessed in conjunction with the nearby wells to provide a prediction of potential impacts.

During construction, vibration of heavy equipment could potentially produce some mobilization of sediment within nearby wells. Equipment vibration has the potential to affect both shallow and deep, drilled or dug wells. Mobilization of sediment could also result in elevated bacteriological analyses on well water. This condition should be temporary and will subside after the cessation of the vibrations.

The application of road de-icing salt has the potential to affect neighbouring wells, in particular shallow wells (i.e. less than 15 metres deep) and is discussed in further detail below.

4.3.3 Potential For Groundwater Contamination

Shallow wells located near the proposed right-of-way may be susceptible to impact by road de-icing salt application. Chloride, which is highly mobile in the subsurface, is a major constituent of road salt. Chloride at high concentrations (greater than 250 milligrams per litre) may produce an impact on the taste of water. Sodium, which is the other major constituent of road salt is less mobile in the subsurface, but is known to contribute to hypertension at elevated concentrations.

Because of the mobility of road salt constituents, mitigation of road salt impacts is difficult. To the extent possible, road salt application in areas of significant groundwater recharge within the right-of-way should be closely monitored and managed to prevent over application of road salt. Further, the presence of dissolved salt in runoff may lessen the desirability of enhanced groundwater recharge structures. The overall impact of road salt application can be mitigated by diverting, to the extent possible, surface runoff to surface water bodies in areas of groundwater discharge.

Mobile vehicle re-fuelling during construction presents a risk of impact to local wells as a result of accidental releases of fuel. In general, only catastrophic, large volume releases (i.e. greater than 100 litres) are likely to have an adverse impact on local water well supplies. The risk can all but be eliminated or managed by allowing re-fuelling only in designated areas preferably situated on a paved, impermeable surface and by having an emergency response plan in place to clean up any releases of fuel. Shallow wells in areas of relatively higher water table such as the northern and southern portions of the project area are potentially the most susceptible to fuel impacts. As well, the Camp Borden sand plain area of the project area forms a significant regional groundwater recharge area. These factors should be considered in the management of fuel impacts.

4.3.4 Impact of Areas of High Water Table

Areas of high water table, as noted above in Section 4.1 and as indicated on Figure 5, may affect construction progress and technique. These areas are, in general, expected to be limited to the northern and southern portions of the site, in particular, the low lying areas associated with the Cookstown Hollow swamp and the Little Lake pond area. The significance of the areas as they may affect construction should be re-assessed once site specific borehole data for the detailed design of the project have been obtained.

As discussed above, dewatering activities in areas of high water table, either active or as part of construction, or passive ongoing dewatering as a result of the construction of buried utilities have the potential to adversely impact nearby shallow water wells.

4.3.5 Temporary and Permanent Construction Impacts

As outlined in the foregoing sections, the proposed Highway 400 widening has the potential to produce some relatively minor impacts on existing groundwater resources. These impacts are expected to be localized and should affect primarily those wells/resources located in the immediate vicinity of the project works (i.e. within 100 metres of actual proposed construction works). Precautions to mitigate the potential impacts have been outlined in the foregoing sections of the report. All of these issues should be re-examined in light of actual subsurface borehole data and a detailed inventory of water wells obtained at the detail design stage of the project. In summary, the expected potential groundwater impacts from the proposed construction works are as follows:

- Some reduction in potential groundwater recharge within the project limits as a result of expanded relatively impermeable pavement surfaces is expected. Given the anticipated area to be paved relative to the regional recharge area, the reduction is not expected to be measurable;

- Some physical interference with shallow water well supplies in the immediate vicinity of the work should be expected to occur as a result of dewatering through excavation and disturbance of sediment from equipment vibration. These impacts are expected to be temporary, unless permanent buried utility excavations, which could have a dewatering effect, are proposed in areas of high water table; and
- Groundwater quality impacts should be anticipated from the proposed work resulting from road salt application and vehicle re-fuelling activities during construction.

Attachments:

Table I
Figures 1 to 5

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TABLE I

MOE WATER WELL RECORDS

Highway 400 Planning Study
GWP 30-95-00

CONCESSION	LOT	WELL NO.	ELEVATION (m)	UTM EASTING	UTM NORTHING	WELL DIAMETER (mm)	WATER FOUND (m)	STATIC LEVEL (m)	PUMP LEVEL (m)	TEST RATE (l/min)	WELL DEPTH (m)	OVERBURDEN/BEDROCK (O/B)	WATER USE
West Gwillimbury Township													
14	5	57-5370	225.6	606501	4894143	101.6	33.5	FLW			45.7	O	DO
14	5	57-7471	225.6	606700	4893340	762.0	3.0	2.1	6.1	75.7	7.6	O	IR
14	6	57-5396	226.2	607116	4894606	177.8	77.7	FLW			77.7	O	CO
14	7	57-8408	230.1	607900	4893245	177.8	89.9	FLW			92.0	O	DO
Innisfil Township													
1	5	57-8774	236.2	606160	4894490	152.4	DRY				61.0	O	
1	5	57-11777	228.6	606217	4894421	101.6	63.7	FLW		75.7	81.7	B	MU
1	5	57-11778	228.6	606244	4894389	254.0	DRY				70.7	O	
1	5	57-866	245.7	606439	4895100	127.0	89.9	7.6	15.2	37.9	90.2	O	ST DO
1	6	57-865	227.1	607316	4894857	152.4	9.8	2.4	23.2	94.6	40.2	O	CO
1	6	57-867	227.1	607321	4894917	152.4	35.1	3.0	30.5	30.3	39.6	O	CO
1	7	57-868	227.1	607315	4895177	203.2	97.5	FLW			103.6	B	CO
1	7	57-869	227.1	607321	4894917	762.0	0.9	0.9		15.1	4.9	O	IN
1	7	57-870	227.1	607471	4895002	101.6	36.6	3.7	13.4	37.9	36.6	O	IN
2	5	57-9351	266.7	606170	4896550	127.0	DRY				56.4	O	
2	5	57-9698	251.5	606180	4896550	127.0	106.7	26.2	45.7	22.7	106.7	O	DO
2	5	57-895	236.2	606186	4895831	762.0	13.4	7.0		7.6	13.7	O	ST DO
2	5	57-14033	251.5	606230	4896500	127.0	147.8	27.1	144.8	15.1	147.8	B	DO
2	6	57-6141	266.7	606340	4896950	762.0	15.2	9.8			15.2	O	DO
3	6	57-6454	280.4	605970	4898160	762.0	3.7	1.8	2.7		3.7	O	DO
3	6	57-7257	283.5	606000	4898080	127.0	12.2	6.7	12.2	7.6	14.6	O	DO
3	7	57-910	263.7	606993	4897606	762.0	15.2	11.6		7.6	15.2	O	ST DO
4	5	57-13595	291.1	605250	4899750	152.4	16.5	6.4	10.4	37.9	16.5	O	DO
4	6	57-938	289.6	605891	4899632	762.0	6.1	6.1		7.6	9.8	O	ST DO
4	7	57-941	291.1	606694	4899028	762.0	19.8	16.8		11.4	19.8	O	ST DO
5	6	57-11926	291.1	605550	4901350	762.0	10.7	4.9			11.3	O	DO
5	7	57-996	294.1	606447	4900356		10.7	9.1		3.8	18.9	O	ST DO
6	6	57-5418	304.8	605150	4902550	762.0	7.6	3.7			13.7	O	DO
6	7	57-1049	294.1	606256	4901938	762.0	11.6	6.7		11.4	11.6	O	ST DO
7	5	57-11502	303.3	604750	4902800	762.0	10.1	3.7			13.4	O	ST
7	5	57-1071	303.3	604777	4903527	152.4	20.1	6.4	18.3	30.3	21.9	O	DO
7	6	57-10894	303.3	604976	4903291	152.4	63.4	26.8		75.7	88.4	O	CO
7	6	57-5630	304.8	605050	4904020	177.8		3.7	31.7	94.6	33.5	O	CO
7	6	57-10050	303.3	605055	4903084		DRY				91.4	O	
7	6	57-1072	304.8	605140	4903971	762.0	9.1	9.1		7.6	13.7	O	ST DO
7	7	57-8737	297.2	605670	4904190	152.4	59.1				94.5	O	
7	7	57-8736	300.2	605680	4903870	152.4	63.7				94.5	O	
7	7	57-8735	300.2	605750	4903900	152.4					74.7	O	
7	7	57-8738	304.8	605850	4903450	152.4		32.3	33.2	41.6	74.7	O	
7	7	57-7421	286.5	605870	4904170	152.4	58.8	23.8		227.1	97.5	O	CO

Golder Associates

MOE WATER WELL RECORDS

CONCESSION	LOT	WELL NO.	ELEVATION (m)	UTM EASTING	UTM NORTHING	WELL DIAMETER (mm)	WATER FOUND (m)	STATIC LEVEL (m)	PUMP LEVEL (m)	TEST RATE (l/min)	WELL DEPTH (m)	OVERBURDEN/BEDROCK (O/B)	WATER USE
8	5	57-5725	304.8	604050	4905150	152.4	20.4	8.5	20.4	15.1	21.3	O	DO
8	6	57-10594	303.3	604623	4904750	127.0	65.2	28.0	33.5	22.7	65.2	O	DO
8	6	57-1127	303.2	604639	4904684	762.0	7.3	6.4		7.6	16.2	O	DO
8	6	57-1126	304.8	604774	4904008	762.0	10.4	7.6		7.6	13.7	O	DO
8	6	57-9006	304.8	605100	4904220	177.8	73.2	26.8		227.1	78.6	O	CO
8	6	57-9007	304.8	605140	4904230	177.8					78.6	O	
8	7	57-8108	301.8	605480	4904650	152.4	73.2	24.4	35.4	768.4	118.9	O	CO
8	7	57-8107	301.8	605480	4904720	50.8		21.9			79.2	O	
8	7	57-8739	303.3	605500	4904825	50.8	57.9	24.4			88.4	O	
8	7	57-11853	298.7	605600	4904900	254.0	57.9	25.6	53.9	1930.4	91.4	O	MU
9	5	57-1176	312.4	603656	4906406	101.6	40.2	31.7	32.6	11.4	40.2	O	ST DO
9	5	57-1177	310.9	604158	4906189	152.4	54.6	33.5	49.4	189.3	77.7	O	DO
10	5	57-14396	310.9	603600	4906550	152.4	44.8	30.5	41.1	22.7	44.8	O	DO
10	5	57-13133	312.4	603660	4907850	914.4	39.9	35.7	39.6	37.9	42.7	O	DO
10	5	57-14349	310.9	603700	4906550	152.4		30.5	39.6	22.7	50.9	O	DO
10	5	57-7034	310.9	603850	4906600	762.0	12.5	7.9			13.1	O	DO
10	5	57-8465	304.8	604090	4907300	152.4	44.2	34.4	42.1	30.3	45.7	O	DO
10	5	57-1252	307.8	604102	4907076	152.4	57.3	31.4	48.8	56.8	58.2	O	DO
10	5	57-1251	310.9	604180	4906760	101.6	39.3	31.7	35.4	30.3	39.3	O	DO
10	5	57-5928	310.9	604200	4906700	152.4	48.8	31.7	36.6	26.5	58.5	O	DO
10	6	57-1255	306.3	604156	4907281		34.7	34.7	37.8	34.1	40.8	O	DO
10	6	57-1253	309.4	604365	4906772	101.6	DRY				42.7	O	
10	6	57-1254	309.4	604415	4906757		42.1	32.0	41.1	26.5	43.0	O	DO
10	7	57-1256	304.8	605240	4907058	101.6	21.3	15.8	16.8	22.7	27.4	O	DO
11	5	57-1390	309.4	603548	4909041	101.6	9.1	9.1	13.1	11.4	15.2	O	DO
11	5	57-1389	309.4	603603	4909042	863.6	7.6	7.6		15.1	11.3	O	DO
11	5	57-1388	309.4	603753	4908316	101.6	36.6	36.6	44.2	30.3	47.9	O	ST DO
11	5	57-1391	310.9	603823	4908442	152.4	43.6	37.2	42.7	30.3	44.5	O	ST DO
11	6	57-8895	312.4	603870	4908800	127.0	41.1	38.7	41.1		46.9	O	DO
11	6	57-1392	307.8	604027	4909405	101.6	13.7	3.7	12.2	30.3	13.7	O	DO
11	8	57-1394	289.6	605197	4909744		33.2	19.5	33.5	7.6	34.1	O	ST DO
12	5	57-14922	313.9	602900	4910550	762.0	4.6	3.0			7.6	O	DO
12	5	57-1443	313.9	602967	4910524	762.0	4.6	4.6		11.4	9.1	O	DO
12	5	57-1435	313.9	603003	4910522		18.0	14.3	16.8	15.1	20.1	O	DO
12	5	57-12158	313.9	603100	4910600	762.0	7.0				10.7	O	DO
12	5	57-1444	313.9	603107	4910602	762.0	7.6	4.6	6.1	7.6	7.6	O	CO
12	5	57-1445	313.9	603190	4910671	152.4	23.5	14.6	21.3	37.9	24.4	O	ST DO
12	5	57-14107	312.4	603400	4910570	127.0	46.6	42.7	44.2	22.7	50.0	O	DO
12	5	57-1441	312.4	603441	4910492	101.6	DRY				36.6	O	
12	5	57-1442	312.1	603446	4910480	127.0	DRY				145.4	O	
12	5	57-1446	309.4	603495	4910215		49.1	42.4	47.2	37.9	52.1	O	DO
12	5	57-1440	310.9	603516	4910035		50.9	42.4	46.9	15.1	50.9	O	DO
12	5	57-8231	307.8	603600	4909630	101.6	17.7	6.7	13.7	18.9	18.3	O	DO
12	6	57-13455	304.8	603740	4910850	127.0	32.0	32.0	39.6	18.9	44.5	O	DO

Golder Associates

Table 1 Continued

MOE WATER WELL RECORDS

CONCESSION	LOT	WELL NO.	ELEVATION (m)	UTM EASTING	UTM NORTHING	WELL DIAMETER (mm)	WATER FOUND (m)	STATIC LEVEL (m)	PUMP LEVEL (m)	TEST RATE (l/min)	WELL DEPTH (m)	OVERBURDEN/BEDROCK (O/B)	WATER USE
12	6	57-8284	304.8	604100	4909550	152.4	42.7	35.4	39.9	53.0	43.9	O	DO
12	6	57-7578	306.3	604150	4909580	762.0	4.6	1.5	4.0	18.9	4.6	O	DO
12	7	57-1447	300.2	603620	4909888	762.0	1.8	1.8		7.6	6.1	O	DO
12	8	57-1448	283.3	605079	4911239	101.6	27.4	13.1	14.6	15.1	27.4	O	ST DO
13	5	57-1708	297.2	602611	4911922	177.8	37.2	28.0	36.0	26.5	38.4	O	DO
13	5	57-1522	262.1	602616	4911918	152.4	15.2	0.6	13.4	26.5	15.5	O	DO
13	5	57-1518	266.7	602702	4911952	101.6	14.9	3.7	8.5	15.1	14.9	O	DO
13	5	57-14481	266.7	602750	4911950	127.0	18.3	3.4	9.1	56.8	19.2	O	DO
13	5	57-7925	266.7	602750	4911980	152.4	21.6	3.7	18.9	37.9	22.6	O	DO
13	5	57-14024	265.2	602900	4912010	152.4	17.7	3.4	13.7	37.9	17.7	O	DO
13	5	57-1523	263.7	602921	4912027	762.0	2.7	2.7		7.6	5.5	O	DO
13	5	57-10466	256.0	602955	4912040	762.0	4.6	2.4		11.4	6.1	O	DO
13	5	57-5942	237.7	602980	4912550	101.6	15.2	5.2	10.7	18.9	15.2	O	DO
13	5	57-13139	297.2	603100	4911680	152.4	33.5	32.0	36.6	75.7	40.8	O	DO
13	5	57-1519	281.9	603108	4911854	50.8	28.7	21.3		37.9	30.5	O	DO
13	5	57-1520	286.5	603117	4911814	101.6	30.5	27.4	30.5	18.9	35.1	O	DO
13	5	57-7953	274.3	603130	4911950	101.6	18.3	11.6	11.6	37.9	18.3	O	DO
13	5	57-1524	285.0	603145	4911809	101.6	27.4	27.4	30.5	15.1	36.6	O	DO
13	5	57-9561	281.9	603150	4911860	101.6	29.9	17.7	21.3	18.9	29.9	O	DO
13	5	57-1525	289.6	603155	4911717	101.6	27.4	24.4	27.4	37.9	30.5	O	DO
13	5	57-1521	298.7	603167	4911787	762.0	DRY				27.4	O	
13	5	57-6557	313.9	603200	4910760	762.0	15.8	12.8	14.6		15.8	O	DO
13	5	57-1516	307.8	603212	4911082	101.6	19.8	17.4	17.4	15.1	22.9	O	DO
13	5	57-1517	313.9	603235	4910774	101.6	20.4	15.5	17.1	15.1	26.8	O	PS
13	5	57-13465	310.9	603250	4910950	127.0	15.2	13.7	17.7	15.1	22.3	O	DO
13	5	57-14003	310.9	603340	4910800	762.0	2.4	1.8	6.1	11.4	7.9	O	DO
13	5	57-14002	310.9	603350	4910840	762.0	2.4	1.8	9.1	11.4	11.3	O	DO
13	6	57-7677	306.3	603390	4911350	762.0	12.2	8.5	11.0	22.7	12.2	O	DO
13	6	57-13194	307.8	603400	4911010	127.0	14.0	11.3	15.7	15.1	18.6	O	DO
13	6	57-8794	306.3	603500	4911330	762.0	5.5	5.5	7.9	7.6	9.1	O	DO
13	6	57-1528	307.8	603510	4910878	101.6	40.2	40.2	44.2		47.5	O	DO
13	6	57-1529	286.5	603547	4911943	101.6	20.7	12.8	16.2	22.7	20.7	O	CO
13	6	57-1532	307.8	603573	4911395	152.4	53.3	44.8	54.3	22.7	54.6	O	DO
13	6	57-7449	306.3	603600	4910900	101.6	51.8	42.7	45.7	18.9	51.8	O	DO
13	6	57-14186	307.8	603600	4910900	127.0	36.6	36.6	44.2	22.7	48.2	O	DO
13	6	57-7567	306.3	603600	4911350	762.0	14.0	11.0	13.4	22.7	14.0	O	DO
13	6	57-11221	307.8	603612	4910827	127.0	40.5	40.5	42.7	26.5	48.8	O	DO
13	6	57-1526	307.8	603619	4911470	101.6	53.9	42.1	43.6	18.9	53.9	O	DO
13	6	57-1527	304.8	603670	4911501	101.6	22.6	22.6	26.2	7.6	28.0	O	DO
13	6	57-1531	265.2	603685	4912112		15.5	9.1	15.2	53.0	16.8	O	ST DO
13	6	57-11172	306.3	603698	4910858	127.0	40.5	40.5	42.7	26.5	49.1	O	DO
13	6	57-8282	289.6	603700	4911750	203.2	36.0	23.8	32.9	75.7	38.4	O	DO
13	6	57-1530	281.9	603764	4911980	152.4	DRY				12.2	O	
13	6	57-13884	306.3	603800	4911400	152.4	64.0	43.9	53.3	30.3	74.7	O	CO DO

Golder Associates

Table 1 Continued

MOE WATER WELL RECORDS

CONCESSION	LOT	WELL NO.	ELEVATION (m)	UTM EASTING	UTM NORTHING	WELL DIAMETER (mm)	WATER FOUND (m)	STATIC LEVEL (m)	PUMP LEVEL (m)	TEST RATE (l/min)	WELL DEPTH (m)	OVERBURDEN/BEDROCK (O/B)	WATER USE
13	7	57-1533	262.1	603715	4912243	152.4	68.3	45.7	51.8	30.3	68.3	O	CO
13	7	57-1538	281.9	604297	4912456	101.6	17.1	13.4	13.7	90.8	23.2	O	PS
13	7	57-1534	292.6	604496	4911434	101.6	30.5	25.9	32.0	37.9	33.8	O	IR
13	7	57-1536	292.6	604544	4911568	203.2	35.1	18.3	27.4	227.1	44.5	O	PS
13	7	57-1535	292.6	604556	4911620	101.6	38.1	18.3	42.7	151.4	43.6	O	IR
13	7	57-1537	280.4	604559	4911970	101.6	29.0	20.1	21.0	60.6	33.8	O	DO
14	5	57-1692	234.7	602375	4913451	152.4	71.0	2.4	56.4	15.1	91.4	O	CO
14	5	57-11200	233.2	602500	4913450	127.0	25.9	7.6	13.7	37.9	28.0	O	DO
14	5	57-1694	234.7	602541	4913510	50.8	18.3	2.4	6.7	30.3	19.8	O	DO
14	5	57-11395	233.2	602550	4913450	127.0	18.9	7.3	14.0	18.9	20.1	O	DO
14	5	57-1711	259.1	602575	4911990	762.0	1.8	1.8		15.1	6.1	O	DO
14	5	57-7116	236.2	602600	4913240	762.0	5.8	3.0	4.3		5.8	O	DO
14	5	57-1693	234.7	602608	4913515	50.8	18.3	2.4	6.7	30.3	19.8	O	DO
14	5	57-1710	262.1	602641	4912049	152.4	16.8	0.3	9.1	26.5	16.8	O	DO
14	5	57-10573	236.2	602650	4913230	762.0	6.1	5.5			9.4	O	DO
14	5	57-10574	236.2	602650	4913230	762.0	2.1	2.1			4.3	O	DO
14	5	57-10575	236.2	602650	4903230	762.0	4.9	4.3			6.7	O	DO
14	5	57-6395	236.2	602650	4913500	127.0	18.9	7.6	16.8	22.7	21.6	O	DO
14	5	57-6999	236.2	602660	4913240	762.0		3.0	5.5		6.1	O	DO
14	5	57-1697	234.7	602682	4913481	127.0	DRY				77.1	O	
14	5	57-1696	234.7	602682	4913512	50.8	19.8	3.7	7.3	94.6	71.6	O	IR
14	5	57-11461	236.2	602700	4913150	127.0	22.9	9.1	12.2	37.9	25.6	O	DO
14	5	57-12388	236.2	602700	4913250	101.6	22.9	11.0	13.7	18.9	24.1	O	DO
14	5	57-7451	234.7	602710	4913520	101.6	21.6	4.6	9.1	37.9	22.9	O	CO
14	5	57-6796	236.2	602720	4913530	127.0	18.9	7.6	16.8	7.6	21.9	O	DO
14	5	57-12390	236.2	602730	4913250	101.6	25.9	11.6	22.9	18.9	27.4	O	DO
14	5	57-12325	239.3	602750	4912850	101.6	24.4	10.7	16.8	37.9	25.9	O	DO
14	5	57-12406	236.2	602750	4913150	101.6	28.0	9.8	25.9	18.9	29.6	O	DO
14	5	57-12557	236.2	602750	4913150	101.6	29.0	12.2	24.4	22.7	29.9	O	DO
14	5	57-12391	236.2	602750	4913250	101.6	25.3	10.1	21.3	26.5	27.4	O	DO
14	5	57-1713	259.1	602774	4912075	101.6	11.3	0.9	1.2	56.8	12.8	O	DO
14	5	57-1720	239.3	602800	4912894	152.4	17.4	6.1	19.8	15.1	20.7	O	PS
14	5	57-12392	234.7	602800	4913270	101.6	28.3	9.1	21.3	30.3	29.9	O	DO
14	5	57-1715	259.1	602806	4912097	101.6	11.0	2.7	6.1	11.4	11.6	O	DO
14	5	57-5552	234.7	602810	4913370	101.6	21.9	10.7	16.8	18.9	23.8	O	DO
14	5	57-12393	234.7	602820	4913300	101.6	24.4	11.9	21.9	22.7	26.5	O	DO
14	5	57-1706	239.3	602838	4912799	152.4	19.8	6.7	17.4	26.5	20.7	O	DO
14	5	57-1704	239.3	602839	4912728	152.4	16.2	4.6	15.2	56.8	17.4	O	DO
14	5	57-9524	240.8	602850	4912770	127.0	17.4	7.6	13.7	37.9	21.6	O	DO
14	5	57-14607	240.8	602850	4912900	101.6	25.9	10.1	15.2	56.8	27.1	O	DO
14	5	57-12553	236.2	602850	4913200	101.6	29.6	10.1	18.3	37.9	30.5	O	DO
14	5	57-12389	234.7	602850	4913300	101.6	20.7	14.0	18.3	22.7	22.3	O	DO
14	5	57-1705	240.8	602853	4912619	152.4	15.8	6.1	14.6	53.0	17.1	O	DO
14	5	57-1703	240.8	602855	4912642	152.4	15.5	4.6	13.7	37.9	16.5	O	DO

Golder Associates

Table 1 Continued

MOE WATER WELL RECORDS

CONCESSION	LOT	WELL NO.	ELEVATION (m)	UTM EASTING	UTM NORTHING	WELL DIAMETER (mm)	WATER FOUND (m)	STATIC LEVEL (m)	PUMP LEVEL (m)	TEST RATE (l/min)	WELL DEPTH (m)	OVERBURDEN/BEDROCK (O/B)	WATER USE
14	5	57-1718	240.8	602865	4912591	152.4	19.5	9.1	20.4	15.1	21.3	O	DO
14	5	57-1716	237.7	602865	4913011	101.6	17.7	6.7	9.1	37.9	17.7	O	DO
14	5	57-1712	237.7	602868	4913055	152.4	12.8	6.1	12.8	15.1	13.7	O	DO
14	5	57-1698	240.8	602881	4912568	152.4	14.0	6.1	13.1	34.1	14.0	O	DO
14	5	57-1699	240.8	602886	4912508	152.4	14.0	6.1	13.7	37.9	14.0	O	DO
14	5	57-9738	236.8	602900	4913050	152.4	12.8	6.1	12.2	15.1	19.2	O	DO
14	5	57-7887	243.8	602940	4912360	762.0		5.5	8.2		9.1	O	DO
14	5	57-14479	251.5	602950	4912300		16.8	FLW	1.5	18.9	20.7	O	DO
14	5	57-11036	237.7	602950	4912850	101.6	12.2	8.5	13.7	11.4	16.8	O	DO
14	5	57-1717	254.5	602955	4912230	152.4	19.2	10.4	20.1	7.6	20.7	O	DO
14	5	57-1707	237.7	602960	4912778	152.4	18.6	5.8	18.3	18.9	19.8	O	DO
14	5	57-9522	237.7	602960	4912290	127.0	18.9	9.1	14.3	15.1	18.9	O	DO
14	5	57-1695	243.8	602963	4912145	152.4	14.3	FLW	9.1	37.9	14.3	O	DO
14	5	57-1700	240.8	602978	4912694	152.4	15.5	4.6	11.9	18.9	16.5	O	DO
14	5	57-1701	237.7	602982	4912715	152.4	13.1	4.3	12.2	37.9	14.0	O	DO
14	5	57-1709	243.8	602994	4912466	101.6	43.9	13.4	16.8	22.7	43.9	O	DO
14	5	57-1714	243.8	603007	4912463	152.4	14.9	7.6	15.8	15.1	17.7	O	DO
14	5	57-7768	243.8	603010	4912390	762.0	9.1	5.5	7.9	22.7	9.1	O	DO
14	5	57-1691	240.8	603015	4912541	101.6	61.0	3.0		15.1	61.0	O	DO
14	5	57-5572	251.5	603050	4912300	152.4	19.5	10.1	19.8	7.6	20.4	O	DO
14	5	57-1719	262.1	603058	4912172	152.4	13.4	1.8	12.2	37.9	14.6	O	DO
Vespra Township													
3	21	57-4613	265.2	604528	4918399	762.0	9.8	9.8		7.6	14.0	O	
3	21	57-4612	262.1	604697	4918520	152.4	DRY				48.2	O	
3	21	57-4611	262.1	604712	4918525	152.4	DRY				86.9	O	
4	20	57-9959	268.2	603970	4918780	152.4	46.6	29.0	46.6	15.1	47.5	O	DO
4	20	57-6133	266.7	604010	4918700	101.6	36.6	32.0	35.1	30.3	41.1	O	DO
4	20	57-5662	266.7	604020	4918700	152.4	62.5	28.3	64.0	11.4	64.9	O	DO
4	20	57-4724	266.7	604027	4918689	152.4	62.5	32.0	45.7	56.8	63.4	O	DO
4	20	57-9419	266.7	604046	4918712	762.0	14.3	14.3	16.2	11.4	18.9	O	DO
4	20	57-4725	269.7	604096	4918601	152.4	47.5	35.1	46.3	22.7	48.8	O	DO
Vespra Township - West of Penetanguishene Road													
1	4	57-4919	266.7	607582	4917937	152.4	39.0	21.3	36.0	30.3	41.1	O	DO
1	4	57-9618	260.6	607600	4918010		DRY				20.7	O	
1	4	57-4918	266.7	607633	4918032	127.0	29.6	29.0	33.5	30.3	44.2	O	IN
1	5	57-4922	243.8	605950	4918787	762.0	2.1	2.1		7.6	7.0	O	DO
1	5	57-4920	236.2	605965	4918859	762.0	1.8	1.8		7.6	5.2	O	MU
1	5	57-4923	239.3	605978	4918829	762.0		3.4		7.6	6.7	O	DO
1	5	57-4921	236.2	605998	4918859	762.0	2.4	2.4		11.4	5.5	O	DO
1	5	57-13399	259.1	606190	4918840	127.0	21.3	21.3	24.4	18.9	29.9	O	DO
1	5	57-4924	239.3	606223	4918854	762.0	12.8	12.8		7.6	20.1	O	DO
1	5	57-10838	242.3	606250	4918850	152.4	21.3	7.3	18.6	75.7	25.9	O	DO

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Table 1 Continued

MOE WATER WELL RECORDS

CONCESSION	LOT	WELL NO.	ELEVATION (m)	UTM EASTING	UTM NORTHING	WELL DIAMETER (mm)	WATER FOUND (m)	STATIC LEVEL (m)	PUMP LEVEL (m)	TEST RATE (l/min)	WELL DEPTH (m)	OVERBURDEN/BEDROCK (O/B)	WATER USE
1	5	57-9039	240.8	606250	4918860	762.0	4.6	4.6	6.1	15.1	8.5	O	DO
1	5	57-9041	240.8	606260	4918850	762.0	5.5	5.5	7.3	15.1	9.1	O	DO
1	5	57-10036	242.5	606265	4918850	762.0	3.0	3.4	5.5	15.1	7.0	O	DO
1	5	57-8875	262.1	606400	4918500	762.0	23.8	2.1	24.7	7.6	25.9	O	DO
1	5	57-3533	259.1	607950	4918394	152.4	42.4	24.4	35.1	56.8	46.6	O	ST DO
1	5	57-6645	261.2	607680	4918580	152.4	34.7	25.9	33.5	37.9	35.1	O	DO
1	6	57-4925	233.2	605770	4918994	101.6	9.4	FLW			9.4	O	CO
1	6	57-4991	233.2	605863	4918974	101.6	9.8	FLW			9.8	O	DO
1	6	57-7297	236.2	606220	4918970	762.0	4.6	3.0	4.0	18.9	4.6	O	DO
1	6	57-6807	236.2	606270	4918940	762.0	1.5	0.3	1.5		3.7	O	DO
1	6	57-4926	236.2	606378	4918909	1397.0	1.5	1.5		7.6	4.0	O	DO
1	6	57-7253	259.1	606710	4918860	152.4	27.4	27.4	32.0	37.9	38.4	O	DO
1	6	57-13400	259.1	606750	4918860	127.0	32.0	29.0	32.9	26.5	42.1	O	DO
1	6	57-6549	251.5	606750	4918880	101.6	45.7	27.4	33.5	30.3	45.7	O	DO
1	6	57-5941	259.1	606850	4918860	101.6	38.1	27.4	30.5	37.9	41.1	O	DO
1	6	57-10077	290.2	606900	4918860	127.0	32.0	30.5	32.3	18.9	36.9	O	DO
1	6	57-4584	262.1	606908	4918868	101.6	36.6	27.4	33.5	18.9	36.6	O	DO
1	6	57-4928	259.7	606913	4918890	101.6	35.1	27.4	30.5	37.9	35.1	O	DO
1	6	57-9525	262.1	606930	4918850	127.0	44.8	30.5	35.1	30.3	44.8	O	DO
1	6	57-8445	262.1	606970	4918840	127.0	54.3	29.3	45.7	18.9	55.5	O	DO
1	6	57-9364	262.1	607010	4918840	101.6	46.9	30.2	33.5	37.9	58.8	O	DO
1	6	57-5646	262.1	607050	4918850	101.6	54.3	36.6	38.1	37.9	54.3	O	DO
1	6	57-5906	262.1	607100	4918830	101.6	27.4	8.2	24.4	11.4	27.4	O	DO
1	6	57-10822	262.1	607150	4918800	127.0	33.5	30.5	33.5	26.5	41.5	O	DO
1	6	57-5905	262.1	607150	4918850	101.6	27.4	25.0	30.5	37.9	35.1	O	DO
1	6	57-6221	262.1	607150	4918820	101.6	36.6	25.9	29.0	37.9	36.6	O	DO
1	6	57-4927	239.3	607724	4918769	127.0	38.1	27.4	27.4	30.3	38.1	O	DO
1	7	57-12307	242.3	607800	4919150	127.0	28.3	9.8	14.3	37.9	30.5	O	DO
1	7	57-14222	236.2	607800	4919300	762.0	14.0	3.7	12.2	18.9	16.5	O	DO
1	7	57-9613	234.7	607800	4919400	152.4	1.8			34.1	5.5	O	
1	8	57-4938	233.2	607069	4919621	203.2					73.2	O	
1	8	57-4941	233.2	607079	4919551	203.2		FLW	0.9	598.0	69.8	O	PS
1	8	57-4940	233.2	607109	4919576	50.8		FLW	0.6	923.5	67.7	O	PS
1	8	57-4934	233.2	607109	4919681	152.4					70.7	O	PS
1	8	57-4937	233.2	607127	4919806	152.4					73.2	O	
1	8	57-4935	233.2	607134	4919673	152.4					73.5	O	
1	8	57-4936	233.2	607140	4919786	152.4					70.4	O	
1	8	57-4932	233.2	607207	4919823	152.4					39.6	O	
1	8	57-4931	233.2	607227	4919836	152.4					33.5	O	
1	8	57-4930	233.2	607245	4919866	304.8					72.2	B	
1	8	57-4939	233.2	607245	4919871	304.8	32.3	FLW	15.2	7.6	57.9	O	
1	8	57-4929	233.2	607247	4919838	152.4					32.3	O	PS
1	8	57-4933	233.2	607252	4919871	152.4					25.9	O	
1	10	57-4943	240.8	607893	4920538	127.0					77.4	O	
1	10	57-4942	240.8	607906	4920530	101.6	26.8	FLW	3.0	41.6	29.6	O	
1	11	57-4944	234.7	607285	4920058			19.5	20.4	11.4	32.0	O	DO

NOTE: Table to be read in conjunction with accompanying report.

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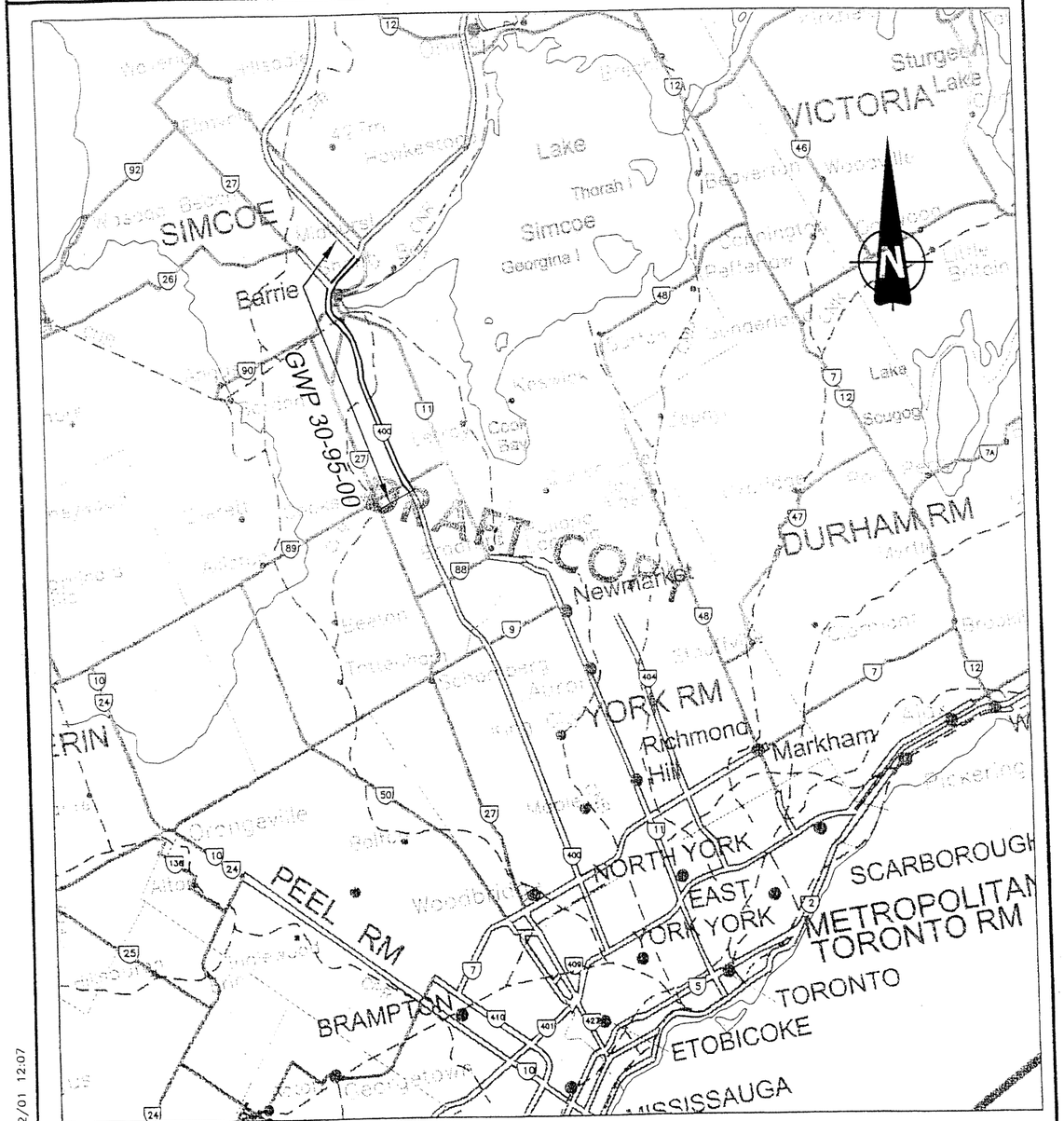
SITE LOCATION PLAN

FIGURE 1



Date MAY 15, 2001

Project 001-3203

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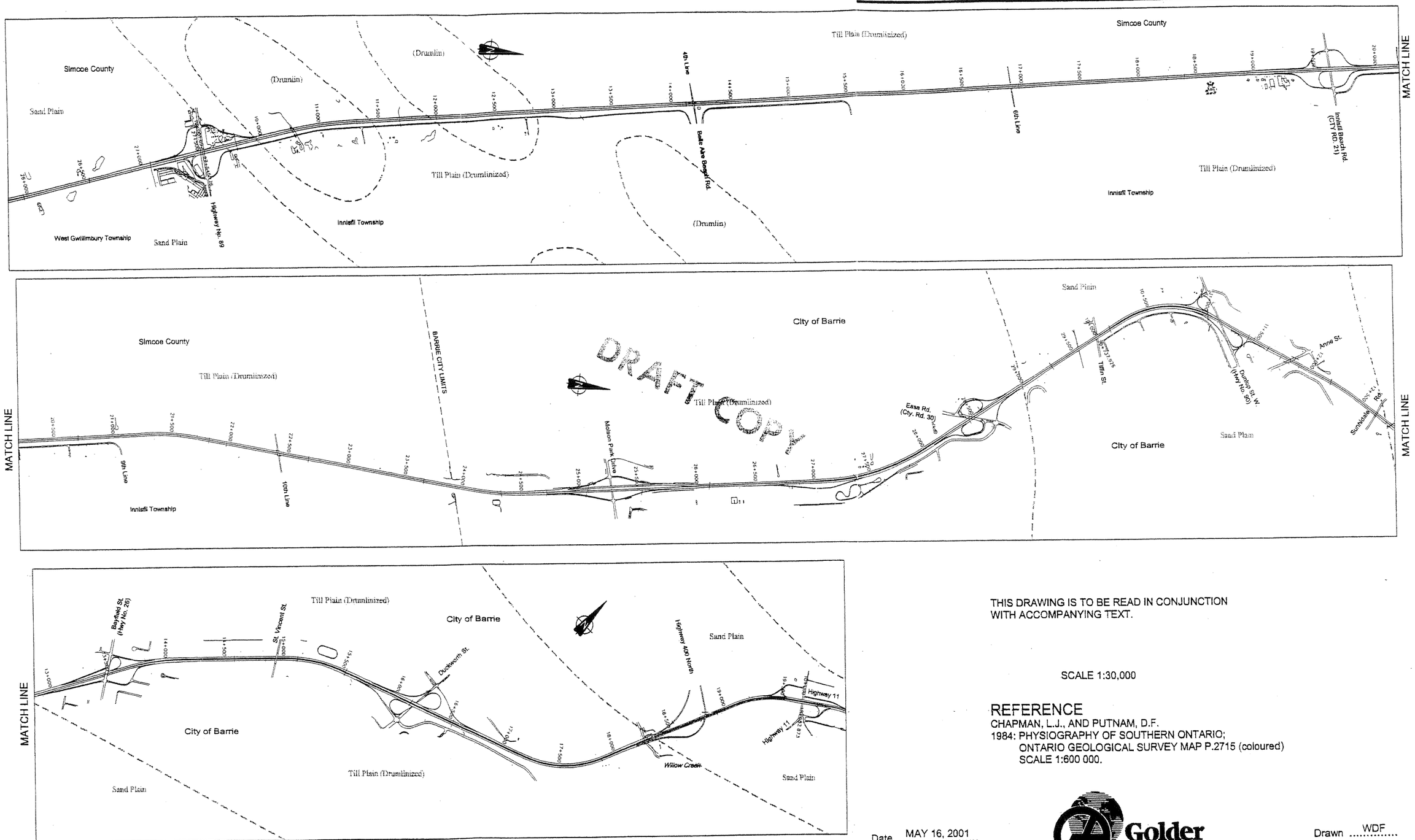
PHYSIOGRAPHY

GWP 30-95-00 - HIGHWAY 400

FIGURE 2



THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING TEXT.

SCALE 1:30,000

REFERENCE

CHAPMAN, L.J., AND PUTNAM, D.F.
1984: PHYSIOGRAPHY OF SOUTHERN ONTARIO;
ONTARIO GEOLOGICAL SURVEY MAP P.2715 (coloured)
SCALE 1:600 000.

Date MAY 16, 2001

Project 001-3203

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Chkd

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REFERENCE
GWYN, Q.H.J., AND WHITE, S.
1973: QUATERNARY GEOLOGY OF THE ALLISTON AREA, SOUTHERN ONTARIO; ONTARIO DIV. MINES, PRELIM. MAP P.835, GEOL SER. SCALE 1:50,000. GEOLOGY 1972

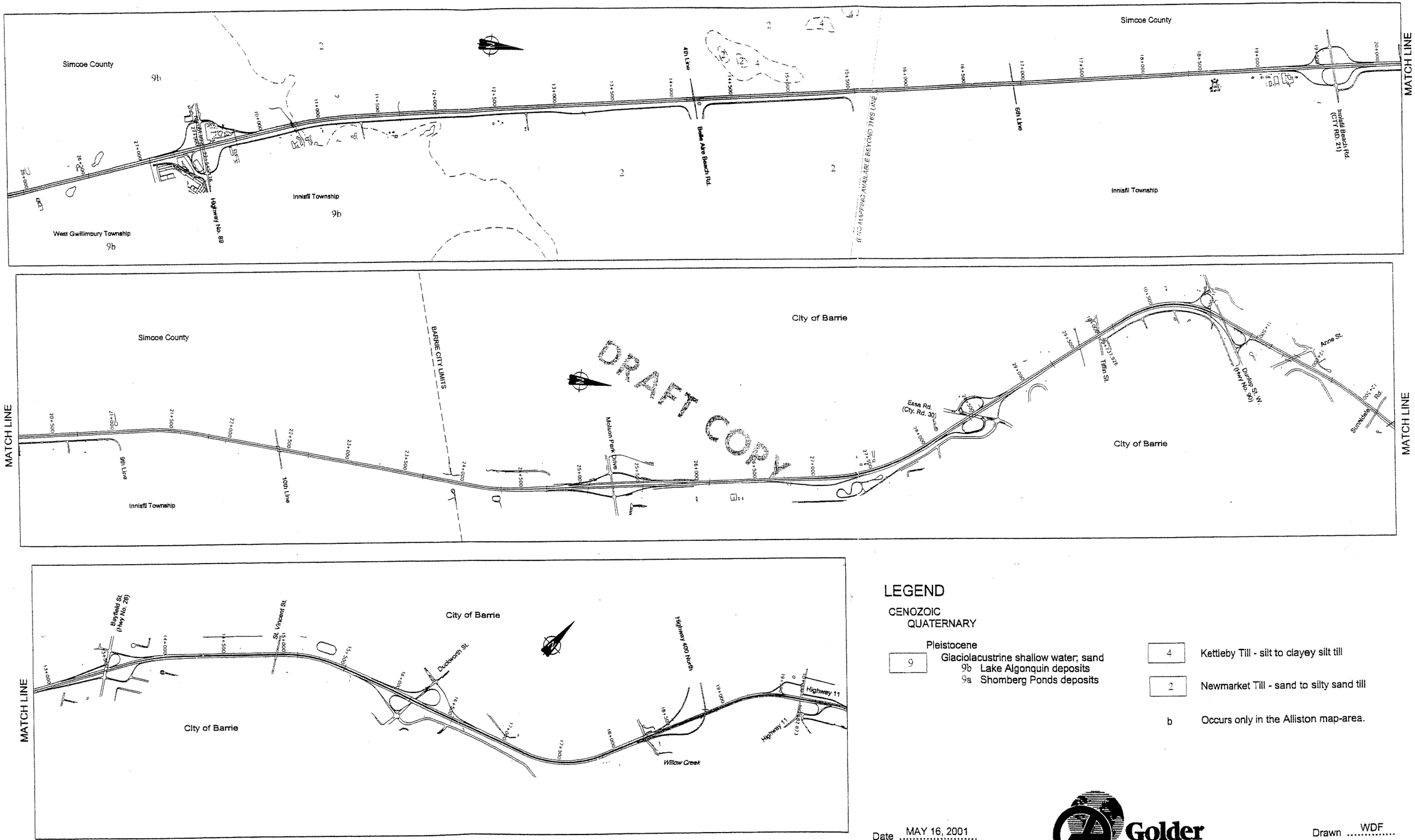
THIS DRAWING IS TO BE READ IN CONJUNCTIO WITH ACCOMPANYING TEXT. .

SCALE 1:30,000

QUATERNARY GEOLOGY

GWP 30-95-00 - HIGHWAY 400

FIGURE 3



LEGEND

CENOZOIC
QUATERNARY

Pleistocene

9	Glaciolacustrine shallow water; sand 9b Lake Algonquin deposits 9a Shomberg Ponds deposits
4	Kettleby Till - silt to clayey silt till
2	Newmarket Till - sand to silty sand till
b	Occurs only in the Alliston map-area.

Date MAY 16, 2001
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REFERENCE
SOIL MAP OF SIMCOE COUNTY
ONTARIO (SOUTH SHEET)
SOIL SURVEY REPORT No. 29
SCALE 1 inch to 1 mile (1:63,360)

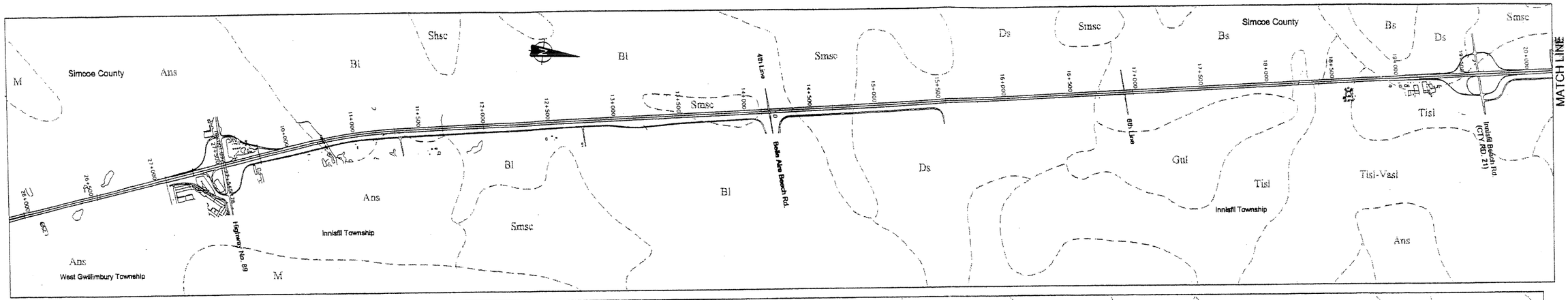
THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING TEXT.

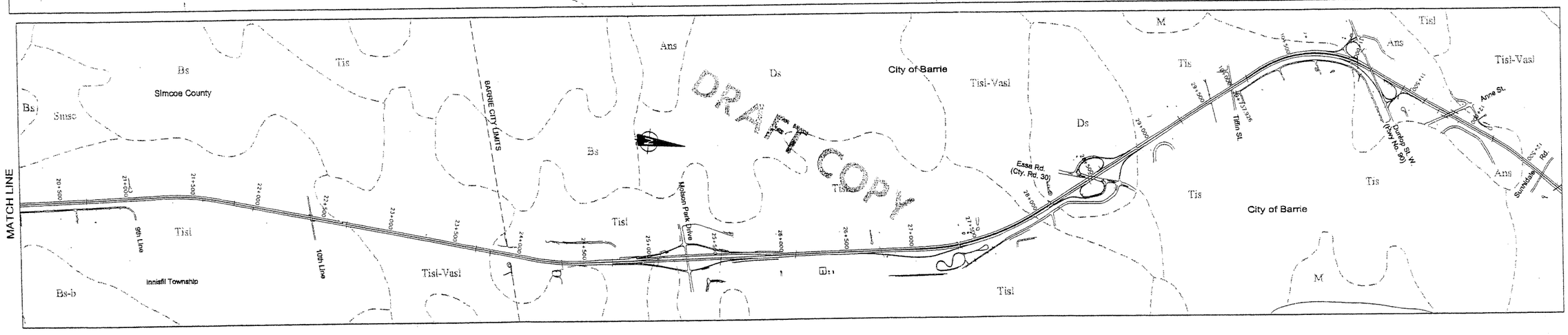
SCALE 1:30,000

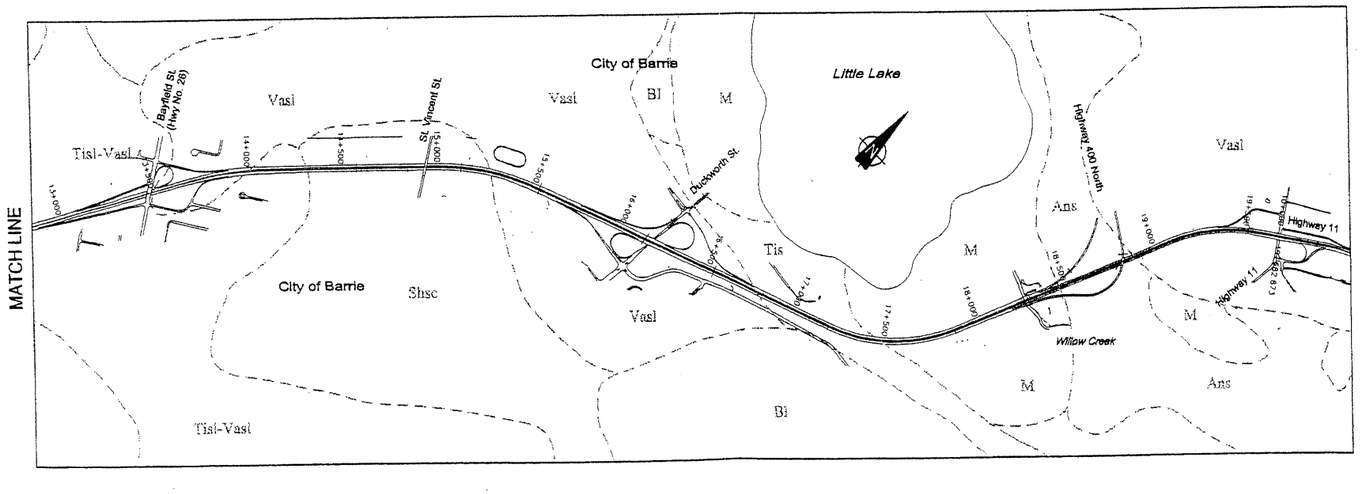
SURFICIAL SOILS

GWP 30-95-00 - HIGHWAY 400

FIGURE 4







LEGEND

Map Symbol	Soil Series	Soil Type
Ans	ALLISTON	sandy loam
Bl	BONDHEAD	loam
Bs	BONDHEAD	sandy loam
Bs-b	BONDHEAD	sandy loam stoney phase
Ds	DUNDONALD	sandy loam
Gul	GUERIN	loam

Map Symbol	Soil Series	Soil Type
M	MUCK	variable
Shc	SCHOMBERG	clay loam
Shsc	SCHOMBERG	silty clay loam
Tis	TIOGA	loamy sand
Tisl	TIOGA	sandy loam
Vasl	VASEY	sandy loam

Date MAY 16, 2001
Project 001-3203

Golder Associates
LONDON, ONTARIO, CANADA

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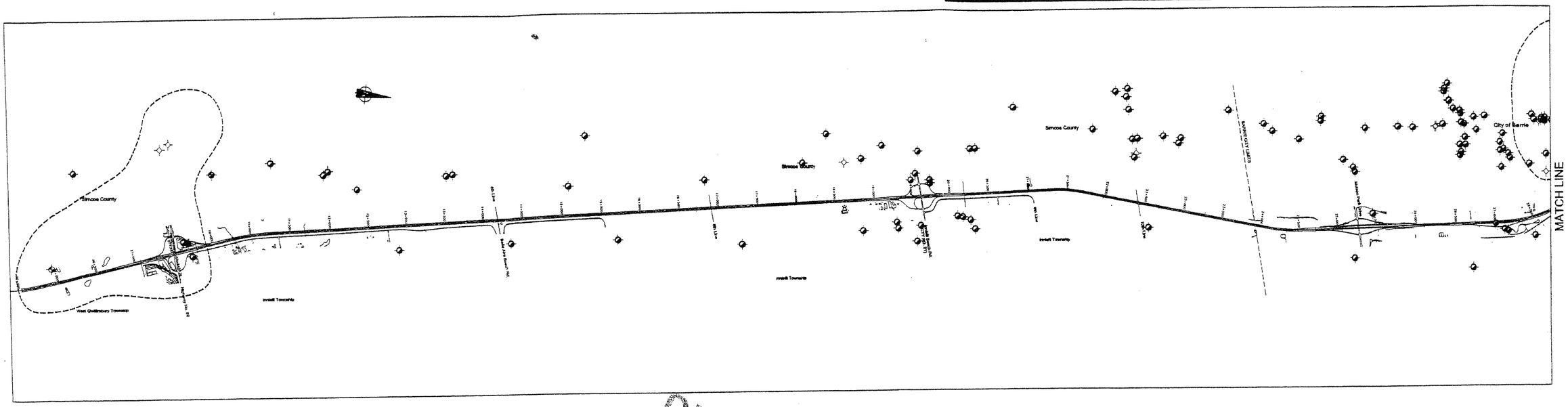
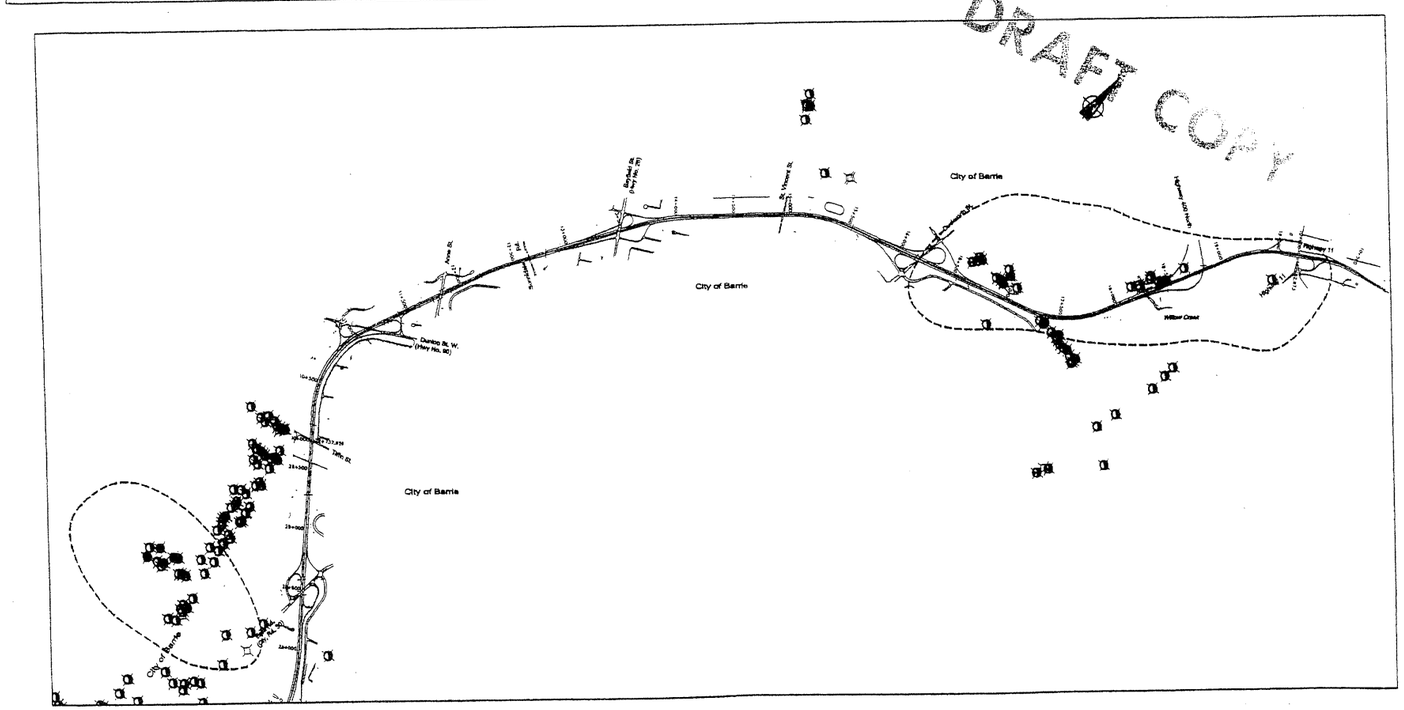
WATER WELL LOCATION PLAN

GWP 30-95-00 - HIGHWAY 400

FIGURE 5





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SCALE 1:50,000

THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING TEXT.

LEGEND

(MOE WATER WELL LOCATION AND DESCRIPTION)

- FLOWING WELLS (ARTESIAN CONDITIONS)
- WATER TABLE LESS THAN 1.5 METRES DEPTH
- WATER TABLE GREATER THAN 1.5 METRES DEPTH
- DRY
- INTERPRETED AREA OF HIGH WATER TABLE

Date MAY 16, 2001

Project 001-3203

Golder Associates

LONDON, ONTARIO, CANADA

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Ontario

Ministry of Transportation
Southwestern Region
Planning and Design Section

HIGHWAY 400 PLANNING AND PRELIMINARY DESIGN STUDY

FROM 1 KM SOUTH OF HIGHWAY 89 NORTHERLY 30 KM TO THE JUNCTION AT HIGHWAY 11

COUNTY OF SIMCOE
TOWN OF BRADFORD WEST GWILLIMBURY
TOWN OF INNISFIL
CITY OF BARRIE
TOWNSHIP OF SPRINGWATER

G.W.P. 30-95-00

APPENDIX T

ALTERNATIVE SERVICE CENTRE LOCATIONS ASSESSMENT

Ontario
Ministry of Transportation
Southwestern Region
Planning and Design Section

Highway 400 Planning Study

GWP 30-95-00 Highway 400 Planning Study
From 1km South of Highway 89 Northerly 30 km
to the Junction of Highway 11

Assessment of Potential Service Centre Locations

September 2003

URS

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2.0 Existing and Future Conditions 2
3.0 Site Requirements 2
4.0 Screening of Potential Candidate Sites 3

Catalogue of Candidate Sites 9

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Exhibit 1. Typical Service Centre Layout 4
Exhibit 2. Potential Service Centre Locations 5
Exhibit 3. Assessment of Potential Service Centre Locations 7

1.0 Introduction

URS was retained by the Ontario Ministry of Transportation (MTO) to undertake a Preliminary Design Study to document the proposed interim and ultimate widening requirements along 30 km of Highway 400 from 1 km south of Highway 89 to the Junction at Highway 11.

There are currently two service centres on Highway 400 within the study corridor. Both service centre locations are owned by MTO and leased to facilities operators. One is located in the northwest quadrant of the Highway 89 interchange. Access to the service centre is available directly from northbound and southbound Highway 400. This service centre will be closed as a result of the Highway 400 improvements. Specifically, the land on which the service centre is located will be required to implement the Highway 89 / Highway 400 interchange improvements.

A second service centre, located between the Molson Park Drive and Essa Road interchanges, serves northbound Highway 400. Closure of the service centre is recommended on the basis that:

- The Essa Road interchange will be modified to address future traffic needs at this interchange. The proposed modifications will reduce the separation between the service centre entrance to Highway 400 and the Essa Road exit from Highway 400 resulting in undesirable traffic operations between these ramp junctions; and
- Many of the services offered by the northbound service centre are readily available in the immediate vicinity of numerous Highway 400 interchanges in the Barrie area.

Alternative configurations to address the reduced separation between the service centre entrance to Highway 400 and the Essa Road exit from Highway 400 were considered but found to be unworkable or not feasible. Closure of the service centre was, therefore, recommended.

The Highway 400 service centres in the vicinity of Kirby Road in the City of Vaughan are the nearest alternatives to the Highway 89 and Essa Road service centres. There is a northbound service centre located north of Kirby Road and a southbound service centre located south of Kirby Road. These service centres are located a significant distance away from the Highway 89 and Essa Road service centres (approximately 36 kilometres from the Highway 89 service centre and 55 kilometres from the Essa Road service centre).

Service centres are an important part of the provincial highway system. Such centres provide motorists with a variety of travel-related services, such as fuel, comfort stations, meals and travel information. They can also serve as a valuable and convenient area for drives to rest and take a break from driving, where no other options are available.

An assessment of feasible service centre locations along the Highway 400 corridor from north of the Holland Marsh to the Barrie Area was, therefore, undertaken.

Candidate sites were identified and the feasibility of the sites to accommodate service centres along this section of Highway 400 was assessed.

2.0 Existing and Future Conditions

To identify a reasonable range of candidate sites, all significant features within the study area limits were identified to determine the sensitive areas and the potential for impact on these areas from the proposed widening alternatives.

Identifying significant features involved the collection of primary and secondary source data derived from surveys, field studies, published and unpublished literature, government sources and consultation with agencies and the public. The data collected was grouped in the following categories:

- Natural Environment;
- Socio-Economic Environment;
- Cultural Environment; and
- Technical Considerations.

This information was based on data collected for the two Highway 400 Preliminary Design Studies (GWP 40-00-00 and 30-95-00) and a windshield survey of the corridor.

Complete discussions of existing and future conditions are provided in the GWP 30-95-00 and GWP 40-00-00 Preliminary Design Reports.

3.0 Site Requirements

Suitable locations along the Highway 400 corridor for siting service centres were identified based on the size, topography, current land use and accessibility to Highway 400:

- Area – A location was suitable if the site provided adequate land to accommodate a service centre similar in size to that of the Essa Road facility. Suitable locations must be large enough to accommodate entrance and exit ramps to and from Highway 400, as well as a rest area on which the service centre facility (gas station, shops, restaurants, etc.) and parking can be situated;
- Topography – Areas of high fills, deep cuts and steep slopes were avoided;

- Land Use – Vacant lands and lands under agricultural uses were considered and developed lands were avoided. Sites were not considered within the City of Barrie, as the services typically provided at service centres are readily available at numerous interchanges in the City; and

- Access – Sites were required to be sufficiently distant from road crossings to provide proper entrance/exit ramp junctions and speed change lanes on Highway 400.

A typical plan of a service centre is provided in Exhibit 1. This typical plan was used for assessment purposes. The ramp geometrics are based on MTO Geometric Design Standards for interchange ramps.

Based on the location requirements mentioned above, six northbound potential candidate sites and seven southbound potential candidate sites, were identified for the screening process. The site locations are summarized in Exhibit 2 and in the Catalogue of Commuter Parking Lot sites, located at the end of this report.

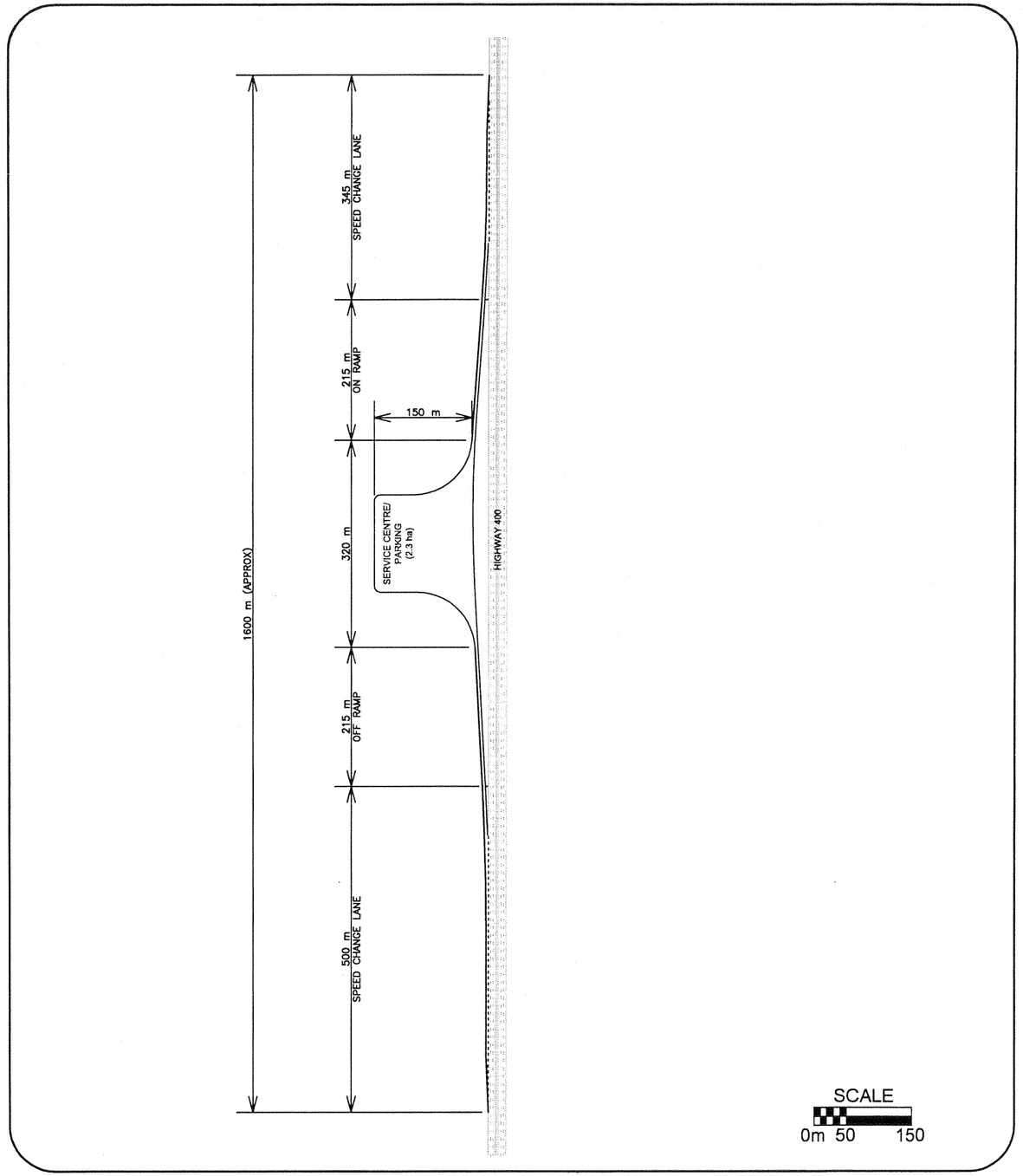
4.0 Screening of Potential Candidate Sites

A screening process was devised to determine if each of the thirteen identified locations should be considered for further investigation.

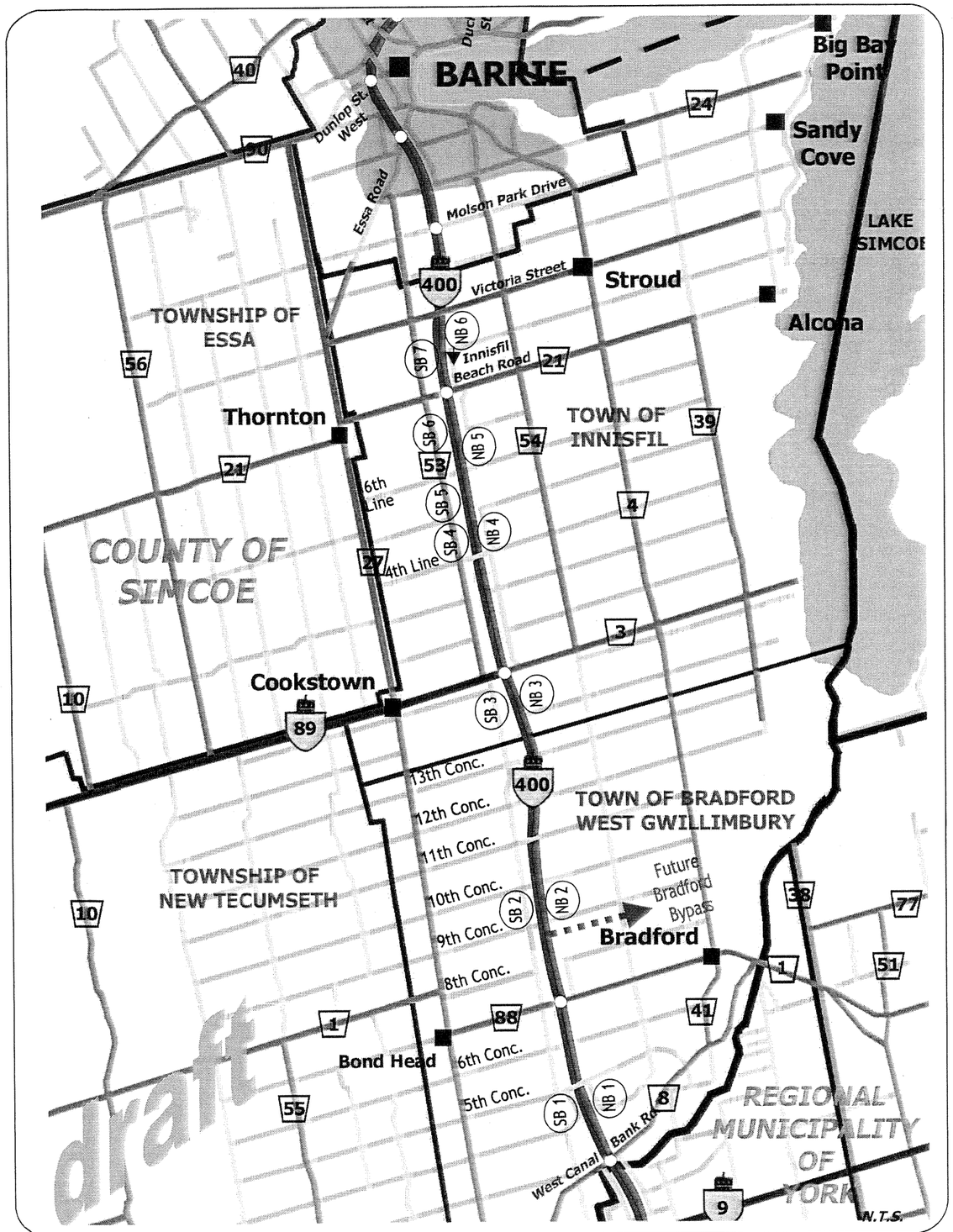
The indicators used in the screening process are consistent with the evaluation criteria typically used in preliminary design studies. They were grouped under the following three factors, which determined whether or not a potential candidate site should be considered for further study:

FACTOR	INDICATORS
Natural Environment	Aquatic habitat impacts Terrestrial habitat impacts Wetlands impacts Groundwater impacts
Socio-Economic Environment	Residences within 300 metres Residences displaced Businesses within 300 metres Agricultural operations within 300 metres Agricultural operations directly impacted
Technical Considerations	Effect on existing road network Proximity to nearest interchange

All significant features within the study area limits were identified so that the impacts of locating service centres at the potential candidate sites to their locations and surrounding areas could be assessed with respect to the above factors.



URS Ontario Ministry of Transportation	HIGHWAY 400 PLANNING STUDY From 1 Km South of Highway 89 to Junction at Highway 11 G.W.P. 30-95-00	Typical Service Centre Layout	EXHIBIT 1



Ontario URS	HIGHWAY 400 PLANNING STUDY FROM 1KM SOUTH OF HIGHWAY 89 TO JUNCTION AT HIGHWAY 11 G.W.P. 30-95-00	Potential Service Centre Locations	EXHIBIT 2

Identifying significant features involved the collection of primary and secondary source data derived from surveys, field studies, published and unpublished literature, government sources and consultation with agencies and the public. The data collected were grouped according to the above four categories.

Once the significant features were identified, an assessment of the impacts was carried out to screen the potential candidate sites.

Exhibit 3 summarizes the impact assessment and screening of the potential sites for locating service centres.

Alternatives SB6 and SB7 had the greatest overall impacts. Site SB6 lacks technical merit and would have significant impacts from a Socio-Economic Environment perspective. From a technical perspective, Site SB6 would be within close proximity to the Innisfil Beach Road interchange, may result in undesirable traffic operations and may affect potential services at the interchange. From a Socio-Economic and Natural perspective, Site SB6 would directly impact two agricultural operations, a forest community and may have significant potential impacts to two significant water crossing. Since Site SB6 would have significant impacts to all of the screening factors, it does not pass the screening process and is not recommended for further consideration.

Site SB7 lacks technical merit as it would be within close proximity to the Innisfil Beach Road interchange, may result in undesirable traffic operations and may affect potential services at the interchange. Although Site SB7 will have minimal overall impacts to the Natural and Socio-Economic environments, it may have potential impacts to a cemetery, which is not desirable. Based on poor technical merit and proximity to a cemetery, Site SB7 does not pass the screening process and is not recommended for further consideration.

The remaining sites are preferable from the perspective of at least two of the three factors. As well, the remaining sites would not impact significant features, such as cemeteries. Therefore, the remaining sites compose an inventory of sites suitable for further investigation for service centre use. The following summarizes the assessment of each site:

- While Sites NB1, NB5, NB6 and SB1 are also in close proximity to interchanges, these sites are being carried forward for further investigation since they have minimal impacts natural and socio-economic impacts.

- Sites NB2, NB3, and SB3 may result in potentially significant impacts to the natural environment. Due to strong technical merits and low Socio-Economic impacts, however, further investigation of these sites is recommended.

ASSESSMENT OF POTENTIAL SERVICE CENTRE LOCATIONS

FACTOR/ Indicator	Site NB1	Site NB2	Site NB3	Site NB4	Site NB5	Site NB6	Site SB1	Site SB2	Site SB3	Site SB4	Site SB5	Site SB6	Site SB7	Comment
NATURAL ENVIRONMENT														
Aquatic Habitat Impacts	No impact ✓	No impact ✓	Potential impacts to Innisfil Creek	No impact ✓	No impact ✓	No impact ✓	No impact ✓	No impact ✓	Potential impacts to Type 2 permanent water crossing.	No impact ✓	No impact ✓	Potential impacts to 2 Type 2 permanent water crossings	No impact ✓	Service centres at sites NB3, SB3 and SB6 would impact significant water crossings.
Terrestrial Habitat Impacts	No impact ✓	Direct Impacts to Reed Canary Grass Marsh	Deer wintering habitat within 300m	No impact ✓	Wildlife linkage/corridor and forest community within 300m	No impact ✓	No impact ✓	No impact ✓	Direct impacts to deer wintering habitat	No impact ✓	No impact ✓	Direct impacts to forest community	No impact ✓	Service centres at sites NB3 and NB5 may potentially impact significant terrestrial habitats. Service centres at sites NB2, SB3, SB4 and SB6 would directly impact significant terrestrial habitats.
Wetlands Impacts	No impact ✓	No impact ✓	No impact ✓	No impact ✓	No impact ✓	No impact ✓	No impact ✓	No impact ✓	No impact ✓	No impact ✓	No impact ✓	No impact ✓	No impact ✓	Locating service centres at any of the sites would have no impact on wetlands.
Groundwater Impacts	No impact ✓	Water well within 300m	No impact ✓	No impact ✓	No impact ✓	Water well within 300m	Water well within 300m	Water well within 300m	No impact ✓	Water well within 300m	No impact ✓	No impact ✓	No impact ✓	Service centres at sites NB2, NB6, SB1, SB2 and SB4 may potentially impact water wells.
SOCIO-ECONOMIC ENVIRONMENT														
Residences within 300m	0 ✓	0 ✓	0 ✓	0 ✓	1 ✓	1 ✓	1 ✓	0 ✓	0 ✓	2	0 ✓	0 ✓	1 ✓	A service centre at site SB4 will potentially impact the greatest number of residences (2 each).
Residences displaced	0 ✓	0 ✓	0 ✓	0 ✓	0 ✓	0 ✓	0 ✓	0 ✓	0 ✓	0 ✓	0 ✓	0 ✓	0 ✓	Residences will not be displaced by locating services centres at any of the sites.
Businesses within 300m	0 ✓	0 ✓	0 ✓	0 ✓	0 ✓	0 ✓	0 ✓	0 ✓	0 ✓	0 ✓	0 ✓	0 ✓	0 ✓	Businesses will not be impacted by the service centres at any of the sites.
Businesses displaced	0 ✓	0 ✓	0 ✓	0 ✓	0 ✓	0 ✓	0 ✓	0 ✓	0 ✓	0 ✓	0 ✓	0 ✓	0 ✓	Businesses will not be displaced by service centres at any of the sites.
Agricultural Operations within 300m	3	2	0 ✓	3	1 ✓	4	4	4	2	4	2	2	1 ✓	Service centres at sites NB6, SB1, SB2 and SB4 will potentially impact the greatest number of residences (4 each).
Agricultural Operations directly impacted	1 ✓	1 ✓	0 ✓	1 ✓	1 ✓	1 ✓	1 ✓	1 ✓	2	1 ✓	1 ✓	1	1 ✓	A service centre at site SB3 will directly impact the greatest number of agricultural operations (2).
Heritage Features within 300m	2	2	0 ✓	3	1 ✓	1 ✓	1 ✓	1 ✓	1 ✓	4	3	4	1 (Cemetery within 300 m)	Service centres at sites SB4 and SB6 will potentially impact the greatest number of heritage features (4 each). A service centre at site SB7 may potentially impact a cemetery.
TECHNICAL CONSIDERATIONS														
Effect on Existing Road Network	Will necessitate realignment of Coffey Road	No impact ✓	No impact ✓	Will necessitate realignment of Reive Road.	Within proximity of Innisfil Beach Road interchange. Potential for future weave condition.	Within proximity of Innisfil Beach Road interchange. Potential for future weave condition.	Will necessitate realignment of Sideroad 5	No impact ✓	No impact ✓	No impact ✓	No impact ✓	Within proximity of Innisfil Beach Road interchange. Potential for future weave condition.	Within proximity of Innisfil Beach Road interchange. Potential for future weave condition.	Service centres at sites NB1, NB4 and SB1 will require modifications to the existing road network. The ramps to/from Sites NB5, NB6, SB6 and SB7 may potentially introduce weave situations with the Innisfil Beach Road ramps.
Proximity to Nearest Interchange	Within 5 km of Highway 9. Within 3 km of Simcoe Road 88.	Within 4 km of Simcoe Road 88. Within 8 km of Highway 89. Within 2 km of future Bradford Bypass Interchange. ✓	Within 2.5 km of Highway 89.	Within 5 km of Highway 89. Within 5 km of Innisfil Beach Road. ✓	Within 10 km Highway 89. Within 2 km of Innisfil Beach Road.	Within 2 km of Innisfil Beach Road. Within 4 km of Molson Park.	Within 5 km of Highway 9. Within 3 km of Simcoe Road 88.	Within 4 km of Simcoe Road 88. Within 8 km of Highway 89. Within 2 km of future Bradford Bypass Interchange. ✓	Within 2.5 km of Highway 89.	Within 5 km of Highway 89. Within 5 km of Innisfil Beach Road. ✓	Within 6 km of Highway 89. Within 4 km of Innisfil Beach Road. ✓	Within 10 km Highway 89. Within 2 km of Innisfil Beach Road.	Within 2 km of Innisfil Beach Road. Within 4 km of Molson Park.	Service centres at sites NB1, NB3, NB5, NB6, SB1, SB4, SB6 and SB7 would be in close proximity (≤3 km) to an interchange, which may impact services at these interchanges.
SUMMARY	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓			

SUMMARY OF EVALUATION

Sites SB6 and SB7 are not preferred as they are within close proximity to the Innisfil Beach Road interchange and may result in undesirable weave conditions and may affect potential services at the interchange. As well, locating a service center at site SB6 will directly impact two agricultural operations, a forest community and may have significant potential impacts to a significant water crossing. Although a service centre at site SB7 will have minimal overall impacts to the Natural and Socio-Economic environments, it may have potential impacts to a cemetery, which is not desirable.

While sites NB1, NB5, NB6 and SB1 are also in close proximity to interchanges, these sites are being carried forward for further investigation since locating service centres at these sites would have minimal impacts natural and socio-economic impacts.

Service Centres at sites NB2, NB3, and SB3 may result in potentially significant impacts to the natural environment. Due to strong technical merits and low Socio-Economic impacts, however, further consideration and further investigation into impacts to the natural environments for these sites is recommended.

Although site SB4 may result in potentially significant impacts to the socio-economic environment, this site is being carried forward for further investigation since it has potentially low impacts to the natural environment and strong technical merits.

Sites NB4, SB2 and SB5 are being carried forward for further investigation as they have minimal environmental impacts and strong technical merits.

THEREFORE, WITH THE EXCEPTIONS OF SITES SB6 AND SB7, ALL SITES ARE SUITABLE CANDIDATE SITES.

✓ Indicates which alternative is preferred.

EXHIBIT 3

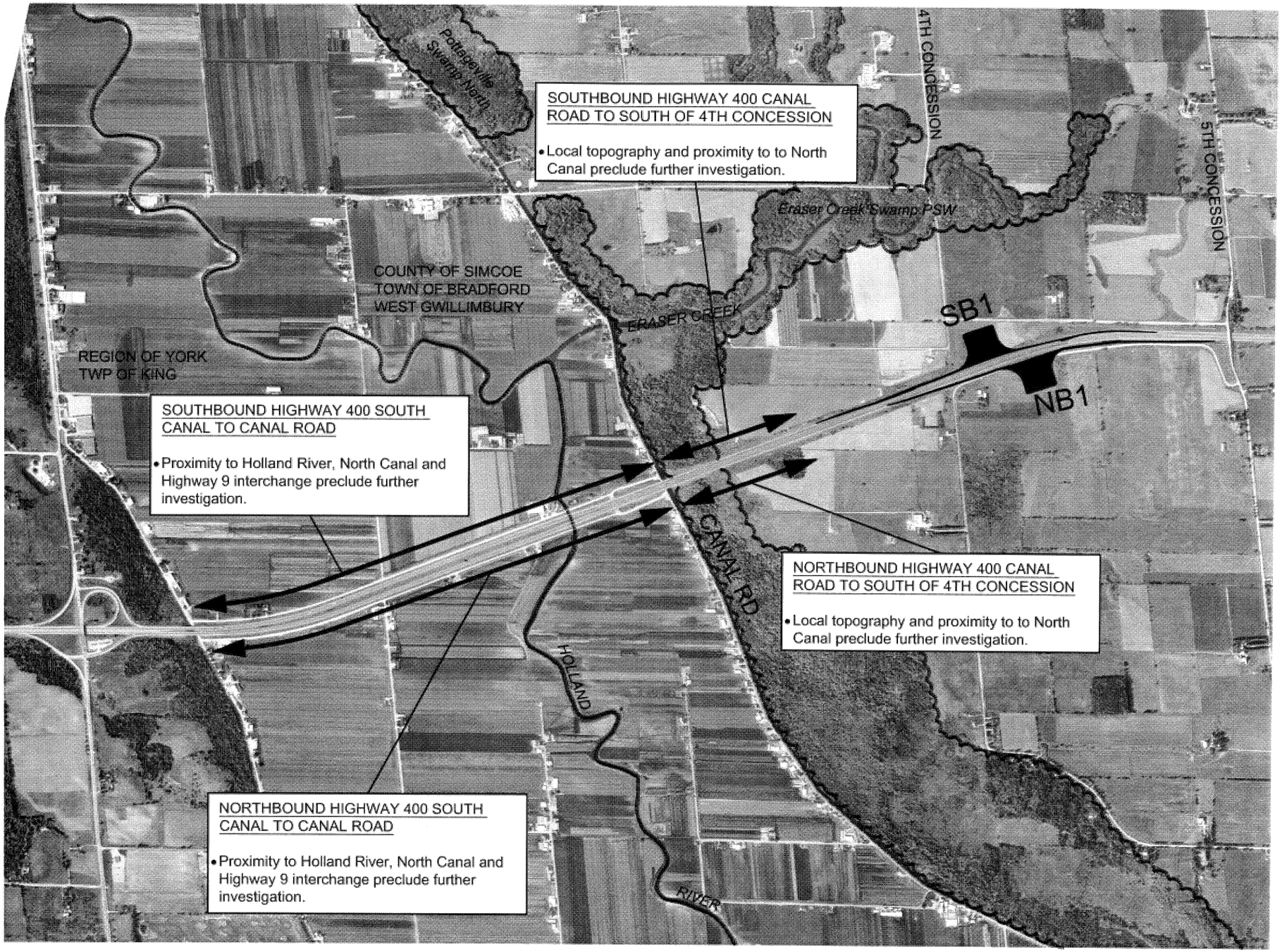
- Although Site SB4 may result in potentially significant impacts to the socio-economic environment, this site is being carried forward for further investigation since it has potentially low impacts to the natural environment and strong technical merits.

- Sites NB4, SB2 and SB5 are being carried forward for further investigation as they have minimal environmental impacts and strong technical merits.

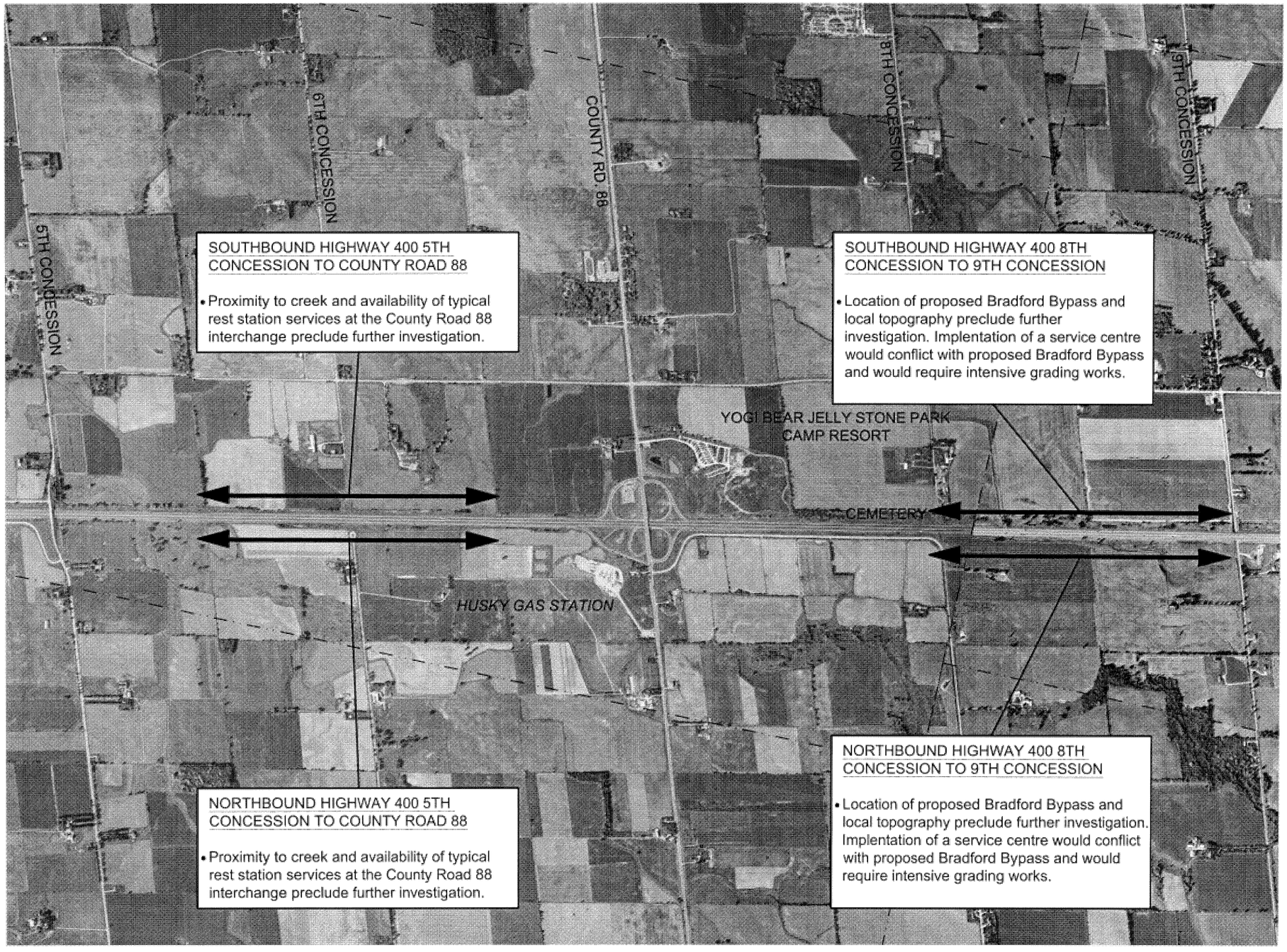
Therefore, with the exceptions of Sites SB6 and SB7, all sites are suitable candidate sites.

The following catalogue provides approximate locations and layouts for the sites recommended for further investigation. The catalogue also locates areas along the Highway 400 corridor where commuter parking lots may not be possible.

Catalogue of Candidate Sites



URS Ontario Ministry of Transportation	HIGHWAY 400 PLANNING STUDY From 1 Km South of Highway 89 to Junction at Highway 11 G.W.P. 30-95-00	Catalogue of Commuter Parking Lot Sites to be Considered for Further Investigation	PAGE 1



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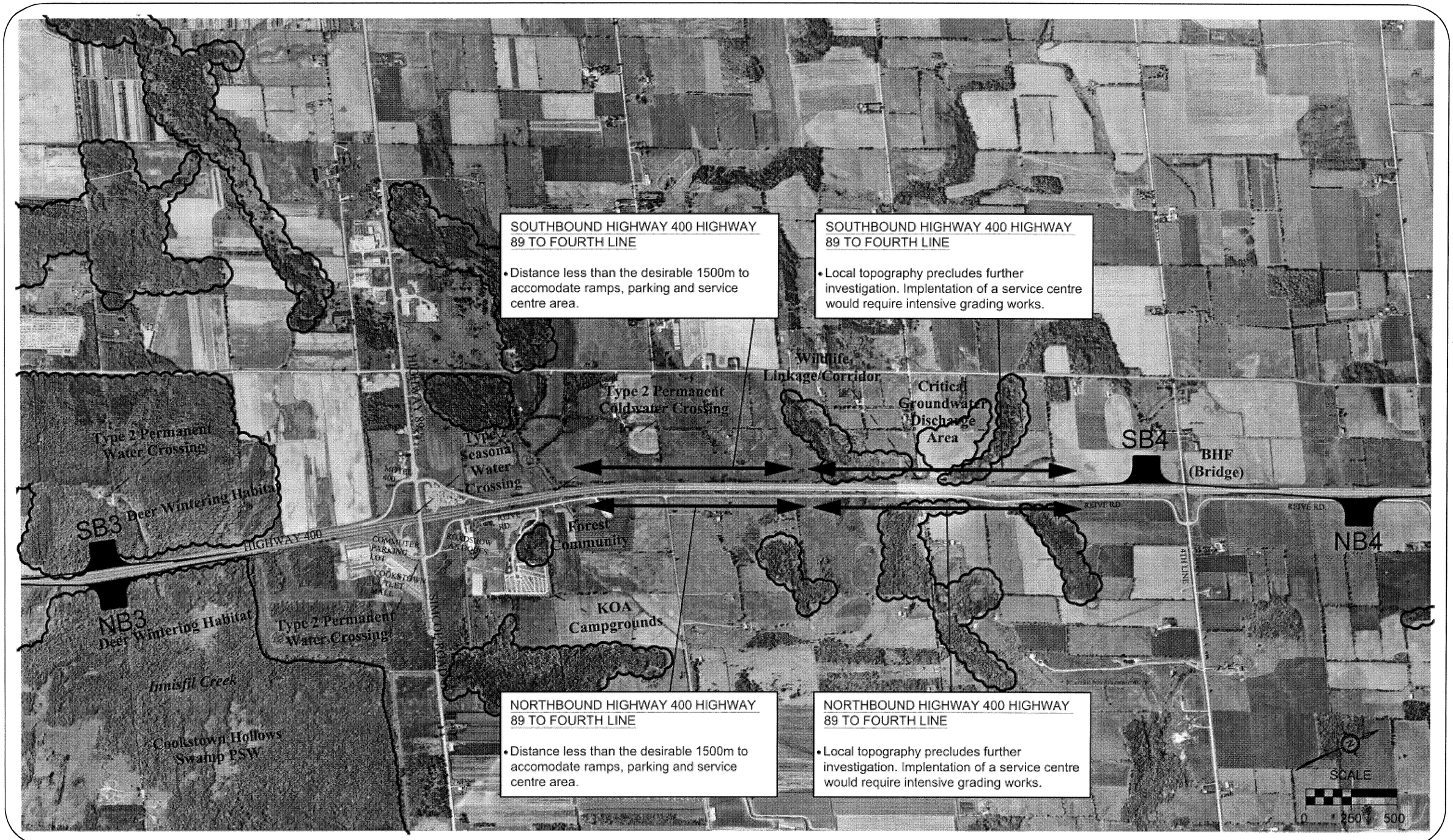
Ministry of Transportation

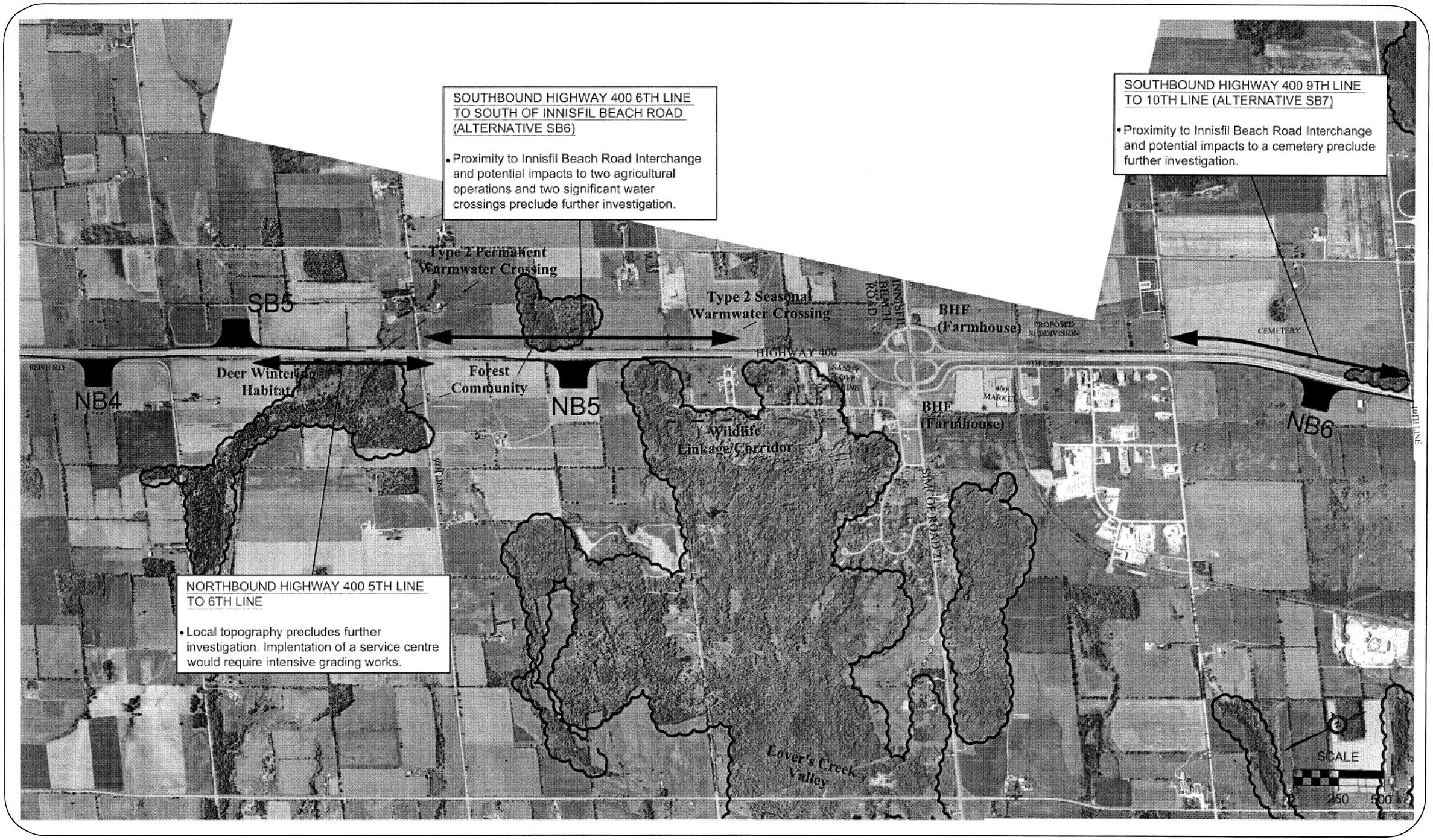
HIGHWAY 400 PLANNING STUDY
From 1 Km South of Highway 89
to Junction at Highway 11
G.W.P. 30-95-00

Catalogue of Commuter Parking Lot Sites to be Considered for Further Investigation

URS Ontario Ministry of Transportation	HIGHWAY 400 PLANNING STUDY From 1 Km South of Highway 89 to Junction at Highway 11 G.W.P. 30-95-00	Catalogue of Commuter Parking Lot Sites to be Considered for Further Investigation	PAGE 3



URS Ontario Ministry of Transportation	HIGHWAY 400 PLANNING STUDY From 1 Km South of Highway 89 to Junction at Highway 11 G.W.P. 30-95-00	Catalogue of Commuter Parking Lot Sites to be Considered for Further Investigation	PAGE 4



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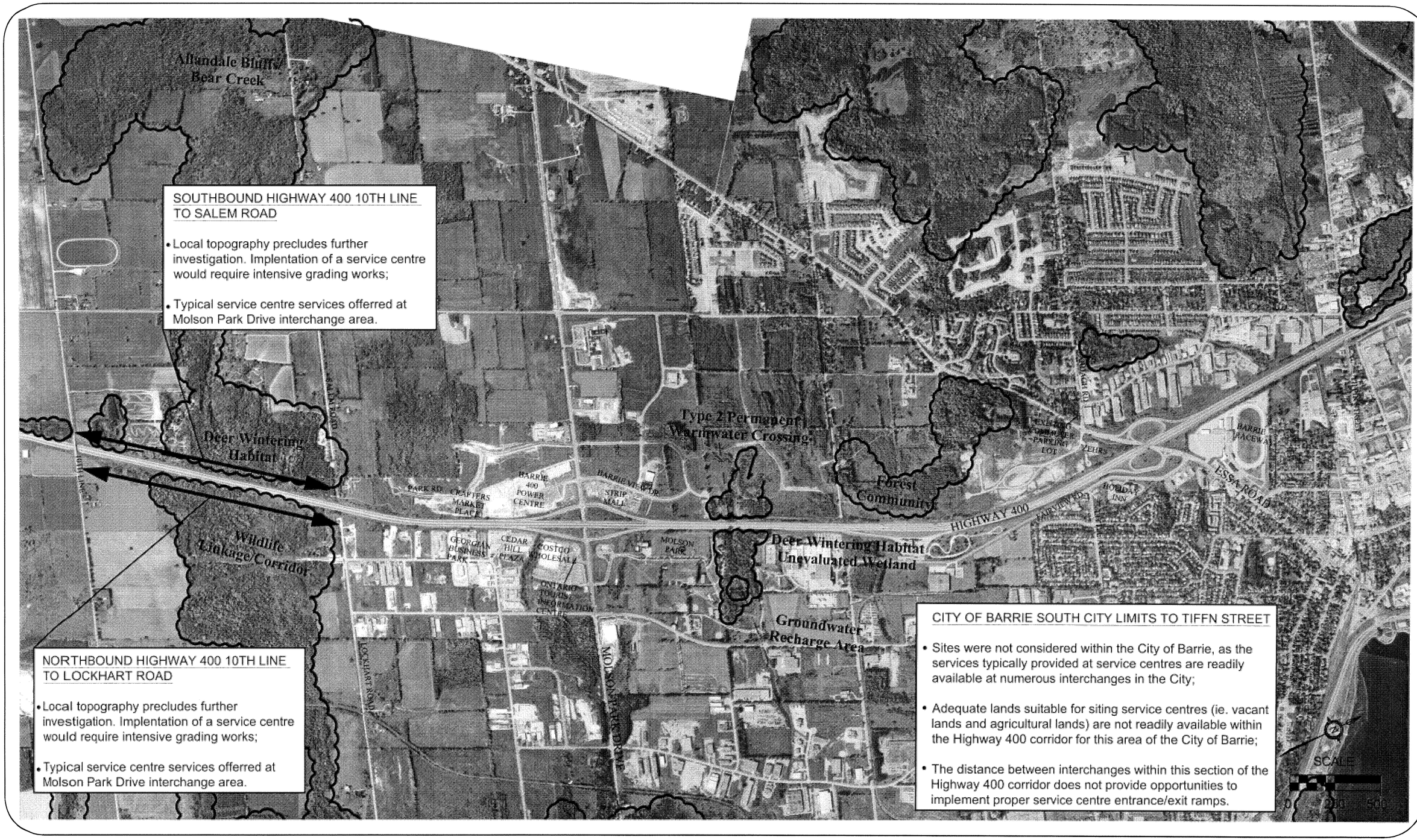
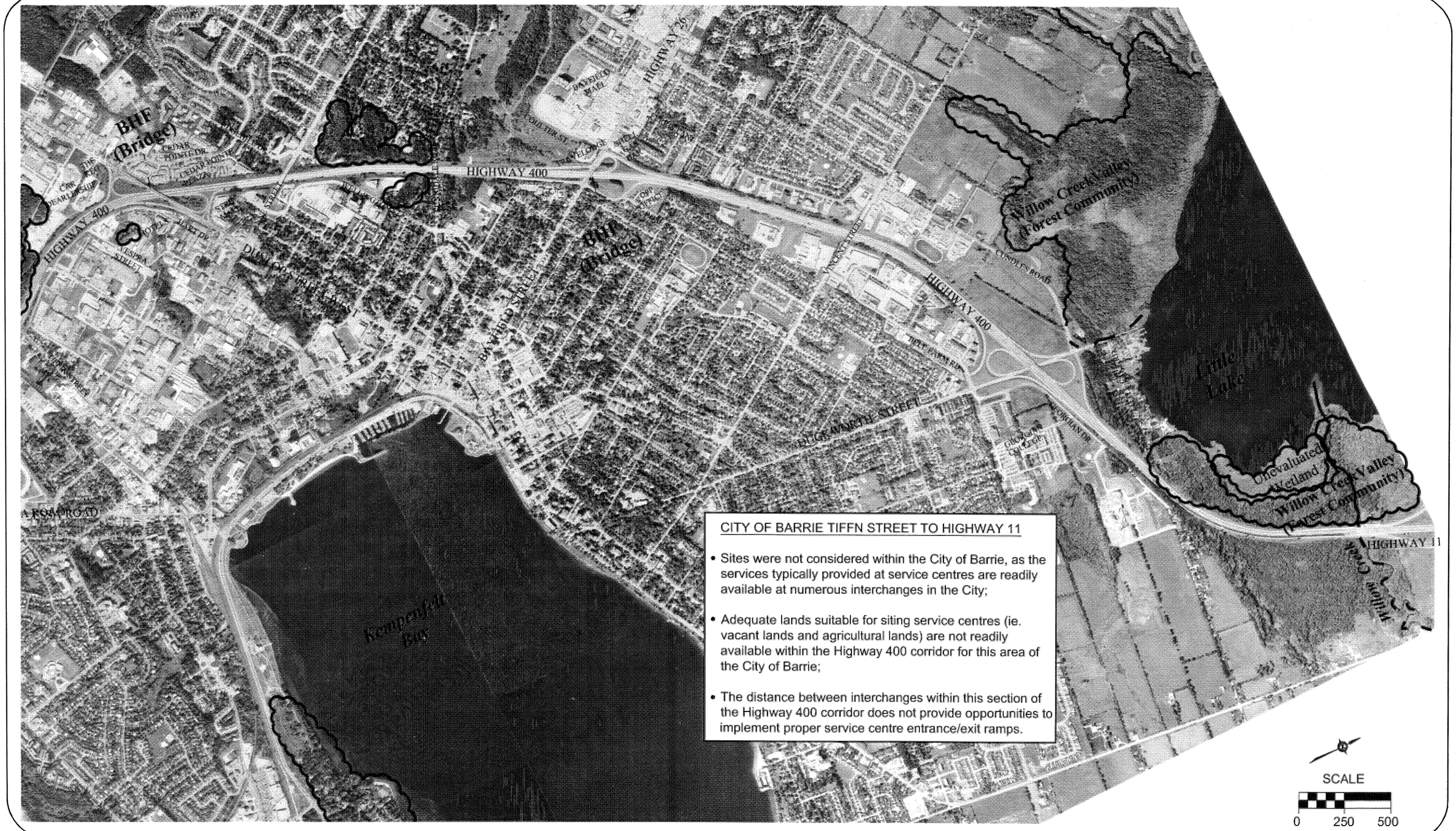
HIGHWAY 400 PLANNING STUDY
From 1 Km South of Highway 89
to Junction at Highway 11
G.W.P. 30-95-00

Catalogue of Commuter Parking Lot Sites to be Considered for Further Investigation



URS Ontario Ministry of Transportation	HIGHWAY 400 PLANNING STUDY From 1 Km South of Highway 89 to Junction at Highway 11 G.W.P. 30-95-00	Catalogue of Commuter Parking Lot Sites to be Considered for Further Investigation	PAGE 6



CITY OF BARRIE TIFFN STREET TO HIGHWAY 11

- Sites were not considered within the City of Barrie, as the services typically provided at service centres are readily available at numerous interchanges in the City;
- Adequate lands suitable for siting service centres (ie. vacant lands and agricultural lands) are not readily available within the Highway 400 corridor for this area of the City of Barrie;
- The distance between interchanges within this section of the Highway 400 corridor does not provide opportunities to implement proper service centre entrance/exit ramps.

SCALE
0 250 500