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 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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OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00

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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 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.

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

Anne S. Poschmann, P.Eng.,
Principal

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

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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.

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

Anne S. Poschmann, P.Eng.,
Principal

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

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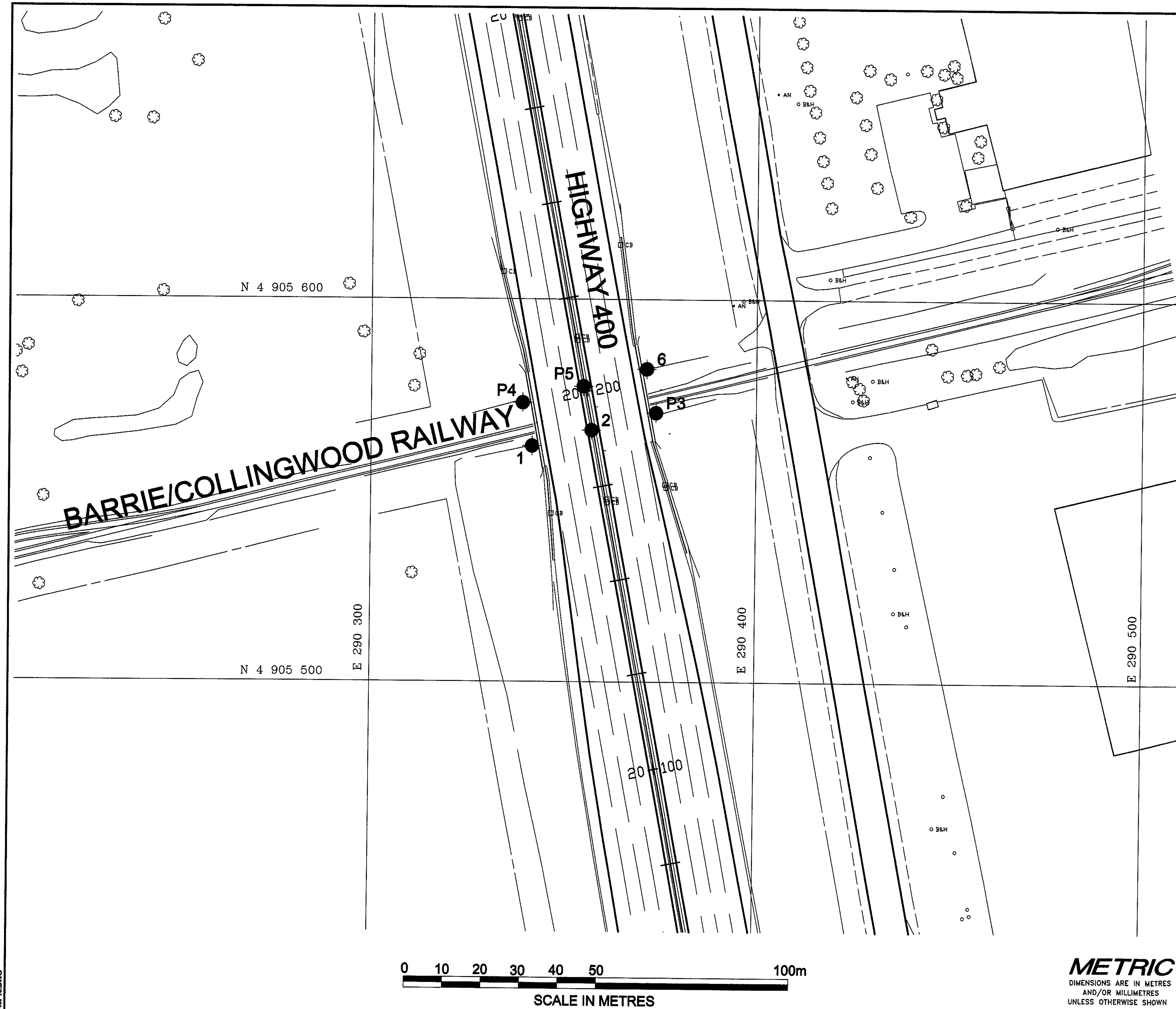


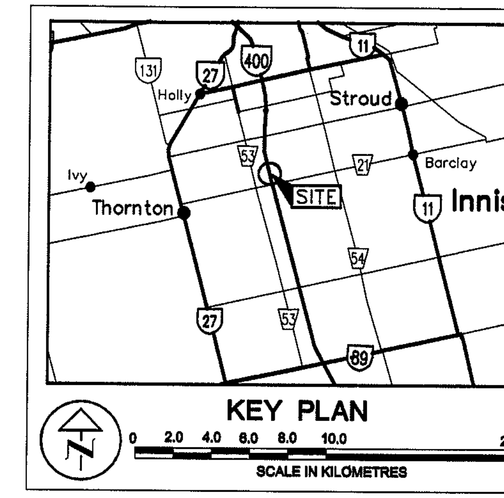
DIST HWY 400
CONT. No.
GWP No. 30-95-00

SHEET

BARRIE/COLLINGWOOD RAILWAY
OVERPASS HWY 400
BOREHOLE LOCATION PLAN

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



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

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.

Driller: M.C. Checked: Date: 26. 3.'58

LEGEND

Shear Strength (C)

Unconfined compression ⊕

Vane test and sensitivity (S) +S

Penetration Resistance (P)

2" Split tube —⊖—⊖—

2" Dia. Cone ———

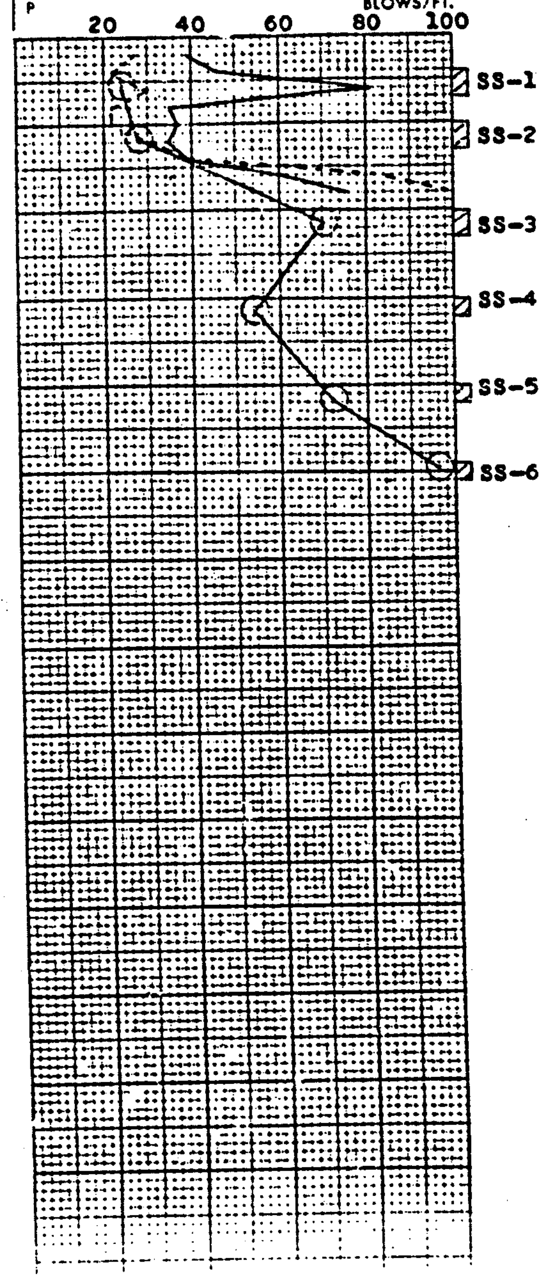
Casing - - - - -

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

C P.S.F.

P BLOWS/FT.



Order No. S-500/T-1128

Enclosure No. 4

RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

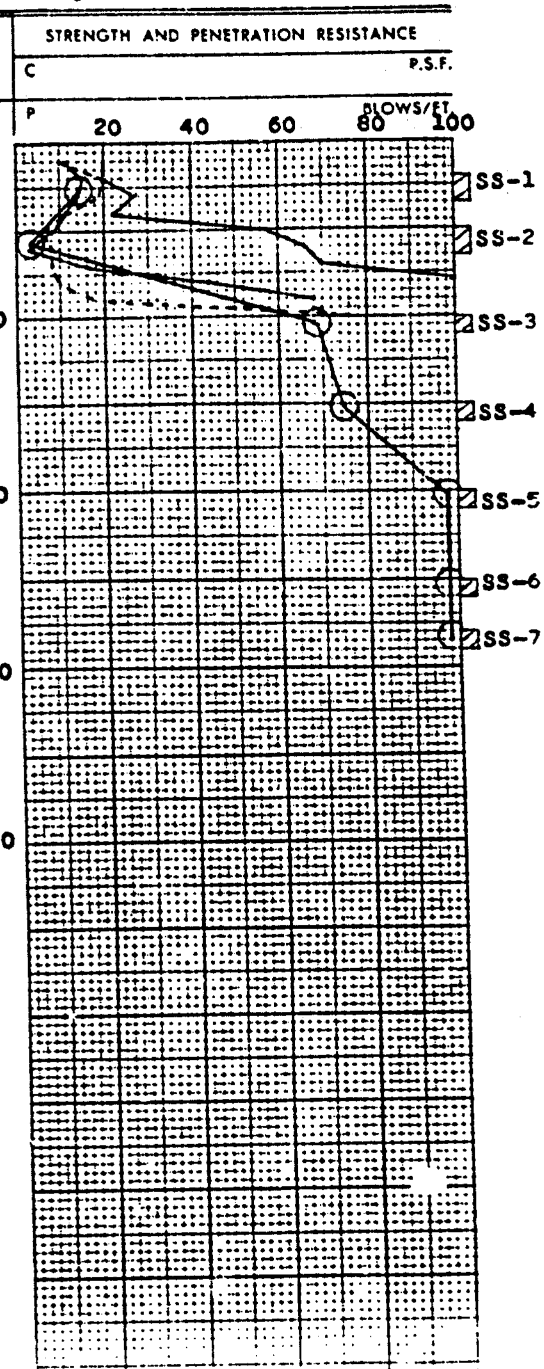
Engineering Data Sheet for Borehole: 2

Project: FOUNDATION INVESTIGATION FOR PROPOSED C.N.R. OVER-PASS
Location: HWY #400, INNISFIL TWP., NEAR BARRIE, ONT.
Hole Location: See Enclosure No 2
Hole Elevation and Datum: 996 Based on contours 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) +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		
294.7m	END OF HOLE	967	30
			40



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

RACEY MacCALLUM AND ASSOCIATES LTD.

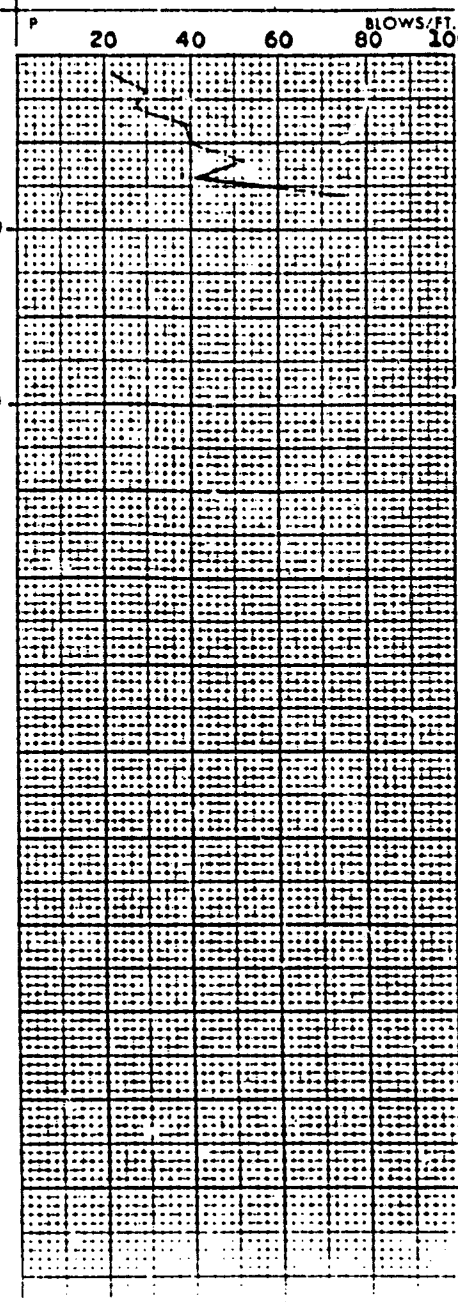
Foundation Engineering Division

Engineering Data Sheet for Borehole: P3

Project: FOUNDATION INVESTIGATION FOR PROPOSED C.N.R. OVER-PASS
Location: ON HWY #400, INNISFIL TWP. NEAR BARRIE, ONT.
Hole Location: See Enclosure No 2
Hole Elevation and Datum: 997 Based on Contours 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) +S
Penetration Resistance (P)
2" Split tube
2" Dia. Cone
Casing

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



Order No. S-500/T-1128

Enclosure No. 6

RACEY MacCALLUM AND ASSOCIATES LTD.

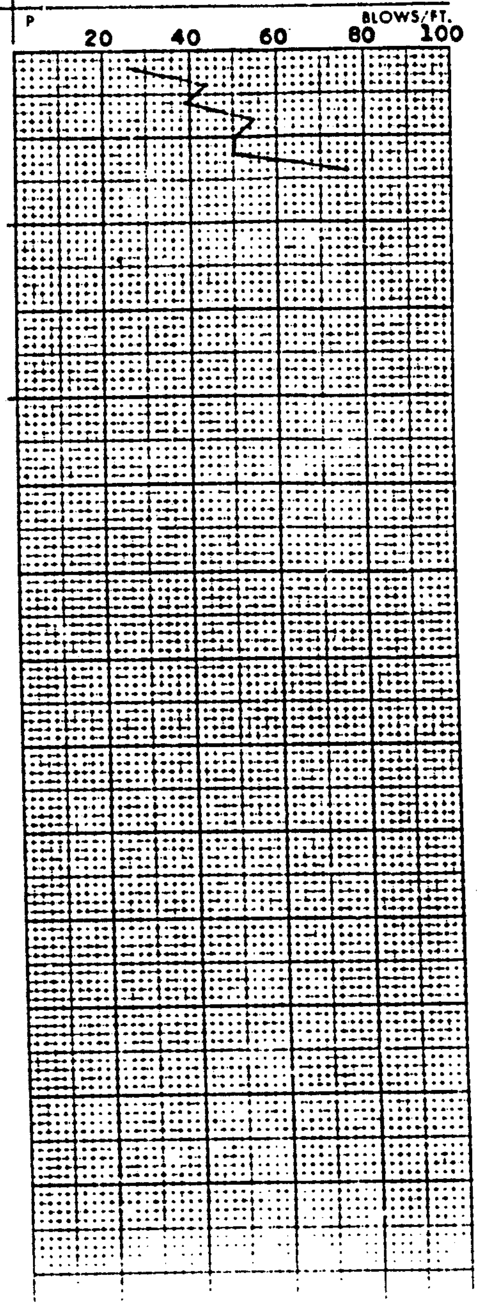
Foundation Engineering Division

Engineering Data Sheet for Borehole: P4

Project: FOUNDATION INVESTIGATION FOR PROPOSED C.N.R.OVER-PASS
Location: ON HWY.#400,INNISFIL TWP.NEAR BARRIE,ONT.
Hole Location: See Enclosure No 2
Hole Elevation and Datum: 997 Based on contours 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) +S
Penetration Resistance (P)
2" Split tube
2" Dia. Cone
Casing

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE
303.9 m		997		
301.8 m	END OF CONE TEST	990		



Order No. S-500/T-1128

Enclosure No. 7

RACEY MacCALLUM AND ASSOCIATES LTD.

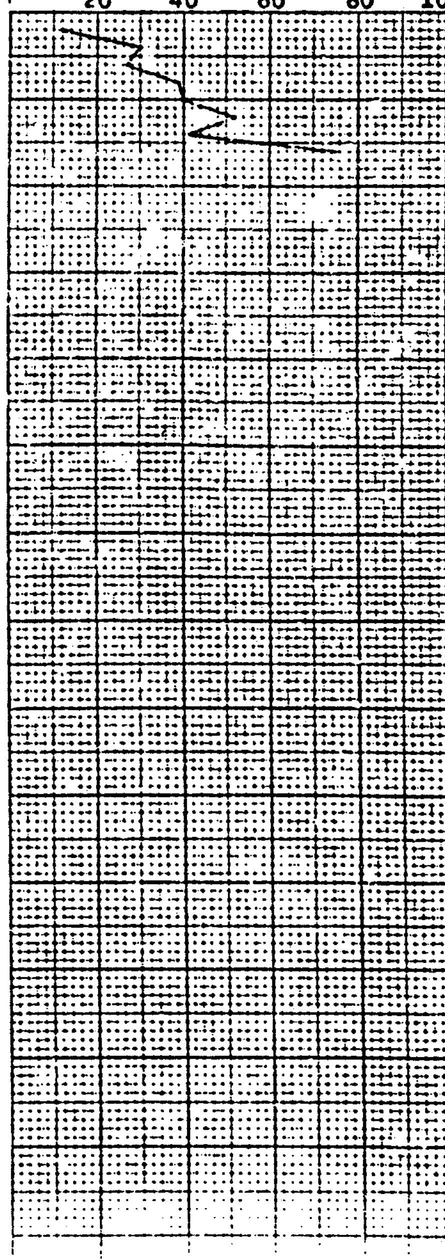
Foundation Engineering Division

Engineering Data Sheet for Borehole: P5

Project: FOUNDATION INVESTIGATION FOR PROPOSED C.N.R. OVER-PASS
Location: ON 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.
Driller: M.C. Checked: Date: 26.3.'58

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

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE C P.S.F. / P BLOWS/FT. 20 40 60 80 100
303.9m		997		
301.4m	END OF CONE TEST	989		
			10	
			20	



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-PASS
Location: ON HWY.#400,INNISFIL TWP.,NEAR BARRIE,ONT.
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) +S
Penetration Resistance (P)
2" Split tube
2" Dia. Cone
Casing

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

STRENGTH AND PENETRATION RESISTANCE