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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:

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February 2002

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## PRELIMINARY FOUNDATION INVESTIGATION REPORT
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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 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.

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# 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 $\delta$, 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.**

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Designated MTO Contact

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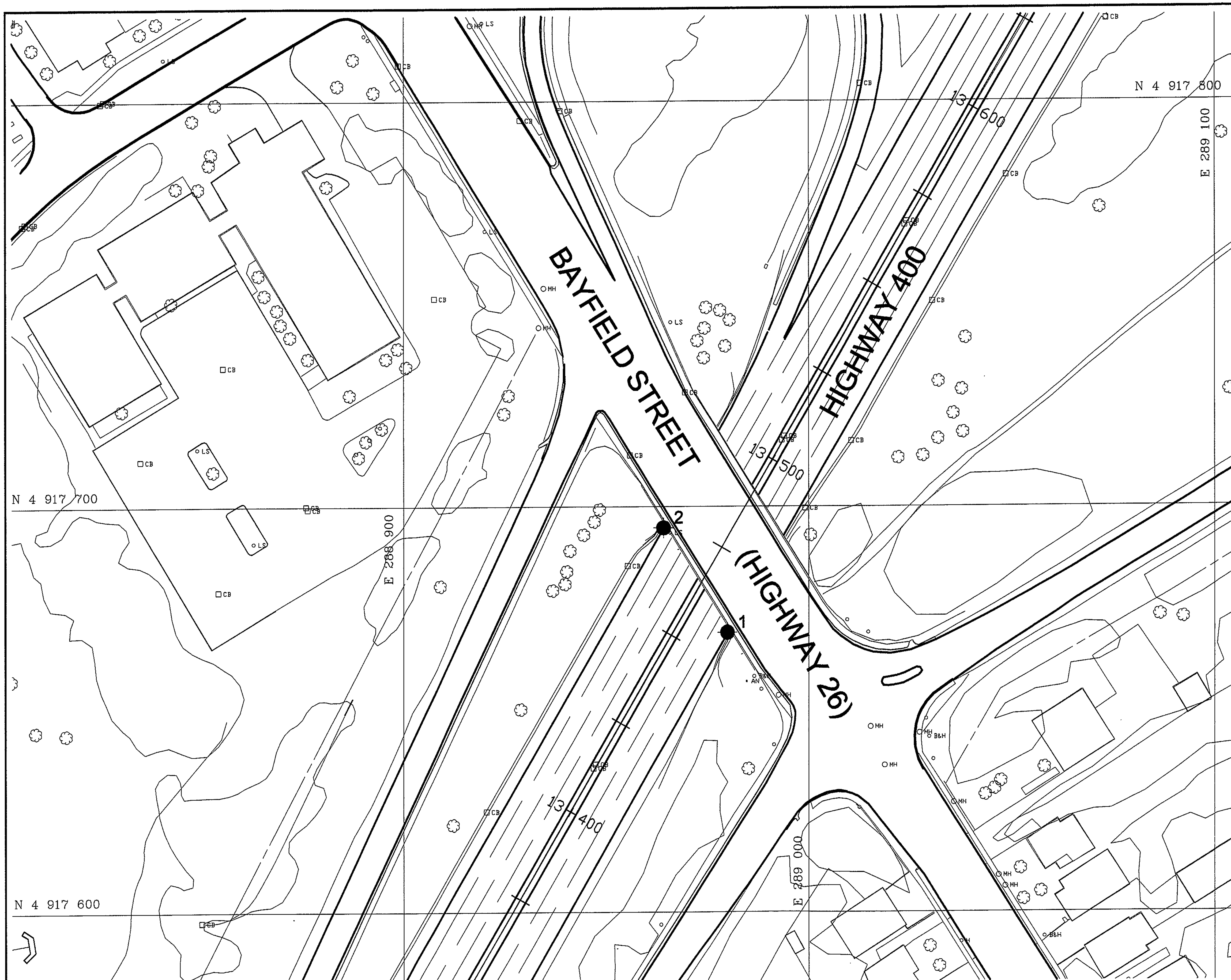
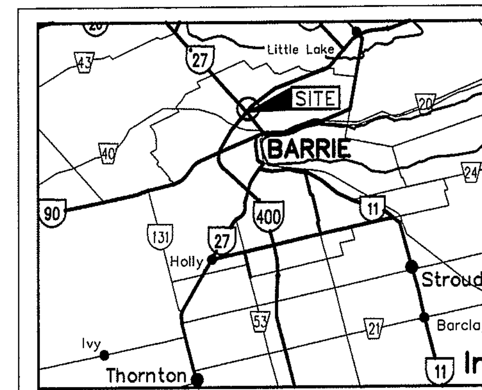
| | |
|---|---|
| DIST HWY 400 | |
| CONT. No. | |
| GWP No. 30-95-00 | |
| BAYFIELD STREET UNDERPASS<br>HWY 400<br>BOREHOLE LOCATION PLAN | SHEET |

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

## MECHANICAL PROPERTIES OF SOIL

| | | |
|---|---|---|
| $m_v$ | $kPa^{-1}$ | COEFFICIENT OF VOLUME CHANGE |
| $C_c$ | 1 | COMPRESSION INDEX |
| $C_s$ | 1 | SWELLING INDEX |
| $C_\alpha$ | 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 |
| $\sigma'_{vo}$ | kPa | EFFECTIVE OVERBURDEN PRESSURE |
| $\sigma'_p$ | kPa | PRECONSOLIDATION PRESSURE |
| $\tau_f$ | kPa | SHEAR STRENGTH |
| $c'$ | kPa | EFFECTIVE COHESION INTERCEPT |
| $\phi'$ | -° | EFFECTIVE ANGLE OF INTERNAL FRICTION |
| $c_u$ | kPa | APPARENT COHESION INTERCEPT |
| $\phi_u$ | -° | APPARENT ANGLE OF INTERNAL FRICTION |
| $\tau_R$ | kPa | RESIDUAL SHEAR STRENGTH |
| $\tau_r$ | kPa | REMOULDED SHEAR STRENGTH |
| $S_t$ | 1 | SENSITIVITY = $\frac{c_u}{\tau_r}$ |

## PHYSICAL PROPERTIES OF SOIL

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| $\rho_s$ | $kg/m^3$ | DENSITY OF SOLID PARTICLES | e | 1,% | VOID RATIO | $e_{min}$ | 1,% | VOID RATIO IN DENSEST STATE |
| $\gamma_s$ | $kN/m^3$ | UNIT WEIGHT OF SOLID PARTICLES | n | 1,% | POROSITY | $I_D$ | 1 | DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$ |
| $\rho_w$ | $kg/m^3$ | DENSITY OF WATER | w | 1,% | WATER CONTENT | D | mm | GRAIN DIAMETER |
| $\gamma_w$ | $kN/m^3$ | UNIT WEIGHT OF WATER | $S_r$ | % | DEGREE OF SATURATION | $D_n$ | mm | n PERCENT - DIAMETER |
| $\rho$ | $kg/m^3$ | DENSITY OF SOIL | $w_L$ | % | LIQUID LIMIT | $C_U$ | 1 | UNIFORMITY COEFFICIENT |
| $\gamma$ | $kN/m^3$ | UNIT WEIGHT OF SOIL | $w_P$ | % | PLASTIC LIMIT | h | m | HYDRAULIC HEAD OR POTENTIAL |
| $\rho_d$ | $kg/m^3$ | DENSITY OF DRY SOIL | $w_S$ | % | SHRINKAGE LIMIT | q | $m^3/s$ | RATE OF DISCHARGE |
| $\gamma_d$ | $kN/m^3$ | UNIT WEIGHT OF DRY SOIL | $I_P$ | % | PLASTICITY INDEX = $w_L - w_P$ | v | m/s | DISCHARGE VELOCITY |
| $\rho_{sat}$ | $kg/m^3$ | DENSITY OF SATURATED SOIL | $I_L$ | 1 | LIQUIDITY INDEX = $\frac{w - w_P}{I_P}$ | i | 1 | HYDRAULIC GRADIENT |
| $\gamma$ | $kN/m^3$ | UNIT WEIGHT OF SATURATED SOIL | | | | [illegible] | [illegible] | [illegible] |

# 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 | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|
| 266.1 | Ground Level | | | | | | |
| 0.0 | Asphalt Pavement | | | | * | 266 | |
| 0.3 | Very Stiff | 1 | SS | 17 | | | 6 63 21 10 |
| | | 2 | SS | 42 | | 264 | |
| | Clayey Silt to Silt With Sand Trace of Gravel Occasional Cobbles Hard (Glacial Till) | 3 | SS | 84 | | | |
| | | 4 | SS | 98 | | | 9 58 23 10 |
| | | 5 | SS | 88 | | 262 | |
| | | 6 | SS | 90 | | | |
| | | 7 | SS | 58 | | 260 | 4 59 25 12 |
| | | 8 | SS | 70 | | | |
| 258.6 | | 9 | SS | 67 | | | 5 59 25 11 |
| 7.5 | End of Borehole<br>* 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 × LAB VANE

PLASTIC LIMIT $W_p$ NATURAL MOISTURE CONTENT W LIQUID LIMIT $W_L$

WATER CONTENT (%) 5 10 15

UNIT WEIGHT γ

Ontario

HIGHWAY ENGINEERING DIVISION-ENGINEERING MATERIALS OFFICE-SOIL MECHANICS SECTION

# RECORD OF BOREHOLE No 2

METRIC

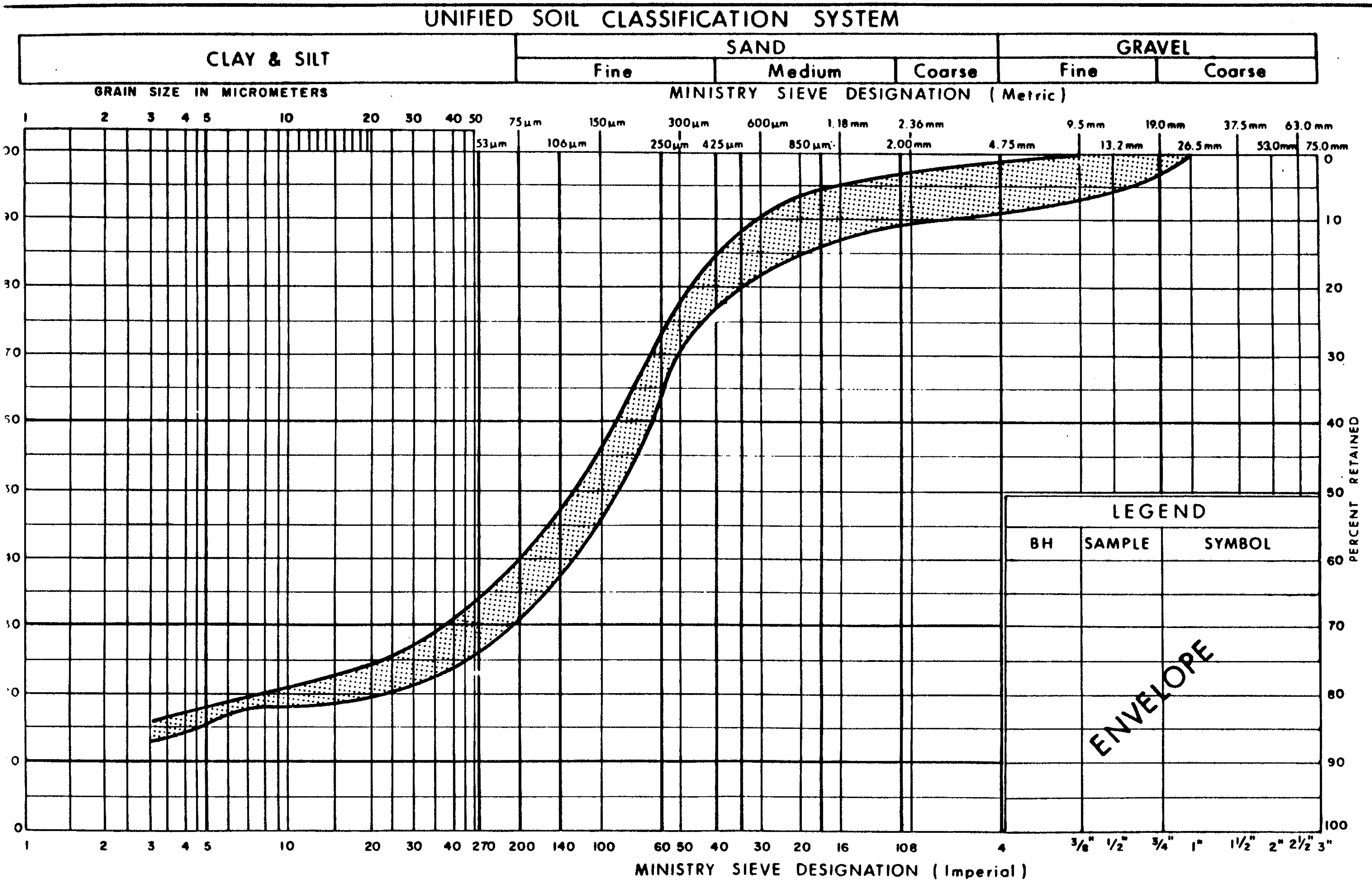
W P 28-78-02 LOCATION Sta. 10+477.1 o/s 12.5 m Lt. ₵ Hwy. 26 & 27 ORIGINATED BY PRK

DIST 5 HWY 400 BOREHOLE TYPE Solid Stem Auger, Cone Test COMPILED BY PRK

DATUM Geodetic DATE 1979 06 07 CHECKED BY RS

| SOIL PROFILE | | | SAMPLES | | | | | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE | PLASTIC LIMIT $W_p$ / NATURAL MOISTURE CONTENT W / LIQUID LIMIT $W_L$ / WATER CONTENT (%) 5 10 15 | | REMARKS & GRAIN SIZ DISTRIBUTIC (%) GR SA SI ( |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | 'N' VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | | | UNIT WEIGHT γ | |
| 266.4 | Ground Level | | | | | | | | | | |
| 0.0 | Asphalt Pavement | | | | | * | 266 | 0.3 m augered through pavement | | | |
| 0.3 | Stiff | | 1 | SS | 13 | | | | o | | |
| | Clayey Silt to Silt With Sand Traces of Gravel Occasional Cobbles Hard (Glacial Till) | | 2 | SS | 77 | | | | o | | 8 55 26 1: |
| | | | 3 | SS | 106 | | 264 | 100/0.08 m | o | | |
| | | | 4 | SS | 63 | | | | | | 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 | | | | | | | | | | |

78 12 M




Ministry of Transportation and Communications

rio

# GRAIN SIZE DISTRIBUTION
## CLAYEY SILT TO SILT
WITH SAND, TRACES OF GRAVEL, OCC COBBLES (GLACIAL TILL )

FIG No 1

W P 28 - 78 - 02