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**PRELIMINARY FOUNDATION
INVESTIGATION AND DESIGN REPORT
THIRTEENTH CONCESSION OVERPASS
STRUCTURE SITE 30-351
HIGHWAY 400 WIDENING
FROM YORK / SIMCOE BOUNDARY
TO 1 KM SOUTH OF HIGHWAY 89
G.W.P. 40-00-00**

Submitted to:

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

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December 2001



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Location Plan

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PART A

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Appendix A Records of Boreholes and Test Results – 1960 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 the York / Simcoe Boundary northerly to 1 km south of Highway 89, in Simcoe County, Ontario. Foundation engineering services are required for the widening and / or replacement of six existing overpass and underpass structures, as well as four structural culverts.

This report addresses the widening and / or replacement of the existing West Gwillimbury Thirteenth Concession overpass structure. Existing subsurface data for this site from a report prepared by Universal Geotechnique Limited, referenced below, were used to determine the subsurface conditions for this preliminary design study:

- Report on Foundation Investigation for Proposed Crossing, Highway 400, Township of West Gwillimbury, County of Simcoe, for Ontario Department of Highways (W.P. 138-60) Report No. T.432/60, dated 1960 – GEOCRE File No. 31D-31.

The terms of reference for the scope of work are outlined in the MTO's Request for Quotation (RFQ) dated September 5, 2000, and in Golder Associates' subsequent letters dated December 13, 2000 and February 15, 2001; these letters are in regard to revisions to the foundation investigation program at the Simcoe Road 88 and West Gwillimbury Thirteenth Concession structure sites.

2.0 SITE DESCRIPTION

The existing West Gwillimbury Thirteenth Concession overpass structure is located about 8.5 km north of the Simcoe Road 88 (formerly Highway 88) interchange, in the Township of West Gwillimbury, County of Simcoe. The MTO has designated this overpass as Structure Site No. 30-351.

At the existing structure, the Highway 400 grade is at about Elevation 237.5 m to 238 m. Thirteenth Concession has been constructed in cut, with its grade at about Elevation 232 m under Highway 400. The existing single-span overpass structure was constructed in the early 1960s. 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 4 m wide spread footings which are founded at about Elevation 230.3 m.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at this site by Universal Geotechnique Limited in 1960. At that time, a total of five boreholes were drilled – Boreholes 1 and 2 were advanced on the west side of the highway, Borehole 3 was advanced at the centreline of Highway 400, and Boreholes 4 and 5 were advanced on the east side of the highway. The boreholes were extended to depths ranging from 9.1 m to 11 m below ground surface, to approximately Elevation 226.5 m to 228.4 m. Dynamic cone penetration tests were carried out adjacent to each borehole.

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. Groundwater conditions were observed in the open boreholes at the time of drilling. Laboratory index and classification tests, consisting of water contents, Atterberg limits and grain size distribution tests, were carried out on selected soil samples.

The borehole locations and elevations, referenced to the geodetic datum, were established by Universal Geotechnique 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 on a figure in the 1960 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 15 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 Schomberg Clay Plains; the Peterborough Drumlin Field; and a second lobe of the Simcoe Lowlands. Along Highway 400, the southern lobe of the Simcoe Lowlands is present from the southern limit of the project at the York / Simcoe boundary (South Canal Road) to the North Canal / Canal Road site. The Schomberg Clay Plains are present north of this site, to 2 km north of Simcoe Road 88 (formerly Highway 88). The Peterborough Drumlin Field extends from 2 km north of Simcoe Road 88 to about 3 km south of Highway 89. The northern lobe of the Simcoe Lowlands extends from about 3 km south of Highway 89 to beyond the northern limit of this project.

The surficial soils in the Schomberg Clay Plains consist primarily of varved clay and silt deposits. These varved deposits overlie till within drumlins as found in the Peterborough Drumlin Field. The drumlins (glacially-shaped hills) are completely or partially buried by the clay and silt deposits, depending on the size of the drumlin. The varved clay and silt deposits are typically about 5 m thick, although deeper deposits have been found in some locations.

The surficial soils in the Peterborough Drumlin Field, in which the West Gwillimbury Thirteenth Concession overpass is located, consist primarily of gravelly sand till or sand and gravel deposits. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins.

Along Highway 400, the Simcoe Lowlands include the Holland River valley, the shores of Kempenfelt Bay, the Nottawasaga River, and Innisfil Creek. The Holland River valley at the southern end of this project extends southwest from Cook Bay, at the south end of Lake Simcoe; it was once a shallow extension of the lake. The floor of the valley is covered by extensive deposits of loose silts and soft clays, which overlie a till sheet. In localized areas, these silts and clays are overlain by a thin, poorly graded sand of deltaic origin. Because the valley is depressed and poorly drained, a surficial cover of peat has formed in many areas. The surficial soils of the northern lobe of the Simcoe Lowlands consist primarily of sand, although silt, clay or peat may be found in low-lying areas.

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the 1960 boreholes, the results of the laboratory testing carried out on selected soil samples, and the results of dynamic

cone penetration testing are given on the Record of Borehole sheets, drawings and tables in the attached 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 west side of the highway, Borehole 3 was advanced at the centreline of Highway 400, and Boreholes 4 and 5 were advanced on the east side of the highway. The approximate locations and ground surface elevations for these borings are shown on the attached Drawing 1.

In summary, the subsoils at the site consist of fill overlying a deposit of loose to dense clayey silt to silty sand till, which contains sand lenses. A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Fill

A layer of fill ranging in thickness from about 0.5 m to 1.4 m was encountered at ground surface in all the boreholes. The fill consists primarily of sand and gravel; clay was also found in the fill at Borehole 1. Standard Penetration Test (SPT) 'N' values measured in the fill ranged from 13 to 23 blows per 0.3 m of penetration, indicating that the fill has a compact relative density.

4.2.2 Sandy Silt to Silty Sand Till

A deposit of till, described as "clayey silty sand" till on the 1960 borehole records, was encountered below the fill to the maximum depth of each borehole; the boreholes were advanced to between 9.1 m and 11 m depth, and terminated between Elevations 226.5 m and 228.4 m. Sand pockets or layers were noted within the till in some of the boreholes. Grain size distribution test results obtained for two samples of the till are shown on Figure 1 in Appendix A; these test results indicate that the till contains a significant proportion of sand, and less than 5 per cent clay-size particles. Based on the 1960 descriptions, the grain size distribution results and the Atterberg limit results (discussed below), this till is considered to range in composition from sandy silt to silty sand, containing some gravel and trace clay.

The natural moisture contents measured on samples of till ranged from 8 to 15 per cent. Atterberg Limits testing on samples of the till from Borehole 3 measured plastic limits of 8 to 10 per cent,

liquid limits of 11 to 13 per cent, and plasticity indices of about 3 per cent. The results of the Atterberg Limits indicate that the tested samples of till are silt of slight plasticity.

The measured SPT 'N' values ranged from 9 to 75 blows per 0.3 m of penetration, but were typically between 20 and 40 blows per 0.3 m of penetration indicating that the deposit is typically compact to dense. A zone of loose till, with measured SPT 'N' values of 9 to 12 blows per 0.3 m of penetration, was encountered between Elevations 233 m and 231 m (between about 4 m and 6 m depth) in Borehole 2. The till is generally dense below about Elevation 231 m.

4.3 Groundwater Conditions

The groundwater conditions were observed in the open boreholes during the 1960 drilling investigation. The measured groundwater levels were at about 0.5 m to 1.5 m depth (approximately Elevation 236 m to 237 m), coincident with the top 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.

GOLDER ASSOCIATES LTD.




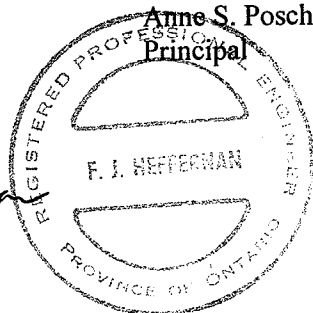
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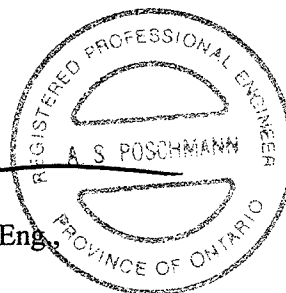
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Anne S. Poschmann, P.Eng.,
Principal



PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
THIRTEENTH CONCESSION OVERPASS
STRUCTURE SITE 30-351
HIGHWAY 400 WIDENING
FROM YORK / SIMCOE BOUNDARY
TO 1 KM SOUTH OF HIGHWAY 89
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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 West Gwillimbury Thirteenth Concession 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 1960 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 ultimate configuration of ten lanes. The primary options under consideration involve widening into the median, and using a 22 m wide open median with widening on the outside of the existing highway; depending on which option is adopted, it is expected that the existing highway will be widened by between 10 m and 29 m. Widening and / or replacement of the existing Thirteenth Concession overpass will, therefore, be necessary.

Based on the general layout drawing for the existing single-span structure, the abutments are supported on spread footings founded at about Elevation 230.3 m, below the Thirteenth Concession cut grade of about Elevation 232 m. The Highway 400 grade at the structure site is at about Elevation 237.5 m to 238 m.

5.2 Bridge Foundation Options

The soils at the site consist of fill overlying a deposit of predominantly compact to dense sandy silt to silty sand till. The till is generally dense, with SPT 'N' values greater than 30 blows ranging up to 56 blows per 0.3 m of penetration, below Elevation 231 m. Based on these subsurface conditions, it is recommended that the widening or replacement structure be founded on spread footings placed on the till deposit below the Thirteenth Concession grade. Consideration could also be given to the use of perched abutments, founded on spread footings placed within the approach embankments. Preliminary recommendations for spread footings, including perched abutments, are given in the following section.

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. However, the 1960 borehole investigation did not encounter “practical refusal” within the drilled depth of 9 m to 11 m (to about 4 m to 5.5 m below the Thirteenth Concession cut grade). Consequently, the preliminary recommendations for driven steel H-piles (given in Section 5.4) have been determined based on shaft friction with some allowance for tip resistance.

5.3 Spread Footings

For preliminary design of the bridge abutment footings, spread footings may be placed at a design founding level of Elevation 230.3 m, to be founded on the dense sandy silt to silty sand 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; in this case, it may be necessary to use a well-compacted granular pad to found the stepped walls.

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

5.3.1 Axial Geotechnical Resistance

Spread footings placed on the properly prepared undisturbed till deposit at the design elevation given above may be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 650 kPa, assuming a 4 m wide footing. The settlement of footings founded on the 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.

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 (OHBD).

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 OHBD. The angle of friction between the concrete and the undisturbed till founding soils should be taken as 24 degrees; the corresponding coefficient of friction, $\tan \delta$, 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 of the replacement structure on steel H-piles driven to found within the dense to very dense silty sand to sandy silt till deposit. The 1960 boreholes were extended to approximately 9 m to 11 m below the Highway 400 grade, or up to about 5.5 m below the Thirteenth Concession cut grade. These boreholes did not encounter “practical refusal” (i.e. SPT ‘N’ values greater than 100 blows per 0.3 m of penetration). For preliminary design, the piles could be assumed to extend to the maximum depth of investigation, at approximately Elevation 227 m. If the pile caps are placed at the Thirteenth Concession cut grade, the piles would be less than about 5 m in length. Therefore, it will likely be necessary to perch the pile caps within the approach embankments; in this case, the piles could be up to about 8 m in length assuming a pile cap base at about Elevation 235 m.

5.4.1 Axial Geotechnical Resistance

For preliminary design of deep foundations, the factored axial resistance at ULS for steel HP 310 x 79 H-piles driven to Elevation 227 m to found within the generally dense till deposit

may be taken as 350 kN. This assumes that the pile caps are perched above the Thirteenth Concession cut grade with a pile length of at least 8 m. The axial resistance at SLS for 25 mm of settlement may be taken as 300 kN. Assuming dense glacial till continues at depth, the axial resistance at ULS for an HP 310 x 79 H-pile driven to Elevation 218 m (i.e. a 17 m long pile) may be taken as 800 kN for preliminary design purposes. The SLS value may be taken as 700 kN. The above geotechnical resistances are based on an extrapolation to depths which are much greater than the existing borehole depths. For detailed design, boreholes should be advanced to at least 3 m below practical refusal or at least Elevation 210 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:

$$k_h = \frac{n_h z}{B} \quad \text{where} \quad \begin{array}{l} n_h \text{ is the constant of subgrade reaction} \\ z \text{ is the depth (m)} \\ B \text{ is the pile diameter (m)} \end{array}$$

Based on the 1960 borehole data, the subsoils in front of the piles will consist of compact to very dense silty sand to sandy silt till. The following range 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
Silty sand to sandy silt till above Elevation 231 m	5 to 10 MPa/m
Silty sand to sandy silt till below Elevation 231 m	5 to 15 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:

<i>Pile Spacing in Direction of Loading</i> <i>d = Pile Diameter</i>	<i>Subgrade Reaction</i> <i>Reduction Factor R</i>
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 of 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 wall 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 Thirteenth Concession overpass are up to about 1.5 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 new or widened portions 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 Thirteenth Concession cut is up to about 5 m in depth, with the road grade at about Elevation 232 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. 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 boreholes was measured at about Elevation 236 m to Elevation 237 m. 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

Groundwater seepage into the footing excavations is expected to occur 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 foundation works. Surface water run-off should be directed away from the footing excavations.

The sandy silt to 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 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 compact to dense silty sand 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 fill and till soils would be classified as Type 3 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.

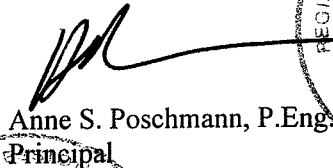
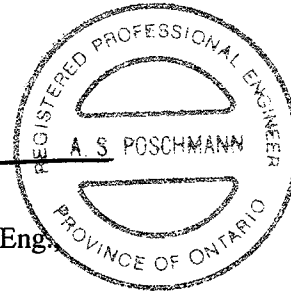
5.7.3 Settlement

Foundation loading will result in deformation of the ground and consequent settlement of the foundation elements and superstructure. Differential settlement will occur between the existing and widened portions of the structure. The potential for differential settlement should be reassessed during the detailed design stage, once the proposed bridge configuration is established.

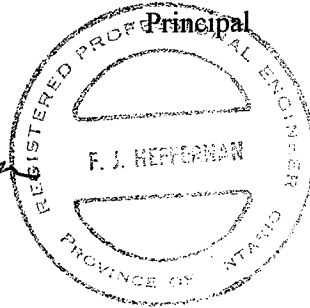
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Designated MTO Contact



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DIST HWY 400
CONT. No.
GWP No. 40-00-00

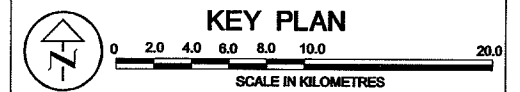
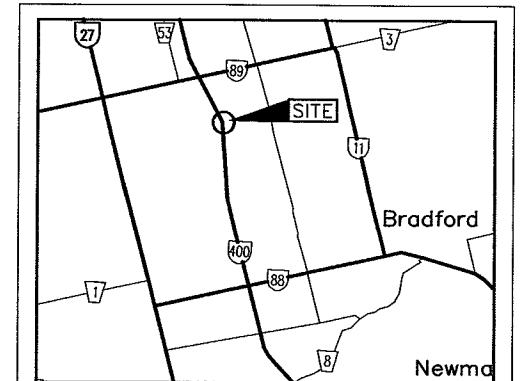


THIRTEENTH CONCESSION OVERPASS
HWY 400
BOREHOLE LOCATION PLAN

SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



LEGEND

- Borehole, previous investigation
- Borehole, present investigation

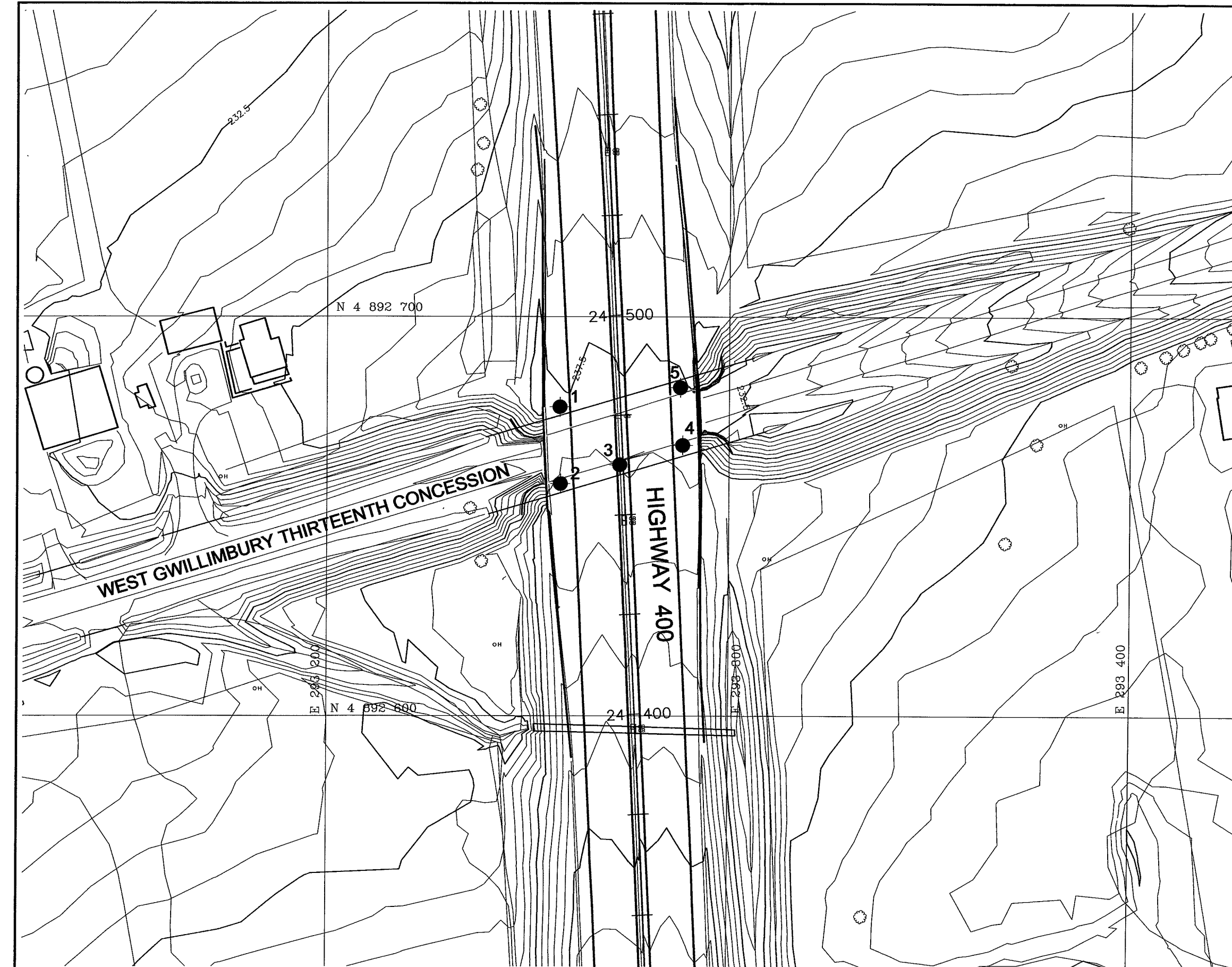
No.	ELEVATION	LOCATION	
		NORTHING	EASTING
1	237.3	4,892,677	293,258
2	237.7	4,892,658	293,258
3	237.5	4,892,663	293,273
4	237.4	4,892,668	293,289
5	237.1	4,892,682	293,288

REFERENCE

This drawing was created from digital file "33813.dwg"
provided by URS Cole Sherman

NO.	DATE	BY	REVISION
Geocres No.			
HWY. No. 400	PROJECT NO.: 001-1151		
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-351
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

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



P1161001.DWG

APPENDIX A

**RECORDS OF BOREHOLES AND TEST RESULTS
1960 SUBSURFACE INVESTIGATION**

UNIVERSAL **GEOTECHNIQUE** LIMITED

SOIL MECHANICS LABORATORY

BOREHOLE LOG

PROJECT Proposed Crossing Highway 400 (W.P. 138-60) ORDER NO. T-432/60

CLIENT Ontario Department of Highways

BOREHOLE NO. BH.1 DIAMETER 2-1/2" CASING 2-1/2"

BOREHOLE LOCATION See Sketch INCLINATION Vertical BEARING

FORM G-1A 900
11/55 Station 01/10

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
	237.3m	778.4		Zero - 0m			
Firm brown sand, gravel and clay FILL.			• 1			23	
Firm light brown clayey silty SAND with fine to medium subangular gravel.			• 2			15	Damp to moist. High dry strength.
do	234.7m	770	• 3			24	Moist. High dry strength.
do			• 4			18	do
do			• 5			23	Moist. Medium to high dry strength.
do			• 6			23	do
	231.8m	760.4		18'-0" - 5.5m			
Dense gray clayey silty SAND with fine to medium subangular gravel.	760		• 7			34	Damp. High dry strength.
do			• 8			56	do
Includes sand pockets.	228.6m	750					
do	228.1m	748.4	• 9	30'-0" - 9.1m		42	do
				End of Borehole			

SCALE 1" = 5'-0" • DISTURBED SAMPLE ■ UNDISTURBED SAMPLE

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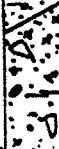
BOREHOLE LOG

PROJECT Proposed Crossing Highway 400 (W.P. 138-60) ORDER NO. T.432/60

CLIENT Ontario Department of Highways

BOREHOLE NO. BH.2 DIAMETER 2-1/2" CASING 2-1/2"

BOREHOLE LOCATION See Sketch INCLINATION Vertical BEARING

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
237.7m Firm brown sand and gravel. FILL.	779.9		• 1	Zero - 0m		23	Damp.
			• 2			20	No recovery
Firm light brown clayey silty SAND with fine to medium subangular gravel.			• 3			21	Damp. High dry strength.
do 234.7m	770		• 4			23	Damp. Medium to high dry strength.
do			• 5			26	do
Loose to firm grey clayey silty SAND with fine to medium subangular gravel.			• 6			9	Moist. High dry strength.
do 231.6m	760		• 7			12	do
do			• 8			11	do
Firm do			• 9			24	No recovery.
do			• 10			22	No recovery.
Dense grey clayey silty SAND with fine to medium subangular gravel.			• 11			34	Damp. High dry strength.
do 228.6m	750		• 12			62	Damp. Medium to high dry strength.
includes sand pockets			• 13			30	do
do 227.3m				34'-0" = 10.4m End of Borehole			










SCALE 1" = 5'-0" • DISTURBED SAMPLE

■ UNDISTURBED SAMPLE

SOIL MECHANICS LABORATORY

BOREHOLE LOG

PROJECT Proposed Crossing Highway 400 (W.P. 138-60) ORDER NO. T.432/60CLIENT Ontario Department of HighwaysBOREHOLE NO. BH.3 DIAMETER 2-1/2" CASING 2-1/2"BOREHOLE LOCATION See Sketch INCLINATION Vertical BEARING

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
237.5m Firm brown sand with some gravel, iron stained. FILL.	779.3		• 1	Zero - 0m		13	Damp. do
do 236.1m Firm light brown clayey silty SAND with gravel.	774.6		• 2	4'-9" - 1.5m		24	Damp - Moist Medium to high dry strength.
Firm light brown clayey silty SAND with fine to medium subangular gravel.			• 3			22	do
do 234.7m	770		• 4			20	do
Dense do			• 5			34	Moist. High dry strength.
Dense grey clayey silty SAND with fine to medium subangular gravel.			• 6	14'-9" - 4.5m		37	do
do 231.6m	760		• 7			38	do
do			• 8			48	Damp. High dry strength.
do 228.6m 228.4m	750		• 9	30'-0" - 9.1m End of Borehole		60	do

SCALE: 1" = 5'-0" • DISTURBED SAMPLE

■ UNDISTURBED SAMPLE

UNIVERSAL GEOTECHNIQUE LIMITED

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BOREHOLE LOG

PROJECT Proposed Crossing Highway 400 (W.P. 138-60) ORDER NO. T.432/60

CLIENT Ontario Department of Highways

BOREHOLE NO. BH.4 DIAMETER 2-1/2" CASING 2-1/2"

BOREHOLE LOCATION See Sketch INCLINATION Vertical BEARING

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
Brown sand with occasional gravel. FILL.	779.0			Zero	0m		Damp.
Firm brown sand with clay and dark organic concentrations. FILL.	237.4m		• 1	2'-0"	0.6m	18	Damp.
Firm light brown somewhat clayey silty SAND with fine to medium subangular gravel.			• 2			19	Moist. Medium to high dry strength.
do	234.7m		• 3			22	do
Includes sand pockets.			• 4			28	No recovery.
do			• 5			28	Moist. High dry strength.
dc	233.0m		• 6	14'-6"	4.4m	29	do
Firm grey clayey silty SAND with fine to medium subangular gravel.			• 7			28	Damp. High dry strength.
do	231.6m		• 8			43	do
do			• 9			33	do
do	228.6m		• 10			46	do
do			• 11			31	No recovery.
do	226.4m		• 12	36'-0"	11.0m	39	Damp. High dry strength.
				End of Borehole			

SCALE: 1" = 5'-0" • DISTURBED SAMPLE

■ UNDISTURBED SAMPLE

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







BOREHOLE LOG

PROJECT Proposed Crossing Highway 400 (W.P. 138-60) ORDER NO. T.432/60

CLIENT Ontario Department of Highways

BOREHOLE NO. BH.5 DIAMETER 2-1/2" CASING 2-1/2"

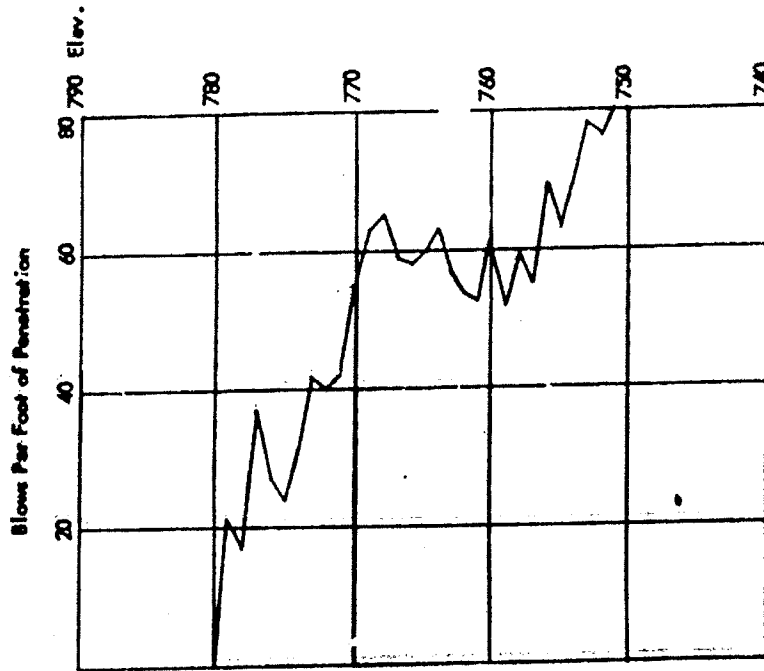
BOREHOLE LOCATION See Sketch INCLINATION Vertical BEARING -

DESCRIPTION OF STRATA	ELEVATION	LOGGED SAMPLE	DEPTH	THICKNESS	N	REMARKS
Brown sand and gravel. FILL. Firm brown sand, gravel and clay. FILL.	777.9 237.7m		Zero	0m	20	Damp.
Firm light brown clayey silty SAND with fine to medium subangular gravel.					27	Moist. Medium to high dry strength.
do 234.7m	770				24	do
do					16	do
do	232.9m				75	High N due to gravel.
Firm grey clayey silty SAND with fine to medium subangular gravel.	764.2		13'-9"	4.2m	20	Moist. Medium to high dry strength.
do 231.6m	760				28	Moist. High dry strength.
do					34	No recovery.
do	228.6m				29	Moist. High dry strength.
Dense do 227.9m	750		32'-0"	9.8m	53 45	Damp. High dry strength. do
			End of Borehole			

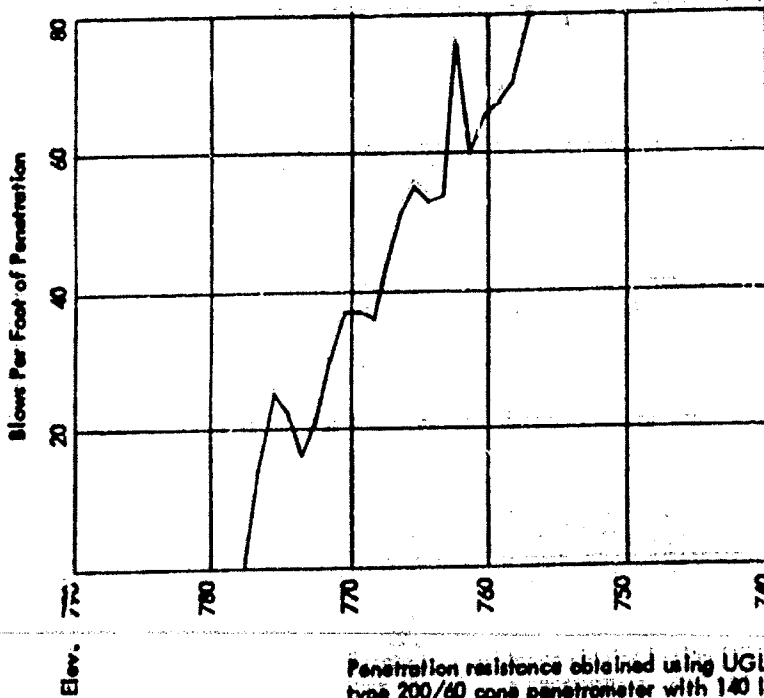
SCALE: 1" = 5'-0" • DISTURBED SAMPLE

■ UNDISTURBED SAMPLE

PT. 2



PT. 1



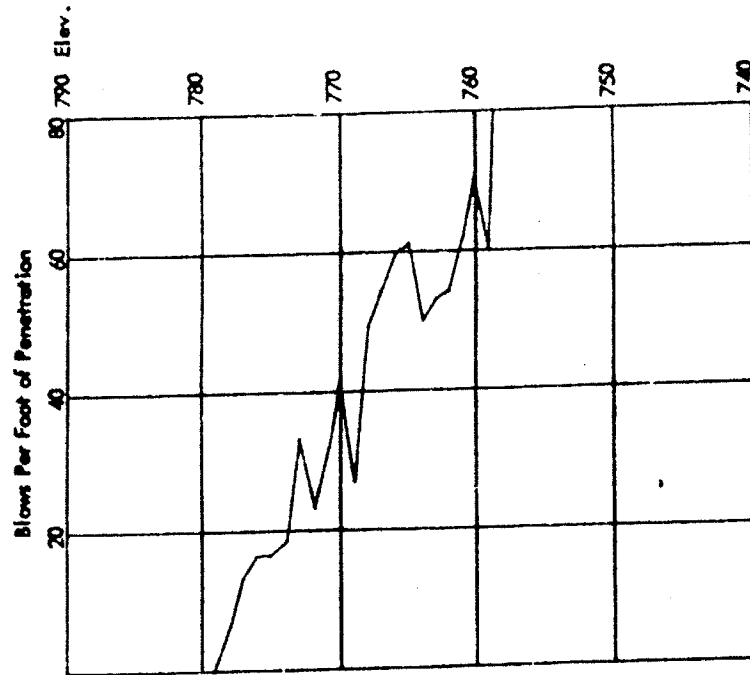
Penetration resistance obtained using UGL
type 200/60 cone penetrometer with 140 lb.
hammer falling 30".

PROJECT Proposed Crossing Highway 400 (W.P. 138-60)
TITLE Dynamic Penetration Test Diagrams
DRG. NO. 4 ORDER NO. T.432/60

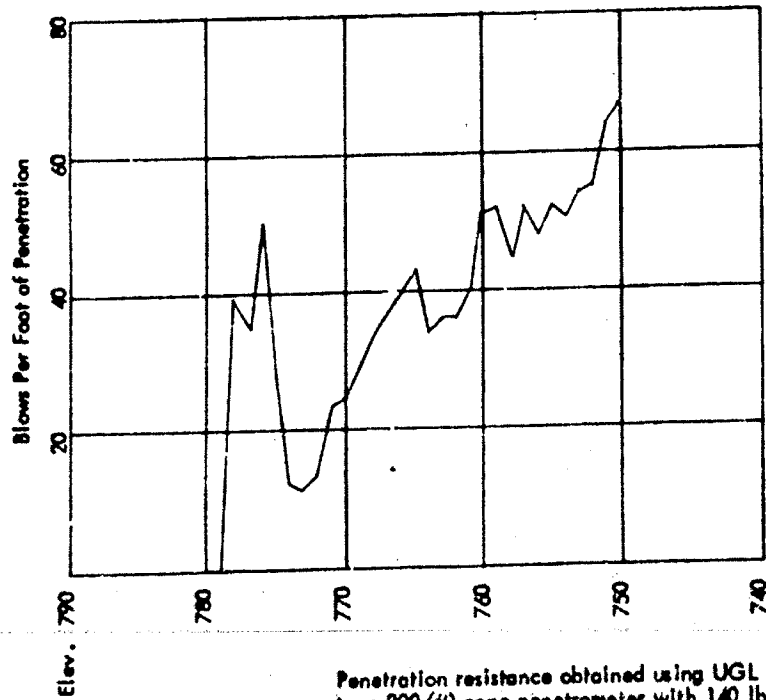


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PT. 3



PT. 2A



Penetration resistance obtained using UGL
type 200/60 cone penetrometer with 140 lb.
hammer falling 30".

PROJECT Proposed Crossing Highway 400 (W.P. 138-60)

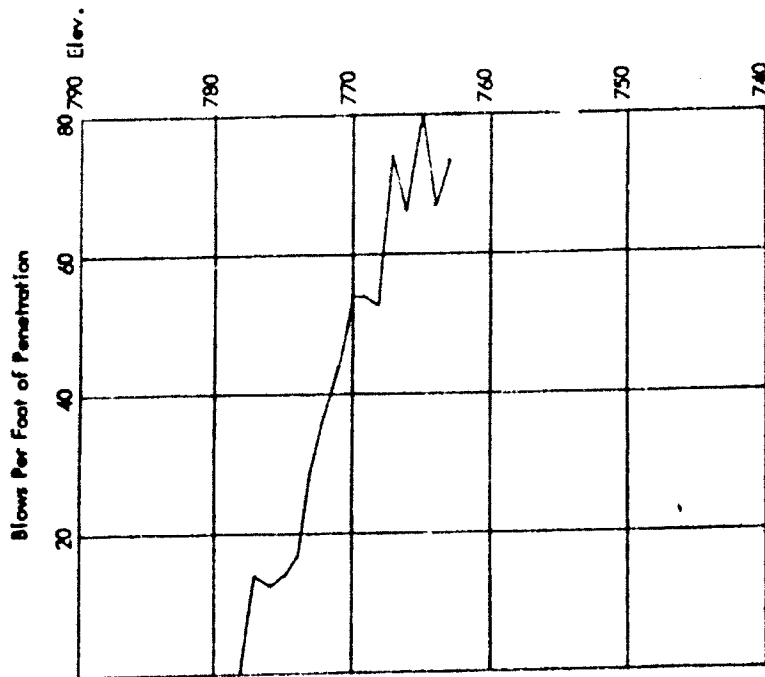
TITLE Dynamic Penetration Test Diagrams

DRG NO 5 ORDER NO T.432/60

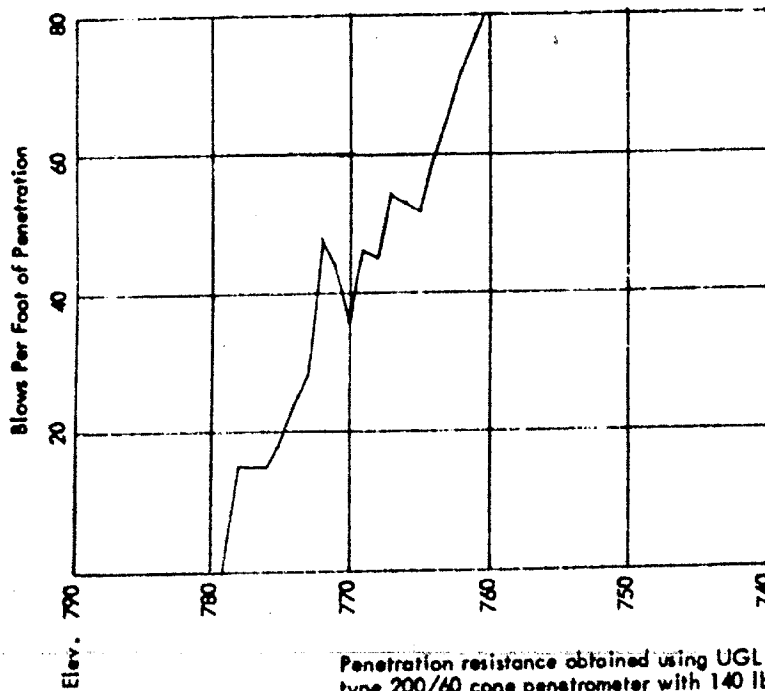


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PT. 5



PT. 4



Penetration resistance obtained using UGL
 type 200/60 cone penetrometer with 140 lb.
 hammer falling 30".

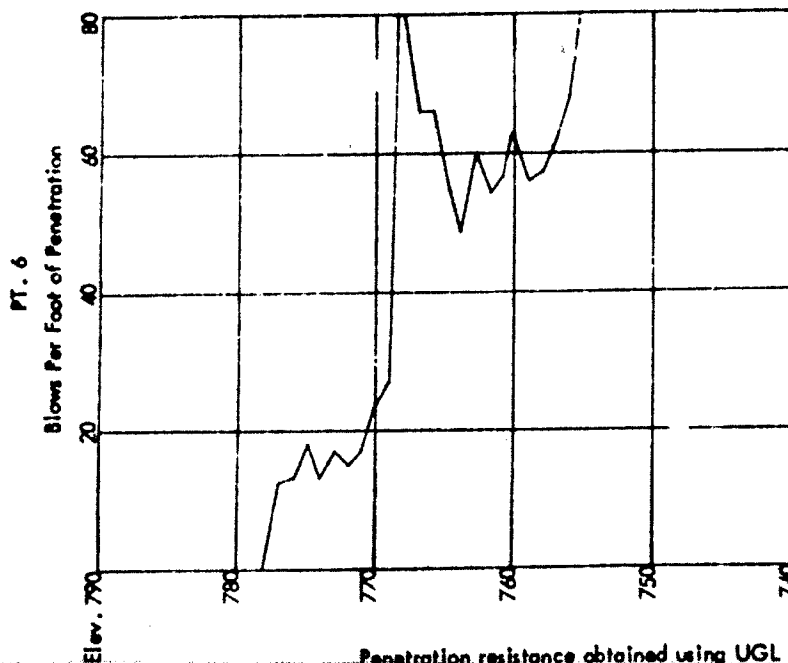
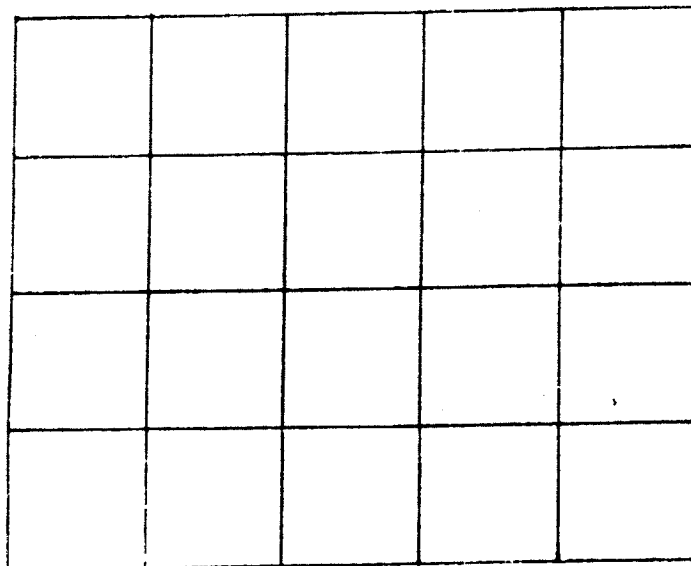
PROJECT Proposed Crossing Highway 400 (W.P. 138-60)

TITLE Dynamic Penetration Test Diagrams

ORG NO. 6 ORDER NO. I. 432/60



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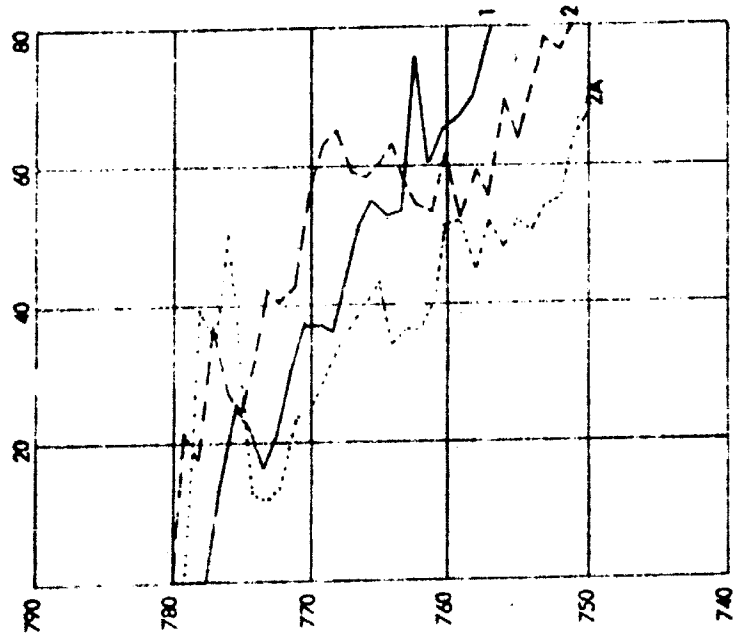
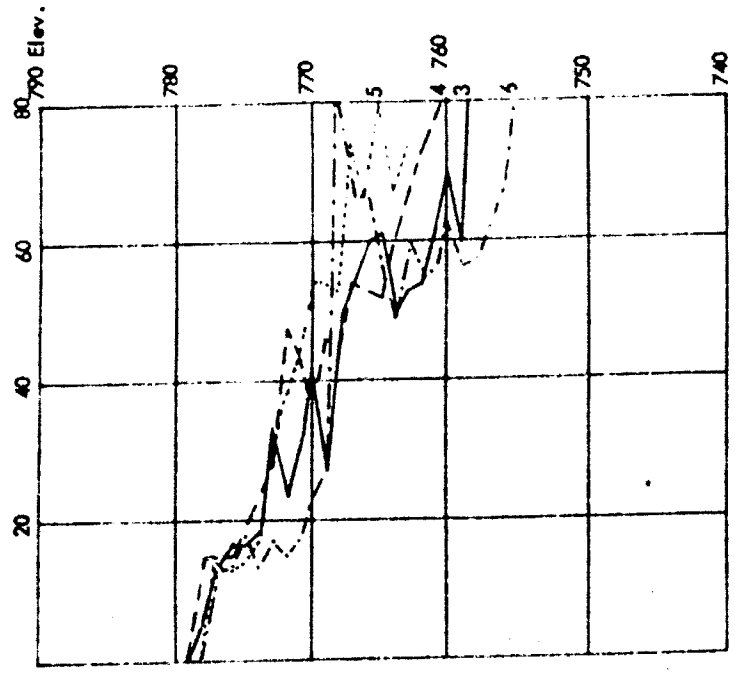


Penetration resistance obtained using UGL type 200/60 cone penetrometer with 140 lb. hammer falling 30".

PROJECT Proposed Crossing Highway 400 (W.P. 138-60)
 TITLE Dynamic Penetration Test Diagram
 DRG. NO. 7 ORDER NO. T.432/60



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COMBINED PLOTTING OF PENETRATION TESTS

Penetration resistance obtained using UGL type 200/60 cone penetrometer with 140 lb. hammer falling 30".

PROJECT Proposed Crossing Highway 400 (W.P. 138-60)
 TITLE Dynamic Penetration Test Diagrams
 DRG NO. 8 ORDER NO. T.432/60

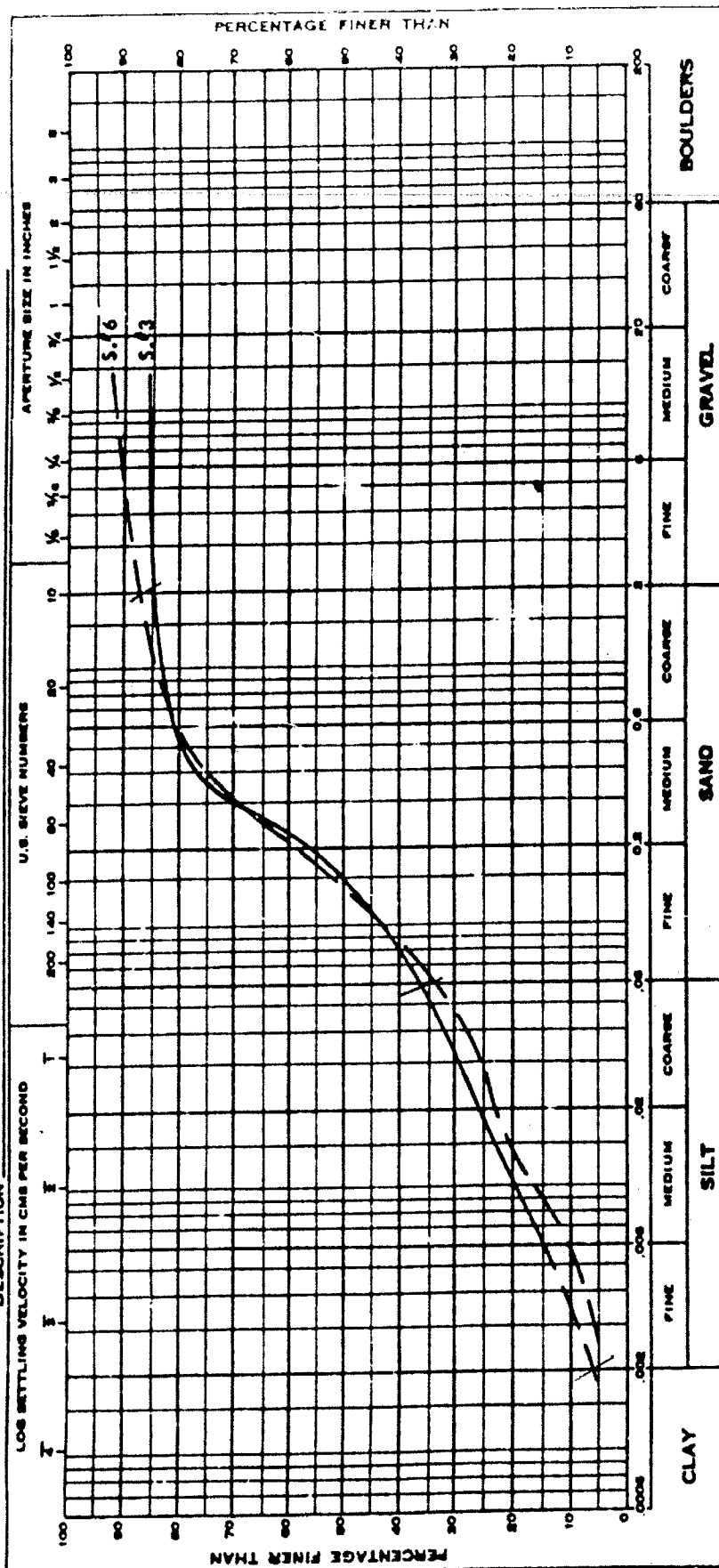


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**SOIL MECHANICS LABORATORY
MECHANICAL ANALYSIS**

T. 432/80

PROJECT Proposed Crossing Highway 400 (W.P. 138-60) BORING NO. BH.3 SAMPLE NO. 3A.6 DATE OF TEST 11/1/68



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PARTICLE SIZE IN MM

Fig. 20.

Page 2-2 104-6-31
Source: Government of

TABLE NO.

SHEET NO. _____

NOTE: ALL SAMPLES OBTAINED WITH 2" DIAMETER SPLIT BARREL.

TITLE _____



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