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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT ST. VINCENT STREET UNDERPASS STRUCTURE SITE 30-394 HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11 G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

Submitted to:

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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 subsurface conditions at the St. Vincent Street underpass structure in Barrie, Ontario. Existing subsurface data for this site from an investigation conducted for the Department of Highways, Ontario (DHO) in 1964 were used to determine the subsurface conditions for this preliminary design study. The 1964 report was prepared by H.Q. Golder and Associates Ltd. ("*Soil Conditions and Foundations, Proposed Highway 400 – St. Vincent Street Underpass, Barrie, Ontario*", dated December 1964 – GEOCRES File No. 31D-207).

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

2.0 SITE DESCRIPTION

The existing four-span St. Vincent Street underpass structure is located about 1.5 km north of Bayfield Street (Highway 26) and 5 km south of Highway 11, in Barrie, Ontario. The MTO has designated this underpass as Structure Site 30-394.

At this structure site, the Highway 400 grade is at about Elevation 261.5 m, rising slightly northward. St. Vincent Street has been constructed in fill, with approach embankments up to 6 m in height. The St. Vincent Street grade is at about Elevation 267.5 m over Highway 400.

The existing four-span underpass structure was constructed in the mid-1960s under Contract 66-149. According to the general layout drawing for this contract, which was provided by Morrison Hershfield (the structural designers for this preliminary study), the abutments, wing walls and piers are supported on spread footings which are founded at Elevation 259.2 m.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at this site for the Department of Highways, Ontario (DHO) in November 1964, by H.Q. Golder and Associates Ltd. At that time, a total of five boreholes, with accompanying dynamic cone penetration tests, were advanced – one borehole at each of the then-proposed abutment and pier locations. Borehole 1 was advanced at the central pier location; Boreholes 2 and 3 were drilled at the west pier and abutment locations, respectively; and Boreholes 4 and 5 were located at the east abutment and pier locations, respectively. The boreholes were advanced to between 5 m and 10.5 m below Highway 400 grade.

Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. The water levels in the open borehole were observed during and following the drilling operations, and a piezometer was installed in each borehole to monitor the groundwater conditions at the site. Laboratory testing, consisting of natural moisture content and bulk unit weight determinations, and grain size distribution analyses, was carried out on selected soil samples.

The borehole locations and elevations, referenced to the geodetic datum, were established by H.Q. Golder and Associates Ltd. Approximate northing and easting co-ordinates consistent with the MTM NAD83 survey system, currently in use on this project, have been determined by Golder Associates based on the borehole locations given in the 1964 report. The approximate borehole locations and northing and easting co-ordinates are shown on the attached Drawing 1.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

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

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

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

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

4.2 Site Stratigraphy

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

Borehole 1 was advanced at the central pier location; Boreholes 2 and 3 were drilled at the west pier and abutment locations, respectively; and Boreholes 4 and 5 were located at the east abutment and pier locations, respectively. These boreholes were advanced to between 5 m and 10.5 m below Highway 400 grade. The approximate locations and ground surface elevations for these borings and probeholes are shown on the attached Drawing 1.

In summary, the soils below the Highway 400 grade at this site consist of road base fill or topsoil underlain by a compact to very dense silty sand till deposit. A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Fill

Up to 1.2 m of silty sand to sand and gravel fill was encountered in the 1964 boreholes; this fill comprised the St. Vincent Street road base. The base of the fill was encountered at about Elevation 260 m to 261 m. This cohesionless fill was considered to have a compact relative density, based on a measured Standard Penetration Test (SPT) 'N' value of 15 blows per 0.3 m of penetration.

4.2.2 Silty Sand Till

Below the surficial fill, the boreholes encountered a till deposit which extended to the maximum investigated depth of 10 m (approximately Elevation 250.5 m). The till composition ranges from silty sand to sand and silt, containing trace to some clay and gravel; layers or lenses of silty clay were noted within the till in one of the boreholes. The grain size distribution test results for samples of the till are shown on the figures contained in Appendix A.

The SPT 'N' values measured in the upper 3 m to 5 m of the till deposit range from 18 to greater than 100 blows per 0.3 m of penetration, but were typically between 20 and 50 blows per 0.3 m of penetration; the upper portion of the deposit has a generally compact to dense relative density. Below this depth (below about Elevation 256 m), the measured SPT 'N' values were generally greater than 100 blows per 0.3 m of penetration, indicating that this portion of the till deposit has a very dense relative density. The SPT 'N' values measured in the 1964 boreholes are summarized on Figure 4 in Appendix A.

4.3 Groundwater Conditions

Piezometers were installed in each borehole to monitor the groundwater levels at the site. The groundwater levels measured in November 1964 were between about 4.5 m and 7.5 m below ground surface, at approximately Elevation 254 m to 257 m. The 1964 report by H.Q. Golder and Associates Ltd. indicates that the year 1964 was relatively dry and consequently the water levels at the site were considered to be below normal levels.

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

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
ST. VINCENT STREET UNDERPASS
STRUCTURE SITE 30-394
HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

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

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

The existing St. Vincent Street underpass is a four-span structure, built in 1964 to accommodate future widening of Highway 400. It is understood that the existing structure will accommodate widening to the ultimate ten-lane configuration. However, if a twelve-lane express / collector system is adopted, or if there are changes in the median width, replacement of the existing structure will be necessary. In addition, if widening of the local road is required, it will be necessary to widen or twin the existing structure.

Based on the general layout drawing for the existing four-span structure, the abutments, wing walls and piers are supported on spread footings which are founded at about Elevation 259.2 m. The Highway 400 grade is at about Elevation 261.5 m, and St. Vincent Street has been constructed on embankment fill, with approach embankments up to 6 m in height and grade at about Elevation 267.5 m over Highway 400.

5.2 Bridge Foundation Options

The soils below the Highway 400 level consist of road base fill overlying a deposit of compact to very dense silty sand to sand and silt till. Based on these subsurface conditions, consideration could be given to founding the replacement structure on spread footings placed on the till stratum.

Consideration could also be given to the use of perched abutments, founded on spread footings placed on a compacted granular pad within the approach embankments.

Alternatively, if integral abutments are under consideration for the replacement structure, the abutments could be supported on steel H-piles driven to found within the very dense silty sand to sand and silt till deposit.

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

5.3 Spread Footings

For preliminary design, spread footings for the abutments and pier may be placed at a design founding level of Elevation 259 m. The footings would be founded on the compact to very dense silty sand to sand and silt till deposit. Any associated wing wall or retaining wall footings may be stepped upward away from the abutments such that a minimum soil cover of 1.5 m is maintained above the underside of the footings. To minimize requirements for groundwater control, the footings should be maintained above the groundwater level, which was measured during the 1964 investigation to be between Elevation 254 m and 257 m; it is noted that 1964 was a relatively dry year and these measured water levels could be lower than the normal levels at the site.

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

5.3.1 Axial Geotechnical Resistance

Spread footings placed on the properly prepared silty sand to sand and silt till deposit at the design elevation given above may be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 900 kPa, assuming a 3 m wide footing. The settlement of footings founded on the silty sand to sandy silt will be dependent on the footing size and configuration, and on the applied loads. The majority of this settlement will take place during construction itself; however, in the case of widening, it should be noted that the settlement will be differential with respect to the existing footings. For preliminary design purposes, the geotechnical resistance at Serviceability Limit States (SLS) may be taken as 400 kPa. The geotechnical resistance at SLS will have to be reviewed following the detailed design stage of subsurface investigation, once the proposed footing size and loadings as well as the configuration with respect to the existing footings are known.

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

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

5.3.2 Resistance to Lateral Loads

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

5.3.3 Frost Protection

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

5.4 Driven Steel H-Piles

Consideration could be given to supporting a new underpass structure on steel H-piles driven to found within the very dense silty sand to sand and silt till deposit. In the 1964 borings, the measured Standard Penetration Test (SPT) 'N' values were generally greater than 100 blows per 0.3 m of penetration below Elevation 256 m. It is possible that a suitable driving resistance may not be achieved above the base of the 1964 boreholes. In this regard, it should be noted that additional borehole investigation work will be required at the proposed foundation locations during detailed design in order to confirm the thickness and relative density of the till stratum and establish design tip elevations.

For preliminary design, the pile tip elevation may be taken at Elevation 250 m, about 17.5 m below St. Vincent Street grade and 11.5 m below Highway 400 grade.

5.4.1 Axial Geotechnical Resistance

For preliminary design, the factored axial resistance at ULS for steel HP 310 x 110 H-piles driven to found within the very dense silty sand to sand and silt till at a design tip elevation of 250 m may be taken as 1,400 kN. The axial resistance at SLS for 25 mm of settlement may be taken as 1,200 kN.

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

5.4.2 Resistance to Lateral Loads

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

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

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

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

For the embankment fill and the upper, compact to dense portion of the silty sand to sand and silt till soils through which the piles would be driven, the range in value of n_h may be taken as 5 MPa/m to 10 MPa/m in the structural analysis.

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

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

5.4.3 Frost Protection

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

5.5 Lateral Earth Pressures

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

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

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

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

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

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

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

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

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

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

5.6 Embankment Design

Based on the topographic information on the Engineering and Title Records plates and on site reconnaissance, the existing approach embankment side slopes are formed at a gradient of about 2 horizontal to 1 vertical (2H:1V). For raising and widening of the existing St. Vincent Street embankment to accommodate a widened underpass structure, the new embankment side slopes should be formed at a maximum gradient of 2H:1V. The widening of the embankment should be carried out using conventional fill placement and compaction practices, and benching of the existing embankment side slopes should be carried out to key in the new fill.

5.7 Design and Construction Considerations

5.7.1 Dewatering

Provided that the spread footing founding level or pile cap underside is maintained above the groundwater level, seepage into the footing or pile cap excavations is expected to be minor. Pumping from properly-filtered sumps or a filtered drain placed at the base of the excavation should provide sufficient groundwater control during foundation works. Surface water run-off, which is expected to be more significant than groundwater seepage, should be directed away from the footing excavations.

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

5.7.2 Excavation

The footing or pile cap excavations will extend a minimum of 1.5 m below the lowest surrounding grade, through embankment fill or road base fill, into the compact to dense portion of the silty sand to sand and silt till deposit. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. The embankment fill and compact to dense till would be classified as Type 2 soil. Temporary open-cut slopes should therefore be made to within 1.2 m of the excavation base, and maintained at a minimum gradient of 1 horizontal to 1 vertical (1H:1V). The remaining depth of 1.2 m may be excavated near-vertical. Where space restrictions dictate, footing excavations could also be carried out within a braced excavation.

5.7.3 Obstructions

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

GOLDER ASSOCIATES LTD.

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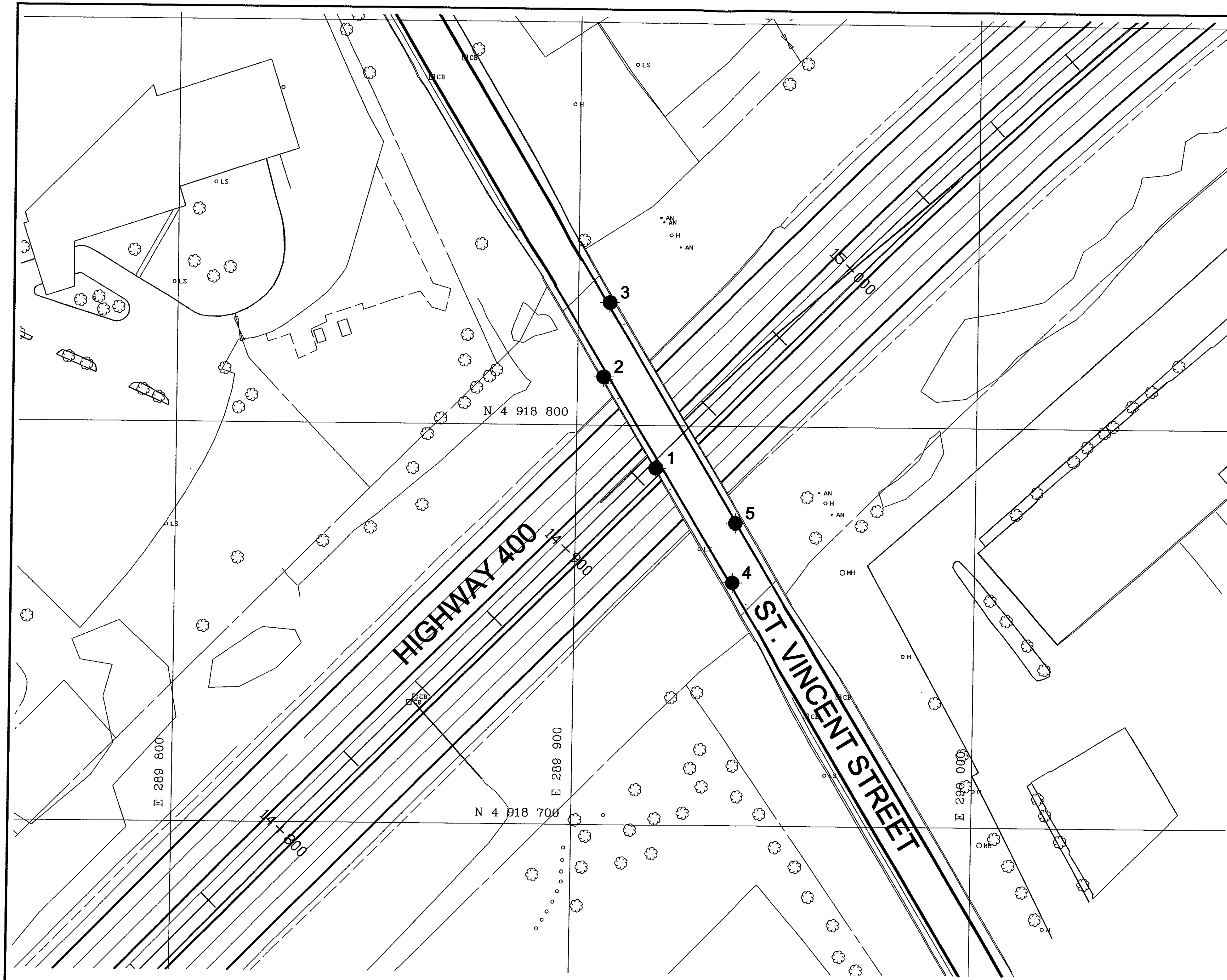
DIST HWY 400
CONT. No.
GWP No. 30-95-00

ST. VINCENT STREET UNDERPASS
HWY 400
BOREHOLE LOCATION PLAN

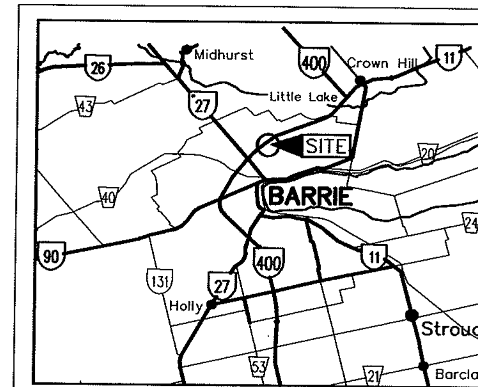
SHEET

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



0 10 20 30 40 50 100m
SCALE IN METRES



KEY PLAN
0 2.0 4.0 6.0 8.0 10.0 20.0
SCALE IN KILOMETRES

LEGEND

Borehole, previous investigation
Borehole, present investigation

No.	ELEVATION	LOCATION NORTHING	LOCATION EASTING
1	261.3	4,918,789	289,920
2	261.2	4,918,812	289,907
3	261.5	4,918,831	289,908
4	260.5	4,918,761	289,940
5	261.0	4,918,776	289,940

REFERENCE
This drawing was created from digital file "50212.dwg" provided by URS Cole Sherman

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

NO.	DATE	BY	REVISION

Geocres No.

HWY. No. 400		PROJECT NO.: 001-1143F	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-394
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

P1143F20.DWG

APPENDIX A

RECORDS OF BOREHOLES AND TEST RESULTS
1964 INVESTIGATION

LIST OF ABBREVIATIONS

The abbreviations commonly employed on each "Record of Borehole," on the figures and in the text of the report, are as follows:

I. SAMPLE TYPES

AS	auger sample
CS	chunk sample
DO	drive open
DS	Denison type sample
FS	foil sample
RC	rock core
ST	slotted tube
TO	thin-walled, open
TP	thin-walled, piston
WS	wash sample

II. PENETRATION RESISTANCES

Dynamic Penetration Resistance: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch diameter, 60 degree cone one foot, where the cone is attached to 'A' size drill rods and casing is not used.

Standard Penetration Resistance, *N*: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch drive open sampler one foot.

WH sampler advanced by static weight—weight, hammer

PH sampler advanced by pressure—pressure, hydraulic

PM sampler advanced by pressure—pressure, manual

III. SOIL DESCRIPTION

(a) *Cohesionless Soils*

Relative Density	*N, blows/ft.*
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) *Cohesive Soils*

Consistency	c_u, *lb./sq. ft.*
Very soft	Less than 250
Soft	250 to 500
Firm	500 to 1,000
Stiff	1,000 to 2,000
Very stiff	2,000 to 4,000
Hard	over 4,000

IV. SOIL TESTS

C	consolidation test
H	hydrometer analysis
M	sieve analysis
MH	combined analysis, sieve and hydrometer[1]
Q	undrained triaxial[2]
R	consolidated undrained triaxial[2]
S	drained triaxial
U	unconfined compression
V	field vane test

NOTES:

[1]Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.

[2]Undrained triaxial tests in which pore pressures are measured are shown as $\bar{Q}$ or $\bar{R}$.

LIST OF SYMBOLS

I. GENERAL

$\pi = 3.1416$
e = base of natural logarithms 2.7183
$\log_e a$ or $\ln a$, natural logarithm of a
$\log_{10} a$ or $\log a$, logarithm of a to base 10

- t time
- g acceleration due to gravity
- V volume
- W weight
- M moment
- F factor of safety

II. STRESS AND STRAIN

- u pore pressure
- σ normal stress
- σ' normal effective stress ($\bar{\sigma}$ is also used)
- τ shear stress
- ϵ linear strain
- ϵ_{xy} shear strain
- ν Poisson's ratio (μ is also used)
- E modulus of linear deformation (Young's modulus)
- G modulus of shear deformation
- K modulus of compressibility
- η coefficient of viscosity

III. SOIL PROPERTIES

(a) *Unit weight*

- γ unit weight of soil (bulk density)
- γ_s unit weight of solid particles
- γ_w unit weight of water
- γ_d unit dry weight of soil (dry density)
- γ' unit weight of submerged soil
- G_s specific gravity of solid particles $G_s = \gamma_s/\gamma_w$
- e void ratio
- n porosity
- w water content
- S_r degree of saturation

(b) *Consistency*

- w_L liquid limit
- w_P plastic limit
- I_P plasticity index
- w_S shrinkage limit
- I_L liquidity index $= (w - w_P)/I_P$
- I_C consistency index $= (w_L - w)/I_P$
- e_{max} void ratio in loosest state
- e_{min} void ratio in densest state
- D_r relative density $= (e_{max} - e)/(e_{max} - e_{min})$

(c) *Permeability*

- h hydraulic head or potential
- q rate of discharge
- v velocity of flow
- i hydraulic gradient
- k coefficient of permeability
- j seepage force per unit volume

(d) *Consolidation (one-dimensional)*

- m_v coefficient of volume change $= -\Delta e/(1+e)\Delta\sigma'$
- C_c compression index $= -\Delta e/\Delta \log_{10} \sigma'$
- c_v coefficient of consolidation
- T_v time factor $= c_v t/d^2$ (d, drainage path)
- U degree of consolidation

(e) *Shear strength*

- τ_f shear strength
- c' effective cohesion intercept
- ϕ' effective angle of shearing resistance, or friction

$\left.\right\}$ in terms of effective stress $\tau_f = c' + \sigma' \tan \phi'$

- c_u apparent cohesion*
- ϕ_u apparent angle of shearing resistance, or friction

$\left.\right\}$ in terms of total stress $\tau_f = c_u + \sigma \tan \phi_u$

- μ coefficient of friction
- S_t sensitivity

*For the case of a saturated cohesive soil, $\phi_u = 0$ and the undrained shear strength $\tau_f = c_u$ is taken as half the undrained compressive strength.

GOLDER & ASSOCIATES

PROJECT No. ...63137......

RECORD OF BOREHOLE 1

LOCATION: SEE BORING PLAN

BORING DATE: NOV 4, 1964

DATUM: GEODETIC

BOREHOLE TYPE: WASH BORING

BOREHOLE DIAMETER: NX, BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE

ELEVN. / DEPTH	DESCRIPTION
857.2 (261.3m)	GROUND LEVEL
0.5	ASPHALT. COMPACT BROWN SILTY SAND WITH GRAVEL AND COBBLES (FILL)
855.2 / 2.0 (260.7m)	DENSE TO VERY DENSE LIGHT BROWN SILTY SAND WITH GRAVEL, TRACE TO SOME CLAY (SILT CONTENT INCREASES AND GRAVEL CONTENT DECREASES WITH DEPTH) (SANDY TILL)
821.7 / 35.5 (250.5m)	END OF HOLE (10.8m depth)

SAMPLES

NUMBER	TYPE	BLOWS/FT.
1	2" D.O.	46
2	"	>100
3	"	55
4	"	89
5	"	49
6	"	45
7	"	87
8	"	>100
9	"	>100
10	"	>100

ELEVATION SCALE: 860, 855, 850, 845, 840, 835, 830, 825, 820

DYNAMIC PENETRATION RESISTANCE BLOWS/FT. (20, 40, 60, 80, 100)

SHEAR STRENGTH C_u, LB./SQ.FT.

>100, >100, >100, >100, >100, >100

END OF PEN. TEST @ ELEV. 836.2

COEFFICIENT OF PERMEABILITY K, CM./SEC.

WATER CONTENT, PERCENT — W_p, W, W_L (10, 20, 30, 40)

ADDITIONAL LAB. TESTING: MH, γ=137; MH, γ=145

PIEZOMETER OR STANDPIPE INSTALLATION: GROUND LEVEL; CEMENT SEAL; GRAVEL FILL; BENTONITE SEAL; PLASTIC TUBING; PIEZOMETER

W.L. IN PIEZOMETER AT ELEV. 835.8 NOVEMBER 20/64 (El. 254.0m)

Percent axial strain at failure

VERTICAL SCALE 1 INCH TO 5'-0"

GOLDER & ASSOCIATES

DRAWN ....R.H....

CHECKED ....

PROJECT No. [illegible]

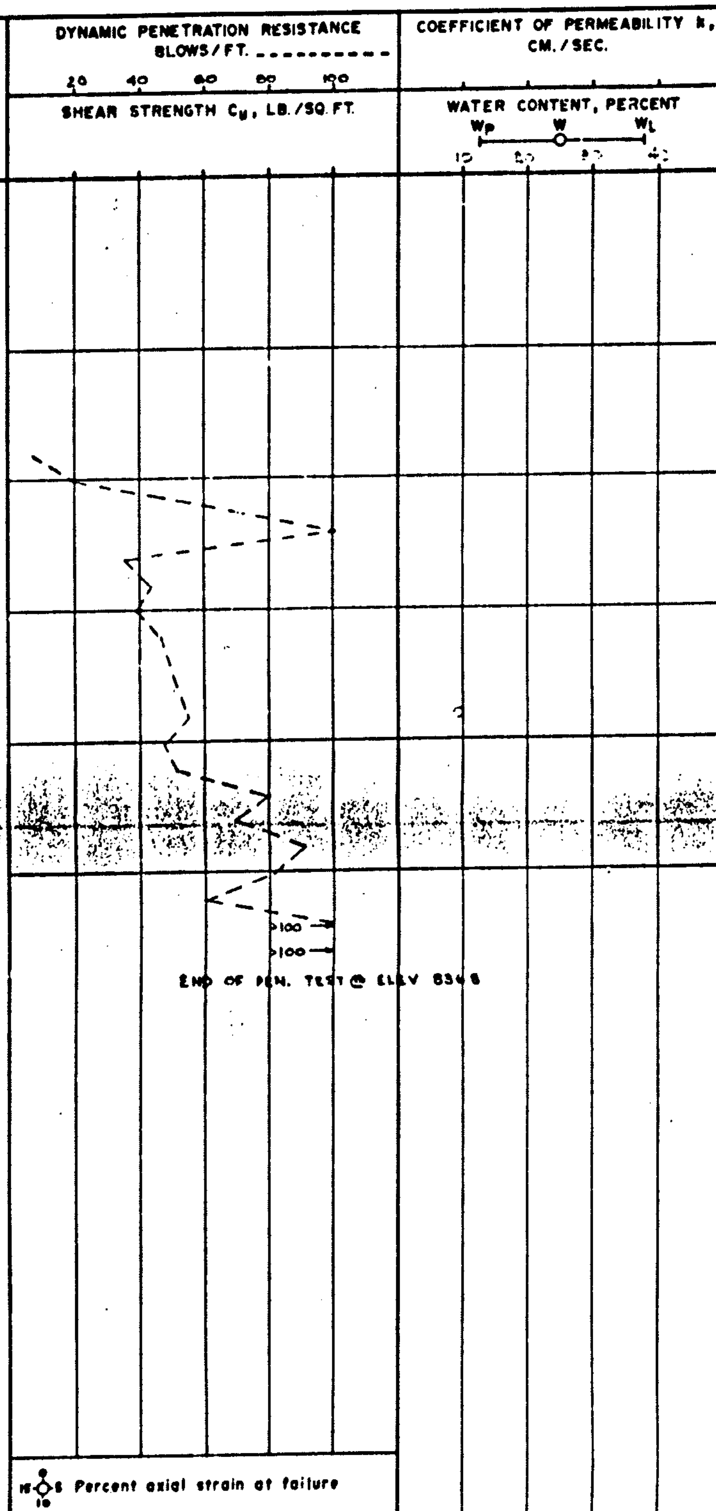
RECORD OF BOREHOLE 2

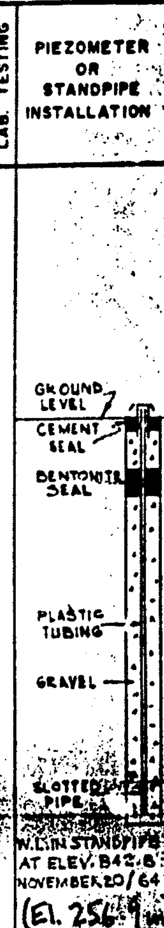
LOCATION SEE [illegible] PLAN BORING DATE NOV. 5, 1964 DATUM GEODETIC

BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER NX BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. DEPTH	SOIL PROFILE DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/FT.	ELEVATION SCALE	ADDITIONAL LAB. TESTING
						860	
856.8	261.2m GROUND LEVEL						
0.0 / 0.4	TOPSOIL 261.0m						
						855	
			1	2" SS	62		
	COMPACT TO DENSE LIGHT BROWN SILTY SAND WITH TRACE TO SOME GRAVEL AND CLAY (SANDY TILL)		2	"	26		MH
			3	"	24	850	
			4	"	23		
			5	"	27		MH
						845	
841.3	256.4m		6	"	46		
15.5	END OF HOLE (4.7m depth)					840	





VERTICAL SCALE 1 INCH TO 5'-0"

GOLDER & ASSOCIATES

DRAWN ... [illegible]

CHECKED ... [illegible]

PROJECT No. [illegible]

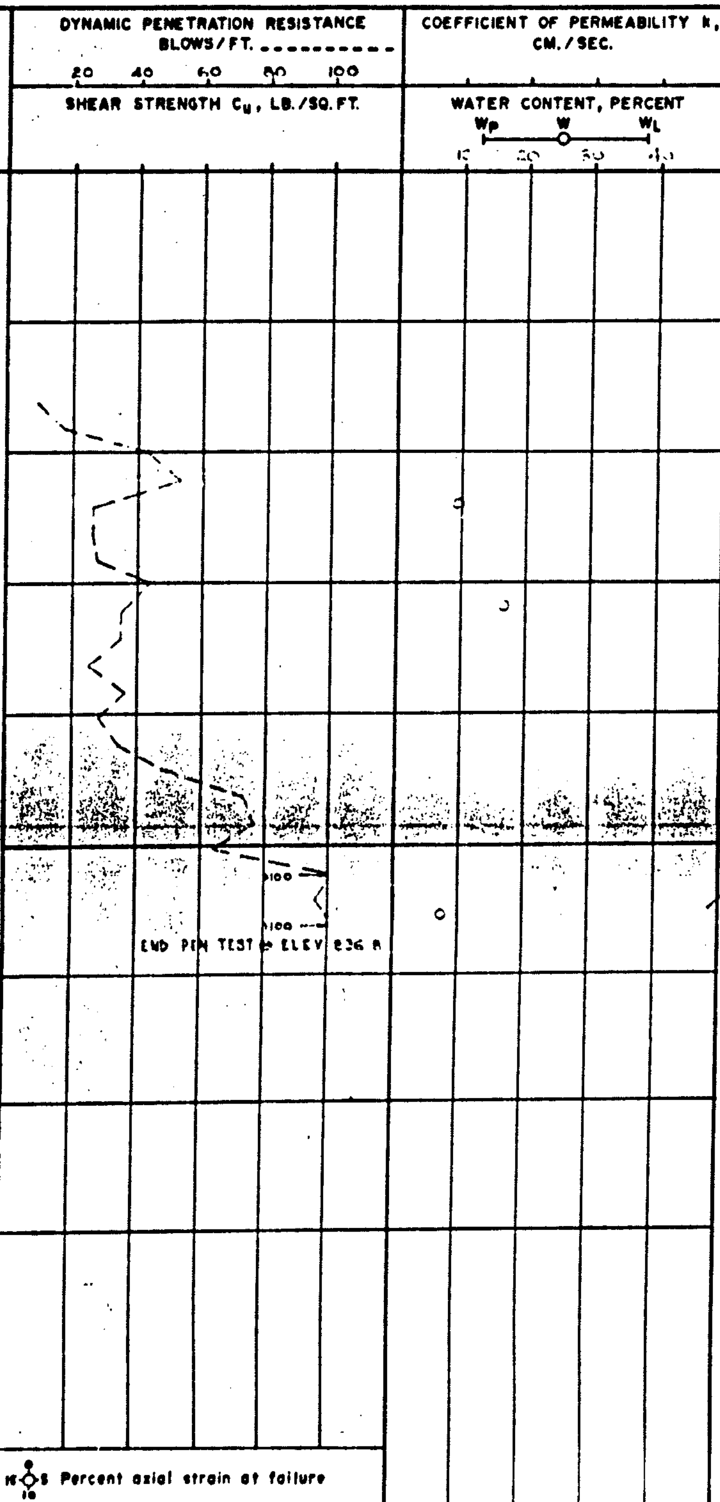
RECORD OF BOREHOLE 3

LOCATION SEE BORING PLAN — BORING DATE NOV 6, 1964 — DATUM GEODETIC

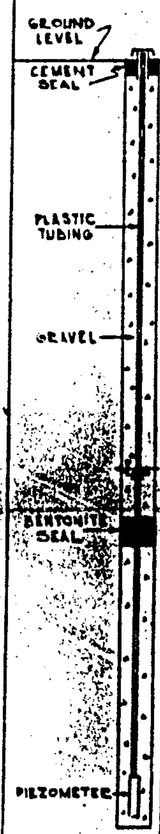
BOREHOLE TYPE WASH BORING — BOREHOLE DIAMETER NX, BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES — PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	BLOWS / FT.	ELEVATION SCALE	ADDITIONAL LAB. TESTING
					860	
857.8	261.5m GROUND LEVEL					
0.0	COMPACT LIGHT BROWN, SILTY SAND WITH GRAVEL NUMEROUS SMALL LUMPS OF GREY SILTY CLAY (FILL)	1	2" DO	15	855	
853.8 4.0	260.2m	2	.	49		MH
		3	.	54	850	
		4	.	35		MH
	DENSE TO VERY DENSE LIGHT BROWN SILTY SAND TRACE TO SOME GRAVEL AND CLAY (SANDY TILL) LAYERS UP TO SEVERAL FEET IN THICKNESS OF VERY STIFF TO HARD SILTY CLAY WITH TRACE TO SOME SAND AND TRACE OF GRAVEL AT ABOUT EL. 849 AND 840.	5	.	34	845	
		6	.	46		
		7	CA SA	—	840	
		8	.	>100		MH γ=150
					835	
		9	.	>100	830	
828.3 29.5	252.5m END OF HOLE (9.0m depth)	10	.	>100	825	



PIEZOMETER OR STANDPIPE INSTALLATION



W.L. IN PIEZOMETER AT ELEV. 841.8 NOVEMBER 20/64.
(El. 256.6m)

VERTICAL SCALE
1 INCH TO 5'0"

GOLDER & ASSOCIATES

DRAWN ....RU.....
CHECKED ....[illegible].....

PROJECT No. ...415...

RECORD OF BOREHOLE 4

LOCATION SEE BORING PLAN   BORING DATE NOV 9, 1964   DATUM GEODETIC

BOREHOLE TYPE WASH BORING   BOREHOLE DIAMETER NX, BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES   PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE: ELEVN. / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	TYPE	BLOWS/FT.	ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT (20 40 60 80 100) / SHEAR STRENGTH C_u, LB./SQ.FT.	COEFFICIENT OF PERMEABILITY k, CM./SEC. / WATER CONTENT, PERCENT W_p W W_L	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
										CAP
854.5	GROUND LEVEL 260.5m					855				GROUND LEVEL
0.0 / 0.5	SAND AND GRAVEL (FILL) 260.3m									SURFACE SEAL
	COMPACT TO DENSE (VERY DENSE BELOW EL. 841) LIGHT BROWN SILTY SAND WITH TRACE TO SOME GRAVEL AND CLAY (SANDY TILL)		1	2" O.D.	18					
			2	"	57	850			MH	
			3	"	26					BENTONITE SEAL
			4	"	19	845				
			5	"	37				MH	GRAVEL
							>100			
						840	>100			
			6	"	>100		END OF PEN. TEST @ ELEV. 839.5			
			7	"	>100	835			MH	STANDPIPE
830.5 / 24.0	253.1m END OF HOLE (7.3m depth)					830				W.L. IN STANDPIPE BELOW ELEV. 835.5 NOVEMBER 20/64. (COULD NOT LOWER WATER LEVEL FINDER PAST ELEV 835.5)
						825				

DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT.

15 ◇ 5 Percent axial strain at failure
10

VERTICAL SCALE 1 INCH TO 5'-0"

GOLDER & ASSOCIATES

DRAWN ....

CHECKED ....

PROJECT No. 64127

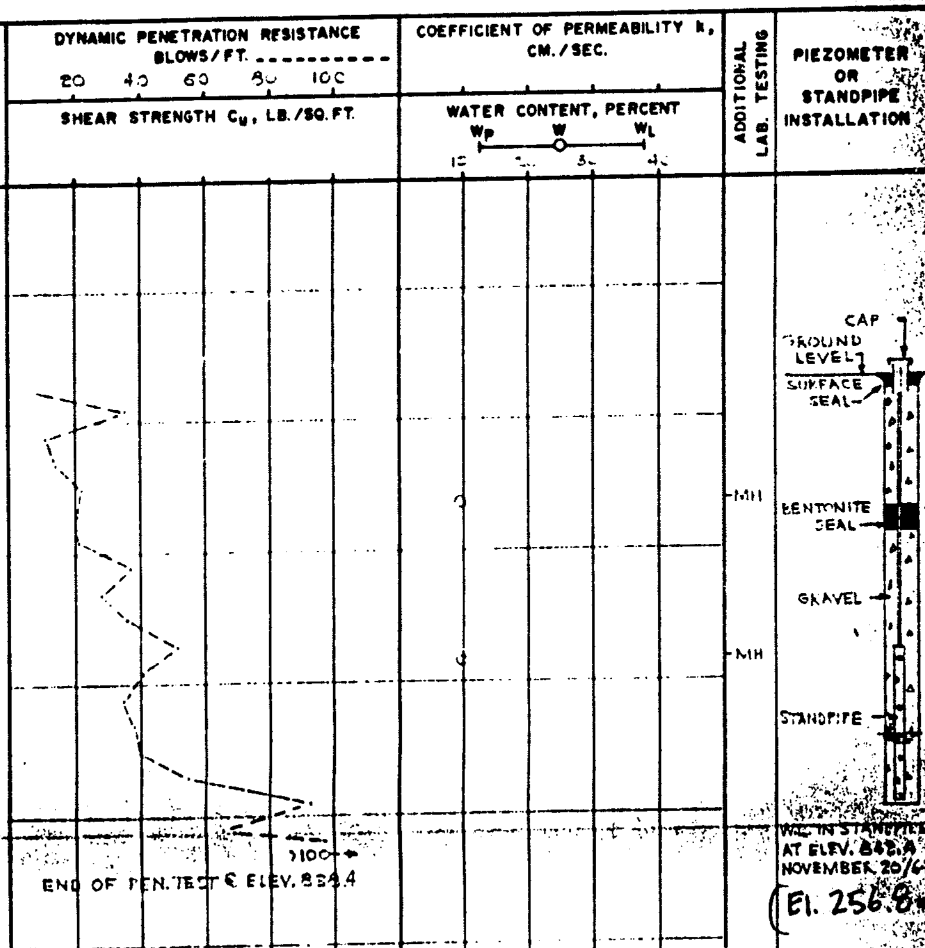
RECORD OF BOREHOLE 5

LOCATION SEE BORING PLAN BORING DATE NOV. 10, 1964 DATUM GEODETIC

BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER NX, BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ELEVN. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FT.	ELEVATION SCALE	ADDITIONAL LAB. TESTING
					860	
856.4	261.0m GROUND LEVEL					
0.5	SAND AND GRAVEL (FILL) 260.9m				855	
	COMPACT TO DENSE LIGHT BROWN SILTY SAND WITH SOME GRAVEL TRACE TO SOME CLAY (SANDY TILL)	1	2" O.D.	46		
		2	"	25		MH
		3	"	26	850	
		4	"	24		
		5	"	21	845	MH
839.9	256.0m	6	"	30	840	
16.5	END OF HOLE (5.0m depth)					
					835	



Percent axial strain at failure

VERTICAL SCALE
1 INCH TO 5'-0"

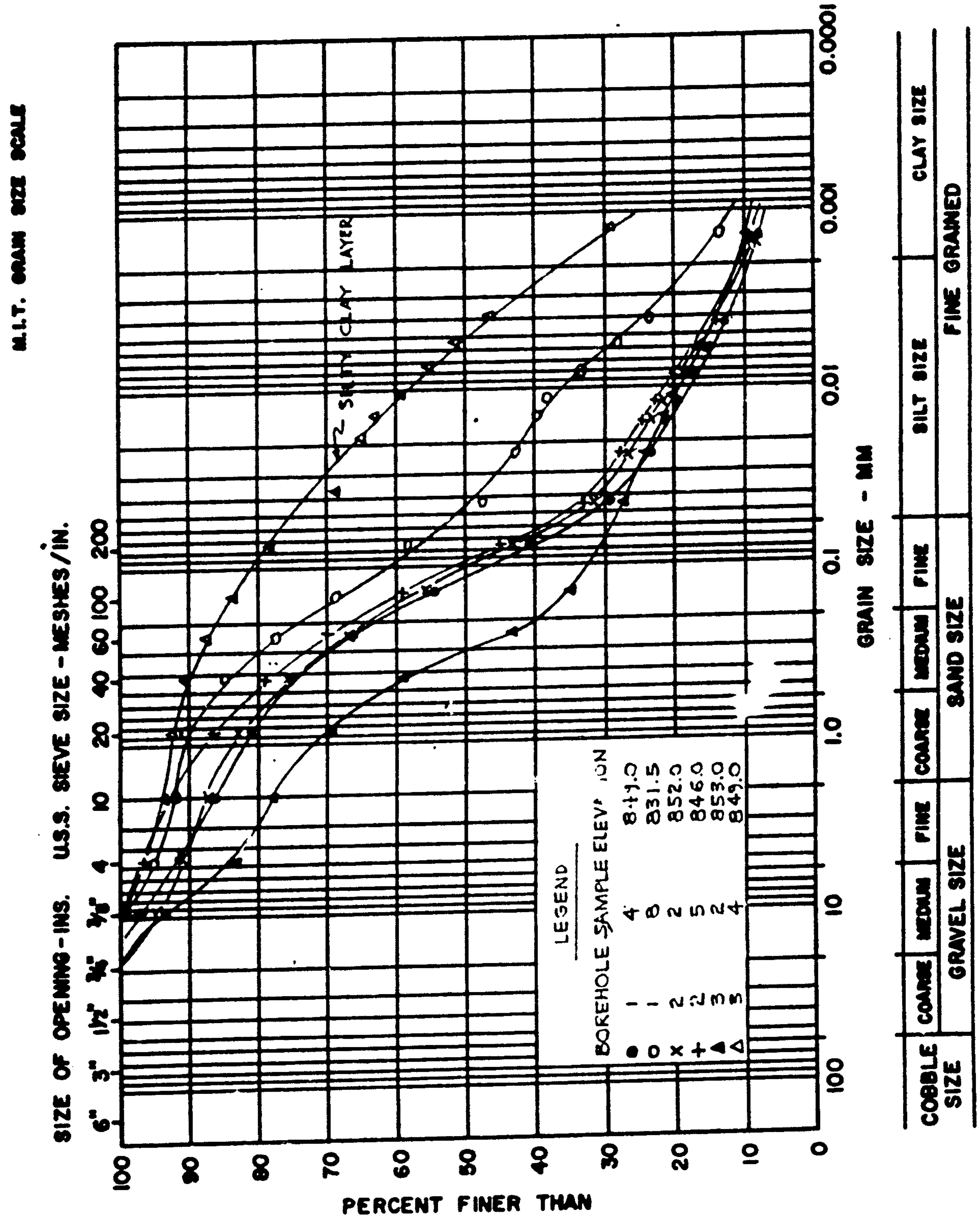
GOLDER & ASSOCIATES

DRAWN ....
CHECKED ....

GRAIN SIZE DISTRIBUTION

TILL

FIGURE 2



GOLDER & ASSOCIATES

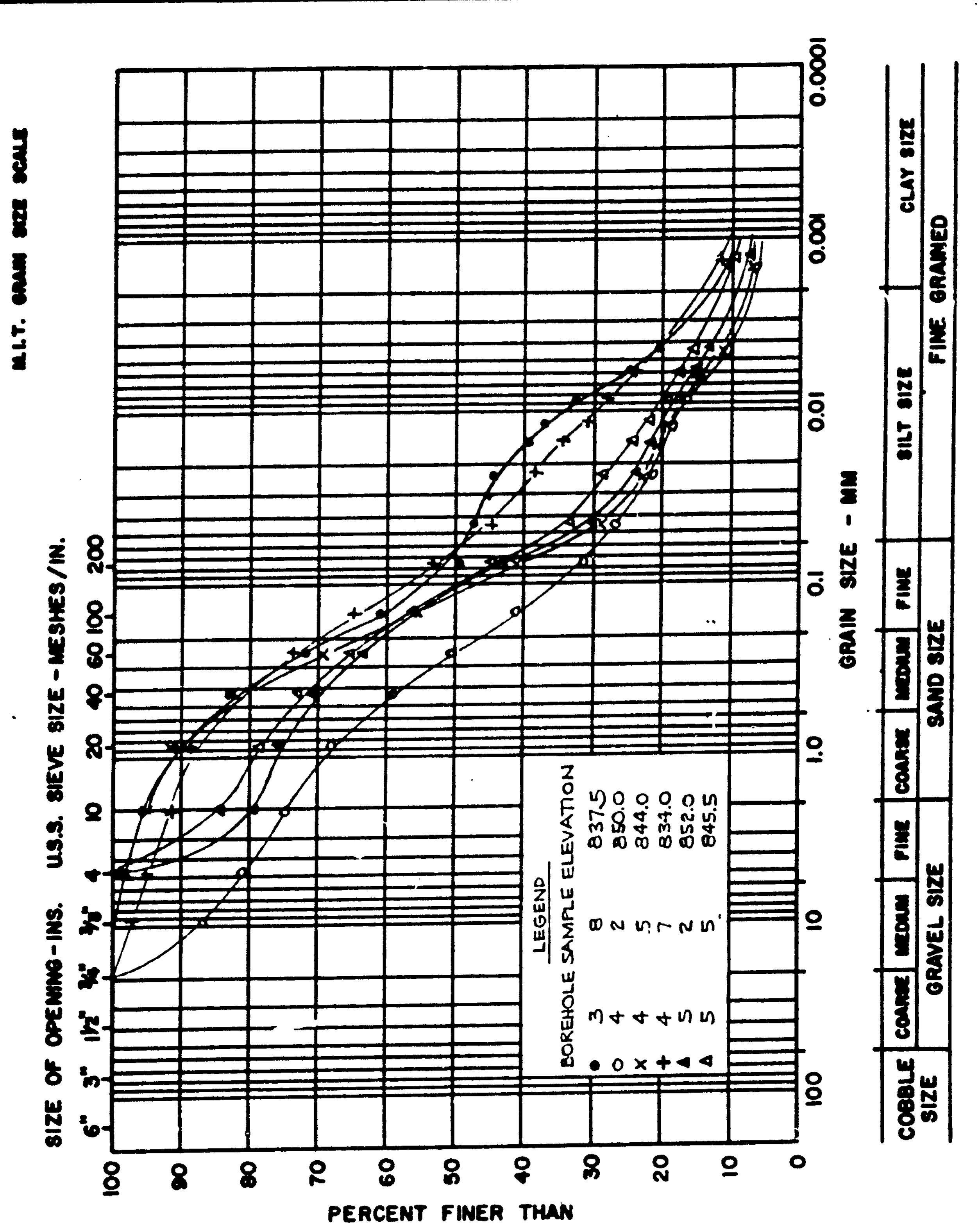
PROJECT No. 64157

PROJECT No. 64157

GRAIN SIZE DISTRIBUTION

TILL

FIGURE 3

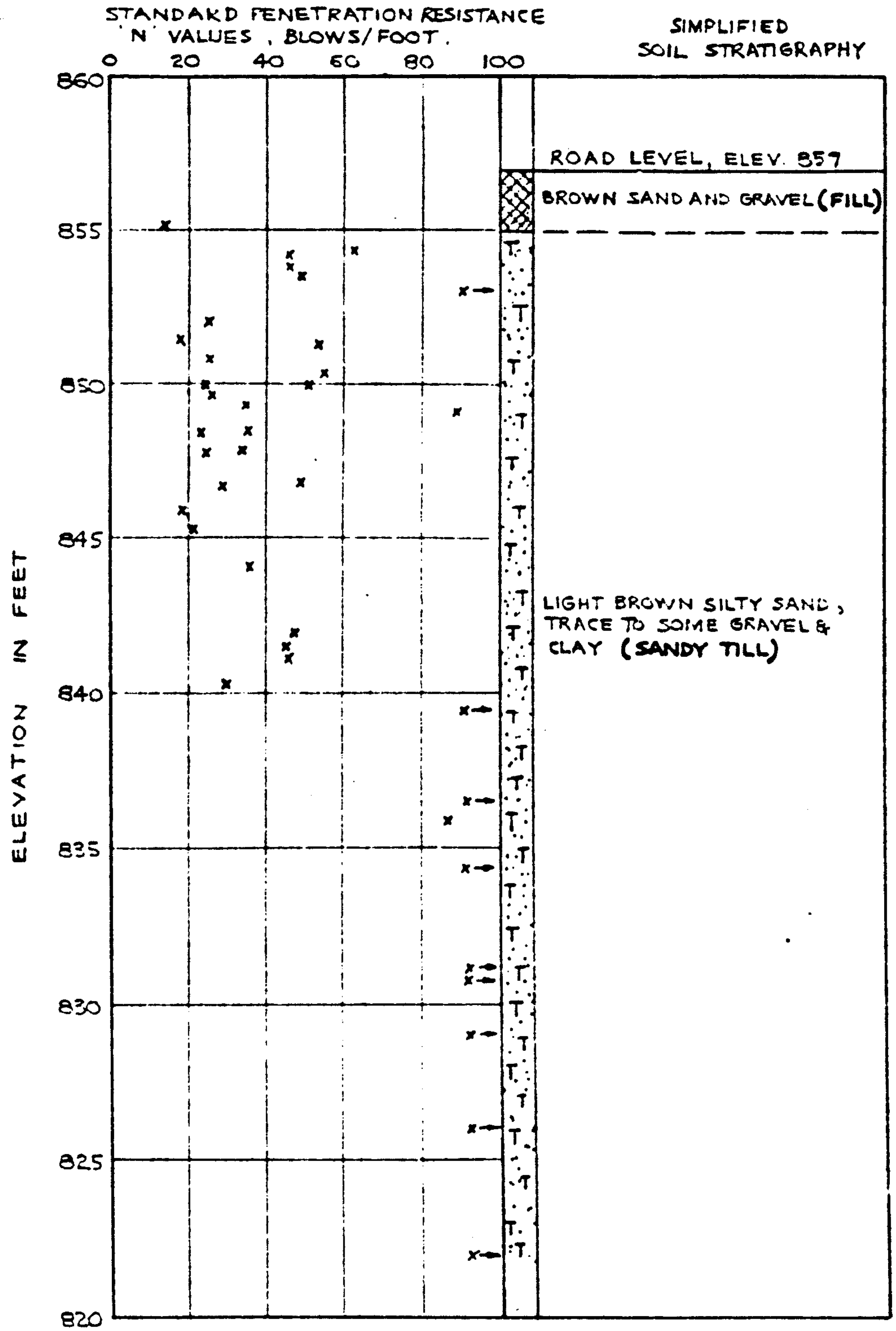


GOLDER & ASSOCIATES

PROJECT No. 64137

SUMMARY PLOT OF STANDARD PENETRATION RESISTANCE VALUES VS ELEVATION

FIGURE 4



GOLDER & ASSOCIATES

Made
Chkd.
Appd.