



DESKTOP REPORT

Desktop Foundation Investigation and Design Report Lancaster Commercial Vehicle Inspection Facility 2.5 km West of County Road 2/34; Highway 401 Township of South Glengarry, Ontario

G.W.P. No. 4045-10-01

P.O. No. 4010-E-0034

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Table of Contents

PART A – DESKTOP FOUNDATION INVESTIGATION REPORT

1.0	INTRODUCTION	1
2.0	SITE DESCRIPTION	1
3.0	SITE STRATIGRAPHY	1
3.1	Overview - Available Information	1
3.2	Regional Geology.....	2
3.3	Site Stratigraphy Overview.....	3
3.4	Surficial Geology	3
3.4.1	Silty Clay	3
3.4.2	Clayey Silt	3
3.4.3	Glacial Till.....	3
3.5	Bedrock Geology.....	4
3.6	Groundwater.....	4
4.0	CLOSURE	4

PART B – DESKTOP FOUNDATION DESIGN REPORT

5.0	ENGINEERING RECOMMENDATIONS.....	5
5.1	General.....	5
5.2	Subsurface Conditions Assessment	5
5.3	Foundations.....	5
5.3.1	Shallow Foundations.....	6
5.3.2	Deep Foundations.....	6
5.3.3	Resistance to Lateral Loads	7
5.3.4	Grade Raise Design Alternatives	7
5.3.5	Site Coefficient.....	8
5.3.6	Slab-on-Grade.....	9
5.3.7	Excavations.....	9
5.3.8	Groundwater Control.....	9
5.4	Desktop Soil Liquefaction Assessment.....	9

5.4.1	General	9
5.4.2	Desktop Preliminary Liquefaction Potential	10
5.5	Additional Investigation Works	10
6.0	CLOSURE	11

FIGURES

Drawing 1 – Lancaster CVIF Highway 401 Site Location
Figure 2 – Physiographic Regions of Southern Ontario Mapping
Figure 3 – OGS Surficial Geology Mapping
Figure 4 – OGS Bedrock Geology Mapping
Figure 5 – OGS Drift Thickness Mapping

APPENDICES

APPENDIX A

GEOCRES Report for the Highway 401 Underpass at Fraser Road Site No. 31-230
GEOCRES Report for the Highway 401 Overpass at Raisin River Site No. 31-231/1&2
GEOCRES Report for the Highway 401 Underpass at County Road 2/34.

PART A

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Dillon Consulting Limited (Dillon) on behalf of the Ministry of Transportation, Ontario (MTO), Eastern Region, to carry out a desktop study as part of the preliminary design stage assignment for the design and construction of the proposed Highway 401 westbound Commercial Vehicle Inspection Facility (CVIF) located southwest of the town of Lancaster, Ontario.

The purpose of the desktop study was to access the subsurface conditions at the location of the proposed Lancaster CVIF.

The scope of work for this report was outlined in Golder's proposal dated December 2018.

2.0 SITE DESCRIPTION

The new westbound CVIF is to be located just north of Highway 401 between Fraser Road and County Road 2/34 in the Township of South Glengarry, Ontario. The proposed location is approximately 2.5 km west of the Highway 401 / County Road 2/34 Interchange. The location of the proposed CVIF is shown on the Key Plan on Drawing No. 1.

The lands surrounding the project limits are typically agricultural with a flat topography. Raisin River Road currently crosses the proposed site in a west-east direction parallel to and north of Highway 401. Brush and trees are also present between the Highway 401 right-of-way and Raisin River Road.

Highway 401 at this location has two through lanes in each direction with paved and gravel shoulders. The eastbound and westbound lanes are generally separated by a wide, vegetated median ditch. Storm water drainage in the area is to existing ditches, culverts and Raisin River.

The site plan drawings provided indicate that the new CVIF will contain driving lanes and parking, a triage area, a static scale, covered and noncovered inspection bays, service bays, a building, and septic system structure and septic field area. Drawing No. 1 illustrates the general layout of the proposed CVIF. It is understood that, due to hydrology requirements, a grade raise of between 1.5 and 3.5 m will be required across the proposed CVIF site.

3.0 SITE STRATIGRAPHY

3.1 Overview - Available Information

The site soil and bedrock model developed for this desktop study was based on from historical data presented in the sources indicated below. No field work was carried out as part of the current study and investigations have not previously been completed at the subject site.

The subsurface information used in the preparation of this report was obtained from historical data provided in the previous Foundation Investigation Reports available from the MTO GEOCRE database and more recent foundation investigation reports prepared for MTO by Golder, as described below. The previous reports reviewed for this Desktop Study are for the investigations carried out for the Highway 401 Underpasses at Fraser Road and County Road 2/34, and the Highway 401 Overpass at Raisin River (all within 2 km east/west of the location of the proposed Lancaster CVIF).

GEOCRES Reports:

- Foundation Investigation Report titled “*Proposed Fraser Road Underpass Highway 401*” dated January 1966 (GEOCRES Reference 31G00-142).
- Foundation Investigation Report titled “*Highway 401 Raisin River Bridge*” dated February 1958, (GEOCRES Reference 31G00-143).
- Foundation Investigation Report titled “*Proposed Crossing Highway No. 401, and Highway No. 2; 1-½ Miles South of Lancaster Township of Charlottenburg, District No. 9, Bridge No. 11*” dated July 1960, (GEOCRES Reference 31G00-144).

Golder Associate Ltd. Reports:

- Foundation Investigation Report titled “*Raisin River Bridge Replacements Structure Sites No. 31-231/1&2*” dated December 2018, (GEOCRES Reference 31G-261).
- Foundation Investigation Report titled “*Replacement of Fraser Road Underpass Highway 401, Site No. 31-230*” dated December 2018, (GEOCRES Reference 31G5-273).

Further descriptions of the existing drift thickness and soil and bedrock geologies at the site were derived from the following sources.

- Chapman, L. J. and Putnam, D. F., 1984. *The Physiography of Southern Ontario*, Ontario Geological Survey. Special Volume 2, Third Edition. Accompanied by Map P. 2715, Scale 1:600,000. Ontario Ministry of Natural Resources and Chapman, L.J. and Putnam, D.F. 2007 and Physiography of Southern Ontario; Ontario Geological Survey, Miscellaneous Release-Data 228.
- Ontario Geological Survey 2010. *Surficial Geology of Southern Ontario*; Ontario Geological Survey, Miscellaneous Release – Data 128-REV.
- Ontario Geological Survey 2011. *1:250 000 Scale Bedrock Geology of Ontario*; Ontario Geological Survey, Miscellaneous Release – Data 126 – Revision 1.
- Bedrock Topography and Overburden Thickness Mapping, Southern Ontario, Ontario Geological Survey, Miscellaneous Release – Data 207.

3.2 Regional Geology

As delineated in The Physiography of Southern Ontario, the proposed CVIF location lies within the minor physiographic clay plain region known as the Lancaster Flats, which lies within the major physiographic region of the Ottawa-St. Lawrence Lowland. Figure No. 2 illustrates the physiographic regions in the area surrounding the proposed CVIF site.

The Lancaster Flats lies in a lowland in which the till plain has been buried under water-laid deposits leaving exposed only the stony crests of a few drumlins. The subsoil consists of poorly drained materials ranging from clay to very fine sand. The clay in this area is known as the Champlain Sea clay or Leda clay, that overlies relatively thin, commonly reworked glacial till and glaciofluvial deposits, underlain by bedrock.

The underlying bedrock is mapped as limestone, dolostone, shale, arkose and sandstone of the Ottawa and Simcoe Groups of the Shadow Lake Formation.

The site falls within the Western Québec (WQ) seismic zone according to the Geological Survey of Canada. The WQ zone constitutes a large area which encompasses the urban areas of Montreal, Ottawa-Hull and Cornwall. Within the WQ zone recent seismic activity has been concentrated in two subzones; one along the Ottawa River and another more active subzone along the Montreal-Maniwaki axis. The two major earthquakes in the WQ zone includes the 1935 Témiscaming event of magnitude 6.2 (i.e., a measure of the intensity of the earthquake, MbLg or MN) and the 1944 Cornwall-Massena event which had a magnitude of 5.6.

3.3 Site Stratigraphy Overview

Based on existing geological mapping and the results of previous investigation reports in the area, the subsurface conditions at the site are anticipated to generally consist of surficial deposits, overlying firm silty clay, overlying loose to compact clayey silt overlying non-cohesive, compact to very dense glacial till, in turn underlain by limestone/shale bedrock.

Groundwater levels were measured at depths at or just below existing ground surface during the previous investigations in the surrounding area.

3.4 Surficial Geology

Figure No. 3 illustrates the surficial geology mapping in the area surrounding the proposed CVIF site.

3.4.1 Silty Clay

Based on the existing geological mapping and previous investigation reports, sensitive marine clay likely extends from ground surface, or the underside of any surficial or sand deposits. Based on the drift thickness mapping (see Figure 5) and the previous investigations in the area, the clay may extend to depths ranging from about 10 to 12 metres below existing ground surface. The upper portion of the deposit, to a limited depth, may be weathered and relatively stiff. Below the depth of weathering the clay is indicated to be grey and unweathered.

The results of Atterberg Limits testing on samples from the nearby investigations indicate the unweathered clay is highly plastic. The undrained shear strength, based on both in-situ and laboratory testing from the original investigations at Fraser Road, may range from about 60 to 25 kPa; corresponding to a stiff to soft clay. The measured shear strength was noted to decrease with depth and the deposit is likely more generally firm to soft. The results of oedometer testing carried out on samples of the silty clay indicate that the pre-consolidation pressure is in the range of 125 to 75 kPa and the clay is indicated to be slightly over consolidated near surface; becoming normally consolidated with depth.

3.4.2 Clayey Silt

The silty clay layer may be underlain by a thin, noncontinuous layer of clayey silt. This layer was encountered during previous investigations in the area had a thickness of between 1.5 and 3 m. Based on the undrained shear strength measured in-situ and the standard penetration test (SPT) results, the clayey silt has a consistency ranging from firm to very stiff, but typically stiff.

3.4.3 Glacial Till

A stratum of glacial till consisting predominantly of sand with silt and gravel underlies the clay and silt strata. The till layer is indicated to have a thickness ranging from 3 to 5 m. Some cobbles and boulders were noted near the base of the till layer. Thin deposits of sand and gravel were also noted within the glacial till layer. SPT results indicate that the glacial till has a compact to very dense state of packing.

3.5 Bedrock Geology

Bedrock geology mapping indicates that the overburden materials at the site are underlain by limestone and shale bedrock of the of the Ottawa and Simcoe Groups of the Shadow Lake Formation. Figure No. 4 illustrates the general bedrock geology in the area of proposed Lancaster CVIF site.

Based on bedrock coring during the previous investigations in the area surrounding the proposed CVIF site, the bedrock ranged from limestone with shale partings to shale. A review of the unconfined compressive strength tests from the various report indicates the bedrock is classified as strong to very strong.

Drift thickness mapping indicates that the depth to bedrock in the area of proposed Lancaster CVIF site ranges from 10 to 15 m (see Figure No. 5).

3.6 Groundwater

Based on the monitoring carried out during the historical investigations, the groundwater is at or near the existing ground surface. It should be noted that slight artesian conditions were encountered within the glacial till layer. Groundwater was measured in the 2018 investigations for both the Fraser Road and Raisin River structures at an approximate elevation of 47.0 m.

The water level at the site is expected to fluctuate seasonally in response to changes in precipitation and snow melt and is expected to be higher during the spring and periods of precipitation.

4.0 CLOSURE

This Desktop Foundation Report was prepared by Mr. Kenton Power, P.Eng. and was reviewed by William Cavers, P.Eng. and Mr. Fintan Heffernan, P.Eng., the Designated MTO Foundations Contact for Golder for this assignment.

Yours truly,

Golder Associates Ltd.



Kenton Power, P.Eng.
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PART B

Desktop Foundation Design
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5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides desktop foundation design recommendations based on our interpretation of the factual information obtained during the desktop study for the proposed Lancaster CVIF site.

The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives for the functional design of the facilities and the structures and canopies indicated on the preliminary site plans provided by Dillon.

The subsurface information used in the preparation of this report was obtained from historical data provided in the previous Foundation Investigation Reports available from the MTO GEOCRES database and more recent foundation investigation reports prepared for MTO by Golder and in accordance with the Canadian Highway Bridge Design Code, version CSA S6-14 (CHBDC). The previous reports reviewed for this Desktop Study are for the investigations carried out for the Highway 401 Underpasses at Fraser Road and County Road 2/34, and the Highway 401 Overpass at Raisin River. (All within 2 km east/west of the location of the proposed Lancaster CVIF).

The desktop foundation report, discussion, and recommendations are intended for the use of the Ministry of Transportation, Ontario (MTO) and shall not be used or relied upon for any other purpose or by any other parties, including construction contractor. The contractor must make their own interpretation based on the factual data in Part A (Foundation Investigation) of the report. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project. Those requiring information on the aspects of construction must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

5.2 Subsurface Conditions Assessment

Based on the results presented in the available GEOCRES reports and the published geological mapping reviewed as part of this Desktop Study, the site is predominantly underlain by a deposit of clay that is generally normally consolidated with depth and is considered to be compressible. Long term settlements should be expected if grade raises are to be constructed across the site; it is understood that due to hydrology requirements a grade raise of between 1.5 and 3.5 m will be required across the proposed CVIF site. Where a grade raise is to be incorporated into the detailed design, further evaluation of the settlement at the site will be required, including its effect on the proposed septic facilities, determining the consolidation parameters of the clay and the development of settlement mitigation measures.

Preliminary discussion with regards to a grade raise at the Lancaster CVIF site from a geotechnical perspective are provided in the following sections.

5.3 Foundations

The site plan drawings provided indicate that the proposed CVIF will contain driving lanes and parking, a triage area, a static scale, covered and noncovered inspection bays, service bays, a building, and septic system structure and septic field area. Drawing No. 1 illustrates the general layout of the proposed Lancaster CVIF.

5.3.1 Shallow Foundations

Based on the subsurface review as part of this desktop study, lightly loaded structures may potentially be founded on conventional spread and/or strip footings bearing on the native clay soils, below any topsoil or fill materials that may be present. However, the feasibility of shallow foundations will depend on the height of grade raise and the mitigation strategies undertaken to address the potential settlements due to the proposed grade raises at the site.

If preloading and surcharging are undertaken on the site (as also discussed further in the sections below), it may be feasible to account for the shallow foundation loading in the surcharge design. However, as the height of the grade raise increases, this approach may not be feasible since the surcharge heights (including foundation loads) may not be feasible due to stability concerns. The risk of poor foundation performance also increases as the height of the grade raise, and associated preload/surcharge fill height increases, due to the potentially very high stresses (exceeding the deposit's preconsolidation pressures) imposed on the compressible clay.

The bearing resistances of shallow foundations placed on the native clay soils will need to be assessed during preliminary design based on an analysis for preloading and surcharging of the site.

Alternatively, shallow foundations may be used for support of lightly loaded structures by providing EPS fill around/within the footprint of the structures. Conceptually the EPS fill would need to extend at least the height of the fill from the foundation walls, across the full width/length of the building and the full depth of fill would need to consist of EPS. The foundations would also need to be placed on top of the EPS fill or would need to extend to the native soil surface.

For functional design, and assuming the use of lightweight fill within the building footprints, the following bearing resistances may be used:

- Ultimate Limit States (ULS) of 100 kPa.
- Serviceability Limit States (SLS) of 50 kPa.

The SLS resistance is based on a maximum of 25 mm of total settlement based on footings up to 2.5 m wide. For these shallow foundations, differential settlement magnitudes of less than 15 mm are expected, provided that proper subgrade preparation is carried out (discussed further in Section 5.6).

Other ground improvement techniques, such as deep soil mixing or rigid inclusions may also provide support for shallow foundations, but the applicability and foundation design values would need to be assessed based on the method chosen (see discussion in Section 5.3.4, below).

Footings should be provided with a minimum of 1.7 m of earth cover (i.e., be 1.7 m below the lowest surrounding grade) to provide adequate protection against frost penetration, per Ontario Provincial Standard Drawing (OPSD) 3090.101 (*Foundation Frost Penetration Depths for Southern Ontario*). The use of rigid insulation (Styrofoam) could be considered as an alternative to, and/or used in conjunction with, earth cover for frost protection purposes (if the foundations are supported on EPS fill, the frost depth requirement may be waived).

Finished site grading should promote drainage away from the structures.

5.3.2 Deep Foundations

Founding of any overhead signs, canopies, settlement sensitive structures such as scales and other moderately to heavily loaded structures will likely require deep foundations founded on the bedrock. If the preloading and surcharging analysis indicates that shallow foundations cannot be supported on the improved ground and that the risks of poor foundation performance are too high or if EPS fill for shallow foundations is not considered acceptable, then lightly loaded structures may also need to be supported on deep foundations.

For functional design, the use of HP 310x110 steel H-piles driven to the limestone bedrock may be considered where higher bearing capacities than those outlined in Section 5.3.1 are required.

Based on the estimated uniaxial compressive strength of the rock at this site and assuming fair rock quality, for an HP 310 x 110 pile, the axial factored ultimate geotechnical resistance (ULS) may be taken as 3,000 kN for assessing the feasibility of pile foundations. The factored ULS geotechnical resistance may be greater than the structural capacity of the pile, which could govern design and should be checked by the structural design engineer. The factored serviceability geotechnical resistance (SLS) does not apply to piles founded on the bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

Similar frost protection measures as noted above for shallow foundations would be required for any pile caps.

Alternatively, concrete caisson foundations extending to or into the bedrock could also be considered. The ULS bearing resistance for caissons end bearing on or within the bedrock may be taken as 5 MPa for assessing the feasibility of these deep foundations. Due to the presence of saturated granular layers, temporary or permanent liners would be required for caisson construction through the overburden soils, to minimize ground loss and to provide groundwater control. It may also be necessary to use a rock core barrel to penetrate the glacial till as cobbles and boulders are known to be present in the overburden material just above the bedrock surface. Installation of liners would therefore be difficult and may not be practical at this site.

5.3.3 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between the concrete spread footings and the subsoil should be calculated in accordance with Section 6.7.5 of the CHBDC. Assuming that the founding soils are not loosened/disturbed during excavation and footing construction, the following parameters may be used:

Interface and Loading Condition	Parameter
Concrete – granular pad: short or long term loading	Effective friction angle = 33°
Granular A pad – silty clay subgrade: short term loading	Undrained cohesion = 25 kPa
Granular A pad – silty clay subgrade: long term loading	Effective friction angle = 28°
Granular A pad – silt and sand subgrade: short or long term loading	Effective friction angle = 32°

These values are unfactored; in accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance.

The above values assume that the subgrade materials will not be disturbed by construction activities or groundwater inflow.

5.3.4 Grade Raise Design Alternatives

Based on the available data in the area of the Lancaster CVIF the soft to firm clay layer is considered highly compressible. Long-term settlements with magnitudes significantly in excess of acceptable values, at both structures and paved areas, are likely should a grade raise be constructed at this site. Further site-specific data will be required in order to estimate the magnitude of the settlement, however given the long-term nature of the settlement, mitigation measures will be required at this site in order to construct the proposed CVIF with the indicated grade raise heights.

As it is proposed to have access lanes to Highway 401, inspection bays and highly loaded multi-axial vehicles accessing the site it is considered that periodic re-paving to correct for the settlement is not a feasible option for addressing/mitigating the settlement effects. Subexcavation and replacing of the clay layer with a non-compressible material would also not be feasible due to its thickness. The use of lightweight fill materials such as expanded polystyrene (EPS) is also not considered cost-effective due to the large footprint area to be raised at the site.

The following options may therefore be considered for mitigating the anticipated settlements: preloading/surcharging; rigid inclusions; or, deep soil mixing (DSM). A site-specific investigation carried out at the proposed location of the Lancaster CVIF during preliminary design would be required to fully assess the viability of these options, but some conceptual guidance is provided in the following paragraphs.

Preloading and surcharging, since preloading alone would likely not be effective in sufficiently reducing the ongoing secondary settlements, could be used to raise the grade in advance of construction of the CVIF structures and roadways. Due to the sensitive nature of the clay and consolidation characteristics, it is expected that the preload/surcharge time would likely be a minimum of 2 years but could be as long as 7 to 10 years based on the available subsurface information. Wickdrains could be installed to shorten the time required for the preload and surcharge. The surcharge will likely need to be at least 1.5 metres in height (i.e., in addition to the grade raise height) and may need to be higher to reduce the post-construction settlements. This conceptual surcharge height does not include any additional surcharge height for foundation loading, if required (i.e., if not all structures are supported on deep foundations).

The installation of Rigid Inclusions (RI's) is another alternative for mitigating settlements and may also provide support for shallow foundations. RI's constructed of ready-mix concrete or stone columns installed within the clay soil using specialty equipment would be suitable for this site. RI's could be installed in the clay deposit, up to the original ground surface, to transfer the stresses down to the glacial till or bedrock. A Load Transfer Platform (LTP) created using granular material and geogrid, would be constructed above the RI's (i.e., beneath the grade raise) to transfer the grade raise loads to the columns. The granular fill of the grade raise could likely form part of the LTP. However, the depth of the clay deposit at this site may be at, or just beyond, the limit of this technology in sensitive clay soils.

Deep soil mixing (DSM) is another alternative for mitigating settlements beneath the site and may also provide support for shallow foundations. DSM consists of in situ mechanical mixing of the native soil through a process that breaks down the soil without extraction while injecting a stabilizing agent in the mix at low pressure.

Both RI's and DSM may not be cost effective, considering the size of the area that may require improvement.

5.3.5 Site Coefficient

The seismic design provisions of the 2012 Ontario Building Code depend, in part, on the shear wave velocity of the upper 30 metres of soil and/or rock below founding level. However, the OBC also permits the Site Class to be specified based solely on the stratigraphy and in situ testing data (i.e., standard penetration test results and in situ vane test results), rather than from direct measurements of the shear wave velocity. Using that methodology, a Site Class of D may be used for functional design of the proposed structures.

Vertical Seismic Profiling (VSP) geophysical testing was carried out at the Fraser Road site during the 2018 investigation to evaluate the average shear wave velocity of the upper 30 m of soil/bedrock at the site. Based on the shear wave velocity profile it was determined that a Site Class of C could be used in design at that site.

A higher site class may be possible based on VSP geophysical testing (and/or any ground improvement undertaken at the site), which should be carried out as the design progresses to confirm the Site Class.

5.3.6 Slab-on-Grade

Prior to the placement of any engineered fill, all topsoil, organic material, existing fill, or loosened soil should be stripped from below the proposed slab-on-grade.

The slab-on-grade for any buildings should be supported by at least 200 mm of OPSS.PROV 1010 Granular A material, for bedding purposes, placed and compacted in accordance with OPSS.PROV 501 (Compaction).

Unless uncontrolled migration of water vapour through the slab is acceptable, a robust polyethylene vapour barrier should be provided between the Granular A fill and the concrete.

5.3.7 Excavations

Excavations for the foundations will be through the surficial topsoil and existing fill materials (if and where present), and potentially into the native clay deposit.

Conventional open cut excavations may be feasible for this project, considering the open nature of the site (i.e., there are no services or utilities present near the potential excavations). According to OHSA, temporary excavations (i.e., those that are open for a relatively short time period) in the native firm silty clay should be made with side slopes no steeper than 1 horizontal to 1 vertical (i.e., 1H:1V) from the base of the excavation. Deeper excavations (i.e., if required and greater than 2 m in depth) that extend into soft clay may need to be provided with flatter side slopes.

5.3.8 Groundwater Control

It is anticipated that groundwater control for excavations to limited depth (i.e., less than 2 metres), can be adequately handled using appropriately sized and filtered sumps in the base of the excavation. Sumps should be located outside of the actual footing limits.

Surface water should be directed away from the open cut excavations.

5.4 Desktop Soil Liquefaction Assessment

5.4.1 General

The preliminary liquefaction assessment for this desktop study was based on historical data provided in the previous Foundation Investigation Reports available from the MTO GEOCRE database and more recent foundation investigation reports prepared for MTO by Golder, as described Section 3.1. No field work was carried out as part of the current study and investigations have not previously been completed at the subject site.

Liquefaction is a phenomenon whereby seismically-induced shaking generates shear stresses within the soil under undrained conditions. These stresses tend to densify the soil (i.e., leading to potentially large surface settlements) and under undrained conditions generate excess pore pressures. The excess pore pressures also lead to sudden temporary losses in strength. Where existing static shear stresses are present, the loss of strength can lead to significant lateral movements (i.e., analogous to a slope failure) often referred to as “lateral spreading” or under certain conditions even catastrophic failure of the slope often referred to as “flow slides”. Lateral spreading and flow slides often accompany liquefaction along rivers and other shorelines.

The site falls within the Western Québec Seismic Zone (WQSZ) according to the Geological Survey of Canada (GSC). The WQSZ constitutes a large area that extends from Montréal to Témiscaming. Within the WQSZ, recent seismic activity has been concentrated in two subzones; one along the Ottawa River and another more active subzone along the Montréal-Maniwaki axis. Historical seismicity within the WQSZ includes the 1935 Témiscaming event, which had a magnitude (i.e., a measure of the intensity of the earthquake) of 6.2 and the 1944 Cornwall Massena event which had a magnitude of 5.6. In comparison to other seismically active areas in the world (e.g., California, Japan, New Zealand), the frequency of earthquake activity within the WQSZ is significantly lower but there still exists the potential for significant earthquake events to be generated.

5.4.2 Desktop Preliminary Liquefaction Potential

The susceptibility to liquefaction of the native silty clay and glacial till materials anticipated at the subject site have been evaluated based on the results of the investigations at the Raisin River Overpass and Fraser Road Underpass sites. Based on the available information, these soils are likely not to be susceptible to liquefaction during the design earthquake event. It should be noted that during detailed design the actual composition and thickness of the soil deposits will need to be determined and shear wave velocity profiling carried out in order to undertake a site-specific evaluation of liquefaction.

5.5 Additional Investigation Works

During the future detailed design, further foundation investigation and analysis will be warranted once the location of the CVIF and the site plan have been finalized to further assess and/or confirm the desktop recommendations provided herein.

The detailed geotechnical investigation should include the following:

- An assessment of the presence, thickness and geotechnical properties of the soil deposits by drilling foundation boreholes at the structures, overhead signs, canopies, inspection and service bays and septic system structures locations should be carried out. It is also recommended that monitoring wells be installed to better define the groundwater levels well as the hydraulic conductivity at the site.
- Undisturbed samples of the clay material should be taken to assess the consolidation characteristics of the clay materials, in order to estimate the magnitude of settlement due the proposed grade raise and the development of settlement mitigation measures.
- It is anticipated that founding of settlement sensitive structures and moderately to heavily loaded structures will likely require deep foundations founded on the bedrock. Boreholes should therefore be advanced into the underlying bedrock and core samples obtained to assess the bedrock type and strength characteristics.
- The potential for and the implications of liquefaction should be confirmed, and this should include carrying out shear wave velocity profiling at the site.

6.0 CLOSURE

This Desktop Foundation report was prepared by Mr. Kenton Power, P.Eng. It was reviewed by Mr. Bill Cavers, P.Eng., a Senior geotechnical engineer and Associate of Golder. Mr. Fintan Heffernan, P.Eng. a Senior Consultant with Golder and the Designated MTO Foundations Contact for this project, carried out an independent quality control review of this report.

Yours truly,

Golder Associates Ltd.



Kenton Power, P.Eng.
Geotechnical Engineer



William Cavers, P.Eng.
Associate, Senior Geotechnical Engineer



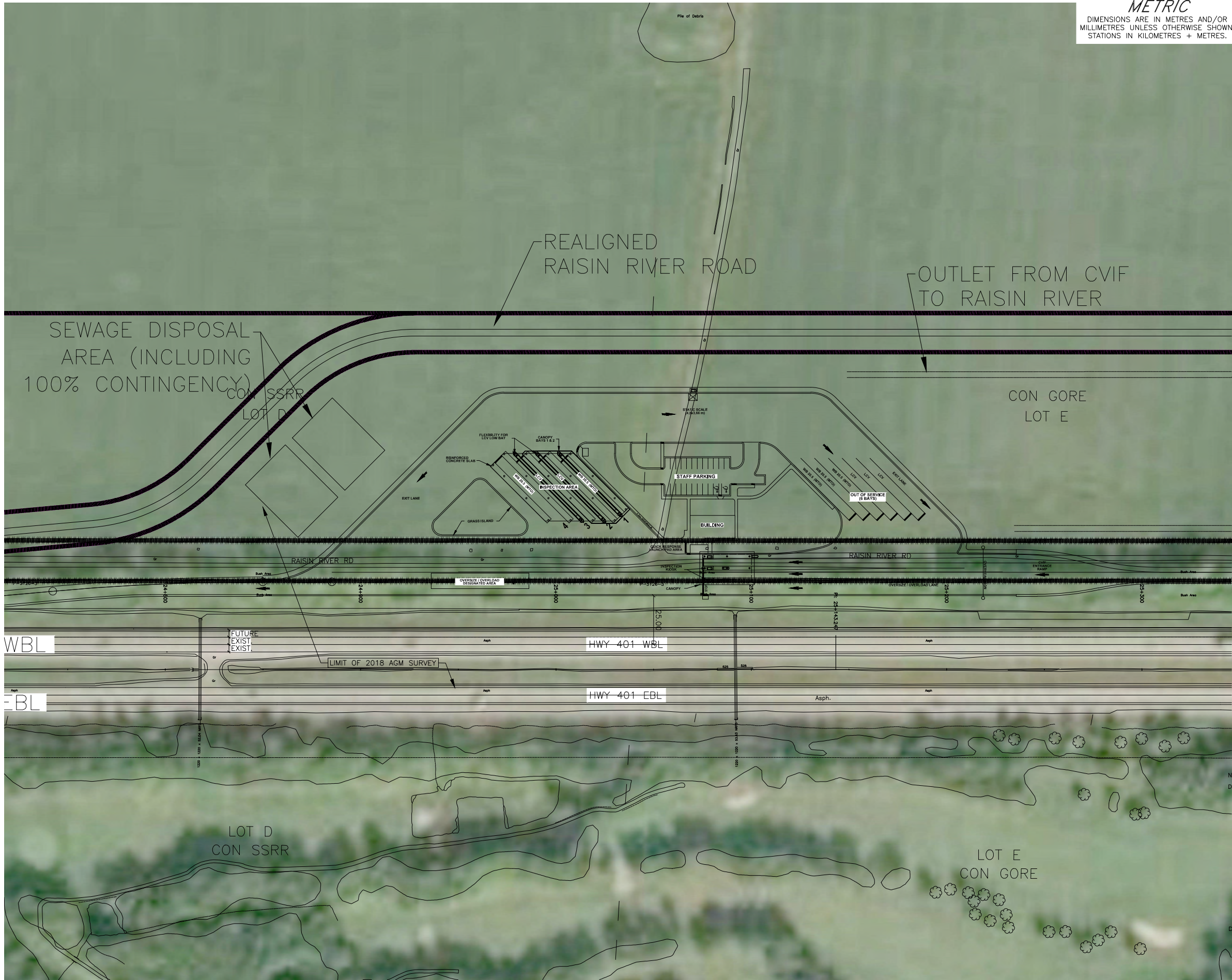
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Designated MTO Foundations Contact



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\\golder.gds\gal\ottawa\active\2016\3 proj\1651503 dillon cvif gananoque & lancaster\08_reports\foundations\lancaster\final\1651503-001-r-rev0-final fidr-mar2019.docx

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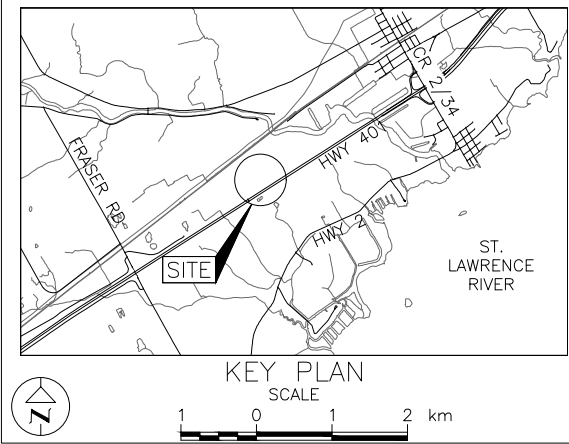


METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 4045-10-01

LANCASTER CVIF
HIGHWAY 401
SITE LOCATION
LAT. 45.124413 LONG. -74.524158

SHEET



NOTES

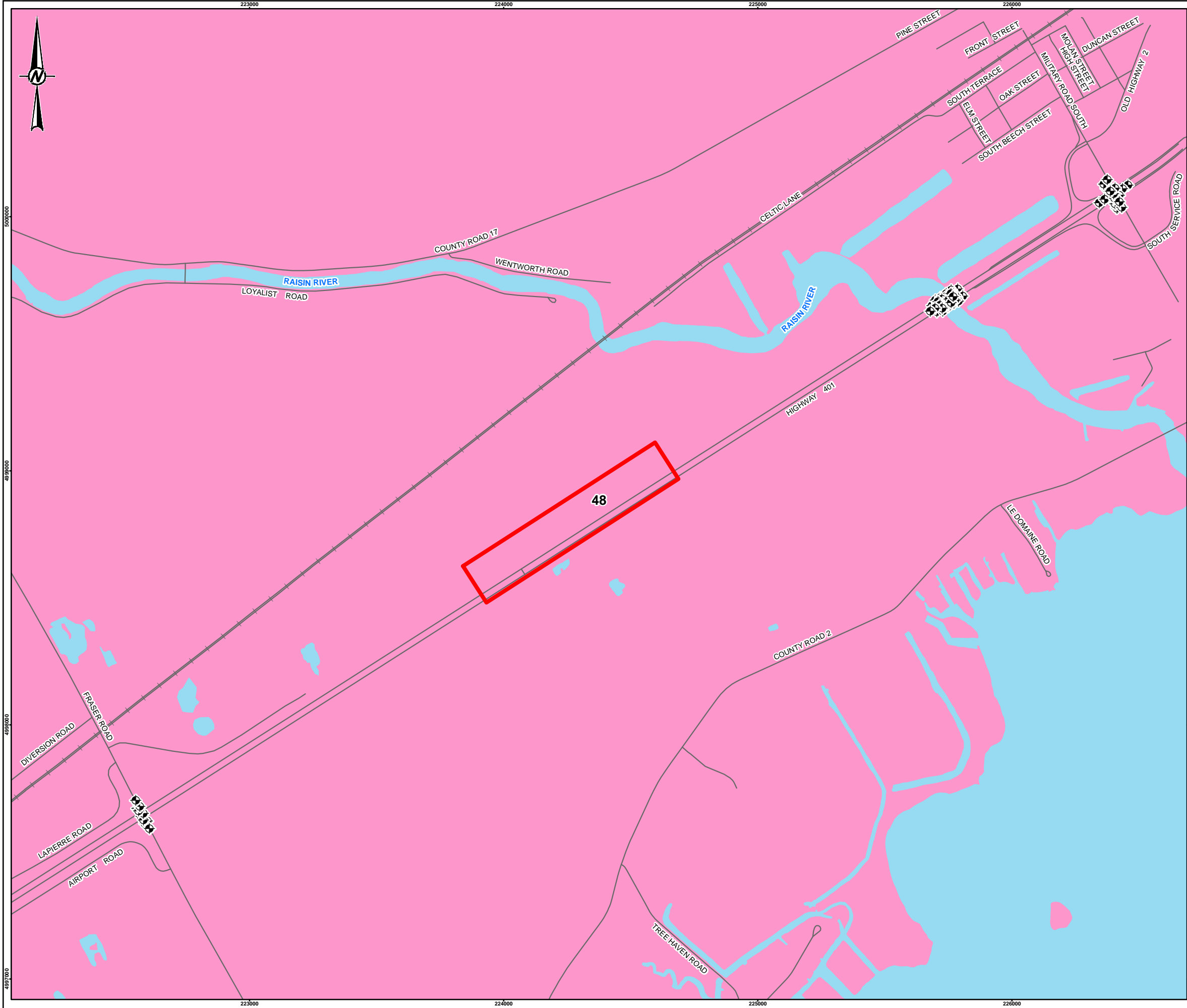
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE

Base plans provided in digital format by Dillon, drawing file no. Lancaster CVIF - Option 2 - 800m.dwg, received March 22, 2019.

NO.	DATE	BY	REVISION
Geocres No. 31G-274			
HWY. 401		PROJECT NO. 1651503	DIST. EASTERN
SUBM'D. KP	CHKD. KP	DATE: 1/15/2019	SITE: .
DRAWN: JM	CHKD. KP	APPD. FJH	DWG. 1



LEGEND

- BOREHOLE LOCATION, PREVIOUS INVESTIGATIONS
- SITE BOUNDARY
- ROADWAY
- RAILWAY
- WATERBODY

PHYSIOGRAPHIC REGION

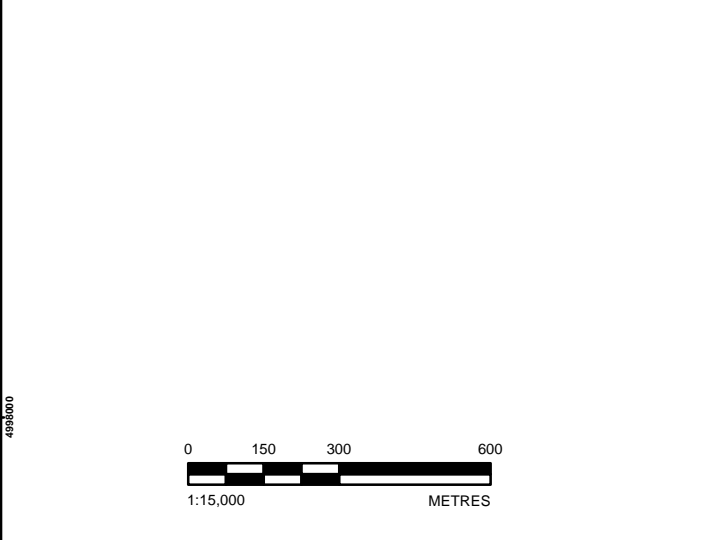
- 48, LANCASTER FLATS

NOTE(S)

1. ALL LOCATIONS ARE APPROXIMATE

REFERENCE(S)

1. LAND INFORMATION ONTARIO (LIO) DATA PRODUCED BY GOLDER ASSOCIATES LTD. UNDER LICENCE FROM ONTARIO MINISTRY OF NATURAL RESOURCES, © QUEEN'S PRINTER 2014
2. CHAPMAN, L.J. AND PUTNAM, D.F. 2007. PHYSIOGRAPHY OF SOUTHERN ONTARIO; ONTARIO GEOLOGICAL SURVEY, MISCELLANEOUS RELEASE-DATA 228
3. PROJECTION: TRANSVERSE MERCATOR, DATUM: NAD 83, COORDINATE SYSTEM: MTM ZONE 8, VERTICAL DATUM: CGVD28



CLIENT

MTO

PROJECT

LANCASTER CVIF

GEOTECHNICAL DESKTOP STUDY

TOWNSHIP OF SOUTH GLENGARY

TITLE

PHYSIOGRAPHIC REGIONS OF SOUTHERN ONTARIO

CONSULTANT

YYYY-MM-DD

2019-01-11

DESIGNED

PREPARED

ABD/BR

REVIEWED

KP

APPROVED

FJM

PROJECT NO.

1651503

CONTROL

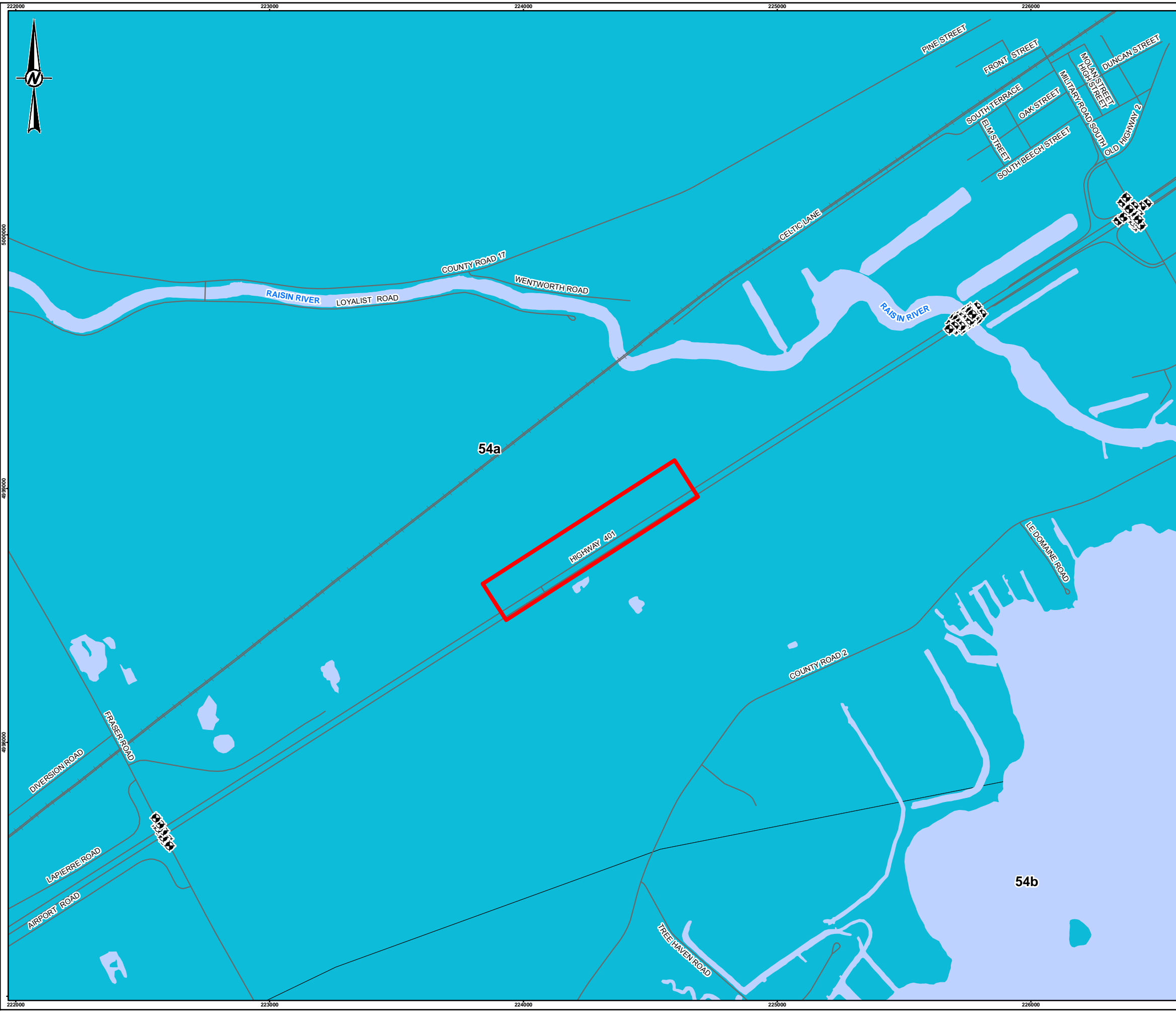
0001

REV.

A


FIGURE


2





LEGEND

BOREHOLE LOCATION, PREVIOUS INVESTIGATIONS


 SITE BOUNDARY


 ROADWAY

 RAILWAY

 WATERBODY

OGS BEDROCK

 54A OTTAWA GP.; SIMCOE GP.; SHADOW LAKE FM.

 54B CHAZY GP.; ROCKCLIFFE FM.

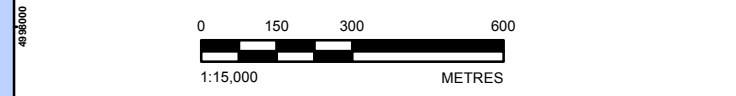
NOTE(S)
1. ALL LOCATIONS ARE APPROXIMATE

REFERENCE(S)

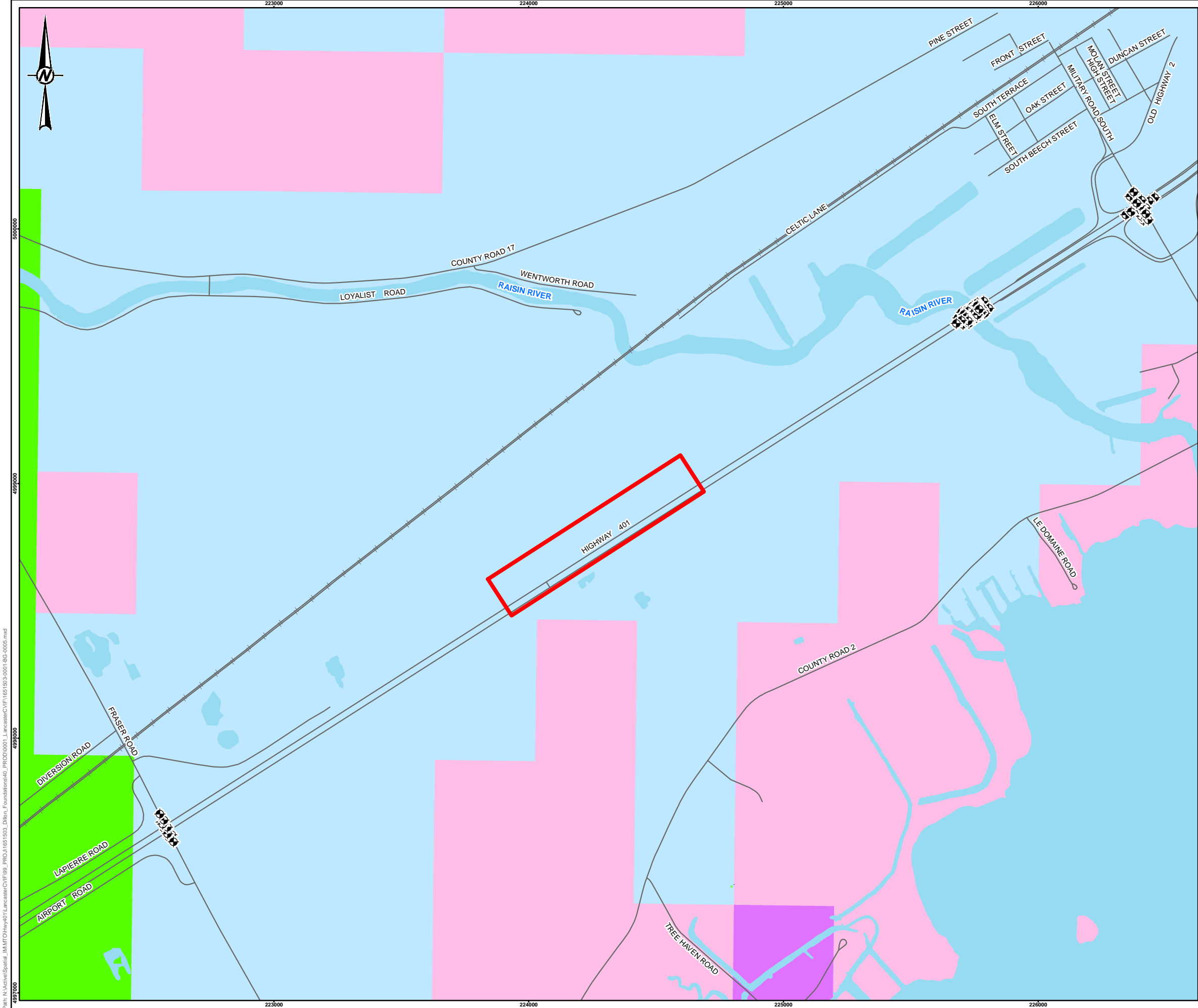
1. LAND INFORMATION ONTARIO (LIO) DATA PRODUCED BY GOLDER ASSOCIATES LTD. UNDER LICENCE FROM ONTARIO MINISTRY OF NATURAL RESOURCES. © QUEENS PRINTER 2014

2. ONTARIO GEOLOGICAL SURVEY 2011. 1:250 000 SCALE BEDROCK GEOLOGY OF ONTARIO; ONTARIO GEOLOGICAL SURVEY, MISCELLANEOUS RELEASE-126 - REVISION 1

3. PROJECTION: TRANSVERSE MERCATOR, DATUM: NAD 83, COORDINATE SYSTEM: UTM ZONE 18, VERTICAL DATUM: CGVD28



CLIENT		
MTO		
PROJECT		
LANCASTER CVIF GEOTECHNICAL DESKTOP STUDY TOWNSHIP OF SOUTH GLENGARY		
TITLE		
OGS BEDROCK GEOLOGY		
CONSULTANT		
YYYY-MM-DD		2019-01-15
DESIGNED		----
PREPARED		ABD/BR
REVIEWED		KP
APPROVED		FJM
PROJECT NO.	CONTROL	REV.
1651503	0001	A
		FIGURE
		4



LEGEND

- BOREHOLE LOCATION, PREVIOUS INVESTIGATIONS
- SITE BOUNDARY
- ROADWAY
- RAILWAY
- WATERBODY

OGS TREND IN DEPTH TO BEDROCK (METRES)

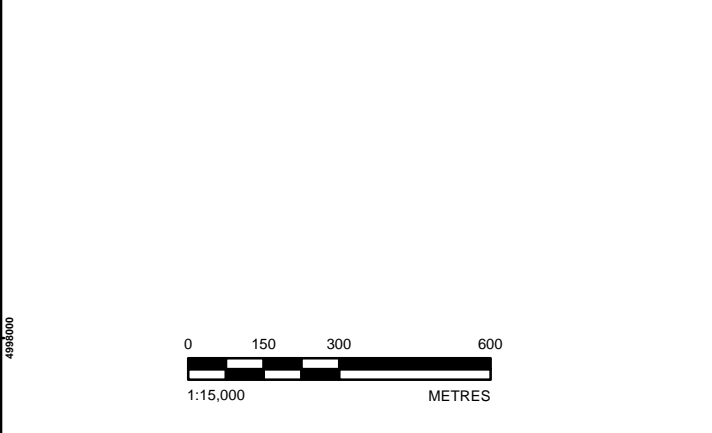
- 5 - 10
- 10 - 15
- 15 - 25
- 25 - 50

NOTE(S)

1. ALL LOCATIONS ARE APPROXIMATE

REFERENCE(S)

1. LAND INFORMATION ONTARIO (LIO) DATA PRODUCED BY GOLDER ASSOCIATES LTD. UNDER LICENCE FROM ONTARIO MINISTRY OF NATURAL RESOURCES, © QUEENS PRINTER 2014
2. BEDROCK TOPOGRAPHY AND OVERBURDEN THICKNESS MAPPING, SOUTHERN ONTARIO, ONTARIO GEOLOGICAL SURVEY, MISCELLANEOUS RELEASE - DATA 207
3. PROJECTION: TRANSVERSE MERCATOR, DATUM: NAD 83, COORDINATE SYSTEM: MTM ZONE 8, VERTICAL DATUM: CGVD28



CLIENT

MTO

PROJECT

LANCASTER CVIF

GEOTECHNICAL DESKTOP STUDY

TOWNSHIP OF SOUTH GLENGARY

TITLE

OGS DRIFT THICKNESS

CONSULTANT

YYYY-MM-DD

2019-01-14

DESIGNED

PREPARED

ABD/BR

REVIEWED

KP

APPROVED

FJM

PROJECT NO.

1651503

CONTROL

0001

REV.

A

FIGURE

5

4397000

APPENDIX A

GEOCRES Report for the Highway 401 Underpass at Fraser Road Site No. 31-230

GEOCRES Report for the Highway 401 Overpass at Raisin River Site No. 31-231/1&2

GEOCRES Report for the Highway 401 Underpass at County Road 2/34 Site No.

Mr. C. E. Robertson,
District Engineer,
Ottawa (District #9).

Foundation Section,
Materials & Testing Div.,
Room 107, Lab. Bldg.

~~Attention:~~ Mr. W. S. Aitken,
Construction Engr.

June 6, 1967

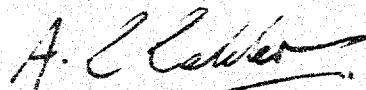
Ray, #401 and Fraser Road Instrumentation
W. J. 66-F-84 -- Contract No. 66-179

At the above mentioned location, we would like to install a vent pipe to the C.I.P. culvert which is now in place.

The details of this vent pipe and the required location are shown on the enclosed drawing.

Would you please make the necessary arrangements to have this work carried out.

ACC/MdeP
Encl.


A. C. Calder,
SENIOR FOUNDATION ENGINEER
For:
N. Davata,
SUPERVISING FOUNDATION ENGINEER

cc: Foundations Files ✓
Gen. Files

Department of Highways Ontario

Copy for the information of

Mr. A. G. Stermac, Principal Foundation Engineer, Room 107, Lab. Building

Mr. A. McKim,
Bridge Control Engineer,
Administration Building.

Bridge Division,
Downsview, Ontario

June 6, 1967

Fraser Road Underpass, W.P. 107-59, Contract 67-18,
Highway 401, Site No. 67-18, District No. 9

The Foundations Section have just requested that the piles at the ends of the wingwalls (4 piles in all) should be battered backwards (i.e. the tips further from Highway 401 than the tops) at 1 in 10. It might be better to ask the contractor to quote the additional cost and then to give written instructions rather than to change the drawings and wait for a claim. The piles involved are a large bored in place tubes costing \$24.00/ft to drive, a fairly large claim could be involved.

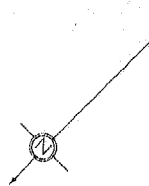
Please discuss this with the District. If you disagree, please give written instructions to change the drawings.

BSR/pr

cc. A. G. Stermac

B. S. Richardson,
Regional Bridge Project Engineer





P22 P23
P21 P20
117' LEFT

P19
P18
97' LEFT

P15 P16
P14 P13
85' LEFT

P12
P11
61' LEFT

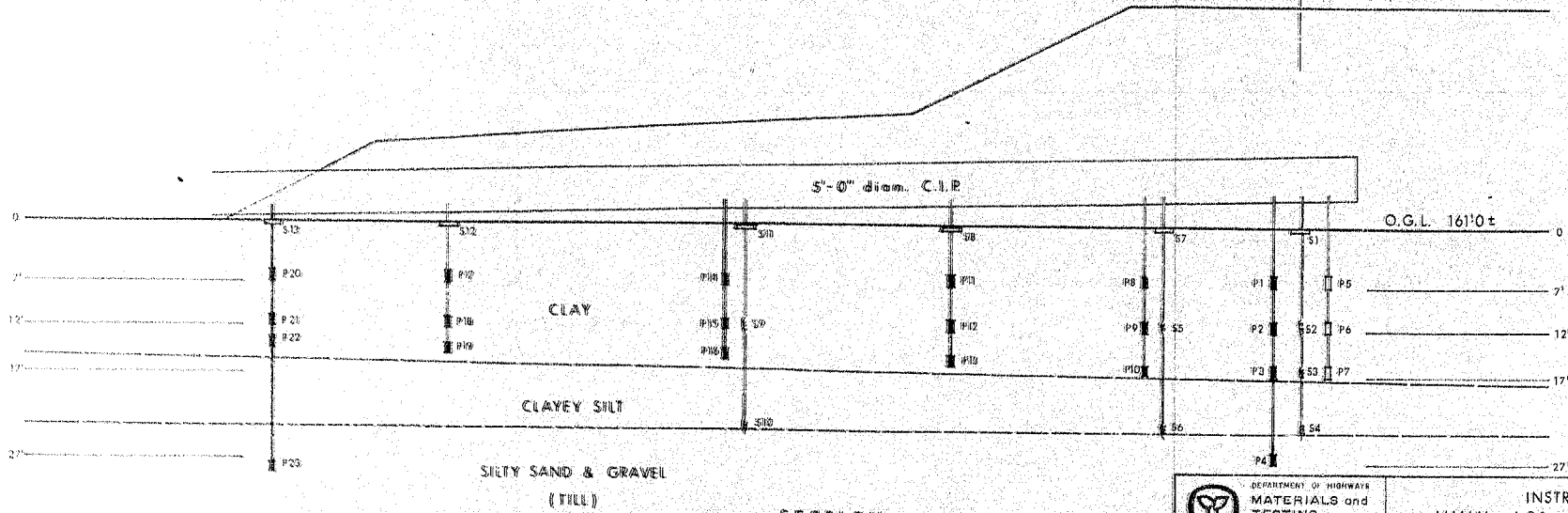
P8 P9
P7 P6
40' LEFT

P5 P6 P7 P8
P3 P2 P1
STN. 26 + 68

PLAN
SCALE : 1" = 10'

LEGEND

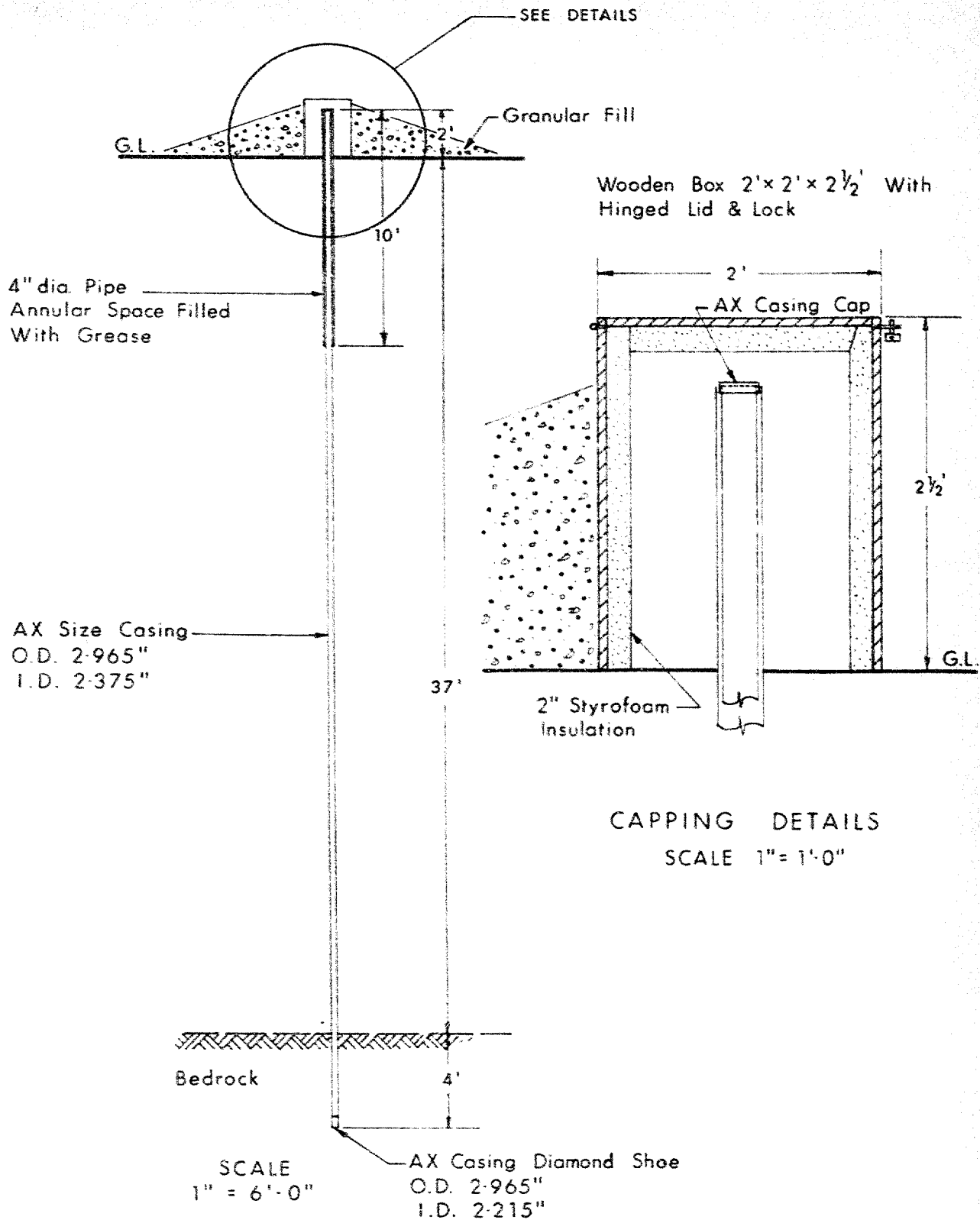
PLAN	PROFILE	QUANTITY
	SETTLEMENT PLATES	8
	GEONOR PIEZOMETERS	20
	PEAKER PIEZOMETERS	3
	SETTLEMENT AUGERS	7



SECTION
SCALE : 1" = 10'



INSTRUMENTATION
HWY. 401 & FRASER ROAD
INSTALLATION DETAILS



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

FRASER ROAD & HWY. 401 INSTALLATION DETAILS OF BENCH MARK

DATE 29 SEPT. 1966

APPROVED *M. Swata*

DRAWING NO. 66-F-84 C

Mr. C. B. Robertson,
District Engineer,
District #9 (Ottawa).

Foundation Section,
Materials & Testing Div.,
Room 107, Lab. Bldg.

Attn: Mr. W. S. Aitken,
Construction Engr.

September 16, 1966

-- Instrumentation --
Fraser Road, Highway 401
District #9 (Ottawa)
W.P. 107-59-1

Further to our telephone conversation, we are enclosing a drawing showing the details of the culvert proposed for the above mentioned instrumentation project. We have also detailed on the drawing the various quantities required for placing of the culvert. We feel that 3-ft. square holes should be cut with a torch in the bottom of the culvert at the predetermined distances prior to placing the culvert over the instrumentation area.

The field work for the installation of piezometers, settlement gauges, etc., will commence on September 19, 1966, and our Project Engineer, Mr. R. Magi, will be in charge of this project.

If there are any other points which need clarification, please let us know as soon as possible.

MD/MdeP
Attach.

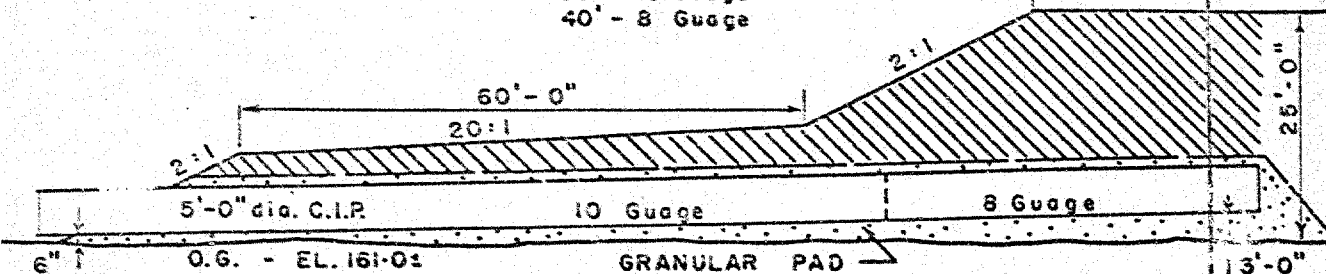
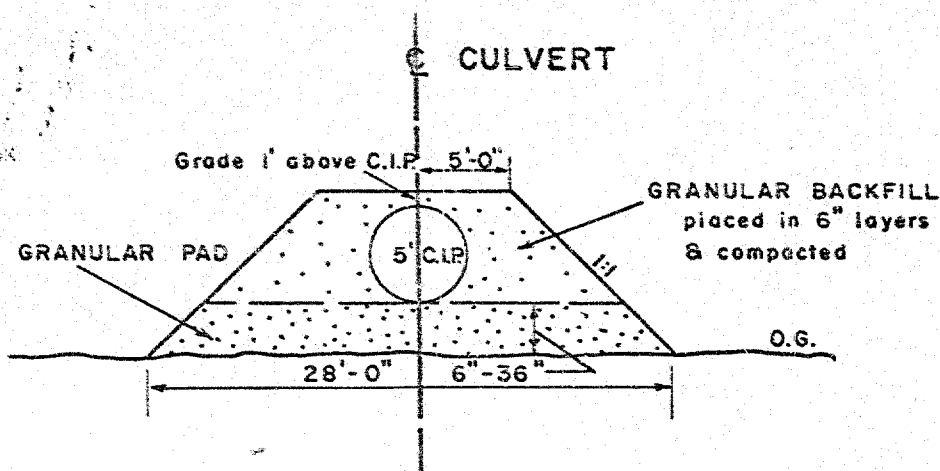
cc: Mr. J. E. Gruspier

Foundations Office ✓
Gen. Files

M. Devata
M. Devata,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

FRASER
ROAD

15-00000

 $\tau = 20^\circ$ $10^3 = 10^3$ 

DEPARTMENT OF HIGHWAYS
MATERIALS AND
TESTING
DIVISION

HWY. 401 & FRASER ROAD INSTRUMENTATION

C.I.P. PLACEMENT DETAILS

DATE SEPT. 15, 1966

APPROVED

DRAWING NO. 66-F-84B

H. Q. GOLDER & ASSOCIATES LTD.

CONSULTING CIVIL ENGINEERS

H. Q. GOLDER
V. MILLIGAN
L. G. SODERMAN
J. L. SEYCHUK

2444 BLOOR STREET WEST
TORONTO 9, ONTARIO
763-4103
767-9201

W. P. 107 - 59

REPORT

TO

DEPARTMENT OF HIGHWAYS, ONTARIO

ON

SOIL CONDITIONS AND FOUNDATIONS

PROPOSED FRASER ROAD UNDERPASS

HIGHWAY 401

GLENGARRY COUNTY

ONTARIO

Distribution:

10 copies - Department of Highways, Ontario,
Toronto, Ontario.

2 copies - H. Q. Golder & Associates Ltd.,
Toronto, Ontario.

January, 1966

65135

*Received Construction
Sept 20/1966*

TABLE OF CONTENTS

ABSTRACT	1
INTRODUCTION	2
PROCEDURE	2
SITE & GEOLOGY	3
SUBSURFACE CONDITIONS	4
DISCUSSION	
General	9
Statement of Problem	10
Stability Analyses	11
Possible Solutions	16
ABBREVIATIONS	In order
RECORDS OF BOREHOLES	Following
	Page 18
FIGURES	
1 Boring Plan and Soil Stratigraphy Section	
2 - 6 Grain Size Distribution Curves	
7 Summary of Engineering Properties Sensitive Clay	
8 Typical Stress-Strain Curves	
9 Consolidated Undrained Triaxial Tests with Pore Pressure Measurements	
10 - 14 Laboratory Consolidation Test Results	
15 Preconsolidation Pressures vs Elevation	
16 Sample Results - Total Stress Stability Analyses	
17 Summary of Results of Total Stress Stability Analyses	
18 Sample Results - Effective Stress Stability Analyses	
19 Summary of Results of Effective Stress Stability Analyses	

ABSTRACT

The results of an investigation to determine the subsurface conditions at the site of the proposed underpass structure to carry the line "A" alignment of Fraser Road over Highway 401 in Charlottenburgh Township near Lancaster, Ontario are reported. Information for the foundation design of the proposed structure and associated roadway approach embankments is also presented.

It was found that the site is underlain by as much as 25 feet of generally firm sensitive marine clay extending down to a 10 foot thick stratum of dense to very dense sandy till. Fairly sound limestone bedrock underlies the till between elevations 120 and 130. The natural groundwater level was found to be at ground surface in the lower lying areas of the site. A slight artesian pressure was encountered in the sandy till stratum with a measured head as high as 2 feet above ground surface.

The sensitive marine clay is the significant subsurface stratum which affects foundation design and controls approach embankment stability. The main design problem at this site is limiting strain and overstressing effects in the sensitive clay resulting from the roadway embankment loading to prevent detrimental lateral movement of the bridge abutments.

Both total and effective stress stability analyses were carried out for the proposed 26 foot high approach embankments and the results of these analyses show that counterbalancing berms some 12 feet high and 80 feet long are required to minimize lateral movements in the clay subsoil beneath the bridge abutment areas to reasonable limits. To resist lateral forces within the subsoil and also negative skin friction forces resulting from consolidation settlement in the clay, it is suggested that a rigid piled foundation be provided for the support of the bridge abutments. With a rigid abutment foundation consideration could be given to reducing the required berm length to about 60 feet. Other possible solutions such as decreasing the height of approach embankments by increasing the slope of the Fraser Road grade line are discussed in this report.

INTRODUCTION

H. Q. Golder & Associates Ltd. have been retained by the Department of Highways, Ontario to carry out a subsurface investigation at the site of a proposed underpass to carry revision line "A" of Fraser Road over Highway 401 in Charlottenburgh Township, Ontario. The purpose of this investigation was to determine the subsurface conditions at the site and to provide information for the foundation design of the proposed structure and associated roadway approach embankments.

PROCEDURE

The field work for this investigation was carried out between November 9 and December 3, 1965. During this period a total of 7 boreholes with adjacent dynamic penetration tests and 5 additional dynamic penetration tests, ranging in depth from about 25 to 50 feet, were put down using a skid mounted machine drillrig supplied and operated by the F. E. Johnston Drilling Co. Ltd., Ottawa, Ontario. Following completion of each boring a standpipe or piezometer was installed for groundwater level observation. The field work was supervised throughout by an engineer from our staff.

The locations of the borings and dynamic penetration tests put down during the investigation are shown on Figure 1 located in a pocket following the Records of Boreholes. A detailed log for each boring and dynamic penetration test is given on the Records of Boreholes following the text of this report. A section of the inferred

subsurface stratigraphy along the proposed centerline of Fraser Road is given in Figure 1.

The samples obtained during the investigation were brought to our laboratory for detailed examination and testing. The results of this testing are shown on the Records of Boreholes and on Figures 2 to 15, inclusive.

The elevations given in this report are referred to Geodetic datum and were determined from a bench mark consisting of a nail and washer in a root of a three foot diameter oak tree located 510 feet to the right of station 21+95. The elevation of this bench mark is 165.58 feet. The borehole elevations and locations were supplied to us by the Department of Highways, Ontario.

SITE & GEOLOGY

The site of the proposed underpass to carry revision line "A" of Fraser Road over Highway 401 is located some 2.8 miles west of Lancaster, Ontario in Charlottenburgh Township, Glengarry County, Ontario.

Except for the existing Highway 401 and Fraser Road the site is generally flat and grass covered. In lower lying areas of the site the ground was covered during the period of this investigation by up to 6 inches of water. The grade of the existing Highway 401 is some

6 feet above the surrounding ground surface and the highway consists of 4 paved lanes with median strip and associated gravelled shoulders. Fraser Road is some 4 feet above the surrounding ground surface and has a 20 foot wide gravelled surface.

The site of the proposed underpass structure is located in the physiographic region known as the Lancaster Flats. Based on available geological information it is known that the subsoil consists of rather poorly drained deposits consisting of water-laid materials ranging from clay to very fine sand. The clay in this particular area is of marine origin and was deposited in the upper reaches of the Champlain Sea which covered the St. Lawrence Lowlands in recent geological time. The marine clay, referred to as "Leda" clay, is generally underlain by a granular till deposit which is in turn underlain at a depth of some 25 to 100 feet, by limestone and shale bedrock.

SUBSURFACE CONDITIONS

The detailed stratigraphy encountered in each boring is given on the Records of Boreholes. The stratigraphy along the proposed centerline of Fraser Road has been interpolated from this data and is presented on Figure 1. Following is a summary account of the inferred subsurface conditions at the site.

The borings put down indicate that the existing Highway 401 roadway fill consists of about 5 feet of compact to dense brown silty sand to sand and gravel. Typical grain size distribution curves for

samples of the roadway fill are shown on Figure 2. Up to 3 feet of black silty topsoil was found beneath the roadway fill and across the remainder of the site.

In the southern portion of the site the topsoil is underlain by as much as 2 feet of compact brown to grey sandy silt to silty sand. This surficial deposit varies in composition to a very stiff brown clayey silt in the northern portion of the site. A grain size distribution curve for a sample of the clayey silt is given on Figure 2.

Underlying the shallow surficial deposits at about elevation 158, the borings encountered some 10 to 25 feet of grey sensitive clay containing a trace to some silt throughout. The clay generally has a fissured or chunky structure but the stratum contains zones where no fissuring is evident. The clay does not have any pronounced stratification or layering pattern. Typical grading curves for samples from the sensitive clay stratum are shown on Figure 3.

Based on thirteen Atterberg limit determinations the liquid limit of the clay varies between about 65 and 85 and the corresponding plasticity index is between 50 and 65. The in situ water content of the clay is between about 60 and 90 per cent resulting in a liquidity index generally between about 0.8 and 1.1. The results of the Atterberg limit tests are summarized on Figure 7. This figure indicates that the liquid and plastic limits of the clay are fairly constant

with depth and that there is a trend for an increase in the in situ water content with depth to about elevation 150 below which there is a slight decrease with depth.

The results of fourteen unit weight determinations, which have been plotted against elevation on Figure 7, indicate that the total unit weight of the clay varies between about 95 and 110 lb/cu.ft. with an average value of about 100 lb/cu.ft.

The undrained shear strength of the clay was determined by in situ vane testing in the field and by undrained triaxial compression tests on relatively undisturbed samples in the laboratory. The results of these tests are plotted on the Records of Boreholes and summarized on Figure 7. Typical stress-strain curves for the undrained compression tests are shown on Figure 8. The test results indicate that the failure strain is as low as 1 per cent and that the undrained shear strength decreases with depth from a value of about 1,500 lb/sq.ft. near the surface of the clay to about 450 lb/sq.ft. near the bottom of the stratum. Based on these strength results, together with standard penetration test values which range from about 2 to 6 blows/ft., the consistency of the clay varies from stiff to firm with depth and is generally firm throughout.

Remoulded field vane tests gave values ranging between about 50 and 200 lb/sq.ft. Based on these results the clay has a sensitivity, which is defined as the ratio of undisturbed strength to

remoulded strength, of between about 5 and 15.

A series of three consolidated undrained triaxial compression tests with pore pressure measurements was also carried out in order to determine the effective or drained shear strength parameters of the clay. The results of these tests are plotted on Figure 9 using the method suggested by Rendulic (1937) and also using the conventional Mohr circle plot.

Five consolidation tests were carried out on relatively undisturbed samples of the clay. The results of these tests are presented as pressure-void ratio curves on Figures 10 to 14, inclusive. A summary plot of the preconsolidation pressure estimated from the pressure-void ratio curves is given on Figure 15. This plot indicates that the clay is overconsolidated by about 1.5 tons/sq.ft. in excess of existing overburden pressure near the surface of the stratum and becomes virtually normally consolidated with depth.

The sensitive clay stratum is generally underlain at about elevation 140 by as much as about 5 feet of grey clayey silt with sand and some gravel. This clayey silt layer is not continuous across the site as indicated by boreholes 3 and 5 where it is absent. Typical grain size distribution curves for the clayey silt are shown on Figure 4. Based on three Atterberg limit determinations the clayey silt has an average liquid limit of about 18 and an average plasticity index

of about 7. The in situ water content is about 12 per cent and is mid-way between the liquid and plastic limits.

Based on one field vane test which gave an undrained shear strength value of about 1,000 lb/sq.ft., together with the standard penetration tests which gave "N" values ranging between about 10 and 20 blows/ft., the clayey silt varies between firm and very stiff in consistency and is generally stiff.

Underlying the clayey silt and sensitive clay strata at a depth of between about 25 and 28 feet, the borings encountered some 10 to 15 feet of sandy till. The till consists of grey silty sand and gravel with a trace to some clay and contains some scattered cobbles and boulders particularly in the lower 5 feet of the stratum. Typical grain size distribution curves for samples of the till obtained using 1½ inch I.D. sampling equipment are shown on Figures 5 and 6. Based on the standard penetration test results given on the Records of Boreholes, the till is in a generally dense to very dense state of packing.

The till below about elevations 120 to 130 is underlain by bedrock which was proved by core drilling in AXT size for up to 10 feet in boreholes 1 to 5, inclusive. The bedrock consists of fairly sound grey limestone with interbedded shale layers.

During the boring operations in the upper portion of the

overburden it was noted that the groundwater level was at ground surface in the area of the site outside the existing roadway fills. Ground surface is as low as elevation 161. In advancing the boreholes through the lower portion of the overburden a slight artesian pressure was encountered in the dense sandy till underlying the relatively impervious sensitive clay stratum. The groundwater level was observed to rise as high as elevation 163 in the casing.

A piezometer which was sealed into the till stratum and a standpipe which was placed in the clay stratum were generally installed in each of the borings following their completion. Periodic readings were taken in these installations during the course of the field work. The installation details together with the latest readings obtained are shown on the Records of Boreholes and on Figure 1.

The readings show that the natural groundwater level across the site is between about elevation 161 and 163 corresponding to existing ground surface in the lower portions of the site, and that the artesian water level in the underlying till is as much as 2 feet higher than the surface or upper groundwater level.

DISCUSSION

General

As presently planned the proposed underpass structure is to consist of four simply supported spans with each central span 66

feet long and the end spans each 40 feet in length. It is understood that the grade of Highway 401 is to remain unchanged and that proposed Fraser Road approach embankment grade over Highway 401 is to be at elevation 186. Thus the roadway approach embankments for the Fraser Road will be some 24 to 26 feet above general ground surface. Spill through abutments supported on piles driven to bedrock are to be used.

Statement of Problem

It is understood that in the general vicinity of the present investigation some difficulties with newly built structures have been and still are being experienced by the Department of Highways, Ontario. During the course of this field investigation some of the underpass structures in the area were studied and it was observed that large settlement of most of the roadway approach fills had occurred. In one case (Brookdale Avenue) a fairly extensive rotational distortion of a bridge abutment had taken place. It is understood that this abutment is founded on non-displacement "H" piles driven to bedrock and the height of the roadway approach fill is about 23 feet. A 50 foot long by about 10 foot high stabilizing berm is present at the front of the approach embankment end slope. The reason for the abutment movement is not clearly known but it was probably caused by consolidation settlement of the subsoil due to the embankment loading resulting in negative skin friction on the piles causing them to settle and by overstress of the clay subsoil whereby strain and creep effects produced lateral movement at depth.

Based on the above experience in the same physiographic region as the site presently under consideration, it was decided to give consideration to preventing or minimizing overstress and creep effects within the sensitive clay subsoil in the stability study of the proposed roadway approach embankments.

Stability Analyses

Stability computations for the proposed 26 foot high roadway approach embankments were carried out using the total stress approach (undrained shear strength of the clay). The results of typical computations to determine the factor of safety against a deep seated rotational type failure of the proposed embankment section using this approach are given on Figure 16. Summary plots giving the results of all the total stress stability analyses carried out are given on Figure 17.

Reference to Figure 17 shows that the factor of safety against a deep seated failure of the proposed 26 foot high embankment with 2 horizontal to 1 vertical side slopes is less than unity and of the order of 0.9. This figure also shows that the provision of a counterbalancing berm in front of the proposed embankment section increases the factor of safety. It is significant to note, however, that for the berm heights studied, namely 12 feet and 16 feet, the factor of safety is a function of the length of berm with the berm height having no major effect. Therefore no appreciable advantage is gained by

placing 16 foot high berms rather than 12 foot high berms. Furthermore, with the provision of a 12 foot high berm longer than about 80 feet no increase in factor of safety is obtained for the embankment section since the most critical circle passes through the berm itself. It can thus be concluded from Figure 17 that the maximum factor of safety obtainable with single berm construction is about 1.7. To increase the factor of safety above this level a double berm system is required.

In addition to the total stress stability analysis discussed above effective stress stability analyses were also carried out. The total stress stability approach is a valid method for predicting the initial stability of the embankment. However, the total stress analysis does not take into account the effects of induced pore water pressures in the clay due to embankment loading or unusual groundwater conditions such as artesian pressures within the subsoil. In order to evaluate the factor of safety for these conditions, the stability has to be analysed using the effective stress approach (laboratory drained shear strength parameters of the clay) incorporating pore pressures. However, it is not always possible to accurately predict the pore pressure build up during construction on the basis of laboratory tests alone. Therefore the most practical approach is to design the embankment initially on the basis of both effective and total stress analyses and to control its rate of construction by the effective stress analysis based on measured field pore pressures.

The results of the consolidated undrained triaxial compression tests carried out on samples of the sensitive clay are presented on Figure 9. In the effective stress stability analyses discussed below, the effective cohesion, c' , and the effective angle of shearing resistance, ϕ' , of the clay were taken to be zero and 20° , respectively. These values were chosen on the basis of a strain criterion such that overstressing of the subsoil due to the embankment loading is minimized. For this criterion a limiting strain of between 1 and 2 per cent was selected for the laboratory test results. Reference to Figure 9 further shows that, for an effective stress failure criterion based on maximum deviator stress, $(\sqrt{1}' - \sqrt{3}') \text{ max.}$, an effective angle of shearing resistance, ϕ' , of 24° is obtained taking $c' = 0$.

As mentioned previously it is not possible to accurately estimate the build up of pore pressure within the subsoil due to embankment loading on the basis of laboratory testing alone. Therefore the effective stress stability analyses were carried out using an overall pore pressure parameter, \bar{B} , of zero, 0.5 and 1.0 to cover the range of possible values.

The results of typical stability computations using effective shear strength parameters for the clay are shown on Figure 18. A summary plot of all the effective stress stability computations carried out is presented on Figure 19. This summary plot shows that for the

proposed 26 foot high embankment, the factor of safety varies from 0.4 to about 1.0 as the overall pore pressure parameter, \bar{B} , is varied from 1.0 to zero. Furthermore the factor of safety is increased by the provision of a berm. With a 60 foot long berm, the factor of safety taking full excess pore water pressure ($\bar{B} = 1$) is 0.6 while for the long term case where all excess pore water pressure in the clay is dissipated ($\bar{B} = 0$) the factor of safety is about 1.4.

A direct comparison between the factor of safety obtained by the total and effective stress approaches cannot be made in this case. The total stress analysis gives higher factors of safety due to the fact that the undrained shear strength value used for the clay represents an ultimate or failure condition, while in the effective stress approach the shear strength parameters used do not represent a failure condition but a limited strain condition, as discussed above. It is interesting to note, however, that the effective stress analysis for the long term case taking full excess pore water pressure dissipation gives a factor of safety of about 1.4 for a 60 foot long berm. The total stress analysis factor of safety for this same stabilizing berm size is about 1.5.

For stability analyses based on total stress approach a factor of safety of the order of 1.7 is required if general overstressing of the subsoil is to be minimized. Therefore, a berm length of

the order of 80 feet would be required if excessive movements within the subsoil are to be eliminated. The results of the effective stress stability analyses indicate that for a berm of this length the factor of safety is of the order of 1.5 for the long term case ($\Delta u = 0$). The results also indicate that if the entire loading imposed by the embankment is initially carried by the pore water ($\bar{B} = 1$), the factor of safety for an 80 foot berm is about 0.7. This condition, however, assumes almost instantaneous placing of the embankment, whereas in the field filling operations are relatively slow. Therefore the $\bar{B} = 1$ condition would not be realized and the induced pore pressure in the subsoil, allowing for some consolidation during construction, would correspond to an overall pore pressure coefficient lower than 1.0 and probably of the order of 0.5. For the $\bar{B} = 0.5$ case the factor of safety of a 26 foot high embankment with 80 foot long berms would be slightly greater than unity. During construction of the embankment and associated berm, the induced pore pressure in the subsoil should be measured by piezometers. The readings obtained in the piezometers would form the control governing the rate of embankment construction.

It must be stressed that the factors of safety based on the effective shear strength parameters ($c' = 0$, $\phi' = 20^\circ$) have been calculated on the basis of limited strain within the sensitive clay and do not represent a failure condition for which $\phi' = 24^\circ$ using $c' = 0$ (see Figure 9).

The total settlement beneath the center of the proposed roadway approach embankment due to consolidation within the sensitive clay stratum has been estimated on the basis of laboratory consolidation tests. Since the clay has not been preconsolidated to the pressures to be imposed by the proposed fill, settlement of the embankment is estimated to be large and of the order of 18 to 24 inches. This settlement should take place within 5 to 10 years following construction with the majority of the settlement occurring in the first 2 to 3 years.

Possible Solutions

To prevent significant movement of the proposed bridge abutments due to lateral strain effects within the subsoil caused by the imposed roadway approach embankment loading, the embankments may be constructed as discussed above using an end stabilizing berm some 12 feet high and 80 feet long. For this case the factor of safety based on total stress analyses and using a failure criterion is 1.7 while the long term factor of safety based on effective stress and a limited strain criterion is about 1.5. This solution would however necessitate a considerable increase in the length of the structure.

Since settlement of the proposed roadway approach embankments may be as high as 2 feet, negative skin friction forces will be imposed on the piles supporting the abutments. These forces combined with movement of the subsoil due to strain imposed by the embankment loading will generally tend to displace the piles laterally and, if

they are not firmly seated in bedrock, vertically downward. It is generally conventional to employ non-displacement piles such as "H" piles in sensitive clay subsoil to minimize remoulding and subsequent loss of strength due to driving. Standard "H" pile sections are however flexible and when subjected to combined vertical and lateral forces may tend to distort. This is a possible explanation of the unsatisfactory performance of some of the existing bridge abutments in the general area.

Therefore consideration should be given to founding the proposed bridge abutments on rigid piles such as tube piles filled with reinforced concrete. To prevent excessive disturbance of the sensitive clay the tube piles should be placed in pre-augered holes. Since the negative skin friction forces may be large, the piles should be set firmly in the bedrock to ensure that no settlement of the abutment occurs.

With the provision of a rigid pile foundation capable of withstanding lateral forces due to movement within the subsoil resulting from the imposed embankment loading, it would not be necessary to design the embankment to as high a factor of safety (1.7 total stress) as discussed previously. A factor of safety based on the undrained shear strength of the clay (total stress) of 1.5 would be adequate to prevent failure of the embankment. Thus with provision of a rigid piled abutment foundation reduction of the berm length to

about 60 feet could be considered.

In conjunction with founding the proposed abutments on rigid piles additional strength may be achieved by employing a rigid structure rather than the proposed simply supported spans.

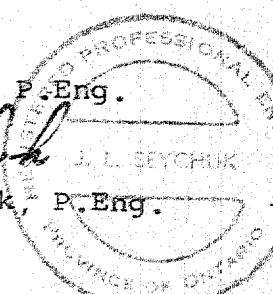
The length of berm required, and hence the length of structure may be further reduced by lowering the grade at which Fraser Road crosses Highway 401. This however is not likely possible due to clearance requirements. However, the slope of the Fraser Road grade line may be increased thus reducing the required height of approach embankments with a consequent reduction in the length of berm.

At a meeting held on January 12, 1966 between Mr. A. G. Stermac of the Department of Highways, Ontario and members of our staff, the results of our study to date on this project were presented and discussed. It was decided at this meeting to present a report covering all of the work carried out to January 12, 1966. Following a study of this report by the Department of Highways, a further meeting is to be arranged to discuss the possible solutions outlined above and to decide on the most practical and economic solution to the problem. Further analyses would be carried out, as required, after the second meeting and a final report presenting the recommended foundation treatment would be prepared at that time.

JBD:JLS:IMB
65135
January 17, 1966

GOLDER & ASSOCIATES

Jor
for J. B. Davis, P. Eng.
J. L. Seychuk
J. L. Seychuk, P. Eng.

A circular professional engineer seal for the Province of Ontario. The outer ring contains the text "REGISTERED PROFESSIONAL ENGINEER" at the top and "PROVINCE OF ONTARIO" at the bottom. The inner circle contains the name "J. L. SEYCHUK" and "P. Eng." below it.

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

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

<i>WH</i>	sampler advanced by static weight—weight, hammer
<i>PH</i>	sampler advanced by pressure—pressure, hydraulic
<i>PM</i>	sampler advanced by pressure—pressure, manual

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

<i>Consistency</i>	<i>c_u, lb./sq. ft.</i>
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

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

NOTES:

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

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

LIST OF SYMBOLS

I. GENERAL

π	= 3.1416
e	= base of natural logarithms 2.7183
$\log_e a$ or $\ln a$	natural logarithm of a
$\log_{10} a$ or $\log a$	logarithm of a to base 10
t	time
g	acceleration due to gravity
V	volume
W	weight
M	moment
F	factor of safety

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Unit weight

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

(b) Consistency

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

(c) Permeability

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

(d) Consolidation (one-dimensional)

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

(e) Shear strength

τ_f	shear strength
c'	effective cohesion
ϕ'	effective angle of shearing resistance, or friction
c_u	apparent cohesion*
ϕ_u	apparent angle of shearing resistance, or friction
μ	coefficient of friction
S_t	sensitivity

in terms of effective stress
 $\tau_f = c' + \sigma' \tan \phi'$

in terms of total stress
 $\tau_f = c_u + \sigma \tan \phi_u$

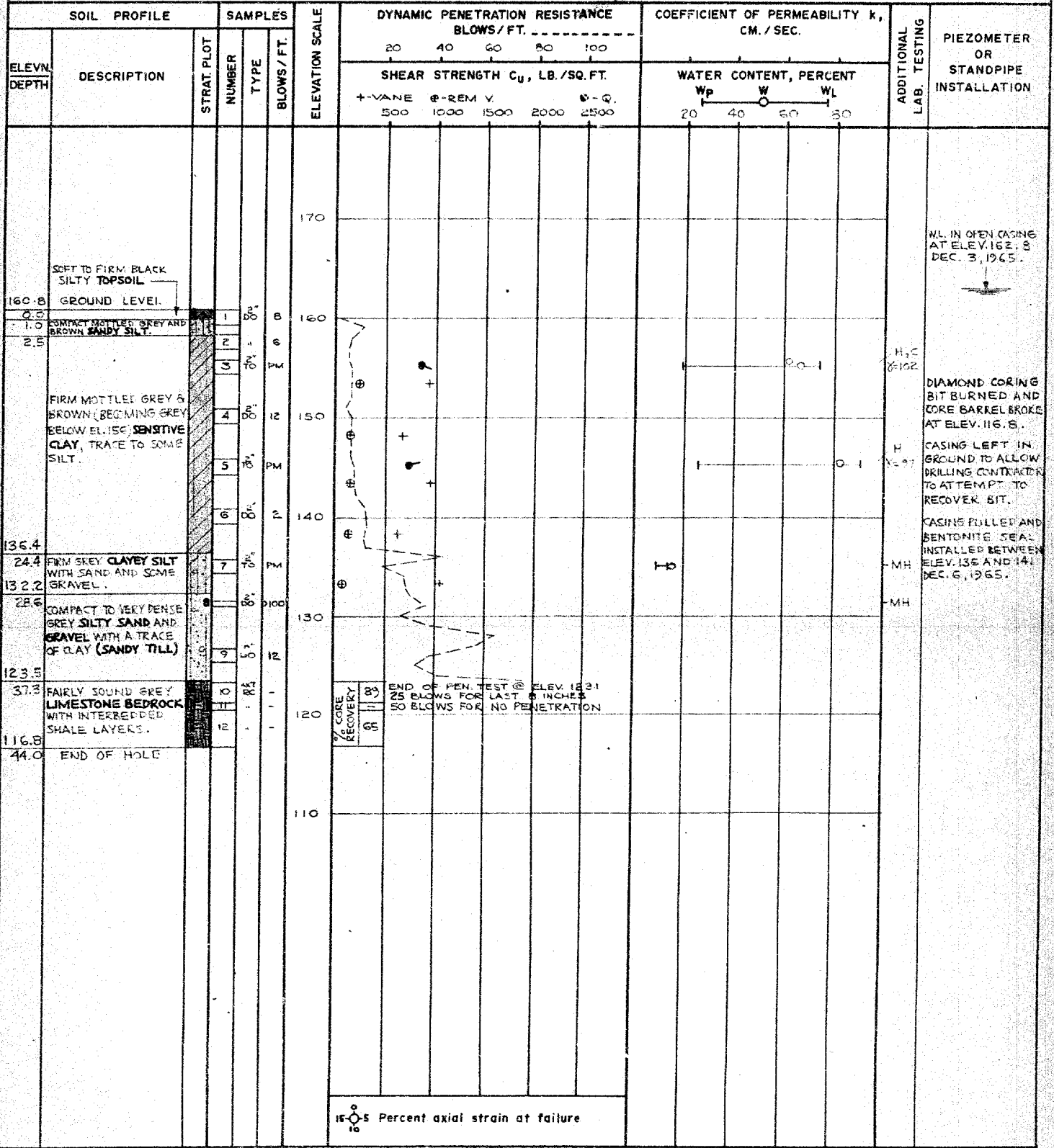
*For the case of a saturated cohesive soil, $\phi_u = 0$ and the undrained shear strength $\tau_f = c_u$ is taken as half the undrained compressive strength.

RECORD OF BOREHOLE 1

LOCATION See Figure 1 BORING DATE NOV 10 - 12, 1965. 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



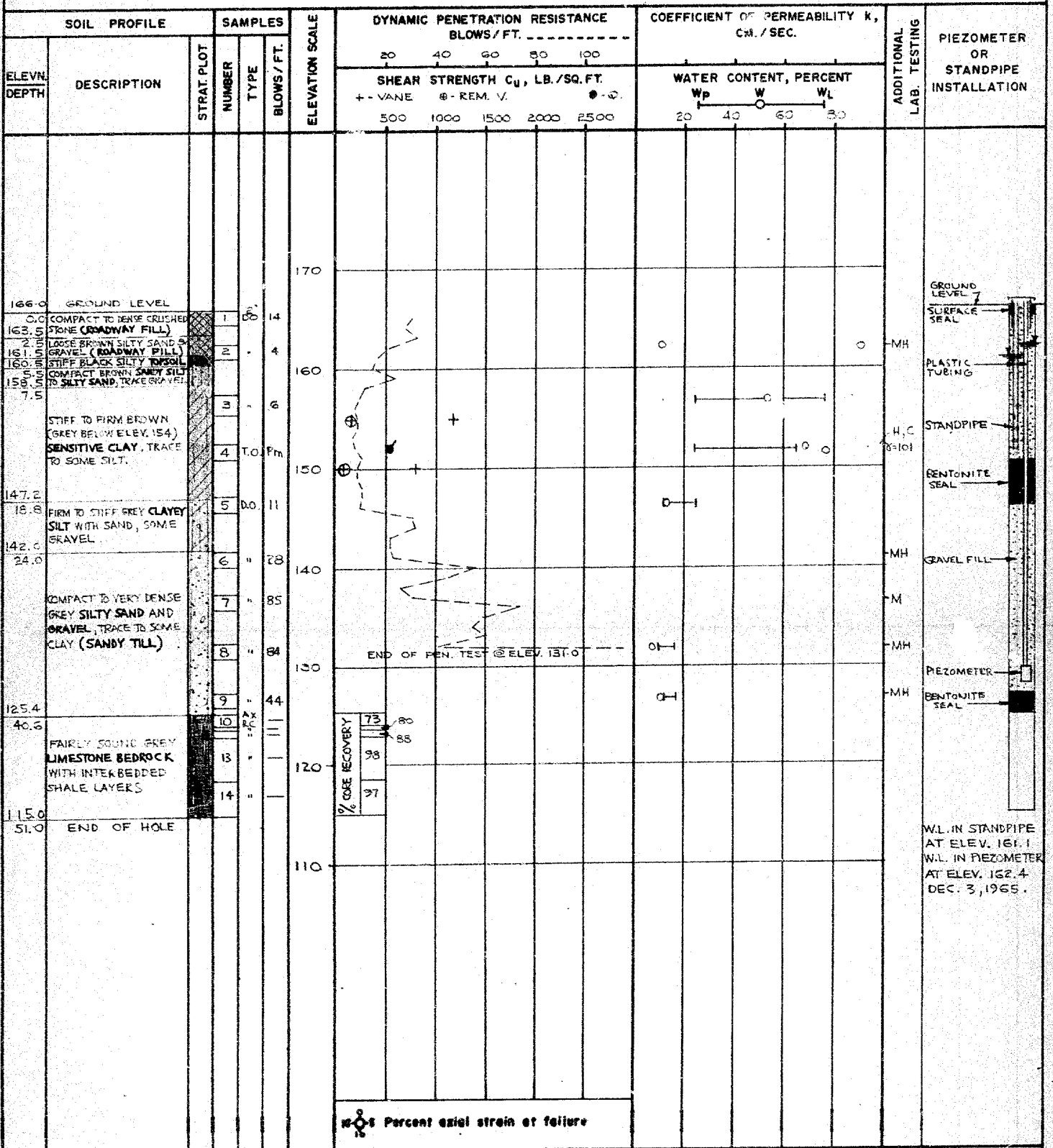
VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN R.H. 1/10
CHECKED JED

RECORD OF BOREHOLE 2

LOCATION See Figure 1 BORING DATE NOV. 13-19, 1945 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



VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN BY *RAH*
CHECKED *ADD*

RECORD OF BOREHOLE 3

LOCATION See Figure 1

BORING DATE DEC. 1-2, 1965

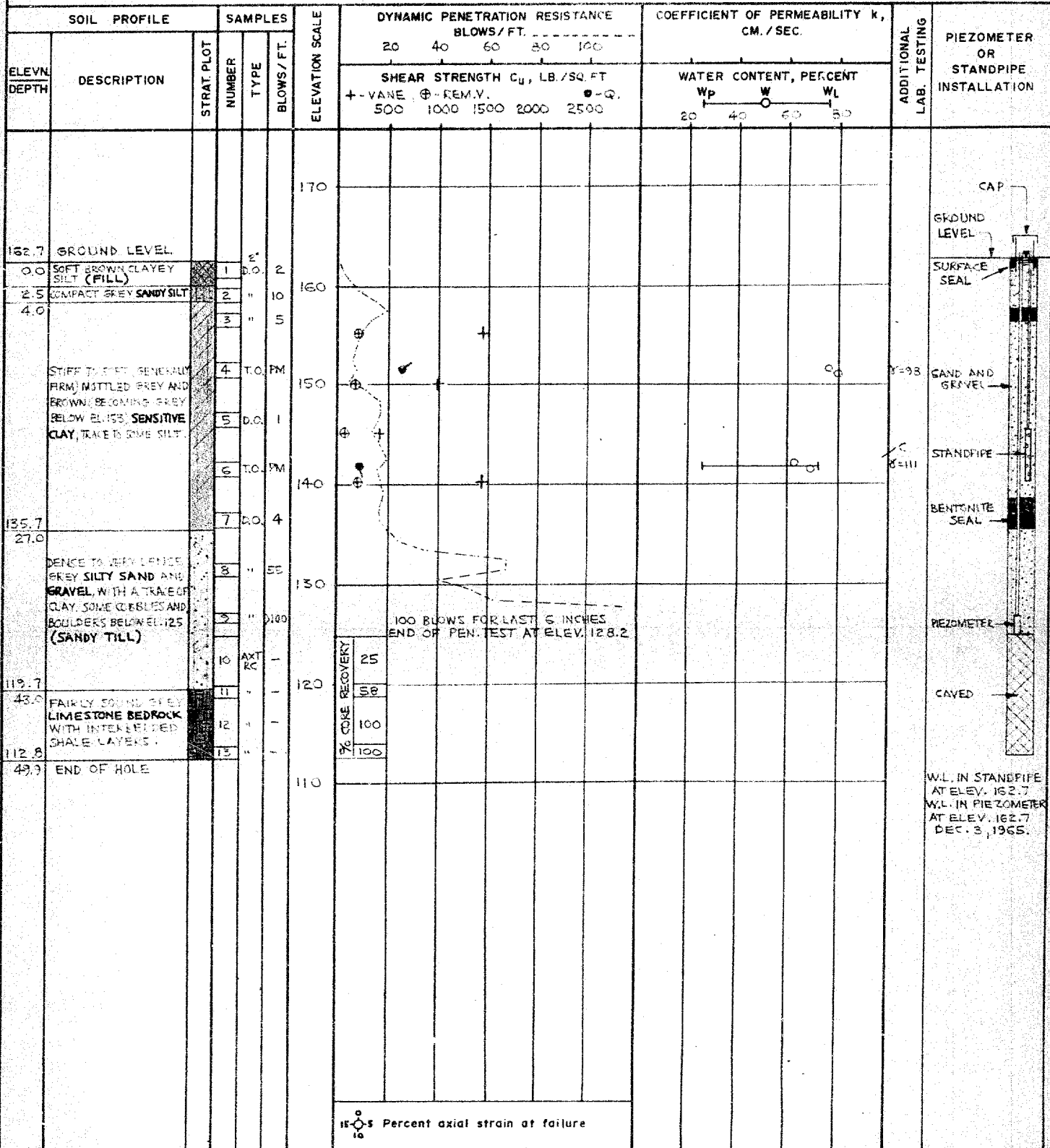
DATUM GEODETIC

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER NX, AX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



GOLDER & ASSOCIATES

DRAWN m.w.

CHECKED J.D.

RECORD OF BOREHOLE 4

LOCATION See Figure 1

BORING DATE NOV. 19-24, 1965

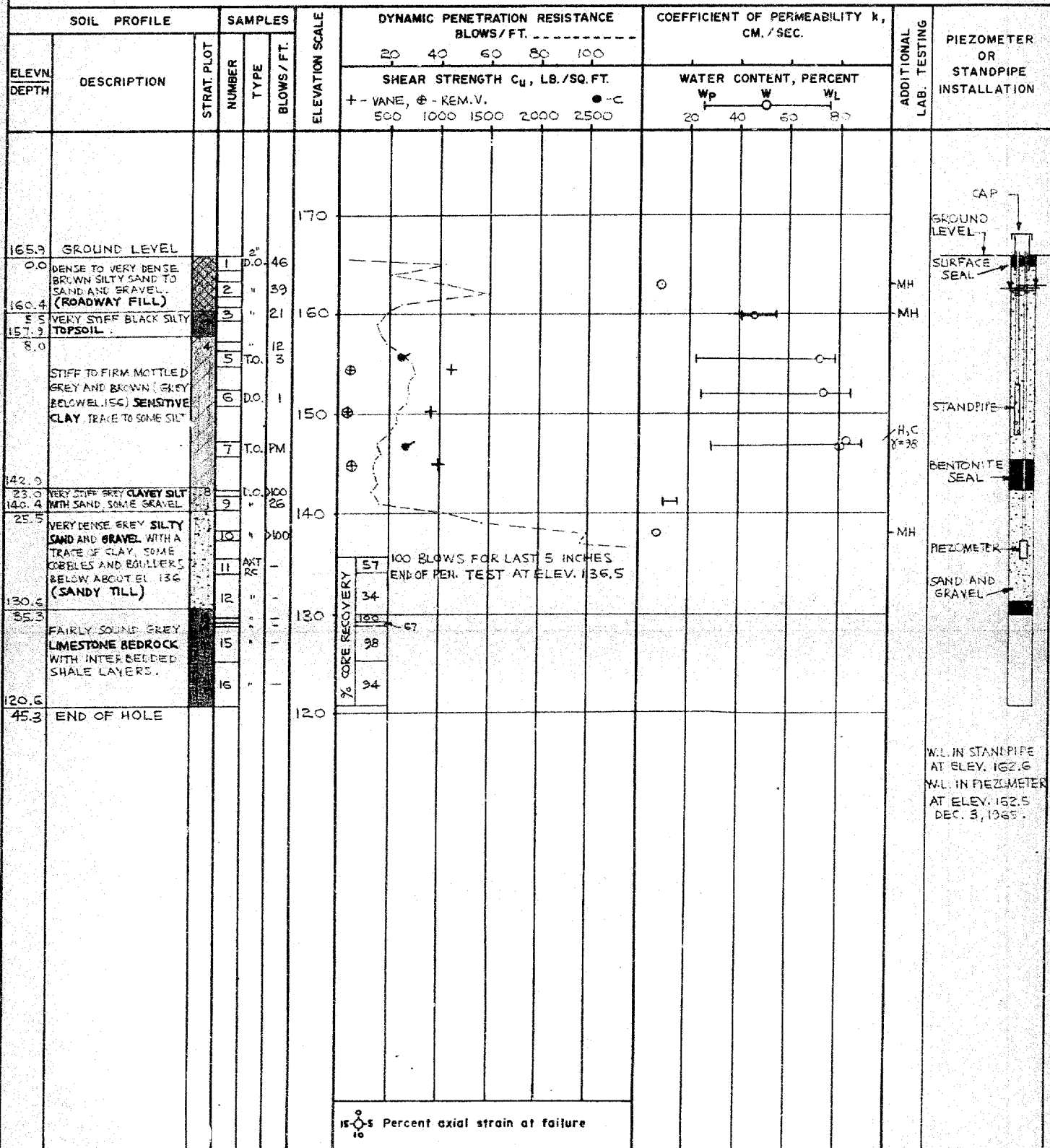
DATUM GEODETIC

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



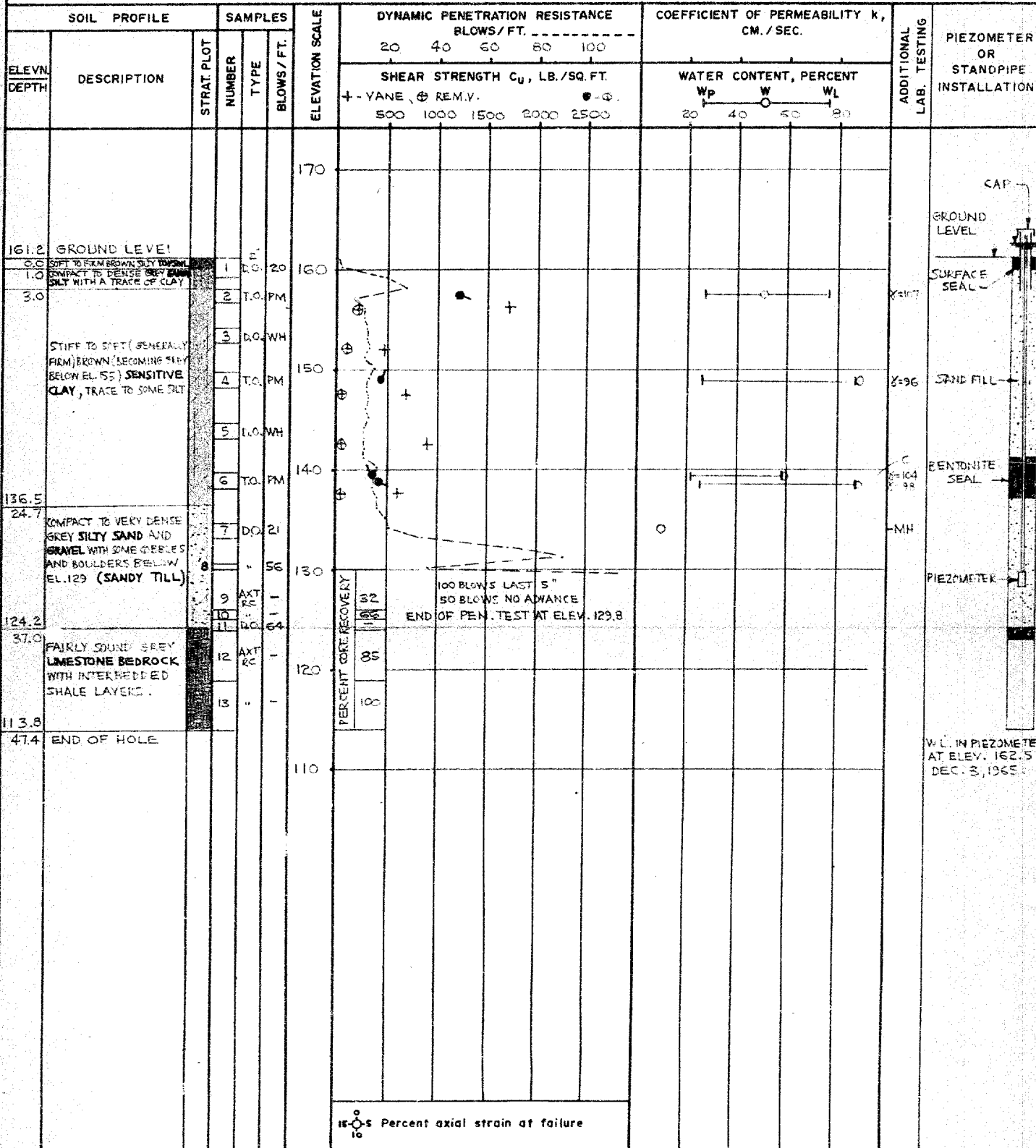
VERTICAL SCALE,
1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

DRAWN *MAS*
CHECKED *MAS*

RECORD OF BOREHOLE 5

LOCATION See Figure 1 BORING DATE NOV. 25 - 29, 1965 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



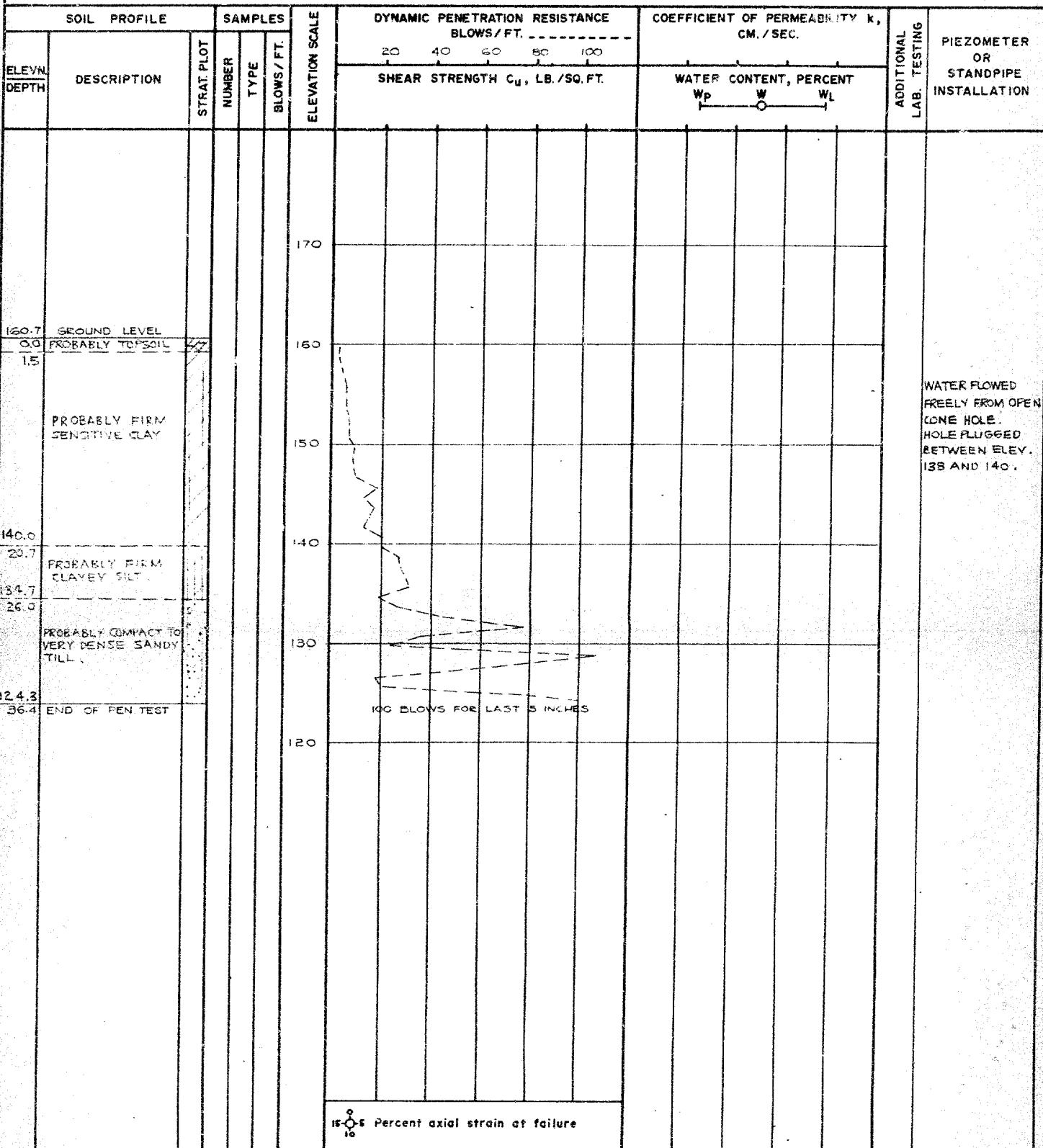
VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN *W.D.*
CHECKED *J.B.D.*

PEN TEST RECORD OF BOREHOLE 6

LOCATION **See Figure 1** BORING DATE **NOV. 3, 1965** DATUM **GEODETIC**
 BOREHOLE TYPE **PENETRATION TEST** BOREHOLE DIAMETER **-**
 SAMPLER HAMMER WEIGHT **- LB.** DROP **- INCHES** PEN. TEST HAMMER WEIGHT **140 LB.** DROP **30 INCHES**



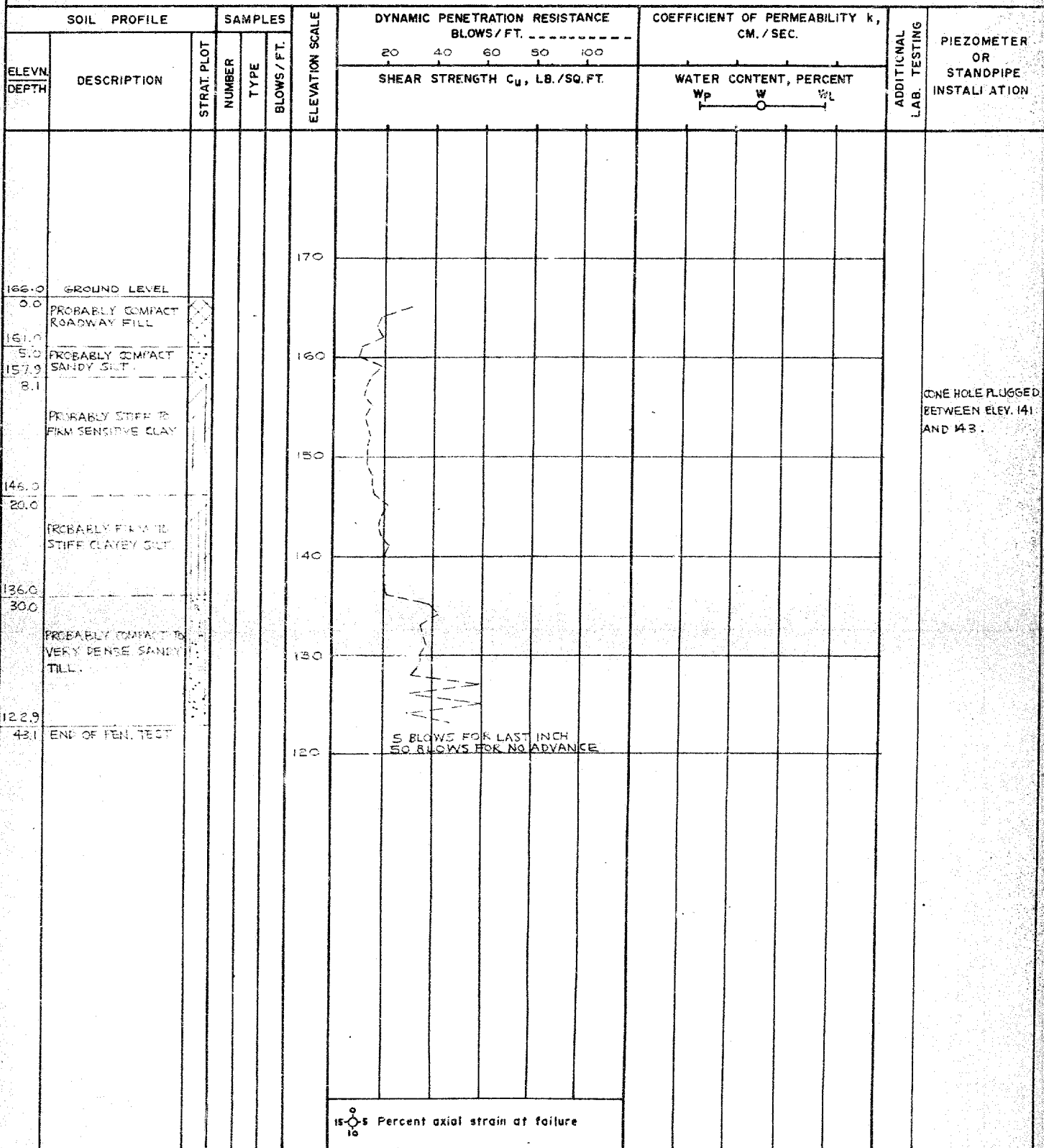
VERTICAL SCALE
 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN RH
 CHECKED JB

PEN TEST RECORD OF BOREHOLE 7

LOCATION **See Figure 1** BORING DATE **NOV. 12, 1965** DATUM **GEODETIC**
 BOREHOLE TYPE **PENETRATION TEST** BOREHOLE DIAMETER **-**
 SAMPLER HAMMER WEIGHT - **LB.** DROP - **INCHES** PEN. TEST HAMMER WEIGHT **140 LB.** DROP **30 INCHES**



ONE HOLE PLUGGED
BETWEEN ELEV. 141
AND 143.

VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN RL
CHECKED TS

PEN. TEST RECORD OF BOREHOLE 8

LOCATION See Figure 1

BORING DATE DEC. 1, 1965.

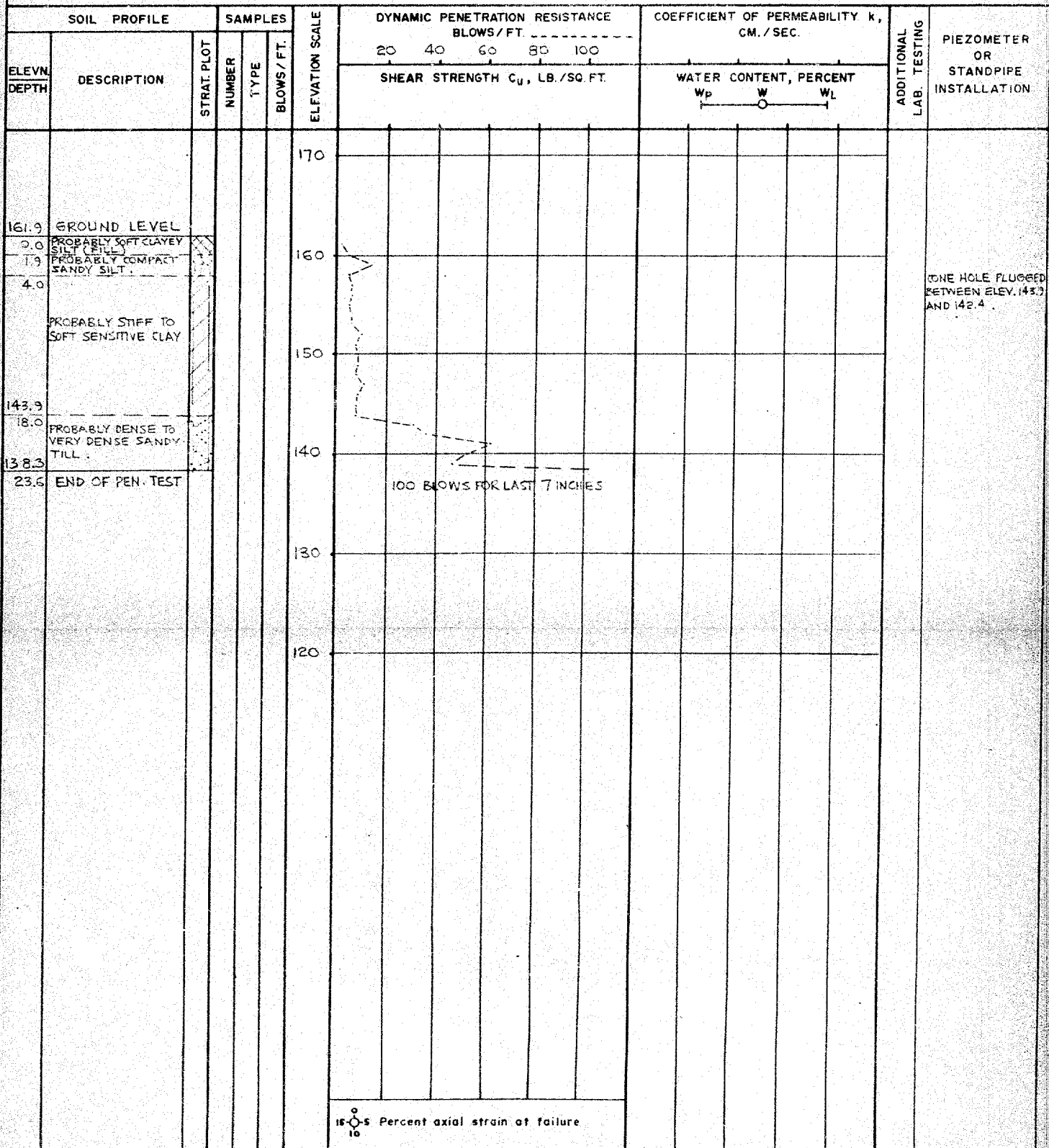
DATUM GEODETIC

BOREHOLE TYPE PENETRATION TEST

BOREHOLE DIAMETER -

SAMPLER HAMMER WEIGHT - LB. DROP - INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES


 BOREHOLE PLUGGED
BETWEEN ELEV. 143.9
AND 142.4

 VERTICAL SCALE
1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

 DRAWN *[Signature]*
CHECKED *[Signature]*

PROJECT NO. 44-254-1324

GEODETIC

BOREHOLE DIAMETER

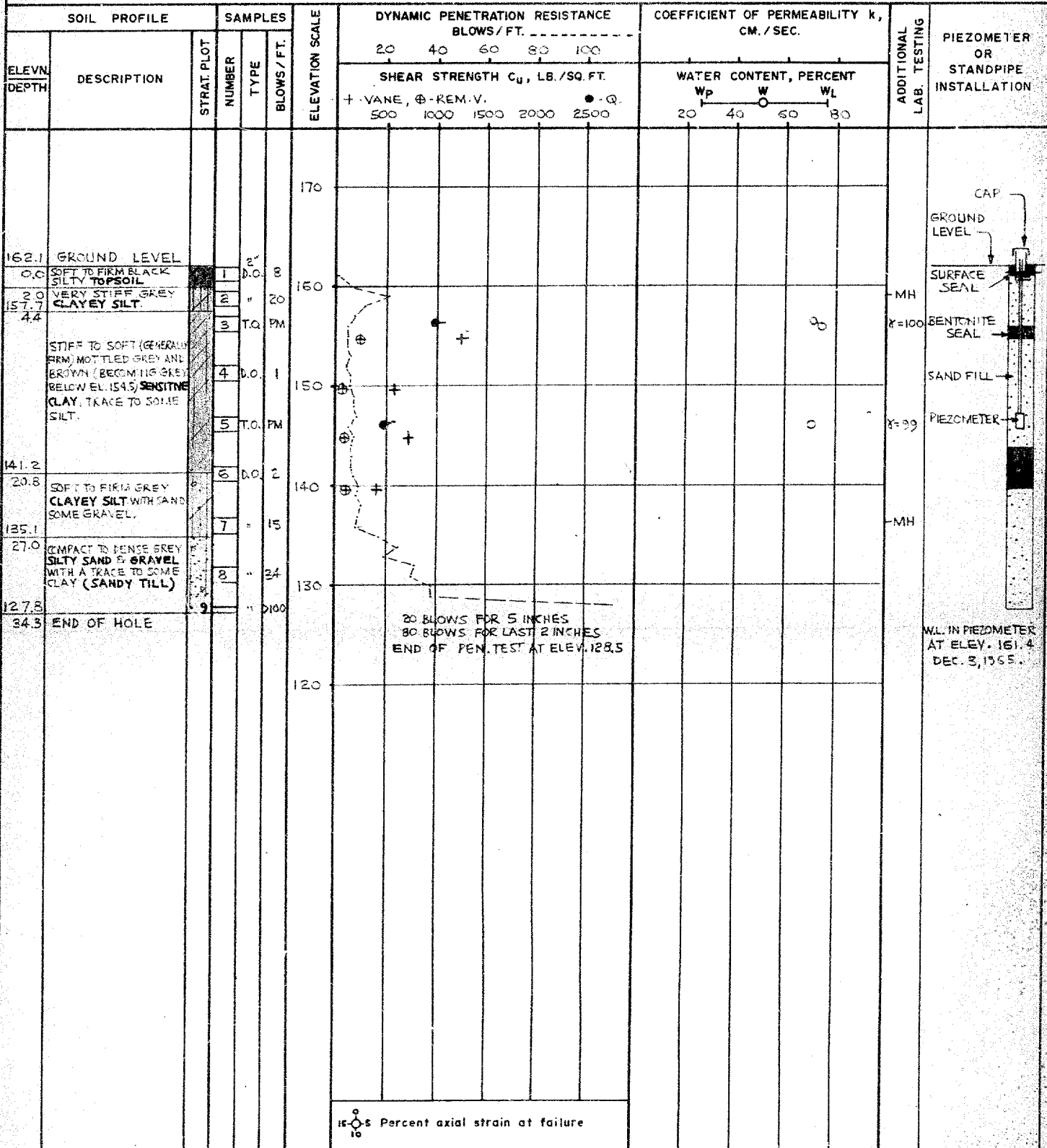
PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

ONE HOLE PLUGGED
BETWEEN ELEV. 137.9
AND 136.4.

DRAWN J.A.
CHECKED TS

RECORD OF BOREHOLE 11

LOCATION See Figure 1 BORING DATE NOV. 30-DEC. 1965 DATUM GEODETIC
 BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER NX CASING
 SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE
 1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

DRAWN M.W.
 CHECKED J.B.

RECORD OF BOREHOLE 12

LOCATION See Figure 1

BORING DATE DEC. 3, 1965

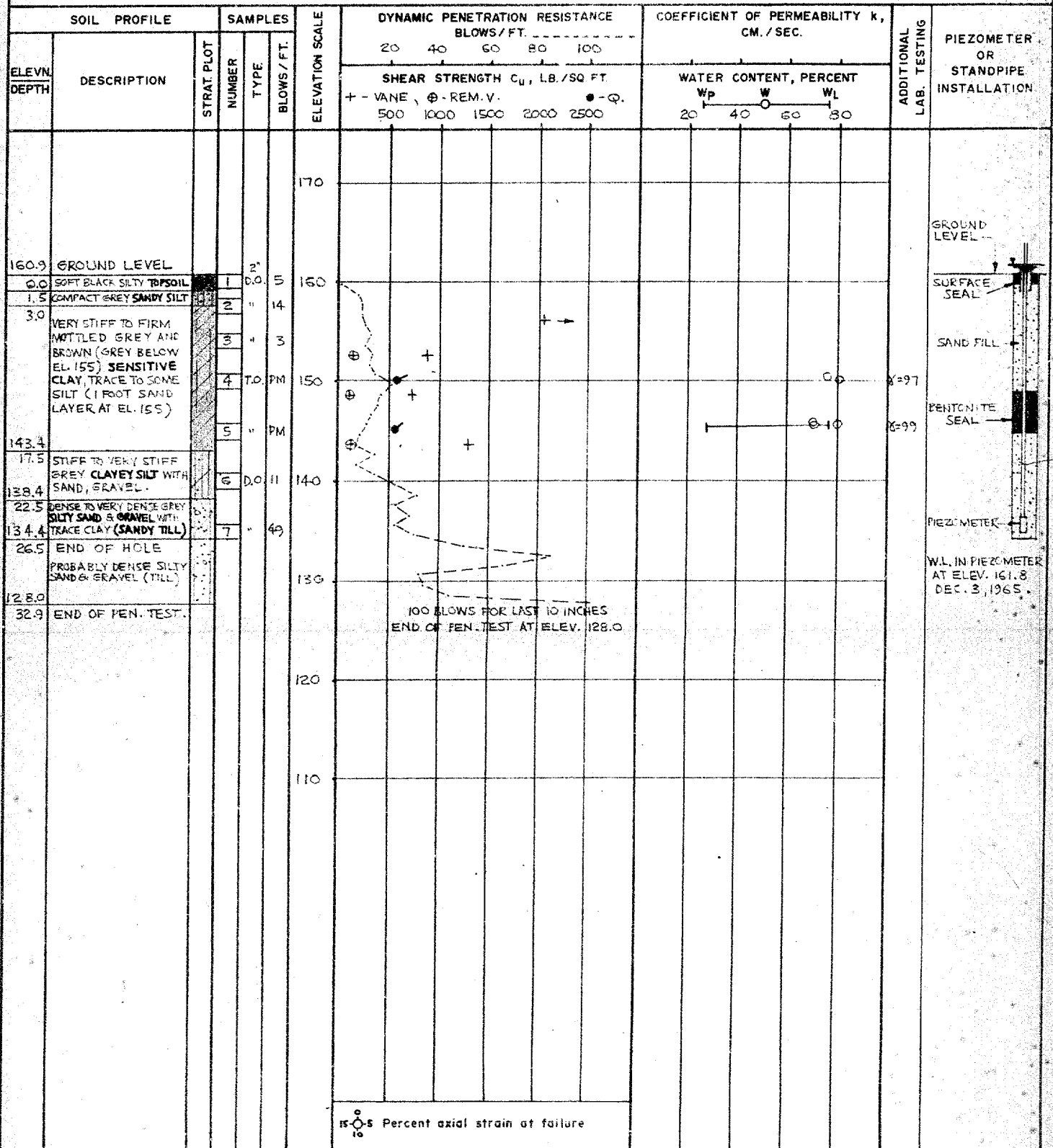
DATUM GEODETIC

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER NX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE

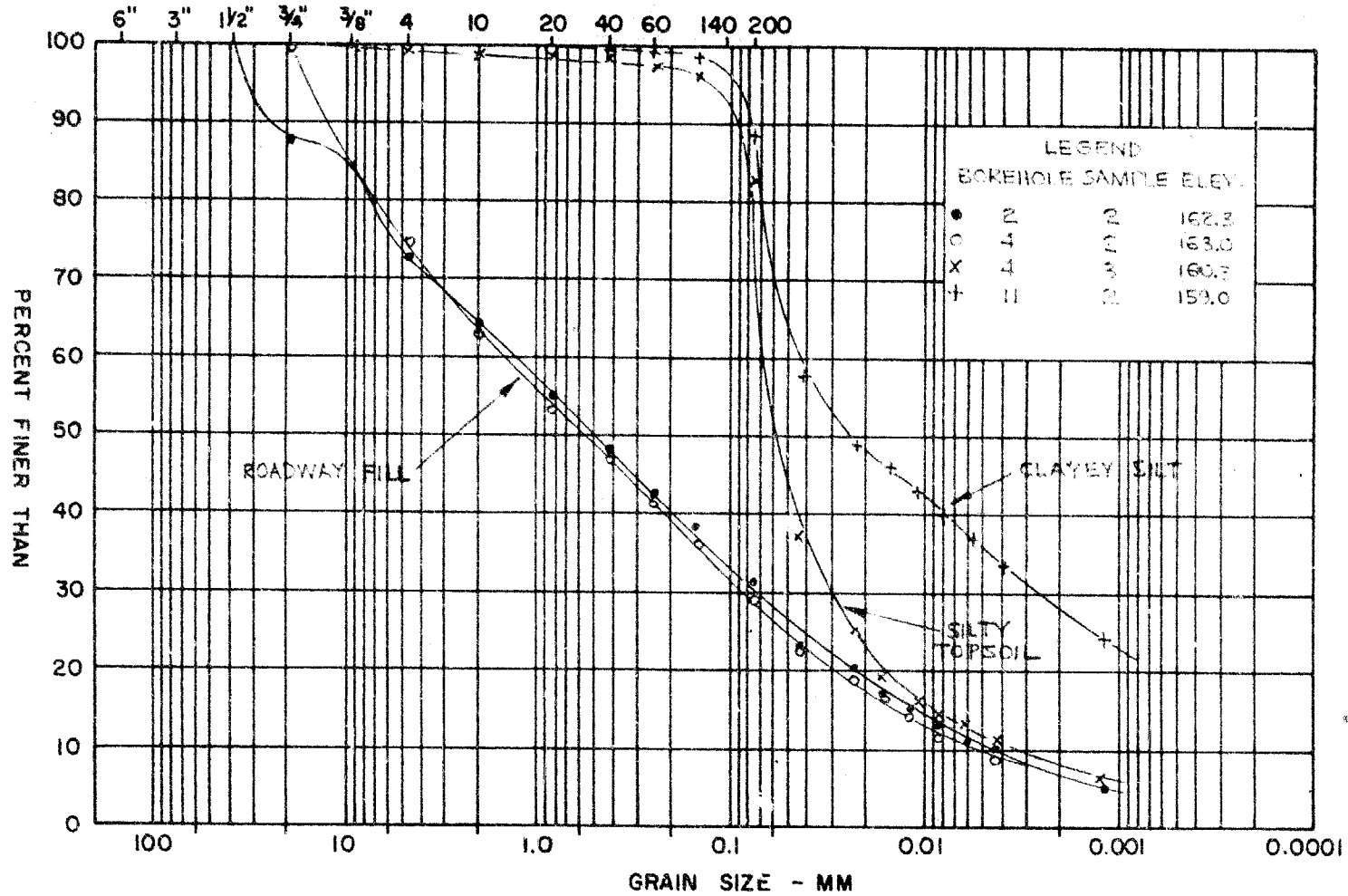
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN *W.W.*CHECKED *T.S.*

M.I.T. GRAIN SIZE SCALE

SIZE OF OPENING - INS. U.S.S. SIEVE SIZE - MESHES/IN.



COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE		CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED		

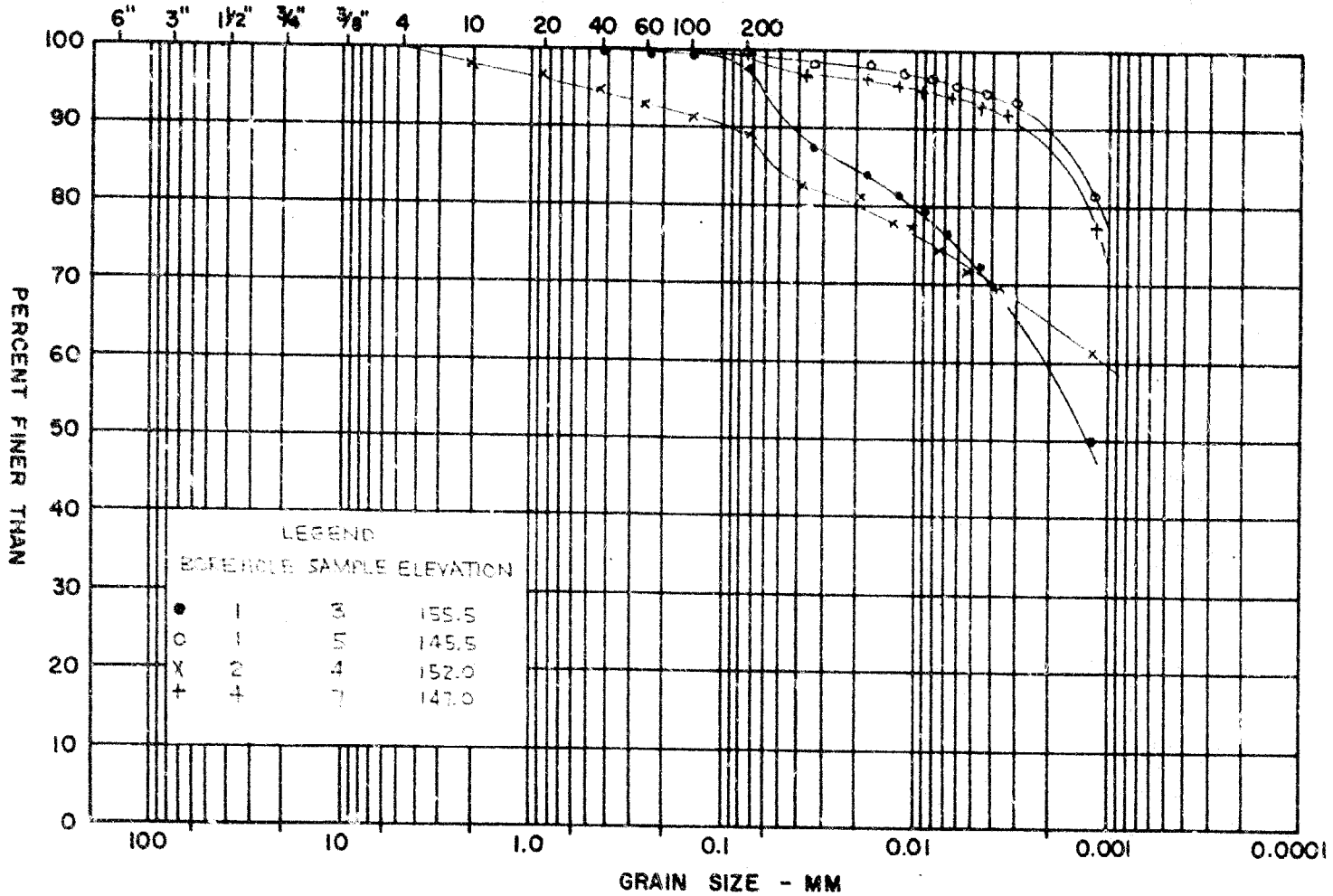
GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION
SEDIMENTAL DEPOSITS

FIGURE 2

M.I.T. GRAIN SIZE SCALE

SIZE OF OPENING - INS. U.S.S. SIEVE SIZE - MESHES/IN.



GOLDER & ASSOCIATES

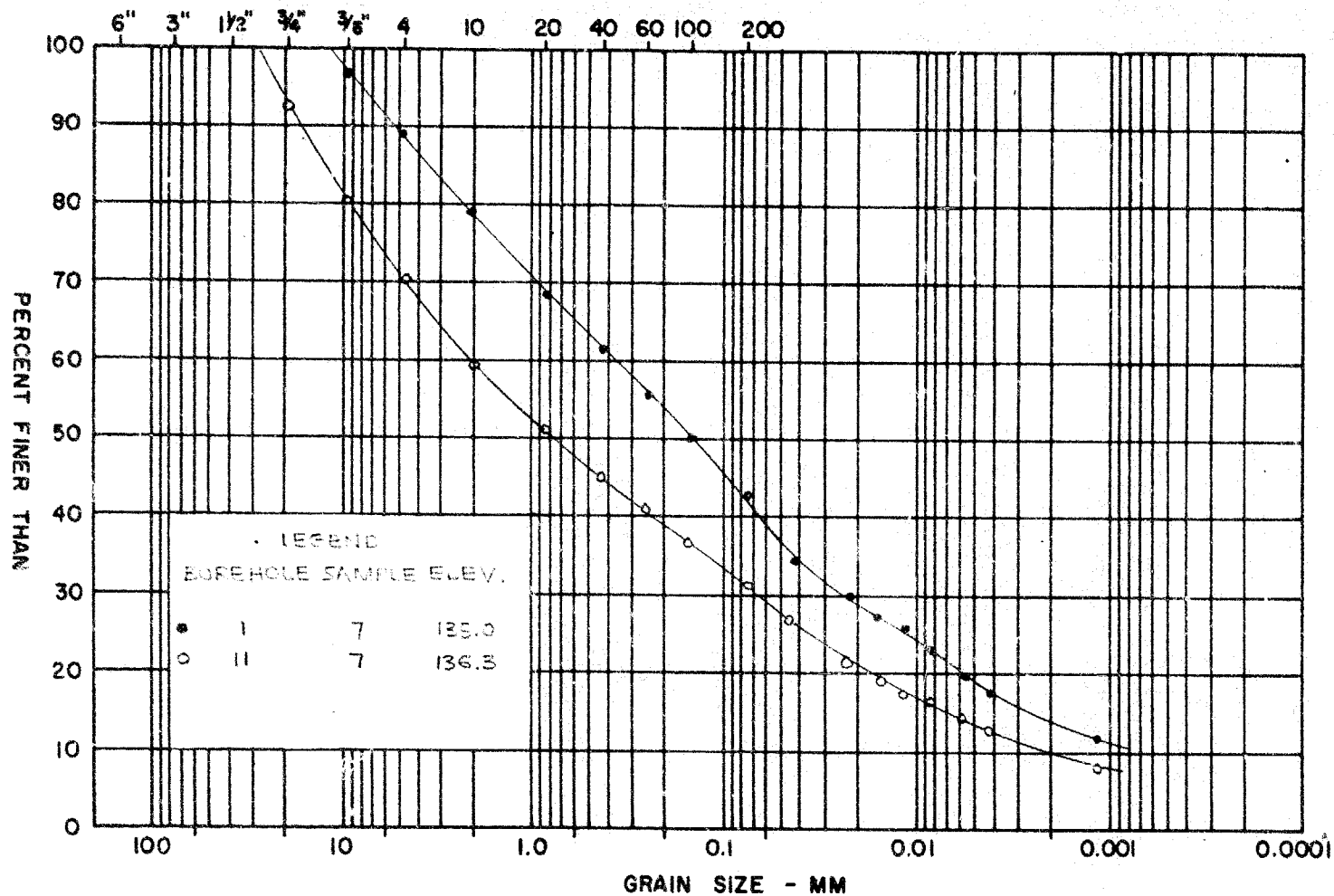
GRAIN SIZE DISTRIBUTION
SENSITIVE CLAY STRATUM

FIGURE

(1)

M.I.T. GRAIN SIZE SCALE

SIZE OF OPENING - INS. U.S.S. SIEVE SIZE - MESHES / IN.



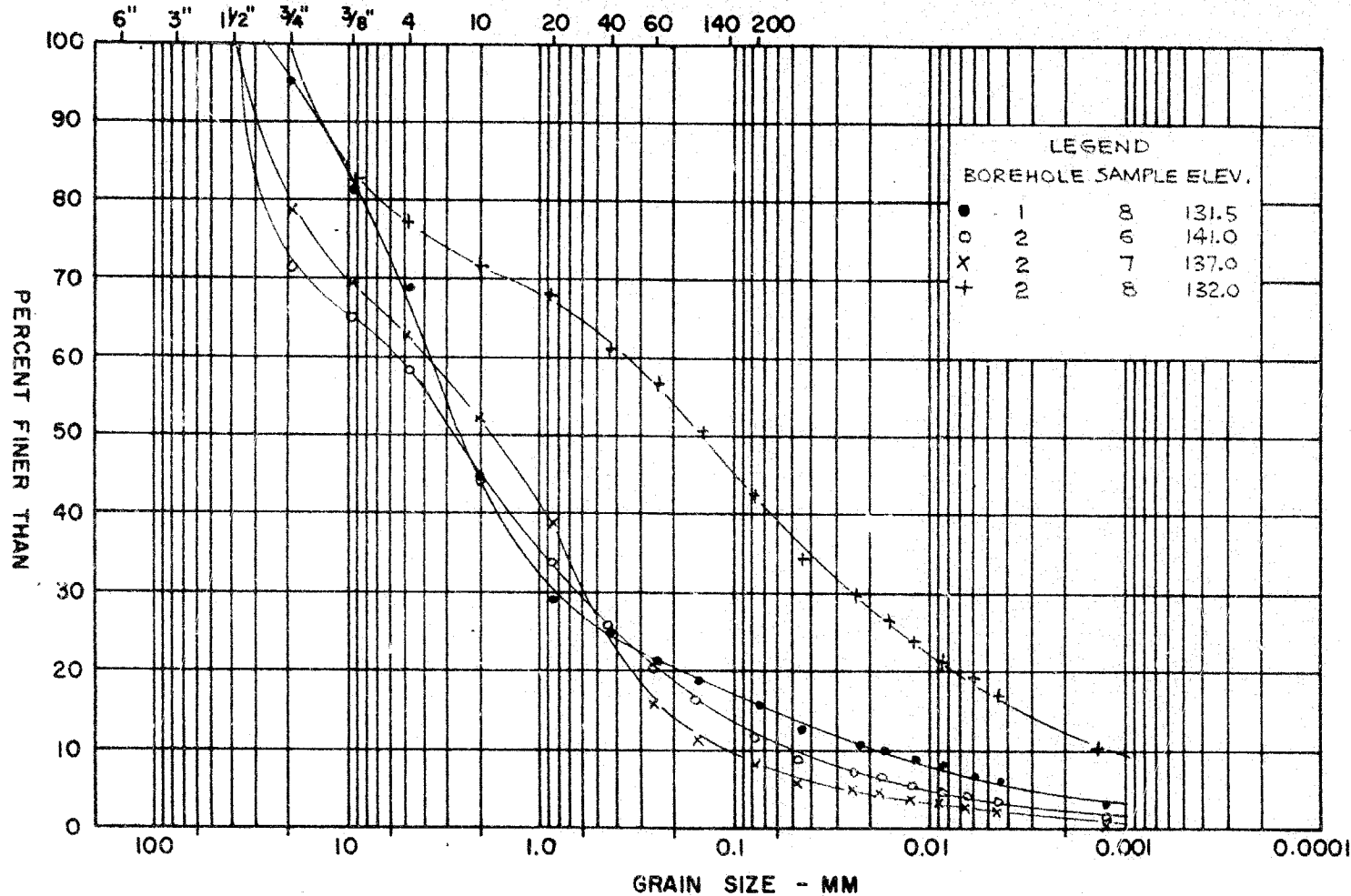
COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE		CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED		

GRAIN SIZE DISTRIBUTION
CLAYEY SILT WITH SAND STRATUM

FIGURE

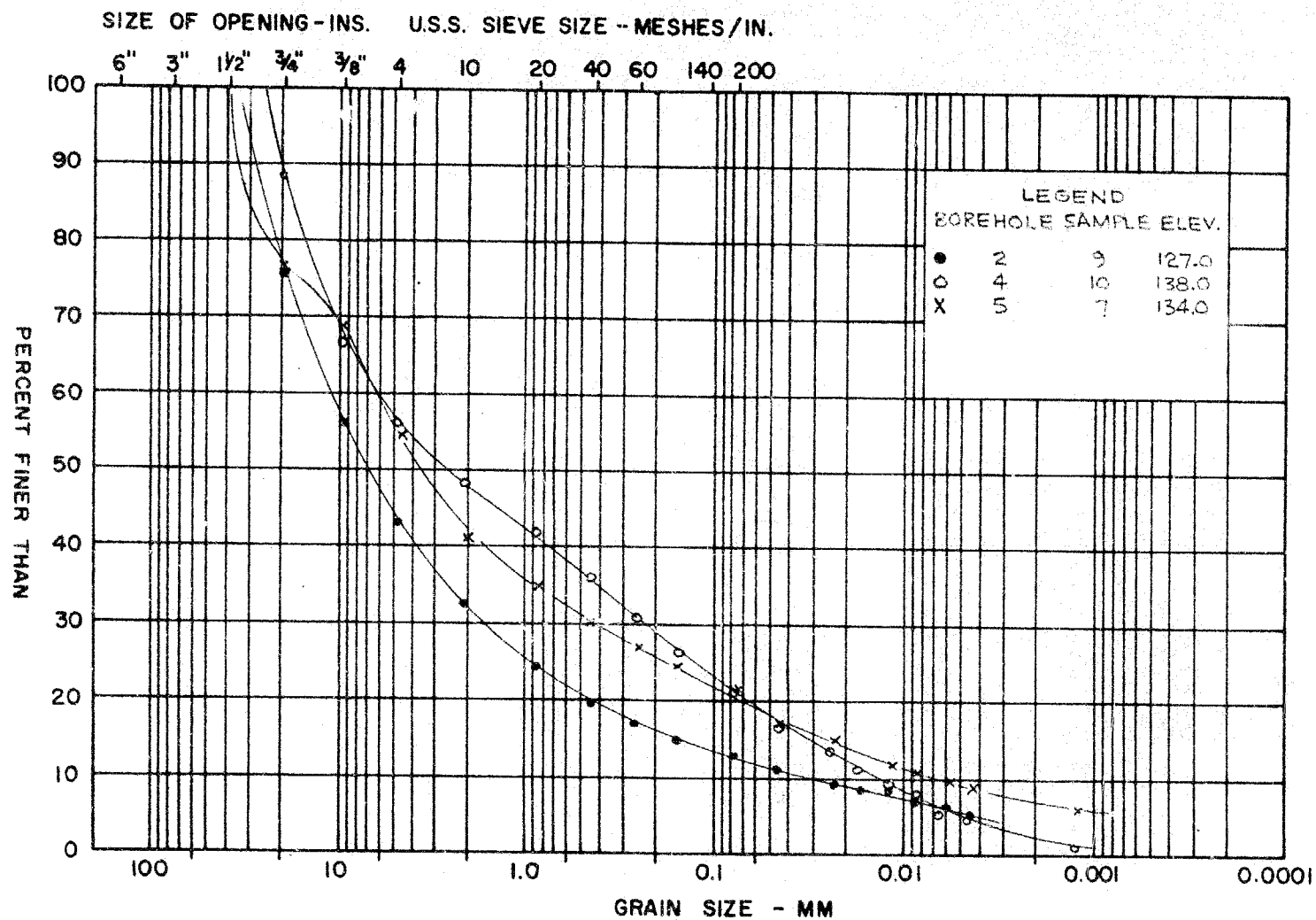
M.I.T. GRAIN SIZE SCALE

SIZE OF OPENING-INS. U.S.S. SIEVE SIZE - MESHES/IN.



GRAIN SIZE DISTRIBUTION
TILL STRATUM

M.I.T. GRAIN SIZE SCALE



GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION
TILL STRATUM

FIGURE 6

SIMPLIFIED SOIL
STRATIGRAPHY

— AVERAGE GROUND SURFACE

LOOSE IN CONTACT
ROADWAY FILL

SILT (MAYBE)

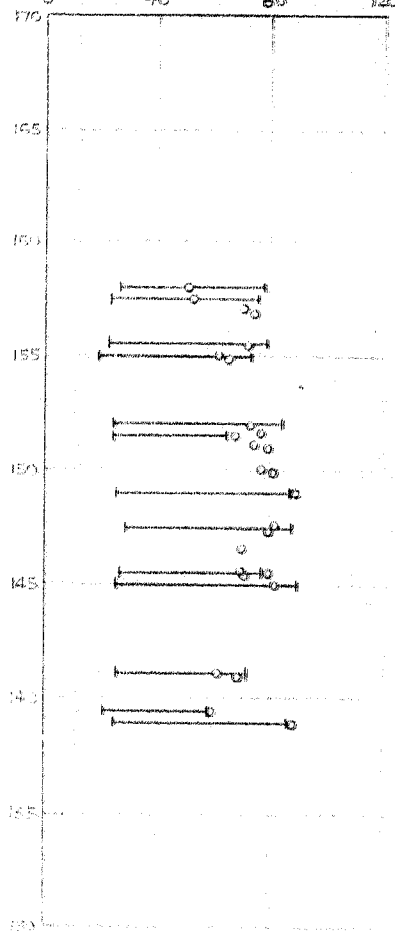
COMPACT SANDY SILT

FIRM
SENSITIVE
CLAY

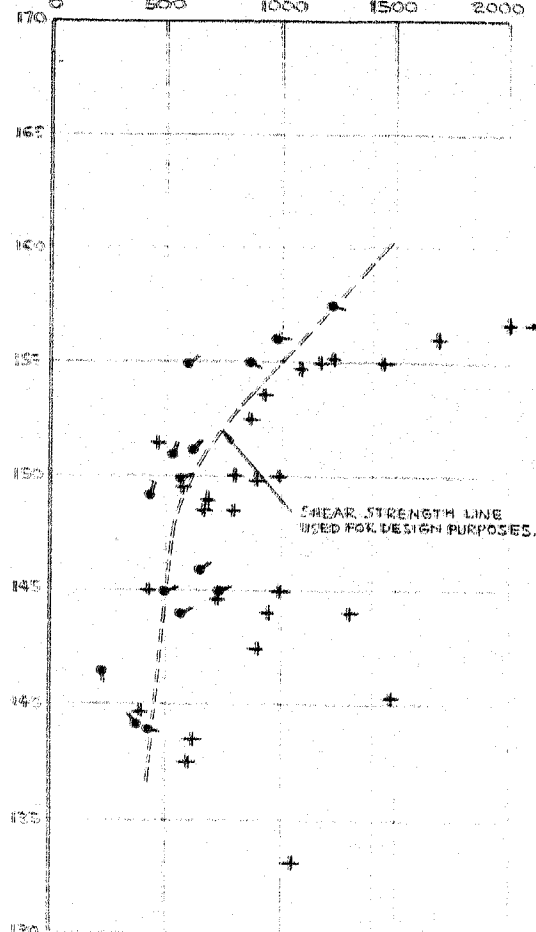
FIRM TO STIFF
SLAYER SILT
WITH SAND, COARSE GRAVEL

LEACH TANK

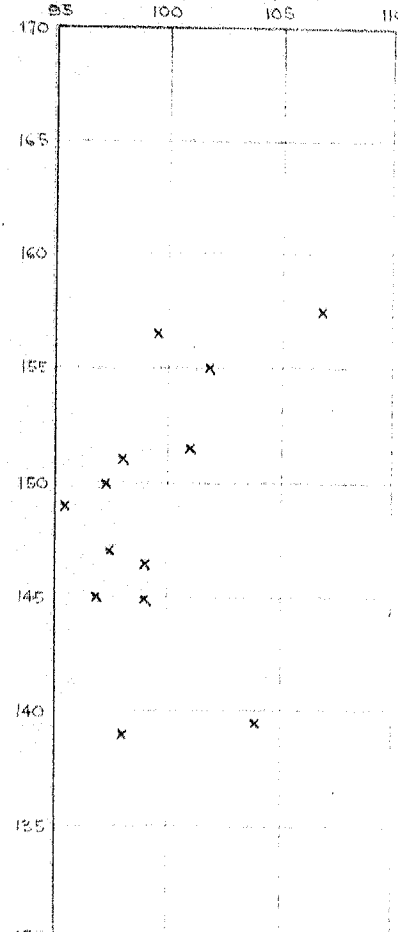
WATER CONTENT, PERCENT



UNDRAINED SHEAR STRENGTH, LB./SQ. FT.



TOTAL UNIT WEIGHT, LB./CU. FT.



W_p = PLASTIC LIMIT
W_L = LIQUID LIMIT
W = WATER CONTENT

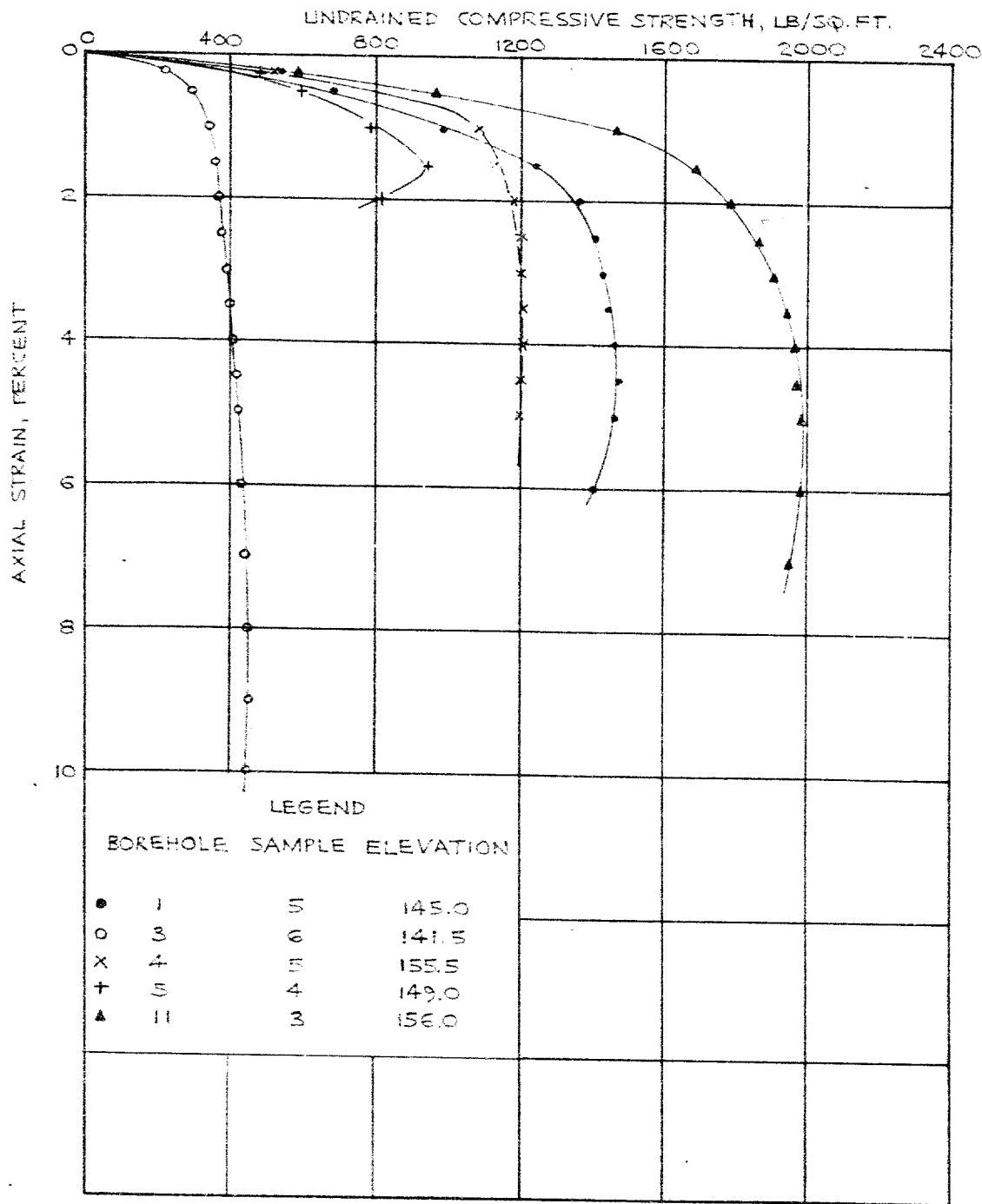
• - UNDRAINED TRIAXIAL COMPRESSION TEST
+ - IN SITU VANE SHEAR TEST
* - PERCENT AXIAL STRAIN AT FAILURE

GOLDER & ASSOCIATES

Made by
Chkd. by
Appd. by

UNDRAINED TRIAXIAL COMPRESSION TESTS TYPICAL STRESS- STRAIN CURVES SENSITIVE CLAY STRATUM

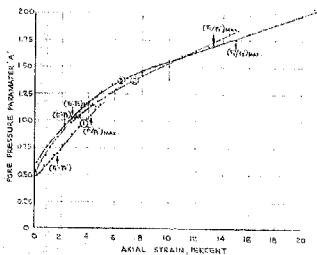
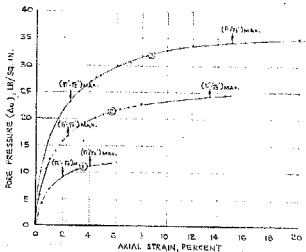
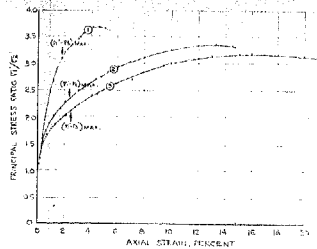
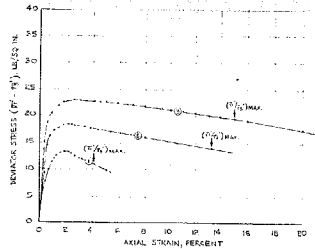
FIGURE 8



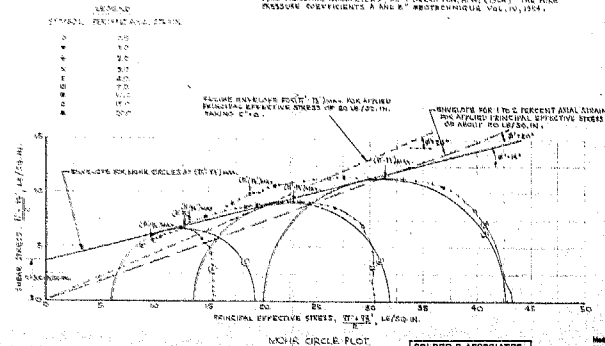
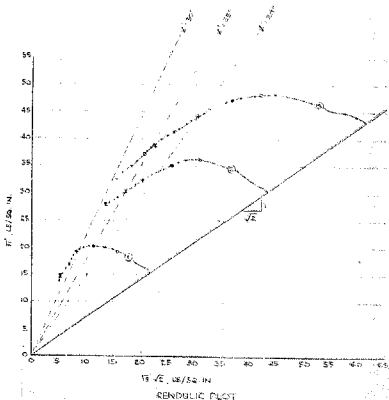
LEGEND					
TEST	HOLE	SAMPLE	ELEVATION	LIQUID LIMIT	PLASTIC LIMIT
1	2	4	151.6	65.0	24.1
2	1	3	144.2	83.0	25.7
3	4	1	146.4	88.0	18.7

TEST	CELL PRESSURE LB/SQ. IN.	AVERAGE RATIO OF STRAIN $(\sigma' - \sigma'_v)_{max} / \sigma'_v$	W ₁ %	W ₂ %	γ %
1	10.4	0.66	76	75	1.00
2	30.5	0.85	81	61	0.73
3	45.3	1.0	81	52	0.38

* PORE PRESSURE PARAMETERS, REF. SKEMPTON, A. W. (1954) "THE PORE PRESSURE COEFFICIENTS A AND B" GEOTECHNICAL VOL. IV, 1954.

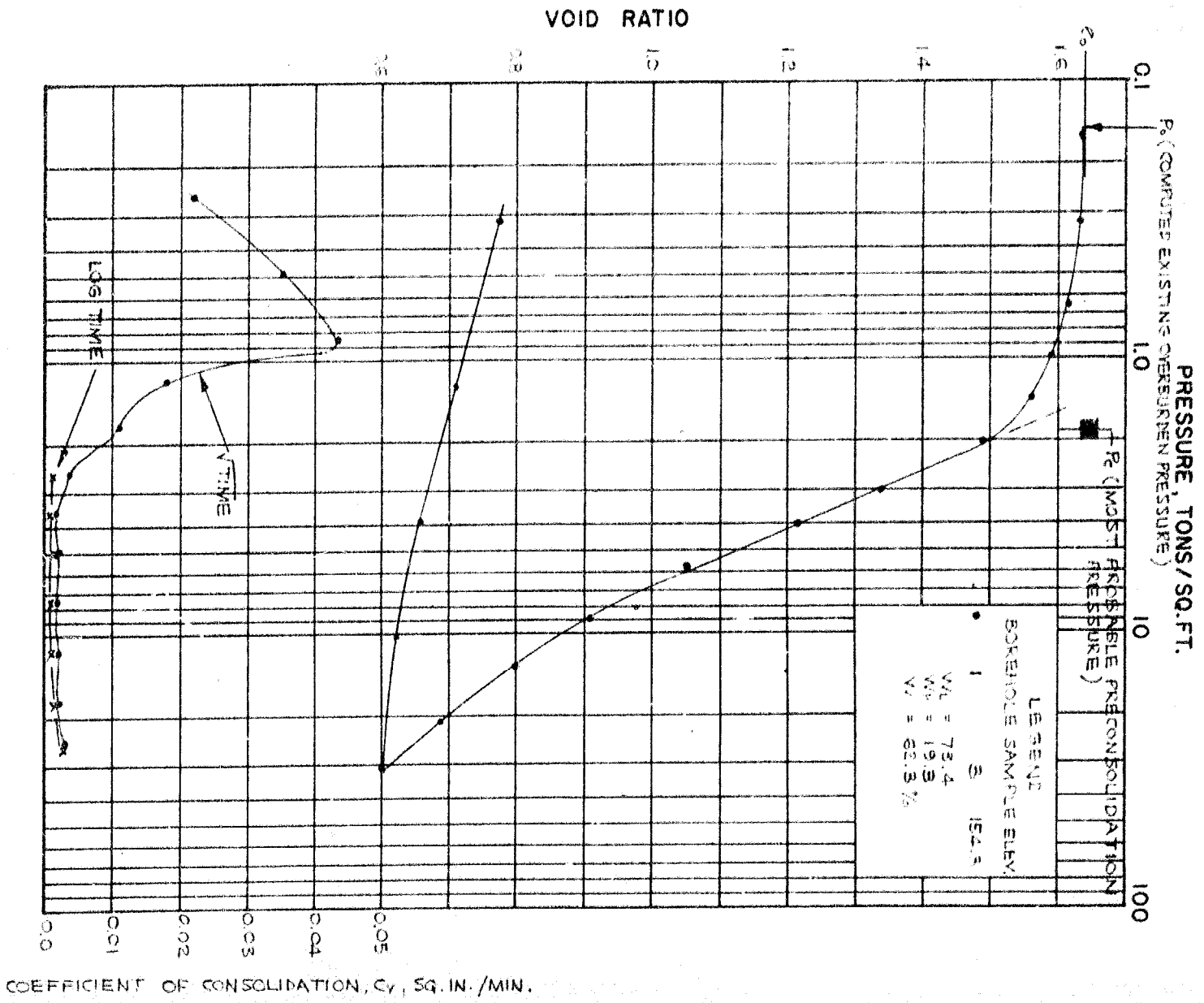


NOTE: PORE PRESSURE PARAMETER 'B' WAS FOUND TO BE BETWEEN ABOUT 0.35 AND 0.6.



VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

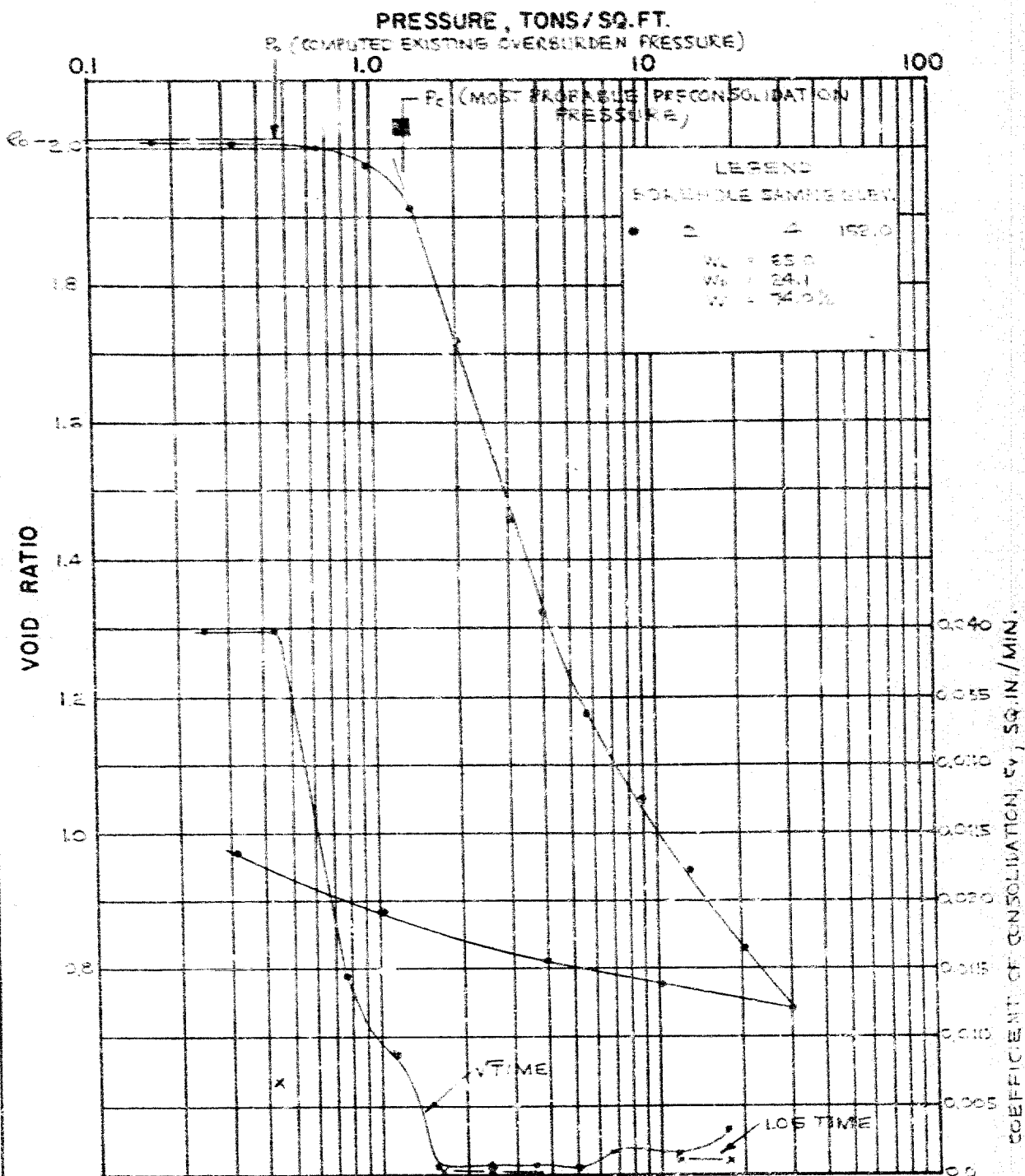
FIGURE 10



GOLDER & ASSOCIATES

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

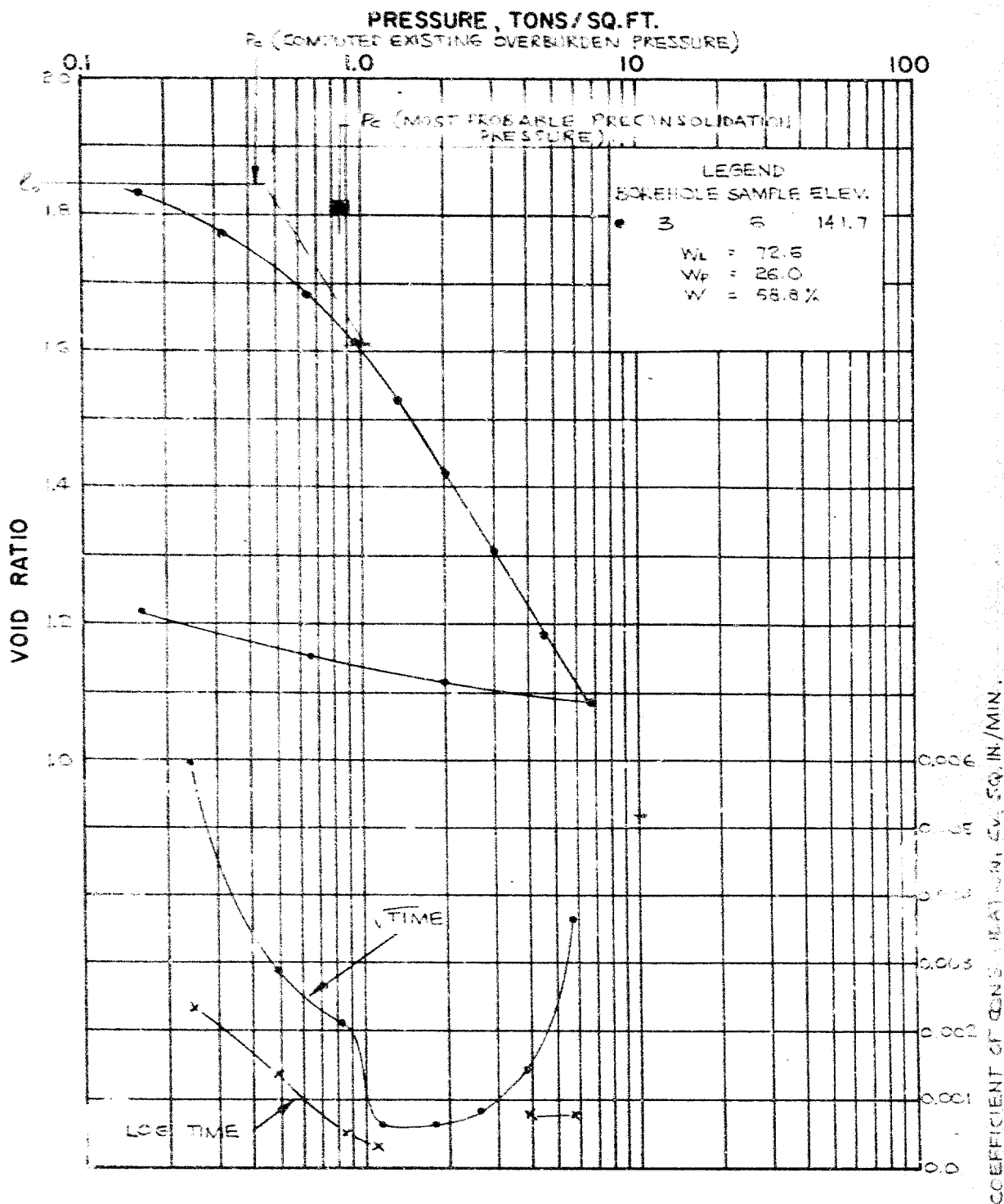
FIGURE 11



GOLDER & ASSOCIATES

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 12

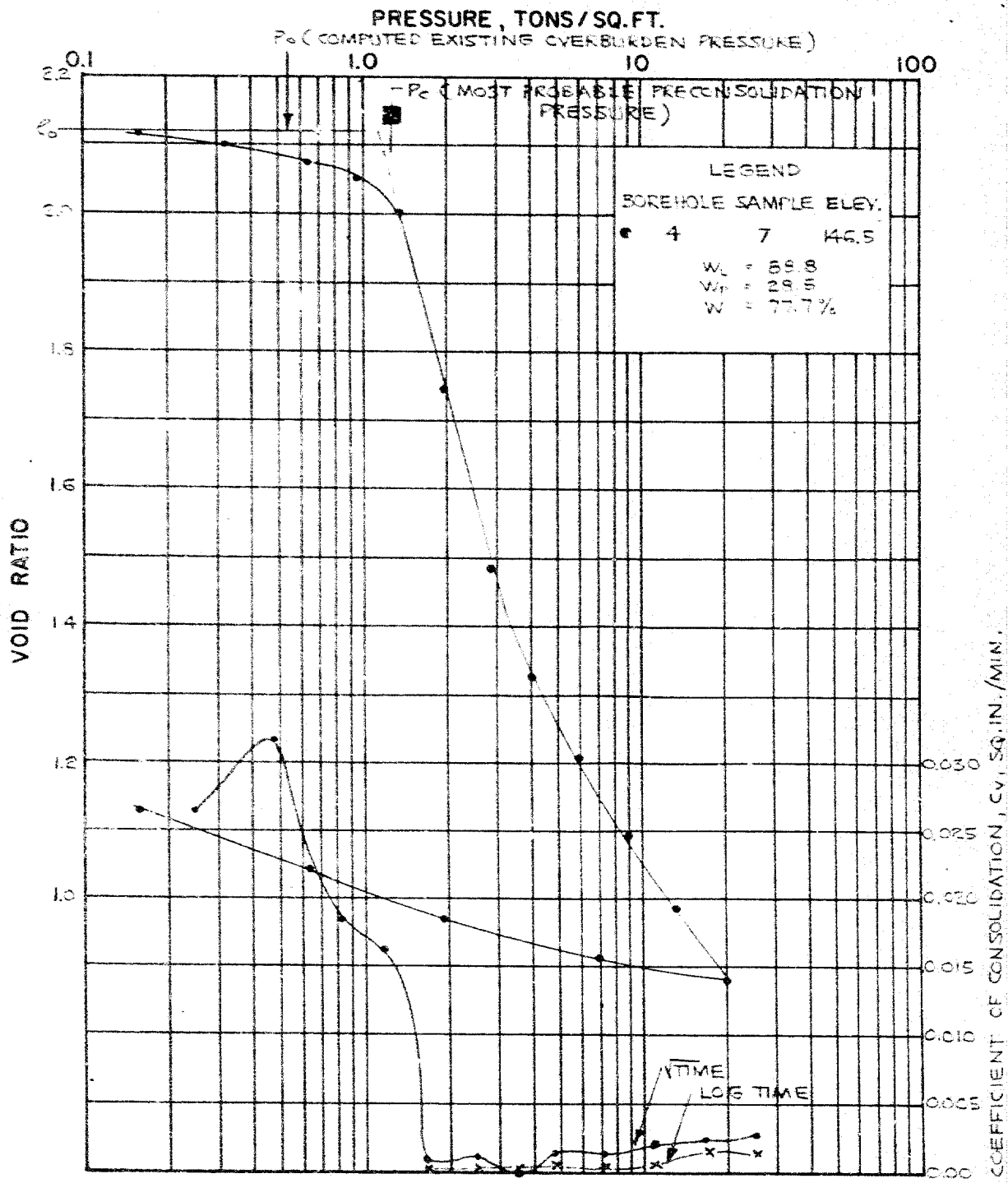


GOLDER & ASSOCIATES

PROJECT NO. 65/35

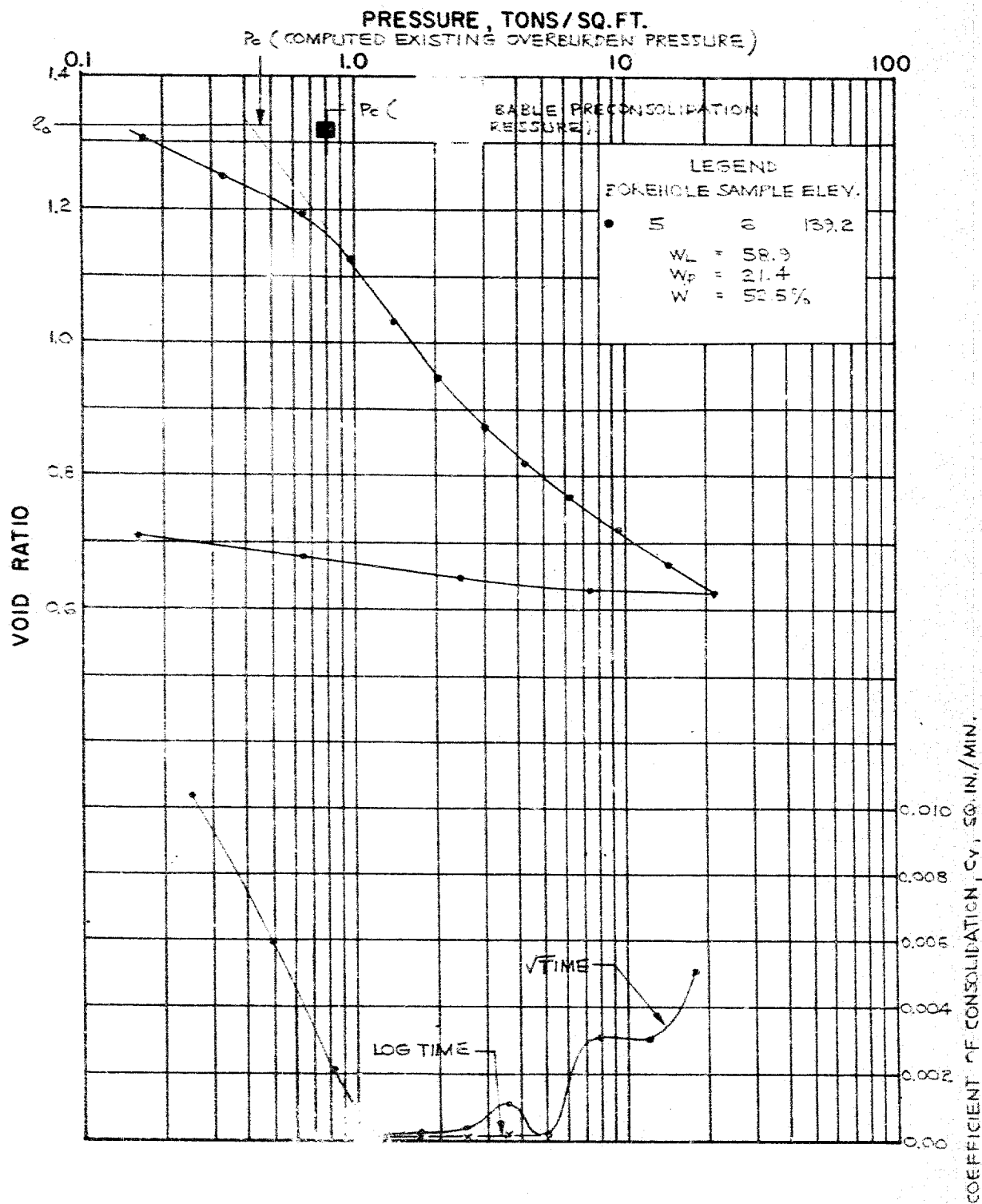
VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 13



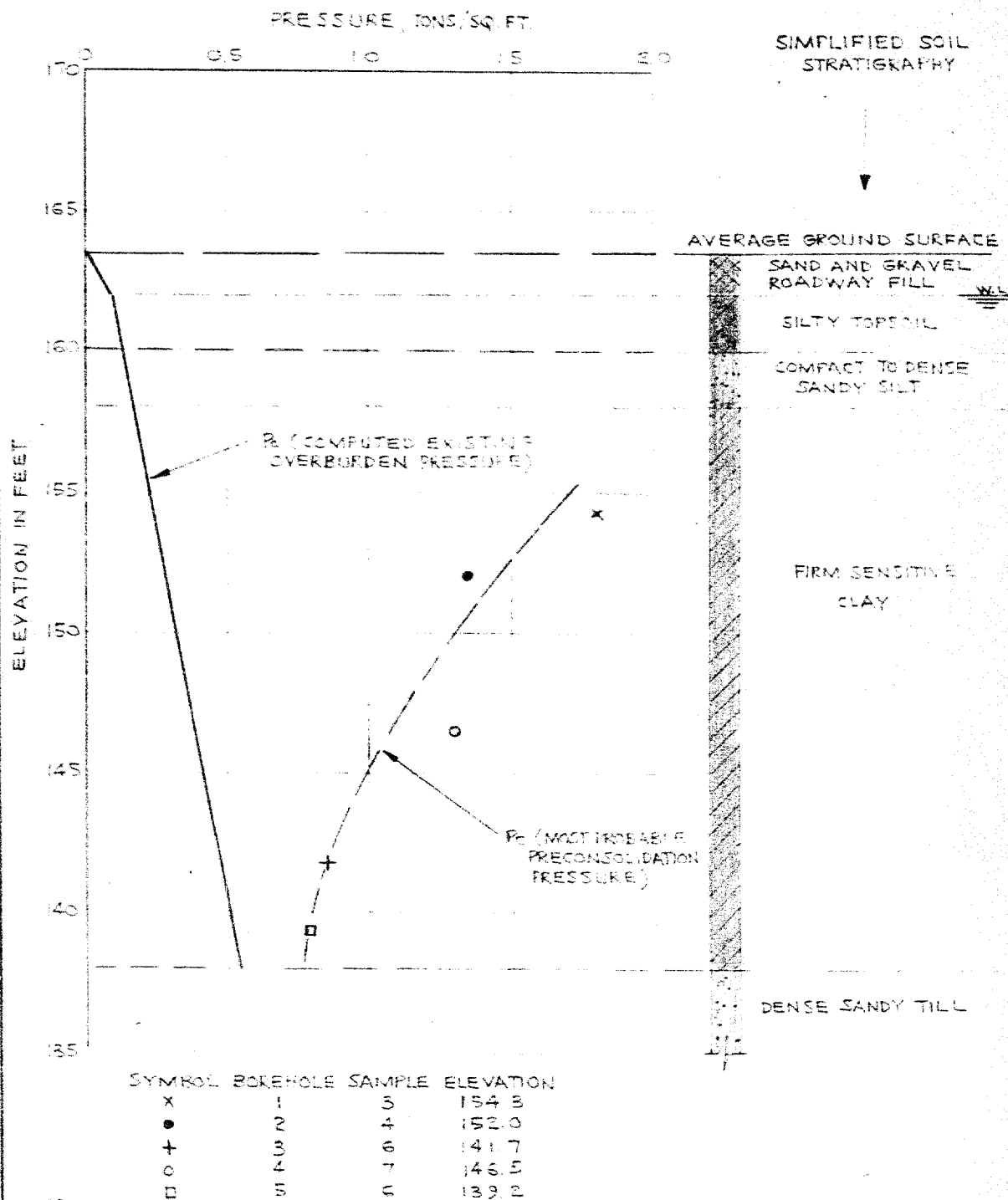
VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 14



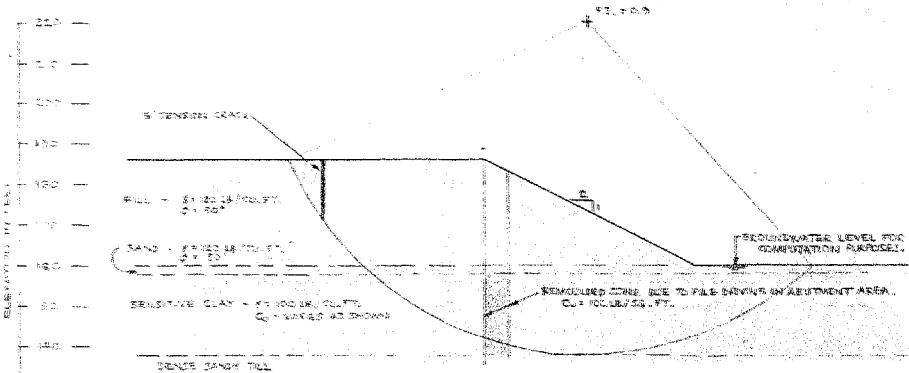
SUMMARY PLOT
PRECONSOLIDATION PRESSURE VS ELEVATION
SENSITIVE CLAY STRATUM

FIGURE 15

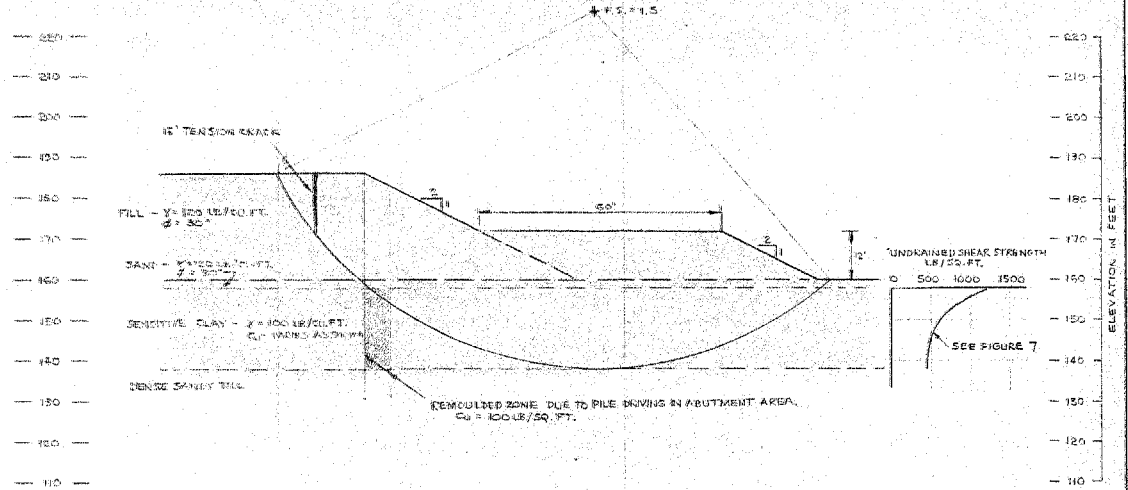


GOLDER & ASSOCIATES

Made *[Signature]*
Chkd. *[Signature]*
Appd. *[Signature]*

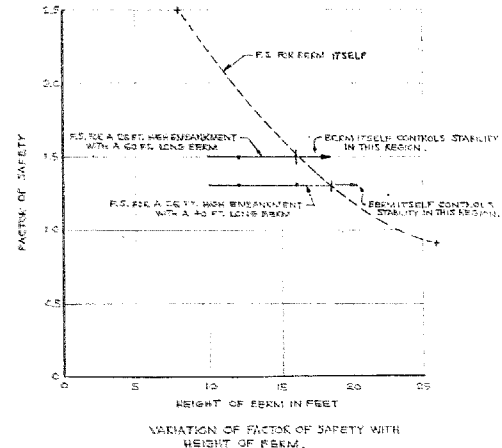
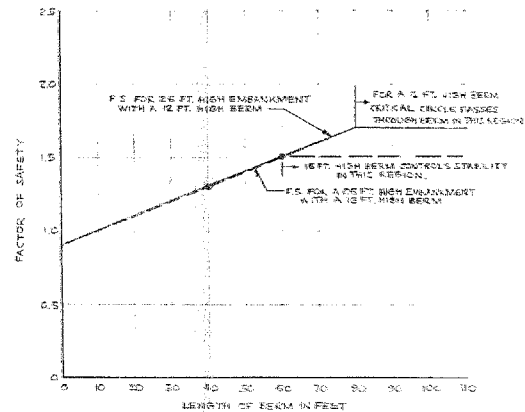
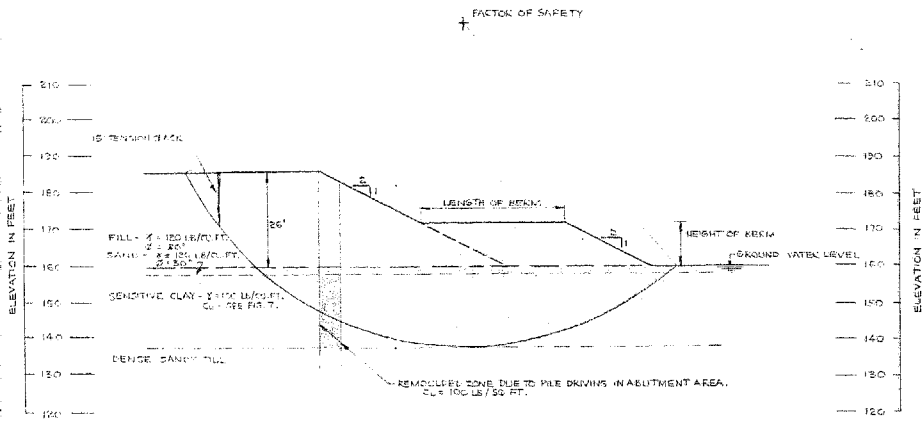


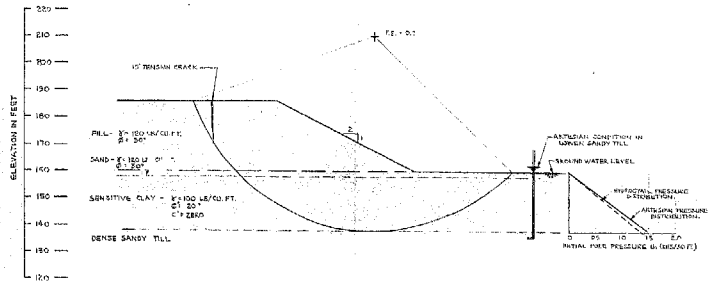
TOTAL STRESS ANALYSIS
26' HIGH EMBANKMENT



TOTAL STRESS ANALYSIS
26' HIGH EMBANKMENT
WITH A 12 FT. HIGH, 60 FT. LONG BERM.

SCALE 1" TO 20'-0"

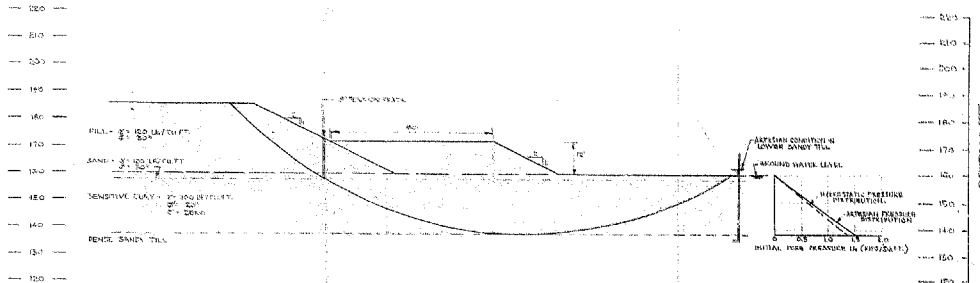




EFFECTIVE STRESS ANALYSIS
22' HIGH EMBANKMENT

EXCESS PORE PRESSURE (Δu) AT ANY POINT ON THE FAILURE CIRCLE WAS COMPUTED AS:
 $\Delta u = B \cdot q$
WHERE B = EFFECTIVE WEIGHT OF EMBANKMENT DIRECTLY ABOVE PORTION OF CIRCLE UNDER CONSIDERATION.
 $B = 0.5$ (OVERALL PORE PRESSURE PARAMETER)

SCALE 1" TO 10'-0"

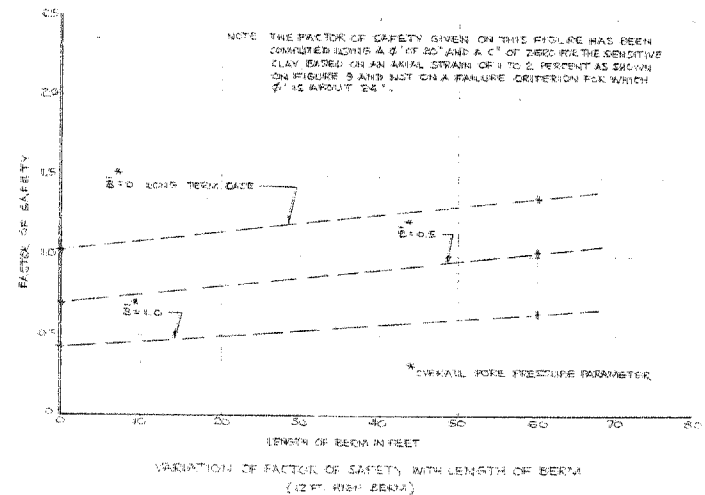
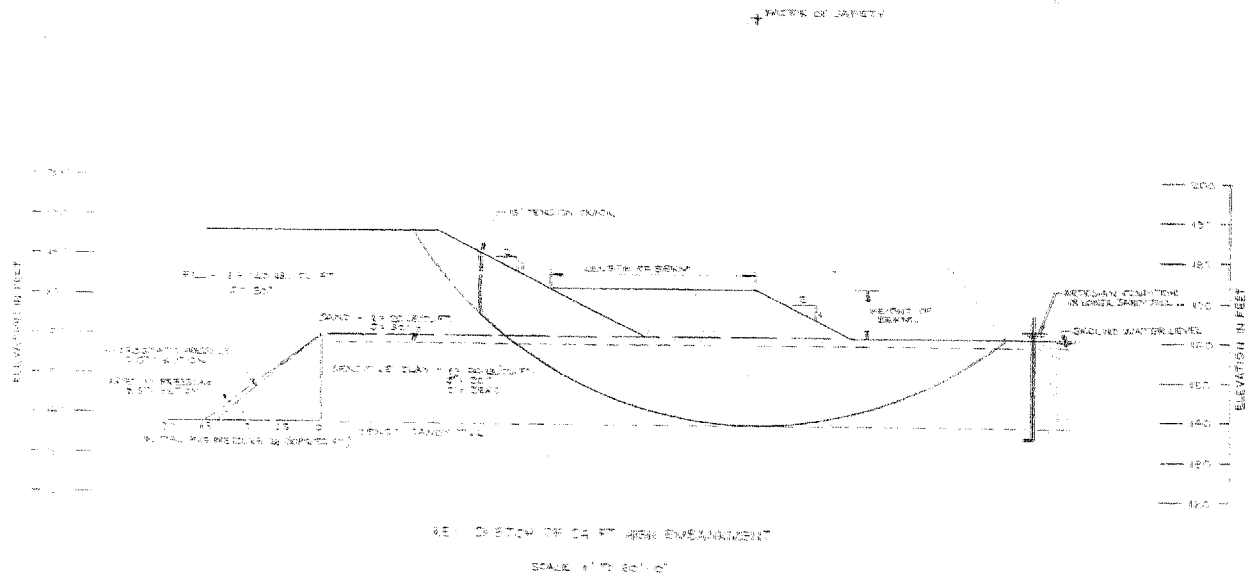


EFFECTIVE STRESS ANALYSIS
22' HIGH EMBANKMENT
WITH 12' HIGH, 6' BY 12' CONCRETE

Drawn JAN. 11, 1966.

GOLDER & ASSOCIATES

Model No. 100
Checked by J. J. G.
Approved by J. J. G.



Box. 401 & Keels St.,
Downsview, Ontario.

November 4, 1965

Materials and Testing Division

M. T. Golder and Associates Ltd.,
2444 Bloor Street West,
Toronto, Ontario.

Attention: Mr. J. Seychuk

Re: R.F. 107-59 -- Site No. 31-230,
Fraser Road Underpass,
2.8 Miles West of Jct. Hwy. 2 & 34,
Hwy. 401 -- District 3 (Ottawa).

Dear Sir:

This is to authorize you to carry out a foundation investigation at the above-mentioned site. The plan showing the locations of the proposed crossing and grade, have been handed to your Mr. J. Seychuk, on November 4, 1965.

Preliminary soil surveys carried out by the Department have indicated the presence of a deposit of very soft clay. It is, therefore, believed that some problems regarding the stability of the approach embankments could be encountered. In the same general area, difficulties with newly built structures have been and are still being experienced.

The above is brought to your attention so that you can organize the necessary field exploration work. It is understood that a qualified Soils Engineer will be in charge of the field work at all times.

Ten (10) copies of the completed report should be submitted to the Foundation Section not later than December 17, 1965. Previous requirements as to preliminary borehole information and laboratory testing program, should be followed.

Because the drawings accompanying the Foundation reports, showing the location of borings, the inferred subsoil conditions, etc., are to become contract drawings, you are requested to prepare them in accordance with the U.E.C. standards. To enable

H. Q. Golder & Associates, - 2 -
Attn: Mr. J. Beychuk.

November 4, 1965

you to do this, we are supplying you with sample drawings with all the necessary explanations, together with linen sheets for your drawings. You are also requested to provide us with Cronaflex copies of the drawings.

Charges for the work performed will be in accordance with your Schedule of Rates, dated October 1, 1965, and invoice to be addressed to the attention of the undersigned.

We are attaching Purchase Order J 34797, covering the purchase of any new material required for this work, in order that you may use this as a basis for exemption from the Federal tax for such purchases. The Exemption Certificate is printed thereon.

Yours very truly,

a.l.

AGS/mdeF
Attach.

A. Rutke,
MATERIALS & TESTING ENGINEER

cc: Messrs. S. McCombie
H. S. Piller
L. E. Walker
J. E. Gruspier
A. Crowley
Mrs. I. Steinberg
B. Konings
H. Brymanski (2) ✓
Foundations Office
Gen. Files (2)

H. Q. GOLDER & ASSOCIATES LTD.

CONSULTING CIVIL ENGINEERS

H. Q. GOLDER
V. MILLIGAN
L. G. SODERMAN
J. L. SEYCHUK

2444 BLOOR STREET WEST
TORONTO 9, ONTARIO
763-4103
767-9201

January 18, 1966.

Department of Highways, Ontario,
Materials & Testing Division,
Hwy. 401 and Keele Street,
DOWNSVIEW, Ontario.

Attention: Mr. A. G. Stermac, P.Eng.,
Principal Foundation Engineer.

RE: SUBSURFACE INVESTIGATION,
W.P. 107-59 - SITE NO. 31-230,
PROPOSED FRASER ROAD UNDERPASS,
HIGHWAY 401 - DISTRICT 9 (OTTAWA),
GLEN GARRY COUNTY, ONTARIO.

Dear Sirs:

Ten copies of our report for the above investigation were delivered to you today by messenger. A Cronaflex copy of Figure 1 from the report was also included with the report shipment.

As discussed during a meeting at our offices on January 12, 1965, this report presents all of the work carried out by us to January 12, 1966. Once you have studied our report a further meeting is to be arranged to discuss the possible solutions outlined in our report and to decide on the most practical and economical solution to the problem.

If you have any questions in the meantime, please call us.

Yours truly,

H. Q. GOLDER & ASSOCIATES LTD.,



J. L. Seychuk, P.Eng.

JLS:hdg
65135

Mr. B. E. Davis,
Bridge Engineer,
Bridge Division.

Attention: Mr. S. MacCombie

Foundation Section,
Materials & Testing Div.,
Room 107, Lab. Bldg.

January 19, 1966

JAN 19 1966

FOUNDATION INVESTIGATION REPORT BY
S. Q. Golder and Associates Limited -
W.P. 107-59 -- Site No. 31-230,
Fraser Road Underpass, 2.8 Miles West
of Jet. Hwy. 2 & 34, Hwy. 401, Twp. of
Charlottenburgh -- District 9 (Ottawa).

Attached, please find the above-mentioned report prepared and submitted by the Consultant, S. Q. Golder and Associates Ltd.

We have reviewed the report and have found that it contains all the necessary factual information, and that all the necessary laboratory work was carried out.

The report deals with the problem of embankment stability in great detail. This is due to the unsatisfactory performance of some bridges in the same area founded on comparable subsoil conditions. The undersigned has discussed with the Consultant, the probable causes of the unsatisfactory behaviour of some of the bridges in the area, and a consensus of opinion was reached that a more rigid type of piled foundation is definitely desirable and that the stability of the approach embankments should be considered also from the strain point of view, rather than only from the failure stress point of view.

It is beyond doubt that counterbalancing berms are needed in order to provide stability of the embankment. The final and exact length of the berms remains yet to be determined. We would, therefore, suggest that a meeting be called at which the entire problem should be discussed, everybody be familiarized with the thinking leading to the design criteria, and everybody be given an opportunity to voice his opinion. At this meeting, representatives of the Bridge Design, Bridge Location, Road Design, and Foundation Section, as well as the representatives of the Consultant, should take part.

AGS/MSF
Attach.

cc: Messrs. B. E. Davis (2)
E. A. Tregaskes
D. A. Farren
A. G. Pillar
L. E. Walker
J. E. Gruebler

A. G. Stersac
A. G. Stersac,
PRINCIPAL FOUNDATION ENGINEER

A. Watt
Foundations Office
Gen. Files

Mr. R. S. Pillar,
Senior Project Design Engr.,
Road Design Division (Kingston)

Foundation Section,
Materials and Testing Division,
Room 107, Lab. Bldg.

February 14, 1966

Your Memo -- Feb. 7, 1966

W.P. 107-59, Underpass at Fraser Road - Hwy. 401,
-- District No. 9 (Ottawa) --

With reference to your memorandum of February 7, 1966, to Mr. W. Wigle, Program Engineer, Program Section, Downsview, we wish to make the following comments:

It is realized by all parties concerned that bridges built in the Lancaster area, which are founded on sensitive clay deposits, have not performed as expected. Large settlements of the approach fills and certain movements of the abutments have necessitated considerable maintenance, and still do. Settlements of the approach fills are understood and were anticipated; however, the abutment movements were not anticipated, and are still not quite well understood. Only certain hypotheses have been put forth to explain these movements, but there is insufficient evidence to prove them. The only fact of which we are all convinced, is that the abutment movements are intimately related to the embankment settlements.

The largest part of the settlement of the approach embankments takes place during the first one to three years after construction. To take advantage of this fact, stage construction is recommended. The longer the staging - i.e., the longer the time between the fill placement and the construction of the bridge, the less maintenance will be required.

Due to the intimate relationship between the fill settlements and the abutment movements, the driving of the piles should be carried out during the second stage. Thus, the influence of the embankment settlements - whatever it may be - on the foundations, is decreased and the possibility of abutment movements becomes rather remote.

The recommendation that pile driving be carried out in Stage II, has also been discussed with Mr. L. E. Walker, District Engineer, Ottawa, and the consultant for the Fraser Road underpass, E. Q. Golder and Associates, and they both concur with our reasoning.

ACS/MEP

A. J. Sternac
A. J. Sternac,
PRINCIPAL FOUNDATION ENGINEER

cc: Messrs. L. E. Walker W. Wigle
 S. McCombie H.Q. Golder & Assoc. Ltd.
 D. Richardson
 C. J. Markiewicz
 Foundations Office
 Gen. Files

MEMORANDUM

To: W. Wigle,
Program Engineer,
Program Section,
Downsview.

From: H.S. Pillar,
Sr. Project Design Engineer,
Road Design, Kingston.

Date: February 7th, 1966.

Attention: D. MacFarlane

Our File Ref.

IN REPLY TO

SUBJECT: W.P. 107-59. U'Pass at Fraser Road - Pwy. #401 - Ottawa District.

A meeting was held on January 27th, 1966, with representatives of the Bridge Division, Road Design Division, and the Foundation Section to discuss anticipated foundation problems at the above-mentioned structure site.

It was the consensus of opinion that 60' earth berms were required and that the approach fills be constructed at least two years in advance of the bridge. Not only would savings be effected on the design of the abutments, but future maintenance costs i.e. asphalt padding, replacement of guiderail, curb and gutter, etc. on the approaches would be greatly reduced. A comparable example would be the structure at Brookdale Avenue where maintenance costs have accumulated to a sizeable sum as a result of the settlements similar to those which we hope to avoid at Fraser Road.

It is our recommendation that the present work project be divided as follows:--

- (1) W.P. 107-59-1 - To include grading and drainage on the approaches (including berms), relocation of service roads in conjunction with a detour for Fraser Road, and placing H piles for the abutments only. The total estimated cost for this work is \$60,000.00 approximately of which \$5,000.00 is allotted to the placing of the steel piles.
- (2) W.P. 107-59-2 - To include granular base, paving, curb and gutter, etc. on the approaches and the complete structure excepting the abutment piles. Minor grading and granular base will also be required in connection with the removal of the detour and completion of the service roads. The total estimated cost for this work is \$110,000.00

At present, this project is scheduled for an early award in 1967, however, I would recommend, providing funds are available, that the approaches be constructed as soon as possible this year and the structure re-scheduled for the summer or fall of 1968. Depending upon the results obtained from instrumentation of the approach fills, it may be possible to award the structure sooner.

In any event, I am presently preparing a property request which will be issued within a few weeks. If you agree, we could have a design prepared for the grading work only by July 1st, 1966.

Any work the Bridge Office desires to include ie: piling, etc. would then have to be completed by June 1st, 1966.

Please advise as soon as possible of your decision in this matter.

R.S. Pillar

R.S. Pillar
Sr. Project Design Engineer.

RSP/ss

C.C. to: L. Walker

A.G. Starnes ✓

A.P. Watt

B. Richardson

S.J. Markiewicz.

R.S. Pillar,
Senior Project Design Engineer,
Road Design Division.
Kingston.

W.G. Wigle,
Program Engineer.

February 25, 1966.


W. P. 107-59, Underpass at Fraser Road
Highway 401 - District 9 - Ottawa

This is in reply to your memorandum of February 7th 1966. The Program Division agrees with your recommendations to split the above work project into two separate contracts.

We have now taken the necessary steps to have the first project, which includes grading and drainage on the approaches, etc., programmed for 1966. The second project which includes the granular base, paving and structure, has been programmed for 1967.

In view of Mr. A. G. Stermac's letter to you on February 14th 1966, we have included the estimated cost of placing steel piles, in the second project.

The appropriate schedule of Pre-Engineering will follow in a few days.


W.G. Wigle,
Program Engineer.

WGW:nf

c. c. A. F. Stermac ✓
c. c. L. Walker
c. c. S. J. Markiewicz
c. c. A. P. Watt

Mr. G. Scott,
Regional Bridge Location Engr.,
Bridge Division, Admin. Bldg.

Foundation Section,
Materials & Testing Div.,
Room 107, Lab. Bldg.

March 15, 1966

Your Memo -- Mar. 8/66

W.P. 107-59, Site 31-230,
Fraser Road Underpass,
2.5 Miles West of Jct. Hwy. 2 & 34,
District #9 (Ottawa) Hwy. 401.

With reference to your memo of March 8, 1966,
concerning Preliminary Plan D 5888-P1, Fraser Road Underpass,
we wish to make the following comments:

It has been decided that stage construction will
be used at this site. The first stage will consist of building
the approach embankments together with the berms, while piles
and the structure itself, will be built in the second stage.

By placing the fill one year earlier, some
detrimental effect of the settlements on the piles will thus
be eliminated. It is essential that the entire fill be placed -
i.e., even at the locations of future abutments. The material
placed at these locations will have to be excavated when piles
are to be driven and abutments built.

Two-foot diameter piles battered (1:4) in two
directions, are presently proposed for the support of the
abutments. In view of the large settlements to be expected,
we are not sure whether this solution is the most appropriate
one. We would like to study this matter further and make our
comments at a later stage. A possible change of type or
arrangement of piles will in no way affect the bridge design
and we feel, therefore, that the final decision on the abutment
piles can be deferred.

AGS/Mief

cc: Foundations Office
Gen. Files

A. G. Sternac
A. G. Sternac,
PRINCIPAL FOUNDATION ENGINEER

MEMORANDUM

To: Mr. A. Stermac,
Principal Foundation Engineer,
Room 107, Lab. Bldg.

From: Bridge Division,
Downsview, Ontario.

Date: March 8, 1966.

Our File Ref.

In Reply To

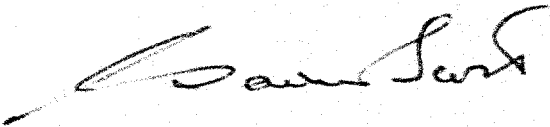
SUBJECT:

W.P. 107-59, Site 31-230,
Fraser Road Underpass,
2.8 miles west of Jct. Hwy. 2 & 34,
District #9, Hwy. 401

We are sending to you herewith one print of preliminary
plan D 5888-P1 for the subject structure.

Would you please let us have your written comments.

GS/ag


G. Scott,
Regional Bridge Location Engineer.

Encl.

Mr. R. S. Pillar,
Sr. Project Design Engineer,
Regional Road Design Office,
Kingston, Ontario.

Foundation Section,
Materials & Testing Div.,
Room 107, Lab. Bldg., Downsview.

May 5, 1966

Your Memo - April 27/66

W.F. 107-59, Fraser Road Underpass, Hwy. #401

With respect to your memo of April 27, 1966, regarding the above subject, we would like to make the following comments:

Both forward slopes of the approach embankments are shown as 2:1. We would suggest that the upper slope - i.e., the slope of the approach fill above the berm, be 1½:1. This portion of the approach fill will be removed for the placing of the abutment, and it would be beneficial to have as much load there as possible until that time.

Because of the settlements of the fill, consideration has to be given to the compaction of the surcharge. It is believed that possibly between one and two feet of settlement could occur and consequently, part of the surcharge could become part of the approach embankment and should, therefore, be compacted as per standard.

It is our intention to place a number of instruments prior to the placement of the fill, and a three weeks' advance notice would be greatly appreciated.

AGS/MdeP

A. G. Stermac
A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

cc: Foundations Office.
Gen. Files.

MEMORANDUM

TO: Mr. T. Stermac,
Principal Foundation Engineer,
Materials and Testing,
Downsview.

FROM: R.S. Pillar,
Sr. Project Design Engineer,
Road Design, Kingston.

DATE: April 27th, 1966.

OUR FILE REF.

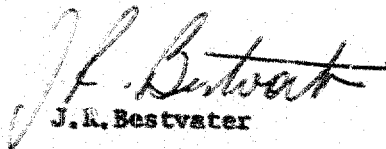
IN REPLY TO

SUBJECT: W.P. 107-59, Fraser Road Underpass, Hwy. #401

Enclosed is a print of a profile No. C-32-17 indicating final grades on which we have shown in red our proposed design for the construction of the berm including the two foot surcharge.

Would you please scrutinize our proposals, and forward your comments in order that they can be incorporated at this stage of the design.

Your immediate concern in this matter is greatly appreciated.


J.R. Bestvater

for:

R.S. Pillar
Sr. Project Design Engineer

JRE/RSP/ss
Encl.

Department of Highways Ontario

Copy for the information of

A.G. STERNAC

L.E. Walker,
District Engineer
Ottawa.

Attention: W. Aitken

*File with report by
H.E. Golden 4/8/66*
R.S. Pillar,
Sr. Project Design Engineer,
Road Design, Kingston.

May 9th, 1966.

W.P. 107-59-1. Fraser Road Underpass, Hwy. #401 - Ottawa District.

Attached is a copy of a memorandum dated April 27th, 1966, from Mr. A.G. Sternac, Principal Foundation Engineer.

I refer you specifically to paragraph 4 where Mr. Sternac has requested 3 weeks advance notice prior to placement of the approach fills. Since this time period will be beyond the design stage, would you please fulfill this request. I might mention that we will incorporate a special provision in the contract advising the contractor that he will be required to safeguard any necessary devices.



R.S. Pillar
Sr. Project Design Engineer

RSP/ss

Att'd.

c.c. to: A.G. Sternac.

MEMORANDUM

*Foundations
Office*

To: Regional Road Design Office -
Kingston.

From: Materials & Testing Division.

Attn: Mr. R. S. Pillar.

DATE: April 5, 1966.

Our File Ref.

IN REPLY TO

SUBJECT: Proposed Fraser Road Underpass,
Hwy. #401, W.P. 107-59.

Spencer

Stage construction has been proposed and accepted for the above-mentioned structure. The longer the period between approach embankment construction and bridge construction, the more beneficial will be the effects of such a staging. However, because of other reasons it is believed that not more than one year of time is available for staging. During this period a certain amount of settlement will take place. To accommodate for that settlement it is suggested that the fill be built 2 ft. higher. These two feet will also act as a small surcharge. It is believed that the approach fills will settle during the one year interval less than 2 ft. and some of the material will therefore have to be removed prior to the final road grading.

Because of possible failure of stock piles it is recommended that they be built not higher than 18 feet.

It is the intention of this Section to instrument this site prior to fill placement. Settlement plates and piezometers are contemplated. It is hoped that the readings of these instruments will enable a better understanding of the problems connected with the building of approach fills on deposits of sensitive marine clays. We would therefore appreciate to be advised of the time of construction commencement so we could carry out our necessary work well ahead of fill placement.

AGS/tt

cc: Messrs. L. E. Walker
S. McCombie
B. Richardson
S. J. Markiewicz
J. E. Gruspier
Foundations Office,
Gen. Files

A. E. Stermac
A. E. Stermac
PRINCIPAL FOUNDATION ENGINEER

MEMORANDUM

To: Mr. S.J. Markiewicz,
Regional Road Design Engineer,
R.D.O., DOWNSVIEW.

From: M.&T. Division, KINGSTON.

Date: October 13, 1965.

Our File Ref.

IN REPLY TO

SUBJECT: Re: Hwy. 401. W.P. 107-59. Fraser Road Underpass 2.8 Miles West of Lancaster.

Attached is the Soils Section Design report and print of the Soils profile 401J 50-1 for the abovenoted Structure on Hwy. 401 located 2.8 miles west of Lancaster.

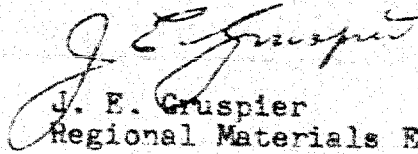
This site is located in a till area where borrow materials of a bouldery nature are likely. It is expected that borrow will be readily available. Some minor grade revisions have been recommended to provide adequate cover over unacceptable subgrade materials.

Granular deposits are generally depleted in the immediate area as are sand deposits. The use of all G.B.C. 'A' is recommended.

A foundation investigation will be required due to the clay subsoil materials encountered. It is possible that the height of the stockpile of granular materials may have to be limited because of the underlying clay but this will have to be determined by the Foundation Section.

Should you have any queries, please contact this office.

JEG:cdr


J. E. Gruspier
Regional Materials Engineer

c.c. D.W. Farren
H.A. Tregaskes
T.C. Muir
W. Wigle
L.E. Walker (2)
S. McCombie
M. Stoyanoff
J.L. Forster
D.A. MacDonald
G.A. Wrong (2)
A.G. Stermac
File

SOILS DESIGN REPORT

Hwy. 401

Fraser Road Underpass
2.8 Miles West of Lancaster

0.57 Miles

V.P. 107-59

Proposed Grading, Granular Base, Paving, & Structure Project

<u>Soils Plan and Profile</u>	<u>Station to Station</u>	<u>Line</u>	<u>Township</u>
401 J 50-1	7+00 to 38+75	'A'	Charlot- tenburgh

GENERAL DATA

This project is located in Charlottenburgh township, approximately 2.8 miles west of Lancaster. It is proposed to pass Fraser Road over Hwy. 401 by means of an underpass to be constructed during the 1966-67 construction program.

DESIGN CRITERIA

A.A.D.T.	- 200
Minimum Vertical Curve	- 500'
Maximum Grade	- 5%
Horizontal Alignment	- 9° Max Curves
Pavement Width	- 20'
Shoulder Width	- 6'
Shoulder Rounding	- 2'

PHYSIOGRAPHY & SOILS DATA

The project is located on the Lancaster Flats Physiographic Region, a lowland area where the till plain has been buried under water-laid deposits leaving exposed only the stony crests of a few drumlins and till ridges. The water-laid materials range from clay to very fine sand.

SOILS INVESTIGATION

A soils investigation was carried out on this project during June of 1965, using a 12" power auger. Borings were

placed to a minimum depth of 4' under the shallow fill sections and to 15' on each side of Hwy. 401 where the high fills are proposed.

The subsoil was found to consist of approximately 2' of moist sandy silt over 4'+ of moist stiff silty clay over 3'+ of moist firm silty clay over wet soft fat clay to depths greater than 15'.

Due to the nature of the subsoil, foundation investigation and stability analysis is required at this site. This has not yet been completed.

EMBANKMENT STABILITY

Throughout this area there have been many other structures built over Hwy. 401 at locations where the foundation soil conditions are similar to those at this site. In some cases berms were required, and in all cases extensive fill settlement adjacent to the structures occurred due to consolidation of the underlying wet clay soil.

At many of the abovementioned sites where extensive fill settlement occurred adjacent to the structures, the concrete gutters are now distorted and useless. It is therefore suggested that gutters not be placed within 50' of the structure at the time of construction.

GRADELINE

To provide an adequate tie-in grade to the county road, the gradeline has been revised from Sta. 7+00 to Sta. 12+00. To provide adequate cover for the unacceptable subgrade soils in the vicinity of Sta. 33+00 and provide a granular lift from Sta. 35+ to Sta. 39+, the gradeline has been revised from Sta. 30+00 to Sta. 41+00.

BORROW MATERIALS

Large quantities of bouldery silty sand till are located on a ridge which crosses the alignment at Sta. 0+00. This is an acceptable earth borrow material and is probably available for use as there is a partially depleted borrow pit located approximately 300' left of Sta. 1+40.

GRANULAR MATERIALS

The crushable gravel sources in the near vicinity of this project were largely depleted during construction of Hwy. 401. However, some crushable gravel still remains in the O'Connor Pit area located approximately 15 miles haul distance from the project. This pit is located 3 miles west of Hwy. 34 and approximately 9 miles north of Lancaster.

RECOMMENDATIONS

1. Type of Granular Contract

Based on the availability of materials, it is recommended that the granular materials on this project consist of G.B.C. Class 'A' only.

2. Depth and Width of Granular Materials

The granular material should be placed full width to a 15" total depth. If the subgrade is constructed of other than the anticipated silty sand till material as indicated under "Borrow Materials" the Materials & Testing Section should be consulted during construction to ascertain whether the 15" granular depth will be adequate over the subgrade as constructed.

3. Structure Backfill

Granular backfill to structures should consist of G.B.C. Class 'A'.

4. Compaction Equipment

It is expected that most of the earth to be compacted on this project will be sandy in nature. For design purposes it is recommended that 80% of the compaction time be allotted for wobble wheel type compaction units and the remainder for sheepsfoot type rollers.

5. Stockpiling Procedures

Due to the soft nature of the clay soils underlying the project site, it is important that any stockpile of granular material required be built on the till ridge in the vicinity of Sta. 0+00.

6. Culvert Types

Due to the nature of the foundation soils any culverts required beneath the proposed fill from Sta. 10+00 to Sta. 35+00 should consist of box type concrete culverts or circular C.I.P. type.

7. Depth and Type of Asphalt Pavement

It is recommended that the pavement consist of the following:

¾" Hot Mix Sand-Asphalt Course

1½" H.L. 3 (Surface Course)

RLS:cdr

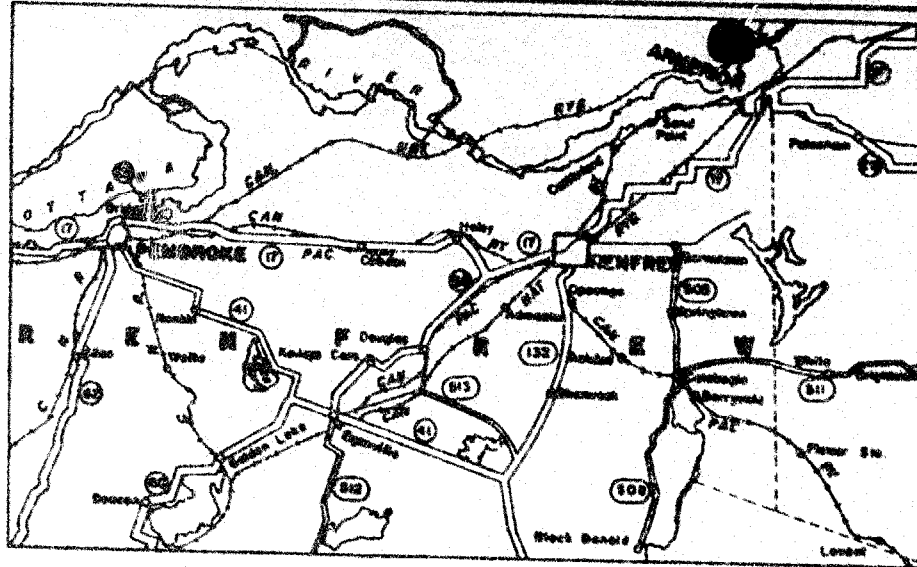
Prepared by:

R. L. Smith
R. L. Smith
Project Soils Supervisor

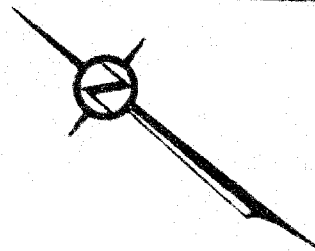
Reviewed by:

H. A. Meyer
H. A. Meyer
Senior Soils Engineer

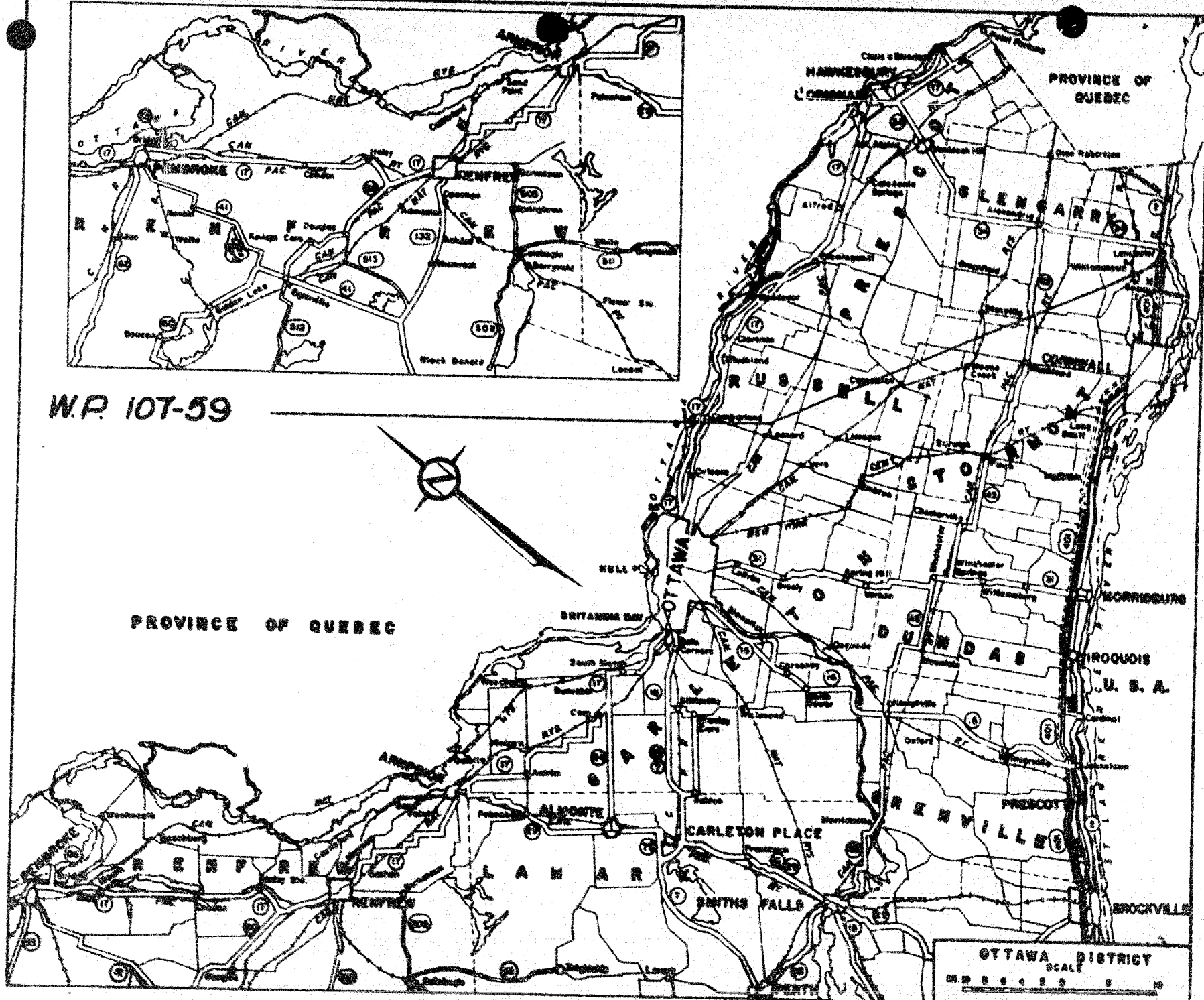
October 13, 1965



W.P. 107-59



PROVINCE OF QUEBEC



Mr. R. S. Pillar,
Sr. Project Design Engineer,
Kingston Regional Office.

Foundation Section,
Materials & Testing Div.,
Room 107, Lab. Bldg.

June 28, 1966

Your Memo -- June 23/66

W.P. 107-59 - Hwy. 401 - Fraser Road Underpass

With respect to your memo of June 23, 1966, regarding the above project, we would like to make the following comment:

The part of the approach embankment where piles are to be driven should not contain any boulders at all. We feel that limiting the size of boulders to 6" maximum could create some serious problems during pile driving.

Since the embankment is going to be built in six-inch layers, the removal of all boulders from a limited area should not represent any problem at all. We would, therefore, strongly suggest that the Special Provisions be altered to incorporate our recommendation.

AGS/MdeF

A. G. Sternac
A. G. Sternac,
PRINCIPAL FOUNDATION ENGINEER

cc: Foundations Office
Gen. Files

MEMORANDUM

To: Mr. A. G. Stermac,
Principal Foundation Engineer,
Lab Building,
DOWNSVIEW, Ontario.

FROM: Mr. R. S. Pillar,
Sr. Project Design Engineer,
KINGSTON, Ontario.

DATE: June 23, 1966

OUR FILE REF.

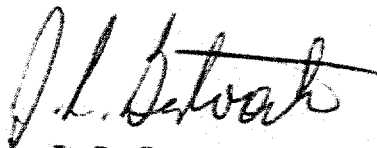
IN REPLY TO:

SUBJECT: Re: W.P. 107-59 - Hwy. #401, Fraser Road Underpass.

Enclosed find one set of our completed Contract Drawings for your information and use.

Special Provisions have been inserted in the Contract as follows:

- (1) Limiting the size of boulders to 6" maximum in the vicinity of the future pile driving areas,
- (2) Providing protection for and allowing access to measuring devices, and
- (3) Limiting the height of granular stockpiles to 18'.



J. R. Bestvater,
For: R. S. Pillar,
SR. PROJECT DESIGN ENGINEER.

Encl.

JRB/RSP/em

SPECIAL PROVISION RE: MEASURING DEVICES

A number of measuring devices (Piezometers, etc.) have been installed by the Department in the proximity of the approach fills.

The Contractor shall:

- A. Provide access to these devices by Department personnel at all times
- B. Provide adequate protection to prevent damage to these devices during his grading operations.

In the event of damage caused to these devices through negligence on the part of the Contractor during operations, all costs arising from repairs or replacement of the devices shall be borne by the Contractor.

#65-F-232
W.P. #107-59
HWY #401
FRASER ROAD

REFERENCE No. E - 4605 - 1

#58-F-235-C

W.P. 72-57

HWY #401

RAISIN RIVER

BRIDGE

E. M. PETO ASSOCIATES LIMITED

Russell 9-1126-7

1287 Caledonia Road,
Toronto 19, Ontario.

CONT. 58-168

23-58-168

Job Number 57147

February 10th, 1958.

SUPPLEMENTARY COPY

58 F 235 C

Office of the Bridge Engineer,
Department of Highways of Ontario,
280 Davenport Road,
Toronto, Ontario.

Attention: Mr. J. C. McAllister.

W.P. 72-57

Dear Sirs,

Re: Raisin River - Highway 401 Bridge
(Soil Site Investigation.)

We have completed the site investigation for the Raisin River - Highway 401 bridge crossing in accordance with our usual terms of reference, and with some amendments to the original program, as explained in the report attached hereto.

For ease of reference, we summarize briefly our recommendations in respect to the soil conditions on this site.

1. The piers and at least the West abutment should be carried on steel piling driven through the sandy till stratum to the bedrock. This type of foundation is particularly desirable if a rigid or continuous girder type superstructure is contemplated.

"Continued"

2. Alternatively, and providing some minor differential settlement can be tolerated, the piers and abutments can be founded in the sandy fill layer, preferably at least 5 ft. below the top surface of this stratum. (i.e. at elevation 136.0 ft., except at the West abutment where this elevation can be raised to about 136.0 ft.).

3. Unless the West approach fill is placed well in advance of the bridge construction (say 6 months to 1 year in advance) it would be advisable to remove the organic fill overlying the sandy fill for a distance of at least 100 ft. beneath the West abutment for the full width of the abutment.

4. There was no indication of any abnormal water problems or difficulty in de-watering any pier or abutment excavation. Some minor difficulty due to small boulder interference may be encountered in driving sheet piling into the sandy fill stratum.

In accordance with our usual practice, we have forwarded one copy of this report complete to your Consulting Engineers, C. C. Barker and Associates Ltd., responsible for the bridge design and this project, in order to allow them to expedite their preliminary design proposal. Some limited information was supplied verbally to the Consultants as the field work was being completed, at their request, in view of the advanced bridge design completion date.

PAGE THREE

Should your Consultants require additional advice with regard to our soils report, perhaps due to a change of pier locations, we shall be pleased to be of additional service.

Yours very truly,

E. M. DETO ASSOCIATES LTD.

E. M. Deto, P. Eng.

EMP/10

SOIL SITE INVESTIGATION

AT

HIGHWAY 401 GABIN RIVER BRIDGE

F O R

DEPARTMENT OF HIGHWAYS OF ONTARIO

B. H. FINE ASSOCIATES LTD.,
1807 California Road,
Toronto 18, Ontario.

TERMS OF REFERENCE

We were instructed by letter dated December 6th, 1957, from Mr. J. C. McAllister, acting for the Chief Bridge Engineer, Mr. A. M. Foye, to carry out the soil investigation for the proposed new Highway 401 bridge across the Raisin River East of Cornwall. The investigation was to be carried out in accordance with our standard practice. A marked site plan, drawing P-3842-1, indicating the location of twelve test holes on this site, was forwarded with the letter of authority.

After examination of the site, and as a result of the preliminary soil test results, it was decided that one additional test hole should be performed some 50 feet back from the waters edge on the highway centre line at the East side of the crossing. In addition, two further test holes were to be put down on the swampy ground along the centre line immediately West of the proposed Western abutment location. These additional test holes were sunk to determine the possibility of failure beneath the bridge approach embankments.

METHOD OF OPERATIONS

This site investigation was carried out by our number 3 unit, a Sullivan "12" skid-mounted diamond drill rig, which commenced work on December 18th, 1957. After a long Christmas break and some delay during the first week of January because of illness of the field crew, work was completed on January 24th, 1958.

Each test hole was sunk by driving and cleaning BX drill casing, sampling ahead of the casing at frequent intervals, with either a 3" split barrel sampler or 2" Shelby tube. Standard penetration test results were recorded whilst sampling with the split barrel sampling tube.

A number of the test holes were diamond drilled in order to prove the reliability and continuity of bedrock. In at least one test hole, an attempt was made to penetrate as far as possible, without core drilling, by means of chopping bit and by running an open end A rod in the diamond drill head.

Several test holes were bailed, or pumped dry, at various stages of the work and the water level reading was recorded the following morning. The natural moisture contents of numerous samples were also determined.

METHOD OF OPERATIONS - Cont'd

The test results are shown on each test hole log, together with the site plan showing test hole locations. The site plan includes a longitudinal section through the test holes located on the Highway Centre Line. The relative depths for the test holes located to both the North and South of the centre line are shown for ease of comparison.

Unfortunately, the D.R.O. reference bench mark on this site had been removed, and a D.R.O. boundary monument was used instead for a reference elevation. After completion of the field work, but during preparation of the drawing it was found that the D.R.O. did not have the elevation of this point. The profile and the ground elevations as shown, are all correctly inter-related, and have been related to an approximate ground elevation of 159.72 feet at station 9+00 as read from D.R.O. drawing T-2242-1. It is therefore possible that there is some slight error up to $\pm .5'$ in the elevations shown.

SITE AND GEOLOGY

This site is located in the physiographic region known as the Glengarry till plain. This till plain has a region of low relief forming the drainage divide between the international section of the St. Lawrence River and the Ottawa basin, from Prescott to the Quebec boundary. The surface is undulating to rolling, consisting of long drumlinoidal ridges and a few well-formed drumlins together with intervening clay flats and swamps.

The drainage pattern is peculiar in that the head waters of the river systems in this area flow sluggishly for long distances between ridges before finding outlets to the main stream.

The Raisin River has its source at Newington in Ganabrock Township. This river flows in an easterly direction approximately parallel to the St. Lawrence River; it has a relatively deep channel and floods only a limited area of low land during periods of high rainfall. The bridge crossing site itself is located only some half to three-quarters of a mile from Lake St. Francis, which forms part of the St. Lawrence River.

The principal soil on this site is the sandy till containing a high proportion of limestone rock fragments with admixture of materials derived from the Precambrian rocks to the North. The area has been glaciated and formed part of the Champlain sea basin.

SOIL CONDITIONS

Organic Silt Stratum

As might be expected, organic silt, in some cases with sand content, constitutes the top stratum in the swamp area on the West side of the river almost as far back as Station 530 + 50. This layer, with variations in sand and clay content, but with decreasing organic content with depth, exists to 13 ft. below surface at test hole 14, to the 15 ft. depth at test hole 1, to the 16 ft. depth at test hole 2, and for the top 2 ft. or 3 ft. only at test hole 10.

The natural moisture contents ranged from 28.3% to 41% at the 7 ft. depth and increased to between 70% and 87% at the 11 ft. depth at test holes 14, and 1 and 2.

The unconfined compressive strengths ranged from a low of 104 p.s.f. to a high of 127 p.s.f. at 20% deformation; failures were uniformly plastic.

The Atterberg Limits for the sample from the 13 ft. depth at test hole 14 gave a Liquid Limit of 85.5, a Plastic Limit of 23.6 and a Plastic Index of 14.9, indicating an inorganic silty clay or clayey silt of low to medium plasticity and low compressibility. The natural moisture content was well over twice the Liquid Limit.

SOIL CONDITIONS - Cont'd

The sample from the 11 ft. depth, at the same test hole, was tested for pH factor and gave a result of 4.8, indicating a weakly acidic condition.

At test hole 8, the silt exists to a depth of about 8 ft., but the organic content is not general throughout, being localized in the form of seams of decayed wood, etc.

Silty Clay Stratum

The grey silt clay stratum shown on the profile as existing from ground surface or ice surface to the 8 ft. 6 inch or 10 ft. depth only at test holes 11 and 13 was also encountered to approximately the same depths below ice surface at test hole 12 beside the East bank, but was not encountered at test hole 10. This stratum also appeared below the organic silt at test hole 1 between the 15 and 19 ft. depths and at test hole 8 below the clayey silt, between the 8 ft. and 17 ft. depths. This stratum also appeared at the river bottom at test hole 4 between 14 and 19 ft. depth (corresponding to test hole 1) and at test hole 9 from the 14 to 17 ft. depth.

SOIL CONDITIONS - Cont 'd

This gray silty clay has a pronounced nuggety texture. The natural moisture contents ranged from 50 to 87%, with a general average of around 64%. The unconfined compressive strength of a sample from the 12 ft. depth at test hole 3 was only 178 lbs. per square foot. Samples tested failed in shear, and quite rapidly. Due to the high moisture contents, there was considerable difficulty in recovering undisturbed samples of this stratum either from Shelby tubes or split tubes with liners. Several laboratory penetrometer tests, on samples in fair condition, gave unconfined compressive strengths ranging from 0.24 tons per square foot to almost 0.5 tons per square foot. The lowest strength results came from samples beneath the river bottom where moisture contents were generally somewhat higher than the average for this stratum.

At test hole 4 only, the sample from the 20 ft. depth appeared to be more silt than clay. The natural moisture content was only 38.7%, the sample gave instantaneous response to the shake test, and had a plasticity index of only 14.3 with a Liquid Limit of 22.2

SOIL CONDITIONS - Cont'd

Sandy Till

The basic subsoil stratum, common to all the test holes, is a gray to gray brown (and in some cases dark grey) fine to coarse sand with grits and angular limestone fragments, generally set in a matrix of silt or clayey silt. Natural moisture contents in this stratum were recorded in several instances at around 9 to 10%.

As might be expected in a till material with quite large rock fragments, there was considerable variation in the standard penetration test results. However, the densities recorded in this stratum varied from compact to dense with standard penetration test blows averaging 37 to 39 per foot. It might be pointed out that a comparatively loose stratum was encountered at test hole 12 from 21 to 22 ft., in test hole 8 from 23 ft. to nearly the 37 ft. depth, and to a lesser extent from 23 feet to 25 feet in test hole 7.

Some minor trouble was encountered with boulders at test holes 5 and 11 at depth, and a few small boulders were encountered at one of the test hole locations immediately below the river bed. No large boulders were encountered at any of the test holes.

SOIL CONDITIONS - Cont'd

Bedrock

As may be seen from the profile and the borehole logs, there was considerable variation in bedrock elevation, or the level of inferred bedrock, due to virtual refusal of the casing and chopping bit at some of the test holes.

In actual fact, careful review of the depth at which diamond drilling and core recovery was carried out suggests that the top surface of the bedrock, within the confines of the river bed, lies somewhere between elevation 121.5 and 123.5. The bedrock appears to be basically a fine grained limestone, grey black in colour and generally hard with little or no reaction with dilute hydrochloric acid.

Some strata of fine grained black shale were found upon examination in some of the rock core. We could not find complete uniformity of this stratification over the site, although a comparatively thin seam of black shale was encountered at the 37 to 38 ft. depth in test holes 2, 5, 8 and 9. Variations in the cleavage plane suggests that the bedrock has been folded in the past.

SOIL CONDITIONS - Cont'd

Considerable difficulty was encountered in penetrating a very hard stratum, which is believed to be a shale layer, at around the 26 ft. depth in a number of test holes. In certain cases, no attempt was made to penetrate this very dense material by drilling after virtual refusal to the casing and chopping bit had been reached, but in others this hard layer was penetrated, usually by diamond drilling, and a further stratum of till material was encountered before final bedrock refusal was reached.

We believe that test hole number 11 (which was the first test hole put down) is an unusual case. With persistent effort, the field crew were able to drive this test hole by alternately driving a chopping bit and running an open end drill rod to the 39 ft. depth, before finally running the diamond drill core barrel. Only wash samples were recovered below the 21 ft. depth due to the very hard driving. Although we have classified the wash samples as a coarse sand and fine gravel, we believe that most of the material below the 27 ft. depth is actually a pulverized bedrock. The till material was encountered between the 25 and 27 ft. depth, overlain by a thin stratum of black shale.

During the diamond drilling operations, wash water was lost only in test hole 9 at the 35 ft. depth. There was no indication of any open seam at any other test hole.

WATER CONDITIONS

Neither artesian water nor any water bearing seam was encountered at any of the test holes, with the possible exception of a water bearing seam at the 85 ft. depth in test hole 9.

After the till stratum had been reached with the casing, the test holes remained dry overnight after being bailed the previous day.

CONCLUSIONS AND RECOMMENDATIONS

1. Due to the extreme variations in soil type and characteristics, it would be unwise to consider placing pier or abutment footings on any soil above the till stratum.
2. If, for reasons of economy, it should be decided to place the pier and abutment footings in the top layers of the sandy till stratum, then a load bearing value of 1.5 tons per sq. ft. should be used for the design, assuming a width of at least 8 ft. for each footing. If the footing is placed at least 5 ft. below the top surface of the sandy till, then the bearing value may be increased to 3.4 tons per sq. ft., assuming

CONCLUSIONS AND RECOMMENDATIONS - Cont'd

a footing width of not less than 5 ft. ; this value should be reduced to 2.8 tons per square foot for footings 10 feet wide or more, with values being more or less proportional for footing widths between 5 ft. and 10 ft.

We wish to point out that there is some risk of minor differential settlements, perhaps exceeding the normal maximum total settlements of $3/4$ " for any pier with the recommended loadings given above, due to the presence of loose pockets in the sandy till stratum, as encountered at 3 of the test holes.

2. There is little doubt that short steel and bearing piles driven into the top surface of the bedrock will provide the most satisfactory foundation for the piers and abutments, certainly for the piers and West abutment.

CONCLUSIONS AND RECOMMENDATIONS - Cont'd

4. Timber pile foundations are not recommended for this site for several reasons, but principally due to presence of small boulders and the high density of the till. Penetration without danger of damage to the piles would be difficult to achieve. Insufficient lateral support at the pile tips could therefore be ~~a~~ problem.
5. Monotube, or similar displacement piles, are not recommended for the foundations on this site for generally similar reasons given in the preceding paragraph.
6. Due to the proximity of the site to the St. Lawrence River system and the generally sluggish nature of the stream flow, scour will not be a major problem in our opinion. This statement is further borne out by examination of the soil profile. Scour exceeding a depth of 5 ft. below river bottom is unlikely, although we recommend that sheeting be driven below this depth, if possible, if the footings are placed on the sandy till instead of on piles.

CONCLUSIONS AND RECOMMENDATIONS - Cont'd

4. With a tentative grade elevation of approximately 168.3 ft. at the East abutment, there is no bank stability problem on this side of the river, assuming normal excavation through the relatively soft surface clay for an abutment located at station 543 + 00.
8. On the West bank, the 10 ft. high approach embankment will present a problem if this fill is placed directly on top of the existing swamp surface. For an embankment with a base width of 120 ft. theoretical calculations show that the new imposed load will exert a pressure exceeding the ultimate shear strength of the soil at the 12 ft. to 15 ft. depth below present surface by about 35%. In actual fact, the presence of the sandy compact to dense till so close to surface beneath such a wide embankment tends to ameliorate the critical theoretical condition. However, some failure of the embankment for a distance of at least 100 ft. back from the West abutment at station 541 + 00 can be expected in the form of a general subsidence as the fill reaches the tentative grade elevation.

CONCLUSIONS AND RECOMMENDATIONS - Cont'd

Under the circumstances, we believe that consideration should be given to removal of the organic silt and clayey silt above the sandy till between chainage 539 + 60 and 541 + 00, for a width of at least 80 ft. (the presumed width of the abutment), and replacement with a basically granular fill material.

8. Excavations for the bridge structure itself will, of course, require close sheeting if dry work is required. However, there was no indication of any potential dewatering problem if the sheeting is driven into the till stratum below the river bottom.

E. M. PETO ASSOCIATES LTD.

E. M. Peto, P. Eng.

EMP/sp

e. m. peto associates ltd.

Toronto 19, Ontario

LIQUID LIMIT TEST

JOB No. 57147 Project Hwy. 401 - Raisin River Bridge

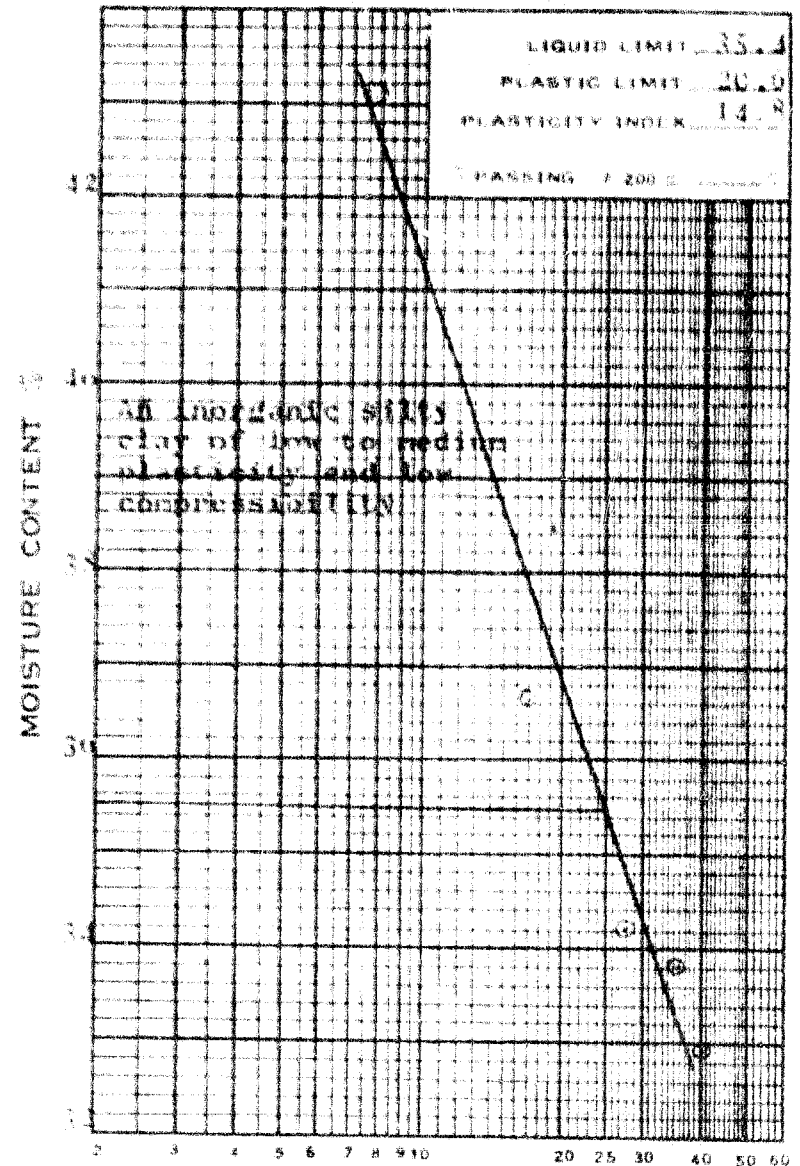
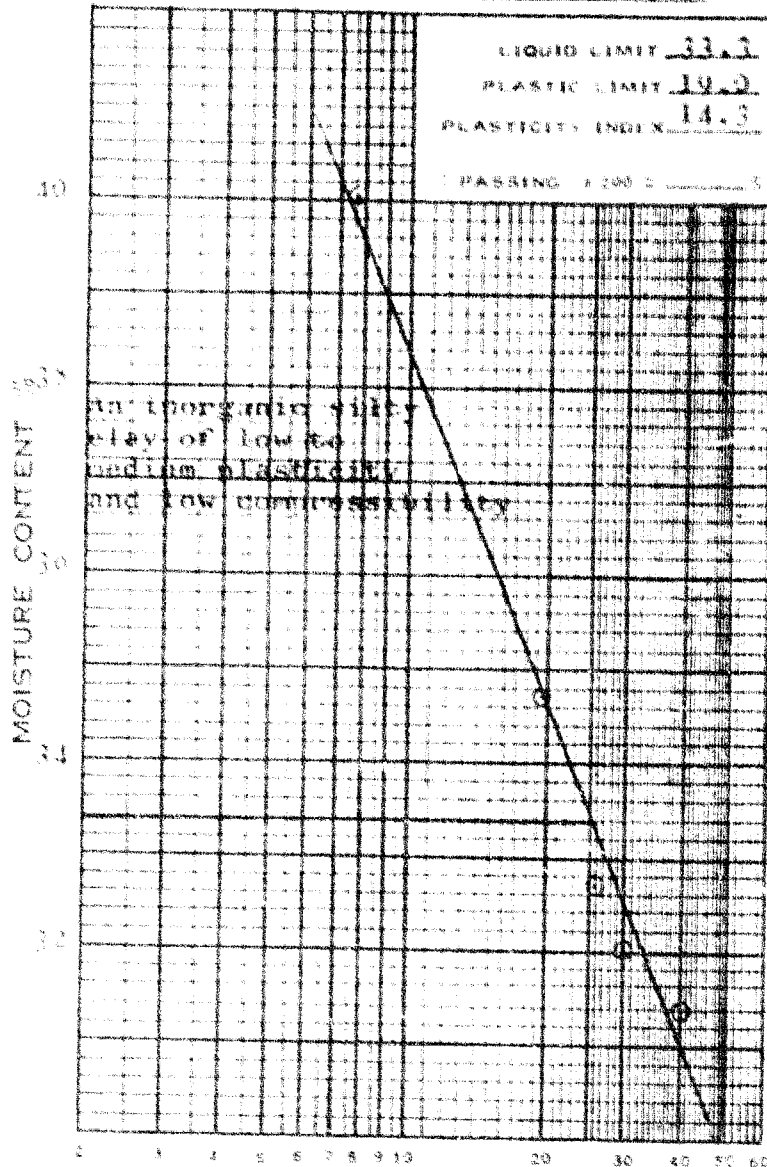
SAMPLE FROM Borehole 1 Sa. 2

DEPTH 20' - 21'

FLOW LINE CHARTS

SAMPLE FROM Borehole 14, Sa. 2

DEPTH 10' - 12'



NO. OF BLOWS (LOG SCALE)

BOREHOLE LOG





Checked By M.M.

[illegible]

e. m. peto associates ltd.
SOIL ENGINEERING SERVICE - TORONTO, ONTARIO
BOREHOLE LOG

Job Name Hwy. 401 Reisin River Bridge Job No. 5747 Borehole No. 2
Client Dept. of Highways of Ontario Casing BX Boring Date Jan. 20th - 22nd, 1958.
Datum D.H.O. Compiled By E. M. Peto Checked By M.M.

SAMPLE CONDITION

 **UNDISTURBED**
 **FAIR**
 **DISTURBED**
 **LOST**

SAMPLE TYPE

S.S. 2" STANDARD SPLIT TUBE SAMPLE
S.L. SPLIT BARREL WITH LINERS
S.T. THIN-WALLED SHELBY TUBE SAMPLE
W.S. WASH SAMPLE
R.C. ROCK CORE

ABBREVIATIONS

V.T. IN SITU VANE SHEAR TEST
Q.U. UNCONFINED COMPRESSIVE STRENGTH
W.L. WATER LEVEL IN CASING
W.T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Consistency	Depth (Feet)	Sample Type	Remarks
Ice Surface			0.0		
Water			0.0		
Sandy and clayey organic silt.	Olive Grey	Very loose	1.0	S.T. PUSHED	Q.U. 127 p.s.f. M.C. 41.0%
Organic silt some sand.	Mixed Brown-Grey	Very loose	1.2	S.S.	M.C. 70.0%
Matrix of clayey silt with numerous rock fragments.	Grey	Compact to Dense	1.6	S.T. PUSHED	M.C. 10.7%
As above with coarse sand (fragments to 1" size).	Grey	Compact to Dense	2.0	S.S.	
Layer of shale 25 - 26 ft.			2.5		
Coarse sand and fine gravel with binder.	Brownish Grey	Extremely Dense	3.0	W.S.	Chopped with open end A rod and chopping bit alternately from 25 to 30 ft.
Fine grained limestone with some fossils	Grey-Black	Hard	3.5	W.S.	
Fine grained shale from 37 ft. to 39 ft.	Black	Hard	3.7		Core recovery 82.4% from 30 ft. to 39 ft.
HOLE TERMINATED					

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Job Name Hwy. 401 Reisin River Bridge Job No. 5747

Client Dept. of Highways of Ontario Casing BX





Datum D.H.O. Compiled By L. M. Peto

Borehole No. 3

Boring Date Jan. 18th, 1958

Checked By M.M.

SAMPLE CONDITION

-  UNDISTURBED
-  FAIR
-  DISTURBED
-  LOST

SAMPLE TYPE

- S.S. 2" STANDARD SPLIT TUBE SAMPLE
- S.L. SPLIT BARREL WITH LINERS
- S.T. THIN-WALLED SHELLY TUBE SAMPLE
- W.S. WASH SAMPLE
- R.C. ROCK CORE

ABBREVIATIONS

- V.T. IN SITU VANE SHEAR TEST
- Q/u UNCONFINED COMPRESSIVE STRENGTH
- W.L. WATER LEVEL IN CASING
- W.T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOR	Consistency	Depth (Elevation)	Level	Sample No. and Location	Sample Type	No. of blows per ft.	WATER LEVEL AND MOISTURE & REMARKS
Ice Surface			0.0					
Water			1.2					
			3.0					
			5.0					
Sandy and clayey silt, organic content and seam of brown decayed wood.	Olive Grey	Very loose	12.0		1	S.T.	10	Q/u 187 p.s.f. M.C. 36.3%
			15.0		2	S.T.	12	Q/u 173 p.s.f. M.C. 80.3% dropping to 59.1% at 14 ft depth
Silty clay	Grey	Very soft	15.0		3	S.S.	6	M.C. 52.7%
Silty clay, slightly nuggety	Grey	Soft to Firm	22.0		4	S.S.	27	
Very silty clay, coarse sand and fine angular gravel. Some decayed wood.	Grey	Very stiff	25.0		5	S.S.	28	
Medium to coarse sand with fine angular gravel, clayey silt binder.	Grey-Black Light Brownish Grey	Compact to Dense	41.0		6	R.C.	-	Limestone fragment 2-1/4" recovered from tip of casing.

VIRTUAL REFUSAL

e. m. peto associates ltd.
SOIL ENGINEERING SERVICE - TORONTO, ONTARIO
BOREHOLE LOG

Job Name Hwy. 401 Raisin River Bridge Job No. 57147

Borehole No. 4





Client Dept. of Highways of Ontario Casing BX

Boring Date Jan. 17th - 18th, 1958.

Datum D.H.O. Compiled By E. M. Peto

Checked By M.M.

SAMPLE CONDITION


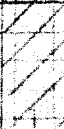




-  UNDISTURBED
-  FAIR
-  DISTURBED
-  LOST

SAMPLE TYPE

- S.S. 2" STANDARD SPLIT TUBE SAMPLE
- S.L. SPLIT BARREL WITH LINERS
- S.T. THIN-WALLED SHELBY TUBE SAMPLE
- W.S. WASH SAMPLE
- R.C. ROCK CORE

ABBREVIATIONS

- V.T. IN SITU VANE SHEAR TEST
- Q/u UNCONFINED COMPRESSIVE STRENGTH
- W.L. WATER LEVEL IN CASING
- W.T. GROUND WATER TABLE IN SOIL





SOIL DESCRIPTION	COLOR	Density or Consistency	Depth (feet)	Legend	Sample No. and Condition	Sample Type	Water Level (feet)	WATER LEVEL, SOIL MOISTURE & REMARKS
Ice Surface			0.0 152.4					
Water			5.0					
			14.0					
Silty clay, slightly nuggety texture.	Gray	Soft	20.0		1 	SS	4	M.C. 82.5% much wetter than Plastic Limit. Approx. Q/u 0.24 tons per sq. ft.
Clayey silt L.C. 33.3 P.L. 19.0 P.I. 14.3	Gray	Loose	22.0		2 	SS	5	M.C. 38.7%. Instantaneous response to shake test. Unable to drive sampler below 25 ft.
		Compact	26.0					
(Casing and chopping bit refused on layer of very hard shale or limestone)								

VIRTUAL REFUSAL

e. m. peto associates ltd.
SOIL ENGINEERING SERVICE - TORONTO, ONTARIO
BOREHOLE LOG

Job Name Hwy. 4 Reisin River Bridge Job No. 57th 47 Borehole No. 5
Client Dept. of Highways of Ontario Casing BX Boring Date Jan. 16th - 17th, 1958.
Datum D.H.O. Completed By E. M. Peto Checked By M.M.

SAMPLE CONDITION






 **UNDISTURBED**
 **FAIR**
 **DISTURBED**
 **LGST**

SAMPLE TYPE

S.S. 2" STANDARD SPLIT TUBE SAMPLE
S.L. SPLIT BARREL WITH LINERS
S.T. THIN-WALLED SHELBY TUBE SAMPLE
W.S. WASH SAMPLE
R.C. ROCK CORE

ABBREVIATIONS

V.T. IN SITU VANE SHEAR TEST
Q.C. UNCONFINED COMPRESSIVE STRENGTH
W.L. WATER LEVEL IN CASING
W.T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
Ice Surface			0 0 152.4					
Water			5 0 10 0 15 0					
Very silty fine to medium sand, some organic matter.	Dark Brownish Grey	Compact	20 0		1	SS	16	M.C. 30.5%
Medium to coarse sand, grits and small angular stones, some binder.	Dark Grey	Compact to Dense	25 0		2	SS	30	
Layer of hard shale.	Black		30 0		3	WS		Unab'e to drive split spoon more than 3"
Silty medium to coarse sand and fine angular gravel.	Grey-Brown	Compact to Dense	35 0		4	SS	38	Slight reaction with dilute hydrochloric acid
As above. Gravel to 1" fine grained dolomitic limestone.	Grey-Black	Hard	38 0		5	R.C.		Creavage slightly inclined from horizontal
As above with some fossils								
very fine grained shale from 37 to 38 ft.	Black	Hard						

HOLE TERMINATED

BOREHOLE LOG

Checked By M.M.

ABBREVIATIONS

Y. T. IN SITU VANE SHEAR TEST

9.4 UNCONFINED COMPRESSIVE STRENGTH

W T GROUND WATER TABLE IN

W. T. GROUND WATER TABLE IN SOIL





R. C. ROCK CORE

HOLB TERMINATED

e. m. peto associates ltd.
SOIL ENGINEERING SERVICE - TORONTO, ONTARIO
BOREHOLE LOG

Job Name Hwy. 401 Kaisin River Bridge Job No. 57147 Borehole No. 7
Client Dept of Highways of Ontario Casing BX Boring Date Jan. 14th - 15th, 1958.
Datum D.H.O. Compiled By E. M. Peto Checked By M.M.

SAMPLE CONDITION









 UNDISTURBED
 FAIR
 DISTURBED
 LOST

SAMPLE TYPE

S.S. 2" STANDARD SPLIT TUBE SAMPLE
S.L. SPLIT BARREL WITH LINERS
S.T. THIN-WALLED SHELBY TUBE SAMPLE
W.S. WASH SAMPLE
R.C. ROCK CORE

ABBREVIATIONS

V.T. IN SITU VANE SHEAR TEST
Q.C. UNCONFINED COMPRESSIVE STRENGTH
W.L. WATER LEVEL IN CASING
W.T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth, Elevation	Legend	Disturb. Condition	Sample Type	No. of Blows per Ft.	WATER LEVEL, SOIL MOISTURE, & REMARKS
Ice Surface			0' 0"					
			162.4					
Water			5' 0"					
			10' 0"					
			15' 0"					
			15' 0"					
Medium to coarse sand many angular rock fragments some binder As above rock fragments up to 1-1/4"	Grey	Compact to Dense			1	S.S.	30	Quite moist.
	Grey	Compact to Dense			2	S.S.	27	Quite moist.
			20' 0"					
Silty coarse sand with considerable angular fine gravel. As above.	Grey-Black	Compact			3	S.S.	19	
	Grey-Black	Dense			4	S.S.	53	Quite moist
			25' 0"					Compact only from 23 ft. to 25 ft.
Silty fine to coarse sand and gravel to 1-1/2"	Grey	Dense			5	S.S.	57	
	Grey	Very dense			6	S.S.	80	
Medium to coarse sand.	Grey-Black	Extremely Dense			7	W.S.	-	Casing refused at 27 ft. Drilled with open end A rod from 27 ft. to 31 ft.
Fragmented shale.	Black	Extremely Dense	31' 0"		8	W.S.	-	





HOSE TERMINATED

e. m. peto associates ltd.
SOIL ENGINEERING SERVICE - TORONTO, ONTARIO
BOREHOLE LOG

Job Name Hwy. 401 Raisin River Bridge Job No. 5747
Client Dept. of Highways Of Ontario Casing BX
Datum D.H.O. Compiled By E. M. Peto

Borehole No. 8
Boring Date Jun. 13th 1958
Checked By M.M.

SAMPLE CONDITION














-  UNDISTURBED
-  FAIR
-  DISTURBED
-  LOST

SAMPLE TYPE

- S.S. 2" STANDARD SPLIT TUBE SAMPLE
- S.L. SPLIT SAMREL WITH LINERS
- S.T. THIN-WALLED SHELBY TUBE SAMPLE
- W.S. WASH SAMPLE
- R.C. ROCK CORE

ABBREVIATIONS

- V.T. IN SITU VANE SHEAR TEST
- Q.C. UNCONFINED COMPRESSIVE STRENGTH
- W.L. WATER LEVEL IN CASING
- W.T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOR	Consistency	Depth (feet)	Legend	Sample No.	Sample Type	Moisture (%)	WATER LEVELS, SOIL MOISTURE & REMARKS
Ice surface.			0' 0"					
			15' 4"					
			5' 0"					
			10' 0"					
			14' 0"					
Very silty clay, many grits and angular pebbles.	Grey	Stiff	20' 0"		1	S.S.	18	Saturated. Fair response to shake test.
Fine to coarse sand, many grits and angular rock fragments, some binder.	Grey	Compact to Dense	20' 0"		2	S.S.	34	
	Grey	Compact to Dense	20' 0"		3	S	27	quite moist. Stratum of loose to compact coarse to fine sand from 23 ft. to 25 ft.
Medium to coarse sand	Grey-Black	Loose to Compact	20' 0"		4	S.S.	11	
Fine to coarse sand, many grits and stones, considerable binder.	Grey	Dense	20' 0"		5	S.S.	52	
Pulverized shale	Black	Soft	20' 0"		6	S.S.	10	
Fine grained shale with thin bands of limestone	Black	Hard	33' 0"			R.C.		Chopped to 33 ft.
Dolomitic limestone	Grey-Black	Hard	35' 0"					
with some fossils 34-1/2 to 36-1/2 ft.	Black							No reaction with weak hydrochloric acid.
Fine grained shale	Black							73% core recovery. Core very badly broken up. Drift hole did not stand up well.
36-1/2 to 37-1/2 ft.								
Medium grained limestone	Grey-Black	Very hard	42' 0"					
Some very thin black layers of fossiliferous limestone	Black		42' 0"					

HOLE TERMINATED

e. m. peto associates ltd.
SOIL ENGINEERING SERVICE - TORONTO, ONTARIO
BOREHOLE LOG

Job Name Hwy. 401 Raisin River Bridge Job No. 57147

Borehole No. 9





Client Dept. of Highways of Ontario Casing B.X.

Boring Date Jan. 12th - 12th, '958

Datum D.H.O. Compiled By E. M. Peto

Checked By M.M.

SAMPLE CONDITION

 **UNDISTURBED**
 **FAIR**
 **DISTURBED**
 **LOST**

SAMPLE TYPE

S.S. 2" STANDARD SPLIT TUBE SAMPLE
S.L. SPLIT BARREL WITH LINERS
S.T. THIN-WALLED SHELLY TUBE SAMPLE
W.S. WASH SAMPLE
R.C. ROCK CORE

ABBREVIATIONS

V.T. IN SITU VANE SHEAR TEST
Q.U. UNCONFINED COMPRESSIVE STRENGTH
W.L. WATER LEVEL IN CASING
W.T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOR	Consistency	Depth (ft.)	Remarks	Sample Type	Number of Tests	Notes
Ice Surface			0				
Water			5				
			12				
			14				
Silty clay, nuggety	Grey	Very soft	17		1	SS	3
Sandy and silty clay, many grits and angular rock fragments.	Dark Grey	Firm to Stiff	20		2	SS	12
As above	Dark Grey	Very stiff	22		3	SS	32
Medium to coarse sand, angular rock fragments	Grey	Dense	25		4	SS	45
Coarse sand and fine gravel in matrix of light grey silt.	Grey	Dense	28		5	SS	52
As above.	Grey	Very dense	30		6	SS	98
As above	Grey	Dense	34		7	SS	34 For 6
2" of black shale above fine grained limestone, some fossils.	Grey-Black	Hard	35			R.C.	Boulder at 30'6". C hop to 32 ft. Ran rod to 33 f Shale cleavage from 33 ft to 33'3".
7" of black shale at bottom of core.							Lost wash water at 35 ft.
Limestone	Grey-Black	Hard	40			R.C.	87.5% core recovery.

HOLE TERMINATED

BOREHOLE LOG

Checked By M.M.

3. GROUND WATER TABLE IN SOIL

[illegible]

BOREHOLE LOG

Borehole No. 11

Boring Date Dec. 22nd - 23rd, 1957

Checked By M. M.

SAMPLE TYPE

ABBREVIATIONS

 UNDISTURBED

 FAIR

☒ DISTURBED

LOST

5.5. 2nd STANDARD SPLIT TUBE SAMPLE

5.2. SPLIT BARREL WITH LINERS

S. T. THIN-WALLED SHELBY TUBE SAMPLE

W.S. WASH SAMPLE

R. C. ROCK CORE

V. T. IN SITU VANE SHEAR TEST

Q/V UNCONFINED COMPRESSIVE STRENGTH

W. L. WATER LEVEL IN CASING

W. T. GROUND WATER TABLE IN SOIL

[illegible]

BOREHOLE LOG

Checked By M.M.

ABBREVIATIONS

W. T. GROUND WATER TABLE IN SOIL

B.C. ROC

[illegible]

BOREHOLE LOG

Checked By M. M.

[illegible]

BOREHOLE LOG

Checked By M. M.

ABBREVIATIONS

W.T. GROUND WATER TABLE IN SOIL.

SOIL DESCRIPTION	COLOR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
Ground Surface	-	-	0' - 8" 153.6	[Symbol]				
Organic silt. Clay and silty fine sand	Black Grey-Brown	Very loose Loose	5' - 9'	[Symbol]	X	S.T. PUSHED		Super saturated 50% recovery from tube.
Sandy organic silt	-	-	10' - 3"	[Symbol]				M.C. 28.5%
Silty and sandy clay - Pockets of organic silt	Light Brown Grey-Brown	Very soft	13' - 9"	[Symbol]	2 [Symbol]	S.T. PUSHED		Q/U 104 p.s.f. L.L. 35.5 M.C. 87.5% P.W. 20.6 P.I. 14.9
Sandy silt with considerable angular gravel	b'l'e Grey	Dense	16' - 0"	[Symbol]	3 X	G.S.	39	Very moist.
Pulverized shale	Grey-Black	-	17' - 0"	[Symbol]	4 X	w.s.	-	
				VIRTUAL REFUSAL				

BOREHOLE LOG

Checked By M.M.

- V.T. IN SITU VANE SHEAR TEST
Q/u UNCONFINED COMPRESSIVE STRENGTH
W.L. WATER LEVEL IN CASING
W.T. GROUND WATER TABLE IN SOIL

[illegible]

OFFICE LOCATION —

DOWNSVIEW AVE.,

KEELE ST. — HIGHWAY 401

TORONTO, ONTARIO.



ONTARIO

DEPARTMENT OF HIGHWAYS

POSTAL ADDRESS —

DEPARTMENT OF HIGHWAYS,

PARLIAMENT BUILDINGS,

TORONTO 5, ONTARIO.

Road Design Division, Toronto, Ontario.
February 8, 1961.

MEMORANDUM FOR:

Mr. A. Rutka,
Materials & Research Engineer,
Department of Highways,
Laboratory Building #2,
DOWNSVIEW, Ontario.

ATTENTION: L. Soderman

RE: W.P. 51-59-2 G.D. Hwy. 401 Summersown to Lancaster

The approaches for the interchange at Highways 2 and 34 (W.P. 108-59) at Lancaster are being included in the project but the structure will not be done until about a year later.

Due to anticipated settlements of the approaches, you requested to be advised when the contract will be awarded in order that you can make arrangements to instrument the fills. I checked with Mr. H. Mosher who advised that the tentative advertising date is September, 1961, and will likely be awarded on November 20, 1961.

We are stipulating that the placing of the approaches at Highways 2 and 34 shall be one of the contractor's first operations but it does not appear likely that the contractor would begin grading operations until the spring of 1962.

[Signature]
S. J. Markiewicz
PROJECT DESIGN ENGINEER

SJM/bcc
c. c. Mr. J. Gruspier

Mr. A.G. Stermac,
Principal Foundations ENgineer,
Room 107, Lab-Bldg..

Mr. L.A. Walker,
District Engineer,
OTTAWA, Ontario.

P. McHatt

Att. Mr. G.A. Hotcalfo

September 21, 1962

108-19
Contract #62-158,
Grading, Granular Base, Concrete Paving,
Hot Mix Paving and Structures and
Approaches,
From Toll Gate Road at Cornwall to one
mile E. of Lancaster,
 Hwy 1401, District #9.

This will confirm a telephone conversation with Mr. H.L. Fraser on the 19th of September. Work may proceed on the abutment footings of the Hwy #2 and Hwy. #34 Interchange immediately. This is contrary to the intent of the Special Provisions. The structures at County Road No. 26 Interchange at Cornwall E. Limits and the County Road to Sumnerstown 4.6 miles west of Hwy #34 may also proceed. It is expected settlement will be a problem at all three structures but particularly at the Hwy #2 and Hwy. #34 Interchange. Treatment will be required at the abutments as and when this settlement takes place. At the moment, it is not felt that any special treatment will be needed but please contact us when there is an appreciable settlement, say about one foot.

Please refer to copy of letter from Mr. T.C. Muir to Wilson Concrete Products Limited dated July 25th and referring to contract #62-78. As far as I can determine "Wilson" have not confirmed agreement of the alteration of completion date to August 15th 1963. I would suggest you estimate when the prestressed beams will be required and contact Mr. T.C. Muir.

P. McHatt

PMck/an

P. McHatt,
Bridge Construction Liaison Engineer.

C.C. H. Tregaskes
A.G. Stermac
S. Davis
T.C. Muir
L. Macie

MISCELLANEOUS DETAIL SHEET

(DO NOT USE FOR GRADING QUANTITIES ETC.)
OR FOR SCRATCH PAD USE

SHEET NO. 1 OF 1 DATE Sept 16/62
CONTRACT NO. 62-15B Hwy 2 & 34 Interchange ITEM NO. (92) Driving Steel A Piles
LOCATION OF MATERIAL ETC. A Piles Centre Pier (See book "26 page 5")

			Pile #	Length	UNIT
C1	I	I D1	C1	42' 10"	
C2	I	I D2	C2	42' 9"	
C3	I	I D3	C3	42' 8"	
C4	I	I D4	C4	42' 8"	
C5	I	I D5	C5	42' 9"	
C6	I	I D6	C6	43' 7"	
C7	I	I D7	C7	43' 8"	
C8	I	I D8	C8	42' 10"	
C9	I	I D9	C9	42' 11"	
C10	I	I D10	C10	42' 11"	
C11	I	I D11	C11	43' 0"	
			D1	43' 3"	
			D2	43' 1"	
			D3	40' 11"	
			D4	41' 7"	
			D5	43' 2"	
			D6	43' 5"	
			D7	43' 7"	
			D8	43' 2"	
			D9	43' 7"	
			D10	43' 3"	
			D11	43' 6"	
Total length Driven 945' 1"					

DETAILED BY Ref. 2 in 11

CHECKED BY Paul Poy

W.J. 61-F-109W.P. 108-59.Nov. 1st, 1961.INSTRUMENTATION OF APPROACHES

An Instrumentation Programme is proposed at the site of the above mentioned structure approach fill. Future fill heights will be in the order of 22'. The foundation report was prepared by H. G. Acres and settlements in the order of 6.0' have been computed. It is now proposed to measure the actual settlement at a point some 50' behind the abutment on the east side. Steel plates will be placed on the original ground just before placing the fill material. When the fill is completed, steel pipes will be drilled down to a contact with each plate. At the same time, it is proposed to place 3 piezometers at each of three locations adjacent to the proposed settlement plates. The piezometers will be placed well in advance of the approach fill. Present intentions are that the fill will be started in the Spring of 1962. The piezometers will be placed early in November 1961.

K. G. Selby.

Mr. A. M. Toya,

September 2, 1960.

Bridge Engineer.

FOUNDATION INVESTIGATION REPORT

Materials & Research Section.

by: H. G. Acres & Company, Limited

Attention: Mr. S. McCombie.

Re: Proposed Crossing, Hwy. 401 & Hwy. 2,
1½ Miles South of Lancaster, Twp. of
Charlottenburg, District No. 9,
Bridge No. 11 - W.P. 138-574-GRADING

108-59-
F.D.N.

Enclosed herewith, are two copies of the foundation report for the above site, submitted by H. G. Acres. The results of the investigation at this site show that the general soil profile consists of 35 to 40 ft. of soft, sensitive marine clay overlying glacial till.

We have reviewed the data presented in Acres' report and checked the stability calculations which they have carried out, and are in agreement with their general conclusions.

We would like to emphasize the following points:-

1. For embankment types not exceeding 24 ft., a standard cross-section using 2:1 slopes can be safely constructed. A safety factor for this fill has been computed as 1.4 against a general base failure.
2. Settlements resulting from consolidation of the foundation soil underlying the embankments, are expected to be at least 12 inches. Experience with fills constructed on marine clay deposits similar to that which exists at this site, indicates that 35% of the total estimated settlement will take place in the first year. Because of this relatively rapid rate of consolidation, it is our recommendation that embankment fills be built as long in advance of the scheduled date of contract for the structure, as practicably possible.

cont'd. /2 ...

3. The structure should be founded on 'H' piles bearing on the underlying bedrock formation. Due to the overstressing of the foundation soil which will result from the construction of the embankments, it is imperative that the fills be placed first.

If the embankment heights are in excess of 24 ft., then consideration will have to be given to the use of berms. The berm sections given in the report attached, should not be used without approval by the Foundation Section.

If you have any queries with regard to the contents of the consultants' report, or our foregoing comments, please do not hesitate to contact our Office.

L. G. Coderman (singly)
for

LGS/MdeF
Attach.

L. G. Coderman,
PRINCIPAL FOUNDATIONS ENGINEER

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
D. G. Ramsay
J. Ford
L. E. Walker
J. E. Crispier
A. Watt

Foundations Office ✓
Gen. Files.

23-62-158.

W.P. 108-59

Counter No 61-18

GRADING W.P.

138-57

51-59-2

ONTARIO DEPARTMENT OF HIGHWAYS
Toronto, Ontario

REPORT
on
FOUNDATION INVESTIGATION

PROPOSED CROSSING
HIGHWAY NO. 401 AND HIGHWAY NO. 2
1-1/2 MILES SOUTH OF LANCASTER
TOWNSHIP OF CHARLOTTENBURG, DISTRICT NO. 9
BRIDGE NO. 11, WP ~~138-57~~

~~108-59~~
~~108-54~~

H.G. ACRES & COMPANY LIMITED
Consulting Engineers
Niagara Falls, Canada

July, 1960

ONTARIO DEPARTMENT OF HIGHWAYS
Toronto, Ontario

REPORT
on
FOUNDATION INVESTIGATION

PROPOSED CROSSING
HIGHWAY NO. 401 AND HIGHWAY NO. 2
1-1/2 MILES SOUTH OF LANCASTER
TOWNSHIP OF CHARLOTTENBURG, DISTRICT NO. 9
BRIDGE NO. 11, WP ~~138-59~~
106-59

Table of Contents

Introduction

Exploratory Work

Site Conditions and Soil Properties

Design Considerations

Conclusions

Recommendations

Appendix A - Program of Work

Appendix B - Summary of Field Vane Test Results

Appendix C - Summary of Laboratory Test Results

Appendix D - List of Plates

ONTARIO DEPARTMENT OF HIGHWAYS
Toronto, Ontario

REPORT
on
FOUNDATION INVESTIGATION

PROPOSED CROSSING
HIGHWAY NO. 401 AND HIGHWAY NO. 2
1-1/2 MILES SOUTH OF LANCASTER
TOWNSHIP OF CHARLOTTENBURG, DISTRICT NO. 9
BRIDGE NO. 11, WP ~~106-57~~
106-54
108-53

Introduction

The present plans of the Department for Highway No. 401 include an overpass structure to carry Highway No. 2 over Highway No. 401 south of the Town of Lancaster. At the request of the Ontario Department of Highways, soil explorations were carried out by H.G. Acres & Company Limited to determine the foundation conditions for the overpass structures and for the approach embankments. The F.E. Johnston Drilling Company Limited was retained to perform the soil drilling and field sampling under the supervision

of Mr. J.A. MacLeod of H.G. Acres & Company Limited. Field work commenced on June 18, and was completed on June 24, 1960.

Exploratory Work

The exploratory work consisted of drilling and sampling five holes, Nos. 889-1 to 889-5 inclusive, the locations of which are shown on Plate I. Two diamond drills were used for the explorations, and the holes were supported by BX casing. Samples of clay were obtained with the use of 2-inch diameter thin-walled tubes and vane tests were performed to measure the in situ shear strength of the clay. Samples of sandy soil were obtained with a 2-inch diameter split-spoon sampler. Bedrock was proved by diamond drilling in holes No. 889-2 and No. 889-4 and AXT core was recovered.

The progress of work is outlined in Appendix A and the drilling reports are given on Plates II to VI inclusive.

Site Conditions and Soil Properties

The site is located on Highway No. 2 near Lancaster, Ontario and approximately one mile from the St. Lawrence River, which in this stretch is known

as Lake St. Francis. It is within the Ottawa-St. Lawrence Lowland, an area which was extensively glaciated and subsequently inundated by the marine Champlain Sea.

The materials which were encountered in the exploratory holes are described in the attached drilling reports, Plates II to VI inclusive. The soil stratigraphy is shown on Plate I and comprises marine clay extending from the ground surface to depths of 35 to 40 feet and sandy till 5 to 10 feet thick overlying a relatively level bedrock surface.

(a) - Clay - The clay was deposited under marine conditions and subsequently uplifted to an elevation at which it has been leached by fresh water and weathered.

From the results of three determinations, it has been found that the liquid limit ranges from at least 47 to 66 per cent and the plastic limit ranges from 26 to 31 per cent. The liquidity indices for these same three samples varied from 1.5 to 2.3 which indicates that the natural soils are extremely sensitive.

The natural shear strength of the clay was found to be relatively uniform, except near the ground

surface where it was somewhat greater than average. The natural undrained shear strength of the uppermost 10 feet of the clay is generally greater than 0.5 tons per square foot, whereas below this depth the average shear strength is between 0.3 and 0.4 tons per square foot. The results of the field vane tests and the laboratory compression tests agree quite well, except at depths of the order of 30 feet where the vane tests gave greater strengths. It is believed that disturbance of the tube samples took place during the drilling and sampling operation, and that this partial remoulding of the samples has resulted in lowering of the natural undisturbed strengths. The results of the vane tests and the laboratory tests are summarized in Appendix B and Appendix C, respectively, and these data are presented graphically on Plate X.

The results of three consolidation tests are shown on Plates VII to IX inclusive, and are compared on Plate XI. The apparent maximum consolidation pressures to which the samples have been naturally subjected, have been estimated and, as shown on Plate XI, it would seem that the clay is slightly overconsolidated. This is also implied by the high natural shear strengths as compared with the shear strength of

- 5 -

a normally-consolidated clay having an $\left(\frac{Su}{P}\right)$ ratio equal to 0.30, as shown on Plate XI. The value of the coefficient of consolidation, C_v , for the stress range in which the clay will be acting, was found to be 6.8 feet per year.

(b) - Till - Below the clay is a granular till which overlies bedrock. This till varies in thickness from 5 to 10 feet over the entire site. It exists in a relatively dense condition as indicated by the fact that the average number of blows in the standard penetration test was 16.

(c) - Bedrock - Bedrock was cored in two holes and was found to be a dark calcareous shale of the St. Martin formation; core recovery was good. The bedrock surface appears to be relatively level, as indicated on Plate I.

(d) - Ground Water Conditions - A porous tip piezometer was installed in hole No. 889-2 at a depth of 25 feet. The elevation of the ground water table was found by means of this piezometer at elevation 154 feet as compared with the ground surface elevation of 156 feet. A ground water surface measurement was also made in hole No. 889-1 where the water in the underlying till was allowed to rise up in the casing. The

- 6 -

elevation of the ground water surface in this hole was also found to be 154 feet.

Design Considerations

(a) - Bearing Capacity

Embankment - In considering the stability of the embankment for the case of a bearing capacity failure, it has been assumed that the embankment fill acts only as a load and has zero shear strength. In fact, however, the shear strength of the embankment fill will in all probability be much greater than that of the foundation, but in order to mobilize this shear strength, large shear strains must take place. In contrast, the foundation soil is an extremely sensitive clay which fails at very small shear strains, and with additional strains its shear strength reduces to a very small value. At the contact between embankment and foundation soil there must be compatibility of strain; therefore, for strains at which the shear strength of the subsoil is fully mobilized, the mobilized shear strength of the embankment fill is small. It is for this reason that it has been assumed that the embankment fill does not contribute any stabilizing influence in the case of a deep foundation failure.

The minimum cross section of the embankment which has been considered is shown on Plate XIII; the crest width is 54 feet and the slopes are assumed to be 2 to 1. Average shear stresses developed along circular-arc failure surfaces were determined, and the results of these calculations are given on Plate XIII. The maximum shear stress was found to be 540 pounds per square foot and this was developed along a deep failure surface which is tangential to the underlying dense till. The average shear strength of the subsoil is 750 pounds per square foot and, therefore, the minimum safety factor against ultimate failure is $\frac{700}{540} = 1.4$.

Embankment cross sections were also designed using safety factors of 1.5 and 1.6. These cross sections, together with the cross section for a safety factor of 1.4, are shown on Plate XIV. It can be seen that berms 40 feet wide and 8 feet high are required to provide a safety factor of 1.6.

From the results of the calculations, it follows that the design of the embankment cross section is dependent upon the choice of the safety factor. In our opinion, a safety factor of 1.4 is adequate. We

realize that there is a possibility that the shear strength of the sensitive clays will decrease if the clay consolidates very slowly, but we believe that this effect of reducing the safety factor will be counterbalanced by such unconsidered factors as the strength of the embankment fill and the high strength of the clay crust.

Bridge Piers - The average natural shear strength of the upper 15 feet of the foundation soil is approximately 900 pounds per square foot. Using a safety factor of 2.5 against ultimate shear failure, the allowable net pressures which could be transferred to the soil at the base of a footing is 1.05 tons per square foot. Assuming a bridge pier load of 50 tons per foot length of pier, the footing required to support this load would have the impracticable width of 8 feet.

The logical alternative is the use of bearing piles driven into the till or to the bedrock surface. Piles of small displacement such as steel H-sections, would be preferable in order to reduce the amount of disturbance to the clay.

(b) - Settlement - The settlement of the end abutments of the bridge and of the embankment will be

primarily governed by the embankment loading. The loading conditions which have been assumed are shown on Plate XV. The consolidation characteristics of the clay are given on Plates VII to X inclusive, and are summarized on Plate XII. The value of the apparent modulus of elasticity which has been used to predict the immediate settlements, is 80 tons per square foot. The predicted settlements are listed on Plate XV and it can be seen that they are extremely large. These settlements were calculated by using currently accepted analytical and laboratory procedures, but we consider them to be larger than the probable actual settlements. However, since there appears to be no valid reason for disregarding them on the basis of experience, they are included in this report.

It has been determined that 50 per cent of the consolidation settlement will take place within 10 years after construction.

Conclusions

(a) - On the basis of the drilling work, which was done at the site, the general soil profile consists of 35 to 40 feet of sensitive marine clay and 5 to 10 feet of dense sandy till overlying bedrock. The

- 10 -

ground level was found to be at elevation 154 feet; i.e., 2 feet below the ground surface.

(b) - The clay is slightly overconsolidated and is extremely compressible because of its sensitive nature. Its average natural undrained shear strength is approximately 750 pounds per square foot, although a somewhat stiffer surface crust exists.

(c) - The stability of the approach embankments was considered assuming a height of 24 feet, a crest width of 54 feet, and side slopes of 2 to 1. For safety factors of 1.6, 1.5 and 1.4, against foundation failure, 8-foot high berms of 40 feet, 16 feet, and zero feet widths, respectively, are required.

(d) - The allowable net bearing pressures which can be used for footings are 1.05 tons per square foot.

(e) - Large settlements below the embankment fills, ranging from 6.0 feet below the centre of the embankment to 0.2 feet at the toe of the embankment nearest Highway No. 401 have been calculated but are considered to be much greater than the probable actual settlements. Fifty per cent of the ultimate consolidation settlement will take place within approximately 10 years of the end of construction.

- 11 -

Recommendations

(a) - We are of the opinion that a safety factor of 1.4 is adequate to ensure the stability of the embankment fills, considering that in the analyses the shear strength of the fill has been completely ignored and the higher shear strengths of the surface crust have not been taken into account. Therefore, we believe that the embankment can be built to full height without the use of berms if the slopes are 2 to 1.

(b) - The allowable net bearing pressure which can be used for footing design is 1.05 tons per square foot. For normal bridge pier loads, this low bearing pressure would result in impracticable footing sizes. We, therefore, recommend that the piers be supported by bearing piles which could be driven into the till and possibly to the bedrock surface. Piles which have small displacement volumes, such as steel H-sections, are preferable.

(c) - Settlements as large as six feet below the embankments have been predicted on the basis of currently accepted methods of calculation. While we consider the magnitudes of these settlements to be unreasonably large, we do believe that large settlements, possibly,

in terms of feet, will occur, and, therefore, in the design of the bridge structure, these settlements must be taken into account.

(d) - Because shearing strains in the foundation are inevitable when embankment loads of this magnitude are placed upon relatively soft soils, we recommend that the embankment be built before the bridge structure.

We also recommend that if piles are used to support the centre piers of the bridge, they should be of high capacity and low displacement so that the foundation soils will be disturbed as little as possible during the pile driving. This is especially important for the piers at the toes of the embankments.

(e) - Vertical retaining-wall bridge piers founded on the natural overburden should not be considered if their height is greater than 10 feet.

APPENDIX AProgram of Work

- June 18, 1960 - Diamond drill arrived at site and started hole No. 889-1.
- June 20, 1960 - Second diamond drill arrived at site and started hole No. 889-2.
- June 21, 1960 - Hole No. 889-1 was completed to a depth of 42 feet. Hole No. 889-3 was started.
- June 22, 1960 - Holes No. 889-2 and No. 889-3 were completed to depths of 49 feet and 47 feet respectively. Hole No. 889-4 was started.
- June 23, 1960 - Hole No. 889-5 was started.
- June 24, 1960 - Holes No. 889-4 and No. 889-5 were completed to depths of 46 feet and 41 feet respectively.

Summary of Work

<u>Work Type</u>	<u>No. of Holes</u>	<u>Total Length Feet</u>	<u>Total Time Machine - Days</u>
Soil Drilling and Sampling	5	215	11

APPENDIX BSummary of Field Vane
Test Results

Hole No.	Elevation Feet	Undrained Shear Strength Tsf		Sensitivity
		Natural	Remoulded	
889-1	147.3	0.49	0.14	3.6
	141.5	0.47	0.09	5.4
	136.8	0.44	0.10	4.3
	131.5	0.71	0.20	3.6
	125.3	0.35	0.20	1.8
889-2	148.0	0.99	0.21	4.8
	143.5	0.27	0.09	3.1
	139.5	0.35	0.14	2.5
	136.0	0.34	0.14	2.5
	131.5	0.32	0.07	4.6
	124.0	0.34	0.14	2.4
889-3	144.4	0.43	0.13	3.4
	138.9	0.36	0.06	6.5
	131.4	0.37	0.06	6.7
	126.9	0.68	0.19	3.7
	121.4	0.38	0.08	4.8
889-4	147.5	0.74	0.12	6.1
	143.2	0.38	0.06	6.3
	139.4	0.40	0.09	4.3
	135.7	0.45	0.11	4.0
	131.5	0.27	0.08	3.3
	127.5	0.46	0.10	4.5
889-5	144.9	0.44	0.10	4.4
	139.0	0.40	0.07	5.4
	134.9	0.50	0.09	5.6
	129.0	0.47	0.09	5.3

APPENDIX CSummary of Laboratory
Test Results

Hole No.	Sample No.	Elevation Feet	Water Content	Liquid Limit	Plastic Limit	Su_n Tsf	ef %	Su_r Tsf	St
889-1	1	150	64.5	-	-	0.80	5.5	0.12	
	3	140	76.9	-	-	0.30	3.0	0	>20
	5	128	67.9	-	-	0.20	7.0	0	>20
889-3	1	151	78.4	51.3	26.5	0.26	7.0	0	>20
	3	141	72.5	46.8	26.7	0.27	5.0	0	>20
	5	129	83.5	66.0	31.2	0.20	5.0	0	>20
	6	125	75.5	70.4	33.1				
889-5	1	152	70.9	-	-	0.57	2.5	0.07	8.1
	2	148	60.8	-	-	1.14	3.5	0.15	7.6
	3	142	71.5	-	-	0.39	2.0	0	>20

Su_n - Natural undrained shear strength.

Su_r - Remoulded undrained shear strength.

ef - Failure strain.

St - Sensitivity.

APPENDIX DList of Plates

- Plate I - Exploratory Holes, Plan and Section.
- Plate II - Drilling Report, Hole No. 889-1.
- Plate III - Drilling Report, Hole No. 889-2.
- Plate IV - Drilling Report, Hole No. 889-3.
- Plate V - Drilling Report, Hole No. 889-4.
- Plate VI - Drilling Report, Hole No. 889-5.
- Plate VII - Consolidation Test, Hole No. 889-3,
Sample Elevation 151 Feet.
- Plate VIII - Consolidation Test, Hole No. 889-3,
Sample Elevation 141 Feet.
- Plate IX - Consolidation Test, Hole No. 889-3,
Sample Elevation 129 Feet.
- Plate X - Consolidation Test, Hole No. 889-3,
Sample Elevation 125 Feet.
- Plate XI - Summary of Drilling and Test Results,
Comparison of All Tests.
- Plate XII - Consolidation Test - Comparison of
All Tests.
- Plate XIII - Summary of Stability Analyses for the
Case of an Embankment without Berms.
- Plate XIV - Comparison of Embankment Cross Sections
Showing the Effect of Varying the
Safety Factor.
- Plate XV - Foundation Settlements Due to Embankment
Load.

DRILLING REPORT

CLIENT Ontario Department of Highways JOB No. 889
 PROJECT W.P. 138-57 HOLE No. 889-1
 SITE Highway 401, Highway 2, Lancaster, Ontario SHEET No. 1 OF 2

CONTRACTOR: F.E. Johnston Drilling Company Limited STARTED 10:00 A.M. June 18 19 60
 FINISHED 1:30 P.M. June 21 19 60
 METHOD SOIL Modified Wash Boring CASING DIAM. BX
 OF
 DRILLING: ROCK Diamond Drill CORE DIAM. AXT

LOCATION: LATITUDE Ch. 507+73 ELEVATIONS: DATUM G.S.C.
 DEPARTURE 5 Feet Left DRILL PLATFORM
 BEARING GROUND SURFACE 155.8
 INITIAL DIP 90 Degrees ROCK SURFACE
 OTHER DIPS BOTTOM OF HOLE 114.3
 WATER TABLE 154.0

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE					PENETRATION TEST
			NO	TYPE*	SIZE	DEPTH	RET'D	
Feet					Inches	Feet	Inches	Blows*
0.0	Clay	Light brown with darker brown spots, weathered, stiff						
3.0	Clay	Grey-green with brown spots stiff.	1	30	2	5.0 6.5	18	Pushed
8.0	Clay	Grey-blue medium stiff becoming softer with depth	Vane Test			8.5		
			2	30	2	11.0 12.5	18	Pushed
			Vane Test			14.3		
			3	30	2	15.0 16.7	17	Sank by own weight
			Vane Test			17.0		
			4	30	2	21.0 22.7	24	Sank by own weight
			Vane Test			24.3		
			5	30	2	27.0 28.8	24	Pushed

SAMPLING METHOD

* A - SPLIT TUBE
 B - THIN WALL TUBE
 C - PISTON SAMPLER
 D - CORE BARREL

E - AUGER
 F - WASH

SHIPPING CONTAINER

N - INSERT
 O - TUBE
 P - WATER CONTENT TIN
 Q - GLASS JAR

R - CLOTH BAG
 S - PLIOFILM BAG
 Z - DISCARDED

INSPECTOR J. MacLeod
 LOGGED BY J. MacLeod

APPROVED *D.H. Macdonald*
 DATE July, 1960

DRILLING REPORT

CLIENT Ontario Department of Highways

JOB No. 889

PROJECT W.P. 138-57

HOLE No. 889-1

SITE Highway 401, Highway 2, Lancaster, Ontario

SHEET No. 2 OF 2

DEPTH Feet	SOIL TYPE	DESCRIPTION COLOUR CONSISTENCY STRUCTURE WATER CONTENT PLASTICITY COMPACTION WATER LOSS OR GAIN ETC.	S A M P L E					PENETRATION TEST Blows*
			NO.	TYPE	SIZE Inches	DEPTH Feet	RETD Inches	
					Vane Test	30.5		
33.0	Till	Grey sand, angular pebbles and stones, quite clayey	6	AQ	2	33.0		6
						33.5		3
						34.0		4
						34.5	5	
				A2	2	37.0		
						37.5		2-1/2
						38.0		4-1/2
						38.5	1/2	7
41.5		Possibly bedrock						
* - Penetration Test This is the number of blows of a 140-pound weight falling 30 inches required to advance the sampler to depth indicated.								

DRILLING REPORT

CLIENT Ontario Department of Highways JOB No. 889
 PROJECT W.P. 138-57 HOLE No. 889-2
 SITE Highway 401 and Highway 2, Lancaster, Ontario SHEET No. 1 OF 2
 CONTRACTOR: F.E. Johnston Drilling Company Limited STARTED 8:00 A.M. June 20 19 60
 FINISHED 10:30 A.M. June 22 19 60
 METHOD OF DRILLING: SOIL Modified Wash Boring CASING DIAM. BX and AX
 ROCK Diamond Drill CORE DIAM. AXT
 LOCATION: LATITUDE Ch. 567+72 ELEVATIONS: DATUM G.S.C.
 DEPARTURE 69 Feet Left DRILL PLATFORM
 BEARING GROUND SURFACE 156.0
 INITIAL DIP 30 Degrees ROCK SURFACE 115.0
 OTHER DIPS BOTTOM OF HOLE 107.0
 WATER TABLE 154.0

DEPTH Feet	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE					PENETRATION TEST Blows
			NO.	TYPE*	SIZE Inches	DEPTH Feet	RET'D Inches	
0	Clay	Brown-grey, weathered						
5.0	Clay	Blue, homogeneous, tenacious	1	30	2	5.0		
						6.5	18	Pushed
				Vane Test		8.0		
			2	30	2	9.0		
						10.5	18	Pushed
				Vane Test		12.3		
			3	30	2	13.0		
						14.5	18	Pushed
		Some shells in wash		Vane Test		17.0		
			4	30	2	17.5		
						19.0	18	Pushed
				Vane Test		20.0		
				Vane Test		24.5		
			5	30	2	29.0		
						30.5	15	Pushed

SAMPLING METHOD

* A — SPLIT TUBE
 B — THIN WALL TUBE
 C — PISTON SAMPLER
 D — CORE BARREL

E — AUGER
 F — WASH

SHIPPING CONTAINER

N — INSERT
 O — TUBE
 P — WATER CONTENT TIN
 Q — GLASS JAR

R — CLOTH BAG
 S — PLIOFILM BAG
 Z — DISCARDED

INSPECTOR J. Bateson

LOGGED BY J. MacLeod

APPROVED

A. H. MacDonald

DATE

July, 1960

H. G. ACRES & COMPANY LIMITED — CONSULTING ENGINEERS
 NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Ontario Department of Highways
 PROJECT W.F. 138-57
 SITE Highway 401 and Highway 2, Lancaster, Ontario

JOB No. 889
 HOLE No. 889-2
 SHEET No. 2 OF 2

DEPTH	SOIL TYPE	DESCRIPTION COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTION, WATER LOSS OR GAIN, ETC.	SAMPLE					PENETRATION TEST
			NO	TYPE	SIZE	DEPTH	RET'D	
Feet					Inches	Feet	Inches	Blows
				Vane Test		32.0		
				AZ	2	36.5		
						37.0		4
						37.5		7
						38.0	0	6
36.5	Till		6	FQ	2	38.0		
						38.5		
			7	AC	2	38.6		
						39.0		11
						39.6		13
						40.0	3	16
41.0	Bedrock	Calcareous Shale						
42.0		Hole Complete						
		BK casing to 41.0 feet						
		AX casing drilled to 42.3						
		feet because of water loss						
		at BX rock contact.						

H. G. ACRES & COMPANY LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT	Ontario Department of Highways	JOB No.	889
PROJECT	W.P. 138-57	HOLE No.	889-3
SITE	Highway 401, Highway 2, Lancaster, Ontario	SHEET No.	1 OF 2
CONTRACTOR:	F.E. Johnston Drilling Company Limited	STARTED	3:00 P.M. June 21, 19 60
		FINISHED	5:30 P.M. June 22, 19 60
METHOD OF DRILLING:	SOIL Modified Wash Boring	CASING DIAM.	BX
	ROCK	CORE DIAM.	
LOCATION:	LATITUDE Ch. 567+97	ELEVATIONS: DATUM	G.S.C.
	DEPARTURE 200 Feet Left	DRILL PLATFORM	
	BEARING	GROUND SURFACE	155.9
	INITIAL DIP 90 Degrees	ROCK SURFACE	108.9
	OTHER DIPS	BOTTOM OF HOLE	108.9
		WATER TABLE	

DEPTH Feet	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	S A M P L E					PENETRATION TEST Blows
			NO	TYPE *	SIZE Inches	DEPTH Feet	RET'D Inches	
0.0	Clay	Gray-brown, weathered medium stiff	1	BO	2	4.0	5.5	18
5.0	Clay	Blue-grey, medium stiff, homogeneous	2	BO	2	7.0	9.0	24 Pushed
13.0	Clay	Blue-grey, soft, homogeneous	Vane Test			11.5		
			3	BO	2	13.5	15.5	24 Pushed
			Vane Test			17.0		
			4	BO	2	21.0	23.0	24 Pushed
			Vane Test			24.5		
27.0	Clay	Blue-grey, stiff, homogeneous	5	BO	2	26.0	27.5	18 Pushed
			Vane Test			29.0		

SAMPLING METHOD

* A — SPLIT TUBE
 B — THIN WALL TUBE
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 F — WASH

SHIPPING CONTAINER

N — INSERT
 O — TUBE
 P — WATER CONTENT TIN
 Q — GLASS JAR

R — CLOTH BAG
 S — PLIOTILM BAG
 Z — DISCARDED

INSPECTOR J. MacLeod
 LOGGED BY J. MacLeod

APPROVED

D.H. MacDonald

DATE

July, 1960

H. G. ACRES & COMPANY LIMITED — CONSULTING ENGINEERS
 NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Ontario Department of Highways

JOB No. 589

PROJECT W.F. 138-57

HOLE No. 589-3

SITE Highway 401, Highway 2, Lancaster, Ontario.

SHEET No. 2 OF 2

DEPTH	SOIL TYPE	DESCRIPTION COLOUR CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTION, WATER LOSS OR GAIN, ETC.	S A M P L E					PENETRATION TEST
			NO.	TYPE	SIZE	DEPTH	RET'D	
Feet					Inches	Feet	Inches	Blows
32.0	Clay	Blue-grey, soft, homogeneous	6	30	2	31.0		
						33.0	24	Pushed
			Vane Test			34.5		
37.0	Till	Blue-grey clay containing small angular pebbles	7	30	2	35.0		
						37.0	24	Pushed
			8	30	2	41.0		
						41.5		14
						42.0		13
						42.5	18	9
47.0	Bedrock	Dark grey calcareous shale						

DRILLING REPORT

CLIENT Ontario Department of Highways

JOB No. 889

PROJECT W.P. 138-57

HOLE No. 889-4

SITE Highway 401, Highway 2, Lancaster, Ontario.

SHEET No. 1 OF 2

CONTRACTOR: F.E. Johnston Drilling
 Company Limited

STARTED 11:30 A.M. June 22, 1960
 FINISHED 5:30 P.M. June 24, 1960

METHOD SOIL Modified Wash Boring

CASING DIAM. BX

OF
 DRILLING: ROCK Diamond Drill

CORE DIAM. AXT

LOCATION: LATITUDE Ch. 567+79
 DEPARTURE 120 Feet Right
 BEARING
 INITIAL DIP 40 Degrees
 OTHER DIPS

ELEVATIONS: DATUM GSC
 DRILL PLATFORM
 GROUND SURFACE 156.0
 ROCK SURFACE 116.0
 BOTTOM OF HOLE 110.0
 WATER TABLE

DEPTH	SOIL TYPE	DESCRIPTION, COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	S A M P L E					PENETRATION TEST
			NO	TYPE *	SIZE	DEPTH	RET'D	
Feet					Inches	Feet	Inches	Blows
0	Clay	Grey-brown, weathered	1	30	2	5.0		
						6.5	15	Pushed
8.0	Clay	Blue, homogeneous, stiff, tenacious.	Vane Test			8.5		
			2	30	2	9.0		
10.0		Medium				10.5	18	Pushed
			Vane Test			12.8		
			3	30	2	13.0		
						14.5	18	Pushed
			Vane Test			16.3		
			4	30	2	17.0		
						18.5	18	Pushed
			Vane Test			20.3		
			5	30	2	21.0		
						22.5	18	Pushed
			Vane Test			24.3		
			Vane Test			28.5		

SAMPLING METHOD

* A — SPLIT TUBE
 B — THIN WALL TUBE
 C — PISTON SAMPLER
 D — CORE BARREL

E — AUGER
 F — WASH

SHIPPING CONTAINER

N — INSERT
 O — TUBE
 P — WATER CONTENT TIN
 Q — GLASS JAR

R — CLOTH BAG
 S — PLIOFILM BAG
 Z — DISCARDED

INSPECTOR J. Bateson

LOGGED BY J. MacLeod

APPROVED

D. H. MacDonell

DATE

July, 1960

DRILLING REPORT

CLIENT Ontario Department of Highways
 PROJECT W.F. 138-57
 SITE Highway 401, Highway 2, Lancaster, Ontario

JOB No. 869
 HOLE No. 889-4
 SHEET No. 2 OF 2

DEPTH Feet	SOIL TYPE	DESCRIPTION COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	S A M P L E					PENETRATION TEST
			NO	TYPE	SIZE Inches	DEPTH Feet	RET'D Inches	BLOWS
			6	30	2	31.0		125
						32.0	6	Pushed
32.0	Sand and Gravel		7	AC	2	33.0		125
33.0	Till					33.5		4
						34.0		7
						34.5	2	5
40.0	Bedrock	Calcareous shale						
46.0		End of hole						
		5' to 40 feet						
		AX drilled 1 foot into bedrock to prevent water loss.						

DRILLING REPORT

CLIENT Ontario Department of Highways

JOB No. 889

PROJECT W.P. 138-57

HOLE No. 889-5

SITE Highway 401, Highway 2, Lancaster, Ontario

SHEET No. 1 OF 2

CONTRACTOR: F.E. Johnston Drilling
 Company Limited

STARTED 8:00 A.M. June 23, 19 60
 FINISHED 5:30 P.M. June 24, 19 60

METHOD SOIL Modified Wash Boring

CASING DIAM. BX

OF
 DRILLING: ROCK

CORE DIAM.

LOCATION: LATITUDE Ch. 567-65
 DEPARTURE 200 Feet Right
 BEARING
 INITIAL DIP 90 Degrees
 OTHER DIPS

ELEVATIONS: DATUM GSC
 DRILL PLATFORM
 GROUND SURFACE 156.5
 ROCK SURFACE 116.5
 BOTTOM OF HOLE 116.0
 WATER TABLE

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE					PENETRATION TEST
			NO	TYPE *	SIZE	DEPTH	RET'D	
Feet					Inches	Feet	Inches	
0.0	Clay	Gray-brown, weathered, medium stiff	1	30	2	4.0		
						6.0	24	Pushed
			2	30	2	8.0		
						10.0	24	Pushed
11.0	Clay	Blue-grey, homogeneous, soft	Vane Test			11.5		
			3	30	2	14.0		
						16.0	24	Pushed
			Vane Test			17.5		
			4	30	2	18.5		
						20.0	18	Pushed
20.0	Clay	Blue-grey, homogeneous, slightly stiffer than above material	Vane Test			21.5		
			5	30	2	24.5		
						26.0	18	Pushed
			Vane Test			27.5		
30.0	Clay	Blue-grey, homogeneous, soft		BZ	2	31.0		
						32.5	0	Pushed

SAMPLING METHOD

* A - SPLIT TUBE
 B - THIN WALL TUBE
 C - PISTON SAMPLER
 D - CORE BARREL

E - AUGER
 F - WASH

SHIPPING CONTAINER

N - INSERT
 O - TUBE
 P - WATER CONTENT TIN
 Q - GLASS JAR

R - CLOTH BAG
 S - PLIOFILM BAG
 Z - DISCARDED

INSPECTOR J. MacLeod

APPROVED *D. H. MacDonald*

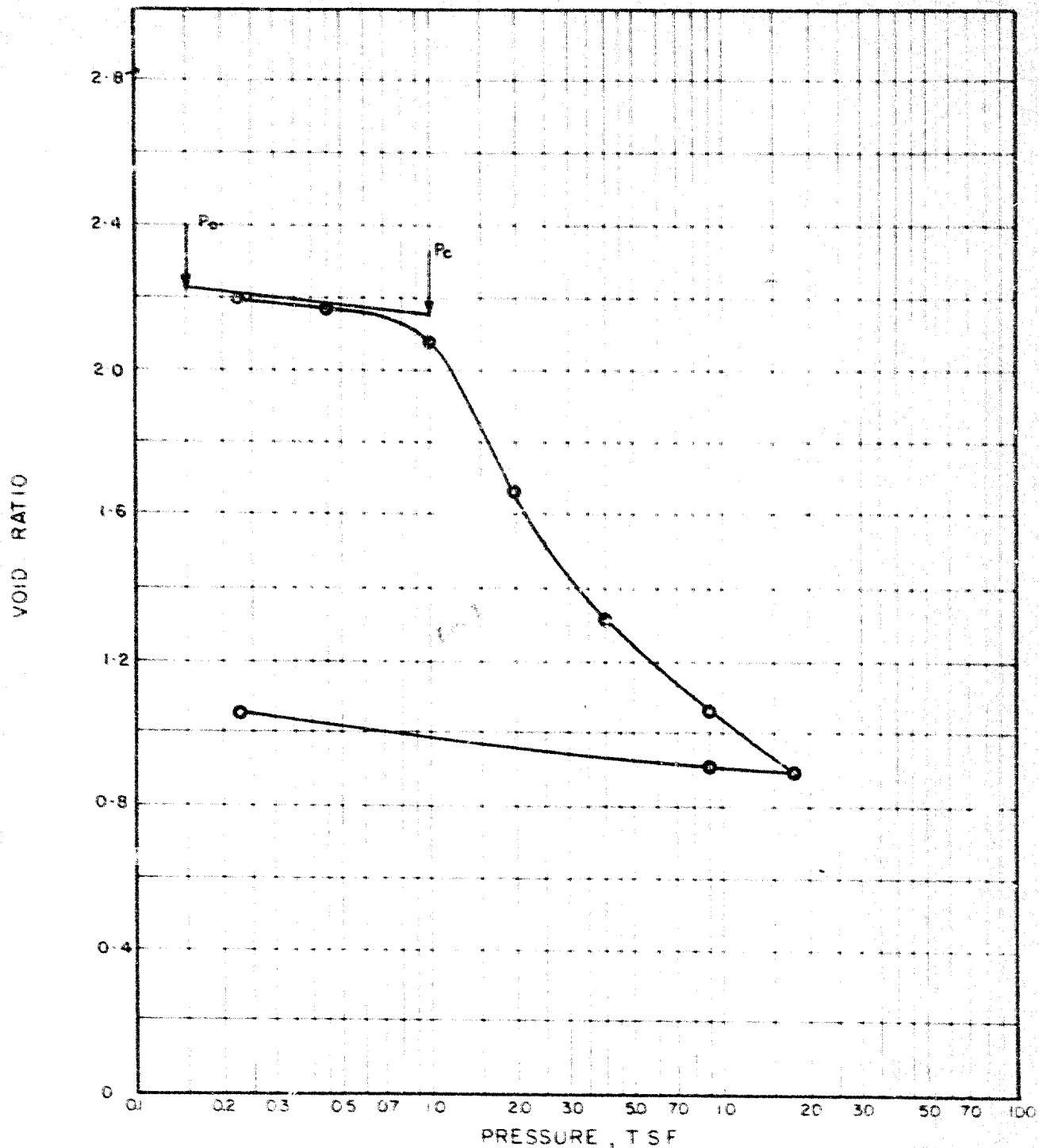
LOGGED BY J. MacLeod

DATE July, 1960

DRILLING REPORT

JOB No. 889
HOLE No. 889-5
SHEET No. 2 OF 2

Figure 1



OVERBURDEN PRESSURE - $P_0 = 0.15$ TSF
 CONSOLIDATION PRESSURE - $P_c = 1.30$ TSF

NATURAL WATER CONTENT 79.1 %
 LOADING INTERVAL 24 HRS.

SAMPLE No 889-B0-28
 TEST No 889-9-1

TEST DATE JULY 6, 1960
 TESTED BY R. L.

H. G. ACRES & COMPANY LIMITED
 CONSULTING ENGINEERS
 NIAGARA FALLS CANADA

CONSOLIDATION TEST

ONTARIO DEPARTMENT OF HIGHWAYS

HOLE No 889-3 SAMPLE ELEV 151 FT.

APPROVED

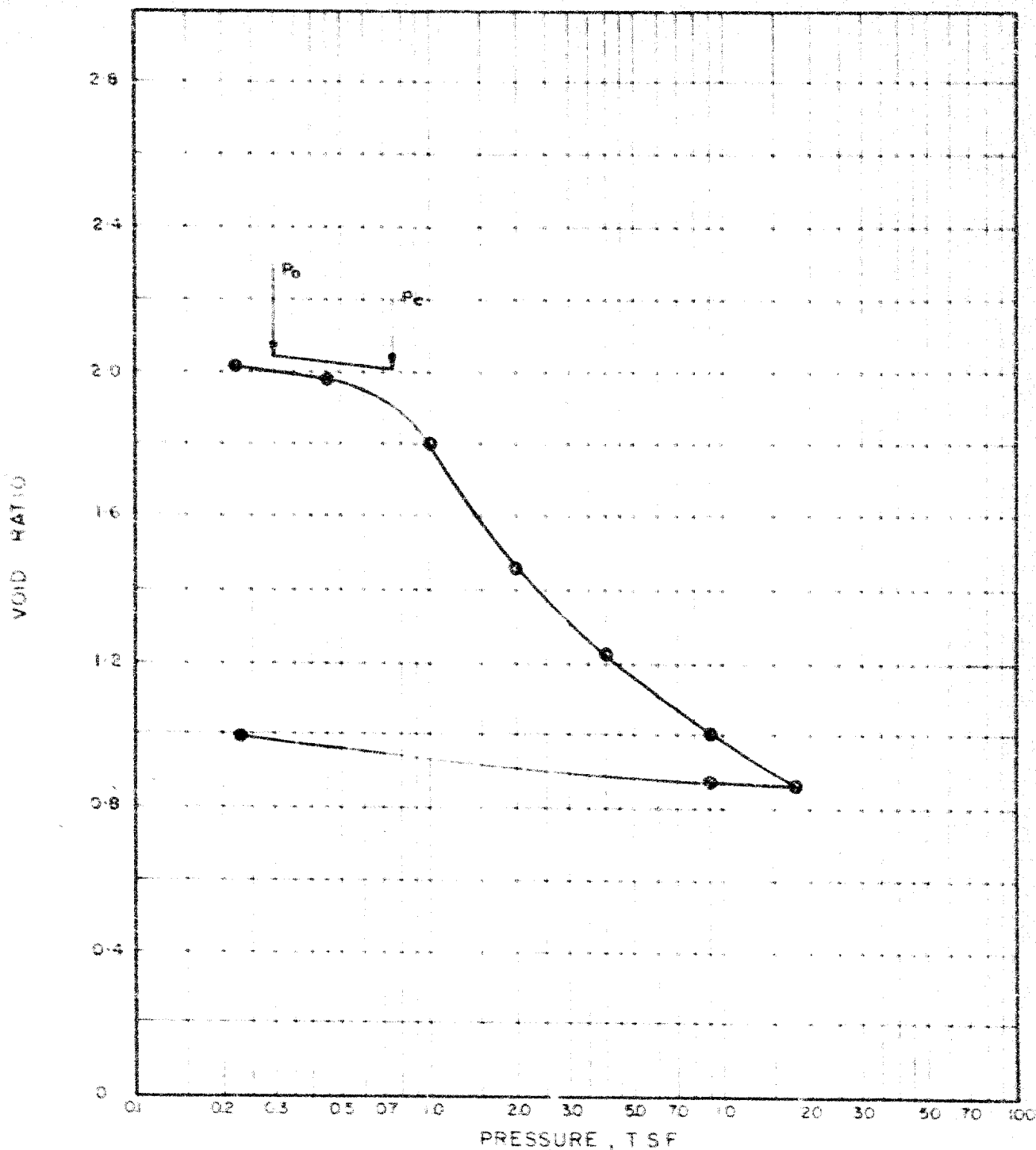
DATE JULY, 1960

W P 138 - 57

R. L. Donaldson
 H. G. ACRES & COMPANY LTD

JOB No 889

PLATE VII



OVERBURDEN PRESSURE - $P_0 = 0.32$ TSF
 CONSOLIDATION PRESSURE - $P_c = 0.75$ TSF

NATURAL WATER CONTENT 73.4 %
 LOADING INTERVAL 24 HRS.

SAMPLE No 889-80-18

TEST DATE JULY 9, 1960

TEST No 889-9-2

TESTED BY R.L. & B.H.

H.G. ACRES & COMPANY LIMITED
 CONSULTING ENGINEERS
 NIAGARA FALLS CANADA

CONSOLIDATION TEST

HOLE No 889-3

SAMPLE ELEV 141 FT.

ONTARIO DEPARTMENT OF HIGHWAYS

APPROVED

DATE JULY, 1960

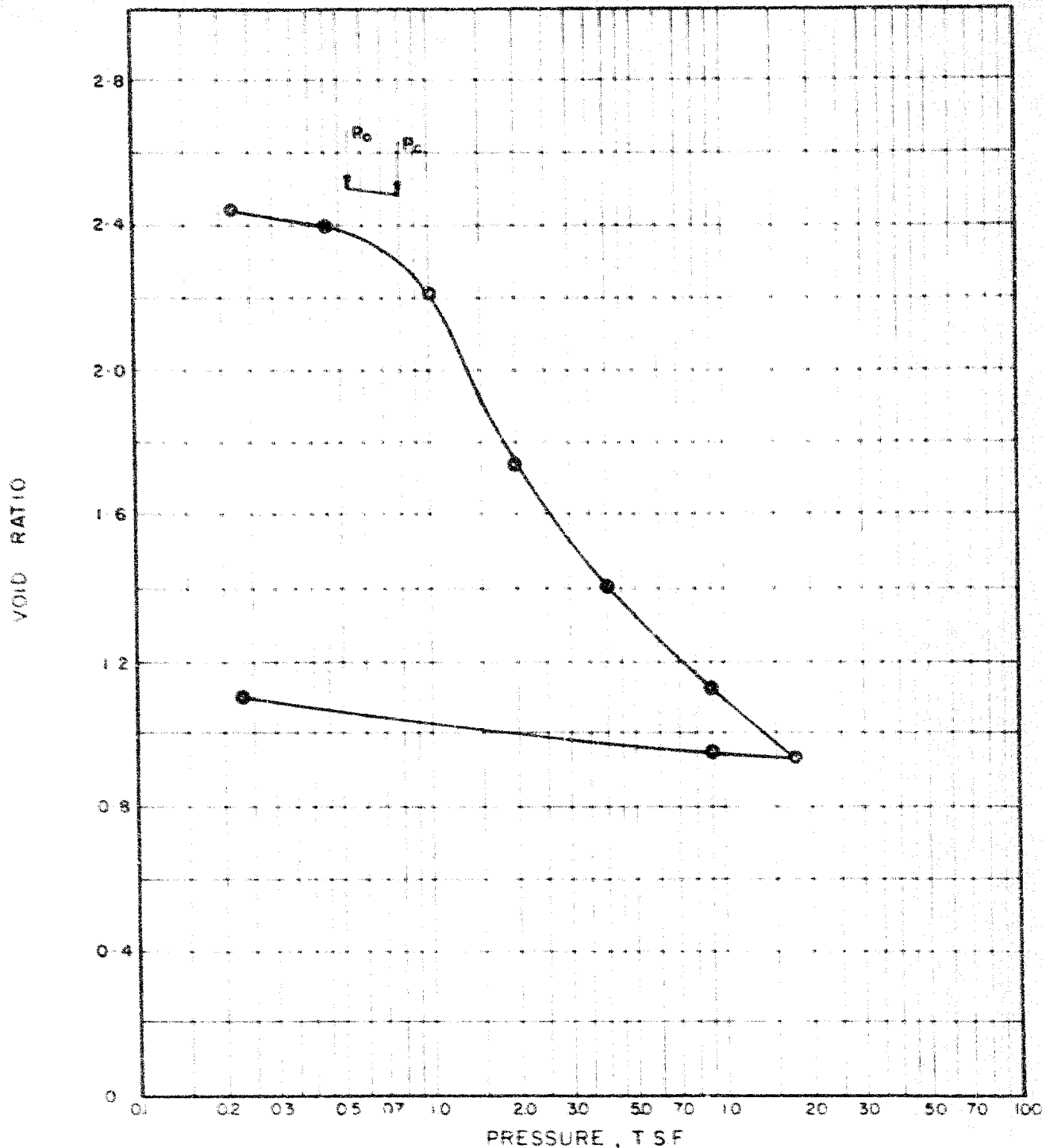
W P 138 - 57

H.G. ACRES & COMPANY LTD

JOB No 889

PLATE VIII

SK-889-LS-2



OVERBURDEN PRESSURE — $P_0 = 0.54$ TSF
 CONSOLIDATION PRESSURE — $P_c = 0.80$ TSF

NATURAL WATER CONTENT 88.6 %
 LOADING INTERVAL 24 HRS.

SAMPLE No. 889-80-20
 TEST No. 889-9-3

TEST DATE JULY 16, 1960
 TESTED BY R.G.

H.G. ACRES & COMPANY LIMITED
 CONSULTING ENGINEERS
 NIAGARA FALLS CANADA

CONSOLIDATION TEST

HOLE No. 889-3 SAMPLE ELEV. 129 FT.

ONTARIO DEPARTMENT OF HIGHWAYS

APPROVED

DATE JULY, 1960

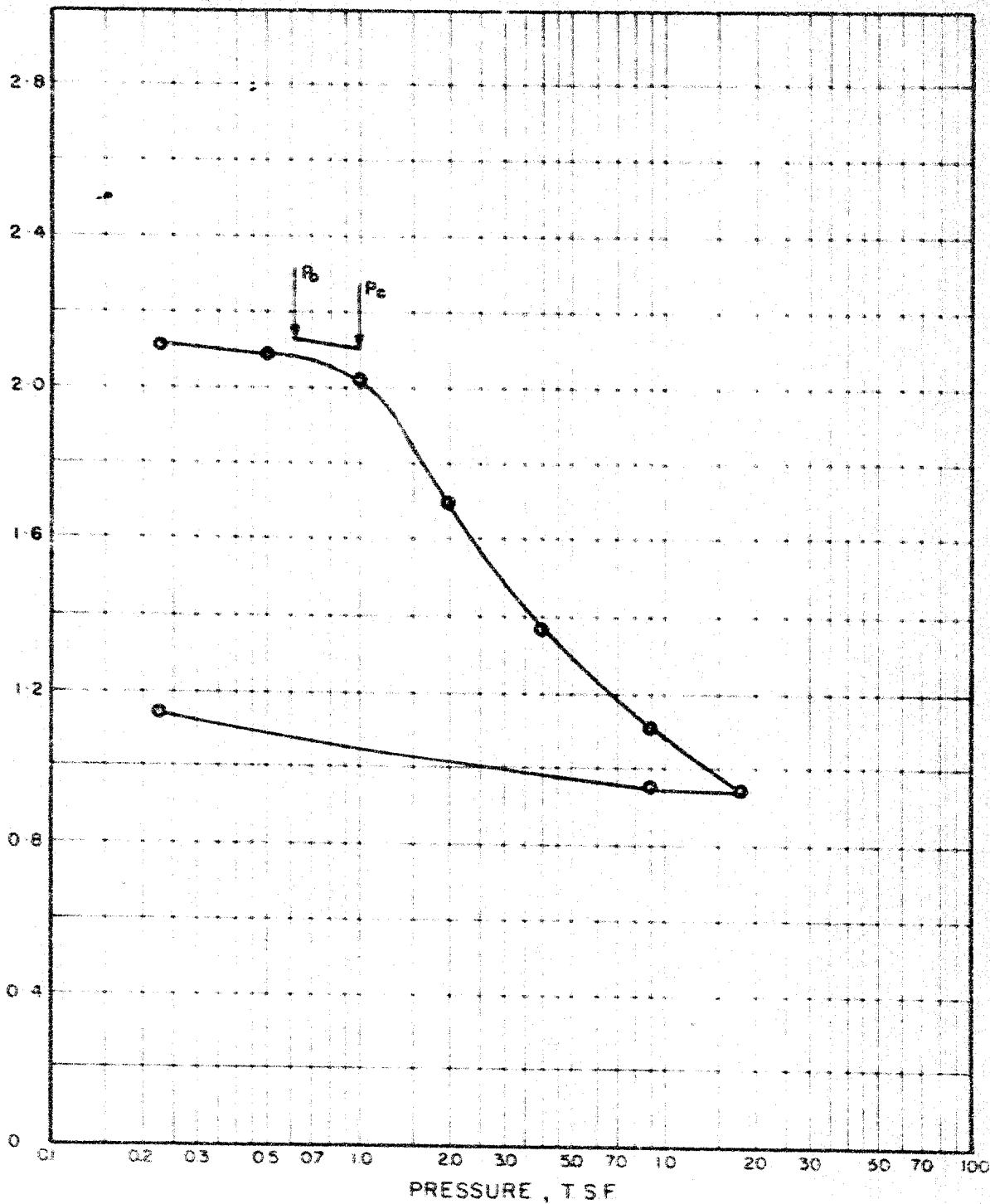
W P 138-57

H.G. ACRES & COMPANY LTD.

JOB No. 889

PLATE IX

VOID RATIO



OVERBURDEN PRESSURE - $P_0 = 0.61$ TSF
 CONSOLIDATION PRESSURE - $P_c = 1.0$ TSF

NATURAL WATER CONTENT 75.5 %
 LOADING INTERVAL 24 HRS

SAMPLE No 889-80-21
 TEST No 889-9-4

TEST DATE JULY 2, 1960
 TESTED BY R.L.

H. G. ACRES & COMPANY LIMITED
 CONSULTING ENGINEERS
 NIAGARA FALLS CANADA

ONTARIO DEPARTMENT OF HIGHWAYS

W P 138-57

CONSOLIDATION TEST

HOLE No. 889-3 SAMPLE ELEV 125 FT.

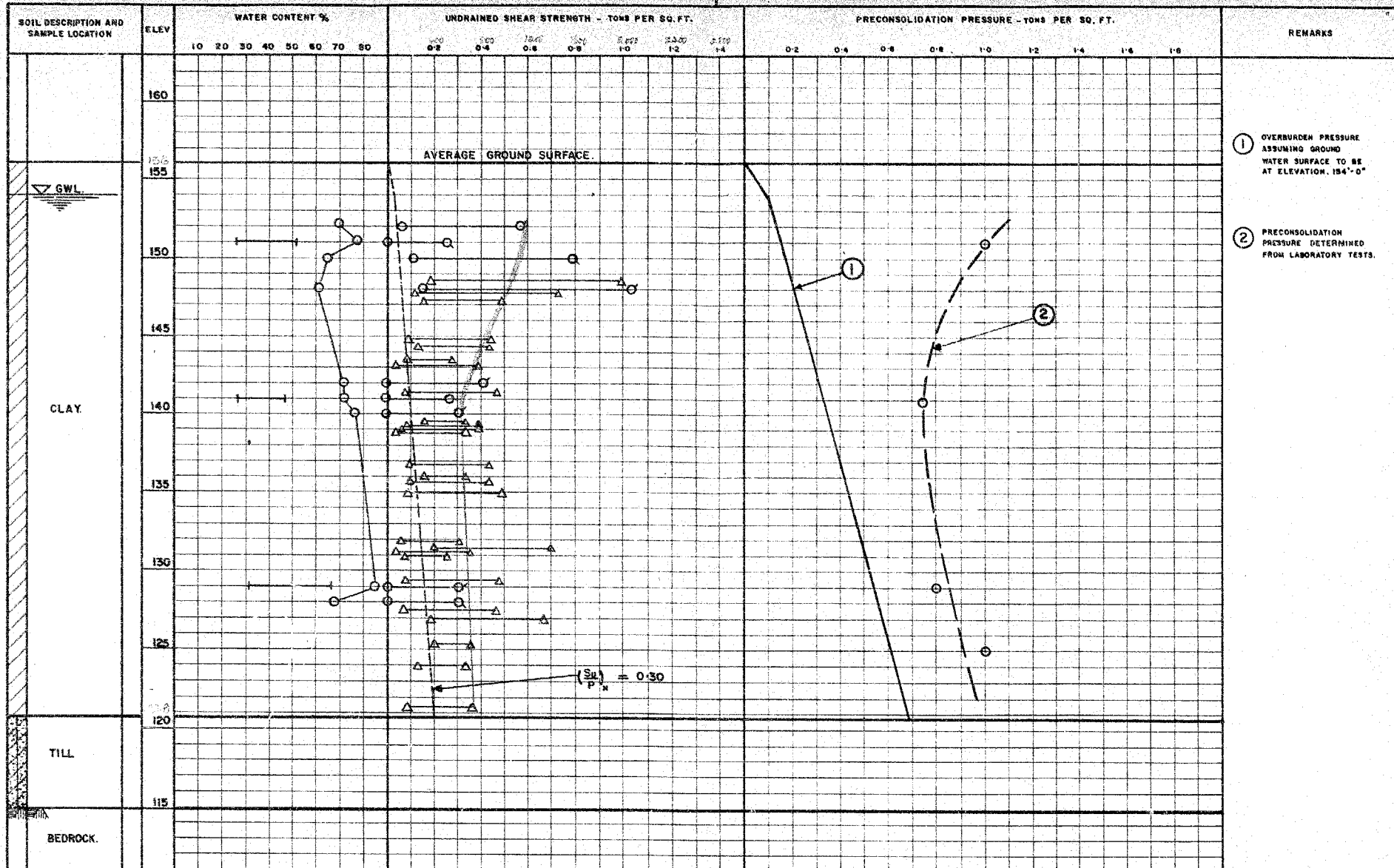
APPROVED

DATE JULY, 1960

JOB No 889

H. G. ACRES & COMPANY LTD.

PLATE X



VOID RATIO

2.8

2.4

2.0

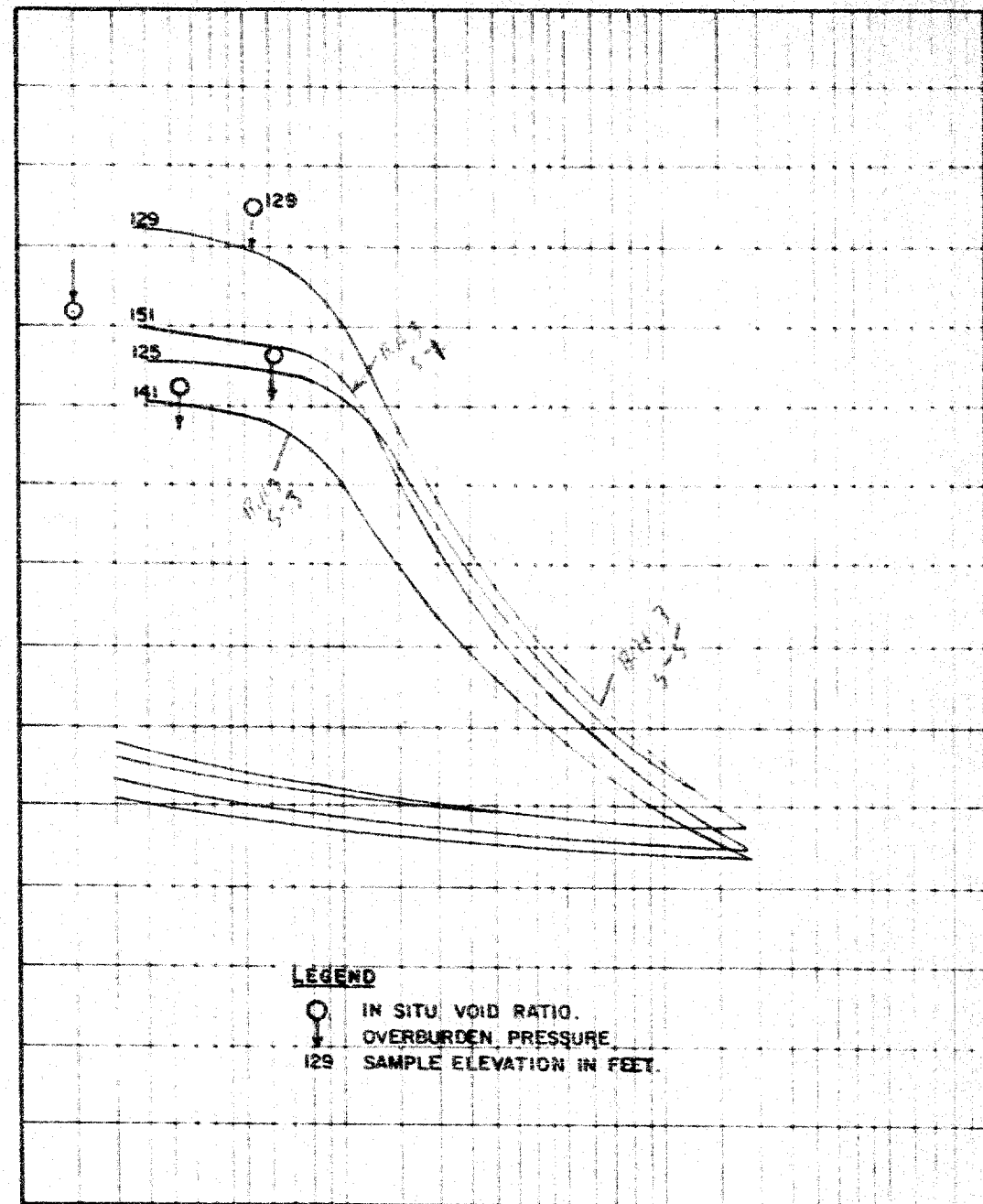
1.6

1.2

0.8

0.4

0



PRESSURE, T.S.F.

OVERBURDEN PRESSURE - P_0 = _____
 CONSOLIDATION PRESSURE - P_c = _____

NATURAL WATER CONTENT _____
 LOADING INTERVAL 24 HRS.

SAMPLE No _____
 TEST No _____

TEST DATE _____
 TESTED BY _____

H. G. ACRES & COMPANY LIMITED
 CONSULTING ENGINEERS
 NIAGARA FALLS CANADA

CONSOLIDATION TEST
 COMPARISON OF ALL TESTS.

DEPARTMENT OF HIGHWAYS OF ONTARIO.

APPROVED

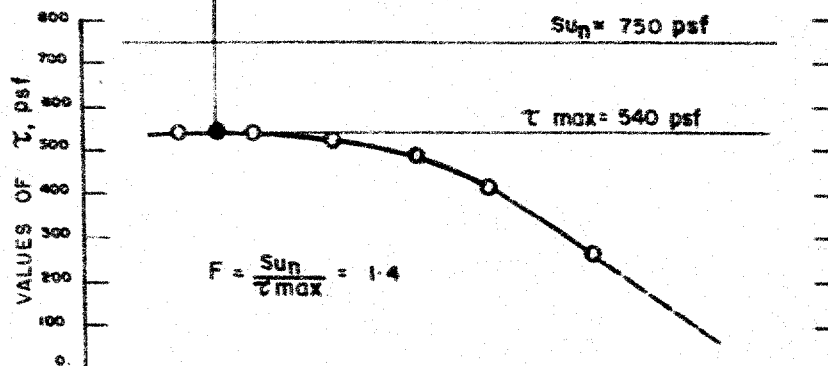
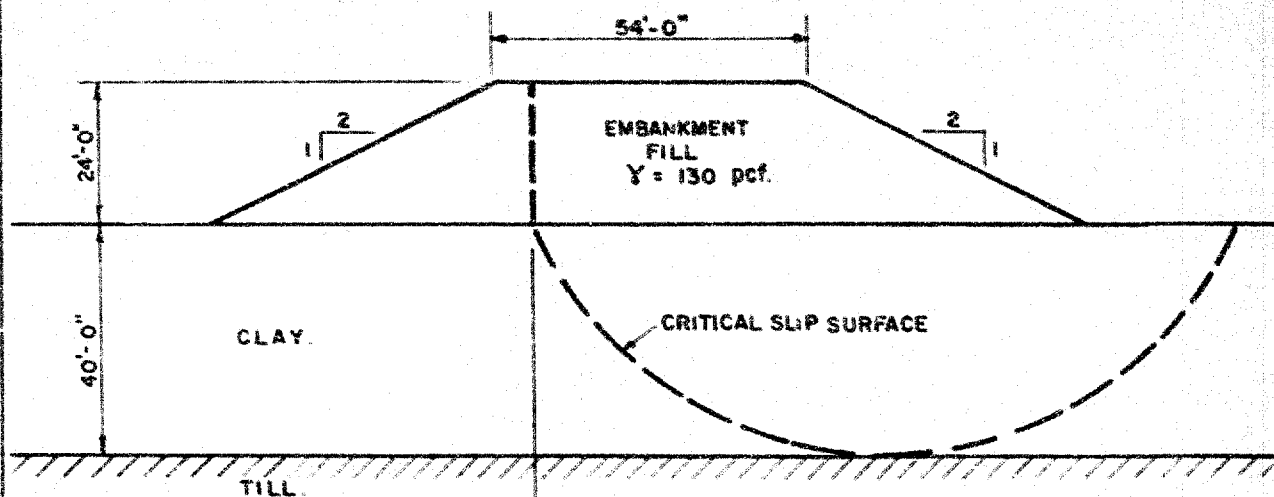
DATE JULY 1960.

JOB No. 889.

WP. 138-57

H. G. Acres
 H. G. ACRES & COMPANY LTD.

PLATE XII



τ DENOTES AVERAGE SHEAR STRESS ALONG SLIP SURFACE

S_{un} DENOTES AVERAGE NATURAL SHEAR STRENGTH OF SUBSOIL

F DENOTES SAFETY FACTOR.

H. G. ACRES & COMPANY LIMITED
CONSULTING ENGINEERS
NIAGARA FALLS CANADA

ONTARIO DEPARTMENT OF HIGHWAYS

W.P. 138-57

SUMMARY OF STABILITY
ANALYSES FOR THE CASE
OF AN EMBANKMENT WITHOUT BERMS.

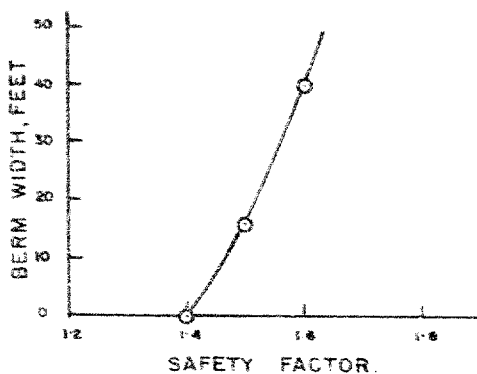
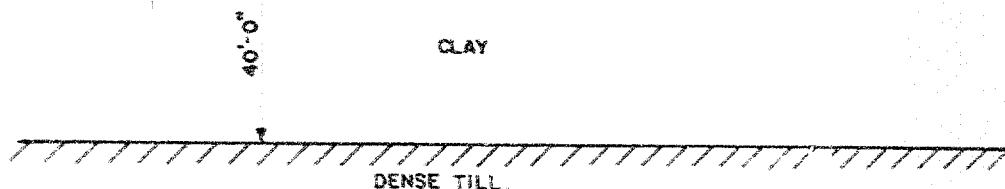
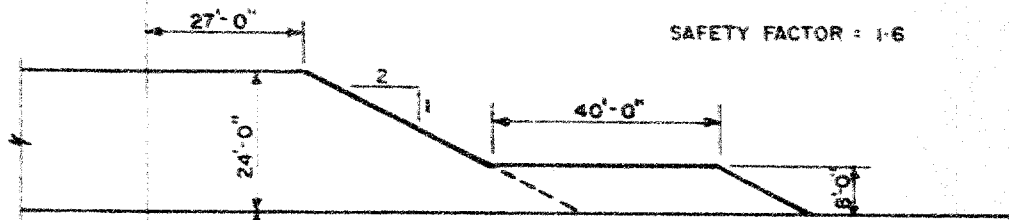
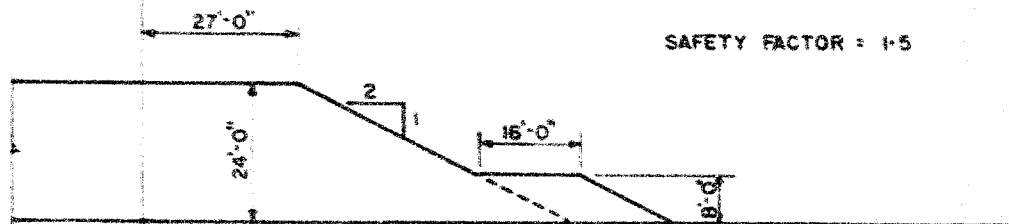
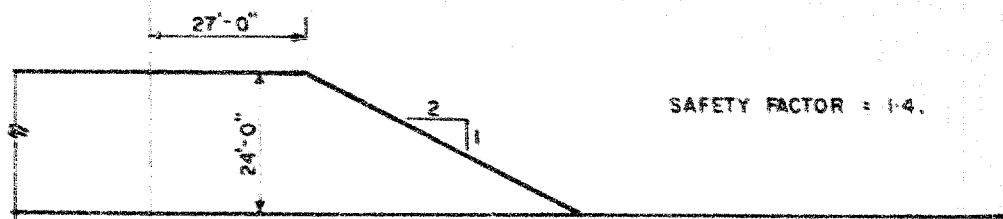
APPROVED

DATE JULY, 1960

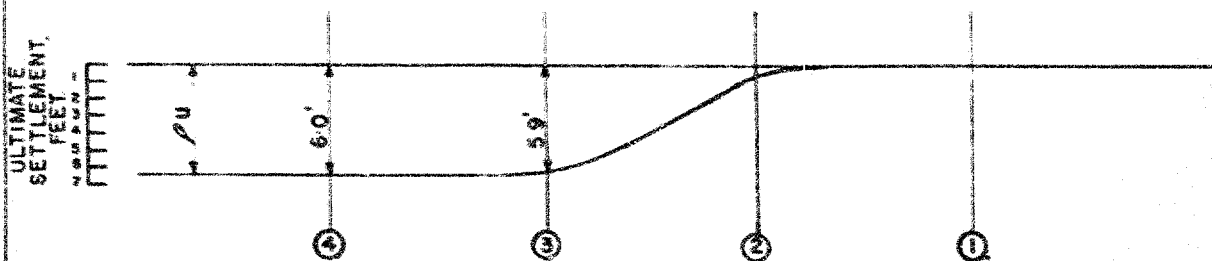
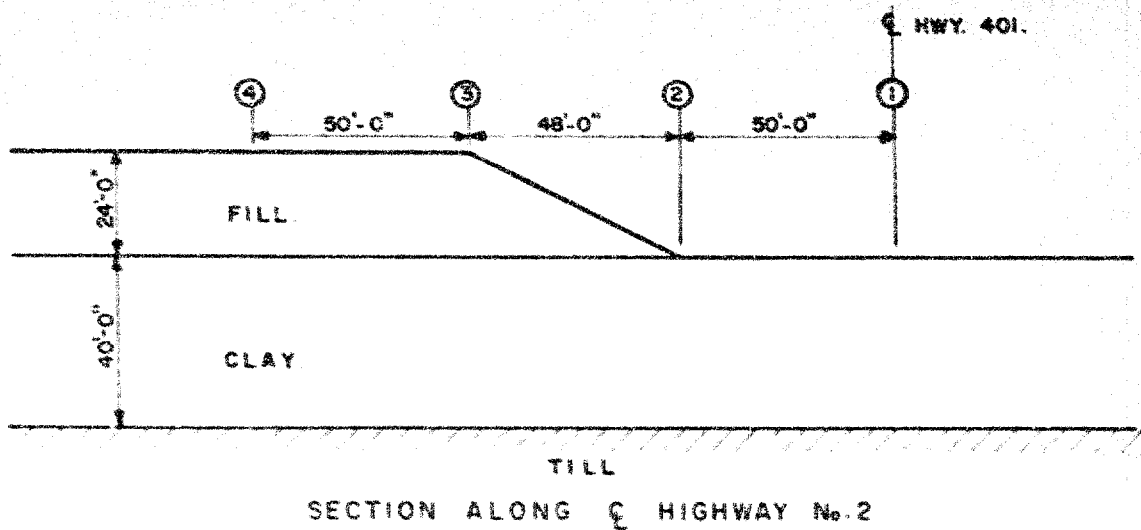
SCALE JOB No.
889

H. G. ACRES & COMPANY LIMITED

PLATE - XIII

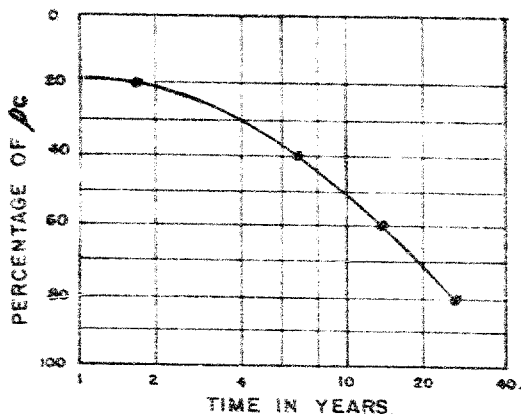


H. G. ACRES & COMPANY LIMITED CONSULTING ENGINEERS NIAGARA FALLS, CANADA	
ONTARIO DEPARTMENT OF HIGHWAYS	
W.P. 138-57.	
COMPARISON OF EMBANKMENT CROSS SECTION SHOWING THE EFFECT OF VARYING THE SAFETY FACTOR.	
APPROVED	DATE JULY, 1960
<i>H. G. Acres</i>	SCALE
H. G. ACRES & COMPANY LIMITED	JOB No. 889
PLATE - XIV	



$p_i = 0.4$	0.4	0	0
$p_c = 5.6$	5.5	0.2	0
$p_u = \underline{6.0}$	<u>5.9</u>	<u>0.2</u>	<u>0</u>

p_i = DENOTES ELASTIC SETTLEMENT IN FEET.
 p_c = DENOTES CONSOLIDATION SETTLEMENT IN FEET.
 p_u = DENOTES ULTIMATE SETTLEMENT IN FEET.



AVERAGE TIME RATE OF SETTLEMENT.

H. G. ACRES & COMPANY LIMITED CONSULTING ENGINEERS NIAGARA FALLS CANADA	
ONTARIO DEPARTMENT OF HIGHWAYS	
W.P. 138-57.	
FOUNDATION SETTLEMENTS DUE TO EMBANKMENT LOAD.	
APPROVED	DATE JULY, 1960
<i>H. G. Acres</i>	SCALE
H. G. ACRES & COMPANY LIMITED	JOB No. 389.
PLATE - XV.	

Calculation

See memo from
Bridge Office dated

November 2, 1960

October

R. J. L.

DISTURBING MOMENT

① 05 (36) (18) (24) 75.00

(S) (1-S) (Q-A) (C-57) 10 A 1175 do

Q 0.7 (1) (1992) 5 Feb

④ 0.1 (1) (27) (77) 1.890

1950年10月10日

④ 2 = (12)(34)(12) 2 70

20310

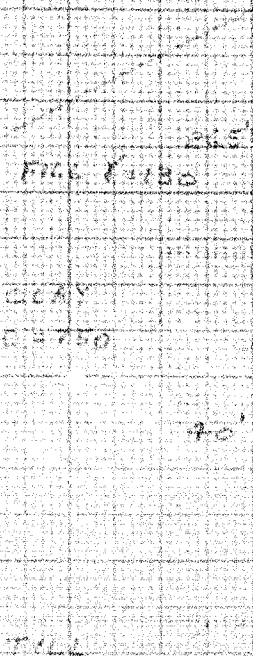
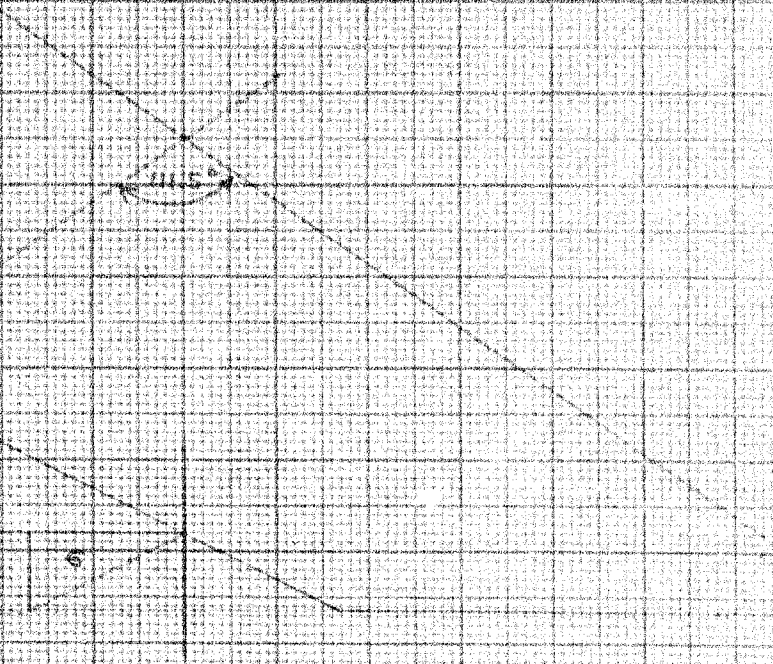
$$M = \frac{R^3 \Delta \rho}{180}$$

$$= \frac{\pi}{180} (9.3) (7.5) (11.5)$$

1950年12月10日

$$\frac{1.23 \times 10^3}{26.30 \times 10^3} = 1.10$$

[illegible]



$$\textcircled{1} \quad 0.5(4.8)(24)(11) = 6,320$$

$$\textcircled{2} \quad \frac{(4.0)(4.4)(51.3)}{66.20} = 69,500$$

$$M_2 = 66,500 \times 130 \text{ AT. LB}$$

$$M_1 = \frac{\pi}{180} (K)^2 (C \theta)$$

$$M_1 = \frac{\pi}{180} (92)^2 (750)(11)$$

$$= 12.3 \times 10^3$$

$$S.F. = \frac{12.3 \times 10^3}{(66.5 \times 130 \times 10^3)} = 1.4 \text{ B}$$

$$① \quad 0.5 (54)(27)(7) = 5,100$$

$$M_1 = \frac{\pi R^3 \rho}{180}$$

$$② \quad \frac{(27)(27)(325)}{33,100} = 28,000$$

$$M_2 = \frac{\pi (77)^3 (34.5)(750)}{180}$$

$$= 6.36 \times 10^4$$

$$M_1 = 33,100 \times 130 \text{ AT-LA}$$

$$F.F. = \frac{1.55 \times 10^4}{33,100 \times 130} = 1.55$$

485

$$① 0.5(34)(27)(7) = 5100$$

$$② 150(77)(51) = 13100$$

73,200

$$M_0 = 73,200 \times 1.22 \text{ FT-LB}$$

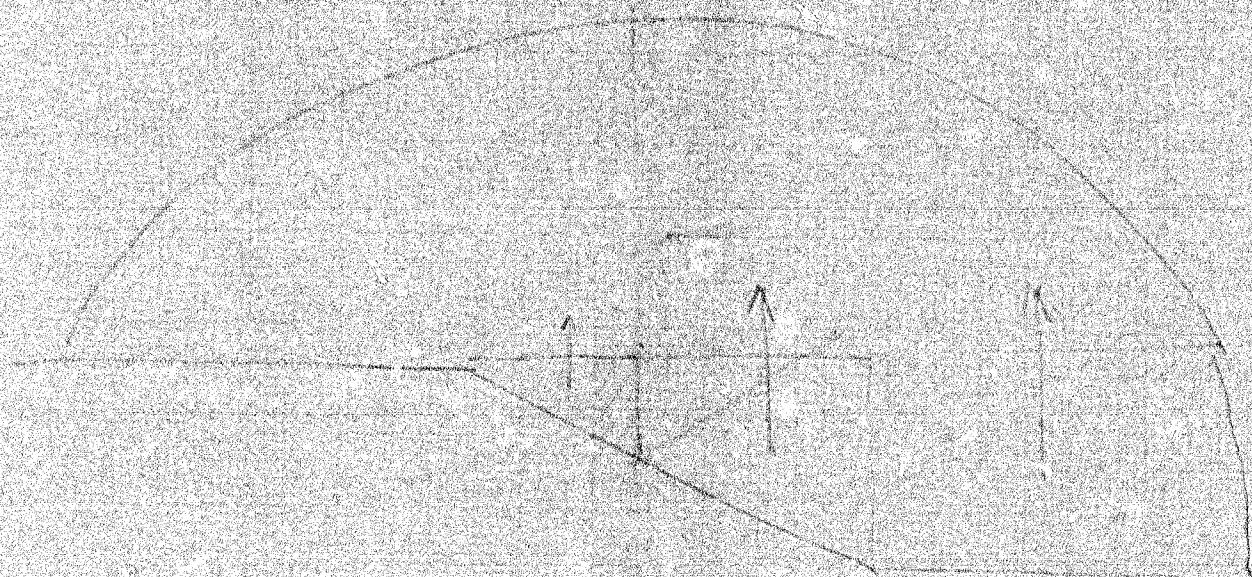
$$M_r = \frac{\pi (R^3) \sigma}{180}$$

$$M_r = \frac{\pi (32)^3 750 (11)}{180}$$

$$= 123 \times 10^3 \text{ FT-LB}$$

$$SF = \frac{123 \times 10^3}{73,200 \times 1.22} = 1.23$$

111



DISTURBING MOMENT

① $0.5(29)(15)(29) = 4,300$

② $(17)(15)(52.5) = 37,000$

③ $(57)(9)(42.5) = 22,300$

④ $0.5(18)(3)(12) = 370$

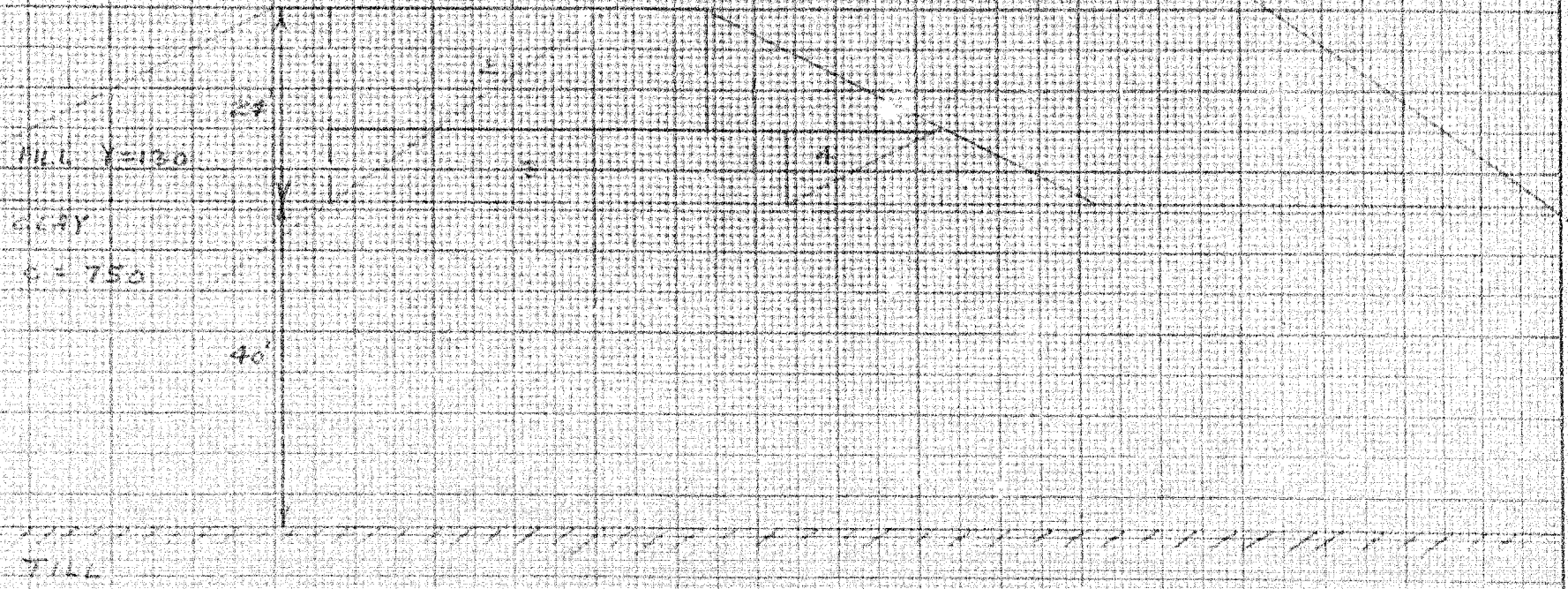
$120 \times 64,579 \text{ FT-LB}$

$M_R = \frac{\pi}{180} R^2 c \theta$

$= \frac{\pi}{180} (92)^2 (750)(111)$

$= 12.3 \times 10^6 \text{ FT-LB}$

$S.F. = \frac{12.3 \times 10^6}{64,579 / 30 \times 10^3} = 1.47$



DISURBING MOMENT $\times 10^3$

- ① - $0.5(29)(15)(130)(20) = 564$
- ② - $(52)(15)(130)(55) = 5,510$
- ③ - $0.5(6)(15)(130)(63) = 423$
- ④ - $0.5(5)(8)(130)(77) = 200$
- ⑤ - $(57)(9)(130)(48) = 3,200$
- ⑥ - $0.5(19)(7)(130)(12) = 123$

$10,133 \times 10^3 \text{ FT-LB}$

$$M = \frac{\pi R^2 (L \cdot S)}{180}$$

$$= \frac{\pi (32)^2 (750)(111)}{180}$$

$$= 12,300 \times 10^3 \text{ FT-LB}$$

$$SF = \frac{12,300 \times 10^3}{10,133 \times 10^3} = 1.213$$



$$\begin{aligned} \textcircled{1} \quad 0.5(34)(27)(7) &= 5,100 \\ \textcircled{2} \quad (52)(27)(51) &= 71,600 \\ \hline &76,700 \end{aligned}$$

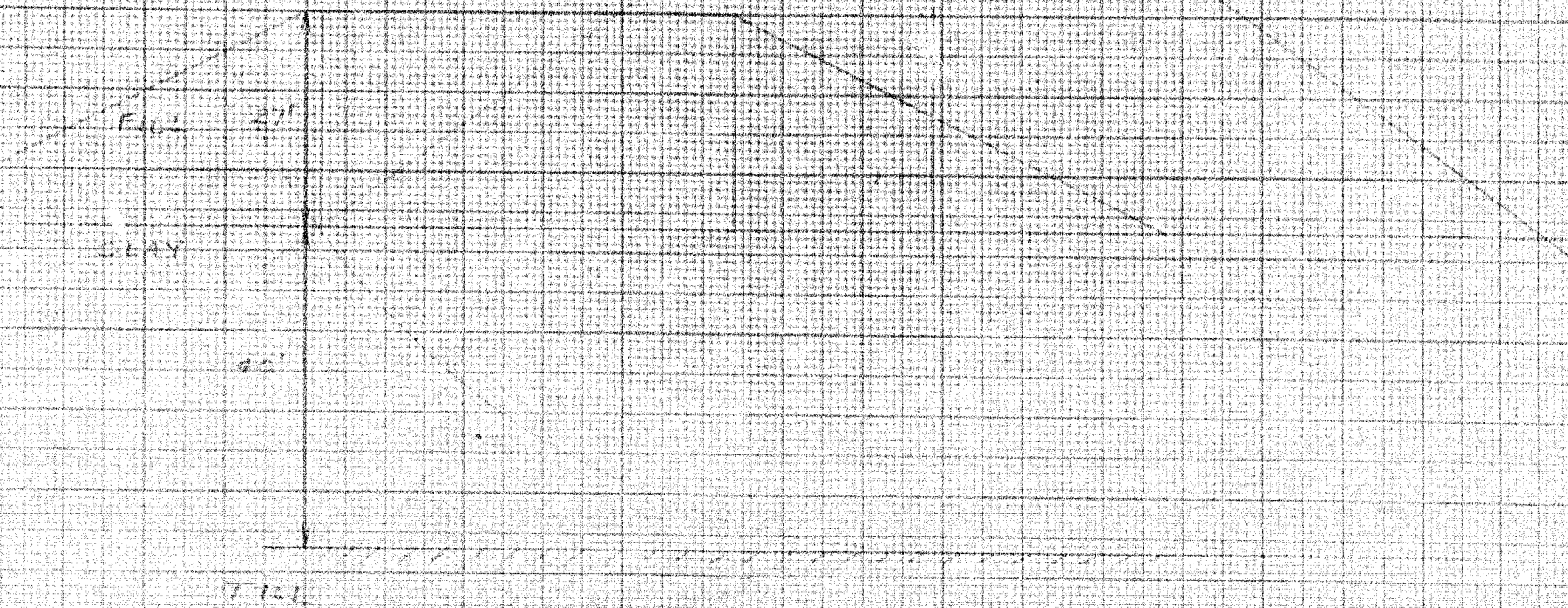
$$M_o = 76,700 \times 130$$

$$M_o = \frac{\pi}{180} (R^3) \phi$$

$$M_o = \frac{\pi}{180} (35)^3 (150)(109)$$

$$= 12.3 \times 10^4 \text{ FT. LB.}$$

$$S.F. = \frac{12.3 \times 10^4}{76,700 \times 130} = 1.29$$



$$① - 5(48)(24)(9) = 5,180$$

$$② - \frac{(51)(24)(515)}{68,180} = 63,000$$

$$M_1 = \frac{\pi R^2 c d}{180}$$

$$M_1 = \frac{\pi (92)^2 (750)(111)}{180}$$

$$= 12.3 \times 10^6 \text{ FT LB}$$

$$S.F. = \frac{12.3 \times 10^6}{68,180 \times 180} = 1.30$$

$$M_1 = \frac{\pi (92)^2 (750)(108)}{180}$$

$$= 12.3 \times 10^6$$

$$S.F. = 1.39$$

FILL

 $\gamma = 130$

CLAY

 $c = 750$

TILL

#60-F-205

W.P.# 108-59

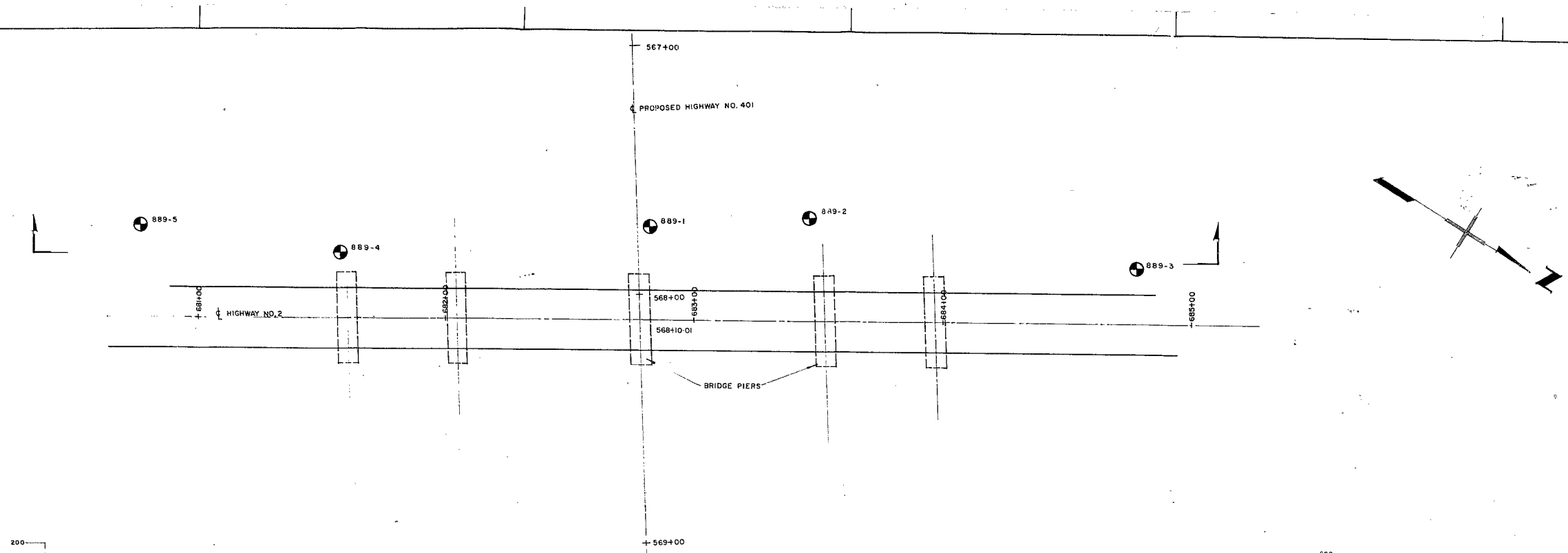
Hwy. # 401

PROP. CROSSING

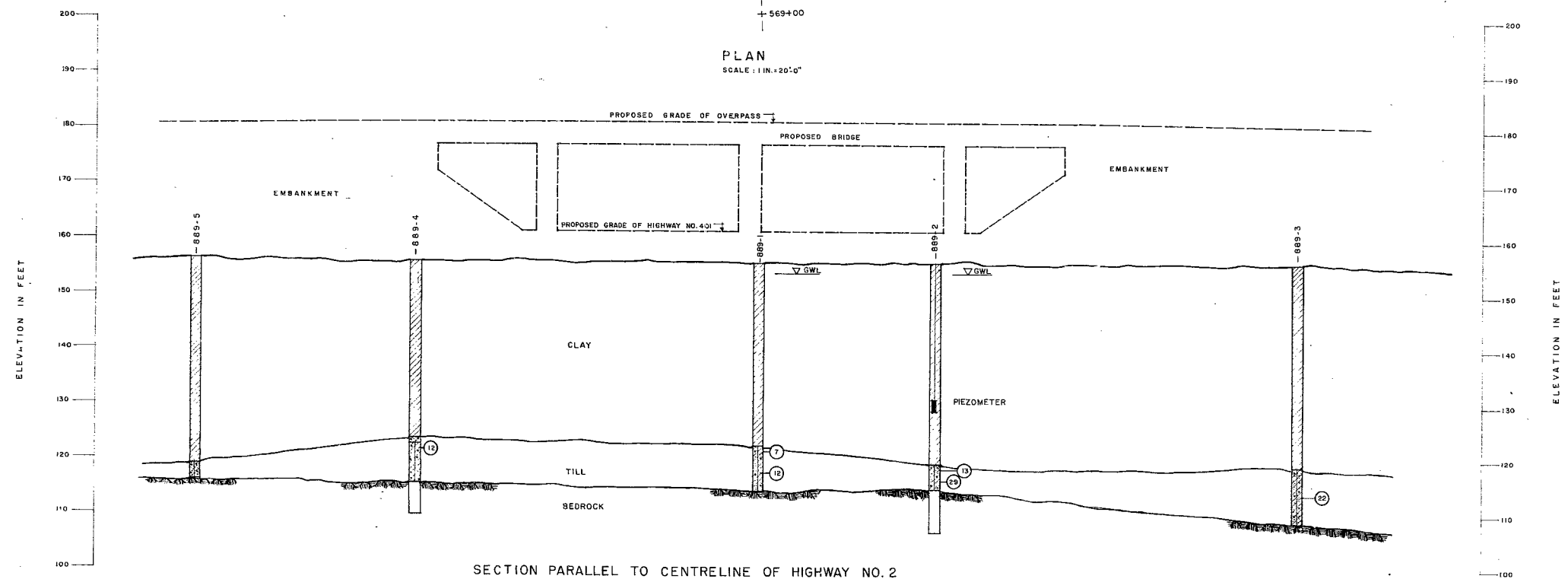
Hwy. # 2

1½ MILES S. OF

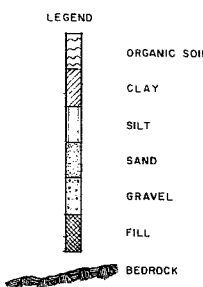
LANCASTER



PLAN
SCALE: 1 IN. = 20'-0"



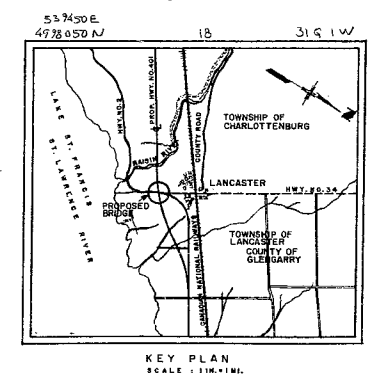
SECTION PARALLEL TO CENTRELINE OF HIGHWAY NO. 2
SCALE: HORIZONTAL - 1 IN. = 20'-0"
VERTICAL - 1 IN. = 10'-0"



- WATER TABLE
- EXPLORATORY DRILL HOLE
 - ⊕ 2 IN. DIA. PENETRATION CONE TEST HOLE
 - ⊙ BLOWS PER FOOT, OR BLOWS FOR NOTED DISTANCE, FOR STANDARD PENETRATION TEST.

- NOTES:
- STANDARD PENETRATION TESTS WERE PERFORMED USING A 2-IN. OUTSIDE DIAMETER SPLIT-SPoon AND A 140 LB. WEIGHT DROPPING 30 INCHES.
 - CONE PENETRATION TESTS WERE PERFORMED USING A 2-IN. DIAMETER D.H.O. CONE AND A 140 LB. WEIGHT DROPPING 30 INCHES.

- REFERENCE DRAWINGS:
- D.H.O. E-2932-1 BRIDGE NO. 11, PLAN AND SECTION
 - D.H.O. F-3158-3 HIGHWAY NO. 401, PROFILES



H. G. ACRES COMPANY LIMITED	
CONS.	ENGIN.
ONTARIO DEPARTMENT OF HIGHWAYS	
WP-138-57	
EXPLORATORY HOLES PLAN AND SECTION	
APPROVED	DATE: JULY, 1960
<i>H. G. Acres</i>	SCALE AS NOTED 889
H.G. ACRES & COMPANY LIMITED	
PLATE - I	



golder.com