Peto MacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION TECHNICAL MEMORANDUM

For

LITTLE BAPTISTE CREEK BRIDGE EBL ON HIGHWAY 401
MTO WEST REGION 59 STRUCTURE REHABILITATIONS
SITE 13-187-1, CONTRACT 7
GWP 3084-11-00
GEOGRAPHICAL TOWNSHIP OF TILBURY EAST
KENT COUNTY, ONTARIO

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For
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MTO West Region 59 Structure Rehabilitations
Contract 7, GWP 3084-11-00
Township of Tilbury East
Kent County, Ontario

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1. INTRODUCTION

The Foundation Engineering Services for the present project involve the detail foundation investigation and design for the rehabilitation of 59 structures in MTO West Region along Highways 4, 6, 401, 402 and 403. Ten (10) Group Work Projects (GWP's) are contemplated to be completed between 2014 and 2020.

This technical memorandum summarizes the factual results of geotechnical data based on the review and compilation of existing subsurface information from relevant reports in the MTO GEOCRES Library for the Little Baptiste Creek Bridge EBL. The Foundation Engineering recommendations from the existing bridge foundation reports are summarized with reference to the "Canadian Highway Bridge Design Code" (CHBDC) and follow in general the "Guidelines for Professional Engineers providing Geotechnical Engineering Services".

From the Minutes of Meeting Report, dated July 24, 2014, it is understood that the rehabilitation of the bridge structure is anticipated and the rehabilitation will be completed in a single stage of construction using median crossovers.

The purpose of the Technical Memorandum is to summarize the subsurface and groundwater conditions and foundation recommendations based on available reports at the bridge location for the design project team's reference.

The elevations in this report are expressed in meters, unless otherwise noted.

2. PROJECT SITE BACKGROUND AND GEOLOGY

The Little Baptiste Creek Bridge EBL on Highway 401 is located in the Geographic Township of Tilbury East, Kent County, Ontario. A key plan map is shown in Figure 1.

The existing structure is a single span reinforced concrete rigid frame structure that carries two lanes of Highway 401 Eastbound traffic. The Little Baptiste Creek is actually a man-made drainage ditch, very straight, and of very low gradient resulting in sluggish flow.

This site is located on a very flat lacustrine clay plain in the physiographic region known as the St. Clair Clay Plain, which consists of relatively deep typically very stiff clayey silt and silty clay till deposits. Topographically, Tilbury is situated in a very flat area with generally poor drainage indicative of deep lacustrine clay plain. Based on geological information, between 37.2 and 61.0 m, most probably about 45.7 m, the site is underlain by limestone of Devonian age. Adjacent areas are underlain by black shale.

3. SOURCE INFORMATION

The following foundation report and drawing, appended in Appendix A, were available for review and provided information for bridge structure, subsoil information and original foundation recommendation.

1. Report on Soil Site Investigation at Highway 401 – Little Baptiste Creek Crossing W.P 162-58, Job No. 58162, Tilbury East by E. M. Peto Associates Ltd. for Department of Highways of Ontario, dated January 28, 1959, GEOCRES 40J08-017. (Reference 1)

2. Little Baptiste Creek Bridge – General Layout, The King's Highway No. 401, District No. 1, Lot 14 and 15, Con. IV, Township of Tilbury East, County of Kent, W.P 162-58, TWP 104-187-1-B, Department of Highways Ontario, dated July 1959. (Reference 2)

4. SITE RECONNAISSANCE

As part of the current foundation engineering assessment study, a site reconnaissance of the Little Baptiste Creek Bridge EBL was carried out on October 20, 2013. A photographic record of the site visit is attached in Appendix B.

The slopes on both sides of the abutments were heavily vegetated at the time of the site reconnaissance (Photographs 1, 2 and 4). No obvious major cracks were observed on the abutment walls except some surficial cracks (Photographs 2 and 3). No erosion or scour on front slopes and slope edges were observed (Photographs 2 and 3). The slopes were observed to be in surficial stable condition. Weeping holes in the abutment walls were open.

5. PREVIOUS FIELD INVESTIGATION AND SUMMARIZED SUBSURFACE CONDITIONS

The site is located on Hwy 401 in the Geographic Township of Tilbury East, Kent County, Ontario. The general subsurface conditions presented in this section are based on the soil investigation Report, GEOCRES 40J08-017 dated February 28, 1959.

The subsurface investigation was carried out in the period from December 16, 1958 to January 15, 1959. The investigation comprised five boreholes which were advanced to depths of 8.0 to 37.2 m, elevation 140.4 to 170.0. The field investigation was carried out by means of a skid-mounted diamond drill rig. Standard sampling procedures were followed. The samples were recovered at frequent interval depths with either 50 mm (2 in.) or 76 mm (3 in.) O.D. split barrel sampling tube, Shelby tube or split barrel sampling tube fitted with brass liners and special sharp cutting nose.

The soil investigation report includes the borehole location plan (Drawing No.F3534-7), Record of Borehole sheets (1 to 5) and summary of the Field and Laboratory tests.

<u>Fill</u>

Adjacent to the creek on both sides was a 0.9 to 1.7 m thick layer of silty clay that was excavated when the drainage ditch was constructed. There was also a 600 to 900 mm layer of sandy gravel fill material encountered on the east side of the creek which constituted the gravel side road.

Silty Clay and Stratified Silty Clay (Desiccated)

The upper most silty clay layer of stiff to very stiff consistency (except for the upper firm zone in borehole 4), which was due to desiccation and chemical weathering, was encountered in all boreholes and extended to depths of 7.0 to 8.5 m, elevation 169.0 to 171.1. The consolidation test performed for the desiccated silty clay layer obtained a compressive index of 0.130 and it appeared that the soil was slightly over-consolidated with an over-consolidation ratio of 1.41. Two grain size analyses results indicated that the silty clay samples consisted of 20 and 22% sand and gravel, 39 and 40% silt, and 39 and 40% clay sized particles.

N values recorded ranged between 9 and 27, locally 6 in borehole 4, which correspond to stiff to very stiff consistency, locally firm; however, the general consistency was in stiff state. Laboratory shear strengths obtained typically ranged from 62.4 to 298.4 kPa (1304 to 6232 psf). The Atterberg liquid limits ranged from 32.2 to 54.2 and plastic limits ranged between 17.5 and 23.0 for the desiccated silty clay samples. The plasticity index ranged from 13.5 to 31.2. Unit weight of the upper silty clay samples varied from 19.2 to 21.5 kN/m^3. Moisture content determinations ranged from 16.4 to 29.9%.

Silty Clay

Below the desiccated silty clay layer, a stratum of firm to stiff silty clay was encountered which extended to depths of 8.0 to 28.9 m, elevation 149.1 to 170.0. N values recorded ranged between 6 and 10 which correspond to firm to stiff consistency. The general consistency was in firm state. Laboratory shear strengths obtained typically ranged from 19.9 to 55.1 kPa (416 to 1150 psf). The Atterberg liquid limits ranged from 29.4 to 34.7 and plastic limits ranged between 16.0 and 19.3 for the desiccated silty clay samples. The plasticity index ranged from 13.4 to 15.8. Unit weight of the silty clay samples varied from 19.4 to 20.8 kN/m^3. The report indicated that this soil has a very low sensitivity, and is generally wetter than, or much wetter than, the plastic limit. Moisture content determinations ranged from 22.6 to 26.6%.

A consolidated undrained triaxial test with pore water measurement was performed on undisturbed silty clay samples of this layer to failure at a rate of strain of 0.2195 m/day (0.006 in./minute). However, very low unit strains of 8.0 to 8.5% developed in the samples and thus, the pore-water pressure built up due to application of axial load was very low. A consolidation test on this soil sample obtained a compressive index of 0.318. The clay appeared to be normally consolidated based on the pressure-void ratio curve from the consolidation test.

Sandy and Silty Clay Till

Below the lower silty clay layer a stratum of sandy silty clay till was encountered at about 27.4 to 28.9 m. elevation 149.1 to 150.1 (489.3 to 492.6 ft.). There was no sampling carried out from this stratum, but the probing indicated that it was of firm to stiff consistency.

Silty fine to Coarse Sand and Gravel

The sandy and silty clay was underlain by very dense silty fine to coarse sand which was encountered in borehole 3 and extended to the termination depth of 37.2 m, elevation 140.4.

Groundwater

Groundwater was not established during the investigation in the five boreholes because of very impermeable silty clay but the soil at this site was found to be saturated virtually throughout its full depth except in samples between 1.8 to 2.4 m (6 to 8 ft.). The final groundwater level was estimated close to the stream level at elevation 175.0.

The report indicated that true water table concept does not exist in the very impermeable soil type that was encountered at the site location. For the purpose of computing effective stresses, submerged unit weights of the soil below creek water level were used.

The following table summarizes the groundwater information obtained at this site location.

Borehole	Date Measured	Depth	Elevation
1	December 19 and 20, 1958	3.4 m (11 ft.) Final groundwater level corresponded closely to creek water level.	174.7 m (573.3 ft.)
2	December 22, 1958	3.8 m (12.5 ft.) Final groundwater level corresponded closely to creek water level.	174.3 m (572 ft.)
3	January 1, 1959	12.2 m (40 ft.) Final groundwater level considerably below creek level	165.4 m (542.7 ft.)
	January 2 and 3, 1959	Hole caved-in. No water.	-
4	January 6, 1959	8.8 m (29 ft.) Final groundwater level considerably below creek level	168.6 m (553.1 ft.)
5	January 16, 1959	No groundwater encountered	-

6. FOUNDATION

6.1 Previous Foundation Recommendations

Engineering Considerations

1. By utilizing the soil moisture content profile versus depth, the correct unit weights and void ratios were computed backwards, assuming 100% saturation because the ‘undisturbed’ soil samples swelled upon removal from the ground.
2. The usual corrections to the consolidation e-log p curve for sample disturbance could not be applied properly due to the odd behaviour of the soil which obscures the pre-consolidation history.
3. It was understood that the final road grade at the crossing would be some 1.8 m (6 ft.) above original grade on both approaches to the creek and that the highway would cross the creek on a skew.

4. The shape of the stress-strain curves for the clay samples tested indicated that the local shear failure condition ought to be assumed and that the total allowable soil bearing pressure including the weight of 1.8 m (6 ft.) of embankment fill would be given by the equation $q_{allowable} = 3.8(C/S.F.)(1+0.3(B/L))$. With recommended values of the ultimate soil shear strength C = 105 kPa (2200 psf) and a safety factor (S.F) of 3, the equation becomes:

   $q_{allowable} = 2780(1+0.3(B/L))$ psf or $q_{allowable} = 133(1+0.3(B/L))$ kPa

   Where B = Width of rectangular footing; L = Length of footing;

   No additional bearing capacity due to surcharge was allowed because of the proximity of the footings to the creek.

5. In order to obtain an estimated magnitude of settlements involved, the following case was assumed: simply supported, single 20 m (66 ft.) span bridge, steel structure with concrete deck and concrete abutments, width 7.9 m (26 ft.), total weight approximately 357 kN (364 tons) and reaction at each end = 1619 kN (364 kips). The report stated that the above assumptions were only valid if two separate bridges were used for the Eastbound and Westbound lanes. Further, dead load, live load, wind load, weight of embankment behind the abutment, etc., were considered to utilize the full allowable soil bearing capacity. In addition, one large footing (1.5 by 7.9 m or 5 ft. by 26 ft.) at each end was assumed to estimate the settlement at various depths.

Depth m (ft.)	**P_o, kPa (psf)**	**Newmark Distribution, Δp kPa (psf)**	**$\frac{P_o + \Delta p}{P_o}$**	**Log $\frac{P_o + \Delta p}{P_o}$**	**$\frac{H_o C_i}{1+e_o}$ mm (in)**	**Settlement, S mm (in) for increment considered**
1.2 (4)	69.1 (1444)	89.8 (1876)	2.298	.3614	203.7 (8.02)	73.6 (2.90)
3.7(12)	81.4 (1700)	64.3 (1343)	1.792	.2533	206.5 (8.13)	52.3 (2.06)
10.0 (33)	146.2 (3054)	7.1 (148)	1.050	.0212	1943.1 (76.5)	39.1 (1.54)
23.3 (67)	249.3 (5206)	3.3 (68)	1.013	.0056	1986.3 (78.2)	11.1 (0.44)
					Estimated Total Settlement	**176.1 (6.94)**

Settlement of sandy and silty clay till soils was considered negligible.

It was anticipated that 20% of the total settlement would be due to recompression, and would occur rapidly during and after application of first load. Further, it was anticipated that a bridge structure would settle uniformly and differential settlements would be within practical limits for a statically determinate structure.

Recommendations and Conclusions

1. The soil report stated that a large multi span reinforced concrete box culvert may be suitable for this site. The large contact area of the culvert base with the soil would automatically insure that low unit pressures would be applied to the soil.

2. If a bridge structure was to be used it could be founded on spread footings placed in the stiff clay crust. The report recommended that the bridge footings be placed at elevation 175.0 (574 ft.), approximately similar to the water level in the creek. It was contemplated that at this elevation, the possibility of footing movements due to shrinkage or swelling accompanying soil moisture changes would be eliminated. Although scouring was not anticipated at this site location, some form of scour protection was recommended.

3. It was recommended to construct a statically determinate bridge structure for this site to eliminate large bending moments which could be induced by possible differential settlement.

4. The report indicated that some differential settlement between the bridge and the highway embankment could be expected.

5. The report suggested safe allowable loadings in the order of 144 kPa (3000 psf) may be used at the site

6. No construction problems were anticipated at this site. It was anticipated that excavation, even below the stream water level, could be maintained in a dry condition because of the impermeability nature of the cohesive soil.

Based on the Little Baptiste Creek Bridge General Layout drawing (Reference 2), the bridge was designed to be constructed as a single span 24.4 m (80 ft.) rigid frame on spread footings placed at elevation 173.1 (568 ft.). The original ground slopes were shown to be cut back and were to be graded at 2H:1V at the bridge site location.

6.2 Assessment of Foundation Parameters

Based on the previous investigation and subsurface conditions encountered, the following table summarizes the foundation design parameters that were recommended in the previous report and the updated geotechnical reaction at SLS and factored geotechnical resistance at ULS are provided.

FOUNDATION DESIGN PARAMETERS

Foundation and Type	Elevation of Footings (m)	Previous Safe Bearing Resistance (psf)[1]	Previous Equivalent Limit State Design Values		Limit State Design Values Updated to current industry practices[2]	
			SLS Geotechnical Reaction (kPa)	ULS Geotechnical Resistance Factored (kPa)	SLS Geotechnical Reaction (kPa)	ULS Geotechnical Resistance Factored (kPa)
East Abutment on Spread Footing	173.1 (568 ft.)	3000	144	215	250	375
West Abutment on Spread Footing						

Notes: 1. Working stress design values. The Ultimate Limit State design values are based on the working stress. No field verifications were made.
2. Resistance Factor = 0.5 for shallow foundation (CFEM 4th edition)
Assumed Factor of Safety is 3 (CFEM 4th edition)

The seismic site coefficient for the conditions at this site is 1.0 (soil profile Type 1, Canadian Highway Bridge Design Code (CHBDC) 2006 Edition, clause 4.4.6). The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.7.4 of the CHBDC. The foundation frost penetration depth at the site is 1.2 m according to OPSD 3090.101.

7. DISCUSSION

The Little Baptiste Creek Bridge EBL on Highway 401 is located in the Geographic Township of Tilbury East, Kent County, Ontario. The existing bridge is a single span reinforced concrete rigid frame structure that carries two lanes of Highway 401 Eastbound traffic.

From a geotechnical point of view, at the present time, foundation work for the Little Baptiste Creek Bridge EBL is not expected provided that the dead load on the overpass does not increase or decrease by more than 10%.

It is understood that rehabilitation of the bridge structure is anticipated and that rehabilitation will be completed in a single stage using median crossovers.

Further, it is suggested that the weep holes in the abutment walls should be maintained and cleaned at a regular basis to prevent any clogging of the holes. Regular maintenance of the weep holes will keep the water flowing from behind the abutment walls and will mitigate hydrostatic pressure to build-up behind the abutment walls.

In addition to the rehabilitating the bridge, the exposed earth in front of the abutment walls may be protected from scouring effects with rock protection, rip-rap or equivalent materials. The aggregate materials should conform to OPSS.PROV 1004 and the construction of the rock protection, rip-rap or equivalent material should conform to OPSS 511.

8. CLOSURE

This Technical Memorandum was prepared by Mr. Nazibur Rahman, P.Eng with the assistance of Mr. Mansoor Khorsand, EIT and was reviewed by Mr. Robert Ng, PhD, P.Eng. Mr. Brian R. Gray, MEng, P.Eng., MTO Designated Principal Contact conducted an independent review of the report.

We trust this memo is sufficient for your immediate needs. Please do not hesitate to contact us if you have any inquiries and/or comments.

Yours very truly,

Peto MacCallum Ltd.

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TABLE 1

LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

DOCUMENT	TITLE
OPSS 511	Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting
OPSS.PROV 1004	Material Specification for Aggregates - Miscellaneous
OPSD 3090.101	Foundation Frost Depth for Southern Ontario

Figure 1 – Key Plan

APPENDIX A

Soil Investigation Report – Little Baptiste Creek (GEOCRES 40J08-017)

General Layout – Little Baptiste Creek Bridge, dated July, 1959

AN INDEPENDENT ORGANIZATION PROVIDING A COMPLETE SOIL ENGINEERING & INVESTIGATION SERVICE

e. m. peto associates ltd.

YOUR REFERENCE:-

OUR REFERENCE:- 58162

850 roselawn avenue,
TORONTO 19, ONTARIO.
RUssell 1 - 4955.

January 28th, 1959.
59-F-205C

Department of Highways of Ontario,
280 Davenport Road,
Toronto, Ontario.

Re: Soil Investigation,
Highway 401 - Little Baptiste Creek Crossing,
W. P. 162-58

Attention: Mr. J. C. McAllister

Dear Sir:

We have carried out a complete soils investigation and engineering analysis of the above site in accordance with your instructions, and are pleased to submit herewith four copies of our report for this project.

For your convenience, we summarize below the salient points covered in the report.

1. The site is located on a very flat lacustrine clay plain, and the soils consist of water-laid sediments of extensive thickness.

2. Apart from 3 to 5-1/2 feet of disturbed upper soil due to excavation of the creek, and some filling for a narrow gravel roadway along the creek, the soils encountered consisted of a 120 feet-thickness of silty clay and silty clay till, underlain by dense sandy till.

The silty clay may be subdivided into three strata, each deposited at a different post-glacial or englacial stage. The uppermost layer is very stiff to stiff, and is approximately 23 to 28 feet. The underlying layers are generally firm, becoming stiff at approximately the 100 foot depth. A soil shear strength versus depth profile has been determined, and is given graphically in Appendix I to the report. The clay soil is of very low to medium compressibility, and the upper layer at least has been pre-consolidated, although we believe that it has all been pre-loaded. The clay soil has a tendency to swell upon removal of the in situ stresses, and develops swelling pressures ranging from 325 to 255 p.s.f.

A silty fine to coarse sand and gravel stratum, in a very dense condition, was encountered at the 121 foot depth in one test hole, and probably underlies the entire site. Bedrock was not encountered.

.3. The soil is virtually saturated throughout its full depth, and the ground water table has been taken to correspond with the stream water levelat elevation 574.3.

4. In our opinion a large box culvert would be suitable at this site, and this alternative merits consideration.

5. If a bridge structure is to be used for the crossing, the bridge can be founded on spread footings placed in the stiff clay crust. We recommend that such footings be placed at elevation 574.0, approximately corresponding to the creek water level. They cannot be placed deeper, since the soil shear strength steadily decreases until elevation 555; they should also not be placed at a higher elevation due to the danger of soil shrinkage and swelling with seasonal moisture content variations.

6. The recommended total safe allowable bearing pressure for rectangular footings at elevation 574.0 is given.

$$q \text{ allowable} = 2780\left(1 + 0.3\,\frac{B}{L}\right)$$

where B = width of footing
L = length of footing.

This provided a factor of safety of 3.0.

7. The total settlements which could be expected to occur under the above given loading would be in the order of 6.1/2 to 7 inches. However, approximately 20% of this would be recompression settlement occuring with the application of first load. Since soil conditions are uniform over the site, differential settlements should not be excessive for a statically determinate bridge structure.

8. There should be no construction difficulties connected with this site. Excavations below stream water level, if required can be maintained in a dry or workable condition due to the impermeable nature of the soil.

We believe this report to be complete and to contain all the information you require. However, should the Consultant require any further advice for his design, we shall be pleased to be of further service.

Yours very truly,

E. M. PETO ASSOCIATES LIMITED,

E. M. Peto, P. Eng.

MM:sb.

DEPARTMENT OF HIGHWAYS OF ONTARIO

SOILS REPORT

for

HWY. 401 - LITTLE BAPTISTE CREEK CROSSING

W.P. 162 - 58

January, 1959

e. m. peto associates ltd.

e. m. peto associates ltd., 850 roselawn avenue, Toronto 19, Ontario

Job No. 58162 Client's Ref. No. Date January 28th, 1959

Report on

SOIL SITE INVESTIGATION
at
HWY. 401 - LITTLE BAPTISTE CREEK CROSSING
W.P. 162 - 58
for
DEPARTMENT OF HIGHWAYS OF ONTARIO

INTRODUCTION:

We were retained, by letter from Mr. J. C. McAllister dated December 2nd, 1958, to carry out an investigation at the above site. At the same time we were issued with D.H.O. plan No. F-3534-7 and profile F-3534-8. Four soil test holes were proposed by the Client.

We were required.

a) to obtain a clear picture of the existing soil conditions and soil stratigraphy at this site.
b) to determine any pertinent ground water information.
c) to perform any useful laboratory soil tests which would provide information for design computations.
d) to make recommendations as to the most suitable foundation type for this site.
e) to advise of any potential construction difficulties.

PROGRAMME OF WORK:

December 11th, 1958:	Soil test holes staked out by Field Engineer, ground elevations obtained.
December 15th, 1958:	Crew and equipment moved to site from Toronto, equipment set up preparatory for working.
December 16th, 1958:	Soil sampling commenced at test hole 1.
December 24th - 30th inclusive.	No field work done.
December 31st, 1958:	Field work continued at test hole 3.
January 5th, 1959;	Original 4 hole boring programme completed. Crew and equipment moved to adjacent job at Baptiste Creek, W.P. 164-58.
January 15th, 1959:	Same field crew returned to site and drove additional test hole number 5.

GENERAL INFORMATION:

a) Our standard soil sampling procedures, described in Appendix II, were followed. When it became apparent that the clay body was very deep and quite uniform, regular sampling was discontinued at approximately the 75 foot depth. Two of the holes were then probed for refusal, using open-end A-drill rod.

b) The generally firm consistency of the clay at depth, and the high content of grits and small rock fragments, precluded the use of vane testing equipment at this site.

c) Once it became apparent that the ultimate solution to the foundation problem at this site was some type of footing founded in the upper stiff crust, an additional shallow hole was driven to the 25 foot depth. Undisturbed samples for shear strength tests were taken exclusively in this additional hole, so that a better soil shear strength profile could be obtained for the critical zone at and below foundation level.

d) Extensive laboratory tests, including consolidation tests for purposes of settlement prediction, were also carried out. A summary of all laboratory results, with supporting graphs, is given in Appendix I.

e) Detailed individual borehole logs, and a site plan showing the borehole locations and soil stratigraphy are also included.

f) The clay at this site is relatively impermeable, and as a result the recorded water levels during the course of the field work have not definitely established the position of the ground water table. All the ground water data is presented in detail in the section of the report sub-titled "Water Conditions".

SITE AND GEOLOGY:

The very flat topography in the Tilbury area, generally poor drainage, and sparse growth of trees, is identical to typical prairie topography and is indicative of a deep lacustrine clay plain.

The site investigated lies in the physiographic region known as the St. Clair Clay Plain. Although the soils consist of water-laid sediments, technically most of the soil at depth is a clay till, and the surface layer is a lacustrine clay deposit which bevelled the till plain and brought about the flat topography.

SITE AND GEOLOGY: (Cont'd)

According to geological research, this area was covered by four post-glacial lakes: an arm of early Lake Maumee, Lake Whittlesey, Lake Warren and finally the early stage of Lake St. Clair. Although the thick clay layer at this site all appears to be a uniform material (having in mind normal variations with depth), a more discerning study reveals three distinct clay types having poorly defined boundaries. As elsewhere in South-western Ontario the Lake Whittlesey and Lake Warren deposits have probably blended into what appears to be one soil type.

Between the 122 foot and 200 foot depths, most probably at about 150 feet, the site is underlain by limestone of Devonian age. Adjacent areas are underlain by black shale. Both of these rock types are important components of the clay soils at this site.

At the crossing site Little Baptiste Creek is actually a man-made drainage ditch, very straight, and of very low gradient and consequent sluggish flow. The growth of vegetation along the sides indicates that the creek has existed at least 20 years. The excavated material was piled up along both banks, and forms a sort of a levee. It does not appear as though flooding of this particular stretch of the creek ever occurs.

SOIL CONDITIONS:

Soil conditions at the site are quite uniform. The soil types encountered were:

a) Excavated Material and Fill

Adjacent to the creek only, on both sides, is a 3 ft. to 5-1/2 ft. thick layer of silty clay excavated when the drainage ditch was put through. In addition, on the Easterly side of the creek only, and parallel to it, there is a 2 to 3 foot thickness of sand and gravel fill which constitutes the gravel side road.

The silty clay material in this stratum is mottled brown in colour, stiff, dessicated and not fully saturated, and contains some sand and organic traces. Its unit weight may be taken to be 125 p.c.f.

b) Silty Clay and Stratified Silty Clay (Early Lake St. Clair)

The uppermost clay layer at this site, which is very stiff to stiff in consistency due to dessication and chemical weathering, was probably deposited as a lacustrine sediment in early Lake St. Clair. This stratum extends to a depth of between 23 ft. and 28 ft. below ground level, but the boundary between this layer and the underlying silty clay is indistinct. The differences between the two layers occurs as an increase in the stiffness and a tendency toward stratification in the upper layer.

SOIL CONDITIONS:

b) Silty Clay and Stratified Silty Clay (Early Lake St. Clair) (Cont'd)

The shrinkage limit at approximately the 10 foot depth at three of the test holes ranges from 16.2% to 18.4%, indicating that soil volume changes would accompany changes in moisture content, but these changes would not be of a serious nature. In general, the lower the shrinkage limit, the more serious the soil volume changes. Unless a very serious drought occurs in the Tilbury area, the presence of the creek at the site considered precludes large variations of soil moisture in the surface layers.

The variations of soil moisture and soil strength in this stratum are clearly shown on the appropriate graphs in Appendix I. The colour is mottled grey-brown to grey. The specific gravity of this soil was determined to be 2.69.

A consolidation test was performed on a typical sample from this stratum, and indicated that this clay has a low compressive index of 0.130, and appears to be slightly over-consolidated, with an over-consolidation ratio of 1.41. However, two separate hydrometer grain size analyses showed this soil to consist of 20 - 22% sand and grits, 39 - 40% silt, and 39 - 40% clay. It is quite likely that the high proportion of sand, grits and silt have tended to obscure the fact that this soil has been more heavily pre-loaded. Despite the results of the consolidation test, the low liquid limit, natural moisture contents near the plastic limit, reasonably high shear strengths, low sensitivity, low compressive index all point to a fairly high degree of over-consolidation. This is also corroborated by the fact that the samples swelled upon their removal from the ground. Swelling pressures in the order of 0.825 k.s.f. can be developed by this material.

The wet unit weight of the material from this stratum was found to be 131.8 p.c.f. on the basis of six actual measurements in our laboratory.

c) Silty Clay, Grits (Lake Whittlesey, Lake Warren)

Beneath the Early Lake St. Clair clay deposit is approximately a 10 foot thick transitional zone followed by a stratum approximately 50 feet thick of silty clay till with numerous grits. This 50 or 60 foot thick stratum was probably deposited in two parts consecutively by glacial Lakes Whittlesey and Warren, but if so the two deposits cannot be distinguished.

The soil in this stratum is grey in colour, and is stiff to firm in consistency. It has a very low sensitivity, and is generally wetter than, or much wetter than, the plastic limit. Variations of soil moisture content and shear strength with depth are shown on the two graphs referred to previously. This clay soil was found to swell even more than the overlying clay upon removal from the ground.

SOIL CONDITIONS:

c) Silty Clay, Grits (Lake Whittiesey, Lake Warren)

A consolidated-undrained triaxial test with pore water pressure measurements was performed on undisturbed samples from this stratum. The samples were allowed to consolidate fully under the applied lateral pressure, w hich required a time of approximately 50 hours, and were then tested to failure at a rate of strain of approximately .006 inches per minute. Since very low unit strains of 8.0% to 8.5% were developed in the samples, the pore water pressure built up due to application of axial load was very low, and the total and effective stress parameters were nearly equal. Results of this test are present in Appendix I.

A consolidation test performed on a typical sample from this stratum indicated that this soil is of medium compressibility, with a compressive index of 0.318. This clay appears to be only normally consolidated, based on the pressure-void ratio curve from the consolidation test, and the pre-consolidation history was obscure on the e-log P curve. However, the comments made for the Lake St. Clair clay also apply to this material. A swelling pressure of .255 k.s.f. was indicated by the consolidation test.

The average wet density of this stratum is 125.7 p.c.f.

d) Sandy and Silty Clay Till (Early Lake Maumee)

Below the 90 to 95 foot depth at this site is a stratum approximately 30 feet thick of sandy and silty clay with many grits. No actual sampling was performed in this stratum, but the probings indicated that it is of firm to stiff consistency, and is grey in colour. This soil can be assumed to be similar to the overlying clay, but with a lower compressive index due to the sand content.

e) Silty Fine to Coarse Sand and Gravel

Underlying the total 121 foot thickness of clay is a grey, very dense, medium to coarse sand with grits and limestone fragments, in a matrix of clayey silt. This sandy till stratum was only reached at borehole 3, but probably underlies the entire site.

WATER CONDITIONS:

The soil at this site was found to be saturated virtually throughout its full depth, with the exception of some 6 to 8 feet at surface. The average ground surface elevation at the site is 583.4, and the stream water level was 574.3 at the time of our investigation.

WATER CONDITIONS: (Cont'd)

Although it is our opinion that a true water table concept does not exist in this type of very impermeable soil, we have used submerged unit weights of the soil below stream water level for purposes of computing effective stresses.

The ground water information obtained at this site is as follows:

Borehole 1

Hole to 30 ft., casing to 25 ft., left overnight. No water.

Hole bailed to 60 ft. before pulling casing, caved in at 36 ft. after pulling casing. Depth to water = 25'0", December 18th, 1958.

Hole 35 ft. deep, depth to water = 11'0", December 19th and 20th.

Final ground water level corresponds closely to stream water level.

Borehole 2

Sampled dry to 10 ft., no water.

Bailed to 60 ft. before pulling casing, December 20th, Depth to water = 12'6" December 22nd, no casing.

Final ground water level corresponds closely to stream water level.

Borehole 3

Sampled dry to 11 ft., no water.

Hole to 35 ft., casing to 30 ft., left overnight, no water.

Hole to 70 ft., casing to 50 ft., hole bailed to 60'0", December 23rd. Depth to water = 20'0" December 31st.

Hole to 122 ft., casing to 60 ft., bailed to 54'0". Casing pulled, hole caved in at 47 ft. Depth to water = 40'0", January 1st, 1959.

Hole caved in at 37 ft. depth, no water, January 2nd and 3rd, 1959.

No gas or excess hydrostatic pressure noted at depth at this hole.

Final water level considerably below stream level.

WATER CONDITIONS: (Cont'd)

Borehole 4

Hole to 31 ft., casing to 25 ft., left overnight, no water.

Hole to 53 ft., casing to 35 ft., hole bailed January 4th. Depth to water = 30'0" January 5th.

Hole to 67 ft., casing pulled, caved in at 40 ft. depth. Depth to water = 30'0" January 5th.

Hole open to 34 ft. depth January 6th, depth to water = 29'0".

Final water level considerably below stream level.

Borehole 5

Hole to 25 ft., casing only to 10 ft. Left overnight. No water.

ENGINEERING CONSIDERATIONS:

1. Because the "undisturbed" soil samples swelled upon their removal from the ground, the densities, void ratios, and degrees of saturation obtained were somewhat erroneous. However the soil natural moisture content is a reliable physical property, determined easily. Once a good soil moisture content profile versus depth was determined, then the correct unit weights and void ratios were computed backwards, assuming 100% saturation.

2. The usual corrections for sample disturbance could not be applied properly to the consolidation test e-log p curves, because of the odd behaviour of the soil which obscures the pre-consolidation history.

3. We understand that the final road grade at the crossing is to be some 6 ft. above present grade on both approaches to the creek. The highway will cross the creek on a skew.

4. The shape of the stress-strain curves for the clay samples tested, indicate that the local shear failure condition should be assumed. The total safe allowable soil bearing pressure including weight of 6 feet of highway fill would then be given by

$$q_{\text{allowable}} = \frac{3.8 \times C}{S.F.}\left(1 + 0.3\frac{B}{L}\right)$$

where C is the ultimate soil shear strength determined by laboratory tests (we recommend from considerations of the shear strength versus depth profile that a value of 2200 p.s.f. be used).

<u>**ENGINEERING CONSIDERATIONS:**</u> **(Cont'd)**

B is the width of the footing
L is the length of the footing
S.F. is the safety factor = 3

The equation then becomes:

$$\text{Total } q_{allowable} = 2780\ (1 + 0.3\frac{B}{L})$$

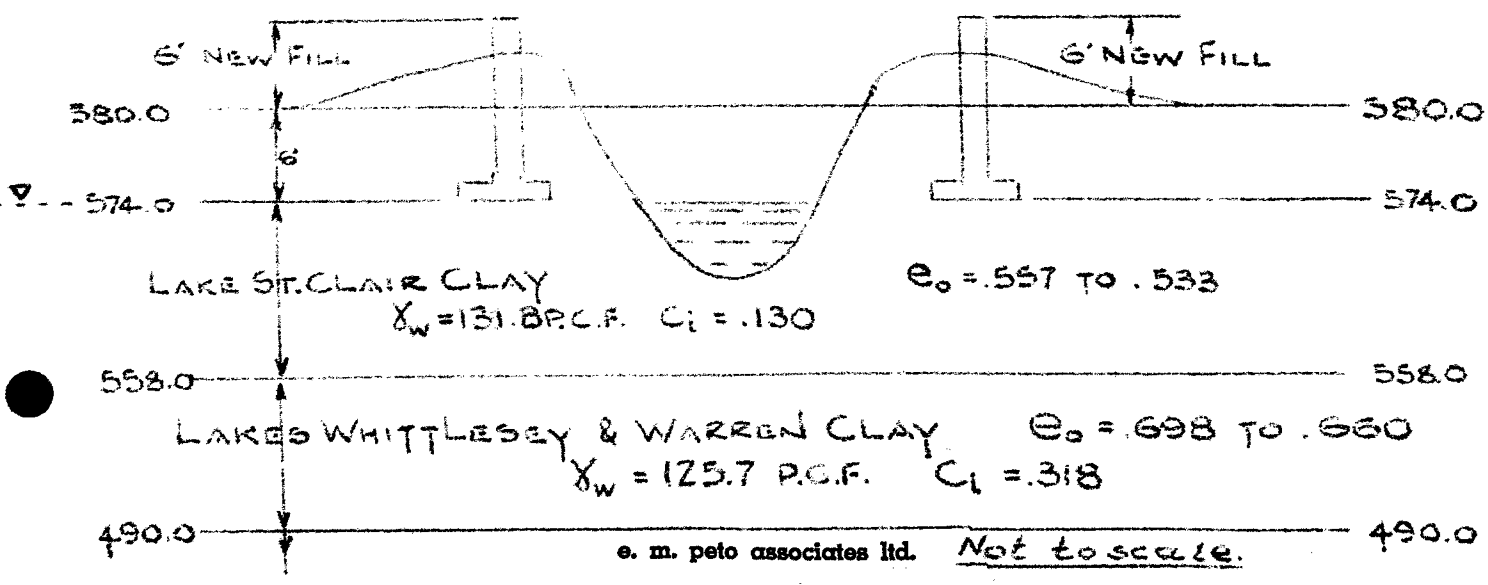
The proximity of the footing to the creek precludes the use of additional bearing capacity due to surcharge.

5. In order to obtain an idea of the magnitude of settlements involved we have assumed the following case: simply-supported, single 66 foot span bridge, steel structure with concrete deck and concrete abutments, width 26 feet, total weight approximately 364 tons, reaction each end = 364 kips. This assumption of a typical case is valid only if two separate bridges are used for the Eastbound and Westbound lanes, but in any case will serve to illustrate the settlements which could arise.

Assume one large footing each end 5' x 26'

$$q_{allowable} = 2780\ (1 + 0.3\frac{5}{26}) = 2940 \text{ p.s.f.}$$

Considering dead load, live load, wind load, weight of embankment behind the abutment, etc., the full allowable soil bearing capacity will be utilized.



Depth Below Footing ft.	p_o ps.f.	Δp p.s.f. Newmark Distribution	$\frac{p_o + \Delta p}{p_o}$	Log $\frac{p_o + \Delta p}{p_o}$	$\frac{H.C_c}{1 + e_o}$	Settlement, S inches For increment considered
4	1444	1876	2.298	.3614	8.02	2.90
12	1700	1343	1.792	.2533	8.13	2.06
33	3054	148	1.050	.0212	76.5	1.54
67	5206	68	1.013	.0056	78.2	0.44

Settlement of sandy and silty clay till considered negligible

ESTIMATED TOTAL SETTLEMENT = 6.94 inches

Approximately 20% of the total settlement would be due to recompression, and would occur rapidly during and after application of first load.

The uniform soil conditions on both sides of the creek lead us to believe that a bridge structure would settle uniformly, and differential settlements should be within practical limits for a statically determinate structure.

RECOMMENDATIONS AND CONCLUSIONS:

1. It is our opinion that a large box culvert would be suitable at this site, and this possibility should be carefully considered. The large contact area of the culvert base with the soil, governed by hydraulic considerations, would automatically ensure that low unit pressures would be applied to the soil.

2. If a bridge structure is contemplated for this site, the bridge can be founded on spread footings placed in the stiff clay 'crust'.

   We recommend that the bridge footings be placed at elevation 574.0, approximately corresponding to water level in the creek, so that the possibility of footing movements caused by shrinkage or swelling accompanying soil moisture changes will be largely eliminated.

   Although scour is unlikely to occur at this site, some form of scour protection should be provided along the inside face of the footings.

3. The settlement computations given above illustrate that it is desirable to use a statically determinate bridge structure for this site, so that no large bending moments will be induced by possible differential settlement.

4. Some differential settlement between the bridge and the highway embankment can be expected.

RECOMMENDATIONS AND CONCLUSIONS: (Cont'd)

5. The bearing capacity formula given in item 4 of "Engineering Considerations" shows that safe allowable loadings in the order of 3000 ps.f. may be used at this site.

6. There should be no construction problems at this site, and excavations even below the stream water level can be maintained in a dry condition because of the impermeable nature of the silty clay soil.

E. M. PETO ASSOCIATES LTD.,

E. M. Peto, P. Eng.

MM:sb

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE TORONTO, ONTARIO

BOREHOLE LOG

Job Name Hwy. 401-Little Baptiste Cr. Crossing Job No. 58162 Borehole No. 1

Client Dept. of Highways of Ontario. Casing BX (2½" diam.) Boring Date Dec. 16-18, 1958.

Datum Geodetic Compiled By M. Mindess Checked By E. M. Peto

SAMPLE CONDITION	SAMPLE TYPE	ABBREVIATIONS
UNDISTURBED	S.S. 2" STANDARD SPLIT TUBE SAMPLE	V.T. IN SITU VANE SHEAR TEST
FAIR	S.L. SPLIT BARREL WITH LINERS	Qu UNCONFINED COMPRESSIVE STRENGTH
DISTURBED	S.T. THIN-WALLED SHELBY TUBE SAMPLE	W.L. WATER LEVEL IN CASING
LOST	W.S. WASH SAMPLE	W.T. GROUND WATER TABLE IN SOIL
	R.C. ROCK CORE	

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft	WATER LEVELS, SOIL MOISTURE & REMARKS
			0' 0" 584.3					
SANDY CLAY	MOTTLED BROWN							
SILTY CLAY, FISSURED, ORGANIC TRACES. AS ABOVE	MOTTLED BROWN, MOTTLED	STIFF			1	S.S.	13	NAT. M.C.=16.4% DRIER THAN PLASTIC LIMIT.
			5' 0"					
SILTY CLAY, FISSURED.	GREY-BROWN	STIFF			2	S.C.	14	NAT. M.C.=21.9% DRIER THAN PLASTIC LIMIT.
			10' 0"					
SILTY CLAY, GRITS AND STONES TO ¾" SIZE.	BROWN	STIFF			3	S.S.	19	NAT. M.C.=17.6% AT PLASTIC LIMIT.
			14' 0" 570.3					NAT. M.C.=18.1%
SILTY CLAY, MANY BLACK GRITS.	DARK GREY	STIFF			4	S.S.	22	WETTER THAN PLASTIC LIMIT.
			20' 0"					
STRATIFIED SILTY CLAY, NUMEROUS BLACK GRITS.	DARK GREY	FIRM TO STIFF			5	S.S.	11	NAT. M.C.=20.4% MUCH WETTER THAN PLASTIC LIMIT.
			25' 0"					
SILTY CLAY, GRITS	DARK GREY	FIRM			6 A B C	2" S.L.	PUSHED	NAT. M.C.=22.1% C=1370 P.S.F. @ 20% STRAIN
SILTY CLAY, GRITS AND LIMESTONE PEBBLES TO ½" SIZE.	DARK GREY	FIRM			7	S.S.	12	MUCH WETTER THAN PLASTIC LIMIT.
			30' 0"					
SILTY CLAY, GRITS	GREY	FIRM			8	2" S.L.	PUSHED	NAT. M.C.=23.3–25.3% C= 1150 — 793 P.S.F. @ 20% STRAIN.
			35' 0"					
SILTY CLAY, BLACK GRITS.	GREY	FIRM			9	S.S.	8	MUCH WETTER THAN PLASTIC LIMIT.
			40' 0"					
AS ABOVE	"	SOFT			10 A B C	2" S.L.	PUSHED	NAT. M.C.=25.1% C=650 P.S.F.
AS ABOVE	GREY	SOFT			11	S.S.	7	MUCH WETTER THAN PLASTIC LIMIT.
			45' 0"					
		"			12	2" S.L.	PUSHED	
			50' 0"					
AS ABOVE	GREY	SOFT			13	S.S.	8	AS ABOVE
			55' 0"					
AS ABOVE	GREY	SOFT			14	2" S.L.	PUSHED	NAT. M.C.=26.6–26.3% C= 416 P.S.F.
			60' 0"					
AS ABOVE, 3" SEAM OF GRITS AND PEA GRAVEL IN MATRIX OF SILTY CLAY.	"	"			15	2" S.L.	PUSHED	NAT. M.C.=22.6% C=650 P.S.F.
			65' 0"					
					16	S.S.	8	
			70' 0"					
					17	2" S.L.	PUSHED	
SILTY CLAY, BLACK GRITS.	GREY	SOFT			18	S.S.	7	MUCH WETTER THAN PLASTIC LIMIT. C= APPROX. 300 P.S.F.
			75' 0"					
			80' 0"					
AS ABOVE	GREY	SOFT			19	S.S.	9	AS ABOVE
			85' 0"					
AS ABOVE						WASHED OPEN-END A-ROD		CASING WITHDRAWN DEC. 18. HOLE CAVED IN TO 35' DEPTH. DEPTH TO WATER=11'0" DEC. 20, 1958.
			90' 0"					
AS ABOVE	GREY	SOFT	95' 0" 489.3		20	DRIVE A-ROD	(6)	MUCH WETTER THAN PLASTIC LIMIT

HOLE TERMINATED. NO STIFFENING OR REFUSAL

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE – TORONTO, ONTARIO

BOREHOLE LOG

Job Name Hwy. 401-Little Baptiste Cr. Job No. 58162 Borehole No. 2

Client Dept. of Highways of Ontario Casing 4" pipe + BX Boring Date Dec. 18-20, 1958.

Datum Geodetic Compiled By M. Mindess Checked By E. M. Peto

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	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
GROUND FROZEN TO 1'3".			0' 0" 584.5					
FISSURED SILTY CLAY, GRITS, ORGANIC TRACES.	GREY-BROWN				1A	SAMPLE FROM CASING		AT PLASTIC LIMIT.
SILTY CLAY, GRITS AND PEBBLES, SILT POCKETS.	MOTTLED GREY-BROWN	STIFF			1B	S.S.	14	NAT. M.C.=20.5 % DRIER THAN PLASTIC LIMIT.
			5' 0"					
SILTY CLAY, GRITS AND PEBBLES, SOME ORGANIC MATTER	AS ABOVE	STIFF			2	S.S.	12	NAT. M.C.=19.8 % DRIER THAN PLASTIC LIMIT.
			10' 0"		2A	SAMPLE FROM CASING	–	
SILTY CLAY, GRITS AND PEBBLES.	AS ABOVE	STIFF			3	S.S.	13	NAT. M.C.=23.7 % DRIER THAN PLASTIC LIMIT.
			14' 0" 570.5					
STRATIFIED SILTY CLAY, BLACK GRITS.	DARK GREY	STIFF			4	S.S.	19	NAT. M.C.= 18.7 % WETTER THAN PLASTIC LIMIT. S.G.=2.69
			20' 0"					
		"			5A B C	3" S.L.	TAPPED	
		STIFF			6	S.S.	18	
			25' 0"					
					7	3" S.L.	PUSHED	
			30' 0"					
SILTY CLAY, GRITS.	GREY	FIRM			8	S.S.	7	MUCH WETTER THAN PLASTIC LIMIT.
			35' 0"					
					9	3" S.L.	PUSHED	
AS ABOVE	GREY	FIRM.			10	S.S.	6	MUCH WETTER THAN PLASTIC LIMIT.
			40' 0"					
					11A B C	2" S.L.	PUSHED	
AS ABOVE	GREY	FIRM.			12	S.S.	7	AS ABOVE.
			45' 0"					
			50' 0"					
" "					13A B	2" S.L.	PUSHED	
			55' 0"					
AS ABOVE	GREY	FIRM			14	S.S.	8	MUCH WETTER THAN PLASTIC LIMIT.
			60' 0"					
					15A B	2" S.L.	PUSHED	
SILTY CLAY, BLACK GRITS.	GREY	FIRM	65' 0" 519.5		16	S.S.	9	MUCH WETTER THAN PLASTIC LIMIT.

HOLE TERMINATED.
NO STIFFENING OR REFUSAL.

CASING WITHDRAWN, DEC. 20, 58.
DEPTH TO WATER= 12'6"
DEC. 22, '58.

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE TORONTO, ONTARIO

BOREHOLE LOG

Job Name Hwy. 401-Little Baptiste Cr. Crossing Job No. 58162 Borehole No. 3

Client Dept. of Highways of Ontario. Casing BX (2¼" diam.) Boring Date Dec. 22-Jan. 1, 1958-9.

Datum Geodetic Compiled By M. Mindess Checked By E. M. Peto

SAMPLE CONDITION	SAMPLE TYPE	ABBREVIATIONS
UNDISTURBED	S.S. 2" STANDARD SPLIT TUBE SAMPLE	V.T. IN-SITU VANE SHEAR TEST
FAIR	S.L. SPLIT BARREL WITH LINERS	Qu UNCONFINED COMPRESSIVE STRENGTH
DISTURBED	S.T. THIN-WALLED SHELBY TUBE SAMPLE	W.L. WATER LEVEL IN CASING
LOST	W.S. WASH SAMPLE	W.T. GROUND WATER TABLE IN SOIL
	R.C. ROCK CORE	

Soil Description	Colour	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	Water Levels, Soil Moisture & Remarks
			0' 0" 582.65					
SILTY CLAY, GRITS.	MOTTLED GREY-BROWN.				1A	SAMPLE FROM CASING		AT PLASTIC LIMIT
AS ABOVE, ORGANIC CONTENT.		STIFF	3' 0"		1B	S.S.	10	NAT. M.C.=29.9 %. WETTER THAN PLASTIC LIMIT
ORGANIC SILTY LOAM.	DARK BROWN.		3'6" 579.6					
	MOTTLED		5' 0"					
SILTY CLAY, GRITS.	GREY-BROWN	STIFF			2	S.S.	9	NAT. M.C.=22.4%. WETTER THAN PLASTIC LIMIT. C = APPROX. 2300 PSF
			10' 0"					
STRATIFIED SILTY CLAY, BLACK GRITS.	BROWN	STIFF			3	S.S.	27	NAT. M.C.=18.2 %. WETTER THAN PLASTIC LIMIT.
			15' 0"					
					4 A B C	2" S.L.	PUSHED & TAPPED	
AS ABOVE.	DARK GREY	STIFF			5	S.S.	19	MUCH WETTER THAN PLASTIC LIMIT
			20' 0"					
AS ABOVE.	GREY	STIFF			6	S.S.	14	AS ABOVE.
			25' 0"					
					7 A B C	2" S.L.	PUSHED	
AS ABOVE.	GREY	STIFF			8	S.S.	16	MUCH WETTER THAN PLASTIC LIMIT.
			30' 0"					
			35' 0"					
					9 A B C	S.L.	PUSHED	
SILTY CLAY, GRITS TO ½" SIZE.	GREY	FIRM			10	S.S.	9	MUCH WETTER THAN PLASTIC LIMIT.
			40' 0"					
			45' 0"					
					11 A B C	2" S.L.	PUSHED	
SILTY CLAY, GRITS.	GREY	FIRM			12	S.S.	7	MUCH WETTER THAN PLASTIC LIMIT.
			50' 0"					
			55' 0"					
AS ABOVE	GREY	FIRM			13 A B C	2" S.L.	PUSHED	
AS ABOVE	"	"			14	S.S.	9	AS ABOVE.
			60' 0"					
			65' 0"					
"	"	"			15 A B C	2" S.L.	PUSHED	
SILTY CLAY, GRITS.	GREY	FIRM			16	S.S.	10	MUCH WETTER THAN PLASTIC LIMIT
			70' 0"					
AS ABOVE	"	"			17 A B C	2" S.L.	PUSHED	
			75' 0"					
SILTY CLAY, NUMEROUS GRITS.	GREY	FIRM			18	S.S.	7	AS ABOVE
					WASHED A-ROD AHEAD TO PROBE FOR REFUSAL. No CASING DRIVEN BEYOND 75'.			
			80' 0"					
			85' 0"					
AS ABOVE	"	"						
GRADING TO			90' 0" 492.6					
SANDY AND SILTY CLAY, MANY GRITS.	GREY	PROBABLY FIRM						
			95' 0"					
			100' 0"					
			105' 0"					
			110' 0"					
								CASING WITHDRAWN JAN. 1, 1959. HOLE CAVED IN TO 37' DEPTH. NO WATER, JAN. 3, 1959
			115' 0"					
AS ABOVE	"	"	121' 0"					
MEDIUM TO COARSE SAND, GRITS AND LIMESTONE FRAGMENTS, IN MATRIX OF CLAYEY SILT.	GREY	VERY DENSE	122' 0" 460.6		19	DROVE OPEN-END A-ROD	(68)	SATURATED

HOLE TERMINATED NO REFUSAL

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE – TORONTO, ONTARIO

BOREHOLE LOG

Job Name Hwy. 401-Little Baptiste Cr. Crossing   Job No. 58162   Borehole No. 4

Client Dept. of Highways of Ontario   Casing 4" pipe + BX   Boring Date Jan. 2-5, 1959.

Datum Geodetic   Compiled By M. Mindess   Checked By E. M. Peto

SAMPLE CONDITION	SAMPLE TYPE	ABBREVIATIONS
UNDISTURBED	S.S. 2" STANDARD SPLIT TUBE SAMPLE	V.T. IN SITU VANE SHEAR TEST
FAIR	S.L. SPLIT BARREL WITH LINERS	Q/u UNCONFINED COMPRESSIVE STRENGTH
DISTURBED	S.T. THIN-WALLED SHELBY TUBE SAMPLE	W.L. WATER LEVEL IN CASING
LOST	W.S. WASH SAMPLE	W.T. GROUND WATER TABLE IN SOIL
	R.C. ROCK CORE	

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft	WATER LEVELS, SOIL MOISTURE & REMARKS
DEPTH OF FROST = 24"			0' 0"					
SILTY CLAY, GRITS AND PEBBLES.	GREY-BROWN.		582.1			SAMPLE FROM CASING		AT PLASTIC LIMIT.
AS ABOVE, ORGANIC CONTENT.	MOTTLED GREY-BROWN	FIRM			1	S.S.	6	NAT. M.C.=24.2% AT PLASTIC LIMIT.
			5' 0"					
SILTY CLAY, GRITS AND PEBBLES.	AS ABOVE	STIFF			2	S.S.	10	NAT. M.C.=24.3% WETTER THAN PLASTIC LIMIT. C=2000 P.S.F.
			10' 0"					
STRATIFIED SILTY CLAY, GRITS AND ROCK FRAGMENTS TO 2" SIZE.	BROWN	STIFF			3	S.S.	14	NAT. M.C.=17.8% DRIER THAN PLASTIC LIMIT.
			15' 0"					
STRATIFIED SILTY CLAY, GRITS.	GREY	STIFF			4	S.S.	12	NAT. M.C.=18.4% WETTER THAN PLASTIC LIMIT.
			20' 0"					
					5	3" S.L.	TAPPED	
					6	3" S.T.	TAPPED	
AS ABOVE	GREY	FIRM TO STIFF	25' 0"		7	S.S.	11	WETTER THAN PLASTIC LIMIT. NAT. M.C.= 20.6%
SMALL BOULDER AT 26'								
			30' 0"					
SILTY CLAY, GRITS	GREY	FIRM			8	S.S.	8	MUCH WETTER THAN PLASTIC LIMIT.
					9	3" S.L.	PUSHED	
			35' 0"					
AS ABOVE	"	"			10	3" S.T.	PUSHED.	
			40' 0"					
AS ABOVE	"	"			11	S.S.	8	NAT. M.C.=26.5% MUCH WETTER THAN PLASTIC LIMIT.
			45' 0"					
					12	3" S.T.	PUSHED	
AS ABOVE	"	"			13	S.S.	9	MUCH WETTER THAN PLASTIC LIMIT.
			50' 0"					
			55' 0"					
AS ABOVE	GREY	FIRM			14	S.S.	7	AS ABOVE.
			60' 0"					
					15	S.S.	8	NAT. M.C.=24.9% MUCH WETTER THAN PLASTIC LIMIT.
			65' 0"					
					16	2" S.L.	PUSHED	
SILTY CLAY, GRITS	GREY	FIRM	67' 0" 515.1		17	S.S.	10	MUCH WETTER THAN PLASTIC LIMIT.

HOLE TERMINATED. NO STIFFENING OR REFUSAL.

CASING WITHDRAWN, JAN. 5, 1959.
HOLE CAVED IN TO 34' DEPTH.
DEPTH TO WATER = 29'.
JAN. 6

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE – TORONTO, ONTARIO

BOREHOLE LOG

Job Name Hwy. 401-Little Baptiste Cr. Crossing Job No. 58162 Borehole No. 5

Client Dept. of Highways of Ontario. Casing BX ($2\frac{1}{2}$" diam.) Boring Date Jan. 15, 1959.

Datum Geodetic Compiled By M. Mindess Checked By E. M. Peto

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	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft	WATER LEVELS, SOIL MOISTURE & REMARKS
EXCAVATED MATERIAL: FISSURED SILTY AND SANDY CLAY, GRITS AND PEBBLES, ORGANIC TRACES	MOTTLED GREY-BROWN		0' 0" 584.4					
ORGANIC SILTY CLAY TOPSOIL	DARK BROWN		5' 0"					
SILTY CLAY, GRITS.	MOTTLED GREY-BROWN.	VERY STIFF			1 A B C	2" S.L.	TAPPED	DRIER THAN PLASTIC LIMIT NAT. M.C. = 21.8 %.
SILTY CLAY, GRITS.	MOTTLED GREY-BROWN.	VERY STIFF	10' 0"		2 A B C	2" S.L.	TAPPED	WETTER THAN PLASTIC LIMIT NAT. M.C. = 19.2 %.
SILTY CLAY, GRITS AND PEBBLES.	GREY	STIFF	14' 0" 15' 0"		3 A B C	2" S.L.	TAPPED	WETTER THAN PLASTIC LIMIT. NAT. M.C. = 18.2 %.
AS ABOVE.	"	FIRM	20' 0"		4 A B C	2" S.L.	TAPPED	WETTER THAN PLASTIC LIMIT. NAT. M.C. = 19.7 %.
AS ABOVE.	GREY	FIRM	25' 0" 26' 6" 557.9		5 A B C	2" S.L.	PUSHED	MUCH WETTER THAN PLASTIC LIMIT. NAT. M.C. = 20.1 % TO 19.7 %.

HOLE TERMINATED.

HOLE TO 25 FT. DEPTH, ONLY UPPER 10 FT. CASED, NO WATER OVERNIGHT JAN. 15 TO JAN. 16, 1959.

APPENDIX I

LABORATORY TEST RESULTS

SHRINKAGE LIMITS

Job No. 58162

Borehole Number	1	2	3	3
Sample Number	3	2A	3	3
Depth	10'-11'	9'6"-10'	10'-11'	10'-11'
Wt. of dish & wet soil - gms.	53.06	50.66	51.84	50.74
Wt. of dish & dry soil - gms.	44.89	40.75	43.00	39.80
Wt. of dish gms.	24.93	24.93	24.93	17.35
Wt. of water gms.	8.17	9.91	8.84	10.94
Wt. of dry soil - gms. (W_o)	19.96	15.82	18.07	22.45
Moisture Content % (W)	41.0	62.6	48.8	48.3
Volume of dish c.c. (V)	15.70	15.70	15.70	19.20
Volume of dry soil c.c. (V_o)	10.79	8.70	9.80	12.30
Shrinkage Volume c.c. ($V-V_o$)	4.91	7.00	5.90	6.90
Shrinkage Limit = (W_s) $W - \left(\frac{V-V_o}{W_o} \times 100\%\right)$	16.4%	18.4%	16.2%	17.6%
Shrinkage Ratio (R) = $\frac{W_o}{V_o}$	1.855	1.820	1.830	1.825
S. G. = $\frac{1}{1/R - W_s/100}$	2.63	2.74	2.61	2.69

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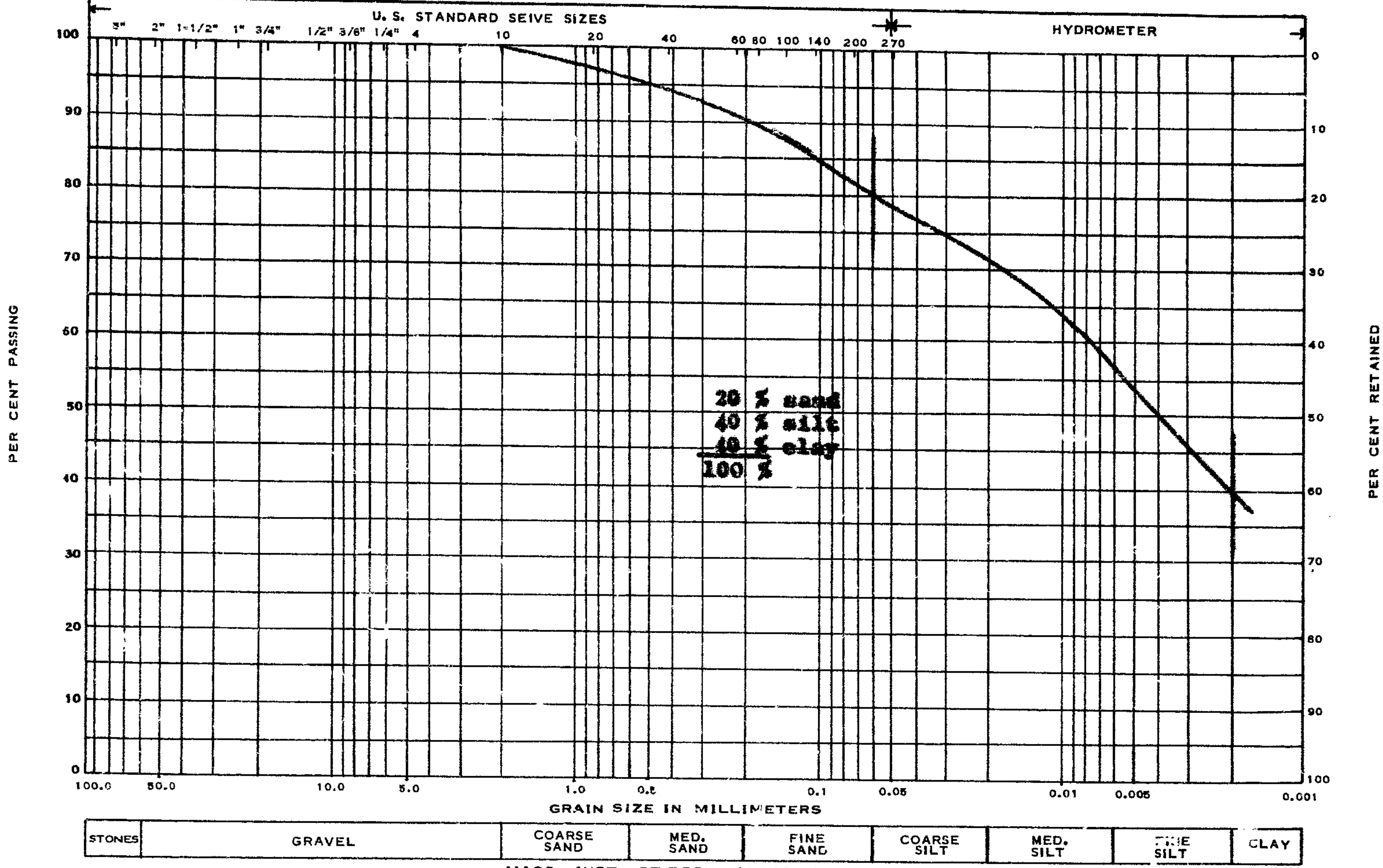
TORONTO, ONTARIO



MASS. INST. OF TECH. CLASSIFICATION

JOB NAME Hwy. 401-Little Baptiste Cr. JOB NO. 58162 HOLE NO. 2 SAMPLE NO. 4

DEPTH 15'-16' ELEVATION 569.0 REMARKS

GRAIN SIZE DISTRIBUTION

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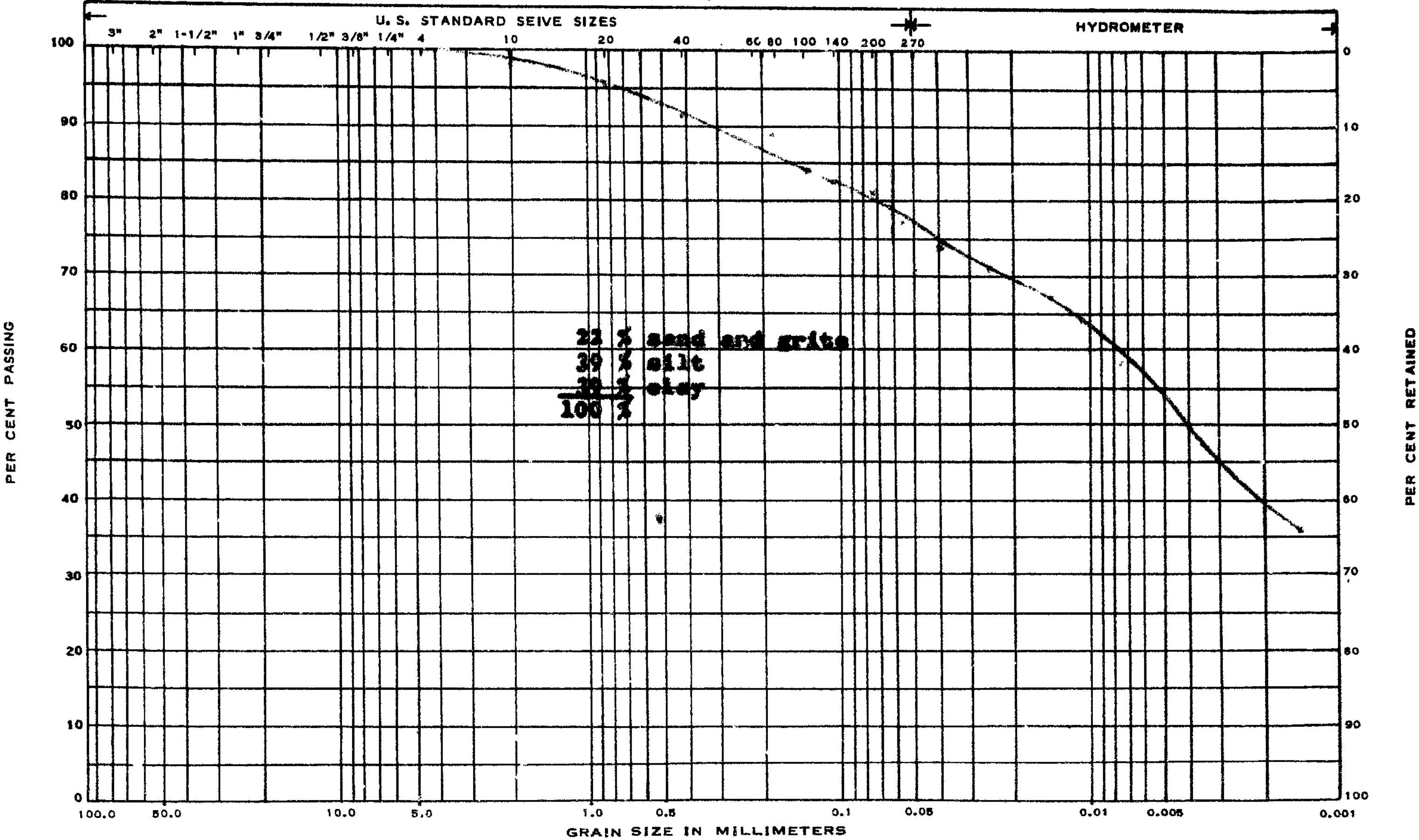
TORONTO, ONTARIO



STONES	GRAVEL	COARSE SAND	MED. SAND	FINE SAND	COARSE SILT	MED. SILT	FINE SILT	CLAY

MASS. INST. OF TECH. CLASSIFICATION

JOB NAME Hwy. 401-Little [illegible] Cr. JOB NO. 58162 HOLE NO. 1 SAMPLE NO. 5

DEPTH 20'-21' ELEVATION 563.[illegible] REMARKS

GRAIN SIZE DISTRIBUTION

e. m. peto associates ltd.
SOIL TESTING LABORATORY

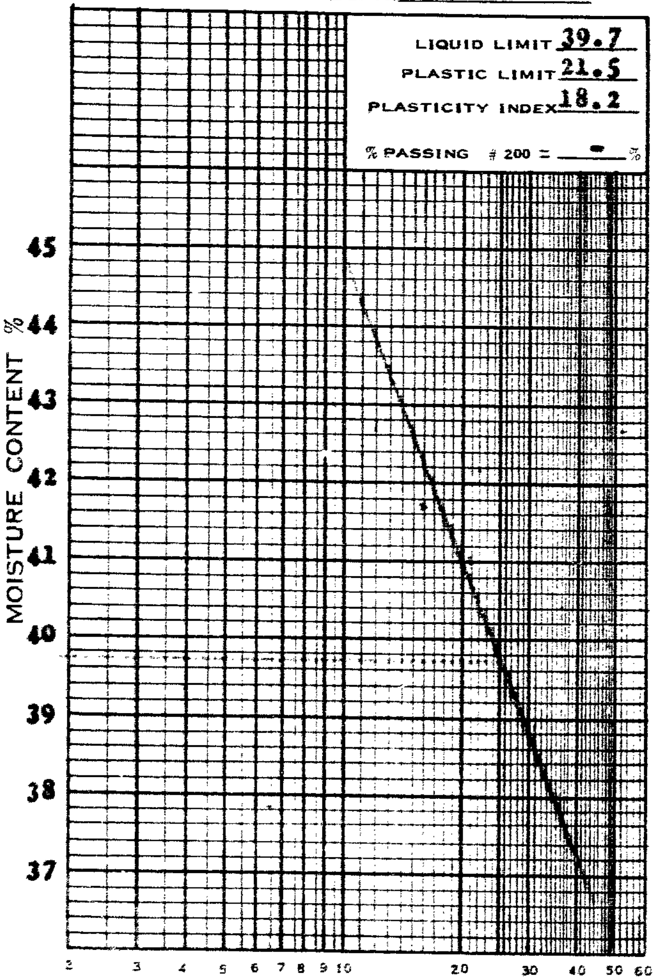
LIQUID LIMIT TEST FLOW LINE CHARTS

JOB No 58162 PROJECT Hwy. 401 - Little Baptiste Creek Crossing

SAMPLE FROM Borehole 1, Sa. 3

DEPTH 10'- 11'

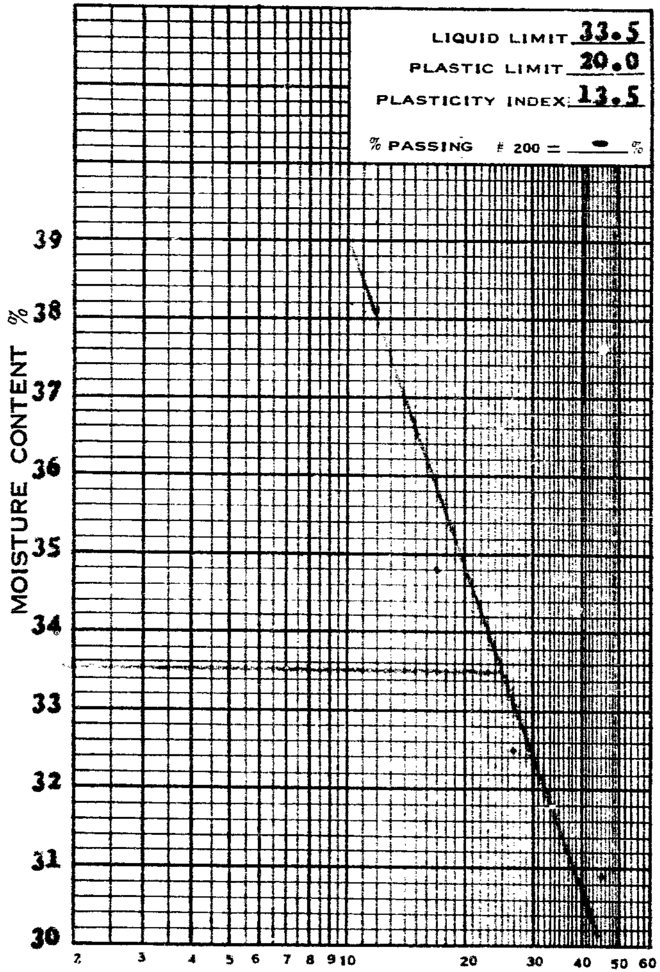
LIQUID LIMIT 39.7
PLASTIC LIMIT 21.5
PLASTICITY INDEX 18.2
% PASSING # 200 = - %



SAMPLE FROM Borehole 1, Sa. 4

DEPTH 15'- 16'

LIQUID LIMIT 33.5
PLASTIC LIMIT 20.0
PLASTICITY INDEX 13.5
% PASSING # 200 = - %



NO. OF BLOWS (LOG SCALE)

e. m. peto associates ltd.

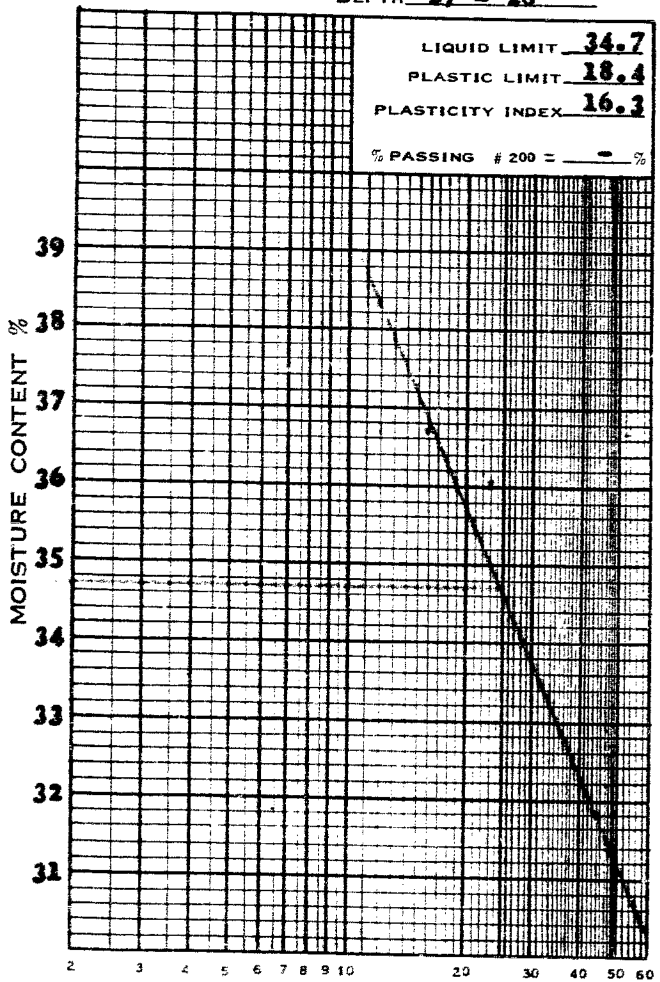
SOIL TESTING LABORATORY

LIQUID LIMIT TEST FLOW LINE CHARTS

JOB No. 58162 PROJECT Hwy. 401 - Little Baptiste Creek Crossing

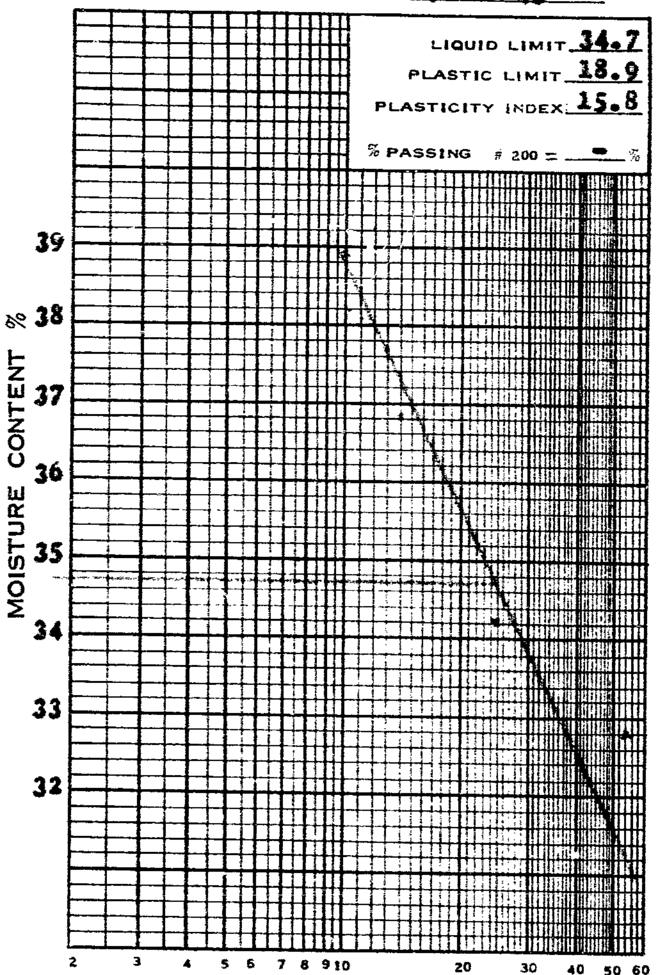
SAMPLE FROM Borehole 1, Sa. 7

DEPTH 27' - 28'



SAMPLE FROM Borehole 1, Sa. 11

DEPTH 42' - 43'



NO. OF BLOWS (LOG SCALE)

e. m. peto associates ltd.

SOIL TESTING LABORATORY

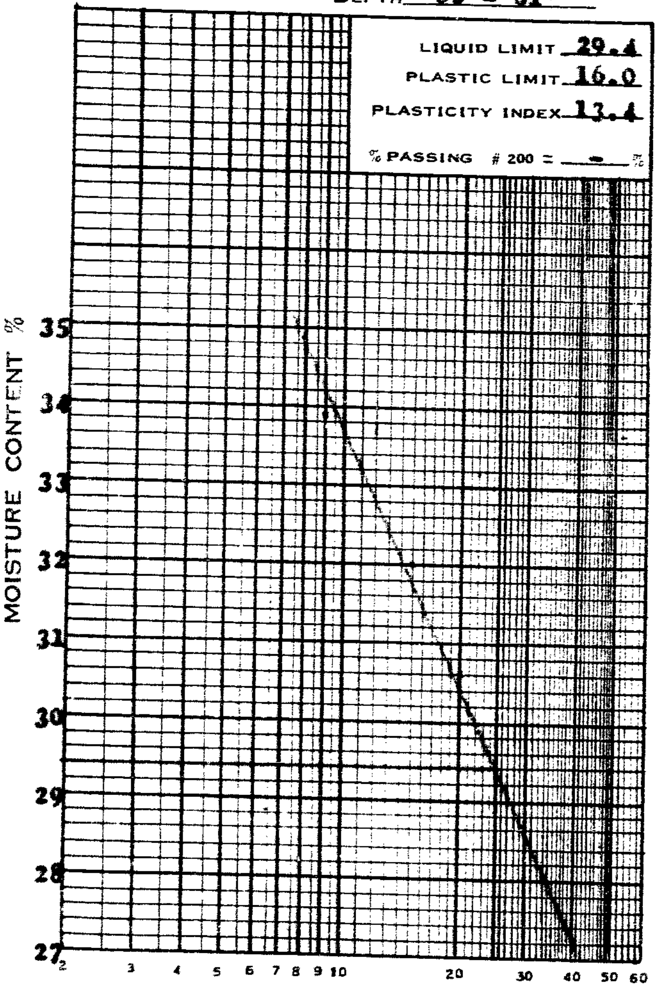
LIQUID LIMIT TEST — FLOW LINE CHARTS

JOB No. 58162 PROJECT Hwy. 401 - Little Baptiste Creek Crossing

SAMPLE FROM Borehole 1, Sa. 19

DEPTH 80'- 81'

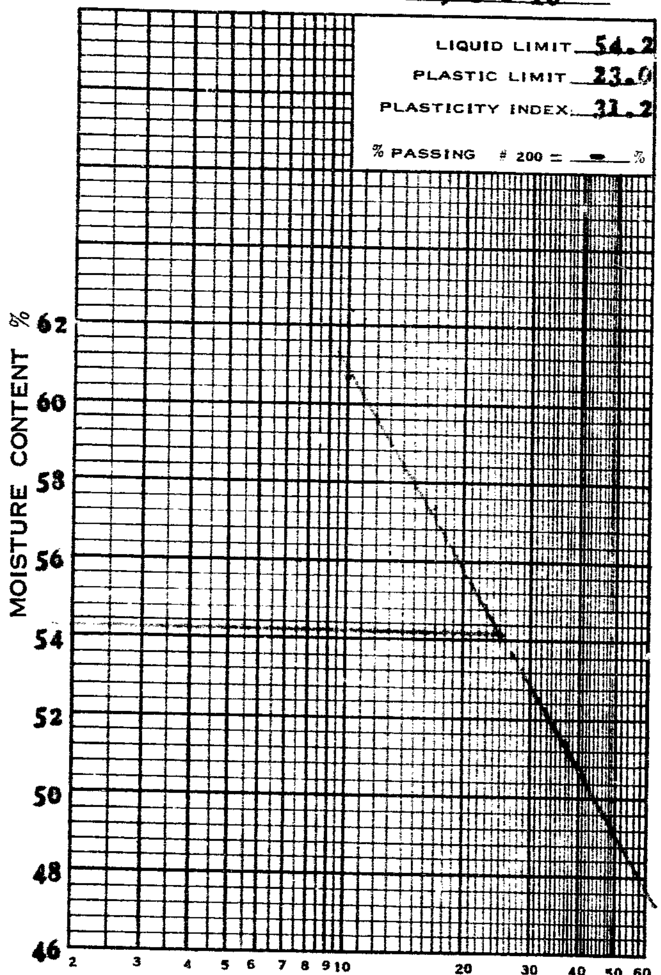
LIQUID LIMIT 29.4
PLASTIC LIMIT 16.0
PLASTICITY INDEX 13.4
% PASSING # 200 = - %



SAMPLE FROM Borehole 2, Sa. 2A

DEPTH 9'6"- 10'

LIQUID LIMIT 54.2
PLASTIC LIMIT 23.0
PLASTICITY INDEX 31.2
% PASSING # 200 = - %



NO. OF BLOWS (LOG SCALE)

e. m. peto associates ltd.

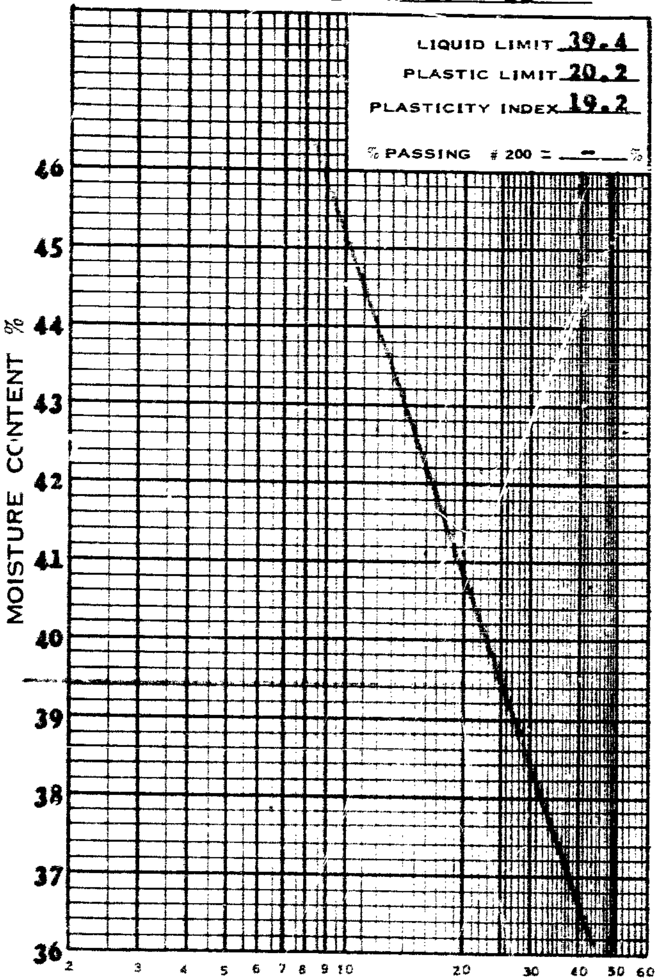
SOIL TESTING LABORATORY

LIQUID LIMIT TEST FLOW LINE CHARTS

JOB NO. 58162 PROJECT Hwy. 401 - Little Baptiste Creek Crossing

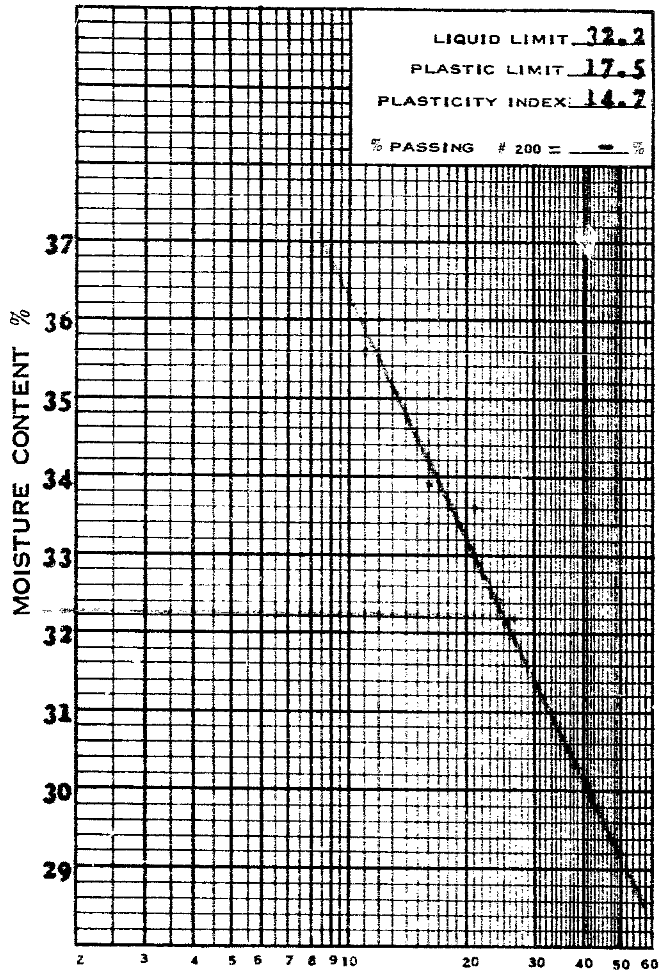
SAMPLE FROM Borehole 3, Sa. 3

DEPTH 10'- 11'



SAMPLE FROM Borehole 3, Sa. 6

DEPTH 20'- 21'



NO. OF BLOWS (LOG SCALE)

e. m. peto associates ltd.

SOIL TESTING LABORATORY

LIQUID LIMIT TEST **FLOW LINE CHARTS**

JOB NO. 58162 PROJECT Hwy. 401 - Little Baptiste Creek Crossing

SAMPLE FROM Borehole 3, Sa. 14

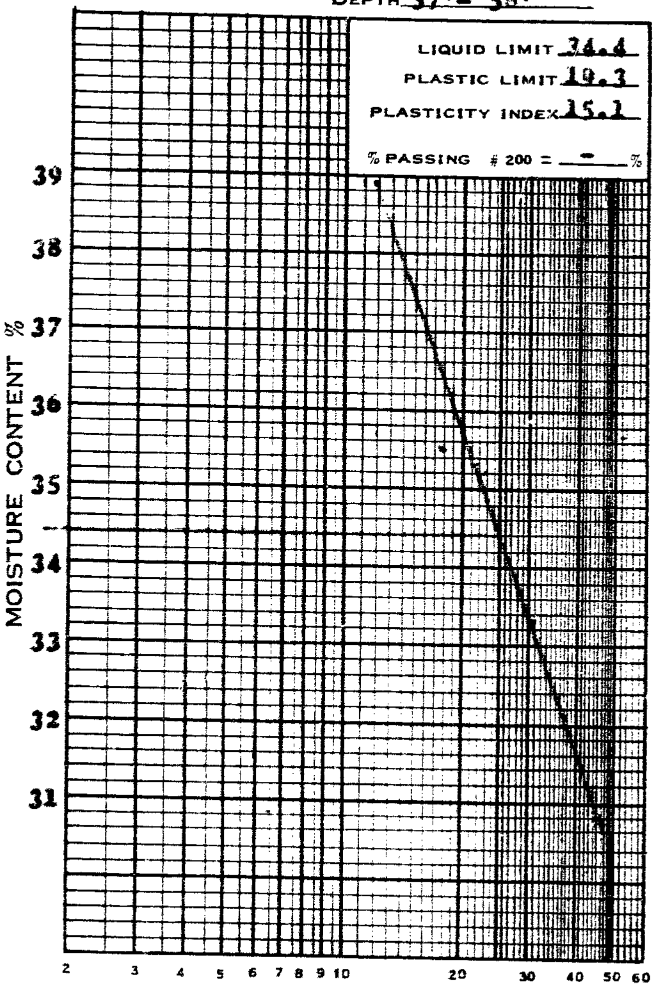
DEPTH 57' - 58'

LIQUID LIMIT 34.4

PLASTIC LIMIT 19.3

PLASTICITY INDEX 15.1

% PASSING # 200 = - %



SAMPLE FROM

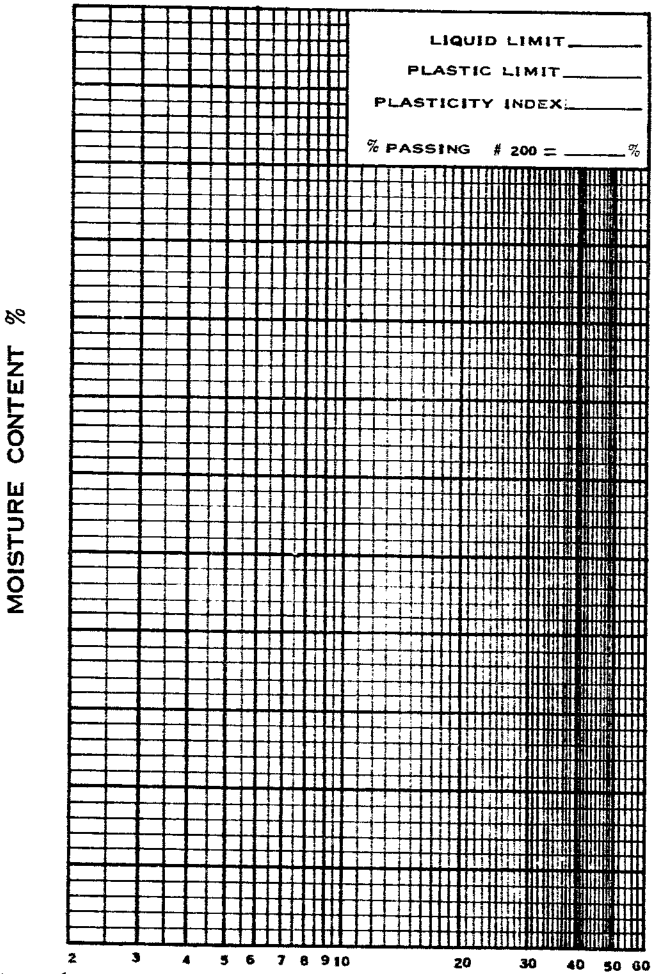
DEPTH

LIQUID LIMIT

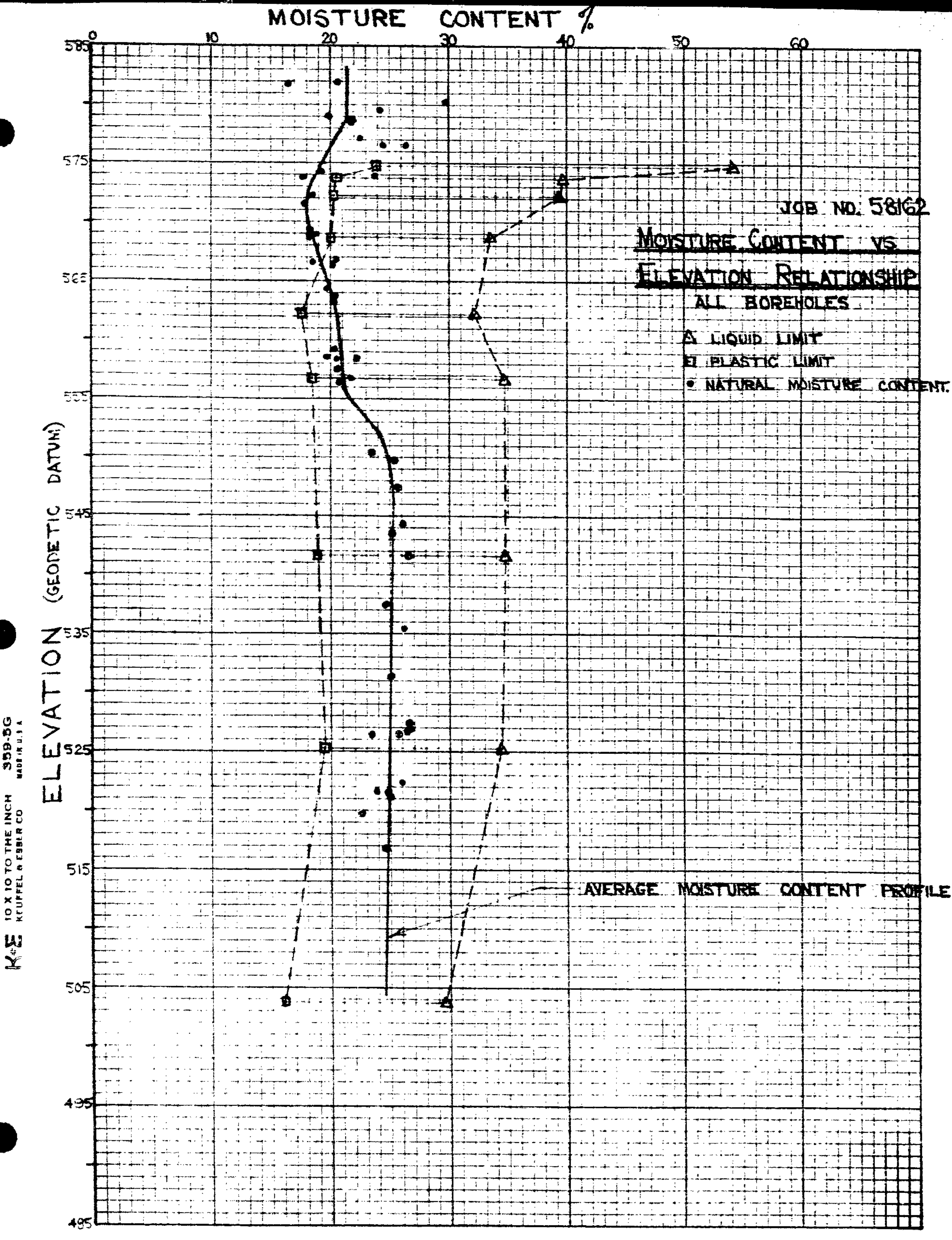
PLASTIC LIMIT

PLASTICITY INDEX

% PASSING # 200 = %



NO. OF BLOWS (LOG SCALE)


MOISTURE CONTENT %
0
10
20
30
40
50
60
585
575
565
555
545
535
525
515
505
495
485
ELEVATION (GEODETIC DATUM)
JOB NO. 58162
MOISTURE CONTENT VS. ELEVATION RELATIONSHIP
ALL BOREHOLES
LIQUID LIMIT
PLASTIC LIMIT
NATURAL MOISTURE CONTENT.
AVERAGE MOISTURE CONTENT PROFILE.
10 X 10 TO THE INCH 359-5G
KEUFFEL & ESSER CO. MADE IN U.S.A.

SUMMARY OF SHEAR STRENGTH TEST RESULTS

Job. No. 58162

Borehole	Sample number	Depth	Elevation	Natural M.C. %	Wet Density p.c.f.	Dry Density p.c.f.	Degree of Saturation	Void Ratio	% Strain at failure	Shear Strength, C p.s.f.
1	6B	25'6"-26'	553.6	22.1	131.0	107.0	100 %	0.560	20%	1370
1	8B	33'6"-34'	550.6	23.3	129.0	104.5	100	0.607	20	1150
1	8C	34'-34'6"	550.1	25.3	128.0	102.0	100	0.644	20	793
1	10B	40'6"-41'	543.6	25.1	125.9	100.6	100	0.667	20	650
1	14B	56'6" -57'	527.6	26.6	125.1	99.0	100	0.682	20	416
										Remould 274
1	14C	57'-57'6"	527.1	26.3	125.2	99.2	100	0.680	20	416
										Remould 216
1	15C	64'-64'6"	520.1	22.6	123.5	100.5	100	0.660	20	650
										Remould 345
2	11A	40'-40'6"	544.3	26.1	126.8	100.5	100	0.660	20	574
2	13A	53'-53'6"	531.3	25.1	125.8	100.4	100	0.660	20	426
2	15A	62'-62'6"	522.3	26.0	125.4	99.6	100	0.683	20	486
2	15B	62'6"-63'	521.8	23.9	127.4	102.9	100	0.629	13.3	686
										Remould 443
2	15C	63'-63'6"	521.3	25.0	124.9	99.8	100	0.690	20	574
3	4B	15'6"-16'	566.9	20.3	135.0	112.2	100	0.493	20	2296
										Remould 1968
3	4C	16'-16'6"	566.4	20.1	135.0	112.4	100	0.492	20	1904
3	7B	25'6"-26'	556.9	21.6	137.1	112.8	100	0.489	20	1640
3	7C	26'-26'6"	556.4	20.8	133.1	110.3	100	0.520	20	1640
										Remould 1500
3	9A	35'-35'6"	547.4	25.7	132.3	105.4	100	0.590	20	450
3	11A	45'-45'6"	537.4	24.6	127.3	102.3	100	0.630	20	541
3	13B	55'6"-56'	526.9	26.5	126.0	99.5	100	0.698	10.0	838x
3	13C	56'-56'6"	526.4	25.7	126.0	100.2	100	0.672	8.5	1030x
3	13C	56'-56'6"	526.4	23.3	126.0	102.1	100	0.643	12.5	861x
3	15B	65'6"-66'	516.9	24.6	126.1	101.2	100	0.658	20	500
4	2	5'-6'	576.6	26.4	123.9	98.0	99.7	0.711	5.7	2000
4	6	23-1/2'-24'	558.4	20.7	134.0	111.1	100	0.500	20	1510

SUMMARY OF SHEAR STRENGTH TEST RESULTS

Job No. 58 162

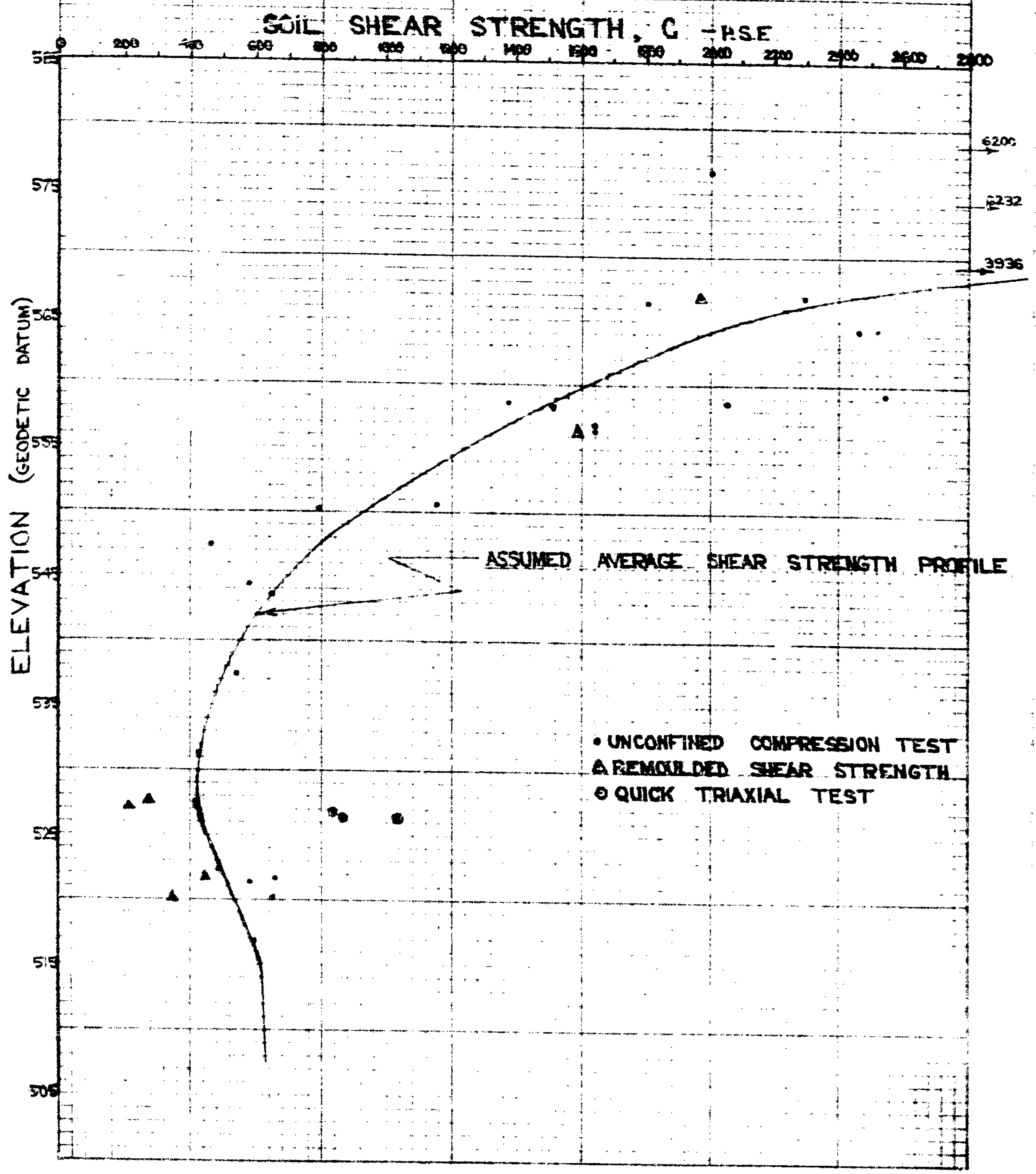
Borehole	Sample Number	Depth	Elevation	Natural M. C. %	Wet Density p.c.f.	Dry Density p.c.f.	Degree of Saturation	Void Ratio	% Strain at Failure	Shear Strength, C p.s.f.
5	1B	5-1/2'-6'	578.7	21.8	122.1	100.3	86.2	0.683	3.3	6202
5	2A	10'-10-1/2'	574.2	19.2	134.8	112.9	100	0.497	12.7	6232
5	3A	15'-15-1/2'	569.2	18.2	134.0	113.3	100	0.488	20	3936
5	4A	20'-20-1/2'	564.2	18.7	135.6	113.3	100	0.488	20	2480
5	5A	25'-25-1/2'	559.2	20.1	132.9	110.5	100	0.524	20	2542
5	5B	25-1/2'-26'	558.7	19.7	131.7	110.0	100	0.532	20	2050

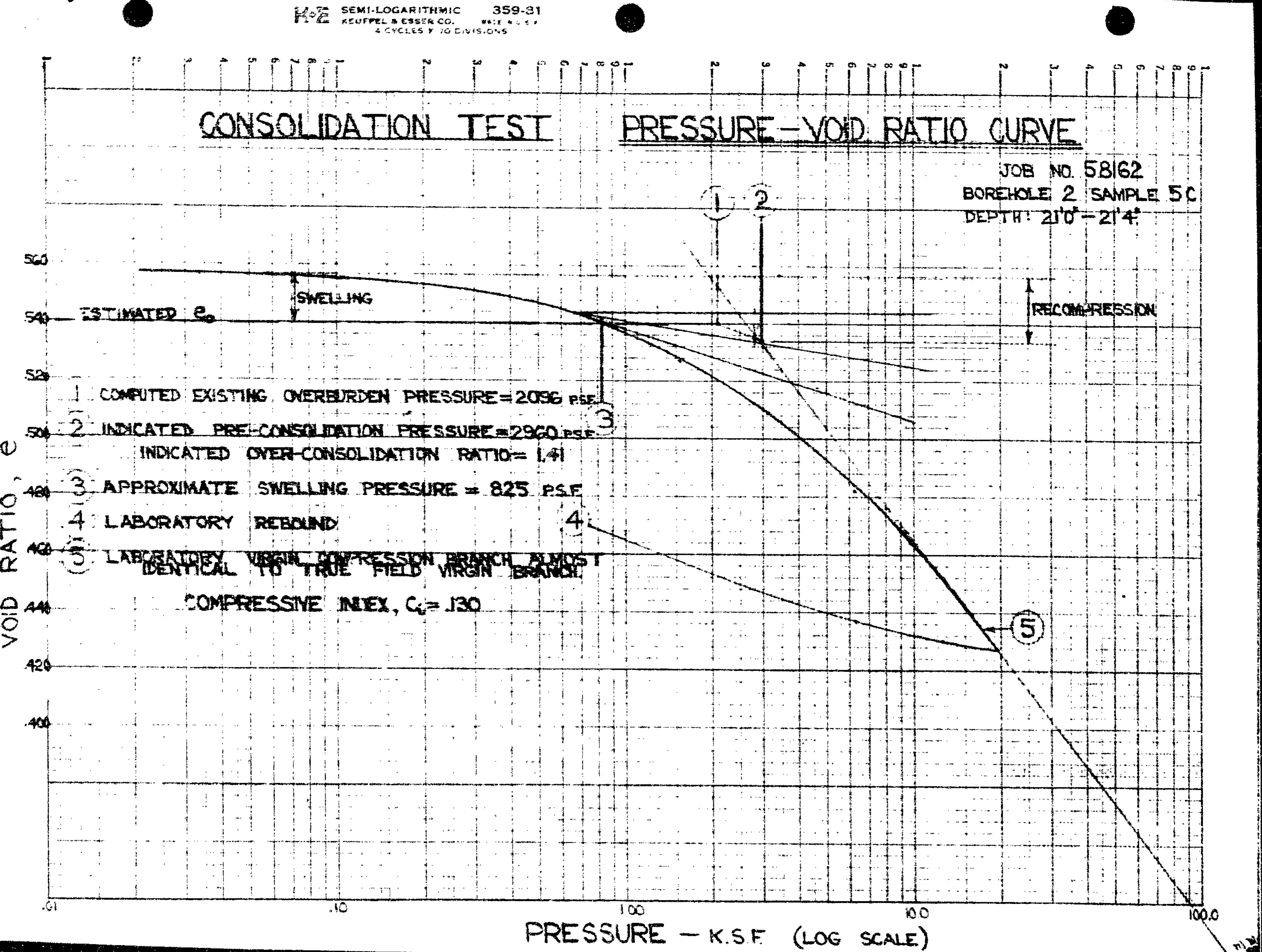
x Denotes 1/2 x deviator stress from quick triaxial compression test.

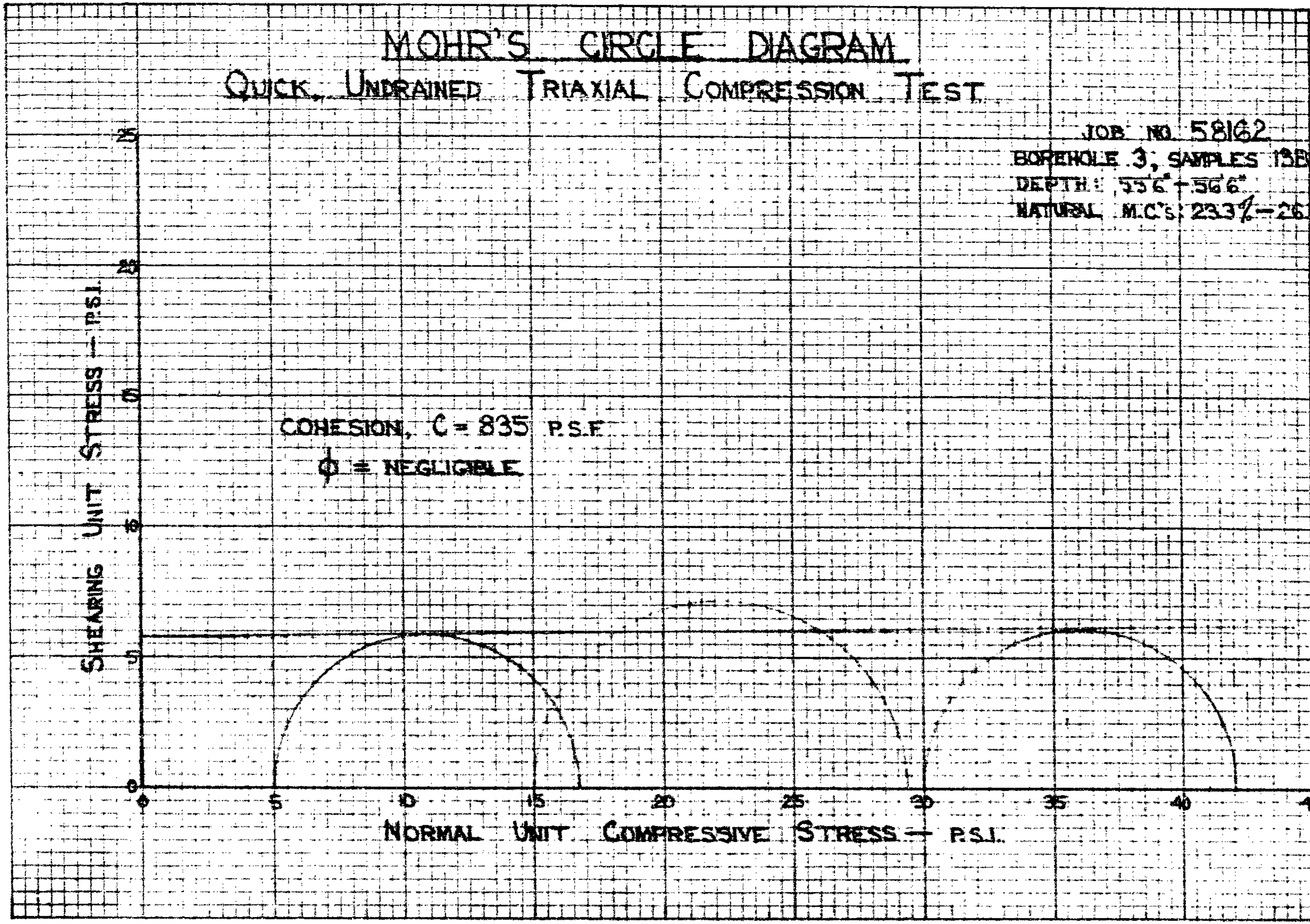
JOB NO. 58162

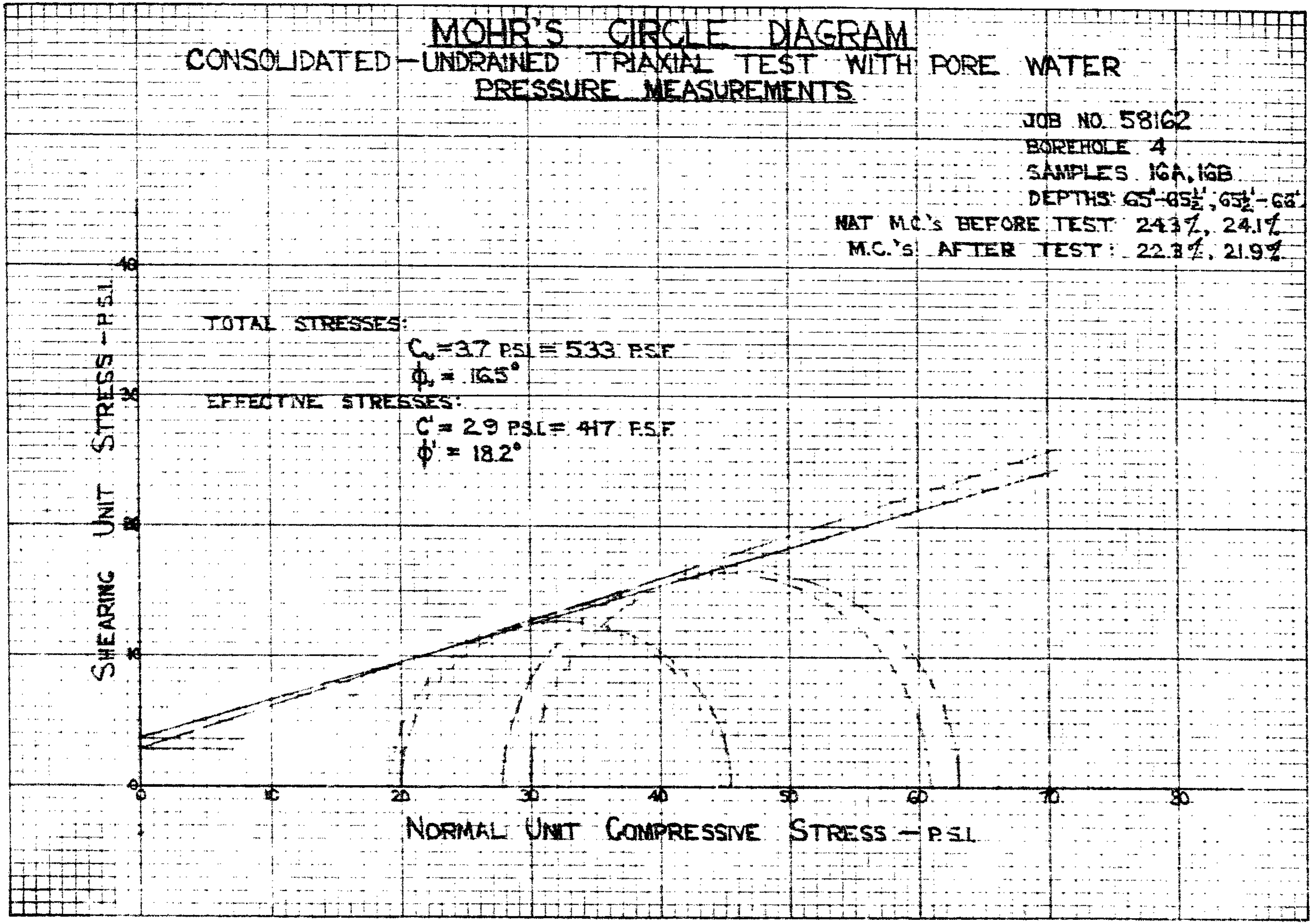
PLOT OF SHEAR STRENGTH vs. ELEVATION

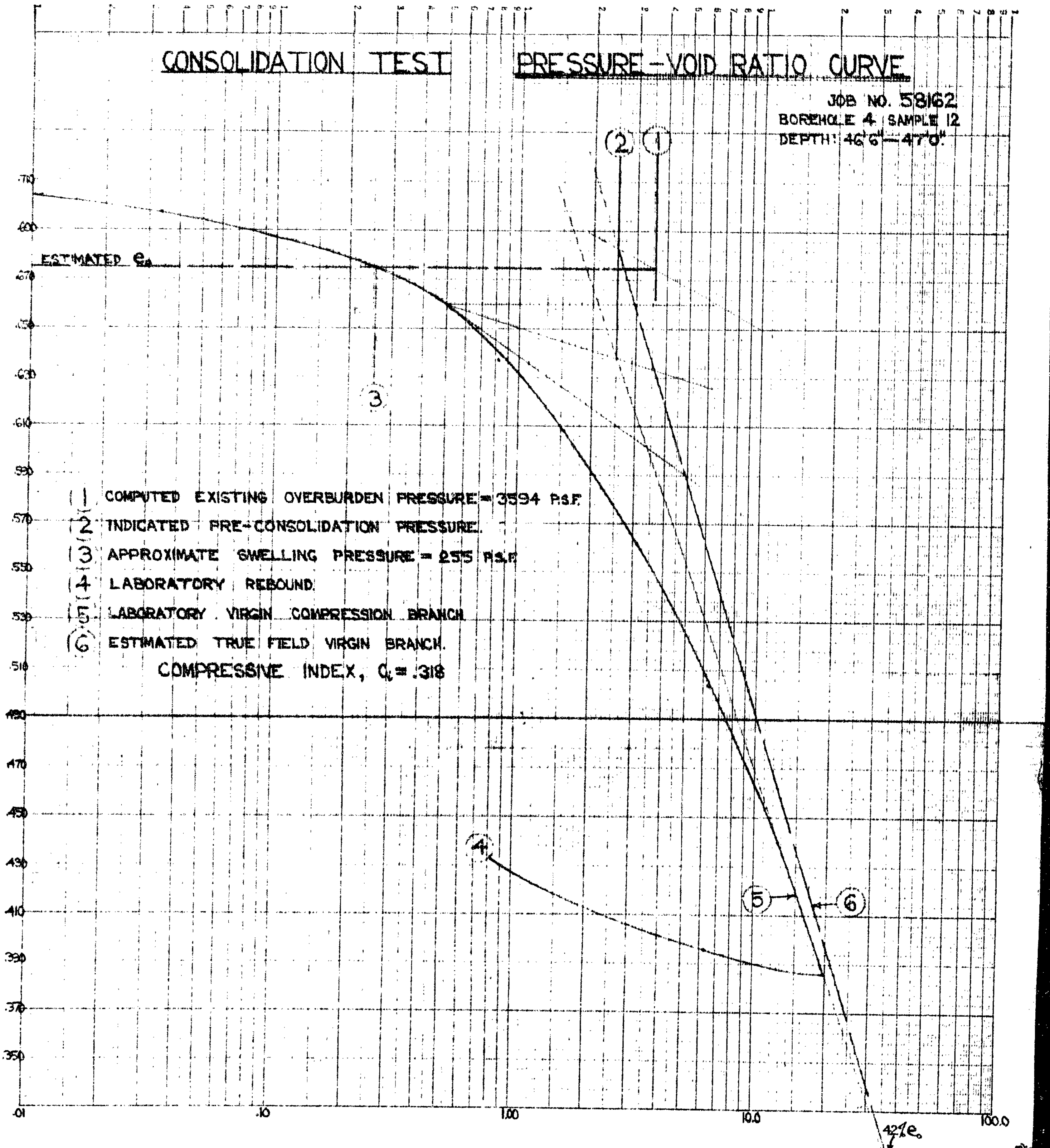
COMPOSITE RESULTS OF ALL ENE BOREHOLES




K+E SEMI-LOGARITHMIC 359-81
KEUFFEL & ESSER CO.
4 CYCLES X 70 DIVISIONS
CONSOLIDATION TEST
PRESSURE-VOID RATIO CURVE
JOB NO. 58162
BOREHOLE 2 SAMPLE 5C
DEPTH: 21'0" - 21'4"
SWELLING
ESTIMATED e_o
RECOMPRESSION
1 COMPUTED EXISTING OVERBURDEN PRESSURE = 2096 PSF
2 INDICATED PRE-CONSOLIDATION PRESSURE = 2960 PSF
INDICATED OVER-CONSOLIDATION RATIO = 1.41
3 APPROXIMATE SWELLING PRESSURE = 825 PSF
4 LABORATORY REBOUND
5 LABORATORY VIRGIN COMPRESSION BRANCH ALMOST IDENTICAL TO TRUE FIELD VIRGIN BRANCH
COMPRESSIVE INDEX, C_c = .130
.560
.540
.520
.500
.480
.460
.440
.420
.400
VOID RATIO, e
.01
.10
1.00
10.0
100.0
PRESSURE — K.S.F. (LOG SCALE)


MOHR'S CIRCLE DIAGRAM
QUICK, UNDRAINED TRIAXIAL COMPRESSION TEST
JOB NO. 58162
BOREHOLE 3, SAMPLES 13B, 13C.
DEPTH: 55'6" - 56'6"
NATURAL M.C.'s: 23.3% - 26.5%
COHESION, C = 835 P.S.F.
φ = NEGLIGIBLE
SHEARING UNIT STRESS — P.S.I.
NORMAL UNIT COMPRESSIVE STRESS — P.S.I.
0
5
10
15
20
25
30
35
40
45


MOHR'S CIRCLE DIAGRAM
CONSOLIDATED—UNDRAINED TRIAXIAL TEST WITH PORE WATER PRESSURE MEASUREMENTS
JOB NO. 58162
BOREHOLE 4
SAMPLES 16A, 16B
DEPTHS: 65'-65½', 65½'-66'
NAT M.C.'s BEFORE TEST 24.3%, 24.1%
M.C.'s AFTER TEST: 22.3%, 21.9%
TOTAL STRESSES:
C = 3.7 P.S.I. = 533 P.S.F.
φ = 16.5°
EFFECTIVE STRESSES:
C' = 2.9 P.S.I. = 417 P.S.F.
φ' = 18.2°
SHEARING UNIT STRESS — P.S.I.
NORMAL UNIT COMPRESSIVE STRESS — P.S.I.
0
10
20
30
40
50
60
70
80



CONSOLIDATION TEST PRESSURE-VOID RATIO CURVE
JOB NO. 58162
BOREHOLE 4 SAMPLE 12
DEPTH: 46'6"—47'0"
ESTIMATED e_o
(1) COMPUTED EXISTING OVERBURDEN PRESSURE = 3594 P.S.F.
(2) INDICATED PRE-CONSOLIDATION PRESSURE.
(3) APPROXIMATE SWELLING PRESSURE = 255 P.S.F.
(4) LABORATORY REBOUND.
(5) LABORATORY VIRGIN COMPRESSION BRANCH.
(6) ESTIMATED TRUE FIELD VIRGIN BRANCH.
COMPRESSIVE INDEX, C_c = .318
.710
.690
.670
.650
.630
.610
.590
.570
.550
.530
.510
.490
.470
.450
.430
.410
.390
.370
.350
.01
.10
1.00
10.0
100.0
42% e_o

APPENDIX II

METHOD OF OPERATION

The field investigation work is carried out by means of a skid-mounted diamond drill rig.

Standard sampling procedures are followed. Casing is driven and cleaned, either by tubes or by wash water.

Samples are recovered ahead of the casing at frequent intervals, with either a 2 inch or 3 inch O. D. split barrel sampling tube, Shelby tube, or split barrel sampling tube fitted with brass liners and special sharp cutting nose.

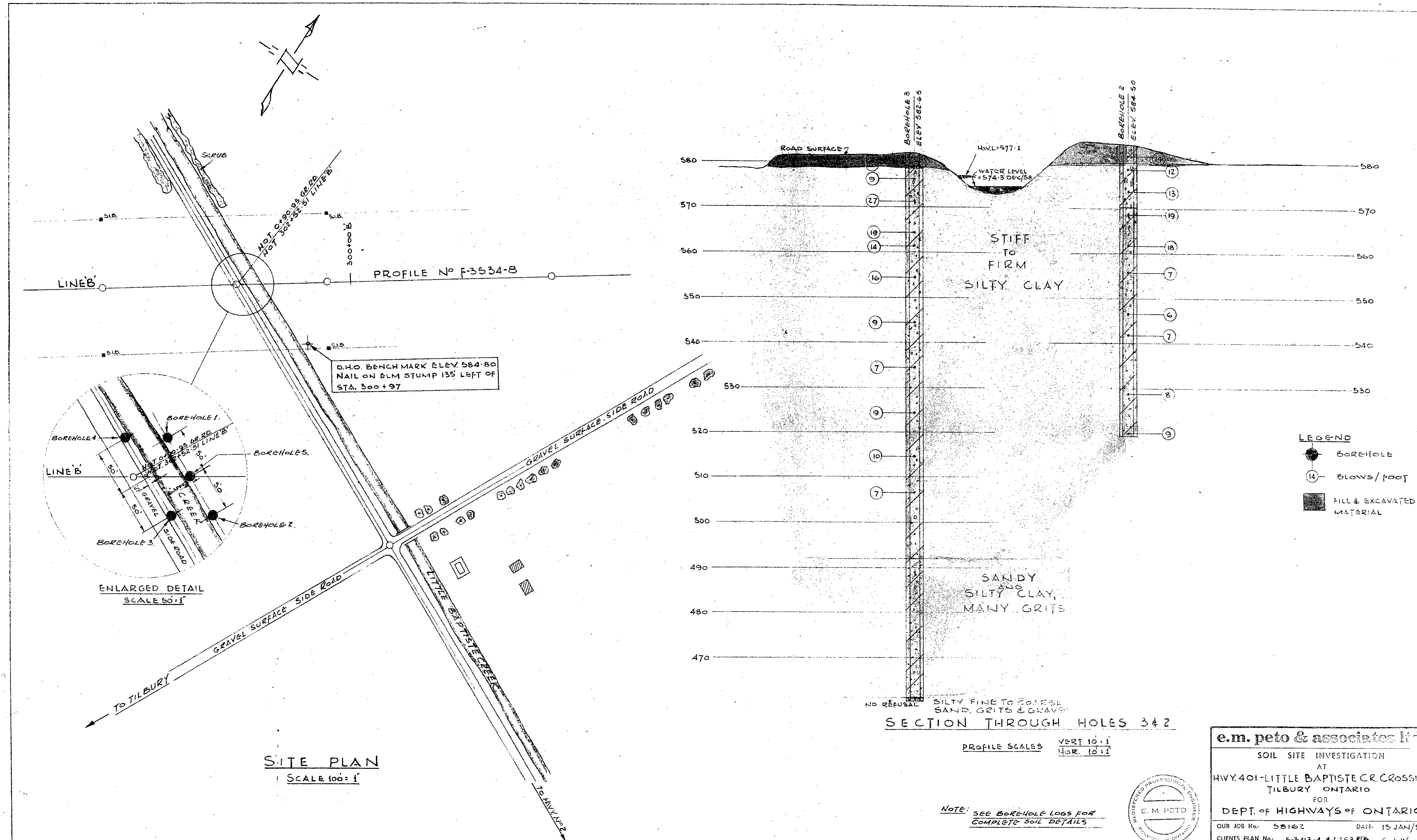
The standard penetration test results are recorded when sampling with the regular 2 inch O. D. split barrel sampler, these being the number of blows of a 140 pound hammer falling 30 inches, required to drive the sampling tube a distance of one foot into undisturbed soil.

The Dutch cone probe test is made by driving the drill rods into the ground with a 2-1/4" - 90° cone tip. The number of 4200 inch pound blows per foot of penetration are recorded, as in the standard penetration test.

Where required, "in situ" shear strength tests are made ahead of the casing, using modified Acker vane test equipment.

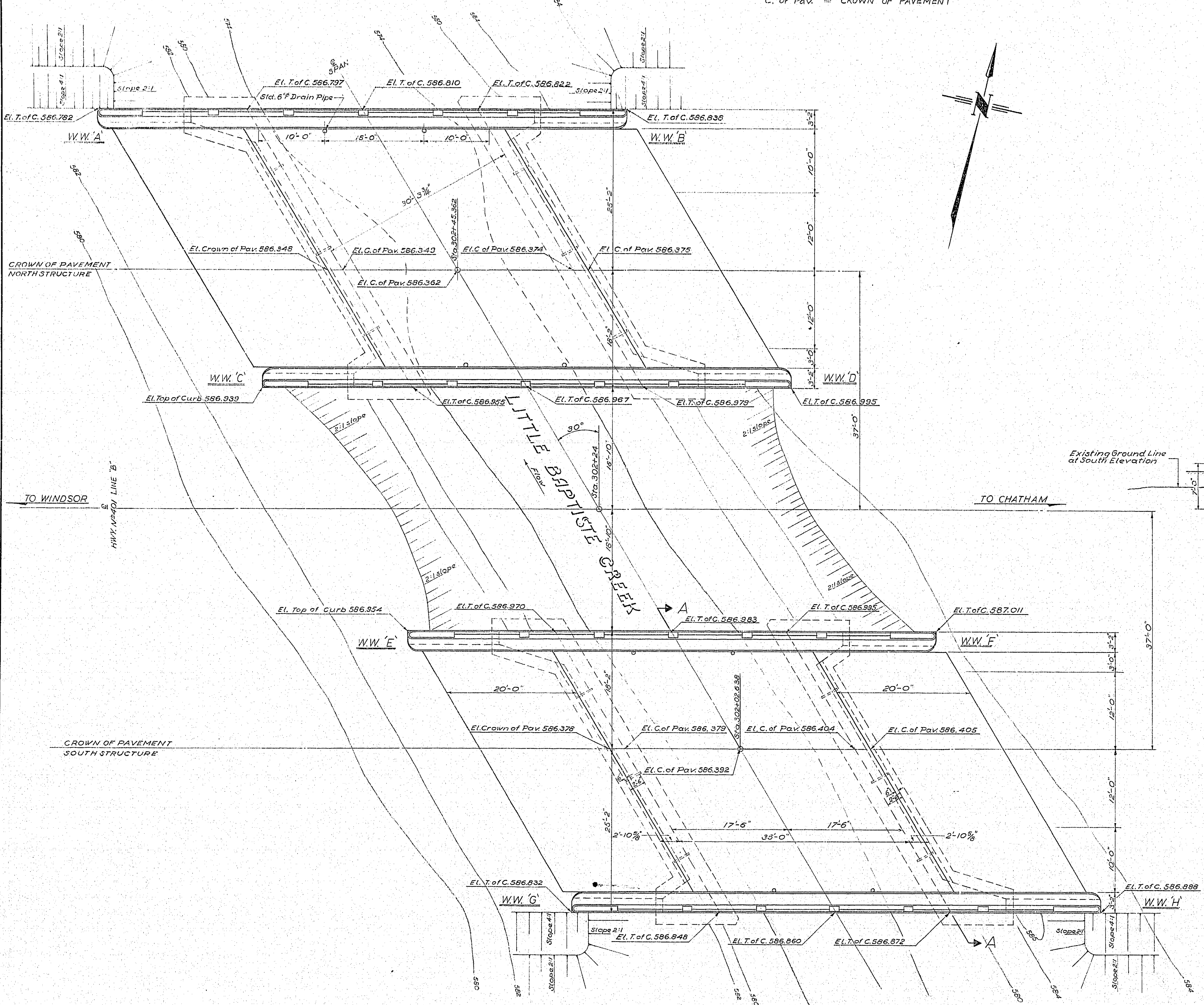
Disturbed samples are visually classified in the field, sealed in sample jars, and are re-examined, and tested as necessary, in the soils laboratory. Undisturbed samples are returned to the laboratory for later examination and testing, as required.

The test holes are bailed at the end of the day and on completion. Subsequent water level readings are taken for the duration of the field work. Water pressure readings are recorded when Artesian water conditions are encountered. Moisture content samples are recovered at frequent intervals to assist in the soil classification and the interpretation of water table results.

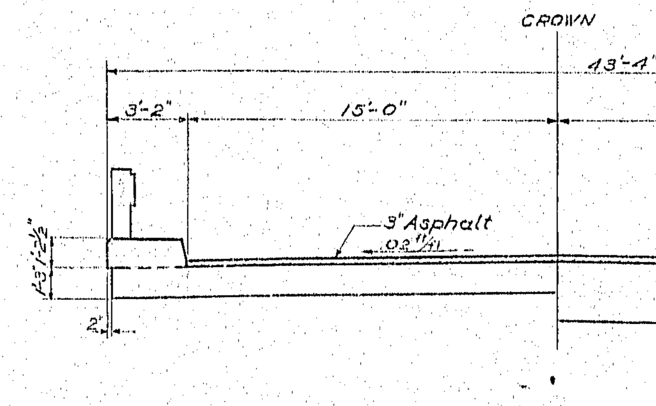


SCRUB
S.I.B.
HOT 0+90.95 GR. RD.
HOT 302+52.51 LINE 'B'
300+00 'B'
LINE 'B'
PROFILE No F-3534-8
D.H.O. BENCH MARK ELEV. 584.80
NAIL ON ELM STUMP 135' LEFT OF
STA. 300+97
BOREHOLE 1.
BOREHOLE 4.
BOREHOLE 5.
BOREHOLE 2.
BOREHOLE 3.
CREEK
GRAVEL SIDE ROAD
ENLARGED DETAIL
SCALE 50 : 1"
GRAVEL SURFACE SIDE ROAD
TO TILBURY
LITTLE BAPTISTE CREEK
TO HWY. No 2
SITE PLAN
SCALE 100 : 1"
BOREHOLE 3
ELEV. 582.65
BOREHOLE 2
ELEV. 584.50
ROAD SURFACE
H.W.L. 577.1
WATER LEVEL
574.3 DEC/58
STIFF
TO
FIRM
SILTY CLAY
SANDY
AND
SILTY CLAY,
MANY GRITS
NO REFUSAL
SILTY FINE TO COARSE
SAND, GRITS & GRAVEL
SECTION THROUGH HOLES 3 & 2
PROFILE SCALES VERT. 10 : 1
HOR. 10 : 1
LEGEND
BOREHOLE
BLOWS / FOOT
FILL & EXCAVATED
MATERIAL
NOTE: SEE BOREHOLE LOGS FOR
COMPLETE SOIL DETAILS
E. M. PETO
e.m. peto & associates ltd.
SOIL SITE INVESTIGATION
AT
HWY. 401 - LITTLE BAPTISTE CR. CROSSING
TILBURY ONTARIO
FOR
DEPT. of HIGHWAYS of ONTARIO
OUR JOB No. 58162
DATE 15 JAN/59
CLIENTS PLAN No. E-3413-1 & F-3534-8
C. J.W.

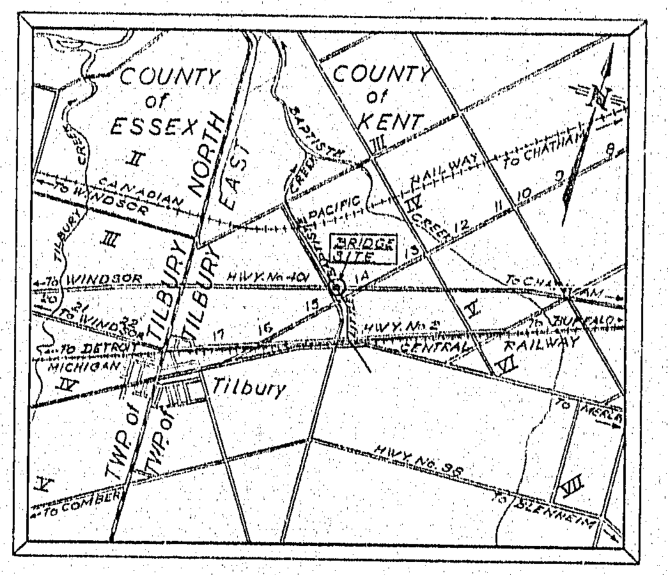
LIST OF ABBREVIATIONS
W.W. = WING WALL
T. of C. = TOP OF CURB
C. of Pav. = CROWN OF PAVEMENT



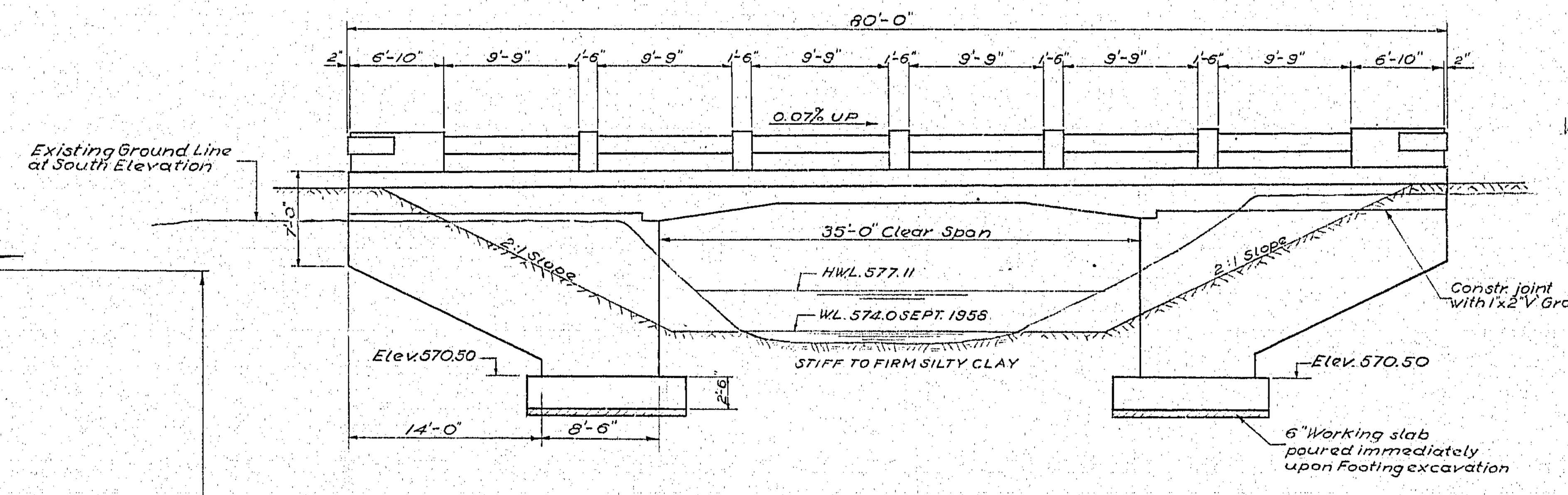
PLAN
Scale: 1/8 in.=1 ft.



SECTION A-A
Scale: 3/16 in.=1 ft.



KEY PLAN
SCALE: 1 IN=2 MI.



SOUTH ELEVATION
Scale: 1/8 in.=1 ft.

NOTE TO DISTRICT ENGINEERS

Concrete work on this structure must not be commenced until monuments to fix control points have been erected and checked by the District Engineer.

NOTE TO CONTRACTOR

Structure to be built in accordance with Form No 9 and the Special Provisions, extra copies of which may be obtained from the District Engineer.

All Construction Joints must be approved by the Bridge Engineer.

CONCRETE MIX

Minimum Concrete Strength at 28 Days
Working Slab & Footins — 2500 p.s.i. - Max Size Aggregate 1½ in
Structure — 3000 p.s.i. - " " " ¾ in.
Above curbs — 3000 p.s.i. - " " " ¾ in.
Approach Slab — 3000 p.s.i. - " " " ¾ in.

An approved admixture supplied by the Department will be added to all concrete as specified by the Engineer.

BORING DATA

The complete soil investigation report BA 861 . . may be examined at the Bridge Office, 280 Davenport Rd. Toronto. The Department does not guarantee the accuracy of this report or the abridged version shown on these plans.

REINFORCING STEEL

Clear Cover in Footings 3 in.
" " " Structure 2 in.
" " above Curbs 1 in.
" " " Approach Slab 2 in.
} or as specified

CONSTRUCTION NOTES

All exposed edges to be chamfered 1 in. unless otherwise noted.

SKEW=30°
sin. 30° = 0.50000
cos. 30° = 0.86603
tan. 30° = 0.57735
cot. 30° = 1.73205
sec. 30° = 1.15470
cosec. 30° = 2.00000

PRINT RECORD		
No.	FOR	DATE
30	TENDER	3.11.59
10	D4 C4 F2	20.1.60
30	TENDER	8.3.61
10	D4 C4 F2	25.5.61

LIST OF DRAWINGS
D-4382-1 GENERAL LAYOUT
D-4382-2 DETAILS OF FOOTINGS, ABUTMENTS & DECK
D-4382-3 DETAILS OF WINGWALLS, APPROACH SLAB, CURB, HANDRAIL & POSTS
D-4382-4 REINFORCING STEEL SCHEDULE
D-4382-5 REINFORCING STEEL SCHEDULE

W.P. 162-58

DEPARTMENT OF HIGHWAYS: -ONTARIO-
BRIDGE OFFICE: -TORONTO

LITTLE BAPTISTE CREEK BRIDGE

THE KING'S HIGHWAY No. 401 DIST. No. 1
CO. OF KENT
TWP. OF TILBURY EAST LOT 14 & 15 CON. IV

GENERAL LAYOUT

APPROVED
BRIDGE ENGINEER
DESIGN ENGINEER

REVISIONS: DATE	BY	DESCRIPTION	REFERENCE PLANS
			Prof. No F-3534-8
			Plan No F-3534-7
			E-3513-1

DESIGN	P.O.L.	CHECK	B.F.	CONTRACT NUMBERS	CATY ELLEY 59-304	APPROACH SLAB 61-109	GENERAL 59-332
DRAWING	P.Sch.	CHECK	B.F.				
TRACING		CHECK		LOADING H20-S16	DRAWING NUMBER ~~D-4382-1~~		
DATE JULY 1959							

TWP 104-187-1-B
~~13-187~~ ~~D4382 1 to 5~~

APPENDIX B

Site Photographs

Photograph 1: Looking at the Little Baptiste Creek Bridge EBL structure from the southeast corner of the bridge. Vegetated slopes on either side of the banks were observed. Erosion of the slope faces was not observed. (October 20, 2013)

Photograph 2: Looking at the Little Baptiste Creek Bridge EBL structure from south of the bridge. The banks were heavily vegetated. No erosion of the slope faces was observed. (October 20, 2013)

Photograph 3: Looking at the west abutment wall from the southeast corner of the structure. Scouring of the exposed earth was observed. Weep holes in the wall were open and wet. (October 20, 2013)

Photograph 4: Looking at the east abutment of the structure from the northwest corner of the structure. Scouring of the exposed earth in from of the abutment wall was observed. Weep holes in the wall were open. (October 20, 2013)