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Underpass

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:



Golder Associates

CONSULTING GEOTECHNICAL AND MINING ENGINEERS

REPORT TO
MINISTRY OF TRANSPORTATION
AND COMMUNICATIONS

CONT. 92-40

FOUNDATION INVESTIGATION
GRADE SEPARATION AT HIGHWAY 407
AND ISLINGTON AVENUE

W.P. 88-78-18

DISTRICT 6

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ABSTRACT

A foundation investigation was carried out by Golder Associates for the Ministry of Transportation and Communications at the site of the proposed grade separation at Highway 407 and Islington Avenue in the Town of Vaughan, Ontario.

The borings indicate that the site is underlain by sand and silt to depths of about 7 m which is underlain by firm to stiff layered clay, stiff clay till and, below depths of about 25 m, by a complex succession of sand, silt and clay deposits. Groundwater in the upper sand and silt is perched above the clay 2 m below ground surface. There is a downward seepage gradient.

The foundations for the bridge should consist of piles driven to depths of about 40 m. Preaugering should be carried out to elevation 145 to minimize pore pressure buildup in the upper silty soils. The cut slopes should be designed at 4 to 1 and must incorporate drains to control groundwater seepage and pressure. These can be installed as part of the construction dewatering operations. Stability analyses indicate that the abutments for the bridge should be moved further from the highway centreline to the top of the 4 to 1 slopes.

ABSTRACT

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1.0 INTRODUCTION

The Ministry of Transportation and Communications has retained Golder Associates to carry out a geotechnical investigation for the proposed grade separation at Highway 407 and Islington Ave. in the Town of Vaughan, Ontario. Authorization for the investigation was received in a letter dated March 22, 1983 from F. G. Allen, Executive Director, Highway Engineering Division. Changes in the proposed alignment of Islington Avenue in 1982 caused a postponement of the investigation until 1983. The investigation was carried out in accordance with the terms of reference set out in Golder Associates' letter dated June 30, 1983.

The purpose of the investigation was to determine the subsurface conditions at the site and, based on this information, to provide recommendations for the geotechnical design of the works.

2.0 SITE AND PROJECT DESCRIPTION

Details of the project were provided on a plan entitled "Islington Avenue Preliminary Plan & Profile" prepared by Marshall Macklin and Monaghan Limited. Subsequently, Plan E-6030-1 prepared by the Ministry of Transportation and Communications was received on July 9, 1983.

The site is located on Islington Avenue about one half kilometer north of Steeles Avenue in the Town of Vaughan, Regional Municipality of York. At this location, the proposed Highway 407 will cross below Islington Avenue at a depth of about 8 metres below existing grade. A new multi-span bridge structure will be constructed along a relocated centreline of Islington Avenue which will correspond roughly to the westerly edge of the existing asphaltic pavement.

3.0 SUBSURFACE CONDITIONS

3.1 Site Geology

The site is located in a level glacio-lacustrine sand plain formed east of the Humber River Valley. The surficial soils consist of sands and silts which, with depth, change to lacustrine clays and clay till. The lacustrine clays in this area are known to contain zones of reworked till and the clays are thought to be derived from the underlying till deposit. Water well records indicate that the site is located in a deep depression in the bedrock surface. Shale bedrock of the Dundas formation has been found at depths of 40 to 60 metres in the area.

3.2 Soil Stratigraphy

3.2.1 Sand

In the upper 2.1 metres of Borehole 2, sand with a trace of silt and some gravel was found. The soil has been weathered to a brown colour and is in a compact condition based on penetration resistance, 'N'* values of 15 and 20 measured in the deposit. The water content of laboratory samples of the soil was about 15 per cent.

3.2.2 Sandy Silt to Silty Clay

Predominantly silty soils were encountered to depths of 7.3 to 8.5 metres in Boreholes 3 and below the upper sand in Borehole 2. The deposit consists of sandy silt to silt with layers of silty clay. The frequency of the silty clay layers increases with depth. Grain size distribution curves of these materials are shown on Figures 1 and 2. The upper weathered zone of the deposit is brown in colour, the unweathered zone below about elevation 156 is grey.

* 'N' Values - Refer to Explanation of Terms

The silty soil was found to be in a compact state at Boreholes 1 and 2 based on 'N' values of 10 to 21. At Borehole 3 lower 'N' values, generally between 3 and 7, were recorded and are indicative of very loose to loose conditions. In the lower portions of the deposit where the silty clay layers were more frequent, in situ vane tests measured undrained shear strengths of 84 kPa or more.

The results of drained simple shear tests on a remoulded sample of the silty soil from Borehole 2 at a depth of 6.4 metres are presented on Figure 15. The tests measured an effective angle of internal friction of 34 degrees. From the unit weights of the sample measured after testing, it is considered that the sample was put into the shear box in a more dense state than its in situ condition. Therefore, the design angle of internal friction has been assumed to be 30 degrees.

The water content of laboratory samples of the soil varied from 21 to 27 per cent. The sandy portions of the silt were found to be nonplastic while zones containing some clay had liquid limits of 21 to 27 per cent and plasticity indices of 3 to 8 per cent, indicative of silts and clays of low plasticity.

3.2.3 Upper Clay

Below the upper silty soils, an irregularly, horizontally layered clay was encountered in Boreholes 1 and 2 to about elevation 143. The layers within the clay contain a higher proportion of silt and occasional thin partings of silt were noted. The clay has zones of light grey silt nodules and traces of gravel giving it a till-like texture in places. A typical grain size distribution curve of the clay is shown on Figure 3.

In situ vane tests in the clay measured undrained shear strengths between 34 and 80 kPa. Laboratory vane tests on shelby tube samples gave strengths as low as 28 kPa at elevation 147. The undrained shear strengths generally increase above and below this elevation. The clay has a sensitivity of about 2 to 5.

The results of a series of consolidated undrained triaxial tests with pore pressure measurements on a sample of the clay from elevation 149 in Borehole 2 are presented on Figure 16. The stress paths followed by each test vary somewhat, probably due to shearing through layers having various silt contents. The minimum effective angle of internal friction of 30 degrees was measured in sample C which was consolidated well in excess of the existing overburden pressures. From this it is concluded that the angle of internal friction for design when failure will take place on a vertical or near vertical plane can be taken as 30 degrees.

A series of three drained direct shear tests were carried out on a sample of the clay from elevation 144.5 in Borehole 2. The specimens were oriented so that shear failure would occur parallel to the layers. The results indicate that the effective angle of internal friction of the clay on horizontal planes is 24 degrees.

The water content of the clay varies from 18 to 44. Atterberg limit tests on the samples measured liquid limits and plasticity indices of 21 to 48 and 9 to 24 respectively indicating the clay to have intermediate plasticity (refer to Figure 11). The unit weight of the clay is generally about 18.7 kN/cu.m.

3.2.4 Silty Clay Till

Below the clay in Boreholes 1 and 2 and below the upper silty soils in Borehole 3, silty clay till was encountered to about elevation 131 to 134. Typical grain size distribution curves of the till are shown on Figure 4.

The till has a firm to hard consistency as in situ vane shear strengths of about 40 kPa or higher were measured within the deposit. The till generally has a low sensitivity of less than 2 although sensitivities as high as 4 were measured in Borehole 3. Penetration 'N' values between 10 and 21 were measured in the till.

The till has low plasticity as the liquid limit is generally about 27 and the plasticity index is between 9 and 12. The water content of the till is generally between 15 and 20 per cent. The unit weight of the soil was measured to be about 21 kN/cu.m.

3.2.5 Lower Silt → Silt to Silty Clay

The silty clay till changes gradually in composition with depth to silt which extends to elevation 128 to 129. The silt has traces of clay and sand and the deposit contains pockets of sand and silty clay. A grain size distribution of the silt is shown on Figure 5.

Penetration values within the deposit were measured to be generally between 10 and 35 indicative of compact to dense conditions. Clayey zones within the soil were found to have a very stiff consistency.

The water content of the soil was measured to be near 20 per cent.

3.2.6 Lower Silts, Sands and Clays

Below elevation 128 to 129 a complex succession of silts, sands and clays were encountered. The strata vary from layered silty clay to sand with some gravel and trace silt. A 2.7 metre thick zone of limestone cobbles or slabs with a silty sand matrix was encountered between elevations 125.3 and 128.0 in Borehole 2.

Within the soil strata, 'N' values of between 10 and 58 were measured above elevation 125 while below this level values were generally in excess of 50 indicating very dense or hard conditions in the lower zone of the deposit. Water contents measured within the strata are typically between 15 and 25 per cent. At elevation 117, the water content of a layered silty clay strata was measured to be 30 per cent. The liquid limit and plasticity index of this lower clay were measured to be 34 and 13 respectively indicating it to be of intermediate plasticity.

3.3 Groundwater Conditions

The groundwater level in the upper sandy and silty soils was at about elevation 157 at the time of the investigation. It appears to be subject to seasonal fluctuations.

Deep piezometers installed in Boreholes 1 and 2 indicate that the water level in the lower sands and silts is at about elevation 141. Therefore, there is a downward seepage gradient at the site.

4.0 DISCUSSION AND RECOMMENDATIONS

4.1 General

The grade separation will require the construction of a bridge within the existing right of way of Islington Avenue. The proposed grade of Highway 407 will be about 8 metres below existing grade and the approaches of Highway 407 to the bridge will be in open cut. Consideration is being given to constructing the bridge before the cuts for the highway are completed.

The proposed bridge consists of two spans with abutments at 37.5 metres from the centreline of Highway 407. Stability analyses indicate, however, that in the long term drained condition, the slopes behind the abutment would be unstable. Accordingly, the abutments for the bridge must be moved further apart and additional spans may be required. Details of the stability analyses are presented in Section 4.3 of this report .

4.2 Foundations

Weak and compressible soils extend to considerable depths at the borehole locations. Therefore, it is recommended that the bridge be founded on piles driven into the dense soils below elevations 120 to 125. Driven steel H piles are considered most suitable. It is anticipated that the minimum cut off elevation will be about elevation 150 i.e. about 1.5 metres below grade at the central pier location. As a guide, an HP 310 x 110 pile driven to an average set of 2.5 mm per blow for a group of 20 blows using a hammer energy approaching but not exceeding 60 kJ will have a factored capacity at Ultimate Limit State of 1350 kN. On reaching the required set, the pile should be subjected to two further groups of 20 blows unless rapid peaking in resistance occurs. The average set

measured for each group should not be greater than the set measured for the preceeding groups. The capacity of a single pile at Serviceability Limit State should be taken as 1,100 kN. The capacity of a group of piles may be lower and should be checked once pile group geometry is known.

The dense soils at the anticipated founding elevation, although variable, are predominantly silty in composition. During driving, these materials will tend to dilate causing a temporary reduction in pore pressure and a corresponding increase in driving resistance. Where these factors are significant, there will be a reduction in the driving resistance and bearing capacity with time. All piles must be subjected to a redriving test, not earlier than seven days since their installation and load tests must not be carried out until a similar period has elapsed. Driving of piles may cause heave of adjacent piles in the same group and this must be checked during and after driving. Because of the possibility of destabilizing pore pressures being induced in the upper loose silts and firm clays during driving, preaugering should be carried out down to elevation 145. During this operation, it will not be necessary to remove the material.

In view of the variable nature of the founding soils, it is recommended that at least three pile load tests are carried out to verify the design capacities and to determine the load/deformation behaviour of the piles.

In view of the soft nature of the soils at pile cap level and the necessity to preauger, all lateral loads should be taken on batter piles. Pile driving operations should be supervised by qualified geotechnical personnel who should maintain pile driving records and monitor the pile load test.

4.3 Slope Design

4.3.1 Stability Analyses

The cut slopes will extend through about 8 metres of silty and clayey soils and some 6 metres below the existing groundwater level. The stability of slopes of various inclinations and the slope behind the abutment have been analyzed for drained and undrained conditions. The method developed by Sarma¹ has been used for the analyses. In the analyses the effective angle of internal friction (ϕ') of the upper silty soils was taken as 30 degrees. The effective angle of internal friction of the clay in which shearing would take place on predominantly horizontal planes was taken as 24 degrees as determined by the direct simple shear tests. For undrained conditions, an undrained shear strength profile based on that shown on Figure 14 was used. A minimum undrained shear strength of 34 kPa was assumed between elevations 147 and 149 m.

In the analysis of the slope behind the abutment, the active wedge behind the abutment was assumed to act on the abutment and therefore not on the failure surface.

4.3.2 Abutment Slope

The drained stability analysis on the slope behind the proposed abutment indicates that deep seated failure through the underlying clay would take place (Figure 18). The abutments for the structure must therefore be moved to the top of the slope so that approaches do not induce failure on the underlying materials. This may also require the construction of additional piers if the bridge spans cannot be increased sufficiently in length.

¹SARMA, S.K. (1973). Stability Analysis of Embankments and Slopes, Geotechnique 23, No. 3.

4.3.3 Cut Slopes

The critical condition for stability of the open cut slopes will be the long term (drained) condition in which pore pressures will have stabilized at their steady state values. Drained stability analyses of the slopes are very sensitive to groundwater levels and pressures in the slope. Without positive drainage of the slope, groundwater would tend to seep from the face of the cut and with a 4 to 1 slope, factors of safety of less than 1 are obtained in the drained stability analyses (Figure 19).

It is anticipated that considerable difficulty could be experienced by the Contractor in excavating the silty soils and maintaining side slopes below the water table. The water table in these materials is likely to be seasonably variable and advantage would be gained from ensuring that the work is carried out in mid-summer.

In assessing the problem of slope stability and the related necessary drainage of the slope, consideration has been given to the installation of drains which would facilitate construction and also provide the necessary permanent drainage.

It is recommended that after excavating to the elevation of the water table, a longitudinal slope drain is placed at the toe of the cut. Leaving a 3 metre wide berm at the location of the drain, the excavation can then proceed down to about half way between the upper berm and the proposed road grade. At that location, another longitudinal slope drain and berm can be incorporated in the slope. Another longitudinal drain should be inserted at the toe of the slope. Some seepage from the slope may still occur between the drains, particularly at the interface between the clay and overlying materials, and provision should be made in the contract for placing gravel blankets or installing

counterfort drains between longitudinal drains in critical areas. The decision on the location of these blankets or drains should be made after inspecting the slopes during spring run-off.

The permanent drains should be 1.5 metres deep and consist of perforated pipe surrounded by a double filter layer of granular material. The layer nearest the drain should consist of a minimum thickness of 150 mm of coarse concrete aggregate having a nominal size of 19 mm. The second granular filter layer surrounding the coarse concrete aggregate should also have a minimum thickness of 150 mm. This material should consist of fine concrete aggregate and will act as a filter to prevent loss of the silty material into the drains. The fine concrete aggregate should be used to fill the drain trenches to within 150 mm of the slope surface. The upper 150 mm of backfill should consist of the topsoil. The slope should be seeded. It is anticipated that the drawdown in the groundwater level at a distance of 50 m from the toe of the slopes will not exceed 1 m.

It should be noted that additional temporary drains will be required in the area between opposite highway side slopes during construction. These can take the form of sumps at spacings and locations convenient to the contractor's operations.

Various slope geometries incorporating the above drainage have been analyzed by computer. For a 4 to 1 slope and assuming the drainage will maintain the phreatic surface at an average depth of about 1 m below the surface of the slope, a minimum factor of safety of 1.3 has been obtained (Figure 20). It is recommended that slopes are excavated to this gradient and that the bridge design is revised accordingly.

An undrained analysis has been carried out to check the construction condition. Assuming a 1.5 m deep excavation at the toe of the slope for the construction of a bridge

pier pile cap, a factor of safety of 1.4 has been obtained (Figure 21).

At the location of the bridge site the slope toe drains are likely to penetrate to the clay. Upward seepage over the area of the highway pavement is likely to be small and a normal granular subbase will provide adequate drainage. At other locations where silt or sand may be present, the road subbase will have to be designed as a drainage blanket.

Excavations for bridge pier pile caps should be made after the slope drainage is in place. Even then difficulty is likely to be experienced in excavating in the silty material. Where space permits in level ground, shallow excavations (>2 m deep) should be stable with side slopes of 2 to 1. In the area of the slopes, it is recommended that positive drainage, such as well points, is installed around the perimeter of the pier prior to excavation, or that steel sheet piling is used to support the slope during excavation.

It is recommended that, if possible, the main highway cut is constructed in advance of the bridge. Where this is not possible, at least that portion of the cut down to the river valley should be constructed in advance to allow gravity drainage and slopes and drainage as recommended for the permanent cut slopes be constructed on the other three sides.

4.4 Earth Pressures

In this project, abutments will be of limited height and probably consist of bank seats at the top of the slopes. These should, however, be designed taking due account of lateral earth pressure and in accordance with the following recommendations.

- o Selected 'free draining' granular fill, in accordance with MTC specifications should be used as backfill immediately behind the structures. The granular fill should be placed in the wedge-shaped zone defined by a 60 degree line extending up and back from the bottom of the rear face of the structures' footings.
- o All granular fill should be compacted in 200 mm thick lifts to 95 per cent of the standard Proctor density of the material. However, heavy compaction equipment should not be used behind any structure within a lateral distance equal to the current height of the fill above the base of the structures.
- o Longitudinal drains located immediately below the base of the walls should be installed to provide positive drainage of the granular backfill.
- o If the abutment support will permit lateral yielding at the top of the abutment equal to not less than 1/2 per cent of the retained height then 'active' earth pressure conditions should be used in the design. If however the structures are not permitted to yield by 1/2 per cent of their height then 'at rest' pressure conditions should be used. It is anticipated that on this project abutments will be of very low height and the following equivalent fluid pressures may be used in the design.

At ultimate limit states

'active' condition	8.0 kPa/m
'at rest' condition	10.0 kPa/m

At serviceability limit states

'active' condition	6.5 kPa/m
'at rest' condition	8.5 kPa/m

- o Highway live loads which act on the soil behind the abutment within a distance defined by a plane rising at 45 degrees from the underside of the rear of the structure's footing should be considered in the design as an equivalent load due to a height of 600 mm of additional fill. The unit weight of the fill should be taken as 20 kN/cu m.

Where a sheet pile is used to support the slope during excavation for shallow pile caps, this can probably be designed as a cantilever, without bracing. For this condition, the earth pressure can be calculated from the formula

$$p = p_a + p_w = K_a \gamma' H + \gamma_w H$$

where p_a = active earth pressure

K_a = coefficient of active earth pressure
 = 0.45 where the slope angle is not greater than 20°

γ' = buoyant unit weight of soil = 10 kN/cu m

H = retained height

γ_w = unit weight of water = 9.8 kN/cu m.

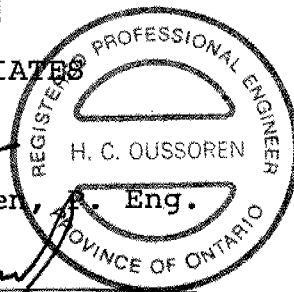
If bracing is used, a trapezoidal distribution should be used with a total earth pressure equal to 1.6 times that given by the above formula.

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APPENDIX 'A'

INVESTIGATION PROCEDURE

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Golder Associates

INVESTIGATION PROCEDURE

The fieldwork for the investigation was begun on June 20, 1983. At that time Borehole 1 was put down to a depth of 40.1 m. Subsequently, Boreholes 2 and 3 were put down between July 7 and 13, 1983. The boreholes were advanced using a Bombardier mounted CME 55 power auger supplied and operated by Master Soil Investigations Limited. Solid and hollow stem augers were generally used to advance the holes to depths of about 20 to 25 m; below this depth, wash boring techniques were used.

At regular intervals of depth, samples of the soil were obtained by a split barrel sampler as part of the Standard Penetration Test. In cohesive soils, in situ vane tests were carried out to determine the soils' undrained shear strength. Thin-walled tube samples of the weaker clays were obtained for detailed laboratory testing. Upon completion, piezometers were sealed into the boreholes to permit monitoring of the groundwater levels.

The fieldwork was carried out under the supervision of a member of Golder Associates staff who logged the boreholes, directed the sampling and testing operations and surveyed the borehole elevations. The elevations are referenced to the geodetic datum.

In the laboratory, the soil samples were identified and tested for their index properties i.e. water content, plasticity, grain size distribution. Samples of the clay and silt were tested in shear to determine their strength characteristics.

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 473 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	>200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	>50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	>3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m ³	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m ³	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m ³	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m ³	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m ³ /s	RATE OF DISCHARGE
γ_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m ³	SEEPAGE FORCE
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL						

RECORD OF BOREHOLE No. 1

W P 88-78-18 LOCATION N 4,847,645 E 298,404 ORIGINATED BY HCO
DIST 6 HWY 407 BOREHOLE TYPE Wash Boring, Hollow Stem Augers, Solid Stem Augers COMPILED BY RWR
DATUM Geodetic DATE June 20 to 23, 1983 CHECKED BY HCO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
159.5	Ground Surface																
0.4	Topsoil - brown, sandy Sandy silt, trace clay, layers of firm to stiff sandy and silty clay in- creasing in frequency with depth. Compact Brown becoming Grey at Elevation 157.1 m		1	SS	15		Water Level Elev. 157.5 m July 7/83 Piezometer dry Aug. 24/83										
			2	SS	18												
			3	SS	18												
			4	SS	15												
			5	SS	11												
			6	SS	14		155										0 22 73 5
			7	SS	15												
152.2																	
7.3	Clay, occasional gravel and sand pockets Stiff to Grey Firm		8	SS	4												
			9	SS	WR		150										
149.1																	
10.4	Interbedded silty clay with some silt nodules and gravel (Till-like) and Clay Firm to Grey Stiff		10	SS	PH												
			11	TW	PM												
			12	SS	1												
			13	SS	WR		145										
143.2																	
16.3			14	TW	PH												
	Silty clay some sand trace gravel (TILL) Stiff Grey		15	SS	13		Water Level Elev. 141.4 m Aug. 4/83										
			16	SS	21		140										
			17	SS	14												
			18	TW	PF												
			19	SS	10		135										
133.9																	
25.6	Silt trace sand and clay to silty clay layered Very Grey Stiff		20	SS	22												
			21	SS	27												
			22	SS	11		130										
129.5																	

30.0 Continued on next sheet

+3, x5: Numbers refer to 20
Sensitivity 15-5 (%) STRAIN AT FAILURE



RECORD OF BOREHOLE No.1 cont.

W P 88-78-18 LOCATION N 4,487,645 ORIGINATED BY HCO
DIST 6 HWY 407 BOREHOLE TYPE Wash Boring COMPILED BY RWR
DATUM DATE June 20 and 21, 1983 CHECKED BY HCO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
129.5	Continued																
30.2	As above																
	Sandy silt, some gravel, trace clay (TILL). Occasional sand seams		23	SS	23												
126.9	Compact Grey		24	SS	16												
32.6	Silty sand, some gravel to gravelly sand some silt, trace clay		25	SS	131												
	Very Dense Grey		26	SS	94/225	225 mm											
			27	SS	127	225 mm											
122.2																	
37.3	Silt, some sand, trace clay and gravel		28	SS	109												
	Very Dense Grey																
119.4			29	SS	162	275 mm											
40.1	End of Borehole																



RECORD OF BOREHOLE No 2

W P 88-78-18 LOCATION N 4,847,575 E 298,428 ORIGINATED BY HCO
DIST 6 HWY 407 BOREHOLE TYPE Hollow Stem Auger, Wash Boring COMPILED BY EFB
DATUM Geodetic DATE July 7-11, 1983 CHECKED BY HCO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)									
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100														
								SHEAR STRENGTH kPa																		
								○ UNCONFINED	+ FIELD VANE																	
								● QUICK TRIAXIAL	x LAB VANE				WATER CONTENT (%)			kN/m ³	GR SA SI CL									
								20	40	60	80	100	10	20	30											
158.5	Ground Surface																									
0.0	Sand, trace silt some gravel		1	SS	15									○												
	Compact Brown		2	SS	20									○												
156.4																										
2.1	Sandy silt, trace clay layered		3	SS	12											○										
	Compact Brown becoming Grey at Elev. 155.3 m		4	SS	21											○										
			5	SS	10											○	0 21 74 5									
			6	SS	15											○										
153.0																										
5.5	Silt, some clay to silty clay, trace fine sand, layered		7	SS	13											○										
	Stiff Grey		8	SS	13											○	0 1 66 33									
150.0																										
8.5			9	TW	PH											○	18.7 0 1 44 55									
	Clay with zones containing silt nodules and traces of gravel, layered		10	TW	PH																					
	Firm to Grey Stiff		11	SS	6											○										
			12	TW	PH																					
143.3																										
15.2			13	SS	11											○										
	Silty clay with some sand and trace gravel (TILL)		14	SS	17											○	0 13 57 30									
			15	SS	26											○										
	Stiff to Hard Grey		16	SS	20											○										
			17	SS	18											○										
			18	SS	20											○										
			19	SS	16											○										
			20	SS	54											○										
131.0																										
27.5	Silty clay, trace to some fine sand		21	SS	10											○										
	Compact to Dense Grey		22	SS	31											○	0 7 80 13									
128.5																										

30.0 Continued on next sheet

+3, x5: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE



RECORD OF BOREHOLE No 2 cont.

W P 88-78-18 LOCATION N 4,847,575 E 298,428 ORIGINATED BY BCO
DIST 6 HWY 407 BOREHOLE TYPE Wash Boring COMPILED BY EFO
DATUM Geodetic DATE July 7-11, 1983 CHECKED BY BCO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
128.5	Continued																
128.0	As Above																
30.5	Limestone cobbles with silty sand		23	SS	33												
	Dense Grey		24	SS	1007	150 mm											
			25	WS	-												
125.3							125										
33.2	Silt, some sand and gravel some clay with occasional sand layers (TILL)		26	SS	48												
	Hard Grey		27	SS	97/250	250 mm											
			28	SS	63												0 46 50 4
			29	SS	60		120										
118.7																	
39.8	Silty clay layered occasional silty sand seams		30	SS	33											19.8	
	Hard Grey		31	SS	117												0 1 59 40
116.4																	
42.1	Sand and silt, some gravel		32	SS	91		115										
	Dense Grey																
114.1			33	SS	1007	90 mm											
44.4	End of Borehole																

OFFICE REPORT ON SOIL EXPLORATION



W P	88-78-18	LOCATION	N 4,847,574 E 298,428	ORIGINATED BY	HCO
DIST	6 HWY 407	BOREHOLE TYPE	Solid Stem Auger, Piezometer Installation	COMPILED BY	EFO
DATUM	Geodetic	DATE	July 12, 1983	CHECKED BY	HCO

[illegible]

+3, x5: Numbers refer to Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION



RECORD OF BOREHOLE No 3

W P 88-78-18 LOCATION N 4,847,617 E 298,429 ORIGINATED BY HCO
DIST 6 HWY 407 BOREHOLE TYPE Hollow Stem Auger, Wash Boring COMPILED BY FFD
DATUM Geodetic DATE July 12-13, 1983 CHECKED BY HCO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL												
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100																					
								SHEAR STRENGTH kPa																					
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE																					
								20 40 60 80 100					WATER CONTENT (%)																
													10 20 30																
159.6	Ground Surface																												
0.0	Fill - sand and gravel																												
158.8																													
0.8	Silt with sand and clay. Layers of silt, some clay increasing in frequency with depth		1	SS	5																								
			2	SS	7																								
			3	SS	15																								
	Very Loose Brown to Compact becoming Grey at Elev.		4	SS	5																								
			5	SS	4																								
154.7	155.9 m		6	SS	3																								
4.9	Silt, trace clay and fine sand to silty clay with silt nodules trace gravel		7	SS	6																								
	Stiff Grey		8	TW	PH																								
151.7			9	TW	PH																								
7.9			10	SS	6																								
	Silty clay trace to some sand and gravel (TILL)		11	SS	6																								
	Firm to Stiff Grey		12	SS	9																								
			13	SS	16																								
			14	SS	12																								
			15	SS	10																								
			16	SS	12																								
			17	SS	11																								
137.1			18	SS	34																								
22.5	Silt, trace clay and sand to silty clay trace sand and gravel. Occasional layers and pockets of sand. Sand content increasing with depth.		19	SS	58																								
			20	SS	36																								
			21	SS	30																								
			22	SS	19																								
129.7	As below																												
30.0	Continued on next sheet																												

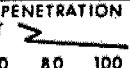
+3, x5: Numbers refer to
Sensitivity

20
15-5 (%) STRAIN AT FAILURE
10

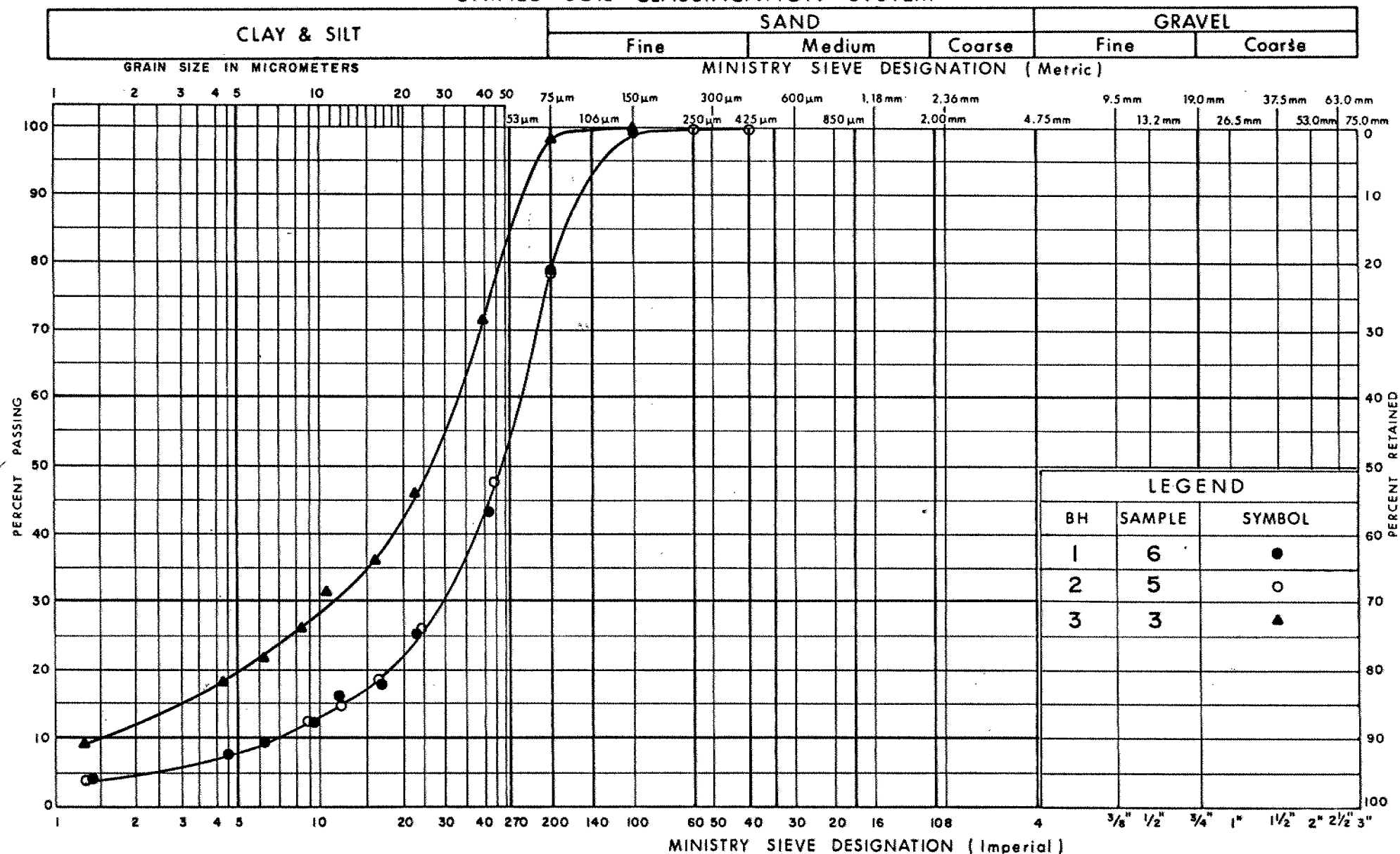


RECORD OF BOREHOLE No 3 cont.

W P 88-78-18 LOCATION N 4,847,617 E 298,429 ORIGINATED BY HCO
DIST 6 HWY 407 BOREHOLE TYPE Solid Stem Auger, Wash Boring COMPILED BY EFO
DATUM Geodetic DATE July 12-13, 1983 CHECKED BY HCO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH					WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE									
								● QUICK TRIAXIAL × LAB VANE									
						20	40	60	80	100	10	20	30				
129.6	Continued																
30.0	Silt, some fine sand, trace clay		23	SS	19								○				
127.9	Compact Grey																
31.7	Sand some gravel and silt with pockets of silty sand		24	SS	48							○	○				
	Compact Grey																
125.2	to Dense		25	SS	20							○					
34.4	Interlayered Silty Clay and Sandy Silt with trace to some gravel and clay (TILL)		26	SS	69								○				
	Dense to Grey Very Dense		27	SS	67								○	○		2 26 65 7 2 8 57 33	
			28	SS	49								○			22.2	
119.5			29	SS	76												
40.1	End of Borehole																

UNIFIED SOIL CLASSIFICATION SYSTEM



Ontario

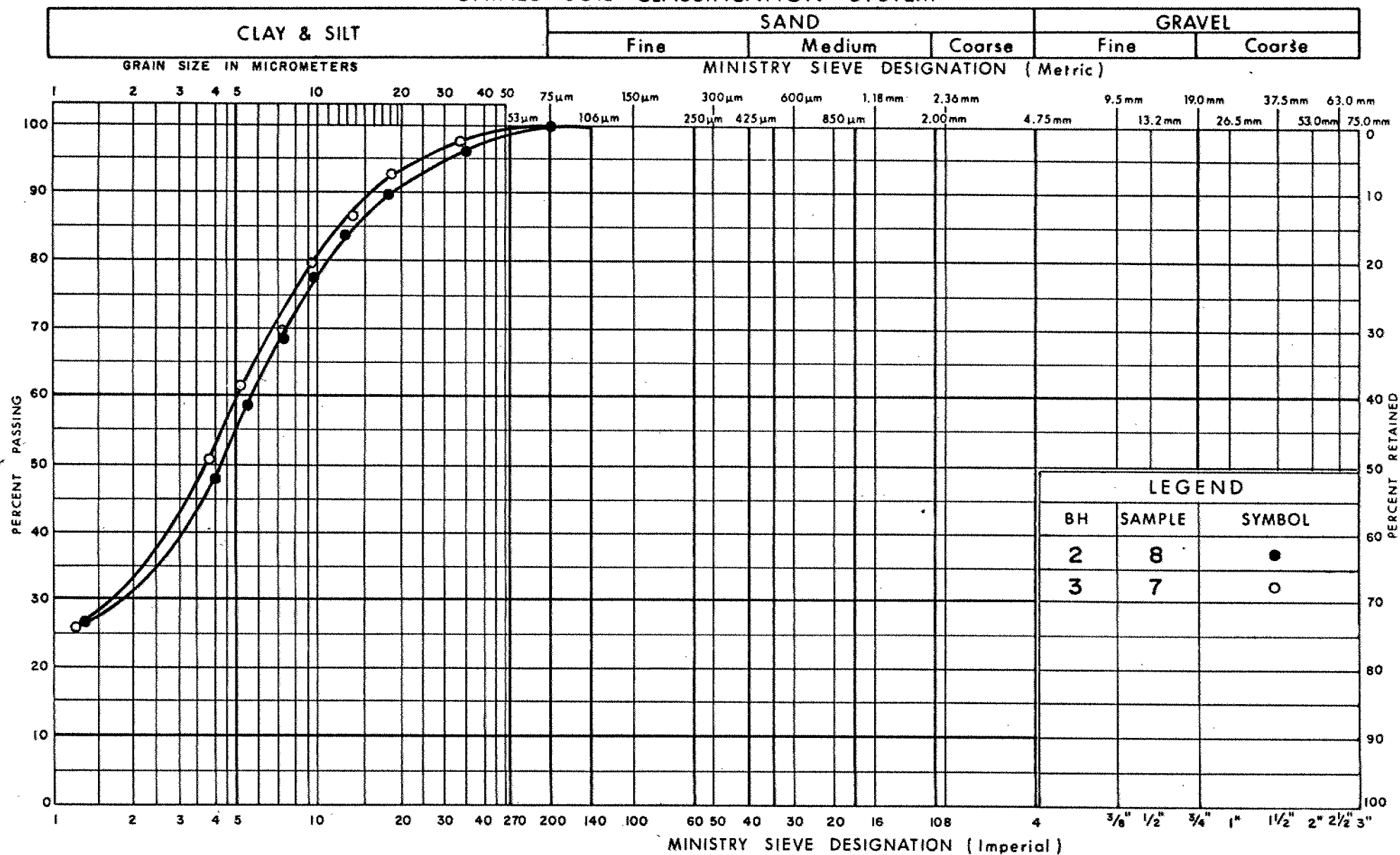
Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION
UPPER SANDY SILT TO SILT

FIG No I

W P 88-78-18

UNIFIED SOIL CLASSIFICATION SYSTEM



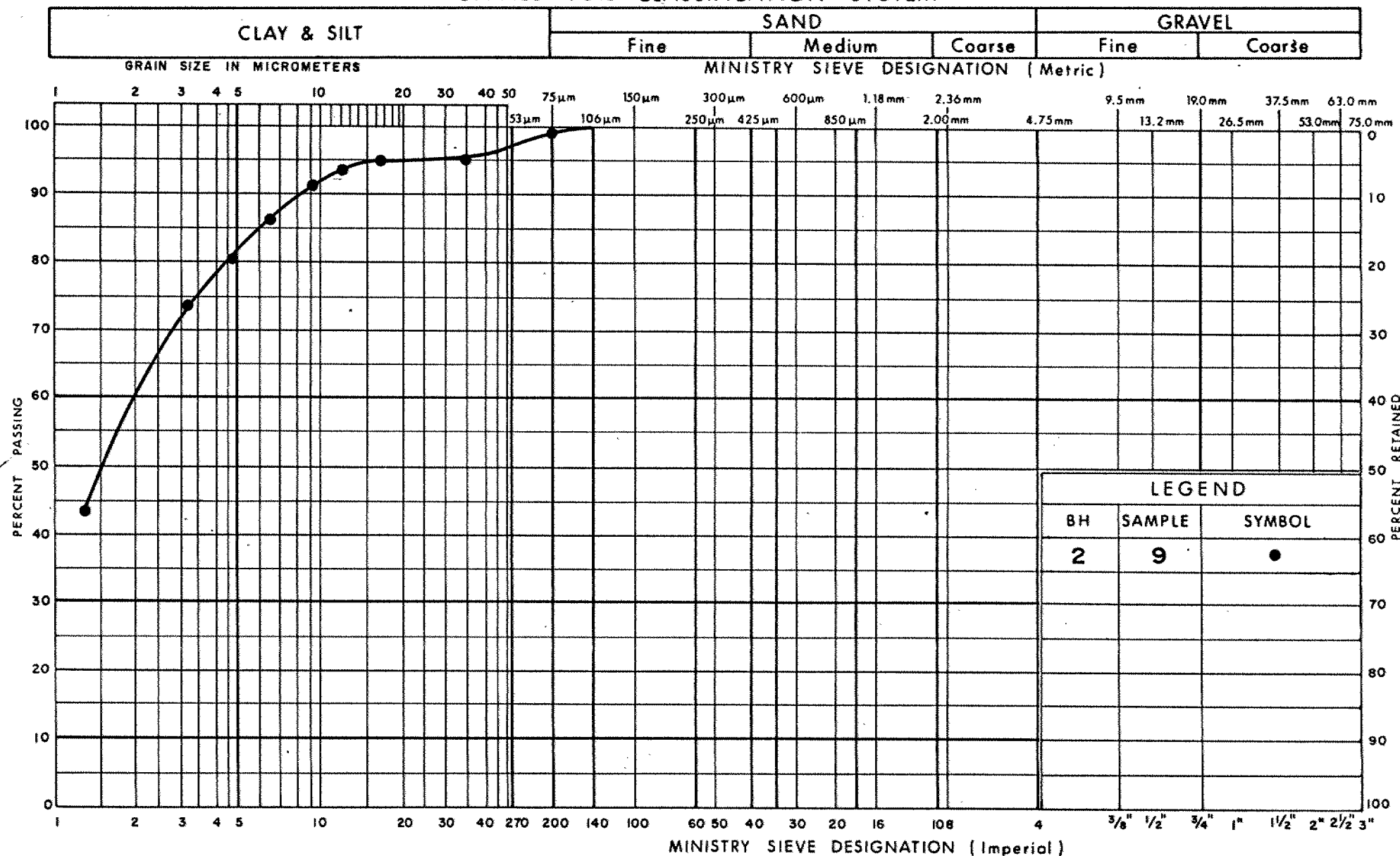
Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION
UPPER SILTY CLAY

FIG No 2

W P 88-78-18

UNIFIED SOIL CLASSIFICATION SYSTEM



Ontario

 Ministry of
Transportation and
Communications

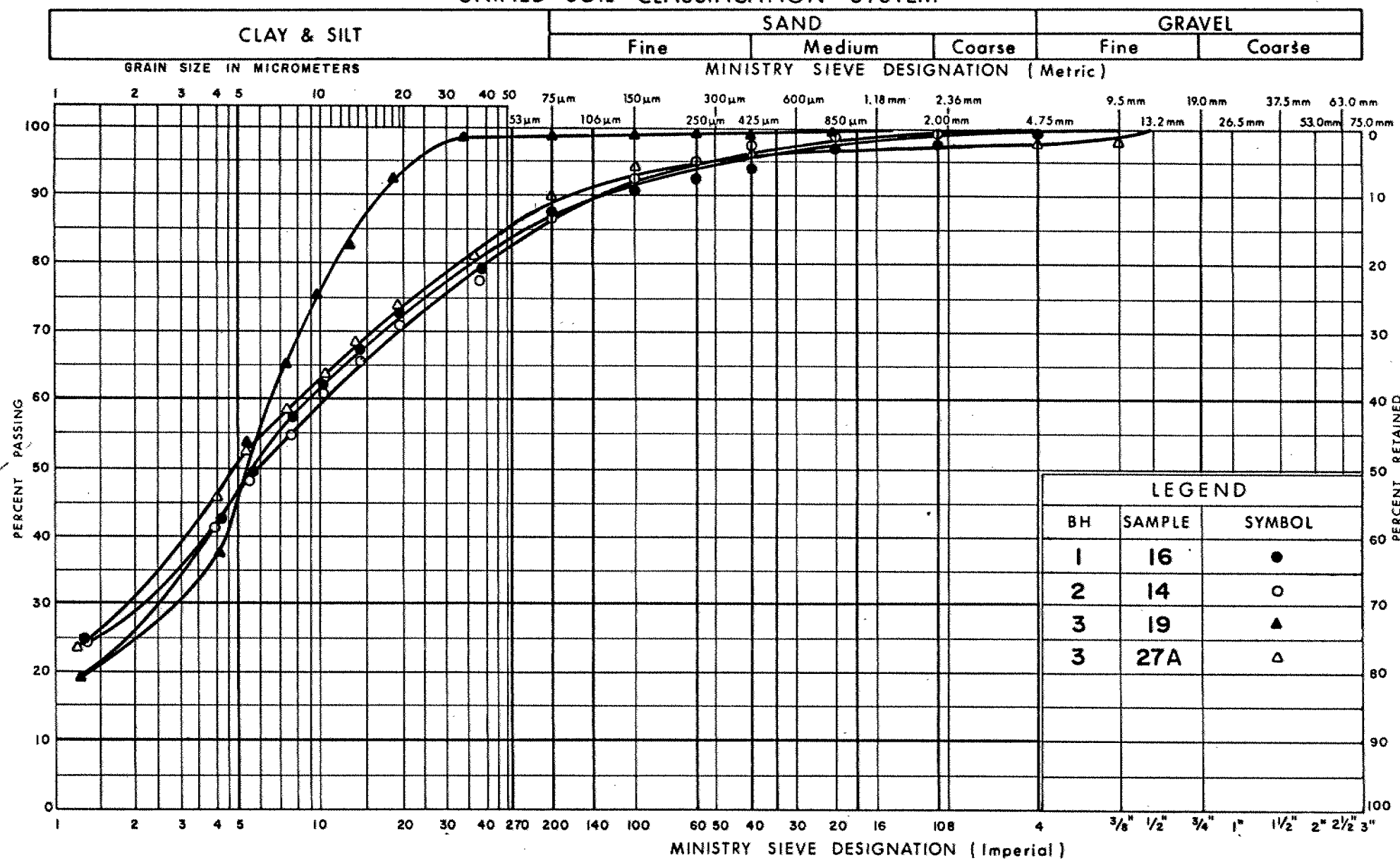
GRAIN SIZE DISTRIBUTION

UPPER CLAY

FIG No 3

W P 88-78-18

UNIFIED SOIL CLASSIFICATION SYSTEM



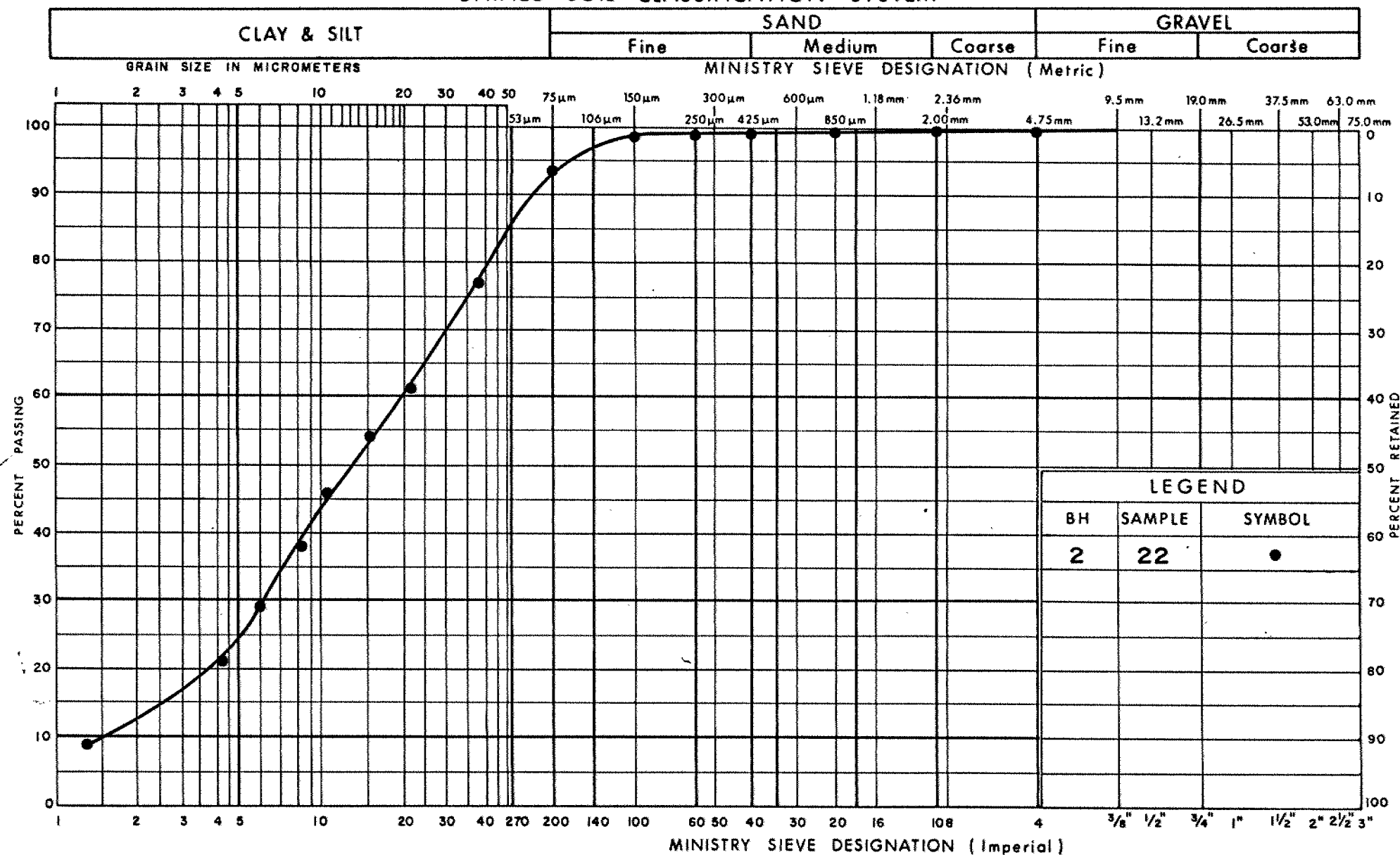
Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION
SILTY CLAY TILL

FIG No 4

W P 88-78-18

UNIFIED SOIL CLASSIFICATION SYSTEM



Ontario

 Ministry of
Transportation and
Communications

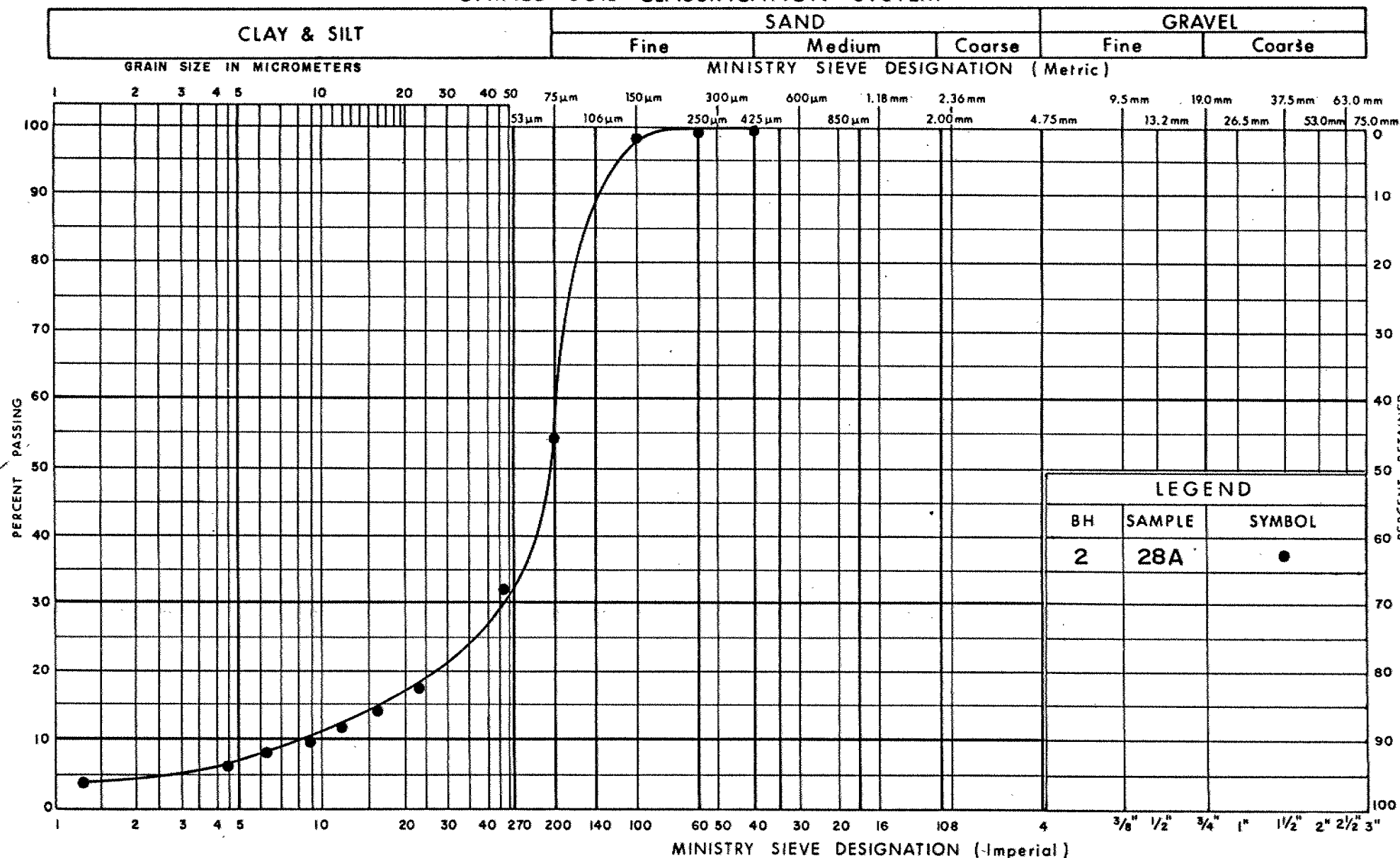
GRAIN SIZE DISTRIBUTION

Lower SILT to Silty Clay

FIG No 5

W P 88-78-18

UNIFIED SOIL CLASSIFICATION SYSTEM



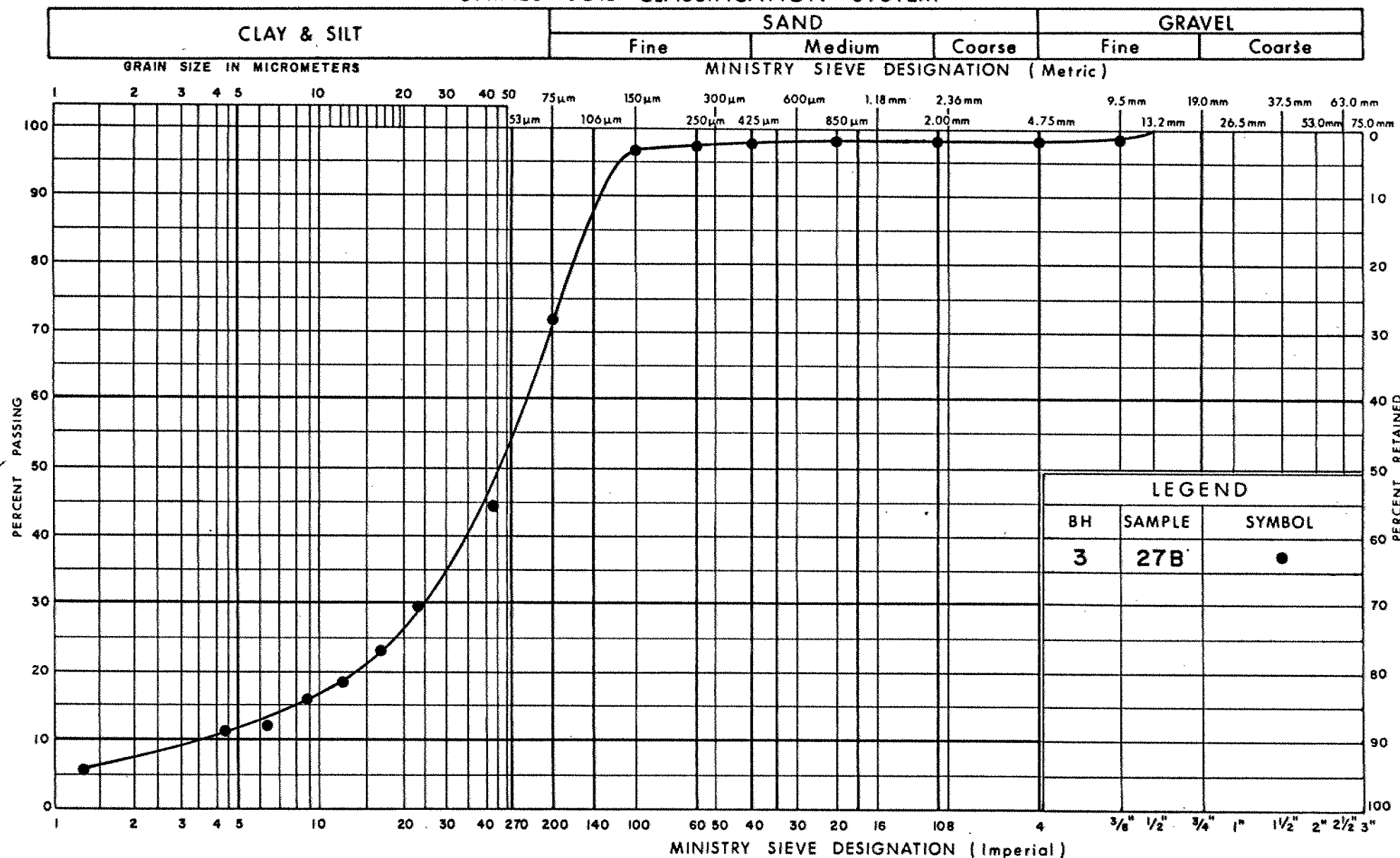
Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION
LOWER SANDY SILT

FIG No 6

W P 88-78-18

UNIFIED SOIL CLASSIFICATION SYSTEM



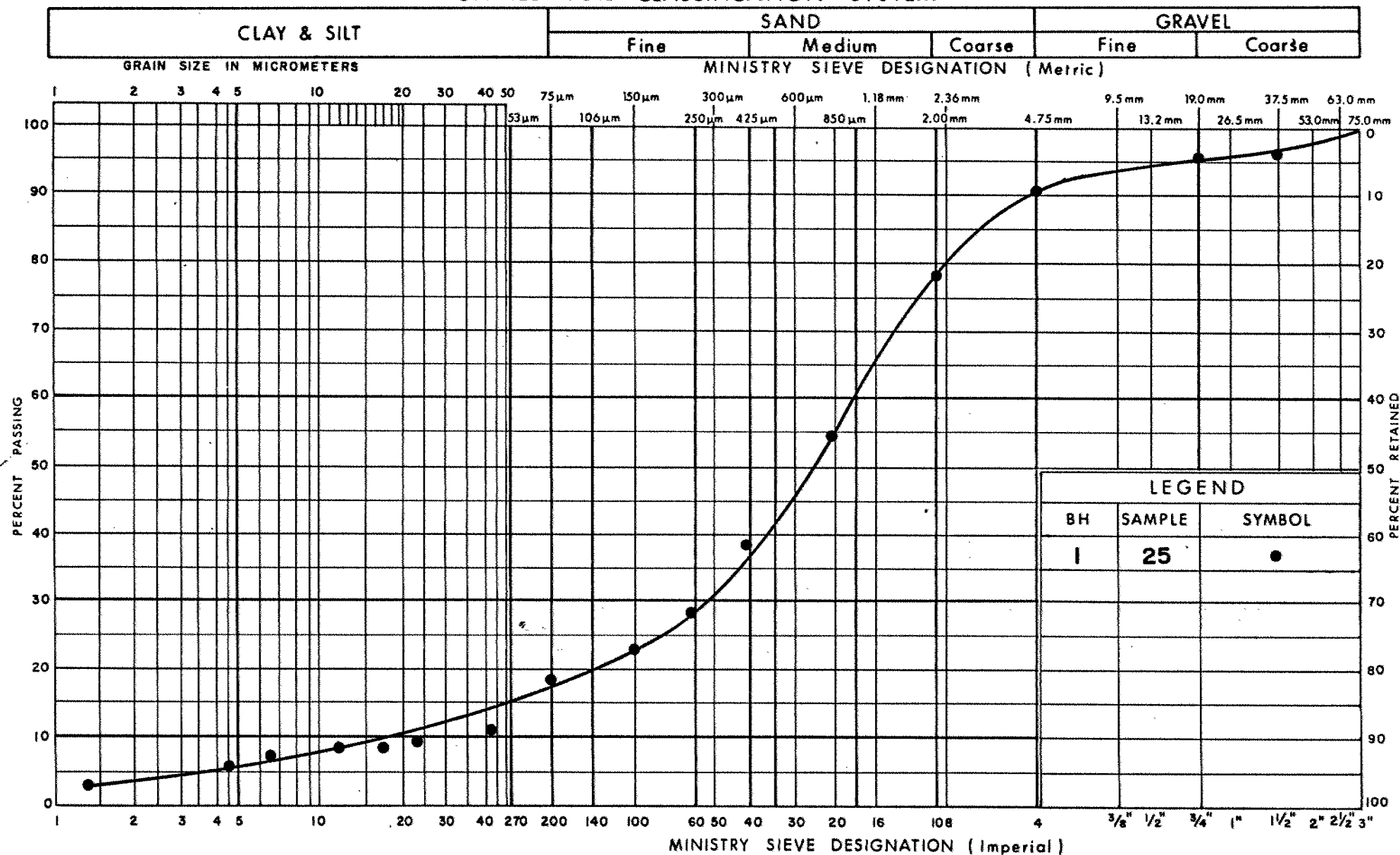
Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION SILT LAYER WITHIN TILL

FIG No 7

W P 88-78-18

UNIFIED SOIL CLASSIFICATION SYSTEM



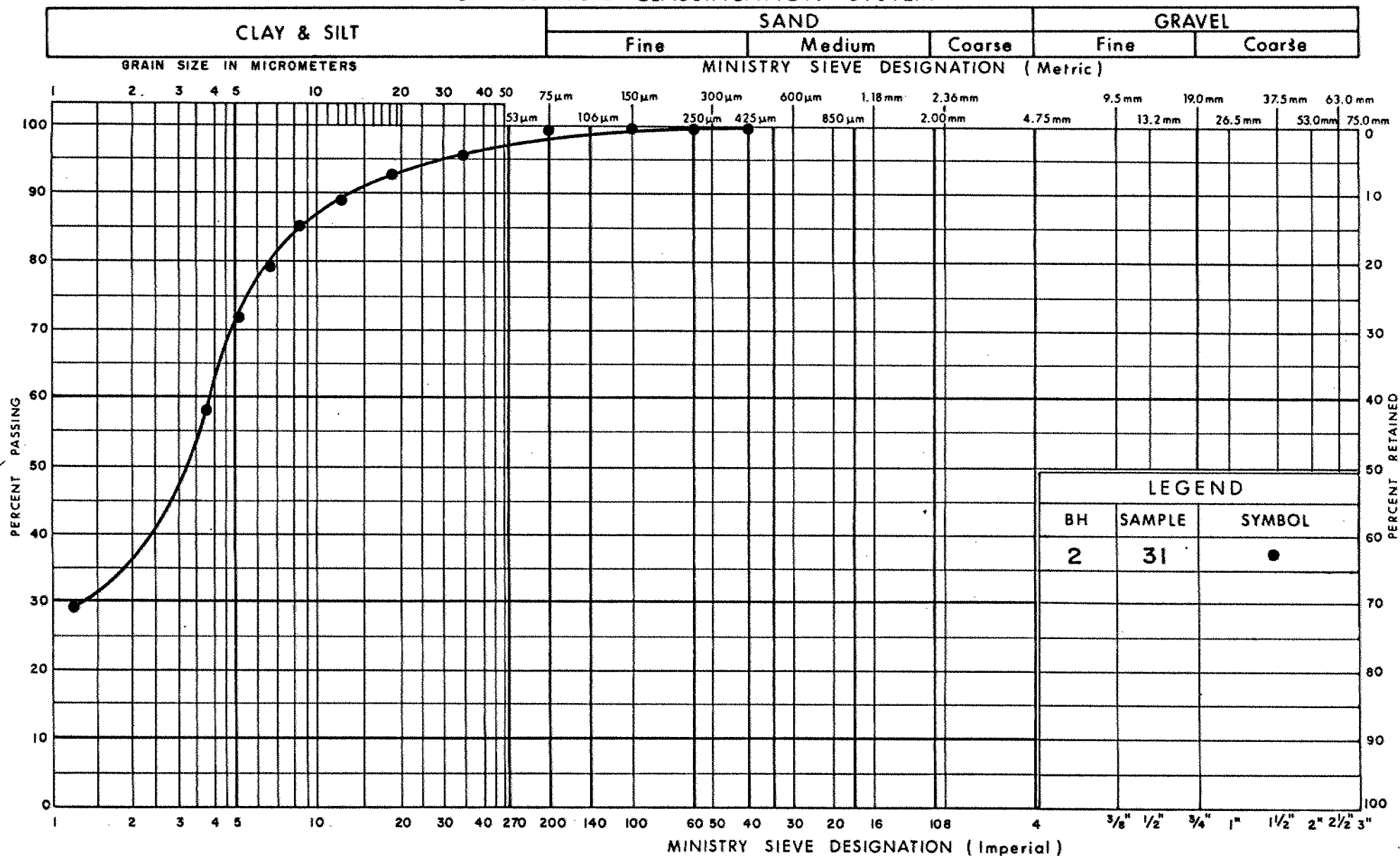
Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION LOWER SAND

FIG No 8

W P 88-78-18

UNIFIED SOIL CLASSIFICATION SYSTEM



Ontario

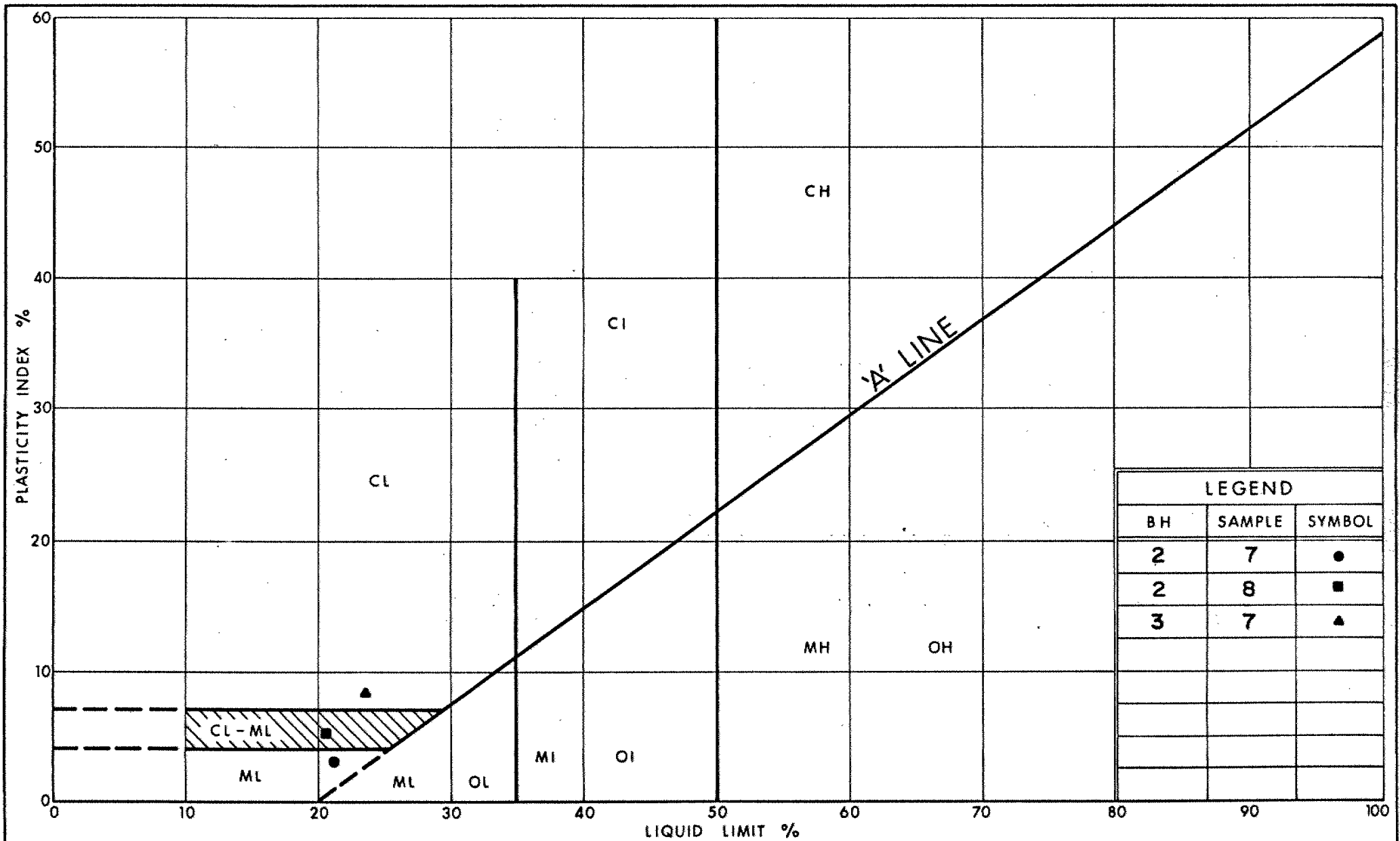
 Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION

LOWER LAYERED CLAY

FIG No 9

W P 88-78-18

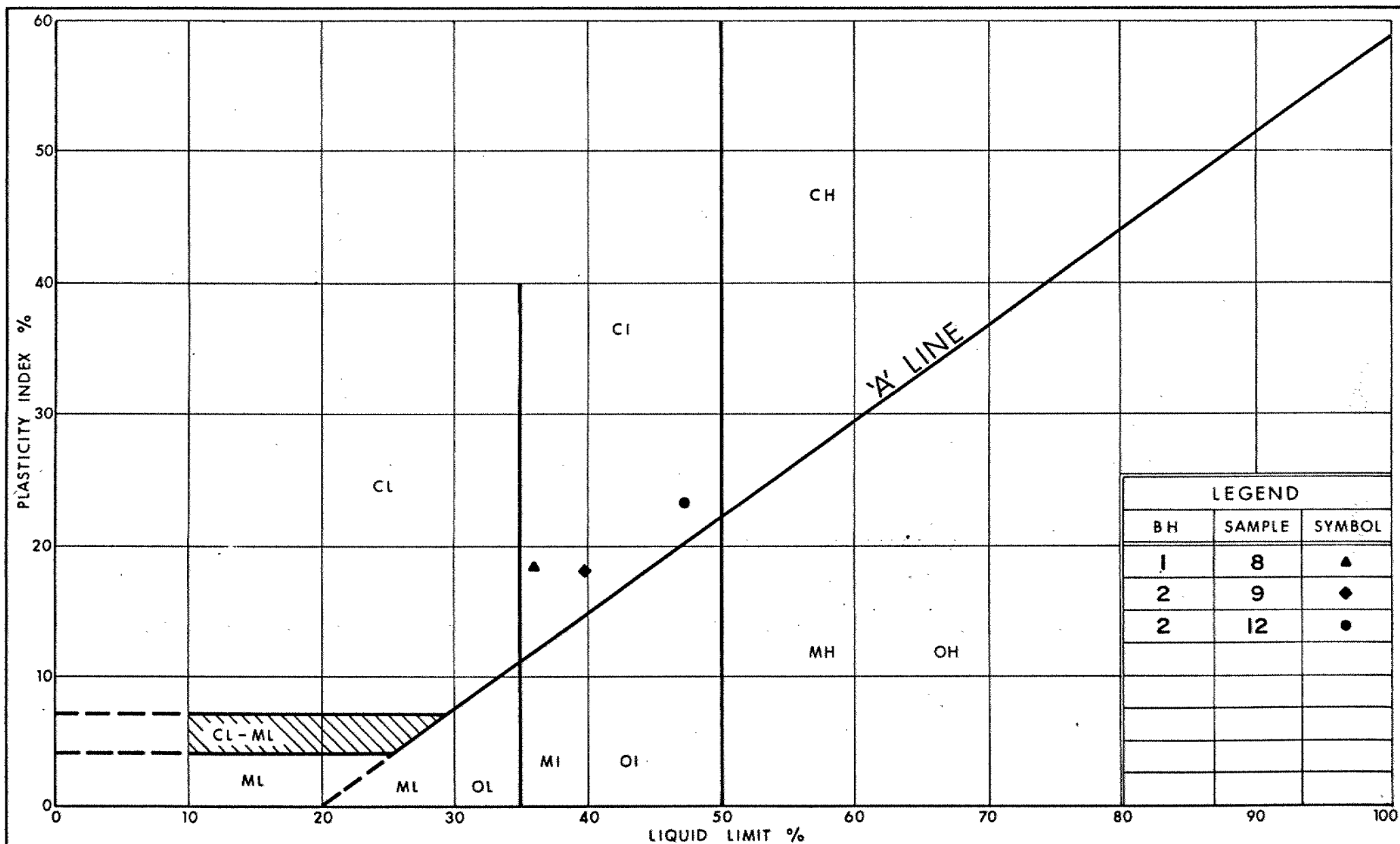


Ministry of
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Communications

PLASTICITY CHART UPPER SILTY CLAY

FIG No 10

W P 88-78-18

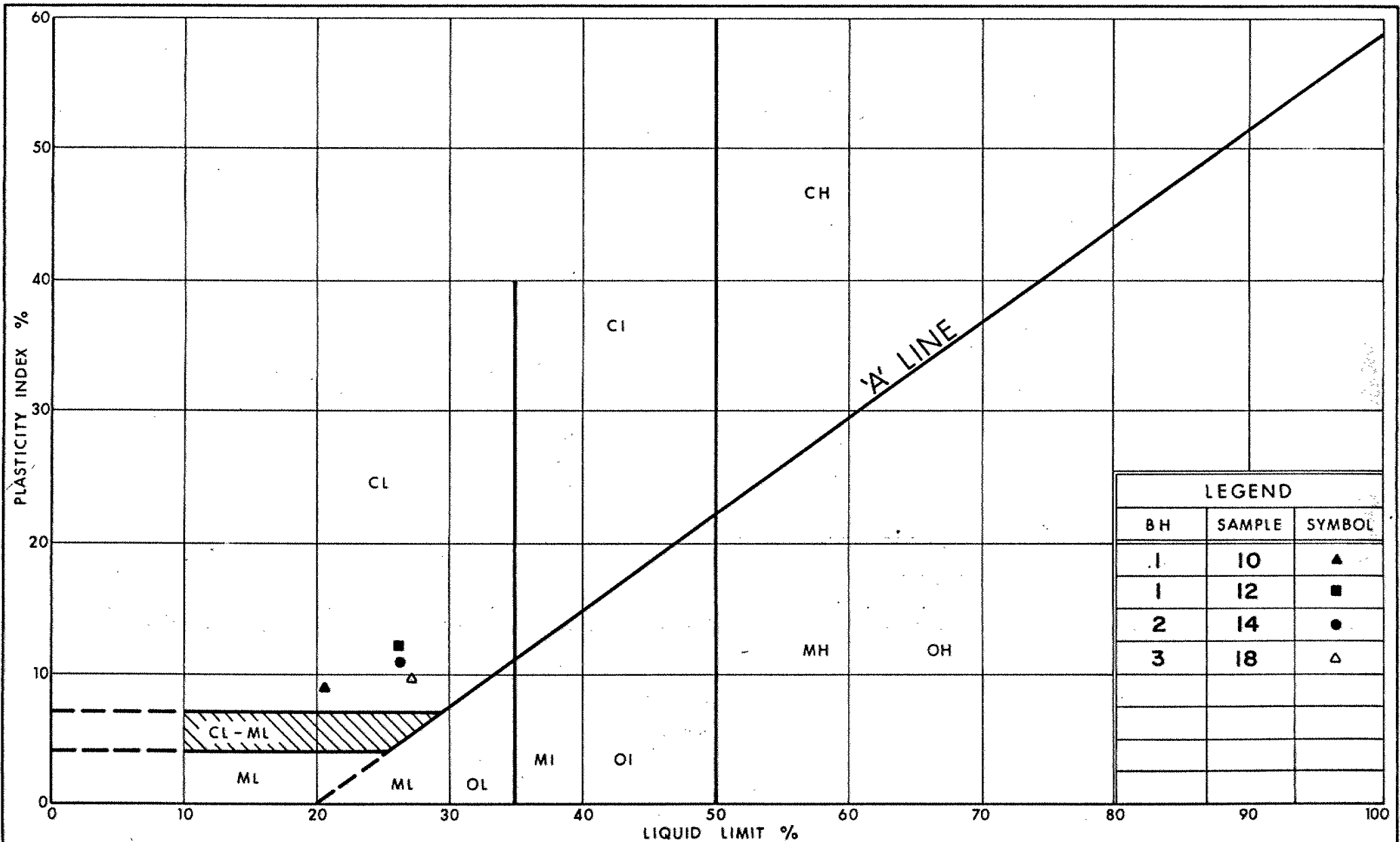


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Communications

PLASTICITY CHART CLAY

FIG No II

W P 88-78-18

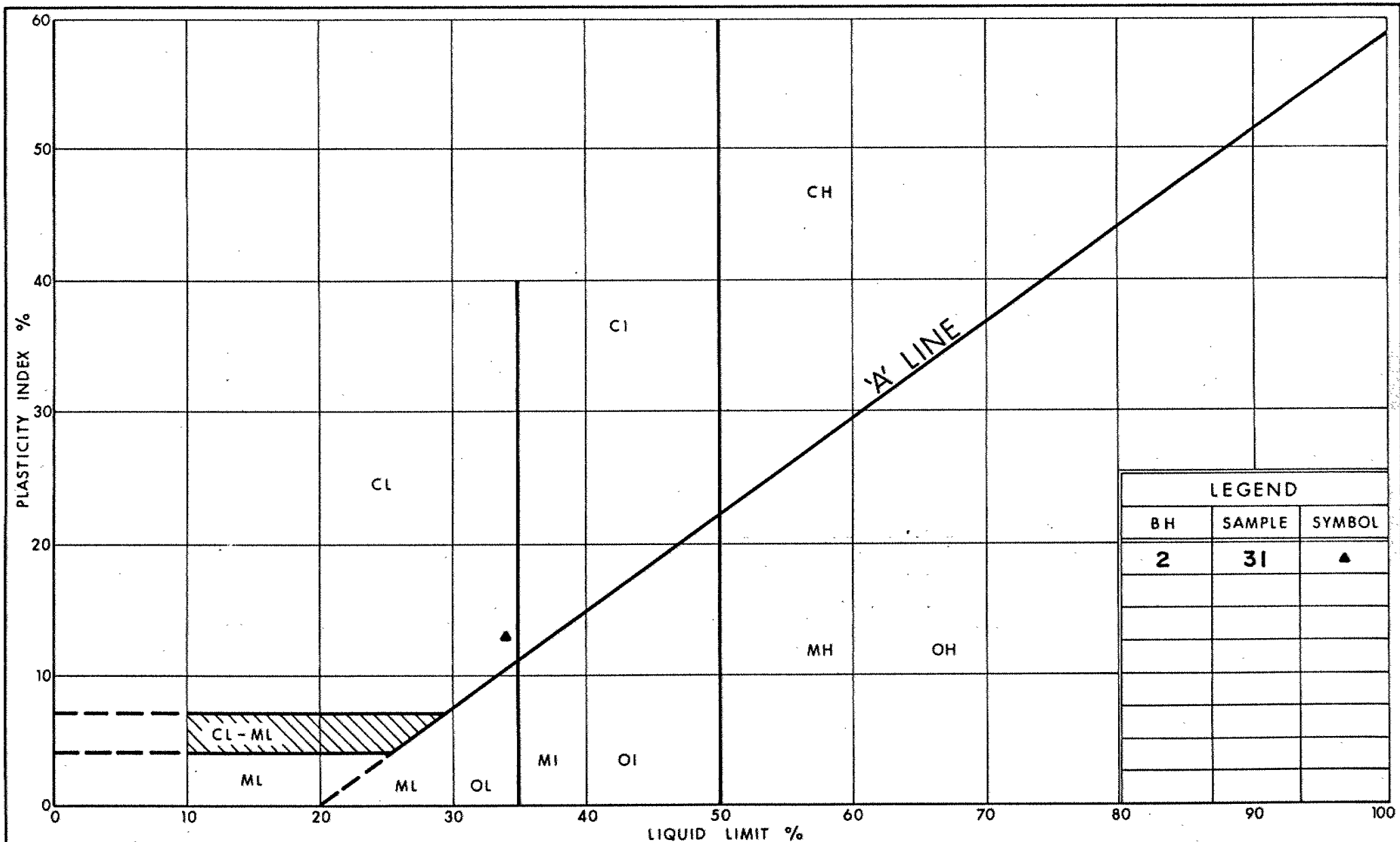


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PLASTICITY CHART SILTY CLAY TILL

FIG No 12

W P 88-78-18



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PLASTICITY CHART LOWER LAYERED CLAY

FIG No 13
W P 88-78-18

METRIC

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

CONT No
WP No 88-78-18



SHEET

BORE HOLE LOCATIONS & SOIL STRATA

GOLDER ASSOCIATES



LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊕ Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- W L at time of investigation June 1983
- ⊕ Bentonite Seal
- ⊕ Piezometer

No	ELEVATION	CO-ORDINATES NORTH	EAST
1	159.5	4,847,645	298,404
2	158.5	4,847,575	298,428
2A	158.5	4,847,574	298,429
3	159.6	4,847,617	298,429

NOTE

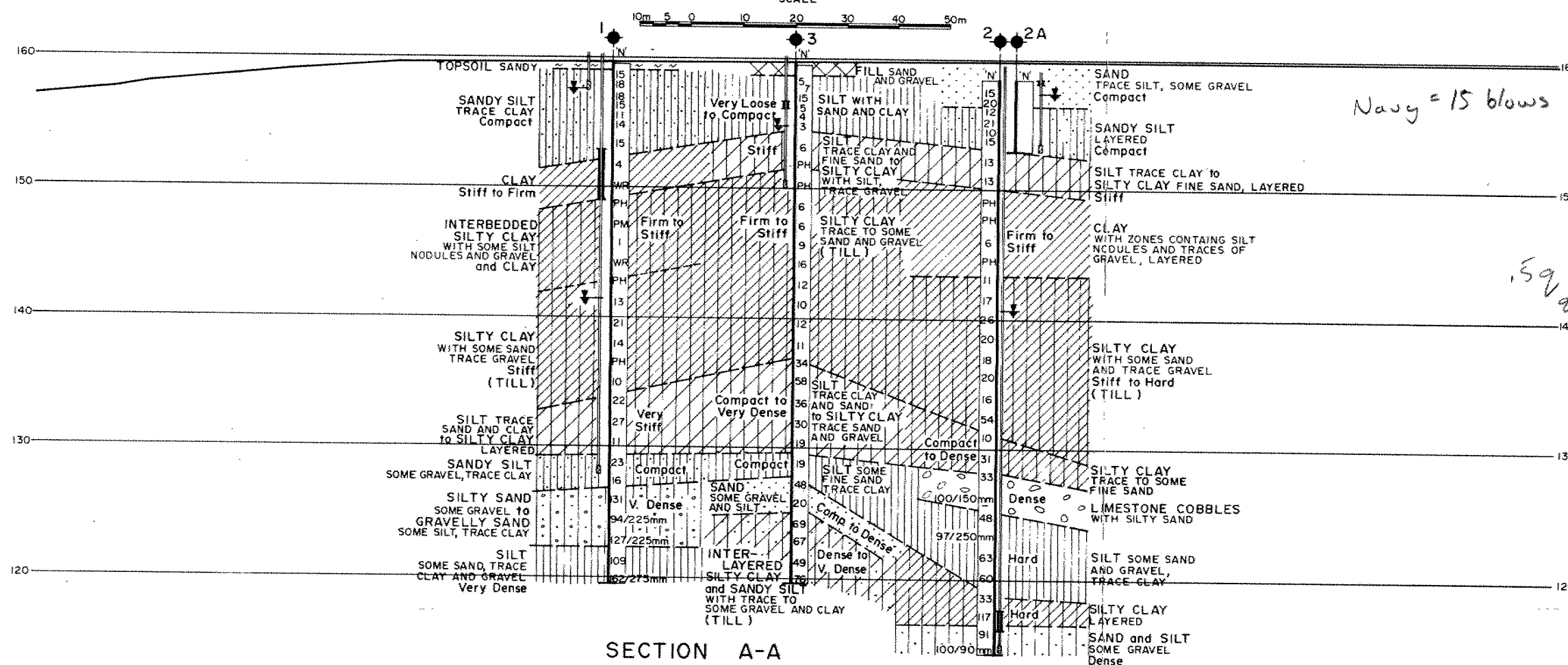
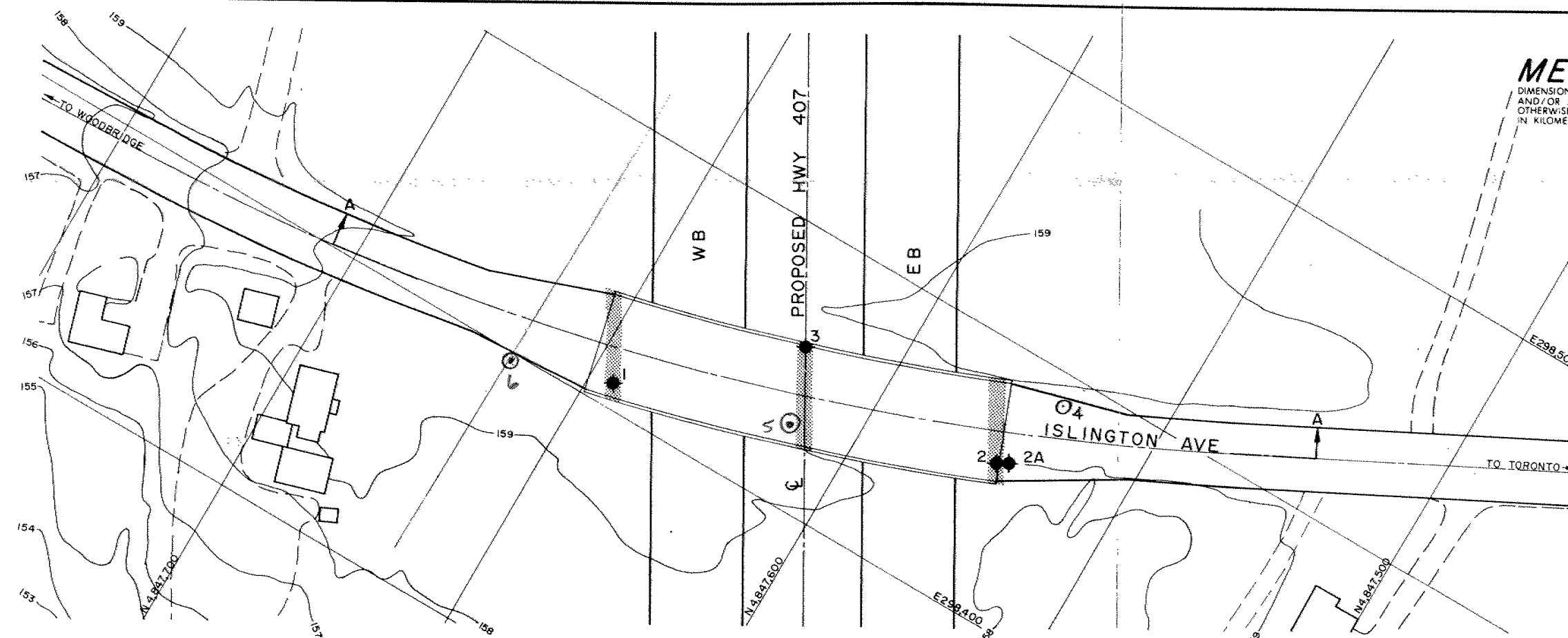
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

REV.	DATE	BY	DESCRIPTION

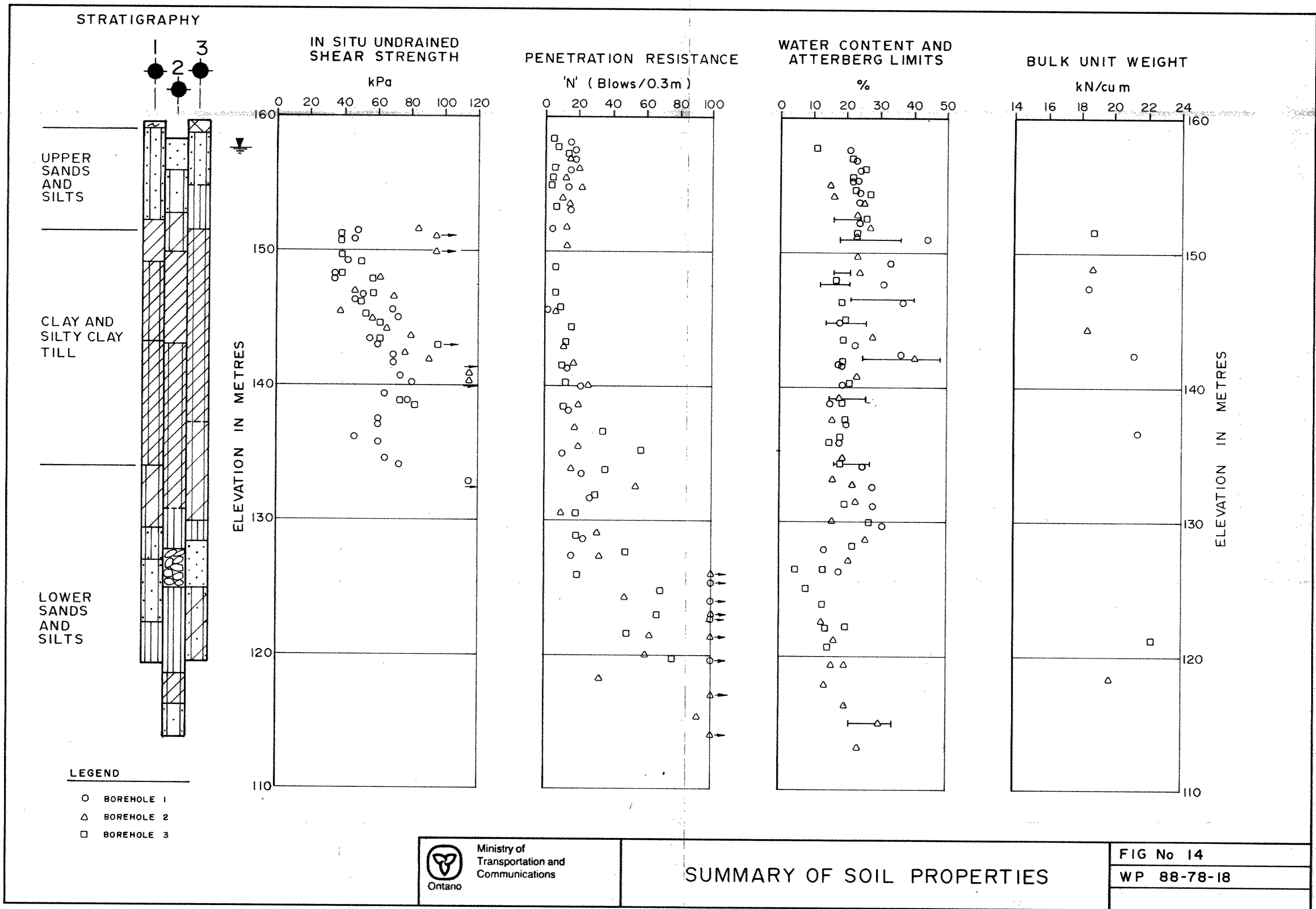
Geocres No

HWY No 407 AT ISLINGTON AVENUE	DIST 6
SUBMD	CHECKED
DRAWN EFO	CHECKED
DATE AUG 16, 1983	SITE
APPROVED	DWG 887818-A

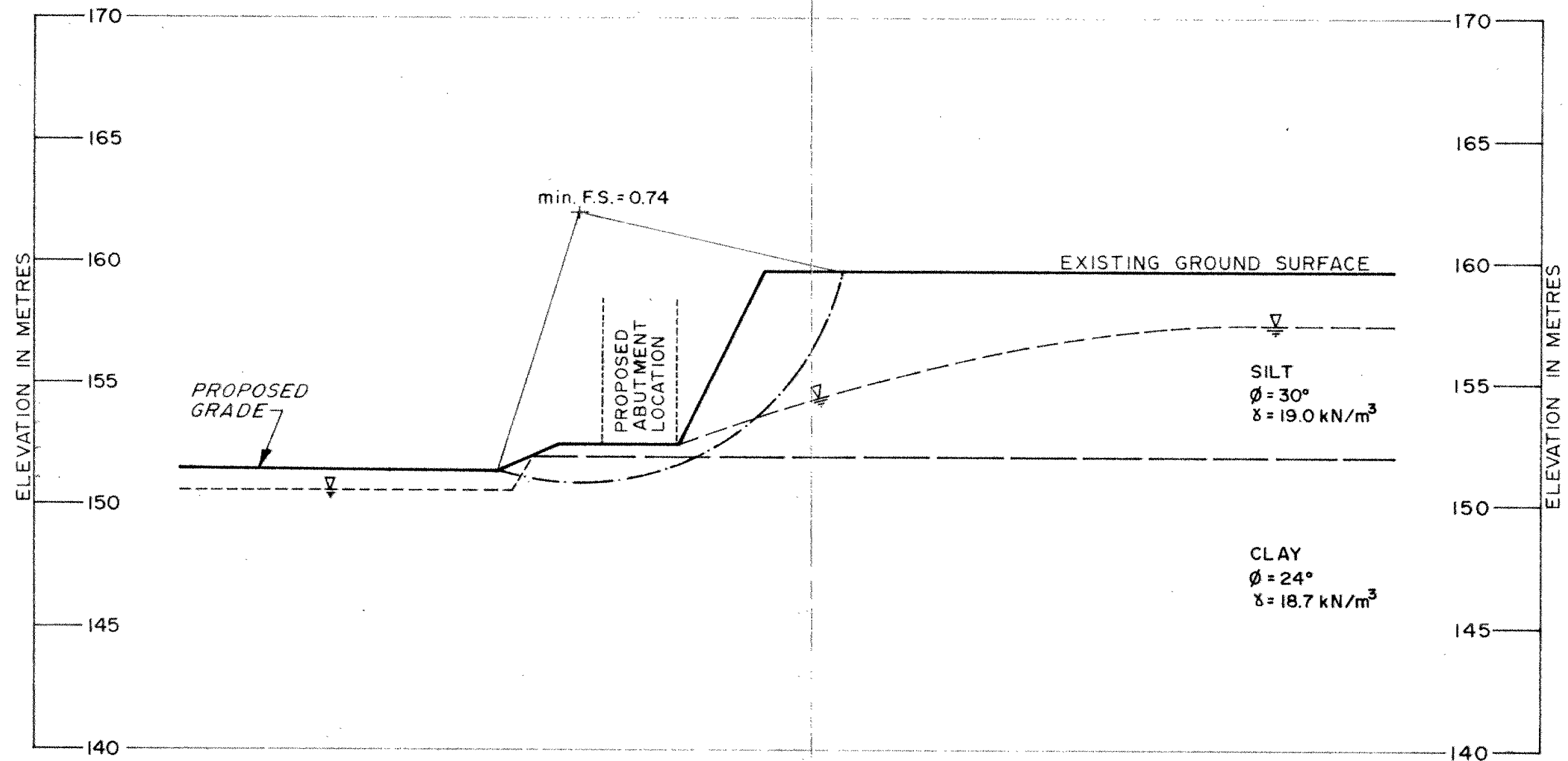


Navy = 15 blows

159 = 80
2 = 80
15
= 160



OVERSIZE DRAWING

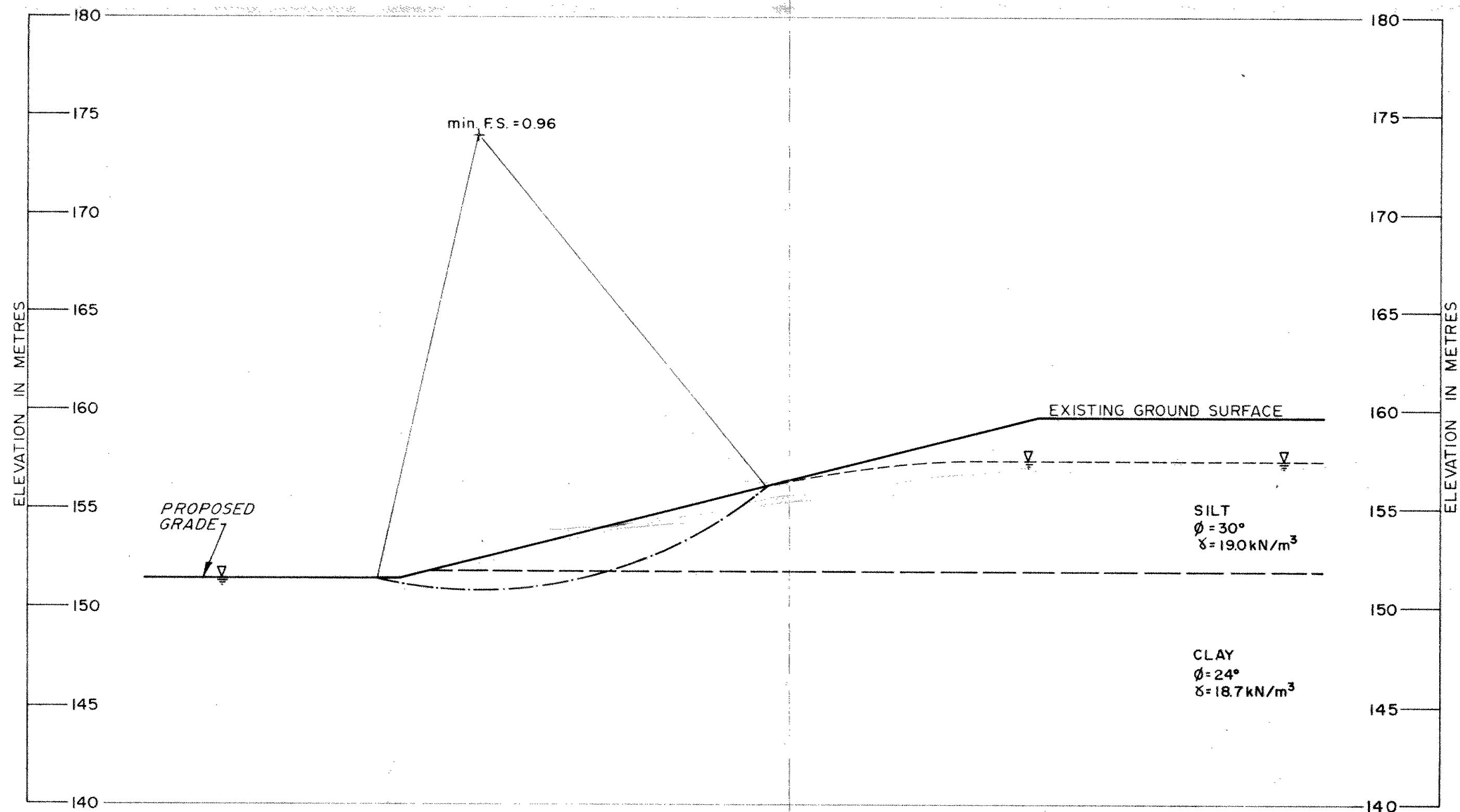


Ministry of
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Communications

STABILITY OF ABUTMENT SLOPE DRAINED CONDITIONS

FIG No 18

WP 88-78-18

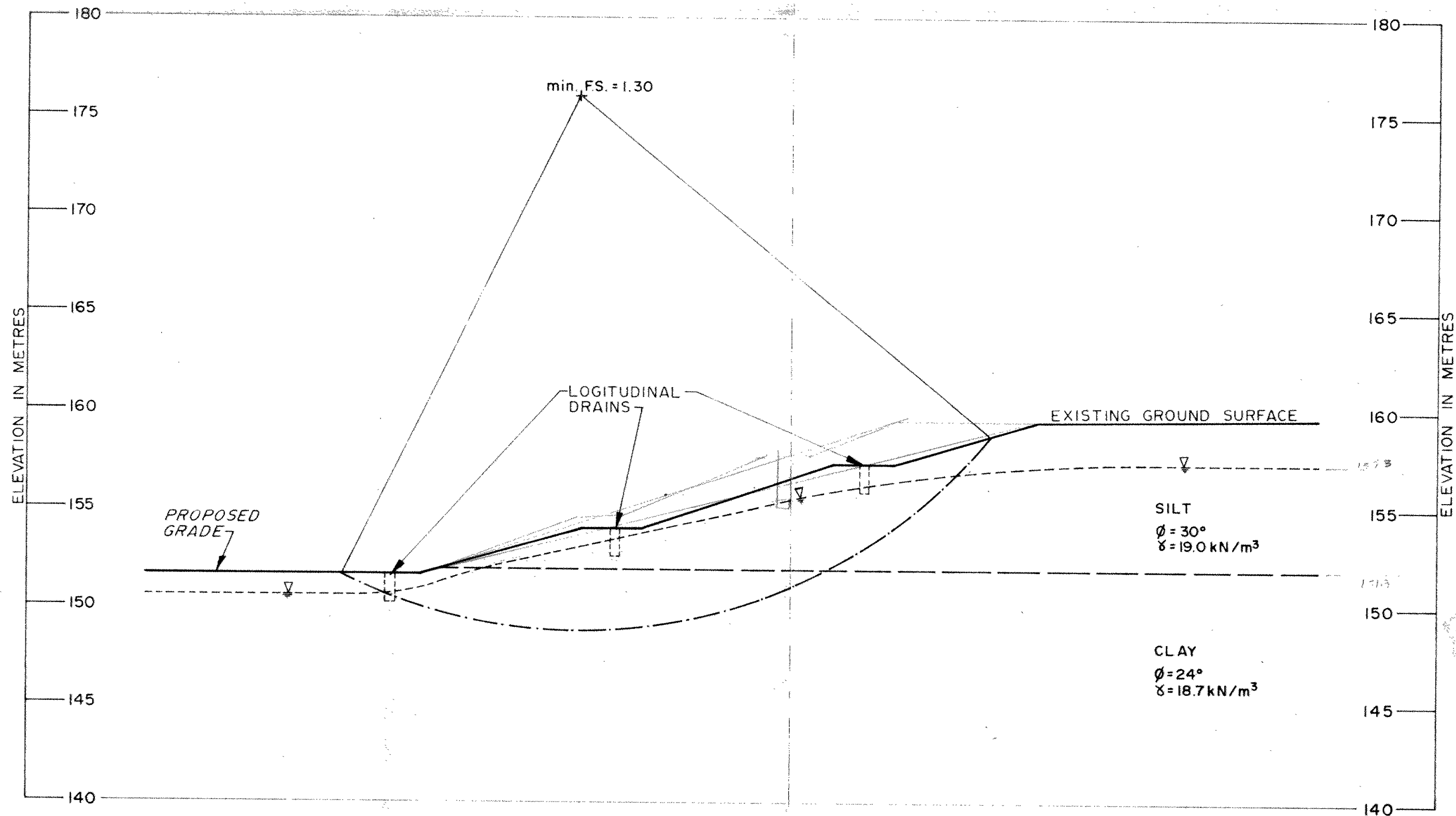


Ministry of
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Communications

STABILITY OF 4 TO 1 SLOPE WITHOUT POSITIVE DRAINAGE

FIG No 19

WP 88-78-18

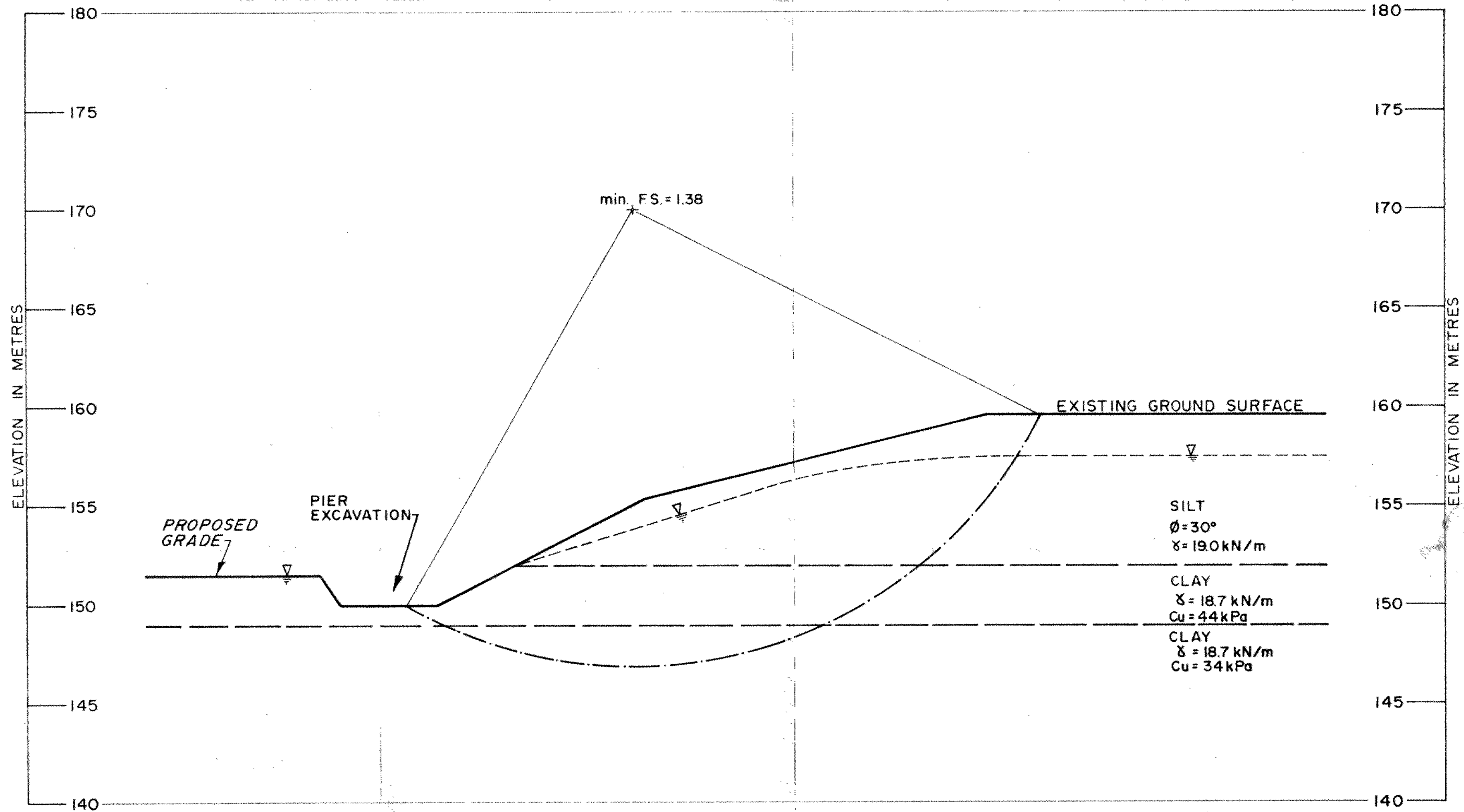


Ministry of
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Communications

STABILITY OF 4 TO 1 SLOPE WITH POSITIVE DRAINAGE

FIG No 20

WP 88-78-18



Ministry of
Transportation and
Communications

STABILITY OF 4 TO 1 SLOPE
UNDRAINED CONDITION
WITH PIER EXCAVATION

FIG No 21

WP 88-78-18

GENERAL NOTES

- *CLASS OF CONCRETE**
- | | |
|------------------------------------|--------|
| Deck, sidewalk & pier columns..... | 35 MPa |
| Remainder..... | 30 MPa |
- *CLEAR COVER TO REINFORCING STEEL**
- | | |
|--|------------------------------|
| Footings..... | 100±25 |
| Abutments & Retaining Walls: Front Face..... | 80±20 |
| Back Face..... | 70±20 |
| Piers..... | 80±20 |
| Deck: Top Slab, Top..... | 70±20 |
| Bot..... | 40±10 |
| Bot. Slab, Top..... | 40±10 |
| Bot..... | 50±10 |
| Walls..... | 60±10 |
| Remainder..... | 70±20 unless noted otherwise |

• REINFORCING STEEL

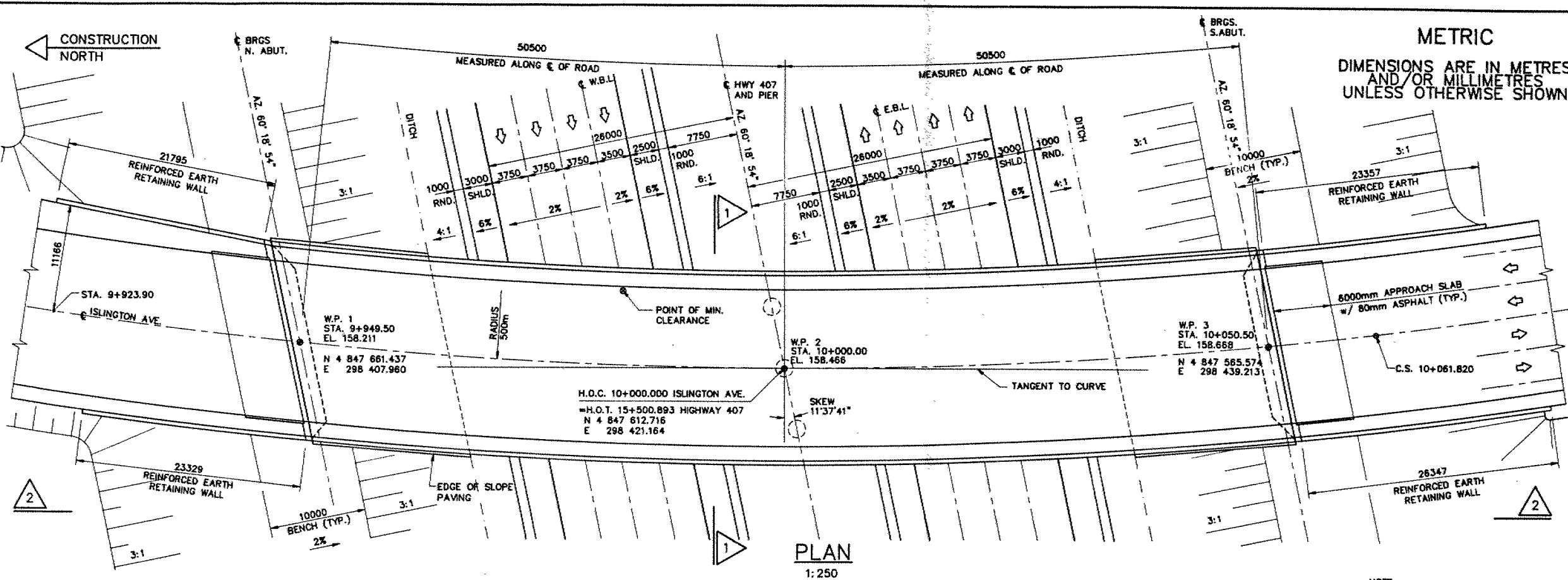
Reinforcing steel shall be Grade 400 unless noted otherwise - bar marks with suffix "C" denote coated bars.

• CONSTRUCTION NOTES

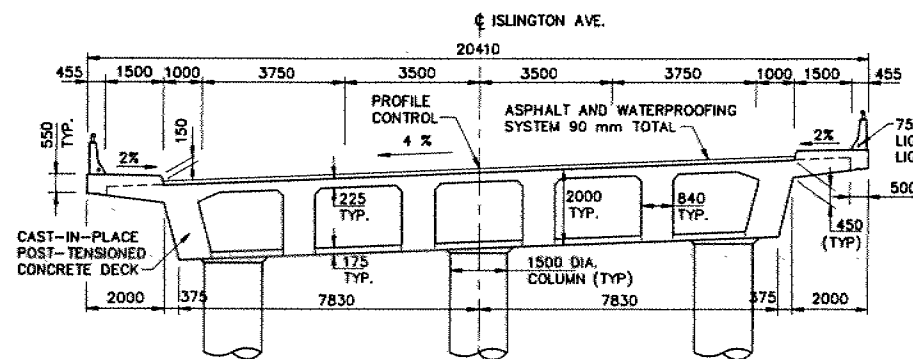
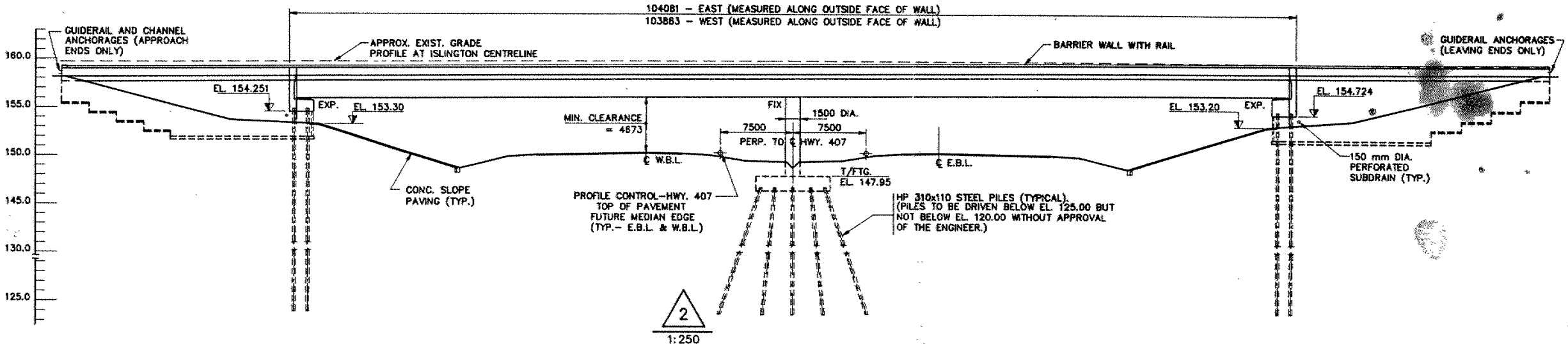
if the actual bearing thicknesses are different from those given in the bearing design data the contractor shall adjust the bearing seat elevations and the reinforcing steel to suit.

LIST OF DRAWINGS

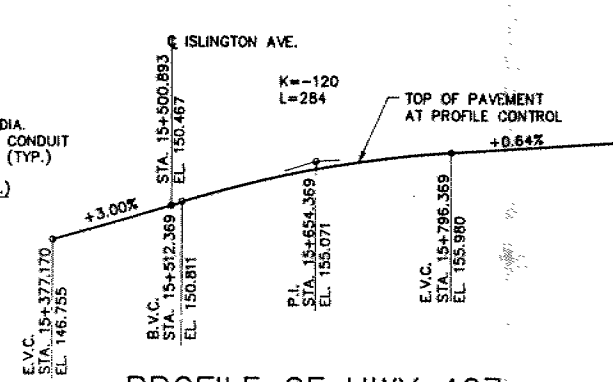
1. GENERAL ARRANGEMENT
2. BOREHOLE LOCATION AND SOIL STRATA
3. PILE AND FOOTING LAYOUT
4. PIER AND PIER FOOTING
5. NORTH ABUTMENT
6. SOUTH ABUTMENT
7. BEARINGS
8. DECK DETAILS
9. LONGITUDINAL TENDONS
10. TRANSVERSE TENDONS - I
11. TRANSVERSE TENDONS - II
12. DECK REINFORCING - I
13. DECK REINFORCING - II
14. DECK REINFORCING - III
15. DECK REINFORCING - IV
16. DECK REINFORCING - V
17. JOINT ANCHORAGE AND ARMOURING
18. BARRIER WALL ON SIDEWALK - EAST
19. BARRIER WALL ON SIDEWALK - WEST
20. BARRIER WALL ON R.E. RETAINING WALL
21. RAILING FOR BARRIER WALL
22. 4000 mm APPROACH SLAB
23. DETAILS OF CONC. SLOPE PAVING
24. ELECTRICAL EMBEDDED WORK
25. AS CONSTRUCTED ELEV. & DIM.
26. PILE DRIVING - STEAM & DIESEL HAMMERS
27. STANDARD DETAILS
28. REINFORCED EARTH WALLS - PLAN AND DETAILS
29. REINFORCED EARTH WALLS - ELEVATION, SECTION AND DETAILS
30. REINFORCED EARTH WALLS - ELEVATION AND SECTION
31. REINFORCED EARTH WALLS - TYPICAL DETAILS
32. QUANTITIES - STRUCTURE I
33. QUANTITIES - STRUCTURE II



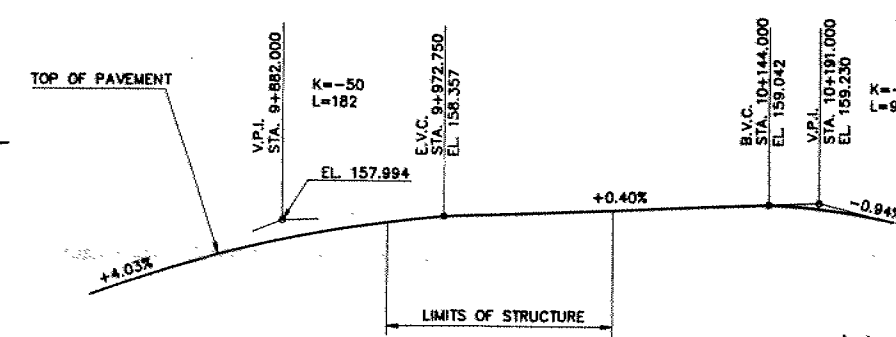
NOTE:
W.P. - DENOTES WORKING POINT
T/P - DENOTES TOP OF PAVEMENT



B.M. 157.924
TOP OF WEST BOLT OF S.W. LEG
HYDRO TOWER - STA. 9+795.00
38.0 m LEFT



PROFILE OF HWY 407



PROFILE OF ISLINGTON AVE.

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DESCRIPTION									
	DATE	BY								
DESIGN	P.D.	CHK	SAM	CODE	CHBOC-83	LOAD	CLASS 'A'	DATE	OCT. 1990	
DRAWN	BOK	CHK	B.D.	SITE	37-1120	STRUCT	SCHEME	DWG	1	

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

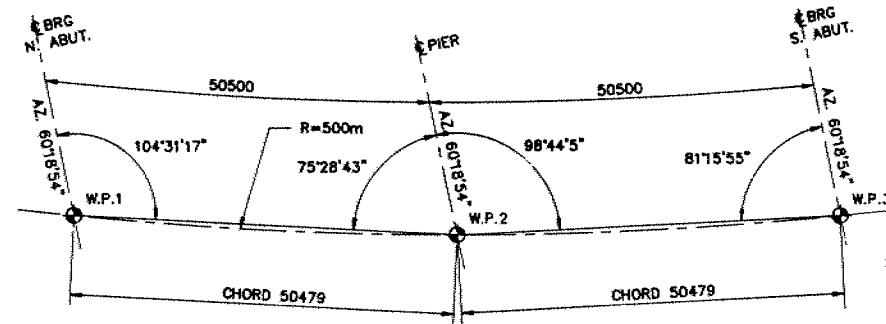
DIST No
CONT No
WP No 88-78-18
HWY. 407 - ISLINGTON AVE.
UNDERPASS
PILE & FOOTING LAYOUT



SHEET



ALIGNMENT LAYOUT N.T.S.



LIST OF HP310 x 110 PILES

	NORTH ABUTMENT		PIER		SOUTH ABUTMENT	
BATTER	LENGTH	NUMBER	LENGTH	NUMBER	LENGTH	NUMBER
VERTICAL	31600	18	23500	5	32200	18
1 : 3	-	-	24700	30	-	-
1 : 10	-	-	23600	10	-	-

NOTES

PILE SPACING MEASURED AT UNDERSIDE OF FOOTING.

PILE LENGTH SHOWN ON THE DRAWING ARE THE THEORETICAL LENGTH BELOW CUT-OFF.

ALL PILES HP 310 x 110.

PILE DRIVING NOTES

- 1.) PILES MUST BE DRIVEN BY A HAMMER DELIVERING AN ENERGY NOT LESS THAN 50 kJ PER BLOW.

PILES TO BE DRIVEN IN ACCORDANCE WITH STANDARD SS 103-10 OR SS 103-11 USING AN ULTIMATE CAPACITY OF 3450 kN PER PILE.

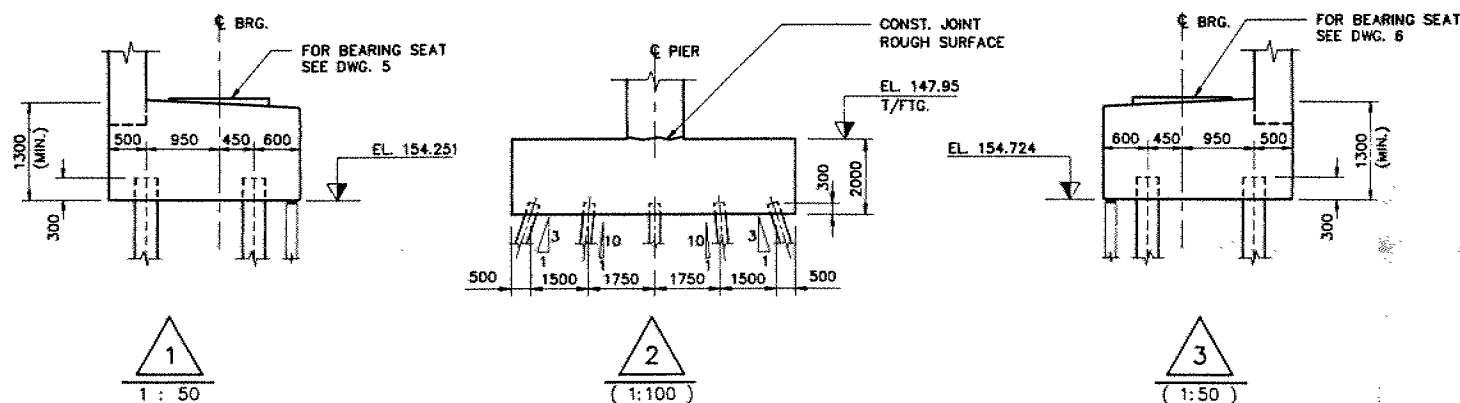
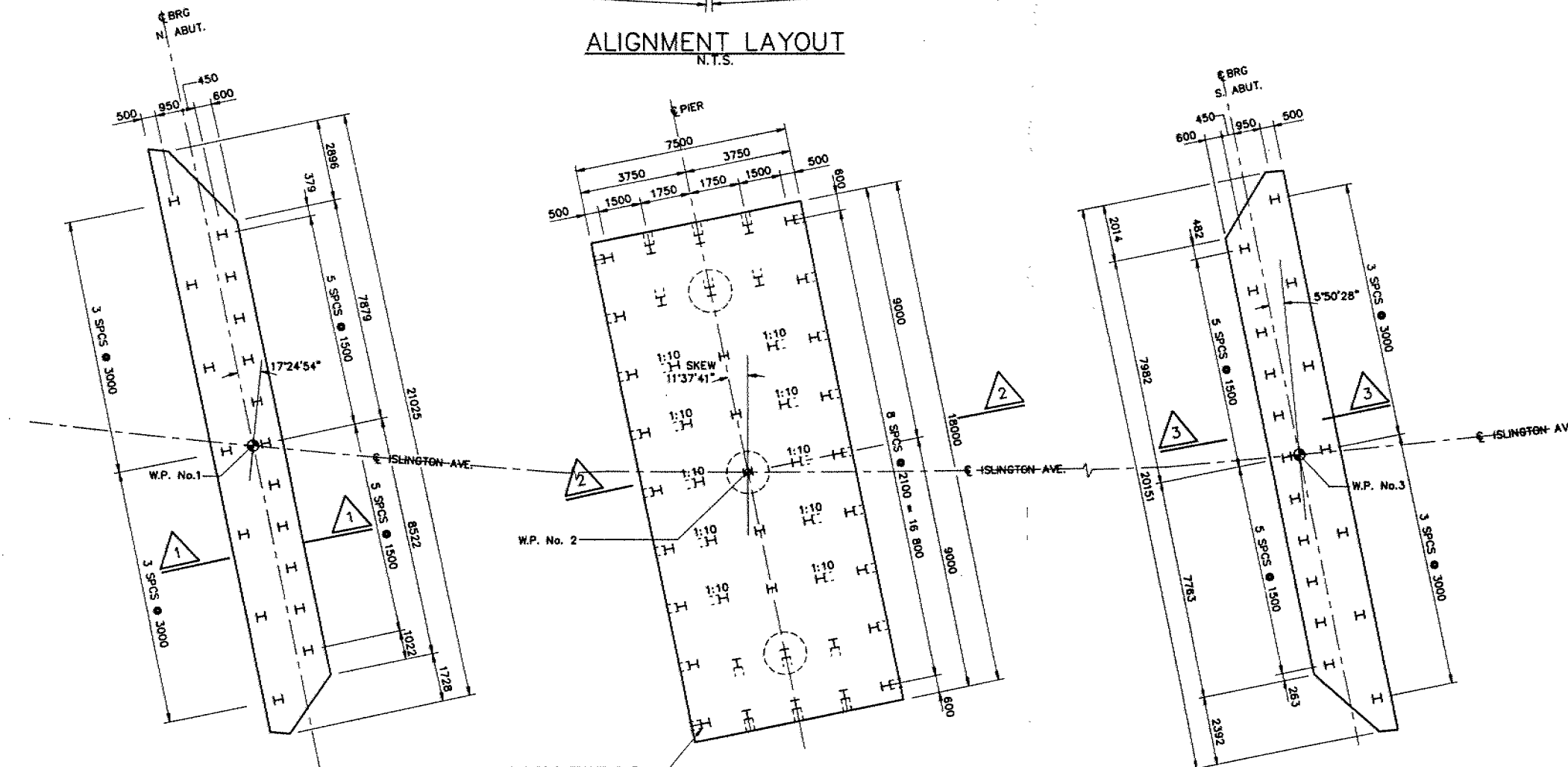
PILE DESIGN DATA

CAPACITY AT SLS TYPE II-----1150 kN
FACTORED CAPACITY AT ULS-----1650 kN

APPLICABLE STANDARD DRAWING
DD 3301-SPlicing & DRIVING SHOE DETAIL
FOR STEEL H-PILES.

1
2
3

PLAN (1:100)



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING



REVISIONS	DATE	BY	DESCRIPTION

DESIGN P.O. CHK. SAM CODE OHBDC-83 LOAD CLASS 'A' DATE DEC 1990
DRAWN BOK JRICHK. B.D. SITE 37-1120 STRUCT. SCHEME IOWG 3

CAD FILE 12835002.DWG
Nov. 20, 1990 10:30 a.m.

ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION

WP 88-78-18

DIST 6

HWY 407

STR SITE 37-1120

CONT. 92-40

DISTRIBUTION

V.F. Boehnke (3)

G. Cautillo

J. Cullen (2)

A. Wittenberg

K.G. Bassi

S.J. Dunham

E.A. Joseph

I. Harrod (Cover Only)

I. Bullen (Cover Only)

File

FOUNDATION INVESTIGATION REPORT
For
Hwy. 407 - Islington Avenue Underpass
W.P. 88-78-18, Site 37-1120
District 6, Toronto

INTRODUCTION

This report summarizes the results of a foundation investigation conducted at the proposed underpass. The fieldwork for this investigation was carried out under two separate mobilizations. Initially, four (4) boreholes were advanced from 83 06 20 to 83 07 13 using a conventional track-mounted auger machine employing solid stem, hollow stem and wash boring techniques. Under a subsequent mobilization, three (3) additional boreholes were advanced from 84 07 23 to 84 07 31 utilizing similar equipment.

SITE DESCRIPTION AND GEOLOGY

The site is located on Islington Avenue, approximately 0.5 kilometers north of Steeles Avenue in the Town of Vaughan, Regional Municipality of York. South of the site along Islington Avenue, a CNR overpass structure is present which is adjacent to the more southerly CPR level crossing. Residential homes and accompanying properties are located immediately west of the site and an agricultural industrial building is located northeast of the site.

The terrain surrounding the site is generally flat but a Humber River tributary valley is present approximately 300 metres west of the site. Land use east of the site is primarily agricultural.

Physiographically, the site is located in the geological domain known as the Bolton area. At the site, deltaic and glaciolacustrine sands and silts, the products of Lake Peel (Karrow 1963), a body of water confined between a lobe of ice projecting up the Humber Valley and the surrounding higher elevation grounds, overly a glaciolacustrine Wildfield Till complex consisting of stratified silty clays, clayey silts and silt nodules, also depositions of Lake Peel. These deposits levelled out the former gently undulating surface of ground moraine, known as the "Peel Plain" (Putnam and Chapman, 1936).

The Bolton area features overburden deposited during the Wisconsin glacialiation of the Pleistocene era. The overburden is underlain by shale bedrock of the Dundas-Meaford formation. Water well records indicate that the bedrock is found at depths ranging from 40 to 60 m.

INVESTIGATION PROCEDURE

Soil data and inherent properties were obtained by in situ and laboratory testing. The procedures employed are discussed below.

Field Investigation

A total of seven (7) boreholes were advanced at the site at the locations illustrated on the attached plan 887818-A. Six (6) of the boreholes were sampled at the intervals and in accordance to the procedures described below. One borehole (BH 2A) was an augered exploratory borehole in which the soil stratigraphy was inferred from the auger cuttings. This borehole was advanced to a depth of 5.5 m.

The six (6) sampled boreholes were advanced to depths ranging from 40.1 m to 49 m using a combination of solid stem, hollow stem and wash boring techniques. In boreholes 1-3 inclusive, subsoil samples were retrieved at 0.7 m intervals for the surficial 6 m and 1.5 m intervals thereafter. At boreholes 4-6 inclusive, subsoil samples were retrieved at 1.5 m and 3.0 m intervals.

Disturbed subsoil samples were retrieved by a split spoon sampler in accordance with the Standard Penetration Test (ASTM D1586). Relatively undisturbed samples were also randomly retrieved within the weaker cohesive deposits at the site using a shelly tube sampler in accordance with standard practice (ASTM D1587). In situ vane tests were also conducted in the weaker cohesive soils to determine the undisturbed and remoulded undrained shear strengths of the soil.

All subsoil samples were identified in the field and returned to the laboratory for further examination and applicable testing.

Water levels were obtained in the open boreholes and also in sealed piezometers within the different soil strata at the site.

Survey information related to the location and elevation of boreholes was provided by Golder Associates and MTO Central Region Surveys and Plans.

Laboratory Analyses

To identify the behaviour, gradation and pertinent properties and characteristics of the soil, various laboratory tests were performed. These tests included:

- 1) Atterberg Limits
- 2) Grain Size Distributions
- 3) Unit Weights
- 4) Natural Moisture Contents
- 5) Consolidated Drained Direct Shear Tests
- 6) Multi-stage Consolidated Undrained Tests with Pore Pressure Measurements

Laboratory test results have been summarized in the subsequent section of this report entitled Subsurface Conditions, and are illustrated on corresponding figures and boreholes included in the attached Appendix.

SUBSURFACE CONDITIONS

The ground surface elevation at the site ranges from 158.5 to 159.6. Topsoil and fill material consisting of sand and gravel extends from the ground surface at some locations of the site.

The native soil stratigraphy throughout much of the site area consists of a surficial deposit of a cohesionless silt to sandy silt interbedded with random layers of clayey silt. The deposit has a thickness ranging from 2.8 to 7.3 m.

At the northerly portion of the site (BH 4), the surficial deposit consists of a cohesive clayey silt. The thickness of this stratum is 2.1 metres.

Underlying the cohesionless silt to sandy silt deposit, a layer of clayey silt to silty clay is present with thickness ranging from 3 to 11.3 metres. This stratum also contains random nodules and layers of plastic silt.

A cohesive deposit of clayey silt with some sand and a trace of gravel underlies the clayey silt to silty clay stratum. This deposit is a glacial till and has a thickness ranging from 9.3 m to 16.8 metres.

The clayey silt, some sand, trace of gravel (Glacial Till) is underlain by a second lower stratum of clayey silt. This stratum also contains occasional layers and pockets of sand, gravel and cobbles. The thickness of this stratum ranges from 3 to 16.2 metres (El. 128 m to 117.3 m respectively).

Below depths ranging from 30.5 m to 37.2 m, a complex deposit that ranges in composition from a silt to sandy silt to a sandy gravel to gravelly sand exists. This deposit also has interbeds of cohesive clayey silt. The full extent of this deposit was not determined during the investigation but thicknesses ranging from 6.4 m to 18.8 m were indeed explored.

The boundaries between the various soil types, in situ and laboratory test results as well as groundwater levels established at the time of investigation, are shown on the attached Record of Borehole sheets in the Appendix. A plan of the site illustrating the locations and elevations of the boreholes and subsoil stratigraphical sections are provided on Dwg. 887818-A.

A detailed description of the subsurface conditions encountered is given below.

Sand and Gravel (Fill Material) and Topsoil

Sand and gravel, a base course fill material was present at BH 3 (unpaved shoulder of Islington Avenue) for a thickness of 0.8 metres. In addition, brown, sandy topsoil was also found at BH 1.

Upper Sandy Silt to Silt

The surficial deposit spread for the most part across the site consists of cohesionless silts, sands and sandy silts. At the southern limit of the site (BH 4), however, the cohesionless deposit is overlain by a thin veneer of clayey silt. In addition, random layers of clayey silt are also interbedded within the deposit. A grain size distribution envelope for the sandy silt to silt material is illustrated on Figure 1 in the Appendix. Figure 1 also illustrates a grain size distribution curve for a medium to coarse grained sand found in this deposit.

The thickness of the cohesionless silt to sandy silt deposit ranges from 3 to 11.3 metres. The deposit is oxidized and hence brown in colour for depths of 2.4 m to 3.7 m below the ground surface. Below these depths, the deposit is unoxidized and grey.

Natural moisture contents determined from samples obtained in this deposit range from 13 to 27 percent. The higher moisture contents reflect the presence of a perched water table within this deposit.

The results of the Standard Penetration Test (SPT) reveal 'N' values ranging from 3 blows/0.3 m to 20 blows/0.3 m indicating a state of denseness ranging from very loose to compact. However, in general, 'N' values range from 10 blows/0.3 m to 20 blows/0.3 m and hence the deposit can be considered as compact.

Clayey Silt (Surficial Veneer)

A surficial veneer of brown clayey silt was found at the most northerly portion of the site (BH 4). The thickness of this surficial stratum is 2.1 metres.

Clayey Silt to Silty Clay with Random Layers/Nodules of Silt

The cohesionless sandy silt to silt deposit is underlain by an irregularly, horizontally layered cohesive stratum consisting of clayey silt to silty clay

ranging in thickness from 3 to 11.3 metres. The stratum, grey in colour, also contains random layers and nodules of light grey plastic silt. Random pockets of fine sand are also randomly present within the stratum. Grain size distribution curves for this material are illustrated in Figure 2 in the Appendix. The curves depict clay and silt percentages ranging from 33 to 55 percent and 67 to 44 percent respectively.

Atterberg Limit tests were conducted to evaluate the behaviour of the material in this stratum and the results are plotted in Figure 3 in the Appendix. A summary of the indices is provided in Table 1, 2 and 3 below.

Table 1 - Clayey silt

	<u>Range</u>	<u># of Tests</u>
Natural Moisture Content (w%)	21-27	8
Liquid Limit (w_L)	21-48	4
Plasticity Index (I_p)	5-8	4

Table 2 - Silty clay

	<u>Range</u>	<u># of Tests</u>
Natural Moisture Content (w%)	18-44	4
Liquid Limit (w_L)	21-48	5
Plasticity Index (I_p)	9-24	5

Table 3 - Silt

	<u>Range</u>	<u># of Tests</u>
Natural Moisture Content (w%)	23	2
Liquid Limit (w_L)	21	2
Plasticity Index (I_p)	3-4	2

The results reveal that the stratum has a plasticity ranging from low to intermediate and hence can be categorized as clayey silt to silty clay. In addition, random layers of plastic silt of low plasticity are also present within this stratum.

Bulk unit weights of the soil obtained from representative samples ranged from 18.5 kN/m³ to 18.8 kN/m³.

Undrained shear strength measurements (cu) of the soil were obtained by conducting in situ vane tests and laboratory vane tests. The results, illustrated on the boreholes in the Appendix, reveal values ranging from 34 to in excess of 96 kPa for the in situ vane test and 28 to 39 kPa for the laboratory vane. The higher values are thought to be attributable to the presence of silty layers in the deposit and consequently, a more representative range of the undrained shear strength is 34 to 80 kPa. The undrained shear strengths generally increase below elevation 147 m. Based on the in situ and laboratory shear strengths, the soil can be classified as having a firm to stiff consistency.

The sensitivity of the soil as defined by the ratio of the undrained strength in the undisturbed state to the undrained strength, at the same water content, in the remoulded state was also determined by the field vane test and the results are identified on the Record of Borehole sheets. Sensitivity values range from 2 to 4.5, indicating that the soil has a low sensitivity.

The 'N' values as determined by the Standard Penetration Test ranged from penetration under the weight of the rods supporting the split spoon sampler to 9 blows/0.3 m. These results confirm the firm to stiff consistency of the soil.

The results of a series of consolidated undrained triaxial tests with pore pressure measurements on a sample of the silty clay from El. 149 in BH 2 are presented on Figure 4. The stress paths followed by each test vary somewhat, probably due to shearing through layers having various silt contents. The minimum effective angle of internal friction of 30 degrees was measured in sample C which was consolidated well in excess of the existing overburden pressures. From this it is concluded that the angle of internal friction for design when failure will take place on a vertical or near vertical plane can be taken as 30 degrees.

A series of three drained direct shear tests were carried out on a sample of the silty clay from elevation 144.5 m in BH 2 and the results are illustrated in

Figure 5. The specimens were oriented so that shear failure would occur parallel to the layers. The results indicate that the effective angle of internal friction of the clay on horizontal planes is 24 degrees.

Clayey Silt, some Sand, trace Gravel (Glacial Till)

Underlying the clayey silt to silty clay stratum, a deposit consisting of a heterogeneous mixture of clayey silt, sand and gravel was encountered to elevations ranging from 131 to 137.1. The thickness of this glacial till deposit ranges from 9.3 m to 16.8 m. Grain size distribution curves as shown in Figure 6 illustrate that the host material consists of clay and silt percentages and that some sand and traces of gravel are also present within the deposit. Although not encountered during the investigation, boulders and are inherent components of glacial till deposits and hence can exist within this deposit.

Atterberg Limit tests were carried out to determined the behaviour of the fine grained portion of the deposit and the results are plotted on Figure 7 and summarized in Table 4 below.

Table 4 - Clayey silt

	<u>Range</u>	<u># of Tests</u>
Natural Moisture Content (w%)	15-27	22
Liquid Limit (w_L)	20-27	4
Plasticity Index (I_p)	8-14	4

The results reveal that the fine grained material is of low plasticity and hence is classified as clayey silt. Natural moisture contents are generally in the 15 to 20% range and hence are less than the liquid limit of the soil.

Undrained shear strength measurements (cu) of the soil obtained by conducting in situ vane tests and also laboratory vane tests. Results are plotted on the Record of Borehole sheets in the Appendix. The results reveal undrained shear strength values ranging from 39 kPa to in excess of 115 kPa but are generally in

the 45 kPa to 75 kPa. Hence, soil consistency ranges from firm to very stiff but is generally stiff.

The sensitivity of the soil as defined by the ratio of the undrained strength in the undisturbed state to the undrained strength, at the same water content, in the remoulded state was also determined by the field vane test and the results are identified on the Record of Borehole sheets. Sensitivity values range from 1 to 3 indicating that the soil has an insensitive to low sensitivity.

Standard Penetration tests carried out in this deposit revealed 'N' values up to 54 blows/0.3 m and as low as penetration induced under the weight of the rods attached to the sampler. In general, however, 'N' values range from 10 blows/0.3 m to 15 blows/0.3 m.

Clayey Silt

Underlying the clayey silt with some sand and a trace of gravel glacial till deposit, a stratum of clayey silt extending for a thickness ranging from 2.5 metres to 16.2 metres exists. The stratum also contains occasional layers and pockets of silt and sand. Grain size distribution curves illustrating the gradation of the clayey silt material present in this stratum is illustrated in Figure 8 in the Appendix.

The clayey silt material has natural moisture contents ranging from 12 to 31 percent. Atterberg Limit tests carried out on two representative samples are illustrated on Figure 9 in the Appendix. The cohesive material has liquid limits ranging from 23% to 27% and plasticity indices ranging from 5 to 9 and hence exhibits a low plasticity. The figure also illustrates the presence of plastic silt interbedded in the deposit.

Standard Penetration tests carried out in this stratum revealed 'N' values ranging from 10 blows/0.3 m to 163 blows/0.25 m. Based on these values, the soil has a consistency ranging from stiff to hard. In general, however, 'N' values are greater than 20 blows/0.3 m and hence, the soil is generally very stiff to hard.

Lower Silts, Silty Sands to Sandy Silts, Gravelly Sand to Sandy Gravel

At an elevation ranging from 129.8 m to 117.3 m, a cohesionless deposit having a composition spanning a wide range of grain sizes exists. This cohesionless material which is irregular and non-uniform in both vertical and horizontal planes, also contains interbeds of cohesive clayey silt. The full extent of this deposit was not explored during the investigation but a maximum depth of 18.8 metres was penetrated.

Figures 10a to 10c inclusive shown in the Appendix illustrate the wide range of gradations associated with this deposit. A 2.7 m thick zone of limestone cobbles and cemented sand was also encountered in this deposit (see BH 2 - El. 128.0 to 125.3 m).

Grain size distribution curves and results of an Atterberg Limit test obtained at BH 2, SS 31 for the interlayered clayey silt is illustrated in Figures 11 and 12 respectively.

Standard Penetration tests carried out in the cohesionless deposit revealed 'N' values ranging from 10 blows/0.3 m to 150 blows/0.05 m. The lower 'N' values are suspected to be the result of soil disturbance introduced by the presence of unbalanced hydrostatic head during the sampling procedure. In general, 'N' values exceed 50 blows/0.3 m and in many instances exceed 100 blows/0.3 m. Hence, the cohesionless material can generally be considered as very dense.

The 'N' values as obtained by the Standard Penetration Test in the cohesive interlayered clayey silt ranged from 32 blows/0.3 m to 163 blows/0.25 m. Hence, this material can be described as having a hard consistency.

GROUNDWATER CONDITIONS

Groundwater levels at the site were obtained by measurement in the open boreholes and in piezometers installed in the different strata present at the site. All piezometers were installed with an upper bentonite seal approximately 0.3 m in thickness. Pea gravel (10 mm) was used to fill the annular space between the piezometer and the borehole.

The groundwater level in the upper sandy and silty soils at the time of the investigation was approximately at El. 157, which is about 1.5 to 2.6 metres below the natural ground surface. Deep piezometers installed in the lower cohesionless deposit indicate that the water level was at an elevation of approximately 141 m. The varying groundwater levels reveals a downward seepage gradient at the site and that a perched water table exists in the upper sand silt to silt.

Groundwater levels, in general, are subject to seasonal fluctuations and hence can vary from the values given in this report.

MISCELLANEOUS

The preceding report combines the factual information obtained from a Foundation Investigation Report prepared by Golder Associates and additional information retrieved by MTO. This report was written by T. Sangiuliano, Foundation Engineer, reviewed by Dr. B. Iyer, Senior Foundation Engineer, and approved by Mr. M.S. Devata, Chief Foundation Engineer.



A handwritten signature in black ink, appearing to read 'T. Sangiuliano'.

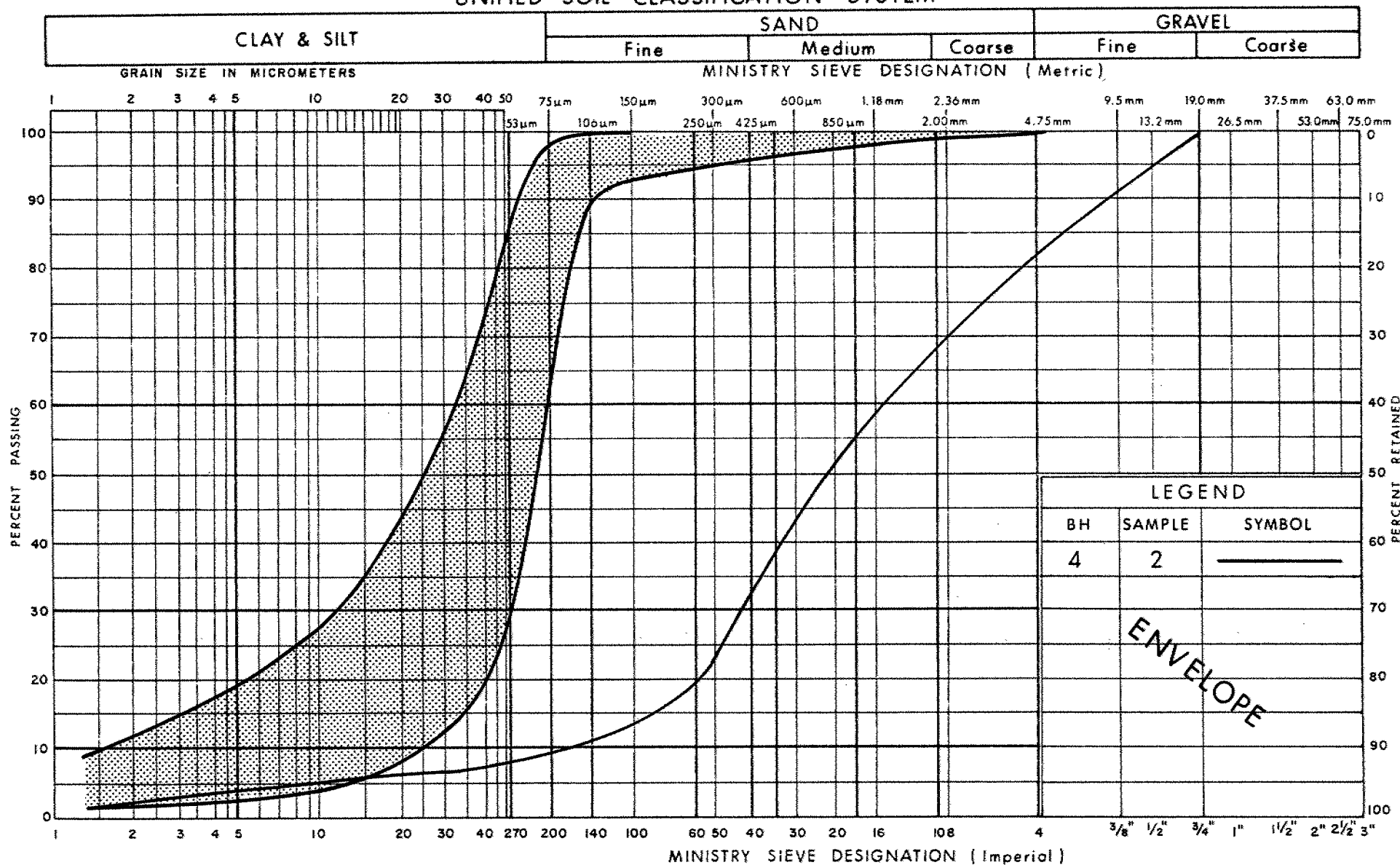
T. Sangiuliano, P.Eng.
Foundation Engineer

A handwritten signature in black ink, appearing to read 'M.S. Devata'.

M.S. Devata, P.Eng.
Chief Foundation Engineer

APPENDIX

UNIFIED SOIL CLASSIFICATION SYSTEM



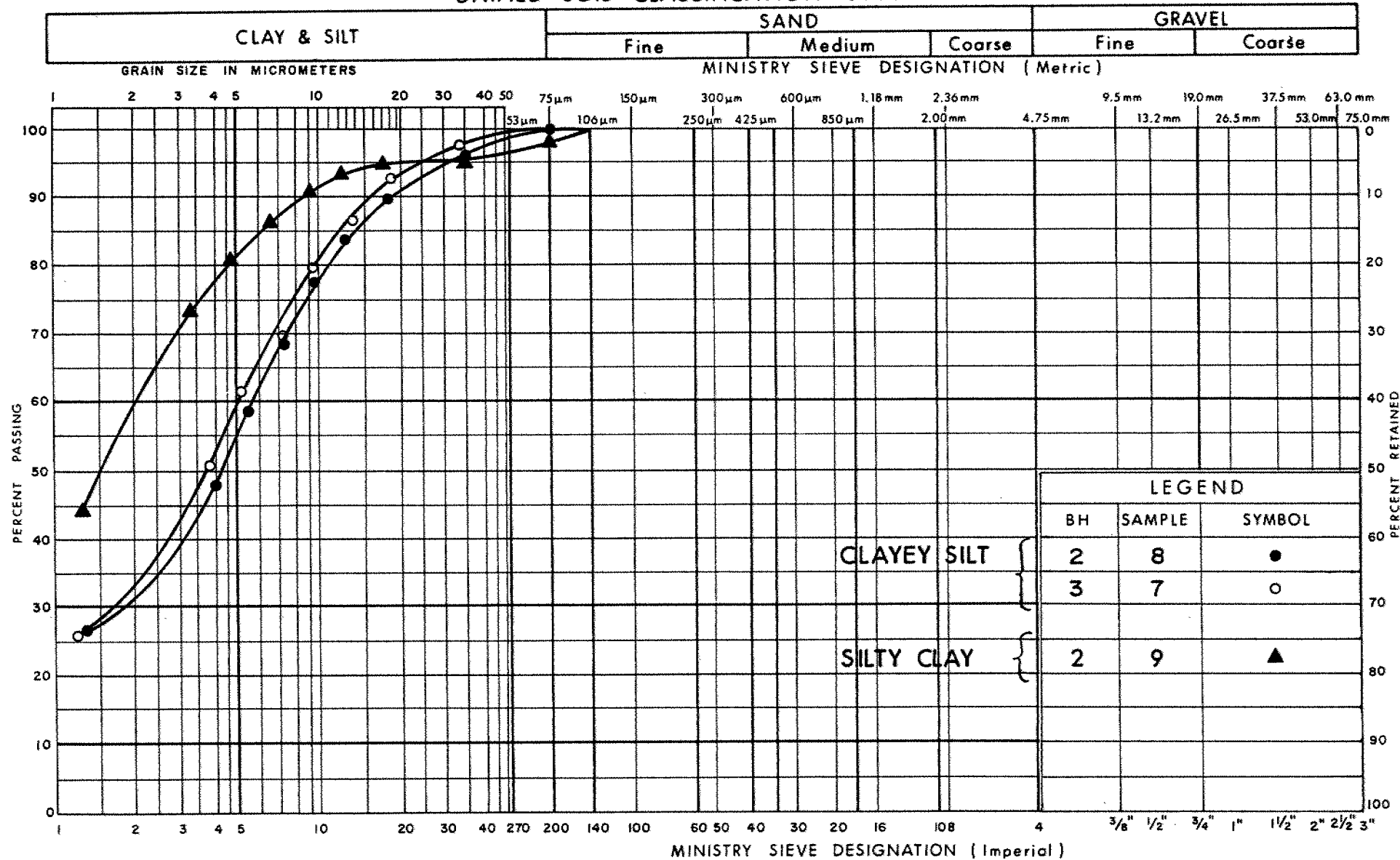
Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION
SANDY SILT TO SILT

FIG No 1

W P 88-78-18

UNIFIED SOIL CLASSIFICATION SYSTEM

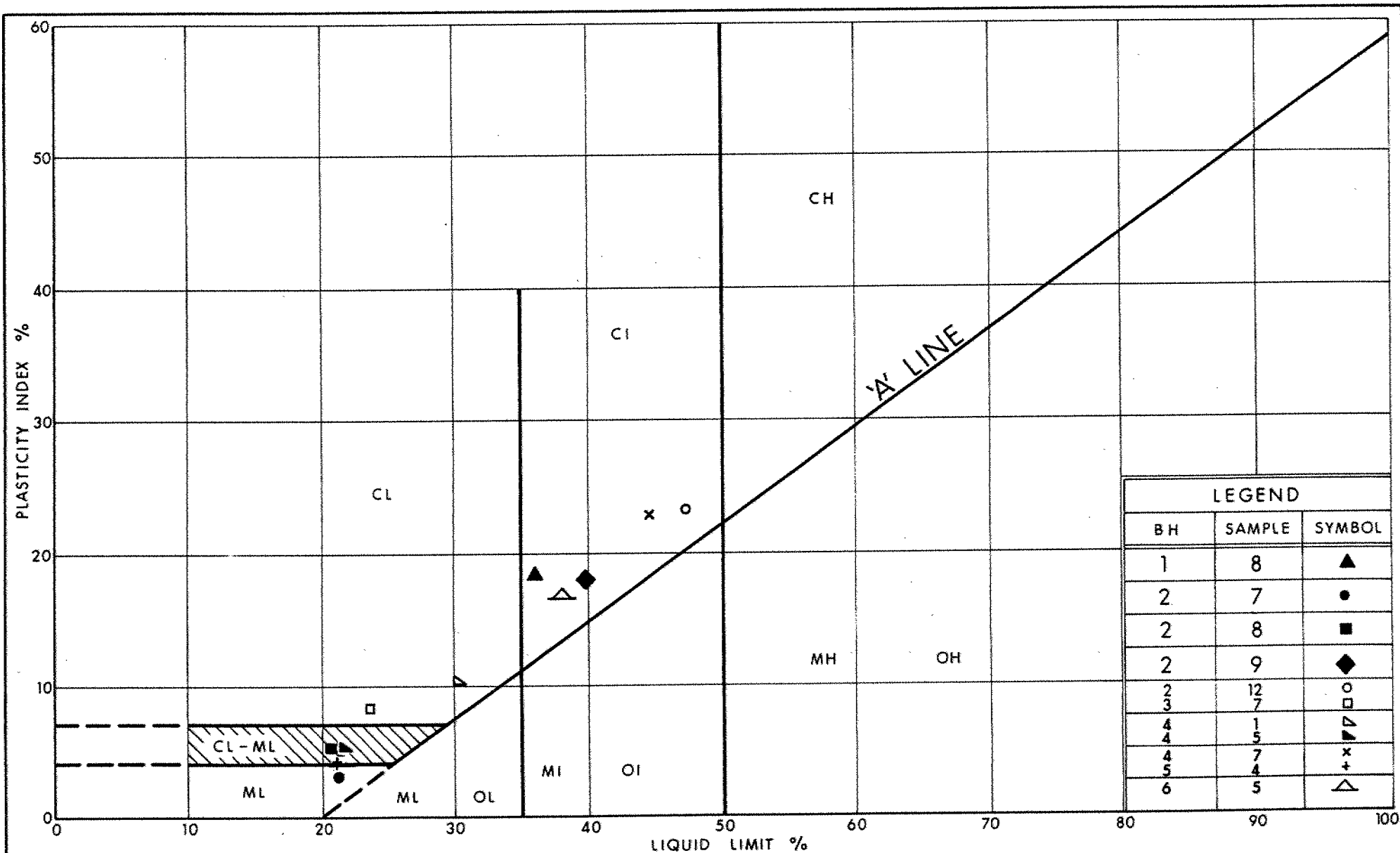


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GRAIN SIZE DISTRIBUTION
CLAYEY SILT TO SILTY CLAY
WITH RANDOM LAYERS/NODULES OF SILT

FIG No 2

W P 88-78-18



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PLASTICITY CHART
CLAYEY SILT TO SILTY CLAY
WITH RANDOM LAYERS/NODULES OF SILT

FIG No 3

W P 88-78-18

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND

GRAVEL

Fine

Medium

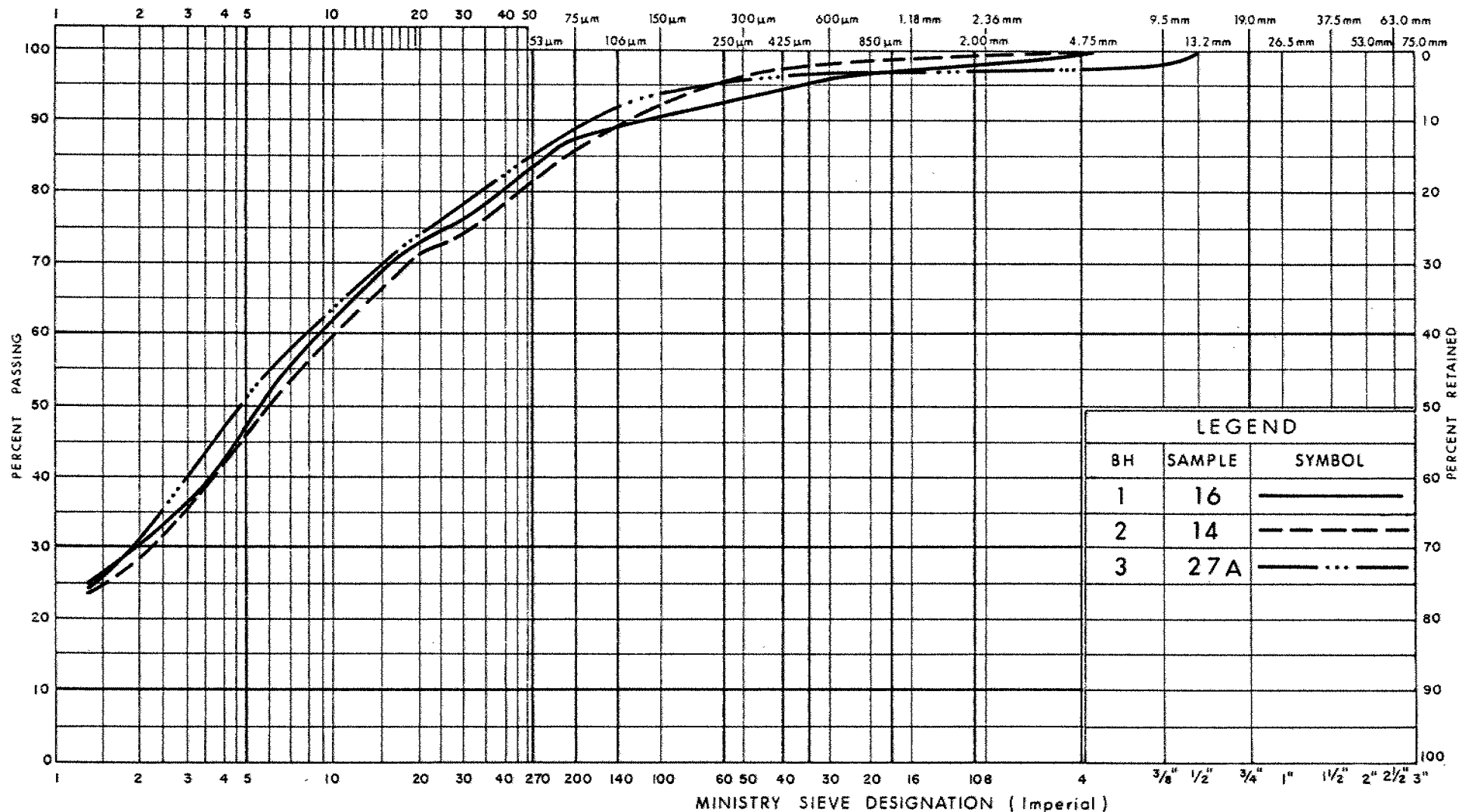
Coarse

Fine

Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)

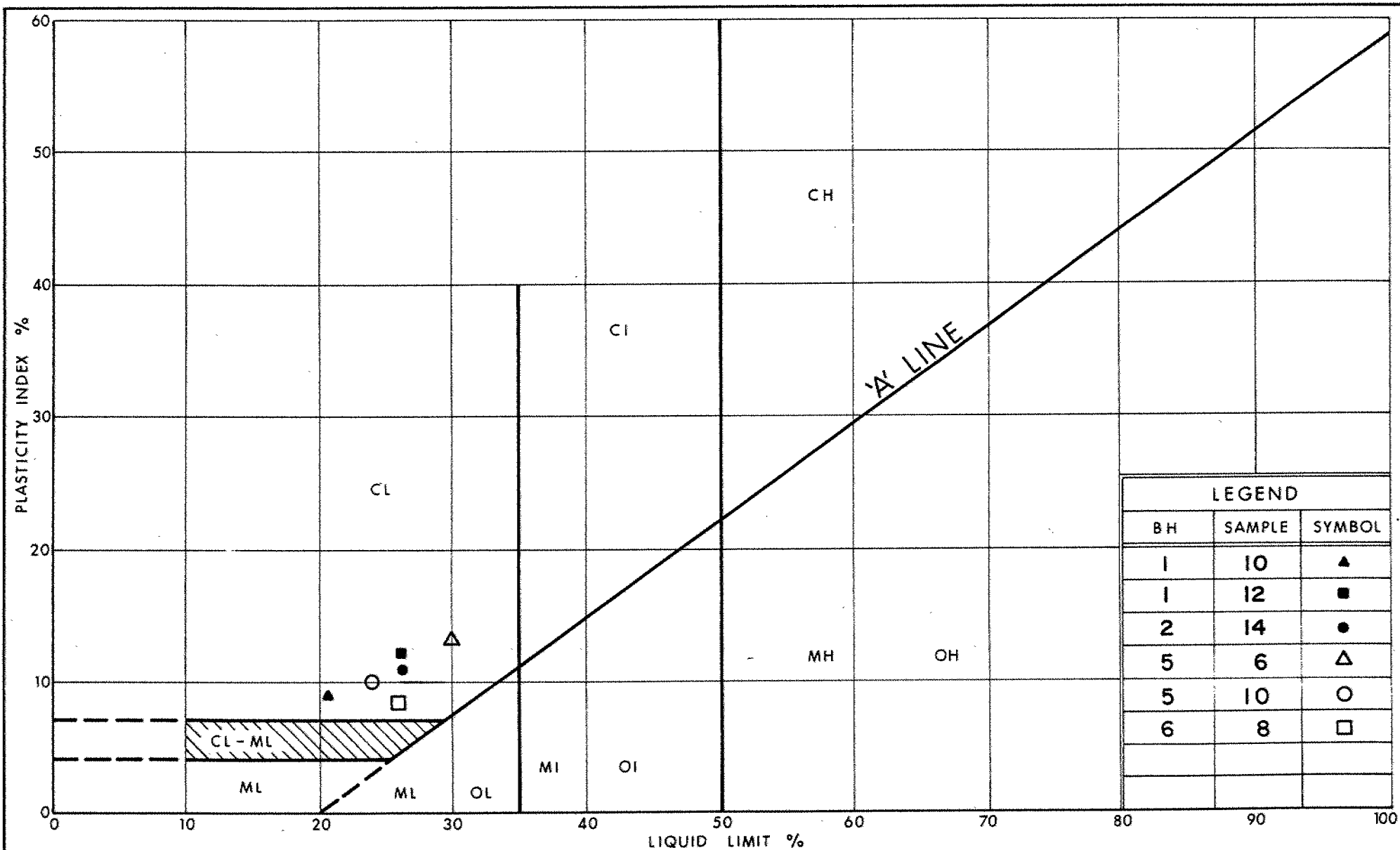


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Communications

GRAIN SIZE DISTRIBUTION
CLAYEY SILT, SOME SAND, TRACE GRAVEL
(GLACIAL TILL)

FIG No 6

WP 88-78-18



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PLASTICITY CHART
CLAYEY SILT, SOME SAND, TRACE GRAVEL
(GLACIAL TILL)

FIG No 7

W P 88-78-18

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND

GRAVEL

Fine

Medium

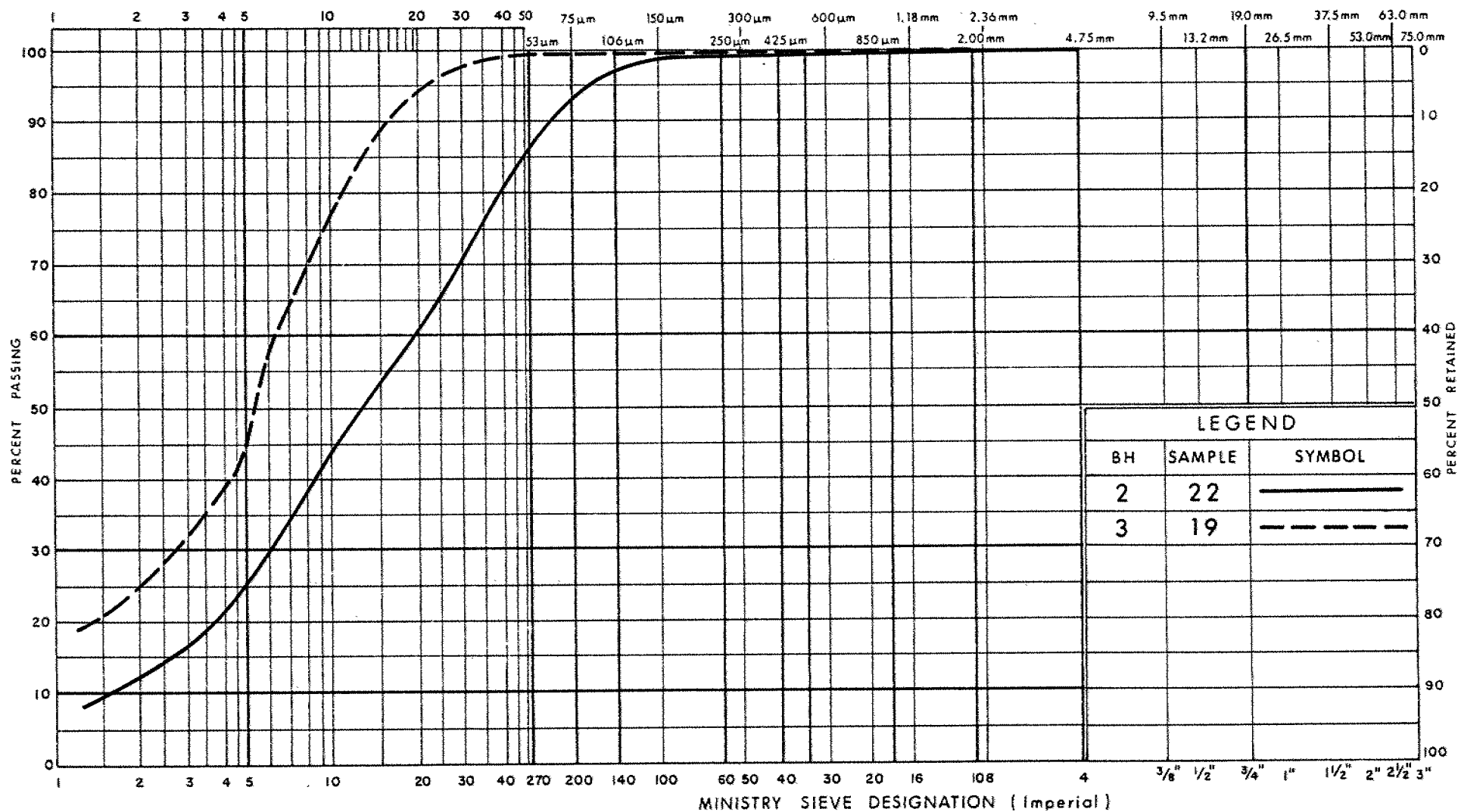
Coarse

Fine

Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



LEGEND

BH	SAMPLE	SYMBOL
2	22	—————
3	19	- - - - -

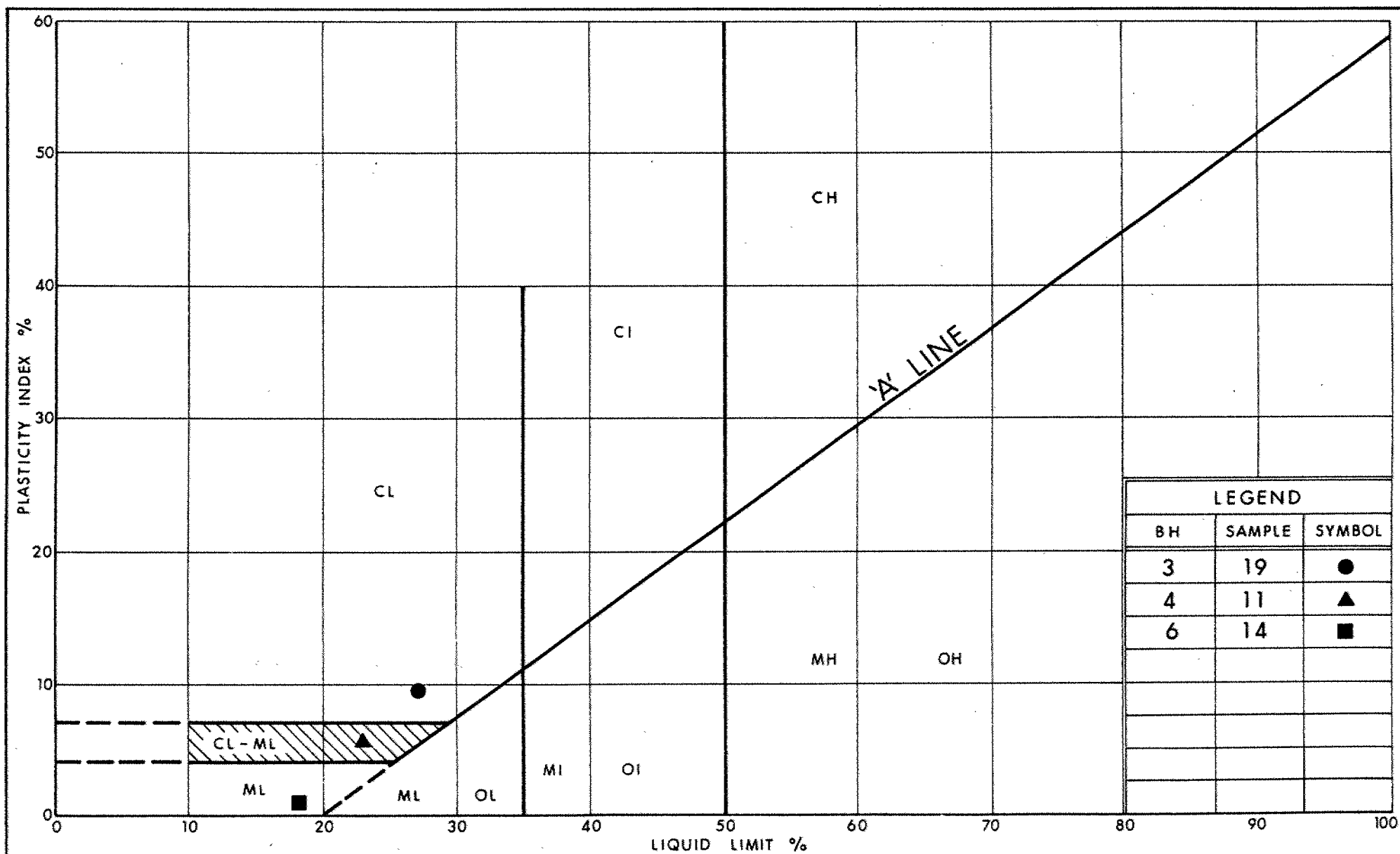


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Communications

GRAIN SIZE DISTRIBUTION
CLAYEY SILT

FIG No 8

W P 88-78-18



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PLASTICITY CHART Lower SILT TO SILTY CLAY

FIG No 9

W P 88-78-18

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND

GRAVEL

Fine

Medium

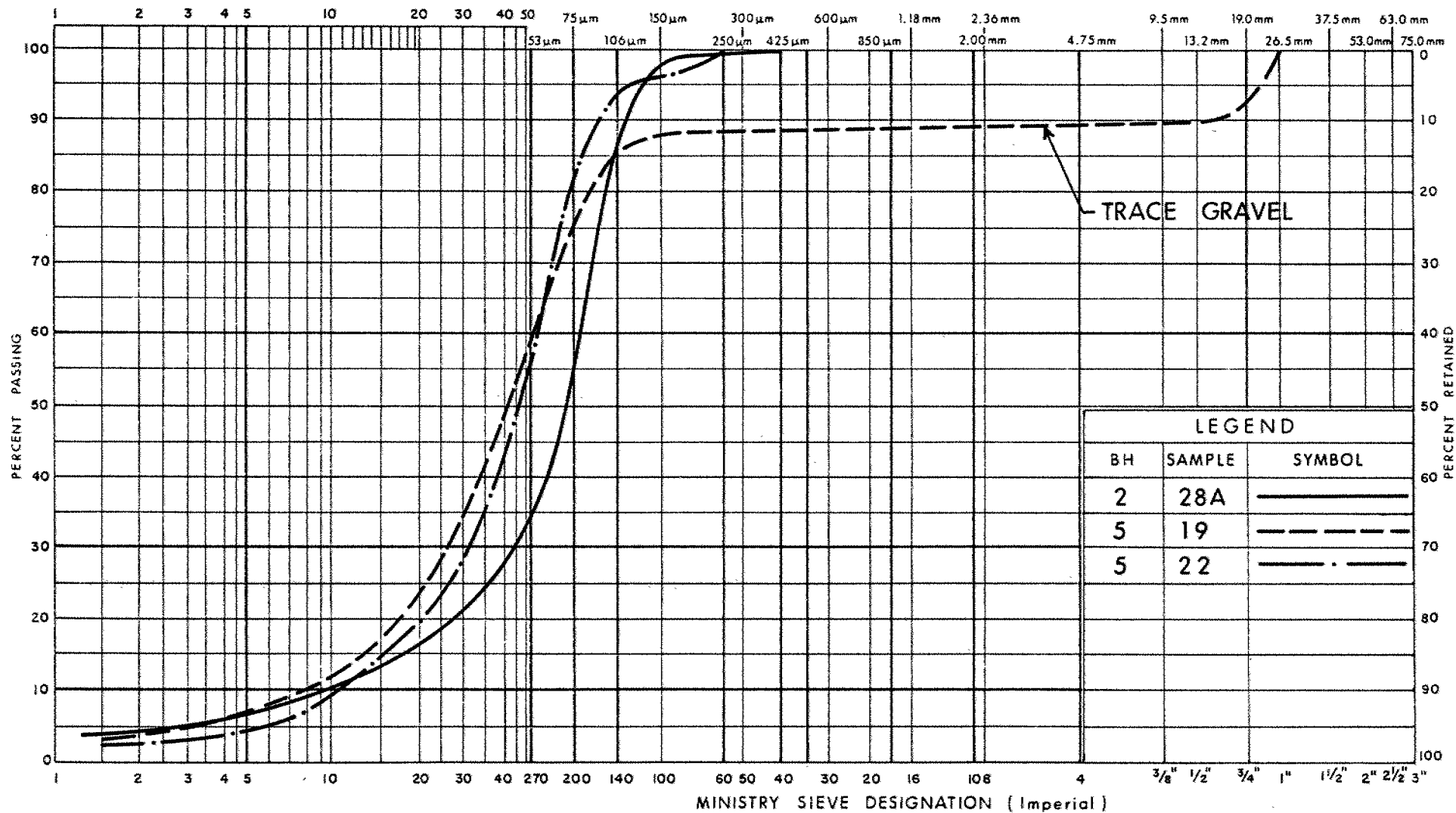
Coarse

Fine

Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



GRAIN SIZE DISTRIBUTION
Lower SILT, SOME SAND

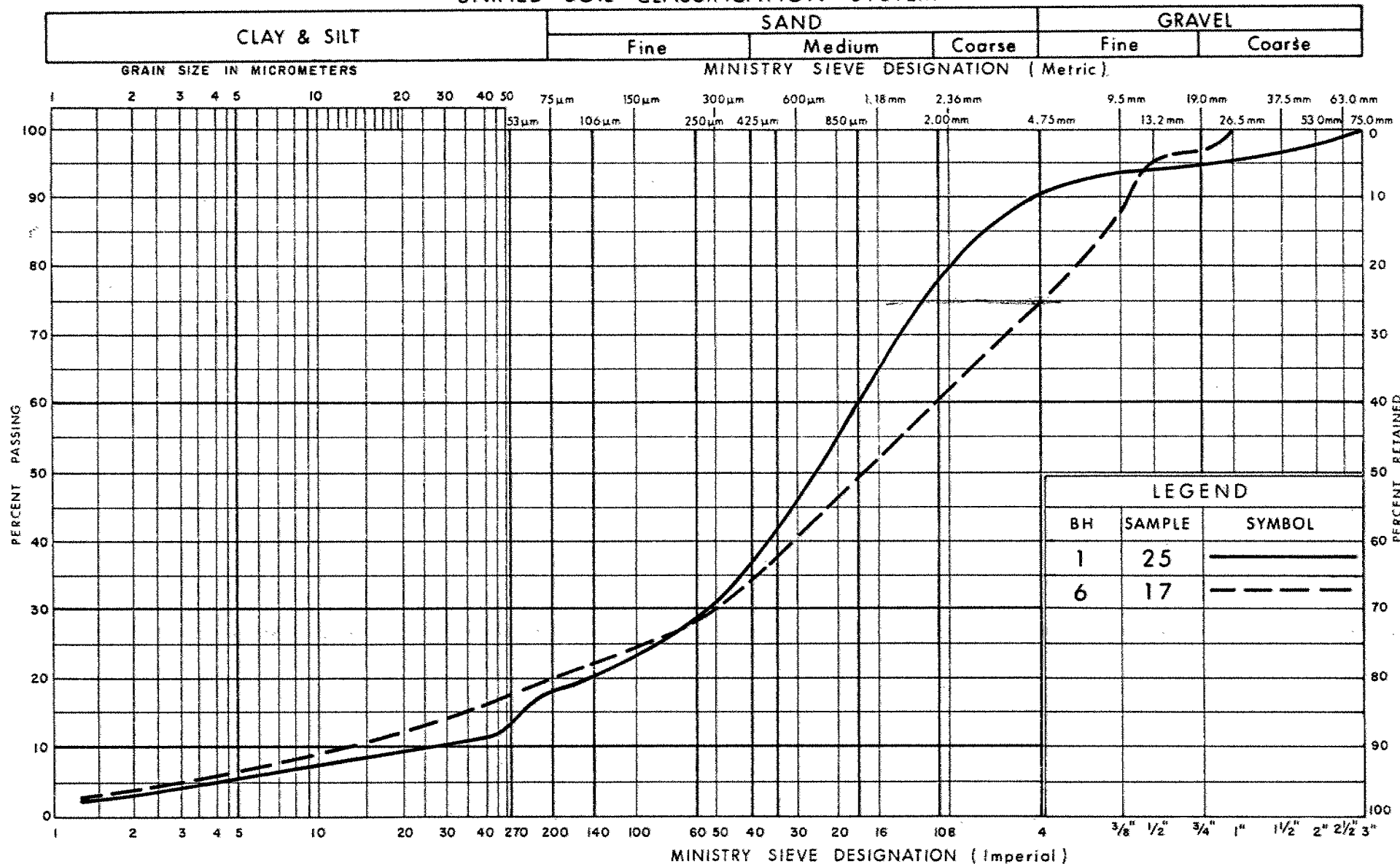
FIG No 10a

W P 88-78-18



Ministry of
Transportation and
Communications

UNIFIED SOIL CLASSIFICATION SYSTEM



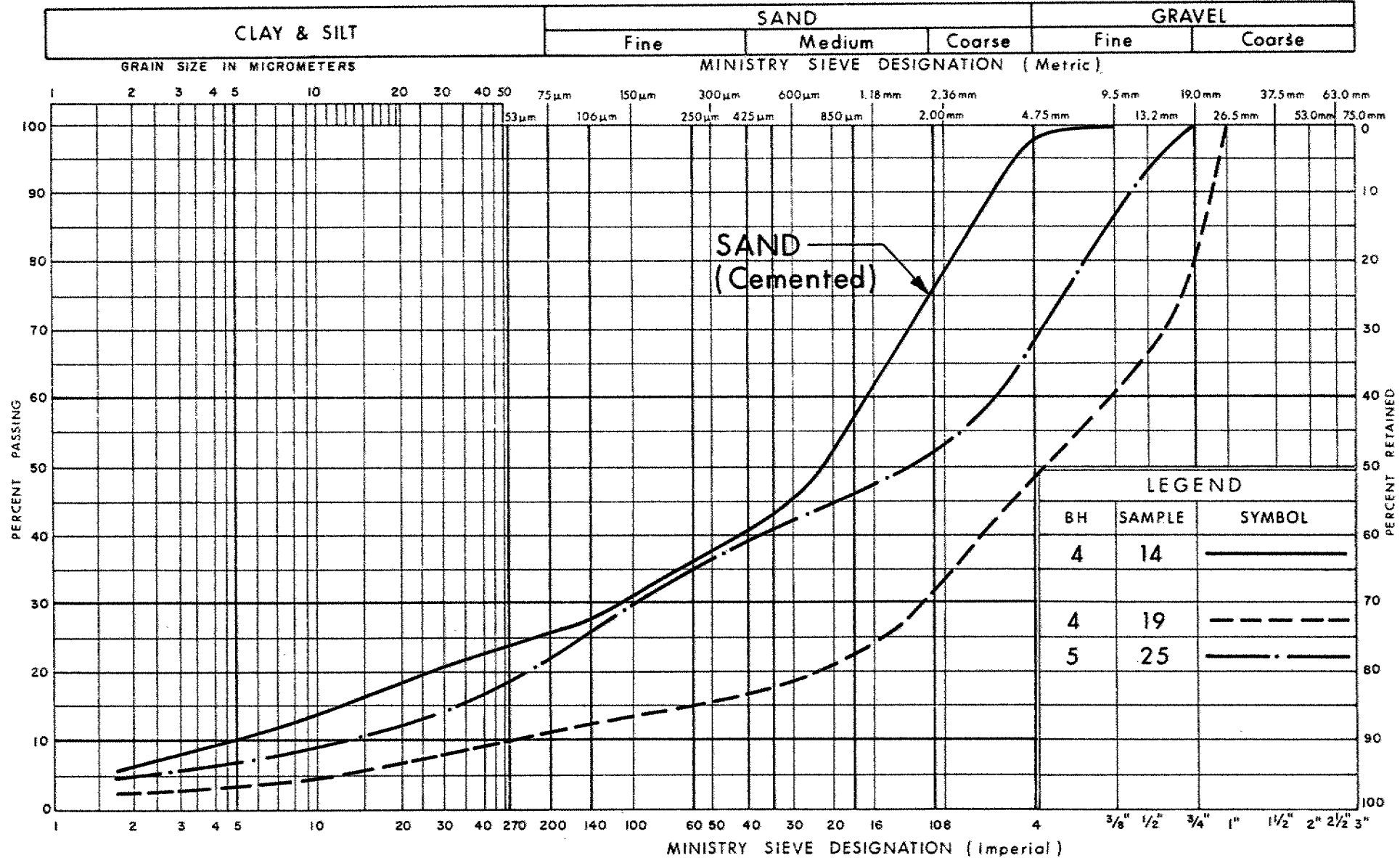
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Communications

GRAIN SIZE DISTRIBUTION
SILTY SAND TO GRAVELLY SAND

FIG No 10b

W P 88-78-18

UNIFIED SOIL CLASSIFICATION SYSTEM



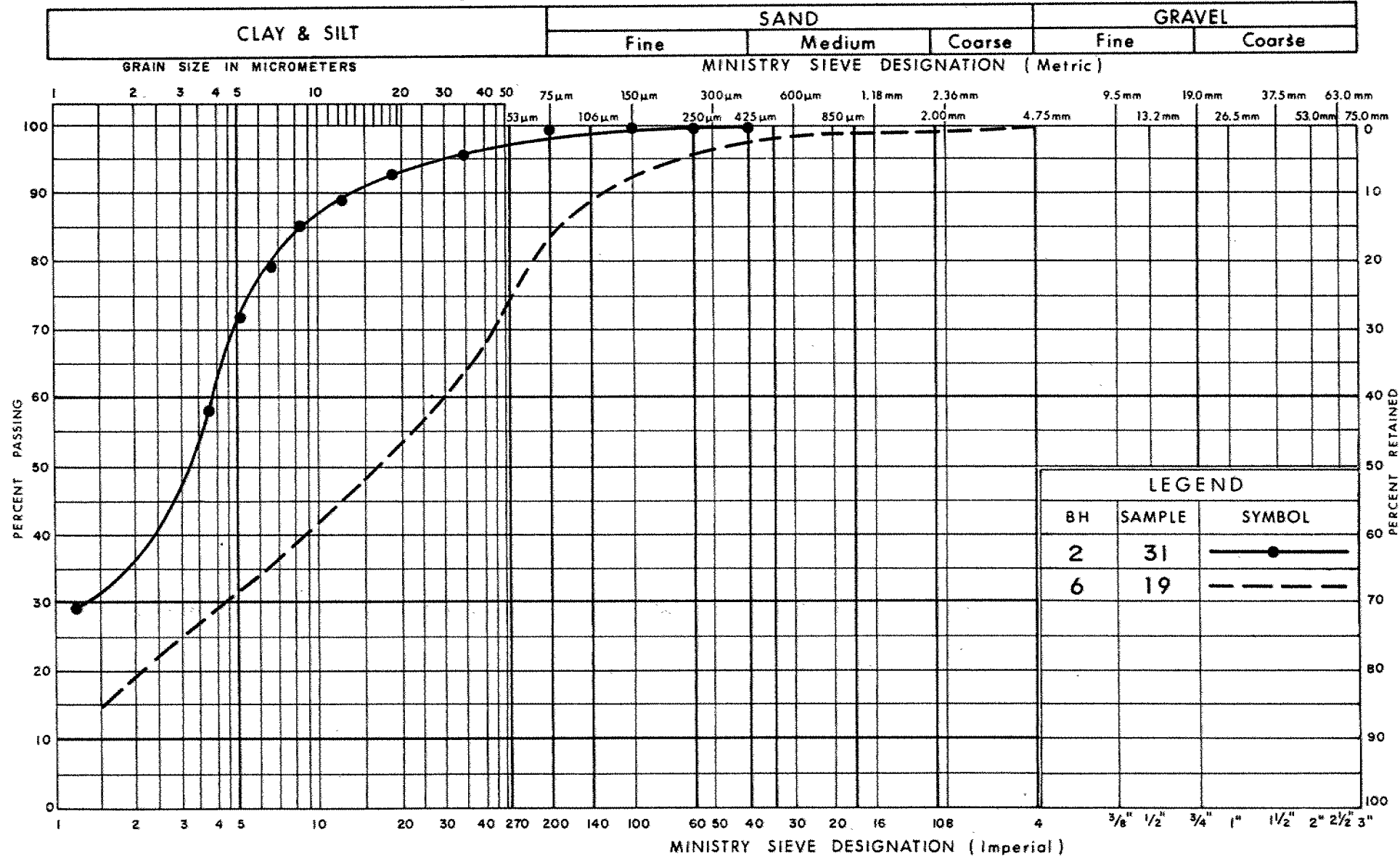
Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION
SANDY GRAVEL SOME SILT TO SILTY SAND

FIG No 10c

W P 88-78-18

UNIFIED SOIL CLASSIFICATION SYSTEM

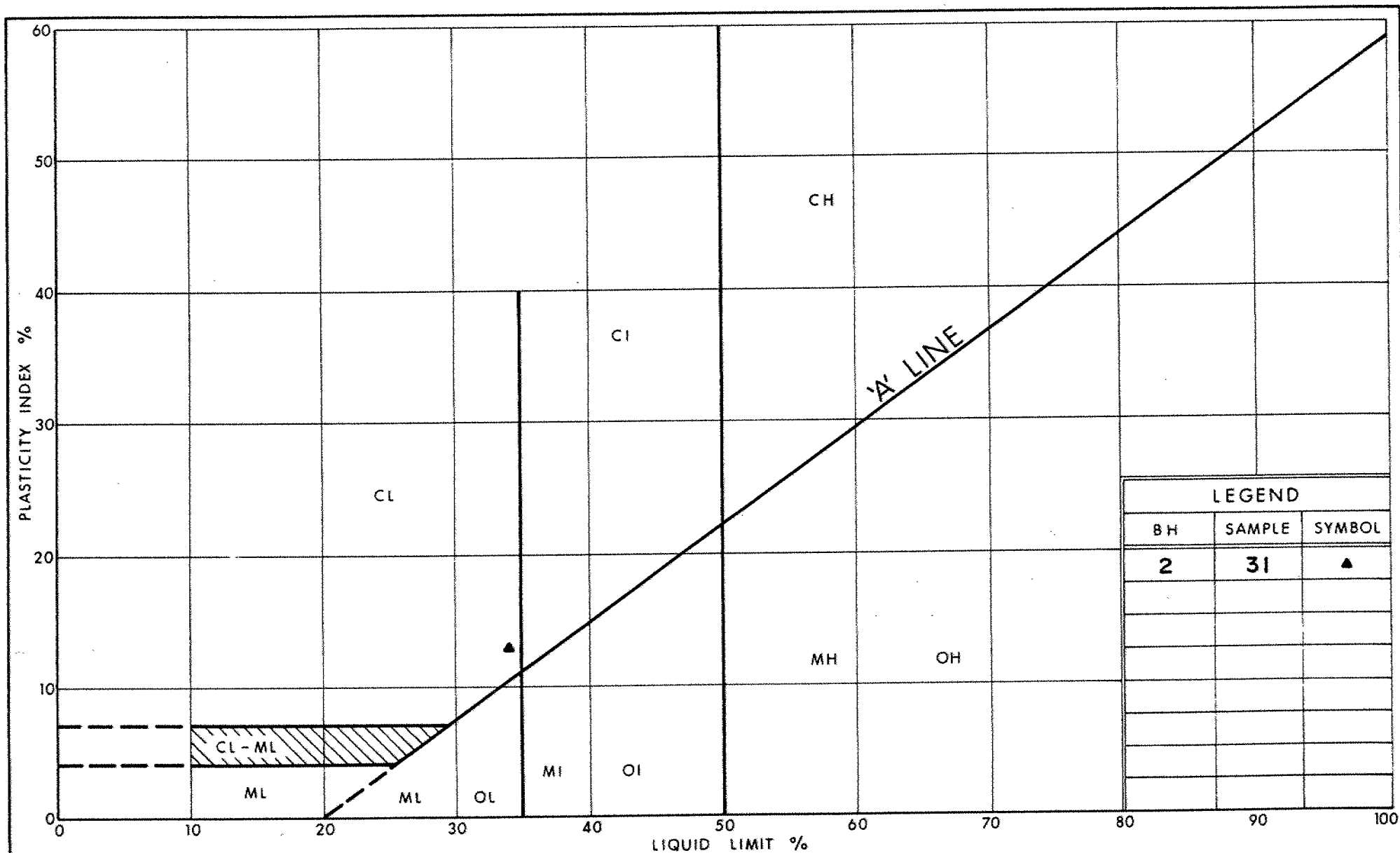


Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION
CLAYEY SILT
INTERLAYERED IN LOWER COHESIONLESS SOILS

FIG No 11

W P 88-78-18



Ministry of
Transportation and
Communications

PLASTICITY CHART CLAYEY SILT

INTERLAYERED IN LOWER COHESIONLESS SOILS

FIG No 12

W P 88-78-18

RECORD OF BOREHOLE No 1

1 OF 2

METRIC

W.P. 88-78-18

LOCATION Co-ords: N 4 847 645 E 298 404

ORIGINATED BY HCO

DIST 6 HWY 407

BOREHOLE TYPE Wash Boring, Hollow Stem Augers, Solid Stem Augers

COMPILED BY RWR

DATUM Geodetic

DATE June 20 to 23, 1983

CHECKED BY HCO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100					
159.5	Ground Surface													
159.1	Topsoil - Brown, Sandy													
0.4			1	SS	15		158							
			2	SS	18									
	Brown		3	SS	18									
	Gray		4	SS	15		156							
	Sandy Silt (Compact)		5	SS	11									
	with interbedded layers of		6	SS	14		154							
	Clayey Silt		7	SS	15									
152.2							152							
7.3			8	SS	4								W _n =44%	
	Clayey Silt		9	SS	WR		150							
	with random layers/nodules		10	SS	PH		148							
	of silt,		11	TW	PM		146						18.5	
	occasional Sand and Gravel		12	SS	1									
	pockets		13	SS	WR		144							
	Firm to Stiff		14	TW	PH		142						21.2	
	Gray		15	SS	13		140							
143.2			16	SS	21		138							
16.3			17	SS	14		136							
	Clayey Silt, Some Sand, Trace		18	TW	PH		134						21.4	
	Gravel		19	SS	10		132							
	(Glacial Till)		20	SS	22									
	Stiff		21	SS	27									
	Gray		22	SS	11		130							
133.9														
25.6														
	Clayey Silt													
	Stiff to Very Stiff													
	Gray													
129.3														
30.2														

Continued

+3, x5: Numbers refer to
Sensitivity

20
15-5 (%) STRAIN AT FAILURE
10

Continued

RECORD OF BOREHOLE No 1

2 OF 2

METRIC

W.P. 88-78-18 LOCATION Co-ords: N 4 847 645 E 298 404 ORIGINATED BY HCO
 DIST. 6 HWY 407 BOREHOLE TYPE Wash Boring, Hollow Stem Augers, Solid Stem Augers COMPILED BY RWR
 DATUM Geodetic DATE June 20 to 23, 1983 CHECKED BY HCO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100										
30.5	<p>Continued</p> <p>Compact ----- Very Dense</p> <p>Sandy Silt to Silty Sand Trace Gravel, Occasional Sand Seams</p>		23	SS	23																	
			24	SS	16																	
			25	SS	131																	
			26	SS	94	/23cm																
			27	SS	127	/23cm																
			28	SS	109																	
			29	SS	162	/27cm																
119.4																						
40.1	End of Borehole																					
	<p>** Water Level Elev. 157.5 m July 7 /83 Piezometer dry Aug. 24 /83</p> <p>*** Water Level Elev. 141.4 m Aug. 4 /83</p>																					
	<p>* GROUND WATER CONDITIONS</p> <table border="1"> <tr> <th>PIEZO. NO.</th> <th>GROUND WATER ELEVATION [Metres]</th> </tr> <tr> <td>1</td> <td>157.5</td> </tr> <tr> <td>2</td> <td>141.4</td> </tr> </table>	PIEZO. NO.	GROUND WATER ELEVATION [Metres]	1	157.5	2	141.4															
PIEZO. NO.	GROUND WATER ELEVATION [Metres]																					
1	157.5																					
2	141.4																					

RECORD OF BOREHOLE No 2

2 OF 2

METRIC

W.P. 88-75-18 LOCATION Co-ords: N 4 847 575 E 298 428 ORIGINATED BY HCO
 DIST 6 HWY 407 BOREHOLE TYPE Wash Boring COMPILED BY EFO
 DATUM Geodetic DATE July 7-11, 1983 CHECKED BY HCO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40						60
128.0	Continued		23	SS	33										
30.5			Limestone Cobbles with Silty Sand	24	SS	100	/15cm								
			Dense to Very Dense, Gray	25	WS	-									
125.3	Silt Some Sand and Gravel, Trace Clay, Occasional Sand layers Very Dense, Gray		26	SS	48										
33.2				27	SS	97	/23cm								
				28	SS	63									
				29	SS	60									
				30	SS	33									
118.7	Clayey Silt layered Hard, Gray		31	SS	117										
39.8				32	SS	91									
116.4	Sand and Silt Very Dense, Gray		33	SS	100	/8cm									
42.1															
114.1															
44.4	End of Borehole														
	** Water Level Elev. 140.7 m Aug 24 /83.														
	* GROUND WATER CONDITIONS PIEZO. NO. 1 GROUND WATER ELEVATION (Metres) 140.7														

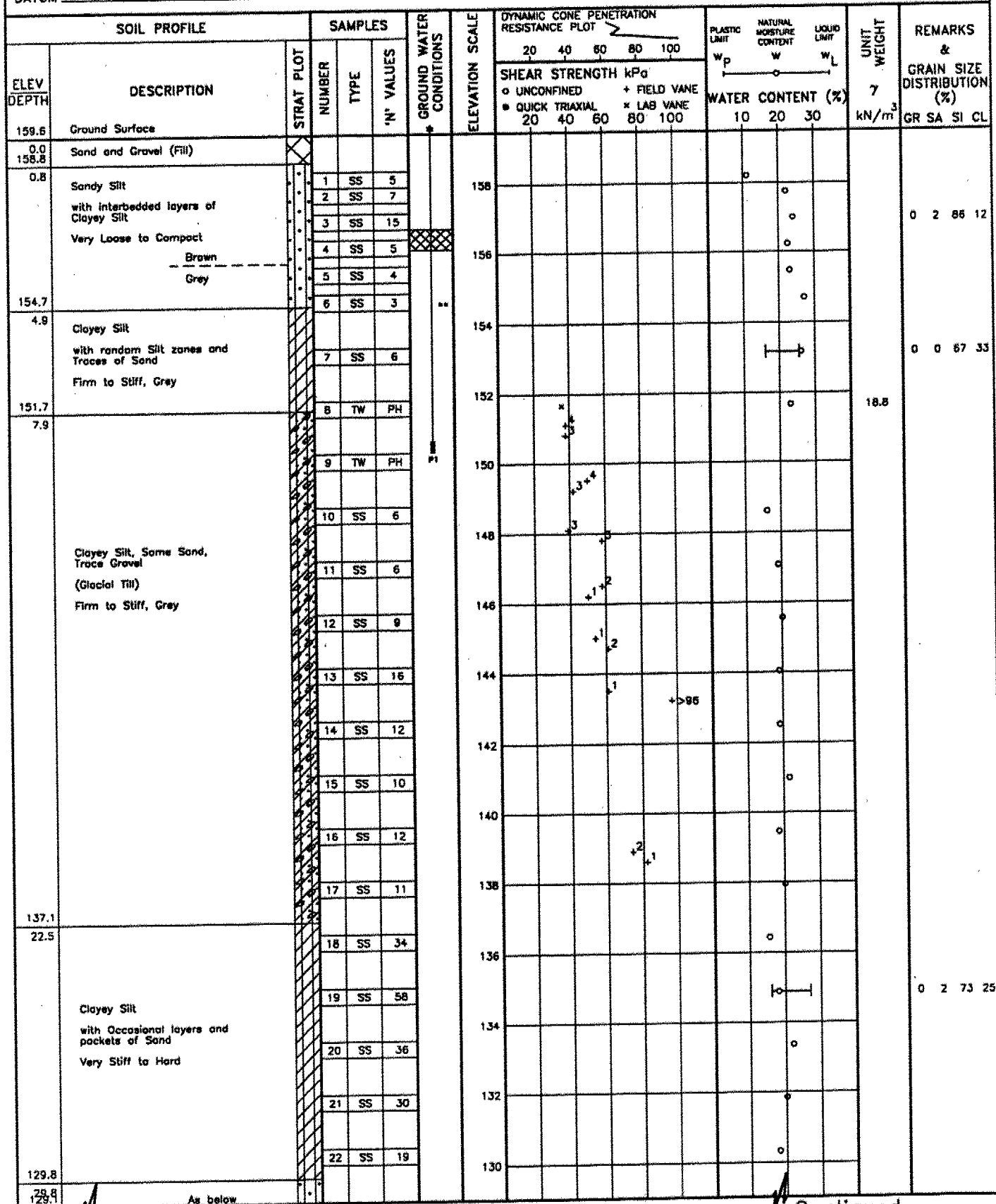
RECORD OF BOREHOLE No 2A										1 OF 1		METRIC					
W.P. 88-78-18		LOCATION Co-ords: N 4 847 574 E 298 428				ORIGINATED BY HCO											
DIST 6 HWY 407		BOREHOLE TYPE Solid Stem Auger				COMPILED BY EFO											
DATUM Geodetic		DATE July 12, 1983				CHECKED BY HCO											
SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		GROUND WATER CONDITIONS	20 40 60 80 100						20 40 60 80 100	WATER CONTENT (%)	10 20 30	7
158.5	Ground Surface																
0.0	Sand and Silt						158										
							156										
							154										
153.0																	
5.5	End of Borehole																
	<p>Note: Unsampler borehole. Soil stratigraphy inferred from auger cuttings.</p> <p>1983 08 24</p> <p>* GROUND WATER CONDITIONS</p> <table border="1" style="width:100%; border-collapse: collapse;"> <tr> <th>PIEZO. NO.</th> <th>GROUND WATER ELEVATION (Metres)</th> </tr> <tr> <td>1</td> <td>1.1</td> </tr> </table>	PIEZO. NO.	GROUND WATER ELEVATION (Metres)	1	1.1												
PIEZO. NO.	GROUND WATER ELEVATION (Metres)																
1	1.1																

RECORD OF BOREHOLE No 3

1 OF 2

METRIC

W.P. 88-78-18 LOCATION Co-ords: N 4 847 617 E 298 429 ORIGINATED BY HCO
 DIST 6 HWY 407 BOREHOLE TYPE Hollow Stem Auger, Wash Boring COMPILED BY EFO
 DATUM Geodetic DATE July 12-13, 1983 CHECKED BY HCO



30.5 Continued

+3, x⁵: Numbers refer to
Sensitivity

20
15-5 (%) STRAIN AT FAILURE
10

Continued

RECORD OF BOREHOLE No 3

2 OF 2

METRIC

W.P. 88-78-18 LOCATION Co-ords: N 4 847 617 E 298 429 ORIGINATED BY HCO
 DIST 6 HWY 407 BOREHOLE TYPE Hollow Stem Auger, Wash Boring COMPILED BY EFO
 DATUM Geodetic DATE July 12-13, 1983 CHECKED BY HCO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT 7 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40						60	80	100
129.1	Continued Sandy Silt to Silty Sand Compact to Very Dense		23	SS	19												
30.5																	
123.0	Clayey Silt Hard		24	SS	48												
36.6																	
121.0	Sandy Silt, Some Gravel Very Dense		25	SS	20												
38.6																	
119.5	End of Borehole ** Water Level Elev. 154.9 m Aug. 24 / 83		26	SS	69												
40.1																	
			27	SS	67												
			28	SS	49												
			29	SS	76												

* GROUND WATER CONDITIONS

PIEZO. NO.	GROUND WATER ELEVATION (Metres)
1	154.9

RECORD OF BOREHOLE No 4

1 OF 2

METRIC

W.P. 88-78-18 LOCATION Co-ords: N 4 847 569.3 E 298 444.7 ORIGINATED BY IR
DIST 6 HWY 407 BOREHOLE TYPE HS Auger and Washboring COMPILED BY HS
DATUM Geodetic DATE 84 07 23 CHECKED BY JP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								20 40 60 80 100								10 20 30		
159.4	Ground Surface																	
0.0	Cloyey Silt with random zones of silt Firm to Stiff, Brown						158											
157.3			1	SS	5		156								23 66 9 2			
2.1																		
	Sand, Trace Silt Compact, Grey		2	SS	16		156											
154.5			3	SS	16		154											
4.9																		
	Cloyey Silt to Silty Clay Firm, Grey		4	SS	8		152											
			5	SS	9		150											
			6	SS	5		148											
							146											
			7	SS	5		144											
143.2																		
16.2	Cloyey Silt, some Sand, trace Gravel (Glacial Till) Stiff to Very Stiff		8	SS	8		142											
			9	SS	13		140											
							138											
							136											
			10	SS	15		134											
133.5																		
25.9	Cloyey Silt trace sand, occasional cobbles very stiff to hard						132											
128.9			11	SS	78		130											

30.5

Continued

+3, x5 Numbers refer to
Sensitivity

20
15-5 (%) STRAIN AT FAILURE
10

Continued

METRIC

1. CHECKED BY JP

+3, x5: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 5

1 OF 2 METRIC

W.P. 88-78-18 LOCATION Co-ords: N 4 847 613 E 294 415 ORIGINATED BY IR
 DIST 6 HWY 407 BOREHOLE TYPE Hollow Stem Auger and Wash Boring COMPILED BY HS
 DATUM Geodetic DATE July 25, 1984 CHECKED BY TS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT 7 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100					
159.3	Ground Surface						159							
0.0	Silt with/some Sand Compact, Brown		1	SS	18		157							0 12 84 4
			2	SS	10		155							0 35 64 1
154.7			3	SS	8		153							
4.6	Clayey Silt with interbedded silt zones and nodules		4	SS	5		151							
152.0	Firm		5	SS	3		149							
7.3	Clayey Silt some Sand, trace Gravel (Glacial Till)		6	SS	5		147							
	Firm Stiff		7	SS	4		145							
			8	SS	11		143							
			9	SS	10		141							
			10	SS	14		139							
	Hard		11	SS	16		137							
			12	SS	34		135							
135.2			13	SS	45		133							
24.1	Clayey Silt trace Sand, trace Gravel Very Stiff to Hard		14	SS	30		131							
			15	SS	22		129							
128.8														

30.5

Continued

+3, x5: Numbers refer to
Sensitivity

20
15-5 (%) STRAIN AT FAILURE
10

Continued

RECORD OF BOREHOLE No 5

2 OF 2

METRIC

W.P. 88-78-18 LOCATION Co-ords: N 4 847 613 E 294 415 ORIGINATED BY JR
DIST 6 HWY 407 BOREHOLE TYPE Hollow Stem Auger and Wash Boring COMPILED BY HS
DATUM Geodetic DATE July 25, 1984 CHECKED BY TS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT 7 KN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
								20 40 60 80 100										
128.8	Continued		16	SS	76													
30.5																		
					17	SS	10											
					18	SS	212											
					19	SS	157											
					20	SS	38											
			21	SS	116													
114.5			22	SS	95													
44.8	Silty Sand to Sandy Gravel some Silt Very Dense		23	SS	185	/23cm												
					24	SS	98	/23cm										
					25	SS	186	/23cm										
110.3			26	SS	185	/18cm												
49.0	End of Borehole																	
1985 04 25 * GROUND WATER CONDITIONS																		
PIEZO. NO.		GROUND WATER ELEVATION (Metres)																
1		157.8																

RECORD OF BOREHOLE No 6

1 OF 2

METRIC

W.P. 88-78-18 LOCATION Co-ords: N 4 847 664 E 294 398 ORIGINATED BY IR
DIST 6 HWY 407 BOREHOLE TYPE Hollow Stem Auger and Wash Boring COMPILED BY HS
DATUM Geodetic DATE July 31, 1984 CHECKED BY TS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
159.4	Ground Surface																
0.0	Silt, trace Sand to Sandy Silt Compact, Brown		1	SS	17		159										0 13 86 1
			2	SS	17		157										
			3	SS	12		155										0 4 90 6
153.9			4	SS	6		153										
5.5	Silty Clay with random zones of Silt and occasional sand pockets Soft to Firm		5	SS	3		151										
			6	SS	3		149										
			7	SS	9		147										
146.0			8	SS	18		145										
13.4	Clayey Silt, some Sand, some Gravel (Glacial Till) Stiff to Very Stiff		9	SS	14		143										
			10	SS	*		141										
			11	SS	25		139										
			12	SS	*		137										
			13	SS	*		135										
133.8			14	SS	14		133										
25.6	Clayey Silt with interbedded layers of Silt Stiff, Grey						131										
							129										

Continued

Continued

+3, x5: Numbers refer to
Sensitivity

20
15-5 (%) STRAIN AT FAILURE
10

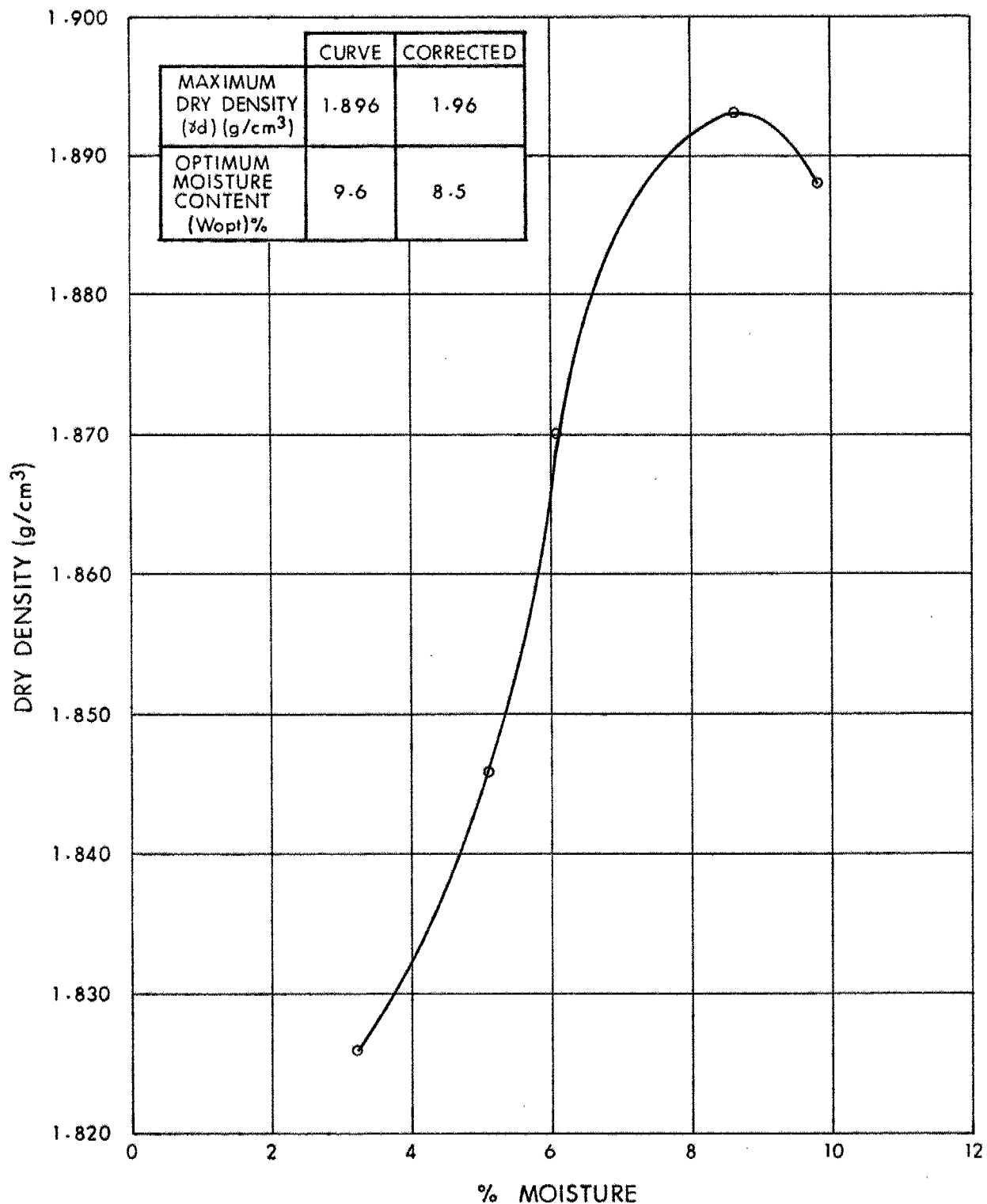
RECORD OF BOREHOLE No 6

2 OF 2

METRIC

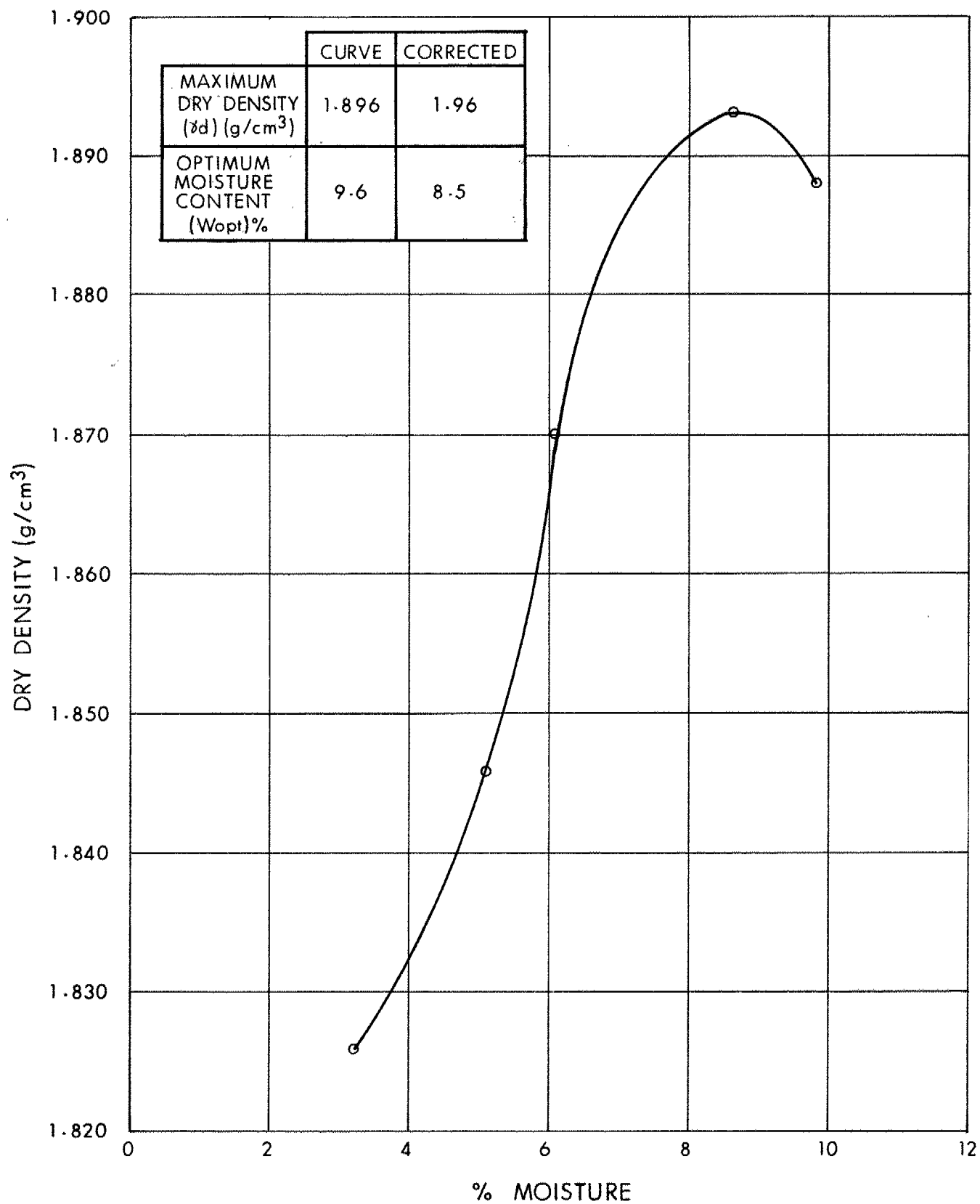
W.P. 88-78-18 LOCATION Co-ords: N 4 847 664 E 294 398 ORIGINATED BY JR
DIST 6 HWY 407 BOREHOLE TYPE Hollow Stem Auger and Wash Boring COMPILED BY HS
DATUM Geodetic DATE July 31, 1984 CHECKED BY TS

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT 7 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20						40	60	80	100
30.8	Continued																
	Silty Sand to Gravelly Sand some Silt		15	SS	100												
	Clayey Silt		16	SS	67												
	Very Dense		17	SS	101								26 55 15 4				
121.6			18	SS	185	/20cm											
37.8	Silt with occasional Clayey Silt seams		19	SS	184								0 16 64 20				
119.3	Very Dense		20	SS	174	/28cm											
40.1	End of Borehole																
<p>1984 08 12 * GROUND WATER CONDITIONS</p> <table border="1"> <tr> <td>PIEZO. NO.</td> <td>GROUND WATER ELEVATION (Metres)</td> </tr> <tr> <td>1</td> <td>dry</td> </tr> </table>														PIEZO. NO.	GROUND WATER ELEVATION (Metres)	1	dry
PIEZO. NO.	GROUND WATER ELEVATION (Metres)																
1	dry																

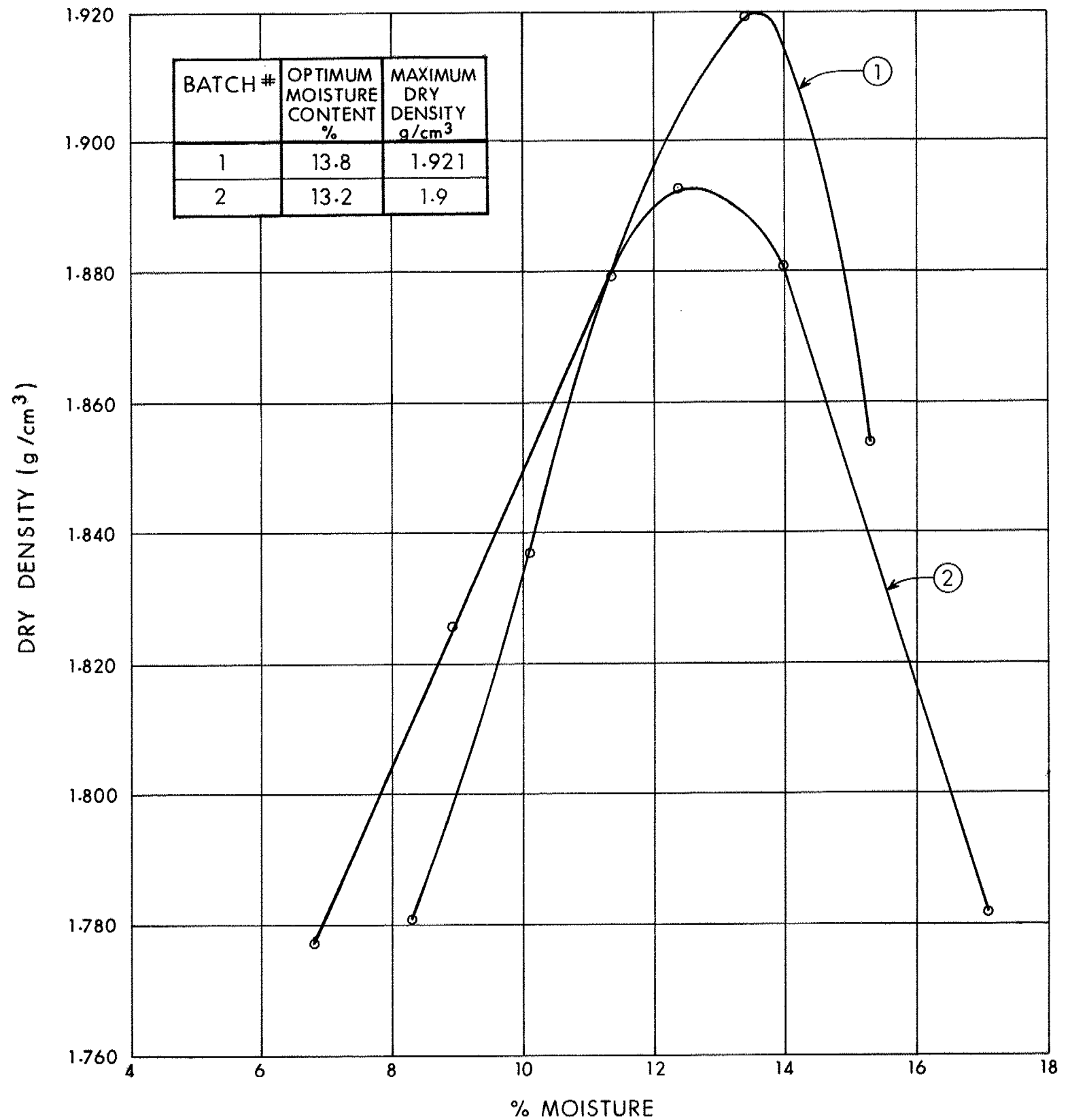


MOISTURE-DENSITY RELATIONSHIP
SAND, TRACE SILT, TRACE GRAVEL

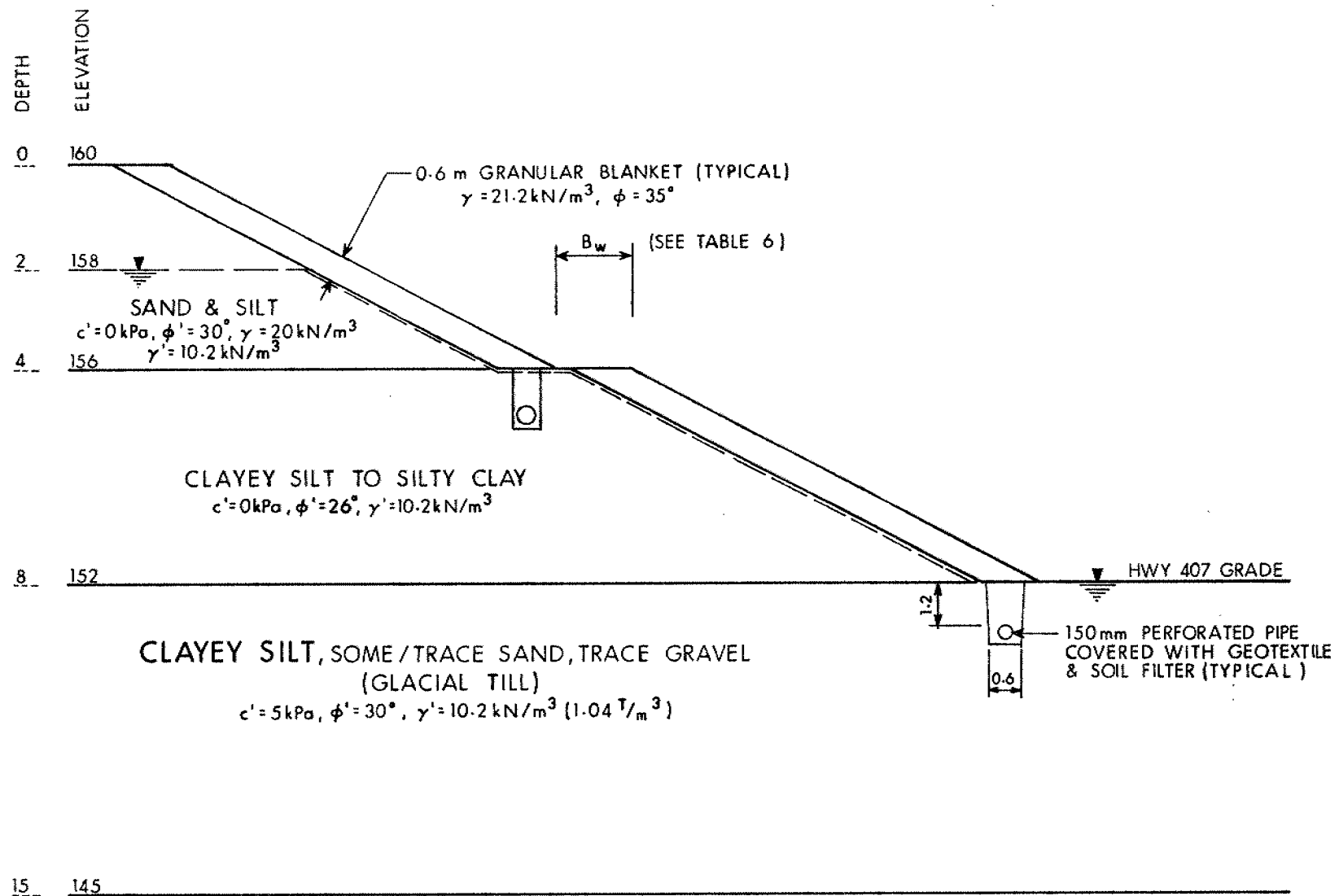
FIG - 3
WP 141-87-00(A)



MOISTURE-DENSITY RELATIONSHIP
SAND, TRACE SILT, TRACE GRAVEL

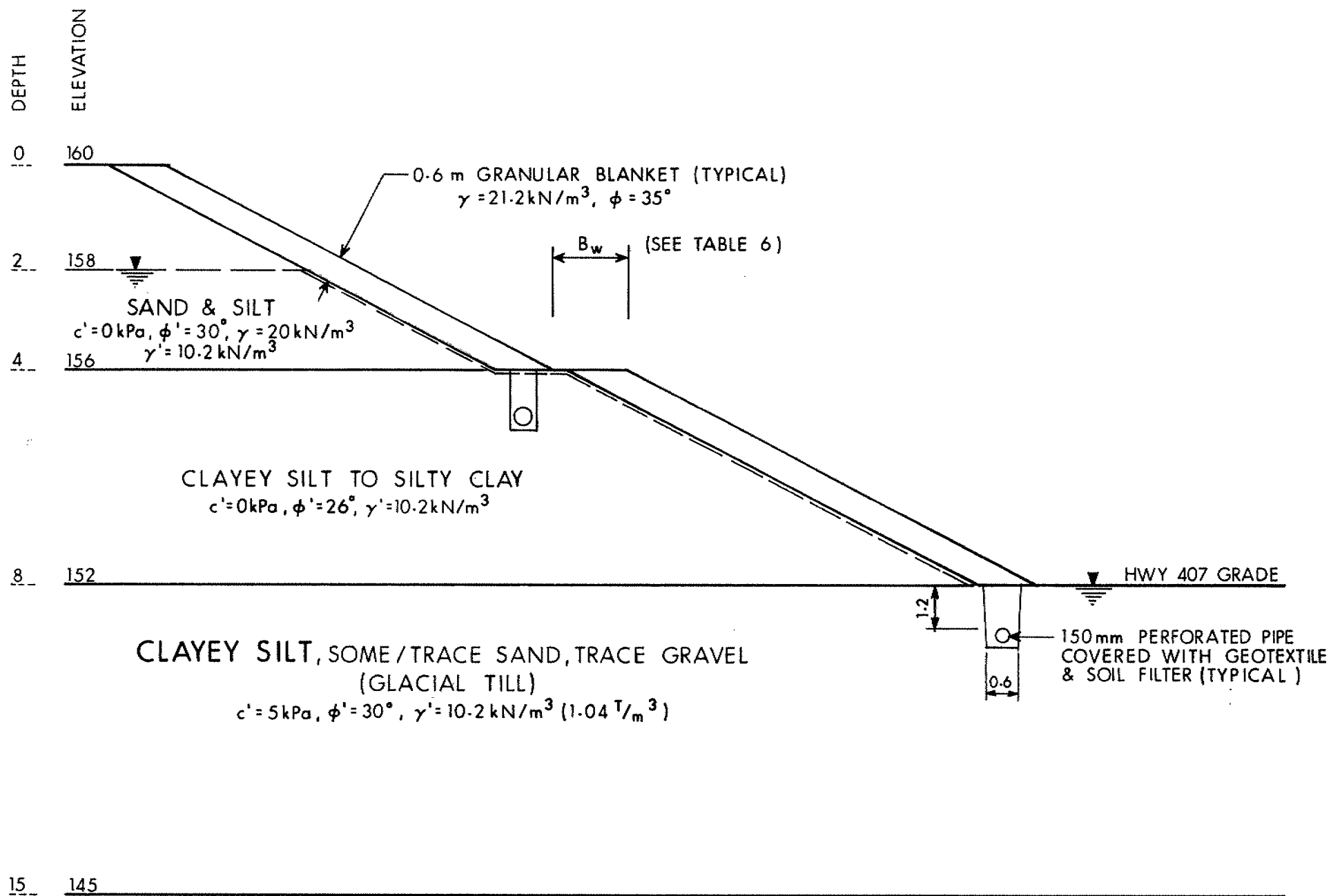


MOISTURE-DENSITY RELATIONSHIP
CLAYEY SILT TO SILTY CLAY
WITH RANDOM NODULES/SEAMS OF SILT



EXCAVATED SLOPE STABILITY DESIGN

FIG - 14
WP 141-87-00(A)



EXCAVATED SLOPE STABILITY DESIGN

FIG - 14
WP 141-87-00(A)

GRAVITY DRAINAGE SCHEME METHOD A

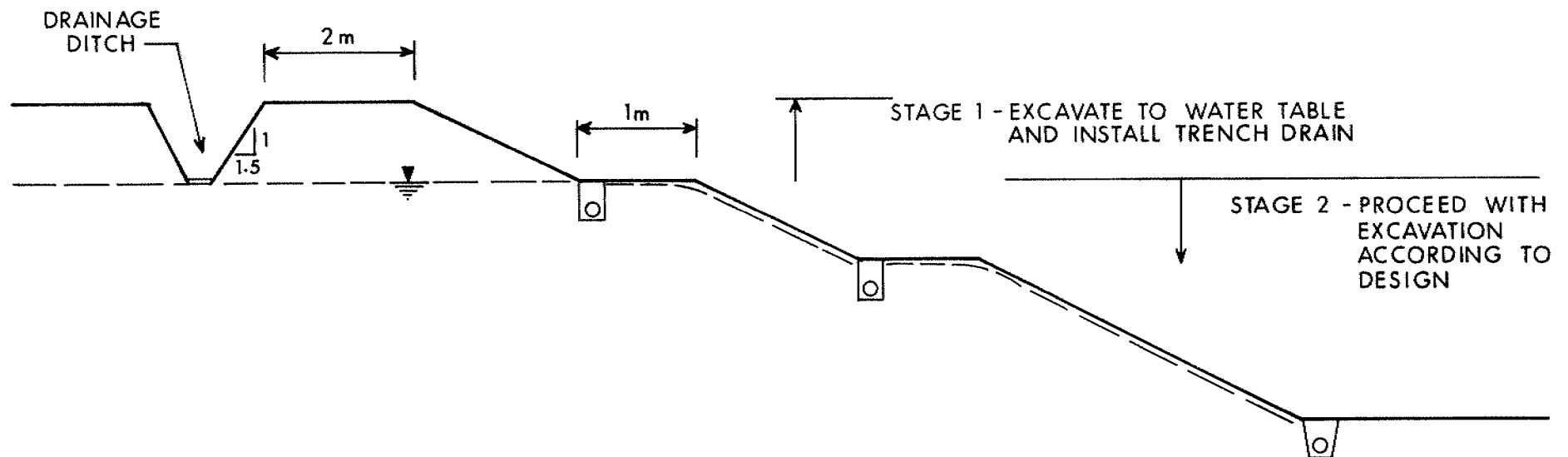


FIG -16A
WP 141-87-00 (A)

CONT No
WP No 88-78-18

ISLINGTON AVE UNDERPASS

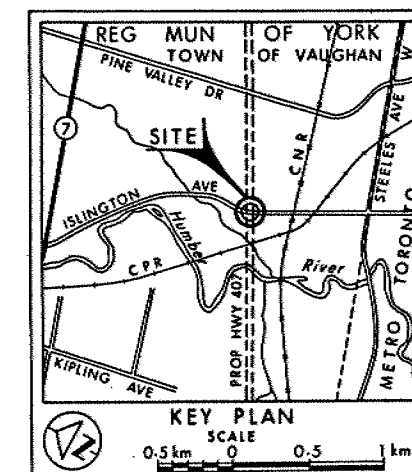
BORE HOLE LOCATIONS & SOIL STRATA



SHEET

METRIC

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



LEGEND

- ◆ Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊕ Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- W L at time of investigation
1983 07 and 1983 08
For BH 4; 84 07 23 For BH 5; 85 04 25
- W L in Piezometer
- Piezometer

No	ELEVATION	CO-ORDINATES NORTH	EAST
1	159.5	4 847 645	298 404
2	158.5	4 847 575	298 428
2A	158.5	4 847 574	298 428
3	159.6	4 847 617	298 429
4	159.4	4 847 569.3	298 444.7
5	159.3	4 847 613.4	298 415.2
6	159.4	4 847 664.4	298 398.6

NOTE

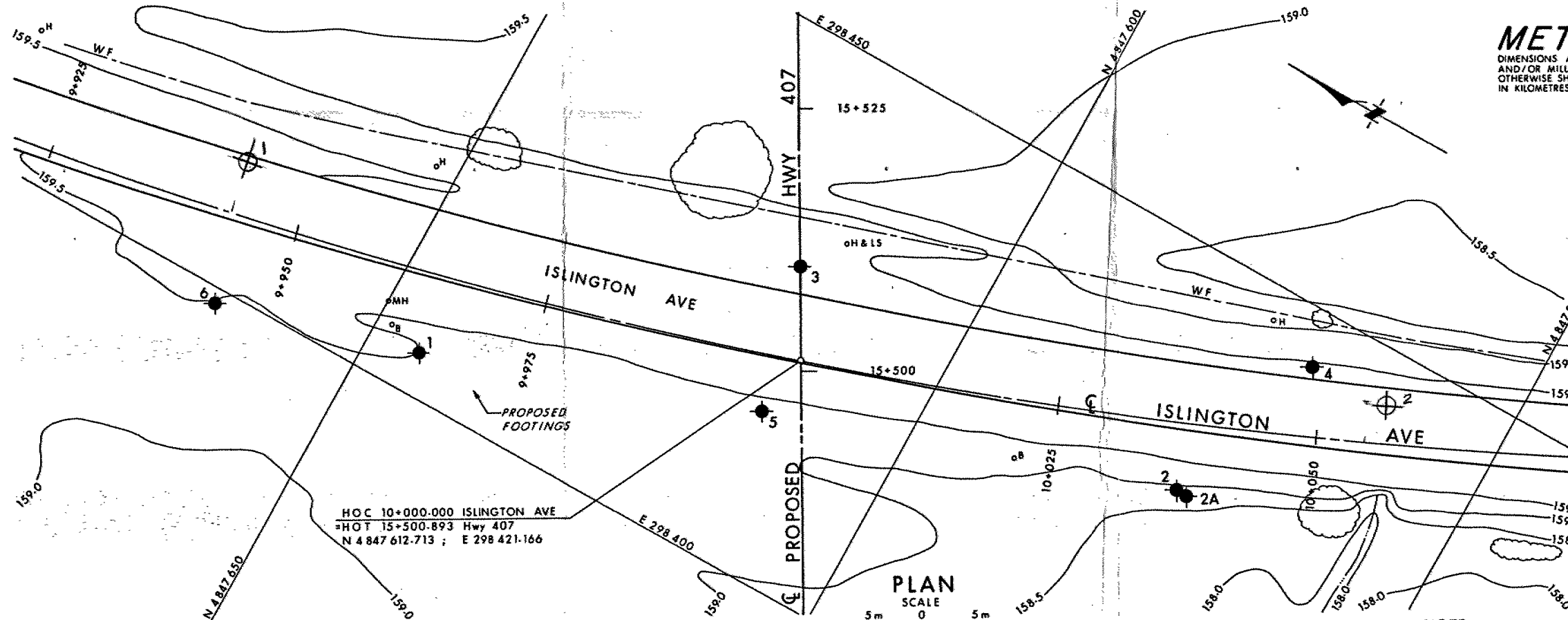
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

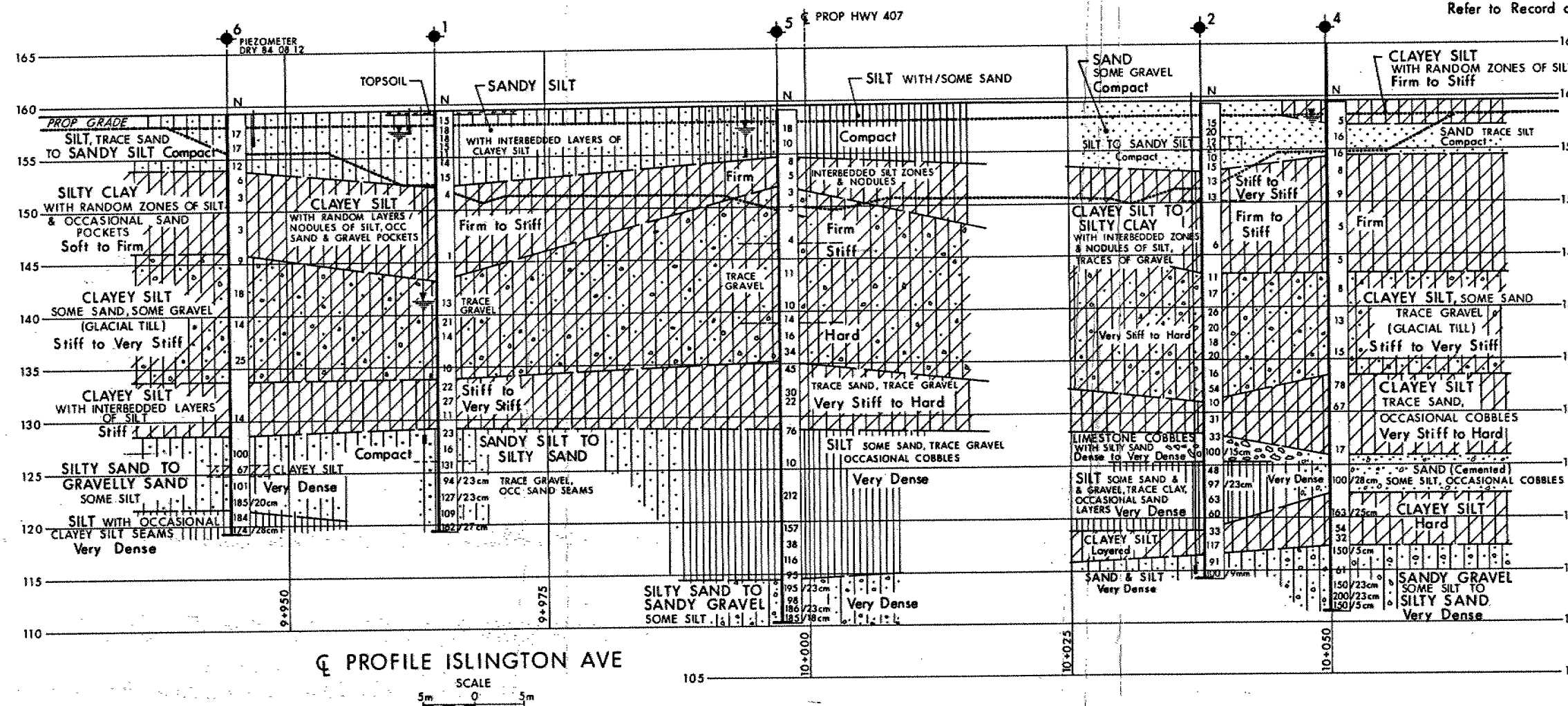
REV	DATE	BY	DESCRIPTION
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Geocres No 30M13-61

HWY No 407	DIST 6
SUBMD HS [CHECKED]	DATE 1985 05 15
DRAWN [CHECKED]	SITE 37-73-1120
	OWG 2



NOTE:
Subsoil Information for BH 2A & 3
Refer to Record of Borehole



PROFILE ISLINGTON AVE

SCALE
5m 0 5m

M E M O R A N D U M



To: T.C. Tam
Construction Services Engineer
Approvals Section
7th Floor, Atrium Tower

Date: May 18, 1993

From: Foundation Design Section
Room 315, Central Bldg.

Tel: 235-3731
Fax: 235-5240

Re: Contract 92-40 ✓
Foundation Report
Hwy 407 - Islington Ave. U'Pass
District 6, Toronto

We have reviewed the above report and find that the contents of the report do not confirm the adequacy of the falsework foundation. There are several conditions listed in the report and only if all conditions are satisfied, the foundation will be suitable to support the expected loads from the mud sills.

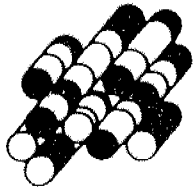
It should be the geotechnical consultant's mandate to verify and comment whether the implied conditions are actually met by the contractor.

Please contact our office if you need elaboration on the contents of this memo.

A handwritten signature in cursive script, appearing to read "B. Iyer", with a horizontal line underneath.

B. Iyer, P. Eng.
Senior Foundation Engineer

BI/jb



Terraprobe Testing Ltd.

Construction and Materials Inspection and Testing

2565 Steeles Ave. E.
Brampton, Ontario
L6T 4L6
(416) 793-2650
FAX: 793-2655

May 5, 1993

Our File No. 929189

Graham Bros. Construction Ltd.
290 Clarence Street
Brampton, Ontario
L6W 1T4

Attention: Mr. Dave Weltz

**RE: FALSEWORK FOUNDATION INSPECTION
HWY 407 - ISLINGTON AVE. UNDERPASS (MTO 92-40)
VAUGHAN, ONTARIO**

Dear Sir:

In accordance with the special provision No. 919S02 of the contract, we hereby submit the following Falsework Foundation report.

Our observations together with field sampling and testing from previous site inspections indicated that the native soils at proposed the mudsill elevations were considered to be unsuitable for support of the mudsills for the underpass falsework. As a result of this, we recommended that excavations be carried out over the site, a layer of filter fabric be placed at the base of the excavations and the excavations backfilled and compacted to a minimum of 98 percent Standard Proctor Maximum Dry Density (SPMDD) utilizing OPSS Granular 'A' materials.

It is our understanding that the mudsills for the underpass falsework are presently founded on imported OPSS Granular 'A' materials at elevations of about 152.4 m adjacent to the north abutment and at about 152.9 m adjacent to the south abutment. Also, the maximum load that is to be applied to any one leg is 70.7 kN.

Also, it is our understanding that the depth of the excavations varied from about 5.5m (18 feet) at the pier locations to about 2.4 m (8 feet) along the spans between the structures, and to about 1.2 m (4 feet) immediately adjacent to the Reco Wall located near to the north and south abutments and that the excavations were backfilled with OPSS Granular 'A' materials.

Based on the above, the Granular "A" materials are considered to be sufficiently thick so as to minimize settlement of the native soils. The expected loads from the mud sills will be adequately reduced as they are transferred through the granulars to the underlying native soils. In order to limit settlement of the underlying soils to less than 12 mm, we recommend that the allowable bearing pressure not exceed 150 kPa (3000 psf). Therefore, the Granular "A" materials, if compacted to 98 percent SPMDD, and the overlying the native soils are considered to be suitable to support the expected loads from the mud sills.

We trust that the foregoing information is sufficient for your present requirements. If you have any questions, please do not hesitate to contact us.

Yours truly,

TERRAPROBE TESTING LTD.



Karl Roechner, P.Eng.



Matthew Julien, P.Eng.

memorandum



To: T.C. Tam
Construction Services Engineer
Approvals Section

Date: 1993 02 19

From: Foundation Design Section
Room 315, Central Building

Subject: Falsework Design - Geotechnical Considerations
Hwy 407 - Islington Avenue
Contract 92-40, W.P. 88-78-18
District 6, Toronto

We have completed our review of the (addendum) report dated 1993 02 15 prepared by the consultant retained by the Contractor. We have the following comments:

1. In the consultant's report, it is stated that, with the excavation from the original grade of 160 m +/- to the present level of 150 m +/-, together with the new load of 120 kPa, the net increase in pressure will be 20 kPa and the resulting total settlement of the underlying soils will be negligible (5 mm).

This would have been the case, provided the excavation was carried out without disturbing/softening the soils underlying the present excavated grade. However, based on the information in the consultant's previous report on this subject, the underlying soils are now in a relatively soft condition. Under the present circumstances, the settlement of the underlying soils would be much larger than 5 mm due to the applied load.

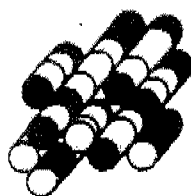
2. As discussed on 1993 02 12, if new fill is placed on frozen ground, the fill should be expected to undergo sudden settlement when the underlying soils thaw.

This memo should be read in conjunction with an earlier memo dated 1993 02 15 issued from this office. Please contact this office if you wish to discuss this topic further.

A handwritten signature in dark ink, appearing to read "B. Iyer", with a horizontal line underneath.

Balu Iyer, P. Eng.
Sr. Foundation Engineer

cc. R. Jeffries
R. Kant



Terraprobe Testing Ltd.

Construction and Materials Inspection and Testing

2565 Steeles Ave. E.
Brampton, Ontario
L6T 4L6
(416) 793-2650
FAX: 793-2655

February 15, 1993

Our File No. 929189

Graham Bros. Construction Limited
290 Clarence Street
Brampton, Ontario
L6W 1T4

Attention: Mr. David Weltz, P. Eng.

**RE: FALSEWORK FOUNDATION INSPECTION
HWY 407 - ISLINGTON AVE. UNDERPASS (MTO 92-40)
VAUGHAN, ONTARIO**

Dear Sir:

This letter presents additional information regarding the proposed foundations for the falsework at the above noted project site. In our letter dated December 22, 1992 (Our File No 929189) we had made recommendations for preparation of the foundation for the falsework. Our letter indicated the following:

- Because of the unstable nature of the native soil, a layer of filter fabric should be placed over the native soils prior to the placement and compaction of the granular materials to form the granular pad. The granular pad placed over the filter fabric may consist of either of the following:
 - (a) a 300 mm thick layer of 50 mm (2 in.) clear stone placed directly on top of the filter fabric over which at least 0.9 m (3 feet) of OPSS Granular 'A' materials are placed and compacted, or
 - (b) a minimum thickness of about 2 m (about 7 feet) OPSS Granular 'A' materials placed and compacted over the filter fabric.

February 15, 19932929189

The granular pad should be extended at least 2 m beyond the falsework envelope and extended a further 3 to 4 m on that portion of the pad where construction traffic will enter on to the granular pad. Also the side slopes of the pad should be at a minimum of about 3 to 1 (horizontal to vertical).

It is our understanding that item (b) above is the preferred option. Therefore, a further evaluation was carried out of the information from the boreholes that were drilled in the vicinity of the Islington Bridge Structure (MTO Contract No. 92-40 - Islington Avenue, WP 88-78-18, Site 37-73-1120). The information indicated that the approximate original ground surface elevation was about 160m. The excavations for the mud sills are to be taken down to an elevation of about 150m. This would result in a net removal of overburden soils of about 10m which is equivalent to about 100 kPa.

The loads that are to be imposed on the foundation are to consist of the weight of the granular pad and the expected weight from the cast-in-place structure on the mud sills, which is equivalent to about 120 kPa. Therefore, the net increase in pressure to the underlying native soils is about 20 kPa

Based on net increase in pressure, the total settlement (of the underlying native soils) is expected to be negligible (about 5mm) which is much less than the 12mm of settlement that may be tolerated.

We trust that the foregoing information is satisfactory. If you have any questions, please do not hesitate to contact us.

Your truly,
TERRAPROBE TESTING LTD.



for Karl Roechner, P.Eng.



Matthew Julien, P.Eng.

Terraprobe

memorandum



To: T.C. Tam
Approval Section
Construction Services Engineer

Date: February 15, 1993

From: Foundation Design Section
Room 315, Central Building

Subject: Falsework Design - Geotechnical Considerations
Hwy 407 - Islington Avenue
Contract 92-40, W.P. 88-78-18
District 6, Toronto

We have completed evaluation of the results of an investigation carried out by the geotechnical consultant retained by the Contractor in connection of the falsework design. Our comments on the Contractor's submissions are as follows:

BACKGROUND

1. As per the falsework drawings submitted by the Contractor, the sills are to be founded at El. 152.421 m at the north end and El. 152.878 m at the south end.
2. The initial foundation investigation(s) carried out at this site revealed the existence of sandy silt to clayey silt/silty clay at or below the design sill elevation(s), together with a groundwater level about 4 m to 5 m above the design sill elevation(s).
3. Prior to the investigation done by the geotechnical consultant regarding the falsework foundation design, the Contractor has already excavated down to El. 149.91 m (or is it 150.78 m, as per Contractor's memo dated 1993 01 27).
4. The falsework foundation report prepared by the geotechnical consultant describes the soil below El. 149.91 m (150.78 m ?) as 'relatively soft' silt and clay.
5. The Contractor's consultant has recommended the use of a layer of filter fabric on excavated ground and then one of the following treatments:
 - a. a 300 mm thick layer of 50 mm (2 in) clear stone placed directly on top of the filter fabric over which at least 0.9 m (3 feet) of OPSS Granular "A" materials are placed and compacted, or

- b. a minimum thickness of about 2 m (about 7 feet) OPSS Granular "A" materials placed and compacted over the filter fabric.

The Contractor's consultant has also recommended that the granular materials should be placed in 300 mm lifts and compacted to a minimum of 98 % of Standard Proctor maximum dry density.

The falsework foundation investigation report does not address the present groundwater level at the site.

COMMENTS

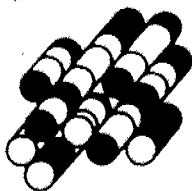
1. If the excavation was carried out with properly designed advance dewatering system, the falsework foundation could have been constructed on a granular pad, about 300 mm in thickness, on native soil using an allowable bearing pressure of 100 kPa.
2. We feel that lack of adequate advance dewatering prior to excavation below groundwater level contributed to the softening of the underlying soils.
3. It is not clear why the Contractor proceeded to carry out the excavation to El. 149.91 m (or 150.78 m ?), when according to his falsework drawings the sills are to be founded at El. 152.421 m / 152.878 m.
4. In the present situation, we agree, in principle, with the recommendations given by the Contractor's consultant.
5. However, we would like some details from the Contractor describing how he proposes to place and compact the granular fill as recommended, considering the present 'relatively soft' condition of the subgrade.

Please contact this office if you wish to discuss this topic further.



Balu Iyer, P. Eng.
Sr. Foundation Engineer

cc. R. Jeffries
R. Kant



Terraprobe Testing Ltd.

Construction and Materials Inspection and Testing

2201 Steeles Ave. E.
Brampton, Ontario
L6T 4L6
(416) 793-2650
FAX: 793-2655

December 22, 1992

Our File No. 929189

Graham Bros. Construction Limited
290 Clarence Street
Brampton, Ontario
L6W 1T4

Attention: Mr. David Weltz, P. Eng.

B. KANT

**RE: FALSEWORK FOUNDATION INSPECTION
HWY 407 - ISLINGTON AVE. UNDERPASS (MTO 92-40)
VAUGHAN, ONTARIO**

Dear Sir:

This letter presents the results of our site inspection of a portion of the area for the proposed foundations for the falsework at the above noted project site. The purpose of the site visit was to assess the condition and suitability of the exposed native soils to support the falsework. It is our understanding that the maximum load that is to be applied to any one leg of the scaffolding is about 70 kN. (15.7 kips)

1. Summary of Site Observations

The site was visited by a member of our engineering field staff on December 14, 1992. At the time of the visit, the contractor was in the process of excavating the overburden soils beneath the proposed mudsills to an elevation of 149.91 m. The excavation was completed in the northern end of the foundation area only. A total of six (6) representative samples of the in situ soils were retrieved from the exposed soils in the completed portion of the excavation. The samples were taken at the following locations:

<u>Sample No.</u>	<u>Location</u>
1	Sta. 15+520 North Shoulder of Proposed Hwy. 407
2	Sta. 15+525 North Shoulder of Proposed Hwy. 407
3	Sta. 15+505 North Shoulder of Proposed Hwy. 407
4	Sta. 15+504 North Shoulder of Proposed Hwy. 407
5	Sta. 15+510 North Shoulder of Proposed Hwy. 407
6	Sta. 15+515 North Shoulder of Proposed Hwy. 407

Also during the site visit, the foundation was manually probed at a few locations utilizing a probe rod. Based on this probing, the soils appeared to be in a relatively soft condition.

2. Laboratory Testing

The samples that were retrieved from the field were brought back to our laboratory for examination and testing. Moisture contents were carried out on all samples, and grain size analyses were carried out on Sample Nos. 1 and 5.

The results of laboratory testing are summarized as follows :

<u>Sample No.</u>	<u>Water Content</u>	<u>Grain Size Analysis</u>
1	35%	10% sand, 37% silt, 53% clay
2	25%.	-
3	30%	-
4	29%	-
5	34%	9% sand, 38% silt, 53% clay
6	31%	-

3. Discussion

Further sampling and testing should be carried out in order to assess the extent of the soft soils. This can be conducted as the contractor proceeds with the excavation to the design subgrade elevation.

Based on our observations and the laboratory test results to date, the soils that were sampled within the excavation may be described as a silt and clay with trace sand, and was in a relatively soft condition. The water contents of the samples were in the range of 25% to 35% (average 31%).

Soils of this consistency and at such a high water content, are not considered to be sufficiently competent to carry the expected loads from the scaffolding. Further, such soils will experience some degree of settlement when loads are applied to the soils. In order to stabilize the foundations and to reduce the settlement when the loads from the mud sills are applied,

Terraprobe

a sufficiently thick granular pad should be placed over the native soils.

Because of the unstable nature of the native soils, a layer of filter fabric should be placed over the native soils prior to the placement and compaction of the granular materials to form the granular pad. The granular pad placed over the filter fabric may consist of either of the following:

- (a) a 300 mm thick layer of 50 mm (2 in.) clear stone placed directly on top of the filter fabric over which at least 0.9 m (3 feet) of OPSS Granular "A" materials are placed and compacted, or
- (b) a minimum thickness of about 2 m (about 7 feet) OPSS Granular "A" materials placed and compacted over the filter fabric.

It should be noted that these recommended thicknesses, are considered to be the minimum thicknesses to stabilize the foundations for the mud sills, and to reduce settlement of the underlying native soils.

The granular pad should be extended at least 2 m beyond the falsework envelope and extended a further 3 to 4 m on that portion of the pad where construction traffic will enter on to the granular pad. Also the side slopes of the pad should be at a minimum of about 3 to 1 (horizontal to vertical).

Most of the settlement of the granular materials is expected to occur during the construction of the granular pad. Settlement of the underlying native materials will also occur as the granular materials are placed over it. The amount of settlement that will occur from the loads imposed by the granular materials, and the time that it will take for most of this settlement to occur, will be evaluated during the construction of the granular pad.

It is our understanding that this work will be carried out over the winter months. As a result of this, OPSS Granular "A" materials were recommended as the construction materials for the granular pad, as it is a free draining material that is non-frost susceptible. Also, because of the relatively high water content of the underlying native soils, a free draining

material must be used directly above the filter fabric in order to adequately stabilize the foundations.

The granular materials should be placed in lifts not exceeding 300 mm, and be compacted to a minimum of 98 percent Standard Proctor Maximum Dry Density (SPMDD). At the end of each day's work, an insulated cover should be placed over the granulars to prevent infiltration of water or snow into the granulars. However, if the granulars are not covered overnight, it may be necessary to remove any snow or frost build up from the surface before the start of the work on each day.

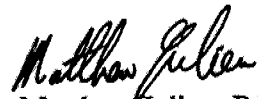
We trust that the foregoing information is satisfactory. If you have any questions, please do not hesitate to contact us.

Yours truly,

TERRAPROBE TESTING LTD.

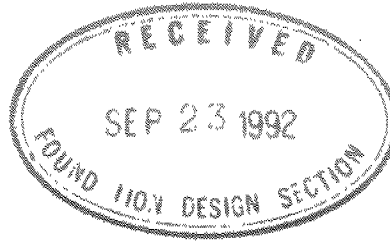


Karl Roechner, P.Eng.



Matthew Julien, P.Eng.

MINISTRY OF TRANSPORTATION
Structural Section
1201 Wilson Avenue
Atrium Tower, 4th Floor
Downsview, Ontario, M3M 1J8
Telephone: 235-5515



memorandum

DATE: September 22, 1992

TO: R. Middleton
Project Supervisor
Contract 92 - 40

RE: Hwy. 407 - Islington Ave. U'pass
Abutment Pile Layout

As discussed previously, the Foundation Office and the Structural Office require a surveyed pile layout for the Islington Ave. U'pass abutments due to the deviations of the piles from the design locations. The locations of the piles 'as constructed' should be referenced to the centre line of bearings and the centre line of the proposed Islington Ave.

We agree with you that the best results for the survey would be attained when the design elevation for the footings/ cap is reached. Please ensure that the contractor is advised that work on the abutments will not be able to proceed beyond that point until the Structural Office has time to assess the information and make any necessary revisions to the design.

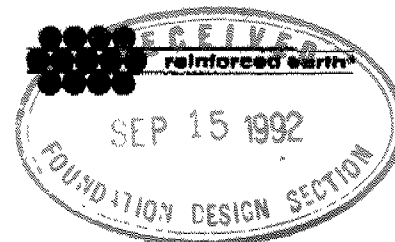
Would you please inform this office as to the time frame when the survey data will be available.

A handwritten signature in cursive script, appearing to read "R. A. Jeffries".

R. A. Jeffries
Structural Supervisor
for:
V. F. Boehnke
Head, Structural Section

:rj

cc: G. Al-Bazi
B. Iyer



-- FAX MESSAGE --

TO : MTO Structural Office
ATTN : Mr. George Al-Bazi, P.Eng.
cc: ~~REDACTED~~

FROM: Bill Brockbank, P.Eng.,
Chief Engineer

Fax # : 235-5657-4068
Pages : 1

REINFORCED EARTH COMPANY LTD.
Toronto - Canada
Telephone: (416) 674-1818
Telefax: (416) 674-7385

DATE: September 14, 1992

RE : Pile Locations Islington/407

___ Original to be mailed

x Original will not be sent

George,

At your request we visited the Islington/407 site today and were able to confirm your observations that the piles in some cases deviated significantly from the specified locations. We would recommend that a survey be done to locate the piles with reference to the center line of bearing and the center line of the proposed Islington Ave. with this information we could superimpose the existing locations on to our plan detail and see what strips and/or panels are affected.

Depending on the severity of conflicts it could be determined if 1) angles could be used to span around the piles, or 2) piles would need to be relocated, or 3) the pile cap would need to be shifted.

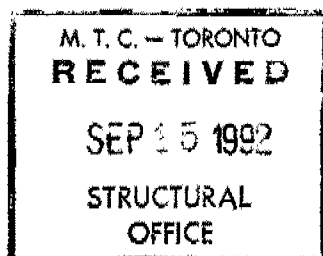
The third option would result in some redesign of ours and recasting of panels to conform to the revised geometry.

We would like to stress the importance of surveying very soon.

Best Regards,

Bill Brockbank

BB/kp



R. Middleton
Senior Project Supervisor
Contract 92-40

1992 09 09

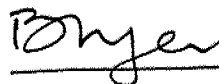
From: Foundation Design Section
Room 315, Central Building

Subject: Abutment Piles
Hwy 407 - Islington Avenue
Contract 92-40, W.P. 88-78-18
District 6, Toronto

During the visits made by the undersigned to the above site during the last 2 - 3 weeks, it was noticed that some of the abutment piles were out of plumb by more than the allowable tolerance of 2 percent. This matter was discussed with you and your inspection staff on 1992 08 25/26. On 1992 08 31, the undersigned discussed this aspect with you in some detail and requested you to carry out detailed measurements of all out of plumb piles and forward that information to our office for review. The information on out of plumbness should include the number and location of the pile and the magnitude and direction of the out of plumbness.

Once we receive the above information, we will review the same with the structural office and forward comments on remedial action, if any, to be taken.

Please call us if we can be of assistance.



Balu Iyer, P. Eng.
Sr. Foundation Engineer

cc: V. Boehnke / R. Jeffries

B:\CONT9240.2

R. Middleton
Senior Project Supervisor
Contract 92-40

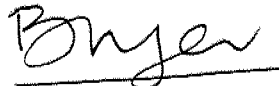
1992 08 31

From: Foundation Design Section
Room 315, Central Building

Subject: Pile Installation
North Abutment
Hwy 407 - Islington Avenue
Contract 92-40, W.P. 88-78-18
District 6, Toronto

We have completed evaluation of the results of the recent borehole put down in the area of the North Abutment. Based on a review of previous and present foundation investigations and the pile driving data to-date, it is recommended that the piles at the North Abutment shall be terminated at a tip elevation of 155 m, unless adequate capacity is reached at higher elevation.

Please contact this office if you wish to discuss this topic further.


Balu Iyer, P. Eng.
Sr. Foundation Engineer

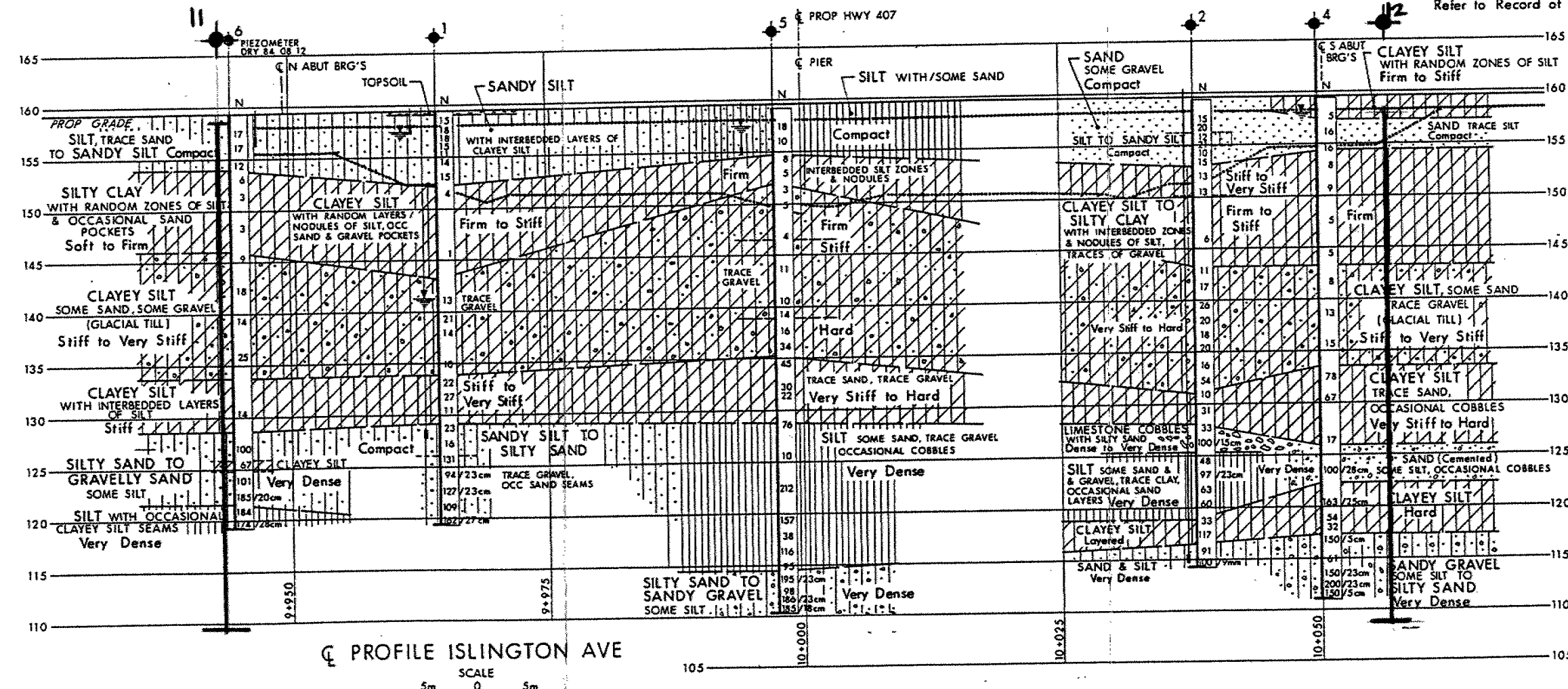
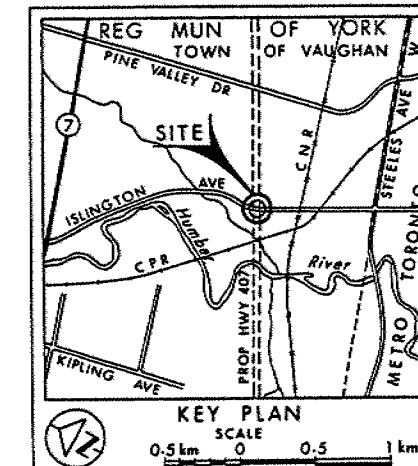
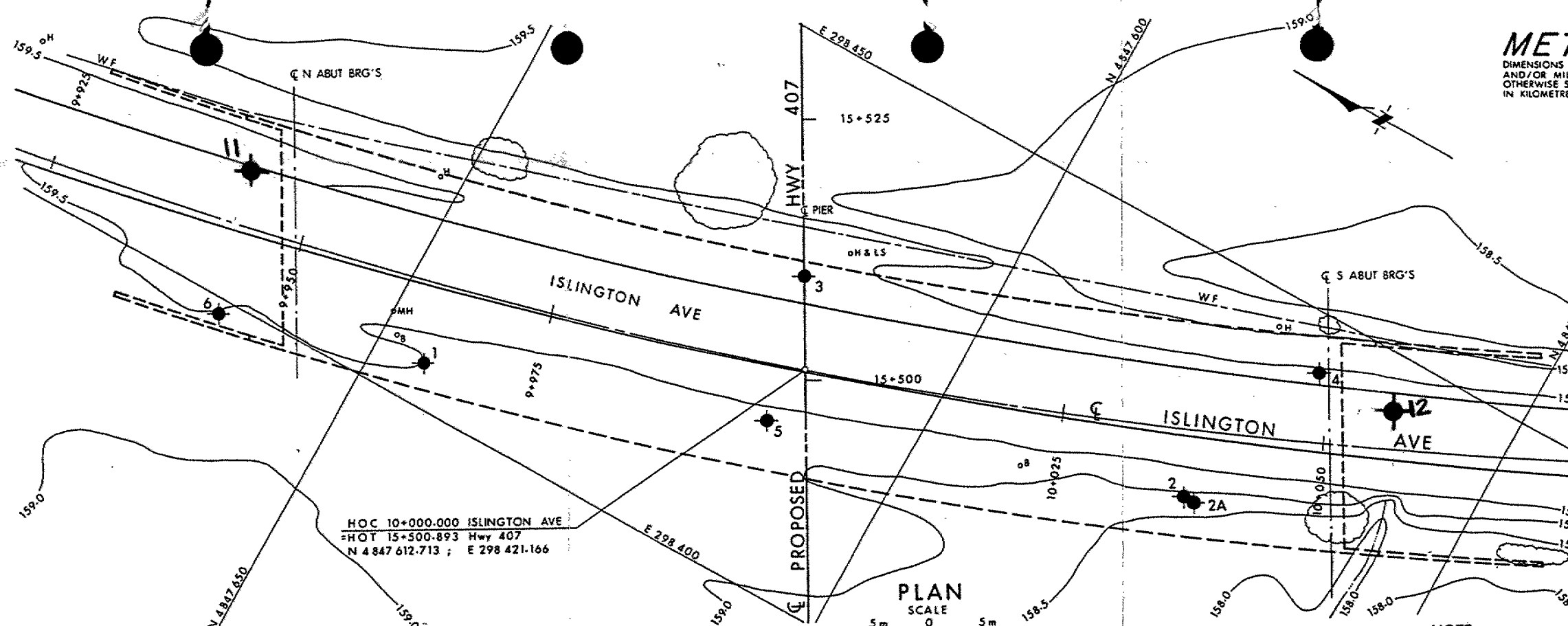
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METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES + METRES.

CONT No 92-40
P No 88-78-18

ISLINGTON AVE UNDERPASS
BORE HOLE LOCATIONS & SOIL STRATA

SHEET
317



NOTE:
Subsoil Information for BH 2A & 3
Refer to Record of Borehole

LEGEND				
◆	Bore Hole			
⊕	Dynamic Cone Penetration Test (Cone)			
⊙	Bore Hole & Cone			
N	Blows/0.3m (Std Pen Test, 475 J/blow)			
CONE	Blows/0.3m (60° Cone, 475 J/blow)			
W	W.L. at time of investigation 1983 07 and 1983 08 For BH 4; 84 07 23 for BH 5; 85 04 25			
W	W.L. in Piezometer			
P	Piezometer			
No	ELEVATION	CO-ORDINATES NORTH	EAST	
1	159.5	4 847 645	298 404	
2	158.5	4 847 575	298 428	
2A	158.5	4 847 574	298 428	
3	159.6	4 847 617	298 429	
4	159.4	4 847 569.3	298 444.7	
5	159.3	4 847 613.4	298 415.2	
6	159.4	4 847 664.4	298 398.6	
11	158.697			
12	157.977			

NOTE:
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

REV.	DATE	BY	DESCRIPTION
Geocres No 30M13-61			
HWY No 407			DIST 6
SUBMD HS CHECKED	DATE 1985 05 15		SITE 37-73-1120
DRAWN CP CHECKED			OWG 2

RECORD OF BOREHOLE No 1

2 OF 2

METRIC

W.P. 88-78-18 LOCATION Sta 9+943.3 O/S 5.2 m Lt ORIGINATED BY DK
DIST 6 HWY 407 BOREHOLE TYPE H.S. Auger COMPILED BY DK
DATUM Geodetic DATE 92 08 25 - 92 08 28 CHECKED BY BI

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
	Continued																
30.5	Heterogeneous Mixture of Clayey Silt, Sand and Gravel Occasional Clayey Silt Layers Grey, Hard		3	SS	94		128										
			4	SS	43		126										
124.9			5	SS	60												
33.8	Silty to Gravelly Sand Occasional Clayey Silt Zones Grey, Very Dense		6	SS	62		124										
	becoming more silty		7	SS	70		122										
			8	SS	60		120										
118.9			9	SS	71												
39.8	Clayey Silt, Some Sand and Gravel, Occasional Boulders Grey, Hard		10	SS	101	/18cm	118										
116.0			11	SS	63		116										
42.7	Silty Sand, Trace Gravel Grey, Very Dense		12	SS	78		114										
113.0			13	SS	119		112										
45.7	Heterogeneous Mixture of Silty Sand and Gravel Occasional Rock Fragments Grey, Very Dense		14	SS	100	/3cm											
109.9	Heterogeneous Mixture of Clayey Silt, Sand and Gravel Occasional Rock Fragments Grey, Hard (Glacial Till)		15	SS	100	/5cm	110										
48.8	End of Borehole																
	* Unstabilized Water Level measured upon completion of drilling																

RECORD OF BOREHOLE No 1

1 OF 2

METRIC

W.P. 88-78-18 LOCATION Sta 9+943.3 O/S 5.2 m Lt ORIGINATED BY DK
 DIST 6 HWY 407 BOREHOLE TYPE H.S. Auger COMPILED BY DK
 DATUM Geodetic DATE 92 08 25 - 92 08 28 CHECKED BY BI

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT 7 KN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	W _P W W _L	WATER CONTENT (%)	10 20 30		
158.7	Ground Surface												
0.0	Probable Silt and Clayey Silt Occasional Organic Inclusions						158						
155.6							156						
3.0							154						
	Probable Silty Clay						152						
							150						
							148						
							146						
							144						
							142						
							140						
							138						
							136						
							134						
							132						
							130						
131.3													
27.4	Sandy Silt Trace Clay, Trace Gravel Occasional Silt Zones Grey, Very Loose to Compact		1	SS	0								
128.8			2	SS	13								
29.9													

Continued

+3, x5: Numbers refer to
Sensitivity

20
15-5 (%) STRAIN AT FAILURE
10

Continued

92-40

METRIC

[illegible]

Continued

Continued

+3, x5: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 11

2 OF 2

METRIC

W.P. 88-78-18 LOCATION Sta 9+943.3 o/s 5.2m Lt ORIGINATED BY
DIST 6 HWY 407 BOREHOLE TYPE Hollow Stem Auger COMPILED BY
DATUM Geodetic DATE 92 08 25 CHECKED BY

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20	40	60	80	100	W _p	W		
30.5	Continued		3	SS	94											
			4	SS	43											
			5	SS	60											
			6	SS	62											
			7	SS	70											
			8	SS	60											
			9	SS	71											
			10	SS	101	/18cm										
			11	SS	63											
			12	SS	78											
			13	SS	119											
110.8			14	SS	100	/3cm										
47.9	End of Borehole															

RECORD OF BOREHOLE No 12

2 OF 2

METRIC

W.P. 88-78-18 LOCATION Sta 10+054.6 o/s 3.9m Lt ORIGINATED BY DT
DIST 6 HWY 407 BOREHOLE TYPE Hollow Stem Auger COMPILED BY DT
DATUM Geodetic DATE 92 08 31 CHECKED BY BI

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT 7 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100	w _p	w	w _L		
30.5	Continued		2	SS	13												
			3	SS	14												
			4	SS	13												
			5	SS	106												
			6	SS	130												
			7	SS	55												
			8	SS	35												
			9	SS	48												
			10	SS	94												
			11	SS	89												
			12	SS	56												
			13	SS	110	/3cm											
108.8			14	SS	110	/5cm											
49.2	End of Borehole																

memorandum



To: T. Tam
Head, Approvals Section
Structural office

Date: 1992 08 21

From: Foundation Design Section
Room 315, Central Building

Re: Approval of Falsework Foundation Report
Hwy. 407 - Pine Valley Drive U'Pass
Contract 92-40
District 6, Toronto

We have reviewed the falsework foundation report for the above site and find it to be satisfactory.

The falsework drawings are returned to you with this memo.

We have no other comments.

A handwritten signature in dark ink, appearing to read "B. Iyer", written over a horizontal line.

Dr. B. Iyer, P. Eng.
Sr. Foundation Engineer

BI/nd



Terraprobe Testing Ltd.

Construction and Materials Inspection and Testing

2201 Steeles Ave.
Brampton, Ontario
L6T 4L6
(416) 793-2650
FAX: 793-2655

August 7, 1992

Our File No. 929189

Graham Brothers Construction Ltd.
290 Clarence Street
Brampton, Ontario
L6W 1T4

Attention: Mr. Dave Weltz

CONT. 92-40

**RE: FALSEWORK FOUNDATION INSPECTION
PINE VALLEY DRIVE UNDERPASS
VAUGHAN, ONTARIO**

Dear Sir:

In accordance with the special provision No. 919S02 of the contract, we hereby submit the following Falsework Foundation report.

It is our understanding that the mudsills are to be founded at an elevation of about 160 m. Therefore, the underside elevation of the mudsills are founded well into the native soils. Also, the maximum load that is to be applied to any one leg is 68.5 kN.

Based on our site inspection, the native soils exposed in the excavations at this site (are similar to those indicated in the borehole logs) are considered to be suitable for support of the mudsills for the underpass falsework.

At the pier locations, where there has been significant excavation, the excavations should be properly backfilled utilizing the native soils. If the excavations are backfilled and compacted to a minimum of 98 percent SPMDD, then the backfill soils at the pier locations will also be able to adequately support the mudsills.

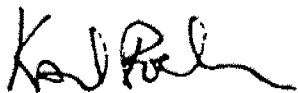
Prior to placement of the mudsills, we recommend that the foundation subgrade should be thoroughly cleaned of all loose and disturbed or wet soils.

August 7, 19922Our File No. 929189

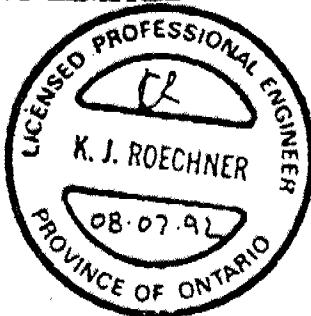
We trust the foregoing information is sufficient for your present requirements.
If you have any questions, please do not hesitate to contact us.

Yours truly,

TERRAPROBE TESTING LIMITED



Karl Roechner, P.Eng.



Matthew Julien, P.Eng.

memorandum



To: Don Lang
Senior Project Supervisor

From: Foundation Design Section
Room 315, Central Building

Re: Pile Notes on Drawing Sheet #352,
Contract 92-40,
W.P. 88-78-20,
Hwy. 407 - Pine Valley Drive Underpass,
District 6, Toronto.

Date: 1992 05 29

Please be advised that the note referring to pile tip elevation shown on the reference drawing should be revised to read as follows:

"Piles to be driven in accordance with STD SS 103-10 or SS103-11 using an ultimate capacity of 3450 kN per pile."

A handwritten signature in black ink, appearing to read "B. Iyer", with a horizontal line underneath.

Balu Iyer, P. Eng.
Sr. Foundation Engineer

BI/bi

cc. Ted Zander

SEND
TO

B. IYER

SENIOR FOUNDATION ENGINEER

FOUNDATION DESIGN SECT. CENTRAL BLD.

DEPT.

FROM

BOB JEFFRIES

STR. SECT.

91/05/29

SUBJECT

407 / ISLINGTON AVE. U'PASS & 407 SIDE SLOPES

THE ULTIMATE 407 MEDIAN REQUIRES A 1 M. WIDENING TO FACILITATE THE LARGER DIAMETER PIERS. THIS .5 M REVISION TO THE SIDESLOPES (BOTH SIDES) NORMALLY WOULD RESULT IN THE ENLARGEMENT OF THE ENTIRE CUT OR FILL CROSS-SECTION AND THE LENGTHENING OF THE STRUCTURAL RETAINING WALLS. IN THE ISLINGTON AVE. U'PASS CROSS-SECTION AND THE SURROUNDING 407 CUT, WE HAVE A 10 M BENCH.

REPLY

IN ORDER TO MINIMIZE THE REVISIONS TO THE 407 AND THE STRUCTURE, A 0.5 M REDUCTION IN THE BENCH WIDTH WOULD ELIMINATE LENGTHENING THE RET. WALLS AND THE OVERALL 407 CROSS-SECTION. WOULD YOU PLEASE REVIEW THE BENCH REQUIREMENTS AND ADVISE IF A 0.5 M REDUCTION WOULD BE ACCEPTABLE.

BOB J.

REPLY FROM

REPLY DATE

memorandum



To: K.G. Bassi
Head, Structural Office
7th Floor, Atrium Tower

Attn: G. Al-Bazi

From: Foundation Design Section
Room 315, Central Bldg.

Re: Hwy. 407 - Islington Ave.
✓ W.P. 88-78-18, Site 37-1120
District 6, Toronto

Date: 1991 08 16

We have reviewed the final drawings and documents submitted to our office and we find them to be in conformity to recommendations made by our section.

Original signed by:

Dr. B. Iyer, P. Eng.
Sr. Foundation Engineer

for

M. Devata, P. Eng.
Chief Foundation Engineer

MD/BI/me

memorandum



To: G. Al-Bazi
Design Engineer
Structural Office
7th Floor, Atrium Tower

From: Foundation Design Section
Room 315, Central Building

Re: Design Pile Capacities
Hwy. 407 - Islington Avenue
W.P. 88-78-18
District 6, Toronto

Date: 1990 11 05

Further to your telephone call of 1990 10 23, we have reviewed the recommendations made in the foundation investigation report for the above project. This memo summarizes our comments regarding pile capacities.

1. 310 X 110 steel H-piles installed to elevation 120 to 125 or lower, shall be designed for factored ULS and SLS values of 1650 kN and 1150 kN respectively.
2. The pile installation should be controlled by the Hiley formula. With this provision, we feel that pile load tests are not required at this site.
3. Standard MTO provision should be allowed for retapping of piles.

We trust that this memo addresses the various items raised by you. Please call if you need further input on this topic.

A handwritten signature in cursive script, appearing to read "B. Iyer".

Dr. Balu Iyer, P. Eng.
Sr. Foundation Engineer

BI/jb

MEMORANDUM

To: K. Bassi
Head, Structural Office

Attn: G. Al-Bazi
Design Engineer

From: Foundation Design Section
Room 315, Central Building

Re: General Arrangement Drawing Review
Hwy. 407 - Islington Ave. U'pass
W.P. 88-78-18, Site 37-1120
District 6, Toronto

90 07 26

The General Arrangement Drawing for the aforementioned structure has been reviewed by this office and the following foundation and geotechnical related comments are provided.

EXCAVATION AND DEWATERING

Excavation for the proposed structure foundations and to the proposed Hwy. 407 grade will be advanced primarily in the surficial cohesionless sands and silts present at the site. In anticipation of a high groundwater level within these materials, as revealed during the Field investigation at the site, an advanced dewatering scheme will be required to facilitate the excavation as previously discussed. (see original foundation report and attached memo dated 90 03 12). It is hereby reiterated that a gravity drainage scheme working from the centre of the proposed excavation and progressing laterally to either excavation slope, in our opinion, is the most effective method of advancing an excavation under the existing described subsoil conditions.

It is recommended that a Non-Standard Special Provision (NSSP) be included in the contract documents to address the requirement of a dewatering scheme. The contents of this NSSP shall include the following.

" The native surficial deposit at the site consists predominantly of saturated cohesionless silts and sands. The groundwater table at the time of investigation ranged from 1.3 to 4.7 m below the existing ground surface (El 157.5 to 154.9 m). These soils are highly susceptible to conditions of unbalanced hydrostatic head and is likely to 'boil' or 'slough' under such conditions. Hence advance dewatering will be required to facilitate excavations below the prevailing groundwater table as is the case for the abutment pile cap construction and Hwy. 407 excavation. The contractor shall render the excavation stable throughout the construction."

APPROACH CUTS

Observation of the overall depth of cut on the drawing reveals a magnitude in the order of 10 metres. Consequently, a mid-depth bench equivalent to 12 metres is required with 2.5H:1V slopes or alternatively a 10 metre bench with 3H:1V slopes. The selected slope geometry shall be coordinated with the Hwy. 407 excavation slopes in advance of the structure. In this regard, we reference a memorandum dated 90 05 08 addressed to G. Cautillo (see attached).

In addition, preservation and performance of the slopes is dependent on adequate drainage and slope protection. As indicated on previous correspondence, it is recommended that granular blankets consisting of a free drainage material such as Granular 'A' material in conjunction with a permanent drainage system be designed to discharge drained water from the slope.

STRUCTURE FOUNDATIONS

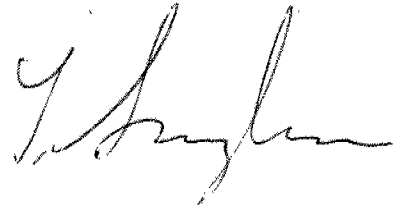
The founding elevations and capacity of the deep foundation steel H-Piles have been correctly given. However, it is recommended that all piles be restriked not earlier than seven days subsequent to their installation to confirm any soil relaxation that may have developed in the end bearing silty stratum.

Excavation for the centre pier pile cap will be carried out predominantly in the cohesive clayey silt with interbedded silt nodules below the proposed Hwy 407 grade. Consequently, no major dewatering difficulty is anticipated in view of the relative impervious nature of this material and conventional sump pumping techniques will suffice in discharging any located seepage of surface runoff. Temporary slopes to facilitate the foundation installation can be constructed no steeper than 1.5H:1V

REINFORCED EARTH RETAINING WALLS

A non-standard special provision shall be prepared to address the construction of the reinforced earth retaining walls. The special provision shall include specifications on the foundation preparation, levelling pad construction, reinforcing strip installation, and backfill material and placement procedures.

If you have any queries regarding the above comments or require any additional information, please do not hesitate to contact this office.

A handwritten signature in cursive script, appearing to read 'T. Sangiuliano'.

T. Sangiuliano, P.Eng
Foundation Engineer

for

Dr. B. Iyer, P. Eng
Sr. Foundation Engineer

cc: V. Boehnke, Head, Structural Section
Central Region

TS/lh

memorandum



To: G. Cautillo
Head, Geotechnical Section
2nd Floor, Atrium Tower

Date: 1990 05 08

Atten: K. Ganesh

From: Foundation Design Section
Room 315, Central Building

Re: Hwy. 407 Deep Excavation Cuts Between
Pine Valley and CPR Tracks
W.P. 141-87-00
District 6, Toronto

In response to your memo dated 1990 04 27 regarding a request for;

- 1) recommendations for excavation cut slope geometries between Islington Ave. and the CPR along the proposed Hwy. 407 (Station 15 + 530 \pm to 15 + 290 \pm)

and,

- 2) an evaluation of applying 3H:IV slopes as an alternative slope geometry to previously submitted slope geometry recommendations between Islington Ave. and Pine Valley Drive,

the following comments are provided;

- 1) Excavation Cuts Between Islington and CPR

The difficulty in accessing this property for a required field investigation is well known and documented by all parties associated with the project. In the absence of site specific data, recommendations for cut geometries are based entirely on extrapolation of soils data and information in conjunction with the Hwy. 407 - Islington Ave. Underpass and the Hwy. 407 - CPR subway projects. Hence, the recommendations provided are preliminary and subject to modification once site specific data is procured.

It is suggested that the slope geometry recommended at the Islington Ave. structure, summarized in a memorandum dated 1990 03 12, be extended to Station 15 + 400. Similarly, it is suggested that the slope geometry recommended at the CPR Subway structure be extended to 15 + 350. The area between the defined limits will be a transition zone between the two different geometries.

Depths of cut in the area bounded by Islington Ave. and the CPR are in the order of 8 to 10 metres. The following table summarizes the proposed geometries within the area.

.... /2

Table 1

<u>Location</u>	<u>Depth of Cut (m)</u>	<u>Bench Width (m)</u>	
		<u>2.5H:IV Slopes</u>	<u>3H:IV* Slopes</u>
Stn. 15 + 530 ± to 15 + 400 ±	8	10	8
	9	11	9
	10	12	10

Stn. 15 + 290 ± to 15 + 350 ± See Table 6, page 18 in
CPR/Hwy 407 Foundation Report

Stn. 15 + 330 ± to 15 + to 400 ± Transition Zone
*alternate 3H:IV slopes also applicable at Islington Ave. U'Pass
(W.P. 88-78-18)

2) 3H:IV Slope Alternative - Excavation Cuts Between Pine Valley Drive
and Islington Avenue

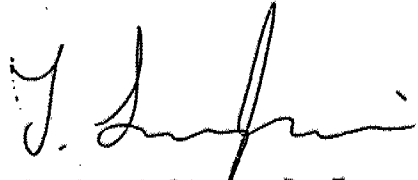
To facilitate any landscaping and associated maintenance on the excavation cut slopes, a request to investigate the application of 3H:IV slopes has been suggested for the area bounded by Islington Ave. and Pine Valley Dr. A subsequent slope stability analyses was implemented incorporating 3H:IV slopes and applying the identical procedures and subsurface conditions described in the original report. The recommended geometries are summarized below.

Table 2

<u>Depth of Cut (m)</u>	<u>Recommended Geometry</u>
> 4 - 8 inclusive	3H:IV
> 8 - 11 inclusive	3H:IV, 2 m Bench

As indicated in the memorandum dated 1990 03 12, the recommended geometry at the Islington Ave. structure shall be extended for a distance of 30 m beyond the east side of the structure.

If you have any queries regarding the above comments, please do not hesitate to contact this office.



T. Sangiuliano, P. Eng.
Foundation Engineer

for
Dr. B. Iyer, P. Eng.
Sr. Foundation Engineer

BI/TS/jb

cc: B. Jeffries

memorandum



To: V. Boenke
Head, Structural Section
Central Region

Date: 1990 03 12

Attn: R.A. Jeffries
Sr. Structural Engineer

From: Foundation Design Section
Room 315, Central Bldg.

Re: Hwy. 407 - Islington Avenue U'Pass
Site 37-1120, W.P. 88-78-18
District 6, Toronto

Subsequent to a review of the original Foundation report produced for the aforementioned structure and evaluation of additional data obtained in conjunction with a geotechnical investigation for the Hwy. 407 deep cut sections east of Islington Ave., additional analyses were carried out for the two sites and the results co-ordinated. Pertinent comments are provided below.

Approach Cuts

The subsoil stratigraphy and corresponding properties applied in the cut stability analyses at the two sites is summarized in Table 1 below. All analyses were conducted in the effective stress state of condition which represents the long term (drained) condition, the critical condition in the evaluation of the proposed cuts. The analysis was implemented using Bishop's method on an in-house mainframe program incorporating a factor of safety of 1.3. The analysis was carried out employing static loading conditions and circular slip surfaces.

TABLE 1

<u>Site</u>	<u>Soil Stratigraphy</u>		<u>Soil Parameters</u>		
	<u>Depth(m)</u>	<u>Description</u>	<u>C'(kPa)</u>	<u>ϕ'(kPa)</u>	<u>(kN/m³)</u>
Islington Structure	0 - 8	Sand and Silt	0	30	19
	8 - 16	Silty Clay to Clayey Silt with silt nodules	0	26	18.7
	>16	Clayey Silt Till	5	26	20
Deep Cuts	0 - 4	Sand and Silt	0	30	19
	4 - 8	Silty Clay to Clayey Silt with silt nodules	0	26	18.7
	>8	Clayey Silt Till	5	26	20

.../2

The phreatic surface was taken at an average depth of 1 m below the surface of the slope.

The different soil stratigraphies encountered at the Islington Avenue U'Pass site location and the deep cut section area as identified in Table 1 dictated separate analyses for the two sites.

The results of the analyses were co-ordinated and the proposed geometry for an 8 metre cut is summarized below in Table 2.

TABLE 2

<u>Site</u>	<u>Recommended Geometry</u>
Structure	*2.5H:IV, 10m mid-depth bench
Deep Cuts	2.5H:IV, 3m mid-depth bench

*This geometry is a revision of the geometry proposed in the original Foundation report. It is our opinion that the revised geometry is more feasible because of the co-ordination difficulties involved in integrating the previous recommended slopes with the proposed adjacent deep cut slopes.

The transition from the geometry at the structure to the adjacent deep cuts shall not be abrupt but rather extended to create a smooth transformation. It is recommended that the geometry at the structure be maintained for a distance of 30 m beyond the east side of the structure. Recommendations for the approach cuts at the west approach remain outstanding until pertinent site specific soils information is obtained. It is our understanding that property permission at this location may not be procured until late summer or early fall 1990.

The performance of the slopes is dependent on adequate drainage and slope protection. Consequently, it is hereby reiterated that granular blankets consisting of a free draining material such as Granular "A" material in conjunction with a permanent drainage system be designed to discharge drained water from the slope. Longitudinal toe/bench drains can be constructed consisting of a perforated pipe encased with a suitable geotextile filter fabric and in turn surrounded by a suitable granular soil filter material. The toe drains should then be connected to an appropriate integrated drainage system.

Normal slope vegetation should be established as soon as possible after completion of the cut in order to control surficial erosion.

Construction of Excavated Slopes

It is anticipated that considerable difficulty could be experienced by the Contractor in excavating the surficial cohesionless sands and silts and maintaining side slopes below the prevailing water table. Consequently, an advanced dewatering scheme is recommended to drawdown the water level to facilitate the slope construction. Two alternatives are summarized below. The alternative that is the most economical and practically feasible should be selected. The dewatering scheme shall conform to OPSS 517 series and subject to review by this office.

Gravity Drainage

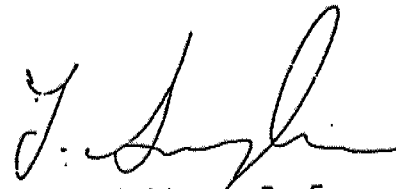
A gravity drainage system consisting of trench drains and sump pumping methods can be employed. One recommended method is to firstly advance the excavation to the elevation of the water table and then install a temporary longitudinal trench drain at the toe of the cut. Subsequent to this installation and observation of drawdown in the native soil, the excavation can proceed down to the proposed grade where another permanent trench drain is to be installed at the toe of the slope.

Gravity drainage can also be conducted by working from the centre of the proposed excavation and progressing laterally to either excavation slope. This can be accomplished by excavating sump pits and staging the excavation as illustrated in Fig. 1 attached. It is our opinion that this is a more effective gravity drainage method because any soil sloughing created by seepage are effectively controlled within the excavation.

Wells and/or Well Points

Vacuum wellpoints and/or wells consisting of large pumps with gravel filters installed concentrically in cased holes can be used to drawdown the water table. The wellpoints and wells should be properly filtered to prevent migration of fines and subsequent potential clogging. This can be achieved by surrounding the wellpoints and/or wells with a properly graded free draining soil filter. Grain size distribution curves for the native cohesionless soils should be applied in establishing the filtration and permeability criteria for the soil filter.

We trust the above comments satisfy your requirements in completing the design of the Hwy. 407 - Islington Ave. U'Pass. If you have any queries or require additional information, please do not hesitate to contact this office.



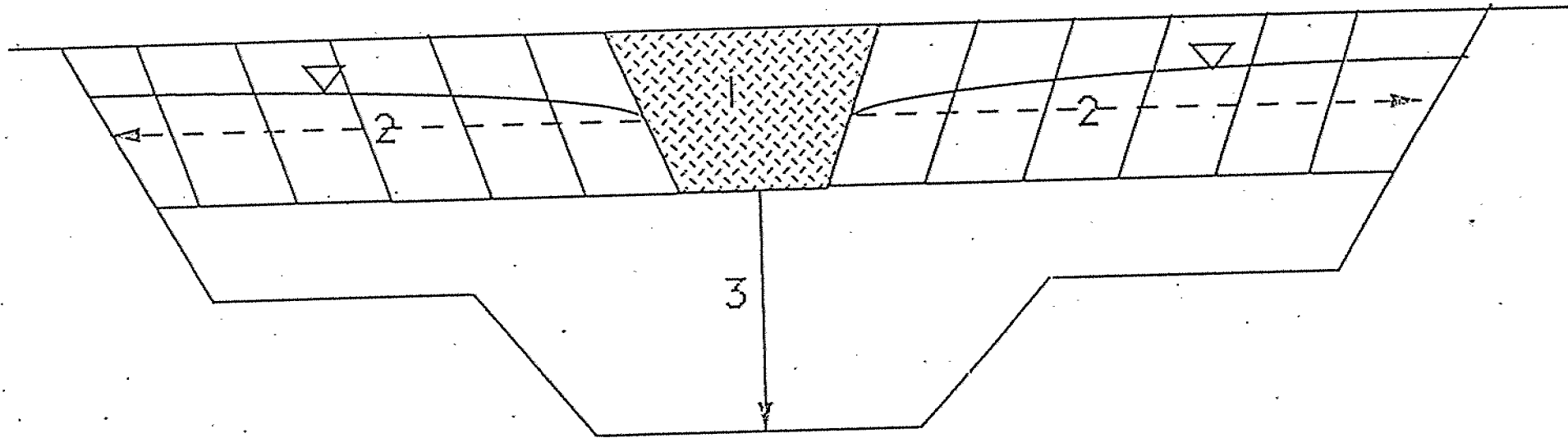
T. Sangiuliano, P. Eng.
Foundation Engineer

for

Dr. B. Iyer, P. Eng.
Sr. Foundation Engineer

BI/TS/jb

FIG. 1 - SLOPE EXCAVATION
ISLINGTON AVE/HWY 407 U'PASS
WP 88-78-18



- 1 EXCAVATE INITIAL SUMP PUMP TRENCH AND ALLOW GRAVITY DRAINAGE
- 2 EXCAVATE Laterally TO EDGE OF EXCAVATION, ALLOWING GRAVITY DRAINAGE IN PROCESS
- 3 PROGRESS TO DESIGNED DEPTH BY REPEATING (1) AND (2)

memorandum



To: V. Boenke
Head, Structural Section
Central Region

Date: 1990 03 12

Attn: R.A. Jeffries
Sr. Structural Engineer

From: Foundation Design Section
Room 315, Central Bldg.

Re: Hwy. 407 - Islington Avenue U'Pass
Site 37-1120, W.P. 88-78-18
District 6, Toronto

Subsequent to a review of the original Foundation report produced for the aforementioned structure and evaluation of additional data obtained in conjunction with a geotechnical investigation for the Hwy. 407 deep cut sections east of Islington Ave., additional analyses were carried out for the two sites and the results co-ordinated. Pertinent comments are provided below.

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.../2

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Construction of Excavated Slopes

It is anticipated that considerable difficulty could be experienced by the Contractor in excavating the surficial cohesionless sands and silts and maintaining side slopes below the prevailing water table. Consequently, an advanced dewatering scheme is recommended to drawdown the water level to facilitate the slope construction. Two alternatives are summarized below. The alternative that is the most economical and practically feasible should be selected. The dewatering scheme shall conform to OPSS 517 series and subject to review by this office.

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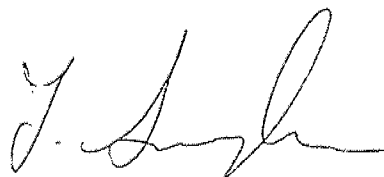
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We trust the above comments satisfy your requirements in completing the design of the Hwy. 407 - Islington Ave. U'Pass. If you have any queries or require additional information, please do not hesitate to contact this office.



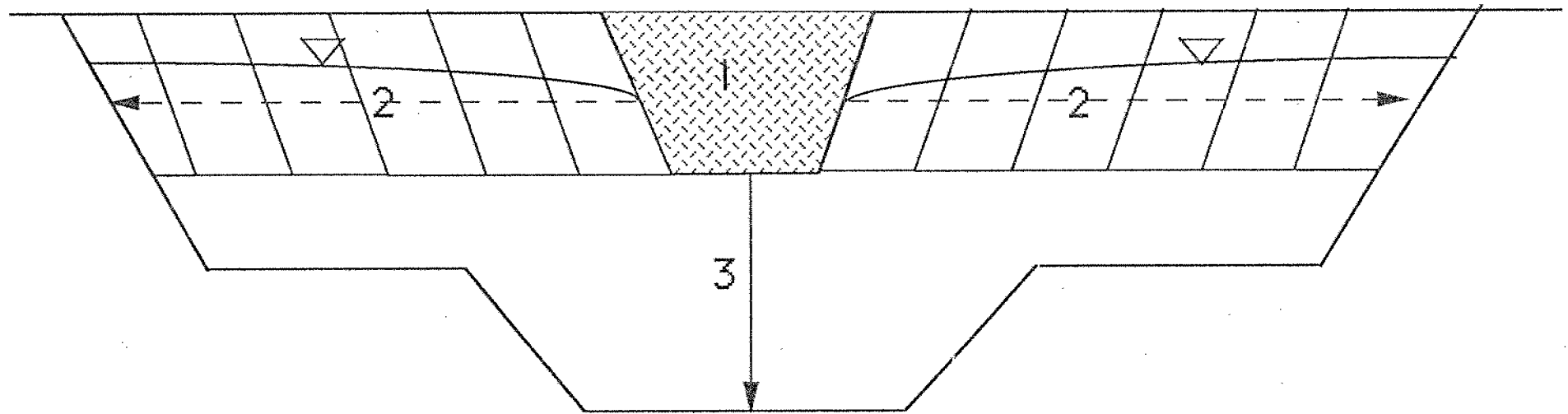
T. Sangiuliano, P. Eng.
Foundation Engineer

for

Dr. B. Iyer, P. Eng.
Sr. Foundation Engineer

BI/TS/jb

**FIG. 1 - SLOPE EXCAVATION
ISLINGTON AVE/HWY 407 U'PASS
WP 88-78-18**



- 1 EXCAVATE INITIAL SUMP PUMP TRENCH AND ALLOW GRAVITY DRAINAGE
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- 3 PROGRESS TO DESIGNED DEPTH BY REPEATING (1) AND (2)

MINISTRY OF TRANSPORTATION
M E M O R A N D U M

To: M. Devata
Chief Foundation Engineer
Foundation Design Section
Central Building
Downsview

Date: October 19, 1989

From: Structural Section
Central Region

Re: Hwy. 407 - Islington Avenue U'Pass
Site 37-1120 W.P. 88-78-18
District 6, Toronto

As part of the 407 Advance Structure, Humber River easterly to Pine Valley Drive project, a underpass is required to carry Islington Avenue over proposed Highway 407 between the Humber River and Pine Valley Drive. This structure was initiated in 1983 with a detailed preliminary drawing produced before the projected was stockpiled.

The original Foundation Report was prepared in September 1983. A preliminary General Arrangement drawing was prepared in May, 1984 depicting a four span structure.

As discussed with Mr. B. Iyer, further investigations in this area are scheduled, primarily to provide recommendations for the deep cut sections of Highway 407.

Please review the original recommendations and provide a addendum or supplement to the original as required, in co-ordination with the Highway 407 recommendations.

The present schedule calls for a completion data for a addendum of January 19, 1990.

Please find attached two of the updated Bridge Site Plan and cross-sections. One copy of the Preliminary General Arrangement Plan is also attached.

R.A. Jeffries
Senior Structural Engineer
for:
G.C.E. Burkhardt
Head, Structural Section

RAJ/vn

cc: D.R. Aron

memorandum



To: Mr. W. L. Lin
Design Engineer (Central)
Structural Office

Date: 1984 10 29

From: Foundation Design Section
Room 315, Central Building

Re: Islington Ave. Underpass
W.P. 88-78-18, Site 37-73-1120
Highway 407, District 6

We have completed 3 additional boreholes for the above mentioned structure as discussed in our memorandum of 1984 07 05. Borings for the deep storm sewer were not carried out at the same time as permission to enter private properties was not obtained.

The subsoil conditions in the additional borings were found to be very similar to those in the initial foundation investigation carried out by Golder Associates. Generally surficial deposits of sands and silts were encountered underlain by lacustrine clays and silty clay glacial till deposits.

The record of borehole log sheets and revised drawing #887818-A will be forwarded at a later date upon completion of the drafting.

We provided the following revised pile tip elevations for use in estimating pile lengths.

	<u>Elevation</u>
North Abutment	123 m
Pier 2	114 m
South Abutment	113 m

For piers 1 & 3 the estimated pile tip elevations as outlined in our memorandum of 1984 07 05 may be used.

A handwritten signature in dark ink, appearing to read "H. Sturm".

H. Sturm, P. Eng.
Project Foundation Engineer

HS/pet

c.c. - R. Kunkel
P. K. Roy

memorandum



To: Mr. R. Kunkel
Senior Project Manager
Planning & Design

Date: 19 07 84

From: Foundation Design Section
Room 315, Central Building

RE: Islington Avenue/Highway 407 Interchange
Drawdown of the Water Table
W.P. 88-78-18

We have reviewed the Foundation Report with regard to the groundwater conditions for the project described above as requested. In our opinion drawdown of the water table in the vicinity of the market garden will be nominal. The estimated drawdown of the water table should not exceed 1 m at a distance of 50 m from the toe of the slopes.

It should also be noted that seasonal fluctuation of the water table of 0.5 m has been observed and may be greater between wet and dry seasons. In view of this we feel there should not be any significant influence on the market gardening operations due to drawdown of the water table.

A handwritten signature in cursive script, appearing to read "H. Sturm".

H. Sturm, P. Eng.
Project Foundations Engineer

for

M. Devata, P. Eng.
Chief Foundations Engineer
(East)

HS/MD/mb

cc: K. Gurney
B. Ogden

memorandum



To: W.L. Lin
Design Engineer (Central)
Structural Office
4th Floor, 3501 Dufferin Street

Date: 1984 07 05

From: Foundation Design Section
Room 315, Central Building

RE: Islington Avenue Underpass
W.P. 88-78-18, Site 37-73-1120
Highway 407, District 6

We have reviewed the General Arrangement drawings No. 37-73-1120 Pl-1 and 37-37-1120 Pl-2 for the structure mentioned above and provide the following comments.

1. We note that the structure has been changed from a 2 span to a 4 span bridge due to the revised cut geometry. Since the foundation investigation was carried out based on a 2 span structure, we are contemplating adding boreholes to provide additional information for the new structure configuration. We understand that a foundation investigation for a deep storm sewer on Highway 407 is required and if there is no urgency we will complete the additional structure investigation in conjunction with the sewer. The sewer investigation will not be carried out until the final structure design is initiated.
2. For preliminary design purposes we suggest that the following pile tip elevations be used for estimating pile lengths.

	<u>Elevation</u>
North Abutment	123*
Pier 1	123
Pier 2	120*
Pier 3	115
South Abutment	115*

* Note: The pile tip elevations may be modified upon completion of the supplementary foundation investigation.

3. The benches on the cut slopes should be dimensioned on the General Arrangement drawing. Slope geometrics should also be indicated on all slopes.
4. Subdrains should be placed beside the pile cap on the north side of Pier 1 and the south side of Pier 3.
5. Original ground should be shown on the elevation.

A handwritten signature in dark ink, appearing to read "H. Sturm".

H. Sturm, P. Eng.
Project Foundation Engineer



Ontario

Ministry of
Transportation and
Communications

Tel: (416) 248-3282

Foundation Design Section
Engineering Materials Office
Room 315, Central Building
1201 Wilson Avenue
Downsview, Ontario
M3M 1J8

1984 06 08

Marshall Macklin Monahan
275 Duncan Mill Road
Don Mills, Ontario
M3B 2Y1

ATTENTION: K. Gurney, C.E.T.
Senior Project Co-ordinator

RE: Storm Sewer Drop Structure
West of Hwy. 407, Islington Avenue
Crossing, W.P. 88-78-18

Dear Sir,

In reply to your request for preliminary foundation recommendations for the above-mentioned structure, we provide the following comments:

It is our understanding that the base of the drop structure will be at approximately elevation 135. Using this elevation and borehole information from the Hwy. 407, Islington Ave. structure we anticipate the founding material will be either a stiff silty clay or compact to dense silt. Based on this preliminary information we feel the drop structure may be designed for a factored bearing capacity at the U.L.S. of 270 kPa and a bearing capacity of 120 kPa at the S.L.S. Type II. The structure should be designed to accommodate a change in the base size if the founding subsoils are found to vary from those as stated above.

The base of this structure may be founded below the water table and on a silt stratum, which would lead to a situation where dewatering would be required to prevent "boiling" of the founding soil.

If you have any further questions please do not hesitate to contact this office.

Sincerely yours,

Harry Sturm, P. Eng.
Project Foundations Engineer

HS/mb

memorandum



To: G.C.E. Burkhardt
Head, Structural Section
5000 Yonge Street
Central Region

From: Foundation Design Section
Room 315, Central Building

Re: Foundation Investigation Report
Grade Separation at Highway 407
and Islington Avenue
W.P. 88-78-18
District 6, Toronto

Date: 1984 01 13

Concerns have been expressed regarding the recommendation of our geotechnical consultant, Golder Associates, for 4 to 1 cut slopes at this location. Accordingly we have reassessed the stability of the proposed 9.2 m deep cuts.

It is our recommendation that the cut slope be constructed with 3:1 side slopes incorporating an additional two level bench, each 3 m wide, located at one third and two thirds the depth of cut. Longitudinal subdrains must be installed at a depth of at least 1.5 m below finished grade at the toe of the cut slope as well as at each bench level. The subdrains must be installed during excavation of the main cut.

It is understood that consideration has been given to employing detour or roadway protection (i.e. sheet pile walls) at this location. Schemes such as these may result in slope instability. Accordingly, please advise our office of the details of any such scheme considered and we will undertake the necessary analysis and provide our recommendations.

We have requested our consultant to assess and comment on the groundwater draw down and the effect on the market garden operation. We will advise you later on this aspect.

M. MacLean

M. MacLean, P. Eng.
Foundations Engineer

MM/mmj

c.c. - R.D. Gunter
R. Kunkel
K. Gurney

memorandum



To: G.C.E. Burkhardt
Head, Structural Section
5000 Yonge Street
Central Region

From: Foundation Design Section
Room 315, Central Building

Re: Foundation Investigation Report
Grade Separation at Hwy. 407
and Islington Avenue
W.P. 88-78-18, District 6 (Toronto)

Date: 1983 10 25

Golder Associates, Consulting Geotechnical and Mining Engineers have been retained by the Engineering Materials Office to carry out a subsurface investigation and provide recommendations with regard to the design of foundations and related earthworks for the above-mentioned project. Enclosed please find their final report containing details of the subsurface conditions with their recommendations for design and construction. The report was reviewed for technical content and format and we provide the following comments.

The presence of groundwater within the sand and silt deposits at a relatively shallow depth as well as the presence of the underlying clayey soils of firm to stiff consistency suggests that steps must be taken to ensure overall stability of the approach cuts. The critical condition for stability of an open cut slope will be the long term condition in which pore pressures will have stabilized at their steady state values. Stability analysis carried out by the consultant suggest that an overall cut slope of 4 horizontal to 1 vertical will be required to ensure the stability of the approaches. In addition these cut slopes should be designed with subdrains to control groundwater seepage.

It should be noted that the analysis was carried out with the assumption that the Islington Avenue will be maintained at the existing ground level (elev. 150) whereas Highway 407 will be at elev. 151.5. In our opinion the stability of the slopes can be improved by one or the combination of the following:

- i) raise the grade of Hwy. 407
- ii) raise the grade of Islington Avenue
- iii) type of structure which requires minimum deck thickness

However, for property requirements one should assume that 4 to 1 cut slopes will be necessary. This also indicates that a longer structure will be required in order to accommodate the flatter cut slopes.

It is recommended that a pilot trench should be constructed at the middle of the cut to the subbase level of Hwy. 407 in order to lower the phreatic surface so that the remainder of the excavation can be carried out in a satisfactory manner by the contractor. The pilot trench should commence at the low end of the grade extending longitudinally towards the high grade to allow gravity drainage. This method of construction proved to be successful in the past by the Ministry where Hwy. 405 cuts were carried out in the silt deposit with high water table near St. Davids Road, Niagara Falls resulting in huge savings for dewatering requirements. In view of this, the main cut of the highway should be constructed in advance of the structure.

The structure should be supported on end bearing Steel 'H' piles driven into the dense glacial till stratum below elev. 125 to elev. 120. For Example, an HP 310 x 110 Steel 'H' pile driven with reinforced tips as per current M.T.C. standards can be designed for the following capacities:

Factored capacity at U.L.S.	1600 KN per pile
Capacity at S.L.S. Type II	1150 KN per pile

We believe the aforementioned comments together with the enclosed report will be adequate for your immediate requirements. Should you require further clarification, or additional information, please contact us.



M. Devata, P. Eng.
Senior Foundations Engineer

MD/mmj

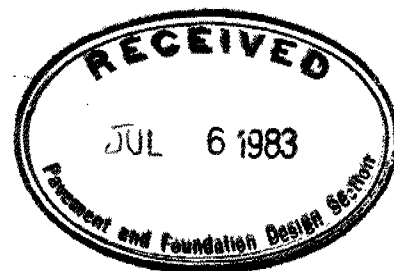
Attach.

c.c. - G.C.E. Burkhardt (3)
R.D. Gunter
A. Wittenberg
J. Smrcka (2)
K. Bassi
B.J. Giroux
R. Hore
R. Fitzgibbon (cover only)
T.J. Kovich (cover only)



Golder Associates

CONSULTING GEOTECHNICAL AND MINING ENGINEERS



July 4, 1983

Our ref: 831-1151

Ministry of Transportation and
Communications
1201 Wilson Avenue
Downsview, Ontario
M3M 1J8

ATTENTION: Mr. M. Devata, P. Eng.

RE: SOIL CONDITIONS
ISLINGTON AVENUE OVERPASS OVER HIGHWAY 407
TORONTO, ONTARIO

Dear Sirs:

Highway 407 at the above site is in a deep cut of about 30 feet. This cut begins about 500 feet west of the bridge and continues well east of the bridge site towards Pine Valley Drive. The Islington Avenue bridge will cross this cut at about the level of the existing ground surface.

Our initial boring at the above site encountered some 24 feet of compact sand with a high groundwater level (5 foot depth). This sand is underlain by a silty clay deposit to about 55 foot depth. The in situ shear strength of the clay was generally of the order of 700 to 1,000 lbs/sq.ft. At this strength, the stability of the slopes for a 30 foot cut would begin to approach a marginal condition. In order to improve the stability of the cut slopes, consideration should be given to raising the grade of Highway 407 by some 5 to 10 feet.

A market garden farm operation exists on the east side of Islington, both north and south of the highway alignment, and extends over towards Pine Valley Drive. It is expected that the high groundwater level within the sandy soil is an important factor in the success of the market garden operation. A deep cut in this sandy soil would depress the groundwater level near the crest of the cut and for a considerable distance back from the cut. It is likely that this would eliminate the market garden operation in a zone adjacent to the cut and it is likely that the market gardener

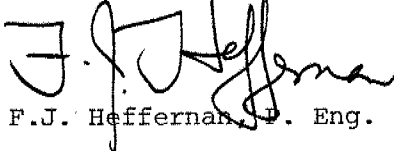
...2

would seek compensation for this change in the size of his operation. With the high water table in sand in the highway cut, the excavation of the material and subsequent use will be difficult. Also, the sand slope would be susceptible to sloughing where the water table approached the slope surface. This gives added reasons for reconsidering the need for a deep cut through these soil conditions. An alternative may be to eliminate or minimize the cut and to elevate Islington Avenue over Highway 407.

We trust that this letter provides an adequate summary of our recent discussions on the engineering significance of the poor soil conditions at this site. Should you have any questions concerning this letter, please call us.

Yours truly,

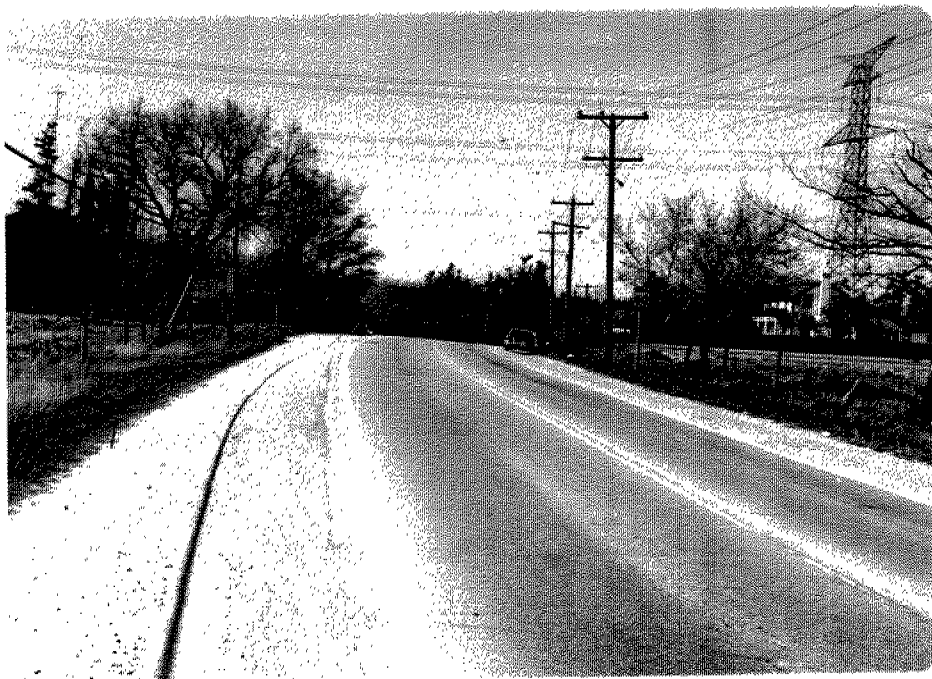
GOLDER ASSOCIATES

A handwritten signature in dark ink, appearing to read 'F.J. Heffernan', is written over the typed name.

F.J. Heffernan, P. Eng.

FJH:baw
831-1151

HIGHWAY 407 - ISLINGTON AVENUE UNDERPASS
W.P. 88-78-18, SITE 37-73-1120
DISTRICT 6, TORONTO



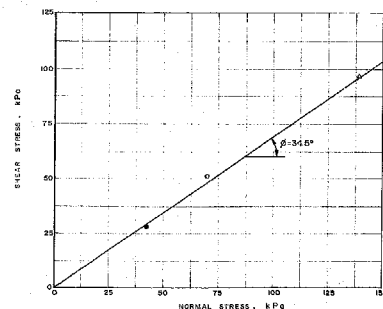
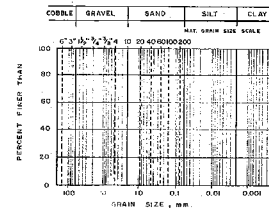
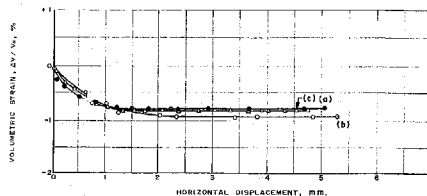
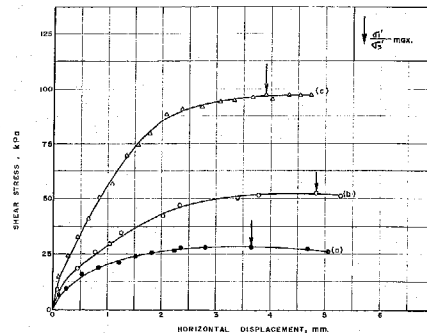
VIEW OF ISLINGTON AVENUE (LOOKING NORTH)



VIEW OF ISLINGTON AVENUE (LOOKING SOUTH)

CONSOLIDATED DRAINED DIRECT SHEAR TESTS

FIGURE 15



	A	B	C	D
BOREHOLE NUMBER	2	2	2	
SAMPLE NUMBER	7	7	7	
SAMPLE DEPTH, m	6.4	6.4	6.4	

SPECIMEN WIDTH, mm	59.4	59.4	59.4
SPECIMEN HEIGHT, mm	25.2	23.9	23.5

WATER CONTENT, BEFORE CONSOLIDATION, %			
NORMAL (CONSOLIDATION) STRESS, kPa	3.1	6.9	13.9
WATER CONTENT, AFTER CONSOLIDATION, %			
AVERAGE RATE OF STRAIN, % / hr.	0.5	0.5	0.5
TIME TO FAILURE, days	1	1	1
WATER CONTENT, AFTER TEST, %			

PEAK SHEAR STRESS, kPa	28.0	51.8	97.1
RESIDUAL SHEAR STRESS, kPa, FIRST PASS			
SECOND PASS			
THIRD PASS			
HORIZONTAL DISPLACEMENT AT PEAK SHEAR STRESS, mm	3.7	4.9	3.9
HORIZONTAL DISPLACEMENT AT RESIDUAL SHEAR STRESS, mm, FIRST PASS			
SECOND PASS			
THIRD PASS			

NATURAL WATER CONTENT, w, %	23		
LIQUID LIMIT, w _L	22		
PLASTIC LIMIT, w _P	19		
DENSITY, ρ, Mg / m ³	2.06	2.17	2.22

REMARKS:
SILT REMOULDED IN SHEAR BOX.

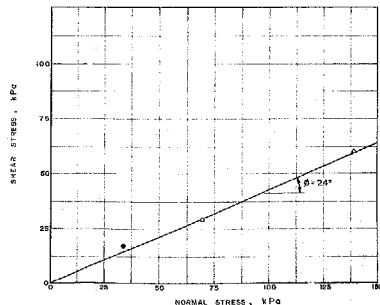
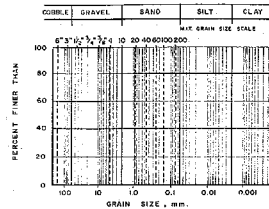
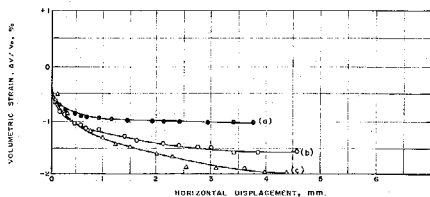
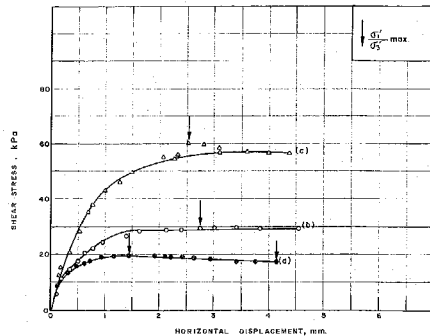
DATE: SEPT. 12, 1963
PROJECT: B31-115

Goldier Associates

DESIGN: EEO
CHECK: CWS

CONSOLIDATED DRAINED DIRECT SHEAR TESTS

FIGURE 17



	A	B	C	D
BOREHOLE NUMBER	2	2	2	
SAMPLE NUMBER	12	12	12	
SAMPLE DEPTH, m	14.0	14.0	14.0	

SPECIMEN WIDTH, mm.	58.4	59.4	58.4
SPECIMEN HEIGHT, mm	25.2	25.2	25.2

WATER CONTENT, BEFORE CONSOLIDATION, %	35	40	42
NORMAL (CONSOLIDATION) STRESS, kPa	34	69	139
WATER CONTENT, AFTER CONSOLIDATION, %			
AVERAGE RATE OF STRAIN, % / hr.	0.5	0.5	0.5
TIME TO FAILURE, days	1	1	1
WATER CONTENT, AFTER TEST, %	39		41

PEAK SHEAR STRESS, kPa	19.52	23.74	60.2
RESIDUAL SHEAR STRESS, kPa	FIRST PASS		
	SECOND PASS		
	THIRD PASS		
HORIZONTAL DISPLACEMENT AT PEAK SHEAR STRESS, mm	12	2.8	2.5
HORIZONTAL DISPLACEMENT AT RESIDUAL SHEAR STRESS, mm	FIRST PASS		
	SECOND PASS		
	THIRD PASS		

NATURAL WATER CONTENT, w, %	35	40	42
LIQUID LIMIT, w _L	48		
PLASTIC LIMIT, w _P	25		
DENSITY, ρ _t , Mg / m ³	1.90	1.69	1.67

REMARKS:
LAYERED CLAY SHEARED PARALLEL TO STRATIFICATION.