

**REVISED DRAFT
FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 17
PANCAKE RIVER BRIDGE REPLACEMENT
STRUCTURE SITE NO. 38S-5
DISTRICT 62, SAULT STE. MARIE
G.W.P. 108-99-00**

Submitted To:

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**December 1999
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GEOCRES No. 41K-52

December 12, 1999.
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Philips Planning & Engineering Limited
P.O. Box 220
3215 North Service Road
Burlington, Ontario, L7R 3Y2
Canada

Attention: Mr. Derk Meyer

Dear Sir:

**Re: REVISED DRAFT
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We take pleasure in enclosing four (4) copies of our Revised Draft Foundation Investigation and Design Report for the above mentioned project and we will be glad to discuss any questions arising from this work.

Soil samples will be retained for a period of one year, and will thereafter be disposed of unless we are otherwise instructed.

We thank you for giving us this opportunity to be of service to you.

Sincerely,

George S. Chow, P. Eng.,
Designated MTO Contact.

GSC/dee

1 copy - Ministry of Transportation Foundations Group

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1.0 INTRODUCTION

AGRA, Consulting Geotechnical Engineers, was retained by Philips Planning & Engineering Limited to conduct a foundation investigation at the site of a proposed bridge replacement that will carry Highway 17 over Pancake River. The site is located about 61 km west of the city of Sault Ste. Marie, in the Townships of Ryan and Herrick, in MTO District 62 - Sault Ste. Marie. The proposed bridge will be an approximately 45 m long, 13.8 m wide, single span, 2-lane structure.

The purpose of the investigation has been to obtain information about the subsurface conditions at the site of the proposed bridge and approach embankments by means of exploratory boreholes, and based on the findings, to provide recommendations for the foundation design of the proposed structure and the approach fills.

2.0 SITE DESCRIPTION AND PHYSIOGRAPHY

The site is located at the existing Highway 17 bridge crossing over Pancake River, about 5 km west of Regional Road 563. The grade along Highway 17 is generally level at Elevation 190 m, sloping gently to the west. The existing highway embankment is about 8 m in height at the abutment locations, decreasing in height to about 2 m at the immediate approaches. Pancake River flows from north to south, with the riverbed in a depression, at about Elevation 182 m \pm .

The existing bridge was originally constructed sometime in the 1940's or 1950's, and widened and rehabilitated in the 1960's. The existing bridge is a nine-span, 42 m long, 12.9 m wide structure, supported by timber piles. The bridge is in poor condition and in need of replacement. The timber piles are damaged or rotted and splited in some areas.

Rip-rap covers the western embankment side slopes, under the east abutment forward slope and along the river banks. A gabion wall supports the northwest embankment slope. Some erosion scour and embankment slope sloughing has occurred along the northeast embankment. The existing northeast ditchline is also eroding.

Based on available geological information¹, the site is in an area of glaciolacustrine/lacustrine shallow deposits (sands), and modern alluvium (sand, silt, gravel and muck). Generally after the last glacial withdrawal which deposited glacial till over the bedrock surface, the area was inundated by glacial Lake Algonquin, depositing sands, silts and clays. Organic (i.e. peat) and alluvial soils were then deposited along drainage courses.

¹Cowan, W.R. and Broster, B.E. 1988. Quaternary Geology of the Sault Ste. Marie Area, District of Algoma, Ontario; Ontario Geological Survey, Map P.3104, Geological Series-Preliminary Map, scale 1:100 000. Geology 1976.

.../...

The bedrock generally consists of the Late Precambrian (Keweenawan) mafic volcanic rocks² (i.e. basalt) of the Southern Province (a structural subdivision of the Canadian Shield).

3.0 INVESTIGATION PROCEDURES

The fieldwork for this project was performed during the period of September 21 to 27, 1999, and consisted of drilling and sampling six boreholes. The plan locations of the boreholes, along with stratigraphic sections are shown on Drawing No. 1.

Due to the presence of reinforced concrete approach slabs adjacent to the existing bridge, the exact proposed abutment locations were not accessible. The boreholes were therefore drilled somewhat offset from the actual proposed foundation element, as close as practicable.

The boreholes were advanced using hollow stem continuous flight augers with a truck-mounted power auger drilling rig (BOA 12M) owned and operated by Groundworks Drilling Inc., under the full-time supervision of experienced geotechnical personnel from AGRA.

Sampling in the boreholes was effected at frequent intervals of depth by the Standard Penetration Test Method (SPT), as specified in ASTM Method D 1586. This consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm diameter o.d. split barrel (split-spoon) sampler into the ground. The number of blows of the hammer to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.30 m is recorded as the Standard Penetration Resistance or the 'N'-value of the soil and this gives an indication of the consistency or the compactness condition of the soil deposit.

Due to the presence of cobbles and boulders within the glacial till deposits, Boreholes 1, 2, 3 and 4 were either advanced by wash boring or rotary core drilling methods through the cobbles and boulders in the overburden utilizing NQ and NW size casing.

The borehole locations were established in the field by our engineering staff, in relation to the existing bridge structure. The borehole geodetic elevations and northing and easting co-ordinates were later taken by surveyors from D.S. Urso Surveying Limited.

The soil samples were shipped in sealed containers to our geotechnical laboratory in Toronto (Scarborough) for further examination and classification. A laboratory testing programme, consisting of natural moisture content determinations, Atterberg Limits tests, consolidation tests, triaxial tests and grain-size analyses, was performed on selected representative soil samples. The results of the laboratory tests are presented on the appropriate Borehole Log Sheets and also in Figure Nos. 1 to 10.

²Giblin, P.E. and Leahy, E.J. 1976. Sault Ste. Marie - Elliot Lake, Districts of Algoma, Manitoulin and Sudbury, Ontario. Ontario Geological Survey, Map 2419, Geological Compilation Series, scale 1:253 440.

.../...

Two standpipe piezometers were sealed in different strata in Borehole 1 to monitor the groundwater level over a prolonged period of time without interference from surface water. The boreholes were backfilled with auger cuttings mixed with a cement-bentonite mixture.

4.0 SUBSURFACE CONDITIONS

The subsurface conditions were explored at six borehole locations (Borehole Nos. 1 to 6, inclusive). The locations of the boreholes are shown on the Plan and Profile on Drawing No. 1 and indicated on the individual Record of Borehole sheets. Cross sections of inferred subsurface stratigraphy are also shown on Drawing No. 1.

In general, the boreholes shows beneath pavement fill and a thin sand layer, the presence of clay overburden. The clay extends to a depth of about 5 to 8 m \pm below existing highway grade. Underlying the cohesive soils is a cohesionless glacial till deposit. Underlying the cohesionless glacial till in Boreholes 2 and 3, at a depth of about 18 to 20 m \pm , is a cohesive glacial till deposit. The groundwater table at the time of our investigation was encountered at depths of about 1 to 3 m \pm below existing highway grade.

Details of the subsurface conditions encountered in the boreholes are presented on the Record of Borehole sheets. The individual strata are briefly described below.

4.1 SANDY GRAVEL TO SAND (FILL)

The boreholes were drilled through the Highway 17 embankment, and encountered 50 to 200 mm of asphaltic concrete, underlain by 0.2 to 0.5 m of sandy gravel fill. The granular base material is underlain by a mixture of sand with gravel fill to depths ranging from 0.8 to 2.0 m. One grain size distribution analysis was conducted on a sample of each of the two types of fill and the results are presented in Figure No. 1. The results indicate 0 and 54% gravel, 96 and 42% sand and 4% silt and clay size particles.

Measured 'N'-values within the fill materials generally range from 6 to 65 blows/0.3 m, indicating a loose to very dense condition, but generally compact to dense.

Measured natural moisture contents range from 4 to 20%.

In our experience the thickness of fill frequently varies in between and beyond the borehole locations.

4.2 SAND

Underlying the fill in Boreholes 1, 2, 3, 4 and 5, is a sand layer ranging in thickness from 0.2 to 0.6 m. A 50 mm thick organic layer was encountered overlying the sand in Borehole 3. 'N'-values of 6 and 7 blows/0.3 m were measured within this layer, indicating a loose condition. Two grain size distribution analyses were conducted on samples of the sand layer and the results are presented

.../...

in Figure No. 2. The results indicate 1% gravel, 97 to 98% sand and 1 to 2% silt and clay size particles. Measured moisture contents range from 24 to 62%.

4.3 CLAY TO SILTY CLAY

Underlying the sand layer or fill in the boreholes is a massive, reddish-brown clay to silty clay deposit. The deposit extends to depths ranging from 4.8 to 8.3 m (or Elevation 185.2 to 181.7 m), or to the termination depths of Boreholes 5 and 6. The cohesive deposit contains frequent grey silt pockets or layers, and occasional gravel inclusions. These silt layers range from 10 to 150 mm in thickness. In Boreholes 2 and 3, interbedded between the massive clay deposit, is a varved stratum less than 1 m in thickness. The red clay varves are about 1 to 2 mm in thickness, whereas the grey silt varves are generally 10 mm in thickness.

Measured 'N'-values range from 1 to 5 blows/0.3 m. The in-situ shear strength of the clay was measured at frequent intervals of depth by the field vane shear test. Measured shear strengths range from 36 to 88 kPa, indicating a firm to stiff consistency.

Seven grain size analyses were conducted on samples from this deposit, resulting in the following grain size measurements.

Gravel:	0 to 1%
Sand:	0 to 14%
Silt:	23 to 40%
Clay:	34 to 73%

The grain size analyses results are presented in Figure No. 3.

Twelve Atterberg Limits tests, six quick triaxial tests and two consolidation tests were conducted on samples from the massive clays, varved clays and silt varves. The following results were obtained:

	MASSIVE CLAY TO SILTY CLAY RANGE	VARVED CLAY	SILT VARVES/LAYERS
Plastic Limit (%)	15 to 20	-	18 to 19
Liquid Limit (%)	41 to 48	-	22 to 26
Plasticity Index (%)	24 to 30	-	4 to 7
Moisture Content (%)	24 to 66	33	24 to 27
Unit Weight (kN/m ³)	15.8 to 22.4	18.9	-
Shear Strength			
Field Vane (kPa)	36 to 88	44 to 68	-
Laboratory Vane (kPa)	10 to 34	-	-
Unconfined Compressive Strength (kPa)	38 to 64	-	-
Coefficient of Consolidation (C _v)	0.06 to 0.23	-	-

.../...

These Atterberg Limits values are characteristic of clayey soils of intermediate to high plasticity (Figure No. 5). Its moisture content exceeds the liquid limit at some locations, indicating that portions of the clays are highly compressible. Based on the field vane and unconfined compressive strength results, the clays are generally firm to stiff in consistency, with occasional soft zones. The laboratory vane results are very low and are likely due to sample disturbance. The results of all field vane and of the quick triaxial compression tests are summarized in Figure No. 7. The results of a consolidation test performed on a sample of this massive clay to silty clay is presented in Figure No. 8. Consolidated undrained triaxial compression tests with pore water pressure measurements were performed on two samples of the massive clay giving an angle of internal friction ϕ' of 28° and a cohesion c' of 8 kPa (Figure No. 9).

The values for the grey silt varves are characteristic of plastic inorganic silt to clayey silt (Figure No. 6). The results of a consolidation test performed on a sample from a massive clay with pockets of silt are presented in Figure No. 10.

4.4 HETEROGENEOUS MIXTURE OF SAND, SILT AND GRAVEL (GLACIAL TILL)

Below the clay deposits in Boreholes 1, 2, 3 and 4, a cohesionless glacial till deposit was encountered. This deposit consists of a heterogeneous mixture of sand, silt and gravel, with frequent cobbles and boulders. The till extends to the remaining depth of Boreholes 1 and 4 (14.2 and 9.4 m or Elevation 175.8 and 180.6 m, respectively). In Boreholes 2 and 3, the till extends to depths of 20.1 and 17.7 m (or Elevations 169.9 and 172.3 m), respectively. Due to the frequent cobbles and boulders (a boulder 1.0 m in thickness was cored in Borehole 1), auger refusal was encountered within the till at depths ranging from 6.7 to 10.6 m. In order to by-pass the obstructions, rock coring and wash boring were conducted. Measured 'N'-values range from 36 to greater than 50 blows/0.3 m indicating a compact to very dense condition, but generally very dense. Measured natural moisture contents range from 6 to 20%. Measured unit weights range from 20.4 to 24.3 kN/m³, or an average of about 23.1 kN/m³.

Seven grain size distribution analyses were conducted on samples from this deposit and the results are presented in Figure No. 11. The results indicate 9 to 41% gravel, 22 to 60% sand, 23 to 54% silt and 2 to 10% clay size particles. Measured moisture contents range from 6 to 20%.

4.5 HETEROGENEOUS MIXTURE OF CLAY, SILT AND GRAVEL (GLACIAL TILL)

Underlying the above cohesionless glacial till, Boreholes 2 and 3 encountered a reddish brown, cohesive glacial till deposit. This glacial till deposit is composed of a heterogeneous mixture of clay, silt and gravel size particles. Due to difficulties in obtaining a sample below the bouldery cohesionless till, no sample of this cohesive till was obtained from Borehole 3, aside from washed samples. A measured 'N'-value of 150 blows/0.09 m was obtained in this deposit in Borehole 2, indicating a hard consistency. An Atterberg Limits test was conducted on the sample obtained and is presented in Figure No. 12. The laboratory test results are presented below:

.../...

Liquid Limit:	19%
Plastic Limit:	15%
Plasticity Index:	4
Moisture Content:	12%
Unit Weight:	23.1 kN/m ³

4.6 GROUNDWATER CONDITIONS

Groundwater levels in the open boreholes were observed during the drilling and at the completion of each borehole. To enable us to measure water levels at the site over a prolonged period of time without interference from surface water, two standpipe piezometers were installed within different strata in Borehole 1.

The recorded values are shown on the individual Record of Borehole sheets. Based on the recorded values in the piezometers installed and moisture contents of recovered samples, the groundwater levels at the time of the investigation generally ranged from 1 to 3 ± m below the ground surface (approximately Elevation 189 to 187 m). It should, however, be pointed out that the groundwater at the site would fluctuate seasonally and can be expected to be somewhat higher during the spring months and in response to heavy rains. Also, a perched water table may be encountered in the pervious fill and sand deposits overlying the relatively impermeable clay deposits.

.../...

5.0 DISCUSSION AND RECOMMENDATIONS

This report contains the findings of our geotechnical investigation, together with our recommendations and comments. These recommendations and comments are based on factual information and are intended only for the use of the design engineers. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The anticipated construction conditions are also discussed, but only to the extent that they may influence design decisions. Construction methods discussed, however, express our opinion only and are not intended to direct the contractors on how to carry out the construction. Contractors should also be aware that the data and their interpretation presented in this report may not be sufficient to assess all the factors that may have an effect upon the construction.

The proposed Pancake River bridge replacement will be an approximately 45 m long, 13.8 m wide, single span, 2-lane structure (about 3 m longer and 1 m wider than the existing structure). The grade along Highway 17 at Pancake River is generally level at Elevation 190 m, sloping gently to the west. The existing highway embankment is about 8 m in height at the abutment locations, decreasing in height to about 2 m at the immediate approaches. Pancake River flows from north to south, with the riverbed at about Elevation 182 m \pm . The existing bridge is in poor condition and requires replacement. No grade raise is anticipated at the immediate approach embankments.

In general, the boreholes show, beneath pavement fill, and a thin sand layer, the presence of clay overburden. The soft to stiff clay is generally massive, and extends to a depth of about 5 to 8 m \pm below existing highway grade. Underlying the cohesive soils is a compact to very dense cohesionless glacial till deposit. Underlying the cohesionless glacial till in Boreholes 2 and 3, at a depth of about 18 to 20 m \pm , is a hard cohesive glacial till deposit. The groundwater table at the time of our investigation was encountered at depths of about 1 to 3 m \pm below existing highway grade.

5.1 FOUNDATIONS

5.1.1 Deep Foundations

It is our understanding that the preferred abutment design of the proposed bridge is of the "integral" type and will be supported on driven steel H-piles.

The boreholes show that for the prevailing subsurface conditions (with bouldery till) the use of a low displacement pile, such as a steel H-pile with a heavy section, such as HP310X110 with reinforced tips as per MTO Specifications, would be better suited than other pile types (i.e. steel tube piles, caissons, steel H-piles with a lighter section, etc.).

.../...

5.1.1.1 Resistance to Axial Loads

The following table summarizes the approximate pile tip elevations and axial resistances that may be utilized for designing HP310X110 steel H-piles.

TABLE 1

SUPPORT LOCATION	REFERENCE BOREHOLE	ESTIMATED DESIGN TIP ELEVATION (m)	FACTORED AXIAL RESISTANCE AT U.L.S. (kN)	AXIAL RESISTANCE AT S.L.S. (kN)
West Abutment	1	179	1,700	1,200
	2	177	1,700	1,200
East Abutment	3, 4	179	1,700	1,200

These values were conservatively selected in view of the fact that some premature refusals may be encountered due to the presence of cobbles and boulders at higher elevations.

The piles should be driven with a suitably heavy hammer capable of delivering a rated capacity of at least 50 kJ/blow.

The driving of the piles should be controlled by a recognized pile driving formula, such as the Hiley Formula. The estimated ultimate resistance of the piles (driven to practical refusal in the overburden) by the Hiley Formula is approximately 3400 kN. This value was arrived by dividing the factored axial resistance at U.L.S. by a resistance factor of 0.5, as per current MTO convention. Because of the presence of frequent cobbles or boulders in the overburden, and the anticipated hard driving conditions as mentioned before, the piles should be equipped with reinforced tips as per MTO Standards (OPSD 3301.00) to minimize the risk of damage to the piles. It may be difficult for the pile to penetrate to the specified elevation due to the presence of cobbles and boulders. Therefore it would be necessary to preauger a small diameter pilot hole (e.g. 200 mm) and possibly core through obstructions to a depth of about 1 to 1.5 m above the acceptable pile tip elevation.

Oversize materials (e.g. greater than 75 mm nominal diameter) should not be used in the fills through which piles would be driven.

During the driving process, piles which have already been driven should be monitored to determine if they are heaving due to effects of driving adjacent piles. If this phenomenon occurs, the affected piles should be re-driven. It is recommended that not less than 15% of the piles and at least three piles in each foundation support element be re-struck one to two days after initial installation, as a precaution against relaxation. If relaxation occurs, then all piles in that foundation element should be re-tapped.

.../...

1700
0.5
= 3400

It is possible that some of the piles may penetrate to one to two metres below the estimated tip elevations and this aspect should be taken into consideration when ordering piles.

The geotechnical resistance at Serviceability Limit States (S.L.S.) is dependent on the settlement of the pile group and, therefore, is governed by the size of the pile group. The pile group configuration is currently not available to us. Provided that the piles are designed and installed as recommended above, it is considered that the quoted S.L.S. value corresponds to no more than 25 mm of settlement of the pile group. We will confirm the estimated settlement once information on the pile group configuration is known.

5.1.1.2 Resistance to Lateral Loads

Laterally applied loads on piles can be resisted geotechnically by the driven piles through passive pressure developed in the soil in which the piles are embedded. The pile tip elevations recommended above indicate that the piles will be in the order of 10 to 12 m in length. Lateral pile resistance may be considered in accordance with Section 6-9.8.1 of the O.H.B.D.C., 3rd Edition.

In cohesionless soils the coefficient of horizontal subgrade reaction may be estimated from;

$$k_s = n_h z/d$$

where k_s = coefficient of horizontal subgrade reaction
 z = depth
 d = pile width
 n_h = coefficient related to soil density as given in the table below

In cohesive soils the coefficient of horizontal subgrade reaction may be estimated from;

$$k_s = 67c_u/d$$

where k_s = coefficient of horizontal subgrade reaction
 d = pile width
 c_u = undrained shear strength of the soil

Also, presented in the same table are the estimated values for angle of internal friction, undrained shear strength of the soil and bulk unit weights.

.../...

AREA/REFERENCE BOREHOLE NO.	APPLICABLE DEPTH FROM EXISTING GROUND SURFACE	SOIL TYPE	BULK UNIT WEIGHT (kN/m ³)	ANGLE OF INTERNAL FRICTION (ϕ) DEGREES	UNDRAINED SHEAR STRENGTH (kPa)	RECOMMENDED n_h VALUE (MN/m ³)
East Abutment						
3	2 - 5 m	firm to stiff clay to silty clay	19	-	30	-
	5 - 18 m	dense to very dense silty sand till	23	37	0	11.0
4	2 - 4.5 m	firm to stiff clay	19	-	30	-
	4.5 - 9.5 m	dense to very dense silty sand till	23	37	0	11.0
West Abutment						
1	2 - 7 m	firm to stiff clay to silty clay	19	-	30	-
	7 - 14 m	very dense silty sand till	23	37	0	11.0
2	2 - 8 m	soft to firm clay	19	-	30	-
	8 - 20 m	very dense silty sand till	23	37	0	11.0

The recommended horizontal resistances for the HP310X110 steel H-piles are as follows:

$$\begin{aligned}\text{Factored Horizontal Resistance at U.L.S.} &= 100 \text{ kN} \\ \text{Horizontal Resistance at S.L.S} &= 40 \text{ kN}\end{aligned}$$

In accordance with MTO requirements (MTO Structural Office Standard), piles for integral abutments require a 3 m long flex zone. In essence the current MTO standard for the flex zone consists of an annular space in between two concentric CSP's. One of the CSP's surrounds the H-pile (i.e. has a diameter of about 600 mm surrounding the pile, while the second CSP has a somewhat larger diameter; typically 800 mm for a 310 mm H-pile). The annular space in between the CSP's is the 3 m long flex zone. After the pile is driven, the space between the H-pile and the inner CSP is filled with coarse sand. An NSSP should be included in the contract documents specifying the gradation of the sands as follows.

.../...

<u>Sieve Size</u>	<u>Percentage Passing</u>
2 mm	100%
600 µm	80 - 100%
425 µm	40 - 80%
250 µm	4 - 25%
150 µm	0 - 6 %

5.1.2 Spread Footing Foundations

Due to the compressible nature of the overlying silty clay, spread footing foundations placed on or within the silty clay would sustain excessive consolidation settlement, and are not considered a feasible option. It is considered impractical to found spread footings on the underlying glacial till as construction would have involved excavation in the order of 5 m to 8 m and extensive groundwater control requirements.

5.2 LATERAL EARTH PRESSURES

Backfill behind abutments and retaining walls should consist of non-frost susceptible, free draining granular materials in accordance with the Ontario Ministry of Transportation Standards.

Free-draining backfill materials (i.e. Granular 'A' or Granular 'B') and the provision of drain pipes and weep holes, etc., should prevent hydrostatic pressure build-up. Computation of earth pressures should be in accordance with O.H.B.D.C. For design purposes, the following parameters (unfactored) can be used.

Compacted Granular 'A'

Unit Weight = 22 kN/m³

Coefficient of Lateral Earth Pressures:

$$K_a = 0.27 \text{ (active condition)}$$

$$K_o = 0.43 \text{ (at-rest condition)}$$

Compacted Granular 'B'

Unit Weight = 21 kN/m³

Coefficient of Lateral Earth Pressures:

$$K_a = 0.31 \text{ (active condition)}$$

$$K_o = 0.47 \text{ (at-rest condition)}$$

These values are based on the assumption that the backfill behind the retaining structure is free-draining and adequate drainage is provided. As well, it is assumed that the ground behind the retaining structure is level.

.../...

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or movements can be allowed such that the active state of earth pressure can develop. If the abutment is restrained and does not allow lateral yielding, then at-rest pressures should be used as per Clause C6-7.1 of the O.H.B.D.C., 3rd Edition. The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients in accordance with Clause 6-7.4.3 of the O.H.B.D.C., 3rd Edition.

Vibratory equipment for use behind abutments and retaining walls should be restricted in size as per current MTO practice.

As an alternative to conventional retaining walls, MTO's Retained Soil System may be used. The following should be included in the Contract Documents:

- identify longitudinal extent in plan of the Retained Soil System.
- identify in plan transverse space constraints (top of wall and bottom of wall)
- identify elevation of top of wall and bottom of wall
- include NSSP for Retained Soil Systems in Contract Documents

The Retained Soil System should be of high performance and medium to high appearance.

5.3 APPROACH EMBANKMENTS

5.3.1 Embankment Stability

5.3.1.1 Existing Conditions

The existing forward slopes are at an inclination of about 1V:1.7H (west abutment) to 1V:2.5H (east abutment), and are continuous from the riverbed to the Highway 11 grade, with no benching. The existing slope along the northwest corner of the west embankment is much steeper, confined by a gabion wall.

The existing west abutment forward slope (1V:1.7H) was back analyzed by the limit equilibrium method, utilizing Bishop's simplified method of analysis. For this purpose the computer programme Slope/W and the assumed soil parameters in Section 5.3.1.2 were used while varying the cohesion value (c') to determine the effect on the Factor of Safety of the slope. The Factor of Safety of the slope approaches 1.0 for cohesion values of 5 to 7 kPa (Factor of Safety's of 0.95, 0.97 and 0.99, respectively), and exceeds 1.0 with a cohesion value of 8 kPa (Factor of Safety of 1.02). The effective internal friction angle used was 27°.

5.3.1.2 Forward Slopes

Due to the height of the existing forward slopes, 8 m in height), a mid-height berm is required by MTO standards. Since no fill is allowed in the river, this would require cutting back the existing slope to provide for a berm, or bench. Due to this unloading of the slope, the short term stability of the embankment is not of concern. Due to the presence of weak clayey soils consideration for

.../...

the long term stability of the approach embankments and of the abutments is required.

The stability of the embankments were analyzed by the limit equilibrium method, utilizing Bishop's simplified method of analysis. For this purpose the computer programme Slope/W and the following assumed soil parameters were utilized. In order to account for the heterogeneity of the clay and the effect of weathering on the embankment, a cohesion value of 5 kPa and friction angle of 27° was used in the following analyses.

Drained Stability Analyses (Long Term Conditions)

SOIL TYPE	FRICTION ANGLE (ϕ) DEGREES	UNIT WEIGHT (kN/m ³)	COHESION (c') kPa
Granular Fill	30	21	0
Sand Layer	30	20	0
Silty Clay	27 ✓	19	5 ✓
Silty Sand Glacial Till	37	23	0

The groundwater table was assumed to be at 2 m below existing highway grade (i.e. Elevation 188 m), intercepting the embankment slope and at the ground surface below this level. The embankment slope stability results can be summarized as follows.

Various forward slope configurations were analyzed for the stability of the 45 m long span structure. A stable forward slope with 1V:2.5H from the riverbed to Elevation 187 m, followed by a 6 m wide bench (or berm), with the existing slope behind the abutment cut back at 1V:2H, will provide Factors of Safety of 1.31 and 1.38 for the west and east forward slopes (see Figure Nos. 13 and 14).

5.3.1.3 Side Slopes

Pavement side slopes of less than 4 m in height should be constructed at 2.5H:1V to maintain stability. Side slopes of less than 2 m in height should be stable at 2H:1V. Side slopes higher than 4 m should have the same configuration as the forward slopes (incorporating a berm at Elevation 187m).

The existing gabion wall appears to be in good condition and not undermined, therefore the wall may remain if desired.

5.3.2 Embankment Construction

All organic and other unsuitable soils should be removed within an envelope given by an imaginary slope not steeper than 1:1 from the toe of the proposed embankment. After stripping, the exposed subgrade should be inspected, approved and where feasible properly compacted from the surface under the supervision of qualified personnel, using a suitable compactor.

.../...

Provided that all organic and otherwise unsuitable materials are removed and the subgrade is properly compacted from the surface as detailed above, the settlement of the foundation materials (i.e. not including the settlement of the embankment material under its own weight) should not exceed 50 mm and should be substantially completed within three weeks of placing the embankment fill to its full height. Such settlements are considered acceptable and will not necessitate preloading or surcharging.

The materials used for the construction of the embankment fills could consist of approved, clean normal earth fill (e.g. Select Subgrade Materials - O.P.S.S. 1010). The existing embankment fill, provided it is carefully removed and not contaminated with the underlying soils could be re-used for this purpose. The earth fills should be placed in lifts not exceeding 300 mm before compaction and each lift should be uniformly compacted to at least 95% of the material's Standard Proctor Maximum Dry Density. The degree of compaction within the top 0.6 m of the fill (i.e. the subgrade immediately beneath the granular sub-base) should be increased to 98%. The selection, placement and compaction of the fill should be carried out under geotechnical control. The time rate of settlement will depend on the materials used for construction. For granular fills it should be mostly elastic (i.e. should be substantially completed during the construction and within a few weeks thereafter) while clayey fills will consolidate over a longer period of time. This quoted settlement would be in addition to the foundation settlements quoted earlier in this section of the report.

The use of light vibratory equipment is recommended for compaction of the slag material. In general, the slag material should be placed and compacted in accordance with OPSS 206.07.

5.4 CONSTRUCTION COMMENTS

Water level measurements indicate that the groundwater level ranges between approximately Elevations 189 and 187 m, or at a depth of about 1 to 3 m below existing Highway 17 grade. Therefore excavations for the pile cap construction and slope reconfiguration will require dewatering. Excavations for the pile cap will require that the groundwater level be lowered to at least 1 m below the anticipated excavation base levels, to minimize risk of base heave within the clay. We recommend that dewatering be implemented by a contractor specializing in this field. Any surface water seepage, if necessary, can easily be handled by gravity drainage and pumping from open sumps. All surface runoff should be diverted away from the excavations.

Allowance should be made to place a work mat consisting of an approximately 75 mm thick layer of concrete on the surface of the clay subgrade.

All excavations should be carried out conforming with the Ontario Occupational Health and Safety Act and its regulations.

Temporary unsupported excavations above the watertable within the fill should be stable at 2H:1V. Temporary excavations below the watertable within the clayey soils should be stable at 2.5H:1V, while excavations within the glacial till should be temporarily stable at 2H:1V.

.../...

5.5 FROST PROTECTION

Design frost penetration for the general area is 2.0 m. Therefore, a permanent soil cover of 2.0 m or its thermal equivalent is required for frost protection of foundations.

5.6 EROSION PROTECTION

The existing ditchline along the northeast corner of the bridge is being eroded, resulting in deepening of the ditchline and sloughing of the embankment toe. The clay in the ditchline was sampled by hand and a grain size of the material is presented in Figure No. 15. To protect the toe and ditchline from further erosion, we recommend that the ditchline be regraded and a rip-rap blanket be placed along the ditch and embankment toe.

Proper erosion control measures should be implemented both during the construction and permanently. This can be achieved by immediate seeding or sodding (OPSS 572). To protect the abutments and riverbanks from scour we also recommend a layer of rip-rap be placed along the forward and side slopes of the embankments.

6.0 CLOSURE

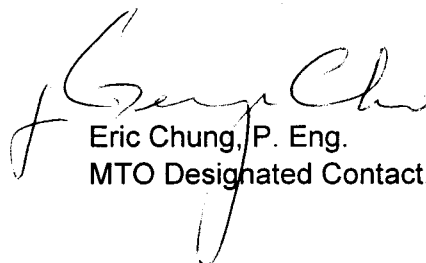
We recommend that once the details of the structure are finalized, our recommendations should be reviewed for their specific applicability.

Sincerely,



Andrew Drevininkas, P. Eng.

ADh



Eric Chung, P. Eng.
MTO Designated Contact.

.../...

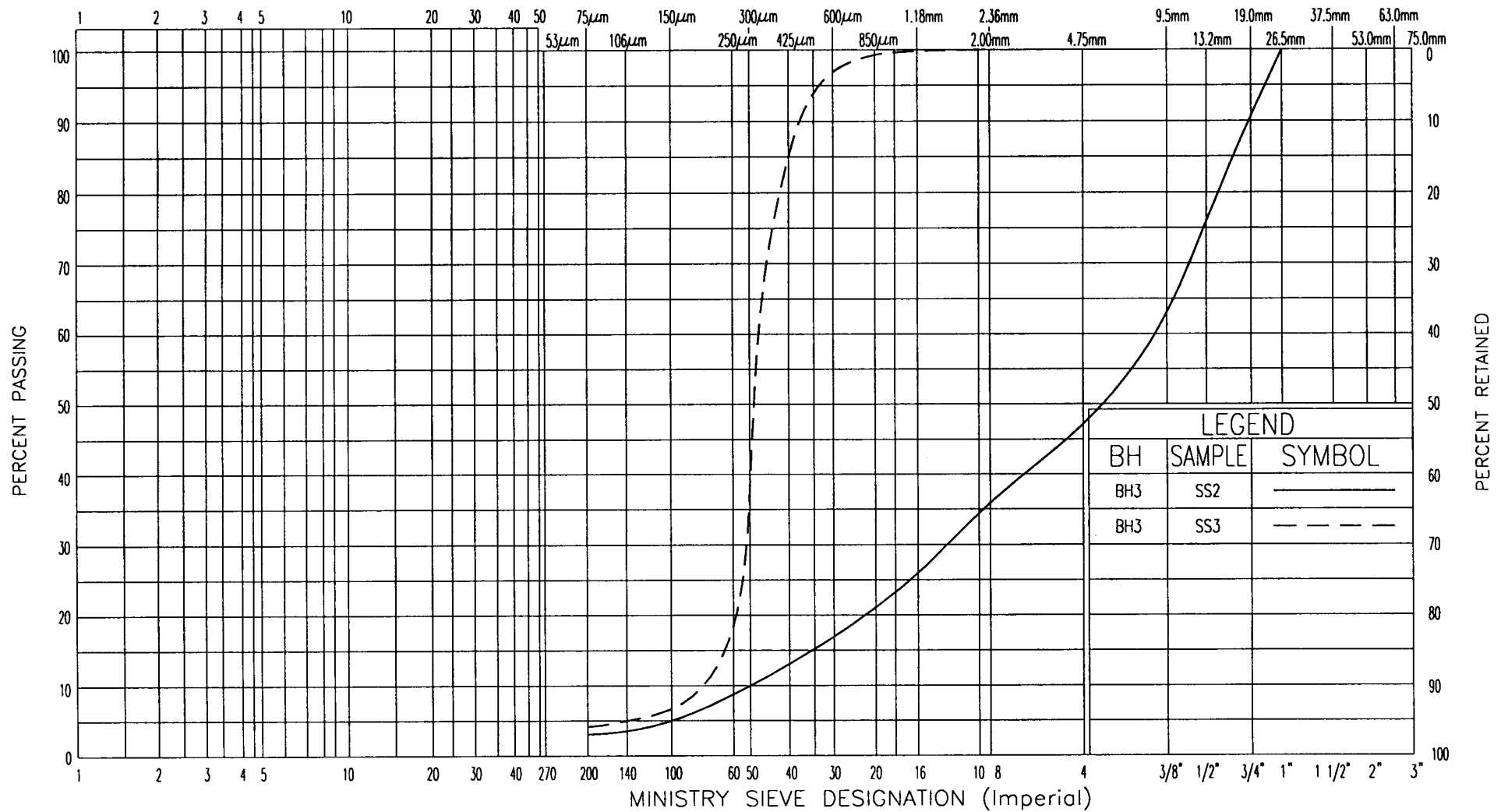
FIGURES

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)

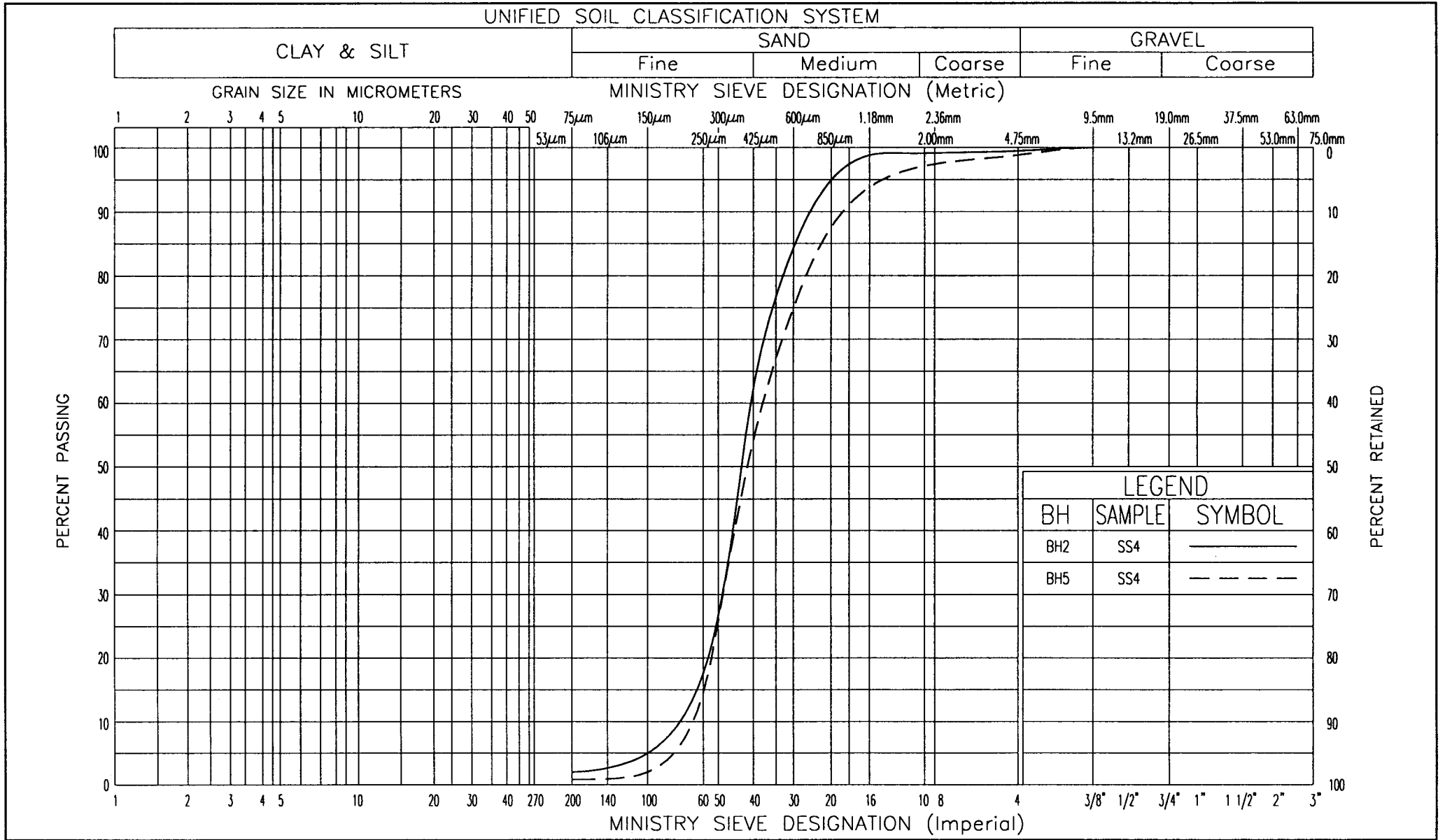


LEGEND		
BH	SAMPLE	SYMBOL
BH3	SS2	————
BH3	SS3	- - - - -



GRAIN SIZE DISTRIBUTION
FILL

FIG. No 1
W. P. 108-99-00

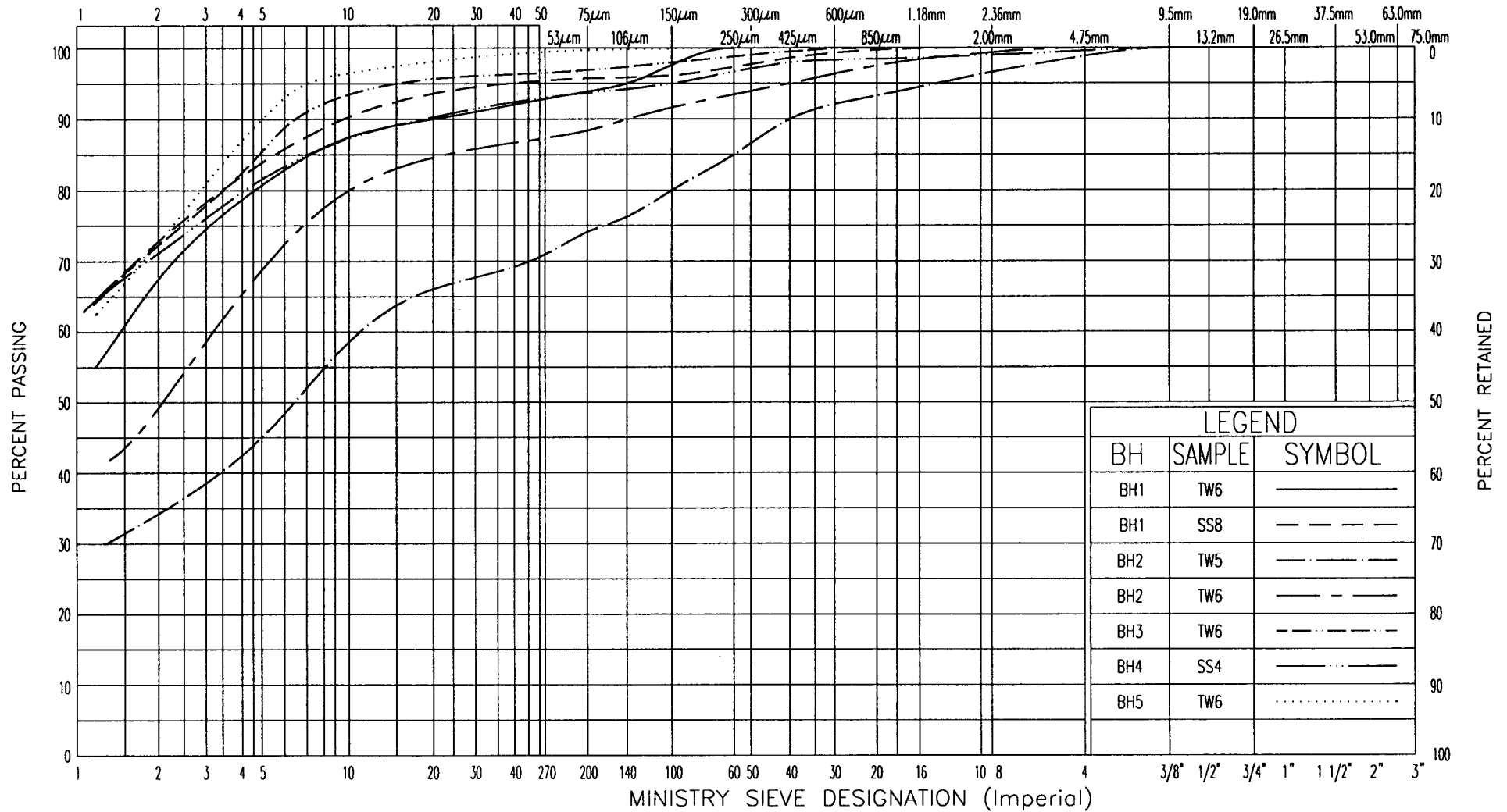


UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



GRAIN SIZE DISTRIBUTION
CLAY

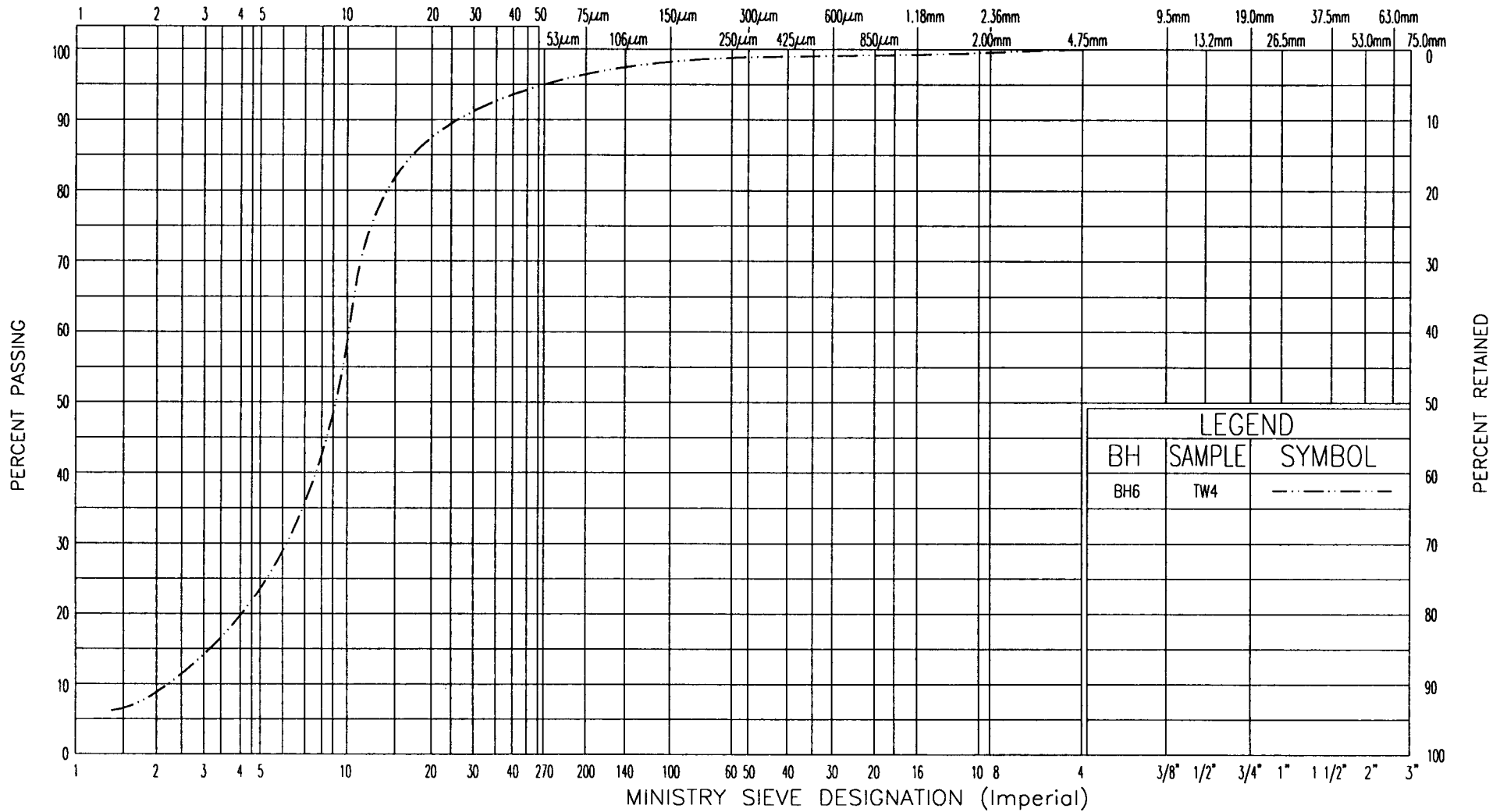
FIG. No 3
W. P. 108-99-00

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL	
		Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)

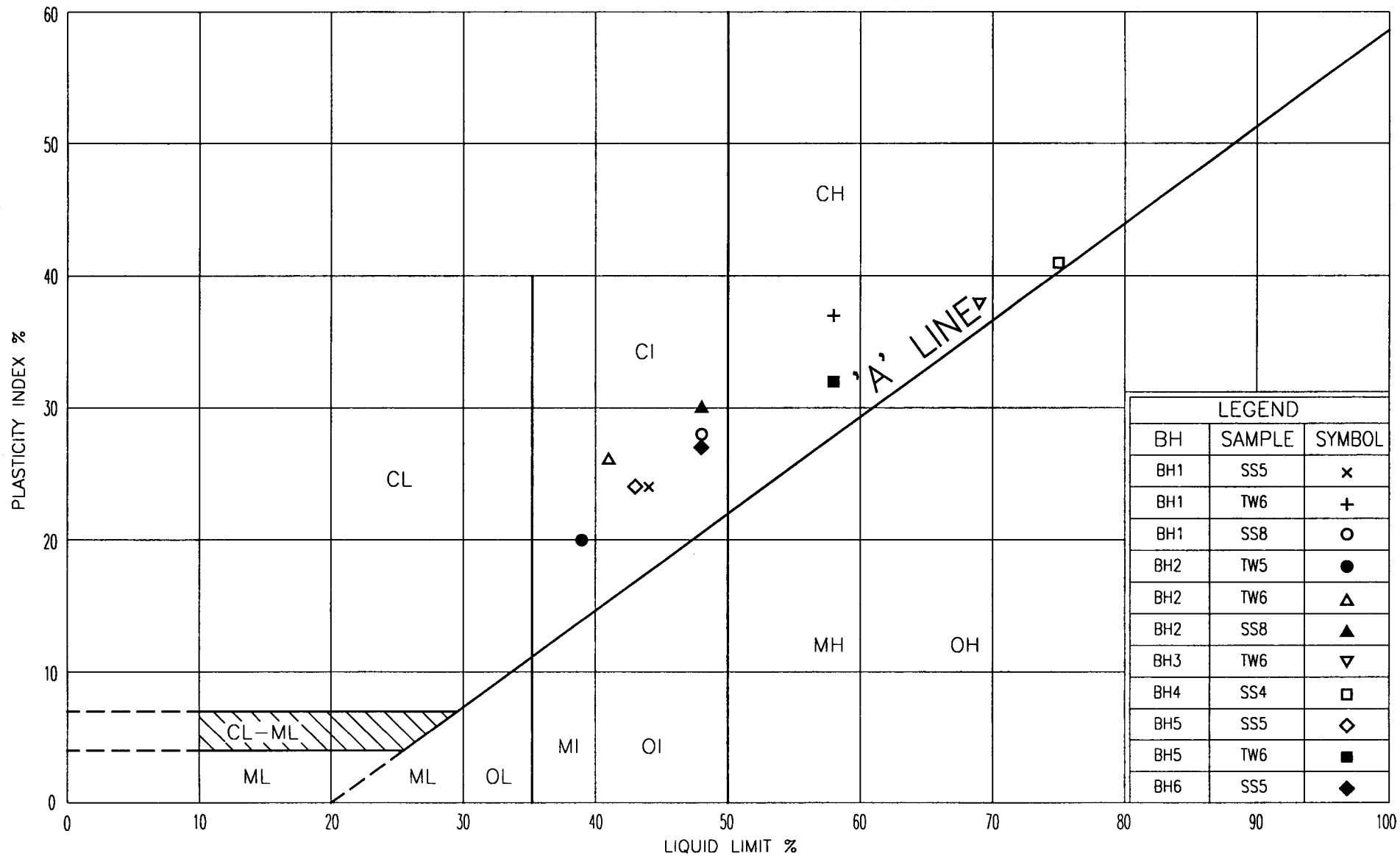


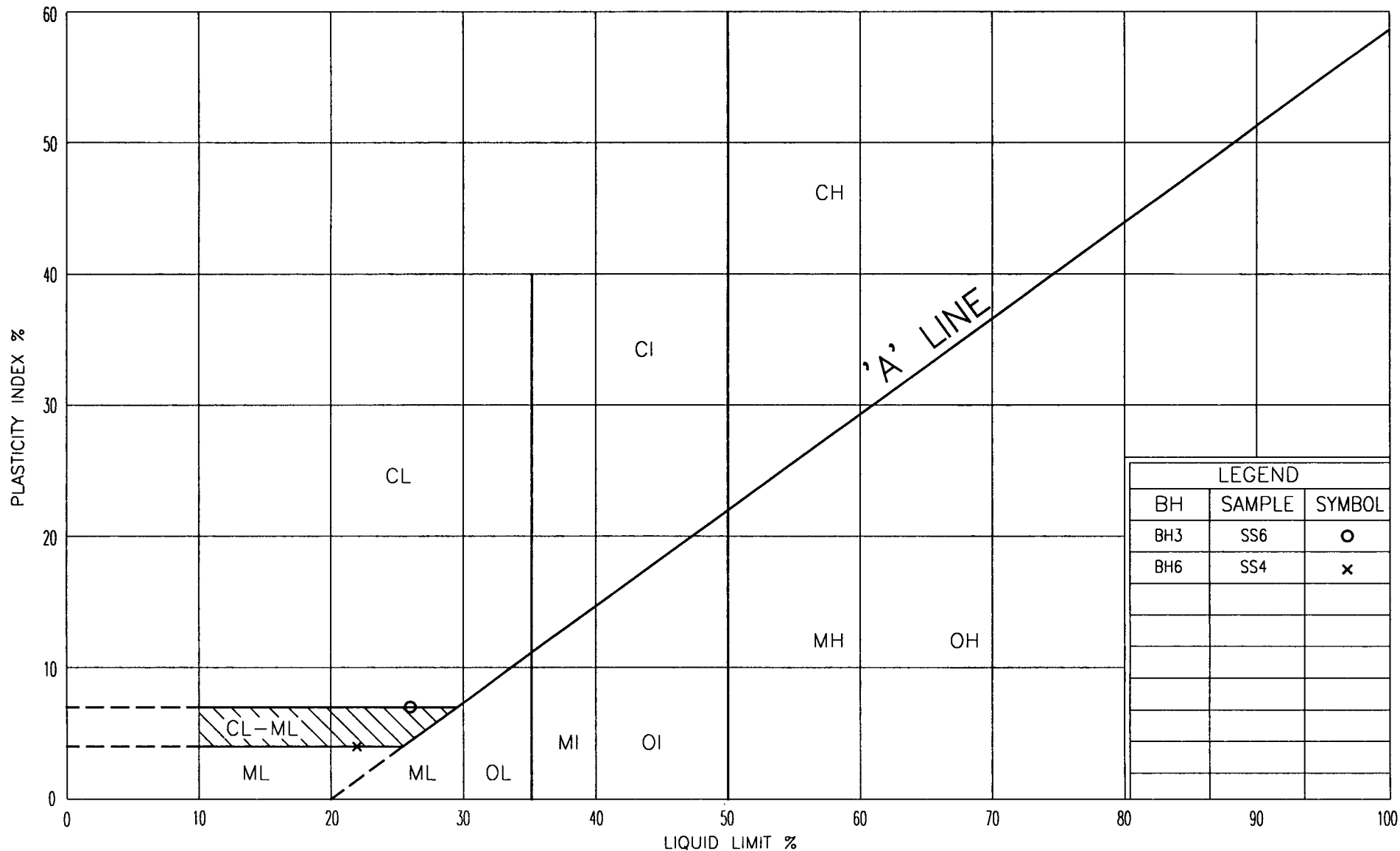
LEGEND		
BH	SAMPLE	SYMBOL
BH6	TW4	-----



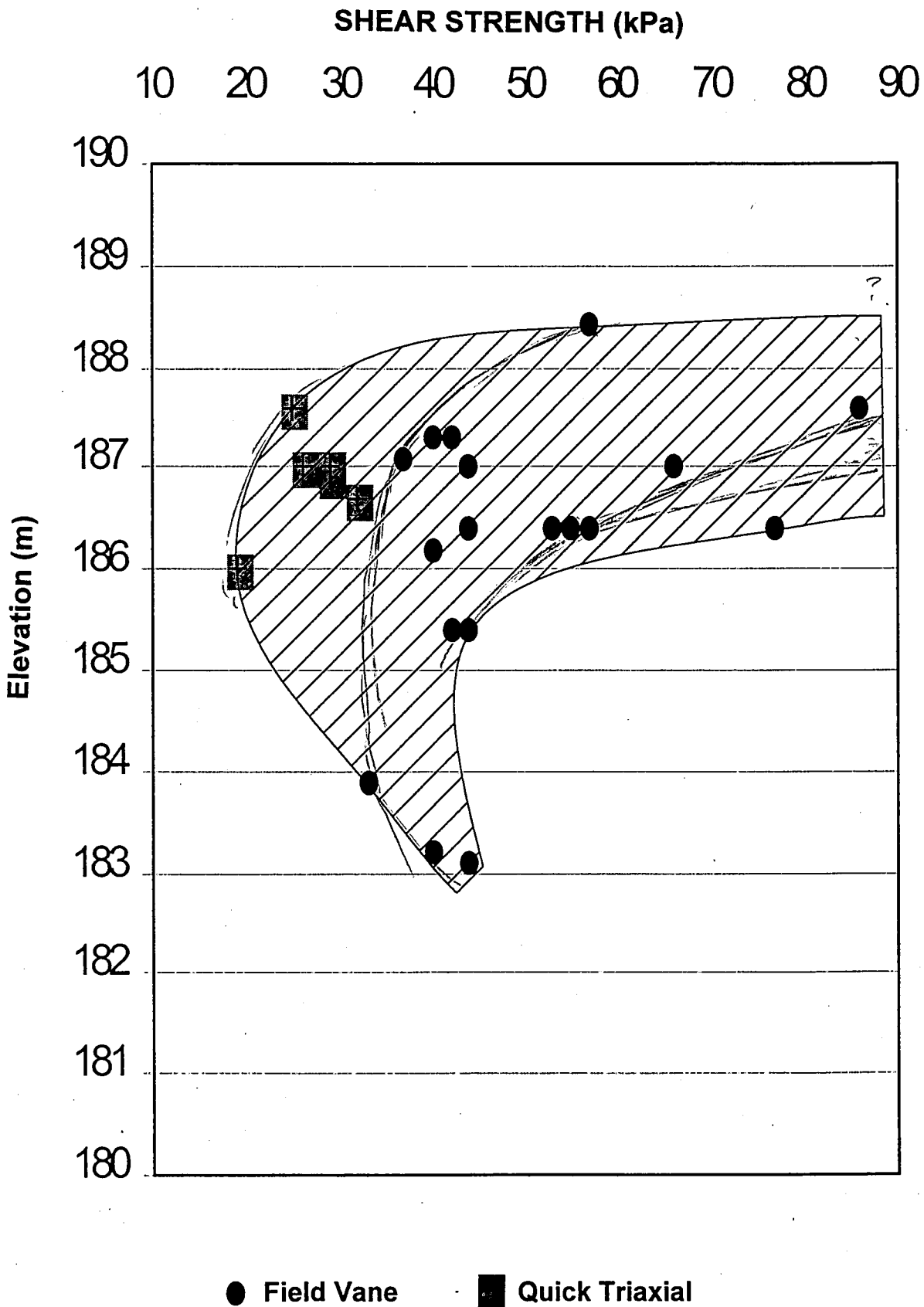
GRAIN SIZE DISTRIBUTION
CLAYEY SILT

FIG. No 4
W. P. 108-99-00





**FIGURE NO. 7: FIELD VANE AND
QUICK TRIAXIAL TEST RESULTS**



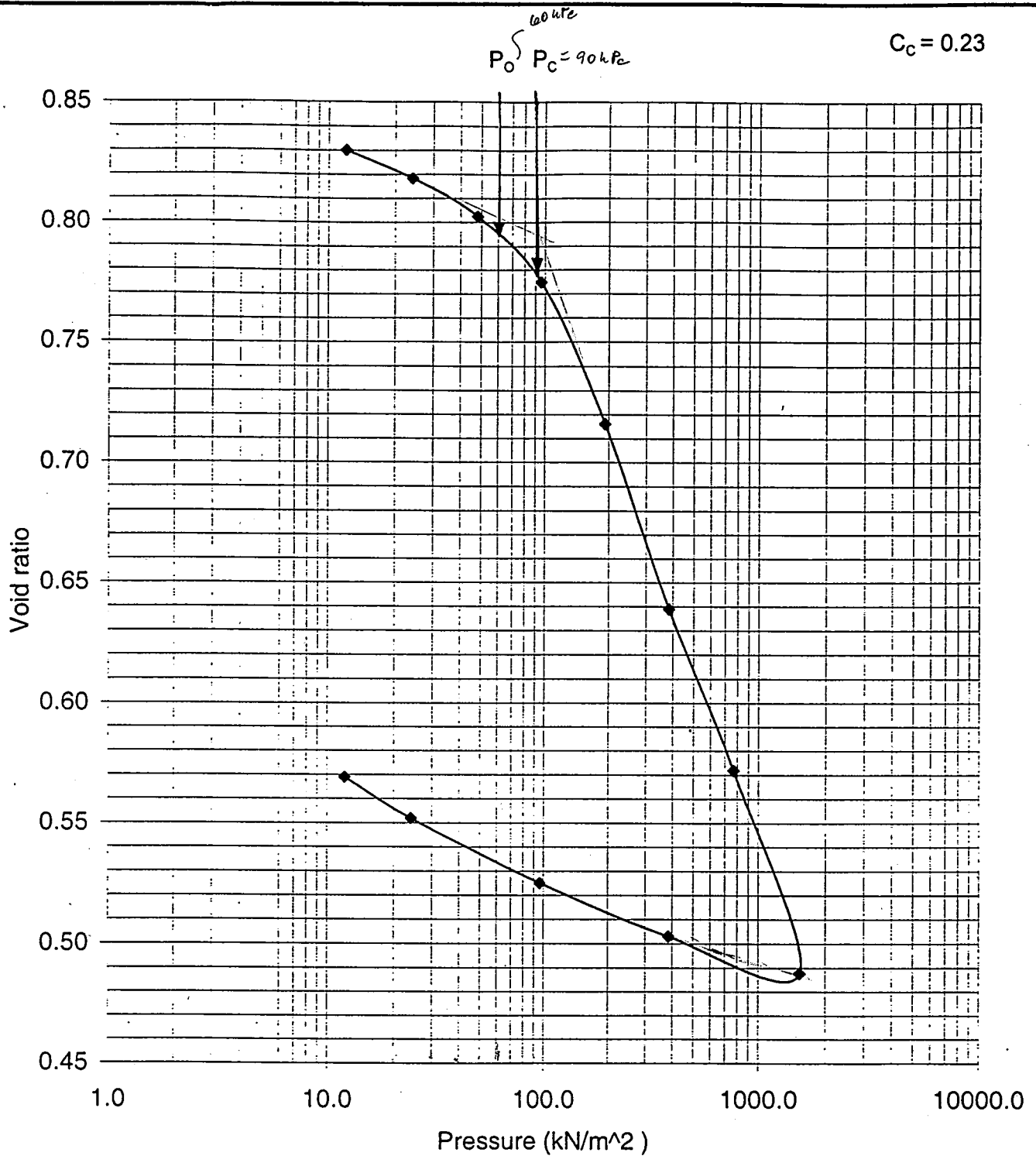
Ring #: 1 Ring Height (in) = 0.71 Wt of dry filter paper (g) = 0.66
 Wet soil + Ring Wt (g) = 122.57 Wt of ring (g) = 59.09
 Wet soil + Wet Paper + Ring (g) = 120.31 Wet Paper (g) = 1.85
 Dry Soil + Dry Paper + Ring (g) = 107.91 Ring Dia (in) = 1.900
 Initial moisture Content (%) = 31.811 Final moisture Content (%) = 23.277
 Area of Ring (in²) = 2.835 Initial Volume (in³) = 2.0131
 Initial Bulk Density (kg/m³) = 1924 Initial Dry Density (kg/m³) = 1460
 Specific Gravity of Soil = 2.7 Eqiv. Thick. of solids (mm) = 9.751
 Final gauge reading for Load 1 = Gauge reading for last Loading =

Trial #	1	2	3	4	5	6	7
Load (tsf)	0	0.125	0.25	0.5	1.0	2.0	4.0
Gauge Reading (in)	0.2597	0.2523	0.24775	0.24165	0.2311	0.20855	0.1789
(H-Hs) mm	8.283	8.095	7.979	7.824	7.556	6.984	6.230
Voids ratio	0.849	0.830	0.818	0.802	0.775	0.716	0.639
t ₉₀ (min)			2.56	3.80	3.42	3.24	3.24
C _v (ft ² /day)			0.406	0.269	0.292	0.294	0.271
k (tsf)		11.993	19.505	29.098	33.649	31.486	47.892
M _v (ft ² / ton)		0.0834	0.0513	0.0344	0.0297	0.0318	0.0209

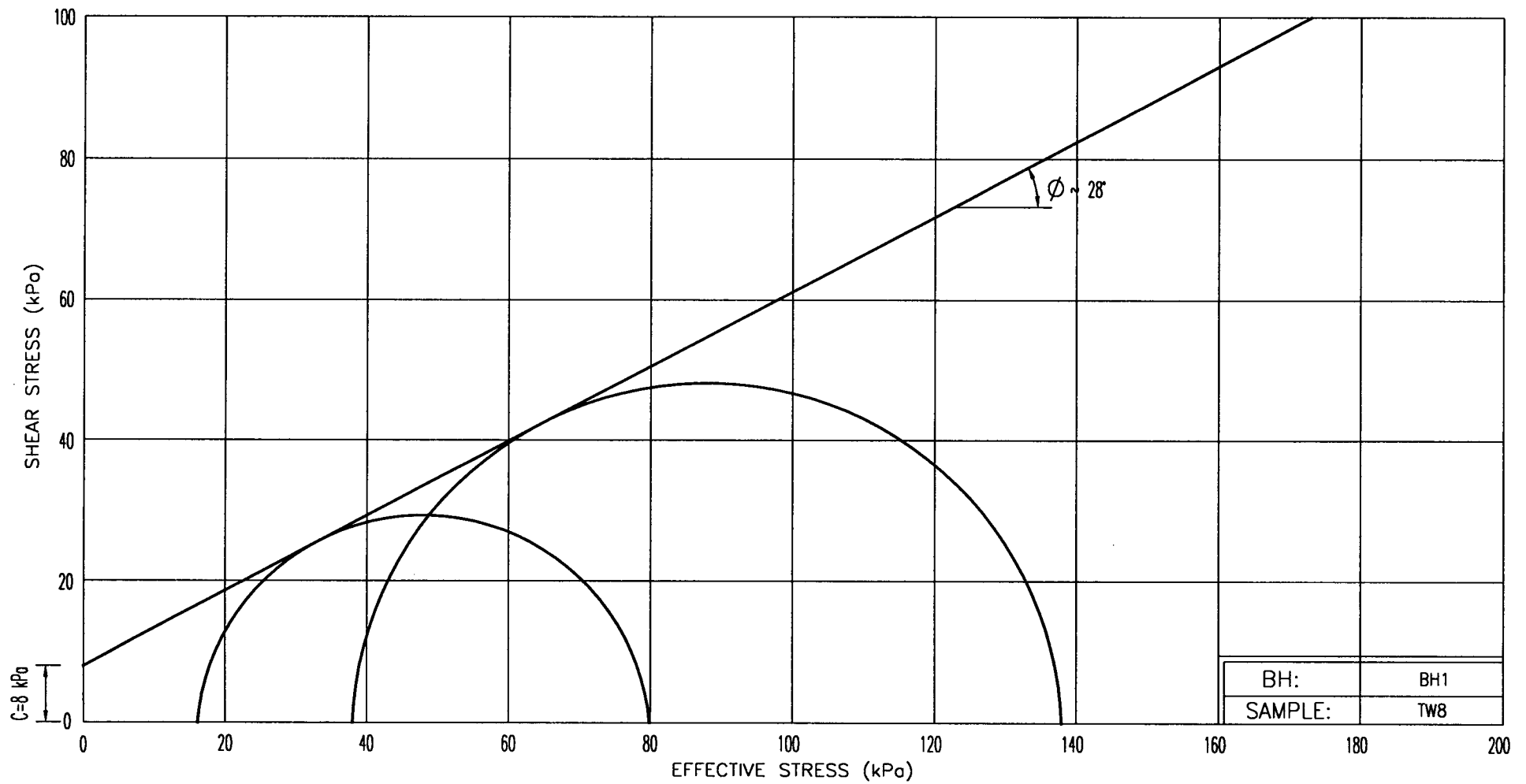
Trial #	8	9	10	11	12	13	14
Load (tsf)	8.0	16.0		4.0		1.0	
Gauge Reading (in)	0.1532	0.1208		0.1267		0.1352	
(H-Hs) mm	5.576	4.755		4.905		5.120	
Voids ratio	0.572	0.488		0.503		0.525	
t ₉₀ (min)	3.8025	1.32					
C _v (ft ² /day)	0.212	0.553					
k (tsf)	110.291	175.580		1444.068		250.588	
M _v (ft ² / ton)	0.0091	0.0057		0.0007		0.0040	

Trial #	15	16
Load (tsf)	0.25	0.125
Gauge Reading (in)	0.1455	0.1520
(H-Hs) mm	5.382	5.547
Voids ratio	0.552	0.569
t ₉₀ (min)		
C _v (ft ² /day)		
k (tsf)	51.699	13.654
M _v (ft ² / ton)	0.0193	0.0732

AGRA EARTH AND ENVIRONMENTAL LIMITED		
November, 1999		Borehole 2
		TW 5
CONSOLIDATION TEST RESULTS		
Ref. No. TT 99853	CLAY TO SILTY CLAY	Figure No. 8



AGRA EARTH AND ENVIRONMENTAL LIMITED		
November, 1999		Borehole 2
		TW 5
CONSOLIDATION TEST RESULTS		
Ref. No. TT 99853	CLAY TO SILTY CLAY	Figure No. 8



BH:	BH1
SAMPLE:	TW8

Ring # : 5 Ring Height (in) = 0.76 Wt of dry filter paper (g) = 0.66
 Wet soil + Ring Wt (g) = 128.31 Wt of ring (g) = 56.83
 Wet soil + Wet Paper + Ring (g) = 127.83 Wet Paper (g) = 1.94
 Dry Soil + Dry Paper + Ring (g) = 114.96 Ring Dia (in) = 1.895
 Initial moisture Content (%) = 24.378 Final moisture Content (%) = 20.167
 Area of Ring (in²) = 2.820 Initial Volume (in³) = 2.1435
 Initial Bulk Density (kg/m³) = 2035 Initial Dry Density (kg/m³) = 1636
 Specific Gravity of Soil = 2.7 Equiv. Thick. of solids (mm) = 11.698
 Final gauge reading for Load 1 = 0.3159 Gauge reading for last Loading = 0.2434

Trial #	1	2	3	4	5	6	7
Load (tsf)	0	0.1	0.25	0.75	1.0	2.0	4.0
Gauge Reading (in)	0.3159	0.298	0.2941	0.28965	0.28295	0.2738	0.2653
(H-Hs) mm	7.606	7.152	7.053	6.940	6.769	6.537	6.321
Voids ratio	0.650	0.611	0.603	0.593	0.579	0.559	0.540
t ₉₀ (min)			1.44	1.21	1.00	1.44	3.61
Cv (ft ² /day)			0.807	0.949	1.131	0.768	0.299
k (tsf)		4.246	29.231	85.393	28.358	83.060	178.824
Mv (ft ² / ton)		0.2355	0.0342	0.0117	0.0353	0.0120	0.0056

Trial #	8	9	10	11	12	13	14
Load (tsf)	8.0	16.0	8.0	4.0	2.0	1.0	0.5
Gauge Reading (in)	0.2567	0.2434		0.2486		0.2540	
(H-Hs) mm	6.101	5.765		5.897		6.033	
Voids ratio	0.522	0.493		0.504		0.516	
t ₉₀ (min)	1.96	1.44					
Cv (ft ² /day)	0.538	0.709					
k (tsf)	351.445	458.868		1753.846		426.168	
Mv (ft ² / ton)	0.0028	0.0022		0.0006		0.0023	

Trial #	15	16
Load (tsf)	0.25	0.10
Gauge Reading (in)	0.2582	0.2603
(H-Hs) mm	6.141	6.194
Voids ratio	0.525	0.530
t ₉₀ (min)		
Cv (ft ² /day)		
k (tsf)	134.118	54.286
Mv (ft ² / ton)	0.0075	0.0184

AGRA EARTH AND ENVIRONMENTAL LIMITED

December, 1999

Borehole 6

TW 4

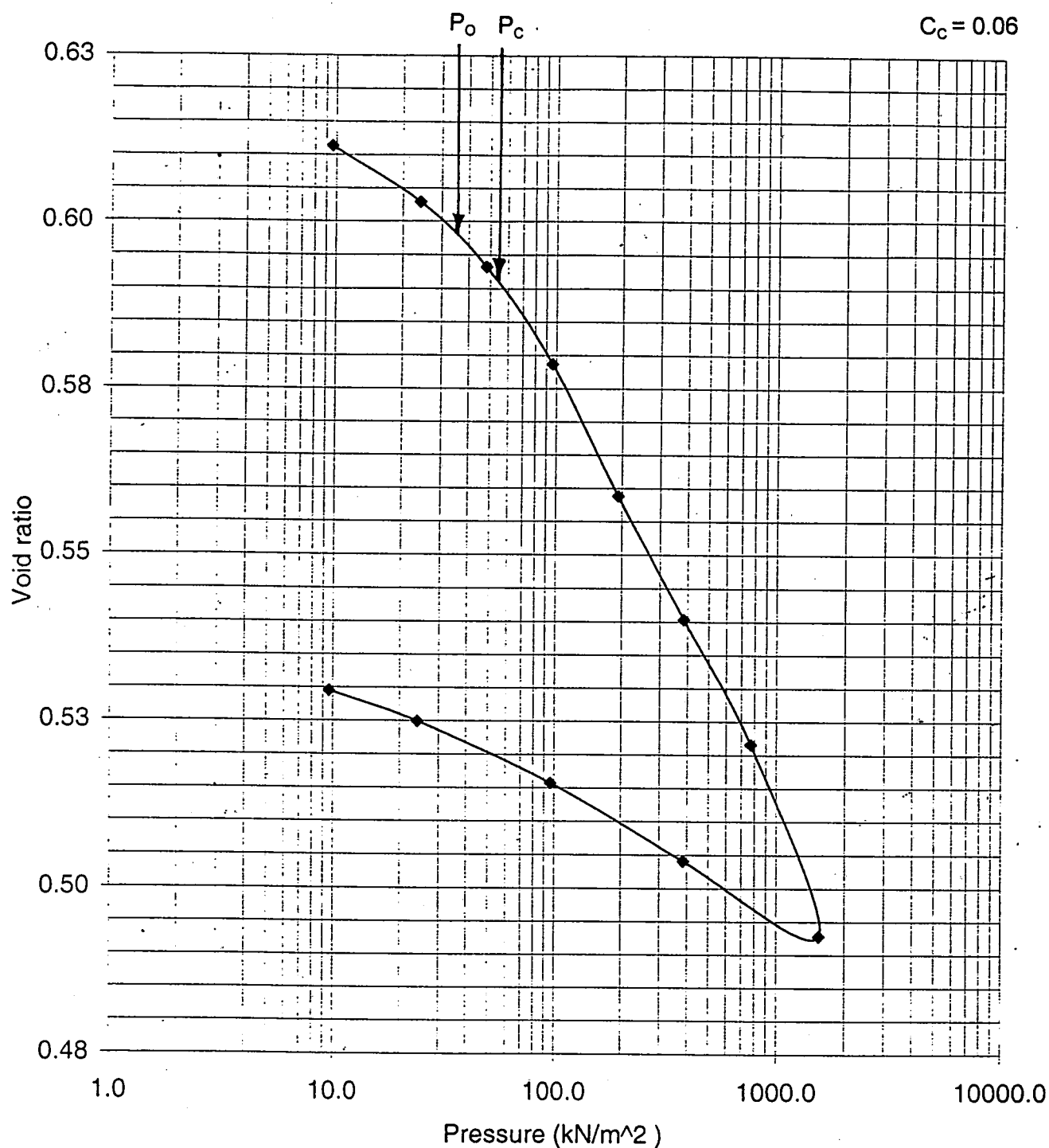
CONSOLIDATION TEST RESULTS

Ref. No. TT 99853

SILTY CLAY WITH SILT

Figure No. 10

$C_c = 0.06$



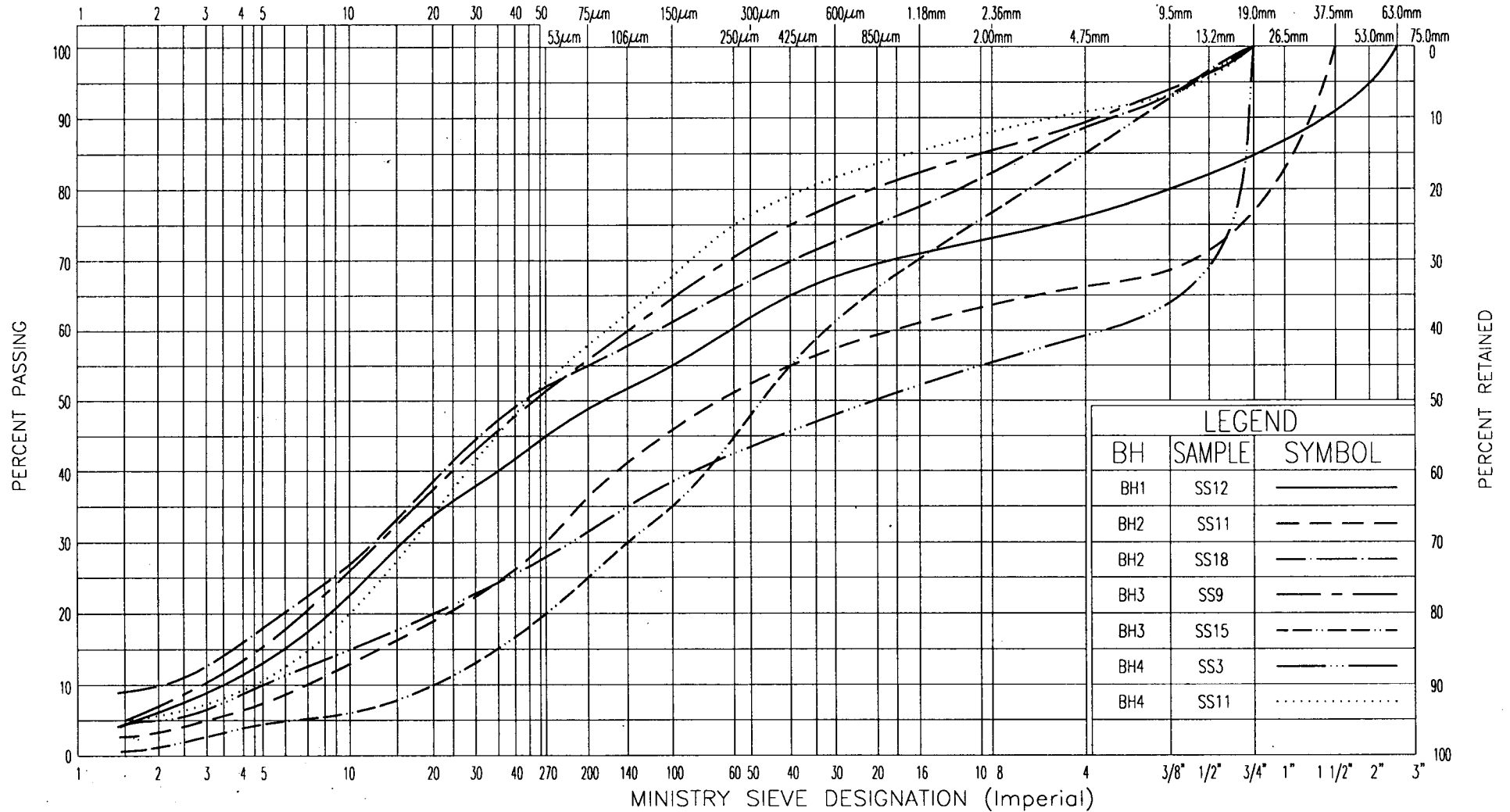
AGRA EARTH AND ENVIRONMENTAL LIMITED		
December, 1999		Borehole 6
		TW 4
CONSOLIDATION TEST RESULTS		
Ref. No. TT 99853	SILTY CLAY WITH SILT	Figure No. 10

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

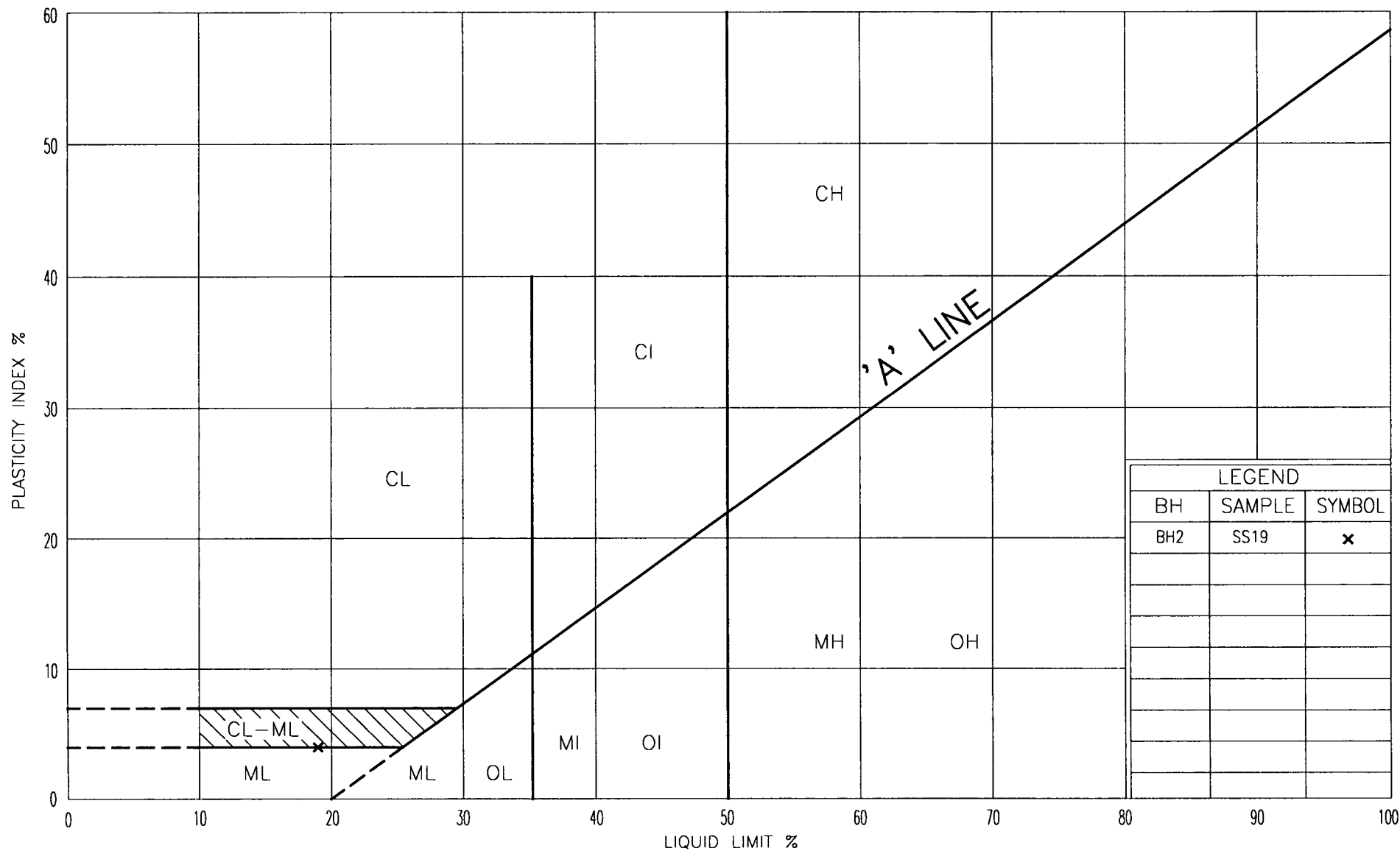
GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)

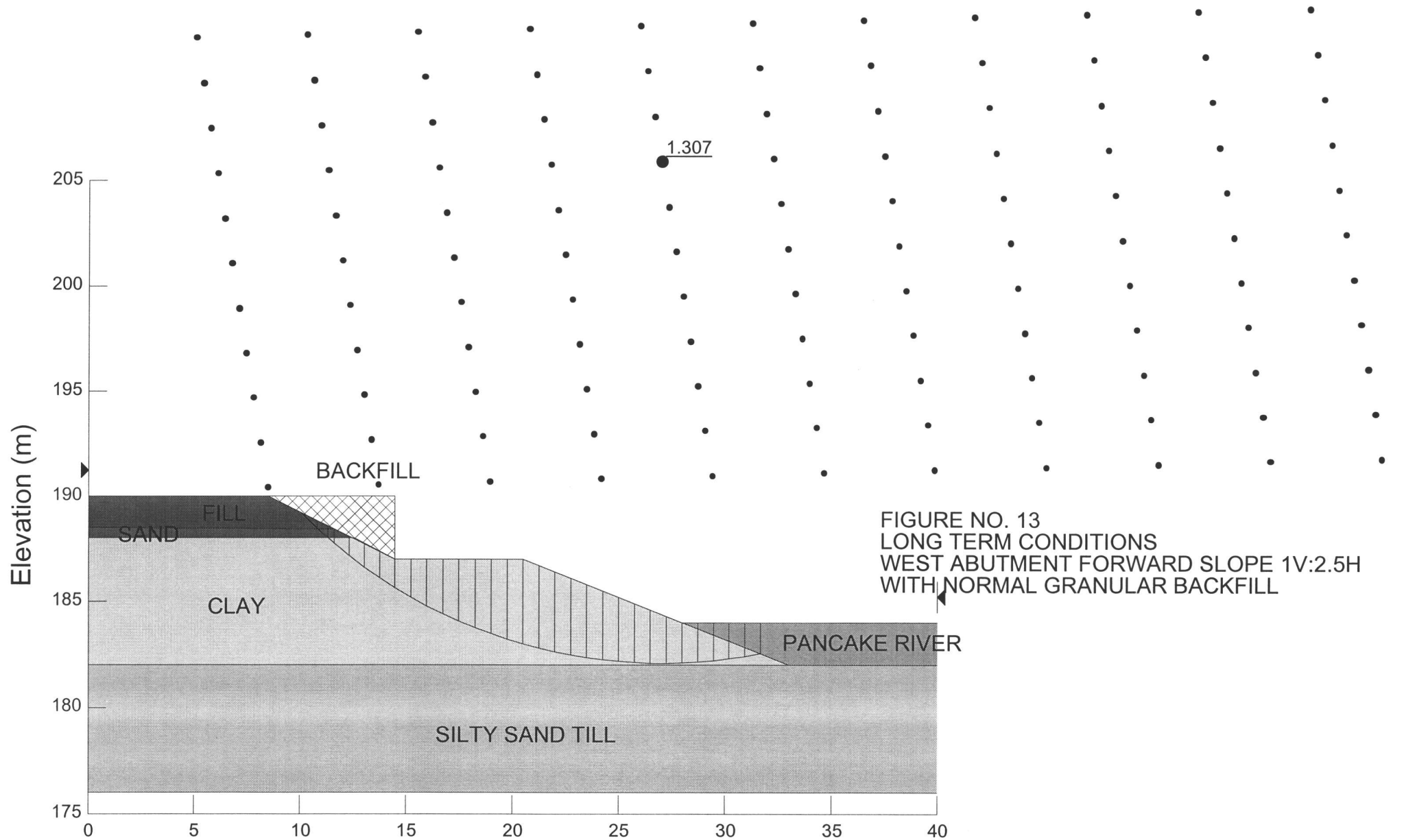


GRAIN SIZE DISTRIBUTION
HETEROGENEOUS MIXTURE OF SAND, SILT & GRAVEL
(GLACIAL TILL)

FIG. No 11
W. P. 108-99-00



LEGEND		
BH	SAMPLE	SYMBOL
BH2	SS19	x



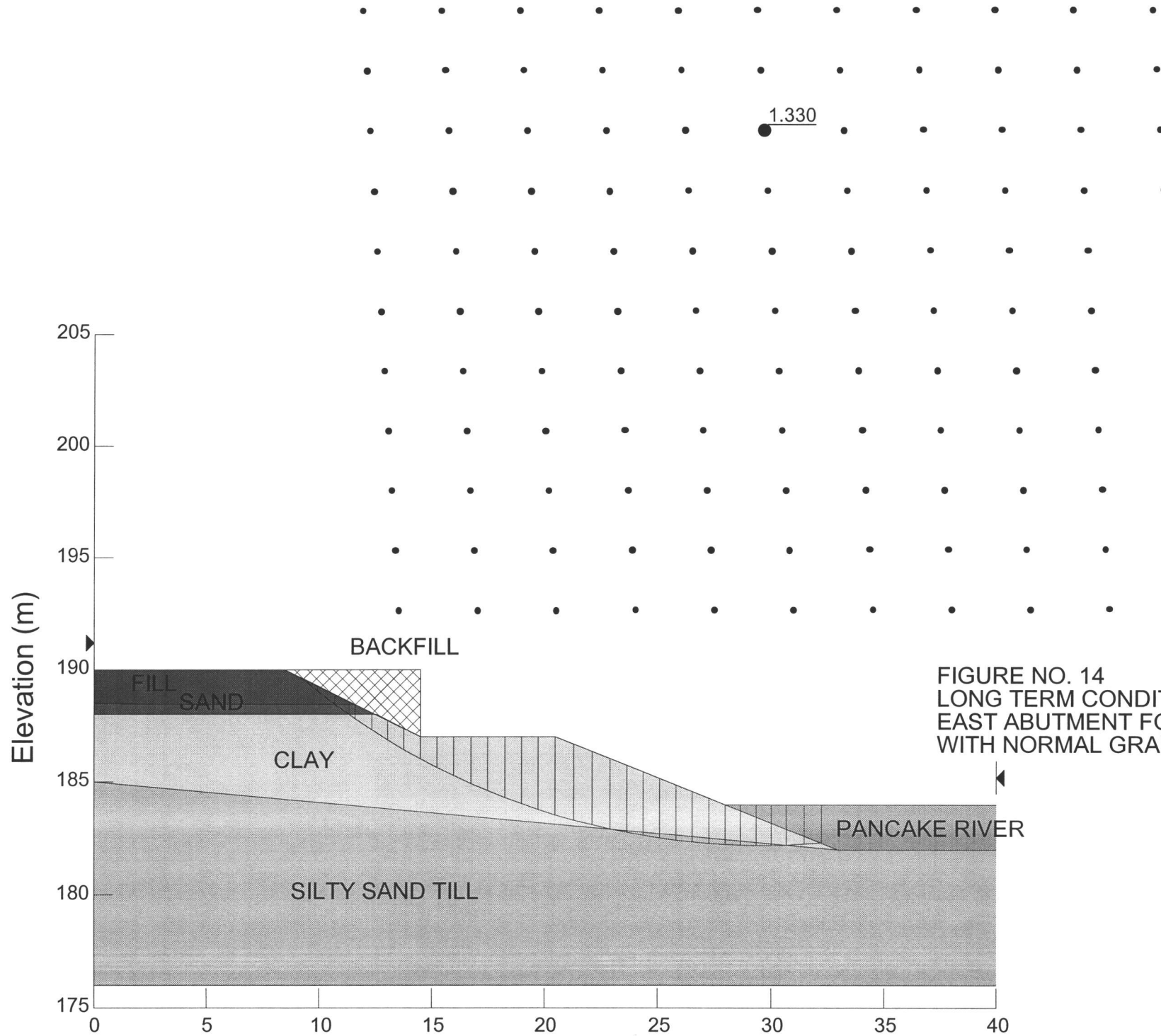
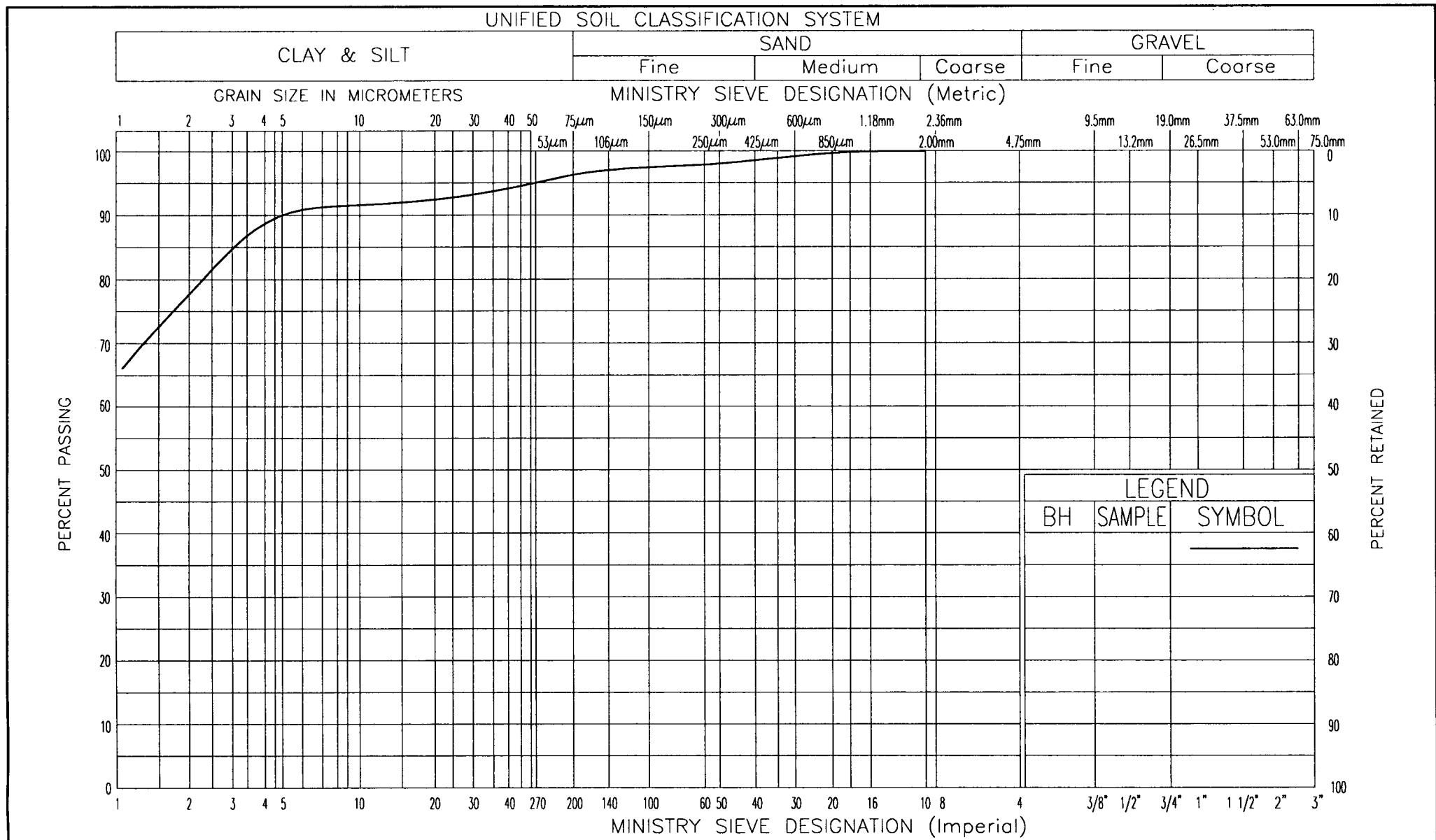


FIGURE NO. 14
LONG TERM CONDITIONS
EAST ABUTMENT FORWARD SLOPE 1V:2.5H
WITH NORMAL GRANULAR BACKFILL



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

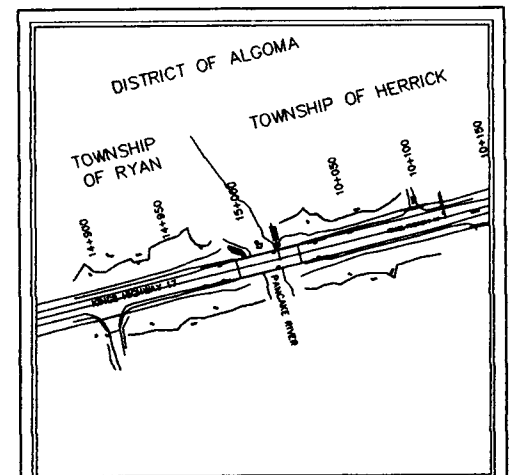
CONT. No.
W.P. No. 108-99-00



PANCAKE RIVER BRIDGE
BORE HOLE LOCATIONS & SOIL STRATA

SHEET

AGRA Earth & Environmental Ltd.



KEY PLAN

25m 0 25m 50m 75m

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)
- WL at time of investigation- Sept. 1999
- WL in Piezometer
- Piezometer

No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
BH1	190.0	5 202 623	254 816
BH2	190.0	5 202 613	254 818
BH3	190.0	5 202 599	254 867
BH4	190.0	5 202 607	254 871
BH5	190.2	5 202 629	254 802
BH6	190.0	5 202 601	254 884

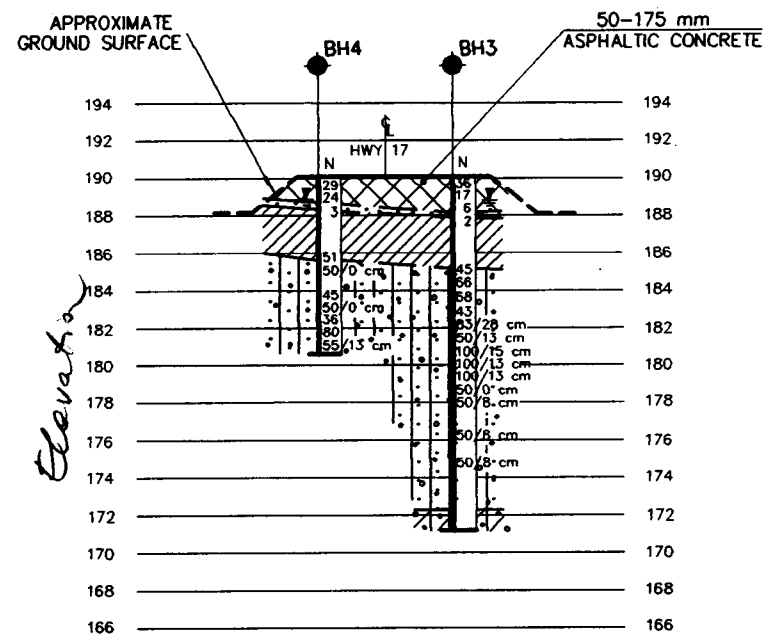
-NOTE-

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

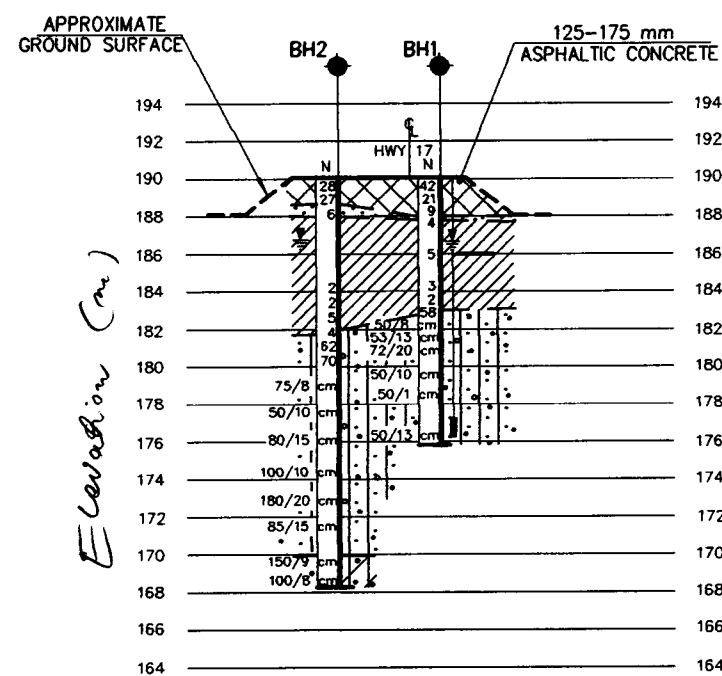
NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen.Cond.

REV	DATE	BY	DESCRIPTION
-----	------	----	-------------

HWY No 17	CHECKED EC	DATE November, 1999	DIST 62
SUBM'D AD	CHECKED	SITE 389-5	
DRAWN MA	CHECKED	DWG 1	



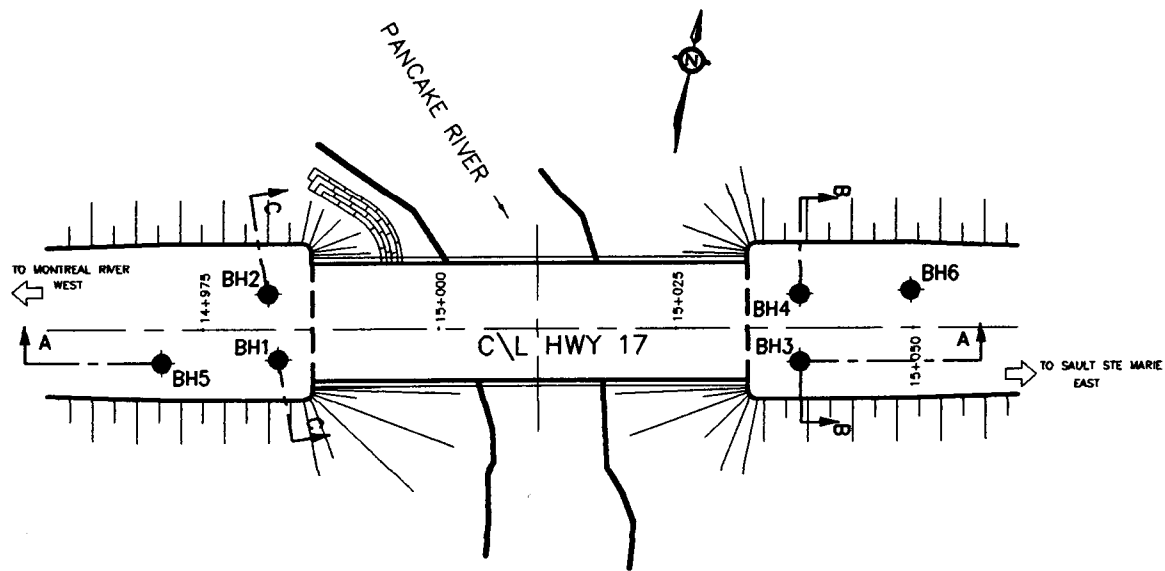
SECTION B-B



SECTION C-C

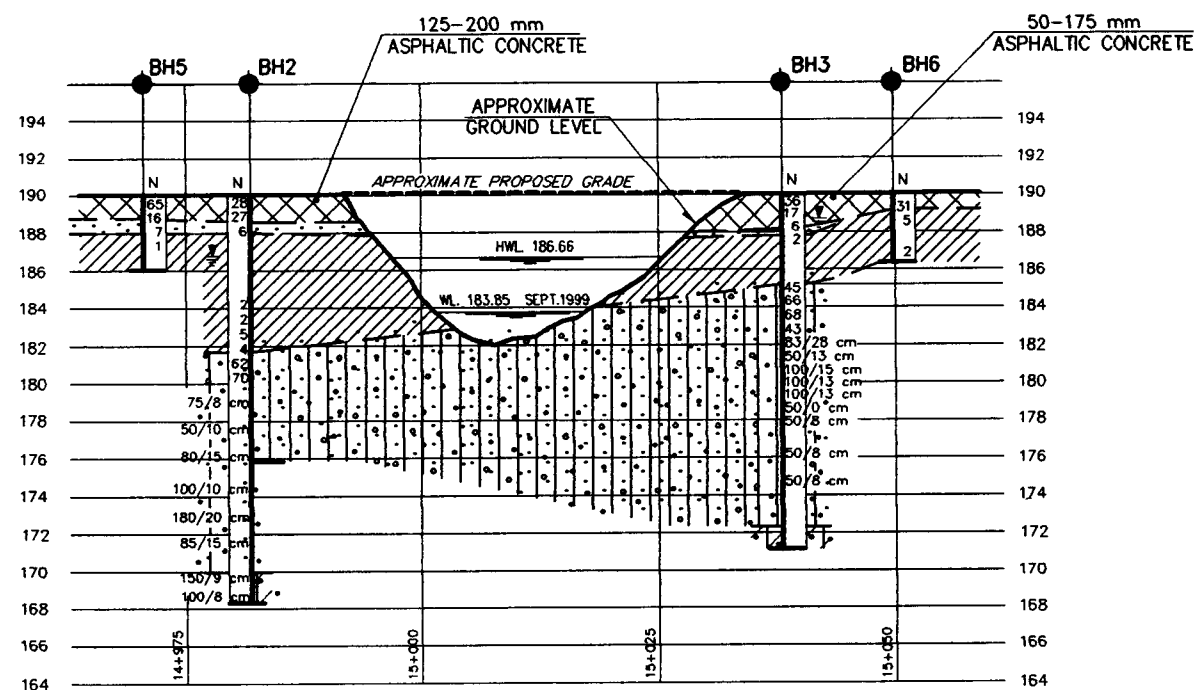
SOIL STRATIGRAPHY LEGEND

- Sandy Gravel to Sand FILL
Loose to Very Dense
- CLAY
Frequent Silt Pockets
Soft to Stiff
- SAND
Very Loose to Loose
- HETEROGENEOUS MIXTURE OF
SAND, SILT & GRAVEL
(GLACIAL TILL)
Frequent Cobbles & Boulders
Dense to Very Dense
- HETEROGENEOUS MIXTURE OF
SILT, CLAY & GRAVEL
(GLACIAL TILL)
Hard



PLAN

4m 0 4m



SECTION A-A

4m 0 4m HOR
2m 0 2m VER

ENCLOSURES

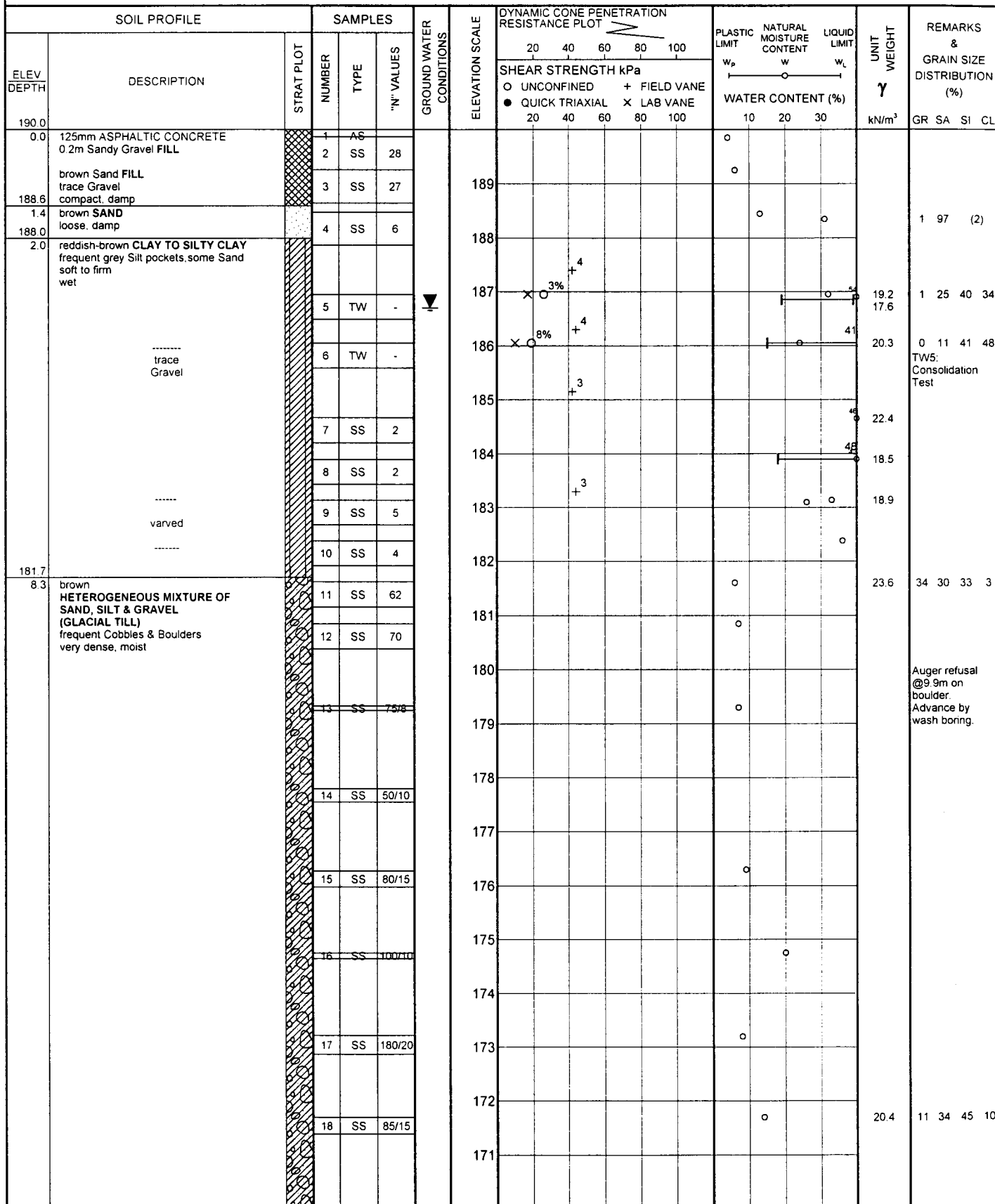
RECORD OF BOREHOLE No 1										1 OF 1		METRIC	
W.P. 108-99-00		LOCATION N 5202622.8 E 254815.6				ORIGINATED BY MA							
DIST 62 HWY 17		BOREHOLE TYPE Hollow Stem Augering/Wash Boring				COMPILED BY AD							
DATUM Geodetic		DATE 21 September 1999				CHECKED BY SP							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		SHEAR STRENGTH kPa						
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
190.0								20 40 60 80 100					
0.0	175mm ASPHALTIC CONCRETE 0.3m Sandy Gravel FILL		1	AS									
	brown Sand FILL trace Gravel loose to dense, damp		2	SS	42								
			3	SS	21								
188.0			4	SS	9								
187.6	brown SAND, damp											20.3	
2.2	reddish-brown CLAY TO SILTY CLAY frequent grey Silt pockets, trace Sand, wet		5	SS	4								
	firm		6	TW	-								0 6 27 67
	stiff												TW6: CU test
	trace Gravel		7	SS	5								
			8	SS	3							18.0	0 4 23 73
			9	SS	2								
182.9			10	SS	58								
7.1	brown HETEROGENEOUS MIXTURE OF SAND, SILT & GRAVEL (GLACIAL TILL) frequent Cobbles & Boulders very dense, damp to moist		11	SS	50/8								
			12	SS	50/13								24 28 42 6
			13	SS	72/20							22.8	Auger refusal @9.9m on boulder. Advance using core drilling and wash boring.
	boulder		14	RC									
	boulder		15	SS	50/10								
			16	SS	50/1								
	boulder		17	RC									RC17: REC=100% R.Q.D.=100%
	boulder		18	RC									RC18: REC=100% R.Q.D.=90%
	boulder												
175.8			19	SS	50/13								
14.2	END OF BOREHOLE												
	Piezometer installed @13.4m Water Level Sept.22/99: 3.3m Sept.24/99: 3.4m Sept.27/99: 3.1m												
	Piezometer installed @8.3m Water Level Sept.22/99: 3.4m Sept.24/99: 3.4m Sept.27/99: 3.1m												

RECORD OF BOREHOLE No 2

1 OF 2

METRIC

W.P. 108-99-00 LOCATION N 5202613.5 E 254818.2 ORIGINATED BY MA
 DIST 62 HWY 17 BOREHOLE TYPE Hollow Stem Augering/Wash Boring COMPILED BY AD
 DATUM Geodetic DATE 26 September 1999 CHECKED BY SP



RECORD OF BOREHOLE No 2

2 OF 2

METRIC

W.P. 108-99-00 LOCATION N 5202613.5 E 254818.2 ORIGINATED BY MA
 DIST 62 HWY 17 BOREHOLE TYPE Hollow Stem Augering/Wash Boring COMPILED BY AD
 DATUM Geodetic DATE 26 September 1999 CHECKED BY SP

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			SHEAR STRENGTH kPa							WATER CONTENT (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
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1 OF 1

METRIC

+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

RECORD OF BOREHOLE No 4

1 OF 1

METRIC

W.P. 108-99-00 LOCATION N 5202606.9 E 254870.8 ORIGINATED BY MA
DIST 62 HWY 17 BOREHOLE TYPE Solid Stem Augering/Wash Boring COMPILED BY AD
DATUM Geodetic DATE 23 September 1999 CHECKED BY SP

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					
190.0	100mm ASPHALTIC CONCRETE		1	AS	-									
0.0	0.4m Sandy Gravel FILL		2	SS	29									
188.6	brown Sand FILL with Gravel, trace topsoil pockets compact, damp to moist		3	SS	24		189							
188.2	brown SAND trace Silt, very loose, wet		4	SS	3		188							
188.2	10 mm Organic layer reddish-brown CLAY TO SILTY CLAY frequent grey Silt pockets, trace Sand wet						187	3						
185.6		firm	5	AS	-		186							
185.6		stiff					185	14						
185.6	grey HETEROGENEOUS MIXTURE OF SAND, SILT & GRAVEL (GLACIAL TILL) frequent Cobbles & Boulders dense to very dense moist to wet		6	SS	51		184							
4.4			7	SS	50/0		183							
			8	SS	45		182							
			9	SS	50/0		181							
			10	SS	36									
			11	SS	80									
180.6			12	SS	55/13									
9.4	END OF BOREHOLE													
	Water Level in open hole on completion: Sept.24/99 1.1m													

RECORD OF BOREHOLE No 5

1 OF 1

METRIC

W.P. 108-99-00 LOCATION N 5202629.3 E 254801.5 ORIGINATED BY MA
DIST 62 HWY 17 BOREHOLE TYPE Hollow Stem Augering COMPILED BY AD
DATUM Geodetic DATE 24 September 1999 CHECKED BY SP

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					
190.2	200mm ASPHALTIC CONCRETE		1	AS	-		190							
0.0	brown Sandy Gravel FILL		2	SS	65									
189.5	very dense, damp													
0.7	brown Sand FILL		3	SS	16		189							
188.8	compact, damp													
1.4	brown SAND		4	SS	7		188							1 98 (1)
188.0	loose, damp													
2.2	reddish-brown CLAY TO SILTY CLAY		5	SS	1		187							
	occasional grey Silt pockets, trace Sand													
	firm wet		6	TW	-									
186.2														
4.0	END OF BOREHOLE													

RECORD OF BOREHOLE No 6

1 OF 1

METRIC

W.P. 108-99-00 LOCATION N 5202601.3 E 254884.4 ORIGINATED BY MA
DIST 62 HWY 17 BOREHOLE TYPE Hollow Stem Augering COMPILED BY AD
DATUM Geodetic DATE 24 September 1999 CHECKED BY SP

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 (%)			
								○ UNCONFINED	+ FIELD VANE							● QUICK TRIAXIAL	x LAB VANE	
190.0							20	40	60	80	100							
0.0	50mm ASPHALTIC CONCRETE		1	AS	-													
189.2	0.2m Sandy Gravel FILL brown Sand FILL		2	SS	31													
0.8	reddish-brown CLAY TO SILTY CLAY frequent grey Silt pockets, trace Sand moist to wet		3	SS	5													
			4	TW	-													
			5	SS	2													
186.4	firm to stiff																	
3.7	END OF BOREHOLE																	