DOCUMENT MICROFILMING IDENTIFICATION

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GEOCRES No. 30M12-240

DIST. CR REGION

W.P. No. 130-99-00(A)

CONT. No.

W. O. No.

STR. SITE No.

HWY. No. 410

LOCATION SANDALWOOD PARKWAY UNDERPASS

No of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

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Transmittal Record

Date: September 13, 2000

Proj. No.: W9921802

WP No. 130-99-00

To:
Ministry of Transportation, Ontario
Pavement and Foundations Office
Room 223
Central Building
1201 Wilson Avenue – Bldg. 'C'
Downsview, ON M3M 1J8

Project
Hwy 410 Extension, Bovaird Drive to Sandalwood Parkway
Location
City of Brampton
Owner
MTO

Attention: Ms. B. Bennett, P. Eng.

We are sending you by courier the following documents:

	No. of Copies	Date of Issue
Highway 410 Extension (Phase 1) 1:1000 Roll Plan	1	N/a

Comments: For your use in revising Borehole & Soil Strata drawings.
Please forward revised mylars directly to the undersigned for incorporation with contract drawing package.

B. R. Bridges.

From: Brian Bridges, P.Eng.
Manager, Bridge Engineering

Distribution: D. Kemper, MTO Highway Engineering
S. Chiu S. Broe
Project File

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FOUNDATION DESIGN REPORT
PROPOSED UNDERPASS STRUCTURE AT SANDALWOOD PARKWAY
HIGHWAY 410 EXTENSION
FROM BOVAIRD DRIVE TO SANDALWOOD PARKWAY
BRAMPTON, ONTARIO
W.P. 130-99-00 (A)

Prepared For:

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FOUNDATION INVESTIGATION REPORT

FOUNDATION INVESTIGATION REPORT PROPOSED UNDERPASS STRUCTURE AT SANDALWOOD PARKWAY INTERCHANGE HIGHWAY 410 EXTENSION FROM BOVAIRD DRIVE TO SANDALWOOD PARKWAY, BRAMPTON WP 130-99-00 (A)

1. INTRODUCTION

As part of the northerly extension of Highway 410 in Peel Region, a new interchange is proposed at Sandalwood Parkway, immediately north of the existing Bovaird Drive interchange. The overall project entails the construction of an underpass structure at the Sandalwood Parkway interchange, along with various culverts. The project also includes several culverts at Bovaird Drive interchange and provision for high mast light poles at and in between the two interchanges. The overall project limits lie in between approximate Stations 17+350 and 19+450.

Shaheen & Peaker Limited (S&P) was retained by the Ministry of Transportation of Ontario (MTO) Pavements and Foundations Section to carry out a foundation investigation for the proposed interchange. This report deals with the proposed underpass bridge structure site. The centerline of the proposed structure is located at the approximate Station of 19+032 (at the intersection with the proposed Sandalwood Parkway extension). The investigation was carried out between approximate Stations 19+016 and 19+048 and about 28 m right and 28 m left of Highway 410 alignment centerline for the foundation locations. Two boreholes were also put down 50 m right and 50 m left of the centerline for the approach embankments. The work was performed in accordance with Consultant Assignment Agreement Number 2005-A-000142, Highway 410: Bovaird Drive to Sandalwood Parkway – Foundation Investigation and Recommendations, WP130-99-00.

The site is located immediately east of Heart Lake Road between Bovaird Drive and Sandalwood Parkway in the City of Brampton. The purpose of the investigation was to obtain information about the subsurface conditions at the site by means of boreholes.

The findings of the investigation are presented in this report.

2. SITE DESCRIPTION AND PHYSIOGRAPHY

The site is located immediately north of the existing intersection of Highway 410 and Bovaird Drive in the City of Brampton, Regional Municipality of Peel.

The land use in the area has recently changed from predominantly farming to industrial and suburban residential. The topography in the area is flat with ground surface sloping gently towards Lake Ontario. The existing ground surface elevations at the interchange sites generally range from about 247 m at the most northerly location (i.e about 500 m north of the Highway 410 and Sandalwood Parkway intersection), falling to about 246 m at the proposed underpass (bridge) site to generally between 245 and 244 m toward the south end of the project at the Bovaird Drive interchange (i.e. an elevation difference of about 2 to 3 m over a distance of about 2.1 km). There are some minor undulations within the site itself where the grade falls locally towards two small watercourses. One of these watercourses, located about 500 m north of the Bovaird Drive intersection, intersects the proposed highway alignment in an east-west direction and here the grade falls to between Elevations 243 and 240 m. The second creek runs in a north-south direction at the north end of the overall project site, some 200 m west of the proposed highway alignment, where it crosses the Sandalwood Parkway extension, turns east and crosses the highway alignment about 200 m south of the proposed bridge site. The grade along this creek generally ranges from about 243 m on the north side to about 240 m southeast.

The site is located within the physiographic region known as the "Peel Plain." In general, this region is underlain by glacial till deposits containing frequent shale and limestone fragments. Much of the surface has been modified by post-glacial, shallow silty clay/clayey silt soils. In the general project area, the significant deposit underlying shallow recent clayey soils is a ground moraine composed of glacial till of generally cohesive nature, laid down during the Wisconsinan glacial age. Silt and sand layers are often found interbedded with the glacial till. The general area is located at the interface of the red Queenston and grey Georgian Bay bedrock formations. These formations belong to the Upper Ordovician Period of the Palezoic Era and are approximately 450 million years old. These shales are interbedded with some limestone, siltstone, sandstone and dolostone layers and seams. These hard layers are usually less than about 150 mm thick but some layers are much thicker.

3. INVESTIGATION PROCEDURES

The fieldwork for the project was performed during the period of October 14 and 25, 1999 and consisted of drilling and sampling eight boreholes (Boreholes 26 through 33). An additional borehole (i.e. Borehole 26A) adjacent to Borehole 26 was also drilled in order to explore the subsurface conditions to greater depths.

The depths of the boreholes ranged from 6.6 m to 23.5 m. In addition, dynamic cone penetration tests were performed from the bottom of some of the boreholes and this increased the maximum depth of testing to 25.3 m below the ground surface. The locations of the boreholes are shown in Drawing No. 1309900A-A and also given on the individual Borehole Log sheets.

The boreholes were advanced using a track-mounted drilling rig owned and operated by Groundworks Inc., under the full time supervision of a geotechnical engineer from Shaheen & Peaker Limited. In general, solid-stem augering was utilized to advance the boreholes but where hydrostatic pressures in cohesionless soil caused excessive cave-ins and/or soil back up, drilling was switched to hollow stem augering and water was used for counter-balancing purposes. Sampling in the boreholes was effected at frequent intervals of depth (i.e. at 0.76 m intervals starting at ground surface to 6 to 9 m depth, and at 1.5 m intervals, thereafter) by the Standard Penetration test method (SPT) as outlined in ASTM Method D1586. In essence, this consists of freely dropping a 63.5 kg. hammer a vertical distance of 760 mm to drive a 51 mm O.D. split-spoon (split-barrel) sampler into the ground. The number of blows required to drive the sampler into the relatively undisturbed ground by a vertical distance of 300 mm is recorded as 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.

Dynamic cone penetration tests (DCPT) were performed in Boreholes 26A, 27 and 29. This test consists of driving a 60 degree point, 51 mm diameter cone attached to the drill rig continuously into the undisturbed ground with the same driving energy as in SPT. The number of blows for each 300 mm of penetration is recorded and this provides an indication of the relative changes in the soil density with depth.

The borehole locations in the field were established by MTO surveyors and ground surface elevations at the borehole locations were provided to us. Several of the boreholes had to be somewhat relocated from their original, staked-out location due to access difficulties. In these instances, allowance was made by our field staff for location and elevation differences.

Water level observations in the open boreholes were made during the drilling and at the completion of each borehole and, wherever possible, several hours thereafter. To enable us to monitor groundwater levels over a prolonged period of time without interference from surface water, piezometers were installed in some of the boreholes and the water level in these piezometers were monitored during subsequent site visits.

The results of drilling, sampling, in-situ testing and water level measurements are summarized on the Record of Borehole Sheets in Appendix A.

The soil samples were shipped to our geotechnical laboratory for further examination and classification. A laboratory testing programme consisting of natural moisture content, bulk unit weight and Atterberg limit tests and grain-size analyses was performed on selected soil samples. The results of the laboratory tests are presented on the appropriate Borehole Log Sheets and also in Appendix B.

4. SUBSURFACE CONDITIONS

The subsurface conditions at the proposed bridge location were explored at eight borehole locations (Boreholes 26 through 33). The locations of the boreholes are shown on the Plan and Profile Drawing No. 1309900A-A and are also indicated on the individual Borehole Log sheets (Appendix A). Cross sections of inferred subsurface stratigraphy are given in the same drawing.

The ground surface at the proposed bridge location is essentially level and ranges at the boreholes from 246.0 m at the west abutment location to about 245.7 at the east abutment location.

In general, beneath a veneer of topsoil and a surficial layer of clayey silt/silty clay, the boreholes contacted a major glacial deposit ranging in composition from cohesive clayey silt till and fine grained granular sandy silt to relatively coarser grained silty sand till with some sand, silt and silty clay interbeds.

At the time of the investigation, the groundwater table was recorded at a depth of about 5 m or at about elevation 241 m.

Details of the subsurface conditions encountered in the boreholes are presented on the Borehole Log Sheets (Appendix A). The individual soil strata are briefly described below.

4.1. TOPSOIL

The boreholes contacted 250 to 500 m of topsoil.

4.2 SURFICIAL SILTY CLAY/CLAYEY SILT

Underlying topsoil, the majority of the boreholes drilled at the bridge site contacted a surficial silty clay/clayey silt deposit with some silt seams. This deposit generally extended to depths ranging from 0.7 to 1.1 m, except in Borehole 28, where it extended deeper to 2.1 m below the ground surface.

This is a cohesive deposit and based on N-values ranging from 5 to 36 blows/0.3 m its consistency is described as firm to hard.

In some of the boreholes, the upper zones of the deposit immediately beneath the topsoil was found to be organic stained, probably due to the tilling of the soil from the farming operations.

4.3 CLAYEY SILT TILL

Clayey silt till was contacted in Borehole 29 from 1.1 (Elev. 244.8 m) to 2.9 m (Elev. 243.0 m) below the ground surface.

This unit consists of a heterogeneous unsorted mixture of silt and sand with some clay and gravel size particles. It is a cohesive deposit with sandy silt till or basically granular interbeds and occasional silt and sand seams/lenses.

Based on Standard Penetration tests, which yielded N-values ranging from 24 to 38 blows/0.3 m, the consistency of the material is described as very stiff to hard.

4.4 SANDY SILT TILL

The predominant soil stratum at the site within the upper 4 to 12 m is a glacial deposit which consists of heterogeneous unsorted mixture of silt and sand with some gravel and traces of clay size particles.

The grain size distribution of samples from the deposit is presented in Figure Nos. B1, B2 and B3, in Appendix B. The curves generally indicate 1-13 gravel, 35-46% sand, 43-61% silt and 0-3% clay size particles. The deposit contains some clayey silt till zones and the grain size distribution of samples from these zones show higher percentage of clay size particles (i.e.12% clay size particles) as shown in Figure B4 in Appendix B. The deposit also contains occasional sand and silt interbeds.

The colour of the deposit is brown to 2.7 to 3.5 m and grey below.

This is a fine grained granular deposit and from N-values which are generally in excess of 30 blows/0.3 m, it is considered dense to very dense, with occasional compact zones.

In Borehole 27 at 5.5 m depth (Elevation 240.5 m), an approximately 0.5 m thick silty clay layer was contacted. This material contains some till lenses and based on a recorded N-value of 16 blows/0.3 m, along with a visual and tactile examination of the recovered soil sample, it is considered to be relatively weaker and more compressible than the sandy silt till. A similar weaker zone (N=19) was found in Borehole 32 at about the same elevation. In addition, in Boreholes 30 and 31, drilled at the east abutment location, a slightly weaker zone was noted between Elevation 239 and 237, with recorded N-values of between 32 and 38 blows/0.3 m.

4.5 SILTY SAND TILL

In Boreholes 26, 27, 29, 30, 32, and 33 the sandy silt till attains at depths ranging from 3.4 to 8.3 m below the ground surface or at Elevation 242.6 – 237.5 m, a somewhat coarser (i.e. silty sand till) texture. The deposit extends to the full depth of these boreholes, including Borehole 26A.

The grain size distribution of samples from the deposit is given in Figures B5 and B6 in Appendix B. The analyses indicate 5-20% gravel, 36-50% sand, 30-52% silt and 1-4% clay size particles. This is a granular deposit and is considered somewhat more pervious than the overlying sandy silt till. It should be pointed out that the presence of cobbles and boulders can always be expected in the glacial till deposits, owing to their mode of deposition. For example, in Borehole 26 refusal to further augering was contacted at 10.6 m or Elevation 235.3 m on a boulder and the borehole had to be relocated and another borehole (BH26A) had to be drilled. N-values recorded in this unit are generally in excess of 30 blows/0.3 m indicating a dense to very dense condition. There are some N-values of between 20 and 30 blows/0.3 m but in most cases, these are believed to be caused by disturbance due to hydrostatic uplift. One exception to this is a weak zone encountered in Borehole 32 at a depth of 5.5 m or at about Elevation 240.5 m, where the soil is somewhat clayey and appeared to relatively be weaker. As was discussed in the preceding section, a somewhat weaker silty clay zone was contacted in Borehole 27 at about the same elevation and at a slightly lower elevation in Boreholes 30 and 31 (i.e. east abutment location).

4.6 GROUNDWATER CONDITIONS

Groundwater conditions in the open boreholes were observed while drilling and at the completion of each borehole. To enable us to monitor the groundwater levels over a prolonged period of time without interference from surface water, piezometers were installed in Boreholes 26, 29 and 30.

The observations made in the boreholes indicate that at the time of the investigation, the groundwater level at the site was at elevations ranging between 241.6 and 240.8 m, or at 4.3 to 5.1 m below the ground surface. The groundwater table can, however, be expected to be subject to seasonal fluctuations and in response to major weather events. Judging from the change of colour of the soil from brown to grey and the moisture contents of the soil samples, it is our opinion that the water level during wet seasons may rise by about 1 m.

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PROPOSED UNDERPASS STRUCTURE AT SANDALWOOD PARKWAY
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5. DISCUSSION AND RECOMMENDATIONS

5.1 THE PROJECT

The details of the proposed bridge structure have not yet been finalized. It is, however, anticipated that the underpass will be an approximately 30 m wide, two-span structure, each span being approximately 30 m long. The existing ground elevation at the proposed bridge site generally ranges from 246.0 to 245.7 m and the anticipated finished grade of the highway is approximately 2.5 m lower than the existing grade or at about elevation 243.5 m. The approach fills for the Sandalwood Parkway ramps can therefore be expected to be about 6 m high.

5.2 FOUNDATIONS

The boreholes show that the proposed bridge can be supported on normal spread footing foundations or if an integral abutment type of structure is to be considered , it can be supported on driven H-piles.

5.2.1 SPREAD FOOTING FOUNDATIONS

As the proposed highway grade is about 2.5 m below the existing grade or at about 243.5 m, the founding grades for the abutments can be expected at about Elevation 242 m or higher, and the elevation for the underside of the central pier foundation between 242.0 and 241.5 m.

Boreholes 26, 26A 27, 28, 29, 30 and 31 drilled at the abutment and central pier locations show that the structure can be supported on normal spread footing foundations placed on undisturbed, competent till, as given in Table 5.2.1.1

Table 5.2.1.1

Borehole Location	Existing Ground Surface Elevation (m)	Recommended Footing Base (bottom) Depth Below Existing Ground Surface (m)	Recommended Footing Base (bottom) Elevation (m)	Factored Bearing Resistance at U.L.S.* (kPa)	Bearing Resistance at S.L.S. (kPa)	Subgrade Material
BH30	245.7	1.2 – 1.7 1.8 – 3.7	244.5-244.0 243.9 – 242.0**	800 900	400 450	Sandy silt till Sandy silt till
BH31 (East Abutment)	245.7	1.1 – 1.7	244.6 – 244.0 243.9 – 242.0**	800 900	400 450	Sandy silt till Sandy silt till
BH28	245.7	3.2 – 4.2	242.5 – 241.5	900	450	Sandy silt till
BH29 (Central Pier)	245.9	3.4 – 4.4	242.5 – 241.5	900	450	Sandy silt till
BH26	245.9	1.9 – 3.9	244.0 – 242.0	800	400	Sandy silt till
BH27 (West Abutment)	246.0	1.5 – 2.7 2.8 – 3.8	244.5 – 243.3 243.2 – 242.2***	800 550	400 300	Sandy silt till Sandy silt till

*Incorporating a resistance factor of 0.5 as per Ontario Highway Bridge Design Code (OHBDC), 3rd Edition.
**The till becomes less competent below elevation 239.0 m to about elevation 237.5 m and therefore the recommended resistance values may have to be reduced for footings placed below elevation 242.0 m.
***Due to the presence of a weaker zone below Elevation 240.5 m, extending the footing below this elevation is not recommended

From the above table, it can be seen that the soil resistance values at the west abutment locations are lower. This is because an approximately 0.5 m thick silty clay zone was contacted in Borehole 27 at Elevation 240.5 m, in which an N-value of 16 blows/0.3 m was recorded. A similar relatively weak zone (N=19 blows/0.3 m) was also contacted at about the same elevation in Borehole 32 which was drilled about 20 m west, for the west approach. Visual examination of the soil samples and recorded N-values in several of the other boreholes also indicate the presence of a somewhat weaker zone at or about 1 to 2 m below this elevation. Due to this and to avoid extending the footings below the water table, it is recommended that the footing elevation be kept as high as possible.

The serviceability condition is based on the premise that the maximum total and differential settlements will not exceed 25 mm and 20 mm, respectively. This can be achieved provided that the founding subgrade is undisturbed during the construction.

Under inclined loading conditions the Bearing Resistance at U.L.S. should be reduced in accordance with Clause 6-8.4.2 of O.H.B.D.C.

The unfactored horizontal resistance against sliding between concrete and approved till surface can be calculated using a friction angle of 29 degrees.

For frost protection, the footings should have a permanent earth cover of at least 1.2 m, or equivalent artificial insulation.

The abutments can also be founded on engineered fill consisting of Granular A type material compacted in thin layers to at least 100% of the material's Standard Proctor Maximum Dry Density. Prior to the placement of the engineered fill, the upper variable, weak and otherwise unsuitable zones of the existing subgrade should be stripped to the surface of the competent stratum. The Granular A pad supporting the spread footing foundations should be at least 1.5 m thick. The suggested highest subgrade elevations at the borehole locations are given in Table 5.2.1.2.

Table 5.2.1.2

General Area	Borehole No.	Existing Ground Elevation (m)	Recommended Stripping Depth (m)	Recommended Stripping Elevation (m)	Soil Type
East Abutment	BH30	245.7	0.9	244.8	Sandy silt till
	BH31	245.7	0.9	244.8	Sandy silt till
West Abutment	BH26	245.9	1.1	244.8	Sandy silt till
	BH27	246.0	1.1	244.9	Sandy silt till

The construction of the Granular A pad and of the earth fill should meet the minimum requirements as per Ontario Ministry of Transportation, as shown on Figure No. D1 in Appendix B. The Granular A pad supporting the spread footing foundations should be at least 1.5 m thick.

For footings satisfying these requirements a factored vertical bearing resistance at U.L.S. equal to 900 kPa and a bearing resistance at S.L.S. of 350 kPa can be utilized. This should however be reviewed when the embankment and foundation details are being finalized.

The unfactored horizontal resistance against sliding between concrete and properly compacted Granular A fill can be calculated using an angle of friction of 35 degrees.

For frost protection, the footings should have a permanent earth cover of at least 1.2 m.

5.2.2 PILE FOUNDATIONS

If an 'integral abutment" type bridge is to be constructed, the abutments may need to be supported on driven steel H-piles. In this instance, steel H-piles with a heavy section such as HP310 x 110 with reinforced tips would be more suitable in order to achieve penetration into the very dense till and also because the presence of cobbles and boulders can always be expected in the glacial till deposits. Oversize materials (e.g. greater than 75 mm nominal diameter) should not be used in the embankment fills through which piles would be driven.

The following table summarizes the recommended axial pile resistance and estimated approximate pile tip elevations.

Table 5.2.2.1

Support Location	Reference Borehole	Estimated Pile Tip Elevation (m)	Recommended Factored Axial Resistance at U.L.S. (kN)	Recommended Axial Resistance at S.L.S. (kN)
East Abutment	30	234.0	1650	1150
	31	234.0	1650	1150
Central Pier	28	236.0	1200	800
	29	233.0	1200	800
West Abutment	27	233.0	1200	800
	26	235.0	1200	800

The minimum pile length beneath the pile cap and/or the highway grade should be 5.0 m. The minimum permissible pile length should also be discussed with the structural engineer.

Higher resistances would be available at greater depths, especially at the location of the central pier and the west abutment. But in view of the variable N-values recorded in Boreholes 26, 26A, 27 and 29, it is our opinion, based on previous experience with similar conditions, that the use of relatively shorter piles will be more economical despite the lower resistance values that were recommended. It is also believed that this approach will reduce potential problems during the driving of the piles, due to differential pile lengths, delays, etc.

The piles should be driven using a suitably heavy hammer capable of delivering a rated energy of at least 55 kilojoules/blow, but not more than 65 kilojoules/blow. The driving of the piles in the field should be controlled by a recognized pile driving formula, such as the Hiley Formula. The estimated ultimate resistance of the piles by the Hiley Formula can be calculated by dividing the recommended axial resistance at U.L.S. by a resistance factor of 0.5 as per current MTO practice. With this criterion, the estimated ultimate axial resistance as per Hiley Formula is approximately 3300 kN for the east abutment (i.e. 1650 ÷0.5 =3300) and 2400 kN for the central pier and the west abutment locations. For this project, however, because of the short nature of the piles, we recommend to use a resistance factor of 0.4 which would yield ultimate axial resistance values of 4125 kN for the east abutment and 3000 kN for the central pier and the west abutment.

In accordance with the above criterion, we recommend that the piles be driven to about 1 m above the quoted design elevation and driving should then be monitored and controlled by employing the Hiley Dynamic Pile Driving Formula in accordance with MTO Standards SS 103-10 or SS 103-11. If the driven pile encounters refusal above the recommended elevations, the engineer should be immediately notified.

During the driving process piles which have already been driven should be monitored to determine if they are heaving due to the effects of driving adjacent piles. If this phenomenon occurs, the affected piles should be re-driven.

10%

At least 15% of the piles (but not less than three piles) driven at strategic locations at each support element should be re-tapped not less than 24 hours after the driving of the pile, as per OPSS-903, to check that relaxation has not occurred. If it has, then all the piles should be re-tapped. It is possible that the piles may drive several metres below the estimated tip elevations. This aspect should be taken into consideration when ordering piles.

If difficulties are encountered, due to the very dense nature of the ground, to penetrate the pile to the required tip elevation then pre-augering may be necessary to an elevation sufficiently above the anticipated pile tip elevation (e.g. about 1.5 m above the pile tip elevation.) As mentioned before, due to the anticipated hard driving conditions and the presence of cobbles and boulders, the piles should be equipped with reinforced tips as per MTO Standards (OPSD 3301.00).

For frost protection, all pile caps should have a permanent earth cover of at least 1.2 m.

In cohesionless soils the coefficient of horizontal subgrade reaction can 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 Table 5.2.2.2.

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

Where the soil is primarily cohesive, the undrained shear strength of the soil is given.

Table 5.2.2.2

Area Reference Borehole No.	Applicable Elevation (m)	Soil Type	Bulk Unit Weight (kN/m3)	Angle of Internal Friction (ϕ) Degrees	Recommended n_h Value (MN/m^3)	Recommended Undrained Shear Strength (kPa)
East Abutment BH30 & 31	245.2-244.7	Clayey silt	19.0			80
	244.7-241.0	Sandy silt till	22.0	35	18.0	
	241.0-237.5	Sandy silt till	22.0	35	11.0	
	237.5-231.9	Sandy silt/silty sand till	22.5	36	12.0	
Central Pier BH28	245.4-245.0	Clayey silt	18.0			40
	245.0-243.6	Clayey silt	21.0			120
	243.6-243.0	Sandy silt till	21.5	34	18.0	
BH29	245.5-244.8	Clayey silt	18.5			50
	244.8-243.0	Clayey silt till	21.5			180
BH28&29	243.0-241.0	Sandy silt till	22.0	34	18.0	
	241.0-237-6	Sandy silt till	22.0	35	11.0	
	237.6-233.3	Sandy silt/silty sand till	22.5	36	12.0	
West Abutment BH26	245.6-244.5	Sandy silt till	20.0	31	4.0	
BH26	244.5-240.6	Sandy silt till	22.0	35	18.0	
BH26	240.6-230.0	Silty sand till	22.5	36	12.0	
BH27	245.6-245.0	Silty clay	18.5			50
BH27	245.0-244.0	Sandy silt till	21.0	32	5.0	
BH27	244.0-240.5	Sandy silt till	22.0	34	18.0	
BH27	240.5-240.0	Silty clay	20.5			130
BH27	240.0-236.0	Silty sand till	21.8	33	11.0	
BH27	236.0-223.0	Silty sand till	22.2	35	11.5	

The recommended horizontal resistances for the HP310x110 steel H-piles are as follows:

Factored Horizontal Resistance at U.L.S. = 130 kN
Horizontal Resistance at S.L.S. = 60 kN

At the central pier locations, and also if integral abutments are not constructed at the abutment locations, the lateral resistances of the piles can be supplemented, if desired, by horizontal components of battered piles.

In order to minimize the effects of any downdrag, and to minimize future settlements of the embankment fill, we recommend that the approach embankments be placed prior to driving the piles.

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 corrugated steel pipes (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 m 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 cement bentonite or uniform coarse sand. An NSSP should be included in the contract documents specifying the gradation of the sand as follows:

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

5.3 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/m^3
Coefficient of Lateral Earth Pressures:
$K_a = 0.27$

$K_o = 0.43$

Compacted Granular 'B'
Unit Weight = 21 kN/m^3
Coefficient of Lateral Earth Pressures:
$K_a = 0.31$
$K_o = 0.47$

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, 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 high appearance.

5.4 APPROACH EMBANKMENTS/CUTS

Based on the borehole results, no foundation failures are anticipated for the proposed 6 m high embankments, provided that all organic soils, weak or otherwise unsuitable materials are removed as per MTO Standards before placing the fill.

Assuming properly compacted, acceptable inorganic earth fill material, 2 horizontal in 1 vertical side slopes can be used. Proper erosion control measures should be implemented both during the construction and permanently. This can be achieved by immediate seeding or sodding (OPSS 572).

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 as depicted by the sketch presented in Appendix E. The average thickness of the unsuitable soils to be stripped can be assumed to be about 0.4 m. After stripping, the exposed subgrade should be inspected, approved and properly compacted from the surface, using a suitably heavy compactor, under the supervision of a geotechnical engineer who is familiar with the findings of this report and appointed by the Contract Administrator.

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 12 mm and should be substantially completed during the construction and within three weeks of placing the embankment fill to its full height. Such settlements are considered acceptable and will not necessitate preloading or surcharging.

Groundwater level was recorded at about 5 m below existing grade and, therefore, we do not anticipate problems due to groundwater seepage during stripping of the subgrade and backfilling for the construction of the embankments.

The materials used for the construction of the embankment fills should consist of approved, acceptable earth fill (e.g. Select Subgrade Materials – OPSS 1010). As mentioned before, oversize materials should not be used in embankment fills through which piles would be driven. The 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 subbase) should be increased to 98%. The selection, placement and compaction of the fill should be carried out under the supervision of a geotechnical engineer who is familiar with the findings of this report and appointed by the Contract Administrator. The settlement of the embankment fills prepared as described above should not exceed 35 mm. The time rate of settlement will depend on the material used for construction and 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. These quoted settlements would be in addition to the foundation settlements quoted earlier in this section. In view of the fact that the anticipated total settlements should not exceed about 45 mm, surcharging is not considered to be necessary.

5.5 CONSTRUCTION COMMENTS

Water level measurements made in the piezometers installed indicate that the groundwater table at the time of the investigation was at about Elevation 241.0 m or about 5.0 m below the existing grade. Therefore, for spread footing excavations extending to 3.5 to 4.0 m below the existing grade (or to about Elevation 242.0 m), no major problems due to groundwater seepage are anticipated. Any surface water seepage can be handled by gravity drainage or by pumping.

If the water table at the time of the construction rises to excavation depth or if a perched water table is encountered, it should be possible to dewater the site by means of gravity drainage and pumping from filtered sumps. Depending on the conditions, a perimeter drain may also be necessary. It should, however, be pointed out that such a system can only be expected to lower the water table by about 0.5 to 0.8 m and if water table is considerably higher or deeper excavations are necessary, the use of a more elaborate system, such as vacuum well points, may be required. It should also be pointed out that the silty deposits encountered at the site are susceptible to disturbance, especially in the presence of water. If foundations are placed on improperly dewatered, disturbed founding soils, excessive settlements could occur after the application of structural loads.

Allowance should be made to place a 150 mm thick lean concrete mudmat in all footing excavations to minimize disturbance. Following the construction of the footings, backfill should be placed to a sufficient height above the footing (i.e. at least 1.2 m) to prevent disturbance and frost penetration.

Up to 2.5 m deep cuts will be required at and in the vicinity of the bridge location. Borehole results indicate that, in general, underlying about 1 m of clayey silt to silty clay soils, the cuts can be expected to be formed through compact to dense sandy silt tills. The groundwater table, which was encountered at a depth of 5 m or at about Elevation 241 m at the time of our investigation, is not expected to rise to the proposed highway elevation (i.e. 243.5 ± m). It is, therefore, anticipated that the cuts will be formed above the permanent groundwater table. These conditions indicate that 2H:1V side slopes should be stable both during the construction and permanently.

All cut slope faces should be inspected during the construction for surficial instabilities. Where necessary, remedial measures, such as gravel sheeting, may be required.

Vegetation should be established on all slope faces to protect against surficial erosion as per OPSS 572.

5.6 FROST PROTECTION

Design frost penetration for the general area is 1.2 m. Therefore, a permanent soil cover of 1.2 m or its thermal equivalent is required for frost protection of foundations.

6. CLOSURE

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

The Limitations of this Report, as quoted in Appendix F, are an integral part of this report.

Shaheen & Peaker Limited

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
PROVINCE OF ONTARIO

Zuhtu Ozden, P.Eng.

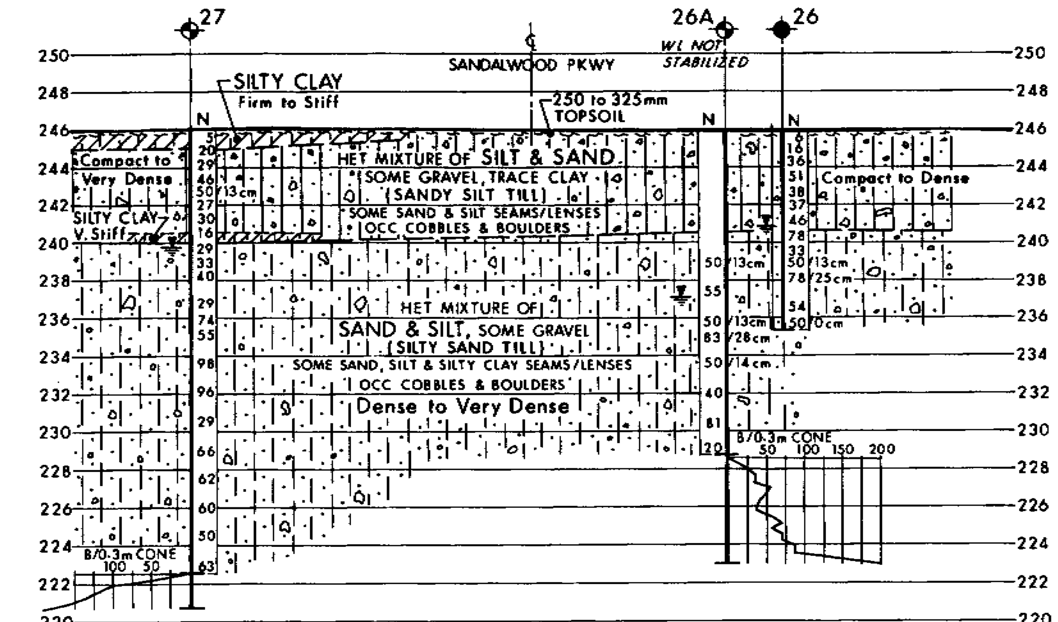
Ivan Lieszkowszky, P.Eng.

REGISTERED PROFESSIONAL ENGINEER
CIVIL
I. P. LIESZKOWSZKY
PROVINCE OF ONTARIO

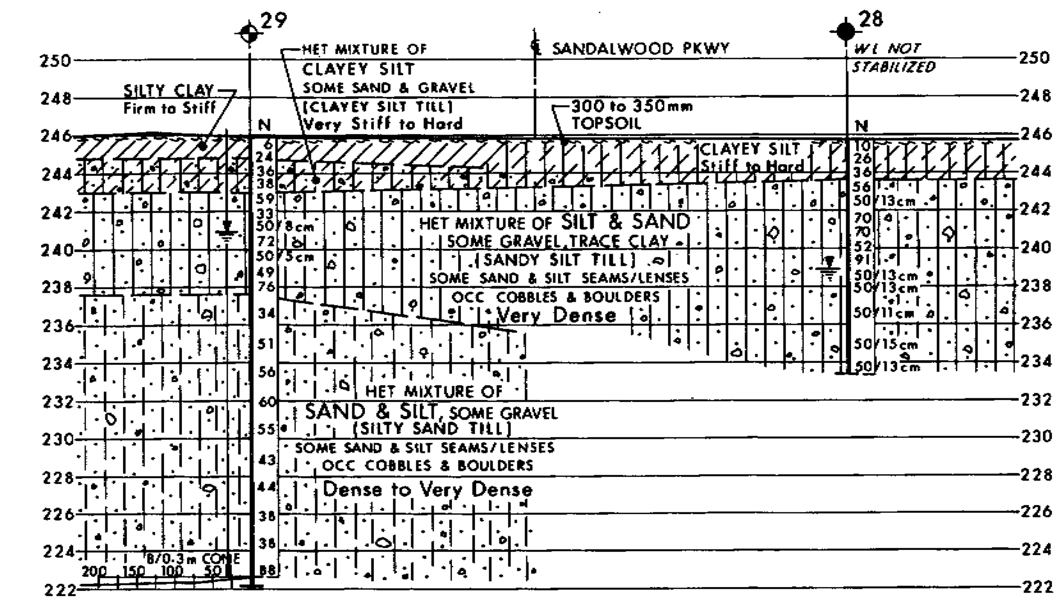
trzip#3

APPENDICES

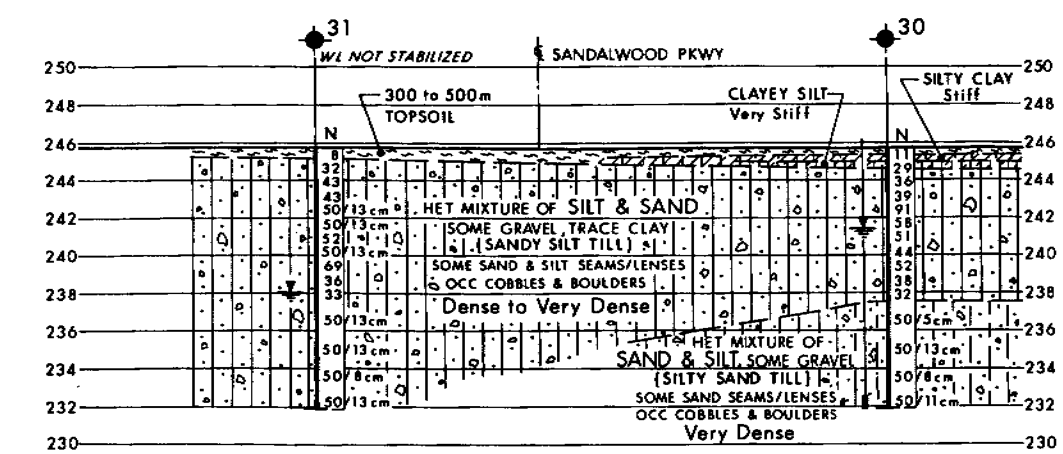
MINISTRY OF TRANSPORTATION, ONTARIO PH-D-207 88 10



SECTION A-A



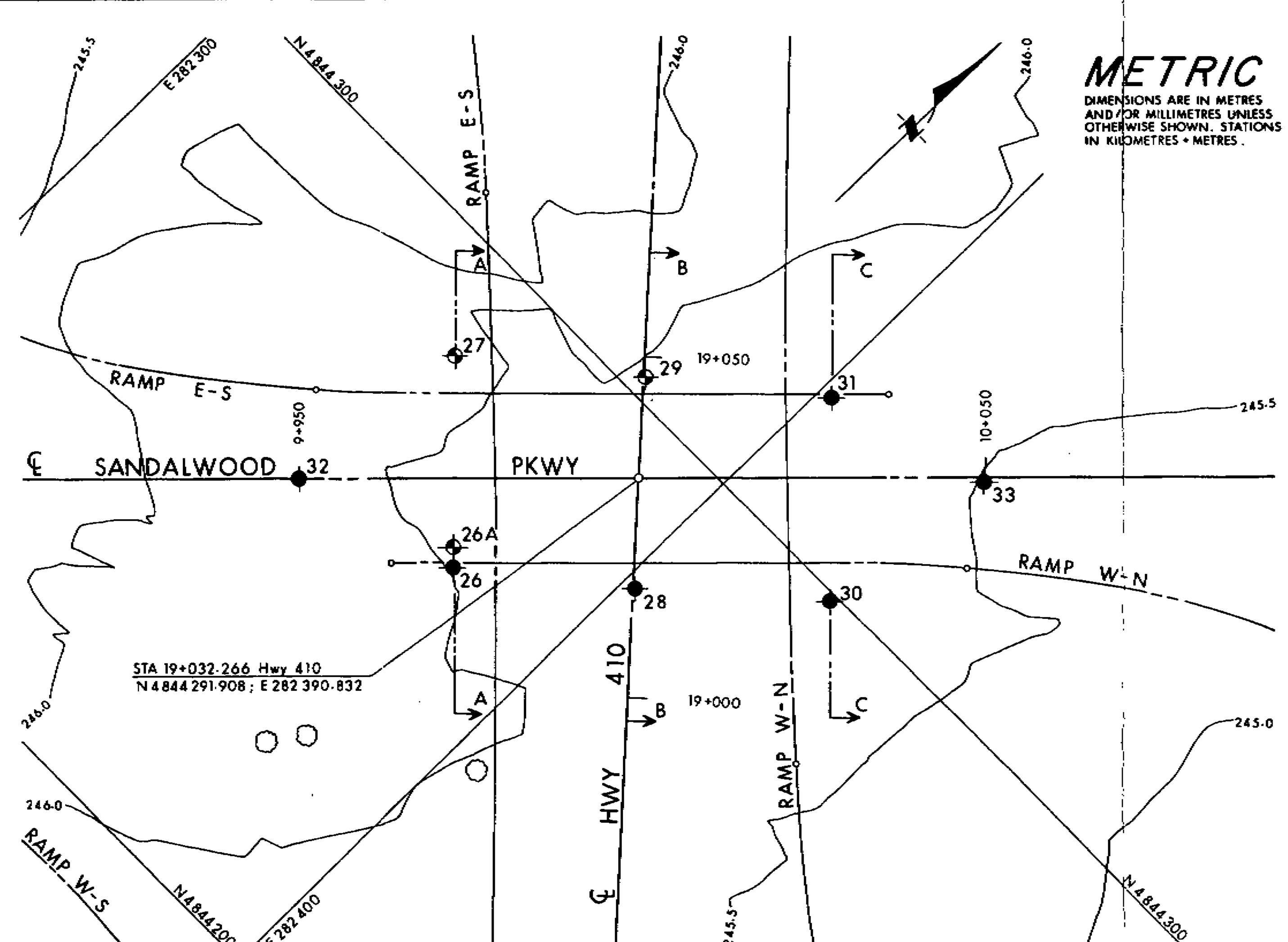
SECTION B-B



SECTION C-C

SCALE FOR SECTIONS

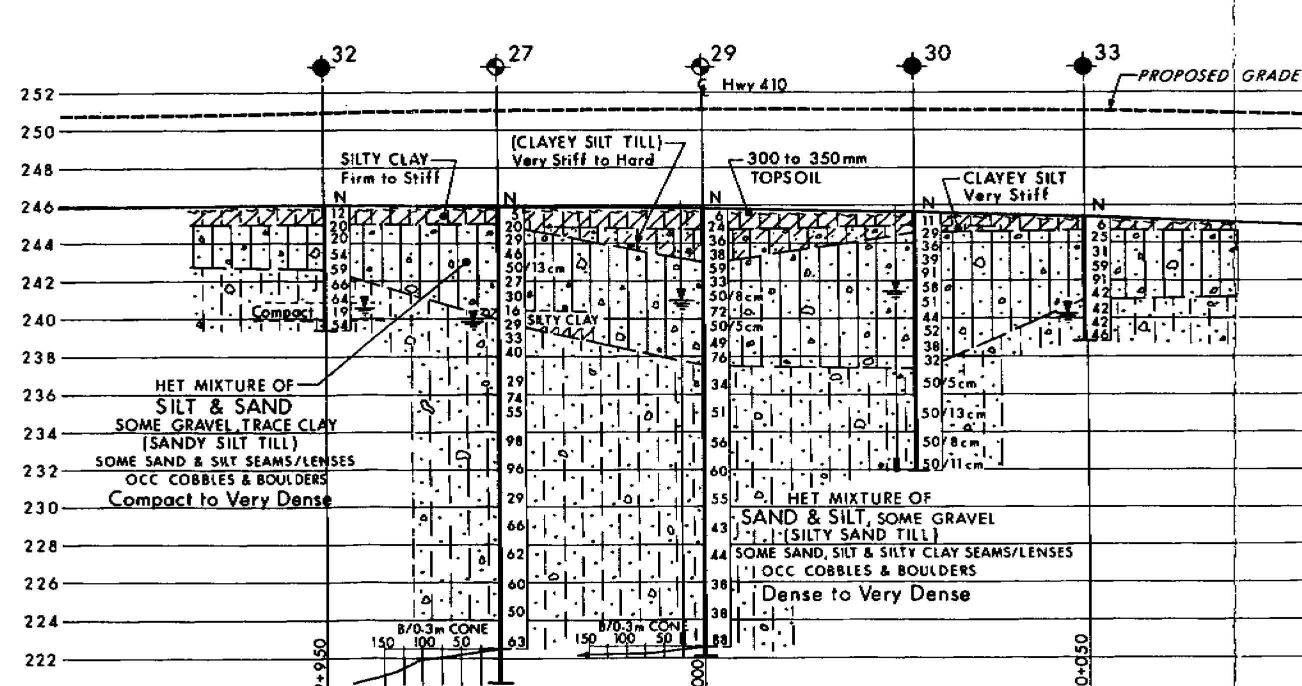
4m 2 0 4 8m



PLAN

SCALE

10m 5 0 10 20m



℄ PROFILE SANDALWOOD PKWY

SCALES

10m 5 0 10 20m HOR

4m 2 0 4 8m VERT

METRIC

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

CONT No

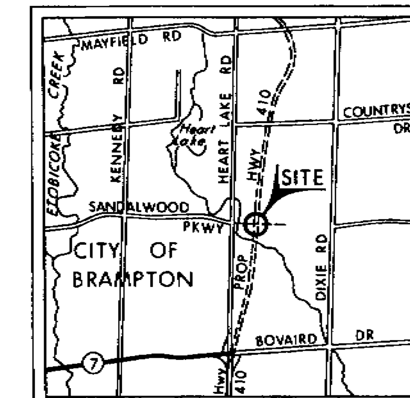
WP No 130-99-00(A)

SANDALWOOD PKWY

SHEET

BORE HOLE LOCATIONS & SOIL STRATA

Shaheen & Peaker Limited



KEY PLAN

SCALE

1km 0.5 0 1 2km

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 Oct. 1999

WL in Piezometer

Piezometer

No	ELEVATION	CO-ORDINATES NORTH	EAST
26	245.9	4 844 263	282 381
26A	245.9	4 844 265	282 379
27	246.0	4 844 285	282 359
28	245.7	4 844 280	282 402
29	245.9	4 844 303	282 381
30	245.7	4 844 299	282 423
31	245.7	4 844 320	282 402
32	246.0	4 844 256	282 356
33	245.4	4 844 327	282 426

=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

Geocres No 30M12-240

HWY No 410			DIST CENTRAL
SUBM'D Z O	CHECKED Z O	DATE Jan 3, 2000	SITE
DRAWN J P	CHECKED S B	APPROVED	DWG 1309900A-A

L. P. LIESZKOWSZKY

Z. S. OZDEN

APPENDIX A

Borehole Log Sheets

RECORD OF BOREHOLE No 26

1 OF 1 **METRIC**

W.P. 130-99-00 LOCATION 4844263N;282381E ORIGINATED BY M.T

DIST Central HWY 410 BOREHOLE TYPE Solid and Hollow Stem Augers COMPILED BY G.T

DATUM Geodetic DATE 18.10.99 19.10.99 CHECKED BY Z.O

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
245.9	Ground surface									
245.6 / 0.0	250 mm **TOPSOIL**	1	SS	6		20.0				
0.3	Heterogeneous mixture of Silt and Sand some gravel, trace clay **(SANDY SILT TILL)** some Clayey Silt Till zones, occasional cobbles and boulders, brown to 2.7 m, grey below, dense to 0.9 m, compact to 1.5 m, dense below, moist to 2.2 m, damp below	2	SS	16	245		8	44	48	0
		3	SS	36	244	21.0				
		4	SS	51	243		7	43	50	0
		5	SS	38		22.5	5	42	53	0
		6	SS	37	242		8	38	52	2
		7	SS	46	241	21.6	4	35	61	0
240.6 / 5.3	Heterogeneous mixture of Sand and silt with some gravel **(SILTY SAND TILL)** some Clayey Silt Till zones, some Sand and Silt seams/lenses, occasional gravelly sand seams, occasional cobbles and boulders, grey, dense to very dense, moist to wet	8	SS	78	240					
		9	SS	33		22.0				
		10	SS	50/13	239					
		11	SS	78/25	238	23.2	19	50	30	1
		12	SS	54	237					
235.3 / 10.6	End of borehole	13	SS	50/N.P	236					

REMARKS: Change to Hollow-Steam augering

End of borehole

Refusal to further augering at 10.6m on a boulder. Standpipe piezometer installed

Water level at 10.0 m upon completion and at 4.8 m on Oct 22/99

Water level in piezometer at 5.1 m on Oct 20 and 21/99

Water level at 5.1 m on Oct 27/99

Sample 13=no recovery

N.P. denotes no penetration

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; REMARKS & GRAIN SIZE DISTRIBUTION (%)

$+^3, \times^3$: Numbers refer to Sensitivity   (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 26A

METRIC

W.P. 130-99-00	LOCATION 4844265N;282379E		ORIGINATED BY M.T
DIST Central	HWY 410	BOREHOLE TYPE Solid and Hollow Stem Augers & D.C.P.T.	COMPILED BY G.T
DATUM Geodetic	DATE 25.10.99		CHECKED BY Z.O

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
245.9	Ground Surface								
0.0	not sampled see log of Borehole 26					245, 244, 243, 242, 241, 240			
239.2 / 6.7	Heterogeneous mixture of Sand and Silt some gravel (SILTY SAND TILL) with Sandy Silt Till zones, occasional Sand and Silt seams/lenses, occasional cobbles and boulders, grey, very dense to 13 m, dense below, wet	1	SS	50/13		239	o	22.2	
		2	SS	55	▼	238, 237			Change to Hollow-Steam augering
		3	SS	50/13		236	o		
		4	SS	83/28		235			15 44 38 3
		5	SS	50/14		234, 233	o	22.7	
		6	SS	40		232, 231			8 46 43 3

Continued Next Page

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE 20 40 60 80 100
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 ◆ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 26A

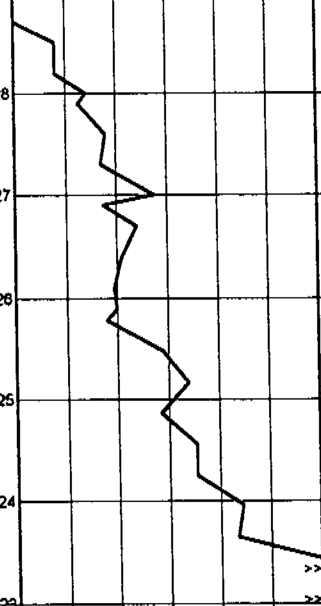
METRIC

W.P. 130-99-00 LOCATION 4844265N;282379E ORIGINATED BY M.T

DIST Central HWY 410 BOREHOLE TYPE Solid and Hollow Stem Augers & D.C.P.T. COMPILED BY G.T

DATUM Geodetic DATE 25.10.99 25.10.99 CHECKED BY Z.O

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Heterogeneous mixture of Sand and Silt with some gravel (SILTY SAND TILL) grey, very dense to compact, wet		7	SS	81				○	22.7	
							230				
228.7			8	SS	20		229				
17.2	End of borehole at 17.2 m Dynamic Cone Penetration Test performed from 17.2 to 22.9m						228				
							227				
							226				
							225				
							224				
223.0								>>			
22.9	End of Dynamic Cone Penetration Test Water level on completion at 8.8 m and hole caved at 9.1 m Water level not stabilized						223	>>			



$+^3, \times^3$: Numbers refer to Sensitivity   20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 27

METRIC

W.P. 130-99-00	LOCATION 4844285N;282359E		ORIGINATED BY M.T
DIST Central	HWY 410	BOREHOLE TYPE Solid Stem Augers & D.C.P.T.	COMPILED BY G.T
DATUM Geodetic		DATE 19.10.99 20.10.99	CHECKED BY Z.O

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
246.0	Ground surface
245.7 / 0.0	325 mm TOPSOIL
0.3	SILTY CLAY with Clayey Silt and Silt seams, brown, firm to stiff
245.0 / 1.0	Heterogeneous mixture of Silt and Sand some gravel, trace clay (SANDY SILT TILL) with Clayey Silt Till zones, some Sand and Silt seams, occasional cobbles and boulders, brown to 3.3 m, grey below, compact to very dense; wet sand and silt seams
240.5 / 5.5	SILTY CLAY with till seams, grey, very stiff
240.0 / 6.0	Heterogeneous mixture of Sand and Silt some gravel (SILTY SAND TILL) some Sandy Silt Till and Clayey Silt Till zones, some Sand seams, occasional Silt and Silty Clay seams/lenses, occasional cobbles and boulders, grey, dense to very dense, wet

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	5		
2	SS	20	20.3	
3	SS	29		
4	SS	46	21.6	
5	SS	50/13		
6	SS	27	22.8	5 39 44 12
7	SS	30		
8	SS	16	20.6	
9	SS	29		
10	SS	33		
11	SS	40		
12	SS	29		
13	SS	74		
14	SS	55		
15	SS	98		10 36 52 2
16	SS	96		Oct 19/Oct20

ELEVATION SCALE: 246, 245, 244, 243, 242, 241, 240, 239, 238, 237, 236, 235, 234, 233, 232, 231

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 27

METRIC

W.P. 130-99-00	LOCATION 4844285N;282359E		ORIGINATED BY M.T
DIST Central	HWY 410	BOREHOLE TYPE Solid and Hollow Stem Augers & D.C.P.T.	COMPILED BY G.T
DATUM Geodetic	DATE 19.10.99 20.10.99		CHECKED BY Z.O

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) | SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100) | PLASTIC LIMIT w_p, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L — WATER CONTENT (%) (20 40 60) | UNIT WEIGHT γ kN/m^3 | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS	GR	SA	SI	CL
	Heterogeneous mixture of Sand and Silt some gravel (SILTY SAND TILL) with Sandy Silt Till zones, occasional Sand seams, occasional cobbles and boulders, grey, dense to very dense, wet				231					
		17	SS	29		Change to Hollow-Steam augering				
					230					
		18	SS	66	229					
					228					
		19	SS	62						
					227					
		20	SS	60	226					
					225					
		21	SS	50						
					224					
		22	SS	63	223		9	42	45	4
222.5 23.5	End of borehole Dynamic Cone Penetration Test extended from 23.5 to 25.3 m				222					
220.7 25.3	End of Dynamic Cone Penetration Test Water level in open hole at 9.1 m and hole caved at 9.1 m on completion Water level in open hole at 6.2 m and hole caved at 6.2 m four hours after completion				221					

+ 3, × 3 : Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 28

METRIC

W.P. 130-99-00 LOCATION 4844280N;282402E ORIGINATED BY M.T

DIST Central HWY 410 BOREHOLE TYPE Solid Stem Augers COMPILED BY G.T

DATUM Geodetic DATE 18.10.99 CHECKED BY Z.O

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
245.7	Ground surface										
245.4 / 0.0	300 mm TOPSOIL	1	SS	10							
0.3	CLAYEY SILT with Clayey Silt Till, Silt and Silty clay seams, brown, stiff to hard, moist					245					
		2	SS	26							
		3	SS	36		244	23.6				
243.6 / 2.1	Heterogeneous mixture of Silt and Sand with some gravel (SANDY SILT TILL) some Clayey Silt Till zones to 6.0 m, frequent Sandy Silt Till zones and some Silt and Sand seams/lenses below, occasional cobbles and boulders, brown to 3.0 m, grey below, very dense, damp to 5.0 m, moist to wet below	4	SS	56				9	42	48	1
						243					
		5	SS	50/13			21.6	6	44	50	0
						242					
		6	SS	70				2	40	58	0
		7	SS	70		241	21.5				
		8	SS	52		240					
		9	SS	91			22.6				
					▼	239					
		10	SS	50/13							
		11	SS	50/13		238	22.5	1	42	56	1
						237					
		12	SS	50/11							
						236					
		13	SS	50/15		235	22.4	13	42	43	2
						234					
233.3		14	SS	50/13							
12.4	End of borehole Hole caved in and water level at 7.0 m upon completion Water level at 6.8 m and hole caved at 7.0 m two hours after completion Water level not stabilized Sand and clay lenses at 5.5 m										

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%): 20 40 60

REMARKS & GRAIN SIZE DISTRIBUTION (%)

$+^3, \times^3$: Numbers refer to Sensitivity; 20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 29

METRIC

W.P. 130-99-00	LOCATION 4844303N;282381E		ORIGINATED BY M.T
DIST Central	HWY 410	BOREHOLE TYPE Solid and Hollow Stem Augers & D.C.P.T.	COMPILED BY G.T
DATUM Geodetic		DATE 20.10.99 21.10.99	CHECKED BY Z.O

SOIL PROFILE | SAMPLES (NUMBER, TYPE, "N" VALUES) | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 | SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE 20 40 60 80 100 | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV DEPTH	DESCRIPTION
245.9	Ground surface
245.5 / 0.0	350 mm TOPSOIL
0.4	SILTY CLAY brown, firm to stiff
244.8 / 1.1	Heterogeneous mixture of Clayey Silt with some sand and gravel (CLAYEY SILT TILL) with Sandy Silt Till zones, brown, very stiff to 1.6 m, hard below
243.0 / 2.9	Heterogeneous mixture of Silt and Sand with some gravel, trace clay (SANDY SILT TILL) occasional cobbles and boulders, grey, dense to very dense, damp with occasional wet zones below 7.0 m
237.6 / 8.3	Heterogeneous mixture of Sand and Silt with some gravel (SILTY SAND TILL) some Sand and Silt seams/lenses, occasional cobbles and boulders, grey, dense to very dense, wet

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT kN/m³	REMARKS
1	SS	6	18.9	
2	SS	24		
3	SS	36	20.7	
4	SS	38		
5	SS	59	22.3	
6	SS	33		
7	SS	50/8	22.4	
8	SS	72		
~~9~~	~~SS~~	~~50/5~~		
10	SS	49		
11	SS	76	22.5	
12	SS	34		
13	SS	51		Change to Hollow Stem
14	SS	56		
15	SS	60	23.1	

Elevation scale: 245, 244, 243, 242, 241, 240, 239, 238, 237, 236, 235, 234, 233, 232, 231

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

$15 \overset{20}{\underset{10}{\oplus}} 5$ (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 29

METRIC

W.P. 130-99-00
LOCATION 4844303N;282381E
ORIGINATED BY M.T

DIST Central HWY 410
BOREHOLE TYPE Solid and Hollow Stem Augers & D.C.P.T.
COMPILED BY G.T

DATUM Geodetic
DATE 20.10.99 21.10.99
CHECKED BY Z.O

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Heterogeneous mixture of Sand and Silt with some gravel (SILTY SAND TILL) some sand and silt seams/lenses, grey, dense to very dense, wet	16	SS	55				
					230			
		17	SS	43	229	o	23.0	
					228			
		18	SS	44	227			
		19	SS	38	226	o		
					225			
		20	SS	38	224			
		21	SS	88	223	o	22.1	
222.6 23.3	End of borehole							
222.1 23.8	Dynamic Cone Penetration Test extended from 23.3 to 23.8 m				222			
	End of Dynamic Cone Penetration Test							
	Standpipe piezometer installed. Water level in piezometer at 4.8 m on Oct 22 and Oct 25 and at 5.0 m on Oct 27/99							
	wet sand layer at 18.3 m							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE: 20 40 60 80 100
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

+³, ×³: Numbers refer to Sensitivity; (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 30

METRIC

W.P. 130-99-00 LOCATION 4844299N;282423E ORIGINATED BY M.T

DIST Central HWY 410 BOREHOLE TYPE Solid Stem Augers COMPILED BY G.T

DATUM Geodetic DATE 14.10.99 CHECKED BY Z.O.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
245.7	Ground surface									
245.4 / 0.0	300 mm TOPSOIL	1	SS	11		19.5				
0.3	SILTY CLAY organic stained, brown, stiff				245					
245.0 / 0.7	CLAYEY SILT with Silt seams, brown, very stiff	2	SS	29						
244.6 / 1.1	Heterogeneous mixture of Silt and Sand with some gravel, trace clay (SANDY SILT TILL) some Clayey Silt Till zones, occasional cobbles and boulders, brown to 3.5 m, grey below, dense to very dense, damp	3	SS	36	244		5	45	47	3
		4	SS	39	243	22.1	6	41	51	2
		5	SS	91	242		10	41	47	2
		6	SS	58		22.5				
		7	SS	51	241					
		8	SS	44	240		9	44	45	2
	wet sand layer at 6.0 m	9	SS	52	239					
		10	SS	38		22.2				
	clayey	11	SS	32	238		10	40	47	3
237.5 / 8.2	wet sand layer at 8.6 m				237					
		~~12~~	~~SS~~	~~50/5~~	236					
	Heterogeneous mixture of Sand and Silt with some gravel (SILTY SAND TILL) with Sandy Silt Till and some sand seams/lenses, occasional cobbles and boulders, grey, very dense, wet	~~13~~	~~SS~~	~~50/13~~	235		5	49	45	1
		~~14~~	~~SS~~	~~50/8~~	234 / 233		20	42	37	1
231.9		~~15~~	~~SS~~	~~50/11~~	232	22.8				
13.8	End of borehole Hole caved-in and water level at 8.9 m on upon completion Standpipe piezometer installed.									

Continued Next Page

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L — WATER CONTENT (%): 20 40 60

REMARKS & GRAIN SIZE DISTRIBUTION (%)

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 30

METRIC

W.P. 130-99-00 LOCATION 4844299N;282423E ORIGINATED BY M.T

DIST Central HWY 410 BOREHOLE TYPE Solid Stem Augers COMPILED BY G.T

DATUM Geodetic DATE 14.10.99 CHECKED BY Z.O

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Water level in piezometer at 4.6 m on Oct 20/99 and 4.3 m on Oct 25 and 27/99										

$+^3, \times^3$: Numbers refer to Sensitivity

15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 31

METRIC

W.P. 130-99-00 LOCATION 4844320N;282402E ORIGINATED BY M.T
DIST Central HWY 410 BOREHOLE TYPE Solid Stem Augers COMPILED BY G.T
DATUM Geodetic DATE 21.10.99 CHECKED BY Z.O

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
245.7	Ground Surface						
0.0 / 245.2	TOPSOIL	1	SS	8			
0.5	clayey Heterogeneous mixture of Silt and Sand some gravel (SANDY SILT TILL) some Clayey Silt Till zones to 8.5 m, frequent Silty Sand Till zones below, some sand and silt seams/lenses, occasional cobbles and boulders, brown to 3.5 m, grey below, dense to very dense to 8.5 m, very dense below, damp to moist to 8.5 m, moist to wet below	2	SS	32		245	
		3	SS	43		244	
		4	SS	43		243	
		5	SS	50/13		242	
		6	SS	50/13			
		7	SS	52		241	
		8	SS	50/13		240	
		9	SS	69		239	
		10	SS	36			
		11	SS	33	▼	238	6 44 38 12
		12	SS	50/13		237 / 236	
		13	SS	50/13		235	7 35 56 2
		14	SS	50/8		234 / 233	
231.9 / 13.8	End of borehole Water level at 7.7 m and hole caved at 8.9 m on completion Water level not stabilized	15	SS	50/13		232	

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m³

+³, ×³ Numbers refer to Sensitivity 20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 32

METRIC

W.P. 130-99-00 | LOCATION 4844256N;282356E | ORIGINATED BY M.T
DIST Central HWY 410 | BOREHOLE TYPE Solid Stem Augers | COMPILED BY G.T
DATUM Geodetic | DATE 19.10.99 | CHECKED BY Z.O

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
246.0	Ground surface						
245.7 / 0.0	300 mm TOPSOIL	1	SS	12			
0.3	SILTY CLAY with Clayey Silt and Silt seams, brown, stiff						
245.0 / 1.0	Heterogeneous mixture of Silt and Sand some gravel, trace clay (SANDY SILT TILL) with Clayey Silt Till zones, some Sand and Silt seams/lenses, brown, compact to 2.6 m, very dense below, damp to moist	2	SS	20		245	
		3	SS	20		244	
		4	SS	54		243	
242.6 / 3.4	Heterogeneous mixture of Sand and Silt some gravel (SILTY SAND TILL) frequent Sand, Silt, and Clayey Silt lenses, occasionl cobbles and boulders, grey, compact to very dense, wet	5	SS	59			7 42 39 12
		6	SS	66		242	
		7	SS	64		241	
		8	SS	19	▼	240	
239.4 / 6.6	End of borehole	9	SS	54			

Water level at 5.5 m four hours after completion
0.3 m thick wet silty sand layer at 1.4 m
Silty clay seam at 5.5 m
Wet sand seam at 6.0 m

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m³

+ ³, × ³: Numbers refer to Sensitivity
20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 33

METRIC

W.P. 130-99-00 | LOCATION 4844327N;282426E | ORIGINATED BY M.T

DIST Central HWY 410 | BOREHOLE TYPE Solid Stem Augers | COMPILED BY G.T

DATUM Geodetic | DATE 14.10.99 | CHECKED BY Z.O

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
245.4	Ground surface											
245.1 / 0.0	300 mm TOPSOIL	1	SS	6		245		20.1				
244.7 / 0.3	SILTY CLAY trace gravel and organics, brown, firm											
0.7	Heterogeneous mixture of Silt and Sand some gravel, trace clay (SANDY SILT TILL) some Clayey Silt Till zones, brown to 3.5 m, grey below, compact to very dense, damp	2	SS	25		244						
		3	SS	31					2	46	50	2
		4	SS	59		243		22.1				
		5	SS	91		242			4	44	51	1
241.0		6	SS	42		241						
4.4	Heterogeneous mixture of Sand and Silt some gravel (SILTY SAND TILL) occasional Sand and Silt seams/lenses, grey, dense	7	SS	42				22.5				
					▼	240						
		8	SS	42								
238.8		9	SS	46		239		22.2				
6.6	End of borehole Borehole dry on completion. Water level in open borehole at 5.2 m one hour after completion											

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

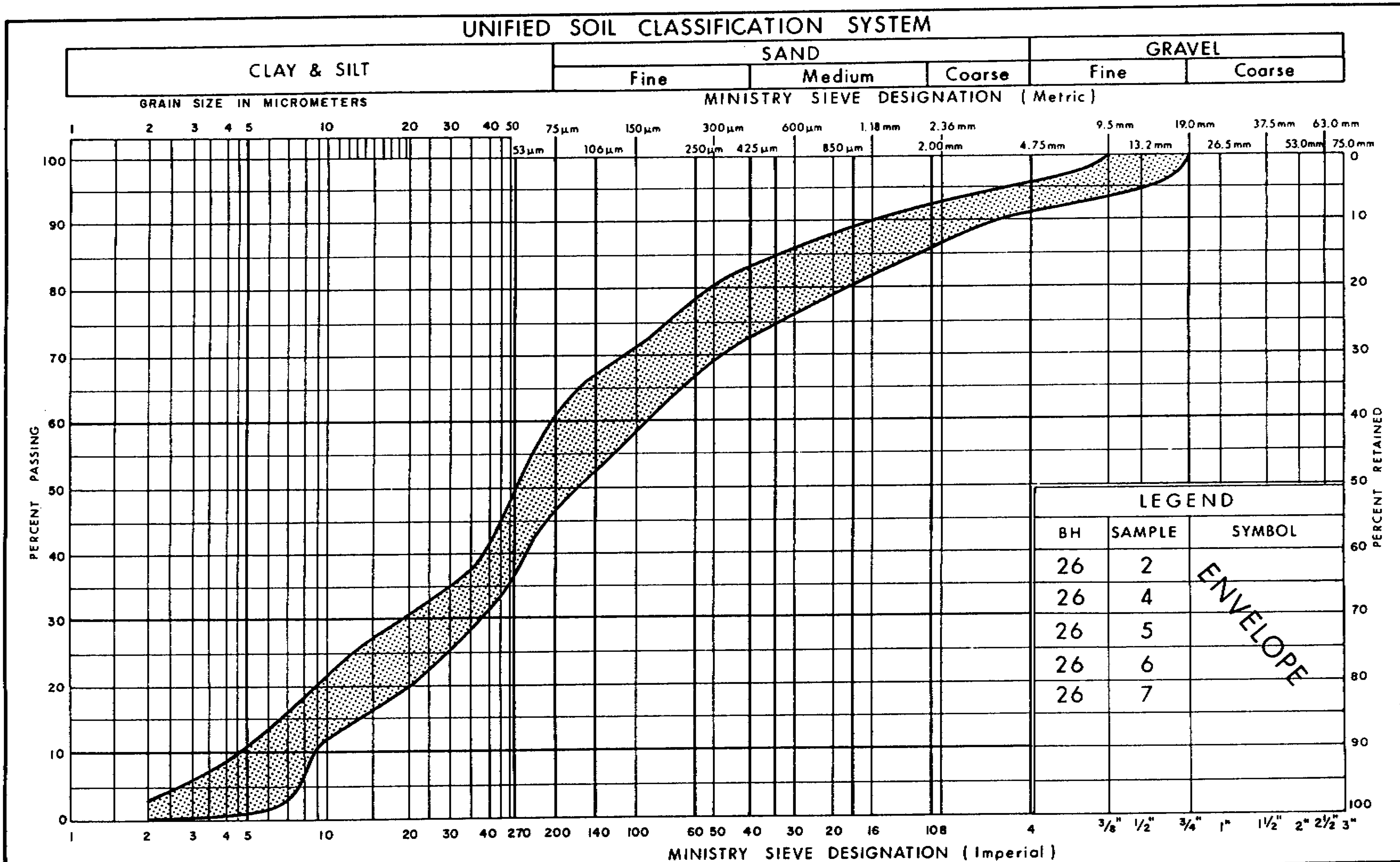
REMARKS & GRAIN SIZE DISTRIBUTION (%)

+³, ×³: Numbers refer to Sensitivity — 20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

APPENDIX B

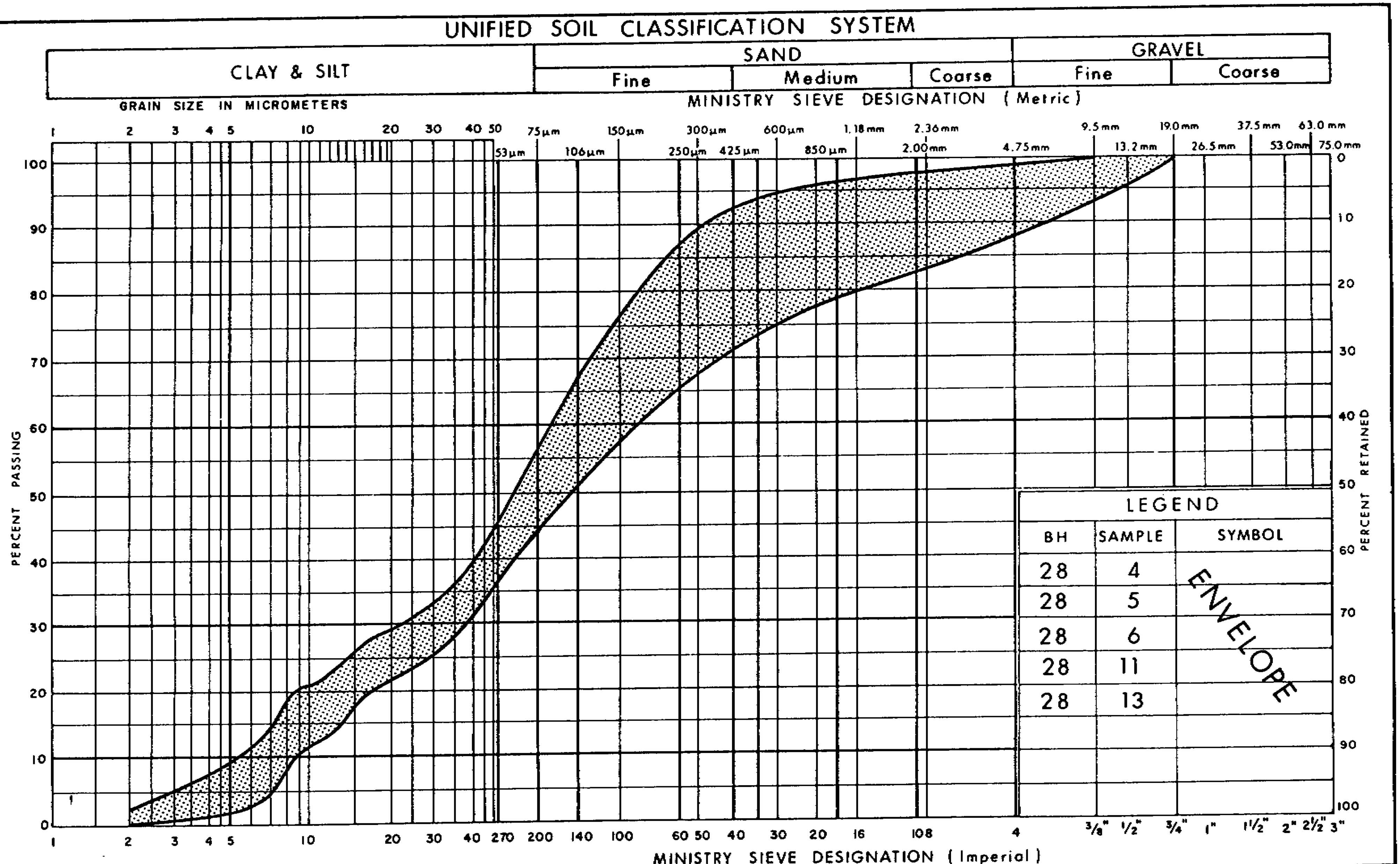
Laboratory Test Results

78 12 M



Ministry of Transportation Ontario	GRAIN SIZE DISTRIBUTION SANDY SILT TILL	FIG No B1 W P 130-99-00 (A) Sandalwood Pkwy

78 12 M



BH	SAMPLE	SYMBOL
28	4	ENVELOPE
28	5	
28	6	
28	11	
28	13	

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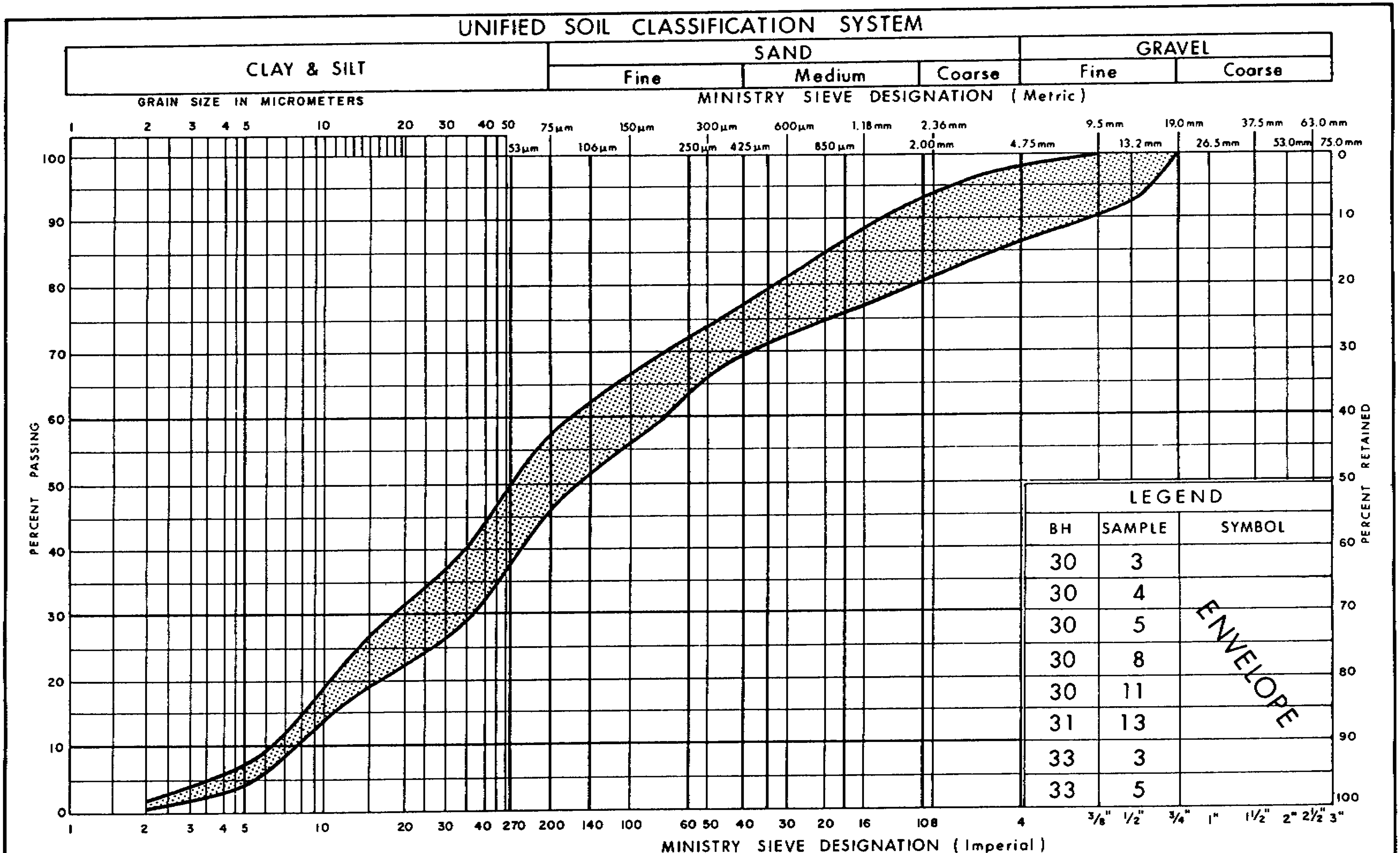
GRAIN SIZE DISTRIBUTION
SANDY SILT TILL

FIG No B2

W P 130-99-00(A)

Sandalwood Pkwy

78 12 M



LEGEND

BH	SAMPLE	SYMBOL
30	3	
30	4	
30	5	
30	8	
30	11	
31	13	
33	3	
33	5	

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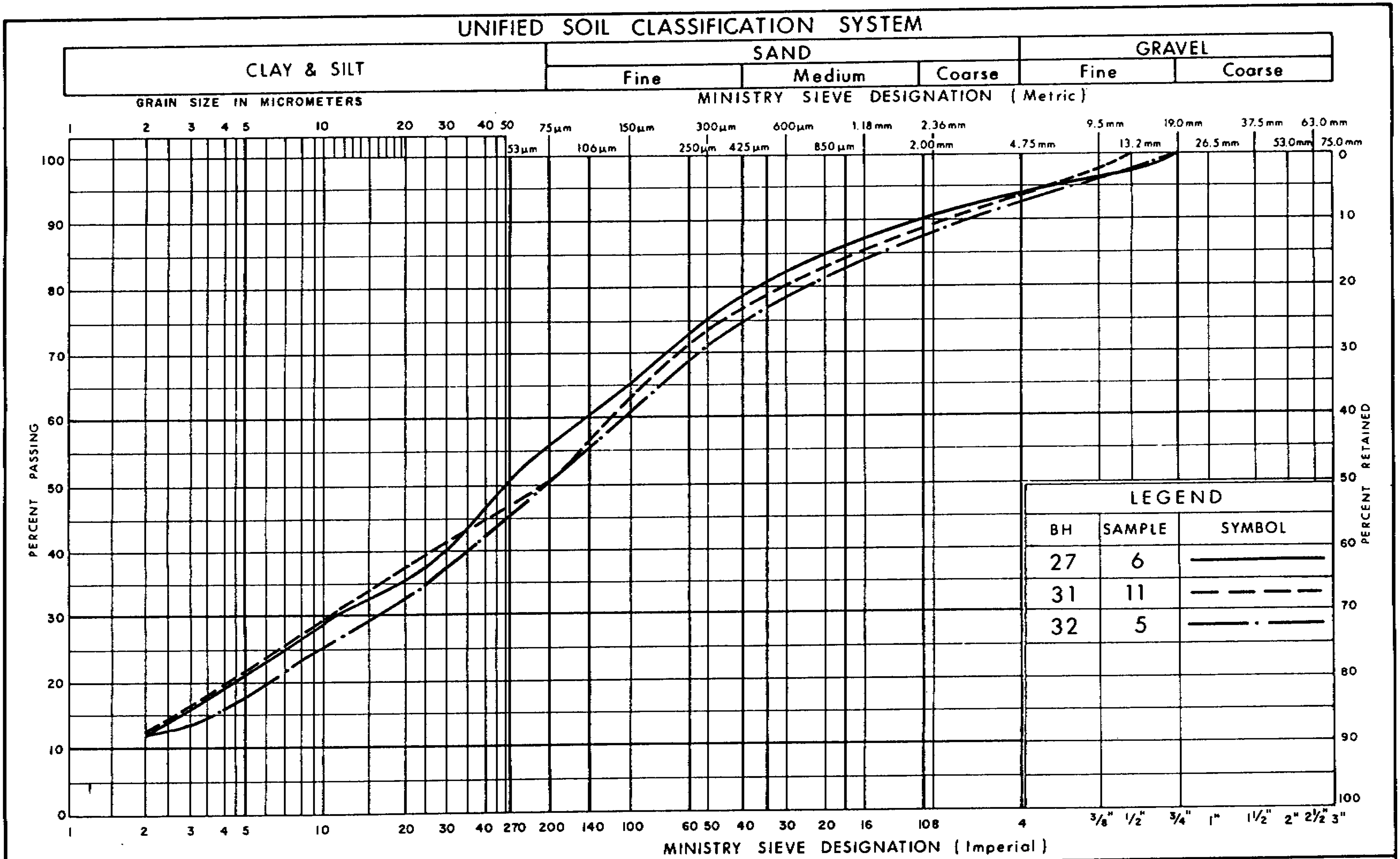
GRAIN SIZE DISTRIBUTION
SANDY SILT TILL

FIG No B3
W P 130-99-00(A)
Sandalwood Pkwy

78 12 M



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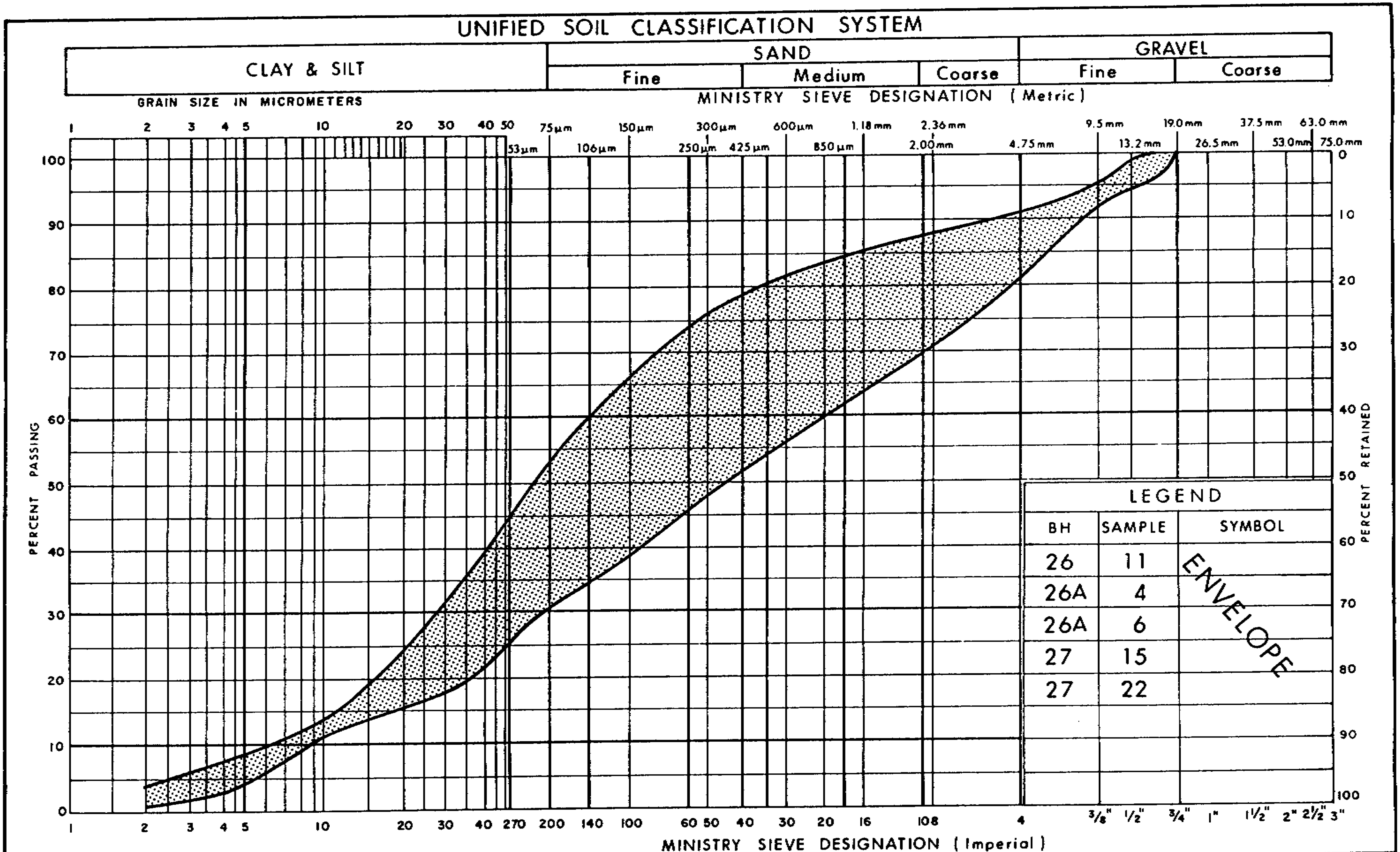
GRAIN SIZE DISTRIBUTION
CLAYEY SILT TILL ZONES IN SANDY SILT TILL

FIG No B4

W P 130-99-00 (A)

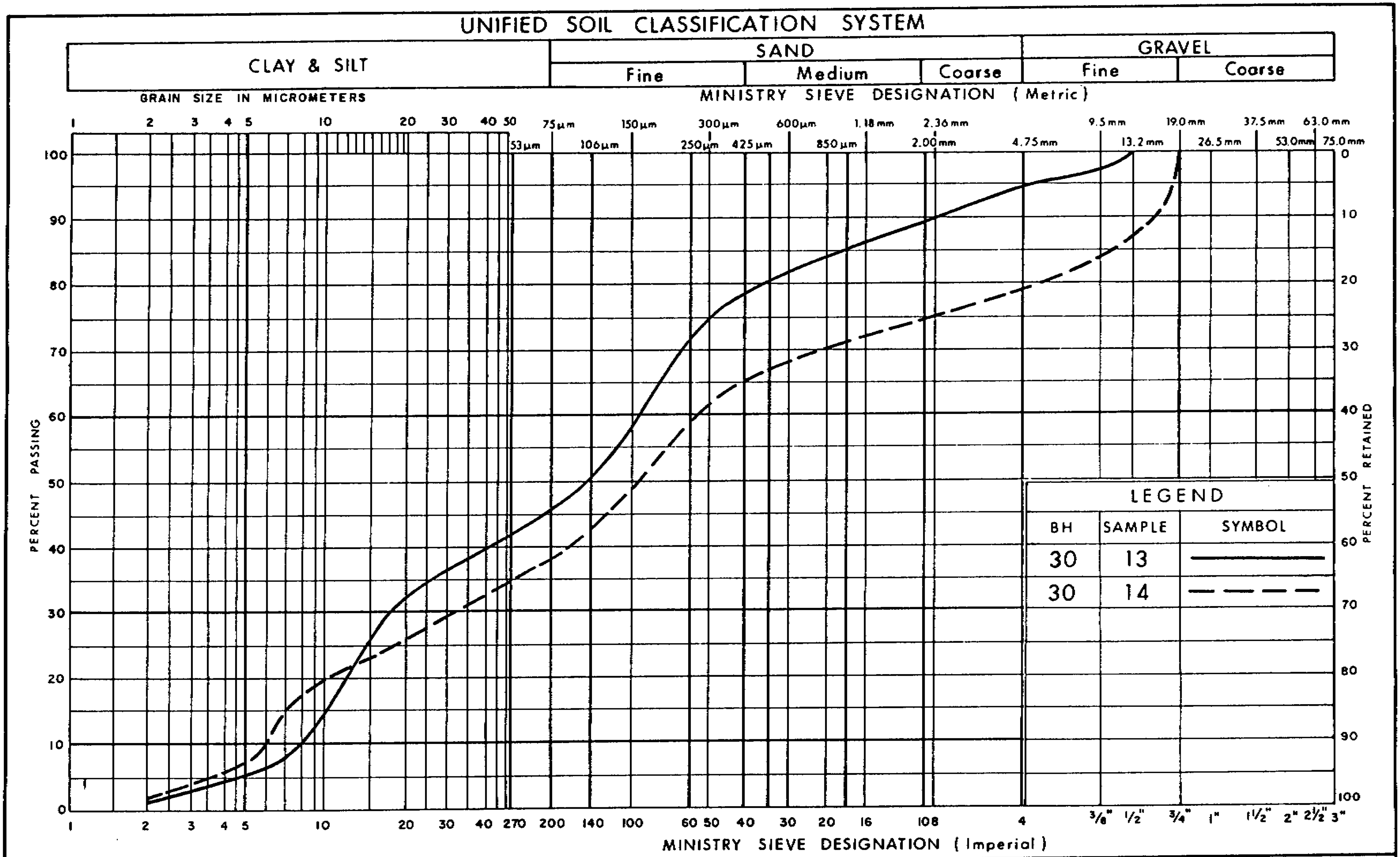
Sandalwood Pkwy

78 12 M



Ministry of Transportation Ontario	GRAIN SIZE DISTRIBUTION SILTY SAND TILL	FIG No B5 W P 130-99-00(A) Sandalwood Pkwy

78 12 M



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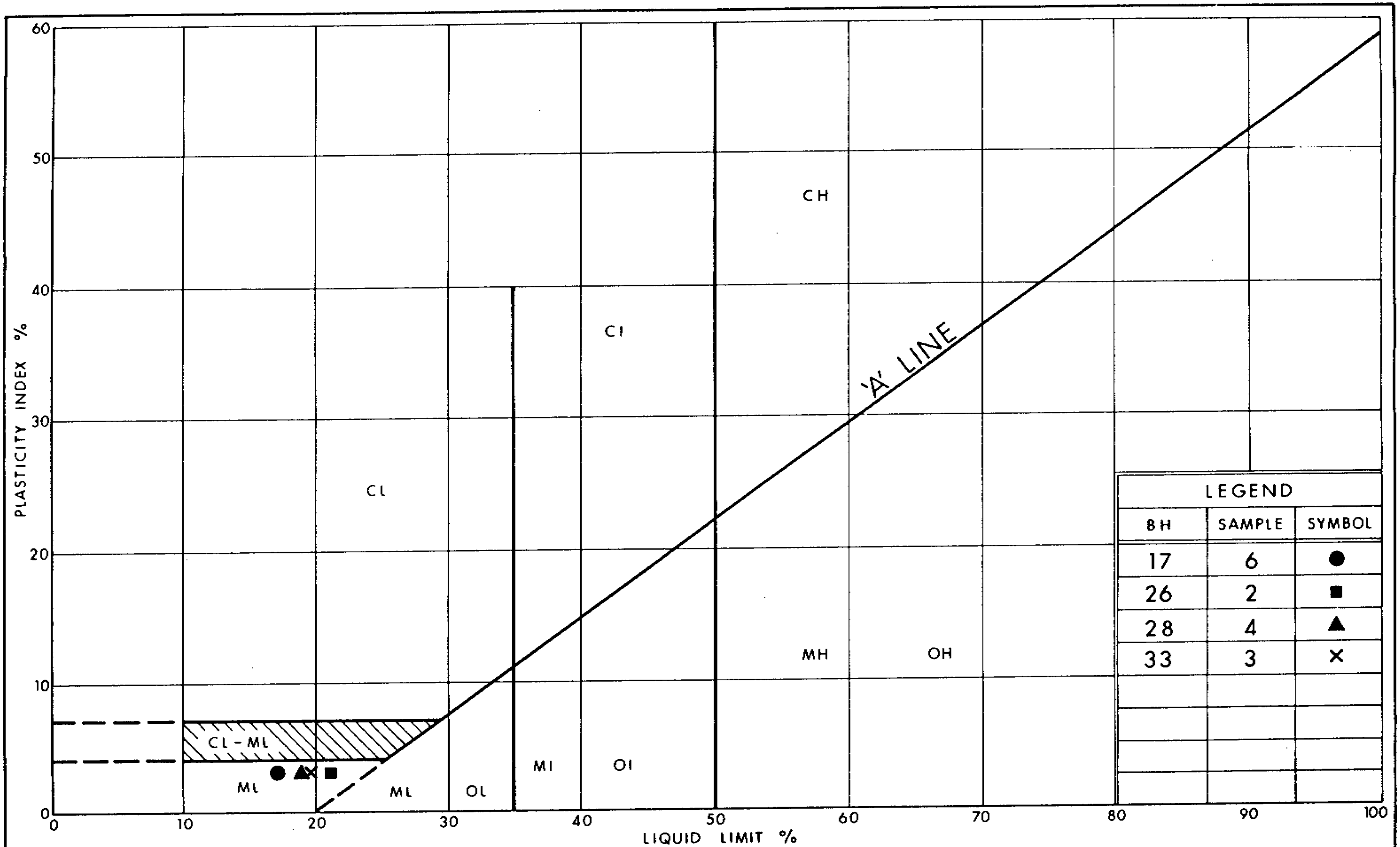
GRAIN SIZE DISTRIBUTION
SILTY SAND TILL

FIG No B6
W P 130-99-00(A)
Sandalwood Pkwy

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
17	6	●
26	2	■
28	4	▲
33	3	×

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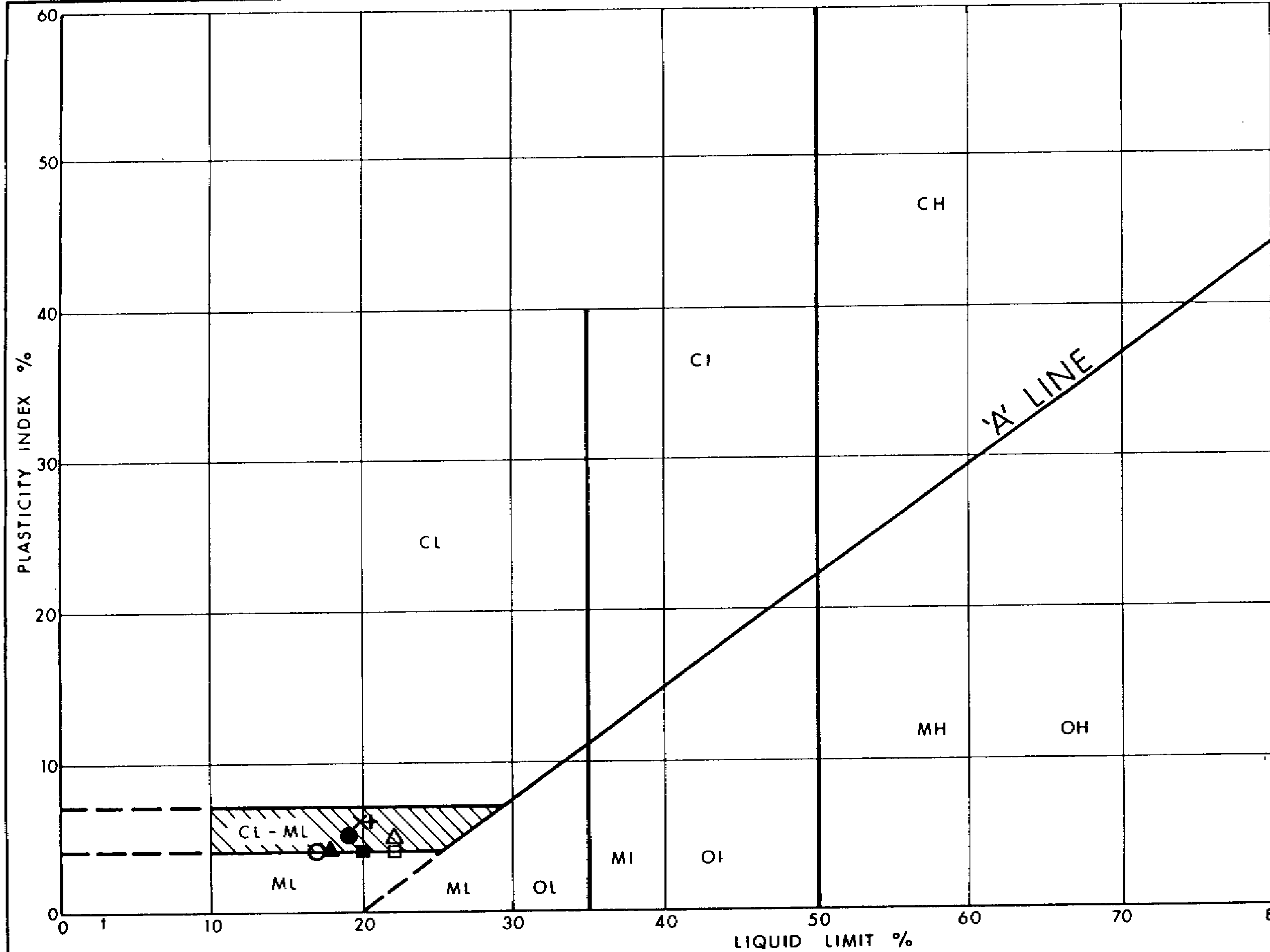
PLASTICITY CHART
SANDY SILT TILL

FIG No B7

W P 130-99-00(A)



LEGEND		
BH	SAMPLE	SYMBOL
9	5	○
9	8	●
10	3	□
12	2	■
13	2	△
21	5	▲
21	6	×
26	7	+

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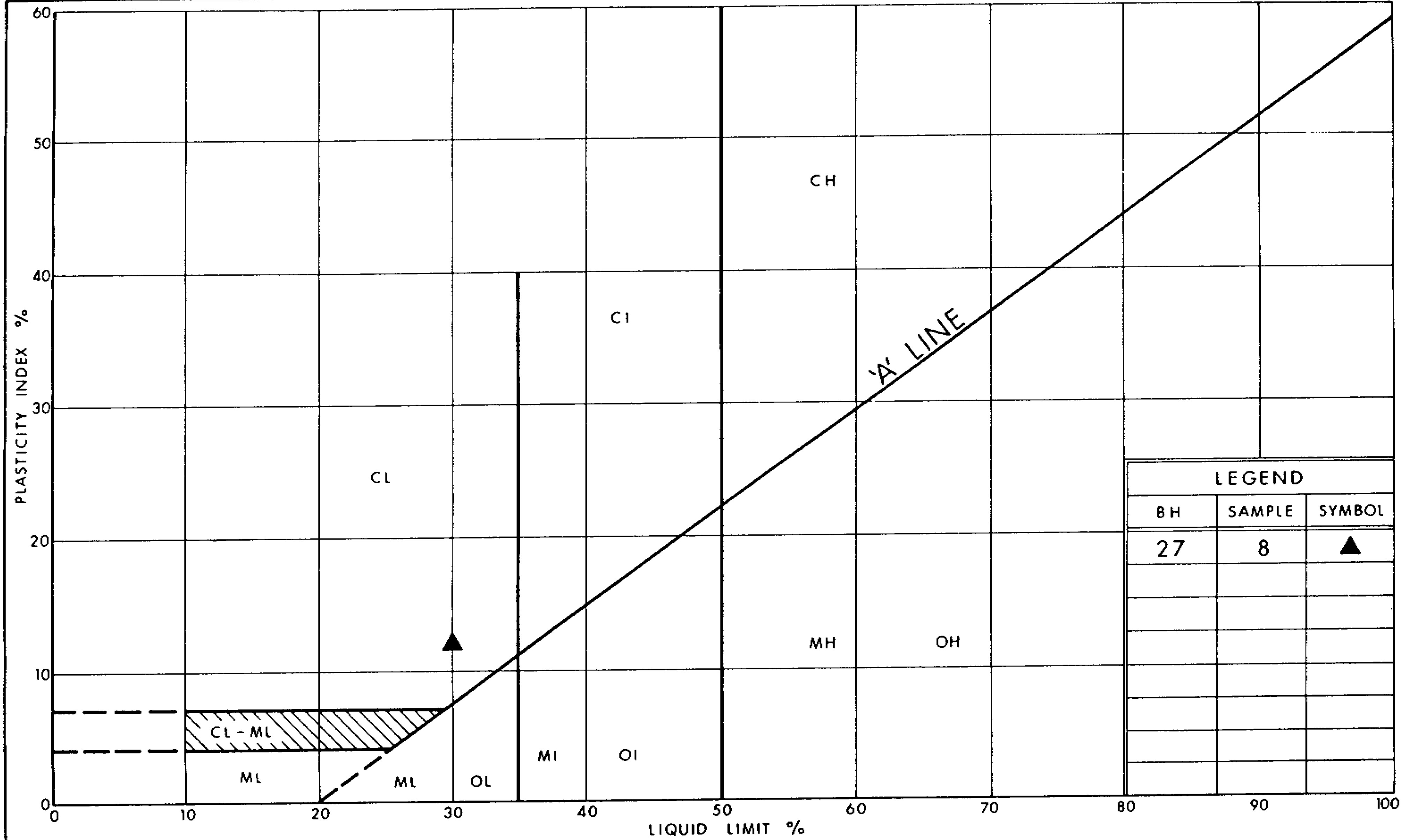
PLASTICITY CHART
CLAYEY SILT TILL

FIG No B8
W P 130-99-00(A)



LEGEND		
BH	SAMPLE	SYMBOL
27	8	▲

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PLASTICITY CHART
SILTY CLAY

FIG No B9

W P 130-99-00(A)

APPENDIX C

Explanation of Terms Used in Report

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 475 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 (R Q D), 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 = $\frac{c_u}{\tau_r}$

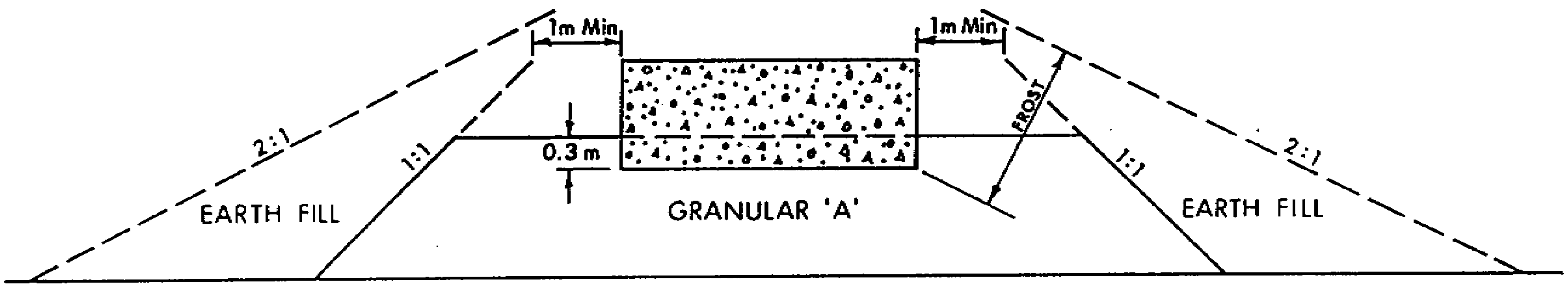
PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL

e	1, %	VOID RATIO
n	1, %	POROSITY
w	1, %	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX = $w_L - w_P$
I_L	1	LIQUIDITY INDEX = $\frac{w - w_P}{I_P}$
I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_P}$
e_{max}	1, %	VOID RATIO IN LOOSEST STATE

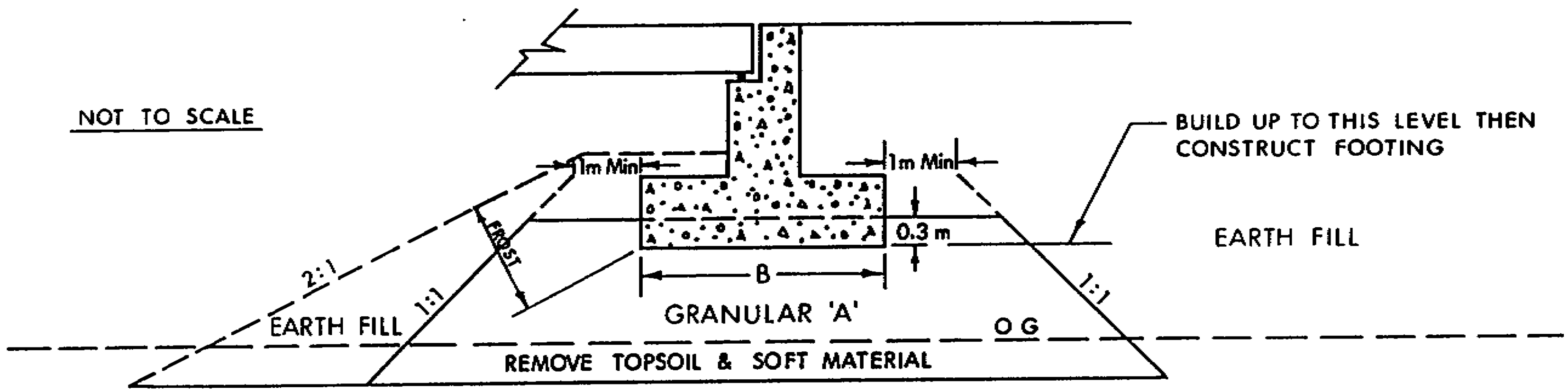
e_{min}	1, %	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_U	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

APPENDIX D
Abutment on Compacted Fill Showing Granular 'A' Core



X SECTION

NOT TO SCALE



LONGITUDINAL SECTION

NOTES:

1 - REMOVE TOPSOIL &/OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' & EARTH FILL.

2 - PLACE GRANULAR 'A' & EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M T O STANDARDS.

3 - CONSTRUCT CONCRETE FOOTING.

4 - PLACE REMAINDER OF GRANULAR 'A' & EARTH FILL AS REQUIRED.

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ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE

FIG No D-1

W P 130-99-00 (A)

APPENDIX E

Sketch Depicting Removal of Unsuitable Soils from Beneath Approach Fills

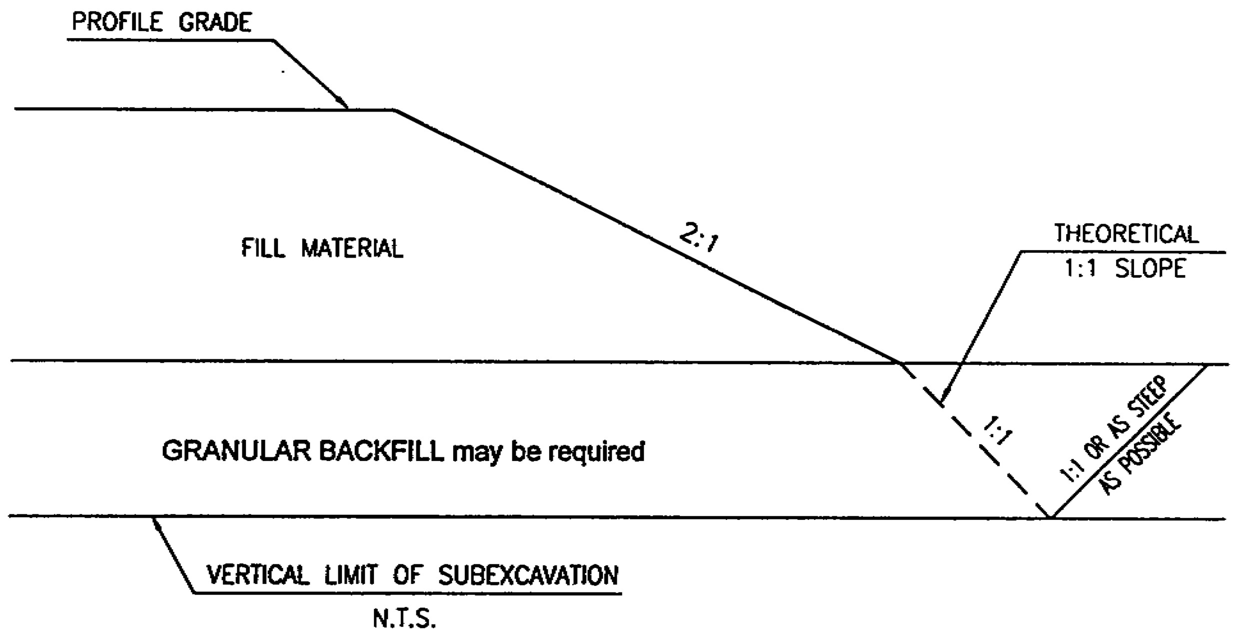

FIGURE NO. E-1
W.P. 130-99-00 (A)

REMOVAL OF UNSUITABLE SOILS
FROM BENEATH APROACH FILLS
N.T.S.

APPENDIX F

Limitations of Report

LIMITATIONS OF REPORT

The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. 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 benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Shaheen & Peaker Limited accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.