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DATE March 15, 1984

FOUNDATION INVESTIGATION
GO-ALRT CORBETT CREEK STRUCTURE
WHITBY, ONTARIO
PROJECT NO. EGG 000-24B-A (REV. 3 LINE)

Town of Whitby
Regional Municipality of Durham
District #6, Central Region

Ref. No. 83-12-8
March 1984

Prepared For:
GO-ALRT

Distribution

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1.0 INTRODUCTION

Dominion Soil Investigation Inc., Consulting Geotechnical Engineers, were retained by GO-ALRT to conduct a foundation investigation at the site of the proposed GO-ALRT line crossing of Corbett Creek, Project No. EGG 000-24B (Rev. 3 Line) in the Town of Whitby, Ontario. Authorization to carry out the work was received from Mr. M.S. Devata, Senior Foundation Engineer, Pavement and Foundation Design Section of the Ontario Ministry of Transportation and Communications.

The purpose of the investigation was to determine the subsoil conditions at the site and, based on the findings, to make recommendations pertaining to the design of the foundations of the proposed bridge structure.

The field work was carried out during the period of December 30, 1983 to January 28, 1984, and consisted of drilling twelve boreholes to depths ranging between 4.8 and 21.6 m and nine dynamic penetration cone tests. The location of the boreholes are shown on Drawing No. EGG 000-24B-A and the subsurface conditions encountered in the borings are presented on the Record of Borehole Sheets.

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2.0 THE PROJECT

The crossing is located west of Thickson Road and north of Highway 401 in an approximately 100 m wide flat valley. The valley has gently sloping sides and the water level in the Creek fluctuates depending on the season of the year. Near the centre, the ground elevation is 83^{\pm} m rising to 85 or 86^{\pm} m near the sides.

The initial design concept was to cross the valley on an 11 to 13 m high embankment. An earlier investigation carried out by our firm in September, 1983, showed this concept not to be feasible because of the presence of a deep, soft and compressible clay deposit. It was recommended that the GO-ALRT lines be carried over a bridge-type structure supported on end-bearing piles except at the abutment locations where normal spread footing foundations could be feasible. Subsequently, the alignment was shifted about 20 m to the south and the proposed embankment grade was lowered by about 6 m. One additional borehole drilled in November, 1983, along this new alignment and our report (Ref. No. 83-8-10A) concluded that this too was not feasible.

The most recent design concept consists of a 166 m long six-span bridge.

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3.0 SUMMARIZED SUBSOIL CONDITIONS

The site is located within the physiographic region known as the "Iroquois Plain". These lowlands bordering Lake Ontario were inundated in the Pleistocene times by Lake Iroquois and the subsoils generally consist of lacustrine deposits overlying glacial till.

In general the boreholes showed the presence of a deep weak clay deposit which thins out towards the sides of the valley. The clay is underlain by a silty sand till with some sand layers. Towards the east side of the valley, a deposit of silt was also encountered which is underlain by a silty clay till.

The groundwater was found to be close to the ground surface.

Details of the subsurface conditions encountered in the boreholes are given on the individual Record of Borehole Sheets and an inferred subsoil profile is presented on Drawing No. EGG 000-24B-A.

The relevant index and engineering properties of the principal strata are briefly discussed in the following paragraphs.

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3.1 Topsoil

The boreholes encountered a 0.2 to 0.6 m thick topsoil layer generally increasing in depth from the sides towards the middle of the valley.

3.2 Silty Clay

Silty clay was encountered in all the boreholes except for Borehole 1 located on the east side of the valley. The clay is deep (approximately 15 m) near the centre of the valley and thins out towards the sides.

Near its surface, and especially towards the sides of the valley, the clay is somewhat more competent and less compressible, probably due to desiccation. This upper somewhat more competent zone is less prevalent towards the middle of the valley where the clay is generally soft. At depth the clay attains a coarser texture with some sand and traces of gravel (the grain size distribution of a sample is shown in Figure 1). The sand and gravel content increases with depth and the clay changes into a sand till with a considerable silt and clay content which, in turn, becomes even coarser (i.e. silty sand) till.

The index properties of the clay were measured in the laboratory as follows:

.../...

	<u>Range</u>	<u>Average</u>
Liquid Limit	35 to 53 %	43%
Plastic Limit	15 to 19 %	17%
Plasticity Index	19 to 34 %	26%
Moisture Content	16 to 86 %	

These values indicate clay deposits of medium plasticity (i.e. CI).

The undrained shear strength of the clay was measured in-situ in boreholes drilled near the sides of the valley where the clay is relatively shallow and more competent. These field vane tests gave values ranging from 23 to more than 120 kPa. Two undrained (quick) triaxial compression tests were also performed in the laboratory which gave values of 30 and 43 kPa. From these results and N-values ranging between 1 and 14 blows/0.3 m, the consistency of the clay near the sides of the valley, is described as generally firm to stiff.

In the boreholes drilled near the floor of the valley, the N-values ranged from less than 1 to 4 blows/0.3 m in the upper zones with somewhat higher Standard Penetration values at depth due to the higher sand and gravel content. From these values and the in-situ vane test results from our previous investigation, the consistency of the clay towards the centre of the valley is described as generally very soft to firm.
.../...

Two consolidation tests were also performed in the laboratory on samples from Boreholes 14 and 16 located towards the east side of the valley and the test results are presented on Fig. No. 7. The test results indicate a relatively compressible structure and that it is somewhat overconsolidated, probably caused by desiccation.

Further discussions on the properties of the clay can be found in our previous report, Ref. No. 83-8-10, dated September, 1983.

3.3 Silty Sand Till

The topsoil layer in Borehole 1 on the west side of the valley and the clay in the rest of the boreholes, are underlain by a silty sand till deposit with frequent sand seams.

The grain size distribution of the till is presented in Figures 2 and 3 indicating 11 to 23% gravel, 44 to 55% sand, 23 to 37% silt and 1 to 5% clay size particles.

Occasionally the till is finer, especially near the interface of the till and the overlying clay. The grain size distribution of a sample from a relatively finer (sandy silt) till is shown in Figure 4. On the east side of the valley, the till at greater depths is considerably finer and attains .../...



a predominantly cohesive character. It is described, on the Record of Borehole Sheets, as 'silty clay' till and the grain size distribution of a sample from this material is presented in Figure 5.

The till is generally dense to very dense with occasional compact zones near the surface of the material. On the eastern half of the valley in Boreholes 9, 14, 16, 18 and 21, weak zones of the till (i.e. loose to very loose) were also recorded at the interface with the overlying clay.

3.4 Sand

In many of the boreholes the silty sand till is stratified with water bearing sand layers of variable thicknesses (e.g. Boreholes 1, 3, 5, 13, 16, 18 and 21). The sand layers are generally dense to very dense and, in some cases, the presence of a subartesian condition was inferred from the back-up of the soil in the augers while drilling.

3.5 Silt

In the eastern half of the valley, a deposit of silt was encountered between the upper silty sand till or sand and the lower finer and hard silty clay till. The thickness of the

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material was found to range from 1.1 m in Borehole 13 to 7.1 m at the location of Borehole 18 further to the east.

The silt is frequently stratified with sandy silt, silty fine sand and silty clay seams. The grain size distribution of the material is presented in Figure 6, indicating 18% fine sand, 77% silt and 5% clay size particles.

4.0 GROUNDWATER CONDITIONS

Groundwater conditions in the boreholes were observed during the drilling. After their completion, where feasible, the boreholes were left open and the water levels in the open boreholes were rechecked. In addition, in Boreholes 1, 3, 9, 16 and 18, piezometers were installed to enable us to monitor the groundwater over a prolonged period of time. The final recorded values are presented on the individual records of borehole sheets.

Based on these observations, the groundwater level at the time of the investigation was generally close to the existing ground surface.

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5.0 DISCUSSION

The proposed bridge will be a six equal span structure with abutments located near the sides of the valley. The existing ground surface elevation at the site ranges from 83⁺ m near the centre of the valley, to 84.3 m and 85.2 m at the locations of the proposed east and west abutments, respectively. The finished grade will range from 89.0 m in the east side of the valley to 89.5 m in the west and therefore the approach embankments will be 4 to 4.7 m high. They will be 11 to 14 m wide and 2 horizontal in 1 vertical side slopes are proposed.

The boreholes have shown that at the present west abutment location (B.H. 1) there is no clay and that the clay is only 2.5 m deep (B.H. 5) at the location of the first pier east of the west abutment (i.e. Pier #1). At the remaining four pier locations the depth of the weak clay ranges from 7.6 to 14.5 m (Boreholes 6, 9, 10 and 13). The clay also extends to the east abutment location where Borehole 16 encountered a firm to stiff material to a depth of 4.4 m.

5.1 West Abutment Foundations

The presently selected location for the west abutment is Station 26+120 in the west side of the valley and here the grade will be raised by about 4 m. The perched abutment .../...



could be supported either on spread footings or on piled foundations.

5.1.1 Spread Footing Foundations

Borehole 1 and Cone Test 2 were drilled at Station 26+120 and here the existing ground surface is at Elevation 85.2 m. As the final grade will be 89.5^{\pm} m, 4.3 m of fill will be placed to build the embankment.

Borehole 1, drilled at the location of the proposed abutment, encountered silty sand till with some sand layers. The till is compact to a depth of 2.3 m (to Elevation 82.9 m) and very dense below.

Based on the findings of Borehole 1, the use of normal spread footing foundations is feasible on the undisturbed very dense till at a depth of 2.3 m (Elevation 82.9 m) below the existing ground surface. The Factored Bearing Capacity at Ultimate Limit States (q_f) is 680 kPa. The recommended Bearing Capacity at Serviceability Limit States Type II is 400 kPa. With this value, provided that the subsoil is not unduly disturbed during the construction, the maximum total settlement should be within 25 mm. About the same magnitude of differential settlements between the abutment and the first pier could occur if the abutment is placed on spread .../...



footing foundations and the pier is founded on driven piles.

If the embankment fill is placed after stripping the topsoil and compacting the exposed subgrade from the surface, settlements of the order of 20 mm could be expected in the native subsoil due to the weight of the embankment fill. The settlement should take place within one month of the placement of the fill.

It is also feasible to establish the abutment further east to shorten the span of the bridge.

Borehole 3 drilled at Station 26+130 (i.e. 10 m east), encountered a firm to stiff silty clay deposit to a depth of 1.1 m below the ground surface, followed by dense to very dense silty sand till. For footings placed in the undisturbed dense to very dense till (at a depth of 1.2 m or more below the present grade), at or below Elevation 83.5 m, the Factored Bearing Capacity at Ultimate Limit States (q_f) 600 kPa. Bearing Capacity at Serviceability Limit States Type II is 350 kPa. Here too, provided that the subgrade is not unduly disturbed during the construction, settlements of the order of 25 mm can be expected and could translate into differential settlements, as discussed before.

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At Borehole 3, the existing grade is about 0.5 m lower than at Station 26+120 and therefore the height of the embankment would be slightly greater (i.e. 4.8 m). Settlements of the order of 25 mm could be expected in the underlying soil, due to the weight of the fill placed after stripping the topsoil and the surficially very weak zones of the clay and compacting the subgrade at the surface. About 80% of the settlements should be completed within one month.

Borehole 20, which was drilled 6 m further east at Station 26+136 m to investigate the possibility of locating the abutment at this station, showed the presence of 1.3 m of clay followed by compact silty sand till, changing to dense to very dense below 1.8 m. At this location too it is possible to place the footings on the dense till at 1.8 m below the existing grade provided that, by means of careful dewatering, the subsoil is not unduly disturbed during the construction. The estimated total settlement is 25 mm and, as discussed before, these settlements could also be equal to the differential settlements if different types of foundations are used, i.e. piles for the pier and spread footing foundations for the abutment.

At this location the existing grade elevation is 84.1 m and since the final embankment grade would be 89.5^{\pm} m, about .../...



5.4 m of fill will be required. The estimated induced settlement under weight of the fill is 35 mm and about 80% of this settlement should take place within one month.

The recommended bearing depths and allowable soil bearing values at each station are summarized in the following table:

Station & B.H. No.	Existing Ground Elevation (m)	Recommended Foundation Depth Below Existing Ground Surface & Elevation (m)	Recommended Soil Bearing Capacity	
			Ultimate Limit States (kPa)	Serviceability Limit States Type II (kPa)
Sta. 26+120 B.H. 1	85.2	2.3 m (82.9)	680 1000	400
Sta. 26+130 B.H. 3	84.7	1.2 m (83.5)	600 700	350
Sta. 26.136 B.H. 20	84.1	1.8 m (82.3)	680 700 700	400 350

Under inclined loading conditions, the bearing capacity at Ultimate Limit State should be reduced in accordance with Clause 6.7.3.3.5 of the Ontario Highway Bridge Design Code, 1979 (OHBDC). For the evaluation of the sliding resistance of the foundations, the ultimate angle of friction between the underside of the foundation and the dense to very dense till, can be taken as 23 degrees.

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The footings must have a permanent earth cover of at least 1.2 m for frost protection. The foundation excavations should be checked and approved by a geotechnical engineer to ensure that the footings rest on undisturbed subsoil capable of sustaining the design pressure.

Due to the high water table and the relatively coarse nature of the till, groundwater seepage into the foundation excavations can be anticipated. As discussed in Section 5.6, by means of careful dewatering techniques, the disturbance of the founding subgrade during the construction will therefore have to be prevented.

Because of the anticipated construction difficulties due to high groundwater level and since, as discussed later, driven pile foundations will be required for the piers and the east abutment (there will be a piling contractor at the site), it is our opinion that driven piles is a better foundation alternative especially from the point of reducing the magnitude of differential settlements.

5.1.2 Piled Foundations

End-bearing steel H-piles could be used to support the abutment perched atop the approach embankment. The pile capacities and other details are discussed in the next .../...



section of the report. From the point of reducing differential settlements, it is our opinion that the use of driven piles is a more suitable foundation alternative than normal spread footing foundations. Due to the fact that the dense to very dense till is located close to the ground surface, however, it may not be possible to drive the piles to a sufficient depth below the pile caps. If the piles cannot obtain a minimum length of approximately 4 m below the pile caps, then other measures such as preboring will be necessary to facilitate further driving or the pile capacities may have to be adjusted. The space between the pile and the sides of the augered hole will have to be filled with low slump fine concrete to provide lateral stability.

5.2 Pier Foundations

The piers could be supported on end-bearing steel H-piles driven into the very dense or hard deposits underlying the weak clay or the weak zones of the till.

The estimated pile capacities for some common sizes of steel H-piles driven to a final set of about 1 blow for 1 mm penetration with a pile driving hammer capable of delivering an energy of 40,000 to 70,000 Joules/blow, are tabulated below. In our opinion, the settlement of the pile head will be small.

.../...

ESTIMATED PILE CAPACITY (kN)

<u>Pile Type</u>	<u>Size</u>	<u>Factored Capacity at Ultimate Limit States (Q_u)</u>	<u>Capacity at Serviceability Limit States Type II (Q_s)</u>
Steel H	HP 310 x 110	1440 → 1600	930 → 1150
	HP 310 x 79	1030 → 1150	670 → 850

It is recommended that the driving of the piles in the field be controlled by a recognized pile driving formula such as the Hiley formula. It is also recommended that the piles should have reinforced flanges for improved driving resistance.

It is estimated that the piles will drive to sufficient depth to mobilize full capacity except possibly at the location of Pier 1 where Borehole 5 indicated the surface of the very dense strata to be only 3 m below the existing ground surface. This pier location will, however, be likely moved somewhat east (i.e. towards the floor of the valley) where the surface of the till can be expected to be deeper and it is unlikely that preboring will be necessary. For piles driven into the silt stratum on the eastern half of the valley, relaxation effects may have to be considered.

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For preliminary estimating purposes the piles can be assumed to drive 1.5 to 2 m into the very dense material before reaching an adequate set.

Unbalanced horizontal forces should be resisted by battered piles and, for frost protection, the underside of the pile caps should be established at least 1.2 m below the finished grade.

5.3 East Abutment Foundations

We understand that the finished embankment grade for the east abutment will be Elevation 89.0 m and that the proposed abutment location is Station 26+286 m where Borehole 16 and Cone Test 15 were extended. Due to the relatively unfavourable conditions, Boreholes 21 and 18 were also drilled further east towards the side of the valley at Stations 26+292 and 26+306, respectively.

The boreholes show that end-bearing steel H-piles would be the best suited foundation type. The capacities and criteria for piles driven to end-bearing in the very dense stratum are given in Section 5.2.

The embankment fill will, however, induce settlements in the underlying weak clay and the weak till zone immediately below
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the clay. In addition to the undesirable effects on the performance of the rail line, these settlements can be expected to cause a down-drag on the end-bearing pile foundations.

The height of the new embankment fill at the location of Borehole 16 (Station 26+286) will be approximately 4.5 m and the weak soils extend to a depth of about 5 m below the ground surface. On the other hand, at the location of Borehole 18, the height of the new fill will be about 3.7 m and the depth of the weak soils below the present grade is only about 4 m. From a geotechnical point of view therefore the location of Borehole 18, i.e. Station 26+306, is preferable to that of Borehole 16. At this location (Station 26+306 where Borehole 18 was drilled), if the fill is placed after stripping the topsoil and compacting the exposed grade from the surface, the estimated settlements due to the weight of the fill is about 45 mm. About three-quarters of the settlement can be expected to take place within 2 months. If the fill is placed about 2 months before driving the piles therefore the down-drag effects on the piles can be ignored.

Although from a geotechnical point of view it is preferable to place the abutment at Station 26+306, it may be more .../...

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economical to locate the east abutment at Station 26+286 (or at a Station between the two) to shorten the span of the bridge.

At Station 26+286 (B.H. 16), the existing ground elevation is 84.3 m therefore approximately 4.5 m of fill will have to be placed and the depth of the compressible soils is also somewhat greater compared with Station 26+306. If the embankment fill at this location is placed after stripping the topsoil and compacting the exposed inorganic subgrade from the surface by a heavy roller, then settlements of the order of 80 mm could be expected in the underlying natural soil under the weight of the embankment fill. Based on the consolidation test results, about 85 per cent of the settlement could be expected to take place in about 3 months. If the embankment fill can be placed about 3 months before driving the piles therefore, the down-drag effects on the piles can be ignored.

If, due to time constraints, a three month waiting period is not considered to be feasible, then the weak clays will have to be removed to the surface of the relatively competent soil and replaced with compacted granular fill. Based on the configuration shown on the Structural Foundation Request Plan .../...

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supplied to us, the soil exchange will have to be effected within an area extending laterally to 21 m on each side of the centreline of the structure (i.e. total width of 42 m) between Stations 26+272 and 26+302. From the borehole results, the average depth of the soil to be removed would appear to be 4.5 m. Due to the high water table, however, the proper compaction of the backfill will be very difficult. At the start, a trial section should be conducted where the removal of the soil and the backfilling are effected in short sections (4 to 5 m wide) and under close supervision. If, in spite of these measures, adequate compaction cannot be achieved below the groundwater table, the embankment fill should be left in place at least one month before driving the piles. The fill used should not contain oversize particles which could obstruct the driving of piles but should be free draining and easily compactable. The 'dynamic consolidation' method could also be employed to compact the fill and the underlying very loose till after the removal of the weak clay.

Place gran.
Backfill

Alternatively, if the three month wait period is too long but a two month period is feasible, then the following approach could be considered. Those portions of the piles which will be within the upper compressible zone of the subsoil could be coated with a thick (say 6 mm) layer of suitable bitumen
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(e.g. at Borehole 16 location, the sections of the pile above Elevation 80 m). In order to prevent the scraping off and removal of the bitumen during driving, it may also be necessary to preauger to a level corresponding to 2 to 3 m below the present grade. The space around the piles can then be filled with a suitable material such as very fine sand or coarse silt.

5.4 Lateral Earth Pressures

Assuming that free-draining granular material and adequate drainage is provided behind the abutments and the wing walls (Figure 6.9.6.1 OHBDC), the lateral earth pressure can be calculated by assuming active earth pressure conditions and using the following equivalent fluid pressures:

At Ultimate Limit State: 8 kPa/m

At Serviceability Limit State Type II: 6.5 kPa/m

The rigid walls of the abutments, however, should be designed to withstand the at-rest pressures which can be evaluated using the following equivalent fluid pressures:

At Ultimate Limit State: 10 kPa/m

At Serviceability Limit State Type II: 8.5 kPa/m

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When using the above values, it is assumed that the slope of the backfill behind the retaining structure is approximately level. Construction joints should be provided between the yielding and restrained parts of the retaining walls.

Care should be given to avoid the overcompaction of the backfill and the use of heavy compaction equipment behind the retaining walls and abutments. Compaction equipment, for use behind retaining structures, must be restricted in size as per current M.T.C. Specifications.

Water accumulation in the backfill behind the retaining structures should be prevented by the use of properly filtered, perforated pipes and weep holes.

5.5 Approach Fills

In the west side of the valley, the height of the embankment fill will be approximately 4.5 m. The depth of the surficial clay deposit here ranges from zero at Station 26+120 to 1.3 m at Station 26+136. Field vane tests in the clay gave undrained shear strength values of 96 and 114 kPa. Below, the subsoil consists of compact to very dense silty sand till. These conditions indicate a high factor of safety (greater than 4) against a shear failure.

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In the east side, the height of the embankment approach fill will be 4.7 m at Station 26+286 gradually decreasing further east (due to the rising ground surface elevation). Here the clay extends to a maximum depth of 4.4 m (at Station 26+286) and undrained shear strength values measured in-situ by field vane tests at Boreholes 16, 18 and 21 ranged from 23 to more than 120 kPa with an average value of about 60 kPa. Two undrained (quick) triaxial laboratory compression tests gave values of 30 and 43 kPa. The clay is underlain by an approximately 0.5 to 1.1 m thick loose to very loose till layer which is in turn underlain by compact to very dense till or sand. From these, it is our opinion that the factor of safety against a shear failure is greater than 2 and that there should be no stability problems. The design of the approach fills should therefore not be limited by the strength of the foundation materials underlying the site.

All organic and unsuitable soils must be removed before placing the fill. The exposed subgrade should be inspected and the approved subgrade should be compacted using a heavy compactor.

If approach fills are constructed from clean earth fills, 2 horizontal in 1 vertical side slopes can be used. The slopes of the embankment should be adequately protected .../...



against surface erosion.

For pile supported perched abutments, rockfill or fill containing boulder or cobble size particles should not be used in that part of the embankment through which the piles are to be driven.

5.6 Construction

Where the excavations extend into the silty sand till below the groundwater table, problems due to groundwater seepage could be expected, especially if sand seams are intercepted. If spread footing foundations are to be used for the west abutment which will be placed on the till, then, by means of careful dewatering techniques, the disturbance of the founding subgrade will therefore have to be prevented. This could probably be achieved by pumping from closely spaced, filtered sumps in conjunction with a skim coat of concrete; but should this method not be sufficient to prevent the disturbance of the subgrade, then other measures such as pumping from deep wells, wellpoints, etc., will be necessary to dewater and stabilize the soil.

The sides of temporary excavations in the properly stabilized till can be expected to be stable at 1 : 1 slopes.

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The excavations for the pile caps will likely be in the clay. Due to its low permeability, seepage through the clay should be moderate and can be handled by pumping from open sumps.

Across the floor of the valley in the soft clay, soft subgrade conditions could, however, be expected. To create a neat working area for the construction traffic, a granular layer of a skim coat of concrete should be placed on the exposed subgrade once the final excavation level is reached.

Temporary excavations deeper than 1.2 m should be sloped at 45 degrees or shored and braced. Probably there will be sufficient space for sloped excavation and therefore no shoring will be necessary. Within the coarse till deposits, below the water table, flatter than 45° side slopes could, however, be necessary.

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6.0 CONCLUSIONS

The boreholes show that because of the presence of weak clay, the use of end-bearing steel H-piles is the most suitable foundation alternative to support the proposed piers and the east abutment. The recommended location for the west abutment is Station 26+136 m where the use of normal spread footing foundations is feasible. It is our opinion, however, here too the use of driven steel H-piles would be preferable.

Due to the variable depth of the weak clay, the length of the piles can be expected to be variable across the valley, increasing from the abutment locations towards the middle of the structure.

With the proposed grades for the structure, the existing grade at the abutment locations will be raised by 4 to 5 m. The boreholes show that there exists an adequate factor of safety against a shear failure under the weight of the new fills. The weight of the approach fills will, however, induce settlements in the underlying weak clay and the upper weaker zones of the till. This settlement, in addition to affecting the performance of the transit line, will also cause a down-drag on the end-bearing pile foundations due to negative skin friction.

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The estimated settlement at the recommended west abutment location (Station 26+136) is 35 mm and the down-drag effects can be ignored if the piles are driven at least one month after the placing of the embankment fill. In the east side of the valley, the estimated settlements are of the order of 45 mm and 80 mm at the locations of Borehole 18 (Station 26+306) and Borehole 16 (Station 26+286), respectively. The down-drag effects on the piles can be ignored if the embankment fill is placed at least two months before placing the fill at Station 26+306 and three months at Station 26+286. Other alternatives such as soil exchange or bitumen coating of the piles are discussed in the text, in case the required waiting period is not possible.

7.0 STATEMENT OF LIMITATION

The Statement of Limitation, as quoted in the Appendix, is an integral part of this report.

DOMINION SOIL INVESTIGATION INC.

Z.S. Ozden, P.Eng.

ZSO:bh



A P P E N D I X

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

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_i	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

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

PHYSICAL PROPERTIES OF SOIL

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



PROCEDURES

The boreholes were located in the field with the aid of the line of stakes representing the proposed GO-ALRT line and Stations (previously established at the site by the surveyors of the Client).

The original drilling programme, consisting of nine boreholes and seven dynamic cone penetration tests, was performed during the period of December 30, 1983, and January 10, 1984 (Boreholes and Cone Tests 1 to 16, inclusive). One additional borehole and two dynamic cone penetration tests (Nos. 17, 18 and 19) were performed on January 10, 1984.

On January 28, 1984, the field work was supplemented by performing field vane tests adjacent to some of the boreholes previously drilled near the sides of the valley. At this time two additional boreholes (Boreholes 20 and 21) were also drilled.

The boreholes were advanced using a power auger drilling rig equipped with hollow-stem augers. Sampling in the boreholes was effected by the Standard Penetration test method and the test results, recorded as N-values or Standard Penetration Resistances, were used to infer the relative density or the .../...

Ref. No. 83-12-8

consistency of the strata. In addition, where the consistency of the soil permitted, the undrained shear strength was measured in-situ by field vane tests and relatively undisturbed samples were retrieved by means of 50 mm i.d. thin-walled (Shelby) tube samplers.

In Boreholes 1, 3, 9, 16 and 18, sealed piezometers consisting of 19 mm i.d. rigid pipe, perforated near the tip, were installed to facilitate long-term groundwater observations.

In addition to the boreholes, dynamic cone penetration tests were performed. This test consists of driving a 60° point, 50 mm diameter cone attached to drill rod continuously into the undisturbed ground with a driving energy of 475 J (625 N hammer falling freely a distance of 0.76 m) per blow. The number of blows for each 0.3 m of penetration is recorded and this provides an indication of the relative changes in the soil density with depth.

The ground surface elevations at the borehole locations were determined with reference to a local benchmark shown to us by the surveyors of the Ministry. This benchmark was "nail and washers in the south face of the second hydro pole west of Corbett Creek Culvert, north side of Highway 401, opposite to .../...

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Sta. 16+115 m, 30 m right". Its elevation was given to us as 85.65 m (geodetic).

The field work was performed under the supervision of technical personnel from Dominion Soil Investigation Inc. The drilling equipment is owned and operated by D.S.I.L. Drilling Inc.

The laboratory work for the project consisted of moisture content, Atterberg limit, bulk density, undrained (quick) triaxial compression and consolidation tests and grain size analyses. The test results are shown on the Record of Borehole Sheets and on Figures 1 to 8, inclusive.

A P P E N D I X
STATEMENT OF LIMITATION

The conclusions and recommendations in this report are based on information determined at the testhole locations. Subsurface and ground-water 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 investigations.

We recommend that we be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the testholes.

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 details of alignment and elevations stated in the report. Since all details of the design may not be known, in our analysis certain assumptions had to be made. The actual conditions may, however, vary from those assumed, in which case changes and modifications may be required to our recommendations.

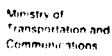
We recommend, therefore, that we be retained during the final design stage to review the design drawings and to verify that they are consistent with our recommendations or the assumptions made in our analysis.

In cases where these recommendations are not followed, the company's responsibility is limited to report accurately the information encountered in the testholes.

The comments given in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of boreholes may not be sufficient to determine all the factors that may affect construction methods and costs. 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 conclusion as to how the subsurface conditions may affect their work.

ENCLOSURES

RECORD OF BOREHOLE No 1										METRIC		
W P EGG 000-248		LOCATION Co-ords. 4,858,733N; 352,270E		ORIGINATED BY S.D.								
DIST 6 HWY GO-ALRT		BOREHOLE TYPE HOLLOW STEM AUGER		COMPILED BY S.D.								
DATUM GEODETIC		DATE 1984.01.05		CHECKED BY Z.S.O.								
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	NUMBER	TYPE			'N' VALUES	20 40 60 80 100					
85.2	GROUND LEVEL											GR SA SI CL
0.0	0.2 m Topsoil	1	SS	14								12 55 30 3
	Compact	2	SS	16								14 55 28 3
	V. dense	3	SS	77								11 51 37 1
	moist to wet	4	SS	68								21 55 23 1
	SILTY SAND	5	SS	58/	0.15 m							
	damp to moist	6	SS	80								
	occ. sand seams	7	SS	80/	0.15 m							Water seep- age into the borehole at 4.3 m from sand layer
	Finer	8	SS	75/	0.15 m							
	Coarser	9	SS	75/	0.15 m							
	Very frequent SAND layers below 8 m	10	SS	50/	0.15 m							
	moist to wet	11	SS	57								
	some gravelly sand & gravel layers	12	SS	65/	0.15 m							Frequent coarse gravel or cobbles be- tween 10.5 and 12 m
	damp	13	SS	100/	0.15 m							
		14	SS	82/	0.23 m							Sample 15: No recovery; auger sample taken
69.8		15	SS	80								
15.4	End of Borehole											



ENCL. 2

HIGHWAY ENGINEERING DIVISION-ENGINEERING MATERIALS OFFICE-SOIL MECHANICS SECTION

W P	EGG 000-24B	LOCATION	Co-ords. 4,858,725N; 352,270,275E	ORIGINATED BY	S.D.
DIST	6	HWY	GO-ALRT	BOREHOLE TYPE	DYNAMIC CONE PENETRATION TEST
				COMPILED BY	S.D.
DATUM	GEODETIC	DATE	1984.01.05	CHECKED BY	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100			W _p
85.3	GROUND LEVEL														
0.0							84								
83.0															
2.3	End of Cone Test									100/0.15m					

+3, x5 : Numbers refer to Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 3										METRIC	
W P EGG 000-24B		LOCATION Co-ords. 4,858,734N; 352,282E		ORIGINATED BY S.D.							
DIST 6 HWY GO-ALRT		BOREHOLE TYPE HOLLOW STEM AUGER		COMPILED BY S.D.							
DATUM GEODETIC		DATE 1984.01.05		CHECKED BY Z.S.O.							
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W _p W W _L		
84.7	GROUND LEVEL										
0.0	0.25 m Topsoil										
83.6	brown, firm SILTY CLAY		1	SS	6	SEAL	84				
1.1			2	SS	30						
	Coarser		3	SS	36						
	moist to wet		4	SS	39						
	Brown		5	SS	57						
	Dense Grey		6	SS	94						
	V. dense damp to		7	SS	95						
	moist		8	SS	50						
	SILTY SAND		9	SS	60						
	TILL										
	Finer										
	occ. sand seams damp										
77.7											
7.0	grey, v. dense		10	SS	64						
	FINE SAND										
	moist.										
75.9											
8.8	grey, v. dense										
75.1	SILTY SAND TILL		11	SS	86						
	damp										
9.6	End of Borehole										

RECORD OF CONE TEST No. 4										METRIC			
W P <u>EGG 000-248</u>		LOCATION <u>Co-ords. 4,858,746N; 352,295E</u>		ORIGINATED BY <u>S.D.</u>									
DIST <u>6</u> HWY <u>GO-ALRT</u>		BOREHOLE TYPE <u>DYNAMIC CONE PENETRATION TEST</u>		COMPILED BY <u>S.D.</u>									
DATUM <u>GEODETIC</u>		DATE <u>1984.01.06</u>		CHECKED BY <u>Z.S.O.</u>									
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	SHEAR STRENGTH					
83.5	GROUND LEVEL						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE						
0.0													
79.2													
4.3	End of Cone Test												



Ministry of
Transportation and
Communications
Ontario

DOMINION SOIL INVESTIGATION INC.
REF. NO: 83-12-8

ENCL. 5

RECORD OF BOREHOLE No 5										METRIC			
W P EGG 000-24B		LOCATION Co-ords. 4,858,737N; 252,300E		ORIGINATED BY S.D.									
DIST 6 HWY GO-ALRT		BOREHOLE TYPE HOLLOW STEM AUGER		COMPILED BY S.D.									
DATUM GEODETIC		DATE 1984.01.06		CHECKED BY Z.S.O.									
SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			VALUES	20					
83.4	GROUND LEVEL												
0.0	0.3 m Topsoil		1	SS	10								
	grey/brown, firm to stiff		2	SS	5								
	SILTY CLAY		3	SS	8								
80.9			4	SS	15								
2.5	Compact wet some clay		5	SS	83								
	V. dense grey												
79.4	SILTY SAND TILL												
	moist												
4.0	grey, v. dense		6	SS	90/	0.25m							
	SAND, mostly coarse, some gravel												
77.9	wet		7	SS	50/	0.1 m							
5.5	grey, v. dense		8	SS	60/	0.15m							
	SILTY SAND TILL												
	moist												
73.8			9	SS	70								
9.6	End of Borehole												

+3, x5 : Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE


RECORD OF BOREHOLE No 6										METRIC	
W P EGG 000-248		LOCATION Co-ords. 4,858,759N; 352,319E		ORIGINATED BY S.D.							
DIST 6 HWY GO-ALRT		BOREHOLE TYPE HOLLOW STEM AUGER		COMPILED BY S.D.							
DATUM GEODETIC		DATE 1984.01.06		CHECKED BY							
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE			'N' VALUES	20 40 60 80 100	W _p W W _L	20 40 60		
83.5	GROUND LEVEL										
0.0	0.2 m Topsoil										
	Brown Grey										
	grey	1	SS	2							Sample 1: No recovery
	v. soft to firm SILTY CLAY	2	SS	1							
		3	SS	1/	0.45 m						
	some sand & traces of gravel	4	SS	2							
75.0		5	SS	3							
8.5		6	SS	85							23 44 28 5
	grey, v. dense	7	SS	52/	0.15 m						
	SILTY SAND TILL	8	SS	65/	0.15 m						
	occasional thin sand seams damp	9	SS	104							
68.0		10	SS	62/	0.15 m						
15.5	End of Borehole										



ENCL. 7

METRIC

W P	EGG 000-24B	LOCATION	Co-ords. 4,858,750N; 352,324E	ORIGINATED BY	S.D.
DIST	6 HWY GO-ALRT	BOREHOLE TYPE	DYNAMIC CONE PENETRATION TEST	COMPILED BY	S.D.
DATUM	GEODETIC	DATE	1984.01.06	CHECKED BY	Z.S.O.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE LIQUID LIMIT CONTENT LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT	PLOT	NUMBER	TYPE			'N' VALUES	20 40 60 80 100	W _p	W	W _L		
83.4	GROUND LEVEL							SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE						
0.0														
74.1														
9.3	End of Cone Test									100/ 0.15 m				

+3, x5: Numbers refer to Sensitivity



ENCL. 8

METRIC

W P	EGG 000-248	LOCATION	Co-ords. 4,858,733N; 352,343E	ORIGINATED BY	S.D.
DIST	6 HWY	GO-ALRT	BOREHOLE TYPE	DYNAMIC CONE PENETRATION TEST	COMPILED BY
					S.D.
DATUM	GEODETIC	DATE	1984.01.09	CHECKED BY	Z.S.O.

[illegible]

+3, x5: Numbers refer to Sensitivity

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 9

METRIC

W P EGG 000-248 LOCATION Co-ords. 4,858,765N; 352,349E ORIGINATED BY S.D.
DIST 6 HWY GO-ALRT BOREHOLE TYPE HOLLOW STEM AUGER COMPILED BY S.D.
DATUM GEODETIC DATE 1984.01.09 CHECKED BY Z.S.O.

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 Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80					
82.9	GROUND LEVEL															
0.0	Topsoil, highly organic															
82.3																
0.6	grey v.soft to firm SILTY CLAY		1	SS	1	PIEZOMETER 0.45m SEAL	82									
			2	SS	1/		80									
			3	SS	1		78									
			4	SS	2		76									
	some sand & traces of gravel		5	SS	6		74									
			6	SS	2		72									
			7	SS	3		70									
	more sandy wet		8	SS	2		68									
68.4	stiff		9	SS	11		66									
14.5	compact SILTY SAND TILL		10	SS	28		64									
67.2	wet		11	SS	77		62									
15.7			12	SS	80/	0.15 m										
	grey, hard SILTY CLAY TILL		13	SS	82	0.15m										
	sandy damp		14	SS	50/	0.10 m										
61.3			15	SS	110/	0.15m										
21.6	End of Borehole															

+3, x5: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 10										METRIC				
W P EGG 000-248		LOCATION Co-ords. 4,858,788N; 352,367E				ORIGINATED BY S.D.								
DIST 6 HWY GO-ALRT		BOREHOLE TYPE HOLLOW STEM AUGER				COMPILED BY S.D.								
DATUM GEODETIC		DATE 1984.01.10				CHECKED BY Z.S.O.								
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					
82.9	Topsoil													
0.0														
82.3														
0.6														
	grey, v.soft to firm SILTY CLAY		1	SS	4									
			2	SS	1									
			3	SS	2									
			4	SS	2									
	some sand & traces of gravel		5	SS	2									
			6	SS	2									
72.9														
10.0	grey, v.stiff to hard SILTY CLAY TILL damp to moist		7	SS	37									
71.7														
11.2	grey, v.dense SILTY SAND TILL with silty sand & sandy silt layers		8	SS	50/	0.10m								
69.9														
13.0	grey, hard SILTY CLAY TILL sandy		9	SS	100/	0.08m								
			10	SS	95/	0.15m								
66.0			11	SS	105/	0.15m								
16.9	End of Borehole													



ENCL. 11

METRIC

W P EGG 000-248 LOCATION Co-ords. 4,858,780N; 352,372E ORIGINATED BY S.D.
DIST 6 HWY GO-ALRT BOREHOLE TYPE DYNAMIC CONE PENETRATION TEST COMPILED BY S.D.
DATUM GEODETTIC DATE 1983.12.30 CHECKED BY Z.S.O.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH							WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE x LAB VANE						
82.9															
0.0															

+3, x5: Numbers refer to Sensitivity

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF CONE TEST No. 12										METRIC				
W P <u>EGG 000-248</u>		LOCATION <u>Co-ords. 4,858,803N; 352,389E</u>				ORIGINATED BY <u>S.D.</u>								
DIST <u>6</u> HWY <u>GO-ALRT</u>		BOREHOLE TYPE <u>DYNAMIC CONE PENETRATION TEST</u>				COMPILED BY <u>S.D.</u>								
DATUM <u>GEODETIC</u>		DATE <u>1983.12.30</u>				CHECKED BY <u>Z.S.O.</u>								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES									
83.2	GROUND LEVEL													
0.0														
74.8														
8.4	End of Cone Test													

RECORD OF BOREHOLE No 13										METRIC					
W P EGG 000-248		LOCATION Co-ords. 4,858,796N; 352,395E		ORIGINATED BY S.D.											
DIST 6 HWY GO-ALRT		BOREHOLE TYPE HOLLOW STEM AUGER		COMPILED BY S.D.											
DATUM GEODETIC		DATE 1983.12.30		CHECKED BY Z.S.O.											
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					
83.2	GROUND LEVEL														
82.6	Topsoil, highly organic														
82.6 0.6															
	grey		1	SS	4										
	v.soft to firm														
	SILTY CLAY		2	SS	1/	0.45m									
			3	SS	1										
	some sand & traces of gravel		4	SS	4										
75.6															
7.6	grey, v.dense		5	SS	50/	0.12m									
	SILTY SAND TILL with SAND layers														
	moist to wet		6	SS	83										
72.9															
10.3	grey, v.dense		7	SS	75/	0.15m									
	SILT with fine sand & silty clay seams, moist														
71.8															
11.4	grey, hard		8	SS	50/	0.08m									
	SILTY CLAY TILL														
	sandy, damp														
69.2			9	SS	50/	0.10 m									
14.0	End of Borehole														

RECORD OF BOREHOLE No 14										METRIC	
W P EGG 000-24B		LOCATION Co-ords. 4,858,810N; 352,406E		ORIGINATED BY S.D.							
DIST 6 HWY 60-ALRT		BOREHOLE TYPE HOLLOW STEM AUGER		COMPILED BY S.D.							
DATUM GEODETIC		DATE 1984.01.04		CHECKED BY Z.S.O.							
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								
83.9	GROUND LEVEL										
0.0	0.35 m Topsoil, highly organic some sand seams stiff to soft brown grey SILTY CLAY		1	AS	-						
			2	SS	5						
			3	SS	7						
			4	SS	6						
			5	TH	-						
			6	SS	3						
			7	SS	1						
79.6											
4.3	wet v. loose compact moist grey SANDY SILT dense TILL		8	SS	1						
			9	SS	23						
			10	SS	32						
76.9											
7.0	grey, v. dense SILT damp to moist		11	SS	50/	0.10m					
74.5											
			12	SS	70/	0.15m					
9.4	End of Borehole										



ENCL. 15

METRIC

W P	EGG 000-248	LOCATION	Co-ords. 4,858,820N; 352,411E	ORIGINATED BY	S.D.
DIST	6 HWY GO-ALRT	BOREHOLE TYPE	DYNAMIC CONE PENETRATION TEST	COMPILED BY	S.D.
DATUM	GEODETIC	DATE	1984.01.04	CHECKED BY	Z.S.O.

[illegible]

+3, x5: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 16

METRIC

W P EGG 000-248 LOCATION Co-ords. 4,858,812N; 352,417E ORIGINATED BY S.D.
DIST 6 HWY GO-ALRT BOREHOLE TYPE HOLLOW STEM AUGER COMPILED BY S.D.
DATUM GEODETIC DATE 1984.01.04 CHECKED BY Z.S.O.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		NATURAL MOISTURE CONTENT		UNIT WEIGHT γ KN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	PLASTIC LIMIT W _p	W	LIQUID LIMIT W _L		
84.3	GROUND LEVEL											
0.0	0.2 m Topsoil some sand seams	1	SS	4		84						
	firm to stiff greyish brown grey SILTY CLAY	2	SS	5			2.7					
	traces of sand & gravel	3	SS	4		82						
	some sand & gravel wet	4	TW	-								
79.9	loose wet some clay	5	SS	1		80	2.5 +3.0 +2.3 +2.5				20.4	
4.4		6	SS	7								
	dense v.dense Boulder	7	SS	46								
	grey SILTY SAND TILL with frequent SAND layers wet to moist	8	SS	62		78						
		9	SS	110								
75.8		10	SS	50/	0.08 m	76						
8.5	grey, v.dense SILT damp to moist	11	SS	50/	0.08 m	74						
		12	SS	58/	0.15 m	72						
71.3		13	SS	50/	0.15 m	70						
13.0	grey, hard SILTY CLAY TILL sandy	14	SS	50/	0.08 m							
68.9		15	SS	85/	0.15 m							
15.4	End of Borehole											

RECORD OF CONE TEST No. 17										METRIC			
W P <u>EGG 000-248</u>		LOCATION <u>Co-ords. 4,858,822N; 352,421E</u>				ORIGINATED BY <u>S.D.</u>							
DIST <u>6</u> HWY <u>GO-ALRT</u>		BOREHOLE TYPE <u>DYNAMIC CONE PENETRATION TEST</u>				COMPILED BY <u>S.D.</u>							
DATUM <u>GEODETIC</u>		DATE <u>1984.01.10</u>				CHECKED BY <u>Z.S.O.</u>							
SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	SHEAR STRENGTH					
85.0	GROUND LEVEL						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE		WATER CONTENT (%)				
0.0													
80.0													
5.0	END OF CONE TEST						100/0.15 m						

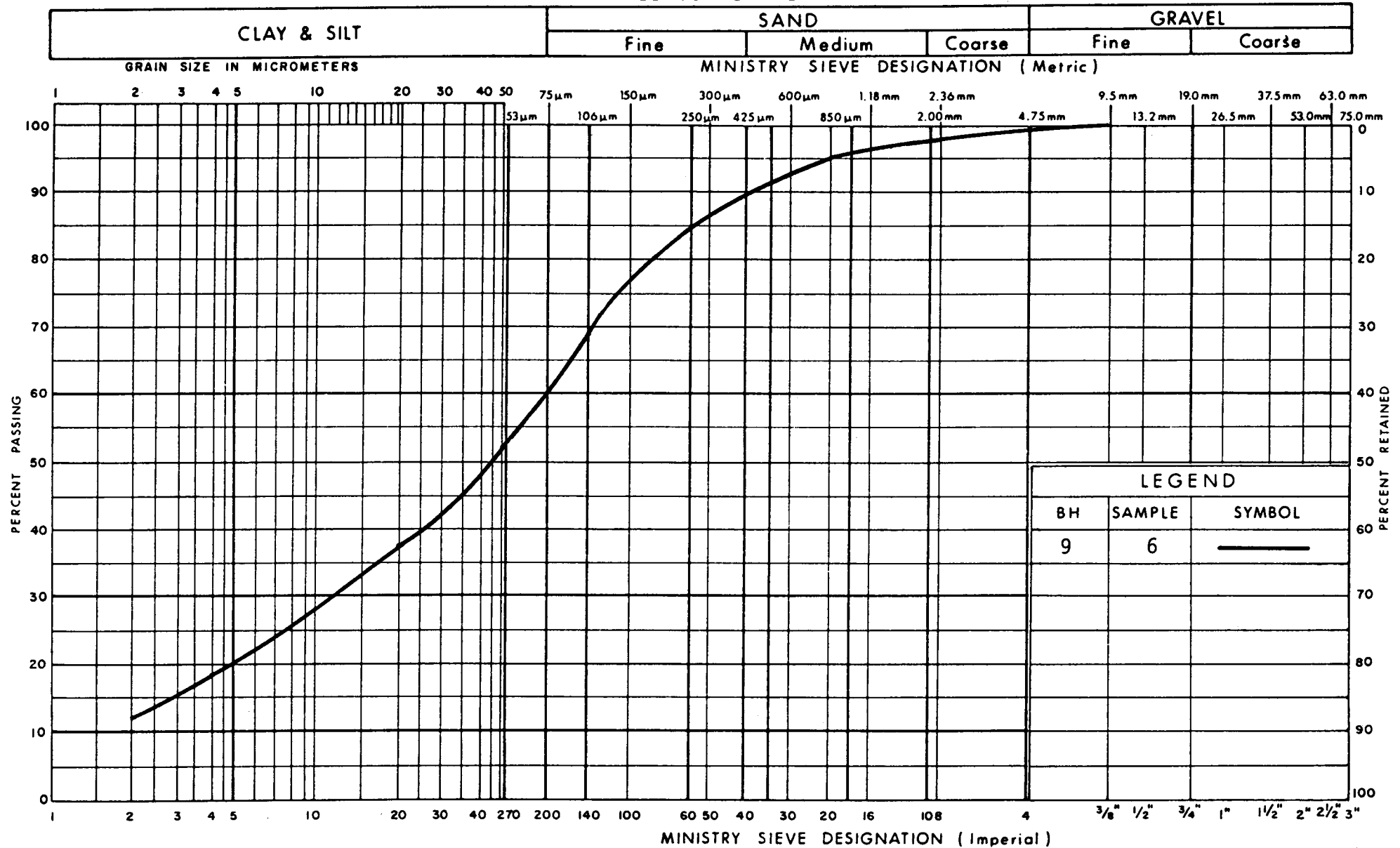
RECORD OF BOREHOLE No 18										METRIC			
W P EGG 000-248		LOCATION Co-ords. 4,858,833N; 352,426E		ORIGINATED BY S.D.									
DIST 6 HWY GO-ALRT		BOREHOLE TYPE HOLLOW STEM AUGER		COMPILED BY S.D.									
DATUM GEODETIC		DATE 1984.01.10		CHECKED BY Z.S.O.									
SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20 40 60 80 100					
85.3	GROUND LEVEL												
0.0	0.3 m Topsoil												
	SILTY CLAY		1	SS	13								
	stiff brown		2	SS	10								
	firm to grey		3	TW	-								
	very sandy, some gravel		4	SS	4								
81.7	grey, loose wet												
3.6	SANDY SILT TILL		5	SS	7								
80.8	grey, v. dense, wet		6	SS	63								
4.5	FINE SAND with silt seams		7	SS	77								
80.0	grey, v. dense moist		8	SS	55/0.15 m								
5.3	SAND with some gravel		9	SS	60/0.15 m								
79.4	grey, v. dense SILT		10	SS	72/0.15 m								
5.9	moist to wet		11	SS	50/0.10 m								
	silty fine sand layer		12	SS	52/0.15 m								
72.3	grey, hard		13	SS	50/0.10 m								
13.0	SILTY CLAY TILL												
71.3													
14.0	End of Borehole												

RECORD OF CONE TEST No. 19										METRIC	
W P <u>EGG 000-248</u>		LOCATION <u>Co-ords. 4,858,825N; 352,432E</u>		ORIGINATED BY <u>S.D.</u>							
DIST <u>6 HWY GO-ALRT</u>		BOREHOLE TYPE <u>DYNAMIC CONE PENETRATION TEST</u>		COMPILED BY <u>S.D.</u>							
DATUM <u>GEODETIC</u>		DATE <u>1984.01.10</u>		CHECKED BY <u>Z.S.O.</u>							
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER								
85.4	GROUND LEVEL										
0.0											
84											
82											
80.5											
4.9	END OF CONE TEST										

RECORD OF BOREHOLE No 20										METRIC				
W P		EGG 000-248		LOCATION		Co-ords. 4,858,736N; 352,287E		ORIGINATED BY		S.D.				
DIST		6 HWY GO-ALRT		BOREHOLE TYPE		HOLLOW STEM AUGER		COMPILED BY		S.D.				
DATUM		GEODETIC		DATE		1984.01.28		CHECKED BY		Z.S.O.				
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		NATURAL MOISTURE CONTENT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100		
84.1	GROUND LEVEL													
0.0	0.3 m Topsoil		1	SS	8									
82.8	SILTY CLAY stiff, br. some sand		2	SS	11									
1.3	wet		3	SS	29									
	compact		4	SS	44									
	dense		5	SS	90/	0.23 m								
	wet		6	SS	50/	0.13 m								
	v. dense		7	SS	50/	0.10 m								
	moist SILTY SAND													
	TILL													
	with sand layers													
79.3	finer													
4.8	End of Borehole													

RECORD OF BOREHOLE No 21										METRIC				
W P <u>EGG 000-248</u>		LOCATION <u>Co-ords. 4,858,820; 352,419E</u>		ORIGINATED BY <u>S.D.</u>										
DIST <u>6 HWY GO-ALRT</u>		BOREHOLE TYPE <u>HOLLOW STEM AUGER</u>		COMPILED BY <u>S.D.</u>										
DATUM <u>GEODETIC</u>		DATE <u>1984.01.28</u>		CHECKED BY <u>Z.S.O.</u>										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH kPa						
84.6	GROUND LEVEL													
0.0	0.25 m Topsoil													
	SILTY CLAY		1	SS	9									
	stiff soft to firm		2	SS	14									
	some sand & gravel		3	SS	5									
81.3			4	SS	3									
3.3	grey, v.loose to loose		5	SS	6									
80.2	SILTY SAND TILL		6	SS	20									
4.4	some clay wet		7	SS	78									
79.0	grey SILTY SAND													
5.6	compact & SANDY SILT													
	v.dense wet													
	End of Borehole													

UNIFIED SOIL CLASSIFICATION SYSTEM



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Communications

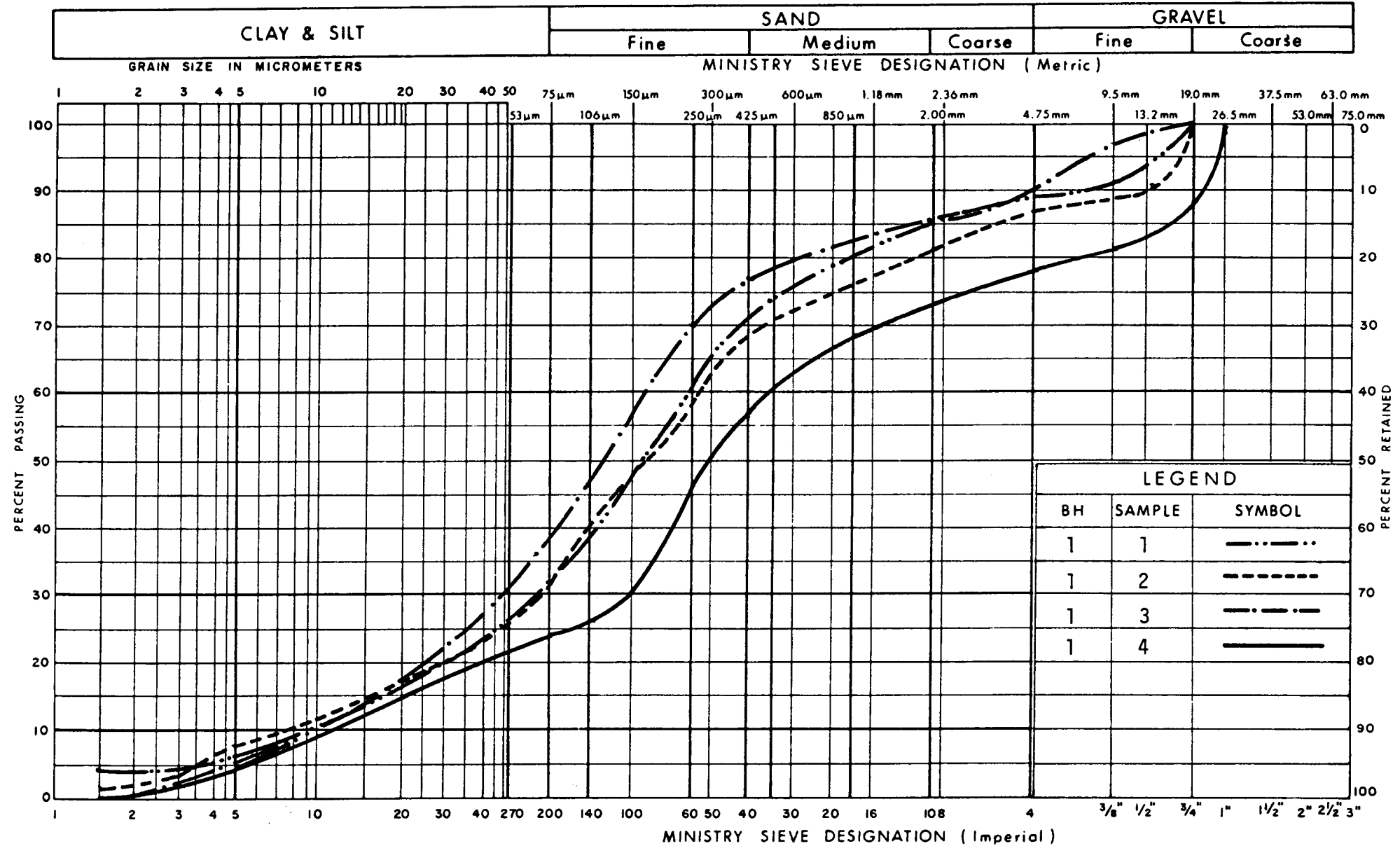
GRAIN SIZE DISTRIBUTION

SILTY CLAY with some Sand &
traces of Gravel

FIG No 1

W P EGG 000-24B

UNIFIED SOIL CLASSIFICATION SYSTEM



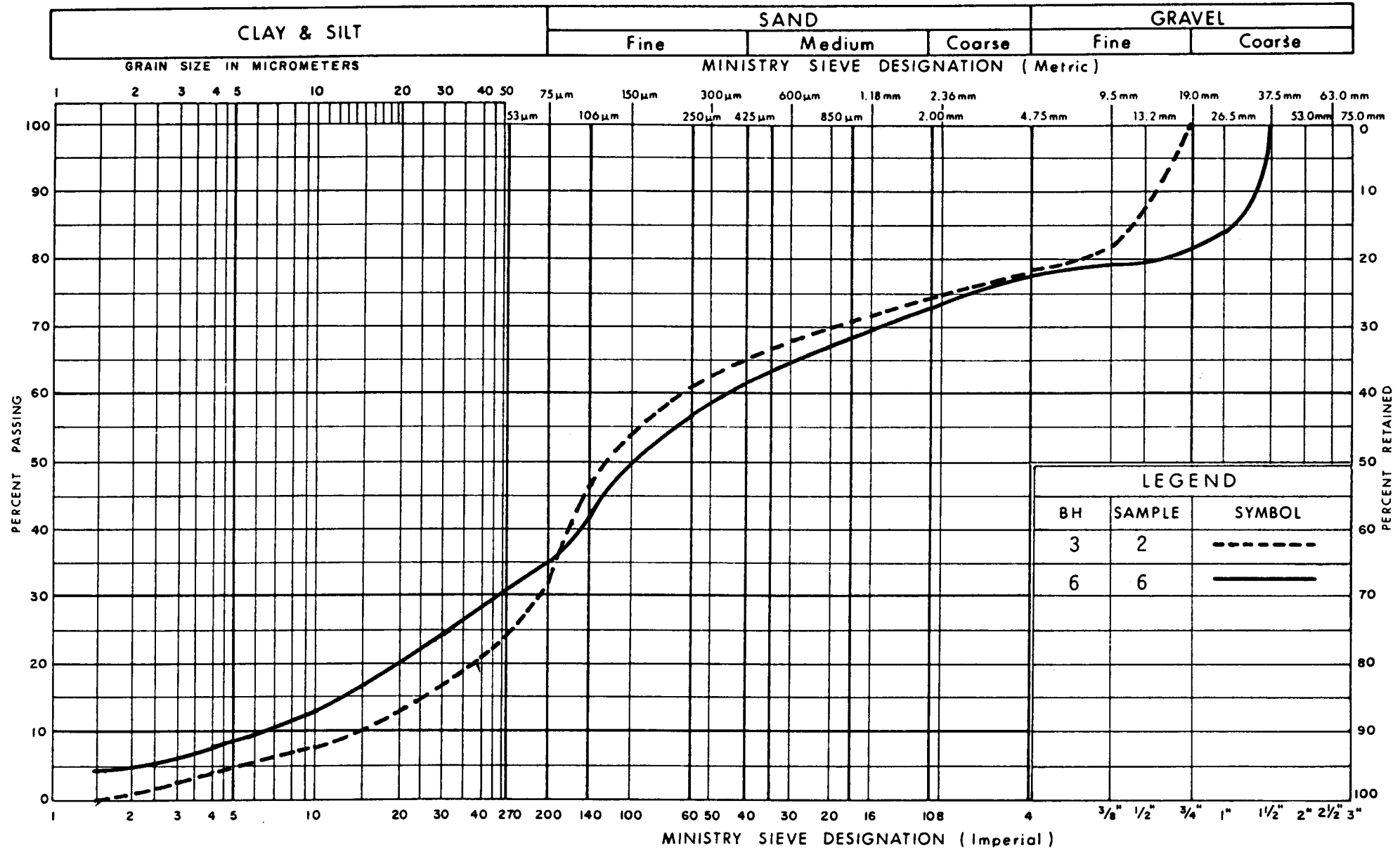
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GRAIN SIZE DISTRIBUTION
SILTY SAND TILL

FIG No 2

W P EGG 000-24B

UNIFIED SOIL CLASSIFICATION SYSTEM



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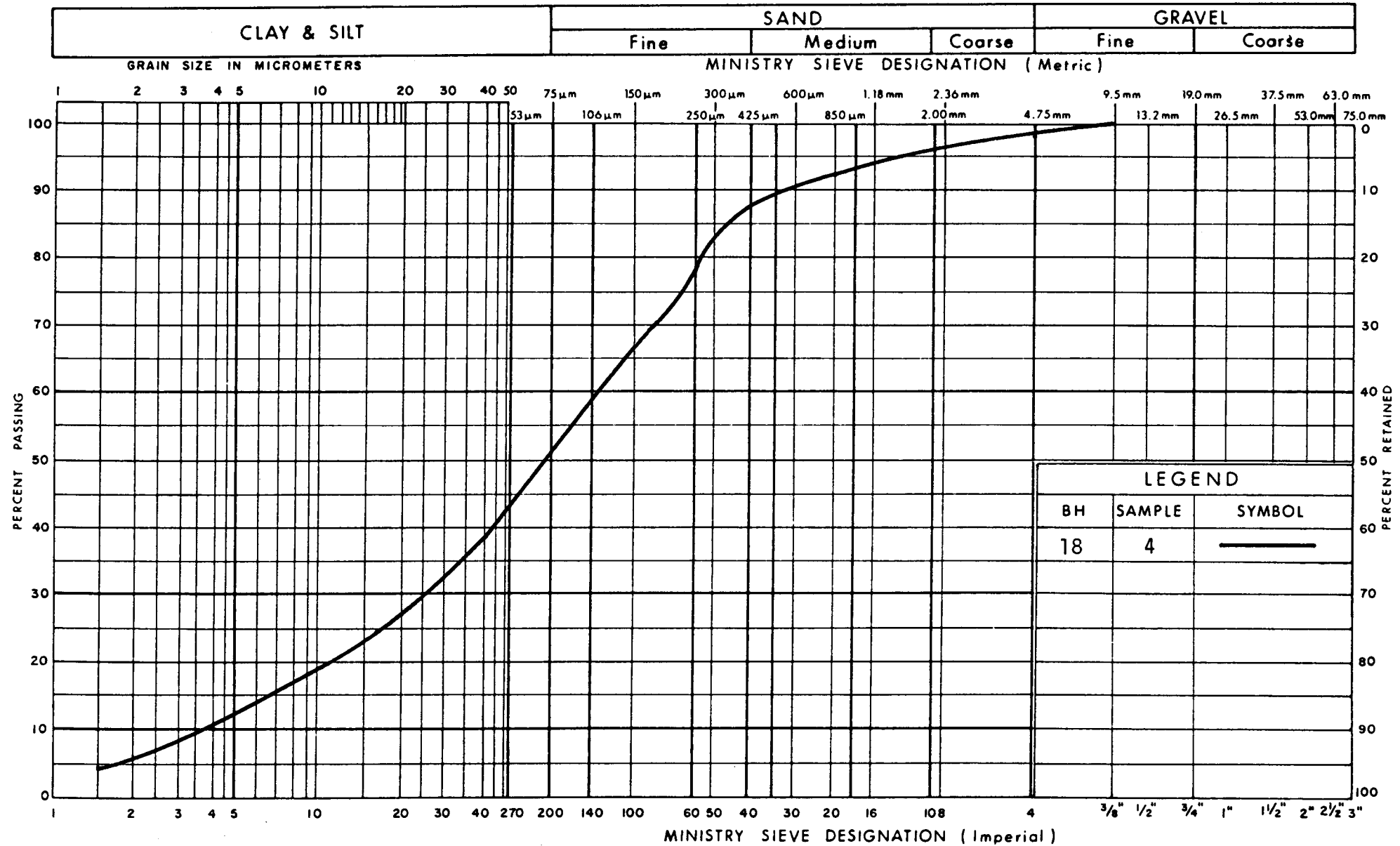
GRAIN SIZE DISTRIBUTION

SILTY SAND TILL

FIG No 3

W P EGG 000-24B

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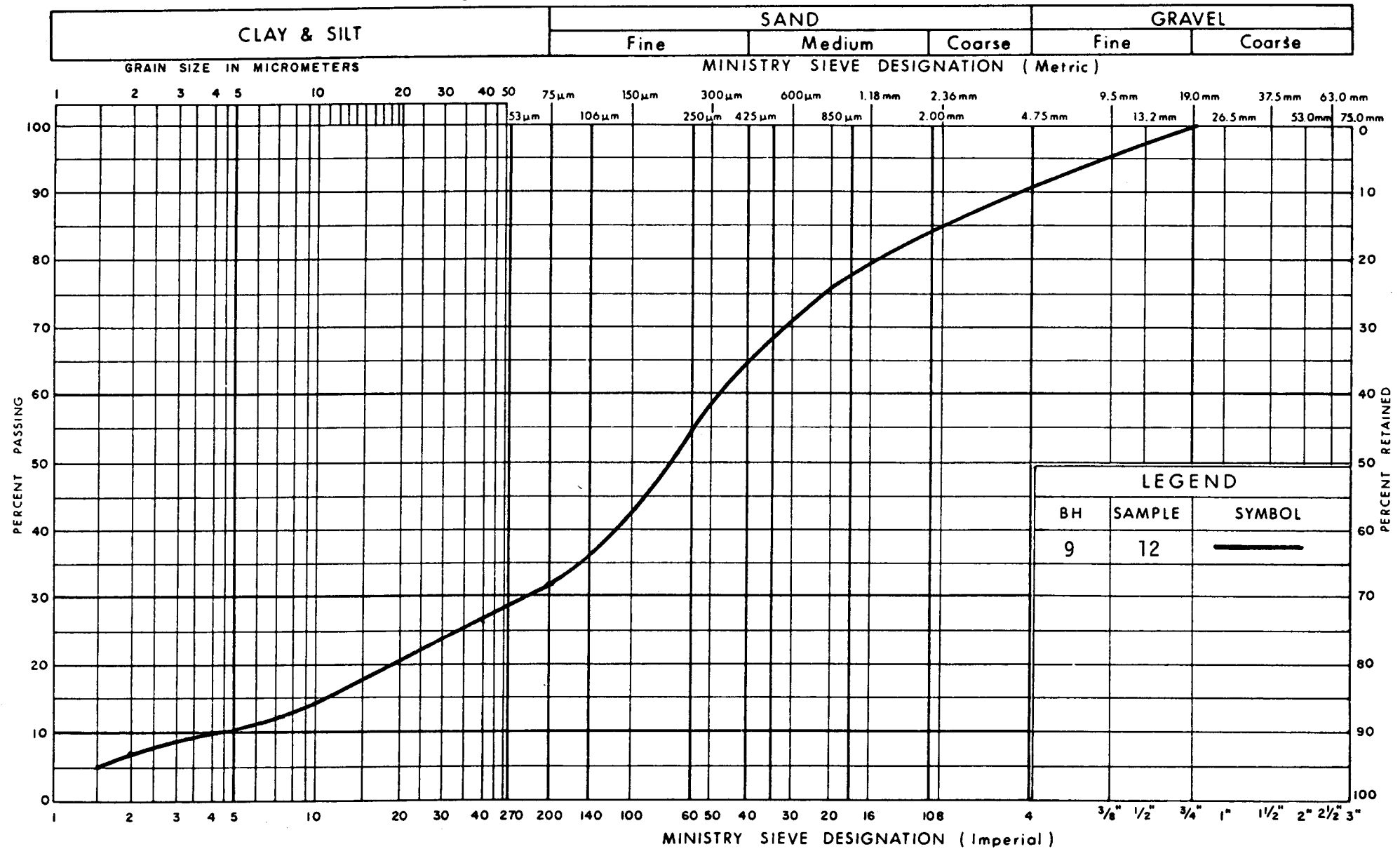
GRAIN SIZE DISTRIBUTION

SANDY SILT TILL

FIG No 4

W P EGG 000-24B

UNIFIED SOIL CLASSIFICATION SYSTEM



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Communications

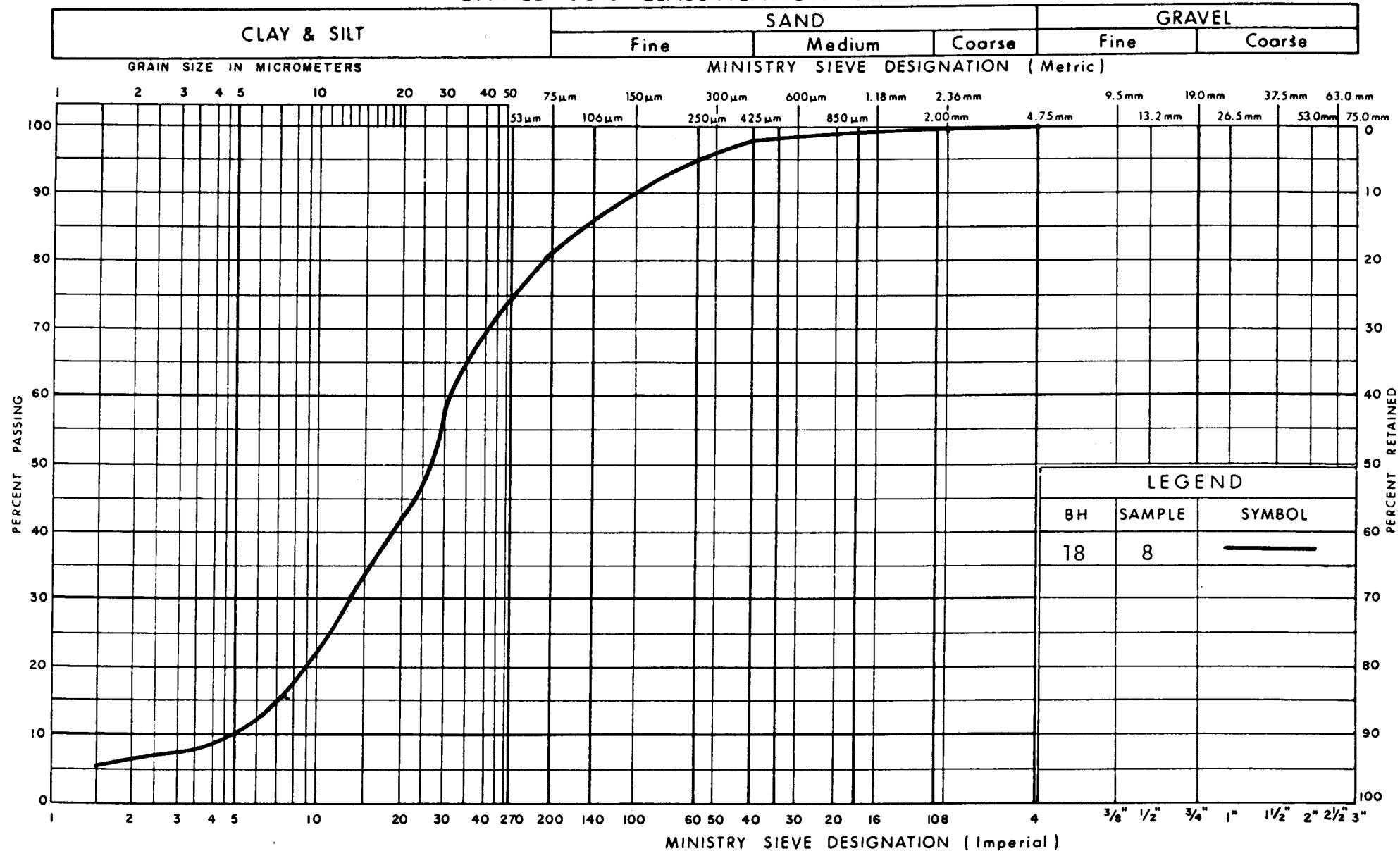
GRAIN SIZE DISTRIBUTION

SILTY CLAY TILL
Sandy

FIG No 5

W P EGG 000-24B

UNIFIED SOIL CLASSIFICATION SYSTEM



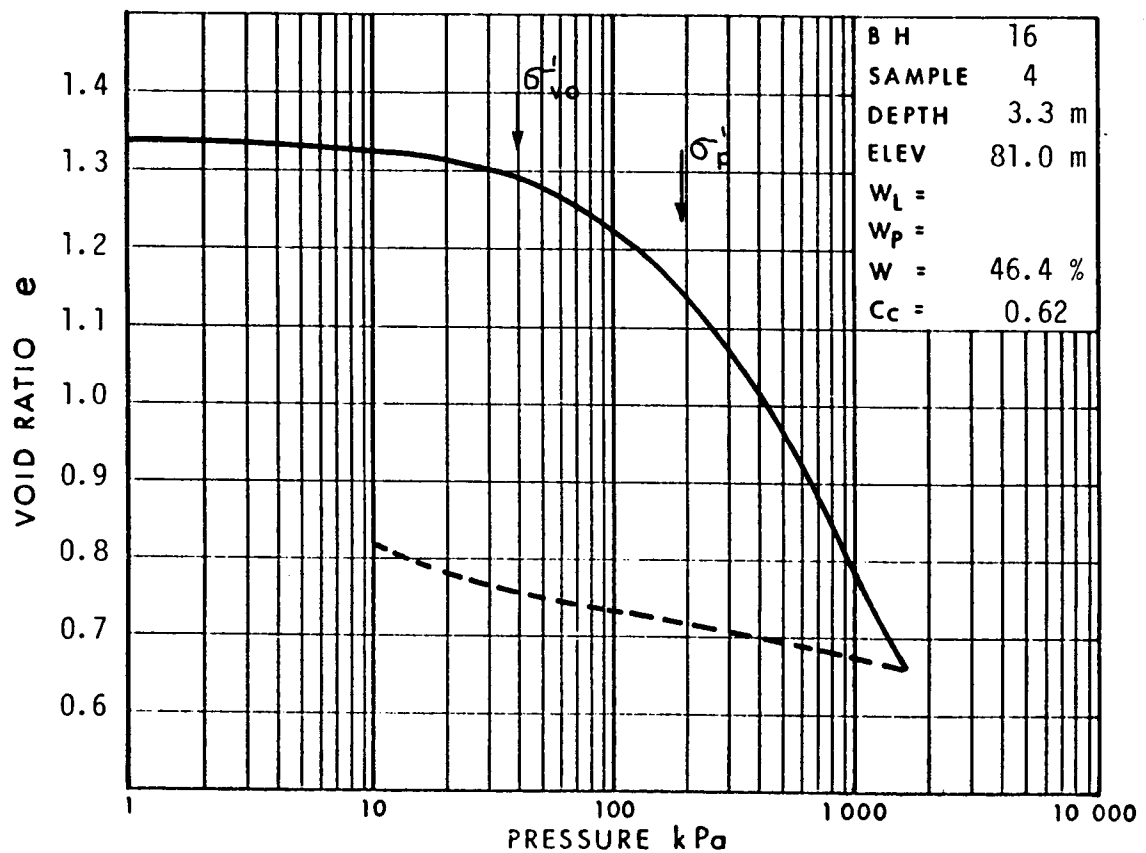
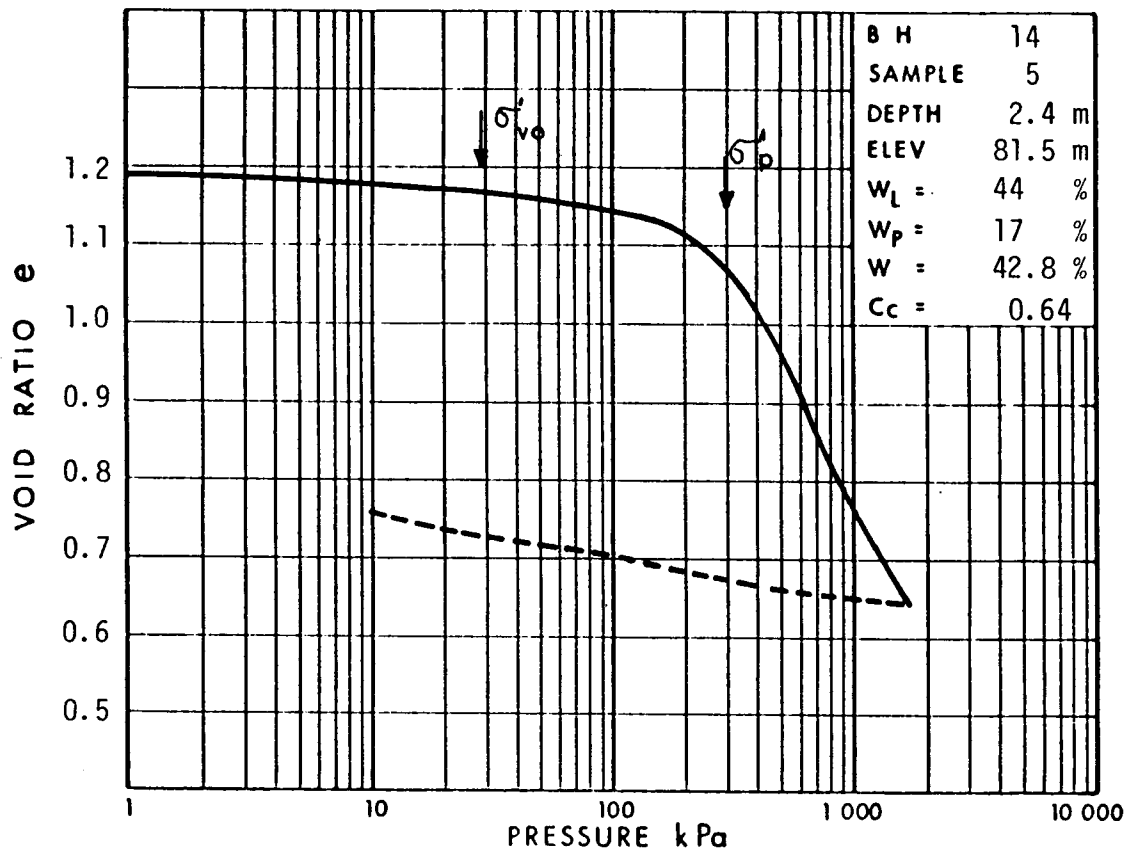
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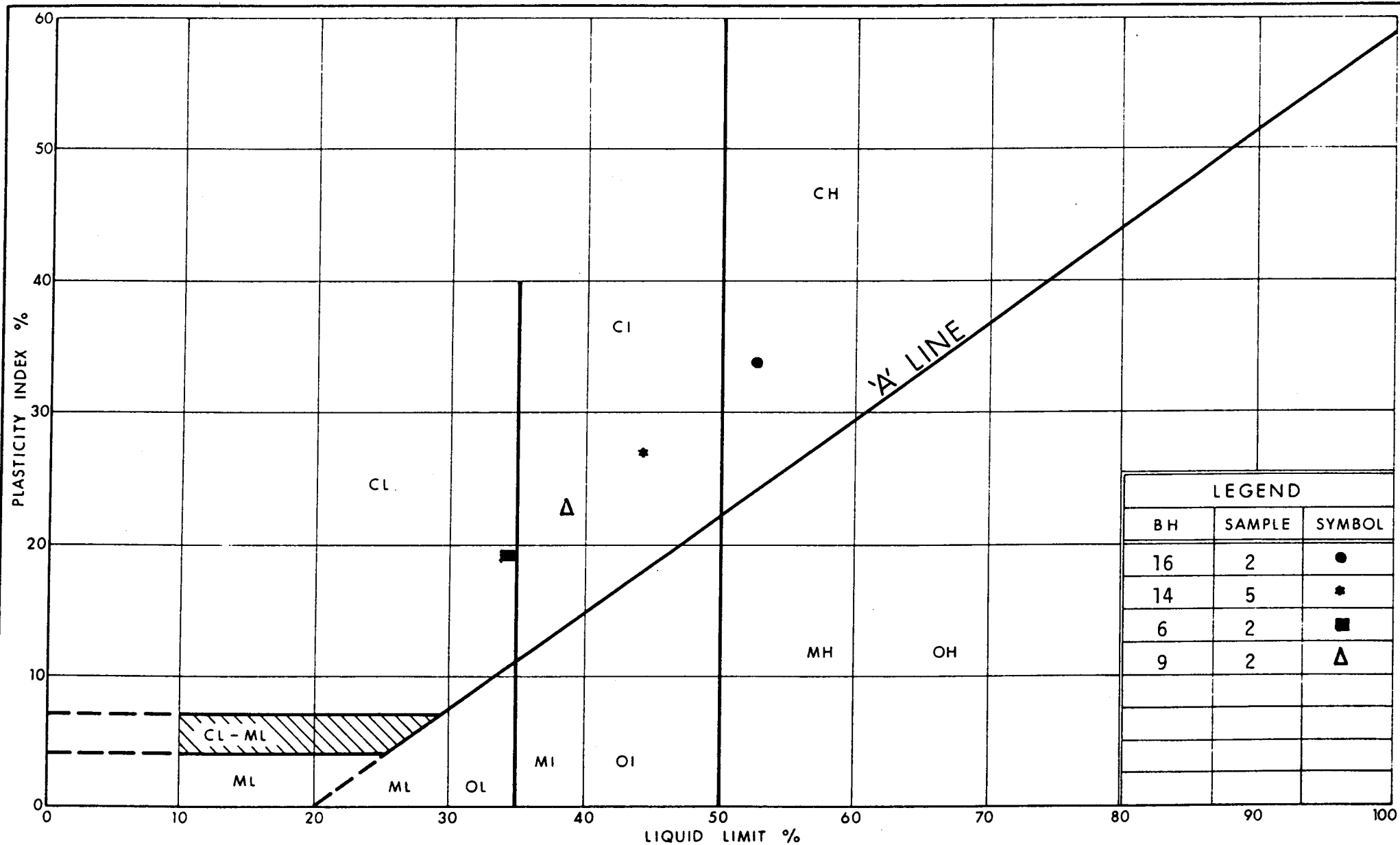
GRAIN SIZE DISTRIBUTION
SILT

FIG No 6

W P EGG 000-24B

VOID RATIO - PRESSURE CURVES





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PLASTICITY CHART SILTY CLAY

FIG No 8

W P EGG 000-24B

ALL DIMENSIONS SHOWN ARE
IN METRES AND/OR MILLI-
METRES UNLESS OTHERWISE
NOTED.



- | No | ELEVATION | CO-ORDINATES | |
|------|-----------|--------------|---------|
| | | NORTH | EAST |
| 1 | 85.2 | 4,858,733 | 352,270 |
| CT2 | 85.3 | 4,858,725 | 352,275 |
| 3 | 84.7 | 4,858,734 | 352,282 |
| CT4 | 83.5 | 4,858,746 | 352,295 |
| 5 | 83.4 | 4,858,737 | 352,300 |
| 6 | 83.5 | 4,858,759 | 352,319 |
| CT7 | 83.4 | 4,858,750 | 352,324 |
| CT8 | 82.9 | 4,858,773 | 352,343 |
| 9 | 82.9 | 4,858,765 | 352,349 |
| 10 | 82.9 | 4,858,788 | 352,367 |
| CT11 | 82.9 | 4,858,780 | 352,372 |
| CT12 | 83.2 | 4,858,803 | 352,389 |
| 13 | 83.2 | 4,858,796 | 352,395 |
| 14 | 83.9 | 4,858,810 | 352,406 |
| CT15 | 84.3 | 4,858,820 | 352,411 |
| 16 | 84.3 | 4,858,812 | 352,417 |
| CT17 | 85.0 | 4,858,822 | 352,421 |
| 18 | 85.3 | 4,858,833 | 352,426 |
| CT19 | 85.4 | 4,858,825 | 352,432 |
| 20 | 84.1 | 4,858,736 | 352,287 |
| 21 | 84.6 | 4,858,820 | 352,419 |

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

PROPOSED CROSSING AT
CORBETT CREEK AND GO-ALRT

CONTRACT NO	DWG NO EGG000-24B-A	REV	SHEET
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-  SILTY SAND AND SANDY SILT
  SANDY SILT TILL
-  SILTY SAND TILL

DRAWN BY: F.L.	DESIGNED BY:
DATE: 1984 02 13	
CHEK'D BY: <i>Richard Lee</i>	APPROVED BY:
SCALE FULL SIZE ONLY	
AS SHOWN	



GO-ALERT
Ministry of Transportation and Communications

PICKERING TO OSHAWA

PROJECT MANAGER