

ACRES INTERNATIONAL LIMITED

WHITBY-OSHAWA SECTION

ENGINEERING MATERIALS OFFICE

FOUNDATION DESIGN SECTION

WO EGG-001-3 DIST 6

HWY GO-ALRT STR SITE

GO-ALRT - CPR - GM SPURS

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1 - INTRODUCTION

Acres International Limited (Acres) was retained by the Ministry of Transportation and Communications of Ontario (MTC) to undertake the geotechnical investigation of a portion of the GO-ALRT east extension adjacent to the CPR-GM spur lines in Oshawa. The study was authorized by Agreement No. EGG-001-3 dated June 14, 1984, however, approval to proceed with the work was not received until February 6, 1985.

The segment of the GO-ALRT line included in this program is from Chainages (CH.) 28+300 to 28+650 which is located just east of Thornton Road in the city of Oshawa. Within this section, the line passes beneath two CPR-GM spur tracks which connect to the GM plant. The twin GO-ALRT rail lines will pass through a cut up to 15 m deep and be carried under the CPR-GM spurs in two concrete box structures.

To simplify descriptions related to the orientation of the project, it will be assumed that the GO-ALRT centerline extends in the east/west direction.

Drilling and soil sampling operations were performed by Longyear Canada Inc. (Longyear) under the supervision and direction of Mr. C. S. Bodimeade, an Acres geotechnical engineer. Fieldwork commenced on February 19, 1985, and was completed March 15, 1985. A plan of the site and the bore-hole locations are shown on Drawing 1 which is included in the pocket at the back of this report.

All soil samples were returned to Acres geotechnical laboratory in Niagara Falls for detailed examination, logging and testing.

The results of the field and laboratory investigations are presented in this report together with an interpretation of the data obtained and recommendations concerning the geotechnical aspects of the design and construction of the proposed works.

2 - EXPLORATORY WORK

The exploratory work consisted of 10 boreholes ranging in depths from 3.7 to 23.2 m and totaling 179.1 m. In performing the drilling and sampling operations, Longyear used a track-mounted CME-55 drill from February 19, 1985, to February 22, 1985. Because of the very hard drilling below a depth of approximately 12 m, and the problem of caving soils in upper part of the holes, the CME-55 drill was replaced by a CME-75 which was used from February 25, 1985, to the completion of the investigations on March 15, 1985.

The borings drilled in this program were numbered in the 200 series to differentiate them from other holes which were being drilled in the vicinity. Six boreholes, 201, 202, 203, 208, 209 and 210 were drilled along the line of the GO-ALRT cut. Boreholes 204, 205 and 206 were located in the CPR cut in the area of proposed concrete box structures. Borehole 212 was intended to study the soil conditions along the west spur line diversion. Boreholes 207 and 211 were deleted from the program.

During the drilling of Boreholes 201 to 203, both hollow and solid stem augers were employed to advance the borings, with the latter being used on the lower part of the holes where the very dense soils were encountered. However, this procedure proved to be ineffective due to the caving of the soils in the upper portion. The balance of the holes were drilled with the hollow stem augers.

In two of the boreholes, 201 and 209, the extremely hard drilling resulted in the shearing off of the bottom section of augers and their subsequent abandonment in the holes. Lengths of auger, 1.52 m long, were left in Boreholes 201 and 209 at depths of 13.7 and 16.8 m respectively.

Piezometers were installed in the following 7 boreholes--201, 202, 203, 204, 205, 206 and 208. At the location of Borehole 202, a separate hole was drilled for the piezometer installation due to the caving of the upper portion of the hole. The locations and details of the piezometer installations are shown on Drawing 1 and on the Record of Borehole forms. Where piezometers were installed, the standpipes have been protected at the ground surface by a protective pipe and a locked cap to avoid vandalism and damage.

During the drilling of some of the initial holes, problems were encountered with the loosening and upward flow of soil into the holes. This resulted in some "N" values which were not completely representative of the deposit. By maintaining a positive water head on the soil, equal to ground surface elevation, this problem was eliminated in subsequent holes.

The boreholes were tied-in by survey and referenced to the MTC grid. Table 1 summarizes the pertinent physical data for each hole.

The laboratory testing program included the following types of tests, the results of which are presented and discussed in subsequent sections of the report

- natural moisture content
- liquid limit
- plastic limit
- sieve and hydrometer analyses
- direct shear
- pH and sulfate content.

Record of Borehole forms, which are included following the report text, summarize all the field and laboratory data obtained.

TABLE 1

SUMMARY OF BOREHOLE
PHYSICAL DATA

<u>Borehole Number</u>	<u>Ground Surface Elevation (m)</u>	<u>Coordinates</u>		<u>Bottom of Borehole</u>	
		<u>North</u>	<u>East</u>	<u>Depth (m)</u>	<u>Elevation (m)</u>
201	106.2	4 860 039.1	353 975.6	18.6	87.6
202	106.7	4 860 074.1	354 034.0	18.7	88.0
203	107.8	4 860 099.9	354 078.8	20.3	87.6
204	103.6	4 860 114.4	354 089.9	17.4	86.2
205	103.6	4 860 112.5	354 109.9	21.7	81.9
206	103.5	4 860 134.0	354 119.4	23.2	80.3
208	108.7	4 860 143.6	354 150.4	20.1	88.6
209	106.5	4 860 179.9	354 211.8	18.3	88.2
210	104.2	4 860 214.1	354 273.4	17.1	87.1
212	103.0	4 860 143.3	354 062.6	<u>3.7</u>	99.3
TOTAL DEPTH DRILLED				<u>179.1</u>	

3 - SITE CONDITIONS

3.1 - General

In the vicinity of the site, the proposed GO-ALRT line is oriented at a slight skew to the CPR main line and is located at a distance of about 100 m to the south. It intersects two CPR-GM spur lines which join to the south of the GO-ALRT centerline and extend to the GM plant as shown on Drawing 1.

The site is located in a gently rolling till plain which slopes downward in the north and east. At the CPR main line, the tracks are almost at grade whereas where the spur lines cross the GO-ALRT centerline they are in a cut about 5 m deep.

The portion of the line covered by this investigation program is located within a cultivated field except for the area in the vicinity of the spur lines. Cut slopes adjacent to the spur lines are standing at approximately 1.5H:1V and appear to be fairly stable, however, there is some minor sloughing. Even though the investigations were undertaken near the end of the winter, while there was generally snow cover on the frozen ground, there were indications that the area near the spur tracks could be somewhat wet and boggy in the spring.

The ground surface elevations in the study area vary from a high of approximately 109 m down to about 104 m at the east end of the site and in the CPR-GM cut.

3.2 - Soil Conditions

The general soil stratification at the site consists of the following four major soil units.

- Topsoil.
- Glacial till which varies from sandy silty clay to sandy clayey silt to silty clayey sand.
- Interbedded silty and gravelly sands.
- Silty clay with silt partings.

The extent of these units along the GO-ALRT centerline is shown on Drawing 1. The contacts between the various units are complex and in many places abrupt changes in material types were observed. No bedrock was encountered in any of the boreholes.

Details of the various soil units encountered at the borehole locations, together with the summary of the laboratory test results, are summarized in the Record of Borehole forms. The grain size distributions obtained for the various soil units are shown in Figures 1 to 4. The plasticity charts for the glacial till and silty clay units are shown in Figures 5 to 7. Consistency limits and resulting soil classification types obtained for both the glacial till and silty clay units are summarized in Table 2.

(a) Topsoil

The cultivated field portion of the site is covered by approximately 0.5 m of topsoil. It generally consists of sandy clayey silt with small fibrous organic roots and exists in a compact condition except for the plowed portion. In the area of the CPR cut there is some sparse vegetation cover and topsoil development to a depth of about 0.3 m.

TABLE 2SUMMARY OF ATTERBERG LIMITS
AND SOIL CLASSIFICATIONS

<u>Borehole</u>	<u>Sample Number</u>	<u>Depth (m)</u>	<u>Liquid Limit (%)</u>	<u>Plastic Limit (%)</u>	<u>Plasticity Index (%)</u>	<u>Soil Classification</u>
201	5	6.3	20.4	11.6	8.8	CL
	6	7.8	12.8	10.7	2.1	ML
	14	16.7	36.9	18.8	18.1	CI
202	1	1.5	15.7	11.3	4.4	CL-ML
	3	3.3	23.2	12.4	10.8	CL
	13	13.8	25.0	12.8	12.2	CL
203	1	1.7	16.6	11.3	5.3	CL-ML
204	11	15.4	57.7	23.5	34.2	CH
205	10	13.9	16.0	10.7	5.3	CL-ML
	14	20.0	16.2	11.3	4.9	CL-ML
206	11	13.9	12.6	10.5	2.1	ML
	16	21.5	16.9	10.8	6.1	CL-ML
208	1	1.7	14.8	11.3	3.5	ML
	11	18.4	15.6	9.9	5.7	CL-ML
209	1	1.7	15.3	10.8	4.5	CL-ML
210	2	3.2	43.7	20.6	23.1	CI
	11	15.4	18.8	11.0	7.8	CL

(b) Glacial Till

The glacial till deposit(s) occur across at the site in irregularly shaped zones and with slightly variable properties. Typical till gradations are shown in Figures 1 and 2. They indicate that the material is well graded with combined silt and clay size fractions varying from 43% to 67%. Based on the consistency limits, density and sand content, the glacial till unit can be divided into four subunits which are described in the following paragraphs in order of increasing depth.

The uppermost till subunit, immediately underlying the topsoil, is about 4.0 m thick. Excavation for the spur lines removed this material from the track area. It can be classified as light brown to yellow sandy clayey silt (CL-ML) west of the spur lines and light brown sandy clayey silt to silty clayey sand to the east of the spur lines. Traces of gravel were found throughout. The material is compact to dense on the west side and very dense on the east. The natural moisture content of the samples varies from 7% to 13% with an average of about 9% which is generally below the plastic limit. At the location of Borehole 210, this unit is overlain by a compact sand layer.

The second subunit of mainly cohesive till exists as thin lenses at approximately el 100 and immediately underlies the uppermost till at the locations of Boreholes 201, 202, 209 and 210. The material on the west side can be classified as gray, firm, sandy, silty clay (CL) of low plasticity whereas on the east side it grades from gray sandy clayey silt to light brown sandy silty clay (ML-CL) of low to medium plasticity with consistency varying from stiff to hard. The natural moisture content, with the exception of material at the location of Borehole 210, is close to the plastic limit and generally varies from 9% to 18% with an average of

about 13%. The material at the location of Borehole 210 is of intermediate plasticity and wet with a natural moisture content of about 38%.

The third subunit appears to be continuous across the site since it was encountered in all boreholes near the centerline. It underlies the second subunit at the west end and the major interbedded sands throughout the central and eastern portions of the site as shown on Drawing 1. The thickness of this unit is defined only in the western and central portions of the site since the eastern boreholes did not completely penetrate the layer. At Boreholes 201 and 206, the layer was 8 m and 5 m thick respectively. The interbedded sand deposit extends into the till unit in the form of fingers at the location of Borehole 202. A lense of sand about 1.5 m thick was also found at the location of Borehole 209 on the east side. The material is very dense and can be classified as gray sandy clayey silt to silty clayey sand (CL-ML-SC) with traces of gravel. The natural moisture content is below the plastic limit and varies from 6% to 13% with an average of about 9%.

The fourth subunit of till occurs below el 84 and was encountered only in the vicinity of the spur lines at the locations of Boreholes 205 and 206. A layer of silty clay exists between the third and fourth subunits of till. The material in this fourth subunit can be classified as very dense, gray, sandy, clayey silt (CL-ML) with trace of gravel. The average natural moisture content is less than the plastic limit and about 7%.

Presence of cobbles or boulders in the till and the interbedded sand units was inferred during drilling but it is not possible to define their exact size and extent on the basis of this investigation. Their presence is certainly consistent with the geological origin of these deposits.

(c) Interbedded Silty
and Gravelly Sands

The interbedded silty and gravelly sand deposit generally occurs as a thick lense between the second and third till subunits. Its thickness varies from a maximum of about 11 m below the eastern portion of the CPR cut to about 1.5 m at the east end near Borehole 210. On the west side, the thickness is only about 4 m and the western extremity appears to consist of a series of lenses separated by till layers as shown at the location of Borehole 202. Uniform fine to medium sands are predominant in this cohesionless unit, but interbedded with these are occasional sandy gravel to gravelly sand beds and occasional silt or gravel layers. The types of sands encountered can be broadly classified as SP, SM and SW. Some of the samples tested indicated appreciable amounts of silt and clay fines in the order of 35% as shown in Figure 3. The density of the different types of sand ranges from compact to very dense, but is generally in the dense to very dense range. The major portion of this deposit, with the exception of a thin layer about 50 m wide adjacent to Borehole 208, is below the groundwater table. The natural moisture contents of the various sand layers range from 7% to 22% and average about 14%.

In Borehole 202, very low penetration resistances were experienced while performing a couple of Standard Penetration Tests between el 100.5 and el 98.5. The drill rods sank under their own weight during tests at el 100.5 and el 98.5. These loose conditions were attributed to the upward flow of groundwater into the borehole and the resulting boiling or loosening of the sand. By maintaining a positive head of water equal to ground surface elevation in subsequent tests, this problem was minimized and more representative penetration

resistance values were recorded in adjacent boreholes such as Borehole 203. A Shelby tube sample of silty fine sand, obtained immediately above the elevation where drill rods sank, was tested for shear in a direct shear box. The results as shown in Figure 8 indicate a friction angle of 41.5° .

(d) Silty Clay With
Silt Partings

Between the third and the fourth till subunits lies a relatively impervious silty clay deposit. Boreholes 201 to 204 penetrated the upper portion of the deposit and terminated within it. Only Boreholes 205 and 206 defined upper and lower boundaries. The extent of this unit on the east side and its contact with the till subunits are not known.

The material in general can be classified as gray silty clay of low to high plasticity (CL-CH). It contains silt partings which possibly exist on fissure surfaces and occasionally traces of fine sand and gravel. Consistency of the material is very hard. The natural moisture contents are close to the plastic limit and range from 11% to 28% with an average of about 20%.

3.3 - Groundwater Conditions

To assess the groundwater conditions across the site, seven piezometers were set in different soil units and the details of their locations and their respective soil units of response are as follows.

TABLE 3PIEZOMETER INSTALLATIONS

<u>Bore- hole</u>	<u>Piezometer Tip Elevation (m)</u>	<u>Piezometer Seal Elevation (m)</u>	<u>Elevation of W.L. Apr 17/85 (m)</u>	<u>Soil Response Unit</u>
201	102.8	104.0	105.5	First till subunit
202	94.5	95.4	103.8	Third till subunit
203	88.9	89.7	103.0	Silty clay unit
204	93.5	97.7	103.4	Third till subunit
205	82.3	83.8	100.0	Fourth till subunit
206	82.0	84.3	103.4	Fourth till subunit
208	89.8	102.7	103.8	Interbedded sand unit

With the presence of an impervious silty clay unit between the third and fourth till subunits there was some concern that a different groundwater regime may exist below the silty clay layer and possibly be artesian in nature. As a result, the piezometers in Boreholes 205 and 206 were set in the fourth till subunit. Piezometers were filled with water up to the top immediately after installation to accelerate the stabilization time and prevent any flowing of fines towards the tip. It is possible that the piezometers installed in Boreholes 203, 205 and 206 had not completely stabilized at the time of the last readings (April 17, 1985).

The last levels are shown graphically on Drawing 1 and on the Record of Borehole forms. All readings taken to date have been tabulated on the latter forms.

The observations indicate that, in general, only one ground-water level exists on the site and it is in the range of el 103 to 104 m. This is very close to the ground surface elevation in the vicinity of the intersection of the GO-ALRT centerline and the CPR-GM spurs.

The relatively high piezometric level recorded in Borehole 201 suggests the possibility of a localized perched groundwater condition with an impervious layer below the piezometer tip. The water levels in the piezometers installed in the silty clay unit and the underlying till unit are still dropping slowly. The piezometric levels in these lower units, even during this stabilizing stage, are generally slightly below the average groundwater levels recorded in the overlying till and interbedded sand unit indicating that the presence of an artesian groundwater situation is unlikely.

There will, no doubt, be some seasonal fluctuations in the groundwater level. The piezometers have been secured with locked protective caps to keep them operative for future readings.

4 - GEOTECHNICAL DESIGN AND CONSTRUCTION CONSIDERATIONS

4.1 - General

This section outlines, in general terms, some of the geotechnical factors which must be taken into account in the design and construction of the proposed works. They are dealt with in more detail in subsequent sections.

It is understood that the CPR-GM spurs must be maintained in operation throughout the construction of the GO-ALRT line. The preliminary structural design indicates that the GO-ALRT tracks will be carried in concrete box sections beneath the CPR-GM spurs. To build the concrete structures, it will be necessary to construct spur detours outside the area of the structures. Because of the confined construction space and the need to maintain the spurs in service, it is proposed that the east and west faces of the excavation for the box structures be retained by vertical walls located as shown on Drawing 1.

Placing the GO-ALRT tracks at the design grade requires excavations for the approach cuts which will be up to 15 m in depth. Since the existing groundwater level is approximately 10 m above the base-of-cut level, a groundwater lowering system will be required to permit the establishment of stable excavation slopes. The groundwater level must remain in the depressed state in the long term thereby requiring a permanent drainage system.

Because of property constraints in the area, it is desirable to employ the steepest feasible excavation slopes in the cuts on the east and west sides of the spur lines.

4.2 - Summary of Geotechnical Site Conditions

The subsurface soils at the site essentially consist of 4 units

- topsoil
- glacial till
- interbedded sands
- silty clay.

These exist in a rather complex stratigraphic sequence as shown on Drawing 1.

The topsoil zone is relatively thin and contains some organic matter. The till is well graded, of low plasticity with combined silt and clay fractions ranging from 43% to 67%. The zone of till at approximately el 100 at the west and east ends is relatively more plastic than the other till units. The interbedded sands range in gradation from silty to gravelly sands so will have some variation in drainage characteristics. With regards to density, both the glacial till and sand deposits generally exist in a dense to very dense state. The silty clay layer is located below the excavations for this project and because of its hard consistency should not have any detrimental influence on the works. The presence of some lenses of sand within the till zone will improve the drainage characteristics of the till. With the exception of a possible minor perched water situation at the west end there appears to be only one groundwater table at the site. The possibility of an artesian groundwater condition beneath the impervious silty clay unit appears unlikely to exist based on data obtained to date.

4.3 - Dewatering and Drainage

4.3.1 - General

Construction of the proposed works in the study area requires an excavation which will extend approximately 10 m below the present groundwater level (el 103 to 104). Lowering of the water table is essential to permit stable excavation slopes both during construction and in the long term. Since it is also an integral part of the problem of slope stability it will be discussed further in Section 4.4. The bulk of the excavation will be carried out in low plastic, sandy clayey silt to silty clayey sand (glacial till) or the interbedded silty and gravelly sand deposits. The former are relatively impervious in comparison with the latter.

4.3.2 - Construction Conditions

To permit the movement of construction equipment in the cut and provide stable slopes, it will be necessary to maintain the groundwater level a minimum of 1 m below the excavation surface. An increase in the moisture content of the glacial till by only a few percent in the range of the optimum moisture content could have a major effect on its trafficability. Furthermore, saturation of the sands will result in them flowing into the excavation thereby impairing the stability of the slopes and the foundation conditions for structures.

There are various dewatering techniques available which can be applied at this site, however, not all methods are equally applicable to all soil conditions. The

less pervious soils, glacial till, will be more difficult to dewater than the sands. They require that the dewatering wells be placed more closely than in the more pervious sands.

Some of the various types of dewatering equipment include well points, eductor wells and deep wells. A single row of well points is generally only effective in lowering the groundwater level by about 6 m in a single stage. A double-stage system would be required to lower the groundwater level by 10 m as required on this site. Eductor or deep wells could accomplish the dewatering in a single stage. Spacing of the wells will be the critical factor in the performance of the system. A group of observation wells is an essential part of the dewatering system to monitor the progress of the groundwater lowering and to provide an indication of required modifications and additions to the initially installed system.

It is anticipated that, except in the sand deposits, the quantity of water to be pumped will be fairly small.

In the glacial till zones, it may prove necessary to supplement the dewatering well system with trenches filled with pervious granular material to achieve the groundwater lowering between wells and dry up the near surface soils.

The performance of a pumping test was not included in the scope of this investigation program, however, such a test performed before the start of construction would provide data which would be extremely valuable in the design of the dewatering system.

4.3.3 - Long-Term (Permanent) Dewatering System

In order to maintain the relatively steep slopes stable in the long term, it is essential that the groundwater remain in a depressed state throughout the life of the cut. It is recommended that this be accomplished by the combination of a granular berm placed on the lower part of the slope and two drain lines installed parallel to the centerline of the cut as shown in Figure 10. These drainage facilities are also discussed in Section 4.4 on slope stability.

The drain lines are located adjacent to the toe of the slope and below the berm level at el 101. They should consist of perforated galvanized drainage pipes surrounded by pea gravel or 6-mm clear crushed stone which in turn is surrounded by a free-draining sand such as concrete fine aggregate. The natural soils in the base and walls of the drainage trenches should be blanketed everywhere with the concrete fine aggregate. As an added precaution against the migration of the natural soils into the drainage system, consideration should be given to placing a filter cloth against the natural soil prior to placing the concrete fine aggregate.

The drain at the toe of the slope is required to also maintain the water level below the depth of frost penetration beneath the tracks to avoid potential frost heave problems. It is recommended that these drains, as well as the ones below the el 101 berm, be placed a minimum of 2 m below the grade.

Construction of these drains results in a temporary oversteepening of the slope while the trench is open. To avoid slope instability problems, it is recommended that these drains be constructed in short sections with the trenches being backfilled with compacted fill before opening up adjacent sections.

Since the base of the cut grades down toward the west, the perforated drains should also slope to the west where they can be connected to drains in the adjacent study area and be directed to a creek west of Thornton Road. Because it is important that these drains remain operative and effective in the long term, it is recommended that manholes be placed on the lines to permit inspection and maintenance of the lines should the latter become necessary.

It is understood that the berm drain in the north facing slope in the portion of the cut west of CH. 28+300 will be at a higher elevation than 101. It is recommended that the drain at el 101, from east of CH. 28+300, be carried down the slope to connect with the toe drain at some point west of CH. 28+300.

4.4 - Stability of Excavation Slopes

4.4.1 - General

Construction of the proposed works involves the excavation of both temporary slopes adjacent to the box structures and permanent slopes throughout the balance of the line. Both types of slopes are in excess of 12 m in height and extend approximately 10 m below the present groundwater level. Lowering of the groundwater table and maintaining it in a depressed state are essential for ensuring the safety of excavation slopes both during construction and in the long term.

The upper three till subunits and the interbedded sand deposits will be involved in these excavations. The till is relatively impervious in comparison with the

interbedded sand deposits, however, the inclusion of some sand lenses in the till is likely to improve its overall permeability and drainage characteristics. After a critical examination of the results of the Standard Penetration Tests and taking into account the fact that the bulk of the excavation will be below the groundwater level, it was considered appropriate to group both the till and interbedded sand units as frictional materials with an angle of internal friction of 36° . This generalization appears reasonable for the major portion of the cut slopes from CH. 28+380 to 28+600. However, near the extremities of the project, particularly at the west end, a softer cohesive zone exists at approximately el 100. The impact of this localized zone on the stability of the slope was checked by assigning it an undrained shear strength of 38 kPa. Considerations were also given to the presence of this weaker zone during the design of cut slopes and the evaluation of measures to improve their long-term stability.

To resolve the potential problem associated with the zones of interbedded sands where the drill rods sank under their own weight, a representative sample of silty sand was tested in direct shear. The results indicated a friction angle of 41.5° which is in excess of the design value. This tends to confirm that the loose sands probably resulted from the upward flow of water as indicated in Section 3.2.

4.4.2 - Temporary Cut Slopes

Temporary cut slopes in the area of the box structures will extend from el 104 to about el 92. The materials involved in the western portion of the excavation will be primarily glacial till and some interbedded sands

whereas on the eastern side the material is mainly the interbedded sands, assuming that the distribution of soils throughout the cut is similar to that in the structure area. Considering both materials as frictional with an angle of internal friction of 36° , a cut slope of 1.6H:1V was considered appropriate with a factor of safety of 1.16.

The materials in the cut slopes will be fairly erodible under the action of precipitation and surface runoff from the CPR lands. If the extent of erosion and migration of material to the bottom of the cut becomes a problem, it may become necessary to provide some erosion protection to the temporary slopes and also ensure that surface waters from beyond the cut area are intercepted and diverted before they reach the crest of the cut slopes.

The groundwater table must be lowered by a suitable system of wells and drains to at least 1.0 m below the face of the excavated slope. Depending on where the wells are located, there may be a tendency for the groundwater to seep from the lower portion of the slopes resulting in sloughing and slumping of the till and flowing of the sands. Installation of the permanent toe drains, and pumping from them, will assist in drawing the water table down and preventing it from exiting on the slope.

4.4.3 - Permanent Cut Slopes

The cut slopes for the approaches to the concrete structures vary in height within the study area. In the western portion, from CH. 28+300 to CH. 28+420, the track grade has been set at approximately el. 95.6. For slope stability computations, the excavation was

considered to extend about 1.5 m below the top of rails to provide for ditches and placement of ballast or select embankment fill. The ground surface elevation in this area varies from approximately 109 m on the south to 105 m on the north. Such a configuration results in slopes approximately 15 m and 11 m high on the south and north sides respectively.

East of the spur lines, between CH. 28+500 and CH. 28+150, the track grade rises slightly and the ground surface elevation lowers. This results in slope heights ranging from approximately 15 m at CH. 28+500 to 8.5 m at CH. 28+650.

The stability analyses for the design of cut slopes were performed using an Acres computer program developed to handle general slope stability problems by adaptation of the modified Bishop method. The program features a special random technique for generating potential failure surfaces and subsequently determining the most critical surfaces and their corresponding factors of safety. The critical failure surfaces generated and the corresponding factors of safety computed for various sections and combinations of the slopes are shown in Figure 9. The analyses indicate that slopes with a minimum amount of drainage provisions that is, only a toe drain, should be cut at 3H:1V to have a factor of safety of 1.3.

On the understanding that the width of the property right-of-way is a critical factor in this area, considerations were given to a number of configurations in an attempt to achieve the most compact and optimum arrangement. The recommended solution involves a relatively steep cut slope of 1.9H:1V, together with a free draining granular berm and 2 drain lines running parallel to the cut as shown in Figure 10. The berm

consists of two material zones, a 0.5-m thick layer of concrete fine aggregate immediately overlying the natural soils with the major fill zone consisting of Granular "B" fill. For the granular berm material, an angle of internal friction of 32° was used in the analyses. The proposed horizontal width of the granular berm at the base of the cut is 4 m. The granular berm height and width were optimized taking into consideration the presence of the softer zone near el 100 and the location of groundwater table at approximately el 103. Based on the analyses, a constant berm elevation of 101 m was considered to be most appropriate. The berm width at el 101 varies, depending on slope height. Where the crest of slope elevation is 106 m or lower, the berm width would be 3 m. For a crest elevation of 109 m, the berm width would increase to 4 m. These parameters result in overall average slopes of approximately 2.18H:1V with minimum factors of safety of approximately 1.25.

Without any erosion protection, the exposed cut slopes will erode and gully under the action of precipitation. To minimize such deterioration, it is recommended that the exposed slopes be covered with a layer of topsoil and hydro seeded in accordance with MTC practice or covered with a layer of granular material suitably graded to prevent erosion.

4.5 - Tieback Walls

Construction of the concrete box structures requires the temporary relocation of the CPR-GM east and west spur lines. During the period of construction both spur lines must be operational at the present level without any interruption in service.

It is proposed to divert the traffic onto detour lines which will be built just outside the area required for construction of the concrete box structures. Such a scheme requires the design and installation of temporary tieback walls to retain the soil supporting the detoured tracks while work proceeds with the excavation and concrete works.

It is further proposed that the site of the new trackwork be regraded to approximately el 104. Before excavation for the new structures can proceed, it will be necessary to dewater the construction area and install the temporary tieback walls. Excavation will extend down to approximately el 92 or 12 m below the general site grade.

The soils to be encountered at the tieback wall locations have been briefly outlined in Section 4.4.2.

The length of the tieback wall on the west side will be about 45 m and about 60 m on the east.

Consideration was given to conventional anchored sheet pile walls for supporting the spur lines. This potential solution was abandoned, however, because of the anticipated problems in driving sheet piles through the dense to very dense soils. A tieback system consisting of steel soldier piles, timber lagging and grouted soil anchors is, however, considered feasible. A typical arrangement of a tieback system suggested for the maximum section of the tieback wall, together with the design earth pressure distributions is illustrated in Figure 11.

The following guidelines are suggested for the design and construction of the tieback system. They do not in themselves constitute a final design.

(a) Soil Conditions

The major soil units to be dealt with in excavation are the till and the interbedded sand unit, and the extent of these units is variable across the site. For design purposes it is recommended that both the till and the interbedded sands be considered as frictional materials with the following parameters

- angle of internal friction $\phi = 36^\circ$
- moist unit weight above GWT, $\gamma = 21 \text{ kN/m}^3$
- submerged unit weight below GWT, $\gamma' = 11.5 \text{ kN/m}^3$
- groundwater table to be lowered to the bottom of cut and maintained there at all times.

(b) Soldier Piles and Timber Lagging

The soldier piles should be installed by predrilling vertical auger holes with a diameter of approximately 600 mm. To keep the holes from collapsing, it will probably be necessary to either temporarily case them or use drilling mud. On setting the pile in the hole, the lower portion, up to excavation grade, would be filled with concrete. Where drilling mud is used, the concrete would be placed by tremie methods. A typical soldier pile spacing of 2 m is considered appropriate. In view of the granular nature of the material and the associated groundwater conditions, it is recommended that the soldier piles be extended and seated on top of the silty clay deposit at approximately el 88. Timber lagging with a thickness of 75 to 100 mm should provide the necessary support between the piles.

(c) Pressure Distribution

(i) Passive Pressure

For computation of passive pressure P_p ignore the effect of wall friction and also the soil resistance to a depth equal to the diameter of the base of the soldier pile "b" below the bottom of excavation. Compute the total resisting force of the soldier piles by assuming the pile to have an effective width of "3b".

(ii) At Rest Pressure

For computation of pressure P_A , consider the wall above the base similar to a braced excavation and the portion below the excavation equivalent to an anchored sheet pile wall. Compute the additional surcharge due to train loading using principles of elastic theory or by assuming an equivalent surcharge multiplied by the applicable earth pressure coefficient as shown in Figure 11. Since tiebacks are prestressed to more than 100% of design load compute pressure based on at rest conditions using an angle of internal friction of 36° .

(d) Soil Anchors

Indications are that three rows of anchors will provide the necessary anchorage to the wall system. These soil anchors rely on friction between the drilled borehole wall and anchor grout. They are installed in a pre-drilled hole at an angle below the horizontal. This angle can be dictated by the depth of soil required above the top anchor zone to provide the necessary resistance. In the computations performed in this

study, an anchor inclination of 25° from the horizontal was required. The limit of the bond length zone as shown in Figure 11 should be at least 0.2 times the depth of the cut away from the active wedge. The design capacity of each of the anchors is anticipated to be in the order of 35 tonnes for a 6-m long bond length. This includes a factor of safety of about 2. For the anchor, particularly the top one, to develop the required ultimate capacity, T_{ult} it should be checked with respect to the overburden cover using the following relationship.

$$T_{ult} = 1.5 (L_o \tan \phi) \pi d P_o$$

where

L_o = bond length

ϕ = angle of internal friction of soil

d = diameter of grout

P_o = effective pressure at midpoint of L_o

Considering the soil at the anchor zone as fine to medium dense sand, a soil anchor with a bond length of 6 m is estimated to provide an ultimate capacity of about 70 tonnes. Nevertheless, if data are not available to confirm such a capacity under these soil conditions, it is recommended that a testing program be undertaken to determine the actual anchor capacity. It would be desirable to carry out these tests prior to construction but if this cannot be scheduled, they should be performed during the initial stages of construction so the wall details can be modified if the test results so indicate.

Anchors should be placed not closer than 6 times the diameter of the anchor grout section to minimize interaction. A stressing length of 9 m appears to be reasonable for the maximum wall section.

The typical arrangement shown in Figure 11 gives an indication of general requirements for the wall but it is not a final design.

4.6 - Structure Foundations

4.6.1 - Concrete Box Structures

The foundation level for the concrete box structures is between el 93 and 94 according to the preliminary structural drawings. At that elevation range, the soils are a combination of the glacial till and the interbedded silty and gravelly sands, both of which exist in a very dense state and are capable of supporting relatively high foundation pressures.

There is, however, one possible problem with these soils related to their potential for frost heave. Typical grain size distribution curves for the glacial till and sand deposits are shown in Figures 1, 2 and 3. The gradations of the till samples and the siltier portions of the sand deposit indicate materials which are capable of supporting frost heave. At the site of the concrete box structures, the other two components required for frost heave, that is, a nearby water source and frozen surface, will also be present.

To avoid any problems which might result from frost heave, it is recommended that the natural soils within the zone of frost penetration below the structures be replaced by a nonfrost-susceptible material. The depth of replacement below the concrete surface should be equivalent to 1.2 m of soil. Since the thermal conductivity of concrete is in the same range as the glacial till, a depth of replacement of 1.5 m below the concrete surface is recommended.

With regard to the backfill material, it is recommended that material conforming to the gradation of MTC Granular A be used. This fill should be placed in layers not exceeding 150 mm in thickness and compacted by a vibratory compactor to a dry density equal to not less than 100% of the maximum dry density as determined by ASTM Test Designation D698.

Considering the bearing capacity design of the structures, it is recommended that a factored bearing capacity of 800 kPa at ultimate limit state be used. The capacity at serviceability limit state should be assumed as 350 kPa. If the base slab is designed on the basis of the Modulus of Subgrade Reaction, a value of 30 000 kN/m³ is recommended.

The structure backfill should consist of granular material in accordance with MTC Standard Special Provision 121, dated October 1983. If Granular A is used, the following properties may be used for design

- unit weight - 22 kN/m³
- angle of internal friction - 35°

If Granular B backfill is to be used, a fairly wide range of values exist for the above parameters. Unless the exact source of the material is known, the computation of the earth pressures should be carried out in accordance with Section 6.6.1.2.2 of the OHBDC.

Since the box structures are assumed to have rigid, nonyielding walls, they should be designed to resist at-rest pressure conditions.

4.6.2 - Retaining Walls

The preliminary structural plan shows a series of retaining walls in the approach cuts and also between

the two box structures. Base slabs for these walls should be placed a minimum of 1.5 m below the finished ground surface to protect against frost action. Alternatively, the bases could be founded on a 0.3-m thick zone of Granular A material, the surface of which is 1.2 m below grade.

The bearing capacity parameters outlined for the concrete box structures are also applicable to the retaining wall design.

Lateral earth pressures acting on the retaining walls will depend on whether the walls are rigid or flexible and free to rotate. If the walls are tied into the concrete box structures, they should be designed for the at-rest pressure condition. Should they be separated from the box structures and free to rotate or deflect at the top, a distance equal to 0.2% of the wall height, they can be designed for the active pressure condition.

For details regarding the backfill of the retaining walls, refer to the backfill recommendations in Section 4.6.1.

With regard to sliding stability of the walls, it is recommended that a sliding friction angle of 24 deg be assumed. This angle represents the ultimate sliding resistance and must, therefore, be factored to provide an adequate factor of safety.

The overall drainage works being provided through the cut should have provisions to drain any water which may seep into the area behind the retaining walls.

4.6.3 - Corrosion Considerations

To determine whether the subsurface environment at the structure locations is detrimental to concrete and steel, a series of tests was undertaken to determine the pH and sulfate content of the soils. Table 4 tabulates the results of these tests.

TABLE 4

pH and Sulfate Test Results

<u>Borehole</u>	<u>Depth (m)</u>	<u>pH</u>	<u>SO₄ (ppm)</u>
204	10.8	8.1	118
204	12.3	8.0	111
205	10.8	8.1	105
205	12.3	8.2	183
205	13.9	8.1	99
205	15.4	7.8	69
206	12.3	8.0	118

The results indicate that the soil conditions are not detrimental to steel and concrete made with normal Portland cement (Type 10).

4.7 - Railway Embankment

It has been recommended that drains be installed along the toes of the slopes to depress the groundwater level below the base of the cut. In addition to these drains, there will, no doubt, be open ditches near the toes to collect and convey surface water out of the cut. Throughout a significant portion of the line investigated in this study, primarily in the western half, the material in the base of the excavation is the glacial till deposit. As mentioned in a previous section, this material has a gradation which has a potential for frost heaving. To avoid such problems, it will be necessary to keep the railroad embankment in a well drained condition. This can be accomplished by constructing the embankment from free draining material which will not support the capillary rise of moisture and also by providing and maintaining adequate ditches.

With regard to construction of the spur line detours, most of the alignments will be in areas of new cut. The subgrade materials will be glacial till or sands. Whereas under the present conditions the ground may be somewhat wet and boggy at certain times of the year, the groundwater lowering for the GO-ALRT works will tend to dry the area and improve foundation conditions. No significant foundation problems are anticipated in the construction of the temporary detour lines.



May 6, 1985

ACRES INTERNATIONAL LIMITED

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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 / 0.3 m)	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_i	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

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

RECORD OF BOREHOLES

RECORD OF BOREHOLE No 201

METRIC

W P EGG-001-3 LOCATION Co-ords. 4,860,039.1 N; 353,975.6 E. ORIGINATED BY CSB
 DIST 6 HWY GO-ALRT BOREHOLE TYPE Hollow and Solid Stem Augers COMPILED BY CSB/PV
 DATUM Geodetic DATE February 19 and 20, 1985 CHECKED BY JSB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100							PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L
								SHEAR STRENGTH									
106.2	Ground Surface																
0.0	Topsoil						April 17/85										
105.7																	
0.5	Sandy clayey silt trace of gravel.		1	SS	26												
	CL-ML (Glacial Till)		2	TW	PH												
	Light brown to yellow		3	SS	31												8 40 (52)
101.7	Compact																
4.5	Sandy silty clay, trace of gravel.		4	TW	PH												
	CL (Glacial Till)		5	SS	6												3 30 (67)
98.7	Firm Gray																
7.5	Sandy clayey silt to silty clayey sand, trace of gravel.		6	SS	22												9 45 (46)
	CL-ML-SC (Glacial Till)		7	SS	48												
			8	SS	90/240 mm												
			9	SS	60/115 mm												
			10	CS	60/35 mm												
			11	SS	106/150 mm												
91.0	Compact to very dense Gray		12	SS	106/150 mm												
15.2	Silty clay with silt partings.		13	CS													
	CI		14	SS	113												
					92/255 mm												
87.6	Hard Gray		15	CS	255 mm												
18.6	End of borehole.																
Water Level Records																	
Date Elevation (1985) of W.L. (m)																	
March 30 104.7																	
April 17 105.5																	

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



Ministry of
Transportation and
Communications
Ontario

RECORD OF BOREHOLE No 202

METRIC

W P EGG-001-3 LOCATION Co-ords. 4,860,074.1 N; 354,034.0 E. ORIGINATED BY CSB
DIST 6 HWY GO-ALRT BOREHOLE TYPE Solid Stem Auger COMPILED BY CSB/PV
DATUM Geodetic DATE February 20 and 21, 1985 CHECKED BY SVB

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					
106.7	Ground Surface																
0.0	Topsoil																
0.5	Sandy clayey silt trace of gravel. CL-ML (Glacial Till)		1	SS	21		105										
103.4	Compact Light Brown		2	TW	PH												
3.3	Sandy silty clay, trace of gravel. CL (Glacial Till)		3	SS	21												
102.2	Very Stiff Gray		4	TW	PH												
101.3	Sandy clayey silt to silty clayey sand, trace of gravel. CL-ML-SC (Glacial Till)		5	TW	PH												
5.4	Dense Gray		6	SS	*												
99.7	Silty fine sand, trace of gravel. SM		7	SS	115												
96.9	Loose to Compact Gray		8	SS	*												
7.5	Sandy clayey silt to silty clayey sand. CL-ML-SC (Glacial Till)		9	SS	68												
97.6	Very Dense Gray		10	SS	75/												
9.1	Sand, fine to coarse with some gravel and trace of silt. SW		11	SS	103/												
96.1	Loose to Very Dense Gray		12	SS	60/												
10.6	Sandy clayey silt to silty clayey sand, trace of gravel. CL-ML-SC (Glacial Till)		13	SS	60/												
95.1	Very Dense Gray		14	SS	149												
11.6	Silty fine to medium sand, trace of gravel. SC		15	SS	129												
92.9	Very Dense Gray		16	SS	68												
13.8	Sandy clayey silt, trace of gravel. CL-ML (Glacial Till)																
	Hard Brownish Gray																
	Silty clay, trace of gravel, occasional silt partings. CL																
88.0	Hard Gray																
18.7	End of borehole.																
	*Rods sank under own weight.																
	Piezometer installed by drilling a separate hole about 2.5 m from BH 202.																
	<u>Water Level Records</u>																
	Date (1985)	Elevation of W.L. (m)															
	March 15	103.7															
	March 30	103.7															
	April 17	103.8															

+3, x5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 203

METRIC

W P EGG-001-3 LOCATION Co-ords. 4,860,099.9 N; 354,078.8 E. ORIGINATED BY CSB
 DIST 6 HWY GO-ALRT BOREHOLE TYPE Hollow Stem Auger. COMPILED BY CSB/PV
 DATUM Geodetic DATE February 21, 22 and 25, 1985 CHECKED BY CSB

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	WATER CONTENT (%)				
107.8	Ground Surface																					
0.0	Topsoil																					
107.3	Sandy clayey silt, trace of gravel.																					
0.5	CL-ML (Glacial Till)		1	SS	37																	
104.4	Dense Light Brown to Yellow		2	TW	PH																	
3.4	Sandy clayey silt to silty clayey sand.		3	SS	99																	
	ML-CL-SC (Glacial Till)		4	SS	99																	
			5	SS	113																	
100.3	Very Dense Gray		6	SS	110																	
7.5	Silty fine sand, trace of gravel. SM		7	SS	48																	
97.6	Very Dense Gray		8	SS	116																	
10.2	Gravelly sand, fine to coarse, some silt.		9	SS	101																	
96.5	SW		10	SS	160/100mm																	
11.3	Very Dense Gray		11	SS	160/100mm																	
	Sandy clayey silt, trace of gravel		12	SS	140/150mm																	
	ML-CL (Glacial Till)																					
90.9	Very Dense Gray		13	SS	83																	
16.9	Silty clay with partings of silt.		14	SS	41																	
	CI																					
87.5	Hard Gray		15	SS	52																	
20.3	End of borehole.																					
<p>Water Level Records</p> <table border="1"> <thead> <tr> <th>Date (1985)</th> <th>Elevation of W.L. (m)</th> </tr> </thead> <tbody> <tr> <td>March 15</td> <td>105.9</td> </tr> <tr> <td>March 30</td> <td>103.5</td> </tr> <tr> <td>April 17</td> <td>103.0</td> </tr> </tbody> </table>															Date (1985)	Elevation of W.L. (m)	March 15	105.9	March 30	103.5	April 17	103.0
Date (1985)	Elevation of W.L. (m)																					
March 15	105.9																					
March 30	103.5																					
April 17	103.0																					

RECORD OF BOREHOLE No 204

METRIC

W P EGG-001-3 LOCATION Co-ords. 4,860,114.4 N; 354,089.9 E. ORIGINATED BY CSB
 DIST 6 HWY GO-ALRT BOREHOLE TYPE Hollow Stem Auger. COMPILED BY CSB/PV
 DATUM Geodetic DATE March 7 and 8, 1985 CHECKED BY JHS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100								
								SHEAR STRENGTH										WATER CONTENT (%)		
						○ UNCONFINED + FIELD VANE					○									
						● QUICK TRIAXIAL x LAB VANE														
103.6	Ground Surface																GR SA SI CL			
0.3	Topsoil						April 17/85													
	Sandy clayey silt to silty clay sand with some gravel. CL-ML-SC (Glacial Till)		1	SS	93								○				17 40 (43)			
			2	SS	60/115mm															
98.9	Very Dense Gray																			
4.7	Silty fine sand. SM Compact to Brownish		3	SS	21									○						
97.6	Very Dense Gray		4	SS	70								○							
6.0	Silty clayey sand, fine to medium, trace of gravel.		5	SS	32												11 63 (26)			
96.0	SC (Glacial Till)		6	SS	120/85mm															
7.6	Dense Brownish Gray																			
	Sandy clayey silt, trace of gravel. CL-ML (Glacial Till)		7	SS	120/100mm								○							
			8	CS	60/115mm												4 37 (59)			
			9	SS	60/100mm															
90.0	Very Dense Gray																			
13.6	Silty clay. CH		10	SS	55															
			11	SS	37															
86.2	Hard Gray		12	SS	53															
17.4	End of borehole.																			
<u>Water Level Records</u>																				
Date (1985)		Elevation of W.L. (m)																		
March 15		103.2																		
March 30		103.4																		
April 17		103.4																		

+3, x5: Numbers refer to Sensitivity

20
15 → 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 205

METRIC

W P EGG-001-3 LOCATION Co-ords. 4,860,112.5 N; 354,109.9 E. ORIGINATED BY CSB
 DIST 6 HWY GO-ALRT BOREHOLE TYPE Hollow Stem Auger COMPILED BY CSB/PV
 DATUM Geodetic DATE March 8 and 11, 1985 CHECKED BY JLB

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					
103.6	Ground Surface													
0.3	Topsoil													
	Sand													
	0.5 to 2.3: Interbedded fine sand with some silt and sandy silt. SM		1	SS	100									
	2.3 to 4.0: Silty fine to coarse sand and some gravel. SM		2	SS	57									
	4.0 to 6.1: Fine to medium sand with trace of gravel and silt. SW		3	SS	41									
	6.1 to 8.5: Inter- bedded silty fine to coarse sand and gravelly sand. SM-SP		4	SS	78									
			5	SS	60/65mm									
			6	SS	107/240mm									
94.2			7	SS	60/100mm									
9.4	8.5 to 9.4: Fine to medium sand with some gravel and trace of silt. SP		8	SS	102/150mm									
	Dense to Light Brown Very Dense to Gray		9	SS	120/145mm									
	Sandy clayey silt, trace of gravel. CL-ML (Glacial Till)		10	SS	71/150mm									
88.2	Very Dense Gray													
15.4	Silty clay with silt partings. CI-CH		11	SS	68									
			12	SS	31									
			13	SS	38									
84.1	Hard Gray													
19.5	Sandy clayey silt, trace of gravel. CL-ML (Glacial Till)		14	SS	60/75mm									
81.9	Very Dense Gray		15	SS	60/100mm									
21.7	End of borehole.													
Water Level Records														
Date (1985)		Elevation to W.L. (m)												
March 15		100.2												
March 30		100.0												
April 17		100.0												

*3, *5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10



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RECORD OF BOREHOLE No 206

METRIC

W P EGG-001-3 LOCATION Co-ords. 4,860,134.0 N; 354,119.4 E. ORIGINATED BY CSB
DIST 6 HWY GO-ALRT BOREHOLE TYPE Hollow Stem Auger. COMPILED BY CSB/PV
DATUM Geodetic DATE March 12 and 13, 1985 CHECKED BY MB

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			20	40	60	80	100					
103.5	Ground Surface															
0.3	Topsoil					April 17/85										
	Sand															
	0.6 to 4.0: Fine sand trace to some silt. SP		1	SS	60/125											
	4.0 to 5.5: Fine to medium sand, trace of gravel and silt. SW		2	SS	103	100										
	5.5 to 6.2: Gravelly sand. SP		3	SS	24											
	6.2 to 10.4: Fine to medium sand, trace of gravel and silt. SW		4	SS	111/265											
			5	SS	23											
			6	SS	60/120											
			7	SS	60/125	95										
			8	SS	120/125											
93.1	Compact to Very Dense Gray to Brown		9	SS	120/125											
10.4	Sandy clayey silt, trace of gravel. CL-ML (Glacial Till)		10	SS	120/175	90										
			11	SS	120/100											
88.4	Very Dense Gray															
15.1	Silty clay with silt partings. CI-CH		12	SS	82	85										
			13	SS	37											
			14	SS	45											
83.8	Hard Gray															
19.7	Sandy clayey silt, trace of gravel. CL-ML (Glacial Till)		15	SS	103/150											
			16	SS	107/150											
80.3	Very Dense Gray		17	SS	120/175											
23.2	End of borehole.															
<u>Water Level Records</u>																
Date		Elevation of W.L. (m)														
(1985)																
March 15		103.7		Above G-L												
March 30		103.5														
April 17		103.4														

*3, x⁵: Numbers refer to
Sensitivity

20
15 → 5 (%) STRAIN AT FAILURE
10



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RECORD OF BOREHOLE No 208

METRIC

W P EGG-001-3 LOCATION Co-ords. 4,860,143.6 N; 354,150.4 E. ORIGINATED BY CSB
DIST 6 HWY GO-ALRT BOREHOLE TYPE Hollow Stem Auger. COMPILED BY CSB/PV
DATUM Geodetic DATE March 1, 5, 6 and 7, 1985 CHECKED BY CSB

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					
108.7	Ground Surface																
108.2	Topsoil																
0.5	Sandy clayey silt to silty clayey sand, trace of gravel. CL-ML-SC (Glacial Till)		1	SS	121											4 49 (47)	
			2	SS	60												
104.5	Very Dense Light Brown Sand																
4.2	4.2 to 7.8: Silty fine sand. SM		3	SS	60											0 68 (32)	
	7.8 to 15.9: Fine to medium sand with some gravel and trace of silt. SW		4	SS	78											16 78 (6)	
			5	SS	102											19 74 (8)	
			6	SS	101												
			7	SS	30												
			8	SS	60												
92.8	Dense to Very Dense Brown to Gray		9	SS	60												
15.9	Sandy clayey silt, trace of gravel. CL-ML (Glacial Till)		10	SS	160												
			11	SS	102												
88.6	Very Dense Gray		12	SS	105												
20.1	End of borehole.																
<u>Water Level Records</u>																	
Date (1985)		Elevation of W.L. (m)															
March 15		104.5															
March 30		104.0															
April 17		103.8															

+3, x5: Numbers refer to Sensitivity
20
15
10
5 (%) STRAIN AT FAILURE



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RECORD OF BOREHOLE No 209

METRIC

W P EGG-001-3 LOCATION Co-ords. 4,860,179.9 N: 354,211.8 E. ORIGINATED BY CSB
DIST 6 HWY GO-ALRT BOREHOLE TYPE Hollow Stem Auger. COMPILED BY CSB/PV
DATUM Geodetic DATE February 27 and 28, 1985 CHECKED BY YAB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100		
106.5	Ground Surface													
0.0 106.0	Topsoil													
0.5	Sandy clayey silt, trace of gravel. CL-ML (Glacial Till)		1	SS	91		105							
			2	SS	60/100									
101.8	Very Dense Light Brown		3	SS	95/150									
4.7	Sandy clayey silt, trace of gravel. CL-ML (Glacial Till)		4	SS	85/150		100							
98.7	Very Dense Gray		5	SS	65/150									
7.8	Sand, fine to medium, trace of silt. SW		6	SS	123/75									
95.7	Very Dense Gray		7	SS	100/115									
10.8	Sandy silt, trace of gravel. ML (Glacial Till)		8	SS	160/25		95							
92.6	Very Dense Gray		9	SS	160/50									
13.9	Sand, fine, trace of silt and gravel. SP													
91.1	Very Dense Gray		10	SS	120/125									
15.4	Sandy clayey silt, trace of gravel. CL-ML (Glacial Till)		11	SS	120/115		90							
88.2	Very Dense Gray													
18.3	End of borehole.													

+3, x5: Numbers refer to
Sensitivity

20
15 \diamond 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 210

METRIC

W P EGG-001-3 LOCATION Co-ords. 4,860 214.1 N; 354,273.4 E. ORIGINATED BY CSB
 DIST 6 HWY GO-ALRT BOREHOLE TYPE Hollow Stem Auger. COMPILED BY CSB/PV
 DATUM Geodetic DATE February 26, 1985 CHECKED BY AB

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					
104.2	Ground Surface																
103.7	Topsoil																
0.5	Sand, fine to medium, trace of silt. SW		1	SS	15												
101.0	Compact Light Brown																
3.2	Sandy silty clay. CI (Glacial Till) Light		2	TW	PH												
99.5	Firm to Stiff Brown																
4.7	Sandy silt, trace of gravel. ML (Glacial Till)		3	SS	13												
97.5	Compact to Dense Gray		4	SS	35												
6.7	Sand, fine, some silt and gravel. SP		5	SS	60/ 100mm												
96.0	Very Dense Gray																
8.2	Sandy clayey silt to sandy silty clay with depth, trace of gravel. ML-CL (Glacial Till)		6	SS	90/ 150mm												
			7	SS	60/ 100mm												
			8	SS	85/ 150mm												
			9	SS	60/ 85mm												
			10	SS	72/ 150mm												
87.1	Hard Gray		11	SS	60/ 35mm												
17.1	End of borehole.																

+3, x5: Numbers refer to 20
Sensitivity 15 5 (%) STRAIN AT FAILURE
10



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RECORD OF BOREHOLE No 212

METRIC

W P EGG-001-3 LOCATION Co-ords. 4,860,143.3 N: 354,062.6 E. ORIGINATED BY CSB
DIST 6 HWY GO-ALRT BOREHOLE TYPE Hollow Stem Auger COMPILED BY CSB/PV
DATUM Geodetic DATE March 14, 1985 CHECKED BY JAS

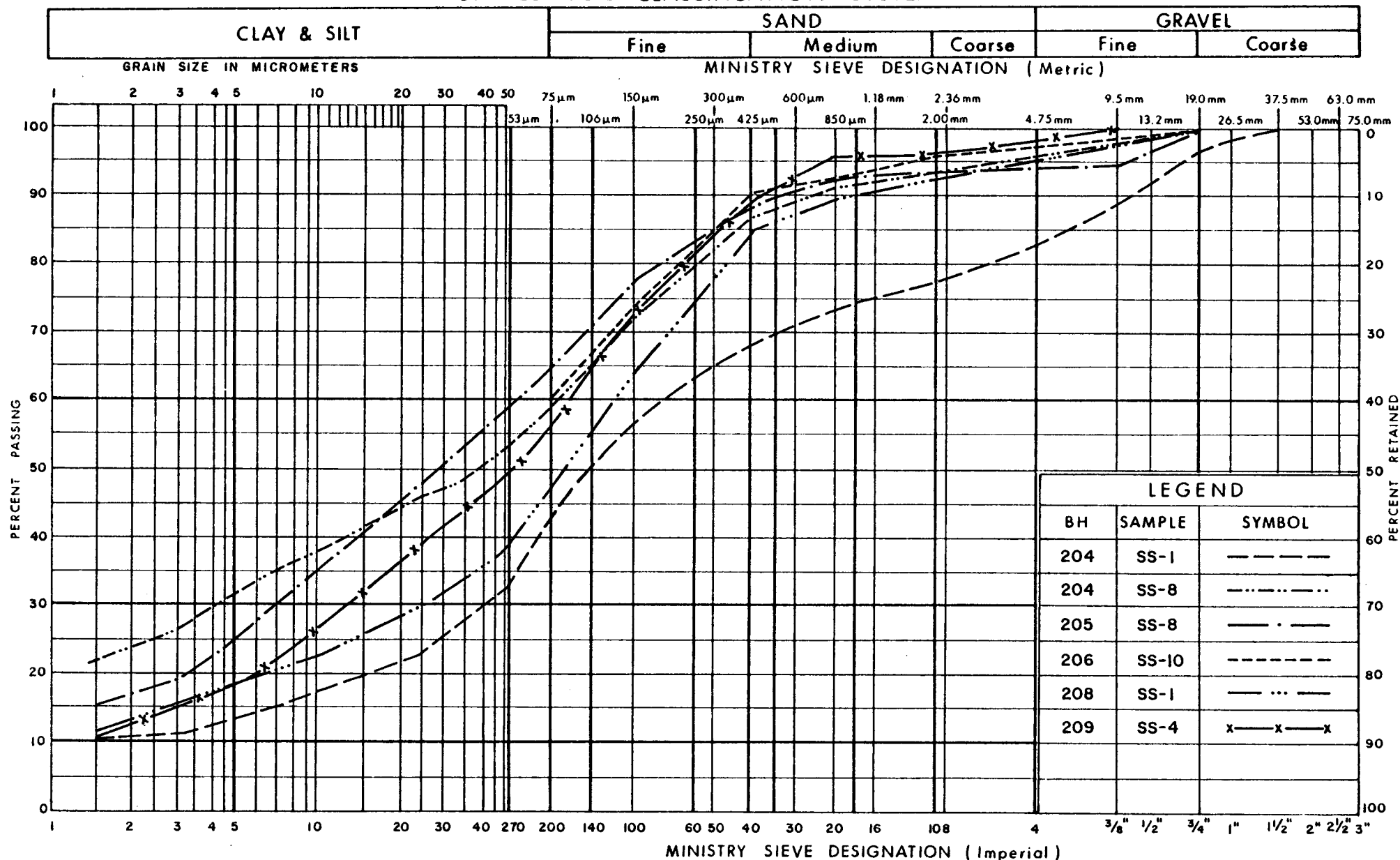
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES								
103.0	Ground Surface												
0.0	Topsoil												
0.3	Sandy clayey silt, trace of gravel. CL-ML (Glacial Till)		1	SS	60								
99.3	Dense to Very Dense Gray		2	SS	45		100						
3.7	End of borehole.												

+3, x5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

FIGURES

UNIFIED SOIL CLASSIFICATION SYSTEM



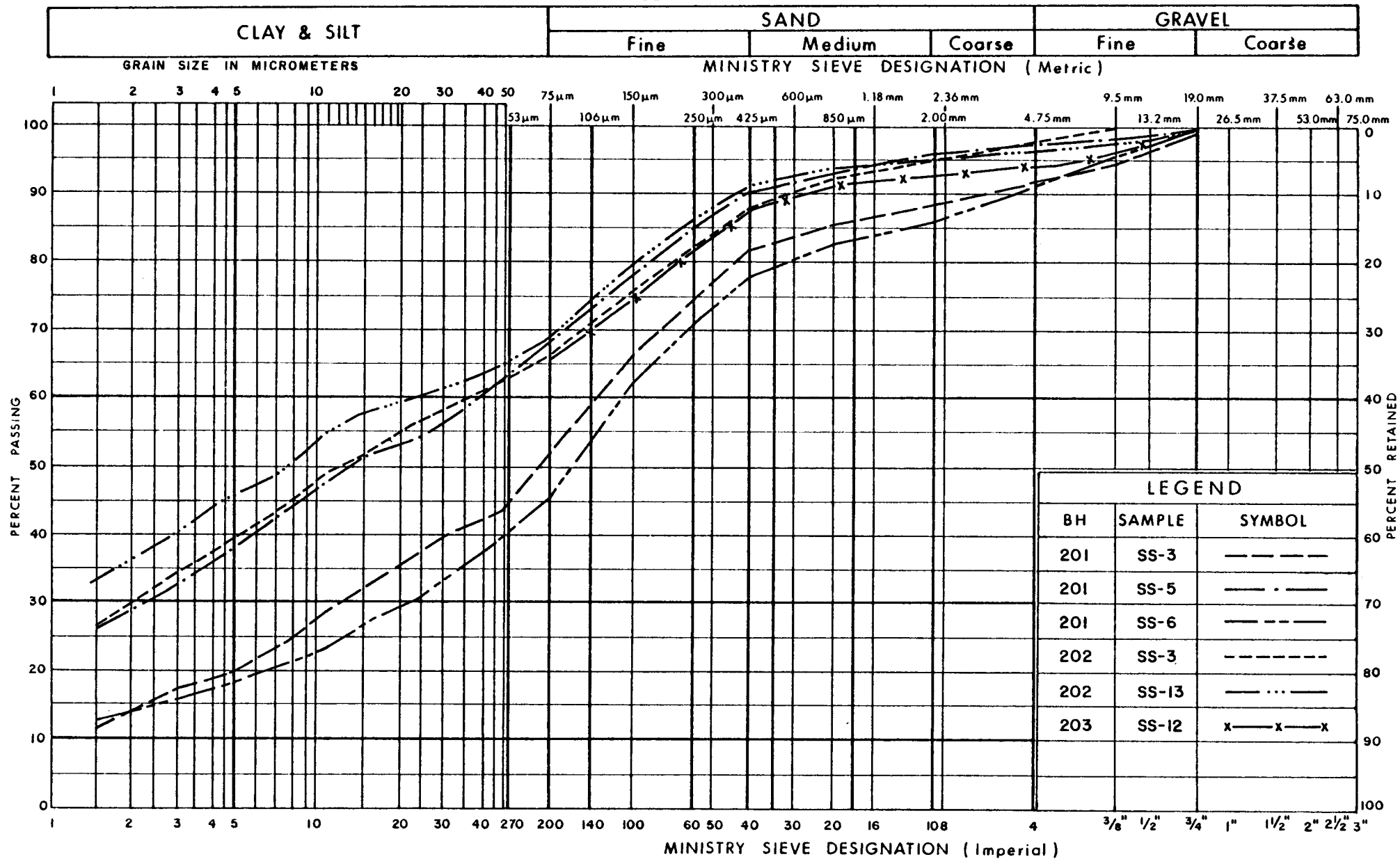
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GRAIN SIZE DISTRIBUTION
GLACIAL TILL - CH. 28+300 TO 28+425
SANDY SILTY CLAY, SANDY CLAYEY SILT, SILTY CLAYEY SAND

FIG No 1

W P EGG-001-3

UNIFIED SOIL CLASSIFICATION SYSTEM



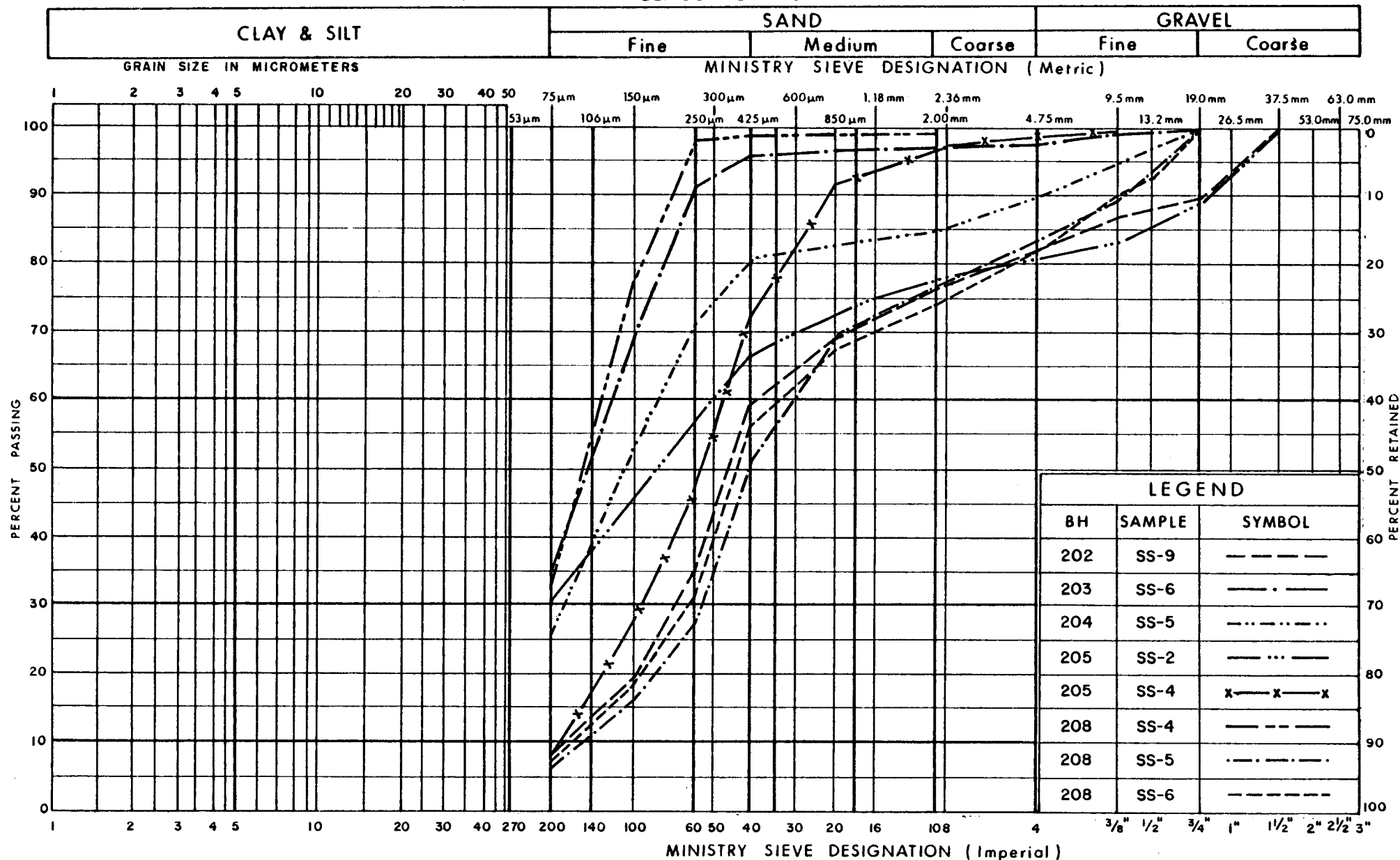
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Communications

GRAIN SIZE DISTRIBUTION
GLACIAL TILL - CH. 28+425 TO 28+650
SANDY SILTY CLAY, SANDY CLAYEY SILT, SILTY CLAYEY SAND

FIG No 2

W P EGG-001-3

UNIFIED SOIL CLASSIFICATION SYSTEM



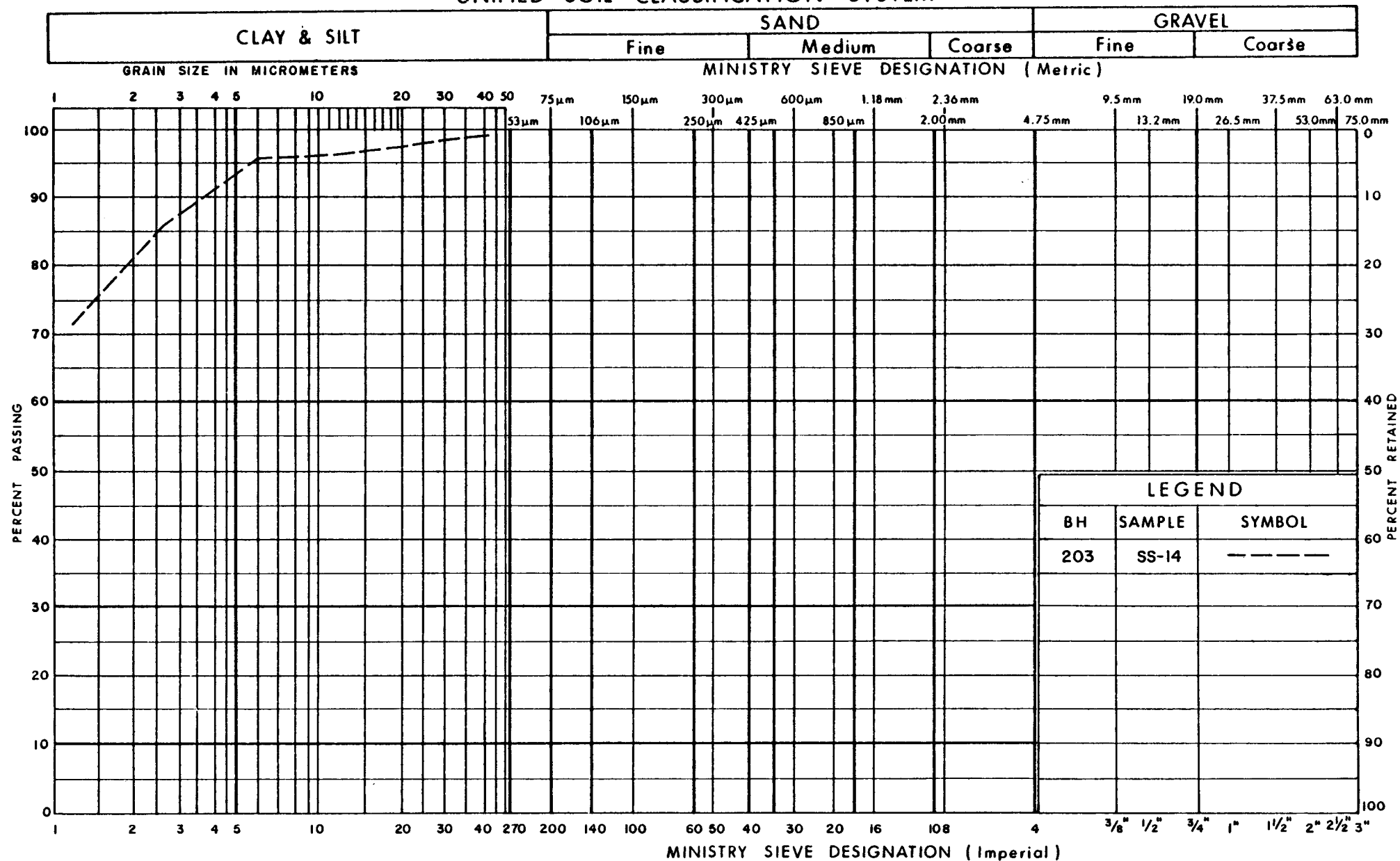
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Communications

GRAIN SIZE DISTRIBUTION
SAND
INTERBEDDED SILTY AND GRAVELLY SANDS

FIG No 3

W P EGG-001-3

UNIFIED SOIL CLASSIFICATION SYSTEM

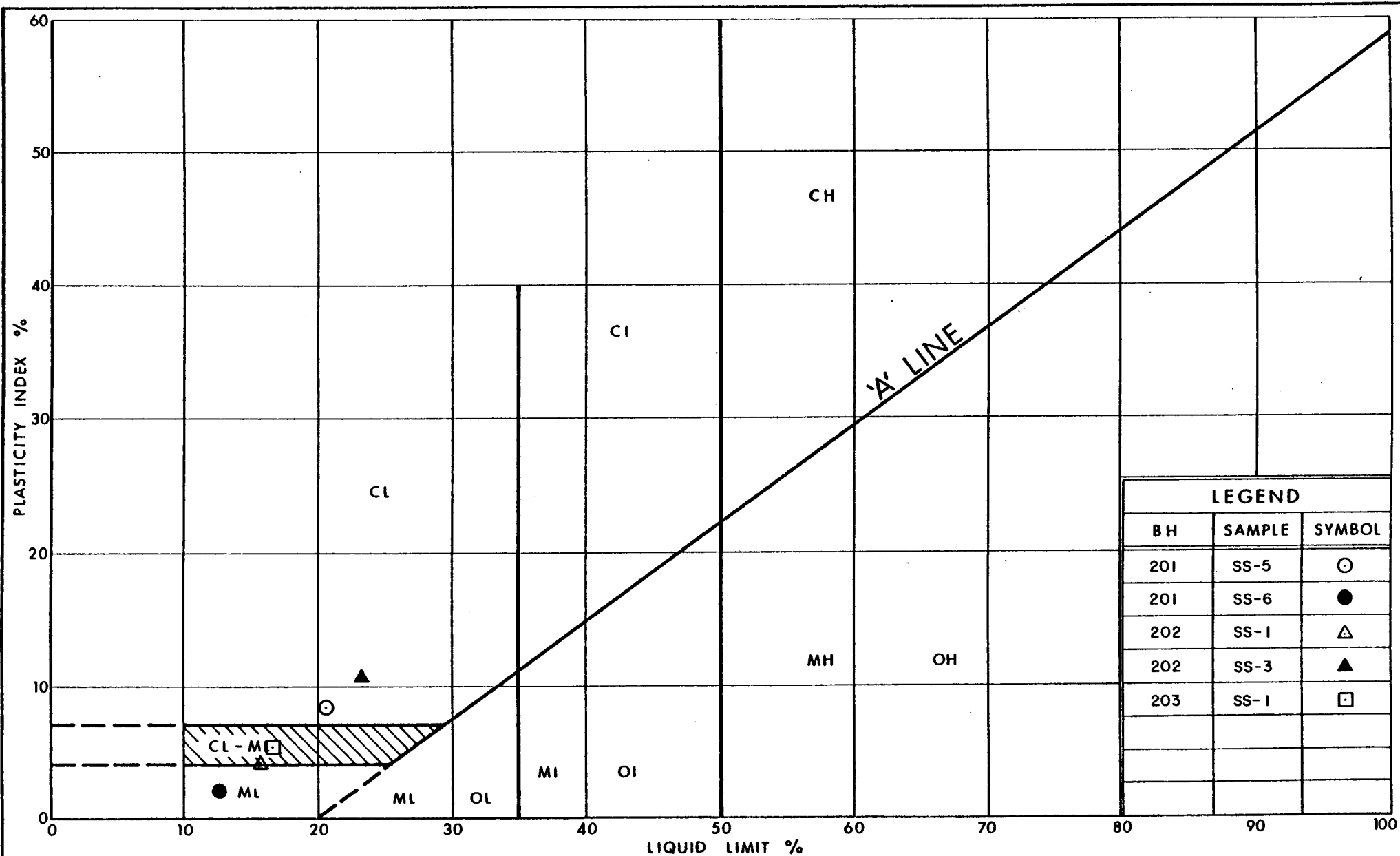


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GRAIN SIZE DISTRIBUTION
SILTY CLAY WITH SILT PARTINGS

FIG No 4

W P EGG-001-3

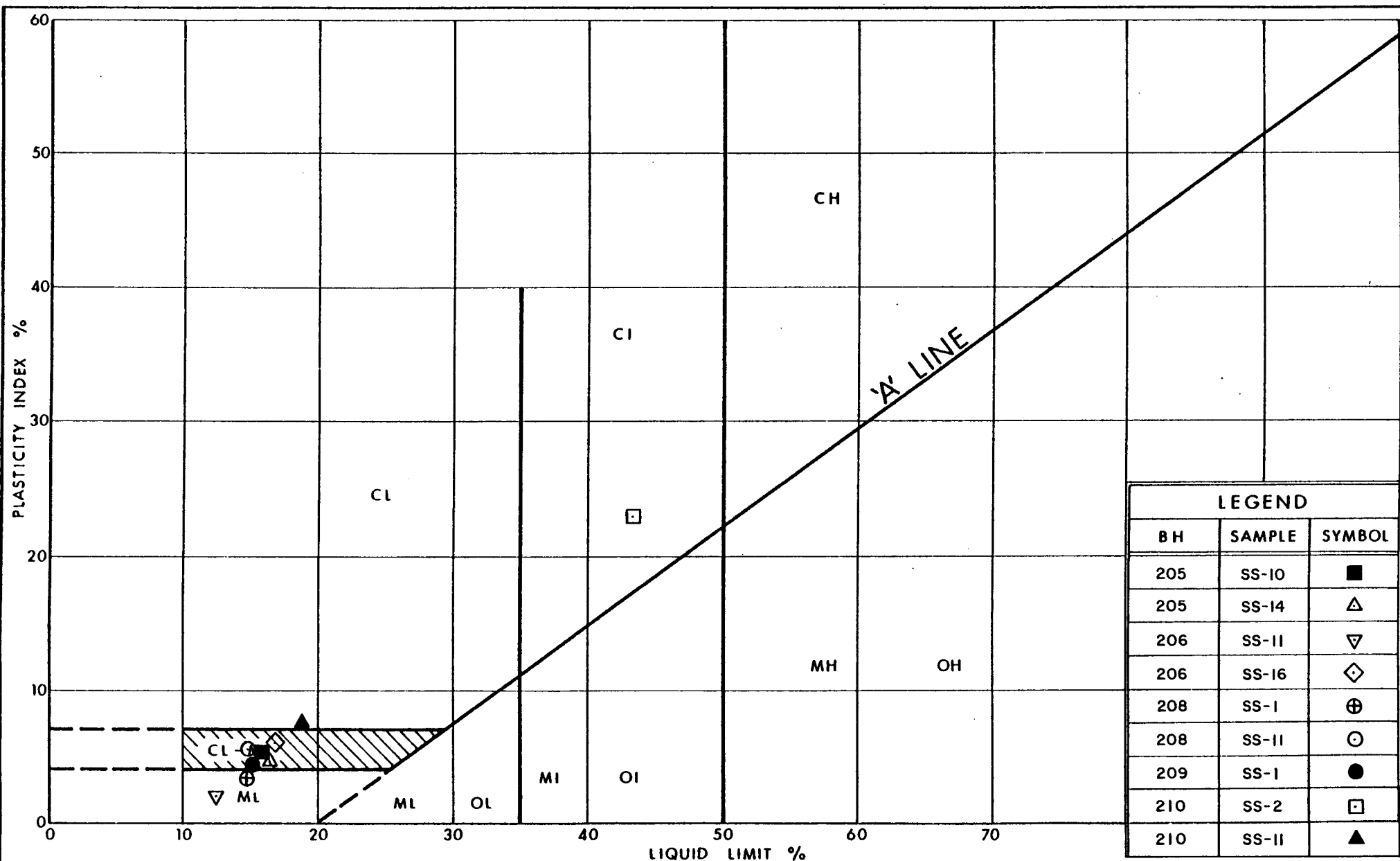


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PLASTICITY CHART
GLACIAL TILL - CH. 28+300 TO 28+425
SANDY SILTY CLAY, SANDY CLAYEY SILT, SILTY CLAYEY SAND

FIG No 5

W P EGG-001-3

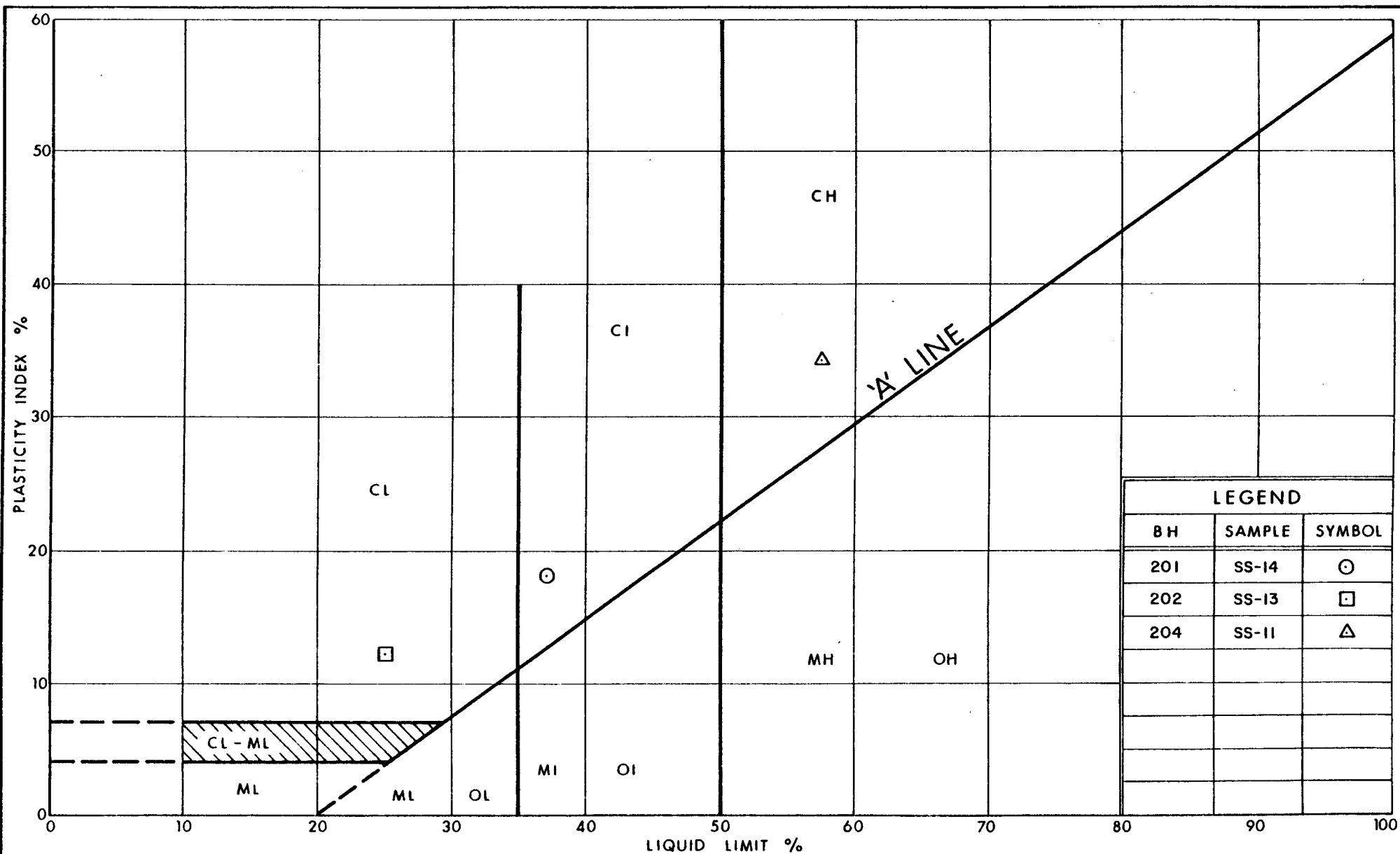


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PLASTICITY CHART
GLACIAL TILL - CH. 28 + 425 TO 28 + 650
SANDY SILTY CLAY, SANDY CLAYEY SILT, SILTY CLAYEY SAND

FIG No 6

W P EGG-001-3

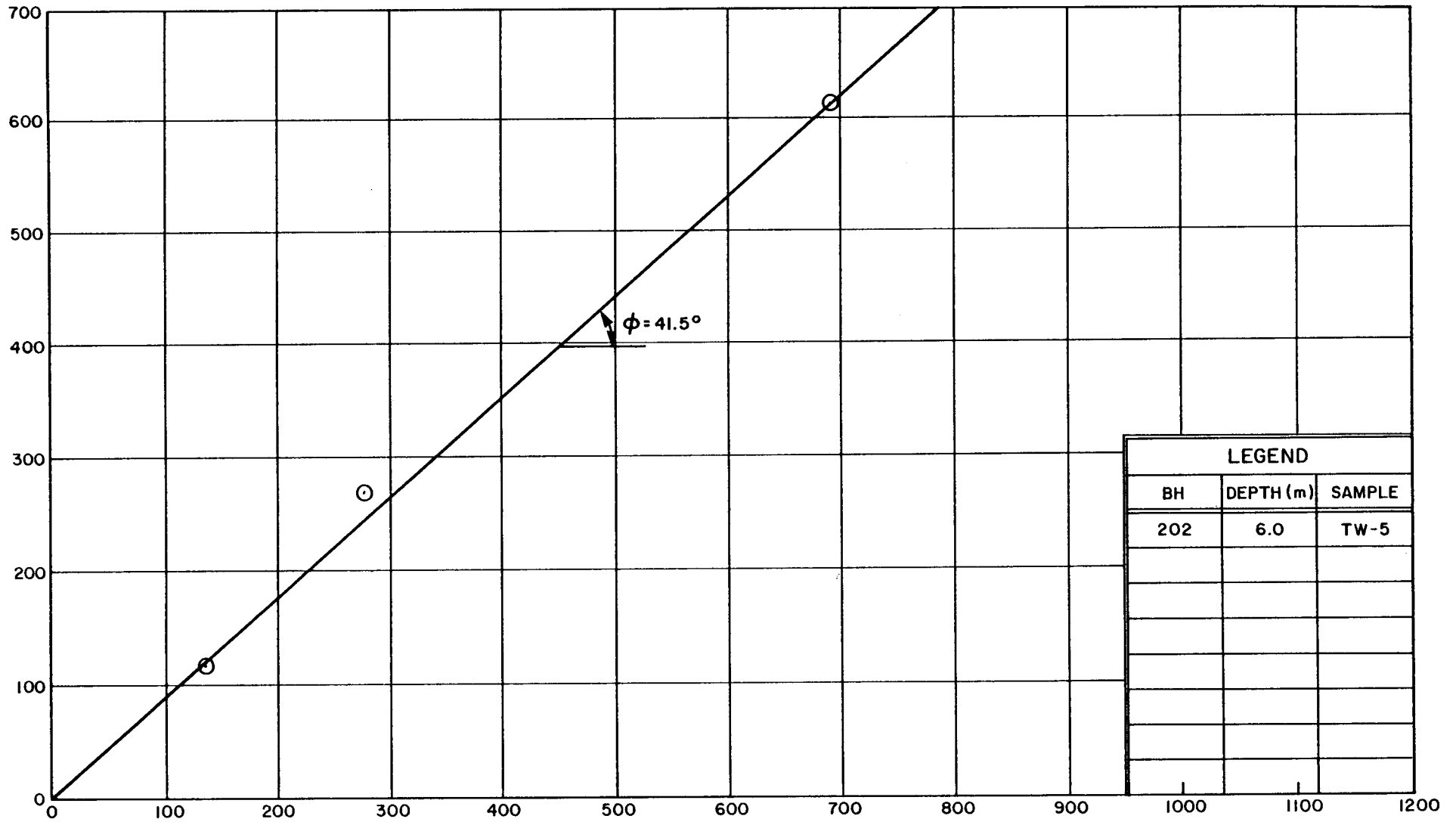


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PLASTICITY CHART SILTY CLAY WITH SILT PARTINGS

FIG No 7

W P EGG-001-3



LEGEND		
BH	DEPTH (m)	SAMPLE
202	6.0	TW-5

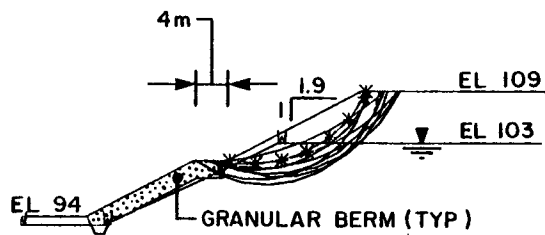


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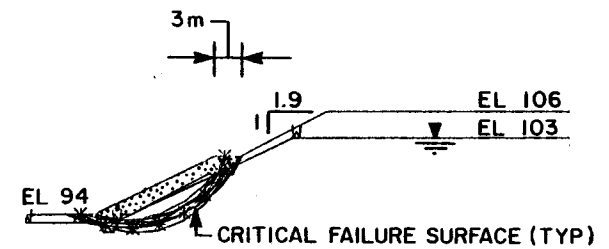
DIRECT SHEAR TEST SILTY FINE SAND

FIG No 8

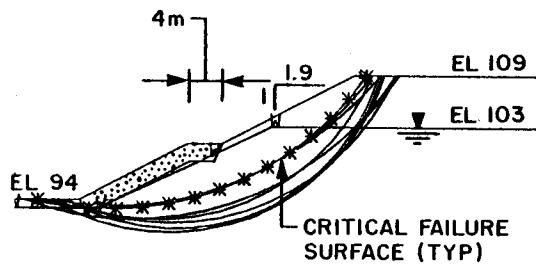
W P EGG-001-3



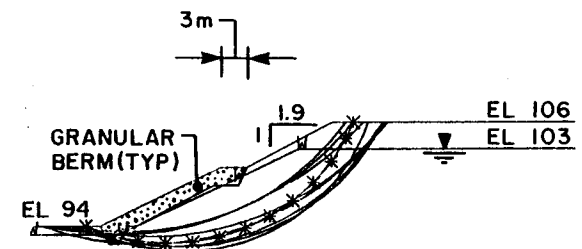
a) SLOPE HEIGHT = 15 m
STABILITY OF UPPER SLOPE
MINIMUM FS = 1.35



c) SLOPE HEIGHT = 12 m
STABILITY OF LOWER SLOPE
MINIMUM FS = 1.42



b) SLOPE HEIGHT = 15 m
STABILITY OF OVERALL SLOPE
MINIMUM FS = 1.27



d) SLOPE HEIGHT = 12 m
STABILITY OF OVERALL SLOPE
MINIMUM FS = 1.25

MATERIAL	FRICTION ANGLE, ϕ	UNIT WT, γ kN/m ³	SUBMERGED UNIT WT, γ' kN/m ³
NATURAL SLOPE MATERIAL	36°	19	10
GRANULAR BERM	32°	18	9

NOTES

- 1 For details of granular berm and drain configurations see Fig 10
- 2 Failure surface shown is the most critical one from 50 surfaces generated

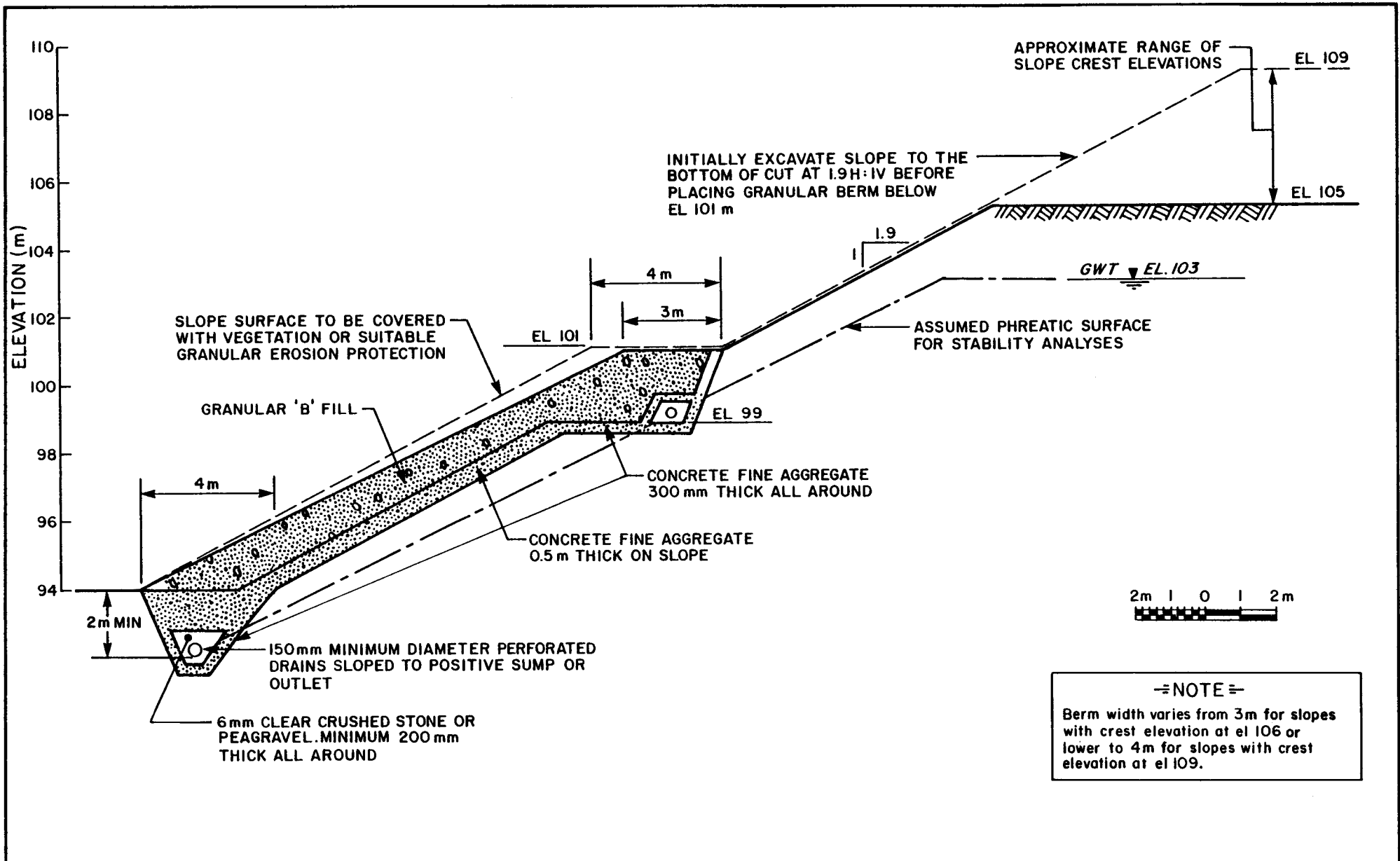


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SLOPE STABILITY ANALYSES

FIG No 9

W P EGG-001-3

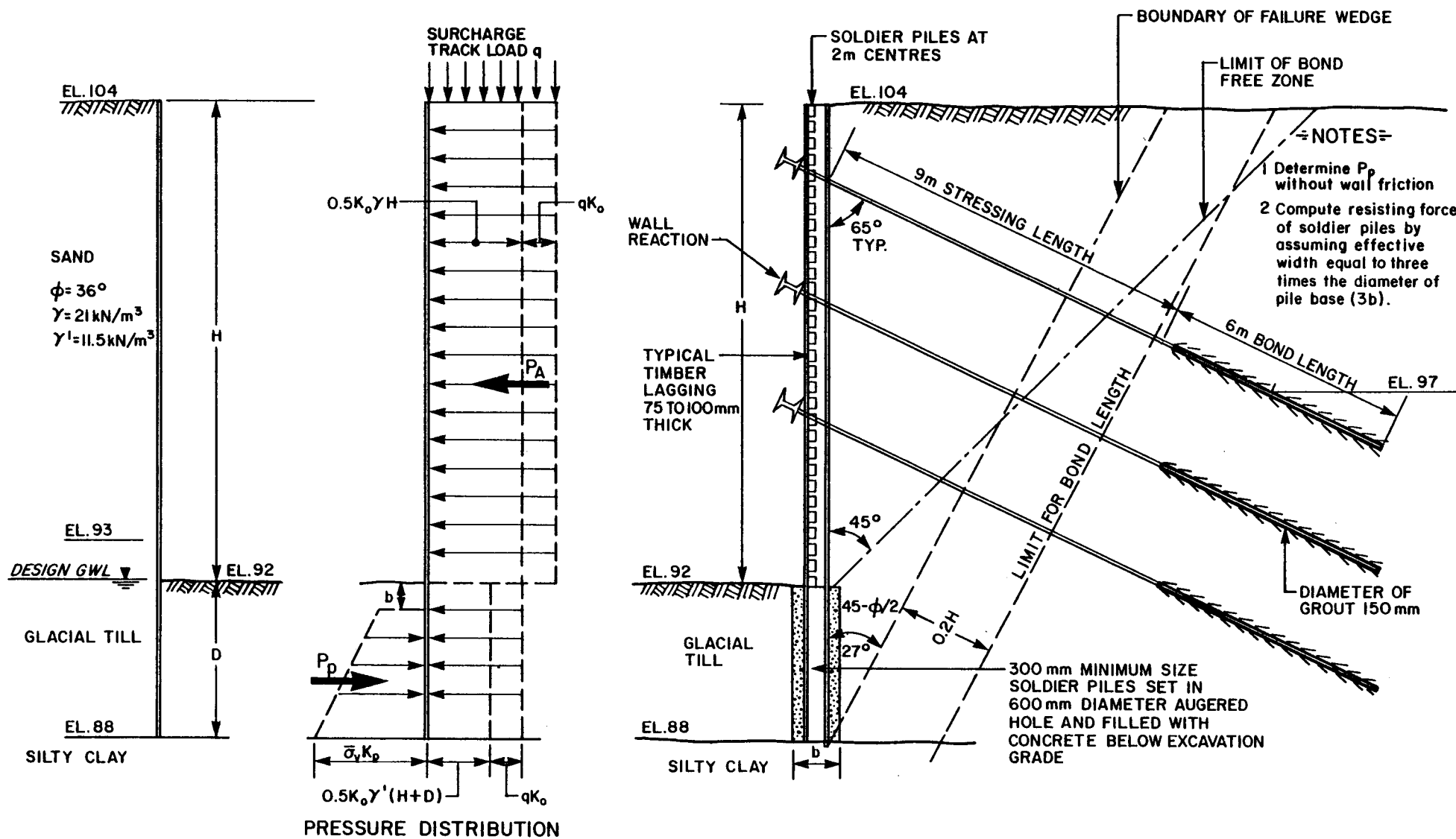


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RECOMMENDED EXCAVATION SLOPE AND DRAIN CONFIGURATIONS

FIG No 10

W P EGG-001-3



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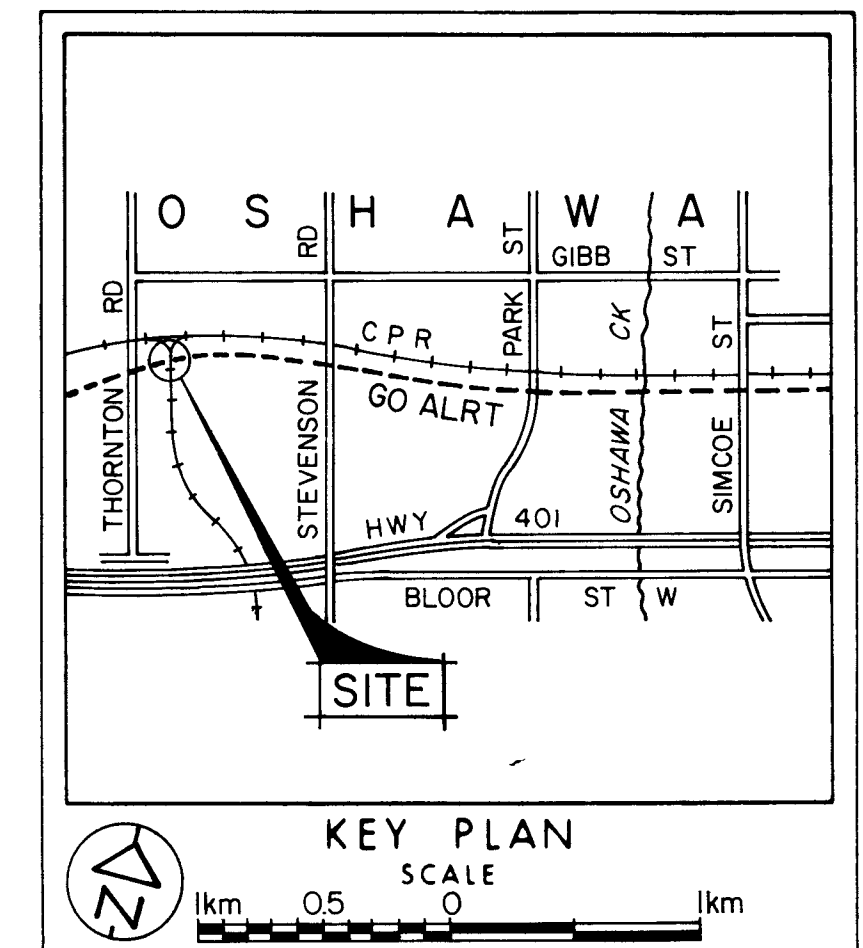
TIEBACK ANCHOR ARRANGEMENT

FIG No 11

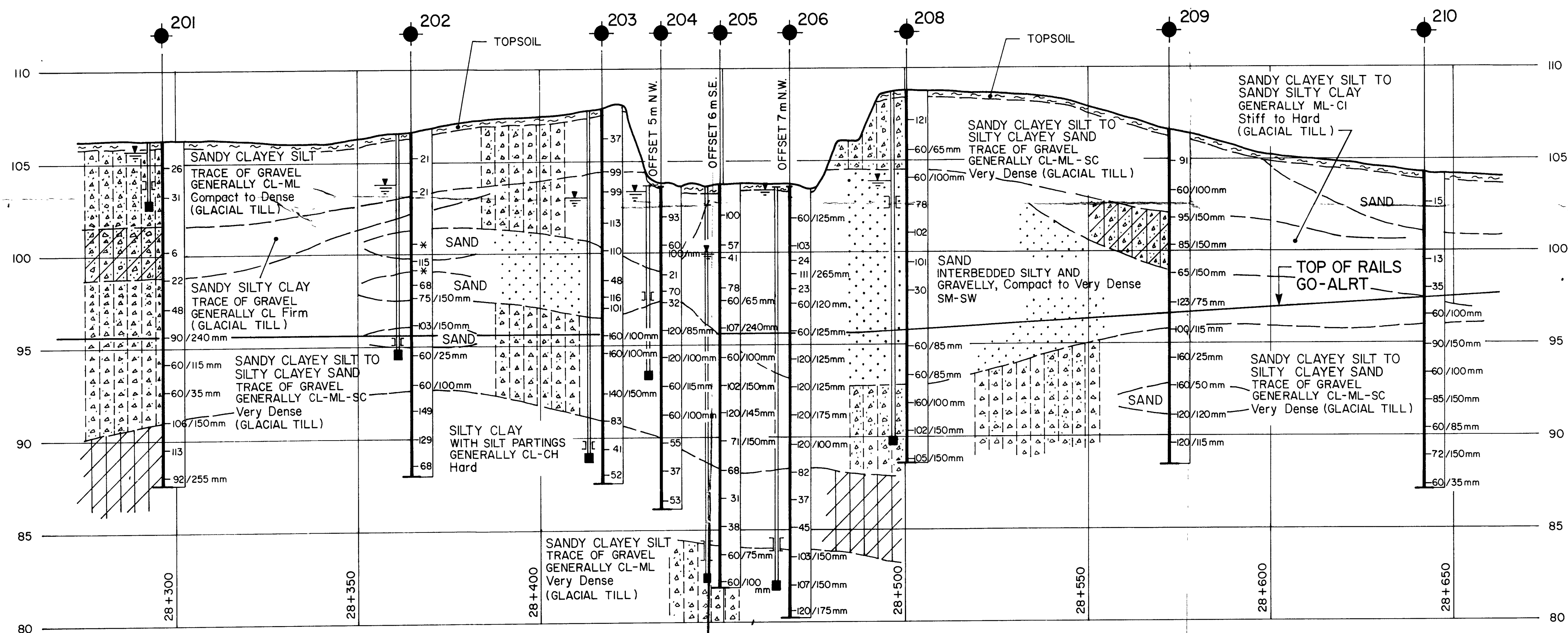
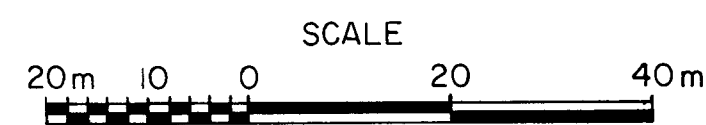
W P EGG-001-3

METRIC

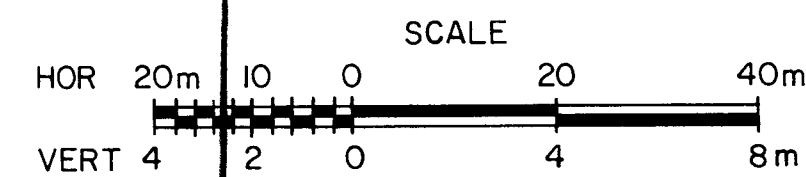
ALL DIMENSIONS SHOWN ARE
IN METRES AND/OR MILLI-
METRES UNLESS OTHERWISE
NOTED.



PLAN



SECTION A-A



LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- W/L at time of investigation 1985.03.8.04
- Not All Piezometers Reached Equilibrium Seal
- Piezometer
- * Drill Rods Sank Under Own Weight

No	ELEVATION	CO-ORDINATES NORTH	EAST
201	106.2	4860 039.1	353 975.6
202	106.7	4860 074.1	354 034.0
203	107.8	4860 099.9	354 078.8
204	103.6	4860 114.4	354 089.9
205	103.6	4860 112.5	354 109.9
206	103.5	4860 134.0	354 119.4
208	108.7	4860 143.6	354 150.4
209	106.5	4860 179.9	354 211.8
210	106.5	4860 214.1	354 273.4
212	103.0	4860 143.3	354 062.6

Geocres No

NOTE

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

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

GO-ALRT REF PDI-601-

REFERENCE DRAWINGS

REVISIONS

DRAWN BY:
T.T.
1985.05.03

DESIGNED BY:
P.V.

CHK'D BY:
P.V.

APPROVED BY:
T.B.

SCALE:
FULL SIZE ONLY
AS SHOWN

GO-ALRT
Ministry of Transportation and Communications

WHITBY-OSHAWA SECTION

GO-ALRT-C.P.R.-G.M. SPURS

BOREHOLE LOCATIONS & SUBSURFACE STRATA

CONTRACT NO DWG NO REV SHEET

PROJECT MANAGER



March 19, 1985
P7407.00

Ministry of Transportation
and Communications
Pavement and Foundations
Design Section
Central Building
1201 Wilson Avenue
Downsview, Ontario
M3M 1J8



Attention: Mr. M. Devata

Gentlemen:

GO-ALRT Program
CPR-GM Spurs - WO EGG-001-3
District 6
Whitby-Oshawa Section
Foundation Investigation

The field investigations for the subject project have just been completed. They took longer than we initially estimated because of the very hard drilling and coring conditions encountered. We are in the final process of logging the soil samples and laboratory testing.

In order that your structural consultants can proceed with their preliminary designs we are providing the following information on site conditions and preliminary design criteria. Should there be any significant changes in any of the conditions or parameters during our subsequent evaluation and finalization of the report, we will keep you advised.

The general stratigraphy at the site consists of five soil units which in the order of increasing depth are as follows.

- (i) Sandy silt to sandy silty clay, very stiff to hard.
- (ii) Interbedded silty sands and gravelly sands, dense to very dense.
- (iii) Silty fine sand, dense to very dense.
- (iv) Silty clay, very stiff to hard.
- (v) Silty fine sand, dense to very dense.

ACRES INTERNATIONAL LIMITED

5259 Dorchester Road, P.O. Box 1001, Niagara Falls, Ontario, Canada L2E 6W1
Telephone 416-354-3831 Telex 061-5107

Vancouver, Calgary, Winnipeg, Toronto, Burlington, Halifax, Sydney, St. John's

March 19, 1985

In the area of the structures the foundations will bear on the third soil unit of silty fine sand at the proposed foundation level of approximately 93 m. This deposit should provide adequate support for the box structure and retaining walls. No unusual problems are anticipated at this level provided the site is properly and adequately dewatered.

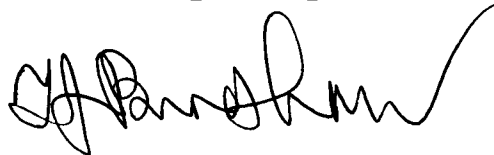
- The upper three soil units will tend to behave as cohesionless materials. In them there appears to be a common groundwater regime with a water level of approximately elevation 103 m at the time of the investigations. The piezometers installed in the impervious silty clay layer (iv) and the underlying pervious silty sand unit (v) have not stabilized to date. It may take another month of monitoring to establish whether the pervious silty sand unit (v) has a different groundwater regime and whether that level will cause any potential uplift problem to the overlying impervious silty clay unit (iv).
- It is anticipated that the control of groundwater will be of major concern during construction and special care and dewatering techniques may be required in lowering the groundwater below the base of the cut.
- Depressurization of the pervious silty sand unit (v) underlying the silty clay (iv) may be required depending on the stabilized groundwater levels.
- The groundwater levels at the site control the excavation slopes and the bearing capacity of various soil units.
- A permanent drainage system must be designed to lower the groundwater level to at least 1.5 m below the excavation and foundation level.
- With regard to stable cut slopes, preliminary analyses indicate that slopes with a minimum amount of drainage provisions should be cut at 3H:1V to have adequate factor of safety. By providing a berm and drainage provision the slope can be steepened to an overall slope of approximately 2.3H:1V. These values are preliminary and require more detailed analyses.
- For the design of spread footings, a factored bearing capacity of 800 kPa at ultimate limit state, Type I, and a bearing capacity of 300 kPa at serviceability limit state, Type II are recommended.
- For the design of base slab of the box structure, a subgrade modulus 200 lb per cu inch is considered reasonable.

March 19, 1985

- The foundations should be placed at a minimum depth of 1.2 m below finished grade elevation to obtain adequate protection against frost action.
- For checking safety against sliding, the coefficient of friction between soil and the concrete base can be taken as 0.45.

We trust that the above brief description of the site conditions and preliminary design parameters will permit your structural consultants to commence their preliminary designs. However, should you have any questions or require clarification of these matters, we would be pleased to discuss them with you at your convenience.

Yours very truly,



T. J. Bradshaw
Deputy Head
Geotechnical Department

TJB:mjg

cc - R. G. Tanner
D. B. Sampson

memorandum



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

Date: 1985 04 19

Atten: P. Roy

RE: Foundation Investigation
CP Rail - GM Spur Structures
EGG-001-3, Pickering to Oshawa

The field investigations for the above-mentioned project was completed during mid March 1985 and it appears that it took longer than the geotechnical consultant, Acres Ltd. initially estimated. Subsequently, a meeting was held in our office attended by T.J. Bradshaw (Acres Ltd.), J. Busbridge (Golder Associates) D. Dundas and M. Devata of M.T.C. to develop some uniformity with respect to recommendations of the proposed cuts related to this project as well as Thornton Road project by Golder Associates. A memorandum dated 85 03 19 containing preliminary recommendations was submitted by Acres Ltd. to this Office. Our review concluded that this memorandum did not cover all the aspects of the foundation and earthwork requirements and in view of this, the submitted information was not released. Acres Ltd. were of the opinion that unless they perform some laboratory testing and further stability analysis, it will be difficult for them to provide reasonable preliminary recommendations. Further to the letter of 85 03 19, Acres Ltd. have undertaken more detailed studies with respect to the long term stability of cuts ranging between 11 to 15 m in depth. We have now received their detailed preliminary recommendations in a letter dated 85 04 15 covering items such as cut slope requirements and bearing capacity factors for concrete structures.

It is understood that a final report will be submitted to us for review by 85 04 26. We believe the enclosed data will be adequate for your design purposes and should you need any further information with regard to this project, please contact us.

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

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

MD/mmj

c.c. - D. Garner
J. Lyle
R. Radolli



April 15, 1985
P7407.00

Ministry of Transportation
and Communications
Pavement and Foundation
Design Section
Central Building
1201 Wilson Avenue
Downsview, Ontario
M3M 1J8



Attention: Mr. M. Devata

Gentlemen:

GO-ALRT Program
CPR-GM Spurs - WO EGG-001-3
District 6
Whitby-Oshawa Section
Foundation Investigation

Further to our letter of March 19, 1985, we have undertaken more detailed studies regarding the stability of the cut slopes for the subject project and several other design factors. This letter outlines our findings and design recommendations for these items.

(a) Design of Cut Slopes

Within the portion of the project included in our investigation program, that is, chainages 28+300 to 28+650, the height of the cut slopes as well as the soil stratigraphy vary. Throughout the western portion of the site, chainages 28+300 to 28+420, the track grade has been set at approximately elevation 95.5 m. In our slope stability deliberation, we have assumed that the excavation would extend about 1.5 m below the T.O.R. elevation to provide for ditches and placement of select embankment fill. The ground surface elevation in this area varies from approximately 109 m on the south to 105 m on the north. Such a configuration results in slopes approximately 15 m and 11 m high on the south and north sides respectively.

ACRES INTERNATIONAL LIMITED

5259 Dorchester Road, P.O. Box 1001, Niagara Falls, Ontario, Canada L2E 6W1
Telephone 416-354-3831 Telex 061-5107

Vancouver, Calgary, Winnipeg, Toronto, Burlington, Halifax, Sydney, St. John's

April 15, 1985

East of the CPR/GM Spur lines, between chainages 28+500 and 28+650, the track grade rises slightly and the ground surface elevation lowers. This results in slope heights ranging from approximately 15 m at 28+500 to 8.5 m at 28+650.

On the understanding that the width of property right of way is a critical factor in this area, we have considered a number of configurations in attempting to achieve the most compact arrangement. The recommended solution involves a relatively steep cut slope of 1.9H:IV together with a free draining granular berm and two drain lines running parallel to the cut as shown on the attached sketch. We are proposing that the horizontal width of the berm at the base of the cut be 4 m. The granular berm height and width were optimized taking into consideration the groundwater table. Based on the analyses a constant berm elevation of 101 m was considered to be most appropriate. The berm width at elevation 101 m would vary depending on slope height. Where the crest-of-slope elevation is 106 m or lower, the berm width would be 3 m. For a crest elevation of 109 m, the berm width would increase to 4 m. These parameters result in overall average slopes of approximately 2.18H:IV.

Of critical importance in the design of the cut is the control of the groundwater profile, not only adjacent to the slopes but also below the base of the cut. The proper design and installation of the drains will be very important to ensure that the groundwater is adequately controlled. It is recommended that manholes be installed on these drain lines, at least the one at toe of the slope, to permit checking and maintenance of the lines for plugging or malfunction.

(b) Bearing Capacity Factors

In our previous letter, the preliminary value of the serviceability limit state bearing capacity, Type II was recommended as 300 kPa. On reviewing the data, this factor can be increased to 350 kPa.

(c) Foundation Zone for Concrete Structures

The soil existing in the foundation zone of the concrete structures has a gradation which makes it susceptible to the support of frost heave. In addition, there is a supply of water below the cut to contribute to the

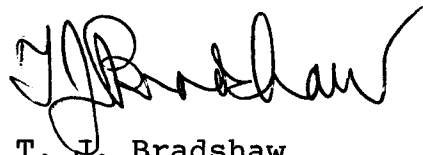
April 15, 1985

formation of ice lenses. It is therefore recommended that the soil existing below the concrete structures, within the zone of potential frost penetration, be removed and replaced by a non frost-susceptible material such as concrete fine aggregate. The depth of replacement below the concrete surface should be equivalent to 1.2 m of soil. Since the thermal conductivity of the concrete is in the same range as the on site soils, a depth of replacement of approximately 1.5 m should be adequate.

We trust that the foregoing amplify and clarify some of the recommendations outlined in our March 19, 1985 letter. We are in the process of preparing our final report which we propose to issue by April 26, 1985.

Should you or your structural consultants have any further questions, we will be pleased to discuss them with you at your convenience.

Yours very truly,



T. J. Bradshaw
Deputy Head,
Geotechnical Department

TJB:mg

Attach

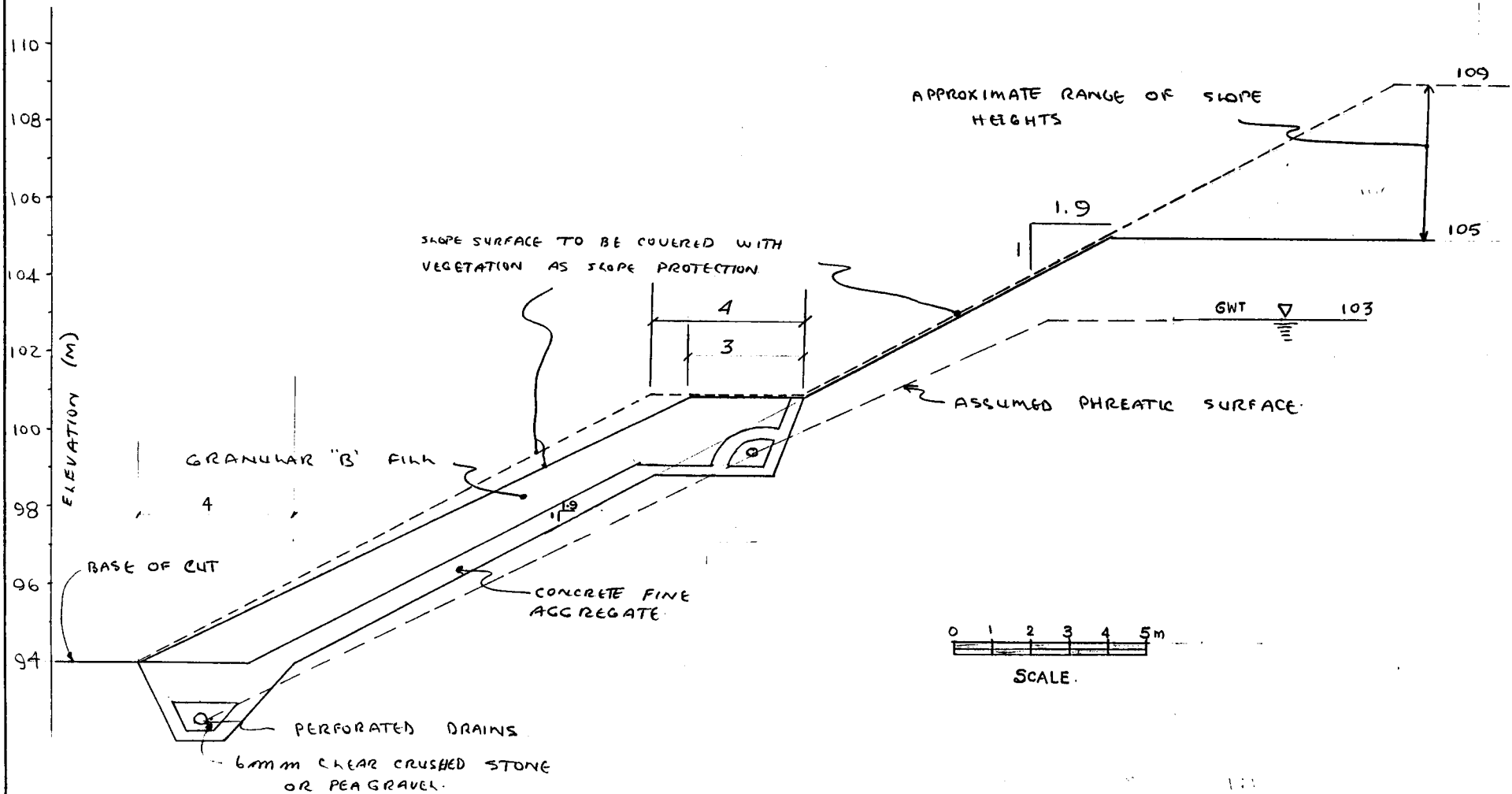


Calculations

SUBJECT:

GO-ALRT / CPR / GM SPURS OSHAWA:
PROPOSED SLOPE AND DRAIN CONFIGURATIONS.

JOB NUMBER P7407.00
FILE NUMBER
SHEET 1 OF
BY P.V. DATE 15/04/85
APP T.J.B. DATE 15/04/85



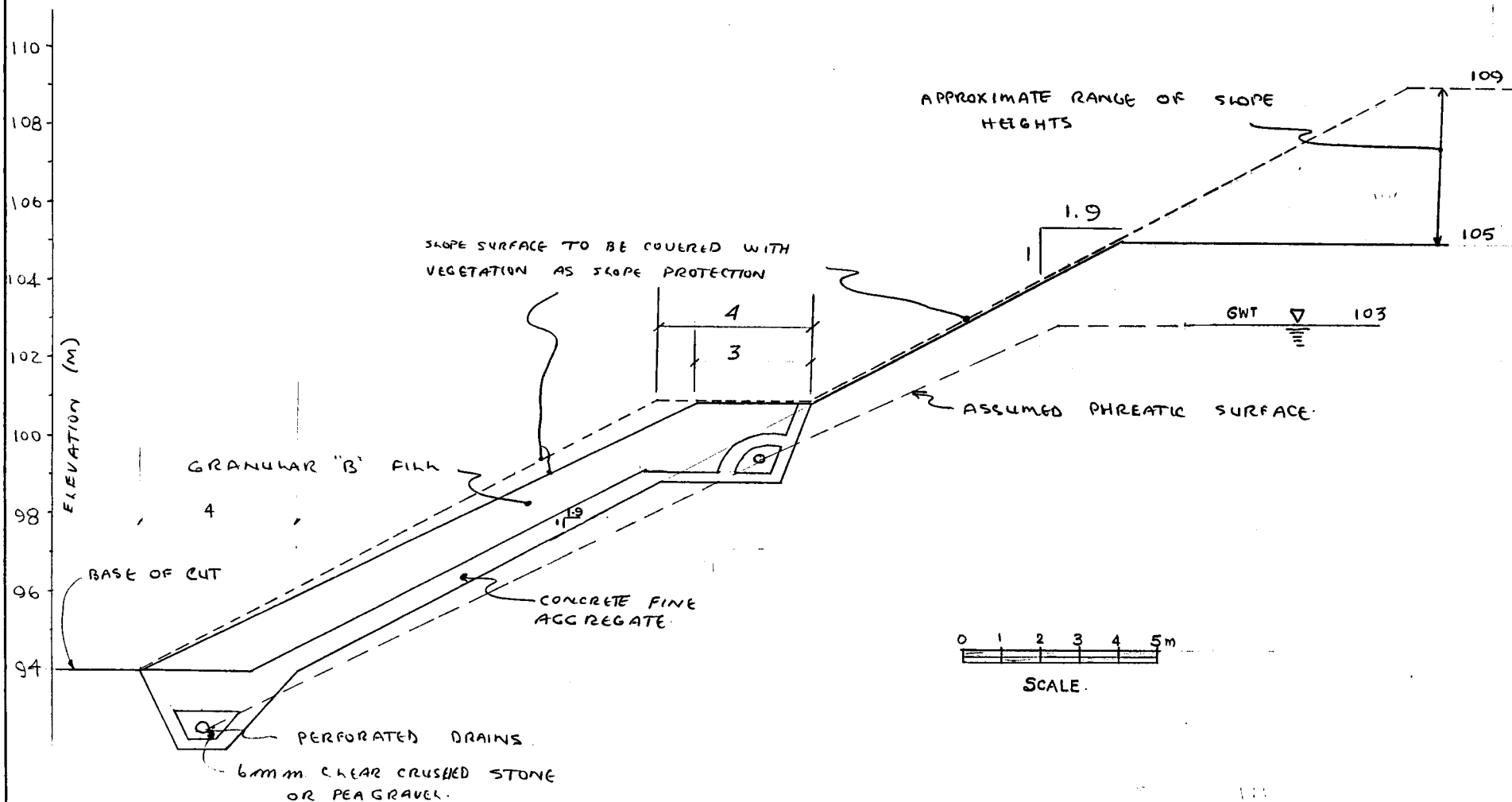


Calculations

SUBJECT:

GO-ALRT / CPR / GM SPURS OSHAWA
PROPOSED SLOPE AND DRAIN CONFIGURATIONS

JOB NUMBER P7407.00
FILE NUMBER
SHEET 1 OF
BY P.V. DATE 15/04/85
APP T.J.B. DATE 15/04/85



memorandum



To: D.P. Garner
Project Director
GO-ALRT
3625 Dufferin Street

Date: 1985 05 13

From: Foundation Design Section
Room 315, Central Building

RE: Foundation Investigation and Design Report
GO-ALRT Pickering to Oshawa
CPR-GM Spurs
Project EGG-001-3
District 6, Toronto

As requested by Mr. G.C.E. Burkhardt, Head, Central Region Structural Section, we have arranged for a foundation investigation to be carried out for the above-noted project. The project, to be carried out under our technical supervision, was assigned to the consulting geotechnical engineers Acres International Ltd.

We have carefully reviewed their progress at each stage of the report's development. This involved reviews of preliminary design recommendations and final report design recommendations. Only the format of the factual portion of these reports has been reviewed, as the descriptions of site conditions are considered to be the responsibility of the consultant. The consultant was advised of our comments at various stages of the project.

Stability of the cut slopes and dewatering, are the most critical and complicated aspects of this project. We have reviewed the consultant's analyses and are satisfied that the recommended design is in accordance with accepted geotechnical engineering practice.

The final Foundation Investigation and Design Report is attached. This report is intended to provide sufficient information to allow the design and construction of this project to proceed, subject to the following comments:

Slope Geometry and Drainage

- Further to our meeting of 85 05 07, regarding cut slope geometry:

To ensure the compatibility of the recommended slope treatments at the Thornton Road and CPR-GM Spurs projects some modifications to the geotechnical consultants' designs were proposed. Figure 1 illustrates the revised design recommendations.

- The 1.9H:1V slopes can be cut before constructing the berms. However, in order to prevent slope distress, the berm should be constructed as soon as possible after excavation after cutting the 1.9:1 slopes. If necessary, the excavation/berm construction may be carried out in sections.

.....2

- The permanent groundwater control scheme requires 1.5 m deep berm drains and 0.5 m deep toe drains. A minimum cover of 1.2 m is required for frost protection for these drains. Suitable outlets, designed to permit drainage throughout the year, are required for both drain types.
- Between Sta. 28 + 200 and Sta. 28 + 300, (Thornton Road) the recommended slope geometry is based on the design recommended by Golder & Assoc. The recommended berm elevation is 3.5 m below the elevation of the crest of the slope (equivalent berm elevation est. 102.5 m). The berm is 1.5 m deep in the vertical direction which corresponds to a minimum berm width of 2.85 m.
- Between Sta. 28 + 350 and Sta. 28 + 650, (CPR-GM Spurs) the recommended slope geometry is based on the design recommended by Acres International Limited. The berm will have a constant elevation of 101 m. Where the crest of the slope elevation is 106 m or lower, the berm width should be 3 m. Where the crest of the slope is above elev. 106 m the berm should be widened (in a linear relationship with the slope crest elevation) to a maximum of 4 m at elev. 109 m.
- Between Sta. 28 + 300 and Sta. 28 + 350, the slope geometry should be determined by a linear interpolation between the recommendations for the slope at the boundaries of this transition zone.
- The stability of the slopes must be maintained during construction of the berm and toe drains. It is suggested that these excavations might be carried out using a trench box to support the slopes. As the drain excavations will create potentially unstable slopes, the excavation/backfilling operation should be carried out in sections with 10± m maximum length.
- The berm material and drain backfill should be free-draining material such as M.T.C. 'Gravel Sheetting'. (Refer to attached SP No. 309 for the grain size distribution for this backfill material).
- The drains should consist of 150 m diameter perforated pipes, wrapped in filter cloth and surrounded by a minimum thickness of 0.3 m of free-draining material such as M.T.C. 'Gravel Sheetting'.

Dewatering

- The Acres report recommends that the groundwater level should be lowered a minimum of 1 m below the face of the excavated slope.
- In our opinion, this dewatering requirement for earthwork can be reduced, provided that a suitable construction sequence is followed. However, the groundwater level should still be lowered a minimum of 1 m below footing excavations. The Contractor should be advised that the project will involve excavations in non-cohesive materials, below the existing groundwater level, and that non-cohesive material is highly susceptible to boiling under conditions of unbalanced hydrostatic head.

The Contractor should design his excavation/dewatering operations to ensure that the foundation soil is not disturbed.

Suggested Sequence of Construction

- In our opinion, the dewatering requirements recommended by the consultant for the earthwork portion of this project can be reduced, provided that a suitable sequence of construction is followed. All excavations should be completed starting at the lowest excavation elevation and advancing toward the highest excavation elevation, in order to take advantage of gravity drainage. Where water accumulates at the low elevations, pumping by sumps can be utilized.

Our suggested sequence of construction follows:

- 1) Complete required excavations above the prevailing groundwater level.
- 2) In our opinion, a pilot trench can be used to depress the groundwater level. The trench should be designed to lower the groundwater as required and will probably involve excavation to approximately the invert elevation of the proposed GO-ALRT ditch.
- 3) Excavate slope geometry to berm elevations.
- 4) Install berm subdrain as described in this memo.
- 5) Excavate slope geometry to toe elevation.
- 6) Install toe subdrain as described in this memo
- 7) Construct ditches.
- 8) Construct granular berm.

Frost Protection

- For frost protection, 1.2 m of earth cover or equivalent is required for earthworks and foundations. It is our opinion, that the Acres report recommendations for 1.5 m of cover (Pg. 28, Para. 4; Pg. 30, Para. 1) can be relaxed to 1.2 m of cover.

Compaction

- Compaction of materials should be carried out as per M.T.C. practice. Restrictions for vibratory rollers near abutments or retaining walls apply.

Earth Pressure Calculations

- Backfill to structures should consist of granular material in accordance with M.T.C. Standard Special Provision #121 (83 10). Computation of earth pressures should be in accordance with Section 6.6.1.2 of the O.H.B.D.C.

For design purposes, the physical properties of the backfill are as follows:

Material	ϕ	γ
Granular 'A'	35°	22.0 kN/m ³
Granular 'B'	30°	21.2 kN/m ³

At this site, the foundation is considered to be yielding and the active condition applies insofar as lateral earth pressures are concerned.

Soil Anchors

- If tieback retaining walls are required for this project, the design criteria for soil anchor bond stress can be provided by this Office. The recommended bond stresses will be based on anchor tests. For estimation purposes, an allowable bond stress of 100 kPa is suggested, rather than the 60± kPa allowable bond stress recommended in the Acres report.

In the event that G0-ALRT may wish to advise the Contractor, we draw to your attention that 1.52 m long pieces of augers were left in the ground at the following approximate locations:

- Sta. 28 + 296, C/L elev. 92.5±
- Sta. 28 + 572, C/L elev. 89.7±

If there are any questions, please contact this office.

D.H. Dundas

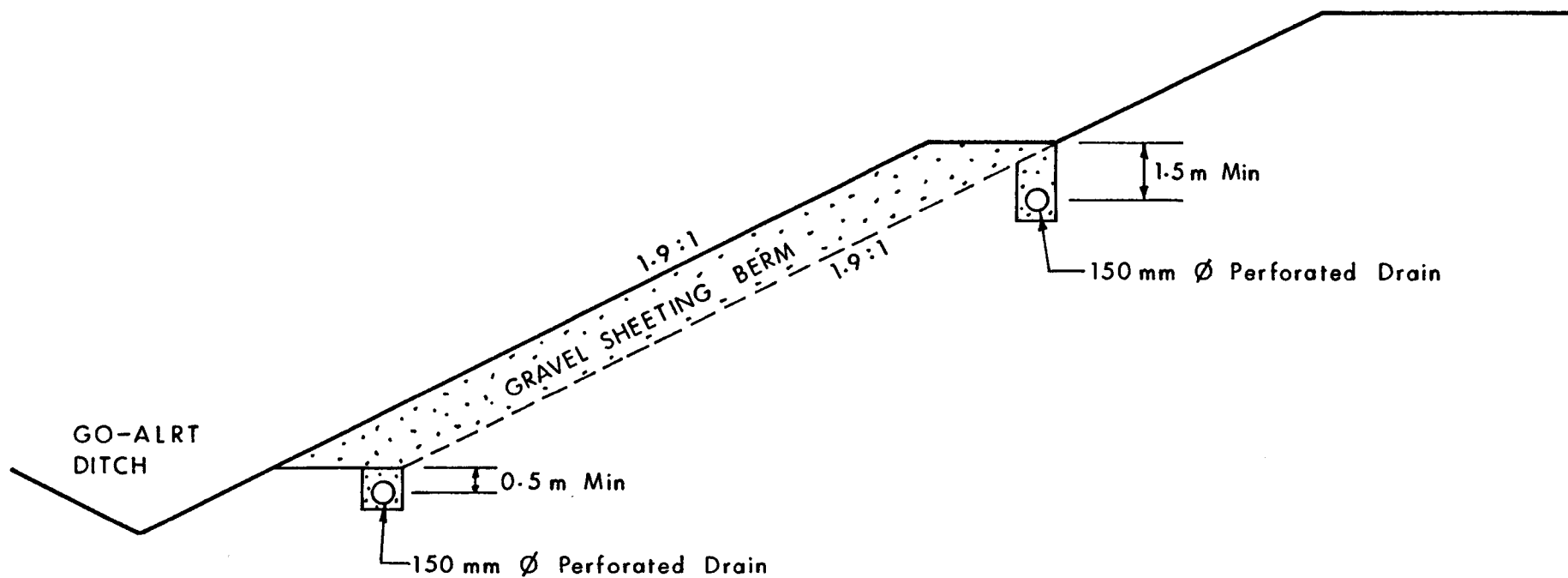
D.H. Dundas, P. Eng.
Foundations Engineer

for

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

DHD/MD/mmj

c.c. - G.C.E. Burkhardt
J. Lyle (2)
J. Ball
H. Clelland
D. Gunter
W. Lin
C. Watson
R. Hore (Cover Only)
T.J. Kovich (Cover Only)
D.J. Zander



N T S

SLOPE GEOMETRY CPR-GM SPURS & THORNTON ROAD .

GO-ALRT PROJECTS EGG-001-3 &
EGG-001-18

FIG. No 1

GRAVEL SHEETING - Item No

Under this item the Contractor shall supply and place granular material as gravel sheeting on the side slopes designated in the contract drawings. Gravel sheeting material shall meet the requirements of Granular 'B' as specified in OPSS 1010 except that the gradation shall conform to the following:

<u>Sieve Size</u>	<u>% Passing</u>
150 mm	100
26.5 mm	50 - 100
13.2 mm	35 - 100
4.75 mm	20 - 80
1.18 mm	10 - 50
300 um	5 - 25
150 um	0 - 15
75 um	0 - 8

In each earth cut where gravel sheeting is required, the Contractor shall schedule his operations in such a manner that the sheeting operation follows the excavation work as closely as is practical and that it is carried out immediately following the completion of the cut.

Compaction of the sheeting material is not required.

Payment at the contract price for the above tender item shall be compensation in full for the supply of all labour, equipment and material required for placing gravel sheeting as described herein.

WARRANT: Always with this tender item