



Golder Associates

CONSULTING GEOTECHNICAL AND MINING ENGINEERS

REPORT TO
MINISTRY OF TRANSPORTATION
AND COMMUNICATIONS

GEOTECHNICAL INVESTIGATION
UPGRADING OF HWY. #11
(CONTRACT #78-56; WP #9-75-04)

ATIKOKAN, ONTARIO

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ABSTRACT

The results of a geotechnical investigation carried out along selected portions of Highway 11 to the east of the town of Atikokan in northwestern Ontario are reported. An assessment of construction techniques and pavement performance was made and recommendations are given related to the proposed reconstruction of other sections of Highway 11 in the area.

The sections of highway investigated traversed relatively deep peat deposits in bedrock depressions. In general, the peat deposit ranged in thickness between 4 and 6 m and was underlain by sandy silt to silty sand material. The groundwater level was at the surrounding swamp level at each site investigated.

Laboratory tests were carried out on relatively undisturbed samples of the peat. These revealed that the peat is very compressible and extremely weak, and except for the more fibrous peat, shows no appreciable gain in strength even after long term loading under the pavement structure. In-situ vane strength testing was also carried out.

For future reconstruction work it is recommended that, in areas of shallow peat deposits (less than 2 m), conventional buttress construction methods can be used. However, in those areas where the peat thickness exceeds 2 - 3 m, it is recommended that a modified buttress approach involving pre-excavation, displacement and surcharging be adopted with the provision that the initial excavation should not extend beyond about 2 m depth below swamp level.

1. INTRODUCTION

Golder Associates has been retained by the Ontario Ministry of Transportation and Communications to carry out a geotechnical investigation along selected sections of Highway 11 to the east of the town of Atikokan in Northwestern Ontario. The purpose of this investigation was to determine the pavement structure and subsurface conditions at specific locations along the highway and to explain the causes of the pavement distress which has occurred in many places. Two of the areas investigated are on a portion of the highway which has recently been widened and realigned and is subsequently showing signs of distress. The other two areas investigated are along a portion of the highway which is due to be reconstructed but under which similar subsurface conditions are known to exist. Recommendations are to be provided for construction procedures to be used in upgrading the section of road which is due to be reconstructed in 1982. Authorization for this work was contained in a letter from the Ministry of Transportation and Communications dated August 4, 1981.

2. BACKGROUND INFORMATION

2.1 Project Description

The sections of Highway 11 investigated are in M.T.C. Northwestern Region to the east and within 80 km of the town of Atikokan. Highway 11 between Thunder Bay and Atikokan was originally constructed some 50 years ago. The increasing traffic usage of the highway has necessitated widening and realignment of most of the highway. Many sections have already been reconstructed while others are planned for reconstruction in the future. Several of the areas which have been reconstructed and which traverse deep peat deposits have shown signs of distress.

2.2 Site Descriptions

The investigation was concentrated along four specific sections of Highway 11. Two of these (sites A and B) are on a reconstructed section of road which constitutes Contract Number 78-56, from junction 11B Atikokan easterly 10.2 miles in the District of Rainy River. The sites are approximately 5 km east of Atikokan at chainages 445 + 00 ft. and 622 + 00 ft. and are shown on the key plans on Figures 1 and 2 respectively. Sites C and D are on a section of road which is due to be reconstructed (WP 9-75-04) from 0.6 km east of Pickerel River easterly 21.8 km in the District of Thunder Bay, at chainages 13 + 500 m and 15 + 200 m, (Figures 3 and 4).

The physical appearance of all 4 sites is similar. At each the highway crosses a peat deposit contained in a relatively shallow bedrock depression. At the edge of these infilled depressions the highway passes through cuts in the rock outcrops.

The natural ground surface across the general area shows little relief and in general varies between elevations 435 and 495 m. At site B low cliffs are present.

3. AREA GEOLOGY AND PHYSIOGRAPHY

The areas investigated lie within the Canadian Shield where bedrock is of Precambrian age, usually at a shallow depth and consists of highly metamorphosed sediments, migmatites, granitized gneisses and gneissic or massive granitic rocks. The overburden consists mainly of discontinuous glaciolacustrine deposits and it is this combination of shallow bedrock, rock outcrops and glaciolacustrine deposits which characterizes the local physiography. Because irregular bedrock terrain is common in the area, organic terrain is widespread. The bedrock surrounding depressions in the rock surface has a low permeability so that surface waters are ponded, become stagnant and are eventually invaded by peat forming vegetation. In due course, the entire basin becomes infilled to form peat bogs or swamps overlying any glaciolacustrine deposits pre-existing in the basin.

4. SUBSURFACE CONDITIONS

4.1 General

This section describes the subsoil stratigraphy and groundwater conditions at each of the four sites.

Stratigraphic sections at each of the four sites are shown on Figures 1 to 4. Grain size distributions of representative samples are shown on Figures 5 to 12. In addition, the index properties together with compressibility and strength characteristics of the peat are described in Section 5 of this report and are summarized on Figures 13 to 20.

A total of 16 sampled borings and 12 vane test boreholes were put down at the site using the procedures described in Appendix I. The Record of Borehole sheets for the sampled borings follow the text of this report. The borehole numbering system is as follows: -

Site A	Boreholes 1 to 4
Site B	Boreholes 10 to 15
Site C	Boreholes 20 to 23
Site D	Boreholes 30 to 33

Unsampled vane test boreholes have been designated by a borehole number followed by a letter (A, B, C or D) depending on the number of vanes used. A different boring was made for each vane size. If vane testing was carried out adjacent to a sampled borehole, the sampled borehole would be designated by a number only, while the vane holes would be designated by the same number followed by a letter (A, B, C or D).

4.2 Stratigraphy

A summary account of the subsurface conditions encountered at the sites is given below. It should be noted that the stratigraphic boundaries shown on the Record of Borehole sheets and stratigraphic sections are inferred from

non-continuous sampling techniques. These boundaries typically represent a transition from one soil type to another and do not necessarily indicate an exact plane of geologic change. In particular, the boundary between the rockfill and the underlying native materials was difficult to determine because the voids in the rockfill increased in size with depth and it was difficult to define exactly when (and if) the lowest rock fragment had been penetrated. It should also be noted that conditions may vary between boreholes and the stratigraphic boundaries shown on Figures 1 to 4 have been drawn as shown for the purposes of illustration only.

The major subsurface feature at all sites is the deep peat deposit which has formed in the bedrock basins. These peat deposits typically extend to depths of between 4.0 to 6.4 m below the swamp level.

A prominent feature of the peat deposits is the surficial fibrous layer which extends to depths of between 1.2 and 2.7 m below ground surface. The character of the peat changes gradually with depth with the fibres becoming finer. Below about 3-4 m the peat is amorphous with little or no fibrous structure but with minor quantities of fibrous materials, wood fragments and soil mineral particles (Figures 5 and 6).

Natural water contents measured in samples range from 300 to 2300 percent with the higher values (in excess of 1000 percent) applying to the upper fibrous materials. Organic contents range between 70 and 90 percent and the density of solids (specific gravity) average about 2100 Kg/m^3 . Total unit weights measured are consistently about 1000 KN/m^3 . As discussed in detail in Section 5 of this report, both the fibrous and amorphous peats are very weak and highly compressible.

The peat is generally underlain by silt deposits which are generally sandy. However in some areas, the basal silt contains significant quantities of clay. The boundary between

the peat and the silty materials is not well defined and typically a thin (0.5 m) transition zone of organic silt material forms the boundary. The following is a detailed description of the basal soils at the individual sites.

Site A

The basal silt at this site contains significant proportions of clay and can be described as silt some clay to clayey silt. This material had a liquid limit, plastic limit and natural water content of 24, 19 and 36 percent respectively indicating a soil of low plasticity. The undrained shear strength of this material as determined by field vane tests is between 20 and 30 KPa.

Auger refusal was experienced in Borehole 2 at a depth of 0.9 m below the pavement surface. Due to difficult drilling conditions in the rockfill in Borehole 3, some equipment was lost in the hole and no further progress was possible below a depth of 7.3 m. The rockfill fragments were more widely spaced at this depth suggesting that the borehole had penetrated to close to the bottom of the fill.

Site B

The material underlying the peat at this site varies from a silt with trace to some clay to silt with some sand or silty sand. The results of gradation analyses carried out on samples of these materials are shown on Figures 7 and 8. Based on results of Standard Penetration tests, the basal silt materials are in a very loose to compact condition.

Borehole 14 was put down in a section of road under which a rockfill buttress was constructed according to M.T.C. SD-4-43. The results of the borehole at this location

indicates that the rockfill forming the buttress is underlain by 3.2 m of amorphous peat. Borehole 12 put down through the main body of rockfill was underlain by only 0.7 m of consolidated amorphous peat.

Site C

At this site the peat is underlain by silt with a trace to some fine sand grading to sandy silt/silty sand with trace to some gravel, (Figure 9). The condition of these materials varies from loose to very dense. Based on the results of Borehole 22, rockfill supporting the pavement structure at this site extends to a depth of 4.4 m below road level.

Site D

Loose to very dense silty fine sand to sandy silt with some gravel underlies the peat at Site D, (Figures 10 to 12). Auger refusal was experienced in Boreholes 30 and 31 at depths of 7.0 and 6.1 m respectively while Boreholes 32 and 33 were terminated without refusal at depths of 10.4 and 8.5 m respectively. In Boreholes 30 and 31, the last 1.0 m of augering before refusal was extremely difficult suggesting the presence of large gravel sizes or boulders. Based on the results of Borehole 33, the rockfill supporting the pavement structure at this site is 5.3 m deep.

In general, the stratigraphy encountered in the boreholes put down during this investigation is in good agreement with that disclosed by earlier auger holes put down by M.T.C. personnel, (Soil Profiles #11TB19-47A and 11TB19-48-2A).

As noted above, at least one borehole was put down through the pavement structure at each site. Typically, 0.1 to 0.2 m of asphalt are present on the road surface. This is underlain by gravelly fine to coarse sand to depths of 0.9 to 1.2 m except at Site B where Boreholes 12 and 14 encountered granular fill to depths of 3.1 and 3.0 m respectively. These depths are inferred from variations in auger resistance. No samples were taken. Underlying the granular fill is rockfill. This material is blasted rock from adjacent rock cuts and based on visual observation of this material in higher embankments, the rockfill fragments appear to have a maximum dimension of 0.6 m.

4.3 Groundwater Conditions

Groundwater levels observed in the standpipe piezometers and in the open holes immediately after drilling are shown on the Record of Borehole sheets and on the stratigraphic sections. At each piezometer location, the groundwater level is at or about the natural ground surface which is typical of organic terrain. The water levels in the holes advanced through the pavement structure also correspond directly with the water level in the surrounding area. Artesian conditions in the basal deposits were not encountered.

5. ENGINEERING PROPERTIES OF PEAT

5.1 General

This section reports the results of laboratory testing carried out on the peat. The index properties derived from this laboratory testing are presented in graphical form. Trends from published data are also shown on these plots. Various engineering parameters derived from consolidation tests and undrained strength tests are also presented graphically. In addition some engineering parameters have been correlated with index properties. In summarizing the consolidation and undrained shear strength characteristics of the peat, three peat zones have been identified depending on the probable degree of disturbance before sampling: virgin peat at a considerable distance away from the pavement structure, "disturbed" peat adjacent to the pavement structure, and peat from directly below rockfill.

5.2 Index Properties

Various index properties (natural water content, organic content, density of solids and total unit weight) obtained from tests on representative samples are interrelated in graphical form on Figure 13. On each of these plots the dashed lines represent published behavioral trends* and it is apparent that the index properties of the peat from the investigated sites follow these trends.

The majority of the natural water contents are in the 400 to 800 percent range although values in excess of 1000 percent were measured, (refer to Record of Borehole sheets). The measured organic contents typically range between 70 and 90 percent although in the transition zone between the peat and the underlying silts, organic contents of about 20 percent were measured. There is a linear relationship between water content and organic content up to water contents of about 500

*MacFarlane, I.C. (ed.) 1969. Muskeg Engineering Handbook (Muskeg Subcommittee of NRC, Assoc. Comm. on Geotechnical Research, Univ. Toronto Press).

percent beyond which no further increase in organic content occurs. The density of solids averages around 2000 Kg/m^3 (specific gravity 2.0) and decreases linearly with increasing water content and organic content. Total unit weights measured average about 10 KN/m^3 or just slightly in excess of the unit weight of water. The total unit weight tends to decrease slightly with increasing water content.

5.3 Compressibility Characteristics

The magnitude of consolidation settlement which occurs as a result of dissipation of excess pore pressures within a loaded soil mass can be estimated from compression indices (C_c) obtained from the results of one-dimensional consolidation tests. The results of these tests can also be used to determine the coefficient of consolidation, (C_v) which describes the rate of pore pressure dissipation. Values of secondary compression index (C_{α}) are also derived from consolidation tests and are used to estimate the magnitude of settlements which will occur over any given time interval from the time that pore pressure dissipation (i.e. primary consolidation) is complete. Further, the coefficient of permeability (K) can be estimated for any given stress level. Consolidation test samples were either 12 mm or 19 mm in thickness. Loads were applied every 24 hours and a load increment ratio of 1 was used. At the end of the last loading period, the sample was allowed to rebound. During reduction of the basic data, primary consolidation and secondary compression strains were calculated for each loading period.

The results of the consolidation tests on representative samples are summarized on Figure 14. A notable feature of the test results is the similarity in shape of the (log) stress-strain curves for all samples of peat regardless of origin. The (log) stress-strain curves indicate some preconsolidation with the preconsolidation pressure p_c (~ 14 KPa) slightly in excess of the in-situ effective overburden

pressure p_o' (~5 - 10 KPa). However, the degree of overconsolidation is small and the assumption can be made that the material is normally consolidated.

The secondary compression index, C_{α} , averages about 2×10^{-1} which is high but consistent with published data.* The compression index, C_c , averages about 2.0 in virgin compression and is again consistent with published data. The ratio of C_{α} to C_c is essentially independent of the stress level and is typically about 0.1 to 0.15 which is in the upper range of published data.**

The coefficient of consolidation, C_v , has been calculated from consolidation test results using both root time and log time plots. In addition, values of C_v have been calculated from the consolidation phases of the consolidated undrained triaxial tests (R tests) described in the next section of this report. These latter values of C_v have also been plotted on Figure 14. There is some scatter in these results but the values lie between 10^{-4} and 10^{-2} cm²/sec with C_v decreasing at the higher stress levels.

The coefficient of permeability, K , derived from measured values of C_v ranges between 10^{-6} and 10^{-9} cm/sec and decreases with increasing stress.

*Mesri, G. (1973) "Coefficient of Secondary Compression". Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 99, No. SM1, pp. 123 - 137.

**Mesri, G. and Godlewski, P.M. (1977). "Time and Stress" - Compressibility Interrelationship." Journal of the Geotechnical Engineering Division, ASCE, Vol. 103 No. GT5, pp. 417 - 430.

5.4 Undrained Strength Characteristics

5.4.1 Field Vane Shear Strength Tests

Field vane (undrained) strengths were determined using different size vanes. The test results are summarized on Figure 15 and indicate an inverse relationship between vane size and measured strength (i.e. the measured strength increases with decreasing size of vane). The size effect is significant with a factor of 4-5 between the strengths measured using the (large) peat vane and the (small) diamond vane. This size effect has been observed previously in peat deposits, (MacFarlane, 1969) and can presumably be attributed to the more representative nature of the larger size "sample" tested using the larger vane, (i.e. the effect of individual wood fragments or unusually strong fibres would be less significant in large samples). Interpretation of field vane shear strengths is difficult in the fibrous peat and even in the amorphous peat since the frequent occurrence of wood fragments can result in unrealistically high strengths. However, it would appear that the peat typically has an undrained shear strength of 10 to 20 KPa.

Figure 15B summarizes the field vane strengths for each vane type. From these results it appears that the peat strength at Site B is lower than at Sites A, C and D. Further, there appears to be little difference in strength between the virgin peat and the peat in the "disturbed" zone adjacent to the embankment. However, although limited data is available, it is clear that the peat under the rockfill has an increased strength as a result of compression under the rockfill loading.

5.4.2 Triaxial Tests

Consolidated undrained triaxial (R) tests were carried out on representative samples of peat. The detailed test results are

shown on Figures 16 to 19 and the data is summarized on Figure 20. In these tests, each sample was consolidated to a specified consolidation pressure (P'). When consolidation was complete, drainage was shut off and the sample was subjected to increasing axial load until failure occurred or a specified strain (20 percent) was reached. When this "failure" occurred, the sample was then allowed to "relax" as the proving ring reverted to its original shape. Another sample of the same material was then consolidated under a higher pressure and the process repeated until three or four tests were carried out at different consolidation stresses. These tests constitute a single set of R tests. On one of the samples a multistage test was performed. In this case, the sample was first consolidated and then subjected to undrained stress to 5 percent axial strain. At this time, shearing was stopped and the sample was consolidated under a higher pressure and sheared again. This procedure was repeated for four successive consolidation pressures.

During consolidation, readings of volume change were taken with time and the results were used to plot the volumetric strain versus time curves. From these curves, C_v was calculated. During undrained shearing, axial stress and deflections were measured and stress-strain curves plotted. After "failure", relaxation tests were carried out. These tests involved recording stress and deflection at various times after termination of shearing. From these data, plots of normalized shear stress versus strain rate were made.

The undrained shear strength, S_u was arbitrarily defined at 5 percent strain and has been plotted against the effective consolidation pressure P' on Figure 20 for all tests. These data show considerable scatter except for the more fibrous sample tested. Some of the samples with larger soil mineral contents also showed a reasonable gain in strength with increasing consolidation pressure. The scatter in test data for the amorphous samples is not related to the zone from

which the samples were obtained (i.e. virgin unstressed areas or disturbed areas adjacent to the rockfill). Despite the scatter in test data for amorphous peat samples, there is no apparent consistent gain in the strength with increasing consolidation pressure. Although the samples were allowed to "consolidate" overnight, when the drainage valves were closed prior to shearing, the porewater pressure increased to a value close to the cell pressure because of the lack of interparticle contacts or structure within the amorphous peat. Therefore the amorphous peat samples were sheared at a low effective confining pressure regardless of the consolidation stresses applied.

The more fibrous sample from Borehole 33 which shows consistent strength gain with increase in consolidation pressure has an undrained strength of approximately 35 KPa, which is due to the consolidating effect of the overlying rockfill in the field.

Also included on Figure 20 is a summary plot of S_u versus water content after consolidation. Again there is significant scatter in the results but as expected, the general trend is for decreasing shear strength with increasing water content. On this basis, the shear strength at water content values occurring in the field is of the order of 5 KPa.

The strain rate effect parameter, $\rho_{0.1}$ has been determined for each test stage. This parameter is defined as the change in shear strength resulting from an order of magnitude change in strain rate, and normalized with respect to the shear strength at 0.1%/hr. Values of $\rho_{0.1}$ are plotted against the water content of the sample after consolidation on Figure 20. The value of $\rho_{0.1}$ lies within the relatively narrow range between 15 and 20 percent and is essentially independent of water content and type of peat. To illustrate the significance of this strain rate effect, the difference in rate of strain during field vane testing (about 5 percent per min.) and

during actual loading in the field (about 5 percent per week) is approximately 10^4 . Therefore, the operating shear strength in the field could be as little as a third of the field vane strength.

5.5 Summary

The results of laboratory and field tests on the fibrous and amorphous peat deposits has indicated the following:

- . The field vane strength of fibrous and amorphous peat decreases with increasing vane size. It is generally accepted that the larger the volume of soil tested, the more representative will be the measured property. Therefore, the field vane strength of 10-20 kPa measured using the largest vane is most applicable for design purposes.
- . In the R triaxial tests, the highly organic amorphous peat had insufficient fibre or soil mineral particle structure to develop increased strength as a result of consolidation. By contrast, fibrous peat can develop relatively high strength as a result of consolidation.
- . Both the fibrous and amorphous peats are highly strain rate dependent. Thus, if field vane strengths are corrected to account for the differences in times to failure in the vane test and full scale field loading situation, the operating strength in the field could be as low as about one third of the vane strength. (e.g. 4 - 8 kPa using the large peat vane strengths).
- . Both the fibrous and amorphous peats are highly compressible with high rates of secondary compression. Computed permeabilities at stress levels associated with embankment loadings are sufficiently low to ensure an undrained response to loading.

Based on the above and visual examination of the samples, it is considered that the amorphous peat is an extremely weak soil which will behave essentially as a viscous "jelly-like" material under field loading conditions. Although the fibrous peat is also weak and highly compressible, its fibrous structure would facilitate strength gain under field loading conditions.

6. PERFORMANCE OF EXISTING PAVEMENT

Figure 21 summarizes the conditions and performance of the two sections of road at each location where the highway traverses a peat deposit. The station, lateral extent and depth of these peat deposits were obtained from M.T.C. data (Soil Profiles #11TB19-47A and 11TB19-48-2A for Contract #78-56 and WP #9-75-04 respectively). These data are verified by the results of the boreholes put down during this investigation. For Contract #78-56 the height of additional fill placed during widening/realignment, the type of widening (i.e. whether on or off the centre line of the existing road) and the construction treatment employed are also given. For WP #9-75-04, the proposed widening details, treatment and height of additional fill are given. This information was obtained from the M.T.C. soil profiles and contract documents. Finally, Figure 21 indicates where pavement distress (if any) was experienced at these various stations. This pavement distress summary is based on visual evidence obtained during field trips by our staff (i.e. presence of overlays, rutting, cracking, slips etc.) and on information and photographs supplied by M.T.C. personnel. It is noted that all distressed areas within Contract #78-56 had been patched under Contract #80-22.

Sites A and B in Contract #78-56 were reported to be severely distressed. In the vicinity of station 443 (Site A), braided longitudinal cracking existed parallel to and 1 m from the centre line on each side of the pavement. The central portion within the longitudinal cracks had settled differentially by as much as 150 mm relative to the surrounding pavement. In the vicinity of station 620, the surface was severely distorted with meandering longitudinal cracks. At all remaining distressed locations, the prominent feature was single or multiple longitudinal cracking.

In general, the pavement distress on WP #9-75-04 appears to be less severe although it is understood that ongoing maintenance has been necessary over the years. Based on comments contained in M.T.C. Soil Profile #11TB19-48-2A, it appears that the most common distress feature is slight to moderate distortion. At two sections, however, "slips" were observed along the shoulder of the pavement.

7. CAUSES OF DISTRESS - CONTRACT #78-56

The amorphous peat which underlies many sections of the highway is an extremely weak material. Even if relatively sophisticated and expensive treatments are employed, it would be difficult to obtain maintenance-free pavement conditions.

Figure 21 summarizes pertinent data from all sections (total of 24) which are underlain by peat. At each section, the pavement has been widened; at 4 locations the widening was off the existing centreline while at the remaining 20 sites, the widening was on the centreline. However, the raising of existing pavement was minimal at most locations (i.e. less than 0.8 m). Of the 24 sections where peat is present, 10 have suffered distress and 14 appear to have performed adequately. Of the 10 distressed sections, 8 were located where the peat thickness is greater than 3 - 4 m while at the remaining 2 sections, the peat was 0.8 and 2.7 m thick. It is noted that one of the 14 non-distressed sections was underlain by 6.4 m of peat. Further, of the non-distressed sections, 2 were underlain by 3.0 and 3.4 m of peat respectively and the rest by less than 2 m of peat.

At 2 sections, Stations 620 and 656 respectively, buttresses were constructed in accordance with M.T.C. SD-4-43, (Figure 21). At Station 620 where the peat thickness was 7.3 m, the results of a boring put down through the buttress encountered 3.2 m of peat below the buttress rockfill at this location. Although this information is limited, it suggests that excavation to construct the buttress in this specific location was unsuccessful in that the basal soils were not reached. At Station 656, the peat was only 2.0 m thick and no distress was observed. At all other sections, no special measures were implemented.

As a result of this pavement performance analysis and the nature of the peat deposits, the following general conclusions can be made: -

- (1) The majority of problems occur when the peat thickness is in excess of 2 to 3 m.
- (2) In the one section where a "buttress" construction was adopted, it was probably not fully effective because the peat was not completely removed and the buttress was not constructed on a firm bottom.

8. ALTERNATIVE APPROACHES TO RECONSTRUCTION WORKS

8.1 Ride on Widening

One approach is to simply ride the peat with the widened pavement without any special measures such as subexcavation and replacement, or preloading being employed. In this approach, which was commonly adopted on Contract #78-56, the fill would be constructed to the final height required. Some displacement of peat would inevitably take place during fill placement on virgin peat, but where the peat is relatively thick (in excess of say 2 m), displacement would not be complete. Therefore, post construction settlement and lateral deformation would be inevitable and would lead to distress of completed pavement. Thus, depending on the peat thickness, the problems which occurred on Contract #78-56 would occur and future maintenance costs would be incurred. This approach could only be successfully adopted in areas of thin peat and even then it may be advisable to employ preloading techniques to minimize post-construction settlements.

8.2 "Buttress" (Subexcavation and Replacement) Approach

This method is essentially the same as the treatments identified as M.T.C. SD-4-32/43/44 (Figure 21) and is shown schematically on Figure 22. Three distinct problems arise with this procedure in deep deposits of amorphous peat: -

- (1) When the buttress excavations are made adjacent to the existing fill, it is inevitable that lateral displacement of the existing fill will occur thereby causing loosening of existing rockfill and damage to the existing pavement. This damage may not in itself be extensive if the peat deposits are shallow. However, with deep peat deposits, there is a risk of an extensive failure which could affect traffic along the existing pavement.

- (2) It is not possible to make deep excavations in very weak amorphous peat without developing very flat side slopes. Further, the actual depth of excavation is not known with certainty and it is likely that not all peat will be excavated. For example, it is reported in the M.T.C. inspector's diary that the excavation for the buttress construction at Station 620 (Contract 78-56) took place over a long time period. Presumably, a large volume of peat was removed but based on limited data, it appears that the excavation did not reach a firm base. It is considered likely that the amorphous peat "flowed" into the excavation area when a critical depth of about 2 - 3 m had been reached.
- (3) The peat adjacent to and below the newly-placed rockfill will be in a disturbed state. Post-construction lateral deformation and settlement of this peat is inevitable with resulting settlement of the fill and distress to the supported pavement. It is understood that consideration is being given to attempting as deep an excavation as possible before backfilling to construct the buttress and surcharging. It is our opinion that any attempt to dig much below the upper fibrous mat will not be successful for the reasons described above and considerable construction cost inefficiencies will arise if deep excavations over a prolonged time period are attempted.

Where amorphous peat deposits are shallow (say 2 - 3 m thick), the problems described above would not be as prevalent (Figure 22.2). Thus, for these peat deposits, it should be possible to completely subexcavate and/or displace any peat remaining in the excavation with rockfill to ensure that the buttress backfill rests on a firm bottom. With the new fill on a firm bottom, post construction lateral deformation and settlement should be minimal. During buttress construction, distress to the existing fill would be limited to its outer extremities only and the main body of this material would remain undisturbed and stable.

It is possible to consider increasing the maximum depth of the buttress-type treatment above the 2 - 3 m range cited above if the peat deposits consist of more competent fibrous peat. In these materials, the sides slopes of excavations would be more stable and there would be less tendency for viscous-type flow which is associated with amorphous peat. However, it is unlikely that the maximum depth of buttress excavation in even

relatively competent fibrous peat could exceed 3 - 4 m. Further, there is risk of extensive damage to the existing trafficked pavement as a result of excessive lateral deformation or outright failure of the existing rockfill.

In summary, it is considered that the buttress-type approach as defined in M.T.C. SD-4-32/43/44 should only be employed in amorphous peat deposits where the peat thickness is less than 2 - 3 m. If the peat is fibrous and more competent, it may be possible to increase this peat thickness limit.

8.3 Displacement Method (With Preloading)

The aim of this approach which is illustrated on Figure 22.3 is to maximize shear stresses in the virgin peat thereby causing large displacements without significantly disturbing the existing fill. By allowing the over-fill to remain as a preload/surcharge, post-construction deformations should be minimized. Preloading/surcharging would be necessary because not all of the virgin peat would be displaced where the peat thickness is relatively large.

The placement method should be such that displacement of the peat will be maximized. The most efficient method would be to employ over-filling in conjunction with blasting. However, this method may not be feasible near a travelled highway since there is a risk that some charges may be left undetonated. In any event, the highway would have to be closed temporarily. Use of blasting would be suitable if a new pavement was to be constructed along an alignment removed from the existing pavement.

If blasting is not possible, then the method shown on Figure 22.3 should be used. Displacement of the peat could be maximized and fill quantities reduced by pre-excavating some of the virgin peat. However, the magnitude of pre-excavation would have to be controlled to avoid cost inefficiencies and excessive

deformation of the existing fill with resulting distress to the existing pavement surface. In placing the fill causing displacement, emphasis should be on a continuous filling operation as opposed to a discontinuous operation even if the continuous rate of filling is lower.

The safest approach is to simply partially excavate, overfill and induce failure. Following completion of displacement, a preload/surcharge would be applied to minimize post construction deformations. A further advantage of this technique is that by utilizing large lateral overfills, the area in which future distress would occur is at some distance away from the existing pavement. If this method were chosen, construction technique would be important and the stages of construction would be as shown on the lower part of Figure 22.3. By following this procedure, the mud-wave generated would be pushed outward and away from the pavement. A disadvantage of the method is that a larger fill width will be required because of the need to maintain traffic along the existing pavement during filling and dumping operations and large quantities of fill would be required.

8.4 Berms

The use of berms to improve the stability of embankments built over deep peat deposits has been proposed by Raymond, (1969)*. The purpose of using berms is to minimize the shear stresses in the virgin peat prior to placing new fill and preload material. A schematic illustration of the method is shown on Figure 22.4 and the dimensions shown are based on Raymond's recommendations. The lower part of this sketch summarizes the placement method. Due to the low strength of the peat, corduroy mats would be necessary to permit movement of construction traffic and also as a base on which to place fill. The fill should be placed in thin lifts and work should progress from the outside in

* Raymond, G.P. (1969) "Construction Method and Stability of Embankments on Muskeg", Canadian Geotechnical Journal, 6, pp 81-96.

toward the existing pavement in order to minimize shear stresses in the virgin peat. The mud wave generated by the fill placement would have to be continually removed from the pocket between the new and existing fills.

The main purpose of this approach is to prevent failures occurring during construction. Since most of the peat adjacent to the new portion of highway would remain in place, preloading would be required and because it would probably be necessary to maintain traffic flow throughout the construction operation, preloading would have to be staged or a large enough berm width constructed to permit use of a traffic diversion. Even with the use of preloading, the significant depth of peat remaining under the fill would increase the potential for continued long term lateral and vertical deformation. Further, allowance would have to be made for continual filling of the preload as settlements occur. These settlements would be significant and in the order of about half of the thickness of the peat remaining below the fill. In addition, continual maintenance would be required to patch differential settlement cracks between the new fill and the old fill which will not settle excessively if the grade is not raised significantly.

A major problem associated with this method is the construction of the berms. It may not be possible for the very weak virgin peat to support even thin berms working from the outside in towards the pavement. Associated with this is trafficability of construction machinery on the peat. The use of filter fabrics is not viable because of the very poor subgrade support and a more rigid support system such as corduroy mats is necessary. The mats would also provide continued support for the berms thereby minimizing cracking due to spreading. Because of the difficulties outlined above and in particular, trafficability problems during construction and preload maintenance, this approach is not considered suitable for reconstruction works on Highway #11.

9. REVIEW OF PROPOSED CONSTRUCTION METHODS ON WP #9-75-04

Figure 21 summarizes the proposed treatments in those sections of the highway on WP #9-75-04 which are to be widened and are underlain by peat. It is noted that raising of the existing pavements will be restricted to a maximum of 0.4 m. At stations 11+340 (m), 18+860 (m) and 12+200 (m), no special measures are to be taken. At Station 11+340, the swamp crossing is very short and incorporates an existing culvert which could be affected by buttress construction operations. At Station 18+860, peat is reportedly present on the right side of the highway. Since widening is to take place on the left side, no special measures are required. At Station 12+200, the peat deposits are very thin (0.5 m) and again no special treatment is required. Based on the above, the proposed approach in these areas is considered acceptable. It is possible that in the thin peat sections, some peat may have been trapped below the existing fill and with the addition of up to 0.4 m of additional fill in these sections, some additional settlements may occur. However, the magnitude of these settlements will be small and they will take place in a relatively short time period, probably during construction.

It is currently proposed to adopt conventional buttress construction methods in areas of thin peat deposits, (i.e. without preloading/surcharging). Following the comments made in previous sections of this report, it is considered that this treatment is suitable in the 8 remaining areas where the peat deposits are relatively thin, (less than 2 m).

In the remaining four sections (Stations 13+090, 13+490, 15+060 and 16+120) which are underlain by deeper (3.6 - 5 m) peat deposits, it is proposed to adopt a modified buttress approach which would involve pre-excavation, displacement and surcharging. This approach is considered appropriate with the provision discussed previously that the initial excavation should not extend beyond about 2 m depth below swamp level to avoid cost ineffective excavation and risk of damage to the

existing road. For an average peat thickness of about 4.5 m, a pre-excavation depth of 2 m and displacement to a minimum of 1 m below the pre-excavation level, it is considered that a preload/surcharge of about 1 m above final grade would result in 0.25 to 0.5 m of settlement in about 4 - 8 weeks. When the surcharge is removed, future settlements should be relatively small. However, it is strongly recommended that proper monitoring of settlements be carried out during the preload/surcharge period to ensure completion of primary consolidation under the preload fill.

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APPENDIX I
FIELD WORK PROCEDURES

February, 1982

811-1291

Golder Associates

The field work for this investigation was carried out in two stages, September 5 to 16, 1981 and October 6 to 9, 1981. During these periods a total of 16 sampled boreholes with a cumulative depth of about 120 m of drilling, and 12 vane test boreholes were put down at the locations shown on Figures 1 to 4. These locations were decided upon following discussions with M.T.C. staff both in Toronto and in the field. It had originally been intended to perform the entire investigation in a single period. However, because of difficulties encountered in advancing the boreholes with the relatively small drilling machine, available at the initiation of the field work, the drilling program was halted temporarily until a more powerful machine became available. The work was completed successfully using this machine in the second stage of the investigation.

The boreholes were advanced to depths of between 0.9 and 10.4 m below ground surface using either a CME 45 B bombardier mounted power auger drillrig or a CME 750 self-propelled power auger drillrig both supplied by local contractors. In both cases the holes were advanced either by augering or, in areas where augering was not possible, by running BW size casing and using wash boring techniques. Drilling methods employed are shown on the individual Record of Borehole sheets, following the text of this report.

Samples were generally obtained at 0.76 m intervals of depth although the sampling frequency was modified in many cases depending on the conditions encountered. These samples were obtained using a conventional 50 mm O.D. split barrel sampler in conjunction with Standard Penetration tests. Relatively undisturbed samples were obtained using 50 mm and 75 mm diameter thin-walled Shelby tube samplers.

Information on the undrained shear strengths of the peat and cohesive overburden materials was obtained using in-situ vanes. In the boreholes advanced by running casing, only the BX size vane was used. To supplement the vane test results from the boreholes, field vane tests were carried out at several locations using two additional vane sizes together with the conventional NX and BX size vanes. One of these additional vanes was a conventional straight vane 88 mm in diameter and 176 mm long while the other was a diamond shaped vane 88 mm in diameter and 88 mm long.

Groundwater levels were monitored at the sites by sealing standpipe piezometers into selected boreholes. These installations were left intact after completion of the field work to permit periodic monitoring of groundwater level fluctuations if required.

The field work was supervised on a full time basis by a member of our engineering staff , who directed the drilling, sampling and testing operations, logged the boreholes and cared for the samples obtained. Following completion of the field work, the samples were brought to our laboratory for detailed examination and testing.

The as-drilled borehole locations were obtained by our field engineer. The elevations were obtained by M.T.C. survey staff and have been referenced to Geodetic datum. The ground surface profiles shown on the cross-sections on each of Figures 1 to 4 were also obtained by M.T.C. survey staff.

EXPLANATION OF TERMS USED IN REPORT

'N' VALUE: AN INDICATOR OF SUBSOIL QUALITY. IT IS OBTAINED FROM THE STANDARD PENETRATION TEST (CSA STD. A119.1). SPT 'N' VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 2 INCH O.D. SPLIT-BARREL SAMPLER TO PENETRATE 12 INCHES INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WEIGHING 140 POUNDS, FALLING FREELY A DISTANCE OF 30 INCHES. FOR PENETRATIONS OF LESS THAN 12 INCHES 'N' VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. 'N' VALUES CORRECTED FOR OVERBURDEN PRESSURE ARE DENOTED THUS N_c .

DYNAMIC CONE PENETRATION TEST (CSA STD. A119.3): CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (2" O.D. 60 CONE ANGLE) DRIVEN BY 350 FT-LB IMPACTS ON "A" SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 12 INCH ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOIL QUALITY: SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSITY.

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

S_u (PSF)	0 - 250	250 - 500	500 - 1000	1000 - 2000	2000 - 4000	> 4000
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

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

'N' (BLOW/FT)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCK QUALITY: 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 DRILLED IN THAT CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE NATURALLY FRACTURED CORE PIECES, 4" IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

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

JOINTING AND BEDDING:

SPACING	2"	2" - 12"	1' - 3'	3' - 10'	> 10'
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS & SYMBOLS

LABORATORY TESTING

TRIAxIAL TESTS ARE DESCRIBED IN TERMS OF WHETHER THEY ARE CONSOLIDATED (C) OR NOT (U) ISOTROPICALLY (I) OR NOT (A) AND SHEARED DRAINED (D) OR UNDRAINED (U) WITH PORE PRESSURE MEASUREMENTS (BAR OVER SYMBOLS) EG. $C\bar{U}$ = CONSOLIDATED ISOTROPIC UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENT UNLESS OTHERWISE SPECIFIED IN REPORT ALL TESTS ARE IN COMPRESSION

FIELD SAMPLING

SS SPLIT SPOON
WS WASH SAMPLE
ST SLOTTED TUBE SAMPLE
BS BLOCK SAMPLE
CS CHUNK SAMPLE
TW THINWALL OPEN
TP THINWALL PISTON
OS OSTERBERG SAMPLE
FS FOIL SAMPLE
RC ROCK CORE
PH T.W. ADVANCED HYDRAULICALLY
FM T.W. ADVANCED MANUALLY

EARTH PRESSURE TERMS

μ COEFFICIENT OF FRICTION
 δ ANGLE OF WALL FRICTION
 k_o COEFFICIENT OF EARTH PRESSURE AT REST
 k_A COEFFICIENT OF ACTIVE EARTH PRESSURE
 k_P COEFFICIENT OF PASSIVE EARTH PRESSURE
 i ANGLE OF INCLINATION OF SURCHARGE
 w SLOPE ANGLE-BACKFACE OF WALL
 β ANGLE OF SLOPE
 N, N_q, N_c BEARING CAPACITY FACTORS
 D_f DEPTH OF FOOTING
 B, L FOOTING DIMENSIONS

INDEX PROPERTIES

γ UNIT WEIGHT OF SOIL (BULK DENSITY)
 γ_w UNIT WEIGHT OF WATER
 γ_d UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
 γ' UNIT WEIGHT OF SUBMERGED SOIL
 G_s SPECIFIC GRAVITY OF SOLIDS
 e VOIDS RATIO
 e_o INITIAL VOIDS RATIO
 e_{max} e IN LOOSEST STATE
 e_{min} e IN DENSEST STATE
 D_r RELATIVE DENSITY = $\frac{e_{max} - e}{e_{max} - e_{min}}$
 n POROSITY
 w WATER CONTENT
 w_L LIQUID LIMIT
 w_p PLASTIC LIMIT
 w_s SHRINKAGE LIMIT
 I_p PLASTICITY INDEX = $w_L - w_p$
 I_L LIQUIDITY INDEX = $\frac{w - w_p}{w_L - w_p}$
 I_c CONSISTENCY INDEX = $\frac{w_L - w_p}{w_L - w_{shrinkage}}$
 A_c ACTIVITY = $\frac{I_p}{w_L - w_{shrinkage}}$
 O_m ORGANIC MATTER CONTENT
 S_r DEGREE OF SATURATION
 S SENSITIVITY = $\frac{S_o (undisturbed)}{S_o (remoulded)}$

STRENGTH PARAMETERS

ϕ ANGLE OF SHEARING RESISTANCE
 τ_f PEAK SHEAR STRENGTH
 τ_R RESIDUAL SHEAR STRENGTH
 c COHESION INTERCEPT
 $\sigma_1, \sigma_2, \sigma_3$ NORMAL PRINCIPAL STRESSES
 u PORE WATER PRESSURE
 u_e EXCESS u
 r_u PORE PRESSURE RATIO
 q_u UNCONFINED COMPRESSIVE STRENGTH
 s_u UNDRAINED SHEAR STRENGTH
 ϵ LINEAR STRAIN
 γ SHEAR STRAIN
 ν POISSON'S RATIO
 E MODULUS OF ELASTICITY
 G MODULUS OF SHEAR DEFORMATION
 k_s MODULUS OF SUBGRADE REACTION
 m, n STABILITY COEFFICIENTS
 A, B PORE PRESSURE COEFFICIENTS

NOTE: EFFECTIVE STRESS PARAMETERS ARE DENOTED BY USE OF APOSTROPHE ABOVE THE SYMBOL, THUS:
 σ' = EFFECTIVE ANGLE OF SHEARING RESISTANCE;
 σ'_n = EFFECTIVE NORMAL STRESS

HYDRAULIC TERMS

h HYDRAULIC HEAD OR POTENTIAL
 q RATE OF DISCHARGE
 v VELOCITY OF FLOW
 i HYDRAULIC GRADIENT
 j SEEPAGE FORCE PER UNIT VOLUME
 η COEFFICIENT OF VISCOSITY
 k COEFFICIENT OF HYDRAULIC CONDUCTIVITY
 k_h k IN HORIZONTAL DIRECTION
 k_v k IN VERTICAL DIRECTION
 α_v COEFFICIENT OF VOLUME CHANGE
 c_v COEFFICIENT OF CONSOLIDATION
 C_c COMPRESSION INDEX
 C_r RECOMPRESSION INDEX
 d DRAINAGE PATH DISTANCE
 T_v TIME FACTOR
 U DEGREE OF CONSOLIDATION
 O_c OVERCONSOLIDATION RATIO (OCR)

RECORD OF BOREHOLE No 1

W P Contract No. 78-56 LOCATION Sta. 445 + 00 o/s 55' Lt. @ Hwy.11 ORIGINATED BY SWC
DIST 19 HWY 11 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SWC
DATUM Geodetic DATE September 10, 1981 CHECKED BY SWC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
435.8	Ground Level																
0.0	Fine fibrous peat. Very Soft Dark Brown to Black.						435										
434.6			1	SS	PM												
1.2	Amorphous peat, some fine fibres & wood fragments.						434										
			2	SS	PM												
							433										
			3	SS	PM												
	Very Soft						432										
			4	CS													
	Dark Brown to Black						431										
			5	SS	PM												
430.0							430										
5.8	Silt some clay to clayey silt.						429										
	Very Soft to Soft Grey		6	SS	PM												
427.9							428										
7.9	Silty fine to medium sand.		7	SS	PM												
427.5	Very Loose Grey																
8.3	End of Borehole Auger Refusal																

RECORD OF BOREHOLE No 3

W P Contract No. 78-56 LOCATION Sta. 445 + 00 o/s 8' Lt. of Hwy. 11 ORIGINATED BY SWC
 DIST 19 HWY 11 BOREHOLE TYPE Hollow Stem Auger & Wash Boring COMPILED BY SWC
 DATUM Geodetic DATE October 6, 1981 CHECKED BY SWC

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					
437.6	Road Level																
0.0	Asphalt																
0.1	Fill - gravelly fine to coarse sand. Compact						437										
436.4																	
1.2							436										
	Rockfill.						435										
							434										
							433										
							432										
							431										
430.4																	
7.2	Borehole Abandoned																


RECORD OF BOREHOLE No 4

W P Contract No. 78-56 LOCATION Sta. 445 + 00 o/s 110' Lt. @ Hwy.11 ORIGINATED BY SWC
 DIST 19 HWY 11 BOREHOLE TYPE Hand Auger COMPILED BY SWC
 DATUM Geodetic DATE October 9, 1981 CHECKED BY SWC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y kN m ⁻³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
435.9	Ground Level																
0.0	Fine fibrous to amorphous peat, some wood fragments. Soft Brown						435										
							434										
433.2			1	TW	PM									0		9.48 10.67 Org. 90%	P _s 1800
2.7	End of Borehole																

RECORD OF BOREHOLE No 10

W P Contract No. 78-56 LOCATION Sta. 622 + 46' o/s 47' Rt. @ Hwy.11 ORIGINATED BY SWC
DIST 19 HWY 11 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SWC
DATUM Geodetic DATE September 11, 1981 CHECKED BY SWC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN m ⁻³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE										500 1000 1500		
							20	40	60	80	100									
440.3	Ground Level						440													
0.0	Fine fibrous peat, some wood fragments.	}					440													
	Very Soft		1	SS	1															
438.0	Dark Brown to Black	}	2	SS	PM	▽	439													
2.3	Amorphous peat, trace of fibres & wood fragments.	}					438													
434.2	Soft	}	3	SS	2		437													
	Dark Brown to Black																			
6.1	Sandy silt, some organics.	}					436													
	Very Loose																			
432.8	Grey	}	4	SS	PM		435										Org. 83%			
7.5	Silt, trace to some clay.	}					434										19.24			
431.6	Stiff Grey	}	5	SS	1		433										1 1 74 24			
8.7	End of Borehole Auger Refusal	}	6	TW	PH		432													

+3, x5: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 11

W P Contract No. 78-56 LOCATION Sta. 620 + 88 o/s 50' Lt. 6 Hwy. 11 ORIGINATED BY SWC
 DIST 19 HWY 11 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SWC
 DATUM Geodetic DATE September 12, 1981 CHECKED BY S.W.C.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ KN m ⁻³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100	Wp	W	Wl		
440.6	Ground Level																
0.0	Fine fibrous to amorphous peat, some silt, sand, gravel & wood fragments.		1	SS	5		440										
439.3	Soft Dark Brown																
1.3	Amorphous peat, trace to some fine fibres, trace wood fragments.		2	SS	2		439										
	Very Soft to Soft						438										
			3	SS	PM		437									Org. 91%	p _s 1870
	Dark Brown to Black						436										
			4	TW	PM		435									10.45 Org. 72%	p _s 1550
434.2			5	TW	PH		434										
6.4	Silty fine sand.																
433.6	Dense Grey		6	SS	34/												
7.0	End of Borehole Auger Refusal				0.15												

+3, x5 : Numbers refer to
Sensitivity

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

RECORD OF BOREHOLE No 12

W P Contract No. 78-56 LOCATION Sta. 622 + 46 o/s 7.2' Rt. & Hwy. 11 ORIGINATED BY SWC
DIST 19 HWY 11 BOREHOLE TYPE Hollow Stem Auger & Wash Boring COMPILED BY SWC
DATUM Geodetic DATE October 7, 1981 CHECKED BY SWC

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					
441.6	Road Level																
0.0	Asphalt																
0.1	Fill - gravelly fine to coarse sand.																
	Compact																
438.5																	
3.1																	
	Rockfill.																
434.3																	
7.3	Amorphous peat, trace fine fibres & wood fragments.		1	SS	11												
433.6	Firm Dark Brown		2	TW	PH												
8.0	Silty fine sand to silt, trace gravel.																
432.3	Dense Grey		3	SS	37												
9.3	End of Borehole																

+3, x5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 14

W P Contract No. 78-56 LOCATION Sta. 622 + 46 o/s 16.3' Rt. of Hwy. 11 ORIGINATED BY SWC
DIST 19 HWY 11 BOREHOLE TYPE Hollow Steam Auger COMPILED BY SWC
DATUM Geodetic DATE October 9, 1981 CHECKED BY SWC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ KN m ⁻³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
441.6	Road Level																
0.0	Fill - gravelly fine to coarse sand.						441										
							440										
	Compact						439										
438.6							438										
3.0	Rockfill.						437										
							436										
436.3	Amorphous peat, trace fine fibres and wood fragments.		1	TW	PH		435									10.04 Org. 90%	p _s 1410
			2	TW	PH		434									12.26 Org. 79%	p _s 1510
	Soft to Stiff Brown		3	TW	PH		433									10.05 Org. 90%	p _s 1510
433.1			4	TW	PH		432										
8.5	Silt, some fine sand grading to silty fine to coarse sand, some gravel.		5	SS	29												0 4 81 15
432.0	Compact Grey																
9.6	End of Borehole																

RECORD OF BOREHOLE No 15

W P Contract No. 78-56 LOCATION Sta. 622 + 46 o/s 101.3' Lt. of Hwy. 11 ORIGINATED BY SWC
 DIST 19 HWY 11 BOREHOLE TYPE Hand Auger COMPILED BY SWC
 DATUM Geodetic DATE October 9, 1981 CHECKED BY SWC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ KN m ⁻³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
440.0	Ground Level																
0.0	Fine fibrous to amorphous peat, some wood fragments. Soft Brown						439										
							438										
437.3			1	TW	PM									0		10.08 Org. 92%	ρ _s 1820
2.7	End of Borehole																

RECORD OF BOREHOLE No 20

W P 9-75-04 LOCATION Sta. 13 + 565 o/s 9.3 m Rt. @ Hwy. 11 ORIGINATED BY SWC
 DIST 19 HWY 11 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SWC
 DATUM Geodetic DATE September 12, 1981 CHECKED BY SWC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN m ⁻³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
483.2	Ground Level																
0.0	Amorphous peat, some fine fibres & wood fragments. Very Soft Dark Brown to Black		1	SS	PM		483									Org. 88 %	ρ _s 1820
			2	SS	1		482										
							481									9.97 10.77 Org. 81% 84%	ρ _s 2020
			3	TW	PH		480										
478.9							479										
4.3	Sandy silt to silty sand trace to some gravel. Dense to Very Dense		4	SS	66/ 0.23									20%			
478.2	Grey																
5.0	End of Borehole Auger Refusal																

RECORD OF BOREHOLE No 21

W P 9-75-04 LOCATION Sta. 13 + 510 o/s 8.1 m Rt. @ Hwy.11 ORIGINATED BY SWC
 DIST 19 HWY 11 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SWC
 DATUM Geodetic DATE September 12, 1981 CHECKED BY SWC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN m ⁻³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
483.0	Ground Level																
0.0	Fine fibrous peat, some wood fragments. Soft		1	SS	PM		482										
481.6	Dark Brown																
1.4	Amorphous peat, trace fine fibres & wood fragments.		2	SS	PM		481										
	Very soft						480									10.39	0 32 54 14
	Dark Brown to Black		3	TW	PM		479	+									
							478	+								Org. 81%	ρ _s 1930
476.9			4	SS	PM		477	+									
6.1	Silt, trace to some fine sand.		5	SS	15		476										
476.0	Compact Grey																
7.0	Sand, trace silt, trace to some fine gravel.																
474.9	Loose to Compact		6	SS	7		475										5 83 11 1
8.1	End of Borehole Sampler Refusal																

+3, x5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 22

W P 9-75-04 LOCATION Sta. 13 + 365 o/s 0.5 m Lt. of Hwy. 11 ORIGINATED BY SWC
 DIST 19 HWY 11 BOREHOLE TYPE Hollow Stem Auger & Wash Boring COMPILED BY SWC
 DATUM Geodetic DATE October 8, 1981 CHECKED BY SWC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100														
								SHEAR STRENGTH										WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE										500 1000 1500				
484.2	Road Level						484															
0.0	Asphalt																					
0.2	Fill - gravelly fine to coarse sand.																					
483.3	Compact																					
0.9	Rockfill.						483															
							482															
							481															
							480															
							479.8															
4.4	Amorphous peat, trace fine fibres & wood fragments.		1	SS	26		479															
	Very Soft		2	SS	PM																	
							478															
	Brown		3	SS	PH																	
477.2			4	SS	PH																	
7.0	Silt, trace clay.						477															
476.5	Compact Grey																					
7.7	End of Borehole Sampler Refusal																					

RECORD OF BOREHOLE No 30

W P 9-75-04 LOCATION Sta. 15 + 140 o/s 7.7 m Rt. of Hwy. 11 ORIGINATED BY SWC
 DIST 19 HWY 11 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SWC
 DATUM Geodetic DATE September 13, 1981 CHECKED BY SWC

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	WATER CONTENT (%)	UNIT WEIGHT γ kN m ⁻³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100						
483.5	Ground Level																	
0.0	Fine fibrous peat. Very Soft						483											
482.1	Dark Brown		1	SS	PM											2385		
1.4	Amorphous peat, trace to some fine fibres & wood fragments. Soft Dark Brown		2	TW	PH		482											
478.9			3	SS	PM		481											
4.6	Silt, some fine sand.		5	TW	PH		480											
478.3	Compact Grey		6	SS	42		479											
5.2	Silty fine sand grading to gravelly sand.						478											
477.4	Dense Grey																	
6.1	Boulders. (inferred)						477											
476.5																		
7.0	End of Borehole Auger Refusal																	

+3, x5 : Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 31

W P 9-75-04 LOCATION Sta. 15 + 276 o/s 26.0 m Rt. 6 Hwy. 11 ORIGINATED BY SWC
 DIST 19 HWY 11 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SWC
 DATUM Geodetic DATE September 14, 1981 CHECKED BY SWC

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					
483.2	Ground Level																
0.0	Fine fibrous peat.						483										
	Very Soft						482										
	Dark Brown		1	TW	PH		481										
480.5																	
2.7	Amorphous peat, some fine fibres & wood fragments.		2	SS	PM		480										
	Very Soft																
479.2	Dark Brown						479										
4.0	Silty fine sand																
	Loose to Compact		3	TW	PH		478										
477.7	Grey																
5.5	Boulders																
477.1	(inferred)																
6.1	End of Borehole Auger Refusal																

+3, x5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 32

W P 9-75-04 LOCATION Sta. 15 + 271 o/s 7.3 m Rt. 11 Hwy. 11 ORIGINATED BY SWC
 DIST 19 HWY 11 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SWC
 DATUM Geodetic DATE September 14, 1981 CHECKED BY SWC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN m ⁻³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100										
								SHEAR STRENGTH					WATER CONTENT (%)					
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE					500 1000 1500					
483.4	Ground Level																	
0.0	Fine fibrous to amorphous peat, trace wood fragments. Soft Brown	}} }} }} }} }} }}	1	TW	PM		483										9.23 Org. 92% 9.83 9.36 Org. 92% Org. 90%	ρ _s 1730 ρ _s 1840 ρ _s 1820
			2	TW	PH													
			3	TW	PH													
			4	TW	PH													
			5	TW	PH													
479.4																		
4.0	Silty fine sand, trace to some gravel. Very Dense Grey	}} }} }} }} }} }}	6	TW	PH		479											
			7	SS	PM													
			8	SS	PM													

RECORD OF BOREHOLE No 33

W P 9-75-04 LOCATION Sta. 15 + 271 o/s 0.65 m Lt. @ Hwy.11 ORIGINATED BY SWC
 DIST 19 HWY 11 BOREHOLE TYPE Hollow Stem Auger & Wash Boring COMPILED BY SWC
 DATUM Geodetic DATE September 15 & 16, 1981 CHECKED BY SWC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN m ⁻³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
485.2	Road Level																
0.0	Asphalt																
0.1	Fill - medium to coarse sand, some gravel.																
484.0	Compact																
1.2																	
	Rockfill.																
480.0																	
5.2	Fibrous peat.																
	Soft Brown																
478.8			1	TW	PH												
6.4	Silty fine sand to sandy silt grading to medium sand some gravel.																
	Compact Grey		2	TW	PH												
476.7			3	SS	19												
8.5	End of Borehole																

+3, x5: Numbers refer to
Sensitivity

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

OFFICE REPORT ON SOIL EXPLORATION

METRIC

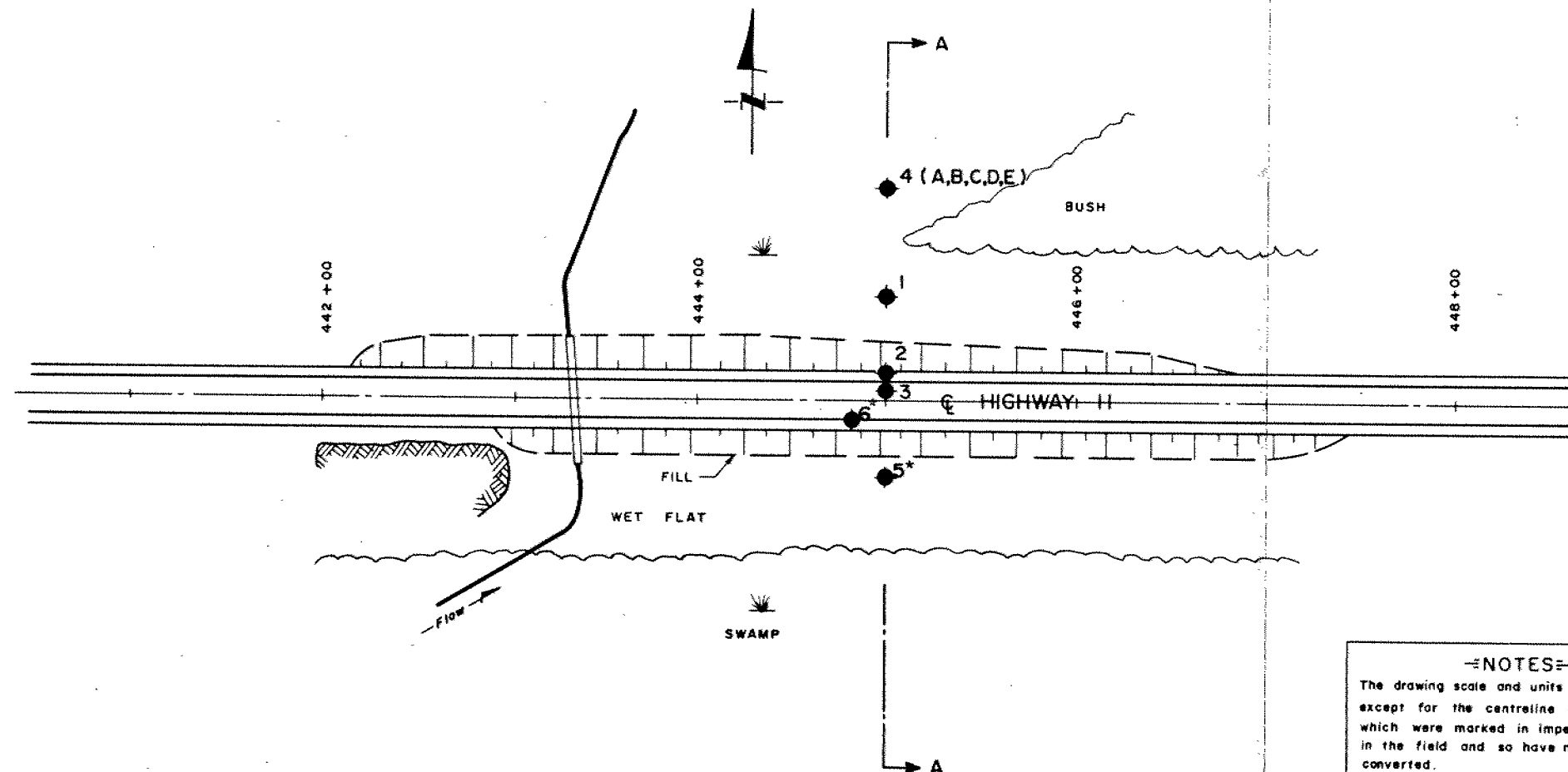
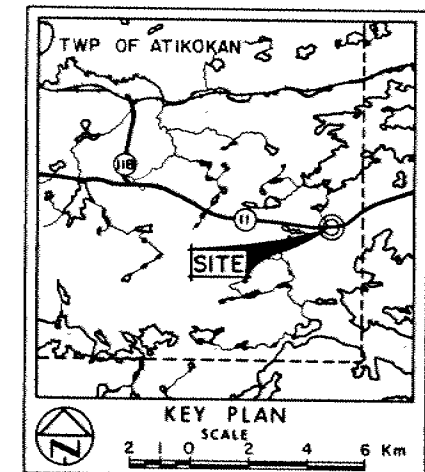
CONT No 78 - 56
WP No



SITE A
BORE HOLE LOCATIONS & SOIL STRATA

FIGURE
1

GOLDER ASSOCIATES



NOTES

The drawing scale and units are metric except for the centreline chainages which were marked in imperial units in the field and so have not been converted.

Off set distances are quoted in both imperial and metric units.

Boreholes shown with an asterisk, 5* etc, were put down during an earlier investigation carried out by M.T.C. staff, the information from which is shown on soils profile No. 11TB19-47A. The elevations of these boreholes are approximate.

LEGEND			
◆	Bore Hole		
⊕	Dynamic Cone Penetration Test (Cone)		
◆	Bore Hole & Cone		
N	shows 0.3m (51d Pen Test, 475 l/blow)		
CONE	shows 0.3m (60° Cone, 475 l/blow)		
W	W.L. at time of investigation Oct 1981		
⊥	PIEZOMETER		

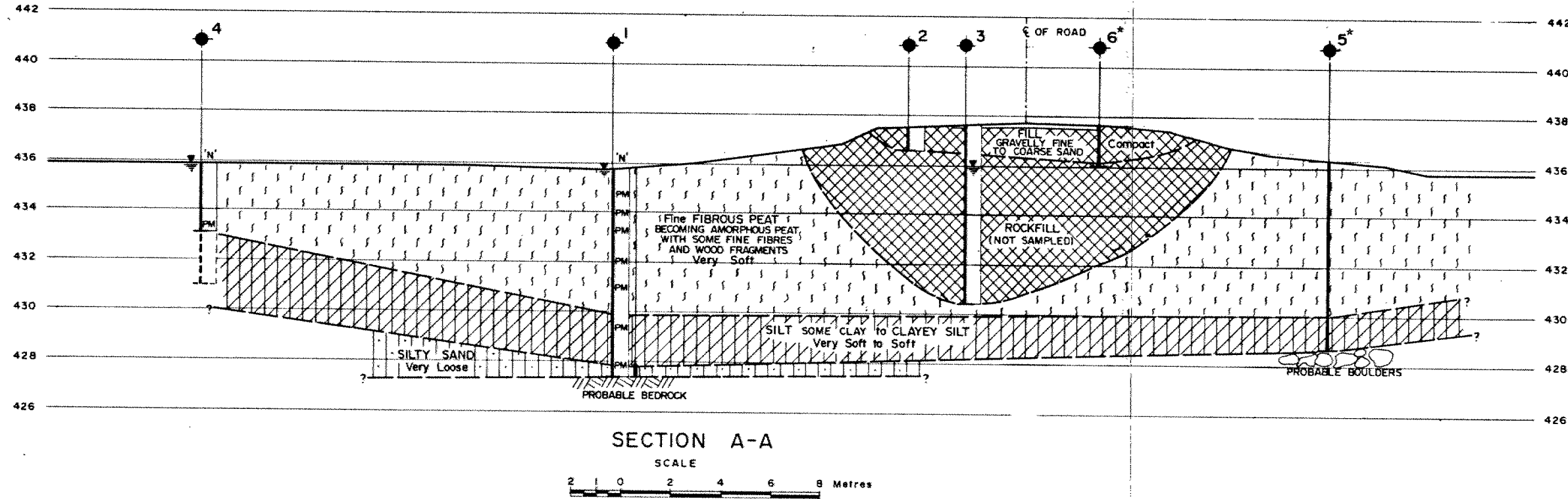
No	ELEVATION	STATION	OFF - SET
1	435.75m	445+00	168m(55ft) L
2	437.44m	445+00	49m(16ft) L
3	437.60m	445+00	24m(8ft) L
4	435.90m	445+00	335m(110ft) L
5*	436.30m	445+00	122m(40ft) R
6*	437.70m	444+80	30m(10ft) R

NOTE

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

DATE	BY	DESCRIPTION

Geocres No	
HWY No 11	DIST 19
SUBWD	CHECKED
DATE FEB. 22, 1982	SITE ATIKOKAN
DRAWN	CHECKED
APPROVED	DWG



METRIC

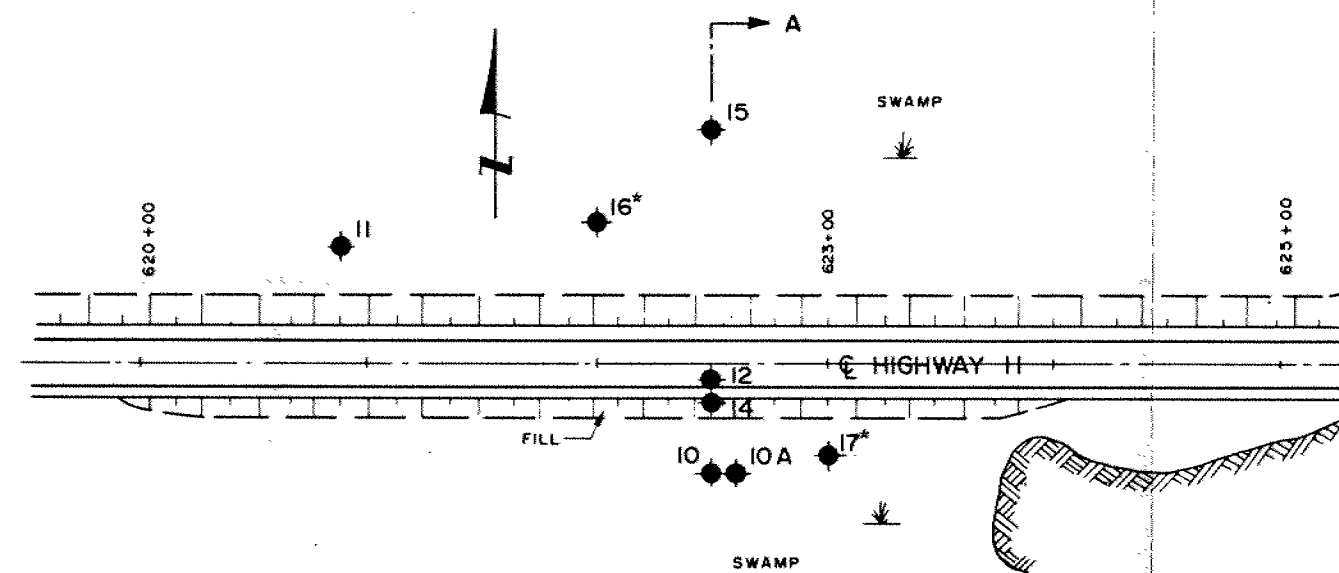
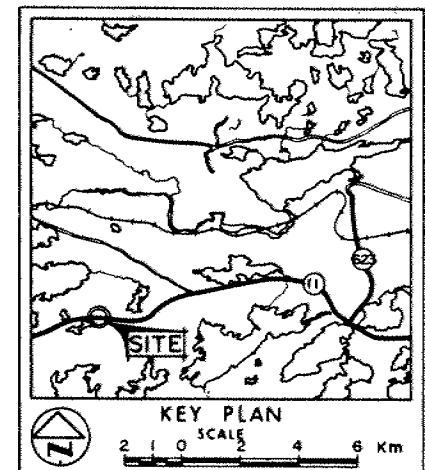
CONT No 78-56
WP No



SITE B
BORE HOLE LOCATIONS & SOIL STRATA

FIGURE
2

GOLDER ASSOCIATES

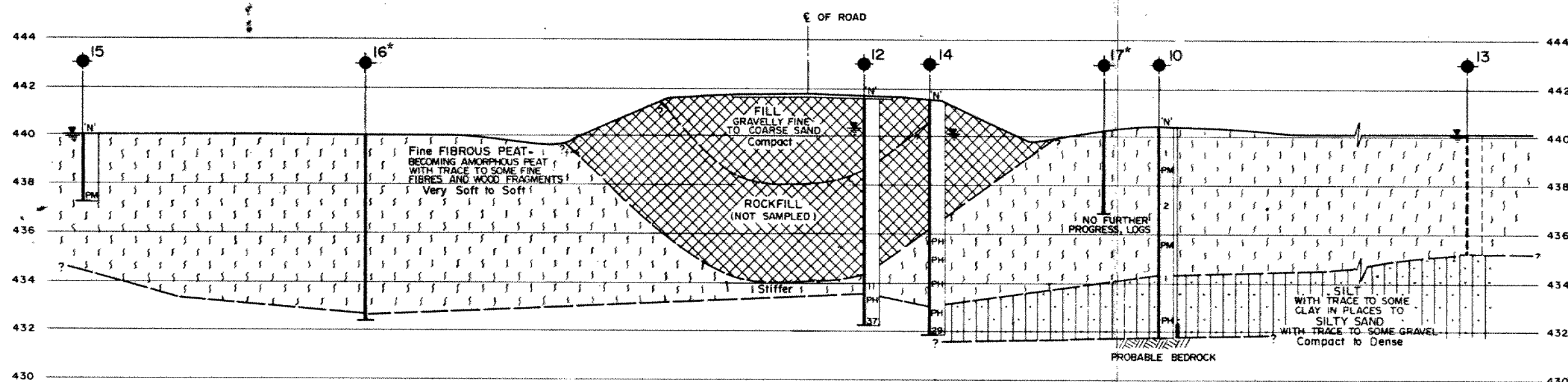
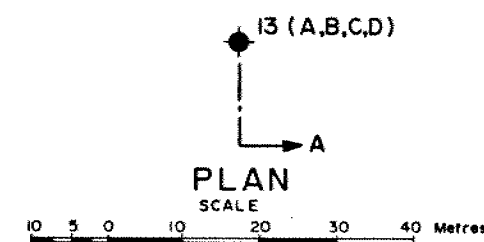


NOTES

The drawing scale and units are metric except for the centreline chainages which were marked in imperial units in the field and so have not been converted.

Off set distances are quoted in both imperial and metric units.

Boreholes shown with an asterisk (16* etc) were put down during an earlier investigation carried out by M.T.C. staff, the information from which is shown on soils profile No 11TB19-47A. The elevations of these boreholes are approximate.



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)
- ⊕ Wt. at time of investigation Oct 1981
- ⊕ PIEZOMETER

No	ELEVATION	STATION	OFF-SET
10	440.34m	622 + 46	14.3m(47ft) R
10A	440.34m	622 + 56	14.3m(47ft) R
11	440.55m	620 + 88	15.2m(50ft) L
12	441.64m	622 + 46	2.2m(7.2ft) R
13	440.10m	622 + 46	14.4m(300ft) R
14	441.61m	622 + 46	4.9m(16.3ft) R
15	440.04m	622 + 46	30.9m(101.3ft) L
16*	440.04m	622 + 00	18.3m(60ft) L
17*	442.00m	623 + 00	12.2m(40ft) R

NOTE

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

DATE	BY	DESCRIPTION

Geacres No

HWY No 11	CHECKED	DATE FEB. 22, 1982	DIST 19
SUBMD	CHECKED	APPROVED	SITE ATIKOKAN
DRAWN RL	CHECKED	APPROVED	DWG

METRIC

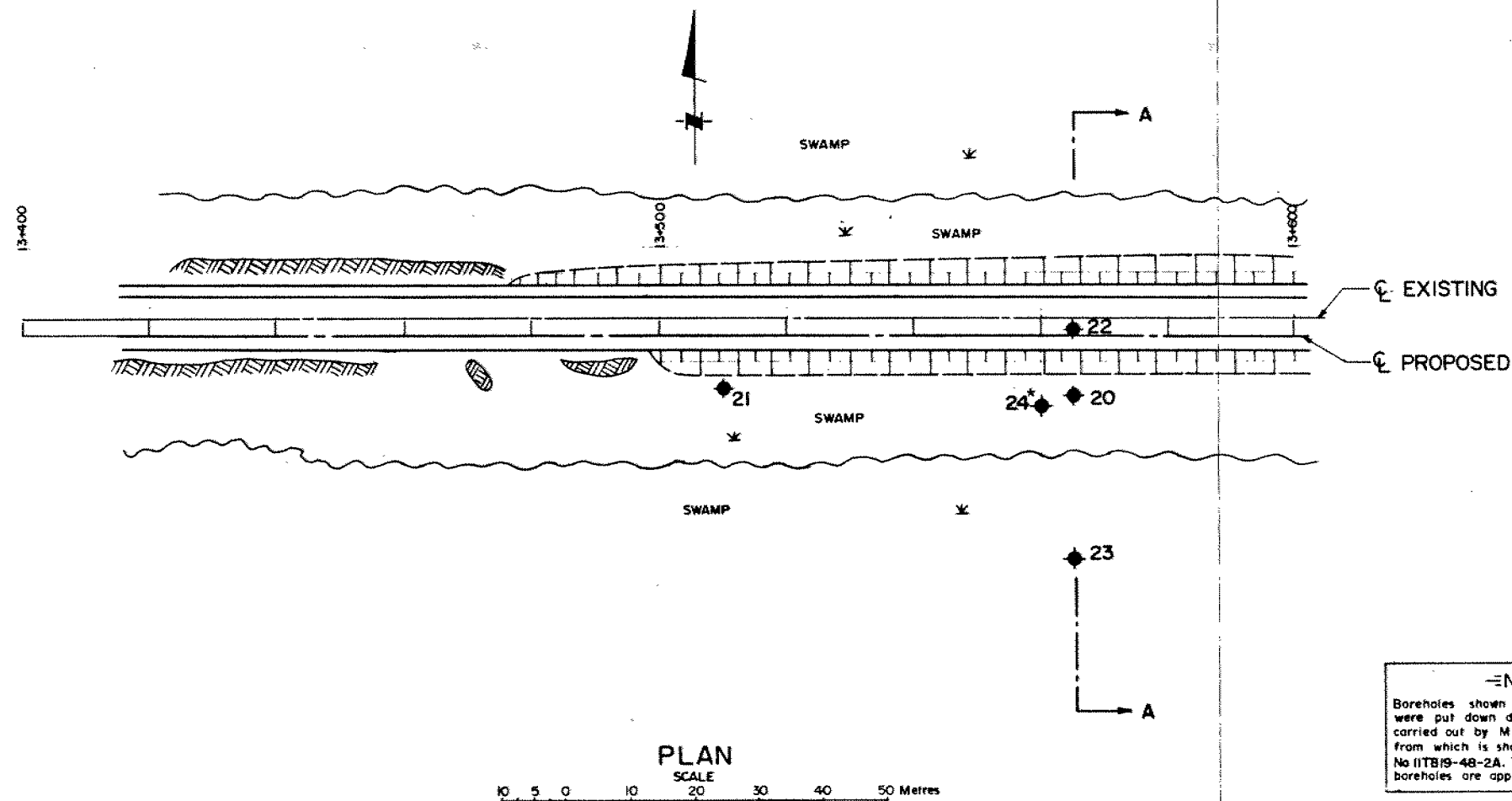
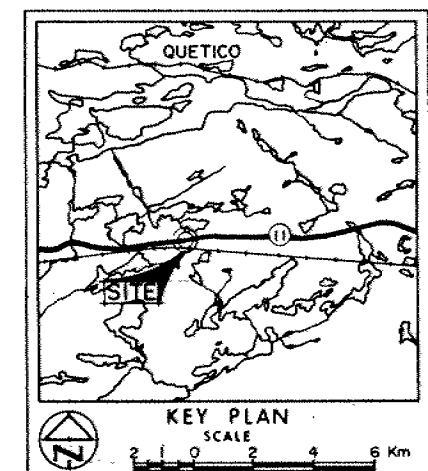
CONT No
WP No 9-75-04

SITE C
BORE HOLE LOCATIONS & SOIL STRATA



FIGURE
3

Golder Associates



NOTE
Boreholes shown with an asterik, 24* etc were put down during an earlier investigation carried out by M.T.C. staff, the information from which is shown on soils profile No IITB19-48-2A. The elevations of these boreholes are approximate.

LEGEND

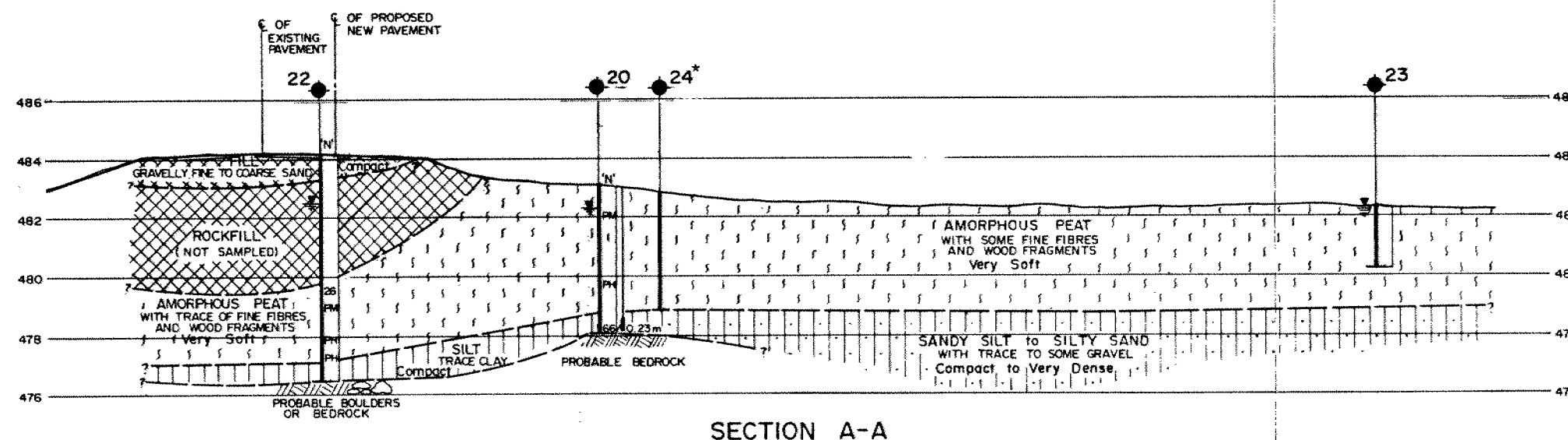
- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ◆ Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- W L at time of investigation Oct 1981
- PIEZOMETER

No	ELEVATION	STATION	OFF-SET
20	483.17 m	13+565	9.3 m R
21	483.01 m	13+510	8.1 m R
22	484.17 m	13+565	0.5 m L
23	482.37 m	13+565	35 m R
24*	482.85 m	13+560	11 m R

NOTE
OFF-SETS ARE FROM PROPOSED

NOTE

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



SCALE 0 2 4 6 8 10 Metres

DATE	BY	DESCRIPTION

Geocres No

HWY No 11	CHECKED	DATE FEB. 22, 1982	DIST 19
SUBWD	CHECKED	APPROVED SWC	SITE ATIKOKAN
DRAWN MV	CHECKED		DWG

METRIC

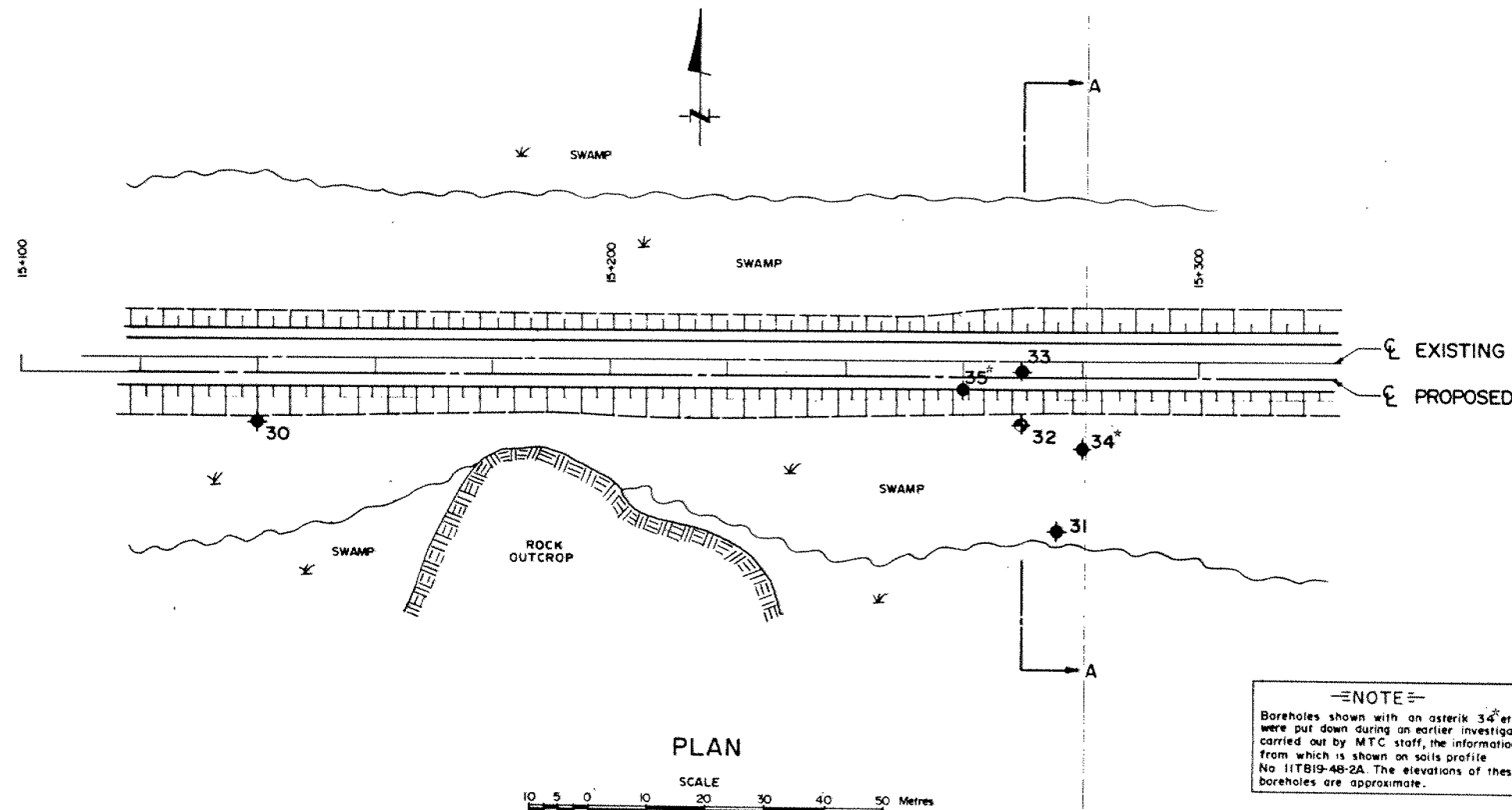
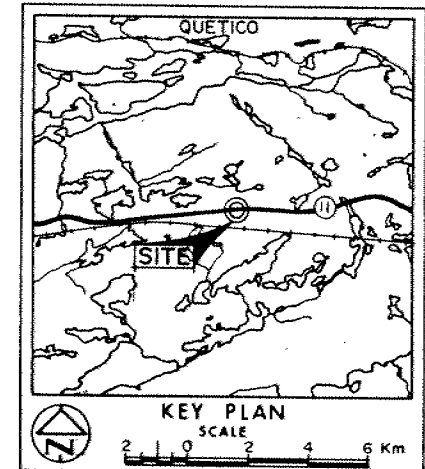
CONT No
WP No 9-75-04



SITE D
BORE HOLE LOCATIONS & SOIL STRATA

FIGURE
4

Golder Associates



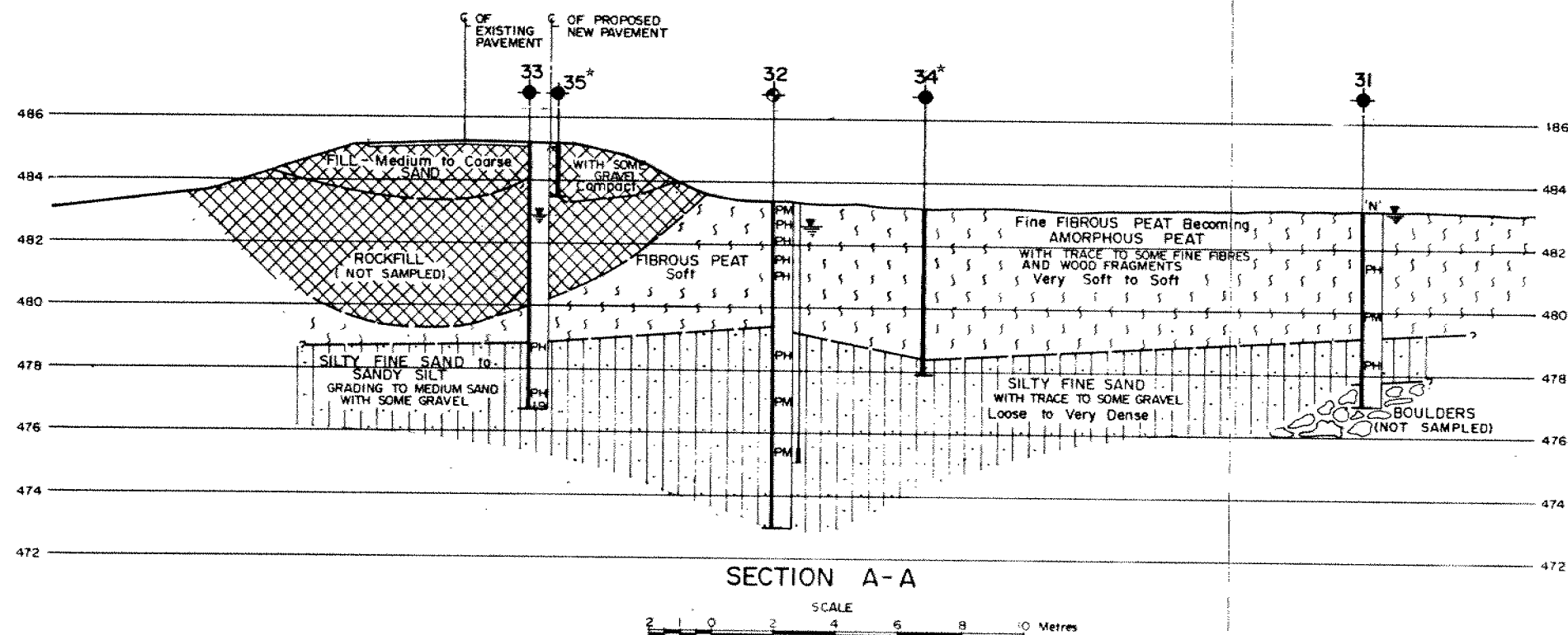
NOTE
Boreholes shown with an asterisk 34* etc were put down during an earlier investigation carried out by MTC staff, the information from which is shown on soils profile No 11TB19-48-2A. The elevations of these boreholes are approximate.

LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ◆ Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 1/blow)
- CONE Blows/0.3m (60° Cone, 475 1/blow)
- W L at time of investigation Sept 1981
- PIEZOMETER

Na	ELEVATION	STATION	OFF-SET
30	483.50m	15+140	7.7 m R
31	483.21m	15+276	26.0m R
32	483.39m	15+271	7.3 m R
33	485.21m	15+271	0.65m L
34*	483.20m	15+280	12.0m R
35	485.20m	15+260	0.2m R

NOTE
OFF-SETS ARE FROM PROPOSED

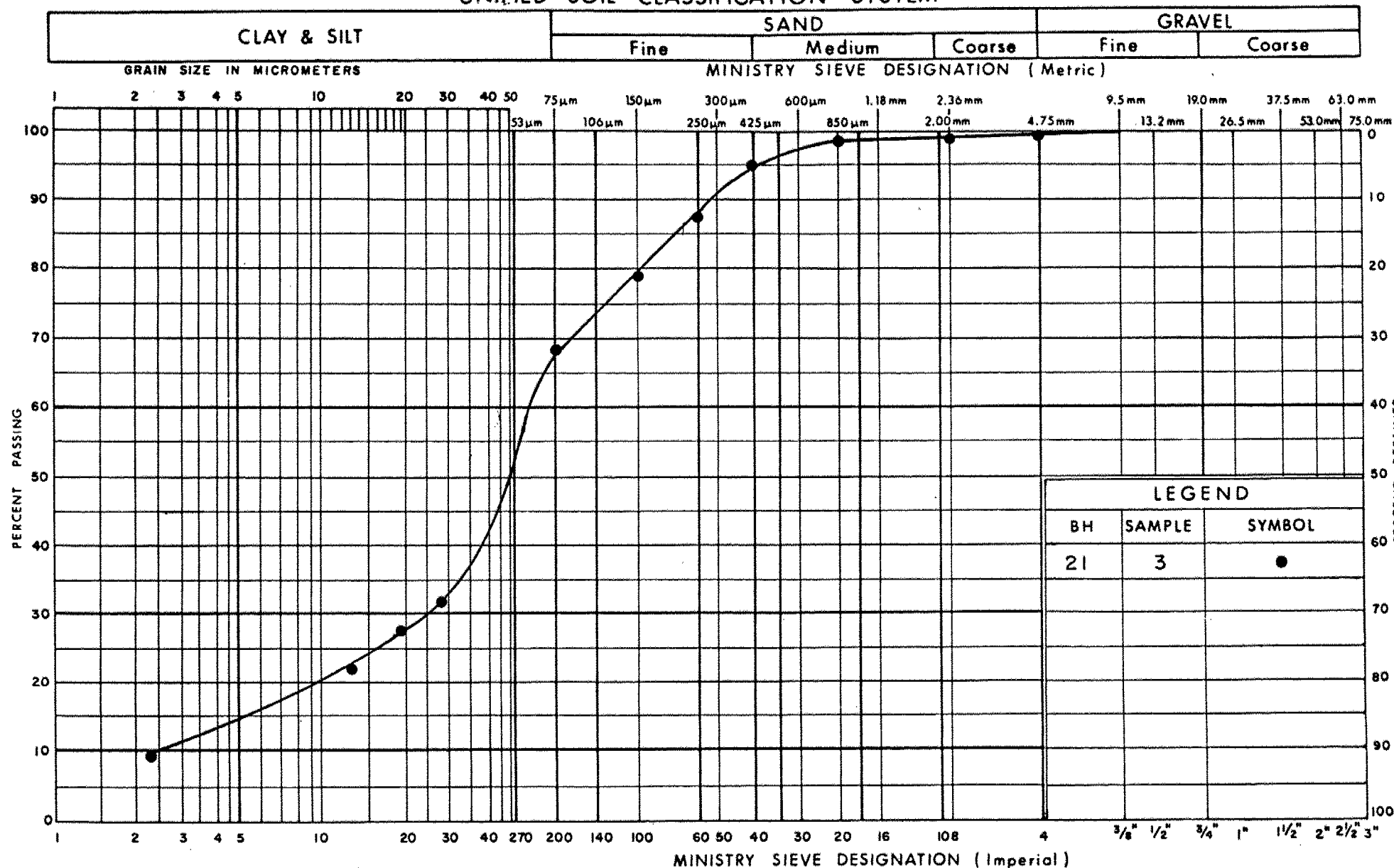


NOTE
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

REVISIONS	DATE	BY	DESCRIPTION

Geocres No			
HWY No 11			DIST 19
SUBMD	CHECKED	DATE FEB. 22, 1982	SITE ATIKOKAN
DRAWN	MV: CHECKED	APPROVED	OWG

UNIFIED SOIL CLASSIFICATION SYSTEM



Ontario

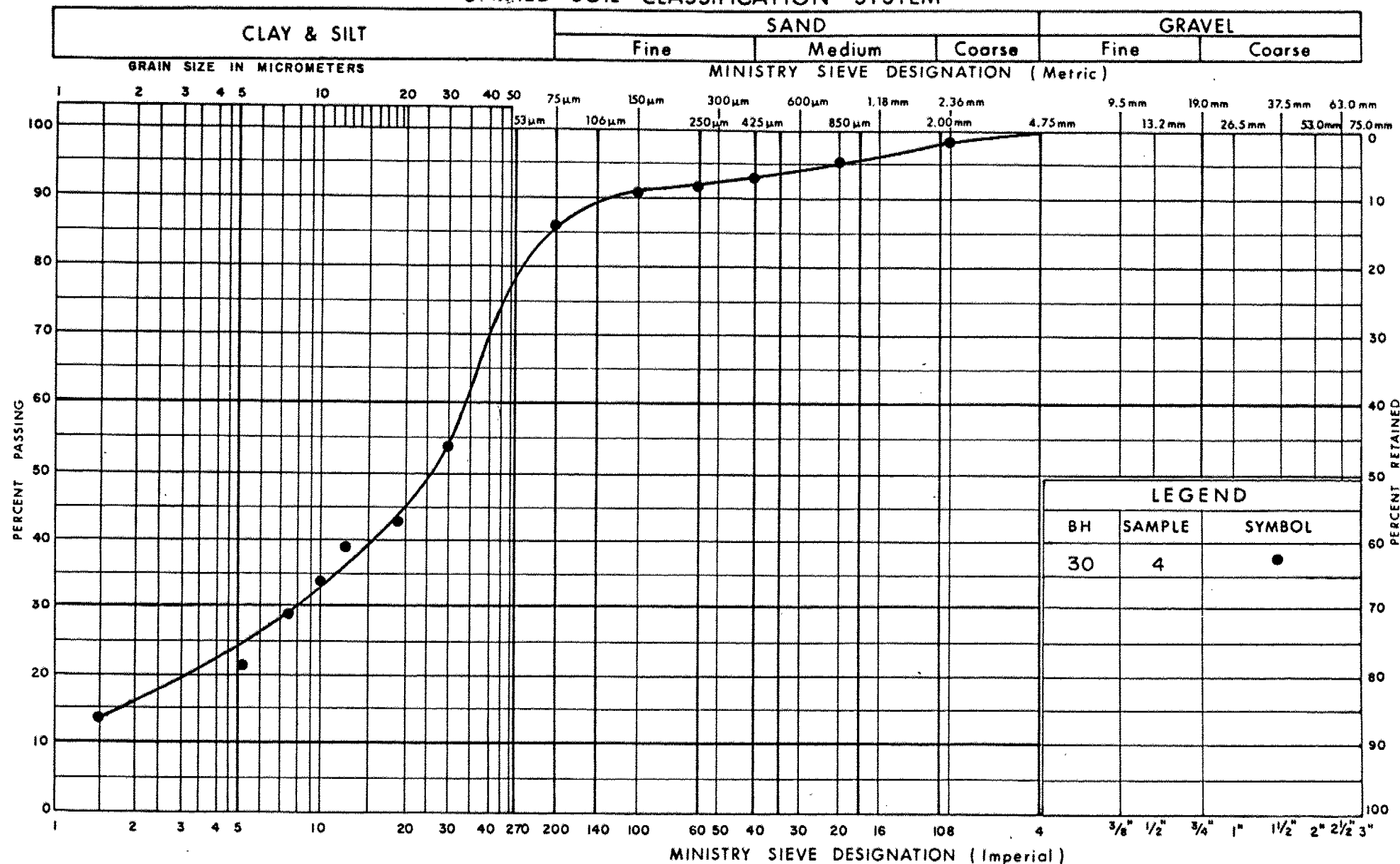
 Ministry of
Transportation and
Communications

 GRAIN SIZE DISTRIBUTION
AMORPHOUS PEAT

FIG No 5

W P

UNIFIED SOIL CLASSIFICATION SYSTEM



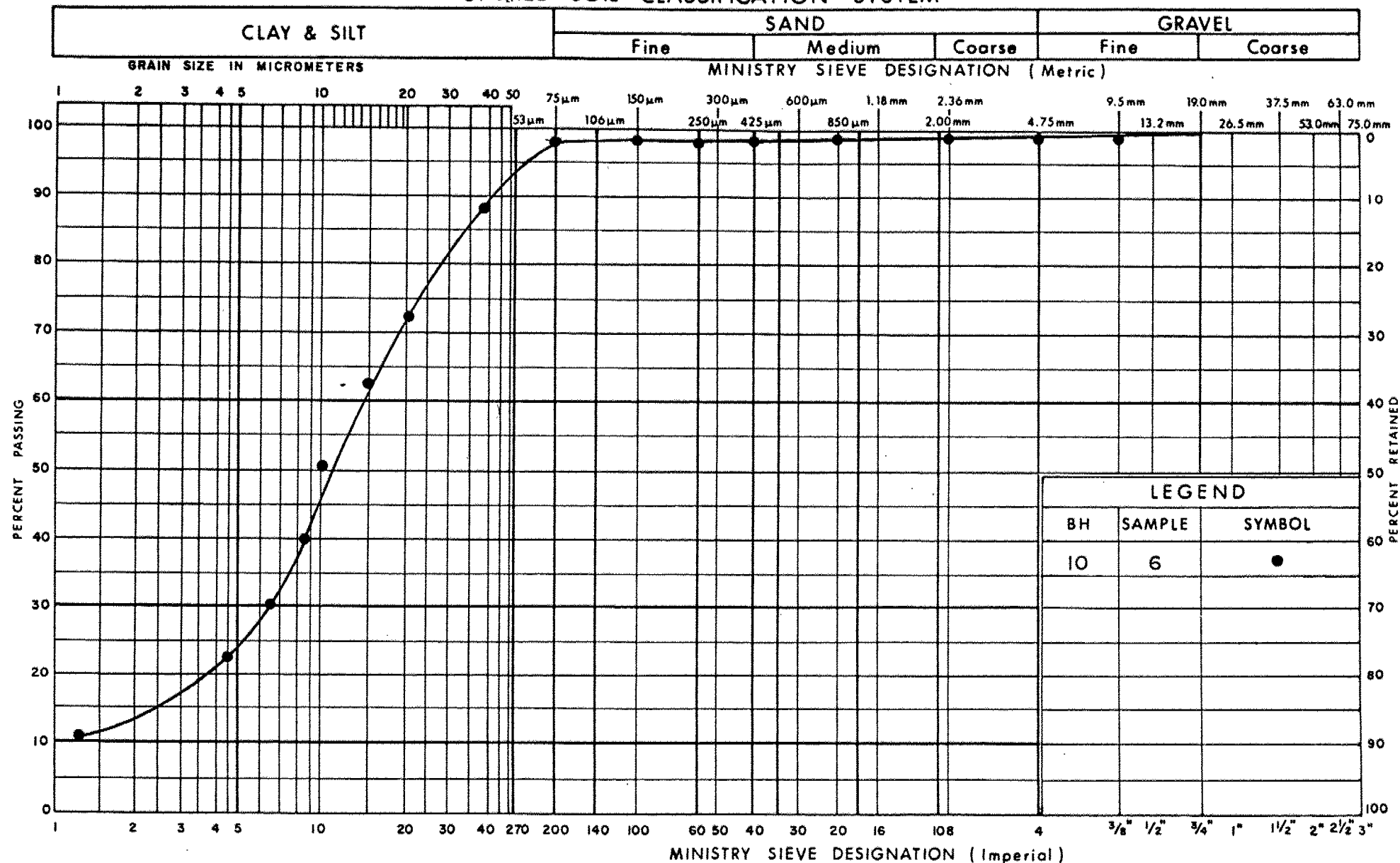
Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION AMORPHOUS PEAT

FIG No 6

W P

UNIFIED SOIL CLASSIFICATION SYSTEM



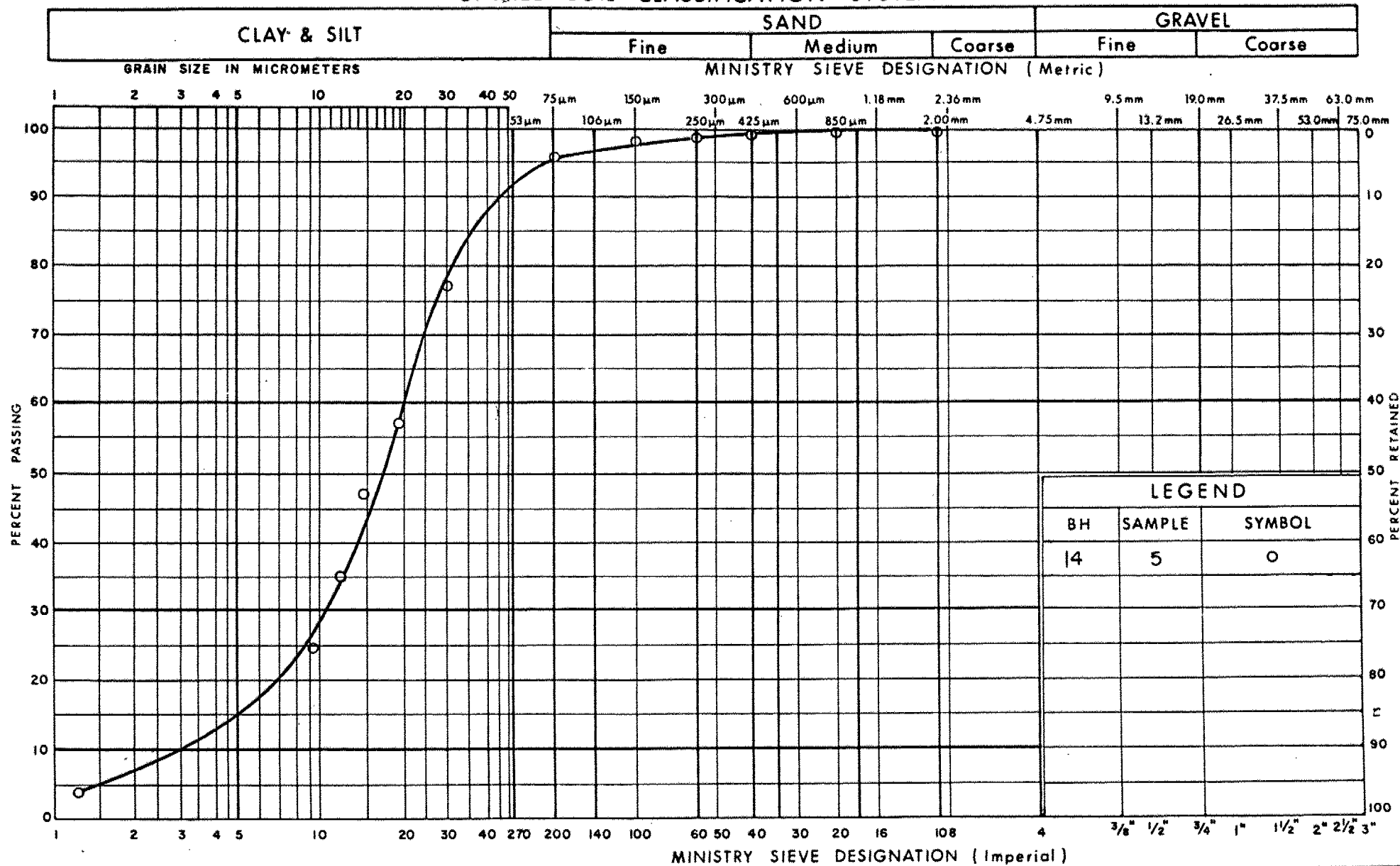
Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION
BASAL SILT

FIG No 7

W P

UNIFIED SOIL CLASSIFICATION SYSTEM



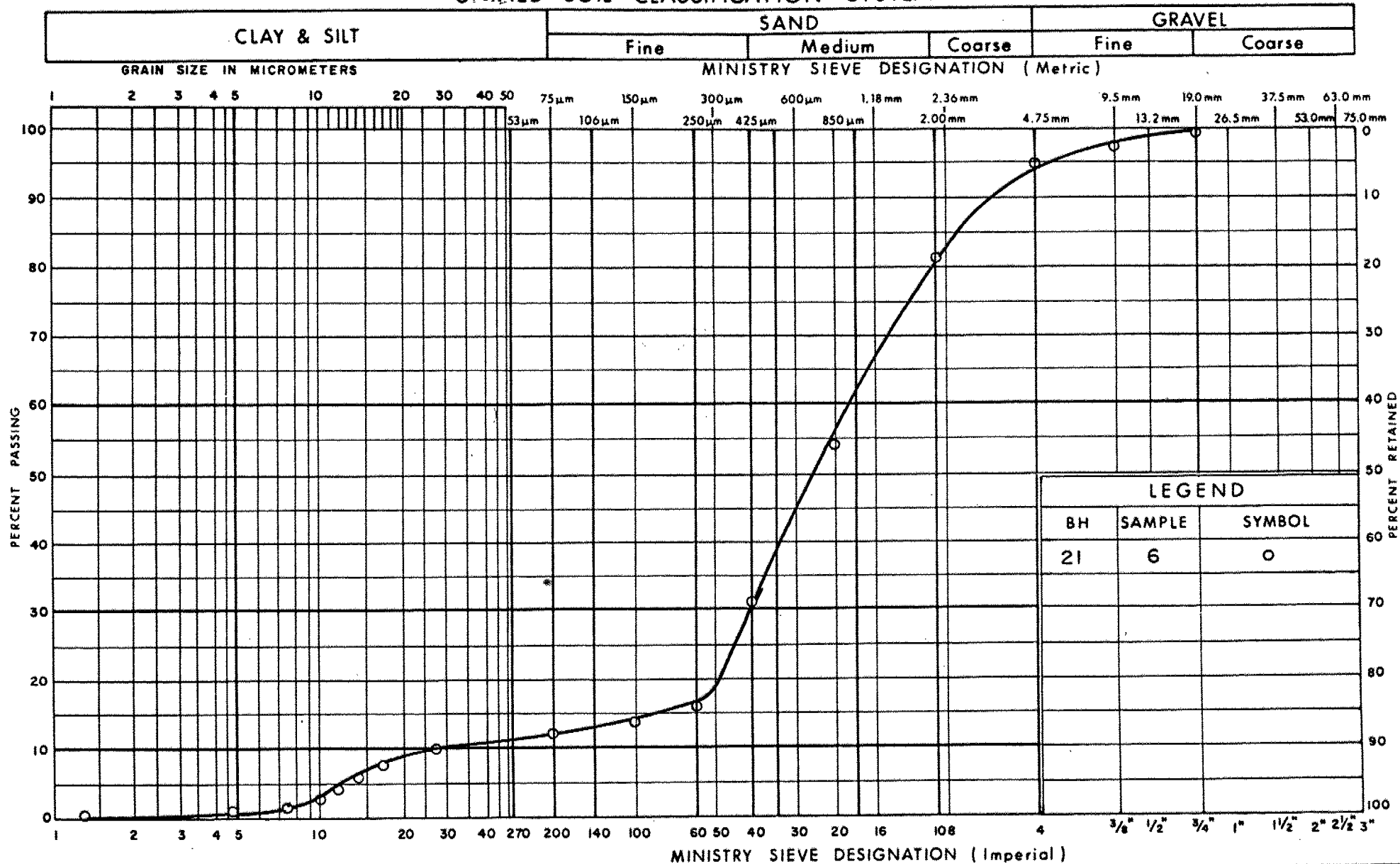
Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION BASAL SILT

FIG No 8

W P

UNIFIED SOIL CLASSIFICATION SYSTEM



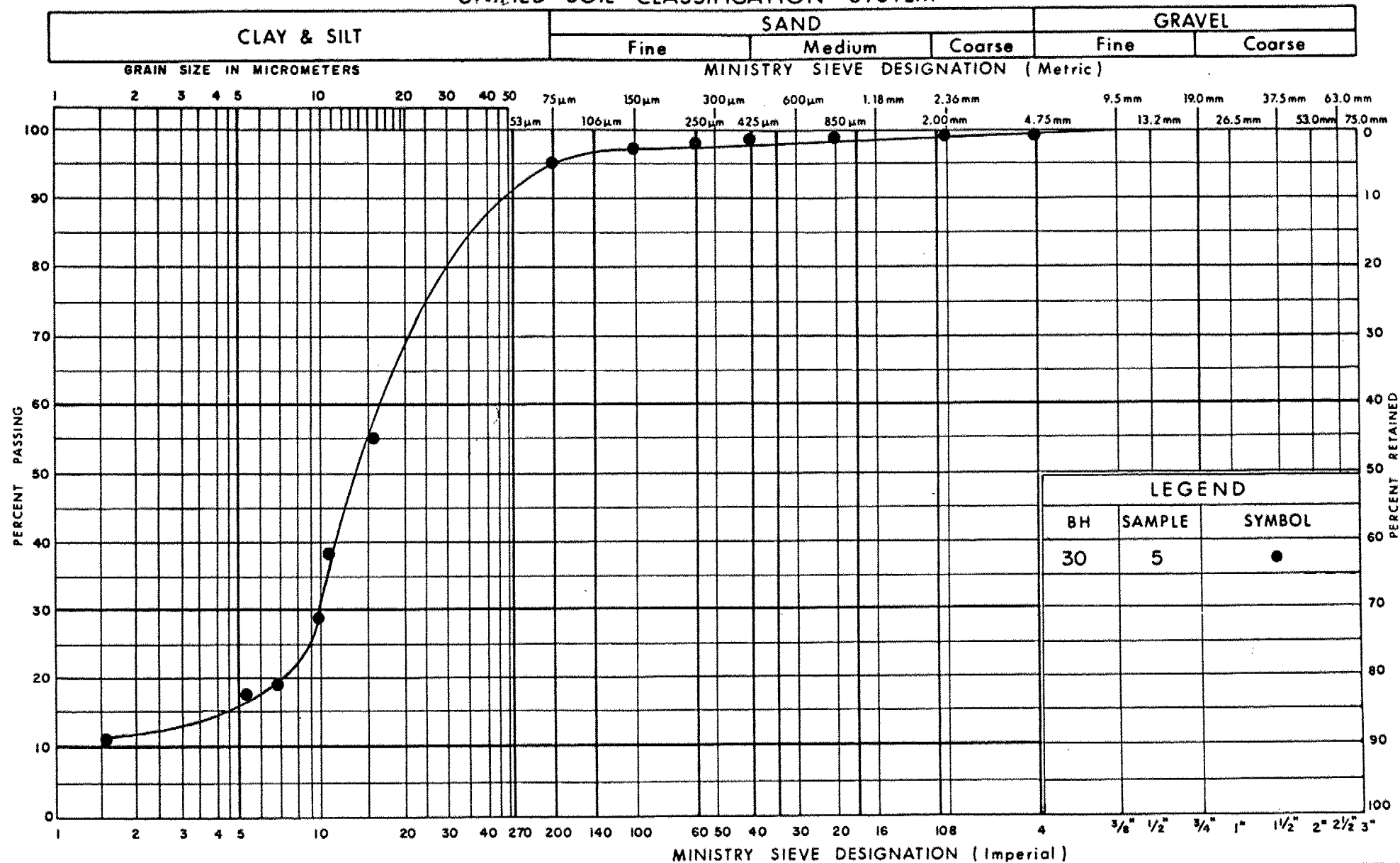
Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION BASAL SAND

FIG No 9

W P

UNIFIED SOIL CLASSIFICATION SYSTEM



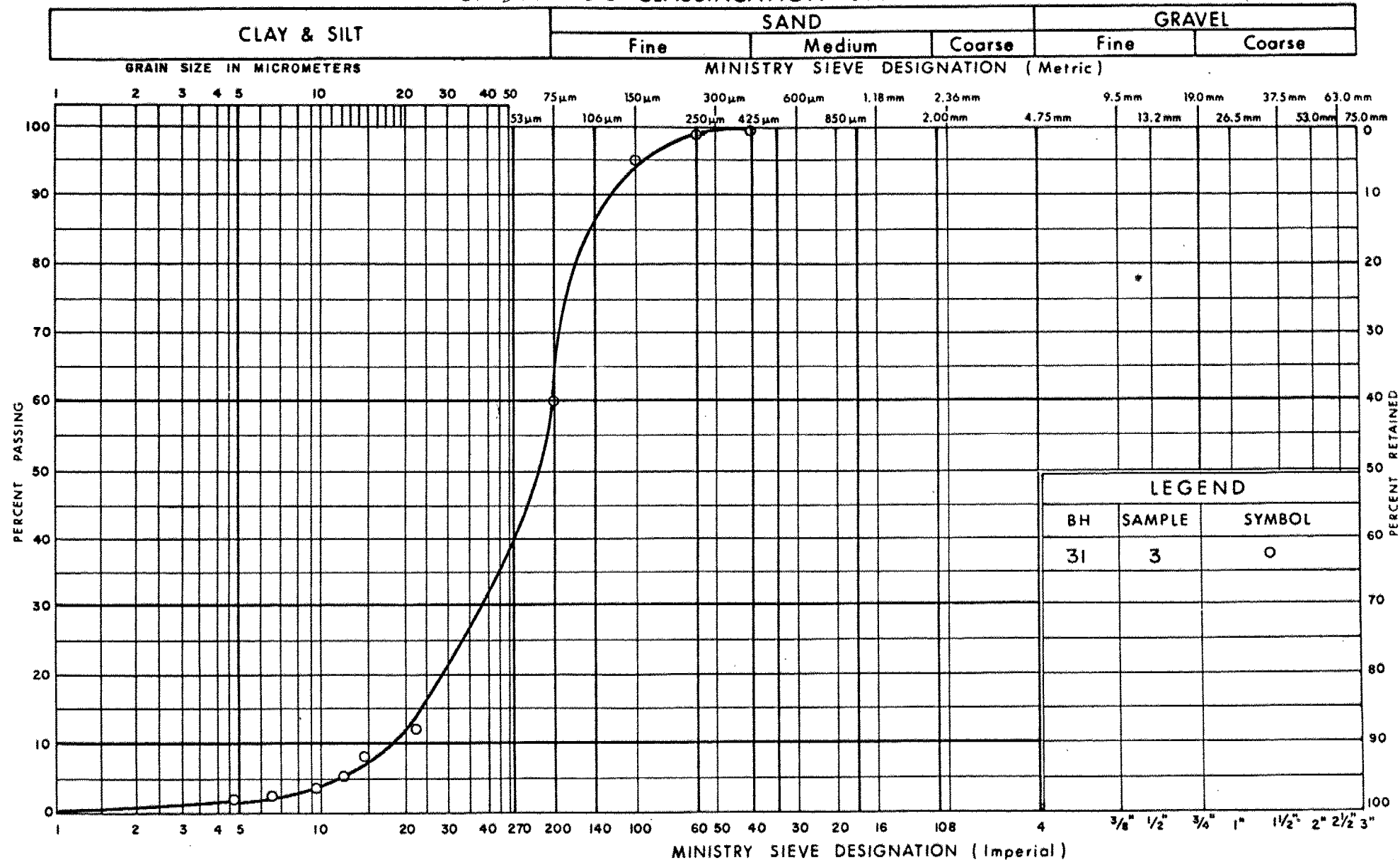
Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION BASAL SILT

FIG No 10

W P

UNIFIED SOIL CLASSIFICATION SYSTEM



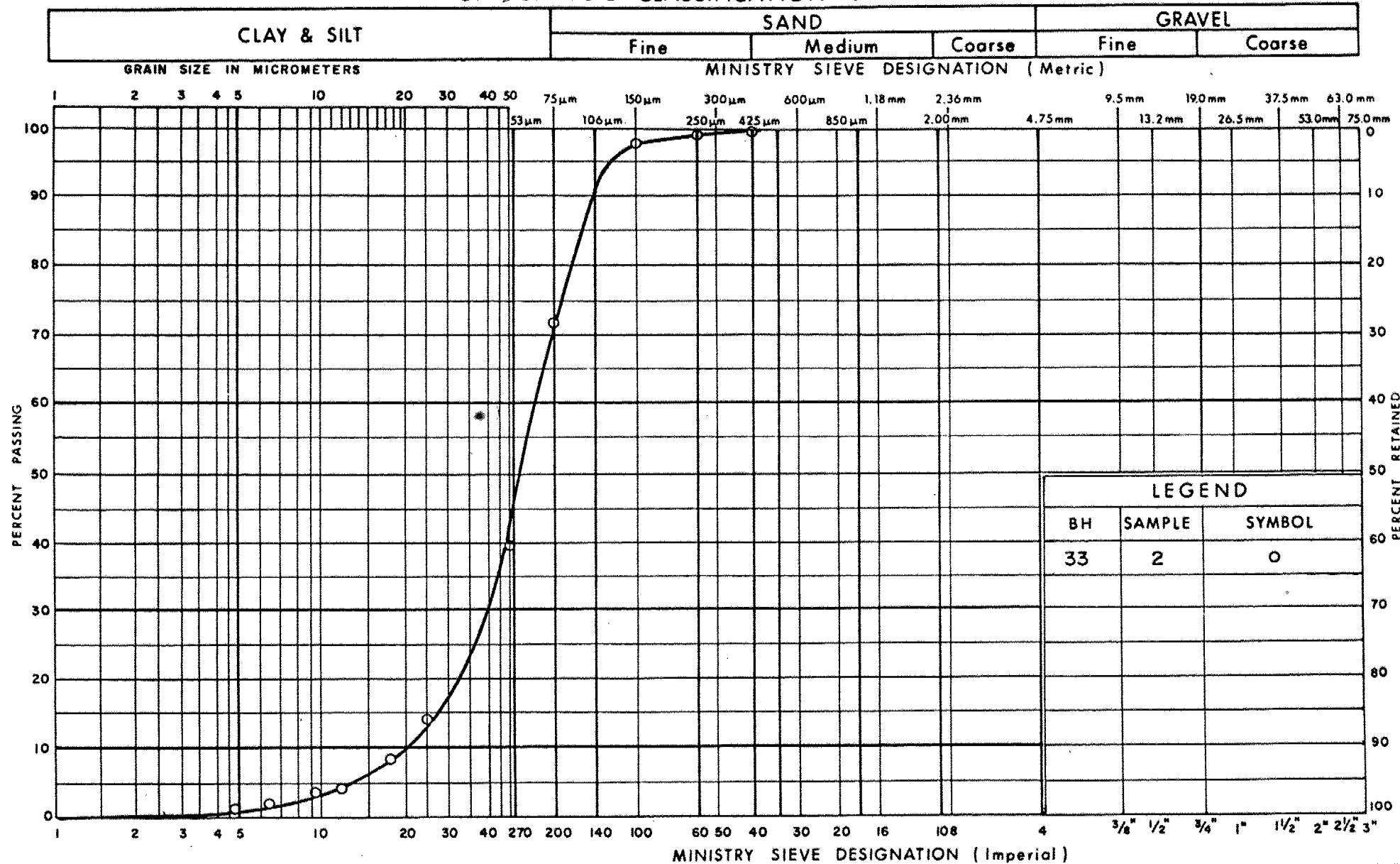
Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION
BASAL SILTY SAND

FIG No 11

W P

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation and
Communications

Ontario

GRAIN SIZE DISTRIBUTION

BASAL SILT AND SAND

FIG No 12

W P

DOCUMENT MICROFILMING IDENTIFICATION

G.I-20 SEPT 1976

GEOCRES No. 52B-7

DIST. 19 REGION Northwestern

W.P. No. 9-75-04

CONT. No. 78-56

W. O. No. _____

STR. SITE No. _____

HWY. No. _____

LOCATION Upgrading Hwy. 11
Atikokan, Ontario

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. —

REMARKS: documents to be unfolded
before microfilming

To: M. Devata
Senior Engineer, Foundations
Central Building
Downsview, Ontario

Date: 81 06 30

From: Geotechnical Section
Northwestern Region

Re: Request for Foundation Investigation
Highway 11

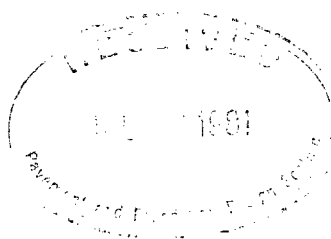
As discussed during our last field trip to Hwy. 11 last Monday and Tuesday June 22 and 23, further investigation will be required at a few swamps. This memo is primarily to request assistance from your Section to undertake a foundation investigation at four swamp locations.

More specifically, two swamps are located on an already completed Contract 78-56, Hwy. 11 from Atikokan E'ly 10.52 miles and two are located on a project which is approaching completion of design, W.P. 9-75-04 some 40 km east of Atikokan E'ly 21.8 km.

I have enclosed soils profile 11TB19-47A showing the location of the two swamps, at stations 445± & 622±. Both are identified by a bold "Req's Invest." notation. The soils profile will also provide a plan, profile and all available soils information. The cross-sections for this contract are presently in Toronto and are available by contacting the Engineering Audit Section. If you absolutely need them, I'm sure they will be glad to help you. I have nevertheless shown on the soils profile a typical cross-section based on the material available on the profile. As you can appreciate, the cross-section of the muskeg is highly interpretive and will have to be established more firmly with your investigation.

Similarly, I have attached Soils Profile 11TB19-48 - 2A showing the location of the two other swamps requiring investigation, at Stations 13+550 and 15+200 (also identified by "Req's Invest." notation). I have furthermore, included the representative cross-sections of the two swamps in this case because they were readily available.

There has been a lot of controversy and discussion regarding the swamps on this Highway 11 over the past few months. A great number of alternatives were evaluated, weighing the benefits of each treatment versus their cost. Corduroy was seriously considered but was later abandoned because of the high cost and the unreliable results. Full excavation (as per DD 406) was also considered but the high cost was not justified. The final Regional decision was to excavate the two swamps in question on widening and take advantage of the much lower costs. If the swamps 'hold' with a minimum of distortion, the pavement can later be removed under some future contract and replaced at a considerable savings compared with full excavation.



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If the swamps don't hold and severe distortion results, we can still place a surcharge under a forthcoming contract to accelerate any settlement and minimize any future distortions. But, in order to place this surcharge, a foundation investigation will be required to ensure that this treatment is possible.

In the case of the contract already completed, the investigation will again evaluate the feasibility of placing a surcharge on the completed and distorted highway.

This methodology was discussed with you on June 24, by telephone, and you generally agreed. You furthermore, advised me that you couldn't do this investigation with your own forces and were going to give this job to a Consultant. You pointed out that the hiring of the Consultant was also a great opportunity to get a third opinion on the treatment of the swamps.

I believe this should explain our position on the swamps and will provide you with sufficient information to conduct your investigation.

Should you need any further information, please feel free to call.



J. B. Girard
Head, Geotechnical Section

JRG/lr
Encls.

c.c. H. Munford
B. T. Darch
W. D. Neillipovitz