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W.P. No. 910-75-01

CONT. No. 77-39

W. O. No.

STR. SITE No.

HWY. No. 17

LOCATION From 8.5 mi W of Raith W'ly 2.5 mi

No of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

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Memorandum

To: Mr. R. Morgenroth (2)
Regional Materials and Testing Engineer
Northwestern Region, Thunder Bay

From: Soil Mechanics Section
Geotechnical Office
West Building, Downsview

Attention: Mr. H. Munford

Date: February 19, 1976

Our File Ref.

In Reply to

FEB 25 1976

CONT. 77-39

Subject:

FOUNDATION INVESTIGATION REPORT

Distress of Embankment
Constructed Over An Organic Terrain
From 8.4 Miles to 11.0 Miles West of Raith
W.P. 910-75-01
Hwy. 17 (T.C.H.) District 19, Thunder Bay

Further to your request, this Section carried out a detailed subsurface investigation to provide various alternatives to remedy the pavement distress on this portion of Highway 17.

This report contains data relative to the past performance of the roadway, its construction history, and subsoil conditions. A summary of possible remedial measures including pertinent recommentations is given herein. The final choice of the remedial measure should be based on economic and other considerations, such as possible icing of the pavement.

This Section is obtaining a detailed proposal regarding the use of urethane foam as an insulating agent against frost action. When received, this data will be forwarded for your consideration as an alternative to other types of insulation.

M. Devata

M. DEVATA
Supervising Engineer

cc: W. Neilipovitz
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FOUNDATION INVESTIGATION REPORT

for

Distress of Embankment
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W.P. 910-75-01
Hwy. 17 (T.C.H.) District 19, Thunder Bay

1. INTRODUCTION

The Soil Mechanics Section was requested to investigate the poor performance of a roadway constructed over organic terrain which resulted in pavement cracking of Hwy. 17. The request was contained in a memo from Mr. R. Morgenroth, Regional Materials Engineer, Northwestern Region, Thunder Bay.

About 60 miles west of Thunder Bay, Hwy. 17 crosses a swamp for a distance of 2.5 miles approximately. This stretch was paved in 1961 and since then longitudinal cracks, which are now up to 12 inches wide, have been occurring every winter, presenting a serious maintenance and pavement performance problem.

A subsurface investigation was undertaken by this Section in November, 1975. This report contains the results of the investigation, discussion of probable causes of pavement distress, and measures to improve the performance of this section of the highway.

2. HISTORY OF CONSTRUCTION AND ROADWAY PERFORMANCE

Before 1959, this portion of the highway existed as a primed surface 20 to 22 ft. wide, with 6 ft. shoulders. No information was available as to when the old roadway was built. It appears that originally the road was constructed by placing granular material directly on the peat deposit . Small longitudinal and transverse cracks were observed in the gravel road.

A soil survey was carried out in 1952 in order to upgrade this portion of the highway. This survey was updated in 1958. A soil design report was issued in August, 1958, and a copy thereof is contained in the

Appendix of this report. As per Mr. A. Rutka's memo to Mr. H.D. McMillan and dated August 15, 1958, "Considerable discussion has taken place since 1952 as to the merits of shifting the centreline and carrying out complete muskeg excavation or of riding the muskeg and using the existing centreline." Finally, the decision was made in favour of the latter alternative, because of a shortage of granular material and in order to take advantage of the consolidation of the existing muskeg underneath the gravel road. It was proposed to raise the grade by about 2 ft. to provide a minimum of 36 in . above the water table in the surrounding swamp. The Soils Design Report recommended that this lift should be achieved in a 28 ft.wide central core by means of 18 in . of GBC class 'B' overlain by 6 in . of GBC class 'A'. Later, GBC class 'B' was changed to sand cushion. In the report it was suggested that acceptable earth material may be used for shouldering.

It was further decided that a 5 ft. deep ditch should be constructed parallel to the highway and 75 ft. south of it, for the purpose of lowering the water table in this area, and, that take off ditches be dug at intervals of 500 ft.

A contract (#59-246) for upgrading this portion of the highway to the TCH standards was awarded to Muskoka Construction Ltd. on August 18, 1959. The Construction Supervisor and Project Supervisor were Mr. F. Caldwell and Mr. N. Maluzynsky, respectively.

Work on the contract began in October, 1959. The initial portion of the work was confined to building a 24-26 ft. wide central core of sand cushion. Widening material was placed when the road was up to the sand cushion grade. About 80% of the sand cushion was placed in 1959. Nearly all the sand cushion was in place by the middle of May, 1960 (Soils Construction Report dated April 7, 1960, p. 3) From the correspondence, it is apparent that most of the materials were"borderline at best and indeed, much of the material did not meet our specifications" (memo from Mr. A. Rutka to Mr. H. Tregaskus, dated April 26, 1960). The sand cushion was mainly end dumped by trucks and bladed to 6 in. lifts using a grader. Compaction of the material was achieved by means of a wobbly-wheel roller. Compactive effort was reported to be good but the moisture content was well above the optimum. The fill material placed in the month of November, 1959 was"freezing as placed". (memos from Mr. A.C. Powell to Mr. J.B. Garland dated October 27, 1959 and to Mr. M. Sinclair dated April 19, 1960). No compaction checks were taken on this contract.

Widening material was placed directly on the muskeg deposit after the centre core was brought up to the sand cushion grade. "Shoulders were placed at the same time as the fill widening". Shouldering material was similar to the fill widening, i.e. acceptable earth borrow varying from bouldery till to fine sand (Soils Construction Report, April 7, 1960, p. 4). Shoulders were well compacted at the surface but no compaction was applied to the fill widening beneath.

GBC 'A' 6" thick was placed on the road during May and June, 1960, using a box spreader and compaction was reported to be good. Crushed gravel was placed on the shoulders at the same time.

The contract for paving (cont. 60-169) was awarded to McLeod Construction Co. Ltd., August 3, 1960 and paving was done in September, 1961. The contract called for HL-4 mix with two lifts of 1½ in. for a width of 24 ft.

It is reported that heaving and cracking of the pavement was observed the following winter (1961-62). Measurements of frost heave and cracking were initiated by the Regional Materials and Testing Office in November, 1963 and were continued to July, 1965. During this period heaving up to 11 in. was recorded. The maximum opening of the longitudinal cracks was about 10 in. No attempt was made to determine the depth of cracks. The pavement required constant maintenance. During the winter, as the cracks opened they were filled with sand. In the spring the badly heaved portion was levelled with a blade and the crack filled with patch mix. At a later date use of wire mesh was suggested to prevent opening of the cracks. In 1967 a binder coat was put on the pavement and a wire mesh 10' wide (gage 10/10, opening 3" x 10") was placed in the middle of the road and it was then covered with a 1½" lift of asphalt (contract 67-156). The cracks reappeared the following winter near the centreline and in addition, along the edges of the wire mesh. It is reported that the wire mesh was split along the longitudinal crack near the centreline.

In April, 1968, a plot of all cracks was compiled by the Regional Materials and Testing Office and measurement points were established at four cross sections where medium to severe longitudinal cracking had occurred. However, the results could not be analyzed because of a survey error. The plot of cracking was updated in April, 1974.

A comparison of the two plots shows cracking has continued unabated and has somewhat increased since 1968.

3. DESCRIPTION OF SITE

The site is located about 60 miles west of Thunder Bay on Highway 17 (Trans Canada Highway). This section extends from 8.5 to 11.0 miles west of Raith. (i.e. old Sta. 1300+00 to 1420+00 or new Sta. 310+00 to 190+00)

At this location the highway crosses a flat swampy area. This area was probably once the flood plain of an older counterpart of Savanne River. The flood plain was later covered with an organic deposit. The depth of the organic deposit, consisting largely of peat, varies from 5 to 15 ft. The area is very poorly drained and free water is at or near the ground surface.

Running parallel to the road is a CP Railway line on the north side and a 5 ft. deep man-made ditch on the south side.

4. FIELD AND LABORATORY INVESTIGATION

The fieldwork consisted of a total of fifteen sampled boreholes. Three boreholes were accompanied by dynamic cone penetration tests adjacent to them. The boreholes were put down to provide subsoil information along a cross section at each of the following stations.

Sta. 231+85	B.H.'s #5, 7, 9 and 12
Sta. 244+75	B.H.'s #4, 6, 8 and 10
Sta. 263+35	B.H.'s #13, 14 and 15
Sta. 273+50	B.H.'s #1, 2, 3 and 11

Selection of the stations was based on the visible damage to the pavement. At Stations 231+85 and 273±50, severe damage has taken place, while at Station 244+75 and 263+35, little damage was visible. At the time of investigation no open cracks were visible but the pavement condition was estimated from the patching and unevenness of the pavement surface. A survey of pavement condition done in 1968 was also utilized in selecting the sections to be investigated.

At each of the above stations one borehole was put down about 5 ft. from the centreline and one borehole on each shoulder. At three stations one borehole was put down on virgin ground between the highway and the railway tracks.

Boring was achieved by means of two trailer mounted hollow stem auger machines adapted for soil sampling purposes. During the field work, disturbed samples were obtained by means of a standard 2" O.D. split-spoon sampler; the energy used in driving it (i.e. 4200 in lbs) conformed to the requirements of the Standard Penetration Test (SPT). The same energy was used for the dynamic cone penetration tests. "Undisturbed" samples were recovered using 2" I.D. Shelby Tubes which were pushed into the peat and underlying cohesive strata by hydraulic means.

Where possible, the in-situ undrained shear strength of the cohesive deposits was determined using an M.T.C. field vane, which has a diameter of 2.8 in., height of 5.6 in. and a 'K' factor of 20.

Details of the borings are given on the Record of Borehole Sheets included in Appendix I of this report. The locations and elevations of the boreholes, together with the inferred soil stratigraphy, are shown on Dwg. No. 9107501-A.

The borings were surveyed in the field by personnel from N.W. Region, Engineering Surveys Section. All elevations are referenced to geodetic datum.

Samples were visually examined and classified at the site as well as in the laboratory. Following this inspection, laboratory tests were carried out on selected samples to determine:

i) Natural Moisture Content
ii) Atterberg Limits
iii) Organic Content
iv) Grain Size Distribution
v) Undrained Shear Strength
vi) Consolidation Characteristics

Laboratory and field test results have been summarized on the Record of Borehole Sheets and are included under Appendix I of this report.

Regional Materials and Testing Office had carried out a soil investigation in April, 1974. Two sections, at Stations 231+75 and 244+75, were common to both investigations. Reference will be made to the findings of this investigation wherever applicable.

5. SUBSOIL CONDITIONS

(5.1) General

In general, the subsoil consists of 4 to 7.5 ft. of fill overlying a 6 to 14 ft. thick peat stratum which is underlain by a clayey silt deposit approximately 50 ft. thick. This is followed by a silt deposit.

(5.2) Fill Material

Fill material was found in all boreholes which were put through the pavement or shoulders. The thickness of the fill (including the pavement) varies from 4 to 7.5 ft. Beneath 6-8 in. thick pavement, the upper 6-12 in. of fill material is well graded coarse sand and gravel and appears to be granular 'A' which was placed for the roadway construction. Below this level, the fill material was finer in grain size composition and was found to be essentially non-uniform granular type of material (sand cushion and the old fill material).

According to design information, granular 'A' is underlain by a sand cushion 12 to 18 in. thick. However, at places, the thickness of sand cushion in the boreholes appears to be greater than 18 in. Below the sand cushion the old fill material, in general, consists of silty sand.

Grain size distribution curves for various boreholes are contained in the Appendix. A review of these curves revealed the following range of distributions: (Figs. 1 to 4)

Location	Gravel (%)	Sand (%)	Silt & Clay (%) (Passing #200)
Severe long.crack	0-20	53-77	9-24 (upper portion) 33-50 (lower portion)
Little or no crack	0-20	58-83	12-30 (upper portion) 24-35 (lower portion)

The above trend was confirmed by the Regional Materials and Testing study.

Cross sections on Drawing 9107501-A show that the fill has settled considerably, because it extends well below the peat surface. The total settlement of 4.5 ft. of fill is in the order of 2-2.5 ft., whereas 7.5 ft. high fill has settled 4-5 ft.

(5.3) Peat

Peat was found in all boreholes. In those boreholes which were put through the road, it was overlain by fill. In three boreholes which were put on virgin ground, it was found from ground surface downwards. The thickness of the peat stratum varies from 5.5 to 13.5 ft. In general, it was found that the thickness of the peat stratum increases in a westerly direction. The contract documents (cont. 59-246, sheets 24 to 29) confirm a similar trend.

The peat is basically fine fibrous in character. Occasional wood pieces or amorphous granular portions were encountered in some places. In general, peat is non-plastic. However, its shear strength can be estimated from field vane tests because of its non-woody character (9).

Results of field and laboratory tests are summarized below:

	Under the Roadbed	Outside the Roadbed
Moisture content	192-537 %	586-757%
In situ vane shear strength	800-1680 psf.	200-600 psf.
Compressibility characteristics	(Fig. 8)	
initial void ratio (e_0)	4.47-5.22 (B.H.'s 1 & 4)	
Compression index (C_c)	2.57-3.05 (B.H.'s 1 & 4)	

It is believed that the field vane test over-estimates the shear strength because of the relative size of the vane (2.8 x 5.6 in.) and fibrous nature of the peat (10).

The above test results show that the peat is very compressible. The reduction in water content and increase in shear strength below the roadbed is due to consolidation of peat under the weight of fill.

(5.4) Clayey Silt

All boreholes except borehole #2, were terminated in this layer. Borehole #2 fully penetrated this layer and its thickness in this borehole was 52 ft.

Atterberg limit tests were carried out on selected samples representative of the material. The physical properties obtained from these laboratory tests are as follows: (Fig. 7)

Liquid Limit	25-34 %
Plastic Limit	17-20 %
Moisture Content	25-40 %

These results indicate that the material essentially consists of clayey silt of low plasticity. In two boreholes (#4 and 6), at some places layers of silty clay of medium plasticity and clay of high plasticity were encountered. The deposit was found to contain occasional silt seams also.

Natural moisture content was always higher than the Liquid Limit, except in one instance.

Besides borehole #2 which penetrated this stratum fully, two other boreholes (#'s 1 and 6) penetrated this stratum for a distance of 30 and 38 ft. Field vane tests in these boreholes, in general, gave shear strength values randomly varying between 800 and 1100 psf. with an average of about 1000 psf. This indicates that the consistency of the deposit is generally firm. In other boreholes also, the consistency was found to be firm, although some deviations were found both on the lower and higher side. A comparison of shear strengths found in the boreholes on the virgin ground and on the highway embankment revealed that unlike peat, there was no increase in shear strength under the fill.

One-dimensional consolidation tests were carried out on samples from the cohesive deposit. One sample from the clay portion gave the following results: (Fig. 8)

Initial void ratio (e_o) = 1.06
Compression index (C_c) = 0.31

The other consolidation test was carried out on a stiff clayey silt sample. It was found to be less compressible as is evident from the following results. (Fig. 8)

Initial void ratio (e_o) = 0.888
Compression index (C_c) = 0.064

(5.5) Silt

Only one borehole (#2) was carried below the clayey silt stratum and into this layer, which was encountered at a depth of 58 ft. Only one sample was obtained in this material. Laboratory tests performed on this sample gave the following results:

Physical Properties (Fig. 7)

Liquid Limit	17%
Plastic Limit	13%

Grain Size Distribution (Fig. 6)

Sand	6%
Silt and Clay	94%

The above results show that the material essentially consists of silt with traces of sand and clay.

6. GROUNDWATER CONDITIONS

Water levels were measured in open boreholes and are shown on respective Record of Borehole Sheets. In the boreholes which were put on the virgin ground the water level was found to be 3 to 18 in. below the ground level.

In three boreholes put through the embankment, the stabilized water level was found to be about 3.0 ft. below the top of the embankment. In other boreholes the water level had either insufficient time to stabilize, or they were filled back due to traffic. It is estimated that the water level in these boreholes was at a depth of 3 ft. approximately.

Boreholes not important.

7. DISCUSSION

(7.1) General

Prior to 1959 this portion of Hwy. 17 existed as a gravel road with primed surface. It is apparent that gravel was placed directly on peat. According to the Soils Design Report (August, 1958), this highway was built about 1 ft. above the prevailing ground or swamp level and the primed surface exhibited some longitudinal and transverse cracking. There are no records to indicate as to how much granular was placed before 1959 and from which sources it was obtained. In 1959 a decision was made to raise the grade by about 2 ft. of granular lift and to pave the road to bring this portion to the TCH standards. About 18 in. of sand cushion overlain by 6 in. of granular 'A' was used for construction. The sand cushion used was at best border line material, was placed at well above optimum and often in frozen condition.

(7.2) Measurement of Heave and Pavement Cracking (1963-65)

Measurements of frost heave and width of cracks were taken during the winter of 1963-1964 and 1964-1965. These measurements were restricted to a small severely affected section near the east end of this project. The precise location could not be pinpointed. These measurements are plotted on Fig. 10 & 11.

An examination of the heave data clearly shows that the entire embankment heaves up, with greater heave being recorded at the centre than at the edges. Unfortunately, the data is very scanty, regarding the progression of heave with time. Frost heaving was slightly higher in 1963-64, although 1964-65 was a more severe winter. "Ontario experience shows that repeated freeze-thaw cycles at the beginning of the cold season will produce large heaves." Weather records show that 1963-64 had more such cycles in the initial part of winter.

references?

During the winter of 1963-64 three readings for the heave were taken on November 19, February 25 and April 1. These readings show that about 90% of the total heave occurred between November 19 and February 25.

The width of cracks was monitored more frequently. These measurements show that the rate of opening of cracks was more rapid in the early part of winter. This is particularly true for 1964-65 when almost all opening was reached by February 5, 1965. The overall width of the worst crack was in the order of 10 in. but the data shows that during one winter its opening increased by about 3 in. In the monitored section the average increase in the opening of the cracks was about 2 in. (Fig. 11).

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During winter these cracks were filled with sand. For the last six years severely cracked portions have been filled with cold mix in the spring.

The data regarding the heave and the width of crack shows that most damage (80-90%) is done in the early part of winter. This indicates that the major source of distress lies in the fill material which freezes in this period.

(7.3) Data on Weather Conditions

Weather records were obtained from the Environment Canada in order to compute the Freezing Index, i.e. the cumulative degree days below 32°F. The nearest weather recording station was at Raith, about 10 miles east of the site. This station came into operation in November, 1969. The second nearest weather station was at Upsala, about 20 miles west of the site. Temperature data from Upsala was used for the winter of 1963-64 and 1964-65, during which pavement cracking measurements were taken. Freezing Indices for the periods 1963 to 1965 (Upsala) and 1969 to 1975 (Raith) are plotted on Fig. 9. The plots show that the average Freezing Index for the site is 3600 degree days. 1963-64 winter had a Freezing Index of 3300, i.e. warmer than average and 1964-65 had a Freezing Index of 4200, the coldest winter on the plot. The pavement measurements taken between 1963 and 1965 appear to be representative of the conditions prevailing over a long period of time.

The Regional Materials and Testing Office carried out a soil investigation in April, 1974. Frost penetration depth was also measured under the pavement during the investigation. It was found that the depth of frozen zone varied from 5.0 to 9.5 ft., with an average of 7.0 ft. However, there was no relationship between the depth of frost and the severity of pavement cracking. 1973-74 was an average winter with a Freezing Index of 3550 degree days and the frost penetration as per MTC formula amounts to 88 in. which correlates well with the observed average penetration.

(7.4) Distress: Causes and Mechanism

The primary cause of frost heave is the formation of ice crystals which later develop into ice lenses in the soil mass.

A frost heave that is greater at the centre of the road than on the sides, may cause bending and tensile stresses in the road structure and result in cracking. If the roadway structure consists of coarse grained material, it does not retain any water in the pores and therefore, lacks adequate tensile strength in both frozen and unfrozen condition. Consequently, there is a risk of cracking in this case. It has been shown that in spite of the fact that a very high strength of soil materials in frozen condition can be obtained by adjusting their composition, this strength is not sufficient to prevent the formation of frost cracks in roads on account of the high stresses which are produced when the frost heave is uneven. (14) At this place, the water level is at a relatively shallow depth of 3 ft. below the pavement surface. It has been shown that relationships occur among rate of heave, rate of penetration of freezing front, heave ratio (=rate of heave/rate of penetration) and water table depth. At a single rate of penetration, deeper water tables limited both rate of heave and heave ratio. As the rate of penetration increased, the rate of heave also increased. A high rate of heave and heave ratio was observed for saturated soils with permeability greater than 1×10^{-5} cm/sec. This was true for even pit gravel. (11) A comparison of the rate of heave of pit gravel (non-frost susceptible) and silt (extremely frost-susceptible) is shown in the following table.

Depth of water table 2.5 ft.
Water content: Pit gravel - 7.9%
Silt -20.7%

Rate of Penetration in/day	Rate of Heave in/day	
	Pit gravel	Silt
0.10	0.047	0.080
0.25	0.055	0.110
0.50	0.090	0.150

This illustrates that even pit gravel will heave considerably if its pores are filled with water. At our site the investigation revealed that the moisture content of the granular fill material ranges from 10-20%.

In our opinion the following sequence of events at this site outlines the reasons for the distress of the roadway. In the early part of winter snow melts and runs off to the shoulders.

Depending upon the composition of fill material, this runoff water may infiltrate under the pavement and get trapped into pockets, or it may drain away from the road. Any accumulation of water beneath the pavement will result in formation of ice lenses and cracking of the pavement. Water getting into the crack freezes when the temperature drops and the ice formation widens the crack. Shrinkage of the pavement structure due to lowering of temperature also helps to widen the crack. When the temperature rises or when the road is salted, ice in the crack melts resulting in slightly greater amount of water in the crack which upon freezing occupies greater volume and widens the crack even further.

As the winter progresses, the freezing front moves downward and penetrates into the fill material. The water in the voids in the fill forms ice lenses and causes frost heave. A maximum heave of 11 in. was recorded in 1963-64, but neither the amount of frost heave nor the severity of cracking was uniform over the entire stretch of the swamp. This non-uniformity is due to soils of variable degree of frost susceptibility in the fill which was placed at different times under different conditions and obtained from different sources. As mentioned earlier, even non-frost susceptible material can undergo considerable frost heaving if its pores are filled with water and the water table is at a shallow depth, as is the case at the present site.

On the edges of the embankment, and away from it, a deep cover of snow retards the freezing front. The freezing front penetrating through the exposed roadway moves ahead of the freezing front from the sides and as a result, the entire roadway heaves but the heave is greatest at the centre. (Fig. 12)

During the winter, cold mix or patch filler which was placed in the preceding spring becomes loose due to the shrinkage of pavement and frost heave that is greater at the centre than at the edges.

This is then gradually removed by the traffic or snow plough. Edges of cracks are chipped by the traffic or snow plough, contributing to the widening of the cracks. Sand used for filling the cracks in the winter prevents the cracks to return to their initial position in the spring. The present width of cracks is the sum of the cumulative effect of the above process over a period of many years and does not reflect the damage done during a single winter. (Fig. 13)

The drainage in the swamp is very poor and for practical purposes, the water in the swamp is stagnant.

The drainage ditch south of and parallel to the road could not lower the ground water table as the ditch water level is reported to be generally at the same level as the water level underneath the pavement. The effectiveness of the ditch was doubtful from the beginning (see memo from Mr. L. Soderman to Mr. J.B. Garrard dated Oct. 26, 1959).

8. REMEDIAL MEASURES

(8.1) General

A review of the causes of distress shows that frost penetration in non-uniform fill material of varying frost-susceptibility and high water table in a very compressible peat deposit are the major contributing factors.

The following table lists many possible remedial measures applicable to the present site, along with the advantages and disadvantages of each.

A combination of more than one alternative can be considered as a practical solution.

The following constraints existing at this site play an important role in the practicality and selection of a suitable solution.

1. Any raise in grade will result in settlements in the order of 35% of the raise. Therefore, grade raise should be kept to a minimum, preferably nil. It is not the stability, but the settlement of the embankment due to the compression of the underlying peat which is a constraint.

REMEDIAL MEASURE	ADVANTAGES	DISADVANTAGES	REMARKS
1. Realign the road, riding the peat.	Use of non frost susceptible material will improve the performance. Existing road can be used as detour.	Peat will settle excessively and therefore, satisfactory performance cannot be guaranteed. However, this may be somewhat improved by the use of synthetic fabric, e.g. Mirafi over the peat. Requires non frost susceptible material for the base course.	Realignment has to be to the south. Subsoil conditions along any minor realignment will not change because of the extent of swamp.
2. Realign the road. Replace peat with granular.	Use of non frost susceptible material for the base course will improve the performance. Use of granular backfill will minimize the long term settlements. Consequently, further improve the performance. Existing road may be used as detour.	Requires granular type for backfill below water level and non frost susceptible material for the base course. Requires excavation up to 15 ft. in depth.	Same as above.
3. Excavate present fill and peat and replace with granular.	Use of proper materials and construction methods will ensure satisfactory performance.	Requires granular and non frost susceptible material as in the preceding case. Requires excavation up to 15 ft. deep. Requires a detour.	
4. Excavate all fill and replace with non frost susceptible material.	Minor settlements if present grade is maintained.	Requires non frost susceptible material for base course. Frost damage may not be completely eliminated because of high water table. Requires a detour.	

REMEDIAL MEASURE	ADVANTAGES	DISADVANTAGES	REMARKS
5. Raise the grade.	Performance will be improved.	Requires non-frost susceptible material for additional fill. Fill will settle excessively and differentially. Frost penetration (depth 9 ft.) into the existing non-uniform granular fill cannot be eliminated without additional fill of sufficient height. Requires a detour.	Settlements will be in the order of 35% of additional fill height. Settlements can be somewhat reduced by light-weight fill. Not advisable to raise grade by more than 3 ft.
6. Use chemical additives.	Present fill material may be utilized minimizing granular requirements. No additional settlements if present grade is maintained.	Results cannot be predicted as they depend upon a large number of variables, e.g. soil type, grain and pore sizes, pore water chemistry, etc. Requires an advance test program to select suitable agent and technique.	Examples: Tetra sodium pyrophosphate - a dispersive agent for coarse grained soils (6) 4-tert-butylcatechol (TBC) - a waterproofing agent for fine grained soils (2)
7. Heat the pavement.	Eliminates heave.	Very expensive and energy consuming.	Heating may be achieved by heating elements or hot water pipes.
8. Encapsulate the fill in waterproof membrane.	Present fill material may be partially utilized minimizing granular requirements. No additional settlements if grade is not raised.	Requires a detour. Will not eliminate frost heave, but will minimize it considerably. Requires strict construction and quality control.	If an acceptable construction procedure is agreed then the roadway may be dug, a waterproof membrane laid and fill replaced, all in a continuous operation.

REMEDIAL MEASURE	ADVANTAGES	DISADVANTAGES	REMARKS
9. Use insulation under the pavement.	Very effective. Can eliminate all frost if sufficient amount used. Minimizes granular requirement. Existing roadway structure may be used. One lane can be used for traffic.	Expensive. May cause icing on pavement.	Styrofoam, rigid urethane foam or foamed sulphur may be used as insulation.
10. Do nothing.	Does not require additional capital expenditure.	Existing performance poor, and presents hazardous conditions for driving. Requires extensive, constant maintenance and repair.	The existing roadway does not break up in spring thaw and is structurally sound.

2. The high water table cannot be lowered because of the lack of relief in the drainage system of the area.

3. Hwy. 17 which forms part of Trans Canada Highway, is the only highway in this area and traffic must be maintained at all times.

(8.2) Use of Insulation

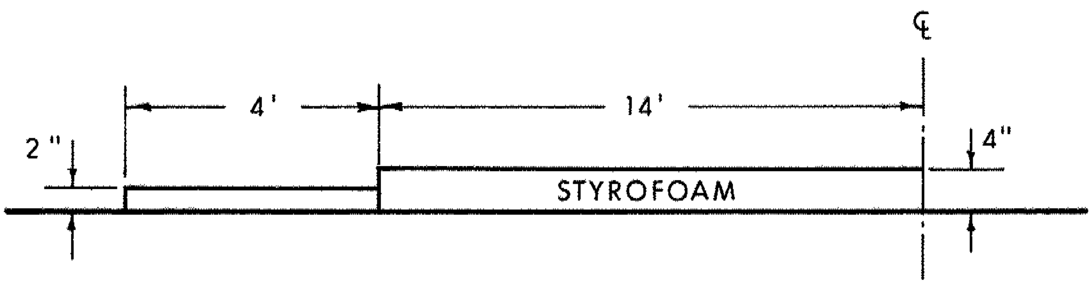
A remedial measure which prevents frost penetration in the fill would be effective at the present site. In our opinion this can be best achieved by the use of insulation under the pavement. Elevation where insulation is placed will have an effect upon the performance of the roadway. The following additional constraints exist in this respect at this site.

1. Insulation should be placed as close to the pavement as possible to eliminate or minimize frost penetration into the frost susceptible fill. However, the closer the insulation is to the pavement, the more icing is liable to occur. This calls for placing the insulation at a lower level. This in turn exposes the overlying fill material to frost action and reduces the effectiveness of insulation.

2. Insulation should not be placed more than 2 ft. below the present grade because of the high water table.

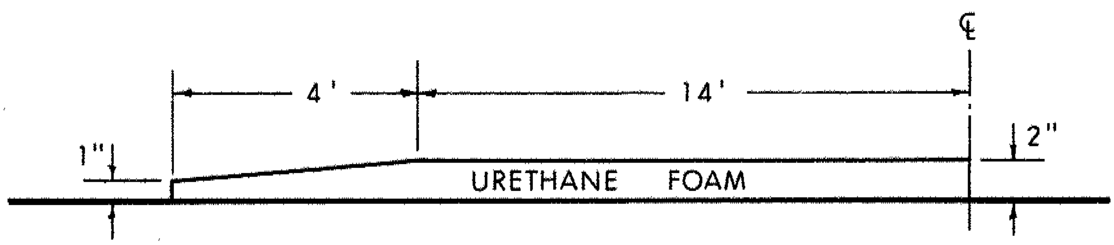
Design Considerations

The design Freezing Index in this area is about 4500 degree days below 32°F. If it is decided to use styrofoam then 4 inches of HI type will be necessary to prevent frost penetration beneath it. The recommended thickness and width of styrofoam for no-frost condition are as shown below.



Styrofoam can be placed directly on the existing pavement or on granular material, as per current MTC standard.

If rigid urethane is used, it should be sprayed in 2 in. thickness over a base width of 35 ft. as shown below. The reduced thickness is due to better insulation value of urethane foam (K=0.13 vs. K=0.25 for styrofoam). Urethane may pick up some moisture over a long period, thus reducing its insulation value. This can be prevented by enclosing the urethane foam in a waterproof membrane. The same membrane can be used to enclose the fill placed on the insulation. This will prevent any infiltration of water in the fill and increase the effectiveness of the method.



This office contacted Witco Chemical Canada Ltd. who manufacture urethane. According to the proposal submitted by the manufacturer, a 2 in. thick layer should eliminate frost penetration below it at this site. For estimate purposes a 2 in. thick foam will cost 67 cents per sq. ft. installed. Enclosing the foam in a waterproof membrane will cost additional 10-15 cents per sq. ft.

The choice of insulation, i.e. styrofoam vs. urethane should be decided on the basis of economic and other practical considerations. However, urethane has a much higher R-factor than styrofoam and it provides a continuous water proof film which styrofoam lacks because of the joints.

The thickness of granular material required and the thickness and type of asphalt pavement should be decided upon the recommendations of the Pavement Design Section. Some asphalts perform better in cold temperatures than others. All granular type material placed above the insulation should be non-frost susceptible.

Depth Below Pavement and Related Considerations

In order to minimize icing, it has been suggested that there should be a 3 ft. cover of granular material over the insulation. This cover can be provided in the following manners.

1. Putting the insulation on the existing pavement and raising the grade by 3 ft. No stability problems are anticipated. If granular material is used, then the total settlements will be in the order of 12 in. where the thickness of peat stratum is 10 ft. The settlements will be correspondingly greater where the peat is thicker. For 3 ft. additional fill, it is recommended that the side slopes should be kept 4 horizontal to 1 vertical or flatter.

   The settlements will be smaller if lightweight fill is used instead.

2. Alternatively, the present roadway can be excavated to a depth of 2 ft., insulation put at this level, and a 3 ft. lift of non-frost susceptible granular placed on top. In this case, the final grade will be 1 ft. higher than the present. The settlements will be in the order of 4 in. The settlements can be reduced by the use of lightweight fill material. The excavation may cause some problems in maintaining traffic.

If increased incidence of icing is acceptable, then the insulation can be placed on the existing pavement. A 12 in. lift of non-frost susceptible material should be placed over it before paving it. The thickness of the insulation should be the same as mentioned earlier. This will result in substantial savings, because no excavation is required and traffic can be maintained much easier by reconstructing one-half the width of the roadway and leaving the other half open to traffic. The fill will settle about 4 in. Some differential settlements will occur.

In Norway a 'top insulation' concept has recently been used, where a new pavement is placed directly on the insulating layer, polyurethane and polystyrene having minimum compressive strength of 114 psi at 5 percent deflection. This increases the danger of ice forming. It was observed that a certain degree of ice formation took place during the autumn which was offset by salting. From December on, the ice formation was more equal to that of uninsulated road. (15) Consideration should be given to its use because of its effectiveness and ease of construction.

(8.3) Grade Raise as an Alternate Solution

If the use of insulation as a remedial measure is unacceptable for economic, icing and other reasons, then consideration should be given to other alternatives. Raising the grade is another alternative from a practical point of view which can be used at this site. However, it should be borne in mind that this method may not completely eliminate the frost heave and distress of the roadway,but will reduce the distress.

The presence of compressible peat immediately below the present roadway fill precludes the construction of high embankments in this area due to problems associated with settlements and performance of the roadway. It is important that post-construction settlements should be kept to a minimum in order to minimize differential settlements. Therefore, it is recommended that the height of additional fill be restricted to 3 ft. above the present grade. The total settlement under 3 ft. of additional fill will be in the order of 12 in. where the thickness of peat deposit is 10 ft. and will occur over a long period of time. Elsewhere, the settlements will be proportional to the depth of peat. It is estimated that 50% of the total settlements will occur within the first 30 days.

The grade may be raised by putting additional fill on top of the existing fill. Alternatively, the upper 2 ft. of the existing fill may be removed, good non-frost susceptible material salvaged and reused. The grade can be raised by placing additional fill. It is recommended that additional fill should consist of non-frost susceptible granular type material. Side slopes of the embankment should be kept 4 horizontal to 1 vertical or flatter. In order to minimize infiltration of water into the fill material. the shoulders in this area should be paved. Moisture content of the fill should be kept on the drier side of the optimum moisture content during placement.

Lightweight fill material e.g. slag or bark will minimize the settlements and associated problems. Bark can provide added insulation against frost penetration.

Post construction settlements can be reduced by putting surcharge and preloading the embankment. At this site, it is not feasible because the road has to be kept open to traffic at all times.

9. MISCELLANEOUS

Subsoil investigation for this project was performed during the period of November 18 to November 26, 1975 under the supervision of Mr. A. Prakash, Senior Engineer. The equipment used for the investigation was owned and operated by Dominion Soil Investigation Ltd. and Dodds Associates Ltd., of Thunder Bay.

Mr. H. Munford, Senior Soils Supervisor, Northwestern Region, provided information regarding the history of the roadway.

During the preparation of this report, discussions were held with Mr. A. Rutka, Manager, Geotechnical Office, Mr. W. Phang, Head, Pavement Research, and Mr. G.A. Wrong, Head, Pavement Design Section. Comments obtained during these discussions are incorporated in the report.

This report was prepared by Mr. A. Prakash and reviewed by Mr. M. Devata, Supervising Engineer.

A. Prakash
Senior Engineer

M. Devata
Supervising Engineer

REGISTERED PROFESSIONAL ENGINEER
A. PRAKASH
PROVINCE OF ONTARIO

February, 1976

APPENDIX

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MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO

ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 1

WP 910-75-01 LOCATION Sta. 273 + 44 16' Lt. ORIGINATED BY AP

DIST 19 HWY 17 T.C.H. BORING DATE November 18, 1975 COMPILED BY NT

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT. PLOT
1513.2	Ground Level	
0.0 / 1509.2	Fill silty sand, traces of gravel. Compact	
4.0 / 1503.7	Peat fine, fibrous, occ. woody.	
9.5 / 1465.2	Clayey silt, occasional silt seams Firm	
48.0	End of Borehole	

SAMPLES

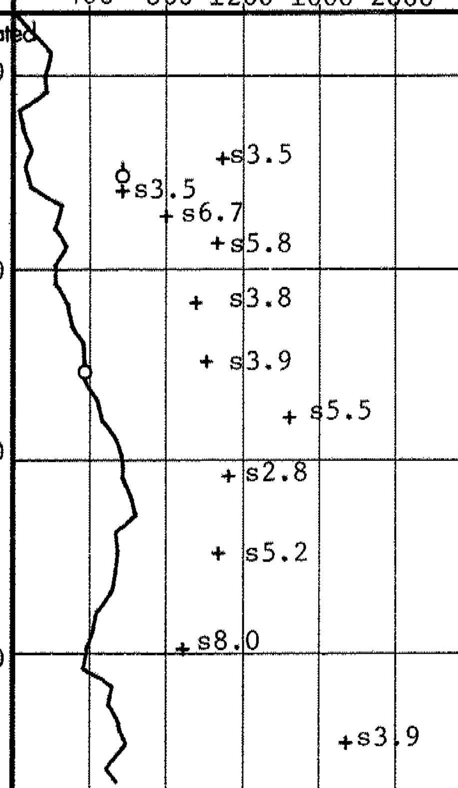
NUMBER	TYPE	'N' VALUES
1	SS	11
2	SS	10
3	SS	3
4	SS	1/9"
5	TW	PH
6	TW	PH
7	TW	PH
8	SS	4
9	TW	PH
10	SS	7
11	SS	11
12	TW	PH
13	TW	PH
14	TW	PH

GROUND WATER: Estimated ∇ 1510

ELEV: 1510, 1500, 1490, 1480, 1470, 1460

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 400 800 1200 1600 2000



Shear strength (field vane)	Water content	UNIT WEIGHT γ P.C.F	REMARKS % GR SA SI CL
s3.5		62	Org. 88.5% wc 392%
s3.5			
s6.7			
s5.8			
s3.8			
s3.9		121	
s5.5			
s2.8			
s5.2			
s8.0			
s3.9			
s8.3			

LIQUID LIMIT — W_L PLASTIC LIMIT — W_P WATER CONTENT — W; WATER CONTENT % 20 40 60

20 15 5 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

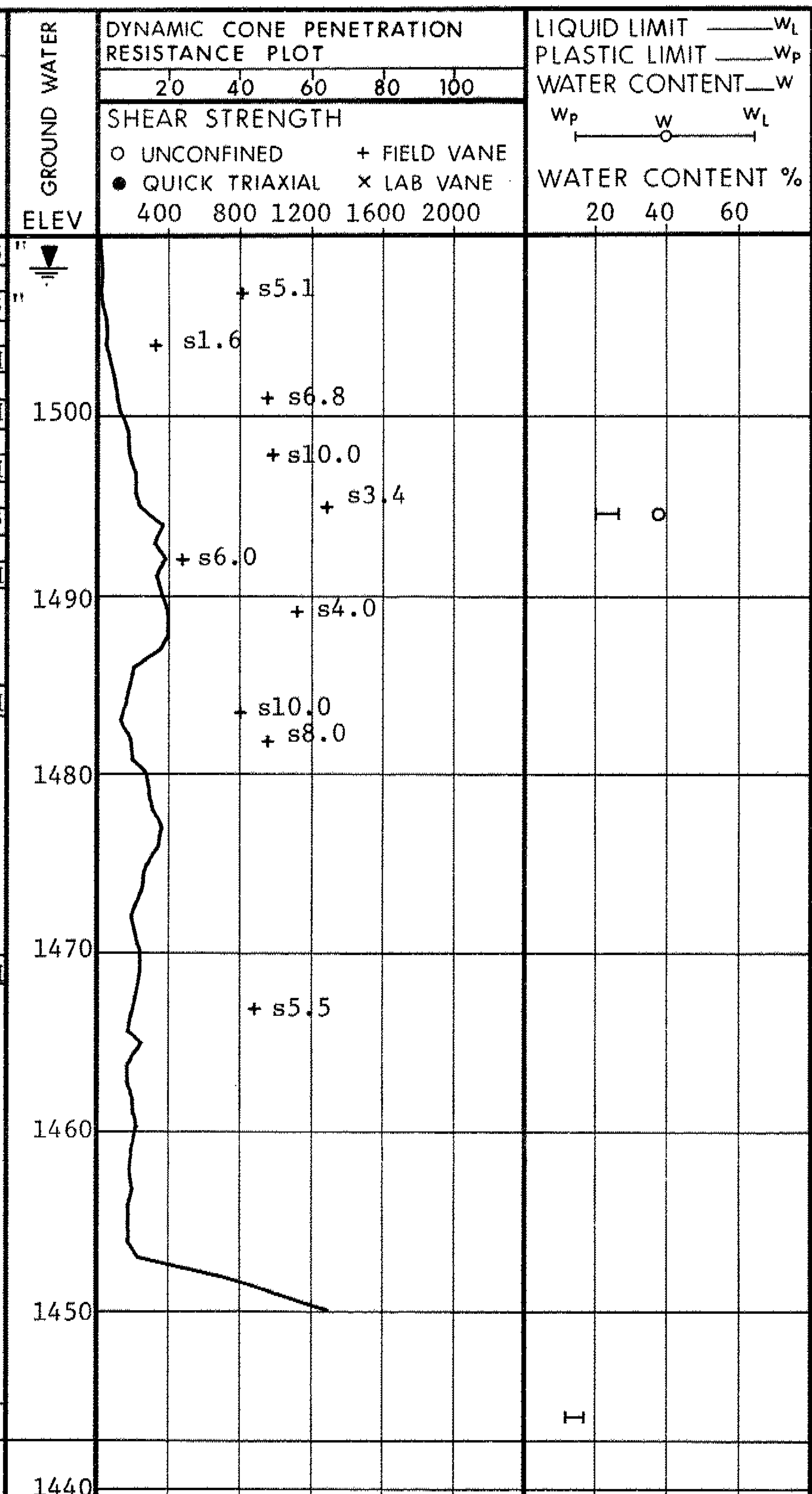
RECORD OF BOREHOLE Nº 2

WP 910-75-01 LOCATION Sta. 273 + 51 44' Lt. ORIGINATED BY AP

DIST 19 HWY 17 T.C.H. BORING DATE November 21, 1975 COMPILED BY NT

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	SHEAR STRENGTH (Field Vane)	WATER CONTENT %	REMARKS (% GR SA SI CL)
1510.0	Ground Level						
0.0	Peat Fine Fibrous	1	SS	1/18"	s5.1		w/c 586% Org.61.9%
		2	SS	1/18"	s1.6		
1504.0 / 6.0	Clayey silt occasional silt seams Firm	3	TW	PH	s6.8		
		4	TW	PH	s10.0		
		5	TW	PH	s3.4		
		6	SS	3	s6.0		
		7	TW	PH	s4.0		
		8	TW	PH	s10.0, s8.0		
		9	TW	PH	s5.5		
1452.0 / 58.0	Silt, traces of sand and clay. Compact	10	SS	-			0 6 (94)
1443.0 / 67.0	End of Borehole						



20 / 15 5 / 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 3

WP 910-75-01	LOCATION Sta. 273 + 51 6' Lt.	ORIGINATED BY AP
DIST 19 HWY 17 T.C.H.	BORING DATE November 22, 1975	COMPILED BY NT
DATUM Geodetic	BOREHOLE TYPE Hollow Stem Auger	CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	SHEAR STRENGTH	WATER CONTENT %	UNIT WEIGHT γ	REMARKS % GR SA SI CL
1513.7	Ground Level									
0.0	~~Pavement~~ Fill-silty sand, traces of gravel and clay		1	SS	90	Estimated ▽				0 79 (21)
			2	SS	94	1510				6 71 (23)
1508.2	sandy silt		3	SS	13					0 49 50 1
5.5	Peat Fine fibrous, occ. woody		4	SS	6					w/c 243%
			5	VS	–		> +			Org. 79.4%
			6	TW	PH		+ s3.0			
1500.7			7	TW	PH	1500				
13.0	Clayey Silt occasional silt seams						+ s4.3			
			8	SS	4		+ s2.9			
	Firm		9	TW	PH					
			10	TW	PH					
1489.2	stiff					1490	+ s6.2			
24.5	End of Borehole									
						1480				

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 400 800 1200 1600 2000

LIQUID LIMIT — W_L PLASTIC LIMIT — W_P WATER CONTENT — W

WATER CONTENT % 20 40 60

20 15 5 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

ENGINEERING SERVICES BRANCH - GEOTECHNICAL OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 4

WP 910-75-01 LOCATION Sta. 244 + 77 6' Lt. ORIGINATED BY AP

DIST 19 HWY 17 T.C.H. BORING DATE November 24, 1975 COMPILED BY NT

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY So

ELEV / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER / ELEV	SHEAR STRENGTH	WATER CONTENT %	UNIT WEIGHT γ	REMARKS % GR SA SI CL
1517.4	Ground Level									
	Pavement					Estimated ∇				
	Fill gravelly fine sand, some silt		1	SS	106					19 70 (11)
			2	SS	31					0 85 (15)
	silty sand, traces of gravel		3	SS	26					6 68 (26)
			4	SS	49					9 68 (23)
1509.9			5	SS	3	1510				
7.5	Peat						+ s2.1			
	Fine fibrous		6	TW	PH					Org.77.2% w/c536%
1503.4			7	TW	PH		+ s2.5			w/c 527%
14.0	Silty clay to clayey silt, occasional silt seams.						+ s6.0			
			8	SS	1	1500	+ s3.5			
	Soft to Firm		9	TW	PH		+ s7.5			
1494.4										
23.0	End of Borehole					1490				

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 400 800 1200 1600 2000

LIQUID LIMIT —— W_L PLASTIC LIMIT —— W_P WATER CONTENT —— W

WATER CONTENT % 20 40 60

20 15 5 10 % STRAIN AT FAILURE

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE № 5

WP 910-75-01 LOCATION Sta. 231 + 87 5' Lt. ORIGINATED BY AP

DIST 19 HWY 17 T.C.H. BORING DATE November 24, 1975 COMPILED BY NT

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY SO

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	SHEAR STRENGTH	WATER CONTENT %	REMARKS % GR SA SI CL
1518.3	Ground Level							
	~~Pavement~~				Estimated ∇			
0.0	sand, some gravel, traces of silt Fill	1	SS	47			o	24 67 (9)
		2	SS	34			o	12 76 (12)
	silty sand, some gravel	3	SS	21			o	14 53 32 1
1511.3		4	SS	20			o	11 55 (34)
7.0	Peat fine fibrous	5	SS	4	1510			
						+ s2.4		
		6	SS	2		+ s4.3		
1504.3		7	TW	PH		+ s5.3		
14.0	Clayey silt, occasional silt seams.	8	TW	PH	1500	+ s4.1		
	Stiff	9	TW	PH		+ s7.3		
1495.3								
23.0	End of Borehole				1490			

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH: o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 400 800 1200 1600 2000

LIQUID LIMIT — W_L PLASTIC LIMIT — W_P WATER CONTENT — W (W_P — W — W_L) — WATER CONTENT % 20 40 60

UNIT WEIGHT γ

20 / 15 5 / 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

ENGINEERING SERVICES BRANCH–GEOTECHNICAL OFFICE–SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 6

WP	910-75-01	LOCATION	Sta. 244 + 73 43' Lt.	ORIGINATED BY	AP
DIST 19	HWY 17 T.C.H.	BORING DATE	November 24, 1975	COMPILED BY	NT
DATUM	Geodetic	BOREHOLE TYPE	Hollow Stem Auger	CHECKED BY	[signature]

SOIL PROFILE: ELEV / DEPTH	DESCRIPTION	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	SHEAR STRENGTH (FIELD VANE, s = sensitivity) / UNCONFINED	WATER CONTENT %	UNIT WEIGHT γ P.C.F	REMARKS
1514.8	Ground Level								
0.0	Peat	1	SS	1	1510				w/c 634%
	Fine Fibrous	2	TW	PH		+ s2.5			
		3	TW	PH		+ s3.3; + s5.0			
1501.3		4	TW	PH	1500	+ s5.0			
13.5	Silty clay to clayey	5	SS	1/18"		+ s5.3; + s4.8	w_P–w_L, w		
	silt, clay layers,	6	TW	PH	1490	+ s7.7; + s4.8			
	occasional silt seams	7	TW	PH		o; + s6.0	w_P–w_L, w	114	
	Firm				1480	+ s8.7			
1470.3		8	TW	PH		+ s4·2; + s2.0			
44.5	End of Borehole				1470				

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH: o UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB VANE — 400 800 1200 1600 2000

LIQUID LIMIT — W_L; PLASTIC LIMIT — W_P; WATER CONTENT — W — 20 40 60

REMARKS % GR SA SI CL

20 / 15 10 5 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

ENGINEERING SERVICES BRANCH–GEOTECHNICAL OFFICE–SOIL MECHANICS SECTION

RECORD OF BOREHOLE № 7

WP 910-75-01 LOCATION Sta. 232 + 01 49' Lt. ORIGINATED BY AP

DIST 19 HWY 17 T.C.H. BORING DATE November 24, 1975 COMPILED BY NT

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY

SOIL PROFILE: ELEV / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER / ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 400 800 1200 1600 2000	LIQUID LIMIT W_L, PLASTIC LIMIT W_P, WATER CONTENT W / WATER CONTENT % 20 40 60	UNIT WEIGHT γ	REMARKS % GR SA SI CL
1515.3	Ground Level									
0.0	Peat									
			1	SS	1					w/c 757% Org. 90.2%
	Fine fibrous					1510	+ s2.2			
			2	TW	PH		+ s4.5			
1504.8			3	SS	5					
10.5	Clayey silt						+ s4.3			
			4	TW	PH					
	occasional silt seams		5	SS	8	1500				
							> +			
			6	TW	PH					
	Stiff						+ s5.4			
	Firm		7	SS	3	1490	+ s6.3			
							+ s15.3			
1485.8							+ s12.3			
29.5	End of Borehole									
						1480				

20
15 ◇ 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO

ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 8

WP 910-75-01 LOCATION Sta. 244 + 79 15' Lt. ORIGINATED BY AP

DIST 19 HWY 17 T.C.H. BORING DATE November 26, 1975 COMPILED BY NT

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY So

ELEV / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (400 800 1200 1600 2000) ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT W_L / PLASTIC LIMIT W_P / WATER CONTENT W — WATER CONTENT % (20 40 60)	UNIT WEIGHT γ	REMARKS % GR SA SI CL
1517.1	Ground Level									
0.0	Fill Silty Sand		1	SS	22	Estimated ∇		○		
			2	SS	8					
1510.6			3	SS	6	1510				
6.5	Peat Fine fibrous						+ s5.3 + s5.5			
1503.6			4	SS	–					
13.5	Clayey silt, occasional silt seams. Firm		5	SS	–			├─┤ ○		
						1500	+ s3.2 + s3.3			
			6	SS	2		+ s3.3 + s6.0			
1492.1										
25.0	End of Borehole					1490				

20
15 ⊕ 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

ENGINEERING SERVICES BRANCH–GEOTECHNICAL OFFICE–SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 9

WP 910-75-01 LOCATION Sta. 231 + 86 19' Lt. ORIGINATED BY AP

DIST 19 HWY 17 T.C.H. BORING DATE November 25, 1975 COMPILED BY NT

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER / ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 400 800 1200 1600 2000	LIQUID LIMIT W_L, PLASTIC LIMIT W_P, WATER CONTENT W / WATER CONTENT % 20 40 60	UNIT WEIGHT γ	REMARKS % GR SA SI CL
1517.5	Ground Level									
0.0	Fill Silty sand		1	SS	14	∇ Estimated				
1512.5			2	SS	2					w/c 194% Org.54.3%
5.0	Peat fine fibrous		3	TW	PH	1510	s4.3			
							s4.3			
			4	SS	1		s2.6			
1501.5										
16.0	Clayey silt, occasional silt seams. Stiff		5	TW	PH	1500	s5.1			
			6	SS	16		s6.3			
			7	TW	PH					
1489.5						1490	s3.3			
28.0	End of Borehole									
						1480				

20
15 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

ENGINEERING SERVICES BRANCH - GEOTECHNICAL OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 10

WP 910-75-01 LOCATION Sta. 244 + 79 16' Rt. ORIGINATED BY AP

DIST 19 HWY 17 T.C.H. BORING DATE November 25, 1975 COMPILED BY NT

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER / ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH	WATER CONTENT %	UNIT WEIGHT γ	REMARKS % GR SA SI CL
1516.2	Ground Level								
0.0	Fill Sand Fine to Med some silt				Estimated ∇				
1511.7	Compact silty sand	1	SS	12					
4.5	Peat Fine fibrous				1510				
		2	SS	2					
1502.7		3	SS	3					
13.5	Clayey silt, occasional silt seams.	4	SS	1	1500				
1497.2	Soft					+s3.8			
19.0	End of Borehole								
					1490				

Legend: LIQUID LIMIT — W_L; PLASTIC LIMIT — W_P; WATER CONTENT — W. Dynamic cone scale: 20 40 60 80 100. Shear strength: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 400 800 1200 1600 2000. Water content %: 20 40 60.

20
15 -○- 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

ENGINEERING SERVICES BRANCH - GEOTECHNICAL OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 11

WP 910-75-01 | LOCATION Sta. 273 + 51 19' Rt. | ORIGINATED BY AP

DIST 19 HWY 17 T.C.H. | BORING DATE November 26, 1975 | COMPILED BY NT

DATUM Geodetic | BOREHOLE TYPE Hollow Stem Auger | CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV
1513.1	Ground Level					Estimated ∇
0.0 / 1508.1	Fill Silty Sand Compact		1	SS	10	1510
			2	SS	4	
5.0 / 1502.4	Peat Fine fibrous		3	SS	3	
			4	SS	3	
10.7 / 1499.1	Clayey silt, occ. silt seams. Firm		5	SS	2	
			6	SS	2	1500
14.0	End of Borehole					1490

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

LIQUID LIMIT — W_L; PLASTIC LIMIT — W_P; WATER CONTENT — W

WATER CONTENT %: 20 40 60

UNIT WEIGHT γ

REMARKS: % GR SA SI CL

20 / 15 5 / 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

ENGINEERING SERVICES BRANCH–GEOTECHNICAL OFFICE–SOIL MECHANICS SECTION

RECORD OF BOREHOLE № 12

WP 910-75-01 | LOCATION Sta. 231 + 86 16' Rt. | ORIGINATED BY AP

DIST 19 HWY 17 T.C.H. | BORING DATE November 26, 1975 | COMPILED BY NT

DATUM Geodetic | BOREHOLE TYPE Hollow Stem Auger | CHECKED BY

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT. PLOT	SAMPLES – NUMBER	TYPE	'N' VALUES	GROUND WATER / ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	LIQUID LIMIT W_L, PLASTIC LIMIT W_P, WATER CONTENT W (%)	UNIT WEIGHT γ	REMARKS % GR SA SI CL
1517.6	Ground Level									
0.0	Fill Silty sand					Estimated ∇				
1513.6			1	SS	14					
4.0	Peat									w/c 192% Org.45.9%
			2	SS	3					
	Fine fibrous					1510				
			3	SS	2					
1503.6			4	SS	2					w/c 537.%
14.0	Clayey silt, occ.silt seams. Firm		5	SS	2					
1499.6			6	SS	4	1500				
18.0	End of Borehole									
						1490				

20 15 5 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO

ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 13

WP 910-75-01 | LOCATION Sta. 263 + 35 5' Lt. | ORIGINATED BY AP

DIST 19 HWY 17 T.C.H. | BORING DATE November 26, 1975 | COMPILED BY NT

DATUM Geodetic | BOREHOLE TYPE Hollow Stem Auger | CHECKED BY [signature]

SOIL PROFILE ELEV / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 400 800 1200 1600 2000	LIQUID LIMIT w_L PLASTIC LIMIT w_P WATER CONTENT w / WATER CONTENT % 20 40 60	UNIT WEIGHT γ	REMARKS % GR SA SI CL
1515.0	Ground Level									
0.0	Pavement gravel					Estimated ∇				
	Fill Silty sand traces of gravel		1	SS	24	1510		○		11 60 (29)
1507.3			2	SS	20			○		8 54 36 2
7.7	Peat Fine fibrous		3	SS	2					
			4	SS	3					
1501.0							+ s3.7			
14.0	Clayey silt, occ. silt seams		5	SS	3	1500	+ s2.2	├─┤		
	Stiff		6	SS	4					
1491.0							+ s6.0			
24.0	End of Borehole					1490				

15 ⌖ 5 (20 / 10) % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

ENGINEERING SERVICES BRANCH–GEOTECHNICAL OFFICE–SOIL MECHANICS SECTION

RECORD OF BOREHOLE № 14

WP 910-75-01 LOCATION Sta. 263 + 35 17' Lt. ORIGINATED BY AP

DIST 19 HWY 17 T.C.H. BORING DATE November 26, 1975 COMPILED BY NT

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT w_L / PLASTIC LIMIT w_P / WATER CONTENT w / WATER CONTENT % 20 40 60	UNIT WEIGHT γ	REMARKS % GR SA SI CL
1514.1	Ground Level									
0.0	Fill Silty sand					Estimated ∇				
			1	SS	14					
1508.1			2	SS	6	1510				
6.0	Peat Fine fibrous		3	SS	2					
1502.6			4	SS	1					
11.5 / 1500.1	Clayey silt, occ. silt seams. Firm		5	SS	3	1500				
14.0	End of Borehole									

20 / 15 ⌖ 5 / 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE № 15

WP 910-75-01 LOCATION Stn. 263 + 35 17' Rt. ORIGINATED BY AP

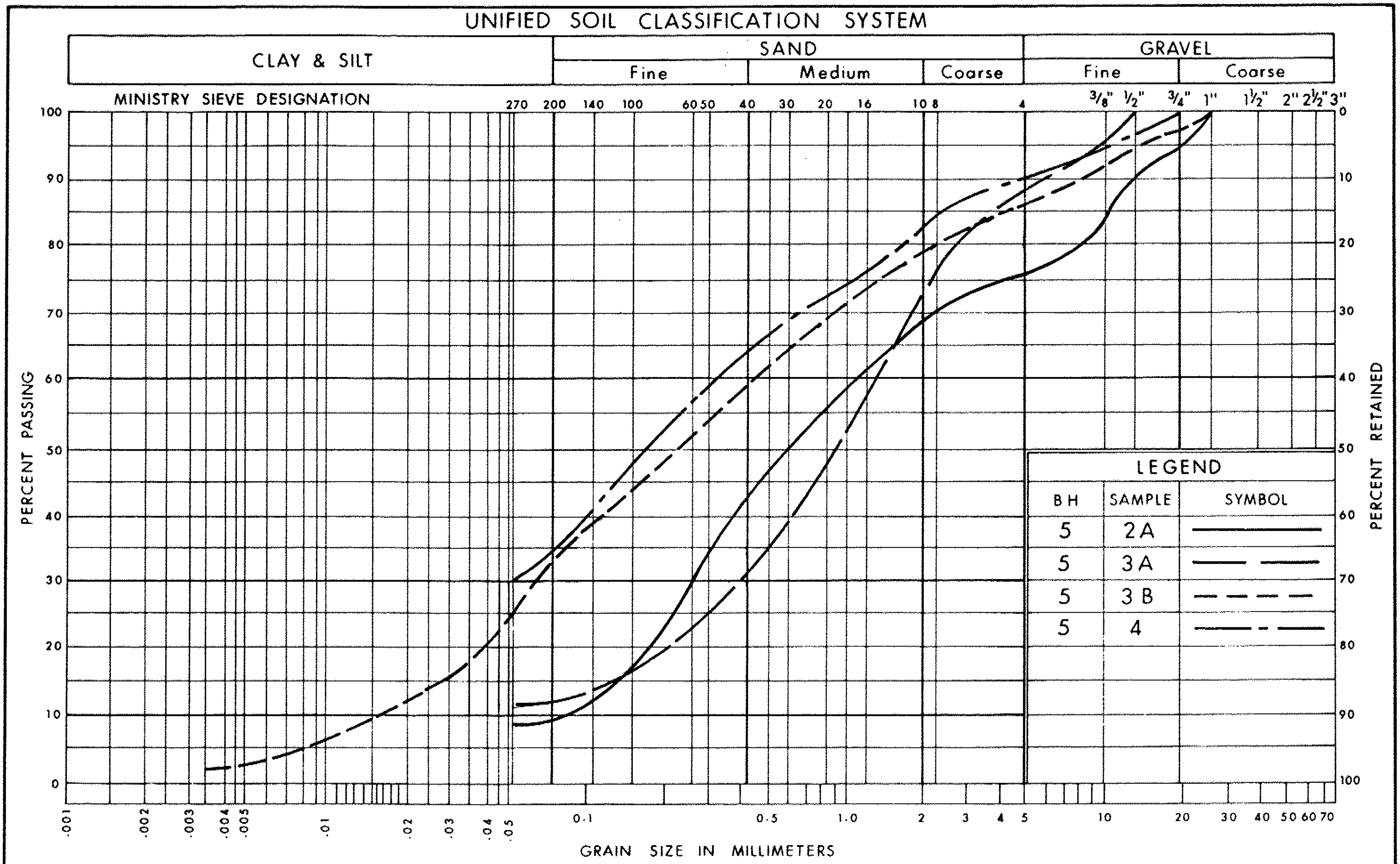
DIST 19 HWY 17 T.C.H. BORING DATE November 26, 1975 COMPILED BY NT

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY [signature]

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT — W_L PLASTIC LIMIT — W_P WATER CONTENT — W / WATER CONTENT % 20 40 60	UNIT WEIGHT γ	REMARKS % GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	ELEV				
1514.0	Ground Level									
0.0	Fill Silty Sand					Estimated ∇				
			1	SS	4	1510				
1509.0 / 5.0	Peat Fine Fibrous		2	SS	3					
1502.5 / 11.5	Clayey silt, occ. silt seams. Firm		3	SS	2					
1500.0 / 14.0	End of Borehole		4	SS	2	1500		W_P ≈ 20, W ≈ 40		

20 / 15 ⌖ 5 / 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION



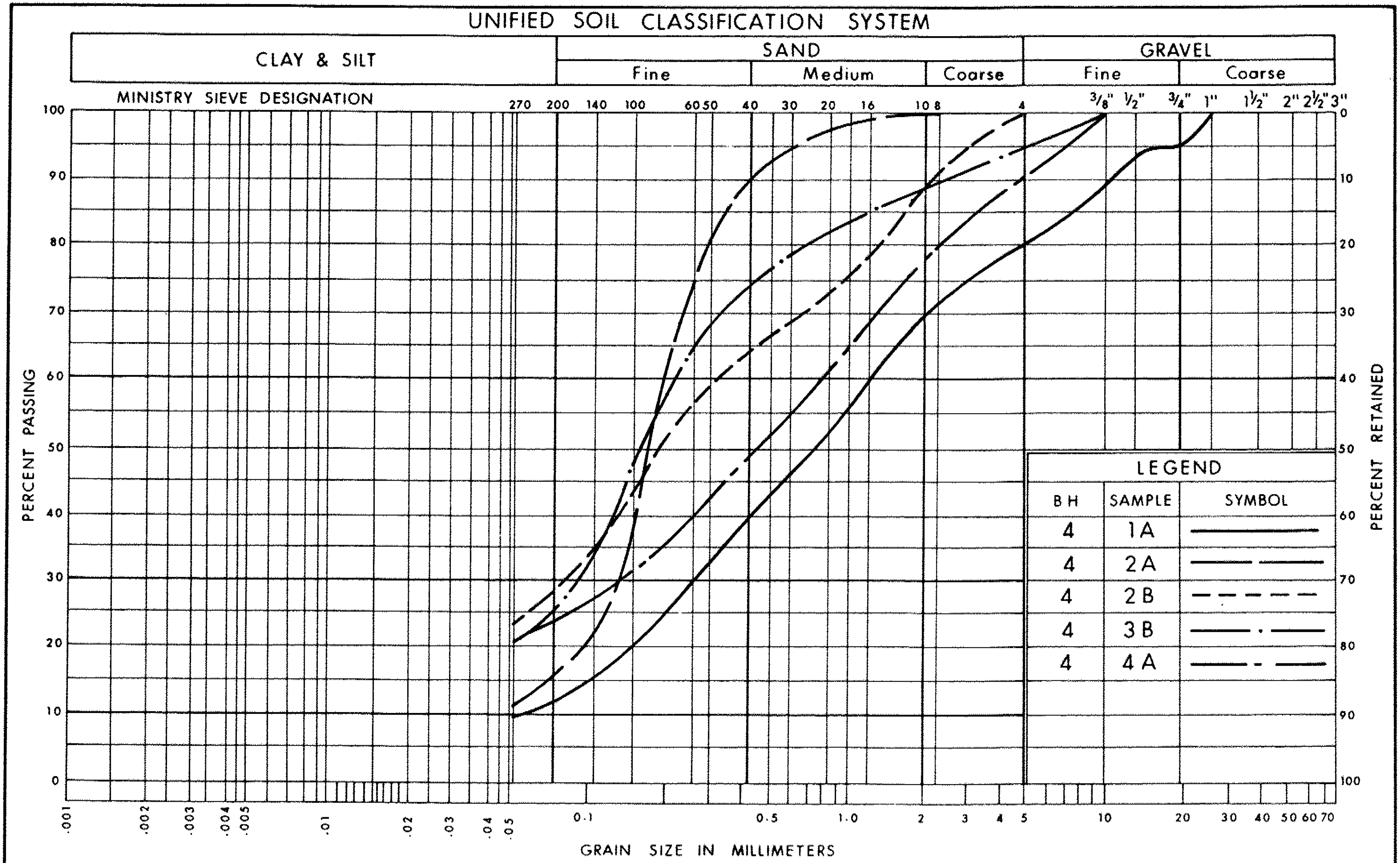
Ministry of Transportation and Communications
Ontario
ENGINEERING SERVICES BRANCH

GRAIN SIZE DISTRIBUTION

STA. 231+87
(FILL MAT'L)

FIG No 1

W P 910-75-01

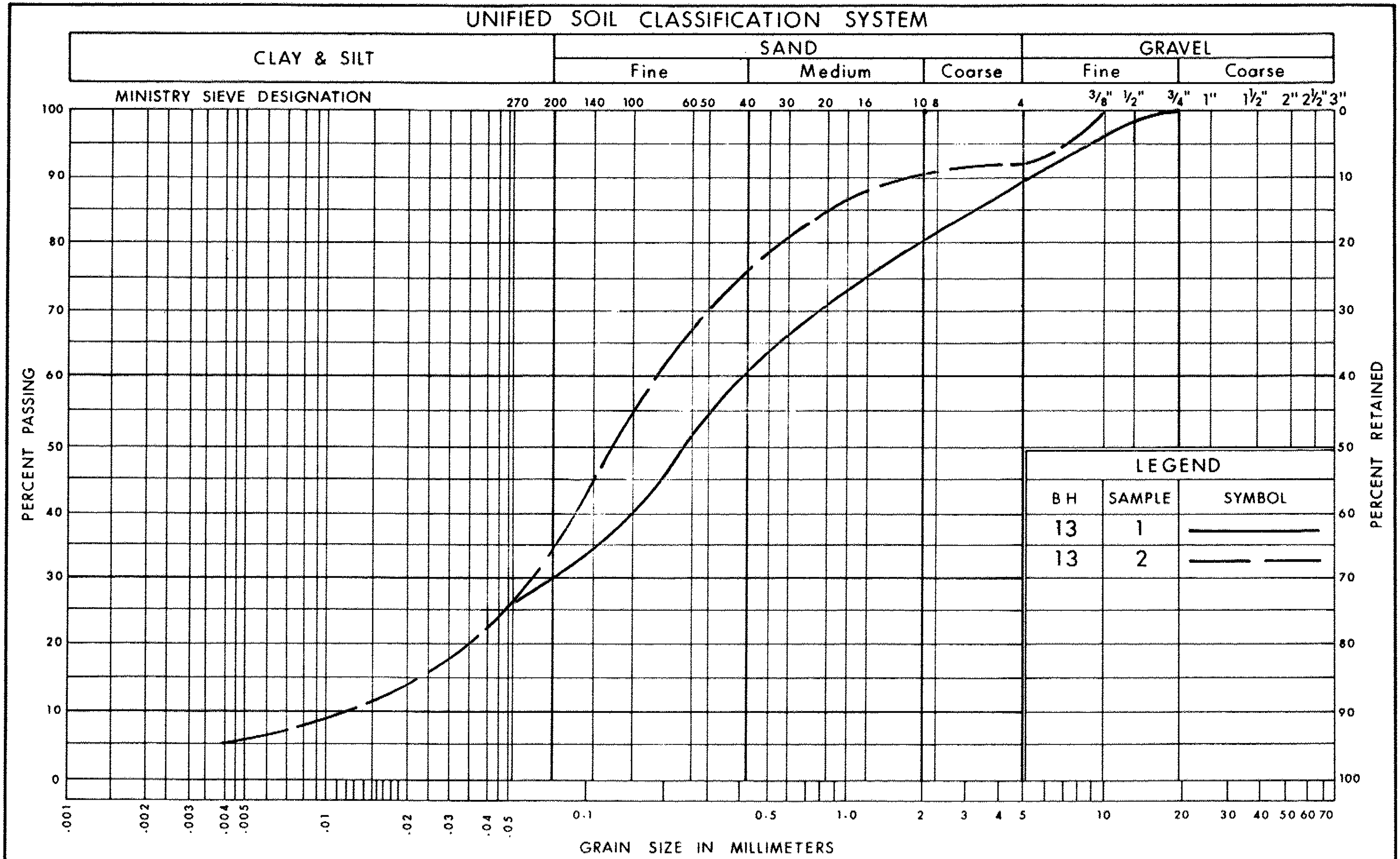


Ministry of Transportation and Communications
Ontario
ENGINEERING SERVICES BRANCH

GRAIN SIZE DISTRIBUTION
STA. 244 + 77
(FILL MAT'L)

FIG No 2

W P 910 - 75 - 01



Ministry of Transportation and Communications

Ontario

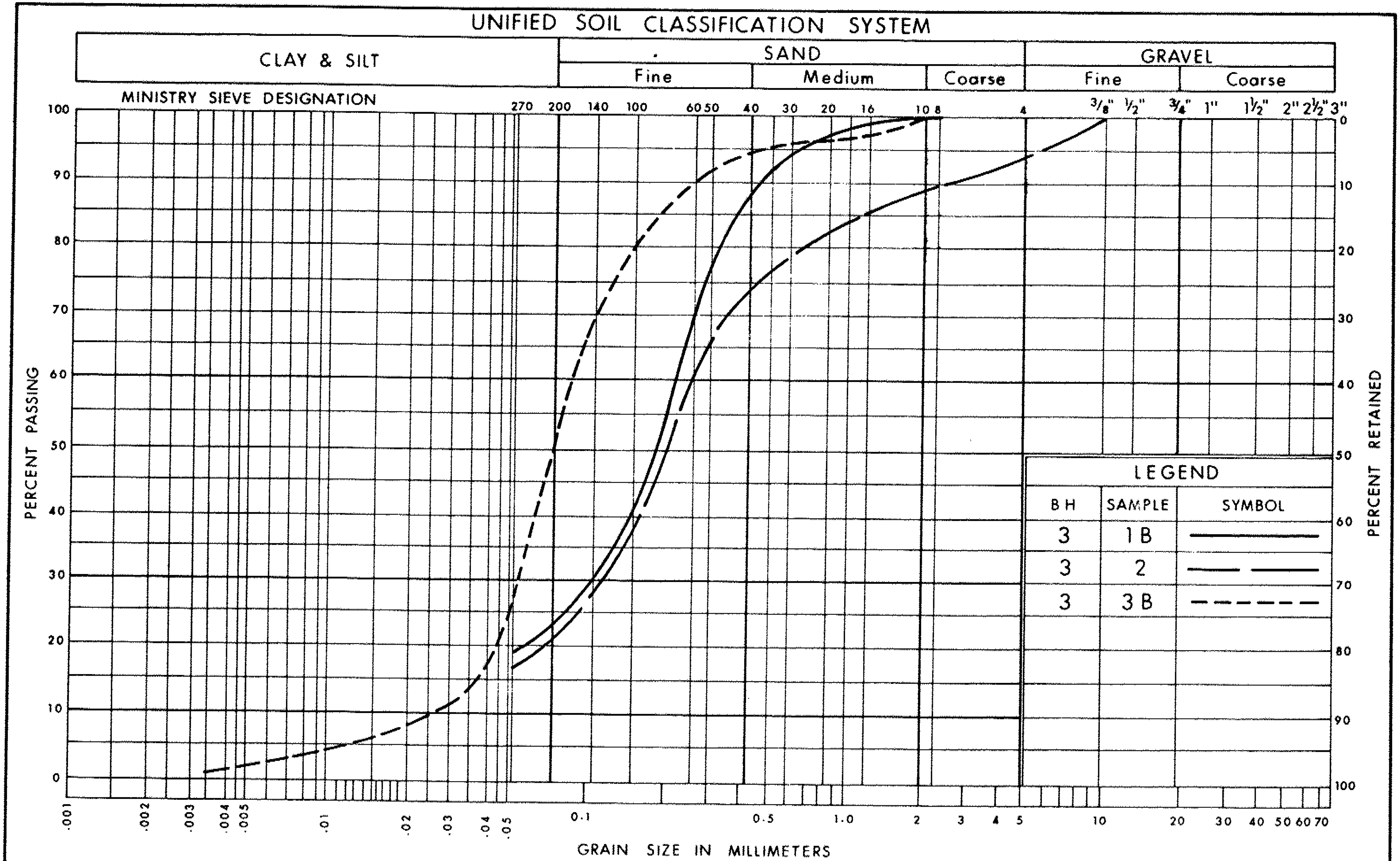
ENGINEERING SERVICES BRANCH

GRAIN SIZE DISTRIBUTION

STA. 263 + 35

(FILL MAT'L.)

FIG No	3
W P	910 - 75 - 01

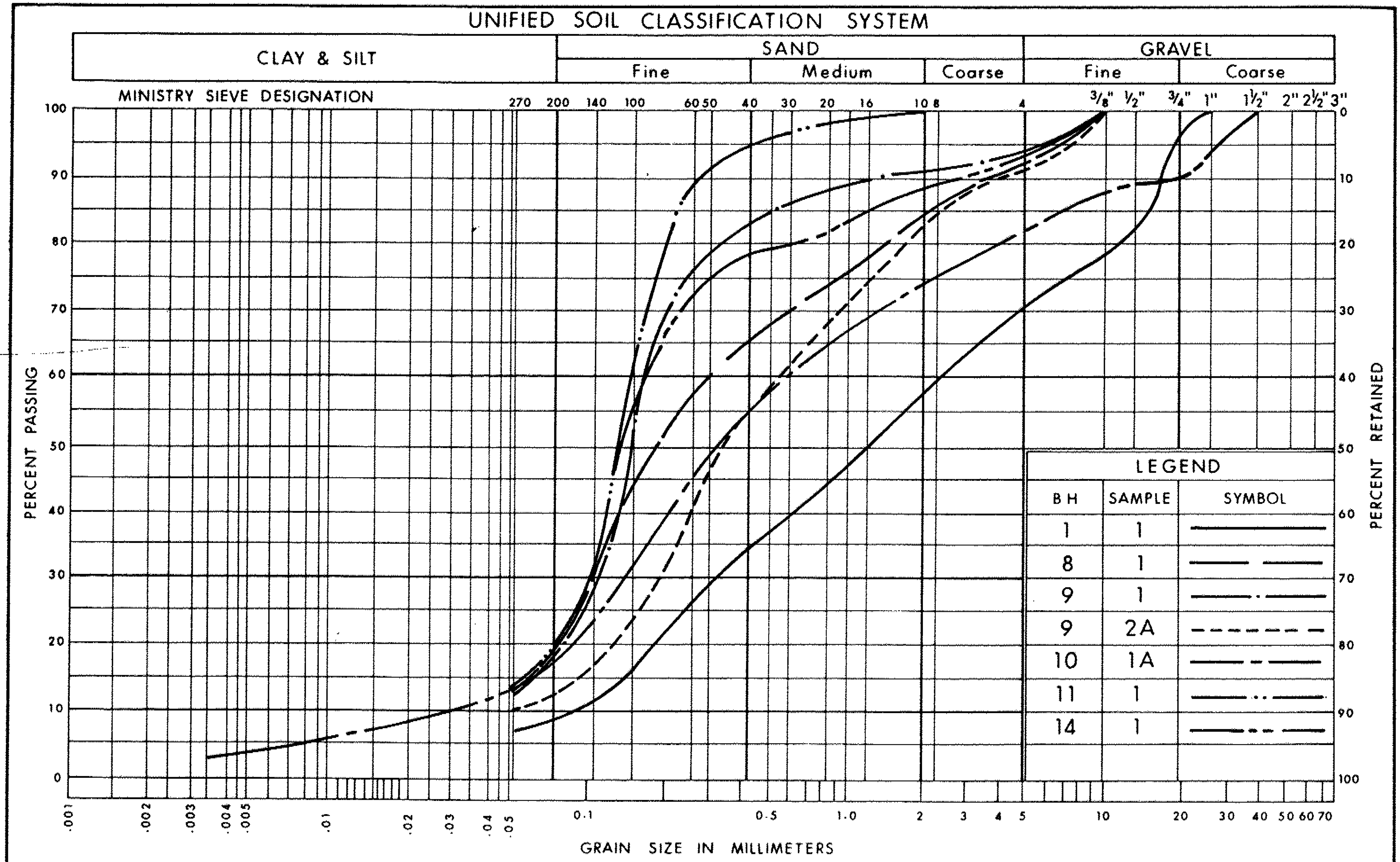


Ministry of Transportation and Communications
Ontario
ENGINEERING SERVICES BRANCH

GRAIN SIZE DISTRIBUTION
STA. 273+51
(FILL MAT'L)

FIG No 4

W P 910-75-01



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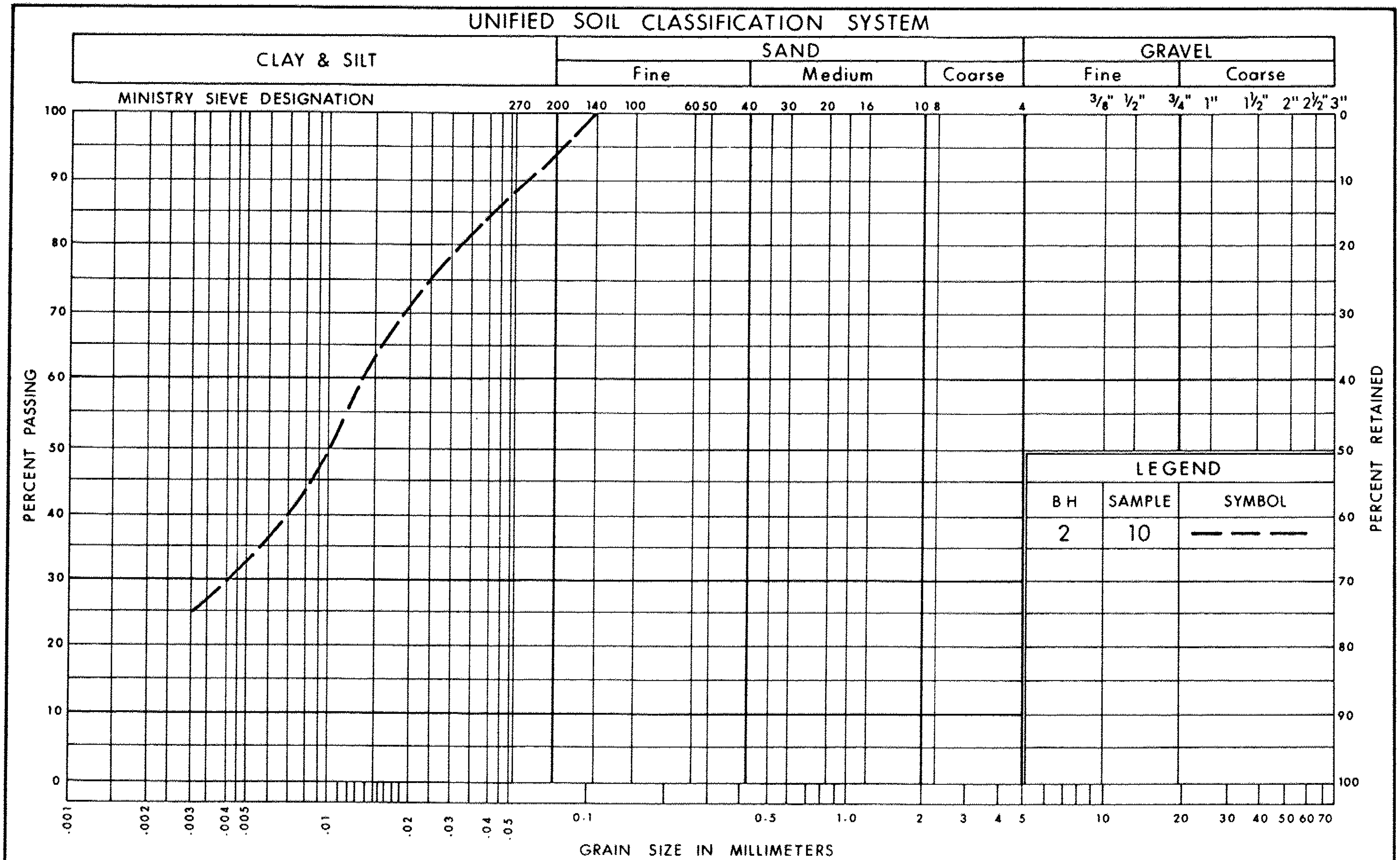
ENGINEERING SERVICES BRANCH

GRAIN SIZE DISTRIBUTION
FILL MATERIAL ON SHOULDER
(FILL MAT'L)

FIG No	5
W P	910-75-01



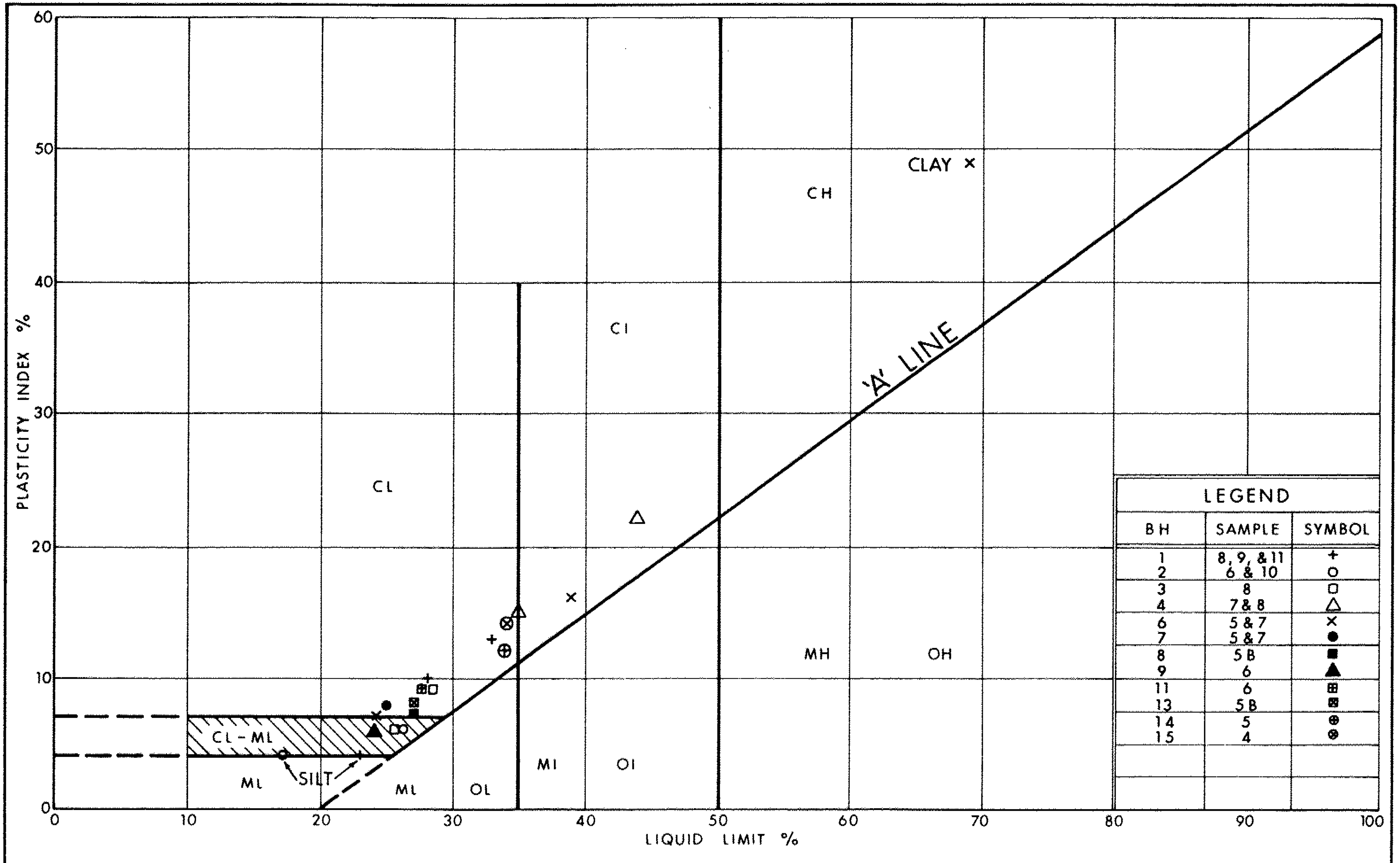
Ministry of Transportation and Communications
Ontario
ENGINEERING SERVICES BRANCH

GRAIN SIZE DISTRIBUTION

STA. 273 + 51
(SILT)

FIG No 6

W P 910 - 75 - 01



Ministry of Transportation and Communications
Ontario
ENGINEERING SERVICES BRANCH

PLASTICITY CHART
CLAYEY SILT
(TO SILTY CLAY IN BH'S 4 & 6 ONLY)

FIG No	7
W P	910 - 75 - 01

VOID RATIO – PRESSURE CURVES

JOB NO. 910 - 75 - 01

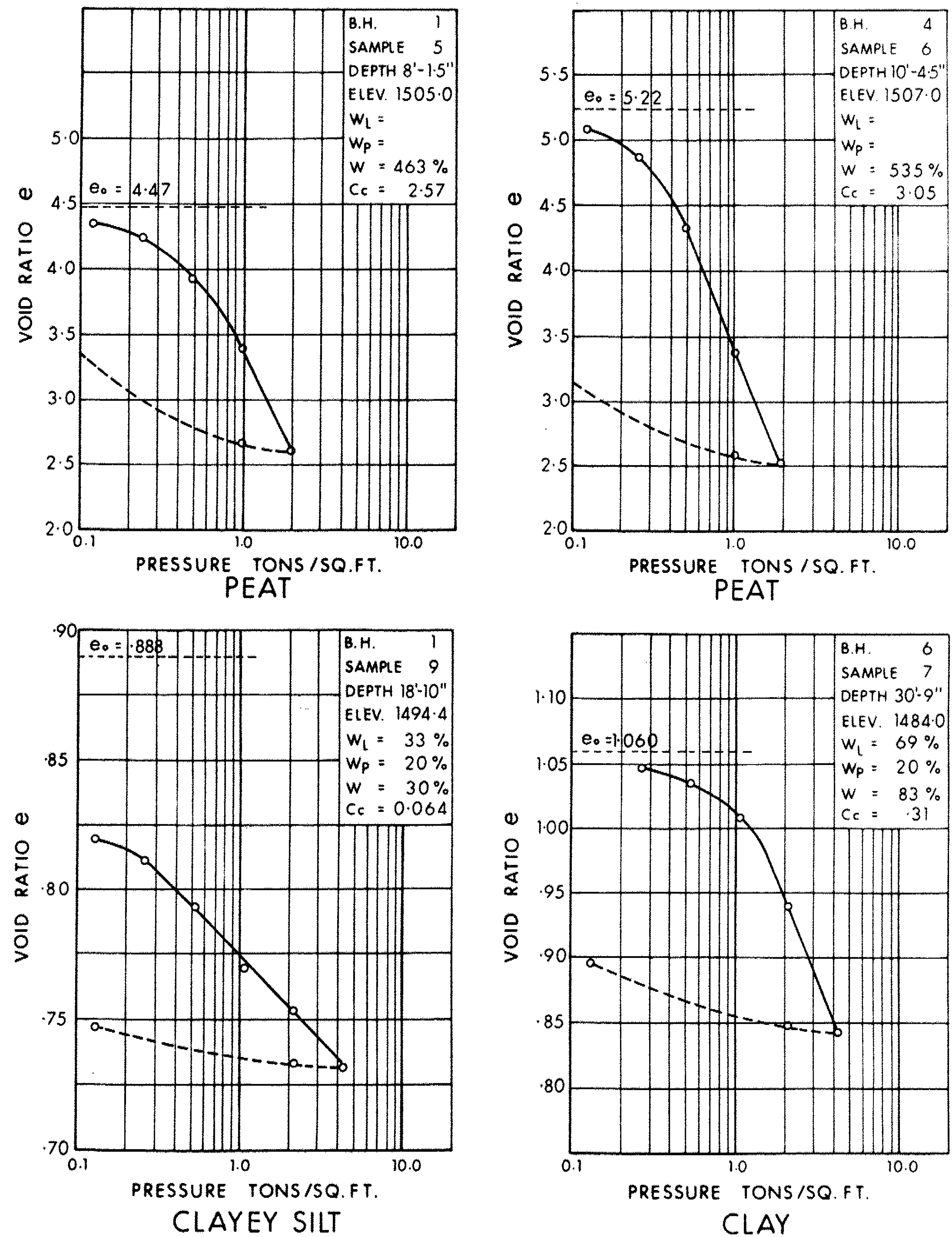


FIG. 8

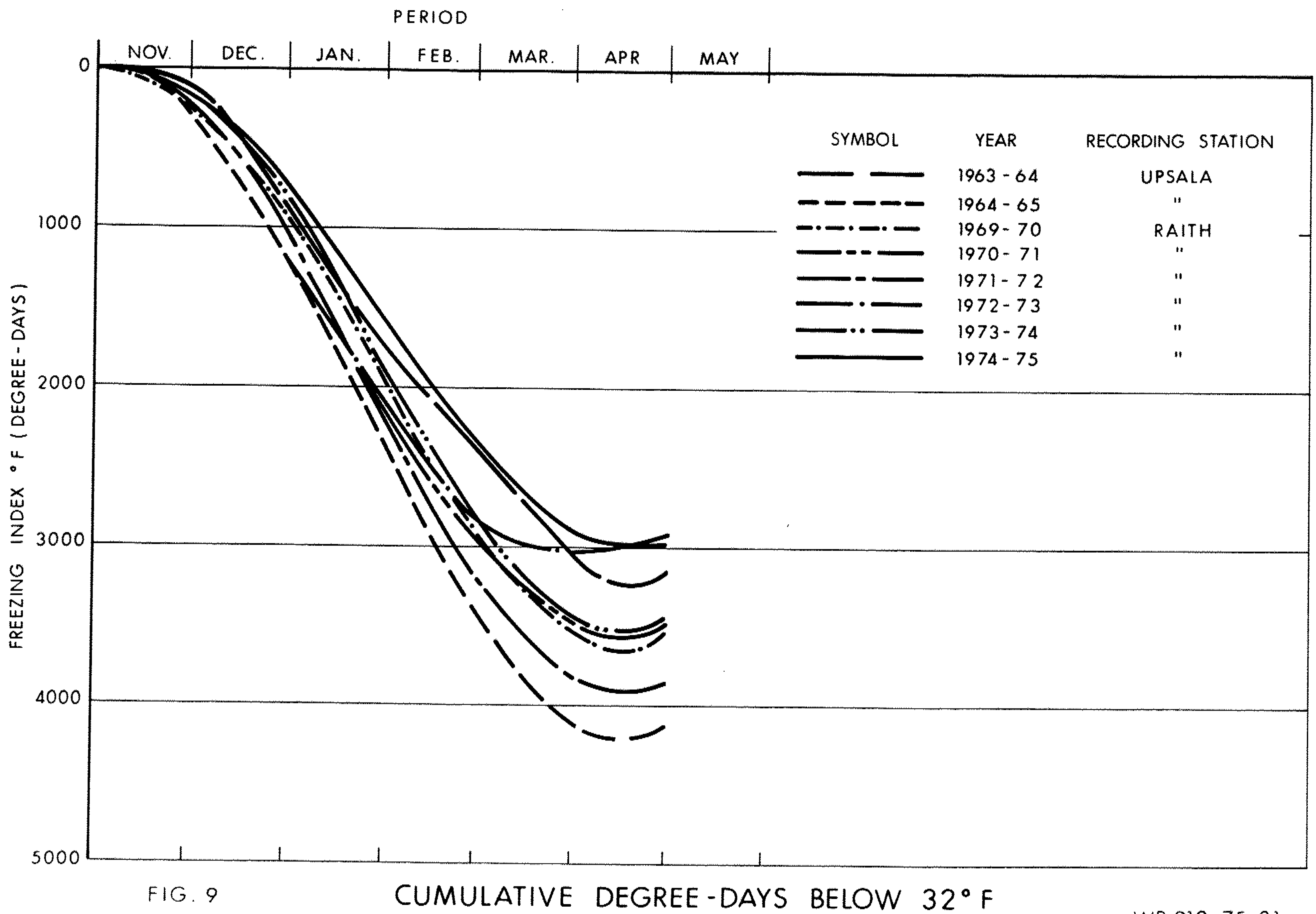
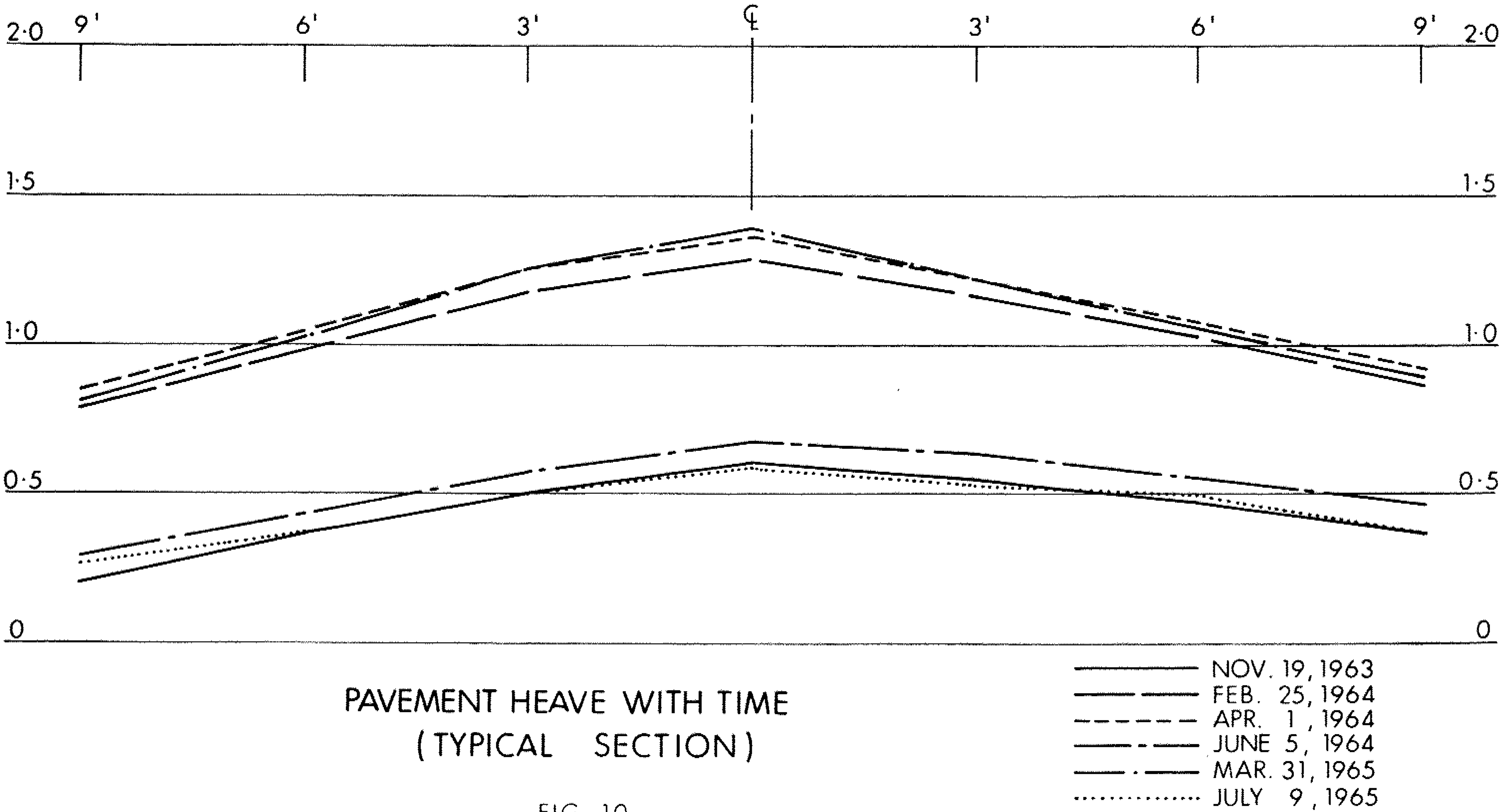


FIG. 9

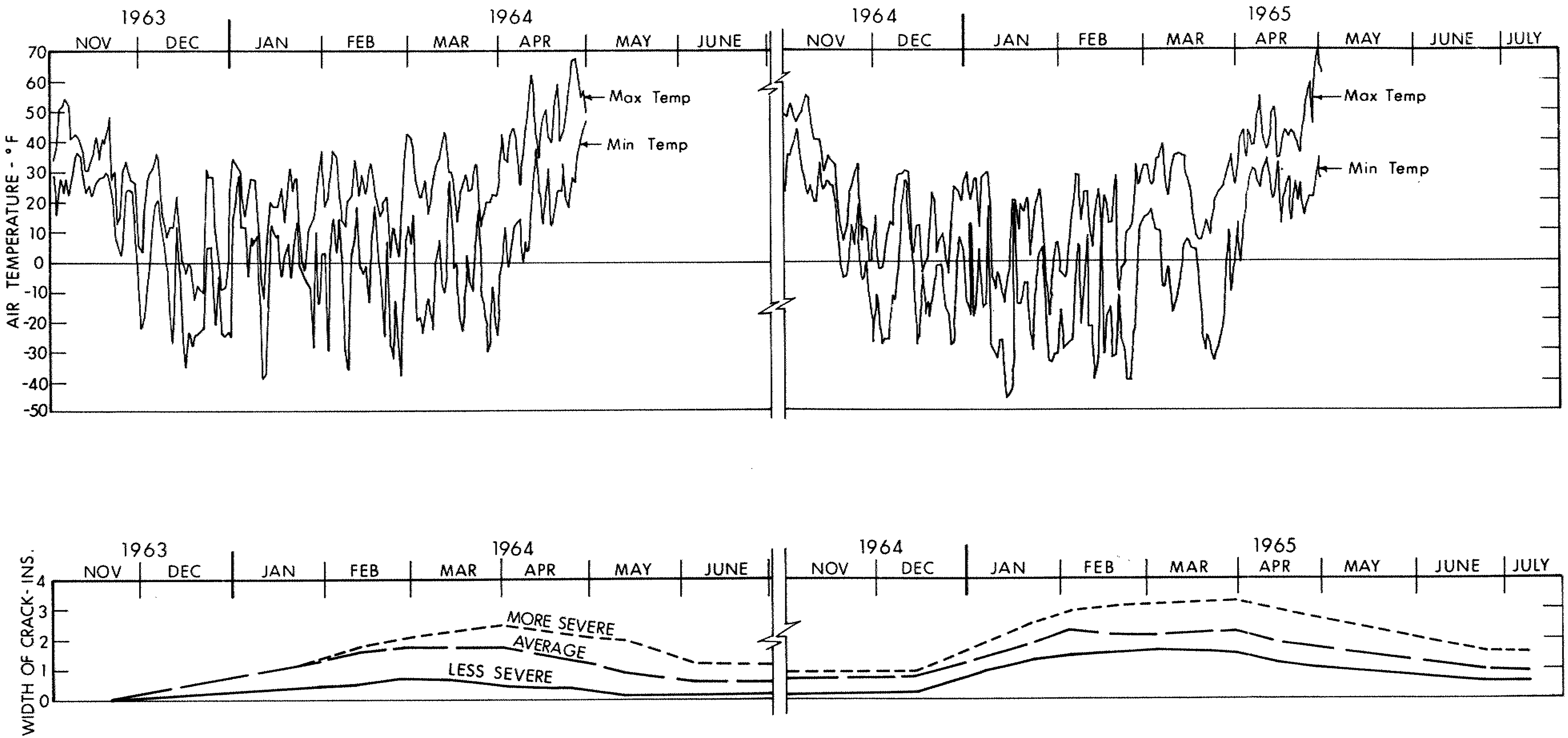
WP 910-75-01



PAVEMENT HEAVE WITH TIME
(TYPICAL SECTION)

FIG. 10

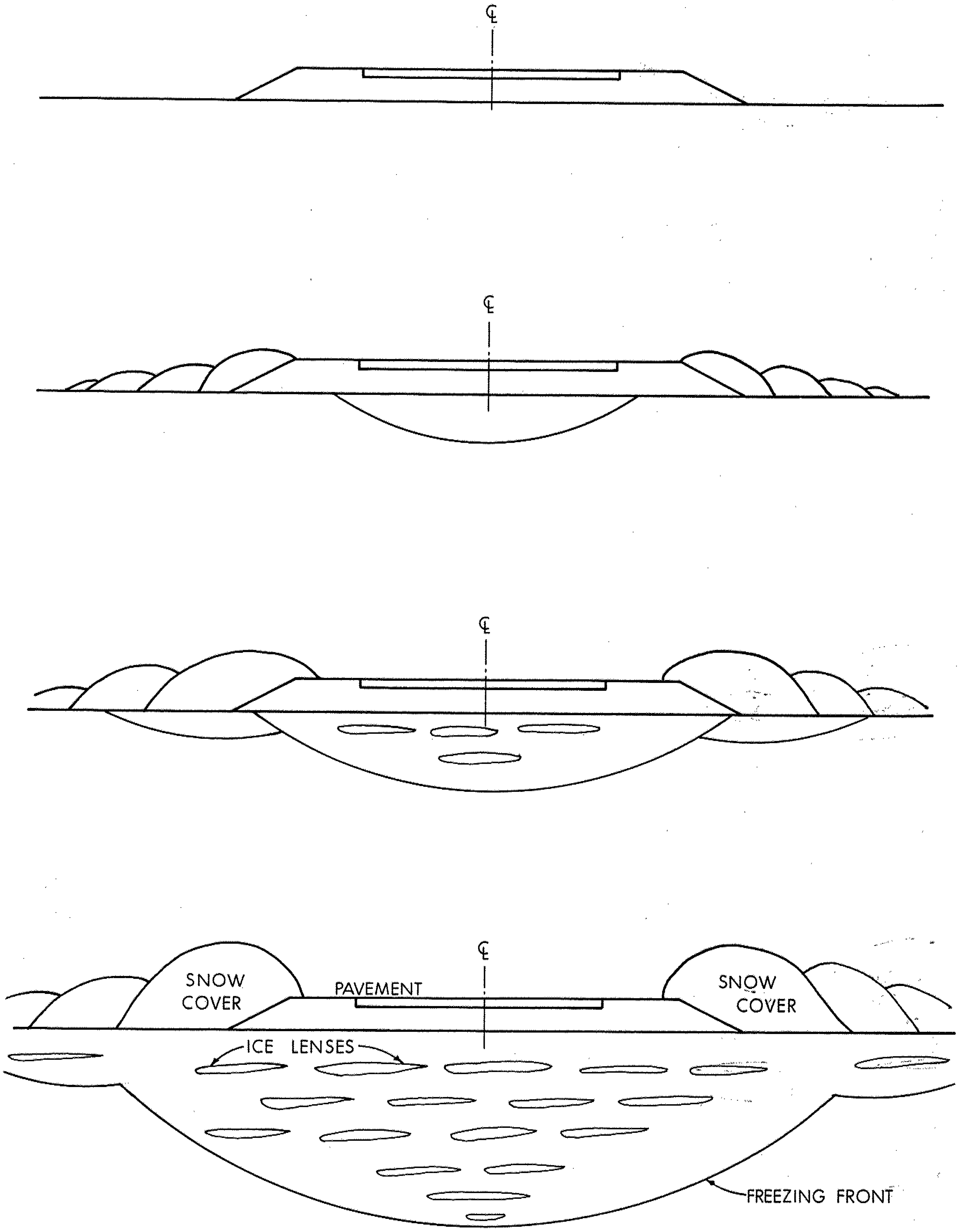
W.P. 910-75-01



WIDTH OF CRACK vs TIME

FIG. 11

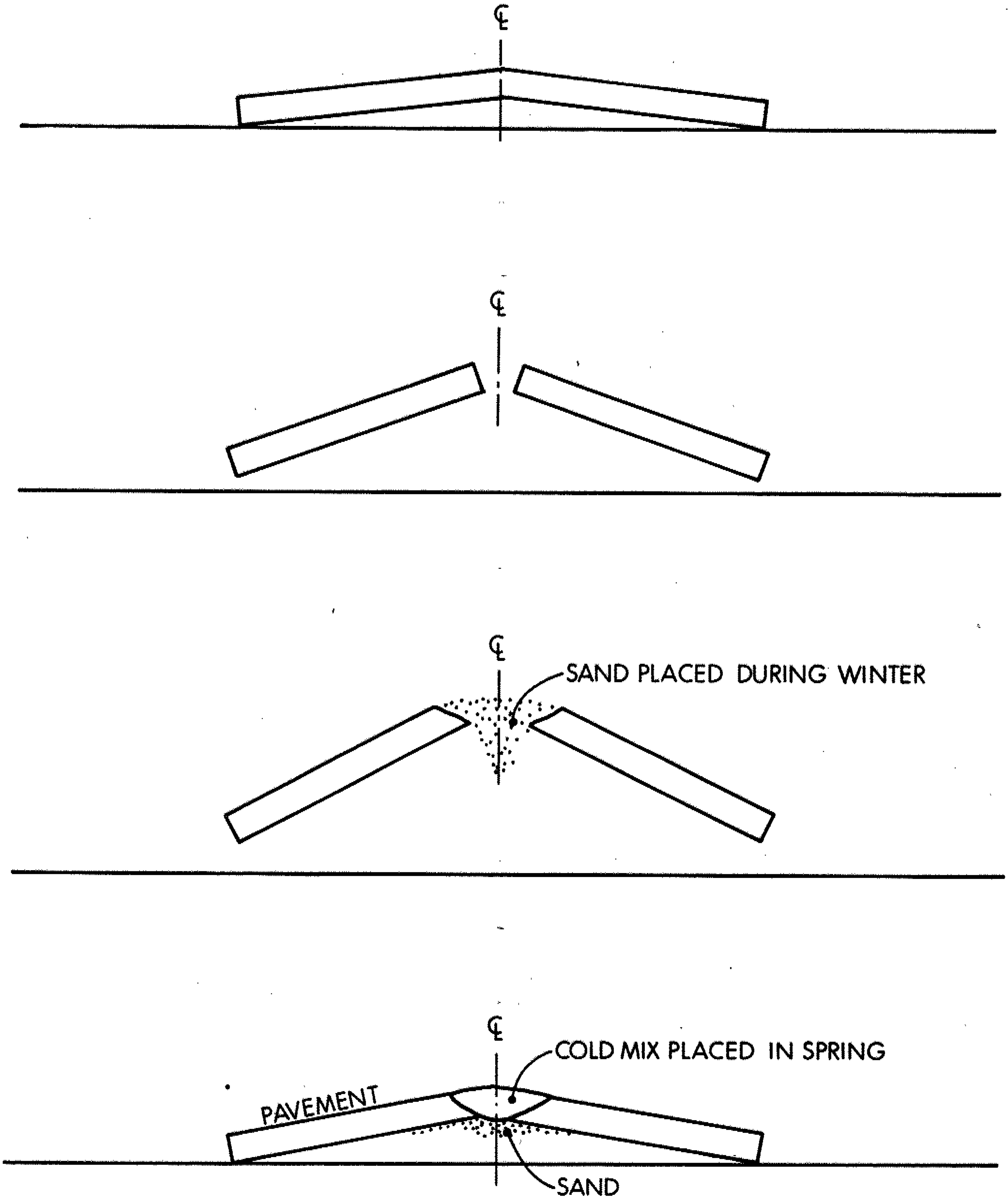
WP 910-75-01



MECHANISM OF DIFFERENTIAL FROST HEAVE

FIG. 12

WP 910-75-01



PROCESS OF WIDENING OF LONGITUDINAL CRACK

FIG. 13

FF-A-24(b) (Rev. Jan.73)

ABBREVIATIONS & SYMBOLS USED IN THIS REPORT

SOIL PROPERTIES

γ UNIT WEIGHT OF SOIL (BULK DENSITY)

γ_s UNIT WEIGHT OF SOLID PARTICLES

γ_w UNIT WEIGHT OF WATER

γ_d UNIT DRY WEIGHT OF SOIL (DRY DENSITY)

γ' UNIT WEIGHT OF SUBMERGED SOIL

G SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$

e VOID RATIO

n POROSITY

w WATER CONTENT

S_r DEGREE OF SATURATION

w_L LIQUID LIMIT

w_P PLASTIC LIMIT

I_P PLASTICITY INDEX

w_S SHRINKAGE LIMIT

I_L LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$

I_C CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$

e_{max} VOID RATIO IN LOOSEST STATE

e_{min} VOID RATIO IN DENSEST STATE

I_D DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$

RELATIVE DENSITY D_r IS ALSO USED

h HYDRAULIC HEAD OR POTENTIAL

q RATE OF DISCHARGE

v VELOCITY OF FLOW

i HYDRAULIC GRADIENT

k COEFFICIENT OF PERMEABILITY

j SEEPAGE FORCE PER UNIT VOLUME

m_v COEFFICIENT OF VOLUME CHANGE $= \frac{-\Delta e}{(1+e)\Delta\sigma'}$

c_v COEFFICIENT OF CONSOLIDATION

C_c COMPRESSION INDEX $= \frac{\Delta e}{\Delta \log_{10} \sigma'}$

T_v TIME FACTOR $= \frac{c_v t}{d^2}$ (d, DRAINAGE PATH)

U DEGREE OF CONSOLIDATION

τ_f SHEAR STRENGTH

c' EFFECTIVE COHESION INTERCEPT

ϕ' EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION

} IN TERMS OF EFFECTIVE STRESS $\tau_f = c' + \sigma' \tan \phi'$

c_u APPARENT COHESION

ϕ_u APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION

} IN TERMS OF TOTAL STRESS $\tau_f = c_u + \sigma \tan \phi$

μ COEFFICIENT OF FRICTION

S_t SENSITIVITY

GENERAL

π $= 3.1416$

e BASE OF NATURAL LOGARITHMS 2.7183

$\log_e a$ OR $\ln a$ NATURAL LOGARITHM OF a

$\log_{10} a$ OR $\log a$ LOGARITHM OF a TO BASE 10

t TIME

g ACCELERATION DUE TO GRAVITY

V VOLUME

W WEIGHT

M MOMENT

F FACTOR OF SAFETY

STRESS AND STRAIN

u PORE PRESSURE

σ NORMAL STRESS

σ' NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)

τ SHEAR STRESS

ε LINEAR STRAIN

γ SHEAR STRAIN

ν POISSON'S RATIO (μ IS ALSO USED)

E MODULUS OF LINEAR DEFORMATION (YOUNGS MODULUS)

G MODULUS OF SHEAR DEFORMATION

K MODULUS OF COMPRESSIBILITY

η COEFFICIENT OF VISCOSITY

EARTH PRESSURE

d DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE

δ ANGLE OF WALL FRICTION

K DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS

K_o COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

B BREADTH OF FOUNDATION

L LENGTH OF FOUNDATION

D DEPTH OF FOUNDATION BENEATH GROUND

N DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY. DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY

k_s MODULUS OF SUBGRADE REACTION

SLOPES

H VERTICAL HEIGHT OF SLOPE

D DEPTH BELOW TOE OF SLOPE TO HARD STRATUM

β ANGLE OF SLOPE TO HORIZONTAL

ABBREVIATIONS & SYMBOLS USED IN THIS REPORT

PENETRATION RESISTANCE

'N'=STANDARD PENETRATION RESISTANCE : - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE :- THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL, THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

CONSISTENCY	c LB./SQ. FT.
VERY SOFT	0 - 250
SOFT	250 - 500
FIRM	500 - 1000
STIFF	1000 - 2000
VERY STIFF	2000 - 4000
HARD	> 4000

DENSENESS	'N' BLOWS / FT.
VERY LOOSE	0 - 4
LOOSE	4 - 10
COMPACT	10 - 30
DENSE	30 - 50
VERY DENSE	> 50

TERMS TO BE USED IN DESCRIBING SOILS:-

TRACE < 10% , SOME 10-25% , WITH 25-40% , > 40% SILTY, SANDY, GRAVELLY, CLAYEY ETC.

TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.T.	SLOTTED TUBE SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE

P.H. SAMPLE ADVANCED HYDRAULICALLY

P.M. SAMPLE ADVANCED MANUALLY

SOIL TESTS

U	UNCONFINED COMPRESSION
UU	UNCONSOLIDATED UNDRAINED TRIAXIAL
CIU	CONSOLIDATED ISOTROPIC UNDRAINED TRIAXIAL
CID	" " DRAINED "
CAU	" ANISOTROPIC UNDRAINED "
CAD	" " DRAINED "
L.V.	LABORATORY VANE
F.V.	FIELD VANE
C	CONSOLIDATION
S	SENSITIVITY

Soils Design Report

Hwy. #17 (TCH) Raith Westerly to Poland 9.5 miles Proposed grading, culverts and GBC Contract work Project 912-58.

Surveys Profiles No. C 1221-1 Composite Soils Profile 17 M 18.
C 1221-2
C 1264-2

General Data

This proposed contract in Fort William district is situated about 60 miles west of the Lakehead on the Trans Canada Highway. No. 17.

The project involves grading in the Townships of Golding, Robson & Fallis. It is the only remaining stretch of primed surface between Fort William and Ignace excluding contract 58-147 which is at present under construction. The limits are the west end of contract 58-147 (Sta 930/00) in the east and the east end of contract 50-74 (Sta 1424/00) in the west.

This project has previously been tentatively listed several times but policy decisions on the method of treatment of the extensive swamps and more urgent work elsewhere have delayed it until now.

No realignments are proposed and no earth excavation on existing highway outside of ditching is proposed. The present gradeline is very low and the performance of the primed surface is poor. An average of 24" of granular is recommended throughout to correct these conditions. No structures are proposed on this work project. The present timber bridge at Sta. 1070/90 will probably be replaced by a 20' x 8' concrete culvert.

Investigations:

The first investigation concerned with this project was done in 1952 by power auger and depths of gravel and fill material were determined at that time. Hand soundings by peat sampler were done outside the roadbed width to determine the depths of muck and the nature of the underlying mat. A granular investigation was done in 1952 and brought up to date in July, 1958. An investigation of possible borrow sources was done in 1955 as a basis for comparing the costs of "riding" or excavating the muck.

Performance:

The present primed surface varies from 20' to 22' wide with 6' shoulders. It is understood that the new pavement will be 22' wide. No actual failures have been noted but small transverse and longitudinal cracks sometimes occur. Roughness and settlements nessitate the oiling and priming of the surface each year and potholes are very much in evidence each spring.

Some very limited research was done by power auger this year in order to predict the expected performance of this portion after it is paved.

Two swampy areas further west of Poland chosen where depths of muck and underlying soil type corresponded closely with those which are encountered between Raith and Poland. Peat moistures and degree of decomposition of the muck were noted and compared with those of a typically good and a typically bad area. The intermediate results obtained between Raith and Poland suggest that we can expect a limited amount of dishing & cracking of the pavement but it will not approach the poor performance noted west of Upsala.

Physiography & Soils Data:

The whole flat area was probably once the flood plain of an older counter part of the present Savanne River. It is composed of fine sand and silt now largely covered by from 5 to 10' of fibrous muck. Rock and glacial drift outcrops are extremely rare but where the Savanne River approaches the highway at Argon there are exposures of sand and silt.

The flat fibrous muskeg area is very imperfectly drained & free water is visible at the surface in several locations. The vegetation varies from dense spruce in drier areas to sparse alders with lichens & mosses in the wet areas. Alders are very prevalent in the ditch line.

The swamps are too extensive to list (covering most of the length of the project) but they are all quite fibrous and only slightly decomposed. The maximum depth of muck is about 15'. The natural moisture content of the material is in the range of 250-450%.

Borrow Materials:

Practically all material used on this project will be borrowed since there is a negligible amount of acceptable ditching material and no cuts.

Three distinct types of earth borrow are available within reasonable haul distance of the project.

1) Fine to very fine sand. Deposits are situated at Argon where 40,000 yards are available within one mile of the highway also at linko where some 30,000 yds. are available within ½ mile of the highway.
2) Sandy loam till. Unlimited quantities are available at Raith and a large quantity is also available at the west end.
3) Silty materials. These are available at Argon but use of this material for shouldering is definitely not recommended (see Recommendations #4).

Granular Materials:

Besides the earth borrow listed above there are three prospective areas for granular materials.

a) A very extensive granular area on Great Lakes Pulp & Paper company property. Four existing pits are known. All have excellent available access roads.

1) 3 miles from Sta. 1150 up the Dog River Road. Some 75,000 yds. of material suitable for GBC "A", "B" and 5/8" crushed.

2) 4 miles from Sta. 1150 on the Dog River Road. 50,000 yards of coarse gravel suitable for crushing.

3) 5 miles from Highway #17 on the Dog River Road. 100,000 yards of sandy gravel acceptable as GBC "B".

4) 3 miles from Highway #17 at Sta. 947 on the Abitibi Woodlands Laboratory road. A very large gravel area containing material suitable for GBC "A", "B" & 5/8 crushed (This deposit probably is on Abitibi property).

b) A very extensive esker on Abitibi road CP 118, 4 miles south of Sta. 947. The esker is 50' wide & 10' high & contains sand & coarse gravel. (A portion of this esker is in Block #2 of Grand Trunk Pacific property and may not be controlled by Abitibi). Part of the haul road here will require maintenance if trucks are used over it but otherwise the access is in good shape.

c) The only other known deposit suitable for use as GBC "B" is situated ½ mile north of highway #17 at Sta. 1112. It is believed that some 25,000 yards are immediately available in addition to the fine sand deposits mentioned earlier.

Pulp and Paper companies control the mineral rights to most of the pits listed above and it is likely that they will request 10¢ per yard for the use of any material from these deposits. Similar changes may be levied for borrow taken from outside the highway right of way.

Gradeline:

A gradeline for this project was set by Soils Branch in 1953 but a recently-taken series of cross sections initiated some further minor grade raises. From 18" to 24" of granular are proposed over the existing road and this should then provide a minimum of 36" above the water table in the surrounding swamps

No appreciable quantities of excavated muck are anticipated so that no special provisions are required to cope with the problem of disposal of the muck. The only muck excavation will be that required for ditching This should be cast away from the embankment.

Construction Features:

It was originally proposed that the centre line should be shifted 50' transversely and the muck could then be excavated without interfering with traffic. Extreme shortage of swamp backfill material has resulted in the abandonment of this treatment. Planning Branch has further proposed a shift of centreline to maintain the north ditch line.

Since this ditch line is completely overgrown (except in limited lengths where some recent maintenance work was carried out) no advantage in maintaining its present location by a shift in centre line would occur. A major disadvantage in any centre-line shift would be that a portion of the new section would be on the existing highway and a portion on virgin muskeg. This would present a serious soils problem.

Recommendations:

Agreed T.P. Cashin (Sand) May 23 1959

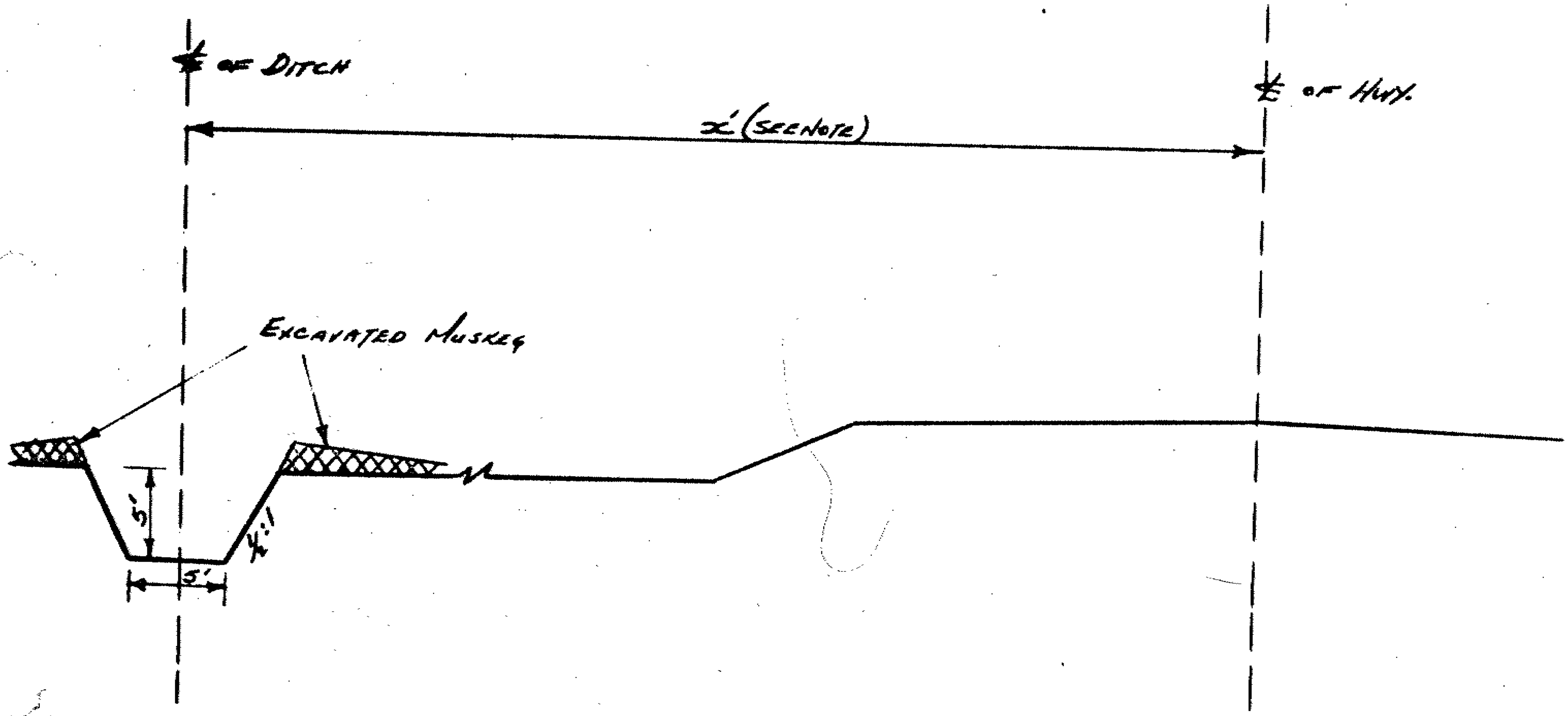
1) It is recommended that the contract be called as GBC Class "B" with 6" of GBC Class "A".
2) It is recommended that the present centre-line be maintained.
3) It is recommended that granular only be used for 28' central core.
4) It is recommended that only acceptable earth material be used for shouldering.
5) Shoulder material is chiefly non-cohesive, hence French Drains should be used in sags only, unless the engineer decides more are required.
6) Existing ditches should be grubbed to remove as much living vegetation as possible before any grading is done.
7) A special section (see attached sketch) is recommended for use between Sta. 980 and Sta. 1104 and also between Sta. 1328 & Sta. 1424. The areas involved are frequently inundated and it is therefore proposed that an auxiliary graded ditch be constructed. It is further proposed that offtake ditches to the roadside ditch be dug at 500' intervals. This special treatment will only be used on the south side of the highway (away from the railroad) but if distances between transverse culverts exceed 1000' additional culverts should be placed at every second offtake ditch.
8) Close cut clearing could be used in lieu of grubbing in the muskeg areas.
9) All excavated muck removed from ditches should be cast beyond the ditches and levelled out. It should not be brought against the backslopes.
10) Due to the high acidity of the water (pH 4.5 to 5.5) all CIP's used should be asphalt coated.
11) a) A 20' x 8' concrete box culvert is suggested for use at this Sta. 1070/90. The subgrade material is a fine silty loam and no foundation problem exists.
b) The culvert at Sta. 1228/00 should either be a flexible type culvert (multiplate if necessary) or a piled structure since the foundation mat'l tends to be soft.
c) The timber structure shown on the profile at Sta. 1405/80 has since been replaced by a perfectly adequate concrete slab on timber piles & rock fill.
12) All compaction will be by 6" layer method. Where bouldery tills are used for shouldering boulders shall be bladed out into the slopes and the boulder-free material compacted.

A.C. Powell
Soils Engineer

ACP/cl

... FOR LONGITUDINAL DITCH ON LEFT

STA 980 TO 1104 AND 13[illegible] TO 1424



NOTE: DISTANCE "x" WILL DEPEND ON R.O.W. - PREFERABLE DISTANCE IS 100' - MINIMUM DISTANCE TO BE 75'.

ACP
Pete Arkema will send you a drainage p'em map which will aid you in laying out the drainage system in lieu of off-take ditch. Please inform
H. Harrell

ACP
The reason for 75' to 100' offset for the ditch are:
(1) To maintain the mat
(2) Possible traffic hazard of big ditch close to Hwy.

ONTARIO
DEPARTMENT OF HIGHWAYS

Memo to Mr. H.D. McMillan
Assistant Rd. Design Engr.

Date August 15, 1958.

Subject Re: WP 912-58; Raith to Poland; Hwy. #17 T.C.H

From Materials & Research Section

Attached find the soils design report for this project. Composite soils profile 17 M 18 is also included.

This project transverses a muskeg deposit (from 5' to 10' deep over 7½ miles of its entire length. Considerable discussion has taken place since the first soils work was done in 1952 as to the merits of shifting the centre line and carrying out complete muck excavation or of riding the muck and using the existing centre-line. It has now been decided to ride the muck providing the present centre-line is maintained. This decision is based on the following factors: (1) Large quantities involved coupled with a shortage of backfill material, (2) Fair performance of existing surface, and (3) 20% to 50% compression of existing muck strata. It is felt that no failure will occur in the future and that performance should be within tolerable limits. It is proposed to keep this portion of highway under critical observation in order to aid in the establishment of a "muskeg policy".

N.B.

Grading on this proposed project will be limited to fill with no cuts except for ditching. The grade line has been set so that there is a continuous GBC "B" fill section throughout for the 28' central core. This depth of granular averages about 24" in depth (including 6" for GBC "A").

It has been found expedient to include a special lateral ditch, over a portion of the project, to carry the large amount of water which is found to drain from the adjacent swamp and is now being carried by an inadequate ditch.

A. Rutka
Acting Materials & Research Engr.

Per: T. J. Kovich

T. J. Kovich
Soils Engineer

cc: H.A. Tregaskes
R.A. Panter
J.B. Garland
H.A. Mantle
H.W. Hurrell
A. Gray
W. Bidell
T.J. Kovich
A.C. Powell
File

TJK/cl

ATTENTION: Mr. W. Wigle

Mr. J.B. Garland,
District Engineer,
Fort William, Ontario.

October 26, 1959

Contr. 59-~~254~~ 246 Hwy. 17 TCH
Raith to Poland Special
Ditch Sections.

Materials & Research

Our original design recommendation on this contract provided for two sections of relief ditching. The easterly section, from sta. 980+00 to sta 1104+00, intercepts water from the swampy area to the south. Before construction of the railroad and highway grades this water percolated to the north into Raith Creek. Under present conditions, however, high runoffs cannot do this since there are insufficient culverts and the lateral relief ditch to the south of the highway is required to intercept this water and lead it into Raith Creek at Sta 1071+00 where it crosses the highway.

The other "Special Section" runs from station 1328+00 to station 1424+00. This ditch was suggested for the purpose of lowering the water table in this area. Mr. Panter has questioned the wisdom of this. (There is a school of thought that with a lowered water table in muskeg, decomposition may follow with a resulting decrease in strength.) This water table could not be lowered appreciably anyway because of the blockage by vegetation of three man-made transverse channels which were constructed to feed water into the Savanne River. After due consideration it does not appear that the lowering of the water table in this area is essential to the good performance of this section of highway.

Briefly therefore, this relief ditch from station 1328+00 to station 1424+00 is not absolutely necessary, and could be omitted from the work performed under the present contract. No improvements are proposed for any of the transverse off-take ditches in this area at the west end either.

One point that should be stressed is that the ditch at the east end should be maintained at least 75 feet from centreline where possible.

L. Soderman
Principal Soils & Foundations Eng.

Per:

ACPowell

A.C. Powell,
Regional Soils Eng.

ACP:cc

CC: B.A. Panter
L. Soderman
T.J. Kovich

NEW.

ATTENTION: Mr. W. Wigle

Mr. J.B. Garland,
District Engineer,
Fort William, Ontario.

Materials & Research Section.

October 27, 1959

Contr. 59-254 Hwy. 17 TCH
Raith to Poland-Suitability
of Granular Material.

I have had cause to inspect the above contract twice in the past week, once in company with Mr. Panter and once with Mr. Wigle. On both occasions the quality of material coming from the pit at Argon, adjacent to Muskaka's camp, left a lot to be desired. Of the eleven samples taken to date, nine have proved unacceptable. It appears therefore that the majority of the material being used does not pass specifications and this is made apparent after rain. The present working is by dragline with a 3 foot face and I cannot suggest any corrective measures which would result in the production of acceptable sand cushion. I tested a much smaller area immediately adjacent to Muskaka's camp and this is rather cleaner material which meets our specification. I advised Mr. Coldwell of this today (Oct. 26/57) and he will allow the contractor to use this material.

Mr. Wigle and myself temporarily suspended any use of the main pit for sand cushion and I would suggest that this pit be condemned. Admittedly a longer haul will be required but I believe that better road performance would result with the use of better material.

With respect to the present use of GBC "A" which is being tailgated to maintain the grade, I feel that a large over-run will result unless the applied quantity is reduced, or the grade shaped before addition of GBC "A". No box-spreader is available for use at present and we feel that no appreciable quantities of GBC "A" should be placed without the use of one.

A.C. Powell

A.C. Powell,
Regional Soils Engineer.

ACP:cc

CC: T.J. Kovich

Mr. M. Sinclair,
Construction Engineer,
Fort William, Ontario.

April 19, 1960

Contract 59-246

Materials & Research, F.W.

Raith to Poland.

This contract was inspected today in company with Mr. Caldwell and Mr. Maluzynsky. Last year's grading was rather a patchwork affair and about 90% of the work was concentrated on building a central core. Most of the sand cushion used was borderline in quality and well above the optimum moisture for compaction. The material placed in October produced problems of traffic maintenance but by November the sand cushion was freezing as placed. The grade remained frozen over the winter and did not give trouble.

N.B.

Now that the frost is beginning to come out of the ground the water is saturating the fills again. The absence of earth widening and shoulders is extremely detremental in that there is no confining of the 24'-28' wide sand cushion core. Traffic has frequently to move to the outside of the core in order to pass oncoming traffic and the outer wheels shove the sand cushion material over the slopes. It is also physically impossible to compact near the edges.

At the present time most of the contractors effort is being expended on maintenance of the sand cushion grade. The tender quantity of GBC "A" is likely to be overrun for this reason. (This was also noted in our letter of October 27, 1959). The sand cushion material is being graded back and forth in an effort to reduce the moisture content. Rutting is not too severe as yet.

It is suggested that no more sand cushion or GBC "A" be hauled; except where required to maintain traffic; until the fill widening is completed over the remainder of the contract. Most of the haul roads are not passable to trucks at present and no serious hardship should result as far as the contractor is concerned. The contractor would be obliged to do this anyway if we enforced DHO Form #200 Section 214. (Under the four-foot layer method of compaction it states-"On no account will the Contractor be allowed to construct a core through the embankment and complete the fill by side dumping". Under the six-inch layer compaction method it states, "In the case of side fill or sloping sections, the low portion shall be constructed as above until a level cross-section is obtained".)

Since the damage has already been done on most of this contract; side dumping will have to be resorted to over the sections which have already received sand cushion. The earth should be dumped over the fill so as not to contaminate the sand cushion. A sheepsfoot roller or some other approved method of compaction should be insisted on once the widening has been brought up to the level of the sand cushion. In the present circumstances a wobbly-wheel roller could not be operated on the earth shoulders now being placed.

The drainage ditch at the east end appears to be functioning well. No inundation of the highway has occurred this spring as it did in preceding years.

Briefly then, it is suggested that the earth widening be brought up to the level of the sand cushion, graded to a good crown and compacted before any more sand cushion or GBC "A" is placed.

A C Powell

A.C. Powell,
Regional Soils Engineer.

ACP:cc

CC: T.J. Kovich
F. Caldwell

c.c. A.C.Powell

Mr. H. Tregaskes,

Construction Engineer.

A. Rutka.

April 26th, 1960.

Re: Hwy #17. T.C.H. Contract 59-246. Raith to Poland.

You will recall that we had a short dicussion recently in connection with the construction of this section of road, which traverses for the most part muskeg terrain.

The contractor commenced operations last fall, generally with material from a very shallow wet deposit. The materials were borderline at best and, indeed, much of the material did not meet our specifications.

I have since received a copy of a letter dated April 19th, 1960, from Mr. Powell, our Regional Soils Engineer, to Mr. Sinclair, Construction Engineer at Fort William. I am sending herewith a copy of this letter, along with a copy of a letter dated October 27th, 1959, and a copy of our Soils Construction Report dated April 7th, 1960, prepared by Mr. Powell.

N.B. It is indeed unfortunate that proper embankment construction was not followed here, because as you may know there was a very big decision that had to be made as to whether the muskeg should be excavated or ridden. A poor quality job could result in a poor performance, even though the riding of the muskeg may not have a direct bearing on the performance.

Your attention is also directed to the fact that a considerable over-run in Class "A" can be expected, due to the fact that the road was maintained during the winter and soft spots will occur which will require some treatment before the contractor gets started this spring.

A.R.

Alan - Please do not get too [illegible]

A. RUTKA
A/Materials & Research Engineer

ar/zw
enc.

c.c. A.Powell. T. Kovich. L. Eadie. File.

SOILS CONSTRUCTION REPORT

Contract 59-246 (Final Report)
Highway 17 (TCH) 9.5 Miles

Raith to Poland
Grading Culverts and Granular Base Contract
Contractor: Muskoka Construction

Construction Supervisor - F. Caldwell
Project Supervisor - N. Maluzynsky
Inspectors - R. Baker, R. Milne and J. Nicholetts

GENERAL DATA:

This Contract was awarded to Muskoka Construction in September 1959. Last years work is described, rather critically, in a previous Construction Report of April 7, 1960.

Work recommenced on this Contract early in April, 1960, while the ground was still frozen. Earth borrow for widening was hauled and the sand cushion haul started a little later.

Most of the GBC "A" and 5/8" crushed were placed in the months of May, June and July.

This Contract was completed on July 23 and there were still 32 working days left at that time.

The construction methods were improved somewhat, but the Contractor still persisted in core construction ahead of his widening operation. The chief damage however, had already been done last year.

DETAILED CONSTRUCTION METHODS:

a) Clearing, Grubbing and Stripping

This was all done last year but some clean up of the right-of-way was completed this year. This clean-up included removal of surface boulders.

b) Culverts

All the concrete culverts were completed last year and are covered in last year's report. However, the box culvert at Sta. 1312+35 settled about 6 inches this summer. In a letter of May 19, 1960, we explained that this could have been due to frost leaving the foundation after heaving the material under the granular pad.

Only a few flexible culverts were installed this year. The remainder were either placed last year or cancelled. The following equalizer culverts were cancelled at the request of Mr. Panter:- Sta. 980+00; sta. 990+00; sta. 1000+00; sta. 1010+00; sta. 1019+00; sta. 1040+00; sta. 1050+00; sta. 1061+00; sta. 1354+00; sta. 1364+00; sta. 1383+00; sta. 1416+00.

A minor failure occurred on one of the CIP's where the widening portion of the fill had settled induly. The end 10' section tore out of the coupling. The section had to be dug out and reinstalled. No further trouble of a similar nature has been noted.

c) Cut Excavation

All the cut widening in earth was completed last year but the rock cut at the east end was not touched until this year. The rock widening was taken out in April, 1960 and a frost heave at the easternmost transition point was sub-excavated at the same time. This sub-excavation was backfilled with sand cushion. The work was recommended in a letter of April 20, 1960.

Additional sub-excavations were required in the vicinity of sta. 1213+00. About 250 cubic yards of material were sub-excavated here; the recommendation was made in a letter dated May 19, 1960.

d) Embankment Construction

This was the worst feature of the Contractor's operation and most of his shortcomings were summarized in last year's "Soils Construction Report".

This year work commenced at the west end, with the widening material being placed where the road was up to sand cushion grade. However, by the time the widening operation reached the east end the road was up to sand cushion grade. (see colour slides on file at Downsview).

Compaction was not attempted until the widening was completed. At this stage, a wobbly-wheel roller was used on the shoulders. Compaction of the surface was quite good but settlements of the shoulders will obviously occur with time.

About 140,000 cubic yards of earth borrow has been placed on widening, most of this was hauled by Euclid or scraper. (Only 116,000 cubic yards were estimated. The overrun can be attributed to consolidation of the virgin muskeg under the fill widening).

e) Swamp Treatment

No swamp treatment was done. The easterly "Special Section" was completed in April and trimmed up in July. The deep ditch appears to be very successful and carries runoff away quite quickly, thus keeping the water table low. (The water table previously was at the old road level quite frequently).

At the west end a similar drainage ditch was cancelled at the request of Mr. Panter. In the vicinity of sta. 1385+00 the water level was checked as suggested on P-8 of last year's Construction Report. On September 8 we recorded the water level within 18" of centreline grade over a 600 foot section between sta. 1382+00 and 1388+00. This suggests that perhaps the special ditch may yet be dug in order to lower the water table.

Ditching throughout was done at the request of the District Engineer. This was not called for in the Contract and was in fact definitely discouraged by our section. However, Mr. Panter agreed to this ditch; with reluctance, I am led to understand. The ditch material, which was mostly organic in character, was dragged back to make a 2:1 or 3:1 slope. Final ditching quantities agreed quite closely with tender quantities.

GRANULAR MATERIALS:

About 80% of the sand cushion had been placed last year. This year the Contractor commenced hauling sand cushion from Linko pit but after depleting this, the material was taken from pits 1 and 9. The sand cushion was placed core width with year also and frequently this Spring the materials were well above optimum. Nearly all the sand cushion was placed by the middle of May 1960.

Compaction of the material was achieved by means of a wobbly-wheel roller..The sand cushion was chiefly end dumped by trucks and bladed to 6" lifts using a grader. Most of the difficulties in handling traffic occurred last year but some of the material placed this year was also well above optimum moisture and this led to problems.

153,000 tons of sand cushion were placed. The tender estimate was for 208,000 tons; an underrun of 53,000 tons (or 25%). The underrun was probably chiefly due to the high conversion factor of 1.9 tons per cubic yard which was used for the sand cushion.

At the time of commencement of operations this Spring very little GBC "A" had been used as GBC "A". About 10,000 tons were stockpiled in pit #1 but this material was not accessible until July; when the haul road was passable.

The Contractor commenced crushing GBC "A" in the 3 mile pit on the Dog River road in April and switched to production of 5/8" crushed in July. The main haul of GBC "A" did not start until the middle of May 1960.

The material was placed using a box spreader and compaction was good.

The 25,000 ton overrun, 30% of the tender quantity can be attributed to the need for maintenance gravel late in 1959 and in the Spring of 1960. (Attention was drawn to this misuse in letters of October 27, 1959, April 19, 1960, the 1959 Soils Construction Report and a letter from A. Rutka to H. Tregaskes.)

g) Shouldering

Shouldering materials varied from bouldery till to fine sand and was similar in character to the fill widening. In actual fact the shoulders were placed at the same time as the fill widening. That is, after the sand cushion core was completed. All the material was hauled from borrow pits by scraper and euclid, some of the hauls were quite long.

N.B

Very few french drains were provided, there does seem to be a tendency for slight dampness against the shoulders after rain but no distress has so far been noted.

The crushed gravel was placed on the shoulders at the same time as it was placed on the roadbed.

Shoulders were well compacted at the surface but no compaction was applied to the fill widening beneath.

COMPACTION:

Wobbly-wheel rollers were used exclusively on this contract for compaction. No compaction checks were taken on this contract.

Compactive effort on the sand cushion was good but much of the material was compacted at moisture contents well above the optimum. Compactive effort on the GBC "A" was also quite good.

Compaction of earth borrow was left until the material was up to grade. 848 hours of wobbly-wheel rental were paid while 125,000 cubic yards of borrow and cut material were moved. This gives well over the specified 125 cu. yds. per hour per compaction unit. In other words insufficient compaction.

BORROW SOURCES:

There were 116,000 cubic yards of borrow used on this contract.

Pit #3. has not been used this year.

Pit #2. At Argon (Sta. 1264+00). This deposit of fine sand was used extensively at the west end of the project.

Pit #6. At Linko (Sta. 1111+75). The bouldery material west of the gravel pit was used for earth borrow.

Pit #5. (Sta. 1145+00) depleted last year.

Pits 7 & 8. Two small borrow pits south of sta. 1064+00 were used on the east end of the project. Both contained fine sand.

GRANULAR SOURCES:

Five sources of sand cushion were used on this Contract.

1) Pit #2 at Argon. ¼ mile south of sta. 1264+00. About 61,000 tons of this material was used last year as sand cushion. Problems connected with this source are outlined in last year's report. This pit was used for shouldering only this year.

2) Pit #4. 1 mile north of sta. 1174+00 on the Dog River Road. Depleted last year.

3) Pit #6. Linko. ¼ mile north of station 1111+75. About 60,000 tons of sandy gravel were removed from this pit. The pit is now depleted.

   Some problems arose with the use of this pit, chiefly because of moisture contents above optimum this Spring. Some faces had to be condemned because of oversize boulders.

4) Pit #1. Abitibi. 2.1 miles south of Hwy. 17 at Sta. 1064+00. - 5,500 tons of gravel were taken from this pit and used at the east end of the contract.

   This pit was also used as a crushing source, some 27,000 tons of material were crushed last year. Part of the material was used for maintenance last winter and the remainder was stockpiled and placed on the road this year.

This deposit is part of a large esker and the material tends to very coarse. (The pit is now being used on Contract 60-169, for hot-mix aggregate).

5) Pit #9. 3 miles north of sta. 1174+00 on the Dog River Road.

Since most of the other pits in this area were depleted, the Contractor moved here early in 1960 for both sand cushion and crushed gravel. About 18,000 tons of sand cushion were taken from this pit. The material was very nicely graded, being a medium gravel with no oversize.

Crushing started in this pit in early April, but very little material was placed on the road before early May. All the GBC "A" was placed by the end of June and the Contractor then crushed his complement of 5/8" crushed. About 80,000 tons of GBC "A" and 17,000 tons of 5/8" crushed were produced.

The production of GBC "A" was too sandy at first but improved with time. The production of 5/8" was very sandy and very few samples met our specifications despite repeated warnings. There is quite a good quality of crushable gravel remaining in this pit. There is at present a 15' face here.

DEVIATIONS FROM SOILS DESIGN REPORT:

Most of the deviations were dealt with in our 1959 report. No grade changes were recommended but some were necessary because of inaccurate original cross-sections. These grade changes were all approved by Mr. Tregaskes in a letter of February 29, 1960.

Two additional sub-excavations were done. A frost heave at sta. 941+40 was sub-excavated, this was recommended in a letter written on April 20, 1960. A series of frost boils between sta. 1212+00 and sta. 1214+25 were also sub-excavated (see letter of May 19, 1960). Together, these amounted to less than 500 cubic yards of sub-excavation.

As noted above, the contract was ditched throughout, including swamps. We do not consider this to be a good idea since it breaks through the fibrous root mat on the surface and could lead to increased settlements.

FOR FUTURE PERFORMANCE STUDY:

1) Next spring, it will be very interesting to compare the east and west ends to see how well the "Special Section" ditch copes with spring run-offs. It is anticipated that the water table at the west end will be quite high and this ditch may yet have to be excavated.

note.

2) When this contract is paved (probably the foundation course will be laid this year) we shall have a datum from which to check the expected consolidation of the "ridden" muskeg. Some of this consolidation is already occurring and this could have been accelerated by the ditch excavations noted above. especially near the ditches in the shoulders.

Some differential settlement can be expected where the widening was placed on virgin muck. It is probable that some cracking may occur on the shoulders and it is almost certain that the soulders will require extensive maintenance over the next few years.

3) Briefly, this contract has recovered from a very shaky start. The finished product appears to be very good but troubles will undoubtedly show up over a period of years.

(This Contract is now complete).

PREPARED BY:

F. Norman,
Project Soils Engineer.

APPROVED BY:

A.C. Powell,
Regional Soils Engineer.

FN:cc

ONTARIO

DEPARTMENT OF HIGHWAYS

Memo to Mr. A. C. Powell,
Soils Engineer,
FORT WILLIAM, Ontario.

Date September 4, 1958.

Subject Re: WP 912-58; Rarth to Poland; Hwy.17 TCH Layout of Relief Ditches as suggested in Soils Design Report.

From Materials & Research Section.

Enclosed find prints of the drainage layout which should help you in making a field check of the feasibility of the scheme as we proposed in our soils design report.

Pete Arkema has studied the area from aerial photographs and he has the following comments to make:

" Sta. 980 to 1104 - it appears that the main drainage outlet for this section is at 1073 and that the water flows westward from the highway. A relief ditch left of C/L will only be of use when this outlet has sufficient capacity, which is doubtful. "

" Sta. 1328 to 1424 - the short arm "A" (about 1200') will require some improvement. The two conducts marked "B" appear man-made and in good condition. However downstream from their junction section "C" is in poor condition. The relief ditch will probably cause section "C" to silt up completely. "

You should make a fairly thorough field investigation keeping in mind that due to the flat terrain and date of photographs it is difficult to appraise the situation accurately from photographs alone.

In the field you should:

(a) Check stream directions at various points and mark flow on the attached plan.

(b) Check condition of streams that are to drain the relief ditch.

(c) Get some idea of the head at various points, particularly at Stas. 1073 and 1328.

(d) Get an idea, from observing the existing ditches, of the amount of stability which can be expected from a ½ 1 slope in muskeg ditch cuts.

G. K. Hunter has indicated that he has a Mr. D. Bews in their Toronto office who will be working on drainage problems generally and therefore he would like to have this man meet you up there with the idea in mind that he may learn something from you during your investigation. Could you, therefore, contact Gord about a week in advance so that the necessary arrangements can be made.

T. J. Kovich

T. J. Kovich,
Soils Engineer.

TJK/jy.

C.C. Mr. A. Rutka
G. K. Hunter
H. W. Hurrell
T. J. Kovich

File.

A. Powell

Department of Highways
COPY
For the Information of:

Mr. T.J. Kovich

Design Division, 1173 Bay Street,
TORONTO, Ontario. Oct. 14, 1958.

MEMORANDUM FOR:

Mr. J.A. Loudon,
Special Projects Design Engineer,
Department of Highways,
1173 Bay Street,
TORONTO, Ontario.

RE: W.P. 912-58, Highway 17 T.C.H., Raith to Poland, Fort William District.

DRAINAGE:

A field inspection of this project was made, with H. Hurrell, Project Design Supervisor, and A. Powell, Soils Engineer, on October 7, 1958.

1. Station 107+02:- Proposed 20 x 8 Concrete Box Culvert will be sufficient.

2. Spacing of Culverts:- It is recommended that maximum spacing between culverts be 1000'. This will involve a number of additional equalizing pipe culverts.

3. Special Lateral Drainage Ditch:- It is recommended that a lateral drainage ditch be constructed as proposed, in the soils design report, by A. Powell. In addition, it is recommended that the centre line of the ditch be located 75' south of the centre line of the Highway. The grade of the ditch is to be governed by the invert elevations of the connecting transverse ditches. All additional culverts should have off take ditches to the lateral ditch. Where the lateral ditches approach the standard side ditches, they should be connected by transverse ditches. All excavated material should be placed between the ditch and the road embankment and graded to provide good drainage away from the road to the ditch. The transverse ditch at station 1372+00 and station 1393+00 should be cleaned out to the south to the intersecting stream.

D.M. Bews.

D.M. Bews,
DRAINAGE ENGINEER,

DMB/ac

59-~~25~~ 246

ONTARIO

DEPARTMENT OF HIGHWAYS

Memo to Mr. J. B. Garland

District Engineer

From Materials & Research Section

Date November 19, 1958.

Subject Re: WP 912-58 Raith to Poland Highway #17.

With reference to your teletype of November 10, 1958 to Mr. H. Tregaskes, we have ~~reviewed~~ the section between Station 982/00 and Sta. 1044/00 on soils profile 17 M 18.

We concur with your remarks on the heavy maintenance cost in this vicinity. There are two chief reasons for this;

1) A very high water table in the vicinity coupled with a low grade line.

2) The shallow lift of granular under the present road directly over the muck.

The water table problem has been aggravated by pipe-line construction on the south side of the highway but we are attempting to offset this by the construction of our proposed interceptor ditch. The gradeline will also be raised by about 2'.

The present granular fill of 18" - 24" has not caused much compression of the underlying muck layer and this fact probably contributes to the high maintenance cost. (The other swamps on this project all have greater lifts of granular). The proposed increase of fill will compress the muck further and will tend to make for a more uniform performance. While we certainly do not expect a perfect surface to result, the scarcity of borrow and granular materials in this area would make the excavation of the muck an expensive alternative. (this is a mentioned in our design report, dated August 15, 1958).

Briefly, therefore, we are relying on the proposed relief ditch to lower an existing high water table and more consolidation of the muck to increase the bearing capacity. Consequently we would recommend no change in our original design.

A. Rutka
Acting Materials & Research Engr.,

Per: A.C. Powell
A.C. Powell
Soils Engineer

cc: Mr. H.A. Tregaskes
H.D. McMillan
R.A. Panter
H.W. Hurrell
T.J. Kovich
A.C. Powell
File

ACP/cl

Ministry of Transportation and Communications

Ontario

Memorandum

To: C. Mirza
Head, Soils Mechanics Section
Downsview, West Tower

Attention:

From: Materials & Testing
Northwestern Region

Date: May 23, 1975

Our File Ref.

In Reply to

Subject: Longitudinal Cracking West of Raith

WP 910-75-01
~~W.P. 915-75~~

During your visit to Thunder Bay on May 21, 1975 we discussed the above section of Highway 17 and I requested verbally that you carry out an investigation to determine the feasibility of and what (if any) failures, settlements or distortion might be expected with a maximum 36 inch grade raise over the existing pavement.

We are proposing to treat the entire 2.5 mile section with styrofoam. Dependent upon your findings, we will be removing the pavement and placing the styrofoam or the granular grade or excavating a portion of the existing roadbed to minimize the fill height.

As the existing pavement is severely distorted due to the longitudinal cracking and some remedial treatment is considered urgent, we would request that you carry out this investigation and issue a report prior to the end of 1975.

Please find enclosed a copy of the report prepared by Mr. R. Girard, Project Manager, Planning & Design, Northwestern Region, outlining conditions encountered during our investigation and alternative treatments which were considered.

H Munford
H. Munford
Senior Soils Supervisor

/mle
Enc.
c.c. G. Wrong
W. Neilipovitz

for R. Morgenroth
Regional Materials Engineer

File copy

Ministry of Transportation and Communications

Ontario

Memorandum

To: R. Franks
District Engineer
Thunder Bay

From: Materials & Testing
Northwestern Region

Attention:

Date: April 22, 1975

Our File Ref.

In Reply to

Subject: Severe Longitudinal Cracking on Highway 17
From 8.5 miles to 11.0 miles west of Raith

As per your request we have investigated the above mentioned section of Highway 17 in an effort to ascertain the causes of the severe longitudinal cracking.

Attached is a preliminary Soils Report written by Mr. R. Girard, Project Manager, Systems Design. In the report he indicates the background of the subject section of highway and outlines the investigation procedures along with the probable causes and some alternative treatments which can be considered.

I would suggest that a meeting be held on ~~Wednesday,~~ FRIDAY May ~~14~~ 16, 1975 in the third floor boardroom at 10.00 a.m. to discuss the possible solutions and their merits and disadvantages with all interested parties in attendance.

When a treatment has been decided on, this Section may require further borings to arrive at treatment limits and design quantities prior to issuing a Soils Design Report.

If you have any questions relating to the enclosed report, please contact this office.

H. Munford
Senior Soils Supervisor

for: R. Morgenroth
Regional Materials Engineer

/mle

c.c. D. Jarvis
W. Lees
D. Lowman
N. Maluzynsky
D. Moorhouse
W. Phang
G. Wrong

RECEIVED
GEOTECHNICAL OFFICE
MAY 27 1975
SOIL MECHANICS
MINISTRY OF TRANSPORTATION & COMMUNICATIONS

SEVERE LONGITUDINAL CRACKING
HIGHWAY 17 - 8.5 to 11.0 MILES WEST OF RAITH

INTRODUCTION

The treatment of the hazardous longitudinal cracking at Raith is now imperative. As indicated on the attached photos, the longitudinal cracking, up to 10 inches wide in some areas, presents a real danger to the travelling public. Numerous attempts have been made over the years to solve the problem, ranging from routine maintenance patching to the laying of reinforcing wire mesh but all have failed.

This report contains the highlights of the chronological development of the present day structure, a description of the investigation carried out, a discussion of the results of this investigation and possible recommendations for the correction of the cracking phenomenon.

LOCATION

This particular section of road is approximately 2.5 miles in length and is about 60 miles west of Thunder Bay on Highway 17, 8.5 miles west of the village of Raith. The problem area extends from Station 1300+00 to 1420+00. The existing grade is low and on tangent and rides ~10 foot fibrous, slightly decomposed, organic deposit with a very high water table.

PERFORMANCE

The 1958 Soils Design Report for the proposed reconstruction of the existing highway describes the performance at that time "...The present primed surface, the only remaining stretch of primed surface between Fort William and Ignace, varies from 20' to 22' wide with 6' shoulders. No actual failures have been noted but small transverse and longitudinal cracks sometimes occur. Roughness and settlements necessitate the oiling and priming of the surface each year and potholes are very much in evidence...."

When the reconstruction of that section of highway became necessary, considerable discussion was presented, evaluating the respective merits of the feasible treatments:

1. Excavate the muskeg and the existing roadbed to solid bottom;
2. Ride the muskeg and the existing road structure;
3. Realign that section of highway around the swamp.

Ultimately, the decision was made. "... It has now been decided to ride the muck providing the present centreline is maintained. This decision is based on the following factors:

1. Large excavation quantities involved coupled with a shortage of backfill material;
2. Fair performance of existing surface and
3. 20% to 50% compression of existing muck strata..."

The Soils Report further states "...It is felt that no failure will occur in the future and that performance should be within tolerable limits..."

Considerable discussion is still generated as to the reason (s) for failure but only serious scrutiny into the design details and the construction practices will reveal the most probable explanation for the failure.

THE DESIGN

The original soils profile was not available for inspection but the Soils Design Report is quite explicit as to the design recommended over the swamp. "... From 18 inches to 24 inches of granular are proposed over the existing road, and this should then provide a minimum of 36" above the water table in the surrounding swamp. It is recommended that granular only be used for the 28 foot central core. It is recommended that only acceptable earth material be used for shouldering. Shoulder material is chiefly non-cohesive, hence Frence Drains should be used in sags only, unless the Engineer decides more are required..." The proposed design was therefore core construction. (This type of design was considered for a short period of time during the mid fifties.)

It was further recommended that elaborate ditching be carried out. "This area is frequently inundated and it is therefore proposed that an auxiliary graded ditch be constructed. It is further proposed that offtake ditches to the roadside ditch be dug at 500 foot intervals."

INVESTIGATION

To establish the reasons for the problem, the entire swamp was visually inspected and typical test sections were laid out representing various degrees of cracking. One test section was chosen to represent the areas where no cracking was observed, one test section was chosen to represent the areas where slight to moderate longitudinal cracking was observed (4 inch max.) and three test sections were chosen to represent the severely cracked areas (4 to 10 inch wide longitudinal cracks). These test sections are an average of 360 feet long and their exact location is shown on the accompanying key map.

During April, 1974, power auger boreholes were placed at both edges of pavement at each test section. That particular time of the year was chosen since it represented the worst frost heaving period of the year. These boreholes were taken down past the organic to the inorganic bottom. The results of these boreholes are shown on the accompanying soils cross-sections. The frost depth limits were also determined during the drilling. Samples and moistures were taken at each different soil type encountered and analyzed in our Regional Materials Laboratory. These results are available at the Regional Materials and Testing Office.

More boreholes were placed at each test location during July, 1974. These were placed in the centre of the westbound lane and at the extreme outer edge of the shoulder. Samples and moistures were again taken, analyzed at the Regional Materials Laboratory, and the results plotted on the cross-sections.

Finally, a complete record of all visible cracks was compiled. These had been done during the month of April 1968 and were brought up to date during April 1974. The locations of the test sections and the boreholes were added to the sketch of the cracks. Cross-sections were taken at each test section during the worst heaving period and also during the summer to establish the type and degree of heaving, but, unfortunately, a survey error was made and the results are ambiguous.

All the samples obtained were analyzed as to two of their physical properties; the percent content of the frost susceptible particle sizes, and the moisture content. The limits of the frost susceptible material were set as that passing the #100 sieve (0.15 mm .) and greater than .003 m m. These limits were established based on the capillarity and the ability of water to move through the soil media of specific particle sizes. As originally established by Atterberg and published by, Gilbert L. Roderick, "Particles from 0.2 - 0.02 mm . possess good capillarity and allow fast capillary movement of water. Materials finer than 0.02 mm . show very high capillarity but the movement of water in the capillaries is retarded." The portion of the samples analyzed therefore represents the susceptibility of that material to frost action. Particle sizes greater than #100 sieve can retain only small amounts of water. Particles sizes smaller than .003 mm. do not readily permit the flow of water. The formation of ice lenses and the consequent frost heaving is therefore a direct relationship to the amount of the frost susceptible portion.

The distribution of the frost susceptible fraction within the soil mass is also a significant factor affecting the degree of frost action in a particular sample but this factor had to be disregarded due to the complexity of the analysis and the huge number of samples that would be required to determine this with some accuracy. The analysis was therefore based entirely on the premise that frost susceptibility is entirely dependent on the amount of a specific particle size range within a given soil mass. It was felt that one of the reasons for the cracking had to do with the core type construction and the lack of uniformity across the road structure. The samples were therefore analyzed with this in mind by comparing the frost susceptible fraction contained at the center of the lane and at the edge of the shoulder. The results for the various test sections are listed in Figure 1, and are tabulated in accumulative 2 foot increments i.e. the frost susceptibility at the centre of the lane is compared to the frost susceptibility at the edge of the shoulder for the top 24 inches for the various test sections and the results are listed on line 0" - 24", the comparison of the frost susceptibility for the top

48 inches is listed under 0" - 48" etc. Inspection of the results reveals the following:

i) The difference in the frost susceptible fraction between the centre of the lane and the edge of the shoulder is consistently greater for the no cracking section than the moderately cracked or severely cracked section.

ii) This difference decreases with depth in all sections and are equal when the top 96 inches are considered.

iii) The amount of frost susceptible soil at the centre of the lane is apparently directly proportional with the severity of cracking for the top 48 inches. The relationship is only very slightly obvious at greater depths and is very doubtful.

iv) The amount of frost susceptible soil is relatively constant with depth in moderately and severely cracked sections but is directly proportional with depth at the centre of lane for the no cracking section and inversely proportional with depth at the edge of the shoulder.

The moisture content results are summarized in Figure 2. They indicate positively that the moisture content is much greater under the shoulder then under the pavement. The results further indicate that the average moisture under the centre of the lane and under the edge of the pavement is directly proportional with the severity of cracking.

DISCUSSION

The results of the moisture content comparison are straight forward and prove to be as expected; the higher the moisture content under the pavement, the greater the amount of cracking.

The frost susceptible fraction comparison is not so straight forward. With the minimum sampling done during this investigation, the exact relationship between the multitude of variables cannot be established with confidence. Trends, relationships and comparisons can nevertheless be established with some certainty. Some facts nevertheless prevail.

1. The results of Figure 1 show that some form of core construction exists since the frost susceptible fraction at the edge of the shoulder is continually greater than at the centre of the lane in all test sections and at all depths. This type of construction is nevertheless not the reason for the serious longitudinal cracking because:

   (a) The serious heaving occurs at the centre of the pavement and not at the shoulders where the greater frost susceptible fraction is encountered.

   (b) The moisture content is greater at the edge of the shoulder than at the centre of the roadway. The argument that the earth shoulders retain the moisture in the core causing severe heaving is false.

2. The results of Figure 1 show that the frost susceptible fraction at the edge of the shoulder is relatively constant with depth and with severity of cracking. This can be explained by reading the contract inspection reports which state that the granular core over the existing roadbed was placed first up to the level of the Granular 'A' and that the earth shoulders were placed after.

3. Figure 1 also indicates that the frost susceptible fraction at the centre of the lane varies with the severity of cracking. Since the reconstruction granular depth was only approximately 18 inches the noted frost susceptible fraction depends greatly on the soils types for the original underlying construction.

It would therefore be reasonable to assume that the severe heaving and the resultant cracking is caused by the frost susceptible soil used in the original roadway construction. This dirty material is shown on the soils cross-sections and is justified by the sample results.

The higher moisture content of these fine grained soils as shown in Figure 2, in conjunction with their high capillarity, the easily accessible supply of water, and the freezing temperatures, will always produce severe frost heaving.

The shoulders don't seem to heave as badly because they are slightly insulated with packed snow and are slightly heated (relative to the travelled portion) by the latent heat supply trapped under the snow banks at the outer edges of the shoulders and under the snow blanket over the swamp.

CONCLUSION

The phenomenon of longitudinal cracking in low lying and wet areas is a common problem but no definite explanation as to its causes and effective treatment has ever been adequately documented. The investigations, as outlined in this report, were carried out, to try to explain the reasons for the severe cracking and to determine an effective treatment.

By systematically reducing the mass of information gathered, we can conclude that the cracking phemonenon, even though it appears as a surface defect, is actually a deep seated problem and can be directly attributed to the frost susceptible material used in the underlying old construction.

RECOMMENDATIONS

Only two possible options are left to treat the cracking problem.

1. Eliminate all frost susceptible material.
2. Prevent the frost from entering the roadway structure.

Under option (1) there are three possible alternatives.

i) Excavate all frost susceptible material and replace it with clean granular type material. This would present severe traffic maintenance problems and might require the construction of a detour.

ii) Excavate the existing roadway and organic deposit to solid inorganic bottom and backfill with clean granular type material. It is very doubtfull if this would be done without the construction of a detour.

iii) Realign the roadway. This could possibly turn out to be the cheapest alternative under this option, since the existing highway could be used as the detour. The new construction would therefore be located to the south of the existing highway and would involve complete excavation to solid inorganic bottom.

Under option (2) basically 2 alternatives are possible.

i) Prevent the frost from entering the roadway structure by using highway insulation. The effectiveness of this insulation could be increased greatly by possibly removing the existing pavement, placing the insulation over the granular grade and covering it up with a minimum of 24 inches of granular material.

ii) Prevent the frost from reaching the frost susceptible soil by increasing the granular depth over the roadway. It is nevertheless very doubtful that this treatment will be successful unless at least three feet of granular material be placed and even so the treatment could be only temporary. Using this treatment, there also exists the problem of increasing the fill to such a height that it would become unstable and present a potential stability problem. The Foundation Office should be contacted for their recommendations if this alternative is seriously considered.

Due to the urgency and the complexity of this problem, the various alternatives should be discussed with the design and construction people in order to reach a concensus on the most effective and economically justifiable treatment.

INVESTIGATION OF LONGITUDINAL CRACKING AT RAITH

W.O. 74-61-309

EXISTING PAVEMENT CROSS-SECTIONS AND BOREHOLE PROFILES OF SAMPLE SECTIONS

APRIL 1974

SCALE: VERTICAL & HORIZONTAL

0 1' 2' 3' 4' 5'

LEGEND

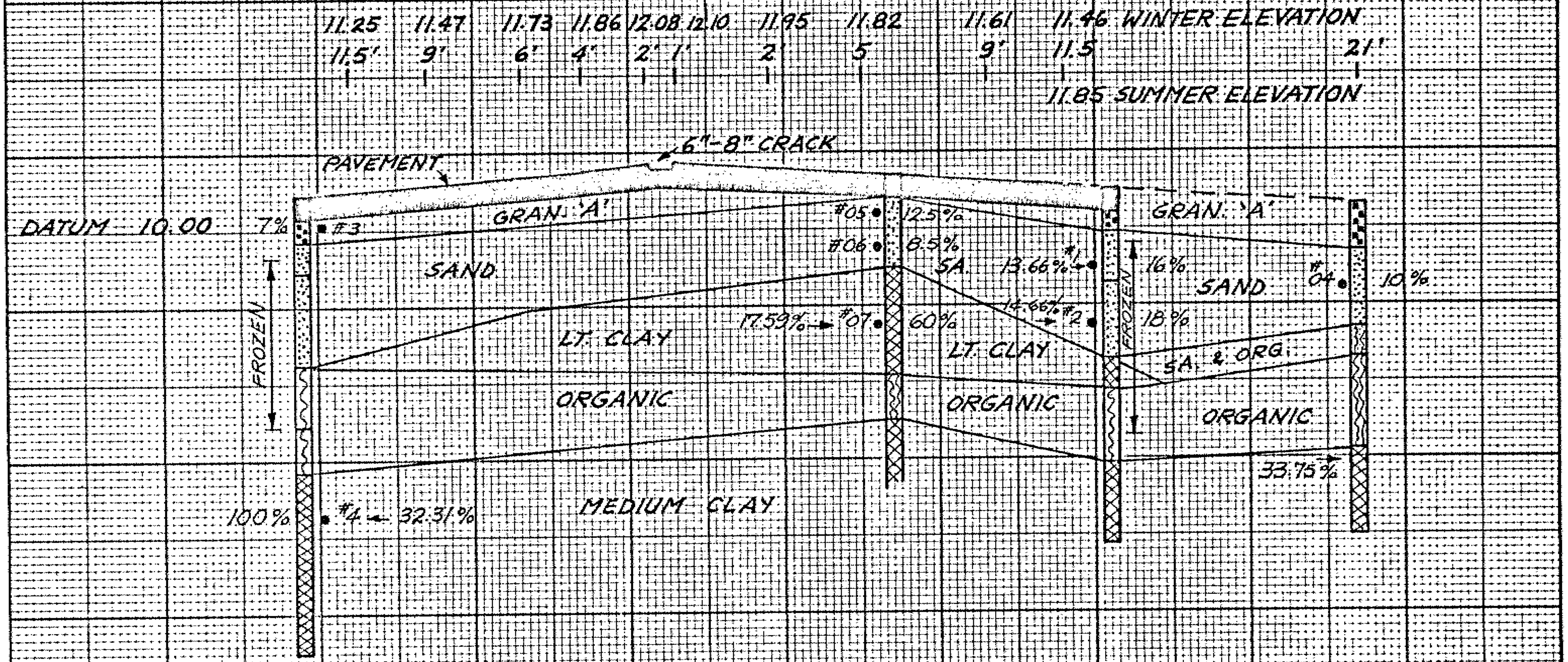
- PAVEMENT
- GRANULAR 'A'
- SAND
- TILL
- SANDY LOAM
- SILTY LOAM
- CLAY LOAM
- CLAY
- BOULDERS
- ORGANIC
- LOCATION SAMPLE OBTAINED
- LOCATION MOISTURE SAMPLE OBTAINED

SEVERE LONGITUDINAL CRACKING

SAMPLE SECTION (1)

STA. 1305+00

℄



SAMPLES: 74-RN-1 (24"-36") RT. ℄ : SAND 16% PASS #200
74-RN-2 (48"-60") RT. ℄ : SAND 18% " "
74-RN-3 (6"-16") LT. ℄ : SAND 7% " "
74-RN-4 (120"-132") LT. ℄ : MEDIUM CLAY 100% "

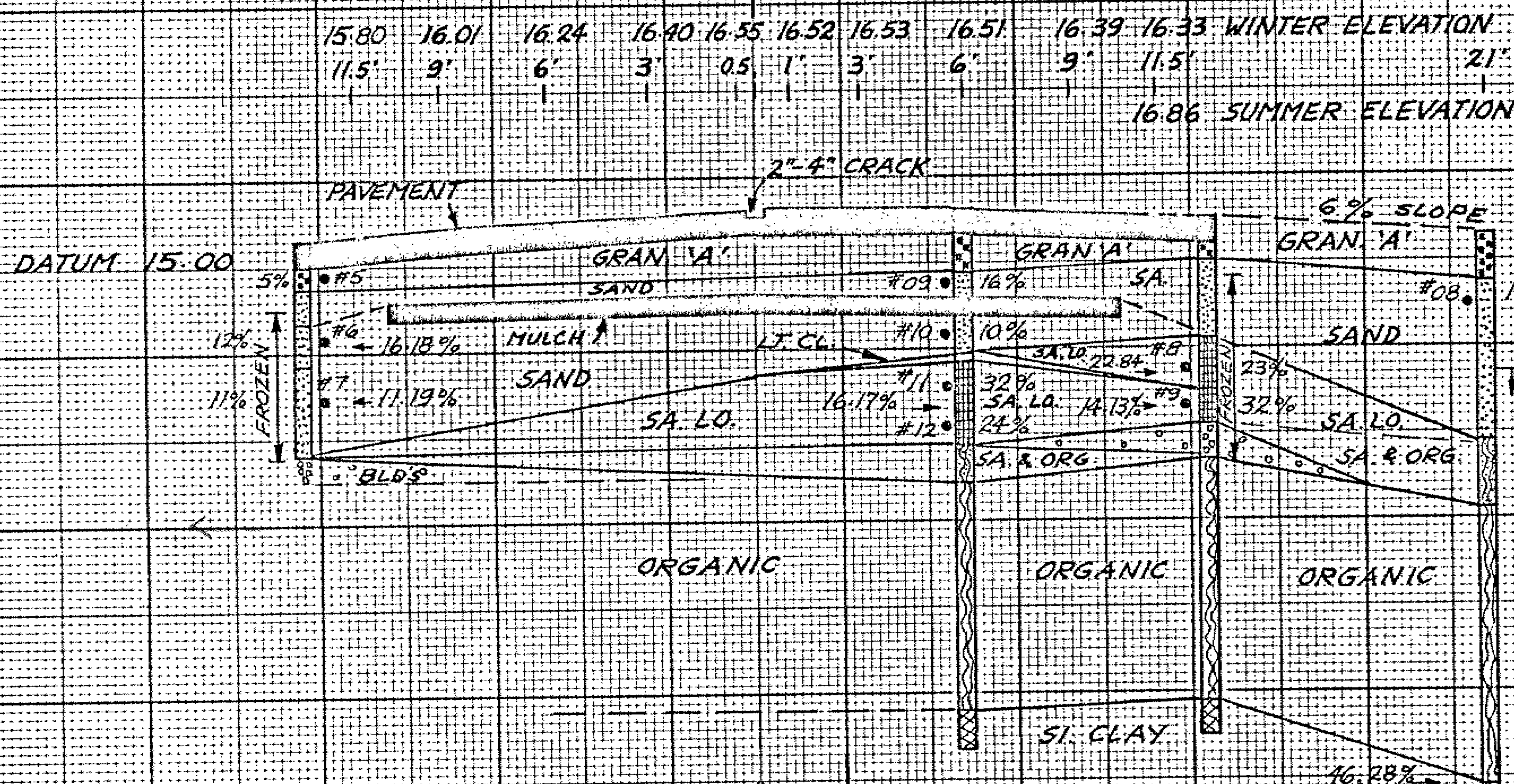
SAMPLES: 74-RZZ-1004 (18"-48") 21' RT. ℄ : SAND 10% PASS #200
74-RZZ-1005 (8"-26") 6' RT. ℄ : SAND 12.5% " "
74-RZZ-1006 (26"-36") 6' RT. ℄ : SAND 8.5% " "
74-RZZ-1007 (36"-73") 6' RT. ℄ : LT. CLAY 60% " "
74-RZZ-1100 (MOISTURE ONLY)

MOISTURES: @ 30" RT. ℄ : 13.66 %
@ 54" RT. ℄ : 14.66 %
@ 126" LT. ℄ : 32.31 %

MOISTURES: @ 60" 6' RT. ℄ : 17.59 %
@ 100" 21' RT. ℄ : 33.75 %

SLIGHT-MEDIUM LONGITUDINAL CRACKING
SAMPLE SECTION

STA. 1358+50



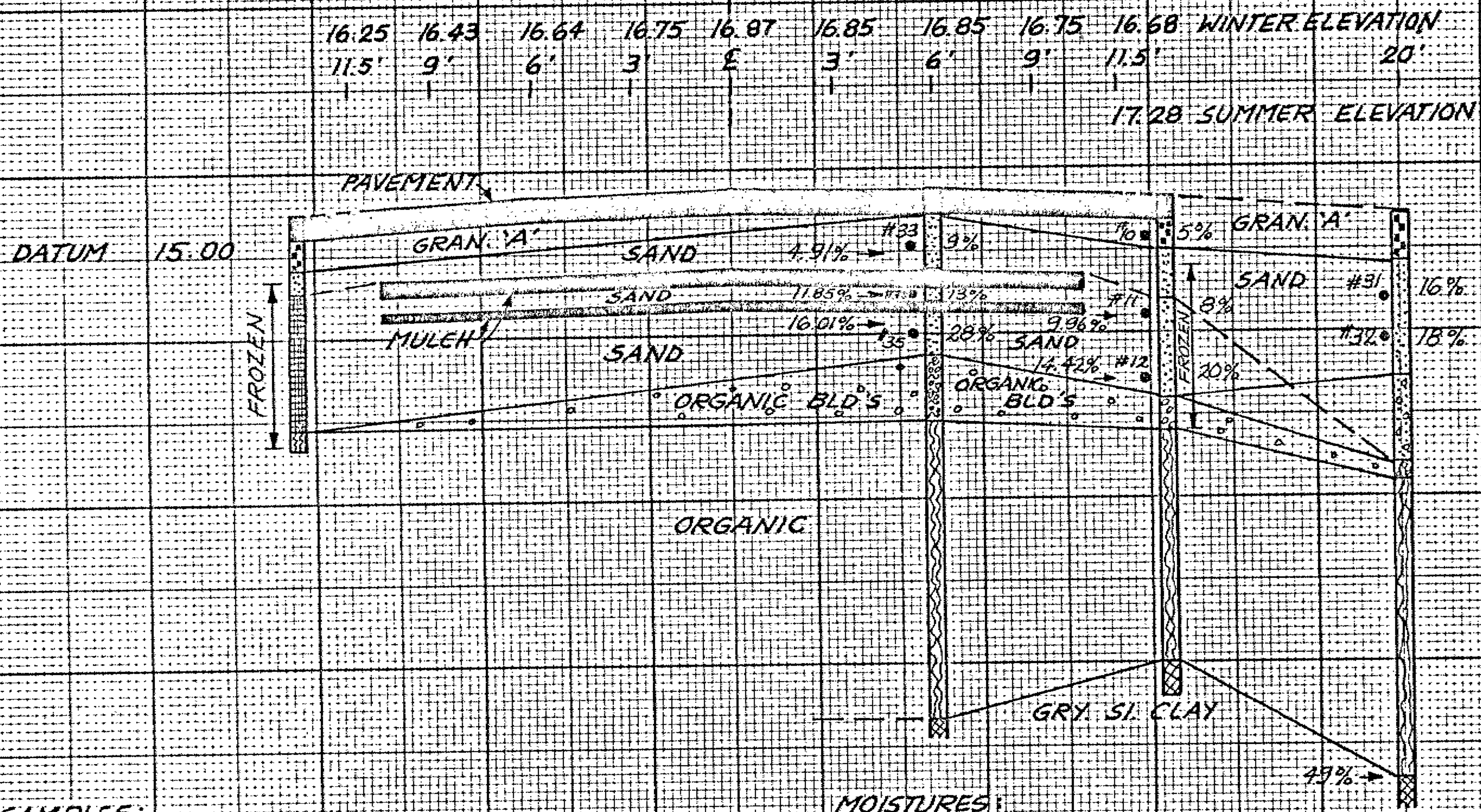
SAMPLES: 74-RN-5 (8"-16") LT. ℄ : SAND 5% PASS #200
74-RN-6 (30"-42") LT. ℄ : SAND 12% " "
74-RN-7 (48"-60") LT. ℄ : SAND 11% " "
74-RN-8 (48"-60") RT. ℄ : SA. LO. 23% " "
74-RN-9 (60"-72") RT. ℄ : SA. LO. 32% " "

SAMPLES: 74-RZZ-1008 (18"-30") 21' RT. ℄ : SAND 14% PASS #200
74-RZZ-1009 (22"-30") 6' RT. ℄ : SAND 16% " "
74-RZZ-1010 (36"-50") 6' RT. ℄ : SAND 10% " "
74-RZZ-1011 (53"-72") 6' RT. ℄ : SA. LO. 32% " "
74-RZZ-1012 (72"-84") 6' RT. ℄ : SA. LO. 24% " "
74-RZZ-1101 (MOISTURE ONLY)

MOISTURES: @ 36" LT. ℄ : 16.18 %
@ 54" LT. ℄ : 11.19 %
@ 54" RT. ℄ : 22.84 %
@ 66" RT. ℄ : 14.13 %

MOISTURES: @ 72" 6' RT. ℄ : 16.17 %
@ 16'0" 21' RT. ℄ : 46.28 %

NO LONGITUDINAL CRACKING
SAMPLE SECTION
STA. 1364+78



SAMPLES:

74-RN-10 (8"–18") RT. ℄ : SAND 5 % PASS # 200
74-RN-11 (36"–48") RT. ℄ : SAND 8 % PASS # 200
74-RN-12 (60"–72") RT. ℄ : SAND 20 % PASS # 200

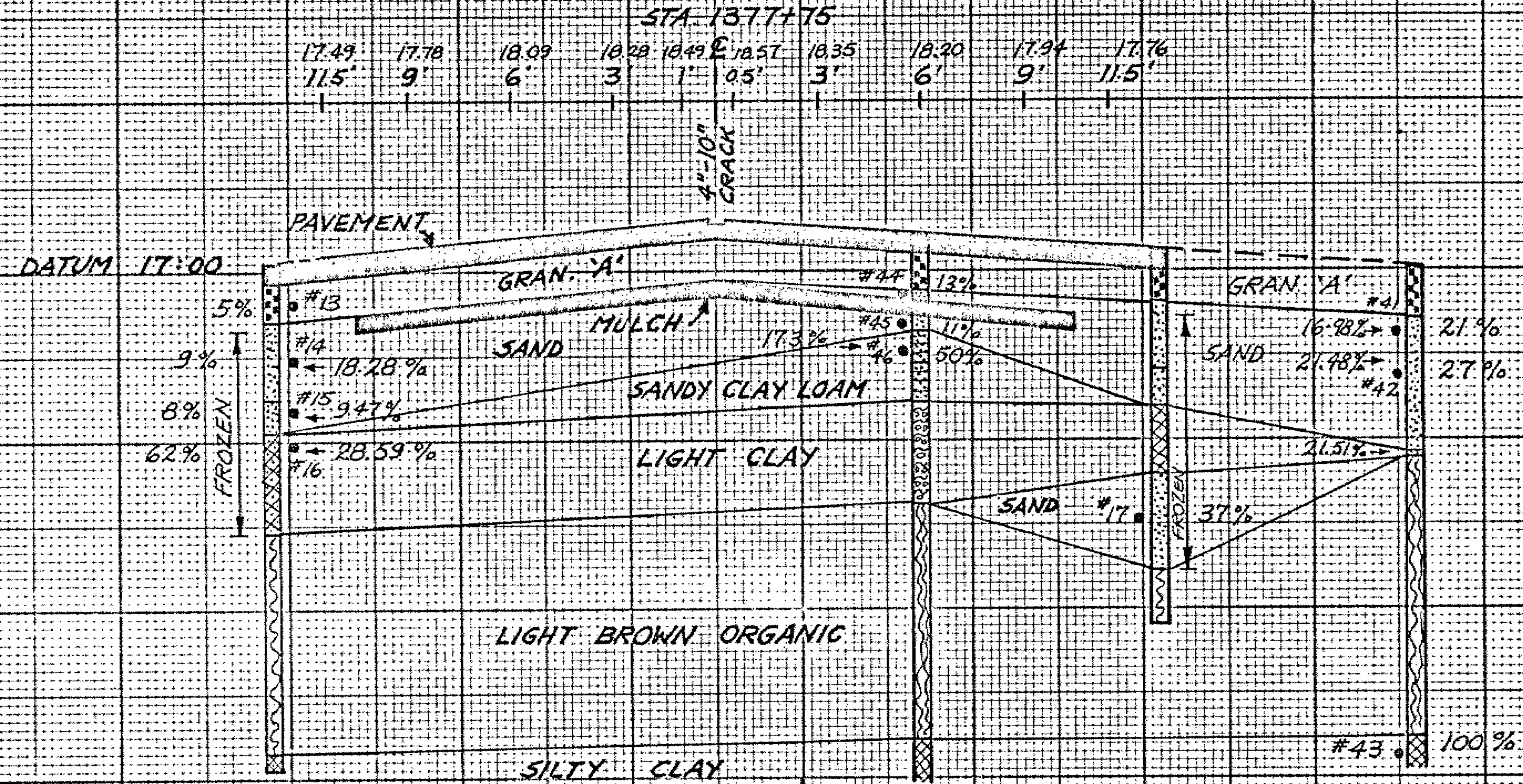
74-RZZ-1031 (18"–40") 20' RT. ℄ : SAND 16 % PASS # 200
74-RZZ-1032 (40"–50") 20' RT. ℄ : SAND 18 % PASS # 200
74-RZZ-1033 (9"–30") 6' RT. ℄ : SAND 9 % PASS # 200
74-RZZ-1034 (36"–42") 6' RT. ℄ : SAND 13 % PASS # 200
74-RZZ-1035 (44"–60") 6' RT. ℄ : SAND 28 % PASS # 200
74-RZZ-1102 (MOISTURE ONLY) 20' RT. ℄

MOISTURES:

@ 42" RT. ℄ : 9.96 %
@ 66" RT. ℄ : 14.42 %

@ 24" 6' RT. ℄ : 4.91 %
@ 40" 6' RT. ℄ : 11.85 %
@ 50" 6' RT. ℄ : 16.01 %
@ 17'0" 20' RT. ℄ : 49.03 %

SEVERE LONGITUDINAL CRACKING
SAMPLE SECTION (2)



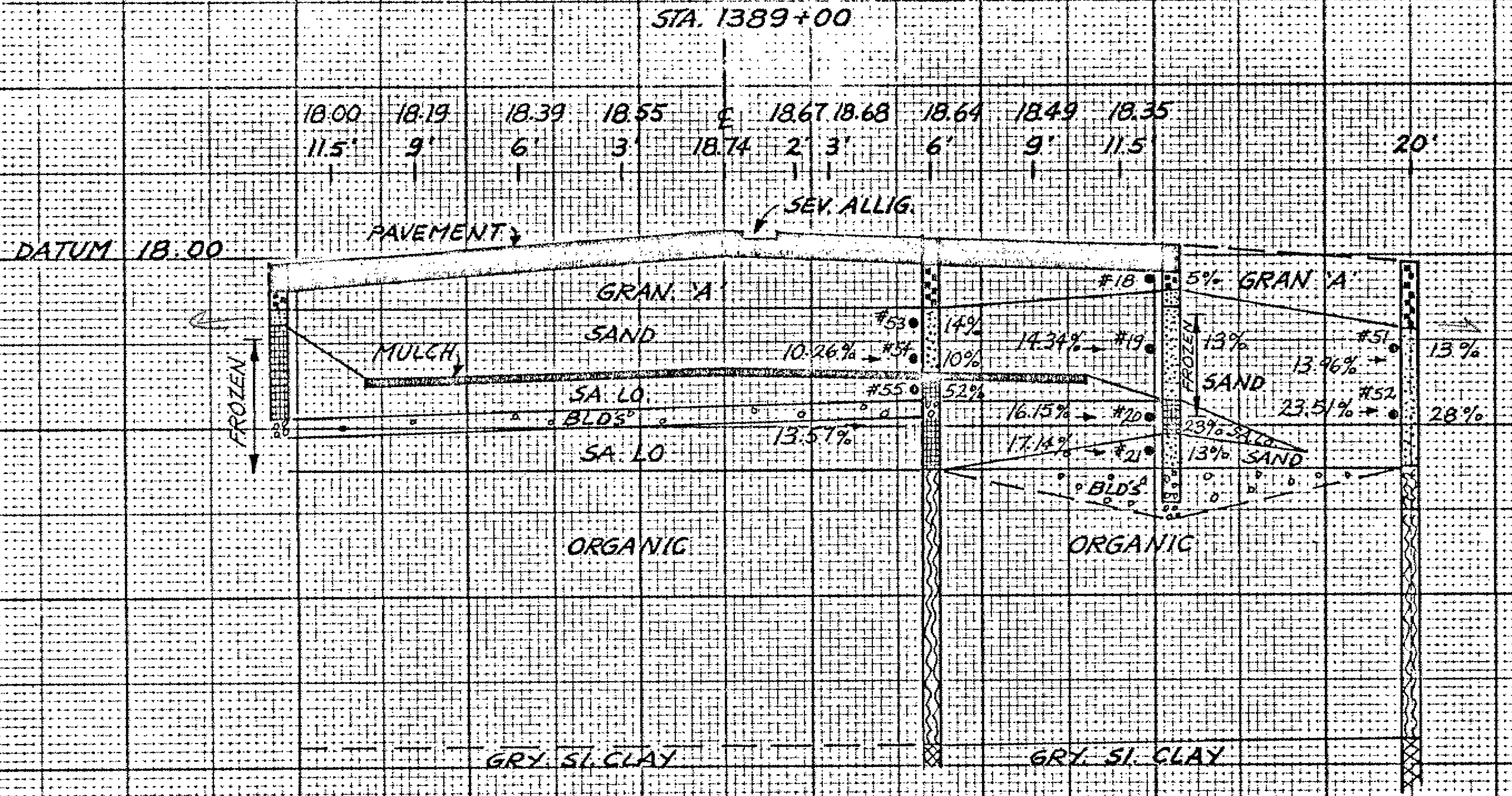
SAMPLES:
74-RN-13 (8"-13") LT. ℄ : SAND 5 % PASS #200
74-RN-14 (30"-42") LT. ℄ : SAND 9 % " "
74-RN-15 (48"-60") LT. ℄ : SAND 8 % " "
74-RN-16 (60"-72") LT. ℄ : LT. CL. 62 % " "
74-RN-17 (90"-102") RT. ℄ : SAND 37 % " "

MOISTURES:
@ 36" LT. ℄ : 18.28 %
@ 54" LT. ℄ : 9.47 %
@ 66" LT. ℄ : 28.59 %

SAMPLES:
74-R22-1041 (18"-30") 20' RT. ℄ : SAND 21 % PASS #200
74-R22-1042 (30"-48") 20' RT. ℄ : SAND 27 % " "
74-R22-1043 (14'-15') 20' RT. ℄ : SI. CL. 100 % " "
74-R22-1044 (18"-24") 7' RT. ℄ : SAND 13 % " "
74-R22-1045 (30"-36") 7' RT. ℄ : SAND 11 % " "
74-R22-1046 (36"-50") 7' RT. ℄ : SA. CL. LO. 50 % " "
74-R22-1102 (MOISTURE ONLY)

MOISTURES:
@ 24" 20' RT. ℄ : 16.98 %
@ 36" 20' RT. ℄ : 21.48 %
@ 42" 7' RT. ℄ : 17.30 %
@ 67" 20' RT. ℄ : 21.51 %

SEVERE LONGITUDINAL CRACKING
SAMPLE SECTION (3)



SAMPLES: 74-RN-18 (8"-14") RT. ℄ : SAND 5 % PASS #200
74-RN-19 (30"-42") RT. ℄ : SAND 13% " "
74-RN-20 (54"-66") RT. ℄ : SA. LO. 23% " "
74-RN-21 (66"-78") RT. ℄ : SAND 13% " "

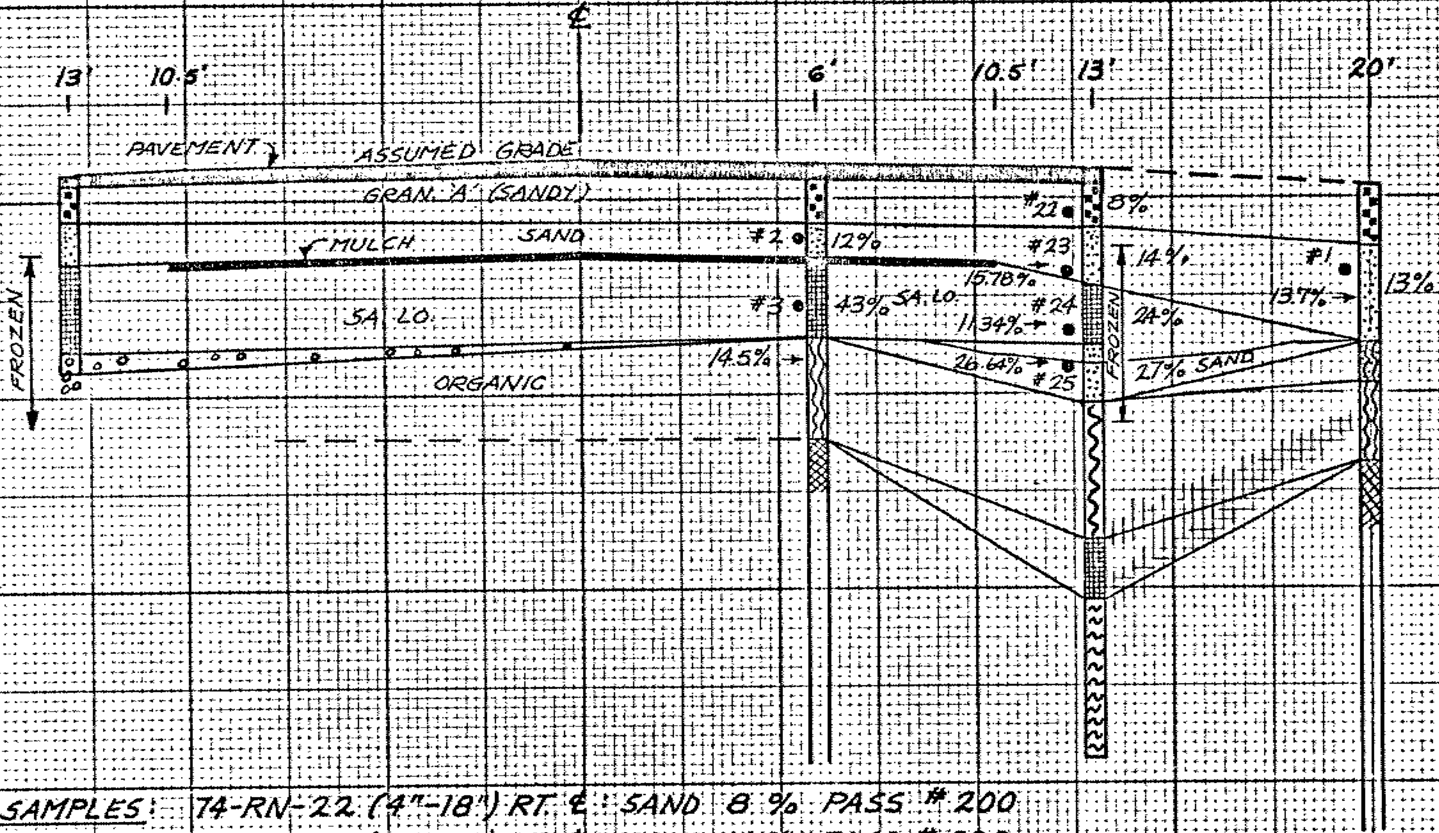
MOISTURES: @ 36" RT. ℄ : 14.34 %
@ 60" RT. ℄ : 16.15 %
@ 72" RT. ℄ : 17.14 %

SAMPLES: 74-RZZ-1051 (24"-48") 20' RT. ℄ : SAND 13 % PASS #200
74-RZZ-1052 (48"-60") 20' RT. ℄ : SAND 28 % " "
74-RZZ-1053 (24"-36") 6' RT. ℄ : SAND 14 % " "
74-RZZ-1054 (36"-48") 6' RT. ℄ : SAND 10 % " "
74-RZZ-1055 (48"-84") 6' RT. ℄ : SA. LO. 52 % " "

MOISTURES: @ 36" 20' RT. ℄ : 13.96 %
@ 54" 20' RT. ℄ : 23.51 %
@ 42" 6' RT. ℄ : 10.26 %
@ 66" 6' RT. ℄ : 13.57 %

COMPARABLE SAMPLE SECTION.

0.51 MILES WEST OF GREAT LAKES PAPER DOG RIVER ROAD



SAMPLES:
74-RN-22 (4"-18") RT. ℄ : SAND 8 % PASS #200
74-RN-23 (24"-36") RT. ℄ : SAND 14 % PASS #200
74-RN-24 (42"-54") RT. ℄ : SANDY LOAM 24 % PASS #200
74-RN-25 (54"-60") RT. ℄ : SAND 27 % PASS #200

MOISTURES:
@ 30" RT. ℄ : 15.78 %
@ 48" RT. ℄ : 11.34 %
@ 60" RT. ℄ : 26.64 %

SAMPLES:
74-R22-1001 (18"-30") 20' RT. ℄ : SAND 13 % PASS #200
74-R22-1002 (18"-30") 6' RT. ℄ : SAND 12 % PASS #200
74-R22-1003 (36"-54") 6' RT. ℄ : SA. LO. 43 % PASS #200

MOISTURES:
@ 36" 20' RT. ℄ : 13.70 %
@ 60" 6' RT. ℄ : 14.57 %

FIGURE 1.

	NO CRACK			SL - MOD CRACK			SEVERE CRACK			AVERAGE		
	℄	E/S	DIFF.	℄	E/S	DIFF.	℄	E/S	DIFF.	℄	E/S	DIFF.
0 - 24	22	46	+ 24	32	42	+ 10	31	41	+ 10	28	43	+ 15
0 - 48	28	44	+ 16	29	42	+ 13	30	37	+ 7	29	41	+ 12
0 - 72	37	41	+ 4	32	42	+ 10	38	42	+ 4	37	42	+ 5
0 - 96	37	41	+ 4	32	42	+ 10	38	42	+ 4	36	42	+ 6

FIGURE 2.

	AVE. MOIST	AVE. DEPTH	NO SAMPLES
℄	13.46	49.50	8
E/P	15.34	50.50	12
E/S	25.69	43.40	5

	NO CRACKING	SL - MOD CRACKING	SEVERE CRACKING
AVE. MOIST	11.43	14.51	15.92
AVE. DEPTH	44.40	54.00	51.50