#65-F-215

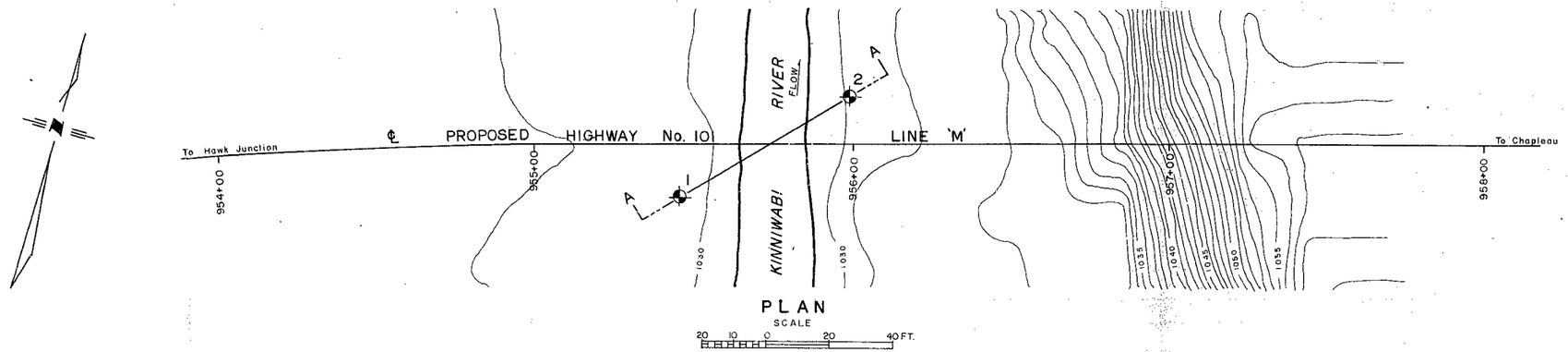
W.P. #83-62

Hwy. #101 &

KINNIWABI

RIVER CENTRE

CROSSING



PLAN
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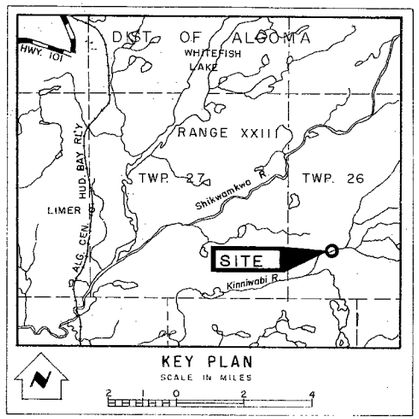

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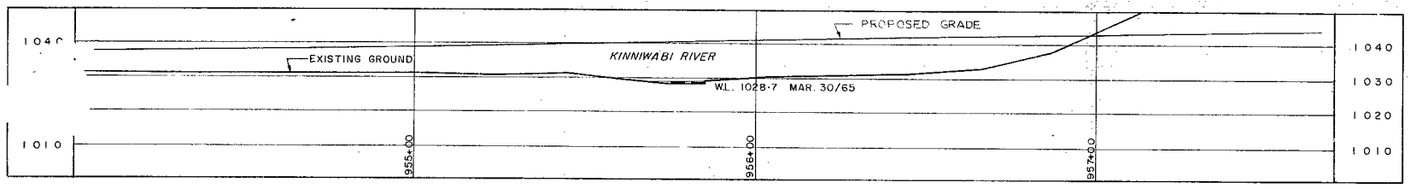

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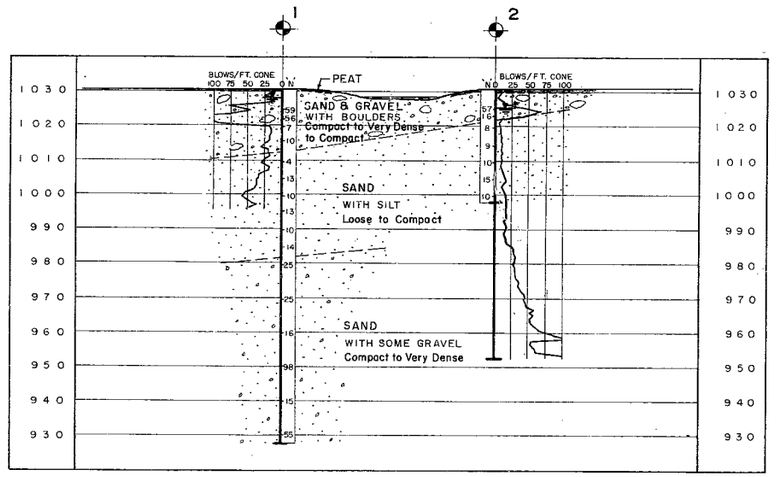

SECTION A-A
SCALE
20 10 0 20 40 FT.

* Measured Ground Water Level at time of field investigation (Insufficient time for ground water level to reach equilibrium at Bore Hole 2)

LEGEND

- Bore Hole
- Cone Penetration Hole
- Bore & Cone Penetration Hole
- Water Levels established at time of field investigation. (MAR. 1965)

NO.	ELEVATION	STATION	OFFSET
1	1030·6	995+46	17' RT.
2	1030·8	995+99	15' LT.

B.M. ELEV. 1037·25
N.W. Top 0·5 Spr. Stump
80' LT. Sta. 951+00

– NOTE –

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

PRINT RECORD		
NO.	FOR	DATE

REVISIONS		
DATE	BY	DESCRIPTION

GEOCON LTD
DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & RESEARCH DIVISION – FOUNDATION SECTION

KINNIWABI RIVER
CENTRE CROSSING

KING'S HIGHWAY NO. 101 LINE 'M' DIST. NO. 18
DISTRICT OF ALGOMA
TWP. No. 26 LOT RANGE XXIII

BORE HOLE LOCATIONS & SOIL STRATA

SUBM'D. H.L.M.	CHECKED D.B.O.	W.P. NO. 83 – 62	DRAWING NO. T7738-1
DRAWN A.E.L.	CHECKED B.T.D.	JOB NO.	
DATE APR. 9, 1965		SITE NO.	BRIDGE DRAWING NO.
APPROVED D. B. Oates		CONT. NO.	

REF. No. E-2839-1

83-62

GEOCON LTD

HEAD OFFICE

420 MICHEL JASMIN, DORVAL, QUEBEC

TELEPHONE 631-9827

DISTRICT OFFICES

14 HAAS ROAD
REXDALE, TORONTO, ONT.
TEL. 244-6476

1425 WEST PENDER ST.
VANCOUVER 5, B.C.
TEL. MU. 1-8926

Rexdale, Ontario,
April 20th, 1965.

Department of Highways, Ontario,
Materials and Testing Division,
Downsview, Ontario.

Attention: Mr. A. G. Stermac, P. Eng.,
Principal Foundation Engineer.

Re: Soil Conditions and Foundations
Proposed Kinniwabi River Crossing
Station 956+20, Township 26
District 18, Ontario

Dear Sirs:

This letter accompanies our detailed report on the above investigation.

We find that aside from a thin surficial cover of peat, the overburden at the site, to the depth explored, consists of a complex succession of sands and gravels. The actual soil and groundwater conditions encountered are discussed in detail in the report.

Based on the findings of this investigation, it is considered that the proposed bridge may be carried on spread footings if measures such as an engineered fill base and scour protection are applied as discussed in the report. Recommendations covering the allowable bearing value for footing design, and other soil mechanics factors pertinent to design and construction are discussed in the report.

We believe that this report covers the information required from this investigation. Should you require further information or wish us to elaborate on any aspect of the discussion given herein, as design progresses, we would be pleased if you would give us a call.

Yours very truly,

GEOCON LTD

M. A. J. Matich per RJBO

MAJM/reb
T7738
Enc.

M. A. J. Matich, P. Eng.,
President.

T7738

REPORT

TO

DEPARTMENT OF HIGHWAYS, ONTARIO

ON

SOIL CONDITIONS AND FOUNDATIONS

PROPOSED KINNIWABI RIVER CROSSING

STATION 956+20, TOWNSHIP 26

DISTRICT 18 ONTARIO

Distribution:

11 copies - Department of Highways, Ontario
Downsview, Ontario

3 copies - Geocon Ltd
Rexdale, Ontario

GEOCON

INDEX

INTRODUCTION

Geocon Ltd has been retained by the Department of Highways, Ontario (D.H.O.) by letter dated March 8th, 1965, Work Permit No. 83-62, to investigate and report on the soil conditions for the Centre Bridge Crossing to be located on Line "M" along proposed King's Highway 101, over the Kinniwabi River, approximately at the location of an existing 30 foot span timber log bridge over the river. The site is located in Township 26, Range XXIII, District of Algoma, Ontario.

The object of the investigation was to determine and interpret the soil and water conditions as they effect design of foundations for the proposed bridge.

SUMMARIZED SOIL CONDITIONS

At the borehole locations, the site is relatively flat lying with a surficial cover of dark brown peat about 1 foot thick. Underlying the peat is a stratum of compact to very dense brown sand and gravel with a high boulder content and a trace of silt; the thickness of this stratum averages about 15 feet, and appears to increase in thickness in the westerly direction. Underlying the sand and gravel stratum is a stratum of loose to compact grey sand with silt, with a trace of gravel at localized areas; the encountered thickness of this stratum was about 32 feet. Directly underlying the sand

with silt stratum is a stratum of brown sand with some gravel, and a trace of silt; this stratum was penetrated for a depth of about 55 feet at borehole 1. Bedrock was not encountered within the depth of exploration.

The ground water level was encountered at depths of 2.5 and 5.5 feet below ground surface level at boreholes 1 and 2, respectively. At Line "M", the Kinniwabi River was approximately 25 feet wide and about 3 feet deep, measured from surface elevation 1028.7 at the time of the investigation. The high water level of the river is understood to be at elevation 1031.7.

DISCUSSION

It is understood that a bridge crossing of the Kinniwabi River is planned along proposed Line "M". The bridge will be located at about chainage 956+20 along Line "M", and is designated as the Centre Bridge Crossing. The proposed bridge will form part of proposed King's Highway 101 between Hawk Junction and Chapleau, Ontario. The bridge will be located approximately in the vicinity of an existing 30 foot span timber log bridge which it will replace. The location of the proposed structure is shown on Drawing T7738-1 at the rear of this report.

As presently planned, the bridge will have a clear span of 44 feet from abutment to abutment and be simply supported, with the abutments founded on concrete strip footings about 5 feet wide. The proposed foundation elevation is 1023, i.e. approximately 7.5 feet below existing ground level at the borehole locations. It is understood that the bridge deck will have an overall width of 36 feet and an elevation of approximately 1040; the underside of the bridge girder will be at approximately elevation 1037. The bridge will incorporate two 15 foot traffic lanes.

The bridge is to have approach embankments, with the maximum height of fill being about 10 feet. Immediately to the east, a cut of about 10 feet is required to reach proposed grade. The proposed grade of the approach embankments is 2 percent increasing to the east. It is understood that the crest width of the approach embankments are to be 40 feet with side slopes of 2 horizontal to 1 vertical (2:1). In the immediate vicinity of the abutments the embankment is to be composed of rock fill with side slopes of 1-1/4:1. This information and other pertinent facts are summarized on the D.H.O. Drawing E-2839-1 and the Johnson-Anderson-Bruce, Consulting Engineers Drawing D-5388-1, both of which were supplied to us.

The discussion which follows deals with the soil mechanics aspects of foundations and earthworks involved in the proposed design of the bridge structure and associated approach embankments.

Foundations

The topsoil is unsuitable as a foundation material and should be removed beneath the location of the embankments and the structure foundations. It is recommended that at least 6 feet of earth cover be provided as frost protection to the bridge foundations and that the latter be adequately protected against possible undermining by scour.

With proposed foundation elevation at 1023, it is considered that the bridge abutments may be carried on spread footings. The investigation has shown that at, and immediately below, this elevation there is a considerable variation in the relative density of the foundation soil. In order to even out possible differential settlements of the structure and also to increase the permissible footing contact pressure, it is recommended that a three foot thick base of engineered fill be placed beneath each footing. The fill should be well graded, minus 3 inch, clean sand and gravel compacted uniformly to at least 100 percent of modified A. A. S. H. O. dry density. With this provision a net allowable bearing value of 2. 0 tons per square foot may be used for footing design and settlements should be within tolerable limits for the type of construction proposed.

Foundations (continued)

It is recommended that the backfill immediately behind the abutments consist of clean free draining well compacted granular material, and that the backfill be provided with positive drainage to avoid build-up of hydrostatic pressure behind the abutments. If this is done the abutments and retaining walls should be designed for a lateral earth pressure coefficient of 0.4. The abutments should be designed for a factor of safety of at least 1.5 against sliding based on a coefficient of friction of concrete to the sand and gravel of 0.45.

The use of spread footings as above would require adequate surface protection against undermining by scour since the natural overburden generally, and particularly the sand with silt stratum, is scour susceptible. The extent of the rip-rapping required to protect both the foundations and the ends of the approach embankments is dependent partly on other than soil mechanics considerations, such as the constricting effect of the structure on the river flow at high stages. The assessment of such effects is beyond the scope of this report.

An alternative foundation solution which may merit study from the points of view of both bearing and scour protection would be the use of piles carried an adequate distance beyond maximum

Foundations (continued)

possible depth of scour. A variety of pile types would be suitable including treated timber piles, precast concrete, steel tube or other displacement type piles. Cast-in-place expanded base concrete piles would also be suitable. Irrespective of type, pile installation will have to contend with boulders in the upper most stratum of sand and gravel, and timber piles in particular will require a suitable shoe and possibly jet assistance to prevent damage during driving.

Construction of the abutment foundations and granular base as above will involve excavation through pervious sand and gravel to a depth of about 8 feet below observed groundwater level. Suitable dewatering measures such as the use of well points or wells, or pumping from sumps from within sheeted enclosures, will therefore be required. In this connection the installation of either well points or sheeting will have to contend with the presence of boulders in the overburden. Sheeting if used should be carried below base of excavation a distance at least equal to the maximum head differential likely to occur during construction. Sumps used for dewatering should be maintained in advance of excavation level. The sand and gravel at base of excavation should be compacted prior to placement of engineered fill base for the footings.

Approach Embankments

The approach embankments will have a maximum height relative to existing river bottom of about 14 feet. If the river bottom is protected against scour and the surficial organic material is removed as discussed, the overall stability of the proposed embankments should be satisfactory. It is assumed for purposes of this discussion that the embankments would be composed of compacted, clean, granular fill with side slopes of 2 horizontal to 1 vertical (2:1) and protected against scour where necessary.

CONCLUSIONS AND RECOMMENDATIONS

1) The site is covered by a layer of peat about 1 foot thick. The main overburden strata beneath this are compact to very dense sand and gravel, loose to compact sand with silt, and compact to very dense sand with some gravel, in this order of occurrence with depth.

2) The Kinniwabi River in the vicinity of the bridge crossing was about 25 feet wide and 3 feet deep at its deepest point, at the time of the investigation. Past surveys in the area indicate that at extreme high water level the river level is about elevation 1031.5. At the time of the investigation river level was at elevation 1028.

3) The bridge abutments may be founded on spread footings at elevation 1023 as proposed providing that suitable protection against possible undermining by scour is applied. With the use of an engineered filled base as discussed, a net allowable bearing value of 2.0 tons per square foot would be permissible for footing design.

4) As an alternative to the use of spread footings, pile foundations might be considered as discussed.

5) A discussion of various soil mechanics factors pertinent to construction of foundations and earthworks is given in the report.

PERSONNEL

The field work was carried out under the supervision of Mr. H. L. MacPhie. This report was written by Mr. B. T. Darch, checked by Mr. D. B. Oates, P. Eng. and reviewed by Mr. M. A. J. Matich, P. Eng.

B. T. Darch

BTD/reb
T7738

B. T. Darch, P. Eng.,
Senior Soils Engineer.

REGISTERED PROFESSIONAL ENGINEER
B. T. DARCH
PROVINCE OF ONTARIO

APPENDIX I

PROCEDURE

SITE AND GEOLOGY

SOIL AND WATER CONDITIONS

OFFICE REPORTS ON SOIL EXPLORATION

PROCEDURE

The field work was carried out between March 23rd, 1965 and March 27th, 1965. Two boreholes, each with an accompanying dynamic uncased cone penetration test, were put down in BX size. For the work a skid mounted diamond drill rig was used.

The granular strata of the overburden were sampled with a 2 inch O.D. split spoon sampler, adapted where necessary with a foot valve to aid in the recovery of the material. The bedrock was not encountered at either borehole put down. The groundwater level was recorded in the cased boreholes during the exploration programme.

Detailed logs of the two boreholes are presented on the Office Reports on Soil Exploration in this Appendix. The location of the boreholes together with the inferred stratigraphy, are shown on Drawing T7738-1, at the rear of this report.

The laboratory testing of selected soil samples was carried out in the Toronto Soil Mechanics Laboratory of Geocon Ltd. The results are plotted on the Figures in Appendix II. The soil samples remaining after testing will be stored until May 1st, 1966 at which time you will be contacted for instructions regarding their disposal.

All elevations in this report are referred to Geodetic datum supplied by the D. H. O. Survey Crew. The benchmark referred to is a nail and washer in the top of 1/2 foot diameter Spruce stump, located 80 feet left of chainage 951+00, along proposed line "M". The Geodetic elevation of this benchmark is 1037.25.

SITE AND GEOLOGY

The proposed D. H. O. Centre Bridge Crossing site is located on line "M" at approximately chainage 956+20, along proposed King's Highway 101 over the Kinniwabi River; approximately at the location of an existing timber log bridge over the river. The site is located in Township 26, Range XXIII, District of Algoma, Ontario. The right-of-way has been cleared for approximately 80 feet on either side of the centre line of line "M", while the remainder of the area is covered by a dense cover of Pine, Spruce and Fir trees. Moving in a westerly direction along line "M" the area is gently undulating terrain; approximately 70 feet east of the proposed bridge structure a hill about 23 feet high is located. In general, the site is surrounded by high ground composed of low mountains which are mainly bedrock outcrops. The surficial cover in the vicinity is a dark brown fibrous peat; with poor surface drainage. At the time of the investigation the Kinniwabi River was about 20 feet wide, and approximately 3 feet deep in the vicinity of the proposed bridge site, and flowed in a northerly direction.

From available geological information and previous investigations in the area, it is known that the general area is covered by silts, sands, gravel and boulder deposits of varying thickness overlying igneous intrusive and related metamorphosed bedrock types of Precambrian origin.

SOIL AND WATER CONDITIONS

The principal soil strata encountered at boreholes 1 and 2 are as follows:

Dark Brown Peat

The surficial cover across the site is a dark brown highly compressible, fibrous peat containing roots, wood and related decayed organics. The ground surface elevation in the vicinity of the proposed bridge structure, which corresponds to the surface elevation of the peat is about elevation 1031. The encountered thickness of the layer was about 1 foot at both boreholes.

Compact to Very Dense Brown Sand and Gravel with Boulders

Underlying the peat surficial cover in both boreholes is a stratum of brown sand and gravel with boulders and a trace of silt. The surface elevation of the stratum is about 1030 in both boreholes, while the encountered thickness of the stratum was 8 feet at borehole 2 and 17.5 feet at borehole 1.

Compact to Very Dense Brown Sand and Gravel with Boulders (continued)

The stratum is basically composed of a matrix of silt and sand binding gravel and boulders. Boulders up to 8 inches in size were visually observed in side slopes near the river; these boulders were of varying mineralogical composition, mainly of an igneous nature. At localized areas the boulder content of the stratum is high; this specifically was noted between depths of 5 to 10 feet below ground surface level. The granular component of the stratum is sub-angular in shape.

A mechanical analysis was carried out on a typical sample from the stratum and the results shown on Figure 1 of Appendix II. The grain size distribution curve indicates that the sample consists of the following; 8 percent silt sizes, 50 percent sand sizes and 42 percent gravel sizes.

Five standard penetration tests carried out in the stratum gave "N" values ranging from 7 to 59 blows per foot. The high values occurred at depths between 5 and 10 feet below the ground surface level, while the lower blows occurred below this depth. The accompanying uncased dynamic cone penetration tests gave values which indicated the variation in composition described above, i. e. the higher values obtained from these penetration tests occurred at about the same depths as noted

Compact to Very Dense Brown Sand and Gravel with Boulders (continued)

above for the standard penetration tests. The relative density of the stratum is believed to be compact to a depth below ground surface level of about 5 feet, changing to very dense between depths of 5 to 10 feet and becoming compact below 10 feet depth. It should be noted that the very dense zone of the stratum corresponds with the area within the stratum, where the boulder content is inferred to be high.

For design purposes the following parameters may be used, where appropriate:

Wet unit weight	130 pounds per cubic foot
Angle of shearing resistance	35 degrees

Loose to Compact Grey Sand with Silt

Directly underlying the brown sand and gravel with boulders stratum in both boreholes is a stratum of predominantly grey sand with silt. The surface elevation of the stratum is about 1013 at borehole 1 and about 1022 at borehole 2. The thickness of the stratum was found to be about 31 feet at borehole 1; the base of the stratum was not proven at borehole 2. The stratum is composed of silts and sands in various proportions, with some stratification and a trace of gravel noted at localized areas. Boulders were not encountered in the stratum at the boreholes but may occur elsewhere.

Loose to Compact Grey Sand with Silt (continued)

Mechanical analyses were carried out on four typical samples from the stratum and the results are shown on Figure 2 of Appendix II. The grain size distribution curves obtained from the tests indicate that the samples consist of the following; between 20 and 30 percent silt sizes, 66 to 80 percent sand sizes and zero to 13 percent gravel sizes. In general, the stratum is mainly a fine sand with silt, with random layering of fine to coarse sand with silt and gravel sizes. It is estimated that the maximum thickness of these layers is about 4 feet. Figure 2 of Appendix II shows a grain size curve of a typical sample from one of these layers in borehole 2.

Eleven standard penetration tests, carried out in this stratum, gave "N" values ranging from 4 to 15 blows per foot with an average of about 11 blows per foot. Based on these values the relative density of the stratum is believed to vary from loose to compact, and to be generally compact.

For design purposes, the following parameters may be used, where appropriate:

Wet unit weight	125 pounds per cubic foot
Angle of shearing resistance	35 degrees.

Compact to Very Dense Brown Sand with Some Gravel

Directly underlying the grey sand with silt stratum at borehole 1, and inferred to underlie the same stratum at borehole 2, is a stratum of brown sand with some gravel and a trace of silt; the boulder content is less than that encountered in the surficial sand and gravel stratum. The surface elevation of the stratum was found to be 982 at borehole 1, the stratum was penetrated for a depth of about 54 feet at this location, however; the base of the stratum was not proven. The stratum is composed mainly of well graded sand with some gravel. A seam of grey silt with fine sand about 3 feet thick, was encountered at about elevation 941 within the stratum. The granular component of the stratum is sub-angular.

Two mechanical analysis tests were performed on samples from the stratum and the results are shown on Figure 3 of Appendix II. The grain size distribution curve for one of the tests on a sample from the stratum indicates that the material consists of 2 percent silt sizes, 90 percent sand sizes and 8 percent gravel sizes. Visual and tactile examination of all the samples from this stratum indicates that the gravel component is generally slightly higher than the indicated percentage from this test. A second grain size distribution curve is shown for a typical sample from the silt seam; this indicates that the material consists of 65 percent silt sizes and 35 percent fine sand sizes.

Compact to Very Dense Brown Sand with Some Gravel (continued)

Six standard penetration tests carried out in the stratum gave "N" values ranging from 16 to 98 blows per foot, with an average of about 35 blows per foot; the values show a general tendency of increasing with depth. Based on these values the relative density of the stratum varies from compact to very dense, and is generally dense.

For design purposes, the following parameters may be used, where appropriate:

Wet unit weight	135 pounds per cubic foot
Angle of shearing resistance	40 degrees.

Water Conditions

The groundwater level observations were taken in the cased boreholes during the period of investigation. The ground water level was encountered at depths below ground surface level of 2.5 feet at borehole 1 and 5.5 feet at borehole 2, these depths correspond to elevations of about 1028 and 1025, respectively. It should be noted that insufficient time was available during the period of investigation to allow the water level to reach equilibrium in borehole 2; it is therefore inferred that the water level at this location may actually be at a higher elevation than noted above. The water levels are plotted on the stratigraphy on Drawing T7738-1. At the time of the investigation the river level was at elevation 1028.7. High river level is belived to be at elevation 1031.7.

EXPLANATION OF THE FORM "OFFICE REPORT ON SOIL EXPLORATION"

The object of this form is to enable a comprehensive study of the soil to be made by combining on one sheet all of the information obtained from the boring. An explanation of the various columns of the report follows.

ELEVATION AND DEPTH

This column gives the elevation and depth of boundaries between the various soil strata. The elevation is referred to the datum shown in the general heading.

WATER CONDITIONS

In this column the water level in the casing at the time of boring or the water table in the ground, determined by a series of observations in a piezometer or standpipe, is indicated to scale by a horizontal line with the symbol W.L. or W.T. above the line. A notation of any complicated groundwater conditions will be made in this column.

DESCRIPTION

A description of the soil, using standard terminology, is contained in this column. The consistency of cohesive soils and the relative density of non-cohesive soils are described by the following terms:

Consistency	U-Strength Tons/sq. ft.
Very soft	0.03 to 0.25
Soft	0.25 to 0.5
Firm	0.5 to 1.0
Stiff	1.0 to 2.0
Very stiff	2.0 to 4.0
Hard	over 4.0

Relative Density	Standard Penetration Resistance, Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

STRATIGRAPHIC PLOT

The stratigraphic plot follows the standard symbols of the National Research Council, Canada.

ELEVATION SCALE

The information in all columns is plotted to a true elevation scale which is shown in this column.

GRAPHS

The main body of the report forms a graph which is used to plot to correct elevation the important soil properties which are obtained through field and laboratory tests. The scales and symbols for the plotting are shown at the head of the column.

OTHER TESTS

In this column are shown, by symbol, the other field or laboratory tests which have been performed on the soil and for which the results have not been plotted on the above graph.

SAMPLES

The first three columns describe the condition, type and number of each sample obtained from the boring. The location and extent of each sample is plotted to scale.

In the last column is shown the penetration resistance in blows of 4200 inch-pounds required to drive one foot of the sampler into the ground. When a 2 inch Drive Sampler is used the result obtained is termed the "Standard Penetration Resistance".

GEOCON

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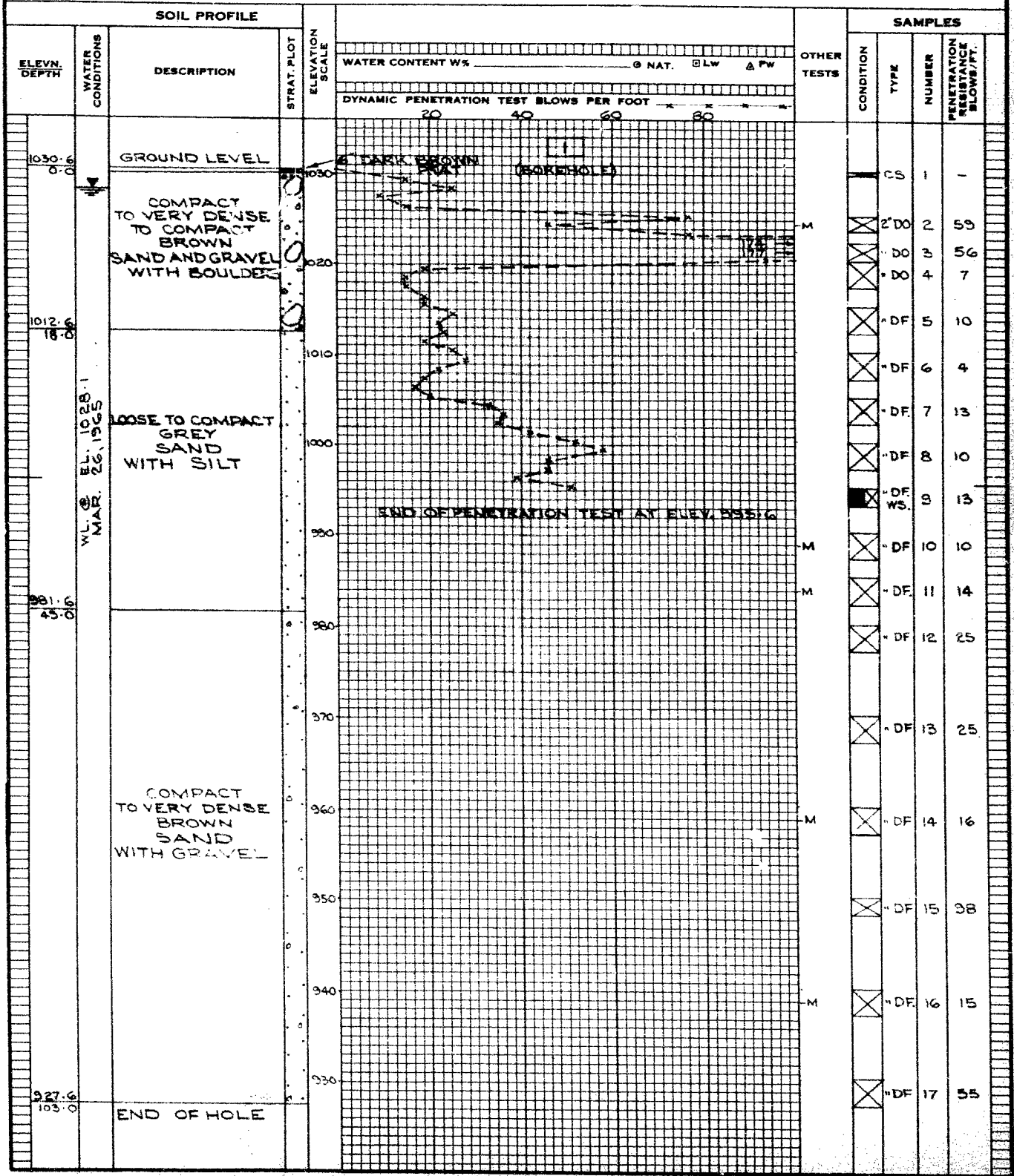
OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T7738 BORING # 1 DATUM GEODETIC CASING BX.
BORING DATE MAR. 23-25/65 REPORT DATE APR. 7, 1965 COMPILED BY A.E.L. CHECKED BY B.T.D.
SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION
DISTURBED
FAIR
GOOD
LOST

SAMPLE TYPES
A.S. - AUGER SAMPLE
S.T. - SLOTTED TUBE
W.S. - WASHED SAMPLE
D.O. - DRIVE-OPEN
D.F. - DRIVE-FOOT VALVE
C.S. - CHUNK SAMPLE
F.S. - FOIL SAMPLE
S.O. - SLEEVE-OPEN
S.F. - SLEEVE-FOOT VALVE
T.O. - THIN WALLED OPEN
R.C. - ROCK CORE

ABBREVIATIONS
V - IN-SITU VANE TEST
M - MECHANICAL ANALYSIS
U - UNCONFINED COMPRESSION
Qc - TRIAXIAL CONSOLIDATED UNDRAINED
Q - TRIAXIAL UNDRAINED
S - TRIAXIAL DRAINED
γ - WET UNIT WEIGHT
K - PERMEABILITY
C - CONSOLIDATION
WL - WATER LEVEL IN CASING
WT - WATER TABLE IN SOIL



APPENDIX I

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T7738 BORING # 2 DATUM GEODETIC CASING BX.
BORING DATE MAR. 26-27/65 REPORT DATE APR. 7, 1965 COMPILED BY A.E.L. CHECKED BY S.T.D.
SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION
DISTURBED
FAIR
GOOD
LOST

SAMPLE TYPES
A.S. - AUGER SAMPLE
S.T. - SLOTTED TUBE
W.S. - WASHED SAMPLE
D.O. - DRIVE-OPEN
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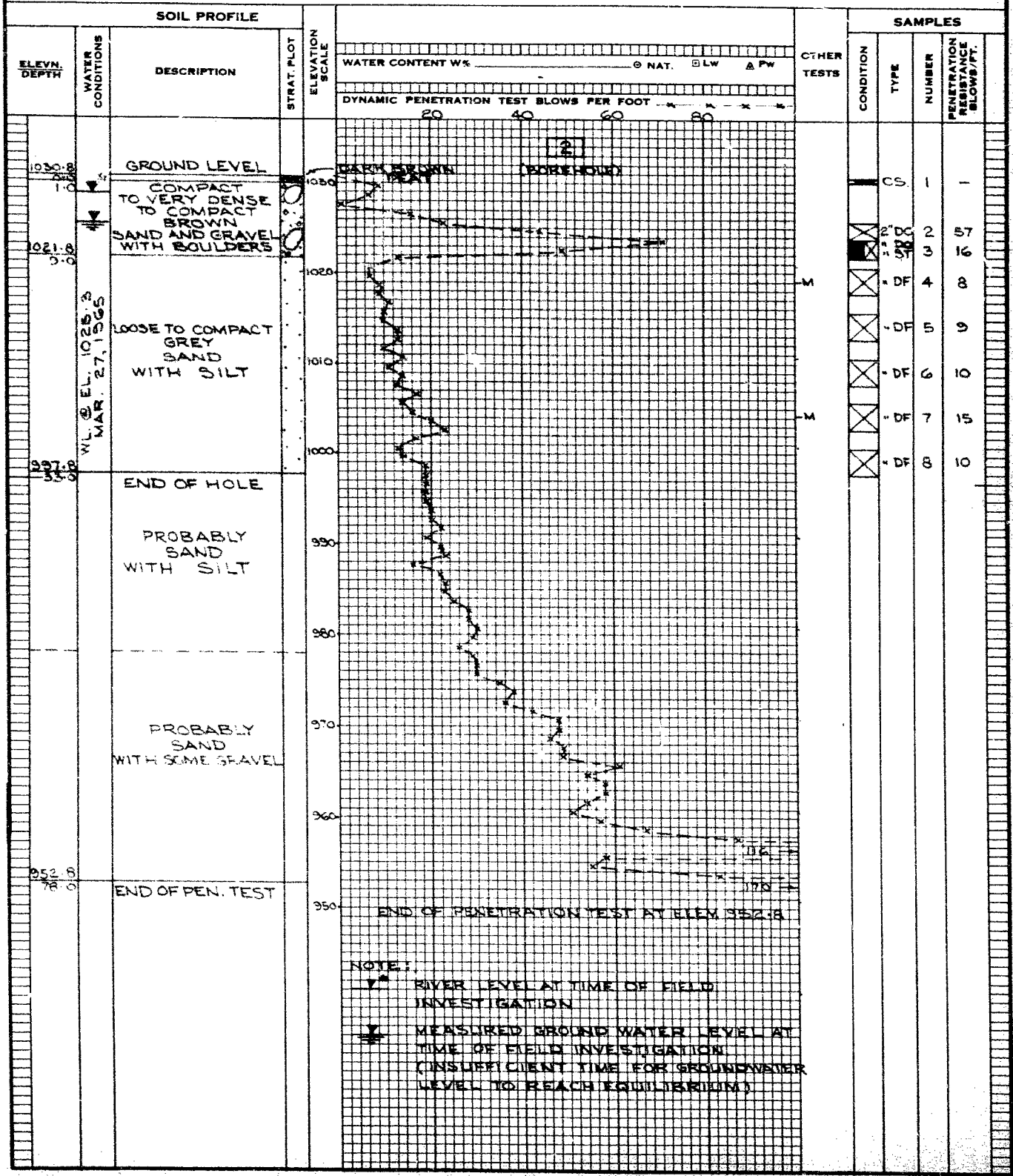
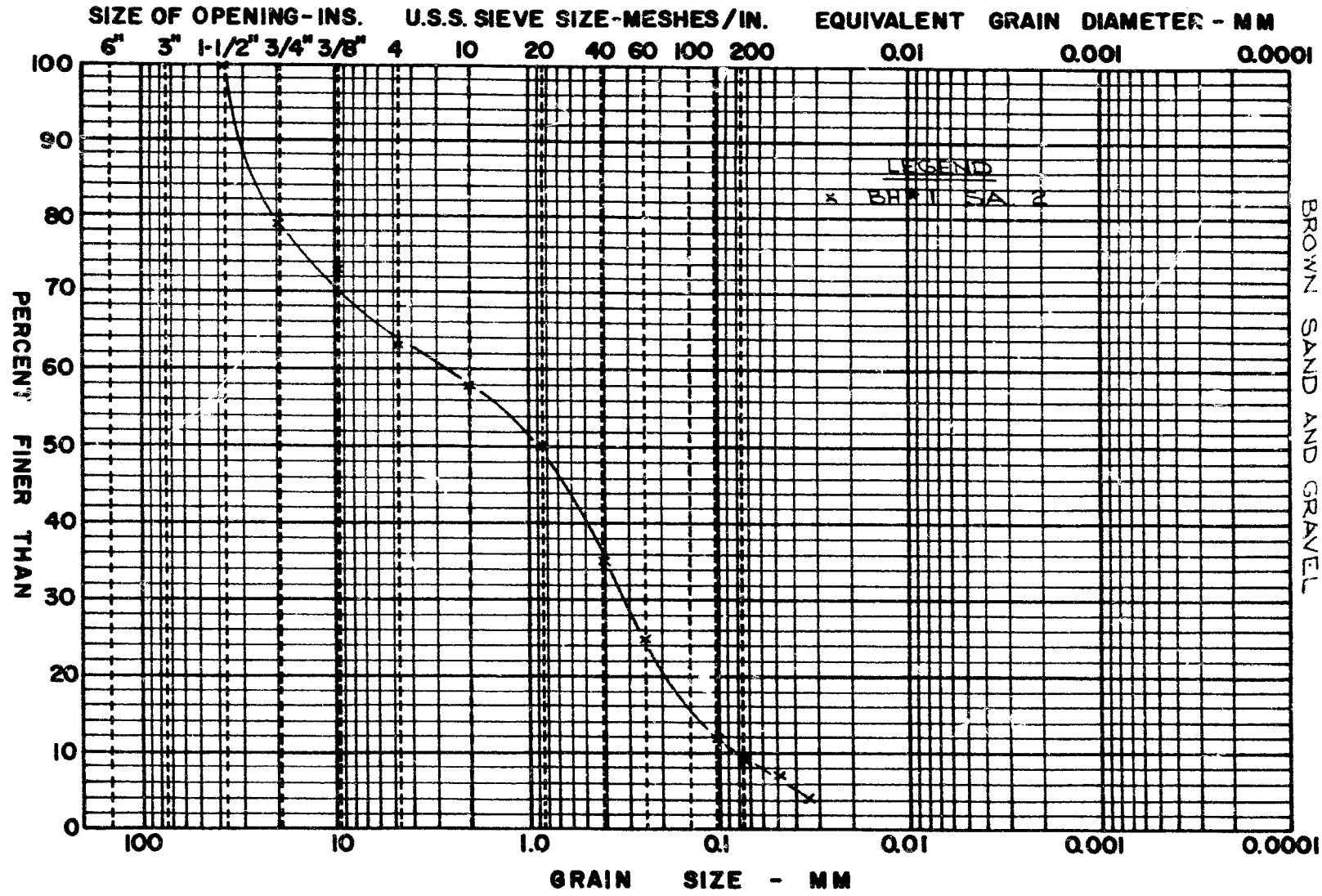


APPENDIX II

FIGURES - LABORATORY TESTING

GRAIN SIZE DISTRIBUTION

COBBLE SIZE →	GRAVEL SIZE			SAND SIZE			FINE GRAINED	
	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE →

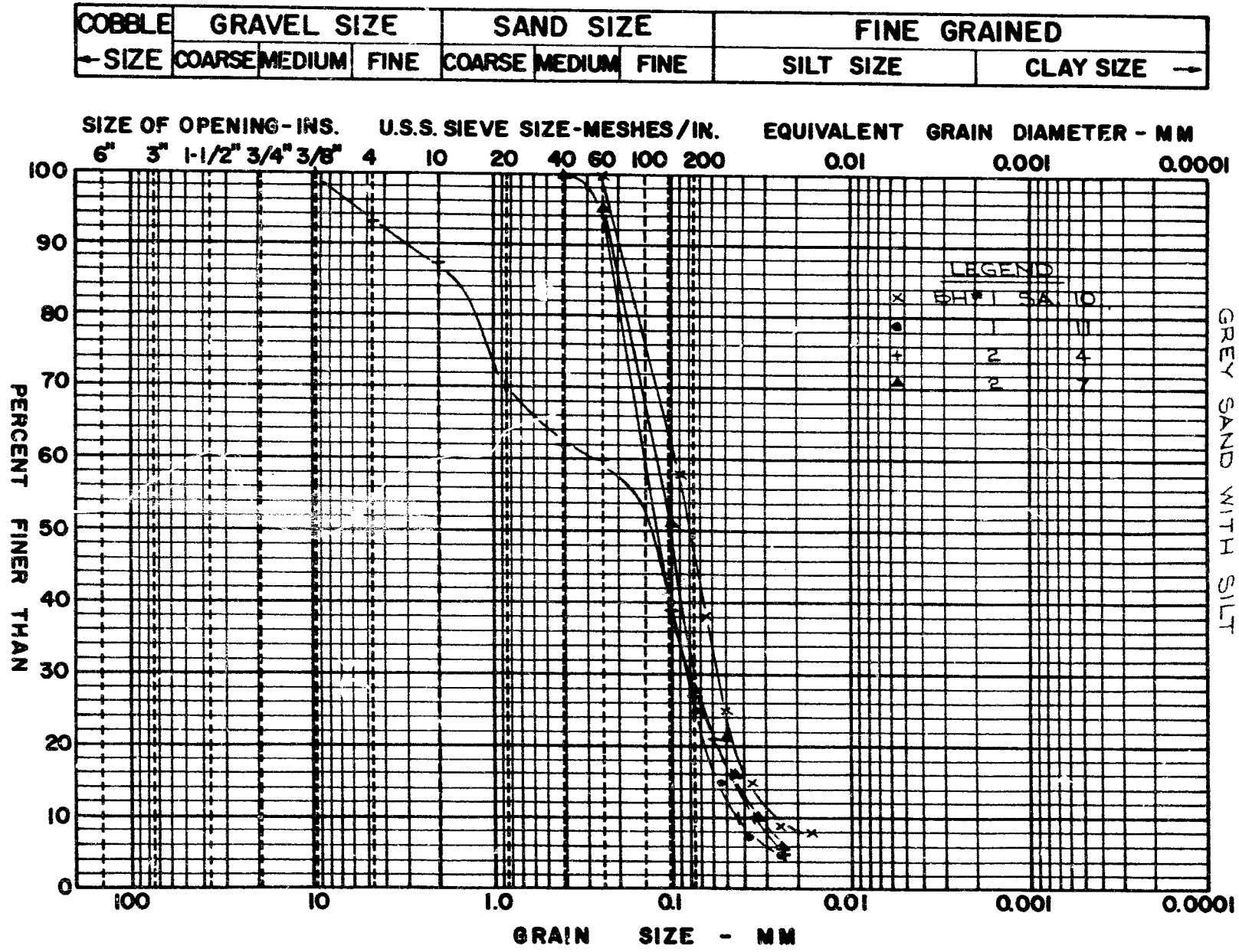


GEOCON

M.I.T. GRAIN SIZE SCALE

APPENDIX II
FIGURE 1
PROJECT T7738

GRAIN SIZE DISTRIBUTION

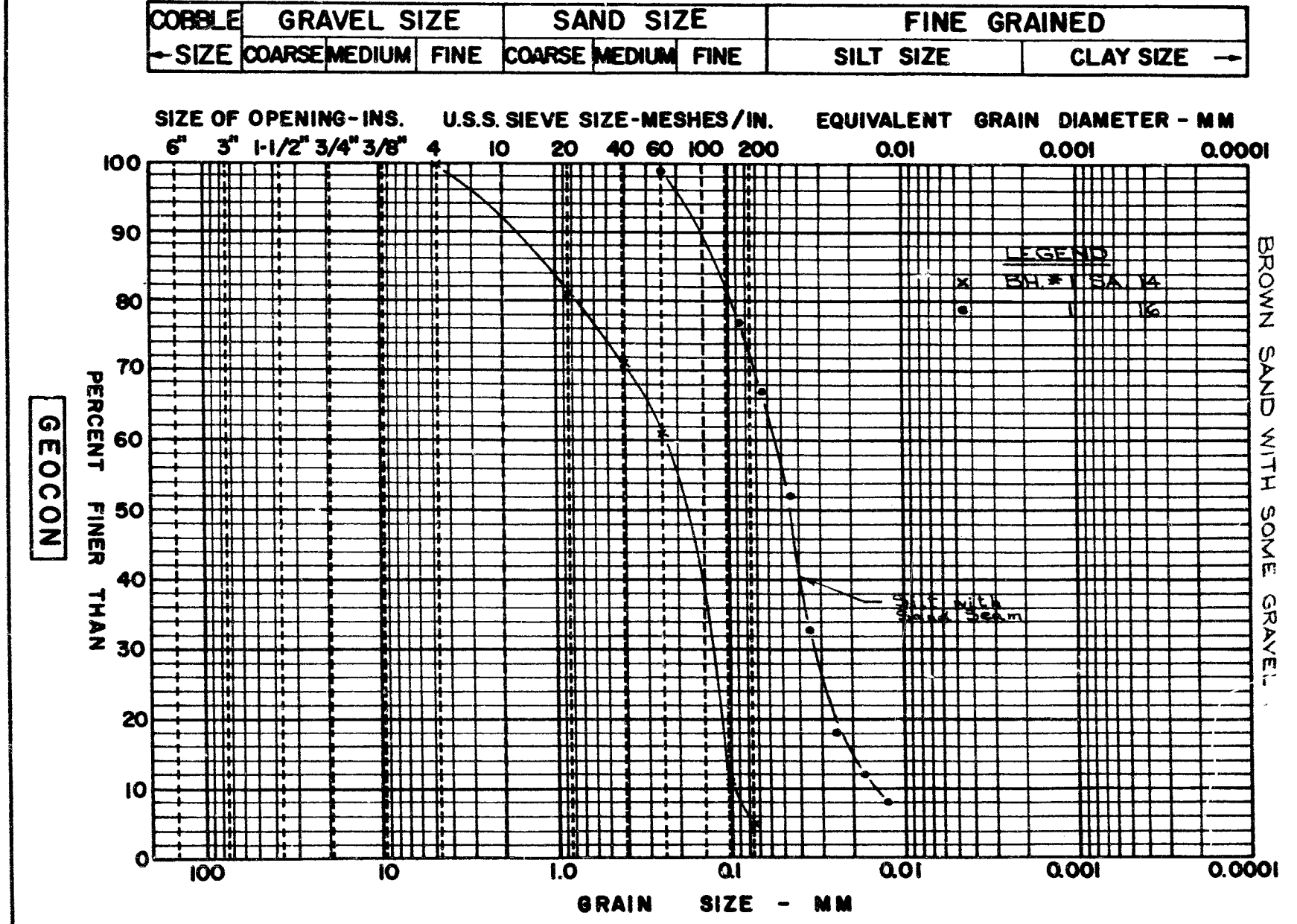


M.I.T. GRAIN SIZE SCALE

GEOCON

APPENDIX II
FIGURE 2
PROJECT T7738

GRAIN SIZE DISTRIBUTION



APPENDIX II
FIGURE 3
PROJECT T7738

cc: Foundations Office (Rm. 110)

Mr. A. M. Toye,
Bridge Engineer,
Bridge Division.

Foundation Section,
Materials & Testing Div.,
Room 107, Lab. Bldg.

Attention: Mr. J. McCombie

April 27, 1965

FOUNDATION INVESTIGATION REPORT BY:
Geocon, Limited, Consulting Engineers, Toronto.
Proposed Kinniwabi River Bridge (Centre Crossing)
District of Algoma, Twp. 26, Dist. 18, Hwy. 101.
W.P. 83-62

The above report prepared by the Consultant, Geocon, Ltd., has been reviewed and we submit the following comments for your consideration:

The recommended bearing pressure for spread footings of 2 t.s.f. appears to be too high for the standard penetration resistance recorded. However, the Consultant has explained the basis for the choice of this value in a letter which accompanies this report.

As for dewatering for construction purposes, an alternative of pouring tremie seal after excavation under water, should be considered since the presence of boulders would render other schemes difficult to be carried out.

We feel the information contained herein, is sufficient for your design purposes. However, should there be further queries, please do not hesitate to contact us.

A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

MYL/vdeF
Attach.
cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
H. R. McMillan
R. McArthur
A. A. Ward
B. R. Saint
F. De Visser
A. Watt

Foundations Office ✓
Gen. Files

GEOCON LTD

HEAD OFFICE

420 MICHEL JASMIN, DORVAL, QUEBEC
TELEPHONE 631-9827

DISTRICT OFFICES

14 HAAS ROAD
REXDALE, TORONTO, ONT.
TEL. 244-6476

1425 WEST PENDER ST.
VANCOUVER 5, B.C.
TEL. MU. 1-8926

Rexdale, Ontario,
April 22nd, 1965.

Department of Highways, Ontario,
Downsview, Ontario.

Attention: Mr. A. G. Stermac, P. Eng.,
Principal Foundation Engineer.

Re: Soil Conditions and Foundations
Proposed Kinniwabi River Crossings (Centre & East)
Township 26, District 18, Ontario

Dear Sirs:

This letter will confirm the discussions between your Mr. Kwan Y. Lo and the writer of April 21st, 1965 concerning the net allowable bearing values given in our reports on the above structures.

The basis for our choice of allowable bearing value for the above structures was given in our report on the Kinniwabi River West Crossing and since all the three structures are located in the same general area the basis for our choice applies to each structure.

As discussed in our report on the West Crossing, previous work by us in the general area and observations of the behaviour of buildings founded on spread footings within a geologically similar deposit locally, indicate two significant factors. Firstly, a similar pattern of "N" values and a relatively low resistance to the dynamic penetration tests at depth after initial high resistances is common. Secondly, it is known from our experience that heavy industrial structures founded on spread footings have performed satisfactorily. It is considered therefore, that the allowable bearing value of 2 tons per square foot as given in our reports, may be used in the design of spread footings for the above structures provided the foundation requirements specified in the reports are satisfied.

We trust that this letter is sufficient for your present purposes. Should you have any questions, however, or if we can be of assistance otherwise, please do not hesitate to call us.

Yours very truly,

GEOCON LTD

D.B.Oates

DBO/reb

D. B. Oates, P. Eng.,
District Soils Engineer.

cc: Foundations Office (Rm. 110)

Hwy. 401 & Keele St.,
Downsview, Ontario.

March 3, 1965

Materials and Testing Division

Geocon, Limited,
16 Mail Road,
Lexdale, Ontario.

Attention: M. E. Vale

Re: W.P. 82-62, Hwy. 101, Kinniwabi River, West Crossing,
✓ W.P. 83-62, Hwy. 101, Kinniwabi River, Centre Crossing,
W.P. 84-62, Hwy. 101, Kinniwabi River, East Crossing.
--- District 6B, Sault Ste. Marie, Ont. ---

Dear Sir:

Please consider this your authority to carry out foundation investigations at the above sites. Plans and profiles were provided to your representative on March 1, 1965.

It is understood that a qualified Soils Engineer will be in charge of the field work at all times.

Eleven copies of the completed foundation report, with one additional copy of each subsoil profile, should be submitted to the Foundation Section prior to May 1, 1965. Previous requirements as to preliminary borehole information and laboratory testing program, should be followed.

Because the drawings accompanying the foundation reports, showing the location of borings, the inferred subsoil conditions, etc., are to become contract drawings, you are requested to prepare them in accordance with the D.H.O. standards. To enable you to do this, we are supplying you with sample drawings with all the necessary explanations, together with linen sheets for your drawings. You are also requested to provide the D.H.O. with Dronefilm copies of the drawings.

cont'd. /2 ...

Charges for the work performed will be in accordance with your Schedule of Rates, dated March 4, 1960, and invoices to be addressed to the attention of the undersigned.

We are attaching Purchase Orders J 34776, J 34777, and J 34778, covering the purchase of any new material required for these projects, in order that you may use these as a basis for exemption from the Federal Tax for such purchases. The Exemption Certificate is printed thereon.

Yours very truly,

A. Rutka,
Materials & Testing Engineer

KAR/MEF
Attach.

cc: Messrs. A. McCombie
F. De Visser
H. McArthur
A. A. Ward
E. R. Saint
H. Konings
N. D. Smith (2)
Foundations Office
Gen. Files (2)

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. A. Stermac,
Principal Foundation Engineer,
Room 107, Lab. Bldg.

From: Bridge Division,
Downsview, Ontario.

Date: March 1, 1965.

Our File Ref.

In reply to

Subject: W.P. 82-62 Kinniwabi R. West Crossing,
W.P. 83-62 Kinniwabi R. Centre Crossing,
W.P. 84-62 Kinniwabi R. East Crossing,
Hwy #101, District # 18.

This will confirm our verbal request for foundation investigations at the above crossings.

You have recieved two prints of the following drawings:-

W.P. 82-62	E.- 2840	- 1
	D - 5389	- 1
W.P. 83-62	E - 2839	- 1
	D - 5388	- 1
W.P. 84-62	E - 2838	- 1
	D - 5377	- 1

and one print of study plan F- 3561.

We would be pleased if you would let us have any information on the sites as soon as it becomes available, in order to conferm the proposed designs.

J.C. McAllister

JCM/ag
c.c. W.D. Smith
R. Fitzgibbon

J.C. McAllister,
for S. McCombie,
Bridge Planning Engineer.

LETTERS OF AUTHORITY, PLEASE.
COPIES GIVEN TO GEOCON WHO ARE
IN THE AREA MARCH 1st. 1965

AGS