

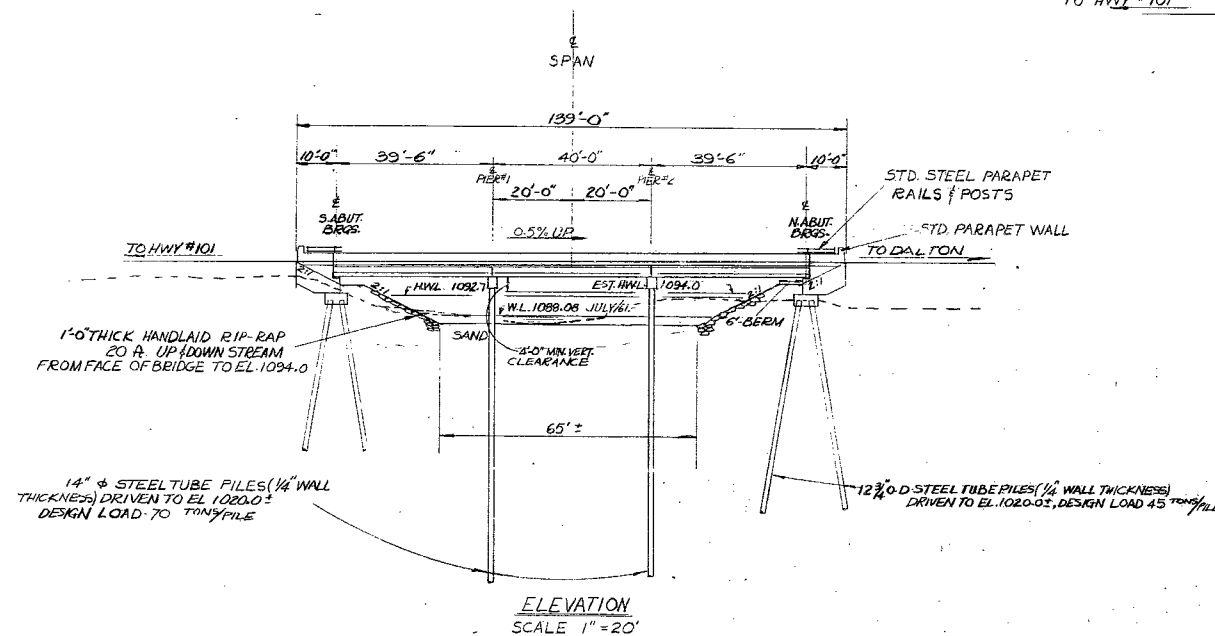
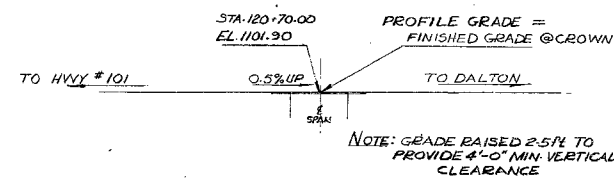
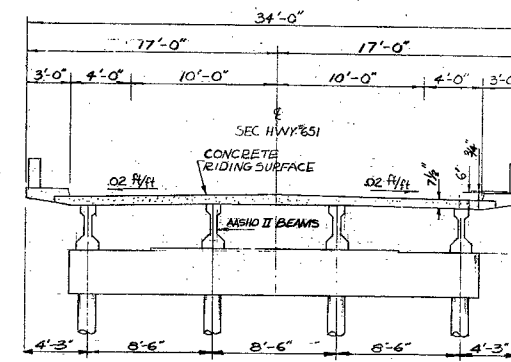
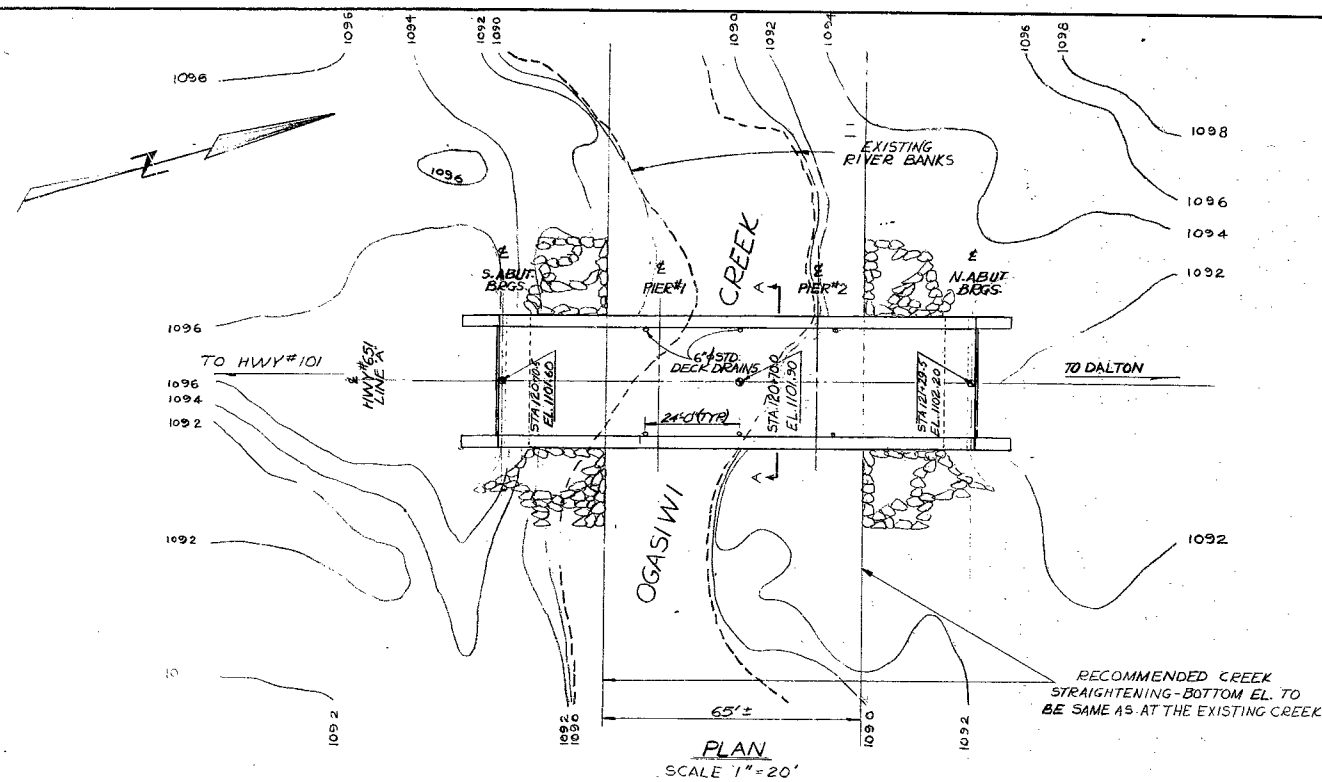
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W.P. #22-65

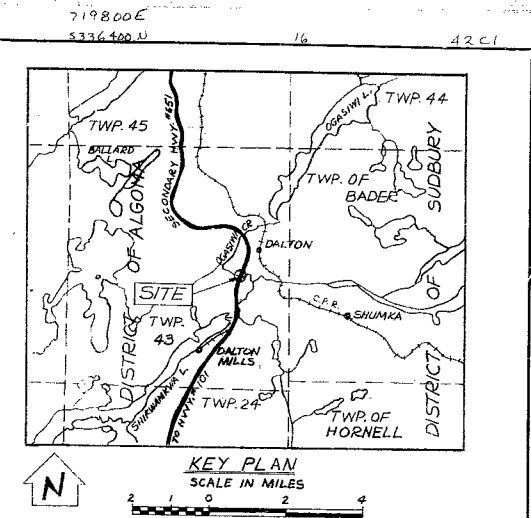
(W.P. #231-61)

Hwy. #651

OGASIWI CREEK

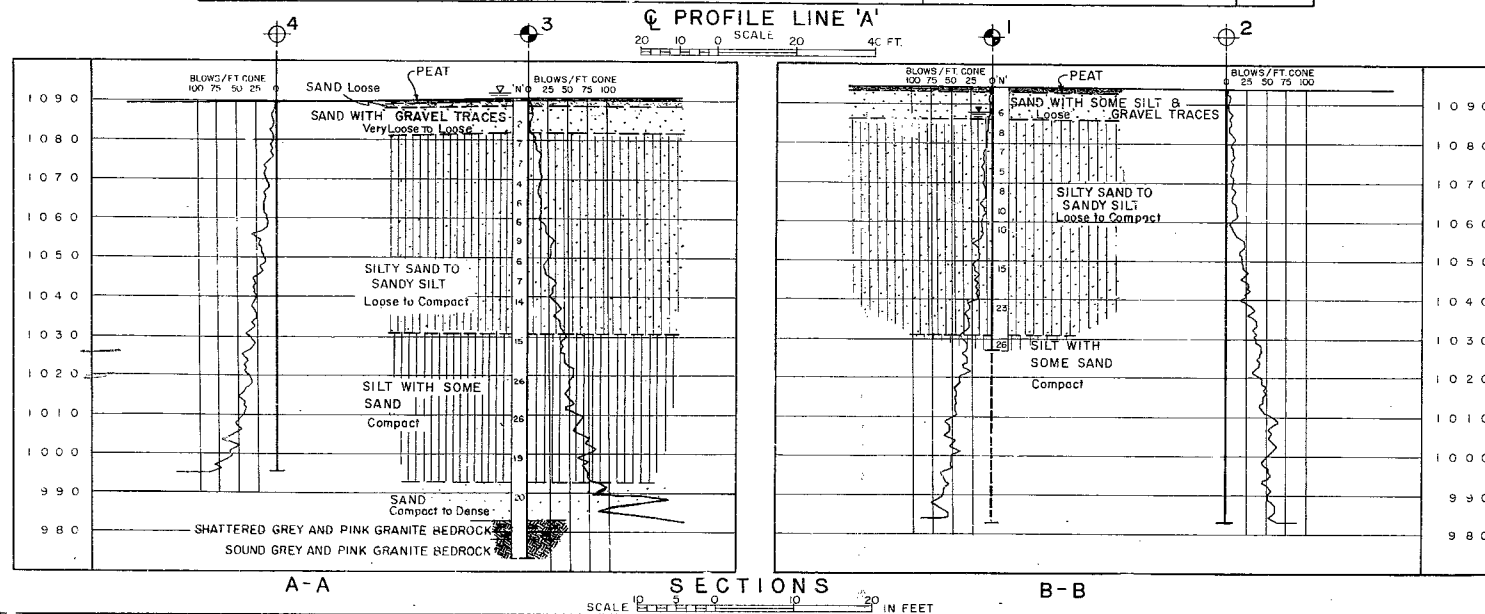
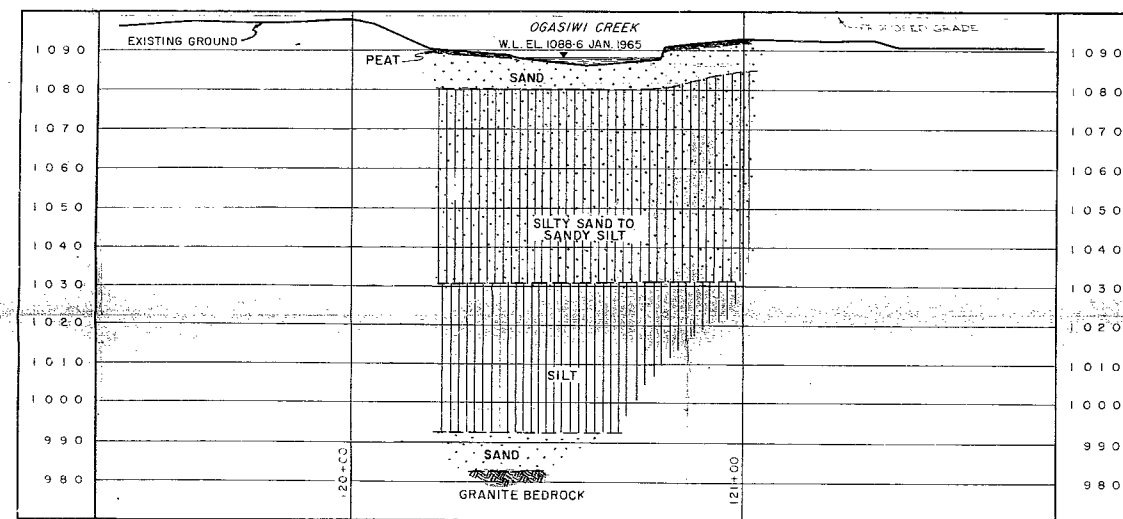
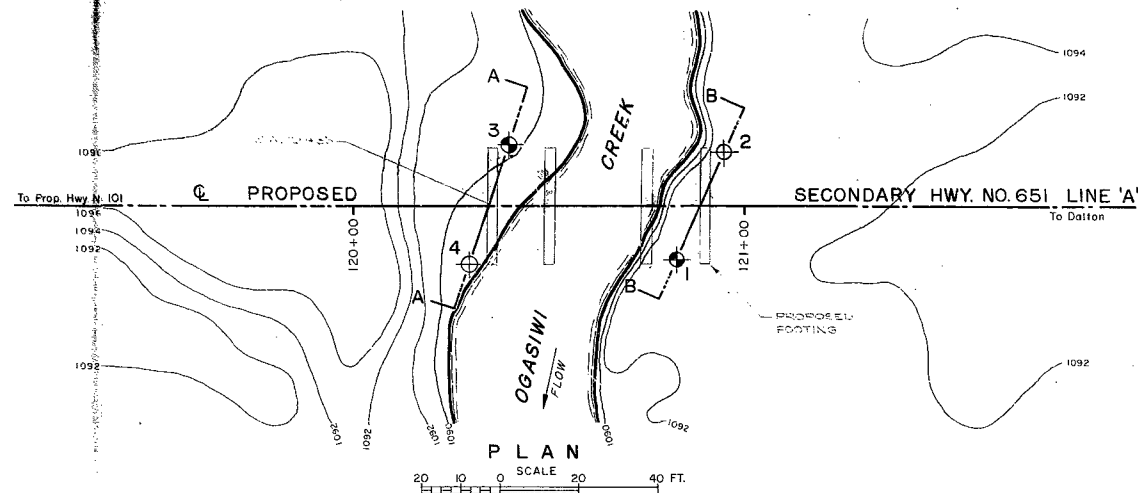


B.M. EL. 1093.81  
GEODETIC DATUM  
N.W. IN W. ROOT OF  
1/2 SPR. 62' RT. OF  
STA. 113+43

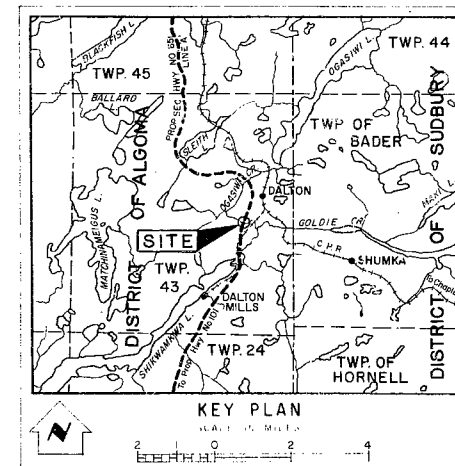


REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO BRIDGE DIVISION			
OGASIWI CREEK BRIDGE 16.4 MILES NORTH OF HWY #101			
KING'S HIGHWAY No. SEC. HWY #651		DIST. No. 18	
DISTRICT OF ALGOMA		TWP. 43 LOT. CON.	
PRELIMINARY			
APPROVED	BRIDGE ENGINEER	SITE No. 38C-20	W.P. No. 22-65
DESIGN A.P.	CHECK F.R.	CONTRACT No.	
DRAWING F.R.	CHECK A.P.	DRAWING No.	D-5673-P1
DATE SEPT 66	LOADING		



**REFERENCE BENCHMARK**  
B.M. ELEV. 1093.81 GEODETIC DATUM  
NAIL & WASHER IN W. FOOT OF 1:2 SPR.  
62' RT. OF STA. 119+49



LEGEND			
	Bore Hole		
	Cone Penetration Test		
	Bore Hole with Penetration Test		
	Water Level established at time of field investigation Jan. 1965		
	Artesian Head Jan. 1965		
NO.	ELEVATION	STATION	OFFSET
1	1094.1	120+83	14 RT.
2	1093.9	120+95	14 LT.
3	1090.3	120+40	16 LT.
4	1089.6	120+30	15 RT.

**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.

**GEOCON LTD.**  
DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS RESEARCH DIVISION

**OGASIWI CREEK**

KING'S HIGHWAY NO. SECONDARY HWY. NO. 651 DIST. NO. 18  
DISTRICT OF ALGOMA  
TWP. 43 LOT — CON. —

**BORE HOLE LOCATIONS & SOIL STRATA**

DATE: FEBRUARY 1, 1965	CHECKED: J.H.	APPROVED: J.H.	PROJECT NO.: 231-61	REPORT NO.: T 7710-1
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# GEOCON LTD

## HEAD OFFICE

420 MICHEL JASMIN, DORVAL, QUEBEC  
TELEPHONE 631-7827

Rexdale, Ontario,  
February 19th, 1965.

## DISTRICT OFFICES

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

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

Department of Highways, Ontario,  
Materials and Testing Division,  
Keele Street and Highway 401,  
Downsview, Ontario.

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

Re: Soil Conditions and Foundations  
Ogasiwi Creek Bridge  
Dalton, Ontario

Dear Sirs:

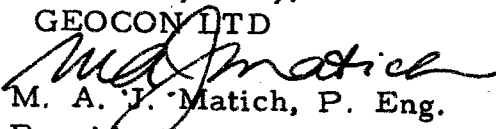
This letter accompanies our detailed report on the  
above work.

We find that the site is covered by 108 feet of granular  
overburden consisting of loose sand, loose to compact silty  
sand to sandy silt, compact silt with some sand and compact  
sand in this order. The actual soil and water conditions encountered  
are described in detail in the report.

Based on the findings of this investigation it is concluded  
that, from a Soil Mechanics standpoint, the most suitable founda-  
tion solution for the proposed bridge will be the use of piles. Data  
to guide pile selection and preliminary design are given in the re-  
port. Recommendations covering scour protection, and foundation  
and embankment construction are also given.

We believe that this report contains all the information  
required from this investigation. Kindly give us a call, however,  
should you have any questions or if we can be of further service  
otherwise.

Yours very truly,  
GEOCON LTD

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

MAJM/reb

DALTON

ONTARIO

Distribution:

- 10 copie - Department of Highways, Ontario  
Materials and Testing Division  
Downsview, Ontario

- 3 copies - Geocon Ltd  
Rexdale, Ontario

# GEOCON

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#### APPENDIX I

Procedure
Site and Geology
Soil and Water Conditions
Water Conditions
Office Reports on Soil Exploration

#### APPENDIX II

Figures - Laboratory Testing
------------------------------

DRAWING - In pocket at rear of report

## INTRODUCTION

Geocon Ltd has been retained by the Department of Highways, Ontario by letter dated December 15th, 1964, Work Permit 22-65 to carry out a foundation investigation for the proposed crossing at Ogasiwi Creek, approximately 1 mile south-west of Dalton, Ontario.

The purpose of the investigation was to determine the soil conditions at the site of the proposed crossing as required for the design of the proposed structure.

## SUMMARIZED SOIL CONDITIONS

A thin surficial peat layer at the site is underlain by an extensive deposit of granular soil which has a total observed thickness of 108 feet. This deposit consists of about 7 feet of very loose sand containing some silt and gravel traces, about 52 feet of loose to compact silty sand to sandy silt, 39 feet of compact silt with some sand and about 10 feet of compact medium to fine sand. The bedrock, which underlies this granular deposit is a grey to pink granite.

The water level at the site corresponds with the level of Ogasiwi Creek which was at elevation 1088.6 at the time of the investigation. Artesian pressure corresponding to elevation 1092 was encountered at depth within the overburden.

General

It is understood that it is proposed to construct a bridge over the Ogasiwi Creek at the site investigated. As presently planned the bridge will consist of a three span structure located on Line A, as shown on Department of Highways, Ontario Drawing No. E-4034-1. It is further understood that the proposed embankment grade will be at elevation 1099.

Foundations

The soil conditions at the site consist generally of very loose to loose sand which overlies a deposit of loose to compact silty sand to sandy silt then compact silt with some sand.

Because of the very loose to loose relative density of the surface strata and the fact that the soil is scour susceptible, the site is not considered practical for the use of spread footings for foundations of the proposed structure. The most suitable foundation solution appears, therefore, to be the use of piles.

A variety of pile types are considered to be practical in this instance such as non-displacement steel H piles deriving their support entirely from end-bearing on the bedrock surface, or displacement types which derive their support partly by side friction and partly by



Foundations, (contd)

point resistance within the granular overburden. Suitable displacement pile types would be treated timber piles, steel pipe piles driven closed end or precast or cast-in-place concrete piles. It is recommended that a pile load test be performed on a representative pile to verify the pile design working load. All pile caps subject to frost action should be provided with a minimum of 6 feet of protective earth cover.

In view of the considerable depth to bedrock it is probable that a displacement type pile would offer the most economical foundation solution for the bridge.

For the purpose of selecting allowable pile diameters and lengths, preliminary design computations have been carried out for straight shaft circular piles of different diameters using two approaches. (Meyerhof 1956. "Penetration Tests and Bearing Capacity of Cohesionless Soils." Journal of the Soil Mechanics and Foundation Division. A. S. C. E. No. SM. 1. - Mansur and Kaufman 1956. "Pile Tests, Low Sill Structure, Old River, La." Journal of the Soil Mechanics and Foundation Division. A. S. C. E. No. SM. 4.) The results incorporating a computed factor of safety of 2.5 are given on Figure 6, in Appendix II. The values indicated are for individual piles. In no case, however, should the working load of the pile exceed the safe load as determined from the

Foundations (contd)

structural properties of the pile material.

All of the overburden at the site is susceptible to scour and therefore, measures should be taken to protect bridge foundations from undermining due to this effect. This could be done by such means as provision of rip-rap protection, or carrying foundations a safe distance beyond maximum scour depth. The extent of scour in both lateral and vertical direction depends on the maximum flood level of the river, the hydraulics of the river channel in the vicinity of the bridge, the orientation of the piers relative to the direction of flow and the like. The hydraulic considerations involved are beyond the scope of this report. Published data (Terzaghi and Peck, "Soil Mechanics in Engineering Practice.") indicates that the depth of scour below the low water channel may be in the order of four times the greatest known rise of river level. Therefore, if no suitable rip-rap protection is provided to the piers, it should be assumed that scouring to at least the depth indicated may occur. The computed pile capacities as given above assume that adequate rip-rap will be provided.

Approach Embankments

It is recommended that surface organic material be stripped from beneath approach embankments. With a proposed grade of 1099 and removal of the surficial layer of peat and organic soil the maximum overall height of embankment will be about 11 feet. In view of the granular nature of the subsoil, it is considered that the stability of the approach embankments with side slopes of 1 vertical to 1.5 horizontal or flatter will be adequate. However, it is recommended that the ends of the embankment be protected, where necessary, by suitable rip-rap protection to prevent scour by river action.

It is recommended that the back fill to the abutments consist of well compaced free draining non-frost susceptible clean granular material. With this provision, a coefficient of lateral earth pressure of 0.5 is recommended for the case of the pile supported abutments.

Construction

Should excavations be required below water level at the pier or abutment locations, they will involve excavation within the loose sand stratum and therefore, some means will therefore be required to control water inflow. For this purpose, a gravity well point system could be

Construction (contd)

used or alternatively the excavation could be sheeted and dewatered by pumping from filter equipped sumps maintained at a sufficient depth below excavation level. Where excavations are required within the creek, provision of a low berm on the creek side of the excavation would be required if a well point system is used. Sheet piling if used, should be carried below excavation level a distance at least equal to the maximum head differential likely to be encountered.

CONCLUSIONS AND RECOMMENDATIONS

- 1) The overburden consist generally of 7 feet of very loose sand overlying 52 feet of loose to compact silty sand to sandy silt, 39 feet of compact silt with some sand, 10 feet of compact sand then bedrock.
- 2) The water level at the site was found to correspond with the creek level, at elevation 1088.6. Artesian pressure, corresponding to elevation 1092 was encountered at depth within the overburden, as discussed.
- 3) The low relative density and scour susceptibility of the surficial soil strata at the site preclude the practical use of shallow spread foundations. The most suitable foundation solution, therefore,

is considered to be the use of friction piles as discussed in the report. Computed pile loads are given as a guide to preliminary design, and it is recommended that the design load be verified by loading a pile of the type selected.

4) Construction of pile caps might require excavation below the ground water level, as discussed. Recommendations are given on possible measures to handle the water inflow.

PERSONNEL

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

Yours very truly,

GEOCON LTD

*D B Oates.*

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



DBO/da

APPENDIX I

PROCEDURE

SITE AND GEOLOGY

SOIL AND WATER CONDITIONS

WATER CONDITIONS

OFFICE REPORTS ON SOIL EXPLORATION

## PROCEDURE

The field work was carried out between January 9th, 1965 and January 23rd, 1965. A total of 2 boreholes in BX size each with an accompanying dynamic uncased cone penetration test, and two additional dynamic uncased cone penetration tests were put down. For the work, a skid mounted diamond drill rig was used.

The soil strata were sampled at intervals not exceeding 5 feet. Two inch split spoon samples, adapted with a foot valve to facilitate recovery, were taken in the overburden. The bedrock was proved in borehole 3 by rock core drilling in AXT size for about 10 feet.

Detailed logs of the 2 boreholes and 2 dynamic uncased cone penetration tests are presented on the Office Reports on Soil Exploration in this Appendix. The locations of the boreholes and dynamic cone penetration tests, together with the inferred stratigraphy are shown on Drawing T7710-1, at the rear of this report.

The laboratory testing of selected soil samples was carried out in the Soil Mechanics Laboratory of Geocon Ltd in Toronto. The results are plotted on the Office Reports on Soil Exploration in

Appendix I and on the Figures in Appendix II. The soil samples remaining after testing will be stored until March 1st, 1966 at which time you will be contacted for instructions regarding their disposal.

All elevations given in this report are referred to Geodetic datum. The bench mark referred to is a nail and washer in the west root of a 12 foot diameter Spruce Tree, 62 feet right of Station 119+49, along proposed Secondary Highway No. 651, "Line A". The Geodetic elevation of the bench mark is 1093.81. The location of this bench mark is shown on the Department of Highways, Ontario Drawing No. E-4034-1.

#### SITE AND GEOLOGY

The proposed D.H.O. bridge site is to be located on the proposed Secondary Highway No. 651, Line "A" over the Ogasiwi Creek, approximately 1 mile south west of Dalton, Ontario. The site is located in Township No. 43, District of Algoma, Ontario. At the time of the investigation the Ogasiwi Creek, which flows in an easterly direction, was about 35 feet wide and about 3 feet deep at its deepest point; indications are that at high water level the depth of water is about 7 feet. Ground level at the site varies between elevations 1090 and 1097, with



the maximum variation occurring close to the edge of the creek. The site area is covered by dark brown peat and other related organic soils to a maximum observed depth of about 2 feet.

From available geological information and inspection of the area, it is known that the overburden cover is composed of sands and silts of post glacial fluvial origin. The bedrock in the area is an igneous intrusive grey to pink granite of the Archean Era of Precambrian Period.

SOIL AND WATER CONDITIONS

The principal soil strata encountered in the boreholes were as follows:

Dark Brown Peat

The surficial stratum across the site is a dark brown highly compressible peat containing roots and other decayed organics. The ground surface elevation across the site, which corresponds to the surface elevation of this stratum, varies from elevation 1090 to 1097. The thickness of the peat is about 1 foot. At borehole 3 the peat is underlain by about 1 foot of organic silty sand and gravel. Because of the highly organic nature of the silty sand it is grouped with the peat

Dark Brown Peat (contd)

material on the stratigraphy shown on Drawing T7710-1. The thickness of the organic material can therefore be considered to vary from 1 to 2 feet across the site.

The dynamic uncased cone penetration tests put down, gave results varying from 1 to 3 blows per foot while penetrating through the organic material, indicating that the material is in a very loose condition.

Very loose to Loose Brown to Grey Sand

Directly underlying the organic material at both boreholes is a stratum of brown to grey sand which contains in places some silt and gravel traces. The surface elevation of the stratum was 1088 at borehole 3 and 1093 at borehole 1 with corresponding thicknesses of 7 feet in both boreholes. The colour change was noted at a depth of about 4 feet in both boreholes. The material consists mainly of medium to coarse sand with some silt and trace of subrounded gravel; both the silt and gravel content is present only at localized areas within the stratum.

Very Loose to Loose Brown to Grey Sand (contd)

Mechanical analysis tests were carried out on three typical samples from this stratum and the results are shown on Figure I of Appendix II. The grain size curves indicate that the samples contained up to 16 percent silt sizes, 84 to 96 percent sand sizes and up to 5 percent gravel sizes. The sand sizes were mainly in the medium to coarse range.

Two standard penetration tests were carried out in this stratum and gave "N" values of 3 and 6 blows per foot. From these results it is estimated that the relative density of the stratum ranges from very loose to loose.

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

Wet unit weight,	115 pounds per cubic foot.
Angle of shearing resistance,	32 degrees.

Loose to Compact Grey Silty Sand to Sandy Silt.

Directly underlying the sand stratum in the boreholes is a stratum which grades from a sand with some silt sizes to a sandy silt. The surface elevation of the stratum was 1081 at borehole 3

Loose to Compact Grey Silty Sand to Sandy Silt (contd)

and 1086 at borehole I. The thickness of this stratum ranged from 50 to 54 feet in the boreholes.

Mechanical analyses were carried out on five typical samples from this stratum and the results are shown on Figures 2 and 3 in Appendix II. The results indicate that the gradation of the stratum ranges from a fine sand with about 20 percent silt sizes to fine sand and silt sizes in approximately equal proportions.

Standard penetration tests carried out in the stratum gave "N" values ranging from 4 to 23 blows per foot with an average value of 9 blows per foot. The relative density is estimated to range from loose to 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 Grey Silt with Some Sand

Underlying the silty sand to sandy silt stratum is a stratum of grey silt which contains some sand sizes. This stratum was only

Compact Grey Silt with Some Sand (contd)

penetrated in borehole 3 where the thickness was about 39 feet.

Mechanical analyses were carried out on four typical samples from this stratum and the results plotted on Figure 4 in Appendix II. The results indicate that the stratum contains 20 percent fine sand sizes and 80 percent silt sizes.

Standard penetration tests carried out in the stratum gave "N" values ranging from 15 to 26 blows per foot with an average value of 22 blows per foot. The relative density is therefore generally compact.

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

Wet unit weight,	125 pounds per cubic foot
Angle of bearing resistance,	35 degrees

Compact to Dense Grey Sand

Directly underlying the silt stratum in borehole 3, where the silt stratum was penetrated, is a stratum of grey medium fine sand. The surface elevation of the stratum was 992 and the thickness was found to be 10 feet at this borehole.

Compact to Dense Grey Sand (contd)

One mechanical analysis test was carried out on a typical sample from the stratum and the result is shown on Figure 5 of Appendix II. The grain size curve indicates that the material consisted of 35 percent medium sand sizes, and 65 percent fine sand sizes.

One standard penetration test carried out in the stratum gave an "N" value of 20 blows per foot indicating a compact relative density.

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

Wet unit weight,	130 pounds per cubic foot.
Angle of bearing resistance,	40 degrees.

Grey to Pink Granite Bedrock

Directly underlying the sand stratum in borehole 3 is a grey to pink granite bedrock. The bedrock encountered in this borehole was proved by diamond core drilling in AXT size for a depth of about 10 feet. The results of borehole 3 and the remaining

Grey to Pink Granite Bedrock (contd)

dynamic penetration tests infer that the surface of the bedrock lies between elevations 982 and 995, but generally at elevation 982 within the area of the proposed bridge. The core recovered exhibited coarse grain sizes, with the majority of the component mineral being composed of the light coloured acidic minerals quartz and feldspar combined with the dark coloured basic hornblendes. The top four feet of the bedrock surface is in a shattered and weathered condition, as evidenced by the relatively poor core recovery. Below this depth the core recovery indicates that the bedrock is in a sound condition.

The bedrock is a massive and coarse grained igneous intrusive variety, with an estimated hardness of about 7 on the Moh's hardness scale. It is believed that the granitic rock type is part of the batholithic formation noted in the geology of the area.

WATER CONDITIONS

Water level observations were taken in the cased holes during the investigation. An Artesian water level was encountered in borehole 3 at a depth of 70 feet below ground level. The artesian water level was about 1 foot above ground level at this time. On

extending the casing to bedrock, the artesian water level increased to about 2 feet above ground level; that is, at elevation 1092.

With the BX casing at a depth of 65 feet in borehole I, the water level was observed at a depth of about 5 feet below ground level. This depth corresponded with the level in the creek. At the time of the investigation the creek level was at elevation 1088.6. Previous recordings in the area inferred that high water level occurs at about elevation 1092.7.



## 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:

<u>Consistency</u>	<u>U-Strength Tons/sq. ft.</u>	<u>Relative Density</u>	<u>Standard Penetration Resistance. Blows/ft.</u>
Very soft	0.03 to 0.25	Very loose	0 to 4
Soft	0.25 to 0.5	Loose	4 to 10
Firm	0.5 to 1.0	Compact	10 to 30
Stiff	1.0 to 2.0	Dense	30 to 50
Very stiff	2.0 to 4.0	Very dense	over 50
Hard	over 4.0		

### 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

## OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T 7710 BORING # 1 DATUM GEODETIC CASING BK  
 BORING DATE JAN. 11, 1965 REPORT DATE JAN. 26, 1965 COMPILED BY J.H. CHECKED BY P.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

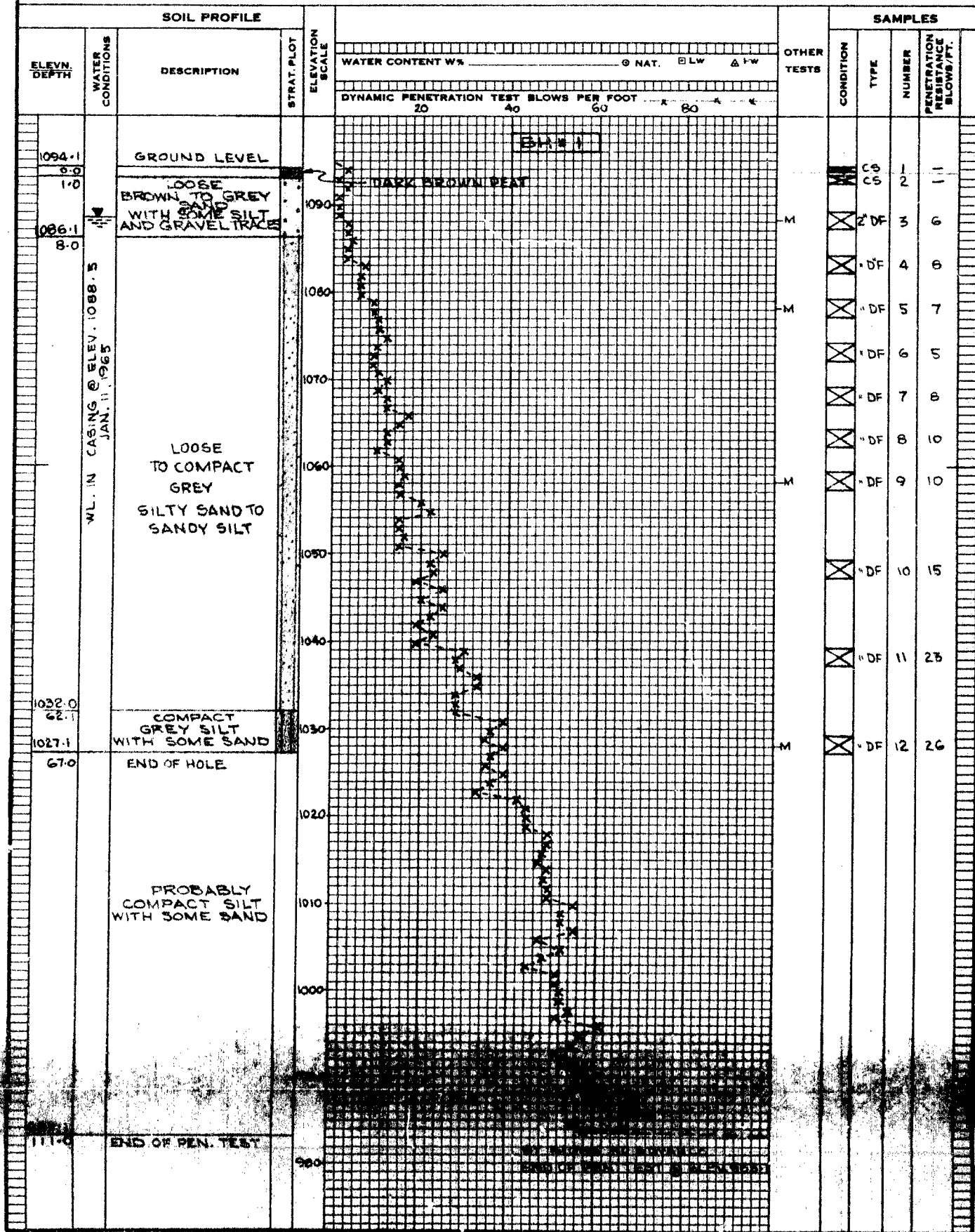
## SAMPLE TYPES

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  
 7 - WET UNIT WEIGHT  
 K - PERMEABILITY  
 C - CONSOLIDATION  
 WL - WATER LEVEL IN CASING  
 WT - WATER TABLE IN SOIL



## GEOCON

## OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T 7710 PEN TEST #2 DATUM GEODETIC CASING -  
 BORING DATE JAN. 12, 1965 REPORT DATE JAN. 26, 1965 COMPILED BY J.H. 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

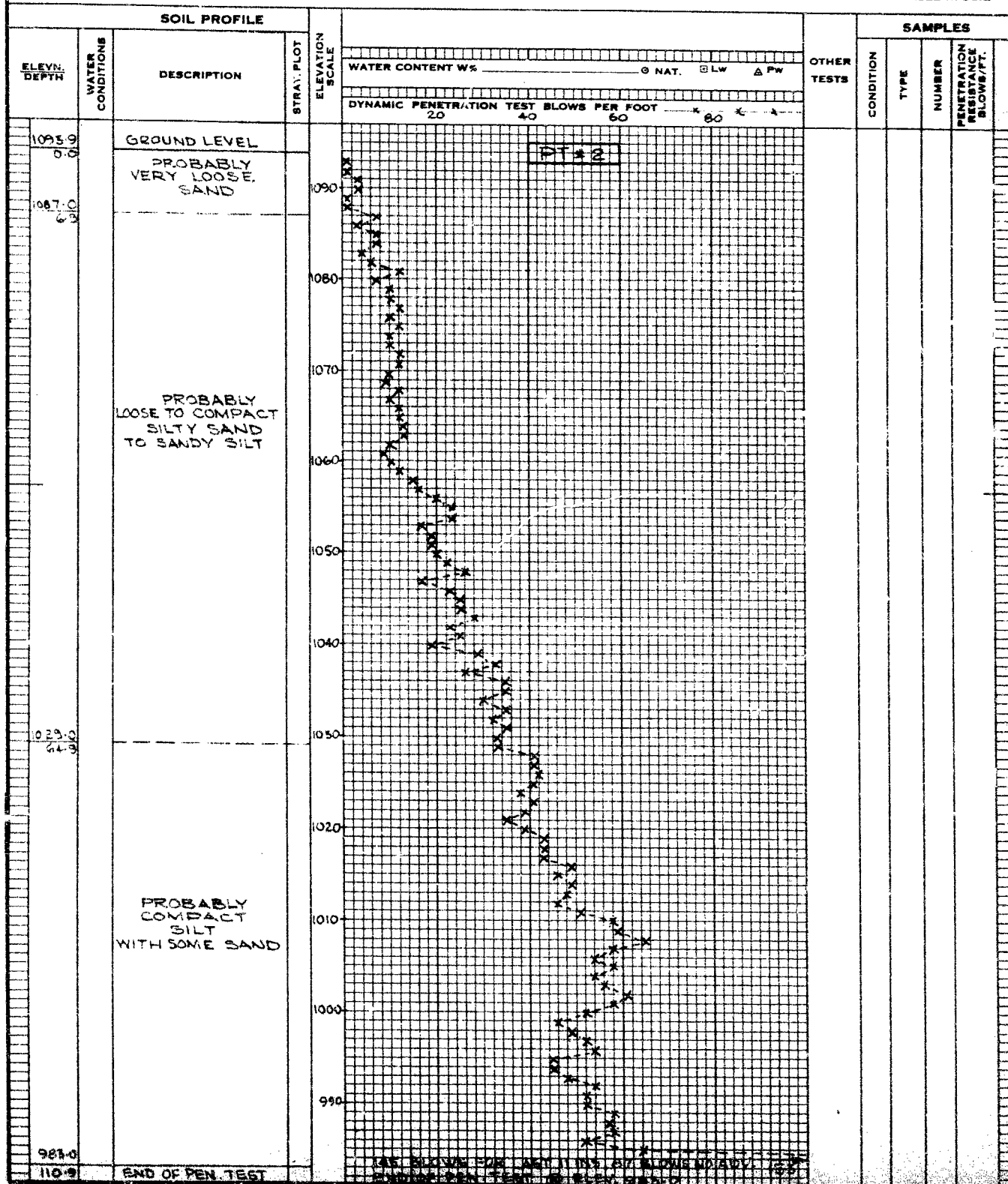
☒ DISTURBED  
☐ FAIR  
☐ GOOD  
☐ LOST

## SAMPLE TYPES

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

## 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



CONTRACT I 7710 BORING # 3 DATUM GEODETIC CASING BX  
BORING DATE 10-22-1965 REPORT DATE 10-26-1965 COMPILED BY J.H. CHECKED BY B.T.D.  
SAMPLER HAMMER WT. 140 LBS. DROP 80 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. (LBS. ENERGY)

### SAMPLE CONDITION

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

V - IN-SITU VANE TEST  
M - MECHANICAL ANALYSIS  
U - UNCONFINED COMPRESSION  
QC - TRIAXIAL CONSOLIDATED UNDRAINED  
Q - TRIAXIAL UNDRAINED  
S - TRIAXIAL DRAINED

Y - WET UNIT WEIGHT  
K - PERMEABILITY  
C - CONSOLIDATION  
INSD  
WL - WATER LEVEL IN CASING  
WT - WATER TABLE IN SOIL

SOIL PROFILE						SAMPLES			
ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLT.	ELEVATION SCALE	OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
1090.3		GROUND LEVEL							
0.0									
2.0		VERY LOOSE TO LOOSE BROWN TO GREY SAND WITH GRAVEL TRACES							
1081.6									
9.0									
		LOOSE TO COMPACT GREY SILTY SAND TO SANDY SILT							
		COMPACT GREY SILT WITH SOME SAND							
		COMPACT TO VERY DENSE GREY CLAY							
		END OF HOLE							



## GEOCON

## OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T 7710        PEN. TEST # 4 DATUM GEODETIC CASING         
 BORING DATE JAN. 22, 23, 1965 REPORT DATE JAN. 26, 1965 COMPILED BY J. H. CHECKED BY B. F. D.  
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

## SAMPLE CONDITION



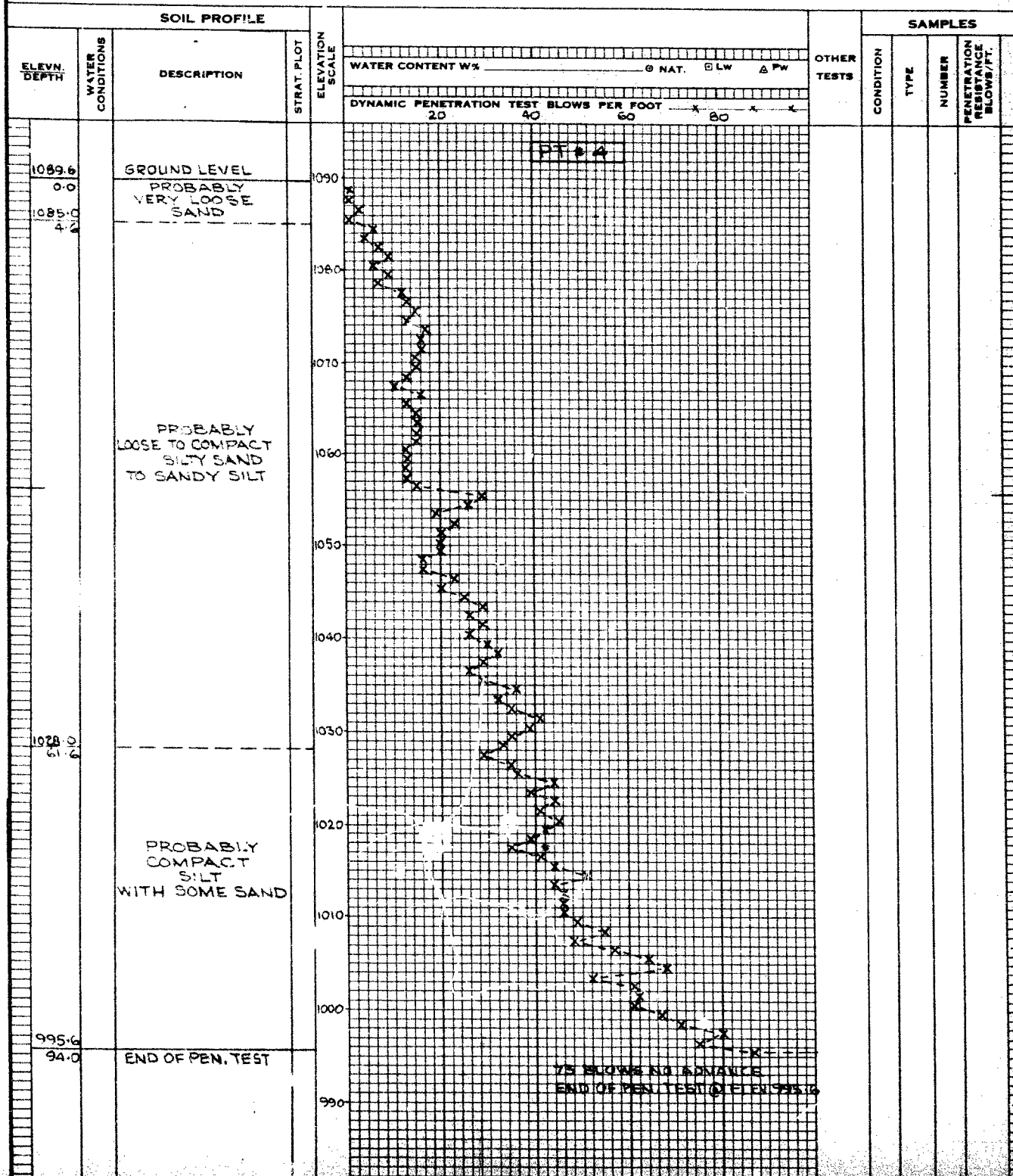
A.S. - AUGER SAMPLE  
 S.T. - SLOTTED TUBE  
 W.S. - WASHED SAMPLE  
 D.O. - DRIVE-OPEN  
 D.F. - DRIVE-FOOT VALVE  
 C.S. - CHUNK SAMPLE

## SAMPLE TYPES

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  
 GC - TRIAXIAL CONSOLIDATED UNDRAINED  
 G - TRIAXIAL UNDRAINED  
 S - TRIAXIAL DRAINED  
 γ - WET UNIT WEIGHT  
 K - PERMEABILITY  
 C - CONSOLIDATION  
 WL - WATER LEVEL IN CASING  
 WT - WATER TABLE IN SOIL



APPENDIX II

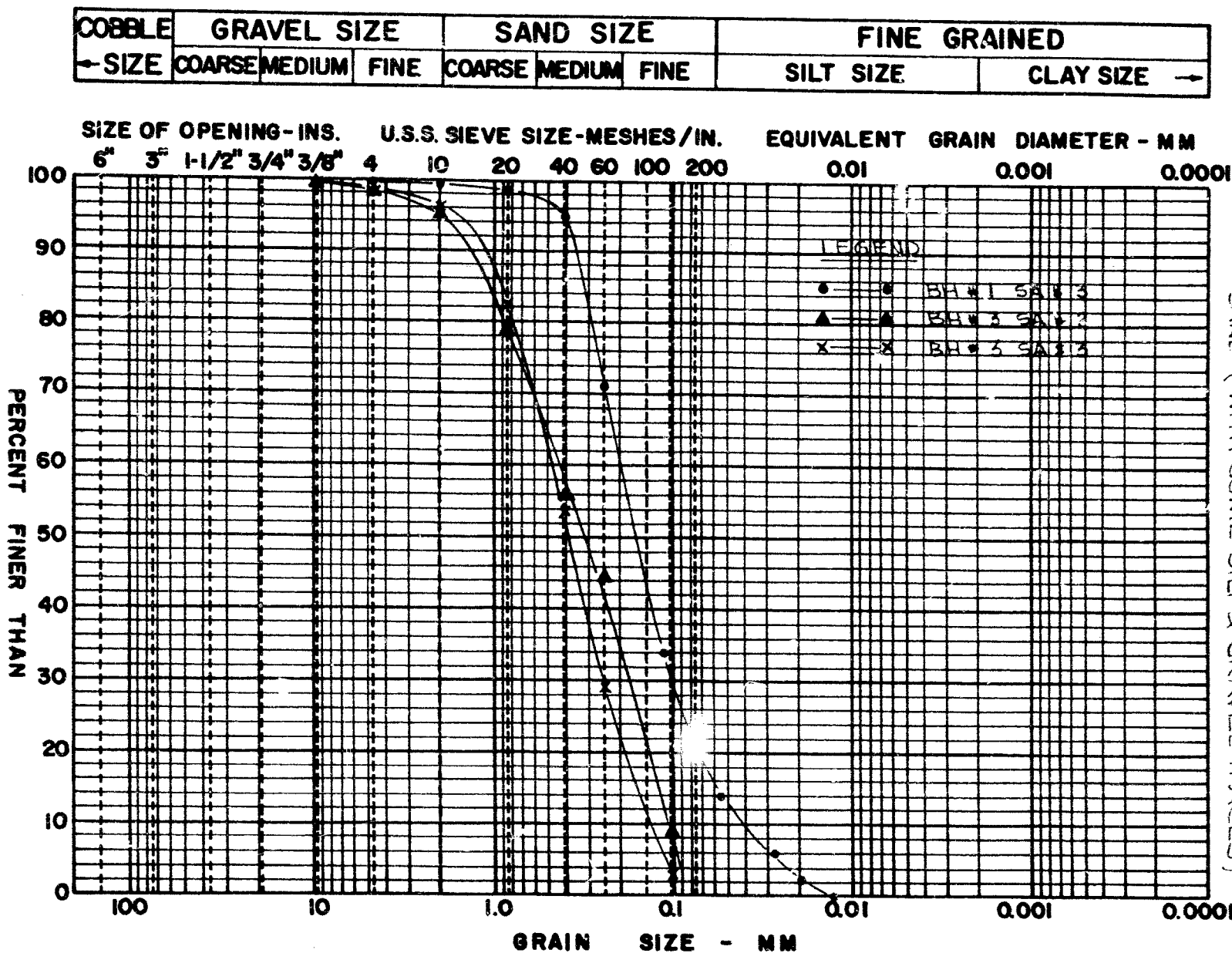
FIGURES - LABORATORY TESTING



# GRAIN SIZE DISTRIBUTION

APPENDIX 11  
FIGURE 1  
PROJECT T 7710

SAND (WITH SOME SILT & GRAVEL TRACES)



M.I.T. GRAIN SIZE SCALE

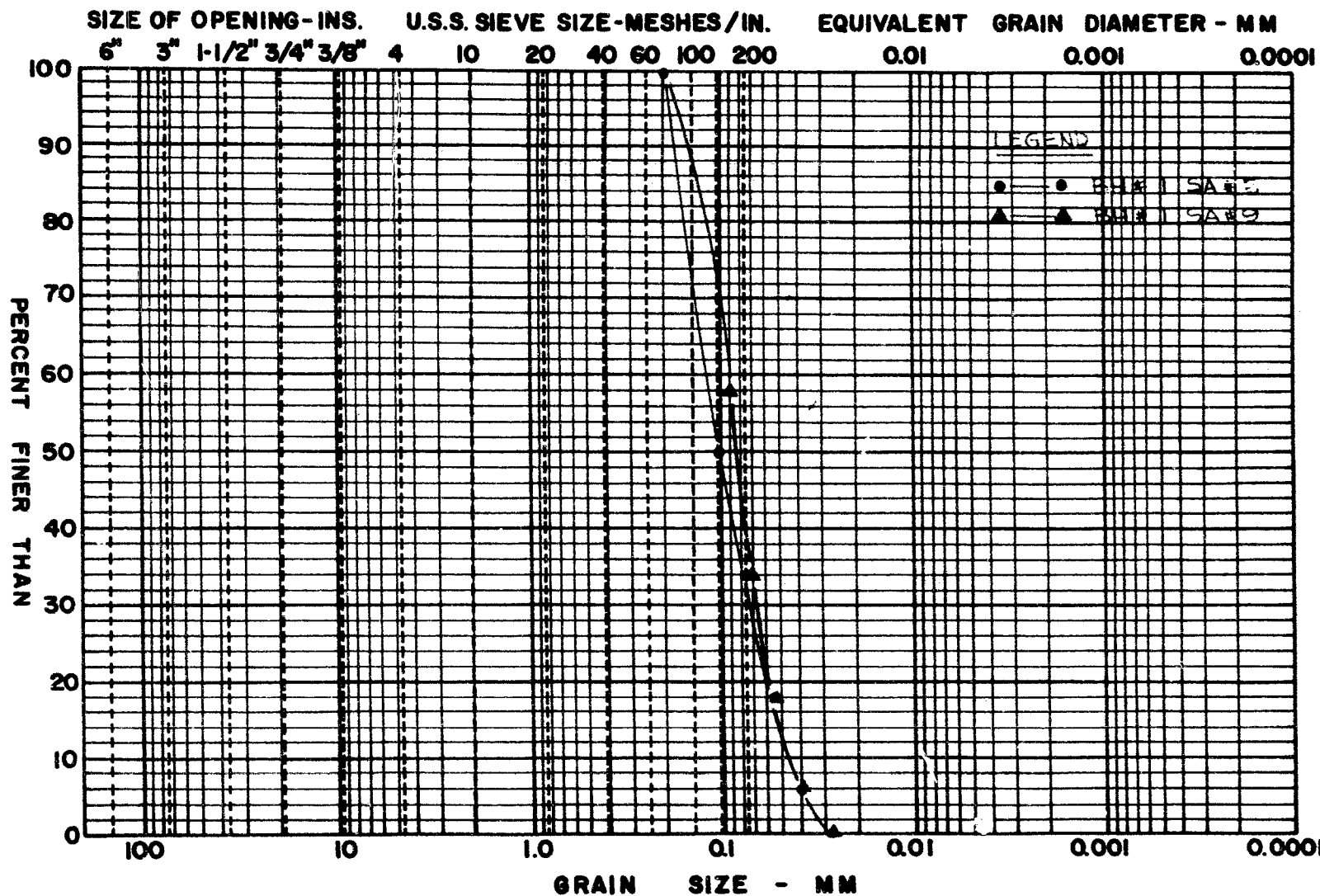
GEOCON



# GRAIN SIZE DISTRIBUTION

APPENDIX II  
FIGURE 2  
PROJECT T 7710

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

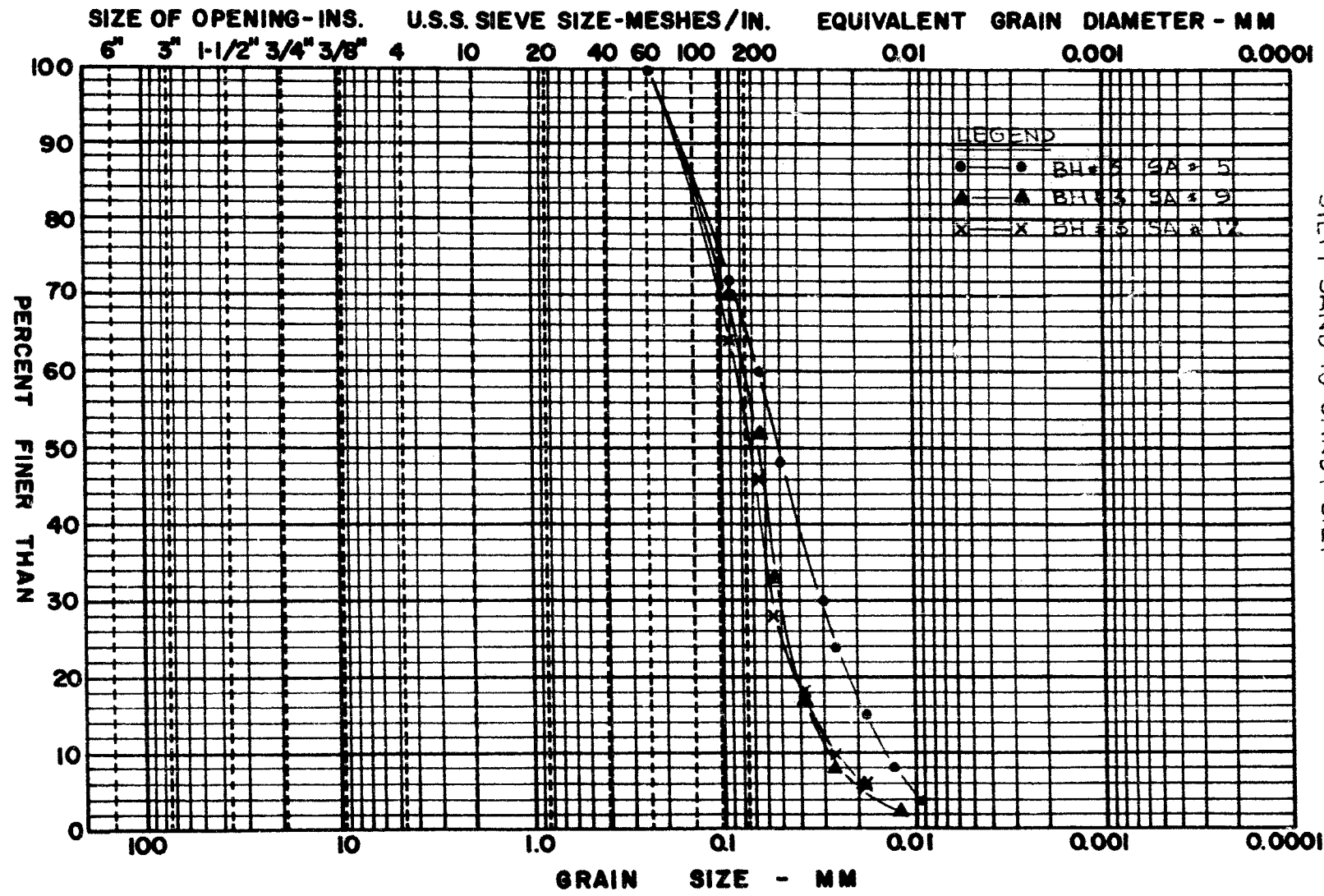


# GRAIN SIZE DISTRIBUTION

APPENDIX 11  
FIGURE 5  
PROJECT T7710

SILTY SAND TO SANDY SILT

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



GEOCON

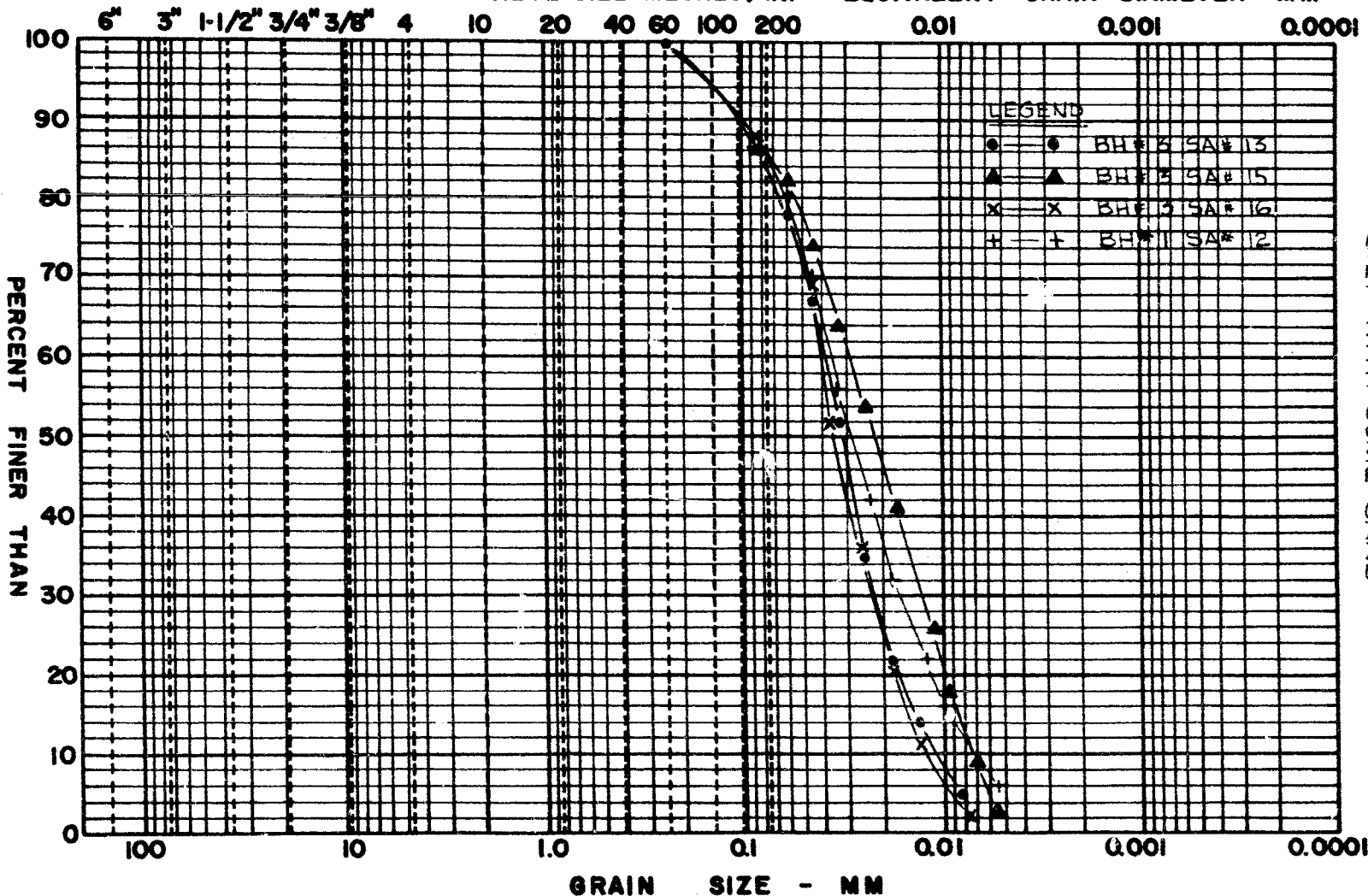
M.I.T. GRAIN SIZE SCALE

# GRAIN SIZE DISTRIBUTION

APPENDIX 11  
FIGURE 4  
PROJECT T 7710

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

SIZE OF OPENING - INS.    U.S.S. SIEVE SIZE - MESHES / IN.    EQUIVALENT GRAIN DIAMETER - MM



M.I.T. GRAIN SIZE SCALE

GEOCON

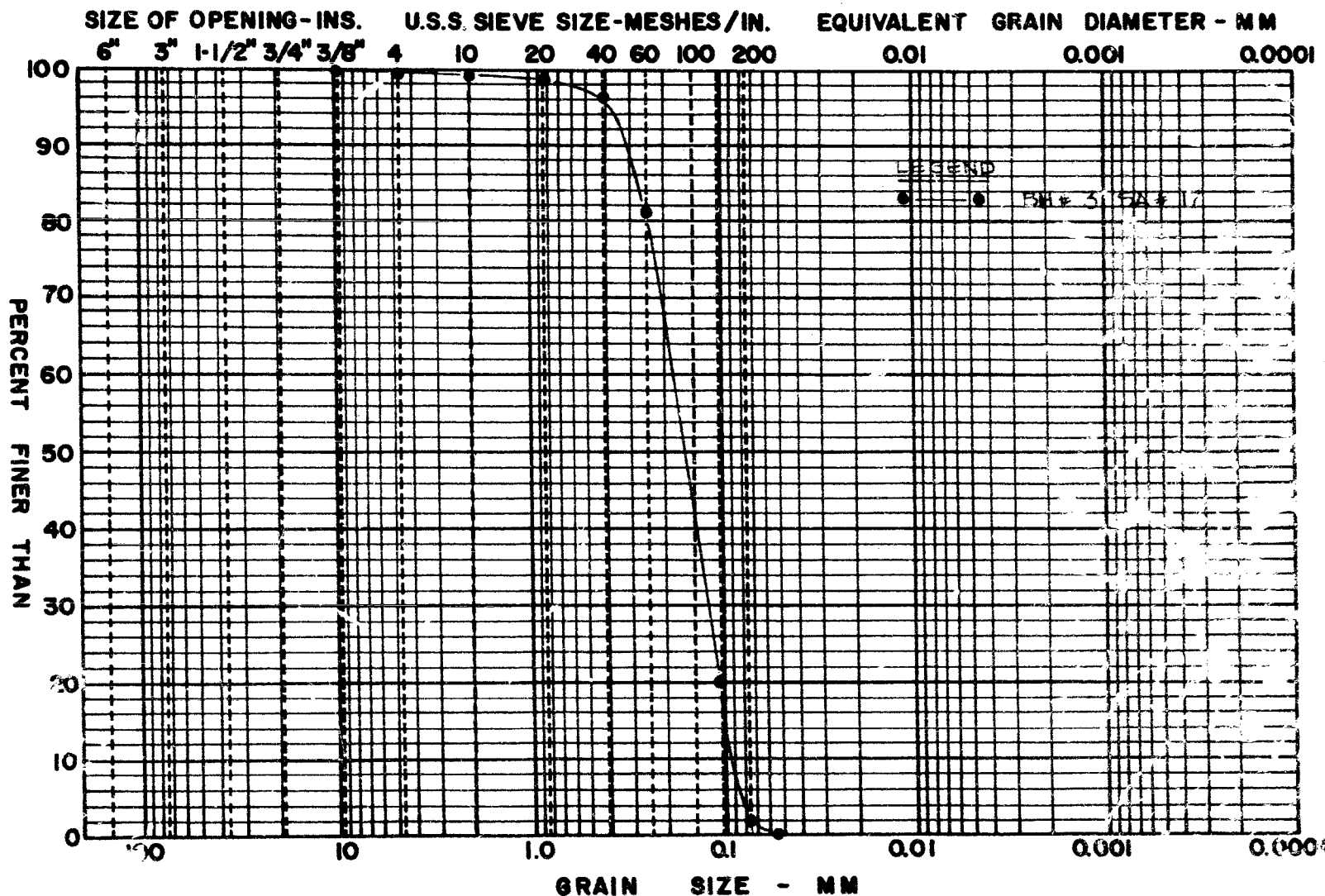
# GRAIN SIZE DISTRIBUTION

APPENDIX II  
FIGURE 5

PROJECT 17710

SAND

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

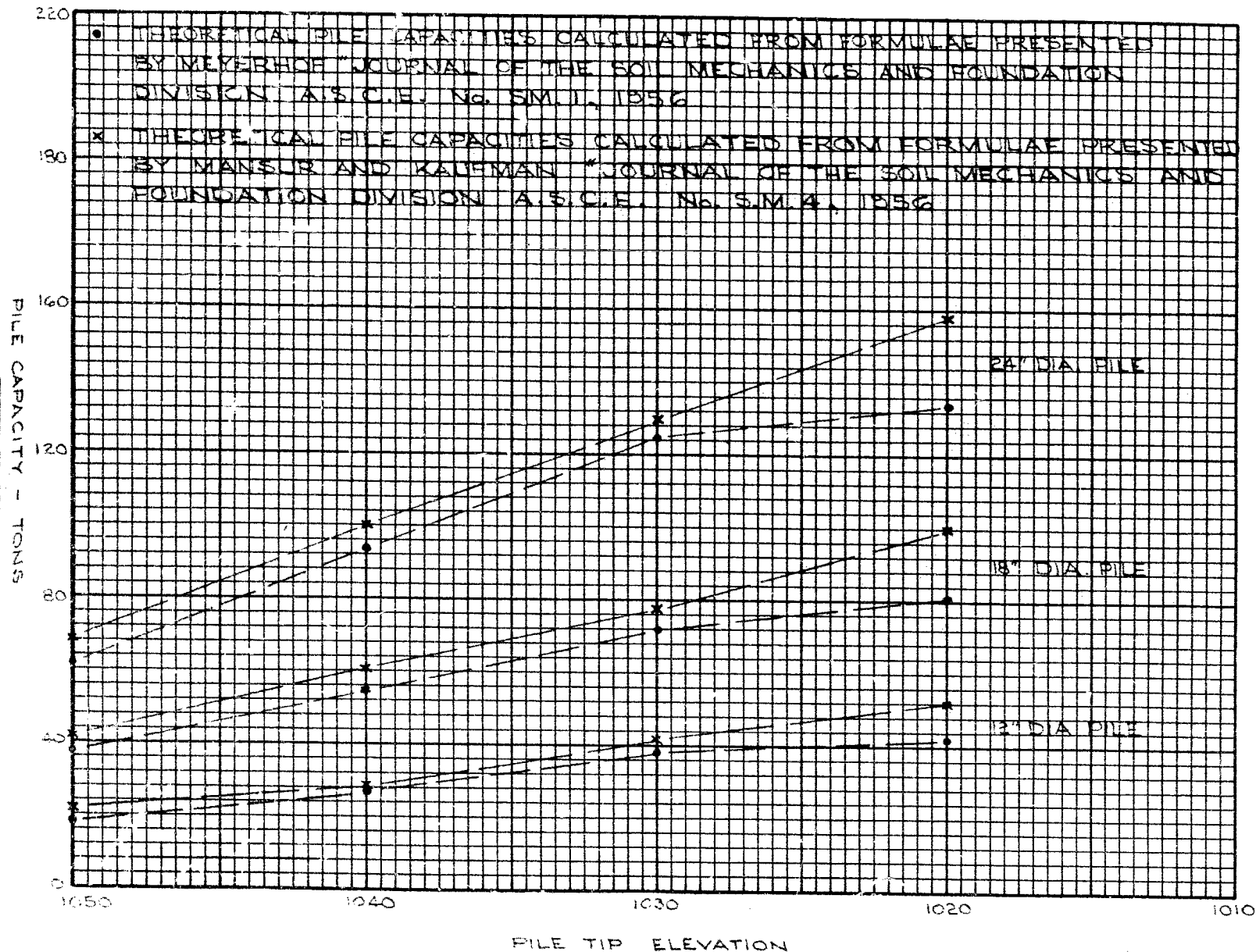


M.I.T. GRAIN SIZE SCALE

GEOCON

# PILE CAPACITY vs. PILE TIP ELEVATION

APPENDIX II  
FIGURE 6  
PROJECT T7710



DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

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

FROM: 208 Simpson Avenue,  
FORT WILLIAM, Ontario.

DATE: November 9, 1964.

OUR FILE REF.

IN REPLY TO

SUBJECT: W.P. 251-61    NOW W.P. 22-65  
Site 38C-19  
Ogasiwi Creek  
16.5 Mi. North of Hwy. 101  
Sec. Rd. 651 - Dist. 18

Attached please find a marked up E-plan for the subject structure, showing the tentative footing lay-out.

Would you please have a foundation investigation carried out.

The site can be reached from Dalton following a bushroad which comes to within 250 yards from the site.

FDeV/sp

*S.M. Combie*  
for F. DeVisser,  
Regional Bridge Location Engineer.

cc. N. D. Smith  
R. Fitzgibbon  
P. Wong  
S. McCombie

*Spent 10/11/64*

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

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

From: 208 Simpson Street,  
Fort William, Ontario.

Date: November 16, 1964.

Our File Ref.

In Reply To

Subject: W.P. 22-65 Site 380-19  
Oxasiwi Creek  
Secondary Road 651  
District 18

Further to my recent request for a Foundation Investigation, please be advised that the elevation of the proposed grade at the structure is 1,099.

FDeV/es

*S. McCombie*  
for F. DeVisser,  
Regional Bridge Location Engineer.

cc. S. McCombie

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

Materials and Testing Division

December 15, 1964

Geocor, Limited,  
14 Haas Road,  
Richdale, Ontario.

Attention: Mr. R. J. Allen

- Re: (1) Proposed Coalain River Patrol Yard, Hwy. 552.  
(2) W.P. 21-65, Sec. Hwy. 651, Windermere River, 15.3 Mi. North  
of Hwy. 101.  
✓ (3) W.P. 22-65, Sec. Hwy. 651, Ogisiwi Creek, 16.5 Mi. North of  
Hwy. 101.

-- District 18, Sault Ste. Marie --

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 December 15, 1964.

It is understood that a qualified Soils Engineer will be in charge of the field work at all times, and that the drill rig will be supplied from Sudbury.

Eleven copies of each completed foundation report, with one additional copy of each subsoil profile, should be submitted to the Foundation Section prior to January 29, 1965, for the report on the Patrol Yard site, and prior to March 1, 1965, for the others. 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 P.E.C. 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 P.E.C. with Cronaflex copies of the drawings.



Gowen, Limited.  
Attn: Mr. D. Bates.

- 2 -

December 15, 1964

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.

Yours very truly,



A. Bates,

MATERIALS & TESTING ENGINEER

WLB/MSF

cc: Messrs. E. McCombie  
E. McArthur  
A. A. Ward  
E. B. Saint  
Mrs. T. Tate  
E. D. Smith (2)  
Foundations Office  
Gen. Files (2)

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

Materials and Testing Division

December 11, 1964

Dominion Soil Investigation Ltd.,  
77 Crockford Boulevard,  
Scarborough, Ontario.

Attention: Mr. A. Bonca.

Re: (1) Proposed Goulais River Patrol Yard, Hwy. 552.  
(2) W.P. 21-64, Sec. Hwy. 651, Windermere River, 15.3 Mi. North  
of Hwy. 101.  
✓ (3) W.P. 22-64, Sec. Hwy. 651, Ogasiwi Creek, 16.5 Mi. North of  
Hwy. 101.  
-- District 18, Sault Ste. Marie --

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 December 11, 1964.

It is understood that a qualified Soils Engineer will be in charge of the field work at all times, and that the drill rig will be mobilized from Sudbury.

Eleven copies of each completed foundation report, with one additional copy of each subsoil profile, should be submitted to the Foundation Section prior to January 29, 1965, for the report on the Patrol Yard site, and prior to March 1, 1965, for the others. 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 Cronaflex copies of the drawings.

cont'd. /2 ...

Dominion Soil Investigation, - 2 -  
Attn: Mr. A. Bonca.

December 11, 1964

Charges for the work performed will be in accordance with your Schedule of Rates, dated July 6, 1964, and invoices to be addressed to the attention of the undersigned.

Yours very truly,



NDS/MdeF

A. Rutka,  
MATERIALS & TESTING ENGINEER

cc: Messrs. S. McCombie  
H. McArthur  
A. A. Ward  
E. R. Saint  
Mrs. T. Tate  
N. D. Smith (2)  
Foundations Office  
Gen. Files

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

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

Attention: Mr. A. McCombie

February 25, 1965

FOUNDATION INVESTIGATION REPORT BY:  
Gecon, Limited, Consulting Engineers.  
Ogisiwi Creek, Sec. Hwy. 651, Dist. #18,  
Sault Ste. Marie, Ont. W.P. 22-65.

Attached, please find the above-mentioned report submitted by the Consultant, Gecon Ltd. of Toronto. We have reviewed the report and have found the factual information adequate and well presented.

Estimating the bearing capacity of piles in such soils is very difficult and a pile loading test is therefore highly recommended. In view of the location, we would suggest that timber piles be given preference. For design purposes, we would recommend a safe pile load of 15 tons per 40-ft. long 14" butt timber pile. A pile loading test should provide the desired final information.

The above, together with the other recommendations contained in the report, should be adequate for your future design work. Should there, however, be any questions that you would like to discuss, please feel free to call on our office.

AGS/MdeP

Attech.

cc: Messrs. A. M. Teye (2)  
H. A. Tregaskes  
H. B. McMillan  
H. McArthur  
A. A. Ward  
E. P. Saint  
F. DeVlaser  
A. Watt

Foundations Office  
Gen. Files

*A. G. Sternac*  
A. G. Sternac,  
PRINCIPAL FOUNDATION ENGINEER

Mr. F. DeVisser,  
Regional Bridge Location Engr.,  
Regional Office (North-Western  
Region - Port William).

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

November 23, 1966

Ogasiwi Creek Bridge,  
16.4 Miles North of Hwy. 101,  
Hwy. 651, District #18,  
W.P. 22-65 (Report by Geocon, Ltd.)

A meeting was held at the Bridge Office on November 14, 1966, between the members of the Foundation Section and the Bridge Design Section, to discuss the Preliminary Plan of the above mentioned structure.

It was agreed that the following recommendations should be adopted in the design of the structure foundations.

The entire structure should be supported on 12-3/4 O.D. tubular piles driven to estimated tip elevations 1025 and 1020 at the abutment and pier locations, respectively. However, during construction, the pile driving should be controlled by the use of the Hiley Formula as per current D.H.O. Standards DD 1218 and DO 1219.

MD/MdeF

cc: Foundations Office  
Gen. Files

*M. Devata*

M. Devata,  
SUPERVISING FOUNDATION ENGR.  
For:  
A. G. Stermac,  
PRINCIPAL FOUNDATION ENGR.

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

Mr. A. Stermac,  
Principal Foundation Engineer,  
Laboratory Building,  
Downsview, Ontario.

FROM: Bridge Division,  
777 Memorial Avenue,  
P. O. Box 1170,  
Port Arthur, Ontario,  
DATE: November 1, 1966.

FILE REF.

IN REPLY TO

SUBJECT:

Site 38C-28, W. P. 22-65,  
Ogisiwi Creek Bridge,  
16.4 Miles North of Hwy. 101,  
Hwy. 651, District 18.

Attached is one print of preliminary plan for  
the subject structure, for your information.

If you have any comments, please let me know  
before November 11, 1966.

*F. DeVisser*

FDV/mcr  
Enc.

F. DeVISSER,  
Regional Bridge Location Eng.

# Ogasini Creek

WP 22-65

## Piers

Given: 18"  $\phi$  piles  $\therefore A_c = 1.77$  ft<sup>2</sup> ;  $\frac{A_s}{50} = \frac{4.71 L}{50} = 0.0942 L$

F.S. = 3

$$q_u = 4 N_c A_c + \bar{N} A_s / 50$$

From BH #3 - Gracon Ltd

Layer	$\bar{N}$	$\frac{\bar{N} A_s}{50}$	$4 N_c A_c$
1020 - 1040	6	28.2	
1040 - 1030	14	13.2	99
1030 - 1000	26	73.5	180

Desire  $q_{all} = 60-70$  tons, say 65 tons  $\therefore q_u = 195$  tons

The pile capacity at 1030 = 140.4 Tons but as tip penetrates the compact silt the capacity rapidly exceeds the desired 195 tons

$\therefore$  Use Pile 18"  $\phi$  to elev. 1025

$$\therefore q_u = 180 + 28.2 + 13.2 + 12.2 = 233.6$$

$$\therefore q_{all} = \underline{78 \text{ Tons}}$$

Abutments

Given  $12\frac{3}{4}''$  piles  $\phi A_t = 0.885''^2$  ;  $\frac{A_s}{50} = 0.0668L$   
 $FS = 3$   
 $q_u = 4N_t A_t + \bar{N} A_s / 50$

Desire  $q_{all} = 40 \text{ to } 45 \text{ tons}$  Say  $42.5 \text{ tons}$   $\phi q_u = 127.5 \text{ tons}$

From BH #3 - Geoscon Ltd

Layer	$\bar{N}$	$\frac{\bar{N} A_s}{50}$	$4 N_t A_t$
1030 - 1040	6	20	
1040 - 1030	14	9.3	
1030 - 1000	26	52.1	92

$\phi$  pile exceeds required capacity as tip penetrates 1030 - 1000 layer.

$\phi$  Use pile  $12\frac{3}{4}''$  to elev. 1025

$$q_u = 92 + 20 + 9.3 + 8.7 = 130.0$$

$$\phi q_{all} = 43 \text{ tons}$$

Pier 1020.0 50 tons /  
 1025.0 Abutments 40 tons /  
 12 3/4" O.D.

Note: In bridge drawing the pile tip elevations are going to be shown and it will be specified that these elevations should be reached. Any prior stoppage should be approved by D&B Foundations. Discussion with Gus Dec 19/66

CAG



OGASIWI RIVER BRIDGE

W.P. 22-65 ; CONTR. Nº 67-63

OGASIWI RIVER BRIDGE W.P. 22-65 CONTR NO 67-63

NUMBER OF PILES DRIVEN	26
NORTH ABUTMENT	7
PIER NO 2	6
PIER NO 1	6
SOUTH ABUTMENT	7

PILES DRIVEN TO PRACTICAL REFUSAL  
BETWEEN ELEVATIONS 984.80 (MAXIMUM)  
AND 970.75 (MINIMUM).

PRACTICAL REFUSAL DEFINED AS APPROX. 30-40  
BLOWS PER INCH WITH A REBOUND OF APPROX.  
0.7-1.0 INCH.

THE PILES SHOULD HAVE BEEN DRIVEN TO  
ELEVATION

1020.0 FOR ABUTMENTS

1025.0 FOR PIERS

AT THE ABOVE ELEVATIONS (PILE LENGTH 71 FT)  
THE RESISTANCE TO DRIVING WAS 13 BLOWS/FT,  
[LENGTH OF PILE DRIVEN WAS 73 FT]

BY THE HILEY FORMULA [D-12 HAMMER] AND AN  
ASSUMED REBOUND OF 0.2 INCHES/BLOW THE  
ULTIMATE BEARING CAPACITY IS 81 TONS  
FOR ANOTHER 10 FT OF PILING [TOTAL LENGTH 83 FT]  
FOR 13 BLOWS/FT OF PENETRATION  $Q_{ULT} = 68$  TONS  
HOWEVER RECORDS SHOW THAT THE BLOWS/FOOT

INCREASED TO 16 GIVING QULT OF 82 TONS.

IT IS BELIEVED THAT THE REBOUND WAS EVEN LESS THAN 0.2 INCH/BLOW WHICH IN TURN WOULD GIVE A HIGHER BEARING CAPACITY VALUE.

FROM THE ABOVE IT IS EVIDENT THAT A CHECK SHOULD HAVE BEEN MADE WHEN THE PILES REACHED THE SPECIFIED ELEVATION.

QULT OF 70 TONS WOULD HAVE BEEN PROBABLY CONSIDERED INADEQUATE BUT A SUBSEQUENT CHECK WITH 16 OR MORE BLOWS WOULD HAVE PROVIDED A VALUE IN EXCESS OF 80 TONS AND THAT WOULD HAVE BEEN THE END.

IT SHOULD ALSO BE NOTED THAT THE SITUATION NOW IS BEARING PILES TO ROCK WITH ONLY 45 TO 50 TONS ON THEM INSTEAD OF 90 T.

IT IS ALSO WORTHWHILE MENTIONING THAT AT EACH SPlicing THE BLOW COUNT WENT UP INDICATING AN IMPROVEMENT OR INCREASE OF BEARING CAPACITY WITH TIME.

REBOUND	No/Blows	PILE	LENGTH	
		73 QULT	82 QULT	73 QULT
0.2	13	61	68	71
0.6	13	51	57	60
1.0	13	44	49	51

0.2	16	73	82	86
0.6	16	60	66	69
1.0	16	50	56	58

# OGASIWI 12 BR.

W.P. 22-65

LENGTH OF PILE

CUT OFF ELEV.

PILE TIP ELEV.

	1		<u>107.51</u>	1091.60	<u>984.10</u>	
	2		116.5	1091.60		
	3		111.25	1091.60		
N.A.	4		115.00	1091.60		
	5		114.50	1091.60		
	6		<u>117.25</u>	1091.60	<u>974.35</u>	
	7	LENGTH IN GEODS	116.50	1091.60		
	8	103	<u>111.75</u>	1096.55	<u>984.80</u> MAX	<u>984.80</u>
	9		113.75			
P.2	10		113.25			
	11		114.25			
	12		104.25			
	13		<u>114.25</u>		<u>981.80</u>	
	14		113.00	1096.35		
	15		115.00			
	16		<u>115.75</u>		<u>980.60</u>	
P.1	17	109	114.75			
	18		112.50			
	19		<u>112.50</u>		<u>983.85</u>	
	20		117.00	1091.00	974.00	
	21		<u>120.25</u>		<u>970.75</u> MIN	<u>970.75</u>
	22		116.25		974.75	
S.A	23		118.00		973.00	
	24		110.00			
	25		117.50			
	26		<u>110.50</u>		<u>980.50</u>	

### PILE DETAILS

STEEL TUBE  $12\frac{3}{4}$ " O.D X  $\frac{1}{4}$ " WALL THICKNESS

LENGTH OF PILE DRIVEN : 83 FT

WEIGHT OF PILE DRIVEN : 1.38 T

### DRIVING DETAILS

HAMMER TYPE : DELMAG D-12 WEIGHT OF HAMMER : 1.38 T

RATED ENERGY : 22,500 LBFT

TYPE OF ANVIL : STEEL

WEIGHT OF ANVIL : 0.25 T.

BLOWS PER FOOT : 17

REBOUND RANGE : 0.2 TO 1.0 IN.

Slide

# OGASIWI RIVER BRIDGE W.P. 22-65 Contr N° 67-63

BEARING CAPACITY OF PILES  
by the HILEY FORMULA

L = 73'    W = 1.21 T

N° of blows = 13

Rebound c	Q <sub>ult</sub>
0.2	71
0.6	60
1.0	51

N° of blows = 16

Rebound c	Q <sub>ult</sub>
0.2	86
0.6	69
1.0	58

L = 83'    W = 1.38 T

N° of blows = 13

Rebound c	Q <sub>ult</sub>
0.2	68
0.6	57
1.0	49

N° of blows = 16

Rebound c	Q <sub>ult</sub>
0.2	82
0.6	66
1.0	56

L = 108'    W = 1.81 T

N° of blows = 13

Rebound c	Q <sub>ult</sub>
0.2	61
0.6	51
1.0	44

N° of blows = 16

Rebound c	Q <sub>ult</sub>
0.2	73
0.6	60
1.0	50

NOTE: FOR THE SAME NUMBER OF BLOWS THE BEARING CAPACITY DECREASES WITH THE INCREASE OF PILE LENGTH [ GREATER WEIGHT TO BE DRIVEN ]

L = 73'

①  
CALCULATION OF ULTIMATE LOAD  
for PILE RESISTANCE  
**OGASIWI RIVER BRIDGE**

128

1.38  
+ 1.48  
1.528

$n = \text{efficiency of blow} = \frac{W + P_u}{W + P}$  where  $W = 1.38 \text{ Tons}$   
 $P = 73 \times 33.3 = 2430.6$   
 $P = \frac{2430}{2000} = 1.215 \text{ T.}$   
 $P = 1.25$   
 $1.465 \text{ T.}$

$n = \frac{1.38 + 1.465 \times .102}{1.38 + 1.465} = \frac{1.528}{2.845} = .536$

WEIGHT OF PILE DRIVEN 1.21T

$R = \frac{nWh}{S + C/2}$  tons. where  $Wh = 28,500 \times 12 \text{ in. lb.}$   
 $S = \frac{12}{13} = .923$

$S = \frac{12}{13}$

$C = 0.2$

$R = \frac{.536 \times \frac{11.25}{22,500} \times 12}{(.923 + 0.1) 2000} = \frac{72.5}{1.023} = 71 \text{ } \underline{70.9 \text{ tons.}} \quad C = 0.2$

$R = \frac{72.5}{1.223} = \underline{59.2 \text{ tons}} \quad 60 \quad \underline{C = 0.6}$

$R = \frac{72.5}{1.423} = \underline{50.9 \text{ tons}} \quad 51 \quad \underline{C = 1.0}$

$S = \frac{12}{16}$



$R = \frac{72.5}{.75 + 0.1} = \underline{85.4 \text{ tons}} \quad 86 \quad C = 0.2$

$R = \frac{72.5}{1.05} = 69.0 \text{ tons} \quad 69 \quad C = 0.6$

$R = \frac{72.5}{1.25} = 58.0 \text{ tons.} \quad 58 \quad C = 1.0$

CONTRACT NO 67-63

W.P. 22-65



$$L = 83'$$

$$n = \frac{W + P_c^2}{W + P}$$

where

$$W = 1.38 \text{ Tons}$$

$$P = \frac{(83 \times 33.3)}{2000} \times 1.25 = 1.38 + 2.5 = 1.637$$

$$n = \frac{1.38 + 1.63 \times 1.02}{1.38 + 1.63} = \frac{1.545}{3.01} = .514$$

WEIGHT OF PILE DRIVEN  
1.38T

$$R = \frac{n W h}{S + \frac{c}{2}} = \frac{.514 \times 11.85 \times 12}{.923 + 0.1}$$

$$\text{when } S = \frac{12}{13} = .923$$

$$c = 0.2$$

$$S = \frac{12}{13}$$

~~R =~~

$$R = \frac{69.5}{1.023} = 67.9 \text{ tons} \quad 68 \quad C = 0.2$$

$$R = \frac{69.5}{1.223} = 56.7 \text{ tons} \quad 57 \quad C = 0.6$$

$$R = \frac{69.5}{1.423} = 48.7 \text{ tons} \quad 49 \quad C = 1.0$$

$$S = \frac{12}{16}$$

$$R = \frac{69.5}{.855} = 81.8 \text{ tons} \quad 82 \quad C = 0.2$$

$$R = \frac{69.5}{1.05} = 66.1 \text{ tons} \quad 66 \quad C = 0.6$$

$$R = \frac{69.5}{1.25} = 55.6 \text{ tons} \quad 56 \quad C = 1.0$$

L = 108'

(3)

$$n = \frac{1.38 + .209}{3.43} = .463$$

WEIGHT OF PILE DRIVEN  
1.81 T

$$R = \frac{.463 \times 11.25 \times 12}{.923 + 0.1}$$

$$R = \frac{62.5}{1.023} = 61.0 \text{ T.} \quad 61$$

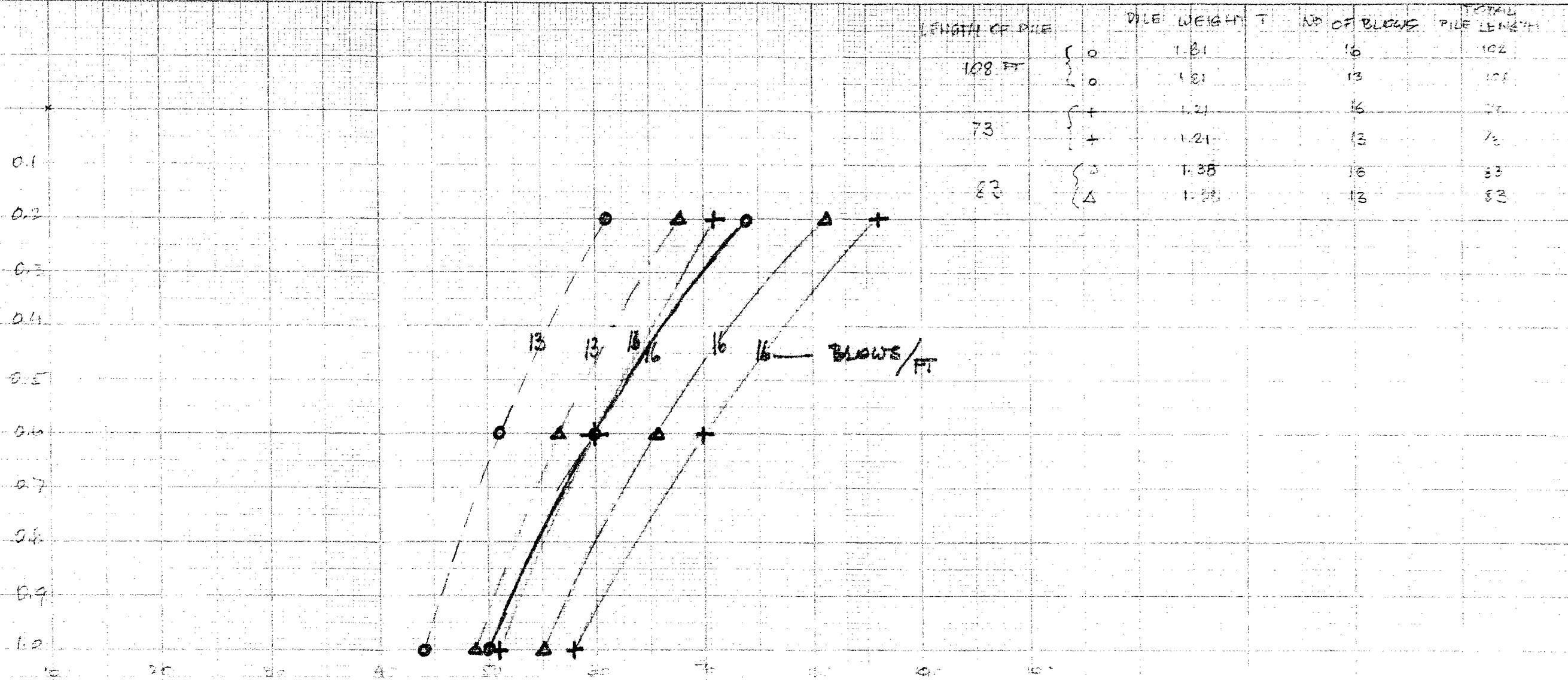
$$R = \frac{62.5}{1.223} = 51.0 \text{ T.} \quad 51$$

$$R = \frac{62.5}{1.423} = 43.8 \text{ T.} \quad 44$$

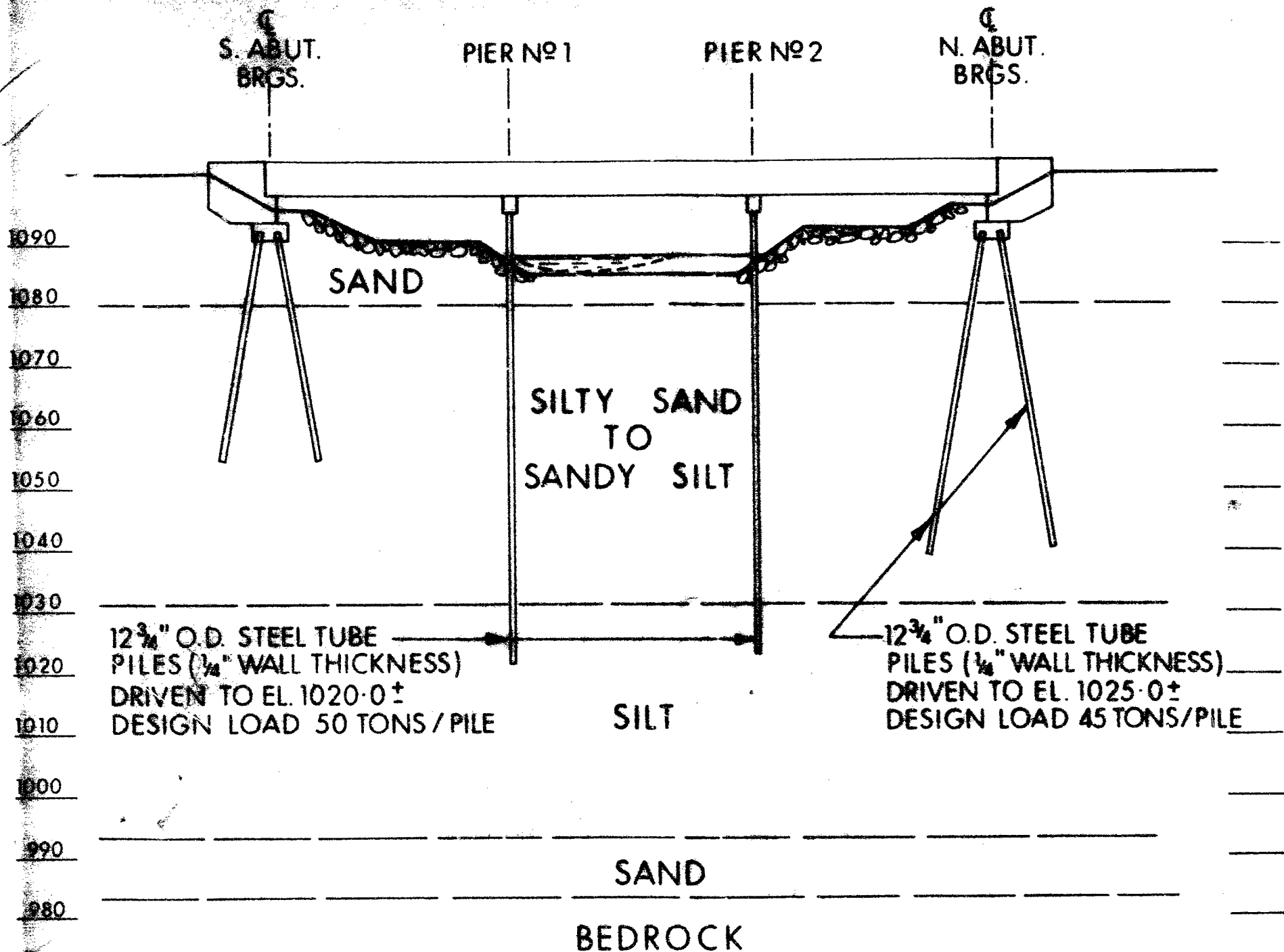
$$R = \frac{62.5}{1.855} = 33.7 \text{ T.} \quad 73$$

$$R = \frac{62.5}{1.055} = 59.5 \text{ T.} \quad 60$$

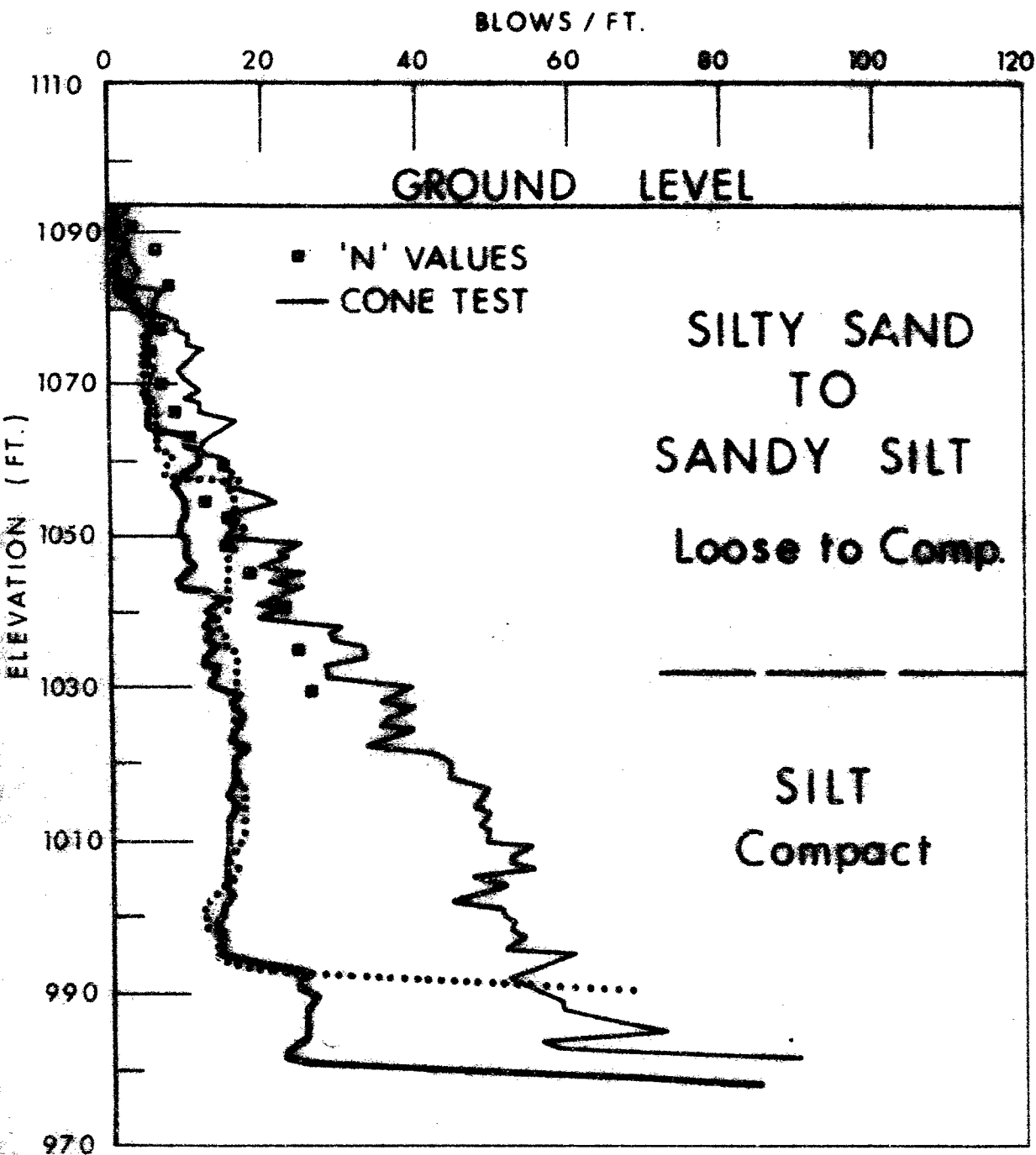
$$R = \frac{62.5}{1.25} = 50.0 \text{ T.} \quad 50$$



OGASIWI RIVER BRIDGE  
 CONTR. NO 67-63  
 W.P. 22-65

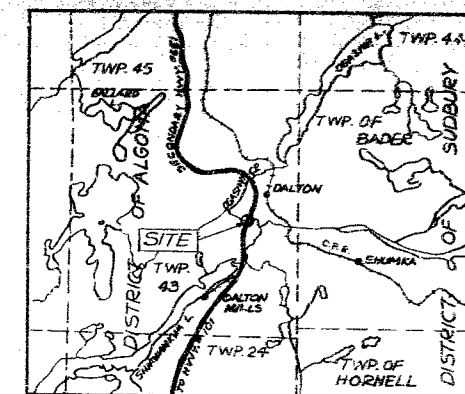
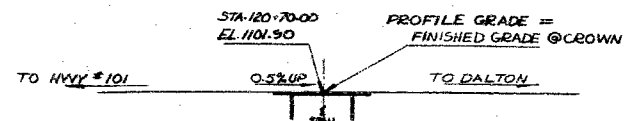
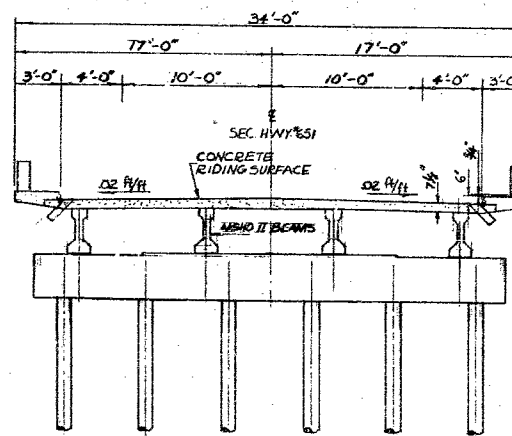
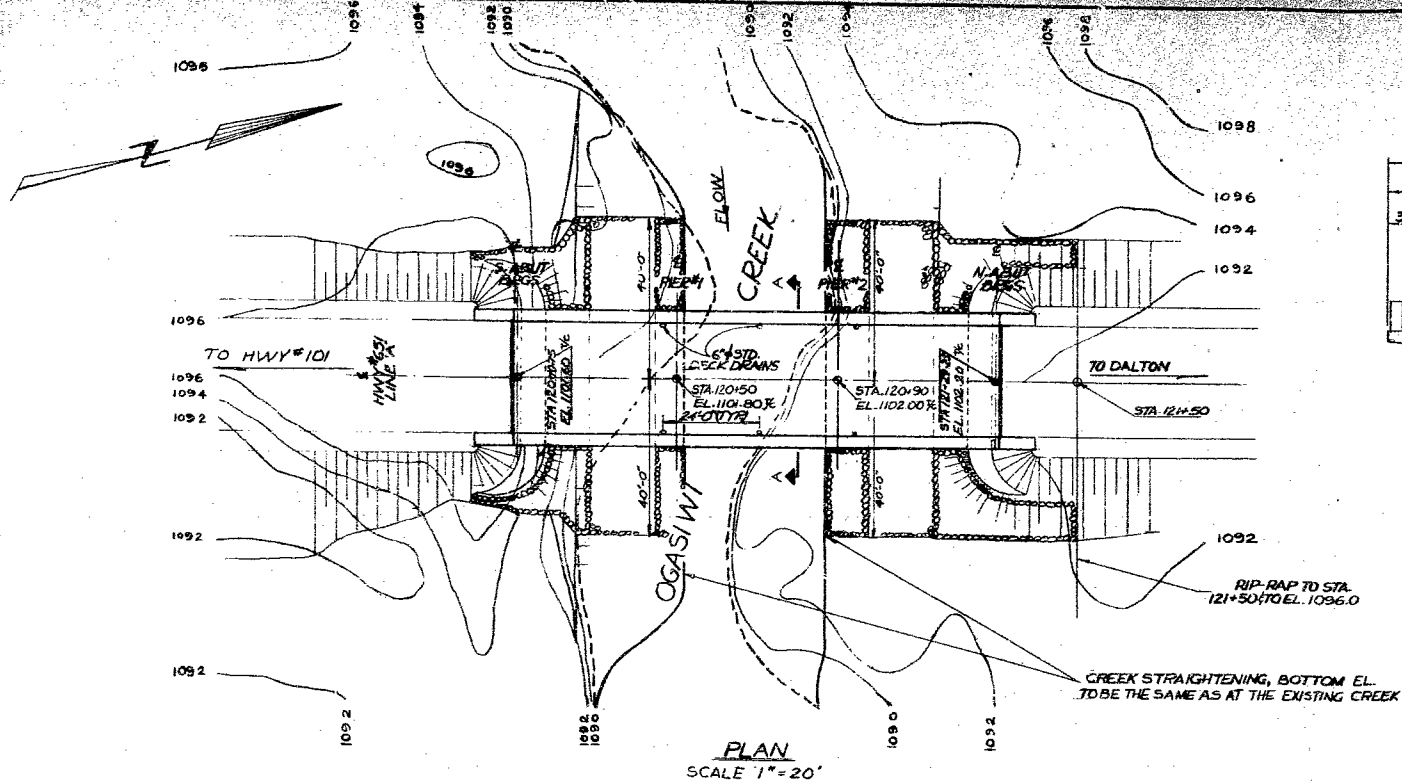


# PILE DRIVING RECORD



— PILE NO 2 - 12 $\frac{3}{4}$ " O.D. STEEL TUBE, 33.38 lb./in.ft.  
1:4 BATTER, SHOE 13 $\frac{1}{2}$ " x 1"  
NORTH ABUT., DRIVEN SEPT. 6&7, 1967

..... PILE NO 8 - 12 $\frac{3}{4}$ " O.D. STEEL TUBE, 33.38 lb./in.ft.  
VERTICAL, SHOE 13 $\frac{1}{2}$ " x 1"  
PIER NO 2, DRIVEN AUG. 30, 1967

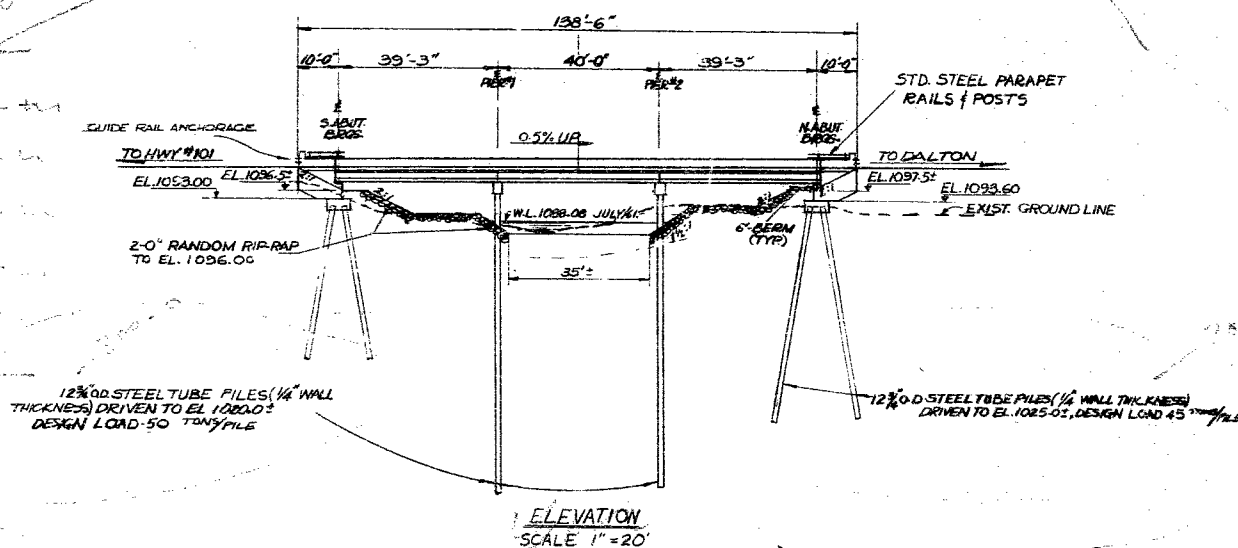


CLASS OF CONCRETE  
DECK & PARAPET WALLS ----- 4000 P.S.I.  
REMAINDER ----- 3000 P.S.I.  
FOR PRESTRESSED GIRDERS SEE DING. D-5673-5

CLEAR COVER ON REINFORCING STEEL  
FOOTINGS ----- 3"  
ABUTMENTS ----- 3"  
PIER CAPS ----- 2"  
DECK TOP ----- 1 1/2"  
BOTTOM ----- 1"  
CURBS ----- 2"  
AND/OR AS NOTED

CONSTRUCTION NOTES  
CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF  $\pm 1/8$  INCH.  
NO CONCRETE SHALL BE PLACED ABOVE THE ABUTMENT BEARING SEATS UNTIL CONCRETE IN THE DECK HAS BEEN PLACED.

B.M. EL. 1093.81  
GEODETIC DATUM  
N. 1/4 IN. W. ROOT OF  
12 SPR. 62' RT. OF  
STA. 113+49



- LIST OF DRAWINGS  
D-5673-1 GENERAL LAYOUT  
-2 BORE HOLE LOCATIONS & SOIL STRATA  
-3 FOOTING LAYOUT, ABUTMENTS, PIER CAP DETAILS  
-4 DECK, DIAPHRAGMS, 15 CREED ELEVATIONS  
-5 PRESTRESSED GIRDERS & BEARINGS  
-6 PARAPET WALL DETAILS  
-7 STANDARD STEEL PARAPET RAIL  
-8 STANDARD DETAILS

DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO  
BRIDGE DIVISION

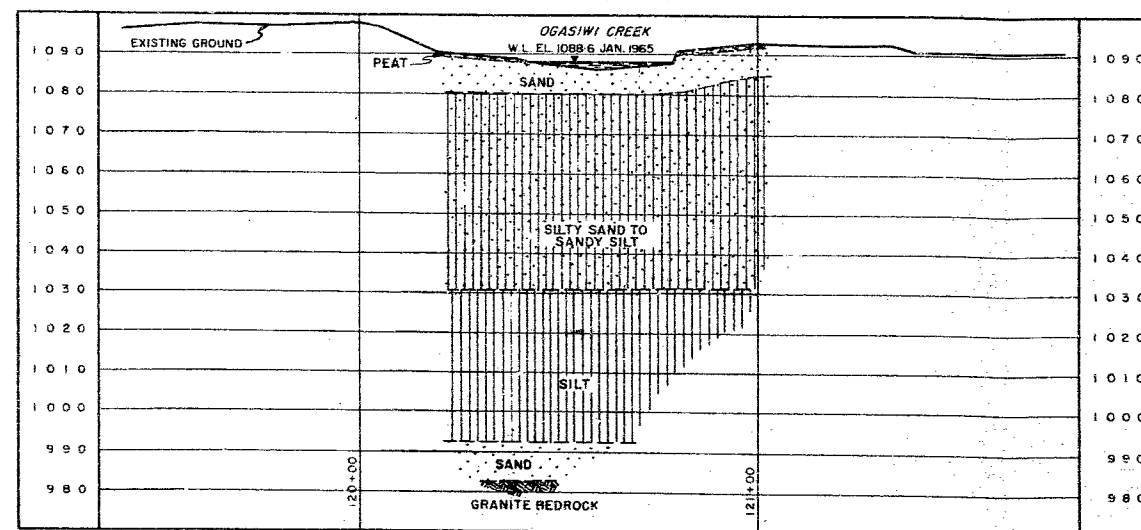
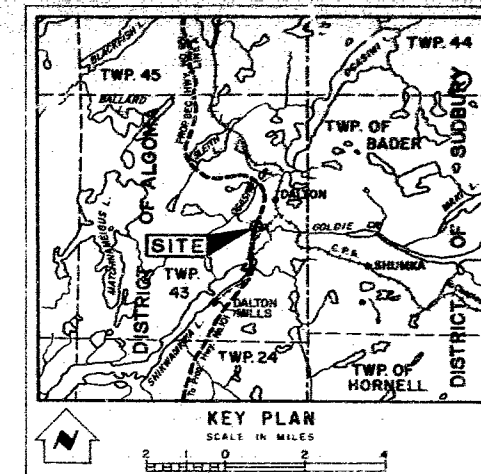
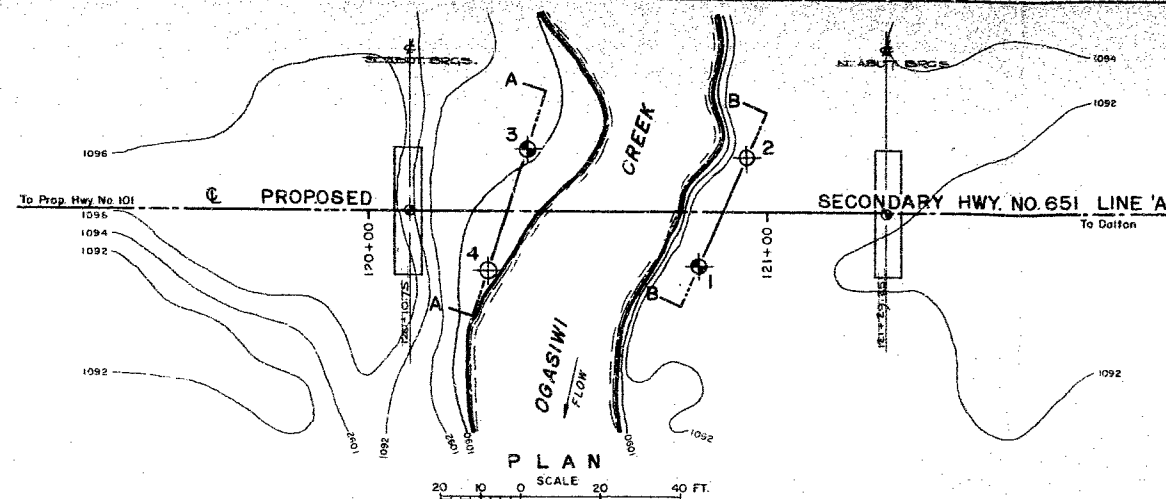
## OGASIWI CREEK BRIDGE

15.4 MILES NORTH OF HWY#101

KING'S HIGHWAY No. SEC. HWY #651 DIST. No. 18  
DISTRICT OF ALGOMA  
TWP. 43 LOT CON.

### GENERAL LAYOUT

APPROVED	DESIGNED	CONTRACT	NO.
DESIGN	CHECK	DATE	LOADING



#### REFERENCE BENCHMARK

B.M. ELEV. 1093.81 GEODETIC DATUM  
NAIL & WASHER IN W. ROOT OF 1-2 SPR.  
62' RT. OF STA. 119+49

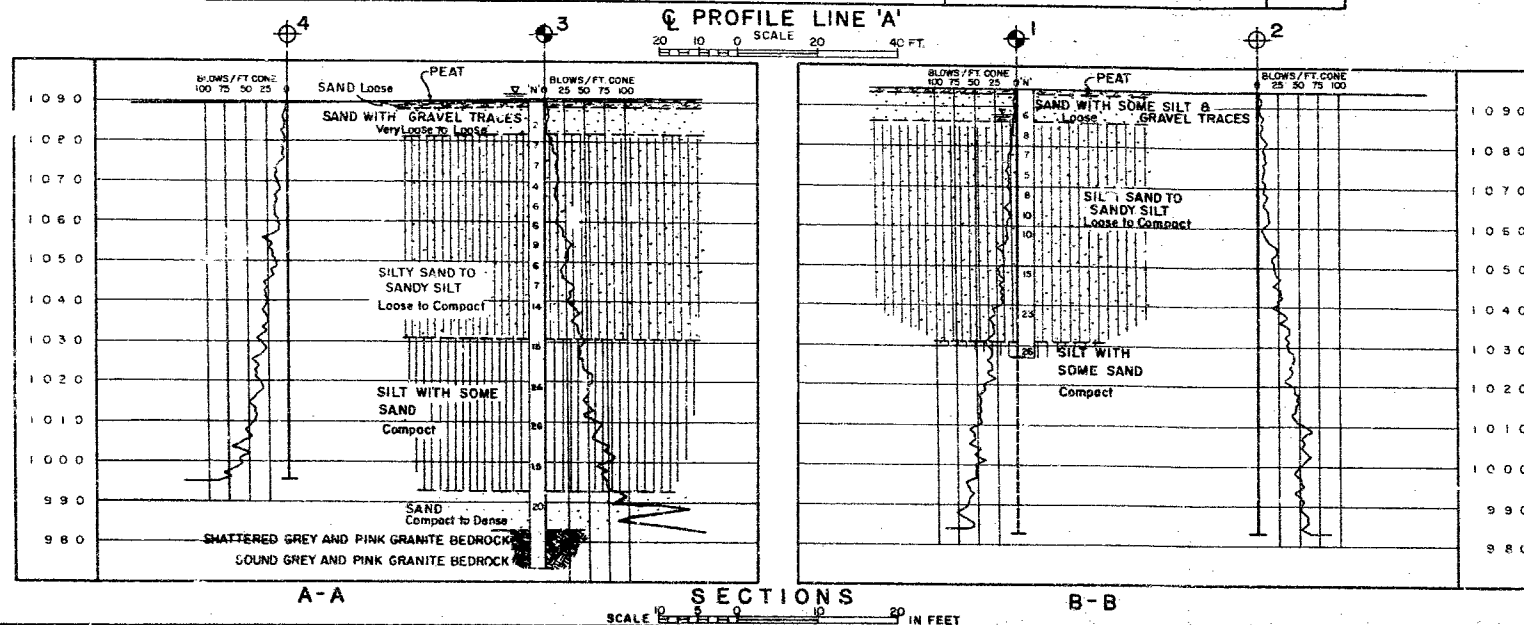
#### LEGEND

- Bore Hole
- ⊕ Cone Penetration Hole
- ⊕ Bore & Cone Penetration Hole
- ≡ Water Levels established at time of field investigation, Jan. 1965
- ≡ Artesian Head Jan. 1965

NO.	ELEVATION	STATION	OFFSET
1	1094.1	120+83	14 RT.
2	1093.9	120+95	14 LT.
3	1090.3	120+40	16 LT.
4	1089.6	120+30	15 RT.

#### 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.



#### GEOCON LTD.

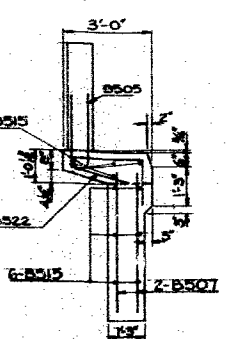
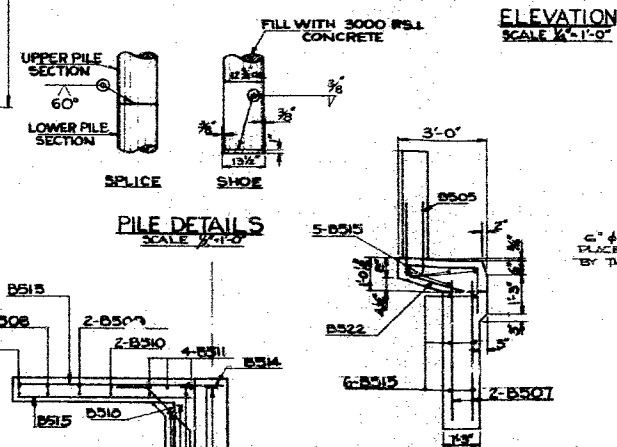
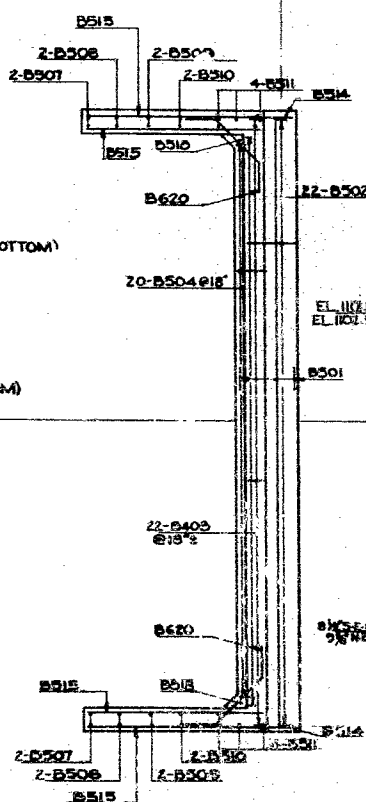
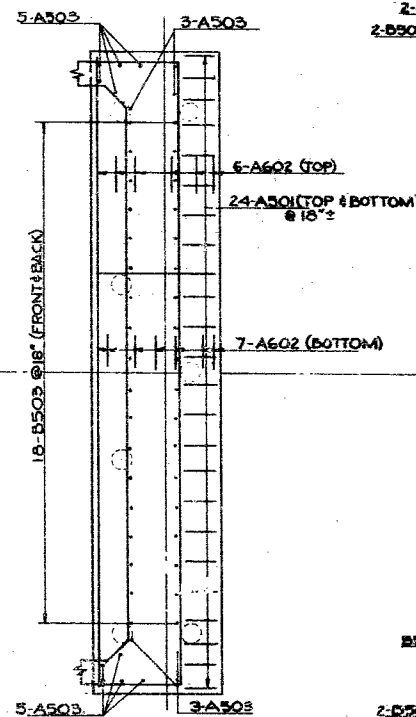
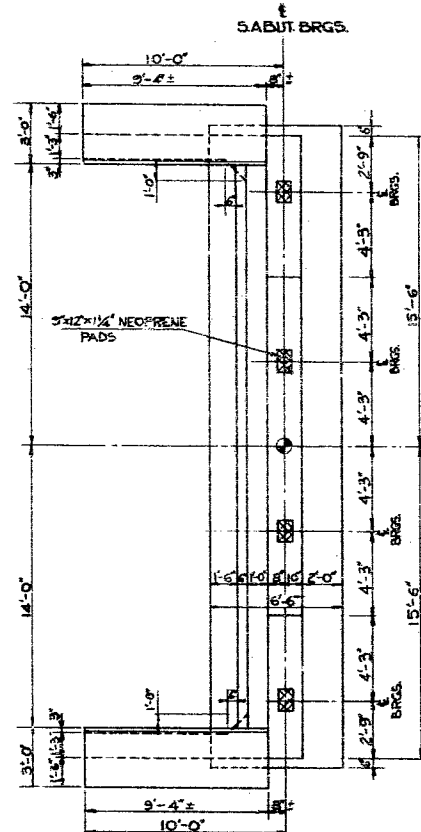
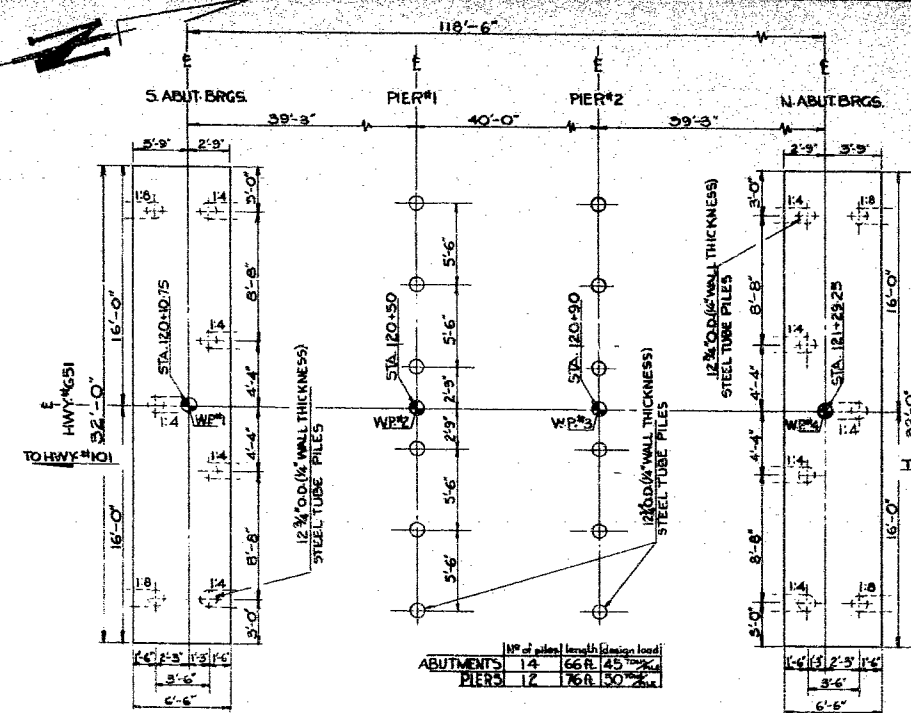
DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS & RESEARCH DIVISION - FOUNDATION SECTION

#### OGASIWI CREEK

KING'S HIGHWAY NO. SECONDARY HWY. NO. 651 DIST. NO. 19  
DISTRICT OF ALGOMA  
TWP. 43 LOT --- CON. ---

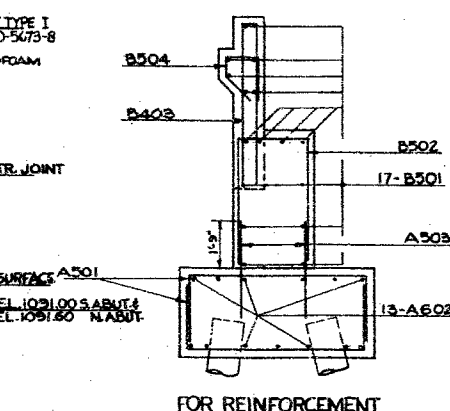
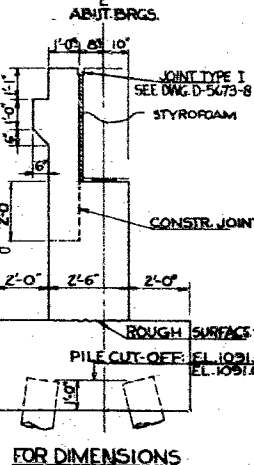
#### BORE HOLE LOCATIONS & SOIL STRATA

SUBM. B.T.D.	CHECKED DBO	W.P. NO. 22-85	DRAWING NO. T7710-1
DRAWN J.H.	CHECKED DBO	JOB NO.	BRIDGE DRAWING NO.
DATE FEBRUARY 1, 1965	SITE NO. 382-21	CONT. NO. 67-62	D5673-2
APPROVED B.T. Dwyer			

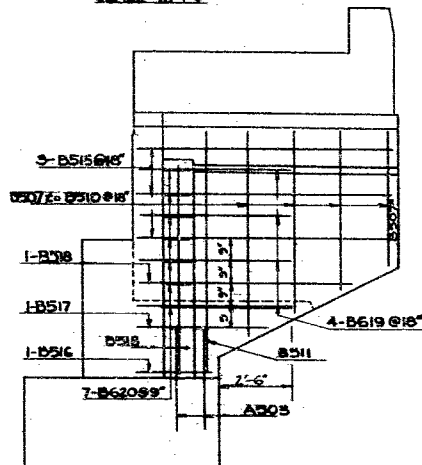


ELEVATION  
SCALE 1/4"=1'-0"

6" & C.I.P. PLACED AS DIRECTED BY THE ENGINEER



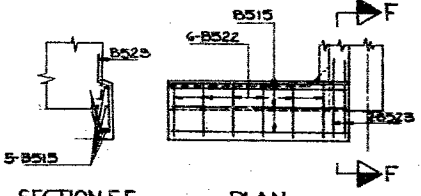
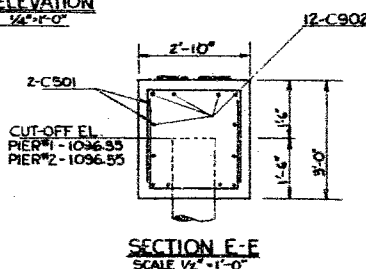
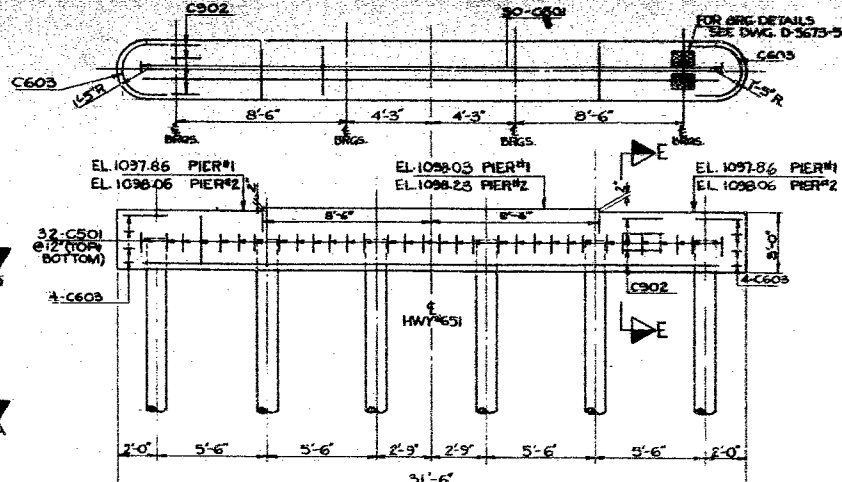
SECTION C-C  
SCALE 3/8"=1'-0"



WING WALLS  
SCALE 1/8"=1'-0"

OUTSIDE FACE

INSIDE FACE



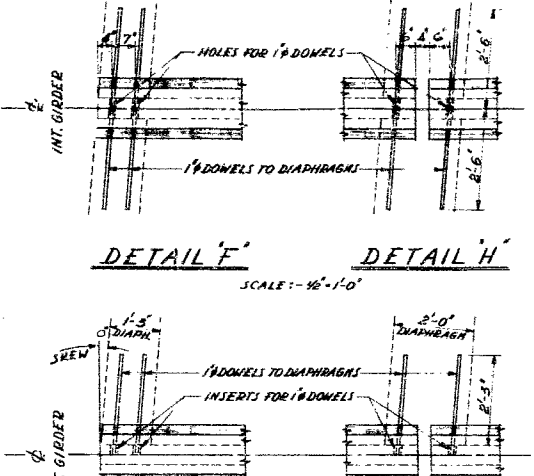
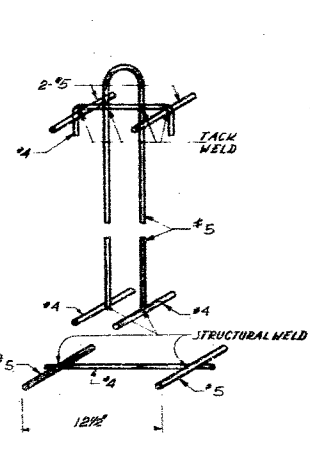
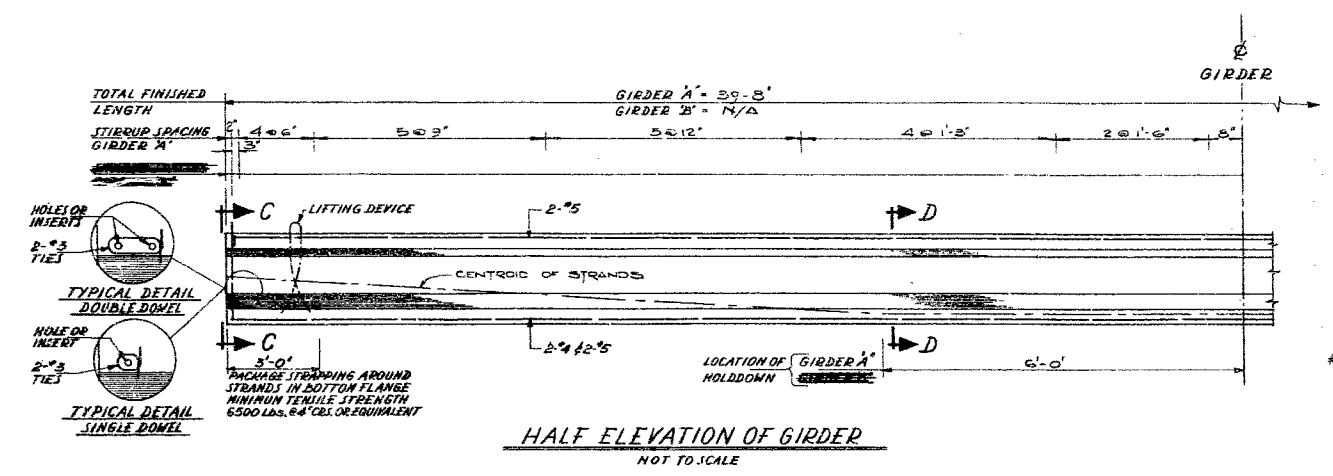
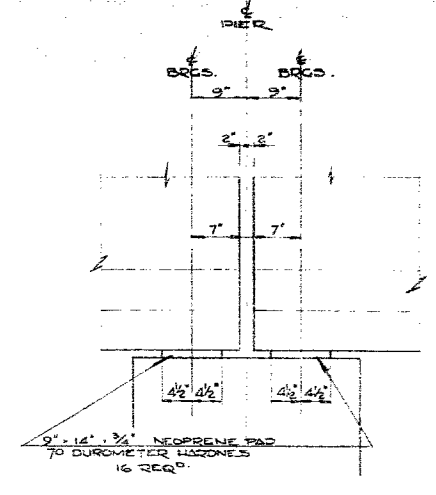
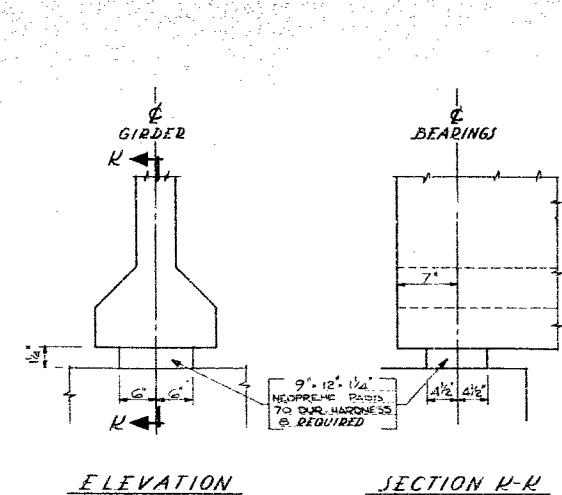
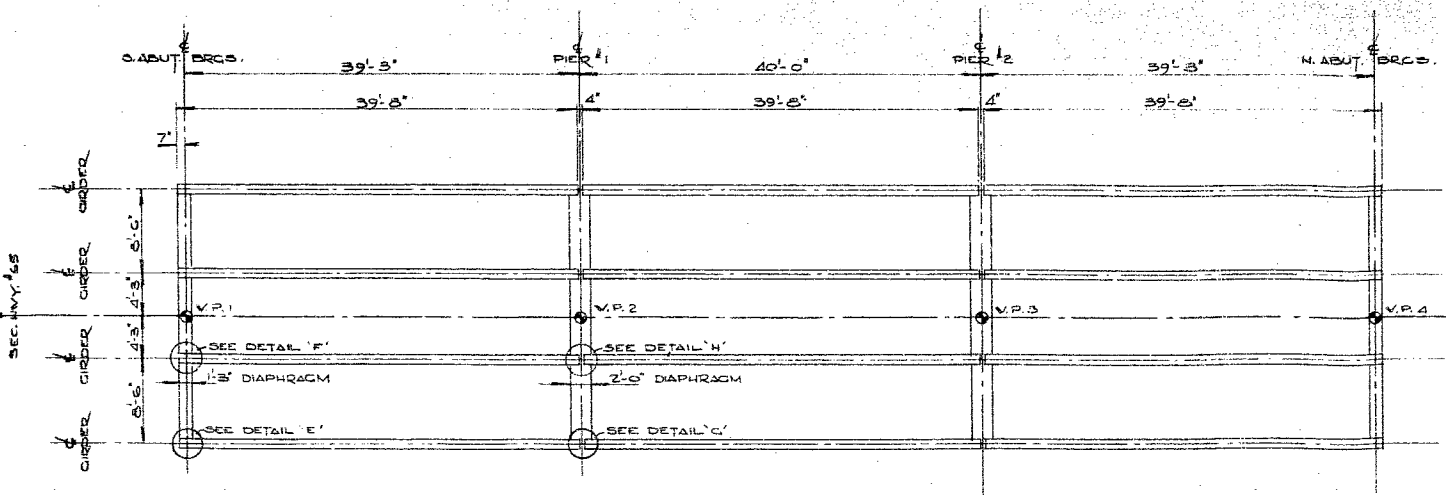
CURB DETAILS  
SCALE 1/4"=1'-0"

DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO BRIDGE DIVISION			
<b>OGASIW CREEK BRIDGE</b> 16.4 MILES NORTH of HWY #101			
KING'S HIGHWAY No. 651		DIST. No. 1B	
E.B. DISTRICT of ALGOMA		TWP. 43 LOT CON.	
FOOTING LAYOUT-ABUTMENTS-PIERCAP DETAILS			
APPROVED	DESIGN	CHECK	P.O.L.
DATE	NOV/88	LOADING	MS20-44
CONTRACT No.		67-63	
DRAWING No.		D-5673-3	

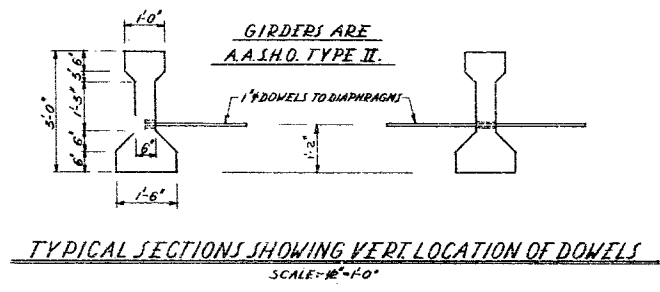
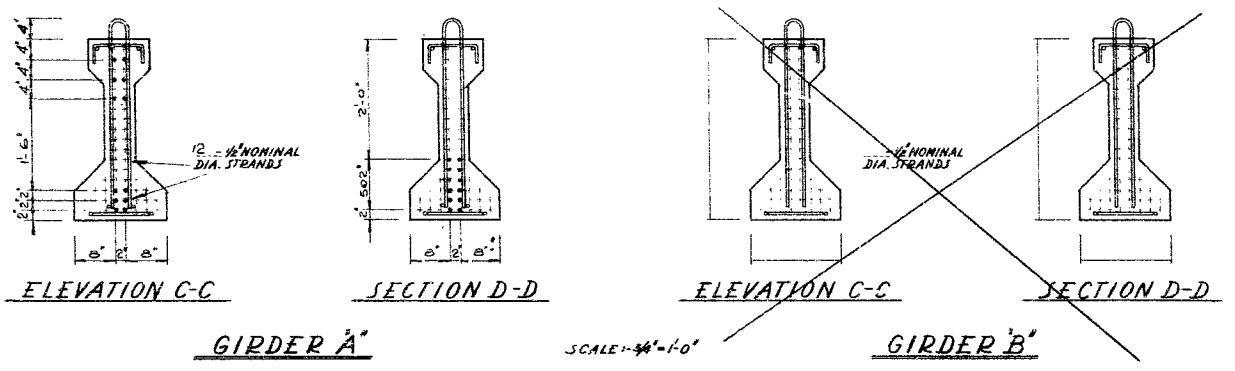






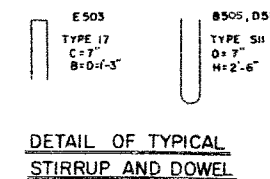
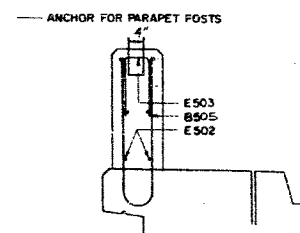
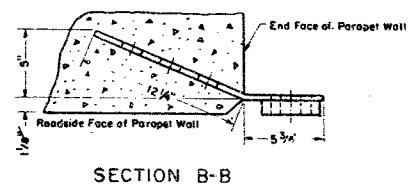
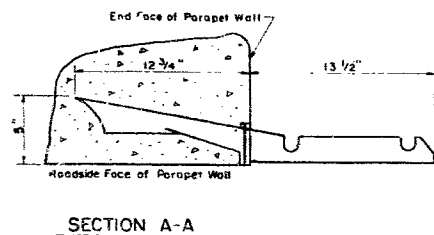
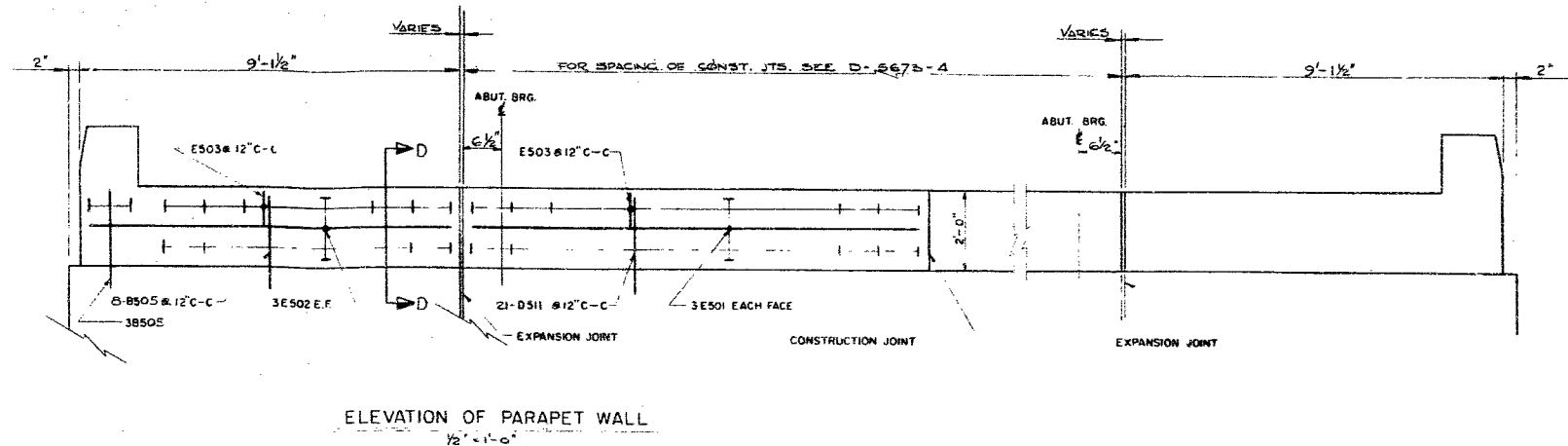
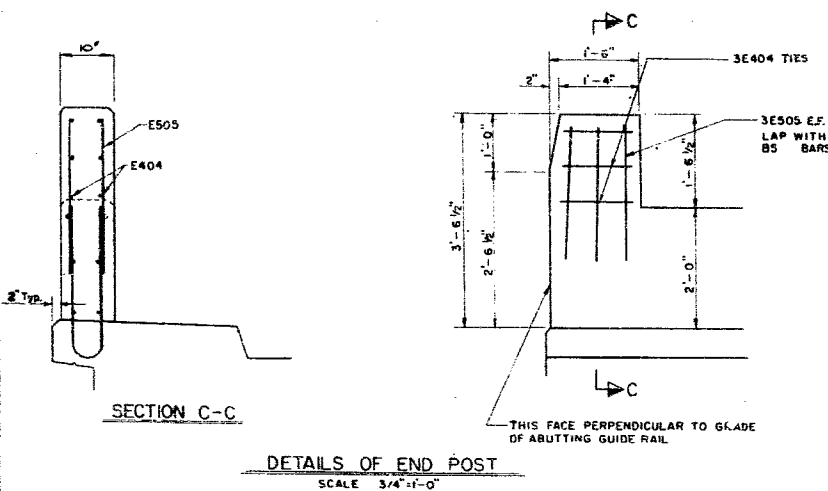
- NOTES FOR PRE-TENSIONED GIRDERS**
- CONCRETE STRENGTH AT 28 DAYS 5000 p.s.i.
  - CONCRETE STRENGTH AT TRANSFER 4000 p.s.i.
  - STRAND TYPE 1/2" NOMINAL DIA. 7 WIRE EXTRA HIGH STRENGTH.
  - MINIMUM ULTIMATE STRENGTH OF STRANDS 41,300 lbs.
  - INITIAL FORCE PER STRAND 30,150 lbs.
  - WORKING FORCE PER STRAND AFTER ALL LOSSES 24,800 lbs.
  - MINIMUM CLEAR COVER ON REINFORCING STEEL 1"

- NOTES FOR DOWELS**
- DOWEL INSERTS SHALL BE CAPABLE OF DEVELOPING FULL STRENGTH OF DOWELS.
  - 1" DOWELS FOR EXTERIOR GIRDERS SHALL BE THREADED AT ONE END TO MATCH INSERTS
  - ALL DOWELS AND INSERTS SHALL BE SUPPLIED AND INSTALLED (OR GROUTED) BY THE CONTRACTOR.

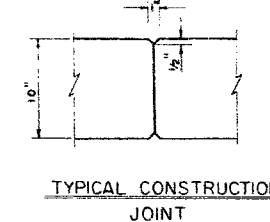
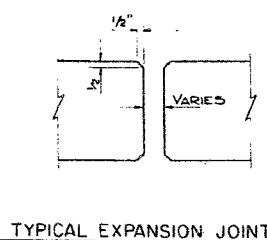
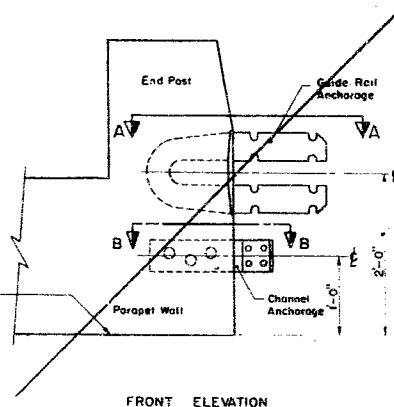
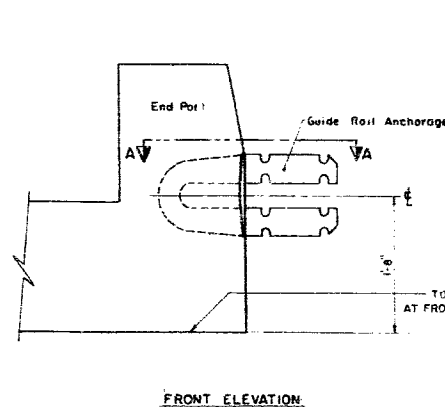


REVISIONS			
DATE	BY	DESCRIPTION	
DEPARTMENT OF HIGHWAYS ONTARIO BRIDGE DIVISION			
OGASIVI CREEK BRIDGE 16.4 MI. NORTH OF HWY. 101			
SEC. HWY. 401		DIST. No. 101	
DIST. OF ALGOMA			
TWP. 45	LOT	CON.	
PRESTRESSED GIRDERS & BEARINGS			
APPROVED	DATE	SITE No.	W.P. No.
		38 C-25	22-65
DESIGN	CHECK	PO. L	CONTRACT
DRAWING			
DATE	NOV. '66	LOADING	HS20-44
DRAWING No.	D-5673-5		

A.A.S.H.O. II.



- NOTES
- 1 FOR SPACING OF PARAPET RAIL POSTS SEE PARAPET RAIL DRAWING
  - 2 REINFORCING STEEL NOT CONTINUOUS THROUGH JOINTS. COVER TO TOP OF WALL FOR BARS E503 & TOP BARS E501, E502, E505 - 1 1/2" MIN. 2" MAX.
  - 3



DETAIL OF GUIDERAIL ANCHORAGE

DETAIL OF GUIDERAIL AND CHANNEL ANCHORAGE

APPROVED 15 Sept 1966  
BRIDGE DESIGN ENGINEER

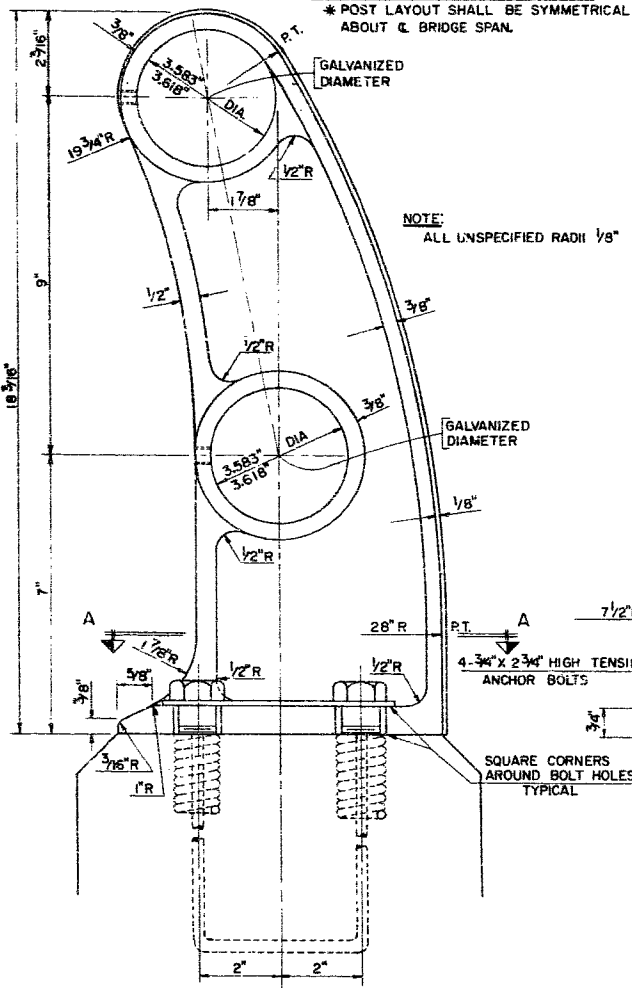
REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO BRIDGE DIVISION			
OCASIMI CREEK BRIDGE 16.4 MI. NORTH OF HIGHWAY No. 101			
KING'S HIGHWAY No. SEC. HWY. # 651		DIST. No. 18	
DIST. OF ALGOMA		TWP. # 43 LOT CON.	
PARAPET WALL DETAILS			
DESIGN	ADAPTED	CHECK	DATE
DRAWING	W.V.	CHECK	DATE
DATE	NOV 1966	LOADING	15 20-66
SHEET No. 38 C-28		W.P. No. 22-65	
CONTRACT No.		DRAWING No. D-5673-6	

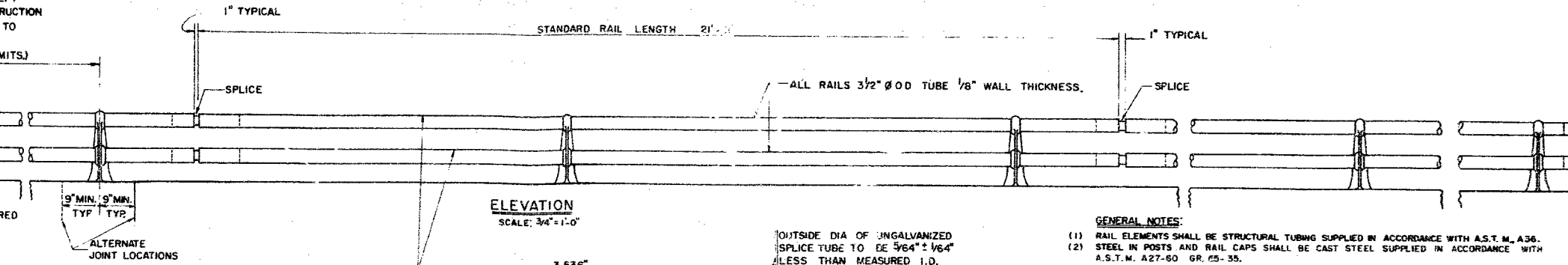
WHERE POST LAYOUT IS NOT SHOWN,  
POSTS SHALL BE EQUALLY SPACED EXCEPT  
THAT POSTS AT EXPANSION AND CONSTRUCTION  
JOINTS SHALL BE MOVED AS REQUIRED TO  
ATTAIN 9" CLEARANCE.  
(SEE TABLE BELOW FOR POST SPACING LIMITS)

DISTANCE END TO END OF RAIL	POST SPACING	
	MINIMUM	MAXIMUM
UNDER 40'	9'-0"	12'-0"
OVER 40'	10'-0"	12'-0"

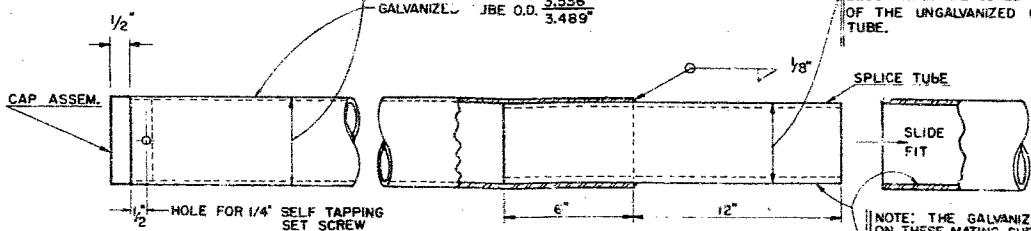
\* POST LAYOUT SHALL BE SYMMETRICAL  
ABOUT A BRIDGE SPAN.



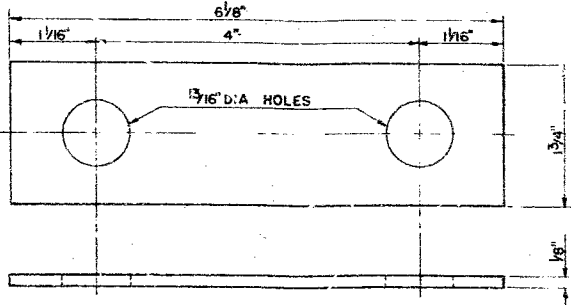
SIDE ELEVATION



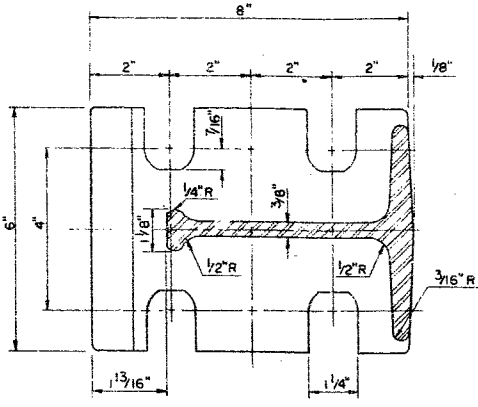
ELEVATION  
SCALE: 3/4" = 1'-0"



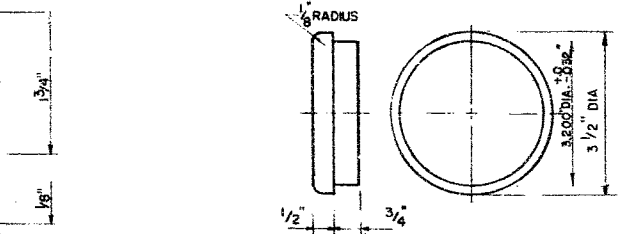
RAIL ASSEMBLY  
(INTS.)



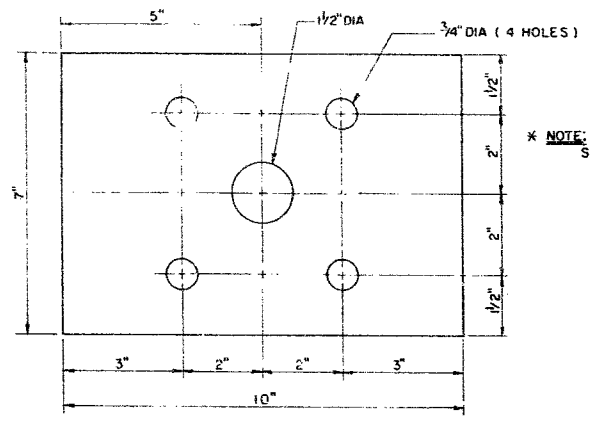
WASHER DETAIL  
(FULL SCALE)



SECTION A-A



RAIL CAP



TEMPLATE  
(1/2" THICK PLYWOOD)

TEMPLATE TO BE INCLUDED AS PART OF ANCHORAGE

GENERAL NOTES:

- (1) RAIL ELEMENTS SHALL BE STRUCTURAL TUBING SUPPLIED IN ACCORDANCE WITH A.S.T.M. A36.
- (2) STEEL IN POSTS AND RAIL CAPS SHALL BE CAST STEEL SUPPLIED IN ACCORDANCE WITH A.S.T.M. A27-60 GR. 65-35.
- (3) RAIL TUBING SHALL BE SUPPLIED WITH SPLICE IN LENGTHS OF 21'-0" (EXCLUDING SPLICE) EXCEPT AS NOTED.
- (4) POSTS, RAILS, WASHERS AND CAPS SHALL BE GALVANIZED IN ACCORDANCE WITH A.S.T.M. A123.
- (5) SET SCREWS SHALL BE CROMIUM PLATED TO .0002 IN. WITH A SPECIAL BRIGHT DIP FINISH TO STD. SPEC. Q Q-P-46A CLASS III. ALL GALVANIZING SHALL BE DONE AFTER FABRICATION.
- (6) ELECTRODES SHALL BE TO A LOW HYDROGEN SPECIFICATION E7015, E7016 OR E7018.
- (7) ANCHORAGE AND BOLTS SHALL BE RICHMOND TYPE D-GRI OR EQUAL, MODIFIED AS SHOWN. INSERTS AND ANCHOR BOLTS SHALL BE GALVANIZED IN ACCORDANCE WITH A.S.T.M. A153-GI. BOLTS SHALL BE GIVEN A LIBERAL COATING OF WHITE NON-STAINING GREASE.

ERECTION NOTES:

- (1) RAIL TUBING SHALL BE BENT TO FOLLOW CURVATURE OF ROAD.
- (2) RAIL POSTS SHALL BE SET PERPENDICULAR TO GRADE.
- (3) WHERE LAYOUT OF POSTS IS NOT SHOWN, POST LOCATION SHALL BE DETERMINED BY THE CONTRACTOR.
- (4) SET SCREWS SHALL NOT BE TIGHTENED ON POSTS ADJACENT TO EXPANSION JOINTS.
- (5) RAIL SHALL BE CUT AS REQUIRED WITH PIPE CUTTERS AND CUT SURFACE TREATED WITH ZINC RICH PAINT.
- (6) WHEN CONNECTING TO EXISTING RAIL, RAIL MUST BE MADE CONTINUOUS AND POST SPACING DETERMINED WITH REFERENCE TO EXISTING POST.

SUMMARY

ITEM	NO.	L.F.	LOCATION	APPROX. L.F. OF RAIL
POST INCLUDING SET SCREWS & ANCHORAGE INCLUDING BOLTS WASHERS AND TEMPLATE	26			
END CAPS (INCLUDING SET SCREWS)	8			
3 1/2" DIA RAIL (21'-0" LENGTHS)	20			
3 1/2" DIA RAIL WITHOUT SPLICE LENGTHS	4	24'-10"		

REVISIONS	DATE	BY	DESCRIPTION

\* NOTE: SCALE: 1/2" = 1' (EXCEPT WHERE OTHERWISE INDICATED)

DEPARTMENT OF HIGHWAYS ONTARIO  
BRIDGE DIVISION

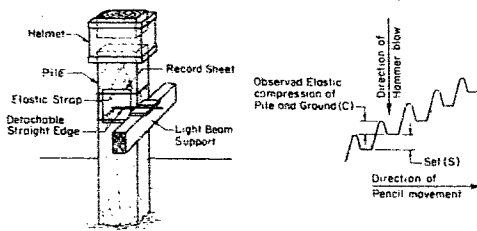
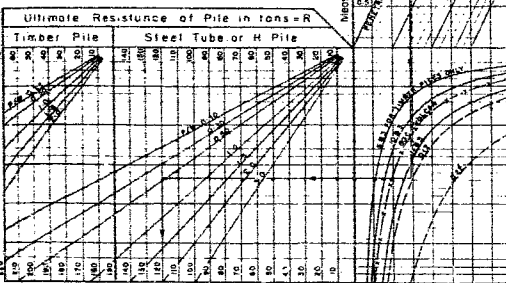
OGASIWI CREEK BRIDGE  
16.4 MILES NORTH OF HWY. N#101

KING'S HIGHWAY No. SEC. HWY. #651 DIST. No. 18  
DIST. OF ALGOMA  
TWP. 43 LOT CON.

=STANDARD STEEL PARAPET RAIL=

APPROVED: [Signature] SITE No. 38C-28 W.P. No. 22-65  
DESIGN: ADAPTED CHECK: [Signature] CONTRACT No. 67-63  
DRAWING: P.SCH. CHECK: [Signature] DRAWING No. 67-63  
DATE: MAR. 67

HAMMERS		
TYPE	WEIGHT (Wt.)	PURPOSE
9B	0.8	For Timber Pile
10B3	1.5	
11B3	2.5	For Steel Tube or H Piles
D12	1.38	
D22	2.4	
50C	2.5	

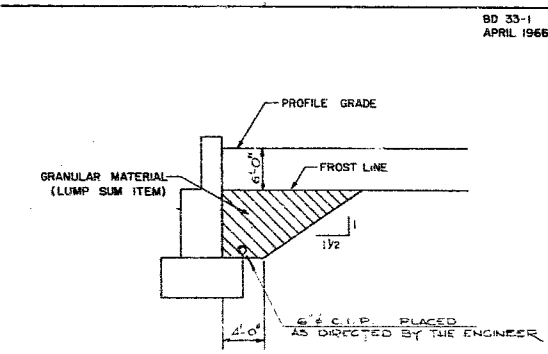
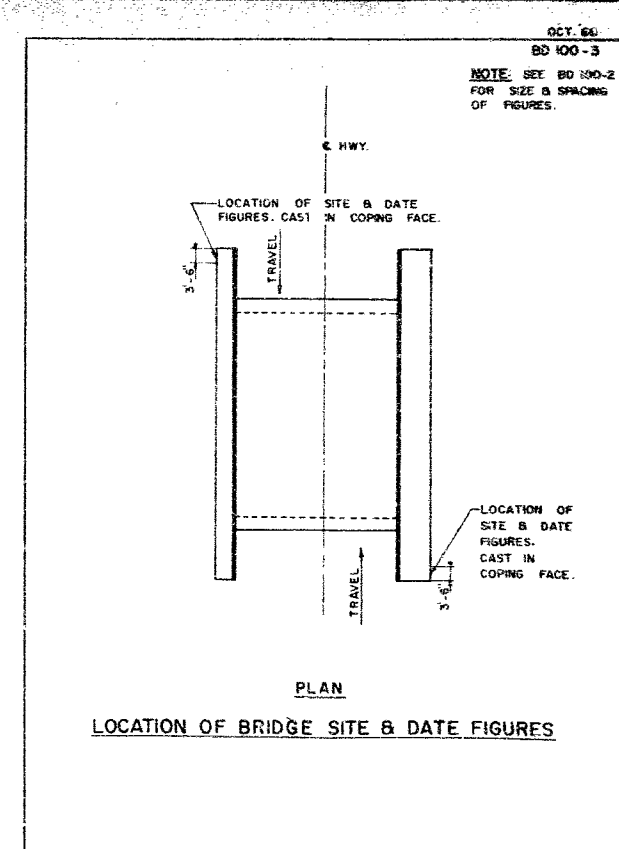
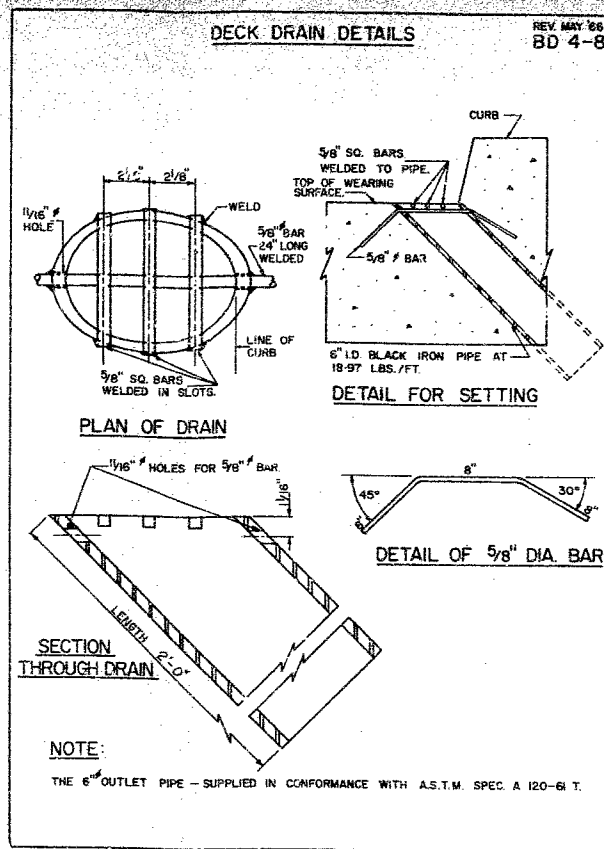


$R = \frac{2Wh}{5+C}$  tons (Hiley formula)  
 where  $R$  = Ultimate load in tons.  
 $W$  = Measured penetration of pile per blow of hammer in inches.  
 $C$  = Measured rebound of pile per blow of hammer in inches.  
 $Wh$  = Gross energy of hammer blow with a reduction due to the effect of single action or double action as against a perfect free fall, this reduction is included in plotting of the curves.  
 $n$  = Efficiency of blow =  $\frac{W+C}{W+P}$   
 where  $n$  = 0.32 for steel (These values of  $n$  have been found by experiment)  
 $n$  = 0.25 for timber  
 $P$  = Weight of pile + 0.25 ton for helmet.  
 $W$  = Weight of hammer in tons.  
 The P/W curves form the required reduction of total energy (Wh) of the hammer blow according to the ratio of P/W.  
 $L = R/O$  tons  
 where  $L$  = Working load on pile in tons  
 $O$  = Factor of safety  
 Use  $O = 3$  unless otherwise authorized by the Bridge Engineer

Example 1:  
 Observed measured rebound =  $C = 0.8$  in.  
 Observed measured set per blow =  $W = 0.33$  in.  
 12 in steel tube of 28 lb per ft. 30 ft. long plus helmet weighing 0.25 ton giving  $P = 0.67$  ton. Delmag D12 hammer,  $W = 1.38$  tons,  $P/W = 0.485$   
 Chart:  
 With  $C = 0.8$  in proceed horizontally to the right to cut line  $S = 0.33$  in and vertically down to cut curve D12 then horizontally to the left to cut  $P/W = 0.485$  and read ultimate load  $R = 120$  tons approximately.

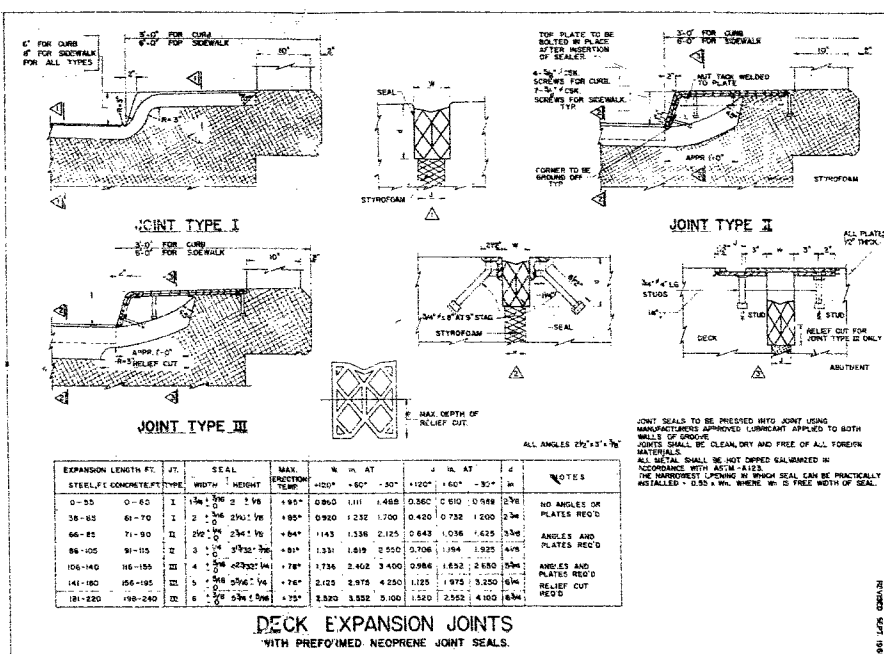
Example 2:  
 Working load on pile is 20 tons, pile is 12 in. tube, 40 ft. long, D/A Mäkelä-Terry hammer 10B3,  $W = 1.5$  ton,  $P = C 54 + 0.25 = 0.79$  ton,  $P/W = 0.525$ . Assume  $O = 3$ , then  $n = 0.25$ .  
 Chart:  
 With  $R = 60$  tons trace up to cut  $P/W = 0.525$  and horizontally to the right to cut curve 10B3 then vertically up. The range of reading will now be between  $C = 0$  in. and  $S = 0.72$  in. and  $C = 1.45$  in. and  $S = 0$  (refusal). A test pile must be driven of a length compatible with the soils branch recommendation (if any). The driving must continue until a pair of readings is obtained corresponding to a pair on the chart, the required pair being decided upon by the Bridge Engineer.

DEPARTMENT OF HIGHWAYS - ONTARIO  
**PILE DRIVING**  
**STEAM AND DIESEL HAMMERS**  
 APPROVED  
 Aug. 4/59  
 Date  
 Bridge Engineer



**DETAILS OF MINIMUM GRANULAR BACKFILL REQUIREMENT**  
 NOTE:  
 SECTION PERPENDICULAR TO ABUTMENT.  
 LATERAL LIMITS - INSIDE FACE TO INSIDE FACE OF WINGWALLS.

DIST. NO.	APPROX. FROST PENETRATION.
1 TO 7	4 FT.
8 TO 11	5 FT.
13, 14, 17 AND 18	6 FT.
16, 19 AND 20	7 FT.



REVISIONS

DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO  
 BRIDGE DIVISION

**OGASIVI CREEK BRIDGE**  
 16.4 MI. NORTH OF HWY. NO. 31

KING'S HIGHWAY No. 631  
 DIST. OF ALCOVA  
 TWP. 14S  
 LOT  
 CON.

STANDARD DETAILS

APPROVED: [Signature]  
 BRIDGE ENGINEER

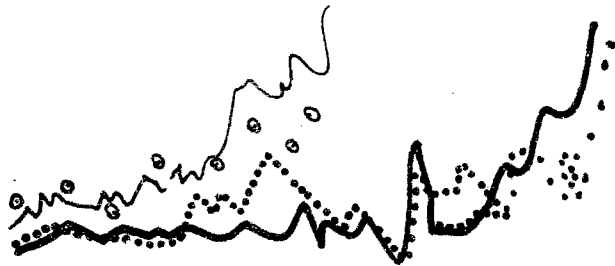
CONTRACT No. 67-63

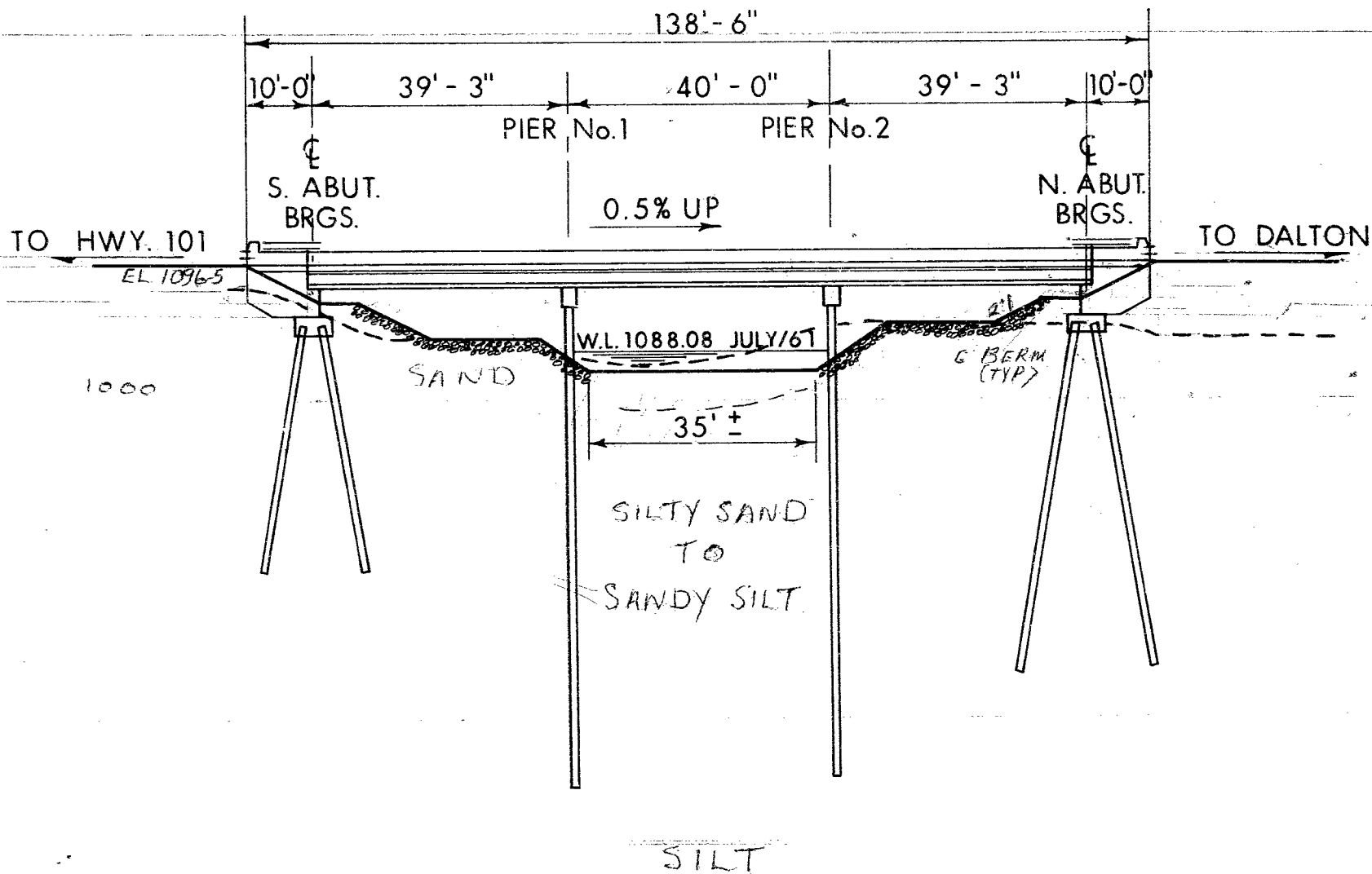
DRAWING No. D-5673-B

DATE NOV 1966 LOADING 115-20-44

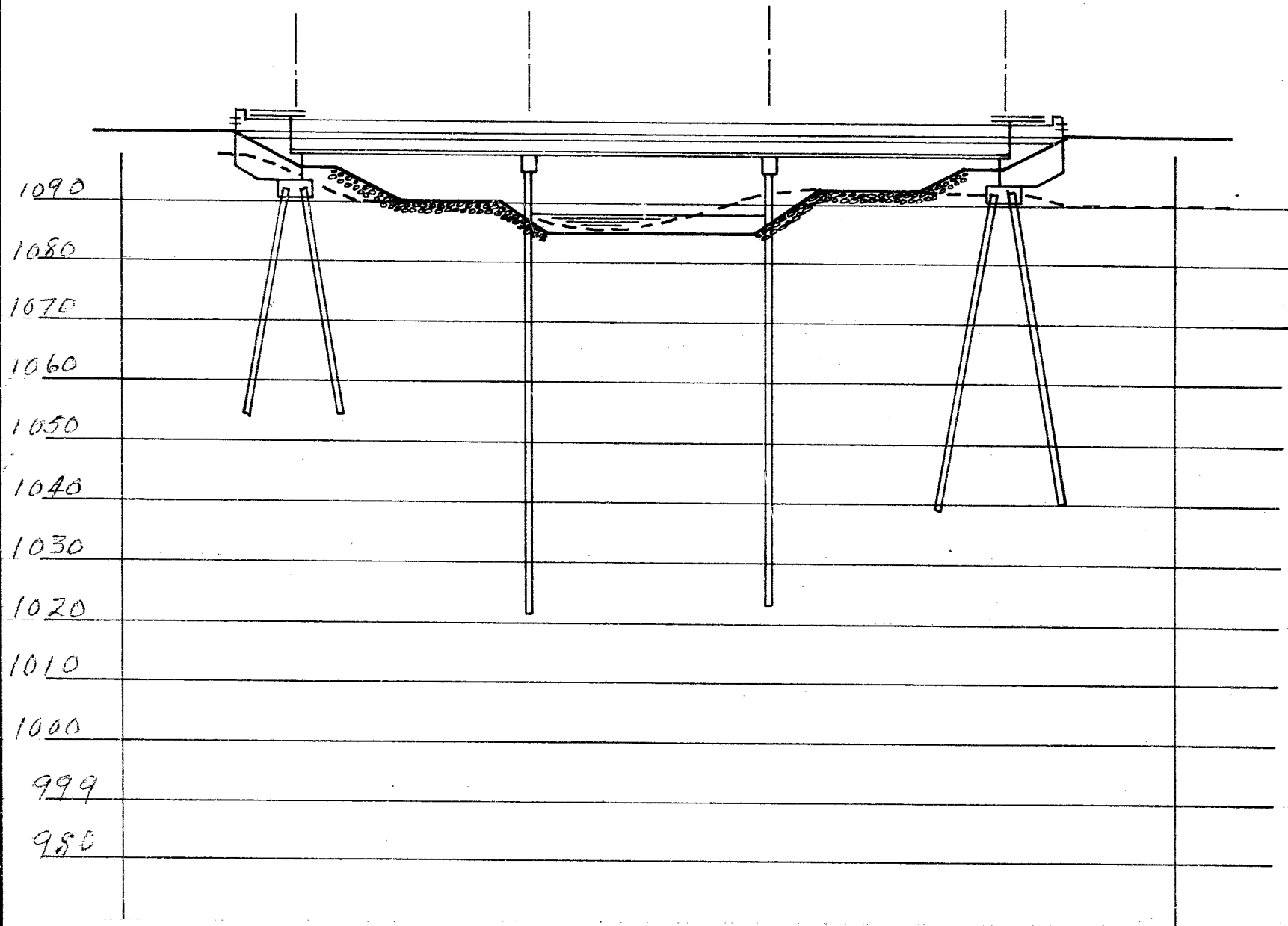
# BRIDGE CONSTRUCTION - PILE DRIVING RECORD

DISTRICT NO. 18 CONTRACT NO. 67-63 STRUCTURE 052nd St. Bridge  
CONTRACTOR W. J. ... DESIGN LOAD OF PILE 25 TONS  
HAMMER DETAILS: TYPE DE-445 D-12 WEIGHT 1.38 T HEIGHT OF FALL OR ENERGY  
TYPE OF ANVIL OR CAP STEEL WEIGHT OF ANVIL OR CAP 0.25 TONS  
PILE DETAILS 12 3/4" O.D. STEEL TUBE 33.38 lbs  
PILE NO. 2 LOCATION NORTH ABUTMENT 1/4 in. ft. DATE DRIVEN SEP. 27/67  
8 FIELD #2 4-6-30/67









CONTRACT NO: 67-63

OGASIW1 CREEK

WP: 22-65

(GEOCON/65)

