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DIST. 18 REGION

W.P. No. 277-85-02

CONT. No. 87-213

W. O. No.

STR. SITE No. 385-21

HWY. No. 556

LOCATION Little Garden River
Crossing #2

No of PAGES -

=====

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:



MIRZA

FILE No._____

DATE _____

REMARKS _____

This image shows a single sheet of white paper with horizontal blue or grey ruling lines. The lines are evenly spaced and run across the width of the page. There is no handwriting or other markings on the paper.

OVERSIZE DRAWING

memorandum



To: Mr. O. Ramakko,
Head, Structural Section,
NORTHWESTERN REGION - Thunder Bay

Date: 86 03 04

From: Engineering Materials Office,
Foundation Design Section,
Central Building, Room 315

Re: Foundation Investigation Report for
Little Garden River Crossing #2
Hwy. #556, District #18, Sault Ste. Marie
W. P. 277-85-02

The geotechnical consulting engineering firm C. Mirza Engineering Inc. was retained by this section to carry out a foundation investigation at the above-note site and provide recommendations for the design and construction of the proposed structure and associated fills.

During the progress of this project, this section reviewed the draft version of the report and comments on the technical content were made. The attached final report satisfactorily addresses our concerns.

In view of the fact that with depth, the subsoil becomes looser, a value of 200 kPa is given in the report for the bearing capacity at the S.L.S. Type II of abutment footings founded on natural ground. This is based on 25 mm settlements. If, however, a greater settlement can be tolerated for the proposed single span structure, then this value can be increased. This section should be consulted if a greater capacity is required.

A handwritten signature in dark ink, appearing to be "L. Politano", written over a horizontal line.

L. Politano,
Project Foundations Engineer

for
M. S. Devata,
Chief Foundations Engineer
(East)

LP:ma

cc: Report Distribution



C. MIRZA ENGINEERING INC.
GEOTECHNICAL SPECIALISTS & CONSULTING ENGINEERS

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FOUNDATION INVESTIGATION

REPORT

for

Little Garden River Bridge # 2 - Hwy. 556

Site No: 38S-21 W.P. 277-85-02

District 18, Sault Ste. Marie
Northwestern Region

Our File: 85-103

Consultant Agreement No: 4238-9085-136

1986 03 03

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GEOCRES # 41J00-044

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FOUNDATION INVESTIGATION

REPORT

for

Little Garden River Bridge # 2 - Hwy. 556

Site No: 38S-21 W.P. 277-85-02

District 18, Sault Ste. Marie

Northwestern Region

1.0 INTRODUCTION

The Foundation Design Section of the Ministry of Transportation and Communications has retained the services of C. Mirza Engineering Inc., under terms of Consultant Agreement No: 4238-9085-136, to carry out a subsoil investigation at the above captioned site. The terms of reference of the study were to report on the factual site conditions and to provide geotechnical engineering recommendations for the design of foundations for the proposed structure.

2.0 SITE AND GEOLOGY

The site is located on the extension of Secondary Highway 556 to Ranger Lake in Hughes Township, District of Algoma, approximately 100 km north-east of Sault Ste. Marie. A key map of the site is shown on Drawing 2778502-A appended.

The geology of this area has been mapped by the Ontario Geological Survey (Mineral Potential Map Series P.1513). Near the bridge site the terrain is shown as an ice-contact deposit area, comprising chiefly of glacio-fluvial deposits, with very low mineral potential. There are no major bedrock outcrops mapped for this area. The bedrocks of the area belong to the Abitibi Belt of the Superior Province of the Precambrian Shield and consist primarily of granite and granitic types of rock.

The terrain at the site and immediate vicinity is relatively flat. Till-ice contact drift "hills" are visible some 500 m to the northwest and to the southeast about 1 km away. Vegetation is mostly cedar with some pine and fir. Many trees along the river bank at the site are leaning into the river, indicating active erosion of the banks.

2.0 SITE AND GEOLOGY - continued:

The existing structure is a timber bridge. The river is about 10 m wide and flows sluggishly towards the west. At the time of investigation, the water in the river was 250 to 400 mm deep. The central portion of the stream was unfrozen.

The timber structure rests on sawn "railway tie size" lumber and is entirely constructed of sawn lumber. The clearance between the road surface at deck and the water at the time of investigation was just over one metre.

3.0 FIELD AND LABORATORY WORK

The field work was commenced on the late afternoon of 1986 02 05 and completed on the afternoon of 1986 02 07. It was originally intended to put down four boreholes to prove bedrock, assuming the bedrock occurred within a depth of 8 metres. Since no bedrock was encountered to a depth of 10 metres in the first borehole, it was decided to conduct a dynamic cone penetration test in the same borehole. Based on the findings of this first borehole, it was decided to change the drilling program to effect economies in the site investigation. Hence, one more sampled borehole was drilled diagonally across from the first borehole; and, in the other two corners of the proposed bridge abutment locations, only dynamic cone penetration tests were carried out. We regret that the MTC could not be contacted to confirm the alteration in the drilling program at the time the decision was taken to terminate the drilling program.

The location of the boreholes is shown on Drawing 2778502-A. The bore and cone holes were tied into the proposed re-alignemnt centreline chainage by distance measuring, and elevations were taken off the E-Plan supplied by the MTC.

In the boreholes, soil samples were obtained in the Standard Penetration Test, the N values being noted at each sampling depth. All recovered samples were placed in moisture proof containers and brought back to our Toronto Laboratories for further visual examinations, classifica-

3.0 FIELD AND LABORATORY WORK - continued:

tion according to the Unified Soil Classification System, and index property tests, such as moisture contents and grain size distribution. The results of the laboratory tests are given on the individual borehole log sheets as well as on Figures 1 to 3 in the Appendix.

4.0 SUBSURFACE CONDITIONS

4.1 General

The soil conditions across the site are fairly uniform. Beneath a surficial cover of peat there is compact to very dense deposit of sands and gravel extending to the final depth of the investigation which was a maximum of just over 11 metres in a borehole and just over 20 metres in a cone hole. No cohesive soils were encountered in either the bore or cone holes. The water table was found to correspond to the river level.

4.2 Peat

Under a cover of snow at the time of the investigation, the soil along the river banks at the proposed crossing location consists of a surficial cover of dark brown peat, mixed with some sand. The peat grades into the underlying sand and gravel deposit at a depth of 700 mm. Therefore the thickness of the peat deposit is about 700 mm. At the time of the investigation, this organic layer was completely frozen. In the unfrozen state, the consistency of the peat is likely to be very soft, since it is quite fibrous and shows no root mat.

4.3 Sand and Gravel

The surficial peat deposit is directly underlain by a 3 to 4 metre thick deposit of sand and gravel. From the drilling, cobbles are inferred to be present within this upper non-cohesive stratum. The sand is of reddish brown colour and the gravel is subangular to sub-rounded, indicating the origin of the sand to be glacio-fluvial, and of granitic parent materials.

The moisture content of this stratum was found to range between 12 and 22 per cent, the lower values being obtained in samples containing a larger percentage of gravel. Typical grain size distribution curves are

4.3 Sand and Gravel - continued:

given in Figure 1. The gravel content is just under 50 per cent.

The N values in this stratum ranged from lows of 13 to an isolated high of 72 blows/0.3 m, indicating the density of the stratum to be generally compact to dense, being occasionally very dense. The N values below 20 blows/0.3 m were measured just below the peat stratum, and it is suspected that both the peat and perhaps unbalanced hydrostatic heads in the hollow stem augering may have caused a boiling of the sand, leading to the lower N values. Below the first sample taken in this deposit, the auger stem was kept filled with water to prevent unbalanced hydrostatic uplift, and the N values were generally in the dense range (over 30 blows/0.3 m). The field experience shows that even such a dense deposit can boil under unbalanced hydrostatic heads.

4.4 Sand with Some Gravel

Below the upper sand and gravel deposit, at about elevation 348, a deep deposit of sand with some gravel was encountered, and was found to extend to the maximum depth of the borehole. The sand is of reddish brown colour generally, becoming greyish below about elevation 343. The sand particle sizes range from medium to coarse, and are generally subangular. The material is typical of glacial outwash deposits. From the augering, boulders and cobbles are inferred to be present in this deposit at random throughout its depth. At the location of Borehole 4, the augers began to tilt, indicating a boulder had been encountered at that depth (8.1 m). Hence the borehole was terminated at that depth.

This sand deposit is inferred to extend below the sampled depth of 12 metres in Borehole 1 on the basis of the dynamic cone penetration tests carried out both within Borehole 1 and at the location of Cone holes 2 and 3. Upon completion of the tests, no clay was found on any of the A rods used to drive the cone tip. However, in Borehole 4, a lens of fine grey silty sand was encountered between elevations 346 and 347. Such lenses are also inferred to be present throughout the deposit in a random fashion. The moisture content of the sand is about 15 per cent on average.

4.4 Sand with Some Gravel - continued:

Grain size distribution curves for the sand deposit are given in Figure 2 and for the silty sand lens in Figure 3. In Figure 3, the presence of occasional gravel has distorted the curve. However, the lens consists of essentially a fine sand matrix, which is non-cohesive.

The N values in the sand deposit ranged between 11 and 68 blow/0.3 m, indicating the deposit to be of compact to very dense relative density. However, some of the higher N values may represent cobbles and boulders which have been pushed aside in the sampling process. Thus, the overall denseness of the deposit is more likely to be compact. However, care is required in excavations to ensure the sand does not boil under unbalanced hydrostatic heads.

5.0 GROUNDWATER CONDITIONS

The groundwater conditions were observed in the open boreholes and were found to correspond to the river level prevailing at the time of the investigation, being generally within 500 mm of the ground level. or elev. 351.0 at Borehole 4 and 351.1 at Borehole 1.

Since the sandy deposits at the site are totally cohesionless, the sand was found to be entering the hollow stem augers when a depth of 1.5 to 2 m below grade was reached, due to unbalanced hydrostatic heads. Since the boreholes were located only 2 to 4 m from the edge of the river, the hydraulic gradient through the deposit is in the order of unity.

6.0 DISCUSSION AND RECOMMENDATIONS

6.1 General

It is proposed to replace the existing timber structure on Hwy. 556 over the Little Garden River with a new structure located some 25 metres downstream of the present location. The proposed structure will be a single span bridge with a span length of 22 m and a deck width of 9 m. The profile grade at the crossing will be at elevation 354±, giving a maximum height of approach fills of no more than 3 m above present grade.

This soil investigation has revealed the presence of 700 mm of a surficial peat cover followed by 3 to 4 metres of a generally sand and gravel deposit which in turn overlies a very deep, almost uniform deposit of sand with gravel of a generally compact nature. No cohesive soils were encountered to depths of 20 m below grade. Occasional cobbles and boulders are inferred from the soil drilling program. Also, isolated lenses of fine sand and silts were encountered at random.

The groundwater table was found to correspond to the river level, and is inferred to be controlled by the river level. Hydraulic gradients of near unity were observed in the upper dense sand and gravel deposit.

6.2 Foundation Design

The subsoil conditions are favourable for the support of abutments on spread footings located either in natural ground or on compacted fill. These two alternatives are discussed below.

From a general design viewpoint, it should be noted that the depth of frost penetration in this area is likely to be 2 metres and this should be used in designing frost protection earth cover above the base of footings.

Also, for calculation of sliding resistance, a value of the coefficient of friction between the granular soil (either natural or compacted fill) and concrete, of 0.4 should be used.

6.2.1. Abutments on Natural Ground

The density of the natural soils at this site has been shown to decrease with depth. In order to found the abutments on natural ground, it would be preferable to have them located within the upper compact to dense sand and gravel deposit. Between elevations 350 and 348, the N values in the sand and gravel stratum decrease from in the order of 70 to between 20 and 30 blows/0.3 m, indicating that it would be preferable to found the abutments at about elevation 350.0. Also, the higher the abutment, the more its load is spread with depth, which would help to minimize total settlements arising out of elastic type compression of the lower compact sand deposit.

Assuming a foundation elevation of 2 m below exterior grade (for frost protection purposes), a 3 metre width of footing and a founding elevation of 350.0, the footings may be designed for the following factored bearing capacities:

@ ULS	750 kPa
@ SLS Type II	200 kPa

The low SLS Type II value is based on settlement considerations, since the water table after construction would be at the base of the footing, and to limit settlements, the contact stress has to be reduced to limit total settlements to less than 25 mm. The factored SLS Type II bearing capacity will decrease as the footings are taken lower into the deposit below elevation 350.0.

6.2.2 Perched Abutments

Abutments founded within the natural deposit will require dewatering, as explained in 6.8.2. To avoid the need for dewatering, and to improve on the factored SLS Type II capacity, it may be preferable to found the abutments as perched footings within compacted granular fill (Figure 4). If rock fill is available, it may be used to form the core of the engineered fill. This core should then be properly "chinked" prior to placement of the Granular A pad which receives the footing itself. Such footings may be designed for the following factored capacities:

@ ULS	900 kPa
@ SLS Type II	300 kPa

6.3 Settlements

The abutment footings, whether founded in the natural soil or as perched units within compacted fill will undergo only elastic type settlements upon application of load. Such settlements are likely to be minor, unless the option of locating footings deeper than elevation 350 is chosen, in which case the factored capacity for SLS Type II would need to be reduced at the rate of 25 kPa for every 0.5 m depth below elevation 350 to a limit of elevation 348, from the value given in 6.2.2 above.

The approach fills are only about 3 m high, and settlements will again be of an elastic nature and minor (<25 mm). Refer to 6.4 below.

6.4 Approach Fills

The approach fills should be constructed from non-cohesive soils for that portion which will be submerged below the prevailing groundwater level. In order to minimize settlements, the organic soils below the plan limits of the approaches should be excavated and disposed of.

6.5 Earth Pressures

Backfill should consist of granular materials. The existing sand and gravel deposit is similar to an MTC Granular B (see Figure 1) and materials excavated from this stratum may be used as backfill to the abutment and wingwalls, in accordance with MTC Standard Special #121 (83-10). Computations of earth pressures should be in accordance with Section 6.6.1.2 of the O.H.B.D. Code. For design purposes, the following physical properties of the Granular backfill may be used:

$$\phi = 30^{\circ} \quad \text{Unit Weight} = 21.2 \text{ kN/m}^3$$

6.6 Stability

Due to the absence of any cohesive deposits within a depth of 20 m below present grade, and the compact to dense nature of the overburden, no stability problems are anticipated for the 3 m high approaches. See 6.4 above regarding excavation and removal of the upper organic soils. Although stability is not a problem, erosion is. Refer to 6.7 below regarding erosion problems and suggested control.

6.7 Erosion Control

Field observations show active erosion of the stream banks, as evidenced by leaning trees along the river bank in the vicinity of the existing and proposed bridge sites. Most of these trees are cedar, which are shallow rooted species. The soil is totally cohesionless and hence greatly susceptible to erosion, particularly at high stream flow velocities, i.e. during spring runoff, and after heavy rains.

Therefore, it is recommended that the design incorporate a stream bank erosion protection scheme for the full width of the abutment and extending to either side as dictated by the hydraulic considerations of the stream regime.

If random rip-rap is used, the sandier portion of the sand and gravel deposit will be eroded away, resulting in future undermining. Therefore, random rip-rap should be underlain by a suitable filter constructed either of granular materials or geotextile. The placement of geotextile filters, especially under water is difficult to control, and it may prove advisable to use a graded granular filter instead. This could consist of Granular A of minimum 300 mm thickness followed by 150mm minimum size rip-rap, covered by the maximum size of armour stone considered appropriate for the stream flow regime and hydraulics.

6.8 Construction Considerations

6.8.1 Stripping

The upper surficial peat cover should be stripped completely from below the plan limits of the approach fill and the abutment locations. For estimating purposes, assume 750 mm of peat and contaminated upper sands and gravel. The excavated material may be used as topsoil dressing on finished approach fill slopes or disposed of outside the right of way.

6.8.2 Dewatering

If dewatering is required to found the abutments within the naturally occurring upper dense sand and gravel deposit (see Section 6.2.1 of this

6.8.2 Dewatering - continued:

report), either heavy interlocking sheet piling or well point dewatering may be used.

If sheet piling is used, the design should allow for the lateral extension of the sheet pile wall beyond the abutment wing wall limits to prevent seepage from the sides. Allow for a minimum extension of at least 3 metres beyond the limit of footing excavation.

For well point dewatering, well points should be placed no further than 1.0 m apart, and should extend to a depth of at least 1.0 m below the base of the abutment footing. Since the soil is dense, the well points may have to be jettied in to reach their design depths. If boulders or cobbles obstruct the advancement of any well point, then additional well points should be installed adjacent to the obstructed one. Due to the low heads involved, vacuum methods are not necessary. However, the pump may have to be located at a sufficient depth to induce flow into the header pipes.

The upper sand and gravel is quite permeable as noted earlier. Drainage is possible by gravity. However, the abutment excavations will be located quite close to the river bank, and there is danger of breaching the natural dyke which would be formed between the river bank and the abutment excavation with such a scheme.

6.8.3 Excavations

In dewatered soils, temporary excavations should be at 1:1 and should conform to the requirements of the Ontario Health and Safety Act.


Excavations below the prevailing water table will be unstable, as the soil has no cohesion. The sand will simply "flow" under the influence of hydrostatic gradients, causing undermining of any overlying strata.

...../11

7.0 ACKNOWLEDGEMENTS

The field work was carried out using drilling equipment rented from Master Soil Investigations Limited, mobilizing out of their Sudbury location. The drilling equipment was a bombardier mounted hollow stem augering unit. Representatives of Terraprobe Limited, who were present near this bridge site for a companion investigation helped with the bulldozing of snow from the site prior to the commencement of the field work for this project. The field work was supervized by Mr. Bruce Polan of this Firm working under the direct supervision of the undersigned.

Respectfully submitted
C. MIRZA ENGINEERING INC.


Cam Mirza, P.Eng.
Principal



APPENDIX

Explanation of Terms Used in Report

Office Record of Boreholes 1 and 4 and Cone Tests 2 and 3
Figures 1 to 4

Drawing 2778502-A



EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

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

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

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

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

f: failure

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

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



Ministry of
Transportation and
Communications
Ontario

RECORD OF BOREHOLE No 1

METRIC

W P 277-85-02

LOCATION Station 12 + 163 o/s 4.6 Rt. @ Hwy 556, Line 'C'

ORIGINATED BY BP

DIST 18 HWY 556

BOREHOLE TYPE Hollow Stem Auger and Dynamic Cone Penetration

COMPILED BY BP

DATUM Geodetic

DATE 1986 02 13

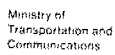
CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100						
								SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE						
351.6	Ground Surface									10	20	30		GR SA SI CL
0.0	PEAT		1	ss	2		351							W.L on 1986 02 16
350.9	Dk. Brown Trace of Sand													
0.7	SAND & GRAVEL Brown Compact to V. Dense occ. Cobbles		2	ss	13		350			○				46 53 (1)
			3	ss	72						○			
			4	ss	46		349							
			5	ss	27						○			43 55 (2)
347.9							348							
3.7	SAND with some Gravel Brown Medium to Coarse Compact occ. Cobbles & Boulders		6	ss	19					○				21 77 (2)
			7	ss	11		347							
							346							
			8	ss	13		345							
343.0			9	ss	20		344							7 91 (2)
8.6	Grey Dense occ. lens of fine Sand						343							
			10	ss	49		342			○				27 68 (5)
							341							
340.5	End of Borehole. Start of Dynamic Cone Penetration Test		11	ss	25					○				
11.1							340							
							339							
							338							
							337							

Continued

+3, x5 : Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE



METRIC

W P 277-85-02 LOCATION Station 12+163 o/s 4.6 Rt. & Hwy 556, Line 'C' ORIGINATED BY BP
DIST 18 HWY 556 BOREHOLE TYPE Hollow Stem Auger and Dynamic Cone Penetration COMPILED BY BP
DATUM Geodetic DATE 1986.02.13 CHECKED BY CM

[illegible]

+3, x5 : Numbers refer to Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

METRIC

W P 277-85-02 LOCATION Station 12+163 o/s 4.4 Lt. @ Hwy 556, Line 'C' ORIGINATED BY BP
DIST 18 HWY 556 BOREHOLE TYPE Dynamic Cone Penetration Resistance Test COMPILED BY SQA
DATUM Geodetic DATE 1986 02 14 CHECKED BY CM

[illegible]

Continued

+3, x5: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 2 Contd.										METRIC				
W P 277-85-02		LOCATION Station 12+163 o/s 4.4 Lt. @ Hwy 556, Line 'C'				ORIGINATED BY BP								
DIST 18 HWY 556		BOREHOLE TYPE Dynamic Cone Penetration Resistance Test				COMPILED BY SQA								
DATUM Geodetic		DATE 1986 02 14				CHECKED BY CM								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT	PLOT	NUMBER	TYPE			'N' VALUES	20					
336.6								SHEAR STRENGTH						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE		WATER CONTENT (%)				
15.0														
331.1								Hammer Bouncing						
20.5	End of Cone Penetration Test Practical Refusal													

+3, x5: Numbers refer to Sensitivity



RECORD OF BOREHOLE No 3

METRIC

W P 277-85-02 LOCATION Station 12+139 o/s 6.8 Rt. & Hwy 556, Line 'C' ORIGINATED BY BP
DIST 18 HWY 556 BOREHOLE TYPE Dynamic Cone Penetration Resistance Test COMPILED BY BP
DATUM Geodetic DATE 1986 02 14 CHECKED BY CM

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE						
351.6 0.0	Ground Surface									
350.9 0.7	Probable PEAT									
348.2 3.4	Probable SAND & GRAVEL									
336.6 15.0	Probable SAND with some Gravel									

OFFICE REPORT ON SOIL EXPLORATION

End of
Cone Penetration Test

+³, x⁵: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 4

METRIC

W P 277-85-02 LOCATION Station 12+141 o/s 4.4 Lt. @ Hwy 556, Line 'C' ORIGINATED BY BP
 DIST 18 HWY 556 BOREHOLE TYPE Hollow Stem Auger COMPILED BY BP
 DATUM Geodetic DATE 1986 02 14 CHECKED BY CM

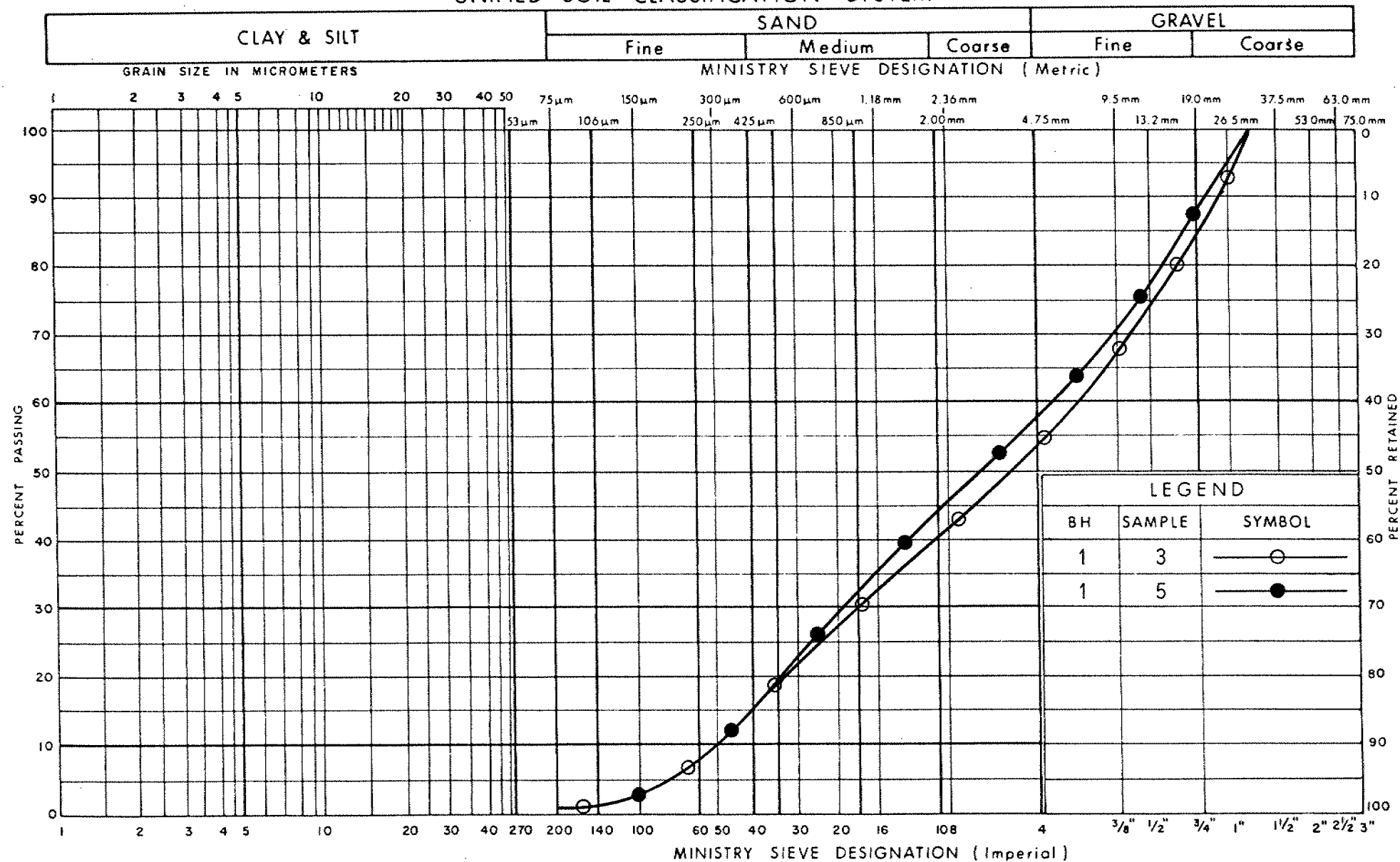
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					NATURAL MOISTURE CONTENT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	W _p	W	W _L		
351.3	Ground Surface															GR SA SI CL
0.0	PEAT		1	SS	2											W.L. on
350.6	Dk. Brown															1986 02 16
0.7	Trace of Sand															
	SAND & GRAVEL		2	SS	17								○			
	Brown															
	Compact to Dense		3	SS	45											
	occ. Cobbles															
			4	SS	42									○		
348.3																
3.0			5	SS	19											18 81 (1)
	SAND with some															
	Gravel		6	SS	15											
	Fine Sand lens		7	SS	68								○			18 65 07 10
	Brown															
	Medium to Coarse															
	Compact to V. Dense		8	SS	33											
	occ.															
	Cobbles & Boulders															
343.2			9	SS	63								○			
8.1	End of Borehole															

+3, x5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

UNIFIED SOIL CLASSIFICATION SYSTEM



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GRAIN SIZE DISTRIBUTION

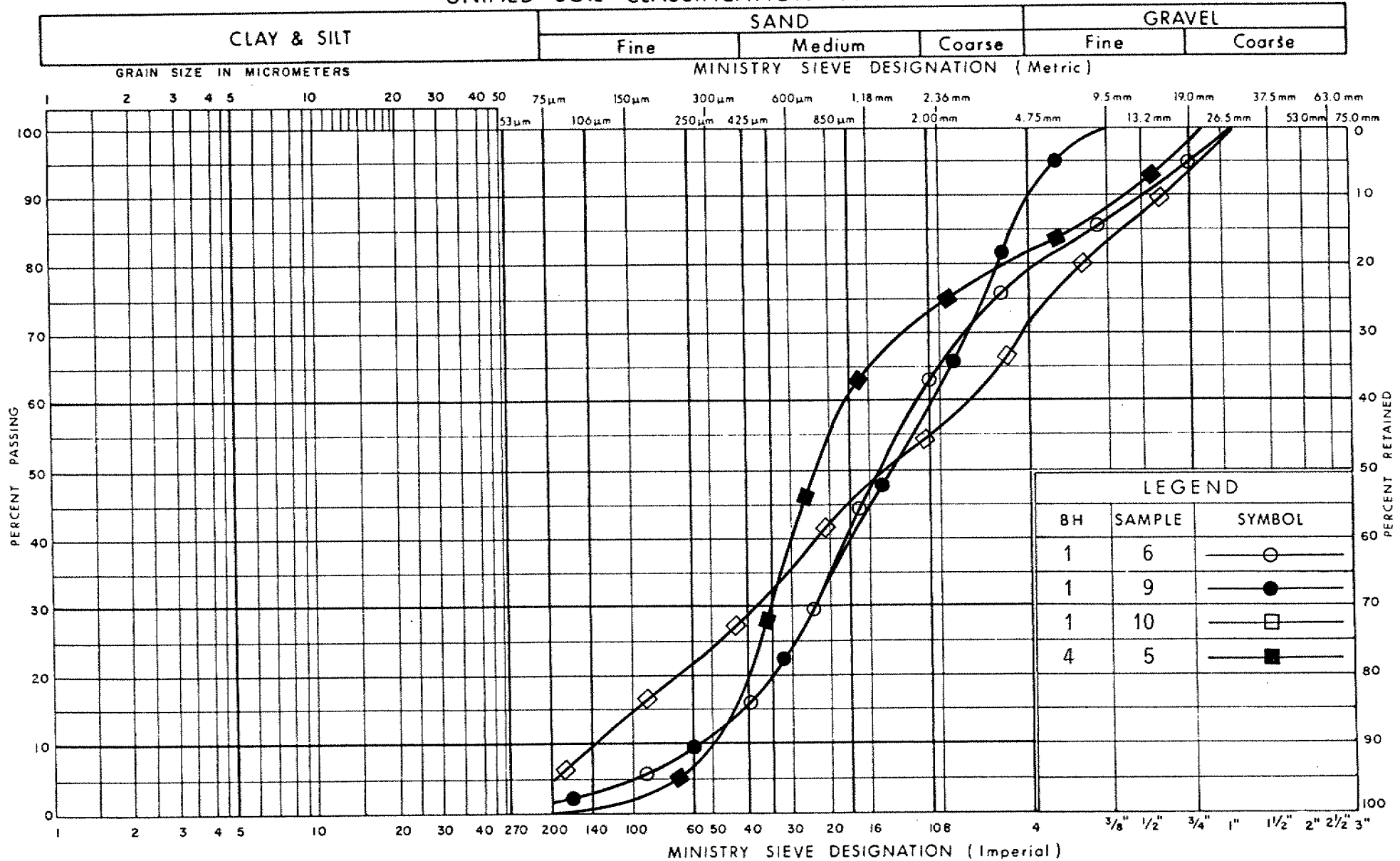
SAND & GRAVEL

FIG No 1

W P 277-85-02

SITE Sault Ste. Marie

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation and
Communications

Ontario

GRAIN SIZE DISTRIBUTION

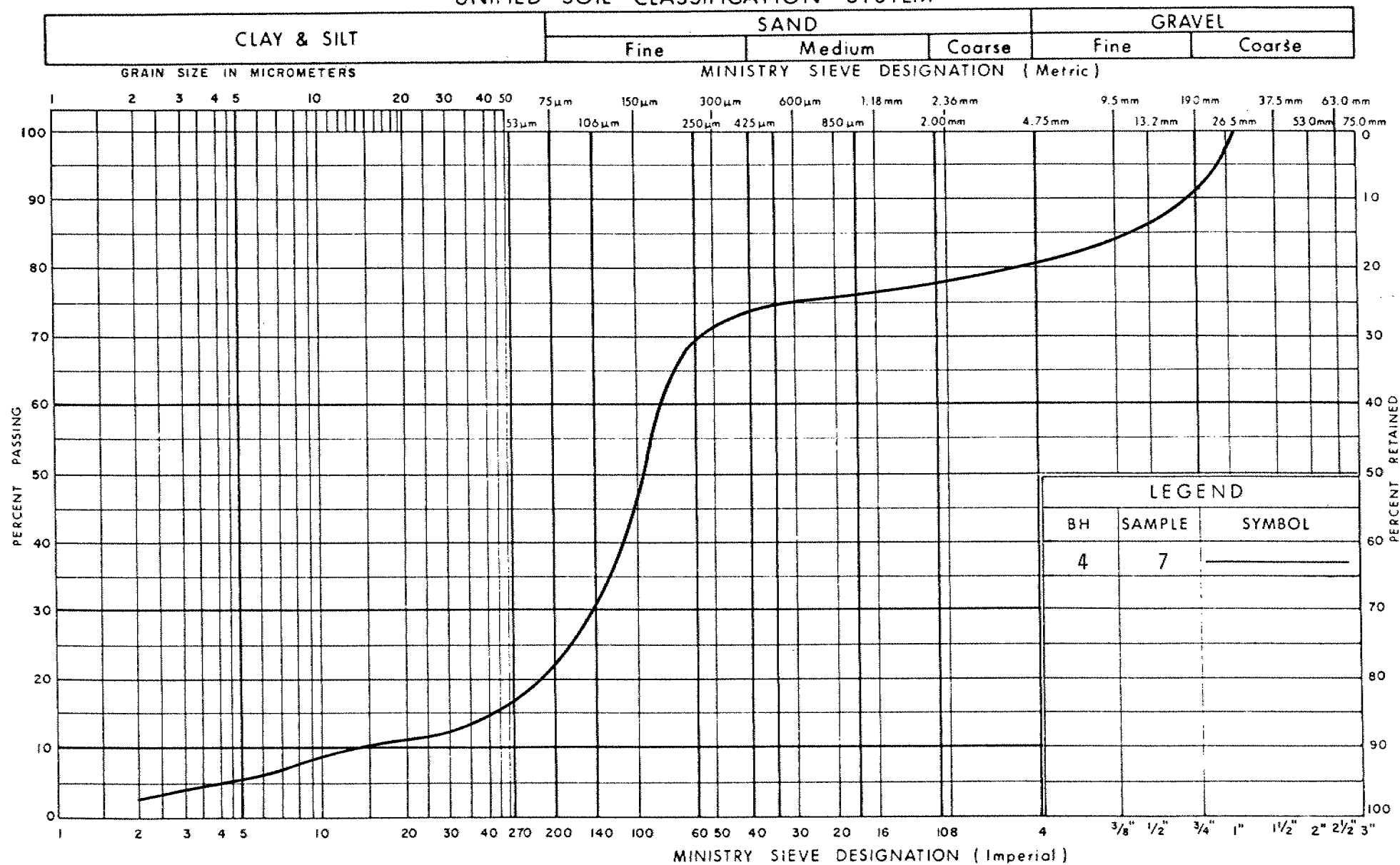
Medium SAND
with Gravel

FIG No 2

W P 277-85-02

SITE Sault Ste. Marie

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation and
Communications

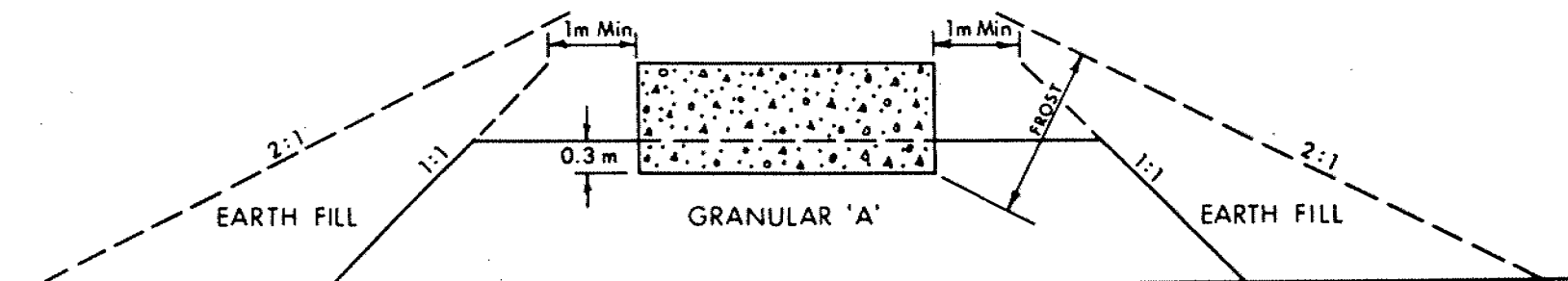
GRAIN SIZE DISTRIBUTION

Fine SAND
with Gravel, and a trace of Silt & Clay

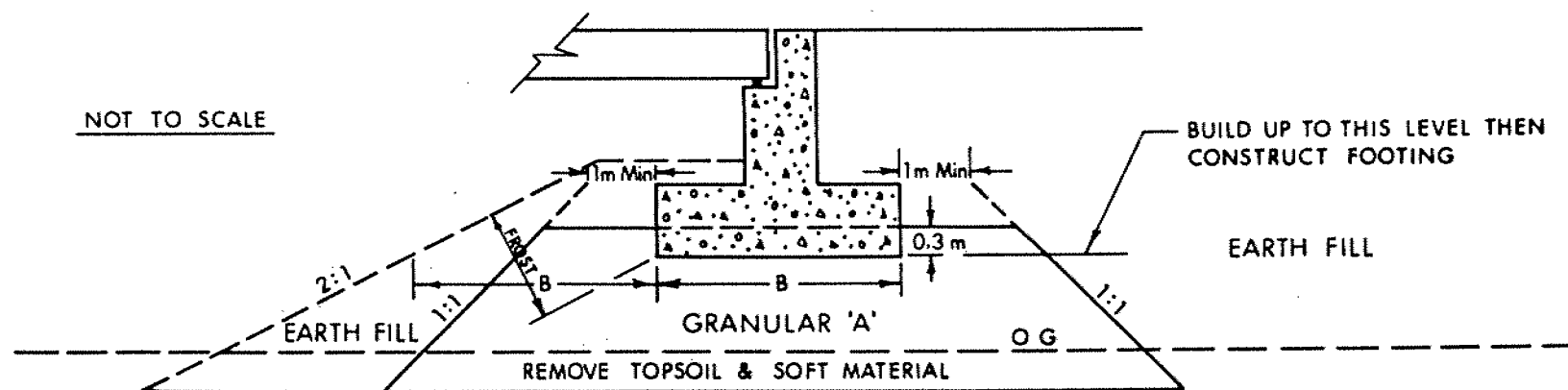
FIG No 3

W P 277-85-02

SITE Sault Ste. Marie



X SECTION



LONGITUDINAL SECTION

NOTES:

- 1 - REMOVE TOPSOIL &/OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' & EARTH FILL.
- 2 - PLACE GRANULAR 'A' & EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M T C STANDARDS.
- 3 - CONSTRUCT CONCRETE FOOTING.
- 4 - PLACE REMAINDER OF GRANULAR 'A' & EARTH FILL AS REQUIRED.



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Communications

ABUTMENT ON COMPACTED FILL
SHOWING GRANULAR 'A' CORE

FIG No 4

W P 277-85-02

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES + METRES

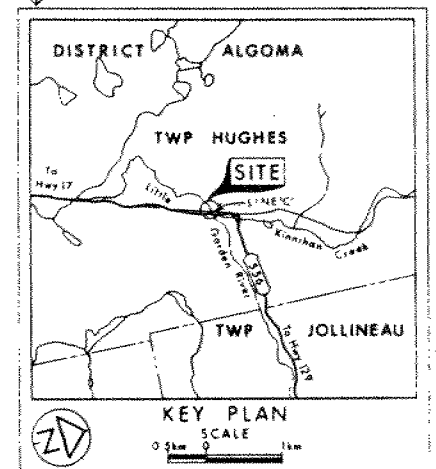
CONT No
WP No 277-85-02



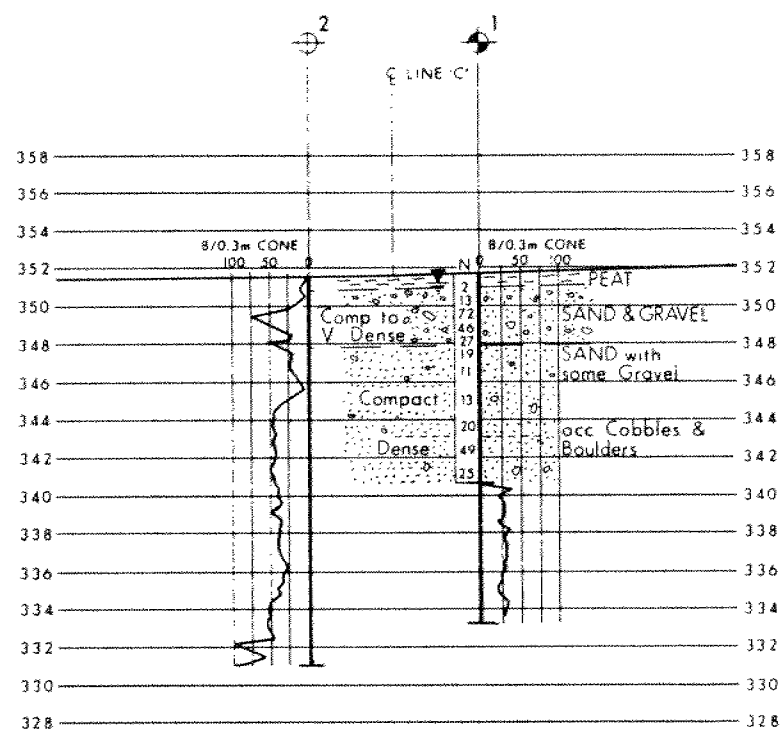
LITTLE GARDEN RIVER BRIDGE
(CROSSING No 2)
BORE HOLE LOCATIONS & SOIL STRATA

SHEET

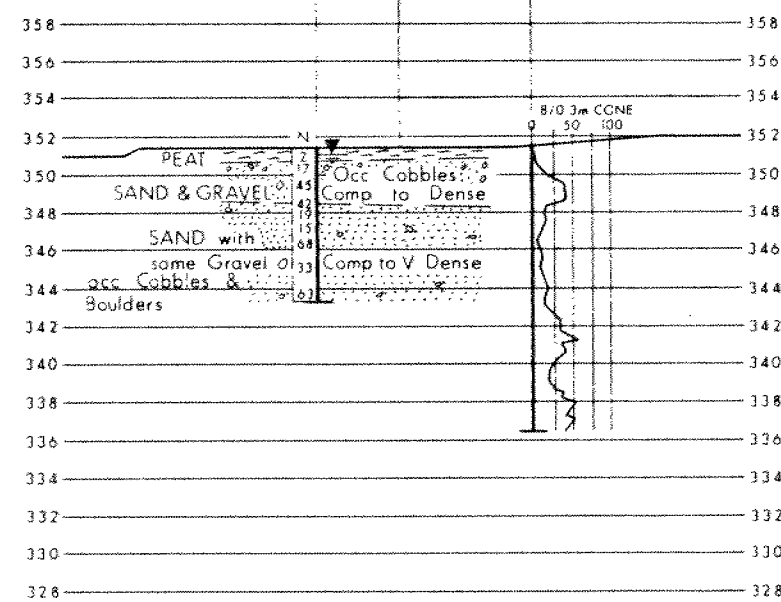
C. MIRZA ENGINEERING INC.
GEOTECHNICAL SPECIALISTS



BM 351.874
N & W in Root 0.20 Spruce
35.4 Rt 12+127.4
GEODETIC DATUM

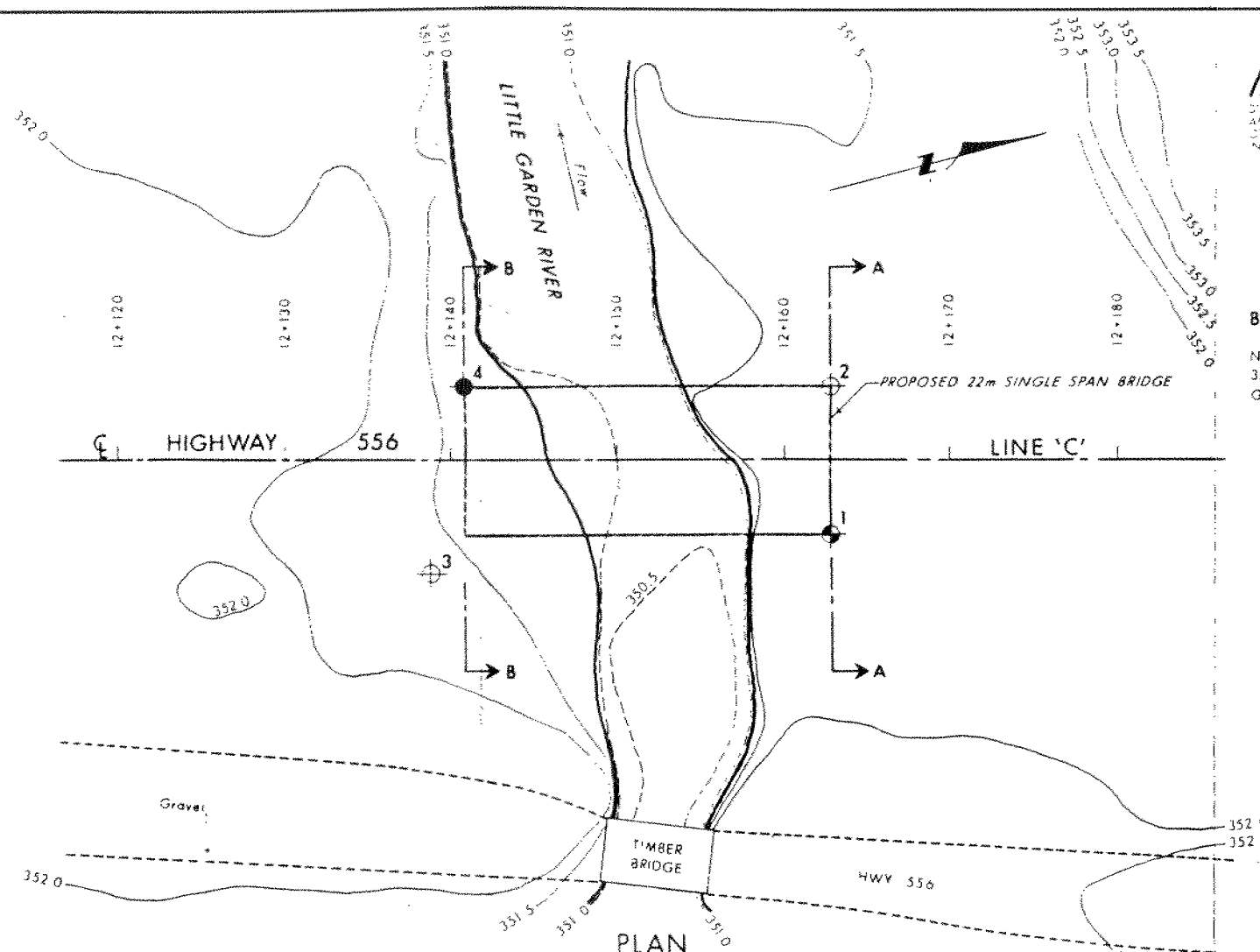


A-A

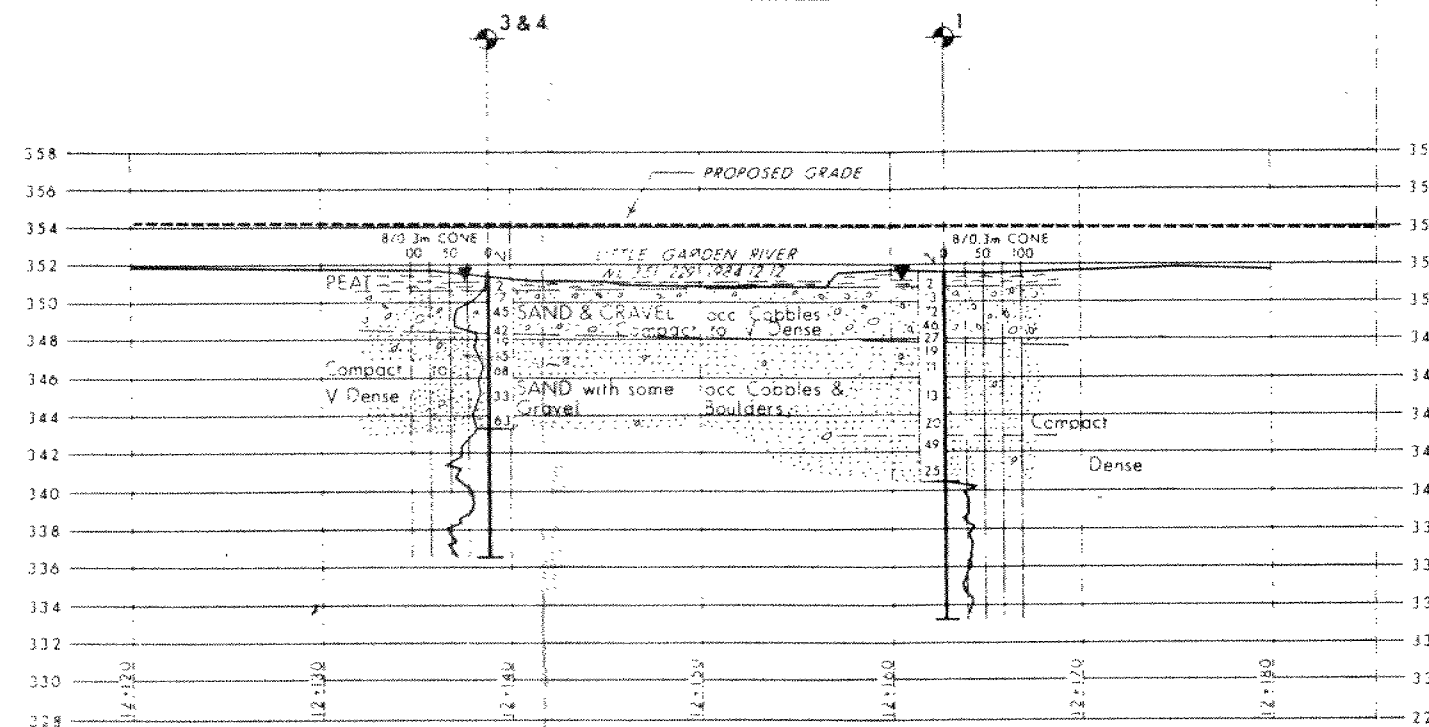
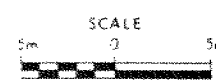


B-B

SECTIONS



PLAN



PROFILE OF LINE 'C'



LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊕ Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- ⬇ W.L. at time of investigation 1986-02

No	ELEVATION	STATION	OFFSET
1	351.6	12+163	4.6 Rt
2	351.6	12+163	4.4 Lt
3	351.6	12+139	6.8 Rt
4	351.3	12+141	4.4 Lt

NOTE

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

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office. Downstream information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 150.

DATE	BY	DESCRIPTION
1985-02-15	SAULT	SAULT
1985-02-15	SAULT	SAULT
1985-02-15	SAULT	SAULT

Geocres No 413-44

HWY No 556 DIST 18 SAULT S.M.
SUBMD BP CHECKED CM DATE 1985-02-15 SITE 385-27
DRAWN SGA CHECKED CM DATE 1985-02-15 DWG 27-8502-A

REF No E-8053-1 1985-01

RECORD OF BOREHOLE No 11

METRIC

W P 277-85-02 LOCATION Sta. 12 + 143 O/S 4.5 RT Q Hwy 556 Line 'C' ORIGINATED BY LP/MJ
DIST 18 HWY 556 BOREHOLE TYPE Hollow stem Auger COMPILED BY LP
DATUM Geodetic DATE 1987 01 13 CHECKED BY *lp*

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT Wp	NATURAL MOISTURE CONTENT W	LIQUID LIMIT WL	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH										WATER CONTENT (%)
								20 40 60 80 100										
351.4	Ground Surface																	
0.0	Peat		1	SS	0	*1	351											
350.6	Trace, sand, gravel																	
0.8	Sand, some gravel, trace silt Occasional cobbles.		2	SS	11			350										
			3	SS	35													
			4	SS	22				349									
			5	SS	37													
	Compact to Dense	6	SS	16	*2	348												
347.6																		
3.8	Fine to medium Sand, trace silt Compact	7	SS	17	*3	347												
		8	SS	12														
								346										
345.3	End of Borehole				*4													
6.1	Notes						345											
	<p>*1 After augering to Elev. 348.4, sand came up augers to Elev. 349.2±. Hole washed back down to Elev. 348.4. 'N' value may not be representative.</p> <p>*2 After augering to Elev. 347.6, sand came up augers to Elev. 248.2±. Hole washed back down to Elev. 347.6. 'N' value may not be representative.</p> <p>*3 After augering to Elev. 346.8±, sand came up augers to Elev. 347.8±. Hole washed back down to Elev. 346.8. 'N' value may not be representative.</p> <p>*4 After augering to Elev. 345.3, sand came up augers to Elev. 346.8±.</p> <p>Borehole was terminated.</p>																	

+3, x5 : Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 12

METRIC

W P 277-85-02 LOCATION Sta. 12 + 143 4.5 LT Q Hwy. 556 Line 'C' ORIGINATED BY LP/MJ
 DIST 18 HWY 556 BOREHOLE TYPE N-Casing, Washboring, Cone Penetration Test COMPILED BY LP
 DATUM Geodetic DATE 1987 01 13 to 16 CHECKED BY LP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100						
351.4	Ground Surface													
0.0	Peat (probable)						351							
350.5							350							
0.9	Sand and Gravel trace silt						349							
	Occ. cobbles		1	SS	31		349							48 45 7 0
	Dense		2	SS	37		348							
	Loose		3	SS	9		348							
			4	SS	9		347							
			5	SS	28		346							36 56 (8)
345.9	Fine to Med. Sand trace silt						345							
5.5			6	SS	34		344							
	Compact to Dense		7	WS	-		343							
344.1							342							
7.3	Sand with gravel, trace silt		8	SS	35		341							
			9	SS	39		340							
	Occ. cobbles		9A	SS	33		339							
							338							
			10	SS	31		337							
336.8	Dense													
14.6	CONTINUED ON SHEET	2												

+3, x5: Numbers refer to
Sensitivity

20
15-5 (%) STRAIN AT FAILURE
10

METRIC

W P 277-85-02 LOCATION Sta. 12 - 143 O/S 4.5 Lt C Hwy. 556 Line 'C' ORIGINATED BY LP/MJ
DIST 18 HWY 556 BOREHOLE TYPE N-Casing, Washboring, Cone Penettation Test COMPILED BY LP
DATUM Geodetic DATE 1987 01 13/16 CHECKED BY LP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100										SHEAR STRENGTH			WATER CONTENT (%)
																		○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE			
336.8	Continued																				
14.6	Sand with gravel Trace silt Occ. cobbles		11	SS	*																
			12	SS	30																
333.3	Dense																				
18.1	End of Borehole Refusal to casing advancement and tricone * Spoon bouncing, probable cobble or boulder																				

+³, x⁵ : Numbers refer to Sensitivity

15 ϕ 5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 13

METRIC

W P 277-85-02 LOCATION Sta. 12 + 159 at Q Hwy. 556 ORIGINATED BY MJ
DIST 18 HWY 556 BOREHOLE TYPE Hollow stem Augers, N & B Casing, Washboring, cone COMPILED BY LP
DATUM Geodetic DATE 1987 01 16 and 17 CHECKED BY [Signature]

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100				
351.3	Ground Surface															GR SA SI CL
0.0	Peat with sand		1	SS	4		351									
350.6																
0.7	Sand and gravel, trace silt		2	SS	19		350									50 45 (5)
			3	SS	35		349									53 43 (4)
	Occ. cobbles		4	SS	36		348									55 40 (5)
			5	SS	10		347									
			6	SS	15		346									
							345									
							344									
			7	SS	23		343									
							342									
							341									
	Compact to Dense		8	SS	77	*	340									
338.8							399									
12.5	Fine to Med. sand Some silt, trace gravel						338									
	Occ. cobbles		9	SS	5	**	337									
336.2																

+3, x5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 13 Cont

METRIC

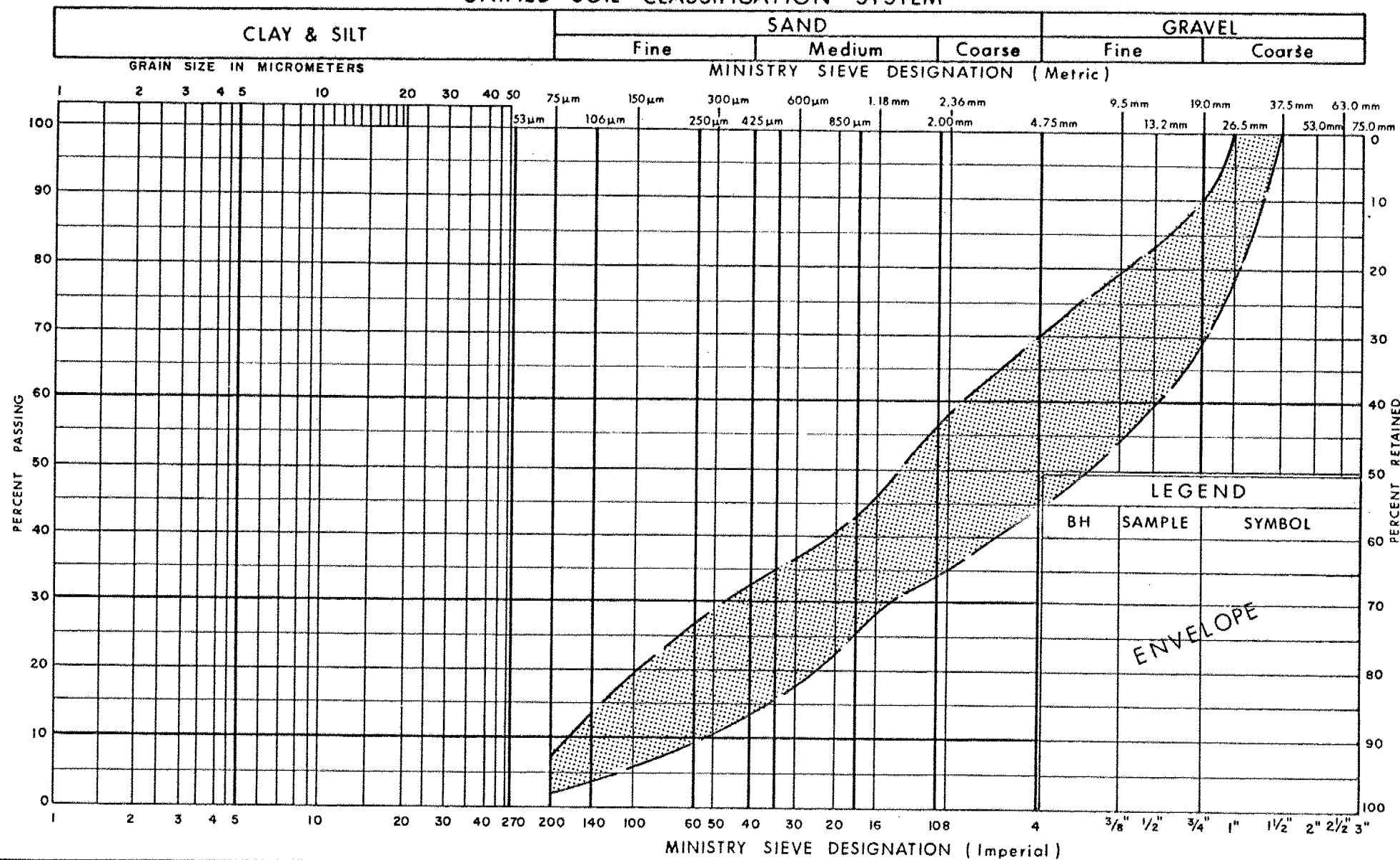
W P 277-85-02 LOCATION Sta. 12 + 159 at Q Hwy. 556 Line 'C' ORIGINATED BY MJ
DIST 18 HWY 556 BOREHOLE TYPE Hollow Stem Augers, N & B Casing, Washboring, Cone COMPILED BY LP
DATUM Geodetic DATE 1987 01 16 and 17 Test CHECKED BY h

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE - PLOT					UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100		
336.2	CONTINUED													GR SA SI CL
15.1	Fine to Med. Sand Some silt, trace gravel						336							
	Occ. cobbles						335							
	Dense						334							
336.6	End of Borehole		10	SS	38									
17.7							333							
							332							
331.2														
20.1	End of Cone Test Refusal to Cone													
	* Spoon bouncing probable cobble													
	** Sample probably disturbed due to unbalanced hydro- static pressure.													

+3, x5 : Numbers refer to
Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10

UNIFIED SOIL CLASSIFICATION SYSTEM



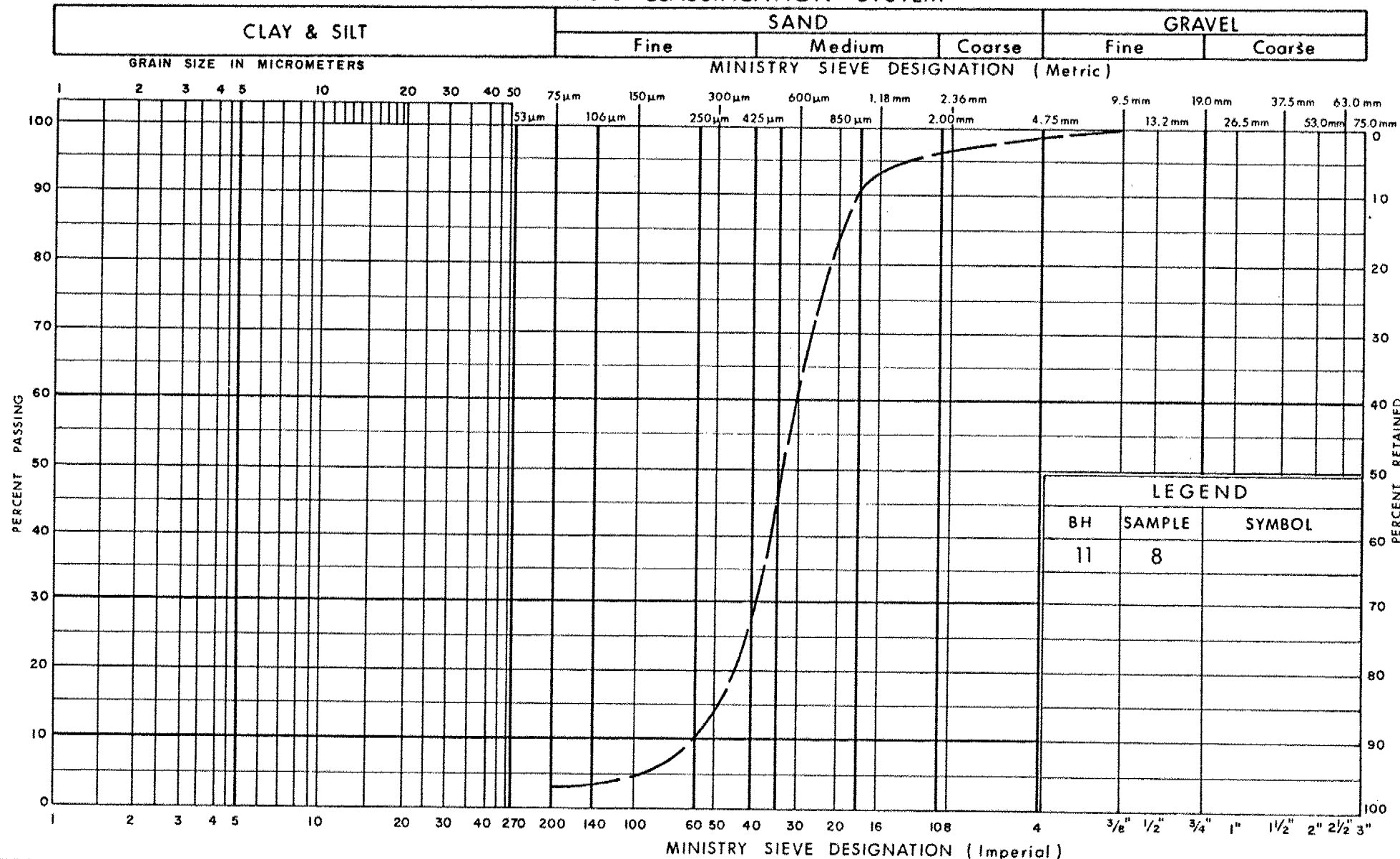
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Communications

GRAIN SIZE DISTRIBUTION
SAND SOME/AND GRAVEL, TRACE SILT

FIG No 4

W P 277-85-02

UNIFIED SOIL CLASSIFICATION SYSTEM

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Communications

GRAIN SIZE DISTRIBUTION
Fine - Med SAND

FIG No 5

W P 277-85-02

RECORD OF BOREHOLE No 11

METRIC

W P 277-85-02 LOCATION Sta. 12 + 143 O/S 4.5 RT Q Hwy 556 Line 'C' ORIGINATED BY LP/MJ
DIST 18 HWY 556 BOREHOLE TYPE Hollow stem Auger COMPILED BY LP
DATUM Geodetic DATE 1987 01 13 CHECKED BY *lp*

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100										WATER CONTENT (%)		
								SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE										10 20 30		
351.4	Ground Surface															GR SA SI CL				
0.0	Peat		1	SS	0	*1	351													
350.6	Trace, sand, gravel																			
0.8	Sand, some gravel, trace silt		2	SS	11		*2	350												
			3	SS	35															
			4	SS	22															
			5	SS	37															
	Occasional cobbles.		6	SS	16			349										38 59 (3)		
	Compact to Dense							348											44 52 (4)	
347.6																31 66 (3)				
3.8	Fine to medium Sand, trace silt	7	SS	17	*3	347														
		8	SS	12																
	Compact					346										2 95 (3)				
345.3						*4														
6.1	End of Borehole						345													
<p><u>Notes</u></p> <p>*1 After augering to Elev. 348.4, sand came up augers to Elev. 349.2±. Hole washed back down to Elev. 348.4. 'N' value may not be representative.</p> <p>*2 After augering to Elev. 347.6, sand came up augers to Elev. 248.2±. Hole washed back down to Elev. 347.6. 'N' value may not be representative.</p> <p>*3 After augering to Elev. 346.8±, sand came up augers to Elev. 347.8±. Hole washed back down to Elev. 346.8. 'N' value may not be representative.</p> <p>*4 After augering to Elev. 345.3, sand came up augers to Elev. 346.8±.</p> <p>Borehole was terminated.</p>																				

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



RECORD OF BOREHOLE No 12

METRIC

W P 277-85-02 LOCATION Sta. 12 + 143 4.5 LT Q Hwy. 556 Line 'C' ORIGINATED BY LP/MJ
DIST 18 HWY 556 BOREHOLE TYPE N-Casing, Washboring, Cone Penetration Test COMPILED BY LP
DATUM Geodetic DATE 1987 01 13 to 16 CHECKED BY So

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES							
351.4	Ground Surface											
0.0	Peat (probable)						351					
350.5							350					
0.9	Sand and Gravel trace silt		1	SS	31		349					48 45 7 0
	Occ. cobbles		2	SS	37		348					
	Dense		3	SS	9		347					
	Loose		4	SS	9		346					
			5	SS	28		345					36 56 (8)
345.9	Fine to Med. Sand trace silt		6	SS	34		344					
5.5			7	WS	-		343					
344.1	Compact to Dense		8	SS	35		342					
7.3	Sand with gravel, trace silt		9	SS	39		341					
	Occ. cobbles		9A	SS	33		340					
			10	SS	31		339					
							338					
							337					
336.8	Dense											
14.6	CONTINUED ON SHEET	2										

RECORD OF BOREHOLE No 13

METRIC

W P 277-85-02 LOCATION Sta. 12 + 159 at Q Hwy. 556 ORIGINATED BY MJ
 DIST 18 HWY 556 BOREHOLE TYPE Hollow stem Augers, N & B Casing, Washboring, cone COMPILED BY LP
 DATUM Geodetic DATE 1987 01 16 and 17 CHECKED BY [Signature]

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100		
351.3	Ground Surface													GR SA SI CL
0.0	Peat with sand		1	SS	4		351							
350.6			2	SS	19		350							50 45 (5)
0.7	Sand and gravel, trace silt		3	SS	35									53 43 (4)
			4	SS	36		349							55 40 (5)
	Occ. cobbles		5	SS	10		348							
			6	SS	15		347							
			7	SS	23		346							
							345							
							344							
							343							
							342							
							341							
	Compact to Dense		8	SS	77	*	340							
338.8							399							
12.5	Fine to Med. sand Some silt, trace gravel						338							
	Occ. cobbles		9	SS	5	**	337							
336.2														

+3, x5 : Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10



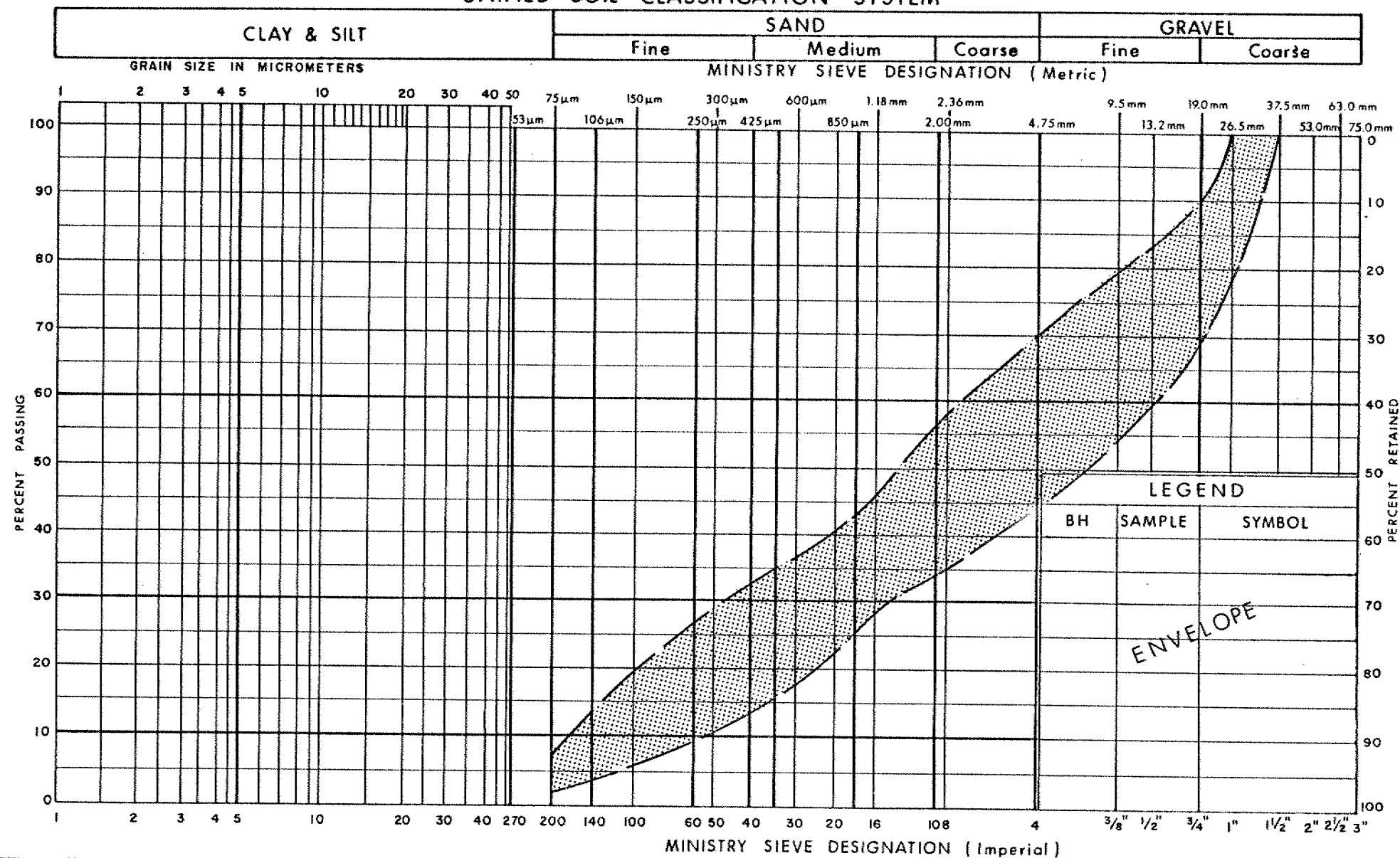
RECORD OF BOREHOLE No 13 Cont

METRIC

W P 277-85-02 LOCATION Sta. 12 + 159 at Q Hwy. 556 Line 'C' ORIGINATED BY MJ
DIST 18 HWY 556 BOREHOLE TYPE Hollow Stem Augers, N & B Casing, Washboring, Cone COMPILED BY LP
DATUM Geodetic DATE 1987 01 16 and 17 Test Test CHECKED BY h

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
336.2	CONTINUED																
15.1	Fine to Med. Sand Some silt, trace gravel						336										
	Occ. cobbles						335										
	Dense						334										
336.6			10	SS	38												
17.7	End of Borehole						333										
							332										
331.2																	
20.1	End of Cone Test Refusal to Cone																
	* Spoon bouncing probable cobble																
	** Sample probably disturbed due to unbalanced hydro- static pressure.																

UNIFIED SOIL CLASSIFICATION SYSTEM



Ontario

 Ministry of
Transportation and
Communications

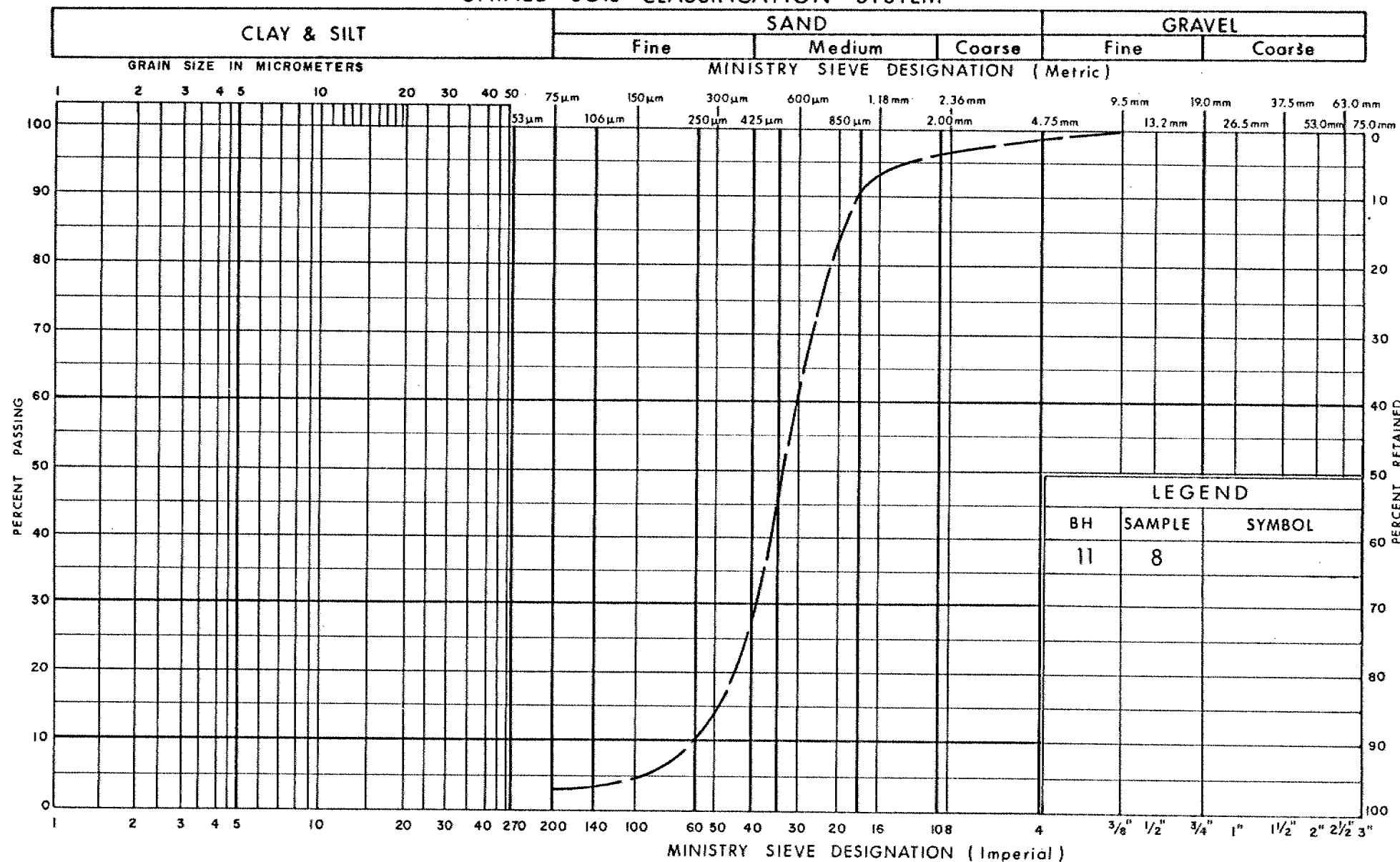
GRAIN SIZE DISTRIBUTION

SAND SOME/AND GRAVEL, TRACE SILT

FIG No 4

W P 277-85-02

UNIFIED SOIL CLASSIFICATION SYSTEM

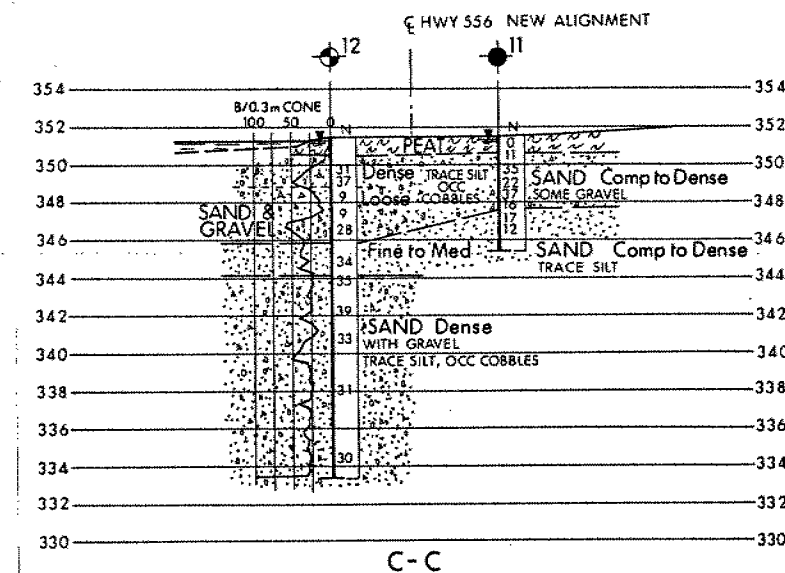
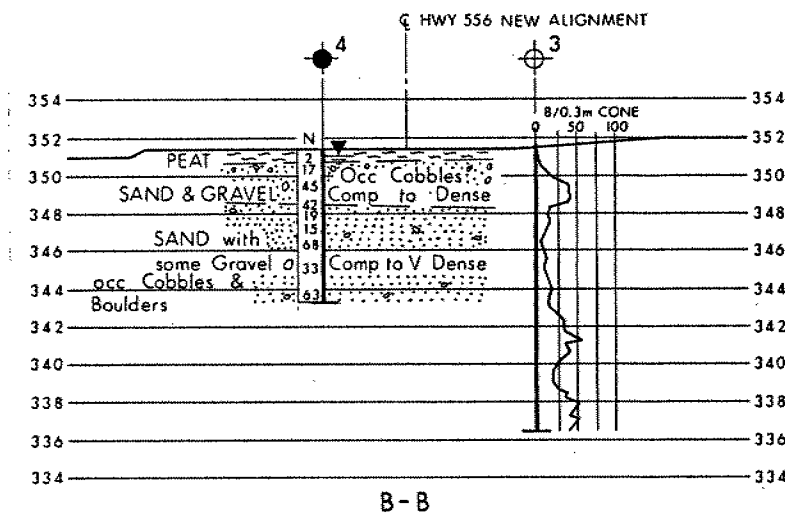
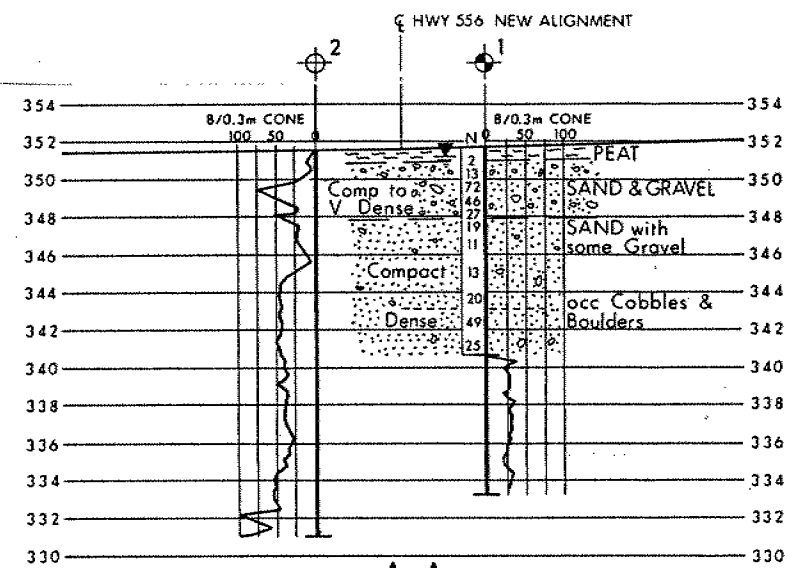


Ministry of
Transportation and
Communications

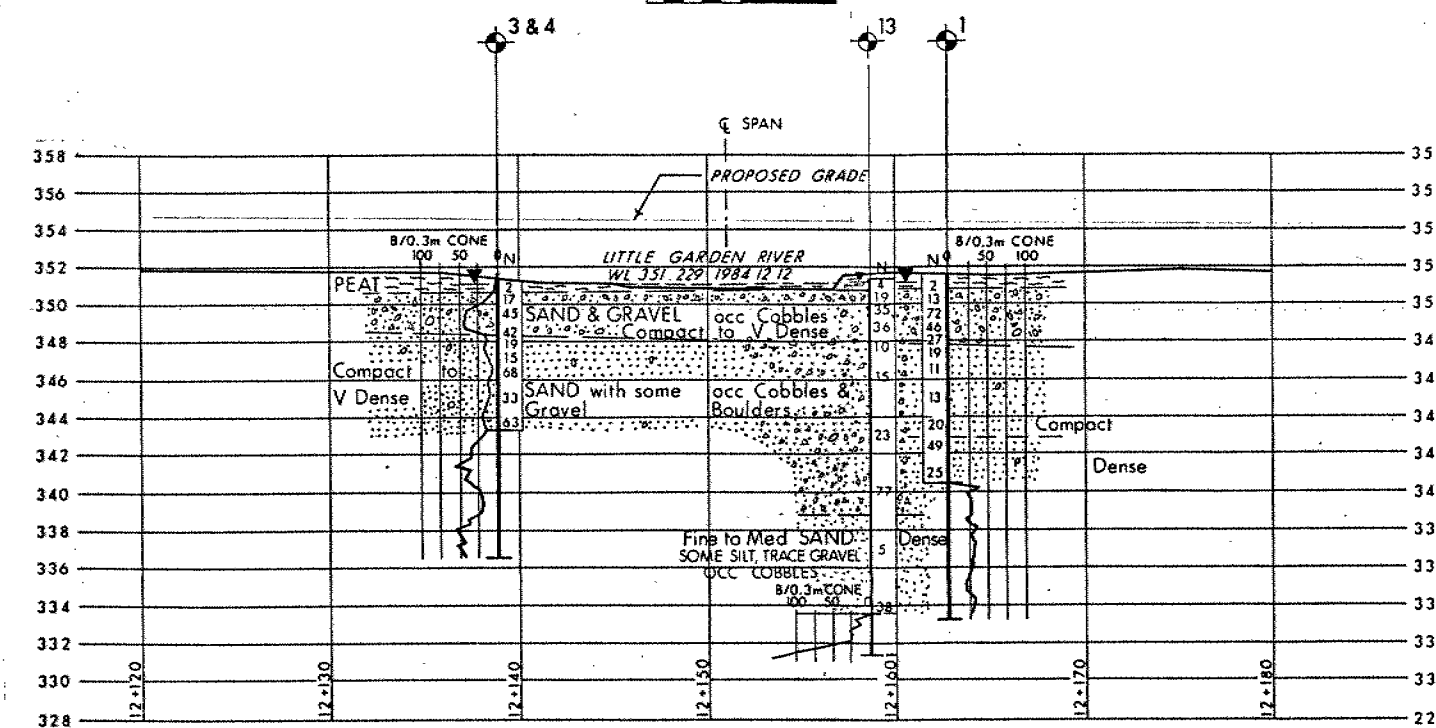
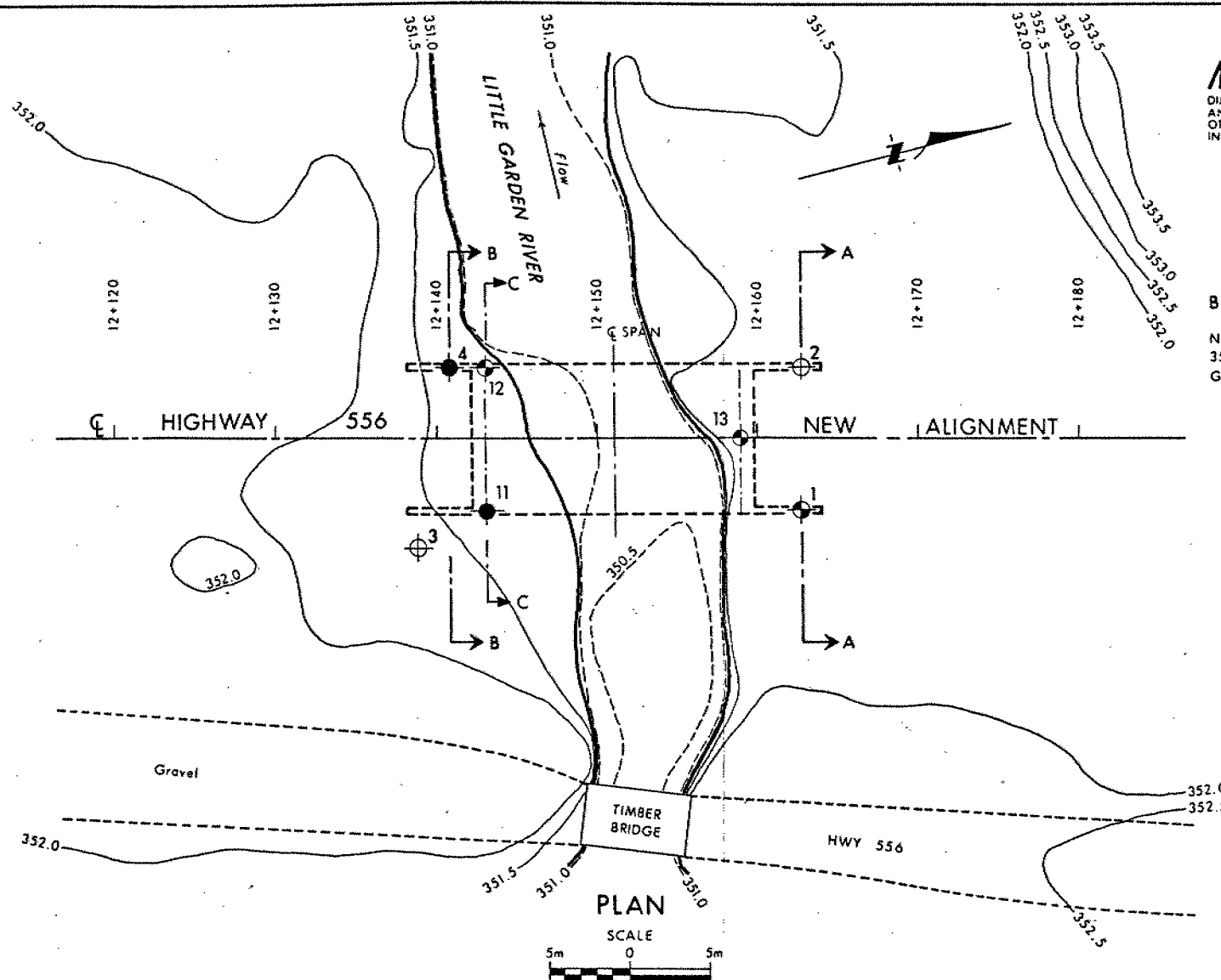
GRAIN SIZE DISTRIBUTION
Fine - Med SAND

FIG No 5

W P 277-85-02



SECTIONS
SCALE
5m 0 5m



METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES + METRES.

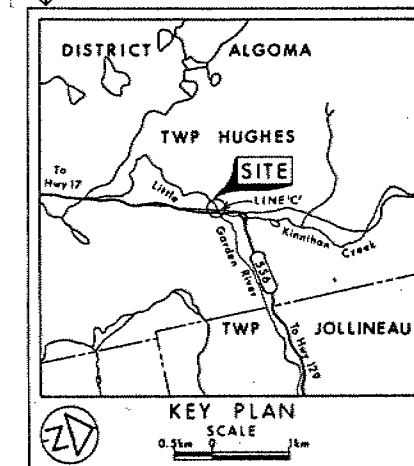
CONT No
WP No 277-85-02

LITTLE GARDEN RIVER BRIDGE
(CROSSING No 2)
BORE HOLE LOCATIONS & SOIL STRATA



SHEET

C. MIRZA ENGINEERING INC.
GEOTECHNICAL SPECIALISTS



LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊕ Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- W L at time of investigation 1986 02 & 1987 01

No	ELEVATION	STATION	OFFSET
1	351.6	12+163	4.6 Rt
2	351.6	12+163	4.4 Lt
3	351.6	12+139	6.8 Rt
4	351.3	12+141	4.4 Lt
11	351.4	12+143	4.5 RT
12	351.4	12+143	4.5 LT
13	351.3	12+159	-

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

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

DATE	BY	DESCRIPTION
870226	SO	8H's 11, 12, 13 & SECTION C-C ADDED
870226	SO	8H's 11, 12, 13 & SECTION C-C ADDED

Geocres No 41J-44

HWY No 556	DIST 18 SAULT 5 M
SUBMITTAL BY CHECKED CM DATE 1986 02 15	SITE 385-21
DRAWN SQA CHECKED CM	BDWG 2

memorandum



To: O. Ramakko
Head, Structural Section
Northwestern Region

Date: 1987 02 26

From: Foundation Design Section
Room 315, Central Building

RE: W.P. 277-85-02, Hwy. 556
Little Garden River Crossing No. 2
District 18 - Sault Ste. Marie

We have reviewed the final Drawing 1 and Drawing 3 (dated Sept.86) for the above-noted structure. We have the following comments:

1. In view of the well-graded nature of the natural soils at the site, we feel that a geotextile is not required below the rock protection. Furthermore, if a geotextile is used, its placement will be difficult to control underwater.
2. In areas where fill is required, peat and other surficial organic material should be completely subexcavated to at least 1 m beyond the plan limits of the embankment. All fill placed below 0.3 m above the prevailing groundwater level should consist of granular material.
3. In view of the presence of occasional boulders across the site, the sheet piles, as indicated on the drawings, may get 'hung-up' and may not be able to advance to the required tip elevation.

It is suggested that as with Little Garden River Crossing No. 1, and Kinnahan Creek, the dewatering scheme be left up to the discretion of the contractor.

The contractor should be informed through a Special Provision of the occasional cobbles and possible boulders which exist across the site. The contractor should be required to submit a dewatering scheme for M.T.C. approval. In addition, the S.P. should include a provision indicating that if sheet piles are used in the scheme, they should be left permanently in place.

If you have any questions or require additional information please do not hesitate to contact this Section.

A handwritten signature in dark ink, appearing to read "L. Politano", followed by a horizontal line.

L. Politano
Project Foundations Engineer
for

M. Devata
Chief Foundations Engineer
(East)

LP/MD/mmj

c.c. - K. Bassi

memorandum



Tel: 7101

To: O. Ramakko
Head, Structural Section
Northwestern Region

Date: 1987 02 24

From: Foundation Design Section
Room 315, Central Building

Re: Additional Borings at
Little Garden River Crossing #2
W.P. 277-85-02, Site 385-21, Hwy. 556
District 18 - Sault Ste. Marie

As discussed in our memorandum dated 86 11 07, this Section has advanced 3 additional boreholes at the above-noted site. The additional borings were necessary because the proposed single span structure was reduced from the original 22 m span to a 16 m span. Consequently, the revised abutment locations are now approximately 3 m closer to the edge of the creek.

The 3 boreholes (B.H. 11, 12, 13) were advanced using hollow stem augers and washboring techniques between 87 01 13 and 87 01 17 and extended to depths ranging between 6.1 and 20.1 m below the ground surface. A cone penetration test accompanied B.H. 12. The attached plan indicates the location of all the borings carried out at this site, including the boreholes (B.H. 1 to 4) advanced for the original investigation in February, 1986.

The following is a brief description of the subsurface conditions encountered in the most recent borings. Draft versions of the Record of Borehole Sheets for B.H. 11, 12 and 13 are attached for your information. It should be noted that all cohesionless materials encountered at this site will 'boil' or will be 'disturbed' when subjected to an unbalanced hydrostatic pressure.

South Abutment (B.H. 11, B.H. 12)

Extending from the ground surface (Elev. 351.4) down to a depth of 0.8 to 0.9 m is a surficial deposit of dark brown peat mixed with traces of sand and gravel. The peat is fibrous in texture and shows no root mat.

Underlying the organic deposit is a non-cohesive stratum of sand, some/and gravel. The thickness of the stratum was found to range between 3 and 3.5 m. From the drilling, occasional cobbles are inferred to be present within this deposit. Based on the interpretation of Standard Penetration Test 'N' values, the deposit is generally in a compact to dense state. However, in B.H. 12, the sand and gravel was found to be loose below Elev. 348.7. At this time, no laboratory testing has been carried out on samples of this cohesionless material.

Underlying the sand, some/and gravel is a deposit of fine to medium sand, some silt. In B.H. 11, this deposit was found to be a minimum of 2.3 m thick since the borehole was terminated at Elev. 345.3.

.....2

In B.H. 12, this stratum has a thickness of 2.9 m. Based on the interpretation of Standard Penetration Test 'N' values, this cohesionless deposit is in a compact to dense state. No laboratory test results are available at this time for samples from this deposit. Extending from a depth of 7.3 m below the ground surface to a depth of 18.1 m is a cohesionless deposit of sand with gravel, trace silt. From the drilling, it is inferred that occasional cobbles are present within this deposit. Based on the interpretation of Standard Penetration Test 'N' values ranging from 30 to 39 blows/0.3 m this deposit is considered to be in a dense state.

The groundwater conditions were observed in the open boreholes and the levels were found to correspond to the creek level prevailing at the time. At the time of the investigation, the groundwater level at the proposed south abutment location was found to be at Elev. 351.3±.

North Abutment (B.H. 13)

The surficial deposit consists of 0.7 m± of dark brown peat mixed with traces of sand. The peat is fibrous in texture and shows no root mat.

Underlying the organic deposit and extending from Elev. 350.6 to 338.8 is a non-cohesive deposit of sand with gravel. From the drilling, occasional cobbles are inferred to be present within the stratum. Based on the interpretation of Standard Penetration Test 'N' values ranging between 10 and 35 blow/0.3 m, it is believed that this deposit is in a compact to dense state. No laboratory tests have been carried out at this time on samples from this deposit.

Underlying the sand with gravel deposit previously described is a stratum of fine to medium sand, some silt, trace gravel. This cohesionless stratum was proven to Elev. 333.6. It is inferred that cobbles may be encountered within this deposit. Based on the interpretation of Standard Penetration Test 'N' values, this deposit is in a dense state. No laboratory tests have been carried out on samples of this material at this time.

Groundwater level at the proposed north abutment location was established to be at Elev. 351.2± at the time of the investigation.

RECOMMENDATIONS

Recommendations pertaining to the design and construction of the foundations for this project were presented in the 1986 report prepared for this Section by C. Mirza Engineering Inc. Generally, the recommendations in this report are still valid. However, in view of the revised footing locations and the findings of the additional borings, the bearing capacities have been modified as follows:

<u>Element</u>	<u>Location (Station)</u>	<u>Proposed Founding Elev.</u>	<u>Capacities</u>	
			<u>U.L.S.</u>	<u>S.L.S.II</u>
South Abutment	12 + 143	349.6±	600 kPa	200 kPa
North Abutment	12 + 159	349.5±	600 kPa	200 kPa

Since the density of the natural soils at this site has been shown to decrease with depth, this Section should be contacted if the founding elevations are revised.

I would request that you append this memorandum to your copy of the foundation report for this project for future reference. If you have any questions or require clarification, please do not hesitate to contact the undersigned.



L. Politano
Project Foundations Engineer

for

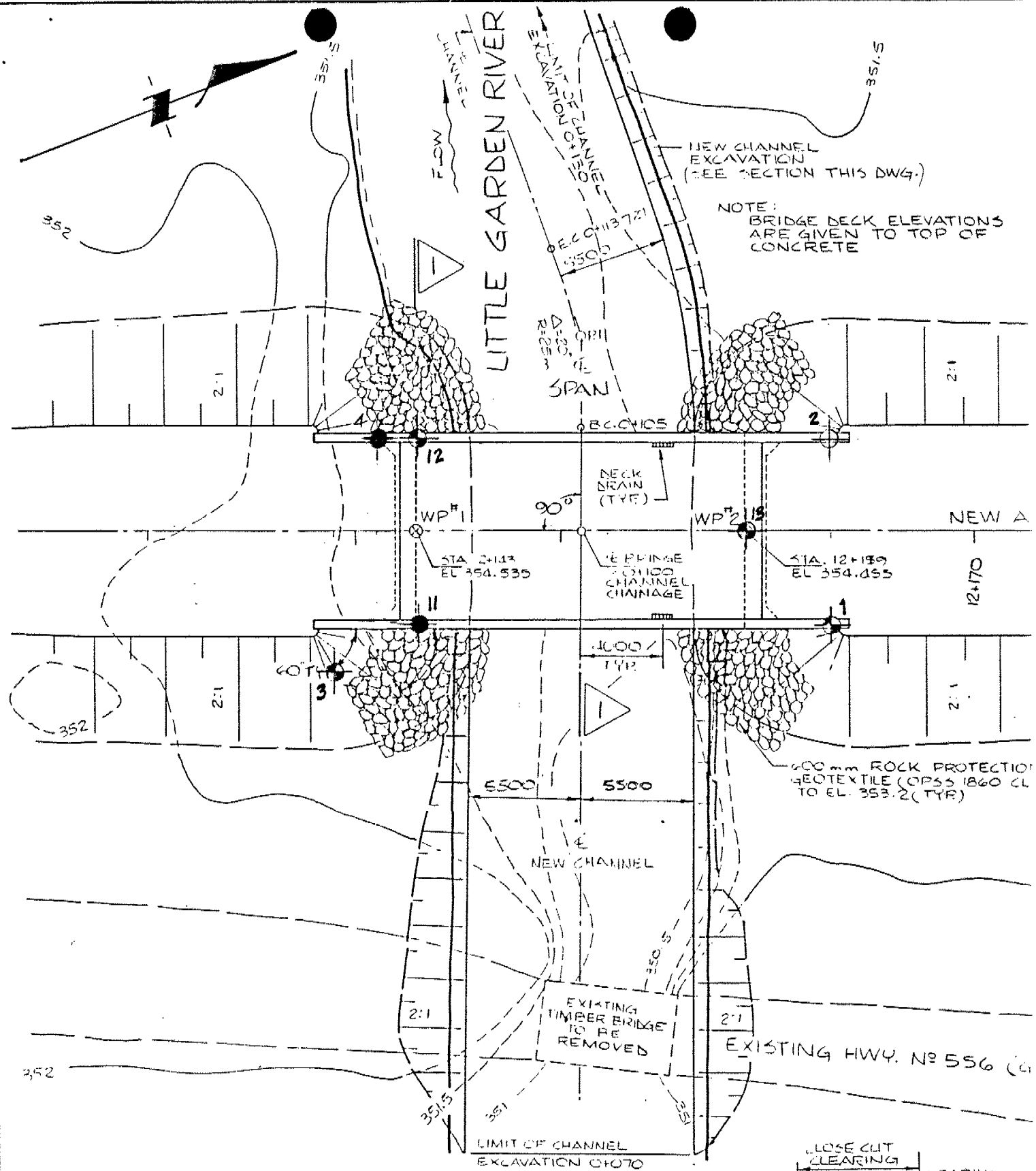
M. Devata
Chief Foundations Engineer
(East)

LP/MD/mmj

Attachs.

c.c. - J.B. MacMaster
G.D. Newell
C.E. Pritchard
K. Bassi
J.H. Peer

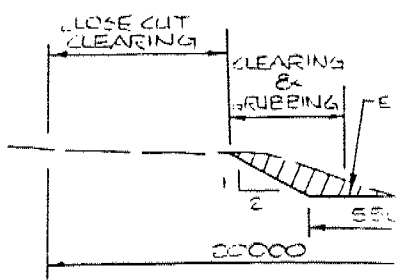
MANUSCRIPT OF TRANSPORTATION AND COMMUNICATIONS CHAIR



Holes done in Jan 86

Holes done in Jan 87

No	ELEVATION	STATION	OFFSET
1	351.6	12+163	4.6 R
2	351.6	12+163	4.4 L
3	351.6	12+139	6.8 R
4	351.3	12+141	4.4 L
11		12+143	4.5 RT
12		12+143	4.5 LT
13		12+150	±



LITTLE GARDEN RIVER
BRIDGE No 2

TYE CHANNEL
N.T.S.

memorandum



Tel: 248-3282

To: O. Ramakko
Head, Structural Section
Northwestern Region

Date: 1986 11 07

ATTN: R.J. Krisciunas
Senior Structural Engineer

From: Foundation Design Section
Room 315, Central Building
Downsview

RE: W.P. 278-85-02
Little Garden River Crossing No. 2
Hwy. 556, Site 38 S-21
Dist. 18, Sault Ste Marie

Further to your memorandum of 86-10-16, we have reviewed the Preliminary General Arrangement drawing (dated September '86) for the above-noted project.

We would like to bring to your attention that the initial E-Plan which accompanied your request for a foundation investigation indicated that the proposed crossing would consist of a 22 m single span structure. Consequently, the boreholes were advanced at locations corresponding to a 22 m span.

Our comments with regards to this project are as follows:

- Limits and extent of organic material subexcavation should be indicated.
- Since the sheet-pile coffer dam will be installed immediately adjacent to abutment footing, it should remain permanently in place. Disturbance of the founding structure and consequent bearing capacity reduction will result if any attempt is made to remove the coffer dam.

The foundation investigation involved advancing 2 boreholes at each of the abutment locations of the original 22 m span structure. Subsequent to the investigation, as previously noted, the structure was reduced to a 16 m span. The abutments were relocated closer to the river banks. Often, the subsurface conditions adjacent to a river are weaker than those at some distance away. Consequently, this Section will arrange for an additional one or two boreholes at this site to confirm that the subsurface conditions at revised abutment locations are consistent to those described in the report.

When the additional fieldwork has been completed, this Section will once again review the preliminary General Arrangement drawing. In the meanwhile, please inform us of the schedule of this project.

A handwritten signature in dark ink, appearing to read "L. Politano".

L. Politano,
Project Foundations Engineer

for

M. Devata,
Chief Foundations Engineer (East)

memorandum

w.p. 277-85-22



To: O.E. Ramakko
Head, Structural Section
Thunder Bay

Date: 1986 09 17

From: Foundation Design Section
Room 315, Central Building

RE: Little Garden River Bridges No. 1 & No. 2
Sites 38S-20 and 386-21
Highway 556
District 18 - Sault Ste. Marie

Further to your memo of 86 09 12 and recent telephone conversations, we understand that consideration is being given to the construction of twin cell(2 - 6 x 3 m) concrete box culverts with retaining walls at the above-noted sites. The proposed elevation of the underside of the culvert at crossing No. 1 is Elev. 344. At crossing No. 2, the proposed founding elevation is Elev.350.

Your concern with this alternative is the potential dewatering problems. The following are our comments with regards to dewatering.

Little Garden River Crossing No. 1

If the excavation is to be advanced to Elev. 344, a dewatering scheme will be required since the predominant deposit consists of gravel, sand and boulders. Since the material is dense to very dense and includes boulders and cobbles, the use of well points or sheet piling is not considered feasible.

An alternative which could be utilized at this site after the creek has been diverted involves advancing the excavation down to Elev. 344 and providing perimeter ditches 0.3 to 0.4 m below the base of the excavation. Water seeping into the excavation would have to be sumped from a number of pumps. The denseness of the material may permit such a scheme to function.

It should be noted, however, that considering the nature of the soil and the water elevation, it is difficult to predict the amount of water which will seep into the excavation. Since a very large excavation is required for the construction of the twin-box culverts, it may be difficult to control the volumes of water seeping into the excavation particularly if there are localized 'less dense' zones or seams in the immediate area.

Little Garden River Crossing No. 2

It is proposed to found the twin-box culverts at Elev. 350. The material across the site is non-cohesive in nature and subsequently is very permeable.

In order to dewater the excavation, a sheet pile coffer dam could be installed around the perimeter of the excavation. The sheet piling would have to penetrate the minimum of 2-3 m below the base of the excavation in order to adequately depress the groundwater level in the area of the proposed excavation.

Pumping from sumps would also be required in order to control the expected nominal amounts of seepage into the excavation.

If you have any questions or require additional information, please do not hesitate to contact me.

A handwritten signature in black ink, appearing to be 'L. Politano', with a long horizontal flourish extending to the right.

L. Politano
Project Foundations Engineer
for

M. Devata
Chief Foundations Engineer
(East)

MD/LP/mmj

MEMORANDUM

TO: M. S. Devata
Chief Foundations Engineer
Engineering Materials Office
Foundation Design Section
Central Building
Downsview, Ontario

DATE: 1986 09 12

FROM: Structural Section
Northwestern Region
807/577-6541 Ext. 247

ATTENTION: L. Politano
Project Foundations Engineer

Re: Little Garden River Bridge No. 1 and No. 2
Sites 38S-20 and 38S-21
Hwy. 556, Sault Ste. Marie

This serves to confirm our recent telephone conversation and my request for additional information about these two sites.

Northland Engineering of Sudbury have recently begun the structural design for these bridges. Part of their assignment requires the assessment of various structure alternatives, and one type in particular shows advantages which have caused me to consider it more closely. That alternative is a twin cell (6 m. x 3 m.) concrete box culvert with retaining walls.

The major concern I have for this type of structure relates to dewatering for the foundations. The Foundation Report notes that the underlying material is dense granular which is quite porous. Mention is also made of the need for a well point system for dewatering. Considering the large area beneath a twin cell culvert to be unwatered, and the need for a stream diversion as well, this alternative may become very expensive.

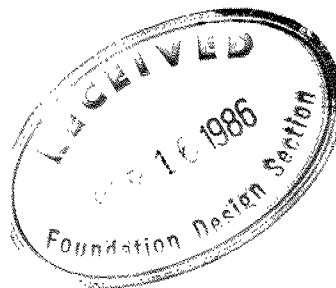
Please review the Foundation Reports for these two structures and advise as to the method of dewatering required, bedding requirements, etc. for the twin cell culvert option. The proposed elevation at the underside of each culvert is as follows:

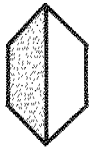
Little Garden No. 1: Elev. 344.0
Little Garden No. 2: Elev. 350.0

O. E. Ramakko

O. E. Ramakko
Head, Structural Section

OER/eu





C. MIRZA ENGINEERING INC.
GEOTECHNICAL SPECIALISTS & CONSULTING ENGINEERS

Suite 31, 1262 Don Mills Road, North York, Ontario, Canada M3B 2W7 • Telephone (416) 441-2560 • Telex 06-966637

1986 02 10

Our File: 85-103

Agreement #:4238-9085-136

M.S. Devata, P. Eng., M.CGS
Chief Foundation Engineer
Ministry of Transportation & Communications
Room 315, Central Building
1201 Wilson Avenue
Downsview, Ontario
M3M 1J8



Attention: Dave Dundas, P.Eng.
Senior Foundations Engineer

Re: Little Garden River Bridge #2, Ranger Lake Road
W.P. 277-85-02, Site # 389-21, Dist. 18
FOUNDATION INVESTIGATION - PRELIMINARY REPORT

Dear Sirs:

The field work for the above project was completed on 1986 02 07. It consisted of two drilled boreholes to depths of respectively 10.7 and 7.6 metres. In the deeper borehole, a dynamic cone penetration test was conducted from the 10.6 m depth to a final depth of 18.3 m, without encountering bedrock. In addition, two cone tests were also carried out to depths of about 10 metres without encountering bedrock.

Summarized Soil Conditions

Beneath a surface organic cover of peat/muck/topsoil (thickness 750mm), the subsoil consists of a brown coarse sand with gravel, and occasional boulders. N Values in this deep deposit ranged from between 10 and 20 to well over 40 blows /0.3 m. At a depth of 2 metres, the N values were over 40 blows/0.3m. These values reduced at a depth of between 3.5 and 5 metres before increasing again. In the dynamic cone test, the blow counts were between 25 and 40 blows/0.3 m.

At depth, the sand contains isolated lenses of finer materials, such as fine sand with some silt and the colour changes to grey. However, no cohesive soils were encountered. In one location, the augers began to tilt, suggesting a large boulder had been encountered.

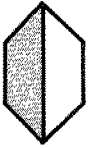
The soil is therefore essentially cohesionless, compact to dense with perhaps minor loose zones due to unbalancing of hydrostatic heads during the sampling operation.

The groundwater was encountered at the level of the river, at a depth of 0.76 m below ground surface. This groundwater connects freely with the adjoining ground.

Foundation Design

The most suitable bearing stratum is located at a depth of 2 metres, where spread footings could be designed for 600 (ULS) and 200 (SLS Type II) kPa. The low SLS Type II value is for the fact that the footings would be submerged. However, such footings would require dewatering. Sheetpiling may be difficult

...../2



due to the presence of boulders. The sand is extremely permeable, and would boil easily under unbalanced hydrostatic heads.

It may therefore be more appropriate to design perched abutments founded within engineered compacted granular fill at values of 900 (ULS) and 350 (SLS Type II) kPa respectively.

There was no identifiable end bearing stratum to 18 metres depth for a suitable pile design. The soil is too dense for safe driving of timber piles, but not dense enough for H piles. Caissons would need to be dewatered and would be dependent on side friction, which would be drastically reduced if casing is used.

Approaches

No stability or settlement problems are anticipated, since the subsoil is generally compact and non-cohesive. Any settlements would be immediate.

Erosion

The subsoil is extremely erosion susceptible. Therefore scour protection is necessary to the high water mark plus a freeboard allowance, depending upon the hydrology of the stream. It is suggested that a graded filter or geotextile be used to protect the washing out of the sandy overburden through the pore spaces of any rip-rap protection.

Stripping

The upper 750 soil cover is very organic and unsuitable as a base below fills. It should be stripped, and used for topsoil dressing on fill slopes.

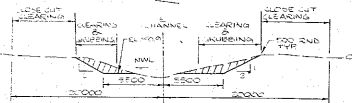
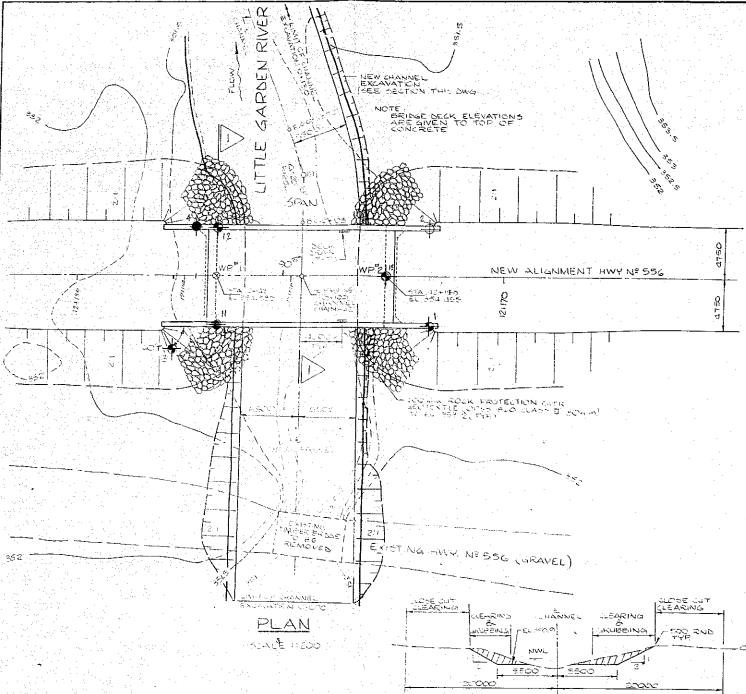
Final Report

We are commencing laboratory testing today, and expect to have the completed report to you, with plan, sections, logs and test results available to you in 12 copies by 1986 02 19. In the meantime, if you have any suggestions, please do not hesitate to call us.

Yours very truly,
C. MIRZA ENGINEERING INC.

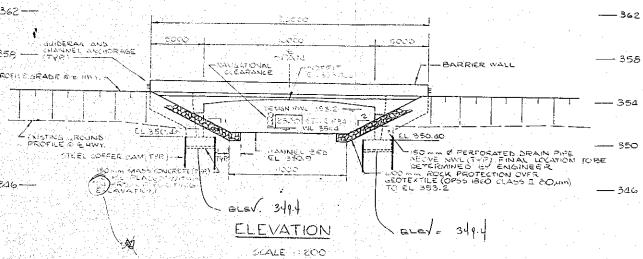
Cam Mirza, P.Eng.

METRIC DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN
 northland engineering limited
 Consulting Engineers and Planners
 1701-1703
 1701-1703



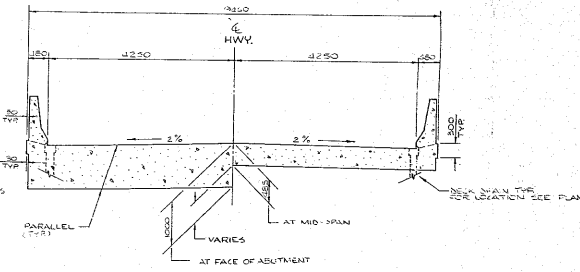
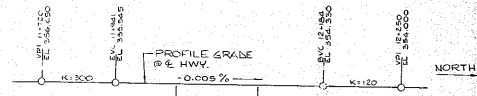
NOTES

- UPON COMPLETION OF CHANNEL EXCAVATION BANKS TO BE REVEGETATED WITH GRASS OR SODS.
- TREES TO BE PLANTED ALONG CHANNEL BANKS BY OTHERS.



BM 351.874
 GEODETIC DATUM
 1984 ADJUSTED
 STA. 12+151

Z-0 FOR CONSTRUCTION



METRIC
 DIMENSIONS ARE IN METRES
 AND/OR MILLIMETRES
 UNLESS OTHERWISE SHOWN
 northland
 engineering
 limited
 Consulting Engineers and Planners
 1701-1703

RECEIVED
 FEB 17 1987
CONT No 87-28
WP No 277-85-00
LITTLE GARDEN RIVER BRIDGE No. 2
 HWY. No 556 STA. 12+151
GENERAL ARRANGEMENT
SHEET 28

- GENERAL NOTES:**
- CLASS OF CONCRETE**
- FOOTINGS AND MASS CONCRETE ----- 30 MPa
 - REMAINDER ----- 20 MPa
- REINFORCING STEEL**
- REINFORCING STEEL TO BE GRADE 400 UNLESS OTHERWISE SPECIFIED.
- CLEAR COVER TO REINFORCING STEEL**
- FOOTINGS ----- 100 mm ± 20 mm
 - ABUTMENTS AND WINGWALLS ----- 50 mm ± 20 mm
 - DECK SLAB TOP ----- 25 mm ± 10 mm
 - DECK SLAB BOTTOM ----- 25 mm ± 10 mm
 - BARRIER WALLS ----- 75 mm ± 20 mm
- EXCEPT AS NOTED OTHERWISE
- CONSTRUCTION NOTES**
- NO CONCRETE SHALL BE PLACED FOR FOOTINGS UNTIL THE SUBTLE OF EXCAVATION AND CHARACTER OF THE FOUNDATION MATERIAL HAVE BEEN APPROVED BY THE ENGINEER.
 - BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND THE ABUTMENTS, KEEPING THE HEIGHT OF BACKFILL APPROXIMATELY THE SAME AS TO THE ELEVATION BE GREATER THAN 500 mm.
 - FALSEWORK SUPPORTING WINGWALLS SHALL NOT BE REMOVED UNTIL CONCRETE IN DECK HAS ATTAINED A MINIMUM STRENGTH OF 30 MPa.

- LIST OF DRAWINGS**
1. GENERAL ARRANGEMENT
 2. BENCHMARK LOCATIONS AND COORDINATES
 3. FOOTING LAYOUT AND REINFORCING
 4. FRAME LAYOUT AND DETAILS
 5. FRAME REINFORCING
 6. WINGWALL REINFORCING AND SECTIONS
 7. BARRIER WALL
 8. AS-CONSTRUCTED ELEVATIONS AND DIMENSIONS
 9. BRIDGE DATA AND SITE NUMBER DATA
 10. STANDARD DETAILS
 11. QUANTITIES - STRUCTURE



REVISIONS	DATE	BY	DESCRIPTION

DESIGN: N.W.C. CHECK: T.S.A. LOADING: C.H.R.C. 85 B. DATE: SEPT/85
 DRAWING: M.D. CHECK: T.S.A. SITE NO: 38 S. 21 DWS

DRAWING NOT TO BE SCALED
 100 mm ON ORIGINAL DRAWING

METRIC
 DIMENSIONS ARE IN METRES
 AND/OR MILLIMETRES
 UNLESS OTHERWISE SHOWN

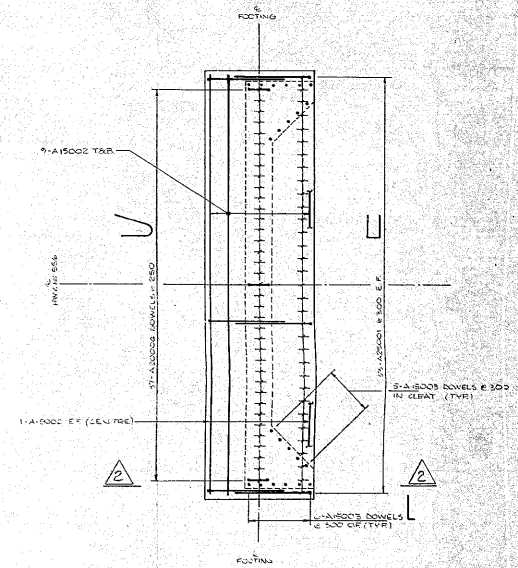
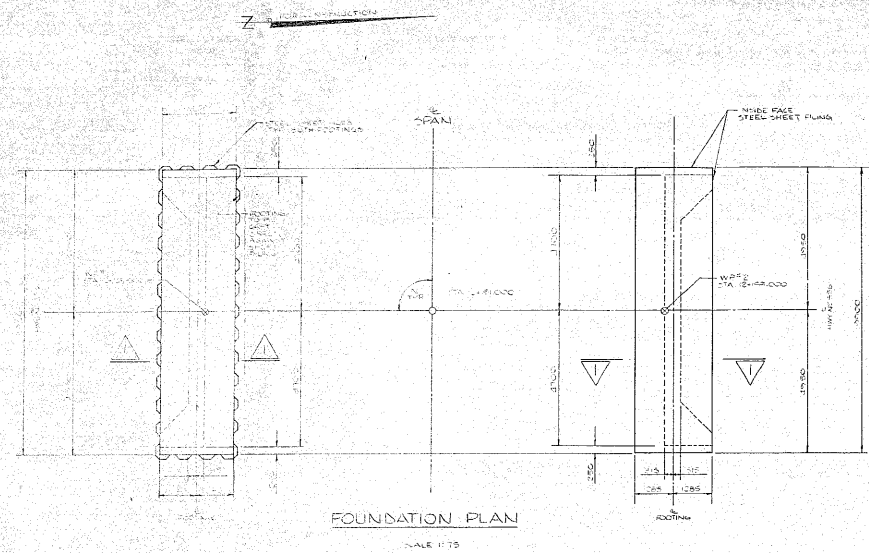
northland engineering limited
 Consulting Engineers and Planners
 NORTH ISY SUBURB

CONT No 87-213
WP No 277-85-00

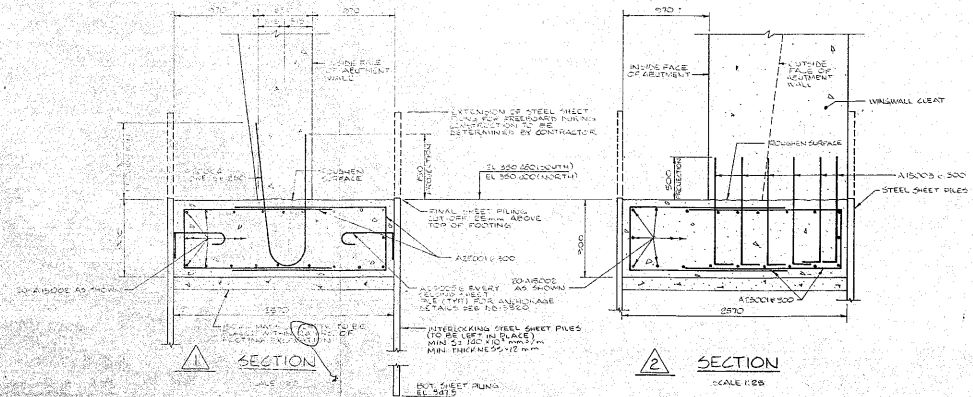
LITTLE GARDEN RIVER BRIDGE No 2
 HWY. No 555 STA. 12+151

FOOTING LAYOUT & REINFORCING

SHEET
30



ABBREVIATIONS
 IF - INSIDE FACE OF
 OF - OUTSIDE FACE
 EF - EACH FACE
 T&B - TOP AND BOTTOM



DRAWING NOT TO BE SCALED
 100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION
DESIGN NAME	CHECK T&B	LOADING	CH150C/BS
DRAWING NAME	CHECK T&B	SITE No	277-85-00
		DATE	06/07/91

