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**FOUNDATION INVESTIGATION REPORT
FOR
LITTLE WHITE RIVER BRIDGE
WP321-85-00, SITE 38-57
HIGHWAY 546, DISTRICT 17, SUDBURY**

Ref. No. G-88.0105
April 1988

Prepared for:

Ministry of Transportation
Foundation Design Section
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Geocres No. 41J-48

Distribution

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FOUNDATION INVESTIGATION REPORT
FOR
LITTLE WHITE RIVER BRIDGE
WP321-85-00, SITE 38-57
HIGHWAY 546, DISTRICT 17, SUDBURY

1.0 INTRODUCTION

This report contains the results of a foundation investigation carried out at the site of the above mentioned project.

The investigation and study was undertaken by Geo-Canada Ltd. on behalf of the Ontario Ministry of Transportation under Consultants Agreement No. 4238-9087-186.

The purpose of the investigation was to determine the subsurface and foundation conditions at the proposed bridge abutment locations and to provide the necessary geotechnical input for the foundation design of the proposed new structure.

.../...

The field work was carried out during the period of 1988 02 04 to 15, and consisted of two sampled boreholes (one drilled on each side of the river), a cone penetration test adjacent to one of the boreholes, two cone penetration tests extended from the base of the boreholes, and two separate cone penetration tests. The borings were advanced by washboring and rotary drilling (tri-coning) techniques to depths ranging between 14.3 and 24.1 m below the existing ground surface. With the additional cone tests, which were put down from the bottom of the boreholes, the depth of exploration was extended to 21.6 and 34.2 m respectively.

2.0 DESCRIPTION OF SITE AND GEOLOGY

The site is located on Secondary Highway 546, near its junction with Highway 639, approximately 40 km north of the Town of Elliot Lake. At present, the Highway is carried over the Little White River on a single span double Bailey bridge. The river, at the points of the existing and proposed crossings follows a generally east to west course.

The topography of the general area is very hilly to small mountainous with steep sided rock formations rising above numerous lakes and small streams and rivers. Several wide valleys wind through the area. In one of these valleys lies the aforementioned bridge site.

.../...

Locally, the river valley is approximately 300 m wide and runs generally in a north-east to south-west direction and the Little White River winds its way through this valley. At the bridge site, the river is approximately 25 m wide and 0.7 m deep at its centre at the time of our drilling. The current is fast and the river bed is lined with cobbles of diameters ranging from 60 to 180 mm (average 125 mm) in size.

At the inside of the river bends, including the north abutment location, berms or narrow ridges consisting of cobble and coarse sand have been deposited to heights of approximately 2.5 m above the river or ice level. At the outside bends of the river (e.g. at the south abutment location) the banks show signs of some scouring and slight undercutting.

The vegetation in the area consists of coniferous (spruce) and deciduous (poplar) mature forests with some sumac and sage brush around the river and wet areas.

Approximately 80 m to the east of the proposed bridge location the existing alignment of Highway 546 crosses the river over a double Bailey bridge. This structure, approximately 30 m long and one lane wide, is supported on large timber cribs at each abutment. The cribs are rock

.../...

filled and appear to be free from the effects of scour and settlement.

The nearest rock outcrop is about 250 m to the east of the proposed bridge and is part of a steep sided ridge of rock running in a north-east to south-west direction. Two outcrops at this location, approximately 50 m apart, were examined by us. One of the outcrops was composed of metasedimentary rock, high in quartzite and was fairly intact and massive. It had nearly horizontal medium spaced bedding planes and two joints, one nearly vertical and the other about 60 degrees to the horizontal. The rock exposed in the other outcrop was more igneous in nature, also high in quartzite and feldspar, and was badly weathered and fractured. It had no apparent bedding plane and no regular joint pattern.

Geologically, the site is located on the Canadian Pre-Cambrian Shield. Geological maps indicate that the Little White River is near the boundary between the early Pre-Cambrian predominantly granitic rocks to the north and the Middle Pre-Cambrian metasedimentary rocks of the Huronian Super Group which comprise conglomerates, metamorphosed sandstones and siltstones to the south. There is a major east-west oriented fault line approximately 10 km south of

.../...

the site, and another north-south fault line about 15 km to the north. During Pleistocene times the area was covered by glaciers which left a mantle of overburden consisting of glacial drift.

3.0 SUMMARIZED SUBSURFACE CONDITIONS

3.1 General

The general subsurface profile at the site consists of a 3 to 4 m thick surficial layer of cobbles, gravel and sand underlain by a deep, over 30 m thick deposit of fine sand. The surface of the Pre-Cambrian bedrock was not encountered at any of the test locations, i.e. within a depth of 34 m (Elevation 269 m) at the north-west quadrant of the site.

The groundwater table at the boreholes was encountered at shallow depths and its level (Elevations 302.3 to 302.4 m) coincided with the river level which at the time of the investigation was at Elevation 302.4 m.

The subsurface conditions are described in detail on the individual borehole logs and the main characteristics of the soil types encountered are discussed briefly in the following sections.

.../...

3.2 Cobbles, Gravel and Sand

The surficial soil deposit encountered in the boreholes is a 3 to 4.3 m thick (average 3 m) coarse granular deposit consisting of cobbles, gravel and coarse sand with the occasional boulder. Grain size analyses performed on particles smaller than 38 mm indicate 72 to 46% gravel, 51 to 26% sand, and 2 to 3% soil fines. The soil fines are non-plastic silt. Grading curves are shown on Figure 1. The cobble size particles range generally between 60 and 180 mm (average 125 mm). The frequency of the cobbles decreases with depth.

The standard penetration resistances in the deposit ranged between 3 and 68 blows per 0.3 m (average > 30), indicating a loose to very dense, but generally dense compactness condition. As some of the high penetration resistances ("N"-values) were undoubtedly affected by the gravel or larger particle sizes, the compactness condition of the deposit is probably less than that inferred from the standard penetration tests.

3.3 Fine Sand

Below the surficial cobble and gravel deposit is a thick bed of fine sand. Its thickness at the test locations is in

.../...

excess of 20 to 30 m.

Near its surface, the deposit consists mainly of fine to medium sand with some gravel (13 to 25%) and a trace of silt (1 to 3%). The results of the grain size analyses performed on two representative samples obtained from this zone are attached as Figure 2.

With depth the sand becomes finer textured and the silt content also increases. The results of the grain size analyses are presented graphically on Figure 3, showing 77 to 98% sand, and 2 to 23% silt.

In Borehole 4, interbedded with the sand at about Elevation 282 m, an approximately 1.0 m thick layer of silt was encountered. The particle size distribution of this material is shown on Figure 4 indicating 25% fine sand and 75% silt.

The standard penetration tests (SPT) performed in the sand gave "N" values ranging from 4 to 29 blows per 0.3 m, but typically between 10 and 15. These values indicate a loose to compact, but generally compact deposit. It is believed that some if not most of the SPT results were affected by the washboring operations and the inevitable disturbance of the soil by the drilling and sampling techniques used. For this

.../...

reason, we are of the opinion that the compactness condition of the sand is higher than that inferred from the SPT results.

3.4 Groundwater

It is expected that the position of the groundwater table in the shallow banks adjacent to the river is governed by the water level in the river. At the time of the investigation the river level was at Elevation 302.4 m, and the free standing water level in the boreholes was recorded between Elevations 302.3 and 302.4 m.

4.0 DISCUSSION AND RECOMMENDATIONS

4.1 General

The proposed new crossing of Highway 546 over the Little White River is located about 80 m upstream (west) of an existing single span double Bailey bridge. The new bridge will be a 28 m long, 10 m wide single span structure. The proposed structural system consists of either concrete or steel girders with a concrete bridge deck resting on reinforced concrete abutments. The height of the approach embankment on the north and south sides will be approximately 3 to 4.5 m

In summary, the investigation established that the general soil profile consists of a 3 to 4 m thick generally dense layer of cobbles, gravel and sand with the occasional boulder. Underlying this is an over 30 m thick layer of compact sand, and the surface of the bedrock was not encountered to a depth of 34 m.

The inferred subsurface profiles are shown on Drawing 3218500-A.

4.2 Foundations

For the support of the structure the following foundation options can be considered:

- a) spread footings on the compacted rock approach fills
- b) timber piles
- c) steel-H piles

These alternatives will be discussed separately below.

a) Spread Footings on Compacted Rock Fill

Consideration could be given to founding the abutments on spread footings resting on the rock fill approach embankments.

For spread footings founded on rock fill the factored bearing capacity at Ultimate Limit State (ULS) is 600 kPa, and at Servicibility Limit State Type II (SLS II) is 250 kPa. The structure should be designed to accommodate total and differential settlements of 50 mm and 25 mm respectively.

The footings should be provided with a minimum of 1.5 m thick earth cover for frost protection.

.../...

Resistance to sliding of the abutment footings can be calculated assuming an unfactored angle of friction (ϕ) of 35° between the underside of the concrete footing and the rock fill. The rock fill pad should extend laterally a minimum of 3.0 m beyond the edge of the footing all around its perimeter. From this point the rock fill could be sloped at an angle not steeper than 1.5 horizontal in 1.0 vertical as shown on Figure 4.2.a.

Before placing the rock fill, the existing material under the plan limits of the rock fill should be subexcavated to Elevation 302 m + (i.e. to the water level). The quality, the placing and the compaction of the rock fill material should conform to current MOT Standards and practice.

We suggest that upon reaching the proposed foundation level the surface of the rock fill in the plan area of the footing be covered with a thin (75 mm) layer of skim coat of lean concrete.

For the spread footing alternative the adequate protection of the embankment and river channel from scour is of utmost importance. The design of the scour protection measures should be based on hydrological requirements.

.../...

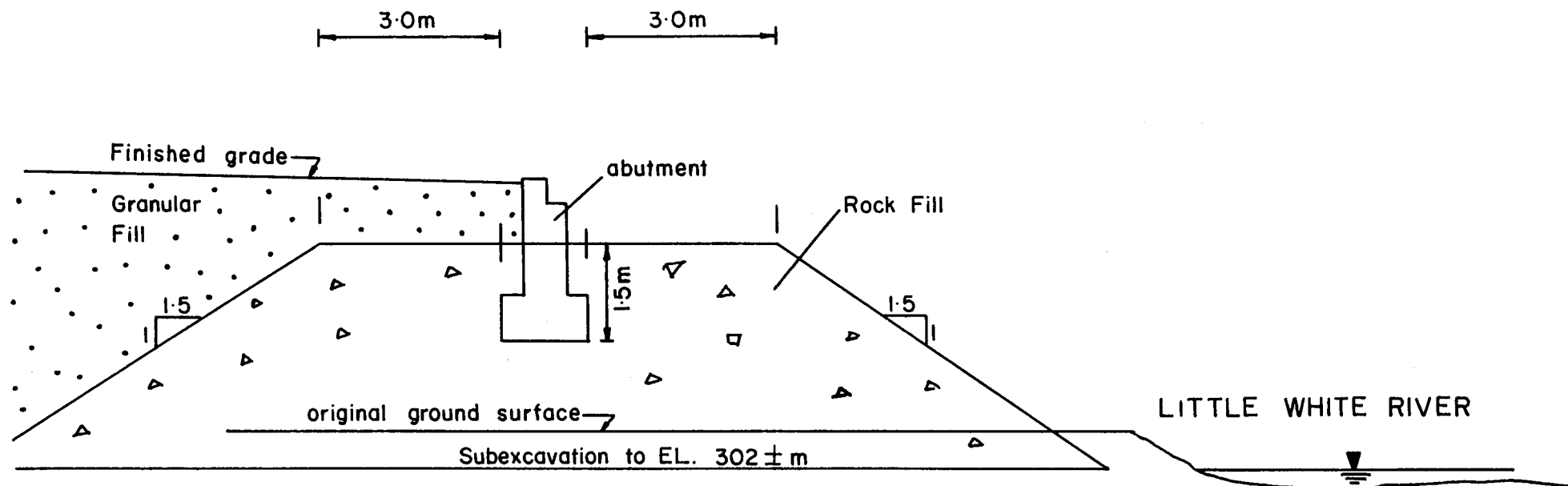


FIG. 4.2.a

FOOTINGS ON ROCK FILL

b) Timber Piles

The capacity of Size 36 timber piles driven to a toe elevation of 290 m is estimated to be as follows:

at ULS - 375 kN

at SLS II - 250 kN

The timber should be pressure treated.

The timber piles, in our opinion, could not be driven through the upper cobble and gravel deposit without the risk of damage. This surficial layer should, therefore, be removed from within the area of the abutments and pile driving. Since the cobble layer is generally only 3 m thick and as the frequency of the cobbles decreases with depth, the depth of subexcavation should not be excessive. The excavation could be carried out under water with ordinary construction equipment. The removed material should be replaced, also under water, with sand and gravel fill in which the maximum particle size is limited to 50 mm. The excavated material, if screened to remove the large particles, could be reused.

.../...

Unbalanced lateral forces should be resisted by battered piles.

Pile driving should be controlled by Ministry of Transportation Standards SS103-10 assuming an ultimate capacity of 750 kN.

c) Steel-H Piles

It is estimated that HP310x79 steel pile sections, if driven to a toe elevation of 275 m, will be able to develop the following capacities:

at ULS - 650 kN

at SLS II - 400 kN

It is possible that higher capacities could be developed, however, these would have to be confirmed by a full scale load test in the field.

Unbalanced lateral forces should be resisted by battered piles.

.../...

Pile driving should be controlled by Ministry of Transportation Standards SS103-11 assuming an ultimate capacity of 1200 kN.

As the driving of the piles could temporarily set up high pore pressures in the fine grained granular deposits present near the pile toe elevation, the initial driving resistance may be lower. In view of this, it is recommended that the piles be retapped 24 hours after initial driving and that the set observed for the first few full blows of the hammer during redriving should be used to determine the capacity of the piles. The use of the Hiley formula is acceptable for the control of the pile driving, however, it would be prudent to carry out a full scale load test not only to confirm, but possibly also to increase the design capacity of the piles.

Although hard driving conditions are expected in the upper cobble and gravel deposit, we believe that it will be possible to drive the steel-H piles through this stratum. It is, however, suggested that at the outset of the project a short test pile be driven and, if considered necessary, the cobble layer be removed from the area of the pile driving similar to that discussed for the timber piles.

In case of perched abutments within the area of the pile driving, the maximum size of particles used for embankment construction should be limited to 75 mm. In anticipation of hard driving conditions, the tips of the piles should be reinforced in accordance with the current MOT standard.

4.3 Lateral Earth Pressures

Backfill to structures should consist of granular material in accordance with Ministry of Transportation Standard Special Provision #121 (83 10).

Computation of earth pressures should be in accordance with Section 6-6.1.2.1 of the O.H.B.D.C. The active condition will govern earth pressure design for the yielding condition while the at-rest condition will govern earth pressure design for the unyielding condition. The following properties for backfill are recommended for design:

Material	φ	γ	K_A	K_o
Granular "A"	35°	22.8 kN/m ³	0.27	0.43
Granular "B"	30°	21.2 kN/m ³	0.33	0.50
Rock Fill	35°	18.1 kN/m ³	0.27	0.43

4.4 Approach Fills

We do not foresee stability problems for the proposed 3 to 4.5 m high approach embankments. The sides of the embankments, therefore, could be constructed with 2 horizontal in 1 vertical side slopes if granular earth fill material is used, or 1.5 horizontal in 1 vertical slopes if rock fill is used. In case of earth fill, the face of the embankment should be adequately protected against surface erosion and where the toe of the fill extends below the anticipated high water level the face of the embankment should also be protected with rip-rap against river scour.

We do not anticipate problems due to the long term settlement of the approach fills as this is estimated to be less than 50 mm.

4.5 Conclusions

In summary, the investigation has established that the bridge site is underlain by a generally 3 m thick layer of cobbles, gravel and sand followed by a deep deposit of compact fine sand which is known to extend to a depth in excess of 34 m.

.../...

The use of spread footings on rock fill, or deep foundations consisting of timber or steel-H piles can be considered as possible foundation alternatives. The most cost efficient alternative should be adopted.

4.6 Miscellaneous

The field work for the investigation was carried out under the supervision of David C. Wismath, P.Eng., using drilling equipment from Canadian Longyear. The report was prepared by Ivan P. Lieszkowszky, P.Eng.

5.0 STATEMENT OF LIMITATION

The Statement of Limitation, as quoted in Appendix "A", is an integral part of this report.

GEO-CANADA LTD.

Ivan P. Lieszkowszky
Ivan P. Lieszkowszky, P.Eng.



IPL:esp

A P P E N D I X

APPENDIX
"A"
Statement of Limitation

The conclusions and recommendations in this report are based on information determined at the borehole locations. Soil and groundwater conditions between and beyond the boreholes may differ from those encountered at the borehole locations, and conditions may become apparent during construction which could not be detected or anticipated at the time of the soil investigation.

The design recommendations given in this report are applicable only to the project described in the text, and then only if constructed substantially in accordance with details of alignment and elevations stated in the report. Since all details of the design may not be known to us, in our analysis certain assumptions had to be made. The actual conditions may, however, vary from those assumed, in which case changes and modifications may be required to our recommendations.

We recommend, therefore, that we be retained during the final design stage to review the design drawings and to verify that they are consistent with our recommendations or the assumptions made in our analysis. We recommend also that we be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the boreholes. In cases where these recommendations are not followed, the company's responsibility is limited to interpreting accurately the information encountered at the boreholes.

The comments given in this report on potential construction problems and possible methods are intended only for the guidance of the design engineer. The number of boreholes may not be sufficient to determine all the factors that may affect construction methods and costs. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work.

ENCLOSURES

RECORD OF BOREHOLE No 1

METRIC

W P 321 - 85 - 00 LOCATION Sta 13+377, O/S 6.3 m Rt. d. Hwy 546, Line 'B' ORIGINATED BY D.W.
DIST 17 HWY 546 BOREHOLE TYPE Tri-Cone; Washboring - Nx Casing; & Cone Test COMPILED BY D.W.
DATUM Geodetic DATE 1988 02 04 to 09 CHECKED BY I.P.L.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			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		
303.9	Ground Surface												GR SA SI CL
0.0	COBBLES GRAVEL AND SAND occasional boulder brown compact to very dense		1	SS	100								Soil frozen to 0.9 m Tri-Cone Wash-boring
			2	SS	17								
			3	SS	56								
			4	SS	68								
299.6			5	SS	32								
4.3	FINE SAND trace gravel some silt greyish brown loose to compact		6	SS	8								
			7	SS	10								
			8	SS	10								
			9	SS	4								
			10	SS	18								
			11	SS	6								
			12	SS	13								
289.6			13	SS	14								
14.3	END OF BOREHOLE												
	SAND compact (inferred)												
282.3	END OF CONE TEST												
21.6													

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 2

METRIC

W P 321 - 85 - 00 LOCATION Sta. 13+374.5; O/S 5.0 Lt. & Hwy 546, Line 'B' ORIGINATED BY D.W.
DIST 17 HWY 546 BOREHOLE TYPE Tri-Cone; Cone Test COMPILED BY D.W.
DATUM Geodetic DATE 1988 02 09 CHECKED BY I.P.L.

[illegible]

+³, x⁵: Numbers refer to Sensitivity



RECORD OF BOREHOLE No 3

METRIC

W P	321 - 85 - 00	LOCATION	Sta. 13+425; O/S 4.7 m Rt. & Hwy 546, Line 'B'	ORIGINATED BY	D.W.
DIST	17 HWY 546	BOREHOLE TYPE	Tri-Cone & Cone Test	COMPILED BY	D.W.
DATUM	Geodetic	DATE	1988 02 10	CHECKED BY	I.P.L.

[illegible]

+3, x5: Numbers refer to Sensitivity

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 4

METRIC

W P 321 - 85 - 00 LOCATION Sta. 13+422; O/S 5.0 m Lt. & Hwy 546, Line 'B' ORIGINATED BY D.W.
 DIST 17 HWY 546 BOREHOLE TYPE Tri-Cone & Cone Test COMPILED BY D.W.
 DATUM Geodetic DATE 1988 02 10 to 14 CHECKED BY I.P.L.

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						
303.3	Ground Surface										
0.0	COBBLES GRAVEL AND SAND occasional boulder loose to very dense		1	SS	3						72 26 2 -
			2	SS	56						46 51 3 -
300.3			3	SS	30						25 72 3 -
3.0	FINE SAND trace gravel some silt compact		4	SS	22						13 86 1 -
			5	SS	16						- 78 22 -
			6	SS	11						
			7	SS	10						
			8	SS	12						
			9	SS	15						
			10	SS	10						- 89 11 -
			11	SS	15						
			12	SS	25						
			13	WS	-						- 98 2 -
			14	SS	9						
			15	SS	15						
			16	WS	-						- 25 74 1
			17	SS	15						
			18	WS	-						- 77 23 -
			19	SS	26						
			20	WS	-						- 98 2 -
279.2			21	SS	29						
24.1	END OF BOREHOLE fine sand compact (inferred)										
274.3											
29.0	CONTINUED										

RECORD OF BOREHOLE No 4 (CONT.)

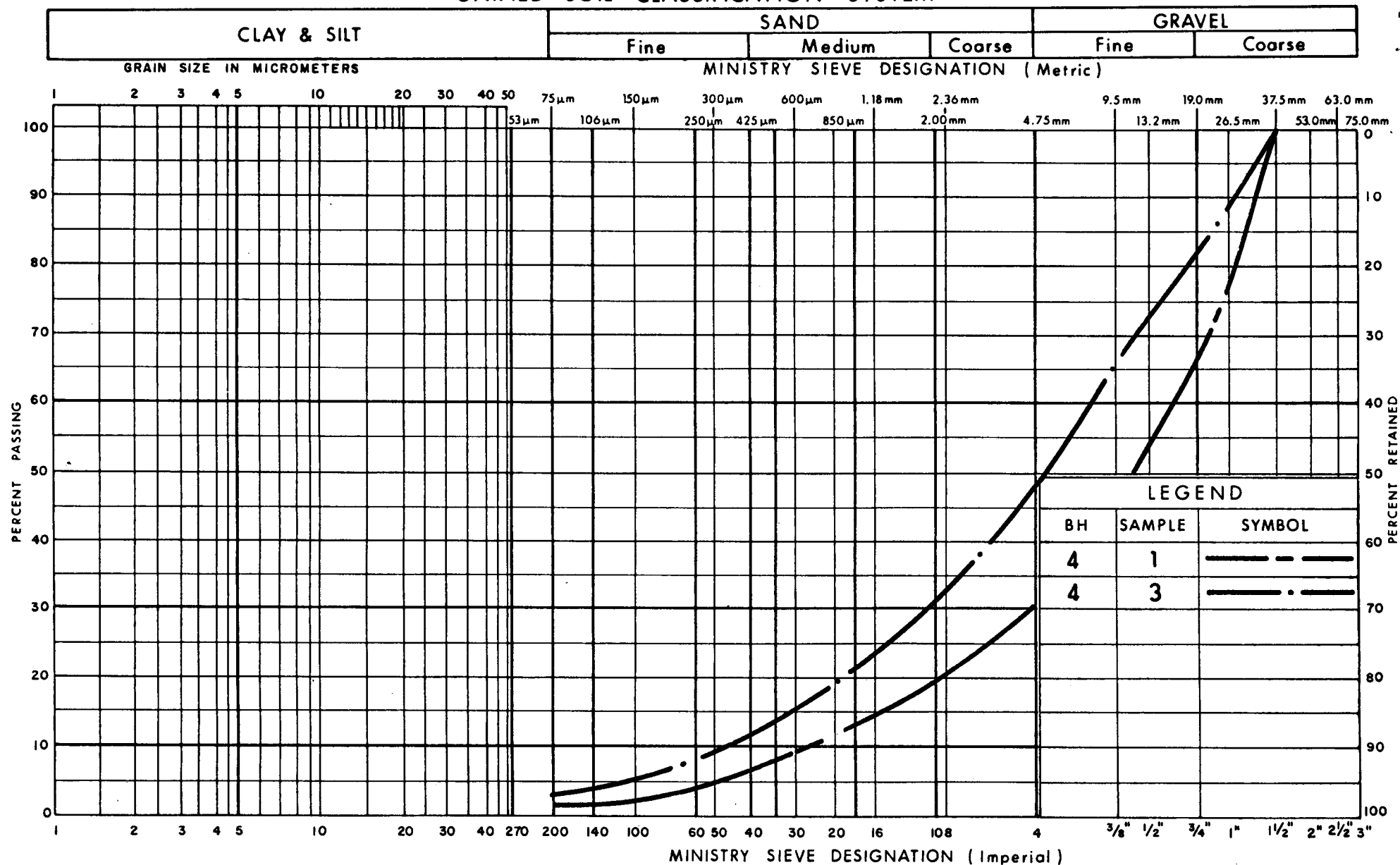
METRIC

W P 321 - 85 - 00 LOCATION Sta. 13+422: O/S 5.0 m Lt. & Hwy 546, Line 'B' ORIGINATED BY D.W.
DIST 17 HWY 546 BOREHOLE TYPE Tri-Cone & Cone Test COMPILED BY D.W.
DATUM Geodetic DATE 1988 02 10 to 14 CHECKED BY I.P.L.

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						
274.3	CONTINUATION										
29.0	FINE SAND compact to dense (inferred)						274				
							272				
							270				
269.1											
34.2	END OF CONE TEST										

OFFICE REPORT ON SOIL EXPLORATION

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

GRAIN SIZE DISTRIBUTION

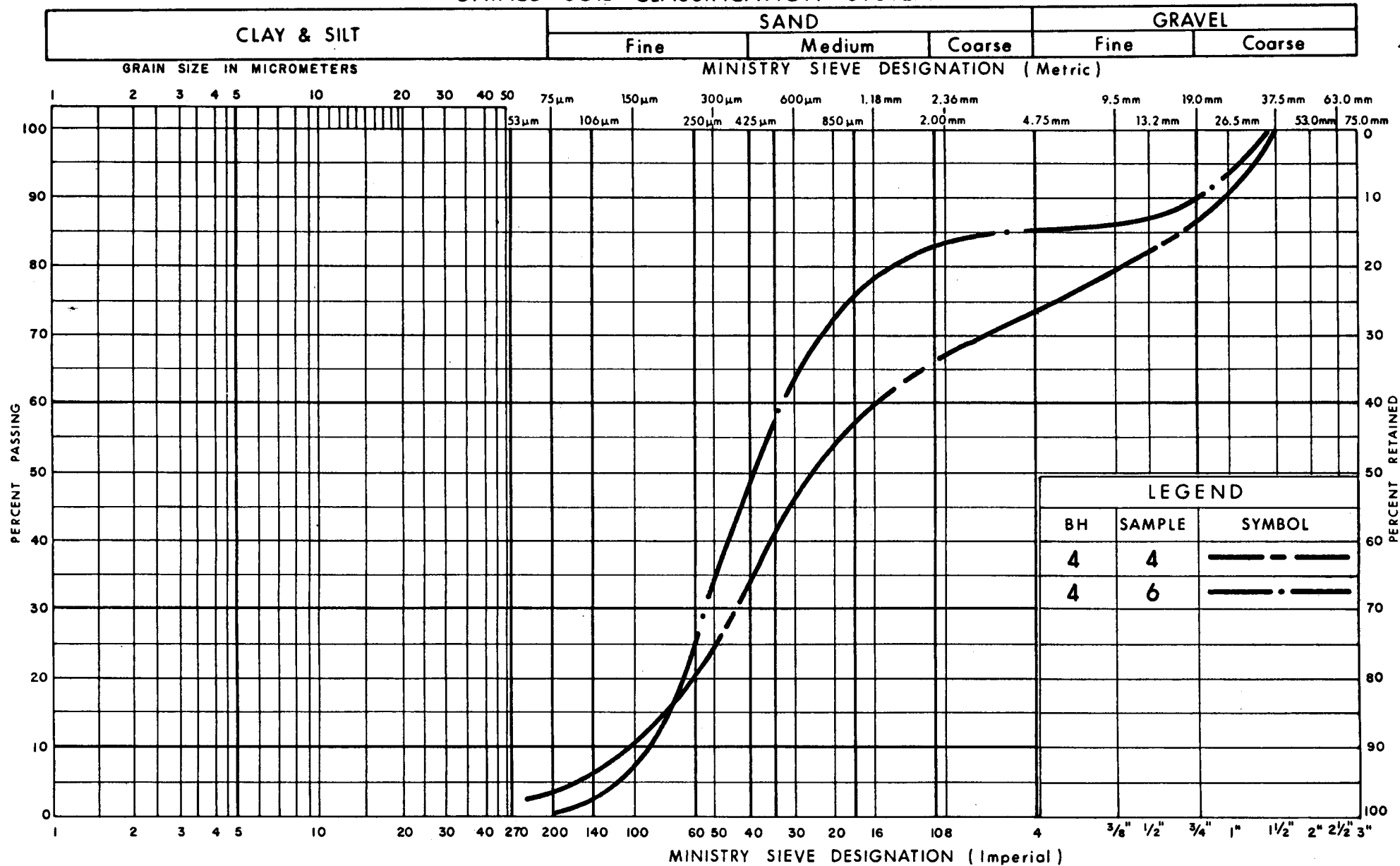
SAND AND GRAVEL

FIG No 1

W P 321-85-00

Febr. 1988

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

GRAIN SIZE DISTRIBUTION

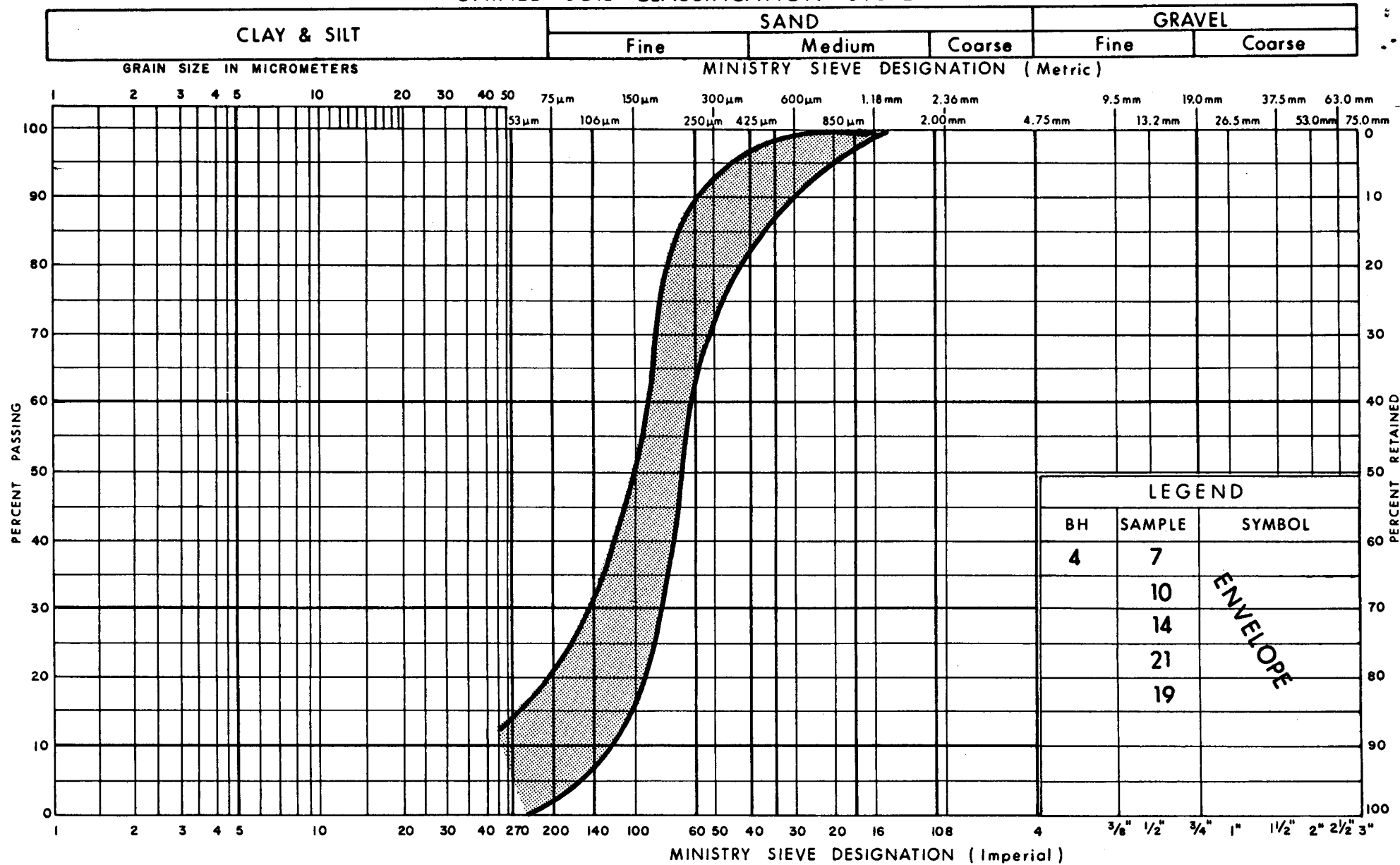
SAND
fine to medium, some gravel

FIG No 2

W P 321-85-00

Febr. 1988

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

GRAIN SIZE DISTRIBUTION

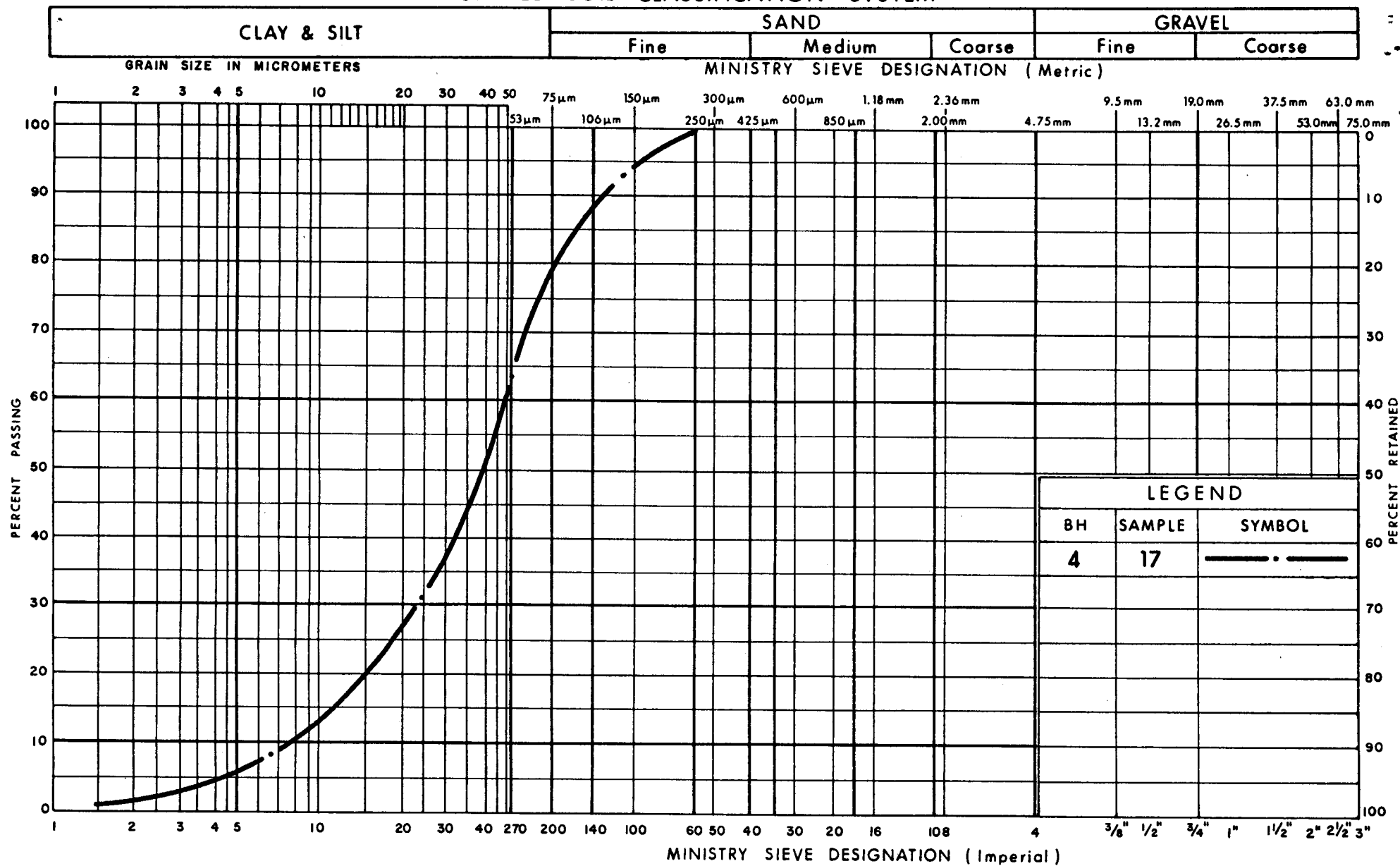
FINE SAND
trace to some silt

FIG No 3

W P 321-85-00

Febr. 1988

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

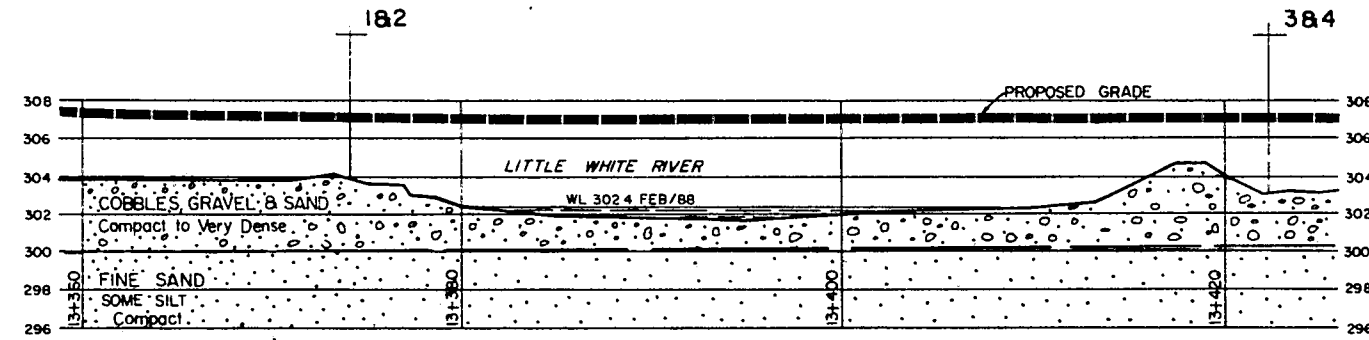
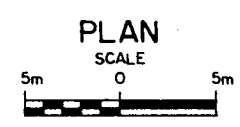
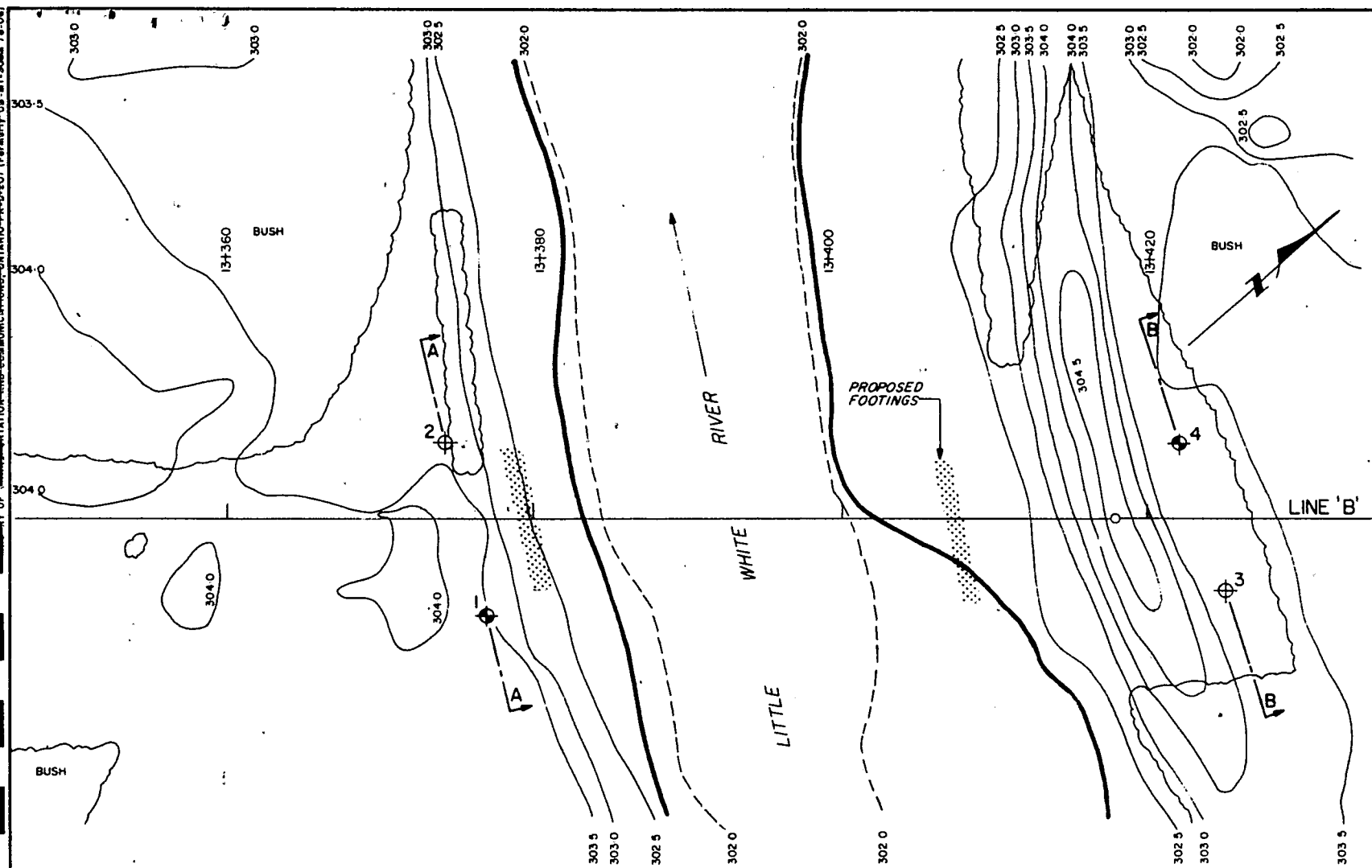
GRAIN SIZE DISTRIBUTION

SILT
Some fine sand

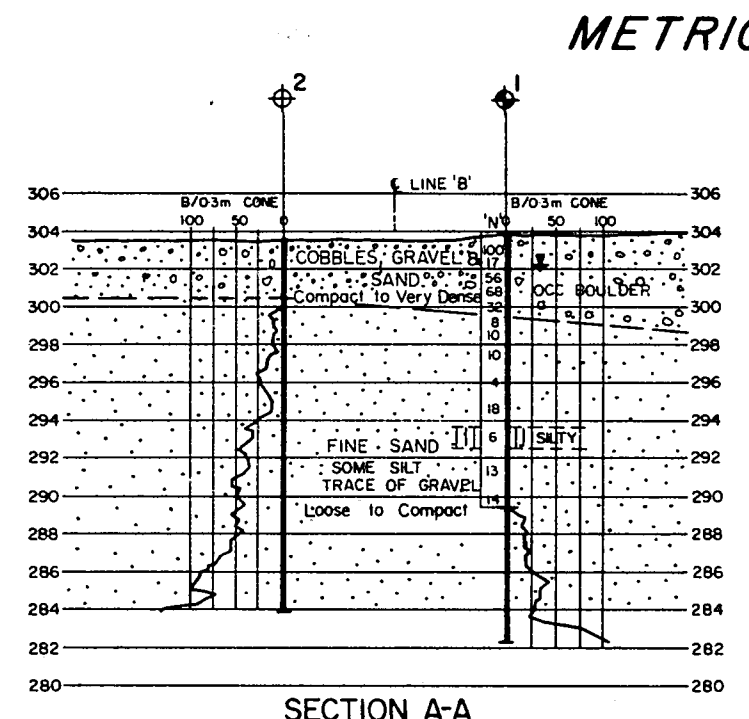
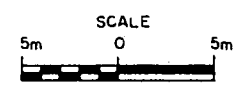
FIG No 4

W P 321-85-00

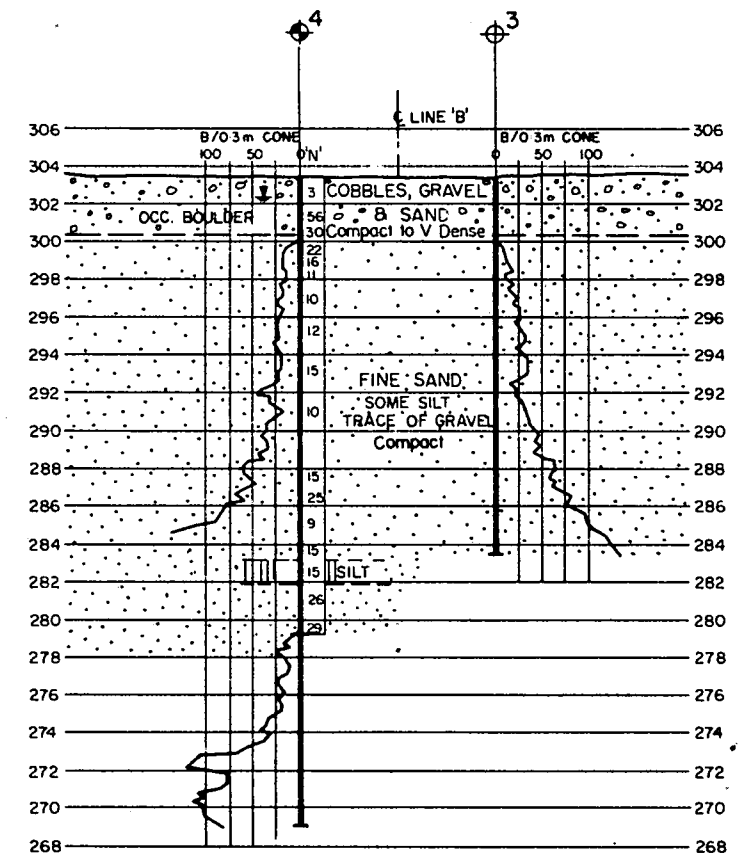
Febr. 1988



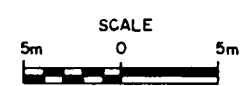
PROFILE LINE 'B'



SECTION A-A

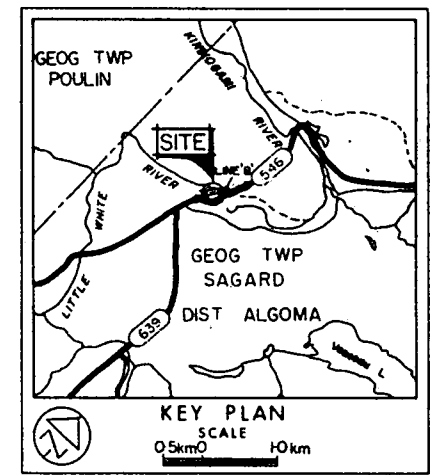


SECTION B-B



CONT No
WP No 321-85-00
LITTLE WHITE RIVER BRIDGE
BORE HOLE LOCATIONS & SOIL STRATA

GEO-CANADA LTD.



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 Feb 1988

No	ELEVATION	STATION	OFFSET
1	303.9	13+377.0	6.3m Rt
2	303.5	13+374.5	5.0m Lt
3	303.3	13+425.0	4.7m Rt
4	303.3	13+422.0	5.0m 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.

REVISIONS	DATE	BY	DESCRIPTION

Geocres No 41J-48	HWY No 546	DIST 17
SUBMD D.W. CHECKED P. DATE Feb 1988	SITE 38-57	
DRAWN H.A. CHECKED	APPROVED	DWG 3218500-A

FOUNDATION INVESTIGATION REPORT

CONTRACT NO 89-239



Ontario

Ministry of
Transportation and
Communications

I N D E X

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	For
	Little White River Bridge
	W.P. 321-85-01; Site 38-57
	Hwy. 546, District 17, Sudbury

NOTE: For the purposes of this contract, this report supersedes all other reports prepared by or for the Ministry in connection with the above-noted project.

EXPLANATION OF TERMS USED IN REPORT

2

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

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 = $\frac{c_u}{\tau_r}$

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

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						

FOUNDATION INVESTIGATION REPORT
FOR
LITTLE WHITE RIVER BRIDGE
WP321-85-01, SITE 38-57
HIGHWAY 546, DISTRICT 17, SUDBURY

1.0 INTRODUCTION

This report contains the results of a foundation investigation carried out at the site of the above mentioned project.

The investigation and study was undertaken by Geo-Canada Ltd. on behalf of the Ontario Ministry of Transportation under Consultants Agreement No. 4238-9087-186.

The purpose of the investigation was to determine the subsurface and foundation conditions at the proposed bridge abutment locations and to provide the necessary geotechnical input for the foundation design of the proposed new structure.

The field work was carried out during the period of 1988 02 04 to 15, and consisted of two sampled boreholes (one drilled on each side of the river), a cone penetration test adjacent to one of the boreholes, two cone penetration tests extended from the base of the boreholes, and two separate cone penetration tests. The borings were advanced by washboring and rotary drilling (tri-coning) techniques to depths ranging between 14.3 and 24.1 m below the existing ground surface. With the additional cone tests, which were put down from the bottom of the boreholes, the depth of exploration was extended to 21.6 and 34.2 m respectively.

2.0 DESCRIPTION OF SITE AND GEOLOGY

The site is located on Secondary Highway 546, near its junction with Highway 639, approximately 40 km north of the Town of Elliot Lake. At present, the Highway is carried over the Little White River on a single span double Bailey bridge. The river, at the points of the existing and proposed crossings follows a generally east to west course.

The topography of the general area is very hilly to small mountainous with steep sided rock formations rising above numerous lakes and small streams and rivers. Several wide valleys wind through the area. In one of these valleys lies the aforementioned bridge site.

.../...

Locally, the river valley is approximately 300 m wide and runs generally in a north-east to south-west direction and the Little White River winds its way through this valley. At the bridge site, the river is approximately 25 m wide and 0.7 m deep at its centre at the time of our drilling. The current is fast and the river bed is lined with cobbles of diameters ranging from 60 to 180 mm (average 125 mm) in size.

At the inside of the river bends, including the north abutment location, berms or narrow ridges consisting of cobble and coarse sand have been deposited to heights of approximately 2.5 m above the river or ice level. At the outside bends of the river (e.g. at the south abutment location) the banks show signs of some scouring and slight undercutting.

The vegetation in the area consists of coniferous (spruce) and deciduous (poplar) mature forests with some sumac and sage brush around the river and wet areas.

Approximately 80 m to the east of the proposed bridge location the existing alignment of Highway 546 crosses the river over a double Bailey bridge. This structure, approximately 30 m long and one lane wide, is supported on large timber cribs at each abutment. The cribs are rock

.../...

filled and appear to be free from the effects of scour and settlement.

The nearest rock outcrop is about 250 m to the east of the proposed bridge and is part of a steep sided ridge of rock running in a north-east to south-west direction. Two outcrops at this location, approximately 50 m apart, were examined by us. One of the outcrops was composed of metasedimentary rock, high in quartzite and was fairly intact and massive. It had nearly horizontal medium spaced bedding planes and two joints, one nearly vertical and the other about 60 degrees to the horizontal. The rock exposed in the other outcrop was more igneous in nature, also high in quartzite and feldspar, and was badly weathered and fractured. It had no apparent bedding plane and no regular joint pattern.

Geologically, the site is located on the Canadian Pre-Cambrian Shield. Geological maps indicate that the Little White River is near the boundary between the early Pre-Cambrian predominantly granitic rocks to the north and the Middle Pre-Cambrian metasedimentary rocks of the Huronian Super Group which comprise conglomerates, metamorphosed sandstones and siltstones to the south. There is a major east-west oriented fault line approximately 10 km south of

.../...

the site, and another north-south fault line about 15 km to the north. During Pleistocene times the area was covered by glaciers which left a mantle of overburden consisting of glacial drift.

3.0 SUMMARIZED SUBSURFACE CONDITIONS

3.1 General

The general subsurface profile at the site consists of a 3 to 4 m thick surficial layer of cobbles, gravel and sand underlain by a deep, over 30 m thick deposit of fine sand. The surface of the Pre-Cambrian bedrock was not encountered at any of the test locations, i.e. within a depth of 34 m (Elevation 269 m) at the north-west quadrant of the site.

The groundwater table at the boreholes was encountered at shallow depths and its level (Elevations 302.3 to 302.4 m) coincided with the river level which at the time of the investigation was at Elevation 302.4 m.

The subsurface conditions are described in detail on the individual borehole logs and the main characteristics of the soil types encountered are discussed briefly in the following sections.

****NOTE:** Refer to BH # 1 to # 4 (Appendix) for specific conditions at each borehole. Refer to Drawing No. 2 of the Contract Drawings for borehole locations and stratigraphical profiles.
.../...

3.2 Cobbles, Gravel and Sand

The surficial soil deposit encountered in the boreholes is a 3 to 4.3 m thick (average 3 m) coarse granular deposit consisting of cobbles, gravel and coarse sand with the occasional boulder. Grain size analyses performed on particles smaller than 38 mm indicate 72 to 46% gravel, 51 to 26% sand, and 2 to 3% soil fines. The soil fines are non-plastic silt. Grading curves are shown on Figure 1. The cobble size particles range generally between 60 and 180 mm (average 125 mm). The frequency of the cobbles decreases with depth.

The standard penetration resistances in the deposit ranged between 3 and 68 blows per 0.3 m (average > 30), indicating a loose to very dense, but generally dense compactness condition. As some of the high penetration resistances ("N"-values) were undoubtedly affected by the gravel or larger particle sizes, the compactness condition of the deposit is probably less than that inferred from the standard penetration tests.

3.3 Fine Sand

Below the surficial cobble and gravel deposit is a thick bed of fine sand. Its thickness at the test locations is in

.../...

excess of 20 to 30 m.

Near its surface, the deposit consists mainly of fine to medium sand with some gravel (13 to 25%) and a trace of silt (1 to 3%). The results of the grain size analyses performed on two representative samples obtained from this zone are attached as Figure 2.

With depth the sand becomes finer textured and the silt content also increases. The results of the grain size analyses are presented graphically on Figure 3, showing 77 to 98% sand, and 2 to 23% silt.

In Borehole 4, interbedded with the sand at about Elevation 282 m, an approximately 1.0 m thick layer of silt was encountered. The particle size distribution of this material is shown on Figure 4 indicating 25% fine sand and 75% silt.

The standard penetration tests (SPT) performed in the sand gave "N" values ranging from 4 to 29 blows per 0.3 m, but typically between 10 and 15. These values indicate a loose to compact, but generally compact deposit. It is believed that some if not most of the SPT results were affected by the washboring operations and the inevitable disturbance of the soil by the drilling and sampling techniques used. For this

.../...

reason, we are of the opinion that the compactness condition of the sand is higher than that inferred from the SPT results.

3.4 Groundwater

It is expected that the position of the groundwater table in the shallow banks adjacent to the river is governed by the water level in the river. At the time of the investigation the river level was at Elevation 302.4 m, and the free standing water level in the boreholes was recorded between Elevations 302.3 and 302.4 m.

NOTE: The preceding report is a copy of the factual information from the Foundation Investigation Report prepared by Geo-Canada Ltd. (consulting geotechnical engineers for this project), under the technical supervision of the MTO Foundation Design Section.



D. H. Dundas
D. H. Dundas, P. Eng.
Sr. Foundation Engineer

M. Devata
M. Devata, P. Eng.
Chief Foundation Engineer

A P P E N D I X

RECORD OF BOREHOLE No 1										METRIC			
W P 321 - 85 - 00		LOCATION Sta 13+377, O/S 6.3 m Rt. d. Hwy 546, Line 'B'					ORIGINATED BY D.W.						
DIST 17 HWY 546		BOREHOLE TYPE Tri-Cone; Washboring - Nx Casing; & Cone Test					COMPILED BY D.W.						
DATUM Geodetic		DATE 1988 02 04 to 09					CHECKED BY I.P.L.						
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	SHEAR STRENGTH					
303.9	Ground Surface												
0.0	COBBLES GRAVEL AND SAND occasional boulder brown compact to very dense		1	SS	100								Soil frozen to 0.9 m
			2	SS	17								
			3	SS	56								
			4	SS	68								
			5	SS	32								
299.6	FINE SAND trace gravel some silt greyish brown loose to compact		6	SS	8								
4.3			7	SS	10								
			8	SS	10								
			9	SS	4								
			10	SS	18								
			11	SS	6								
			12	SS	13								
			13	SS	14								
289.6	END OF BOREHOLE												
14.3	SAND compact (inferred)												
282.3	END OF CONE TEST												
21.6													



OFFICE REPORT ON SOIL EXPLORATION

CONE TEST

135/ 0.25 m

METRIC

W P 321 - 85 - 00 LOCATION Sta. 13+425; O/S 4.7 m Rt. & Hwy 546, Line 'B' ORIGINATED BY D.W.
DIST 17 HWY 546 BOREHOLE TYPE Tri-Cone & Cone Test COMPILED BY D.W.
DATUM Geodetic DATE 1988 02 10 CHECKED BY I.P.L.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIMIT LIQUID W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH		WATER CONTENT (%)			
303.3	Ground Surface							20 40 60 80 100					
0.0	COBBLES GRAVEL AND SAND (inferred)												
300.3			1	WS	-								
3.0	FINE SAND compact (inferred)												
283.5	END OF CONE TEST												
19.8													

20 40 60 80 100

○ UNCONFINED + FIELD VANE

● QUICK TRIAXIAL x LAB VANE

W_p W W_L

WATER CONTENT (%)

302

300

298

296

294

292

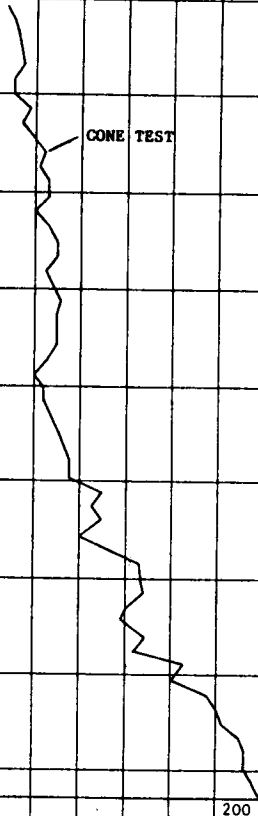
290

288

286

284

CONE TEST



Tri-Cone Cone Test

200

+3, x5: Numbers refer to Sensitivity

20
15 
10

RECORD OF BOREHOLE No 4

METRIC

W P 321 - 85 - 00 LOCATION Sta. 13+422; O/S 5.0 m L.L. & Hwy 546, Line 'B' ORIGINATED BY D.W.
 DIST 17 HWY 546 BOREHOLE TYPE Tri-Cone & Cone Test COMPILED BY D.W.
 DATUM Geodetic DATE 1988 02 10 to 14 CHECKED BY I.P.L.

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						
303.3	Ground Surface										
0.0	COBBLES GRAVEL AND SAND occasional boulder loose to very dense		1	SS	3		302				72 26 2 -
			2	SS	56						
300.3			3	SS	30						
3.0	FINE SAND trace gravel some silt compact		4	SS	22		300				46 51 3 -
			5	SS	16						25 72 3 -
			6	SS	11		298				13 86 1 -
			7	SS	10		296				- 78 22 -
			8	SS	12		294				
			9	SS	15		292				
			10	SS	10		290				- 89 11 -
			11	SS	15		288				
			12	SS	25		286				
			13	WS	-		284				- 98 2 -
			14	SS	9		282				
			15	SS	15		280				- 25 74 1
			16	WS	-		278				- 77 23 -
			17	SS	15		276				- 98 2 -
			18	WS	-						
			19	SS	26						
			20	WS	-						
			21	SS	29						
279.2	END OF BOREHOLE										
24.1	fine sand compact (inferred)										
274.3											
29.0	CONTINUED										

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 4 (CONT.)

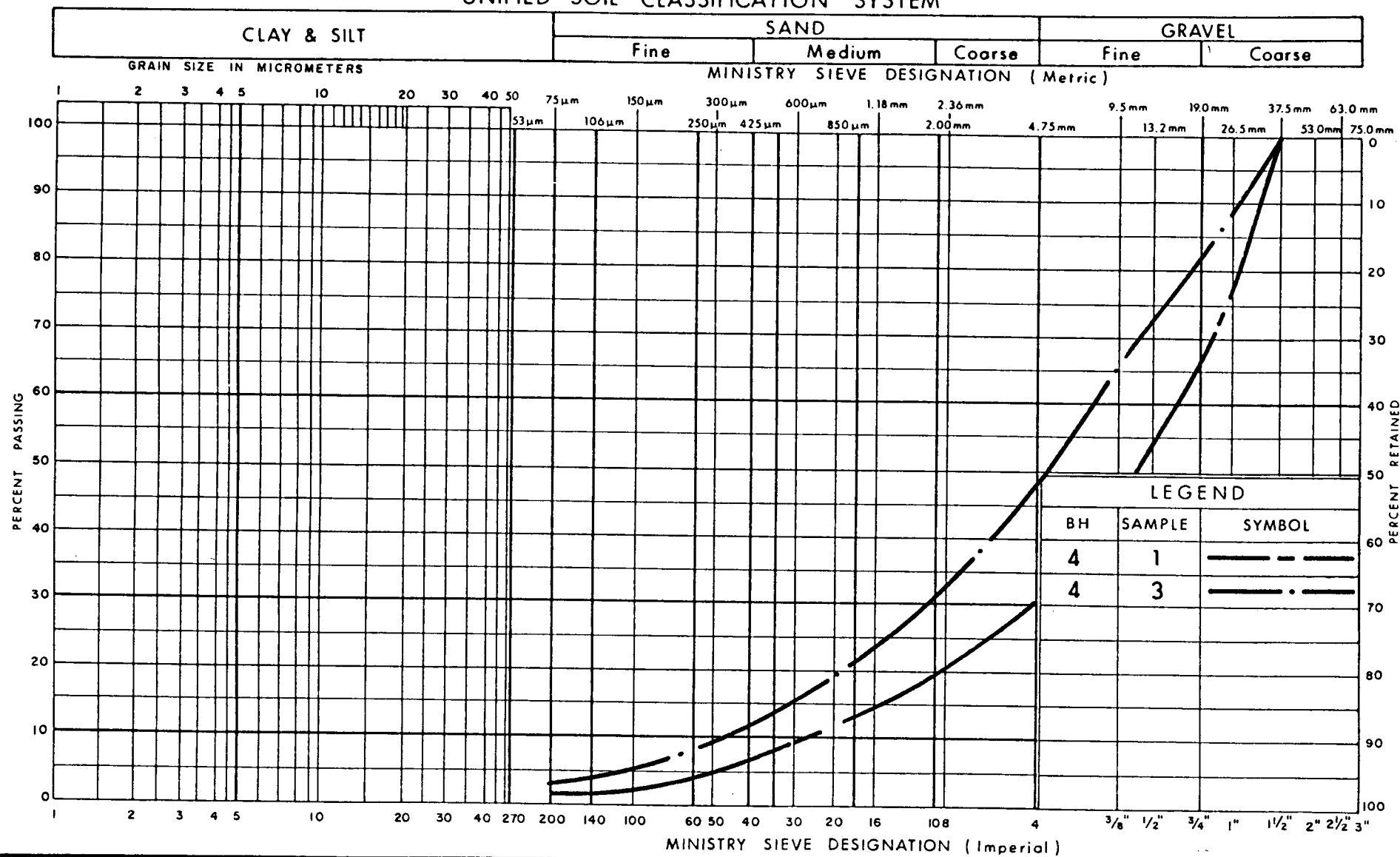
METRIC

W P 321 - 85 - 00 LOCATION Sta. 13+422; O/S 5.0 m L.L. & Hwy 546, Line 'B' ORIGINATED BY D.W.
 DIST 17 HWY 546 BOREHOLE TYPE Tri-Cone & Cone Test COMPILED BY D.W.
 DATUM Geodetic DATE 1988 02 10 to 14 CHECKED BY I.P.L.

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						
274.3	CONTINUATION										
29.0	FINE SAND compact to dense (inferred)						274				
							272				
							270				
269.1											
34.2	END OF CONE TEST										

OFFICE REPORT ON SOIL EXPLORATION

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

GRAIN SIZE DISTRIBUTION

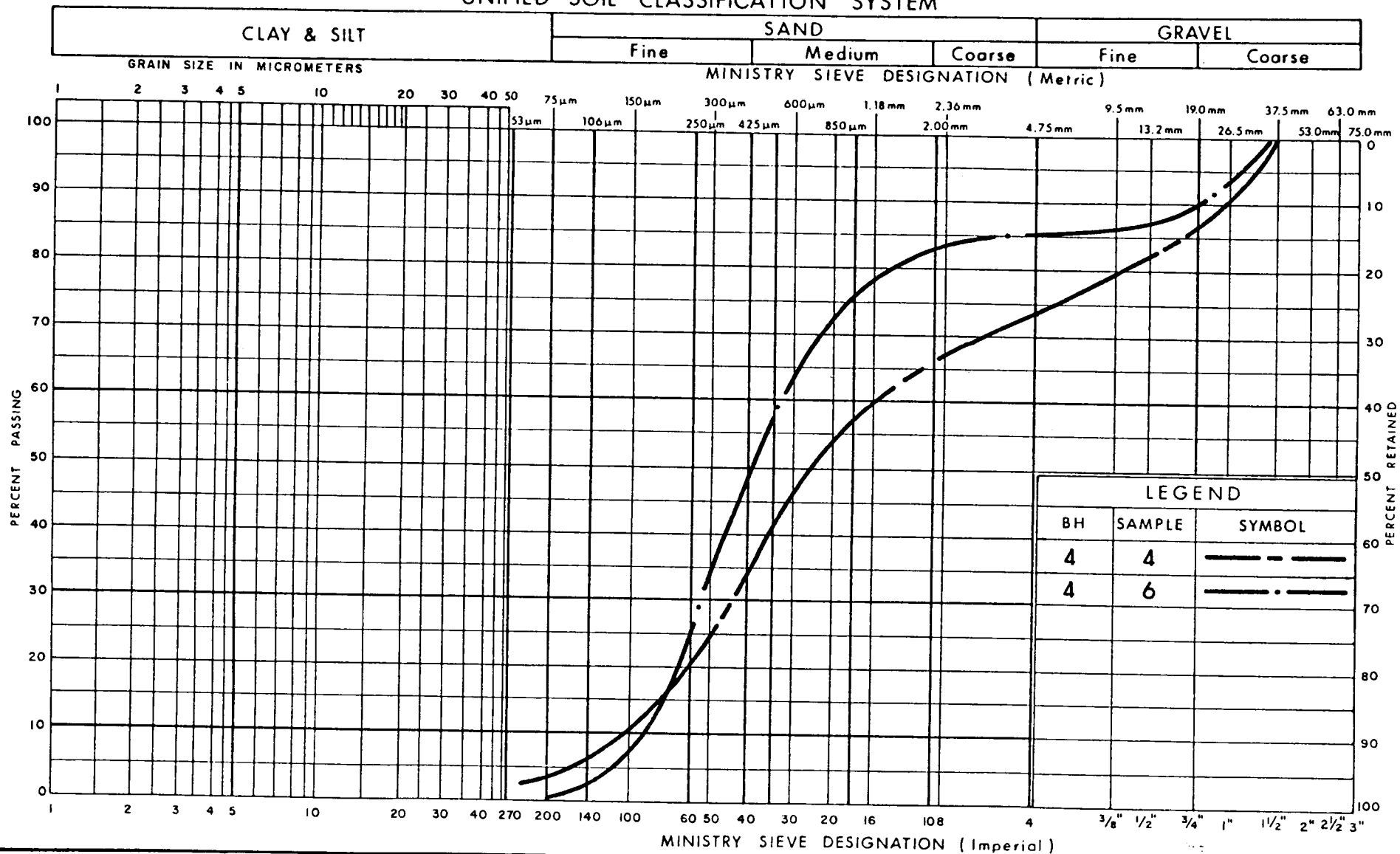
SAND AND GRAVEL

FIG No 1

W P 321-85-00

Febr. 1988

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
SAND
fine to medium, some gravel

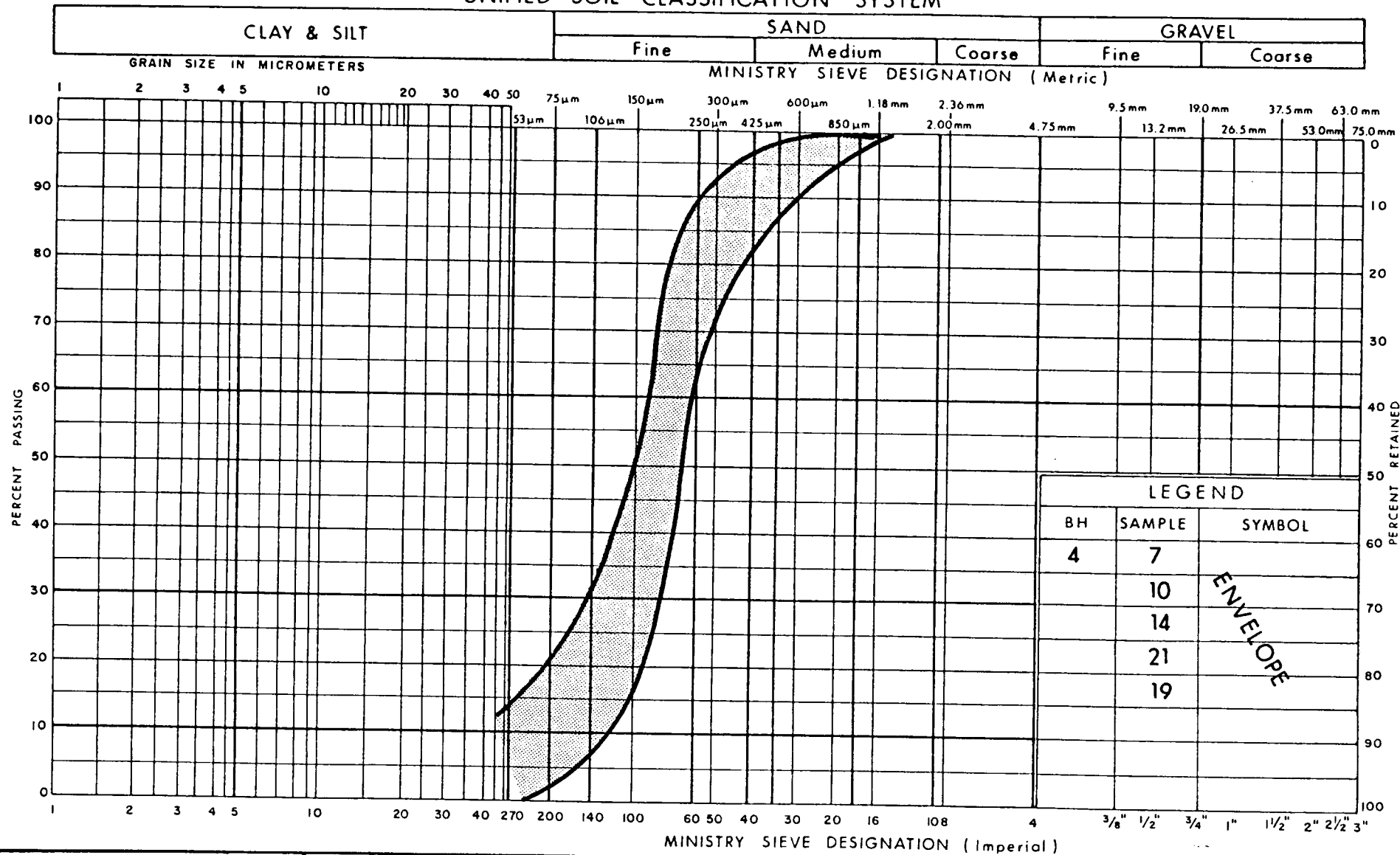
FIG No 2

W P 321-85-00

Febr. 1988

88

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
FINE SAND
trace to some silt

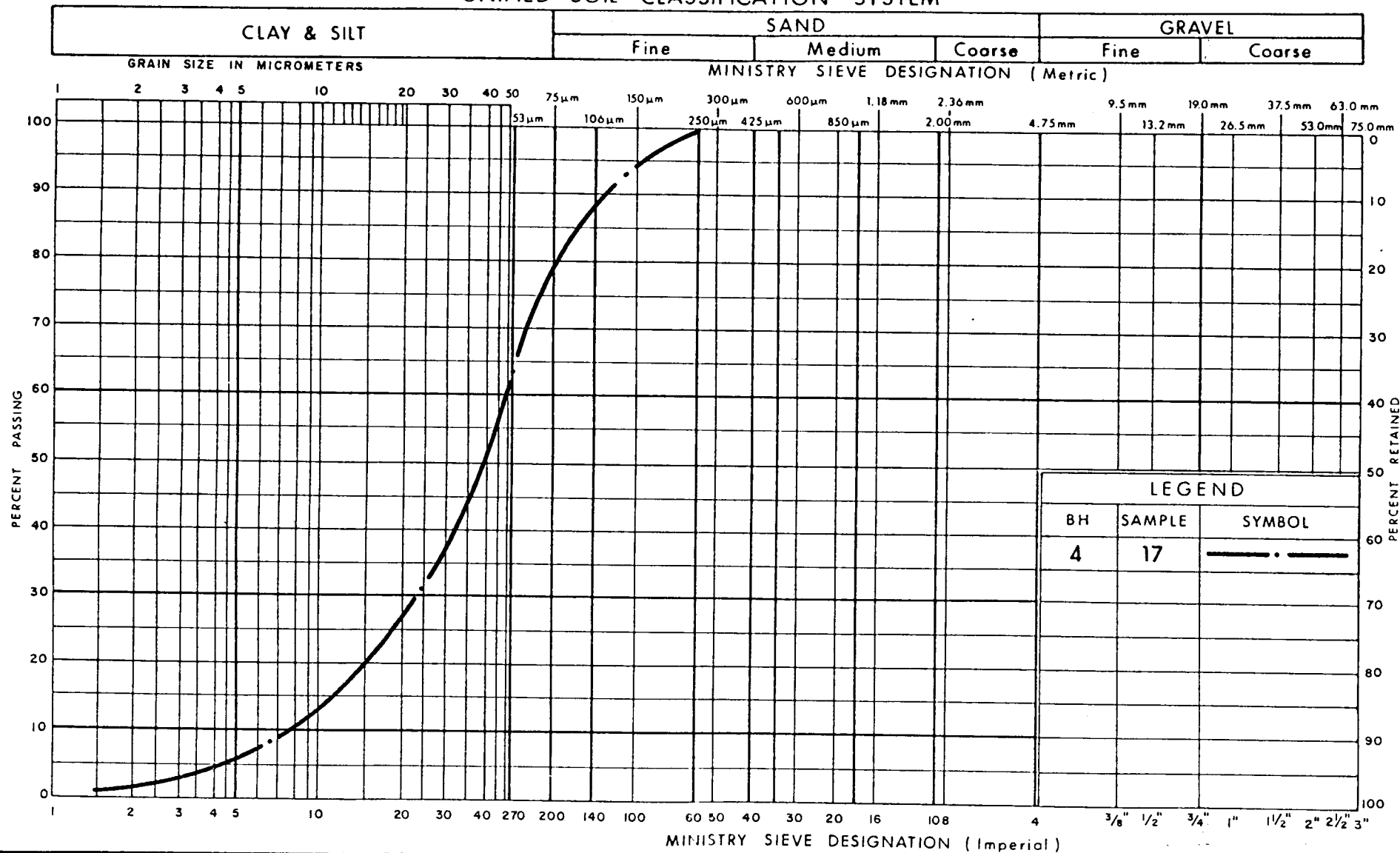
FIG No 3

W P 321-85-00

Febr. 1988

61

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

GRAIN SIZE DISTRIBUTION

SILT

Some fine sand

FIG No 4

W P 321-85-00

Febr. 1988

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

T. 17
CONT No
WP. No 321-85-01



LITTLE WHITE RIVER BRIDGE
GENERAL ARRANGEMENT
SHEET

LIST OF DRAWINGS

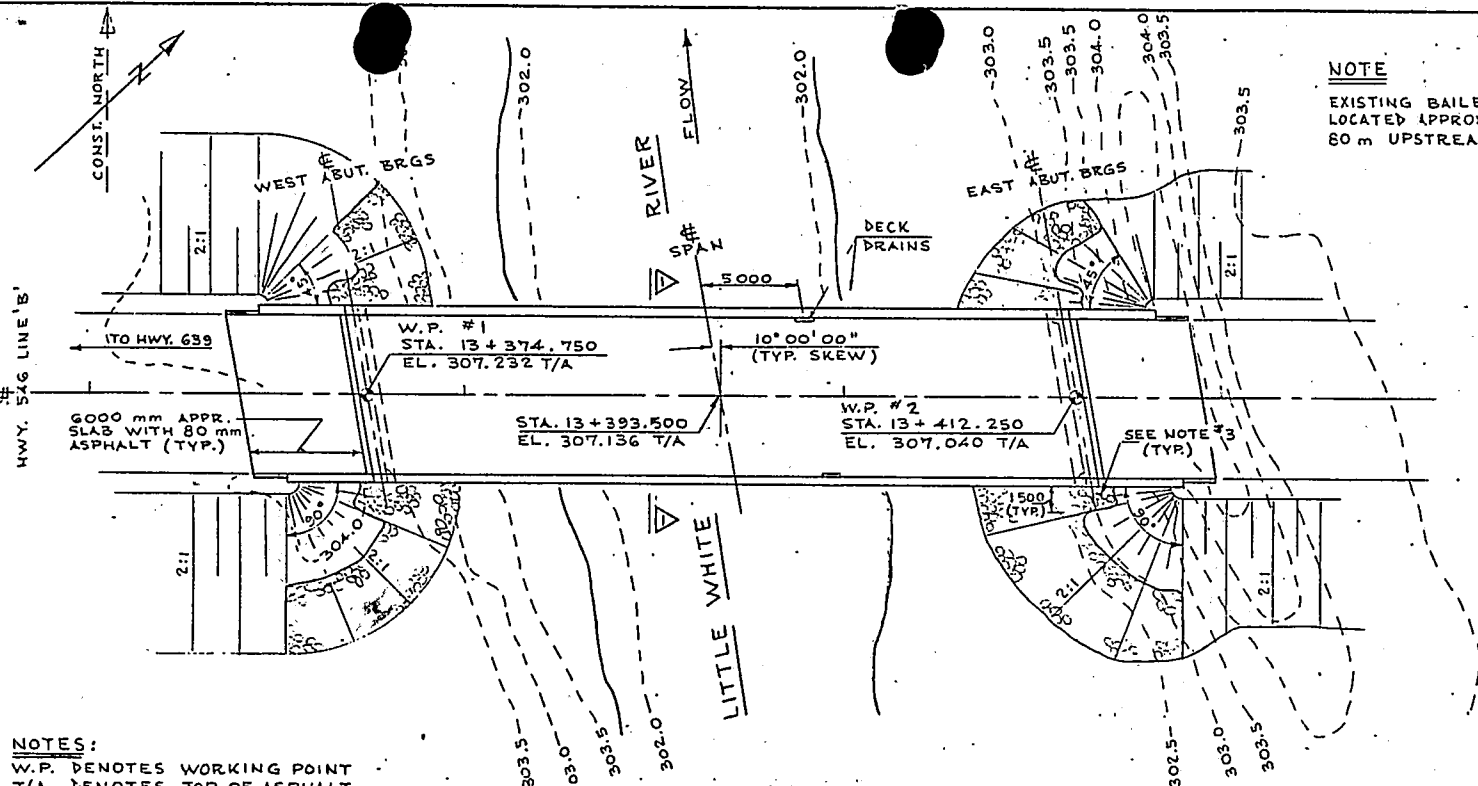
- 38-57-1 GENERAL ARRANGEMENT
- 2 BORE HOLE LOCATIONS & SOIL STRATA
- 3 FOOTING LAYOUT & DETAILS
- 4 ABUTMENTS
- 5 STRUCTURAL STEEL
- 6 SPLICE DETAILS & BEARINGS
- 7 DECK LAYOUT & DETAILS
- 8 BARRIER WALL
- 9 JOINT ANCHORAGE AND ARMOURING
- 10 6000 mm APPROACH SLAB
- 11 BRIDGE DATE & SITE NUMBER DATA
- 12 PILE DRIVING- STEAM & DIESEL HAMMERS
- 13 AS CONSTRUCTED ELEV. & DIM.
- 14 STANDARD DETAILS
- 15 QUANTITIES - STRUCTURE - I
- 16 QUANTITIES - STRUCTURE - II

NOTES:

REINFORCING STEEL
REINFORCING STEEL SHALL BE GRADE 400
UNLESS OTHERWISE SPECIFIED.
BAR MARKS WITH SUFFIX 'C' DENOTES
COATED BARS.

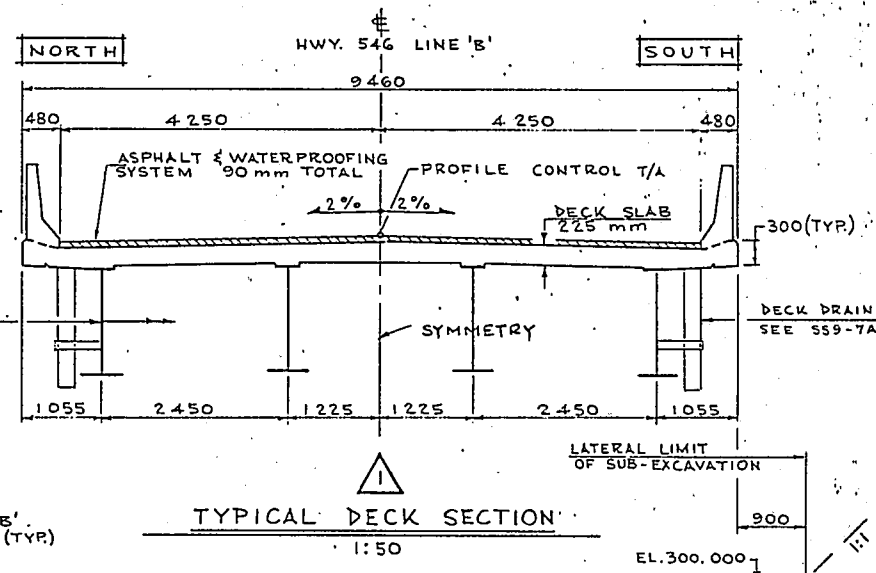
CLASS OF CONCRETE 30MPa
FOOTINGS 20MPa
REMAINDER 30MPa

CLEAR COVER TO REINFORCING STEEL
FOOTINGS 100 ± 25 mm
ABUTMENTS & WINGWALLS
FRONT FACE 80 ± 20 mm
BACK FACE 70 ± 20 mm
DECK TOP 70 ± 20 mm
BOTTOM 40 ± 10 mm
BARRIER WALLS 70 ± 20 mm
APPROACH SLABS 80 ± 20 mm
UNLESS OTHERWISE NOTED ON DRAWINGS.

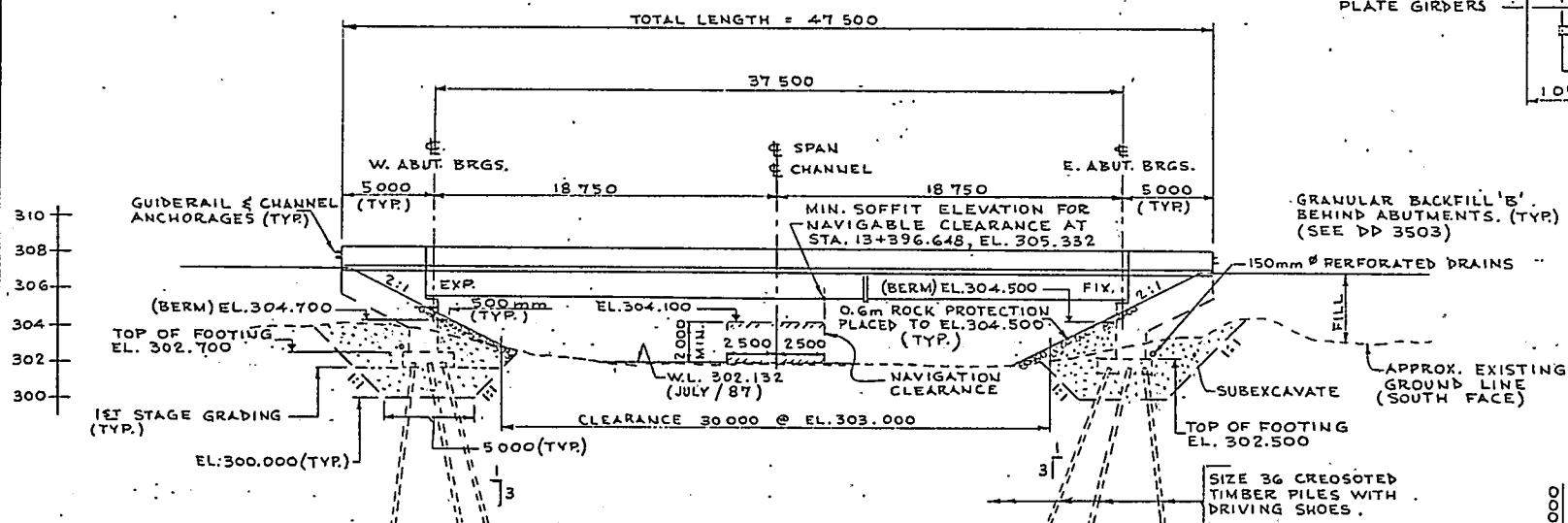


NOTES:
W.P. DENOTES WORKING POINT
T/A DENOTES TOP OF ASPHALT.
BM DENOTES BENCH MARK

PLAN
1:200

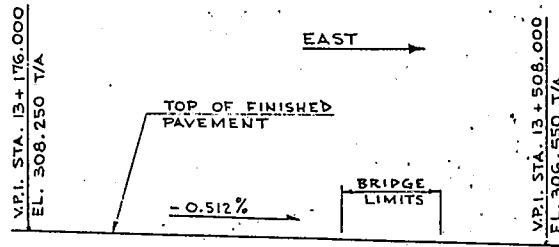


TYPICAL DECK SECTION
1:50

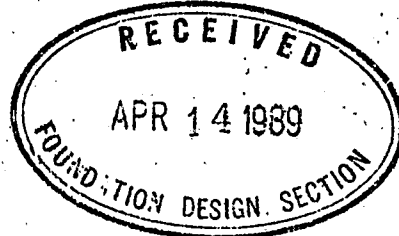


ELEVATION
(SOUTH FACE)
1:200

- NOTES
- #1 TO FACILITATE PILE DRIVING SUBEXCAVATE TO EL. 300.000 AND BACKFILL WITH SAND AND GRAVEL. (MAX. PARTICLE SIZE 50mm DIA.) (SEE GRADING DRAWINGS)
 - #2 FOR LATERAL TRANSVERSE LIMIT OF SUB-EXCAVATION SEE SECTION A
 - #3 PROVIDE ROCK PROTECTION TO EL. 305.400 AND FOR A HORIZONTAL DISTANCE OF 1.5m FROM EXPANSION JOINT DOWNSPOUT DRAINS. (SEE DWG. 14)



PROFILE AT HWY. 546 LINE 'B'
N.T.S.



APPLICABLE STANDARD DRAWINGS
OPS 508.02 (BRIDGE DECK WATERPROOFING)
DD-3503 MINIMUM GRANULAR BACKFILL REQUIREMENTS

B.M. 304.154
RAILWAY SPIKE IN FACE 0.3 SPRUCE
36.2 LT. 13+344.0

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

DATE	BY	DESCRIPTION
DESIGN 13/SS/CHK	CODE 04/DC-EZ	LOAD 11/11/87
DRAWN 6/FW/CHK	J.B.	SITE 38-57
		STRUCT
		SCHEME
		DWG. 1

memo

To : File

Date : 90 07 13

Re: Contract 89-239

WP 321-85-00

Little White River Bridge

Hwy 546, Dist 17, Sudbury

The site is near Eliot Lake

On July 12/90, N. Reg. Quality Assurance referred me to John Goguen, construction supervisor for this project.

John advised that the contractor (Birmingham) was experiencing difficulty in driving the timber piles at this site.

I called Per Forst who referred me to Bert Farago of Struct. Office. On July 13 I tried reaching Bert or John Brown but both were on vacation. I spoke to George Al-Bezi who subsequently concurred with our recommendations.

After discussions with George Meynold and Basil Gallant I determined (w/ John Goguen)

- 1) they are working on east abut.
- 2) the west abut has not started
- 3) they are encountering generally hard driving (15 blows per inch)
- 4) 3 piles were not driven to spec.
4th from S in middle & approx 6 from N in outside row had compound batter & only 6.5 m penetration instead of 12[±] m
the 6 from S in centre row was broken at 4 m
- 5) the site had been prepared by subexcavating 2[±] m (bouldery zone) and backfilling with sand

c) the subsoil is sand

/ recommended

- 1) generally to drive from inside out to minimize classification of sand
- 2) they are probably better buildings after investigation digging them out or prearranging in such a confined space in soft soil (need casing) was not practical
- 3) the two deflected piles are acceptable (they have good set)
- 4) the broken pile should be removed & the remaining 8 m piece redriven
- 5) if the pile can't be removed drive beside it
- 6) if the redriven pile doesn't go more than 4 m or doesn't get good set, drive another pile between it and the problem pile in the outside row.

These recs were discussed with M. Dente & George Al-Bazji & given to John Gallant the George Maynard of Birmingham.

D. Dardas
Sr. Febr Eng.

memorandum



To: B. Farago
Design Engineer
Design Section
4th floor, 3501 Dufferin St.

Date: 1989 04 17

Attn: J. Brown
Project Engineer

From: Foundation Design Section
Room 315, Central Bldg.

RE: Final Design Drawing Review
Little White River Bridge, Hwy. 546
W.P. 321-85-01, Site 38-57
District 17, Sudbury

The final design drawings have been reviewed by this Section, and appear to be in conformance with geotechnical related recommendations and requirements.

A handwritten signature in black ink, appearing to read "T. Sangiuliano".

T. Sangiuliano, P. Eng.
Foundation Engineer

for

M. Devata, P. Eng.
Chief Foundation Engineer

MD/TS/jb

memorandum

Tel: (416) 235-4959



To: Mr. M. Devata
Chief Foundation Engineer
Foundation Design Section

Date: 89 04 04

Attn: Mr. T. Sangiuliano
Foundation Engineer

Re: Little White River Bridge (Foundations)
W.P. 321-85-01, Site 38-57

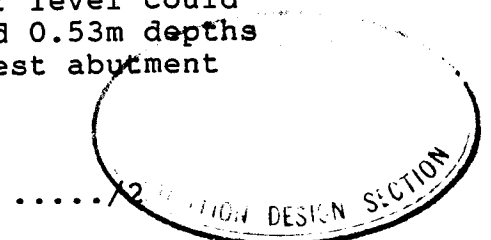
This letter will acknowledge consideration of the alternative recommendations made in Mr. Sangiuliano's memo of 1989-02-13 for the above structure.

A. The following points are noted:

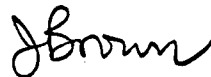
- i) The footing design was originally based on the recommendations in Mr. Devata's memo of 1988-05-09.
- ii) The depth of frost cover included consideration for the rock protection and was originally taken from the Structural Manual DA 2-3.
- iii) The preliminary drawing distribution for the General Arrangement Drawing was done on 1988-11-07.
- iv) The design was undertaken in house and the drafting was completed out of house by Feb. 1989.

B. Comments in regards to your memo are as follows:

1. As shown on attached Table #1, any advantage obtained by increasing the capacity of the piles (by lowering the pile tip to Elev. 285.0) appears to be offset by increased (length) material ~~and thus not cost effective~~, even though the total quantity of piles and shoes would be reduced.
2. a. Although the seasonal river level could vary there is only 0.3m and 0.53m depths of water at the east and west abutment footings respectively.



- B. 2. b. Because of the shallow water depths it was originally considered feasible to pour the bottom half of the footings underwater (tremie) although the design presently includes a tender item for unwatering.
- c. The footings could be raised in order to avoid unwatering but in order to maintain adequate frost cover the span would require lengthening by approximately two meters.
- d. The cost of increasing the span length by two meters has not been compared with the cost of possible unwatering.
- 3. The economic feasibility of preaugering while advancing a steel liner in lieu of subaqueous excavation was not determined as there was little known precedent and corresponding data base for comparative cost study.



J. Brown
Project Engineer

JB/sl
Encl:

c.c. B. Farago
P. Furst

Little White River Bridge.
W.P. 321-85-01, Site 33-57

TABLE * 1

Cost Effectiveness of pile tip elevation vs. capacity.

	Tip. Elev.	Total Length	Capacity (kN)	
			SLS	ULS
OPTION (A). GEO CANADA (1988)	El. 290.0	12m ±	250	375
OPTION (B). (as above, Page 1)	El. 285.0	17m ±	330	550
ratio $\frac{B}{A}$		1.41	1.32	1.33

Pile length is increased by approximately 41%
but capacity by only 32%.

MARCH 1980

MINIMUM FROST PROTECTION TO FOOTINGS

DA 2-3

STRUCTURAL MANUAL

No	District	Depth
1	Chatham	1200 mm
2	London	
3	Stratford	
4	Hamilton	
5	Owen Sound	
6	Toronto	
7	Port Hope	
8	Kingston	1500 mm
9	Ottawa	
10	Bancroft	
11	Huntsville	
13	North Bay	$\begin{array}{r} 1800 \text{ mm} \\ \text{rock } 600 \div 2 \\ \hline 2100 \text{ total} \end{array}$
14	New Liskeard	
17	Sudbury	
18	Sault Ste Marie	
16	Cochrane	2100 mm
19	Thunder Bay	
20	Kenora	

memorandum



Tel: (416) 235-4959

To: Section Heads, Structural Office Date: 89 03 28
Section Heads, Regional Structural
Sections

Structural Office Policy Memo 89-03

SUBJECT: Frost Penetration Depths - Structures

PURPOSE:

To establish new frost penetration depths for Structures.

REFERENCE:

STRUCTURAL MANUAL

BACKGROUND:

For sometime now there has been a conflict between the frost penetration depths recommended in the Foundation Investigation and Design Reports and the values given in the Structural Manual and the DD Standards for Granular Backfill Requirements for Structures. After discussions between the Structural Office and the Foundation Design Section, it has been agreed to base the frost penetration depths on the November 1978 publication "Proposed Design Depths for Frost Penetration" issued by the Ministry's Engineering Research and Development Branch.

POLICY:

The frost penetration depths for structures shall be as follows:

<u>District</u>	<u>Frost Penetration, m</u>
1,2,4,6	1.2
3,5,7,8	1.5
9,10,11	1.8
13,17	2.0
14,18	2.2
16,19,20	2.2 - 2.8*

*Refer to Foundation Investigation and Design Report recommendations.

Page 2

For rock fills or rock protection, the depth effective for frost protection, shall be taken as 0.5 times the depth of rock.

IMPLEMENTATION:

The new frost penetration depths shall be implemented for all new designs started on or after April 1, 1989 and wherever possible, also for designs currently underway. Stockpiled designs may conform to the previous frost penetration depths.

Recommended by:

Approved by:



K.G. Bassi
Head, Design Section



R.A. Dorton
Manager, Structural Office

KGB/sl

c.c. M. Devata
R. Verscheure

memorandum



Tel: 3731

To: B. Farago
Design Engineer
Structural Office
3501 Dufferin Street

Date: 1989 02 13

Atten: J. Brown, Project Engineer

From: Foundation Design Section
Room 315, Central Building

RE: Little White River Bridge
Preliminary Drawing Review
W.P. 321-85-01, Site 38-57
District 17, Sudbury

As requested, the preliminary design drawings for the aforementioned structure have been reviewed by this Section and the following geotechnical related comments are provided.

Structure Foundations

The drawings illustrate the selection of supporting the structure on Size 36 creosoted timber piles. The current design should be modified such that all piles are founded at an elevation of 290.0 m as recommended in the original foundation report. According to the provided drawings, the centre row piles at the east abutment have an incorrect length of 11.270 m. This dimension should be corrected to 12.265 m to achieve the appropriate founding elevation at the 1:4 batter.

As an alternative, it is recommended that the cost-effectiveness of lengthening the timber piles be evaluated. Based on test results of a pile load test recently implemented with similar subsoil conditions at the Root River Bridge site (W.P. 279-85-01), the following recommendations are provided.

Founding Elevation (m)	Factored Capacity @ U.L.S. (kN)	Bearing Capacity @ S.L.S. (kN)
285	500	330

Pile driving should be controlled using the Hiley Dynamic Formula in accordance with standard SS103-11.

Foundation Construction

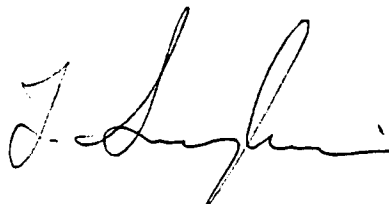
The preliminary drawing reveals a subexcavation scheme to facilitate pile driving in view of the presence of the surficial layer of cobbles, gravel and sand. Although this scheme solves the problem of pile driving impediment, a dewatering method will still be required in the construction of the pile cap within the replacement fill material. A gravity drainage system employing a gravel-filter fabric trench drain(s) is recommended as a method of dewatering.

.....2

As an alternative to subaqueous excavation and construction, the economical feasibility of advancing a steel liner through the surficial deposit of cobbles, gravel and sand and applying preaugering techniques to penetrate this layer should be considered. The piles can then be driven and the liner withdrawn following the driving process. The depth of penetration of the liner should be equivalent or greater than the unbalanced hydrostatic head to prevent seepage and boiling. This alternative, however, still requires a gravity drainage system for the pile cap construction, if the elevation of the pile cap remains as shown on the drawings.

Regardless of the method of construction selected, the dewatering problem can be eliminated if the pile cap is elevated above the prevailing water table and this alternative certainly warrants deliberation.

If you have any queries regarding the above comments or require additional information, please do not hesitate to contact this office.

A handwritten signature in dark ink, appearing to read 'T. Sangiuliano', with a stylized, flowing script.

T. Sangiuliano, P. Eng.
Foundation Engineer

TS/mmj

memorandum



Tel: 3731

To: B. Farago
Design Engineer
Structural Office
3501 Dufferin Street

From: Foundation Design Section
Room 315, Central Building

RE: Little White River Bridge
General Arrangement Drawing
W.P. 321-85-00, Site 38-57
Hwy. 546, District 17, Sudbury

Date: 1988 12 14

As requested in your memo dated 88 11 07, we have reviewed General Arrangement Drawing 38-57-P1 for the above-mentioned project. The drawing was found to comply with our foundation recommendations. However, it failed to provide the estimated pile tip elevation of 290 m. Please include this recommendation in subsequent drawings.

BB/mmj

B. Bennett

B. Bennett, P. Eng.
Jr. Foundation Engineer

memorandum



Tel: 235-3731

To: P. Stuart
Head, Structural Section
Northern Region

Date: 1988 05 09

From: Foundation Design Section
Room 315, Central Building

RE: Foundation Investigation for
Little White River Bridge, Hwy. 546
W.P. 321-85-00, Site 38-57
District 17, Sudbury

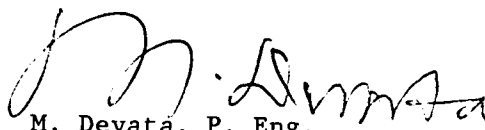
The Foundation Design Section has retained Geo-Canada Ltd. to carry out a foundation investigation at the above-mentioned location. Due to the urgency of this project, a summary of subsurface conditions together with complete foundation recommendations for the design and construction requirements were submitted to you in a memorandum. Subsequently, a draft report was submitted to this office and our comments were incorporated in the final report.

The complete foundation investigation report submitted by the geotechnical consultant was reviewed by this office and our comments are as follows:

In our opinion the most economical and viable deep foundation alternative is timber piles driven to tip Elev. 290 m. Consideration should also be given for spread footings on rock fill.

In our opinion pile load tests are not warranted since load tests at this site will not provide improved capacities and further such tests are very expensive.

We believe the aforementioned comments and the enclosed foundation investigation design report will be adequate for your design requirements. Should you require any other information related to this project, please contact us.


M. Devata, P. Eng.
Chief Foundations Engineer
(East)

MD/mmj

c.c. - J. McDougall
K. Williams
D. Barnes (2)
K. Bassi
J.H. Peer
T. Yakutchuk
A. Szekreny

memorandum



Tel: 3731

To: P. Stuart
Head
Structural Section
Northern Region

Date: 1988 04 15

From: Foundation Design Section
Room 315, Central Building

RE: Little White River Bridge
W.P. 321-85-00, Site 38-57
Hwy. 546, District 17, Sudbury

This memo reports the status of the project, a summary of site conditions, and preliminary recommendations.

The project has been assigned to the geotechnical consulting firm, Geo-Canada Ltd., under the technical supervision of this Section. Fieldwork has been completed, and a draft report has been submitted to this office for review. Revisions to the draft report are required following which the final Foundation Investigation and Design Report will be submitted.

SITE CONDITIONS

The soil profile consists of a 3 to 4 m thick, generally dense, layer of cobbles, gravel and sand underlain by over 30 m of compact sand. An end-bearing stratum was not encountered. The groundwater elevation is at Elev. 302± m, typically within 2 m of the surface.

FOUNDATION RECOMMENDATIONS

Structure construction can be simplified by avoiding dewatering. Two foundation alternatives are considered applicable:

- 1) abutments founded on spread footings on rock fill
- 2) abutments founded on timber piles

Spread Footings on Rock Fill

For abutments on spread footings on rock fill the following design values are recommended:

Factored Bearing Capacity at U.L.S. = 600 kPa

Bearing Capacity at S.L.S. Type II = 250 kPa

The rock fill pad should extend horizontally a minimum of 3 m from the edge of the footing and then at a slope of 1.5 H:1 V minimum to the base of the pad. Material under the plan limits of the pad should be subexcavated to elevation 182 m. The minimum pad thickness is 2 m.

Even at the recommended loadings, small settlements may occur within native material beneath the footing and also within the rock fill. Consequently, if this alternative is selected, the bridge should be designed to accommodate these anticipated settlements.

Resistance to sliding of abutment footings can be calculated assuming an unfactored ϕ of 35° between the underside of the concrete footing and the rock fill.

The requirements for frost protection at this site are 1.8 m of cover to the underside of footing.

For this alternative, it is essential that sufficient scour protection is provided.

Timber Piles

The abutments may be supported on treated Size 36 timber piles.

Pile driving should be controlled by Ministry of Transportation Standard SS 103-10 assuming an ultimate capacity of 750 kN. The recommended pile capacity is expected to be achieved above a pile tip elevation of 290 m.

The following design values are recommended:

Factored Capacity at U.L.S. = 375 kN per pile

Capacity at S.L.S. Type II = 250 kN per pile

Lateral forces may be resisted by battered piles.

The requirements for frost protection at this site are 1.8 m of cover to the base of the pile cap.

To facilitate pile driving, the cobbles, gravel and sand stratum under the plan limits of the abutment, should be subexcavated and replaced with sand and gravel fill in which the maximum particle size is limited to 50 mm. It is estimated that subexcavation will be required to elevation 300 m. This excavation/backfill operation should be completed without dewatering.

If there are any questions, please advise.

DHD/mmj

D. H. Dundas
D.H. Dundas, P. Eng.
Sr. Foundations Engineer