

DOCUMENT MICROFILMING IDENTIFICATION

G.I.-30 SEPT. 1976

GEOCRES No. 31E-117
31E-118

DIST. 52 REGION

W.P. No. 96-86-02

CONT. No.

W. O. No.

STR. SITE No. 40-05

HWY. No. 35

LOCATION HWY 35 & BLACK RIVER
(8 KM SOUTH OF HWY 117)

=====

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:



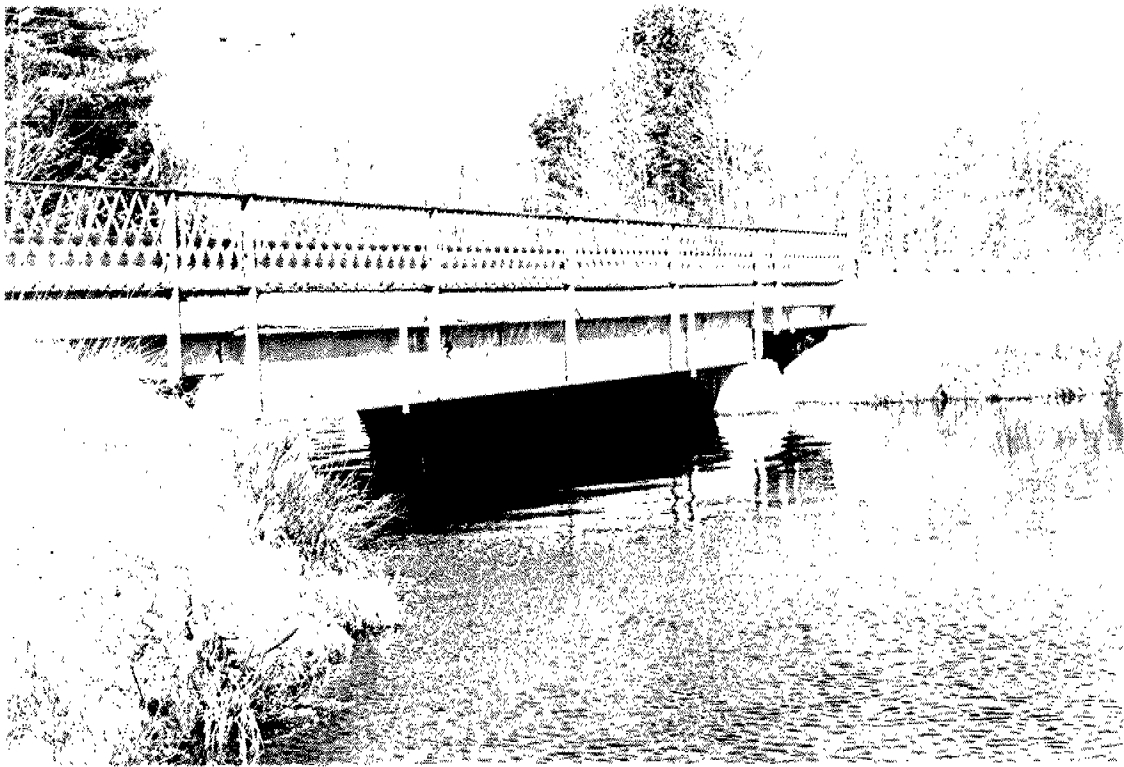
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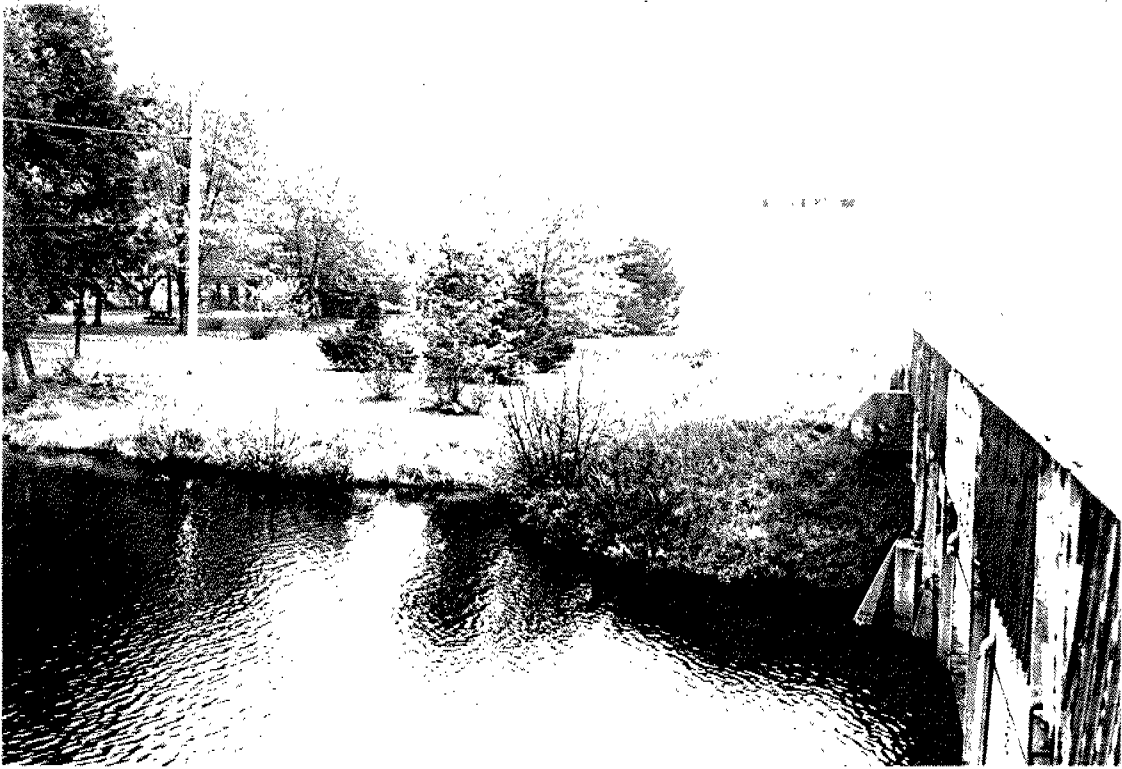
REMARKS _____

Consultant, Mc Clement & Rock

Agreement No. 4246-756-159



40-05
BLK. R. BR -
HWT ³⁵
upstream elevation



40-05

Black R Br

-0FL-07101 MNNRPI

Hwy 35

looking South

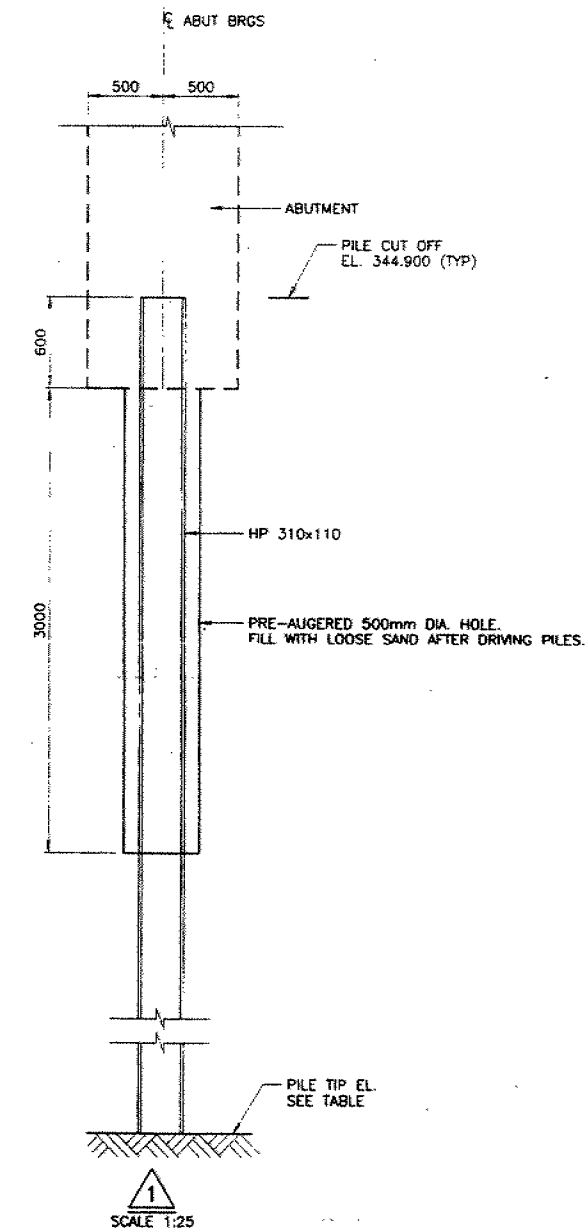
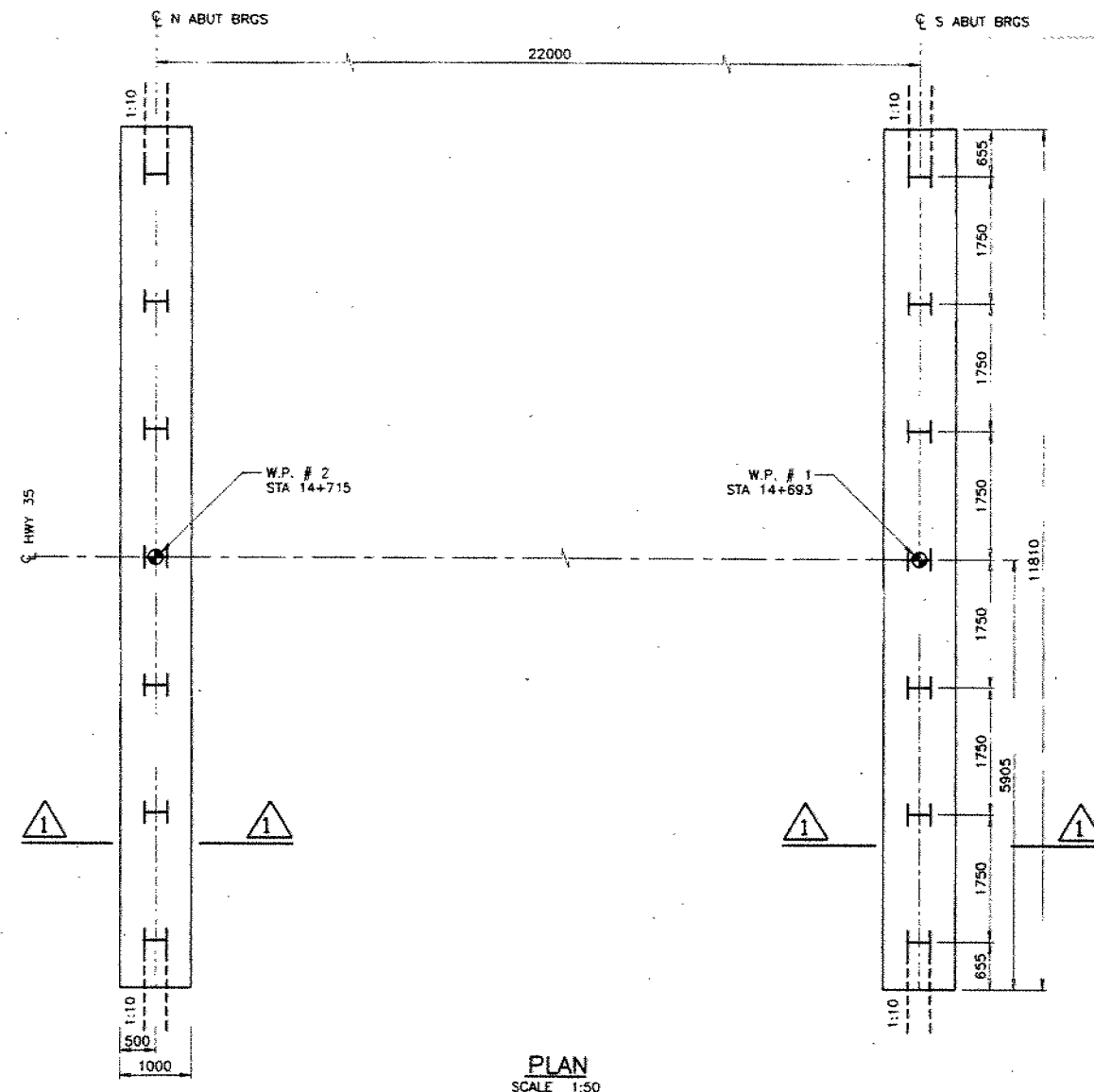
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No 97-58
WP No 96-86-02

BLACK RIVER BRIDGE
FOUNDATION LAYOUT



SHEET
24



HP 310 X 110 PILE DATA				
TYPE	NORTH ABUTMENT		SOUTH ABUTMENT	
	VERTICAL	BATTER 1:10	VERTICAL	BATTER 1:10
NUMBER REQ'D	5	2	5	2
ESTIMATED LENGTH - EACH	40.5	40.7	43.2	43.4
ESTIMATED PILE TIP ELEVATION	304.4		301.7	
DESIGN CAPACITY AT U.L.S. II (KN)	1600		1600	
DESIGN CAPACITY AT S.L.S. II (KN)	1150		1150	

- PILES TO BE DRIVEN TO BEDROCK.
- PILE SPACING IS MEASURED AT THE UNDERSIDE OF ABUTMENTS.
- PILE LENGTHS SHOWN ARE THE THEORETICAL LENGTHS BELOW CUT-OFF.



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

APPLICABLE STANDARD DRAWINGS

OPSD 3301.00 SPLICE AND DRIVING SHOE DETAILS FOR STEEL 'H' PILES

REVISIONS		DESCRIPTION			
DESIGN	M.M. CHK	S.S.B. CODE OHBDC'91	LOAD OHBD	DATE	NOV 1996
DRAWN	V.Z. CHK	M.M. SITE 40-05	STRUCT	SCHEME	DWG 3

GENERAL NOTES

1. CLASS OF CONCRETE.....30 MPa
2. CLEAR COVER TO REINFORCING STEEL:
- | | |
|-------------------------------|-------|
| -ABUTMENTS AND WINGWALLS..... | 70±20 |
| -DECK: TOP..... | 70±20 |
| BOTTOM..... | 40±10 |
| -REMAINDER..... | 70±20 |
| UNLESS OTHERWISE NOTED | |
3. REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED. BAR MARKS WITH PREFIX C DENOTE COATED BARS.
4. UNLESS OTHERWISE SHOWN, TENSION LAP LENGTHS NOT INDICATED ON THE DRAWINGS, SHALL BE CLASS B. BAR HOOKS SHALL BE MINIMUM LENGTH AND STIRRUPS SHALL HAVE MINIMUM HOOKS, UNLESS INDICATED OTHERWISE.

CONSTRUCTION NOTES

1. IF THE ACTUAL BEARING HEIGHTS ARE DIFFERENT FROM THE ASSUMED BEARING HEIGHTS GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE BEARING SEAT ELEVATIONS AND THE REINFORCING STEEL TO SUIT THE ACTUAL HEIGHTS.
2. NO GRANULAR A BACKFILL TO BE PLACED UNTIL CONCRETE IN DECK HAS REACHED 75% OF ITS SPECIFIED STRENGTH.
3. GRANULAR A BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH ABUTMENTS KEEPING THE HEIGHT OF THE BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN THE BACKFILL HEIGHT BE GREATER THAN 500mm.

LIST OF ABBREVIATIONS

T/A DENOTES TOP OF ASPHALT
W.P. DENOTES WORKING POINT

LIST OF DRAWINGS

1. GENERAL ARRANGEMENT
2. BOREHOLE LOCATIONS & SOIL STRATA
3. FOUNDATION LAYOUT
4. NORTH ABUTMENT
5. SOUTH ABUTMENT
6. WINGWALLS
7. STRUCTURAL STEEL I
8. STRUCTURAL STEEL II & BEARINGS
9. DECK DETAILS
10. BARRIER WALL WITH RAILING
11. RAILING FOR BARRIER WALL
12. 6000mm APPROACH SLAB
13. QUANTITIES - STRUCTURE

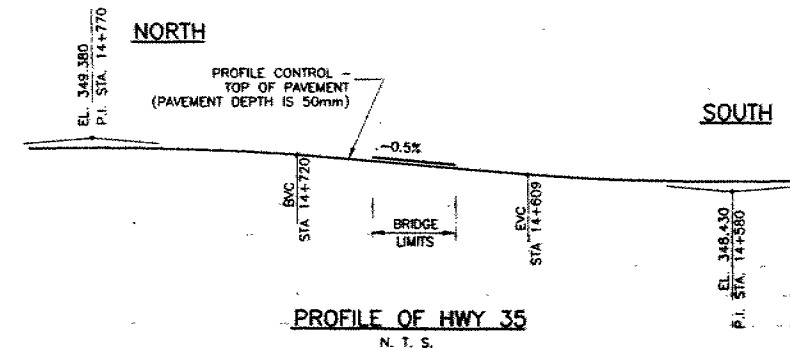
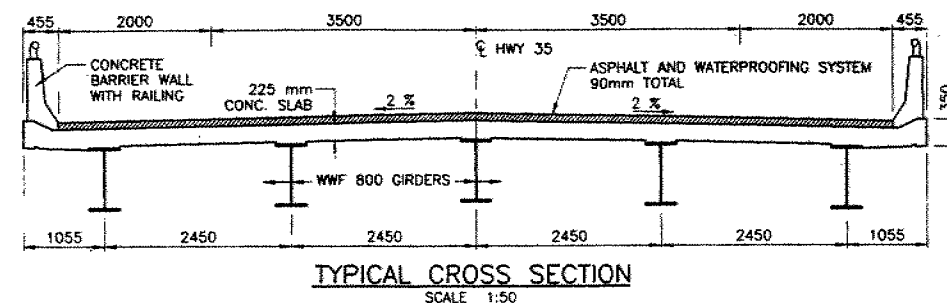
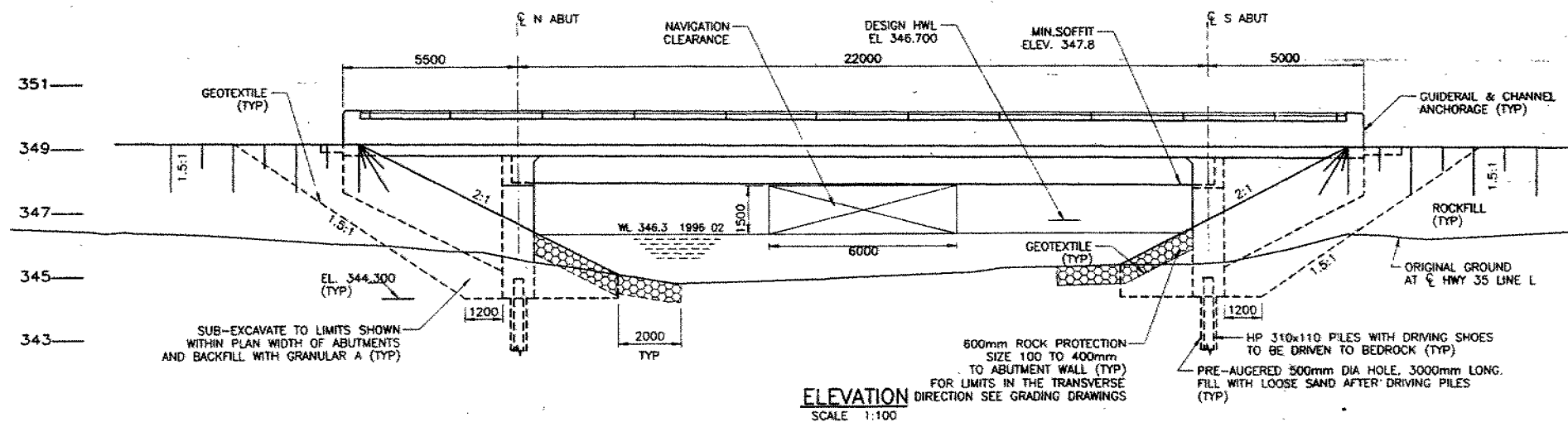
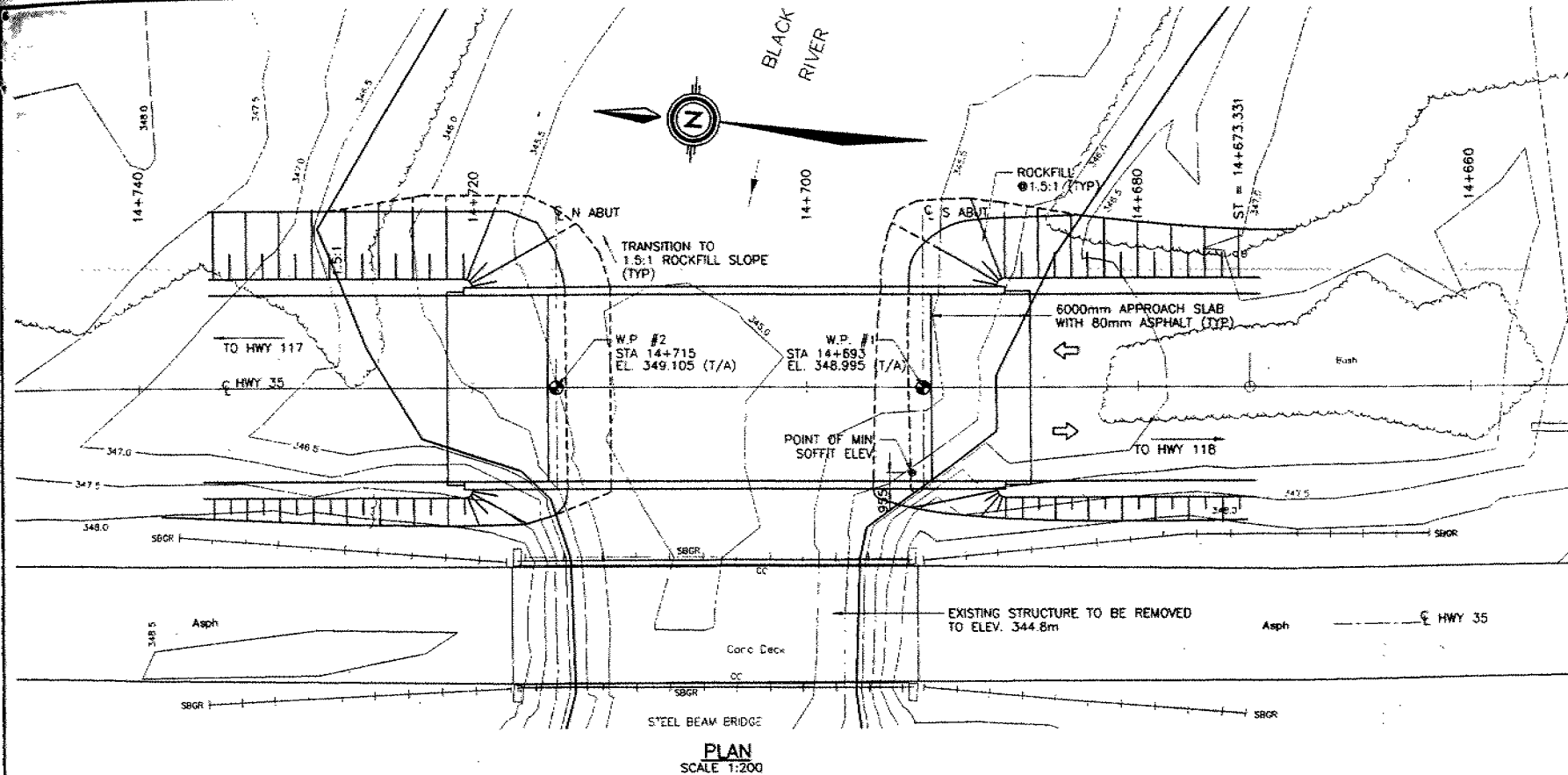


DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

APPLICABLE STANDARD DRAWINGS

OPSD 902.09 EMBEDDED CONNECTION FOR NEW STRUCTURES
OPSD 3906.02 BRIDGE DECK WATERPROOFING
OPSD 3908.03 BRIDGE DECK WATERPROOFING DETAILS
OPSD 4670.00 TYPICAL JOINT DETAILS

REVISIONS										



B.M. 347.228
NAIL AND WASHER IN ROOT
0.3 MAPLE-5.4 RT. STA 14+626.5

C1210

DECEMBER 1990

FOUNDATION INVESTIGATION REPORT
FOR
BLACK RIVER (WREN LAKE) STRUCTURE
W.P. 96-86-02 SITE 40-05
HWY 35, DISTRICT 11, HUNTSVILLE

DISTRIBUTION: 12 COPIES - MINISTRY OF TRANSPORTATION
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PREPARED FOR:

MINISTRY OF TRANSPORTATION OF ONTARIO
1201 WILSON AVENUE
ROOM 315, CENTRAL BUILDING
DOWNSVIEW, ONTARIO
M3M 1J8

31E-117
GEOCRGS - 31D-332

FOUNDATION INVESTIGATION REPORT
FOR
BLACK RIVER (WREN LAKE) STRUCTURE
W.P. 96-86-02 SITE 40-05
HWY 35, DISTRICT 11, HUNTSVILLE

INTRODUCTION

This report summarizes the results of a foundation investigation for the proposed crossing of Black River (Wren Lake), at Highway 35. The investigation was carried out at the request of the Foundation Design Section of Ministry of Transportation of Ontario.

This report contains geotechnical information pertaining to structure foundations, approach embankments and related earthworks between Sta. 14 + 660 and Sta. 14 + 730.

SITE DESCRIPTION

The site is located at the crossing of Black River and Highway 35, about 8.5 m west of the present structure on Highway 35, in District 11, Huntsville, Ontario.

The existing structure is a three span bridge, consisting of steel girders and a concrete deck. The central piers are located within the river channel. The type of foundations used is, however, not known at the present time.

The surrounding area is covered with heavy vegetation and scattered cottages. The water from Wren Lake flows into the Black river in the westerly direction. The river channel, at the crossing, is approximately 28 m wide and about 2.0 m deep. At the time of the investigation, November 8, 1990, the water level in the river was at Elevation 346.3 m, about 1.0 m deep. Both the banks are free from heavy vegetation, except minor bushes at the south end, with a gentle to almost flat slope, rising out of the bed, of 2 H to 1 V at the north bank, and 5 H to 1 V at the south bank.

PROCEDURE

The field investigation was carried out between the period of November 5 and 9, 1990. The fieldwork consisted of drilling seven boreholes and four cone tests. The boreholes, on land, were advanced using a track mounted auger machine equipped with 83 mm I.D. hollow stem augers and B size casing. The boreholes, in water, were advanced using wash boring/casing assembly mounted on a raft.

Samples were recovered by means of a 50 mm O.D. split spoon sampler driven into the soil according to the specification of the Standard Penetration Test (ASTM D 1586-83).

Laboratory testing was carried out on representative samples to identify and determine the physical properties of the overburden including:

Natural moisture content
Grain size distribution

The elevations of the boreholes were referenced to a geodetic benchmark, established by the MTO office. (N & W in N root, 0.3 Maple, Sta. 14 + 613.2 at Elevation 347.228 m).

SITE GEOLOGY AND SUBSURFACE CONDITIONS

Physiographically, the site lies in the area known as Algonquin Highlands. Although the general area is characterized by shallow till and rock ridges, the exceptions are the sand and gravel hills and alluvial sand plains.

The subsurface conditions across the site are composed of very loose to compact, fine to medium sized sand with trace to some silt and gravel. The boreholes were terminated within this deposit, which was explored to the maximum depth of 27.8 m. The upper material, at the existing embankments, consist of sand fill with trace of organics. Organic specs/traces are also evident in the upper layers of the native sand deposit.

The boundaries of the different strata, together with the field and laboratory test results, appear on the Record of Borehole sheets appended to this report. Also refer to the sheets for the locations and elevations of the boreholes. Stratigraphical sections of the subsurface conditions are shown on Drawing 968602-A. A detailed description of the different strata is provided below.

Sand, some silt and gravel, trace of organics (Fill)

The surficial sand fill material was encountered at Boreholes 1 and 9. This fill appears to have been placed during the construction of the existing structure and embankments. This fill depth varies from 2.0 to 2.7 m below road surface, i.e. Elevation 346.4 to 345.5 m.

The 'N' values vary from 2 to 8 blows, indicating a state of compaction varying from very loose to loose. Ignoring the 'N' values at the surface, this deposit is typically in a very loose state. The moisture content varies from 8 to 21 percent. Typical gradation curve is shown in Figure 1.

Sand, trace to some silt and gravel (Native)

This predominant deposit was encountered in all the boreholes. All the boreholes were terminated within this deposit, and it was explored to the maximum depth of 27.8 m. Generally, the upper 0.6 to 1.2 m of this material contains organic specs, seams and small pieces of decomposed wood and plants. The gravel content is also greater in the upper soils, at Boreholes 7 and 9. Typical gradation curves, for the upper material, are shown in Figure 2.

The sand is generally fine to medium sized with trace of silt and gravel, and a typical gradation envelope is given in Figure 3. The silt content increases, in the range of 16 to 19 percent, approximately below 18.2 m depth, i.e. Elevation 330 m, and this grain size distribution is shown in Figure 4.

The 'N' values vary from 3 to 23 blows, indicating a state of compaction varying from very loose to compact. It is probable, however, that some of the relatively low 'N' values reported are due to disturbance caused by unbalanced hydrostatic head. The moisture content varies from 12 to 31 percent (average about 20 percent).

Groundwater conditions

Observation of the groundwater level was carried out by measuring the water levels in the open boreholes. Groundwater was at river level, i.e. Elevation 346.3 m on November 8, 1990. It should be noted that the groundwater is subject to changes with the fluctuations in the river levels.

DISCUSSIONS AND RECOMMENDATIONS

It is proposed to upgrade the existing road to the current MTO standards. This requires a new crossing to the west of the existing structure. At present, three alternative schemes are being considered:

- 1) 6.0 m culvert
- 2) 12.0 m rigid frame bridge
- 3) 33.0 m single span girder bridge

The proposed profile grade, in the vicinity of the crossing, will be at approximate Elevation 349.7 m. The approach fills will thus have a maximum height of 4.2 m, above the elevation of the river bed. At the river banks, the embankments will be 1.7 m to 3.2 m high.

Based on the subsoil conditions, namely very loose to compact sand to the explored depth of 27.8 m, the 33.0 m single span bridge, founded on rock fill, appears to be the most suitable structure from both foundation, as well as construction efficiency point of view. However, alternatives should be assessed based on cost, as well as construction and environmental considerations.

Culvert

From a foundation viewpoint, the culvert option is the least favourable due to the problems and considerable costs associated with dewatering, and construction in 'dry' conditions. If this option was adopted, the culvert could be supported on spread foundations at Elevation 344.5 m. Full foundation design details can be provided, if this option is pursued.

12.0 m Rigid Frame Bridge

Two foundation options should be considered; friction timber piles and spread footings placed in native sand. However, due to the relatively weak nature of the foundation soils, spread footings may not be economically feasible. As a result, it would appear that friction piles are the most viable alternative.

Timber piles

The proposed structure can be supported on driven, friction type, timber piles. The piles should be driven to about Elevation 335.0 m, with the corresponding embedded length of about 10.0 m. For the recommended Pile Size 36-treated timber, the allowable load at S.L.S. Type 2 is 200 kN/pile, and the factored load at U.L.S. is 250 kN/pile.

Excavations, up to 2.0 m below the river level, may be required for construction of pile caps. This may require sheet piling enclosure to be constructed around the excavations.

Spread footings

The footing depth will be based on the scour and frost protection requirements. For purposes of O.H.B.D.C., at the anticipated founding elevations of 343.8 to 343.0 m, the bearing capacity at S.L.S. Type 11 state is 125 kpa, and the factored bearing capacity at U.L.S. state is 450 kpa.

The scour protection can consist of properly designed 600 mm rip-rap, placed at selected areas.

For dewatering purposes, the construction of spread footings would also require sheet piling enclosure.

33.0 m Single Span Girder Bridge

The proposed 33.0 m single span girder bridge can be supported either on spread footings founded on rock fill, or on friction type timber piles. However, the footings founded on rock fill, appear to be the most feasible foundation scheme - subject to comparative cost analyses.

Rock Fill

It is assumed, under this alternative, that the very loose fill with organic traces, in the vicinity of the existing road embankments, will be sub-excavated to Elevation 346.0 m. The exposed surface should then be proof rolled with a non vibratory roller. Any soft spots revealed should be sub-excavated and backfilled immediately with rock fill. The rock fill core must have a minimum thickness of 1.7 m.

The recommended bearing capacities and the founding elevations, as per the O.H.B.D.C., are as follows:

Material	Bearing Capacity at S.L.S. Type 11 (kpa)	Factored Bearing Capacity at U.L.S. (kpa)	Elevation (m)
Rock Fill	250	600	347.7

The rock fill should extend out at least 2.0 m beyond the footing's underside. The slope of the rock fill at 1.5 H to 1.0 V, is expected to be stable. The rock fill should be left exposed, i.e. not covered with earth or vegetation. In addition, the rock fill in the upper 0.6 m depth, should be limited to 300 mm in size. It is further recommended that the surface of the rock fill should be covered by a 120 mm thick concrete mat, of a low slump concrete, to provide good working base.

The settlement of the rock fill is estimated to be about 1.0 percent of the height of the fill, i.e. about 2.0 to 4.0 cm. The settlement of the underlying soils will be elastic and will occur upon application of the loads. In order to minimize the post construction settlement, it is recommended the fill be placed at least 3 months prior to the construction of the abutments. Considerations can also be given to a static surcharge, 1.0 m high, in order to minimize the settlements, and the time period required prior to construction of the abutments.

Timber piles

Alternatively, the structure can be supported on driven, friction type timber piles. The piles should be driven to Elevation 335.0 m, with an embedded length of about 11.5 m. For Pile Size 36-treated timber, the allowable load at S.L.S. Type 2 is 200 kN/pile, and the factored load at U.L.S. is 260 kN/pile.

No major dewatering scheme is required as the pile caps can be constructed in 'dry' above the river level.

OTHER CONSIDERATIONS

Approach Fills

The height of fill, required to achieve the proposed profile grade, will be maximum 4.5 m above the river bed. Prior to placement of fill, all surficial topsoil or any other organic material, in the upper 0.3 m of depth, should be removed within the plan limits of the embankments. The surface should then be proof rolled with a non vibratory roller. Any soft spots revealed during rolling should be sub-excavated, backfilled and compacted. The sub-excavated material can be replaced with rock fill or native sand.

Temporary excavations, above the water level, can be carried out at 1:3 H : 1.0 V. Below the water level, the excavations must be carried out at 3.0 H : 1.0 V, or flatter.

The embankment fill consisting of rock fill can be placed at 1.5 H : 1 V. The local native sand is also suitable as fill material for embankment construction. The sand can be placed at 2.0 H to 1 V.

If sand fill is used, the lower slopes must be protected from erosion. The erosion control can consist of vegetation, or 600 mm of rock protection to the potential high water level, as given by the hydrology department. With surface run-off, gullies are generally formed in the sandy embankment slopes. As a result, consideration should be given to curb and gutter system, with armoured outlets, for embankments over 3.0 m in height.

There are no settlement concerns for the embankment constructed with the above mentioned materials. Settlements will be elastic in nature and should occur during construction.

No stability problems are anticipated for the proposed height of permanent embankments.

Lateral Earth Pressure

Free draining granular material such as Granular 'A' or 'B, or rock fill is recommended as appropriate backfill to abutment walls to prevent hydrostatic pressure build-up.

Lateral earth pressures should be computed in accordance with Section 6.6.1.2 of the O.H.B.D.C. The design parameters are as follows:

	Granular 'A'	Granular 'B'	Rockfill
Angle of Internal Friction (degrees)	35	30	35
Unit Weight (kN/cu.m.)	22.8	21.2	17.6

An active condition (K_a) may be assumed to apply for a yielding structure. For rigid and unyielding structure, at rest condition (K_o) is applicable.

Lateral Resistance

For footings placed on rock fill, the sliding resistance between the concrete footing and rock fill may be calculated using an unfactored ϕ value of 35° . For footings placed on sand, an unfactored ϕ value of 30° , may be used.

Frost Protection

The general soil cover for frost protection, in the subject region, is in the order of 1.8 m. It is noted that the native sand, with trace of silt, is considered as a non frost susceptible material. Therefore, the insulation value of rock fill can also be taken as equivalent to the soil cover.

Dewatering

The river level was at Elevation 346.3 m, on November 8, 1990, and any excavations below the river level will require positive dewatering.

For rigid frame structure, sheet piling enclosure may be required to assist in dewatering. The sheet piling recommendations are given on the following page.

For a single span girder bridge, with footings on rockfill, sub-excavations to Elevation 346.0, can be backfilled with rock fill without major dewatering.

For excavations up to 1.0 m below the river level, it is expected that a relatively impervious dyke, constructed around the perimeter of the footing excavation, would be a suitable solution. The dyke could be constructed of imported clayey soils - subject to locating a suitable borrow.

Any seepage, or surface run-off, into the excavations could probably be controlled by pumping from screened sumps. The extent of dyke construction would, of course, be dependent on the type of structure and foundation scheme selected.

The type and the integrity of dewatering scheme is the responsibility of the contractor.

Sheet Piles

Sheet piling enclosure will be required, to assist in dewatering, for construction of footings, below river level, where impermeable dikes will not be adequate. Sheet piling may be required for footing construction of a rigid frame bridge alternative.

Lateral earth pressures, acting on a sheet pile wall, should be computed in accordance with Section 6.6.1.2 of the O.H.B.D.C. The design parameters are as follows:

Angle of Internal Friction (degrees)	30
Unit Weight (kN/cu.m.)	18.5
Submerged Unit Weight (kN/cu.m.)	8.5

The river level was, at the time of the investigation, at Elevation 346.3 m. For the anticipated excavation depth of 2.0 m below the river level, and given the nature of the river bed soils, the required embedment length would be 1.8 m, where placed in combination with an earth berm.

Other design/construction considerations

Due to the close proximity of the proposed culvert to the existing bridge structure, the upstream wing walls of the culvert will have to be constructed after the demolition of the existing bridge.

MISCELLANEOUS

The fieldwork for this investigation was carried out under the supervision of S. Magdolen, Geologist. The equipment was owned and operated by Master Soils Investigation Limited.

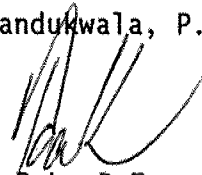
The project was carried out under the supervision of S. Bandukwala, Project Engineer. The report was written by S. Bandukwala, and reviewed by L. Rak, Principal Engineer.

Submitted by

MCCLYMONT AND RAK ENGINEERS INC.



S. Bandukwala, P.Eng.

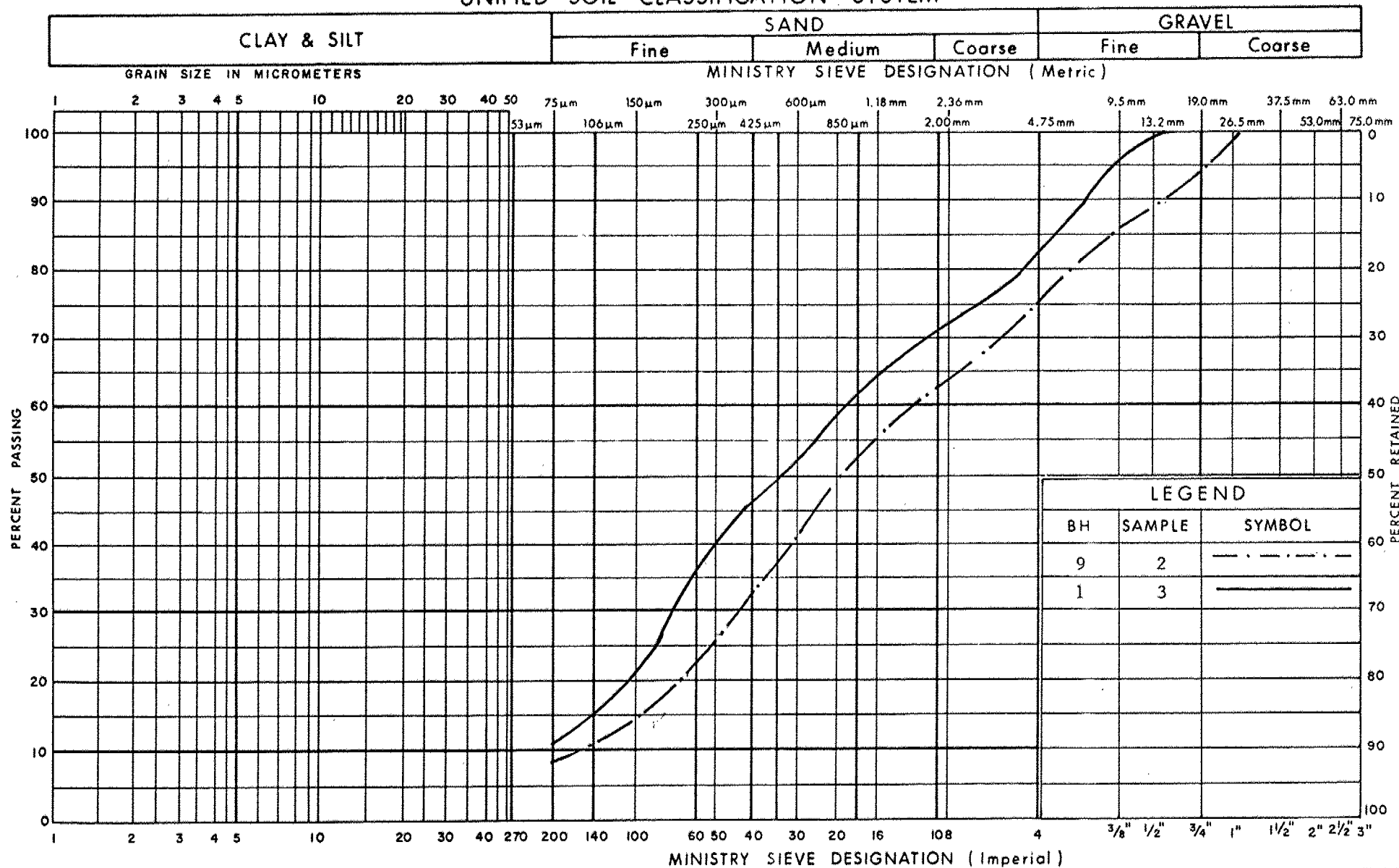


L.J. Rak, P.Eng.



11/11/11.

UNIFIED SOIL CLASSIFICATION SYSTEM

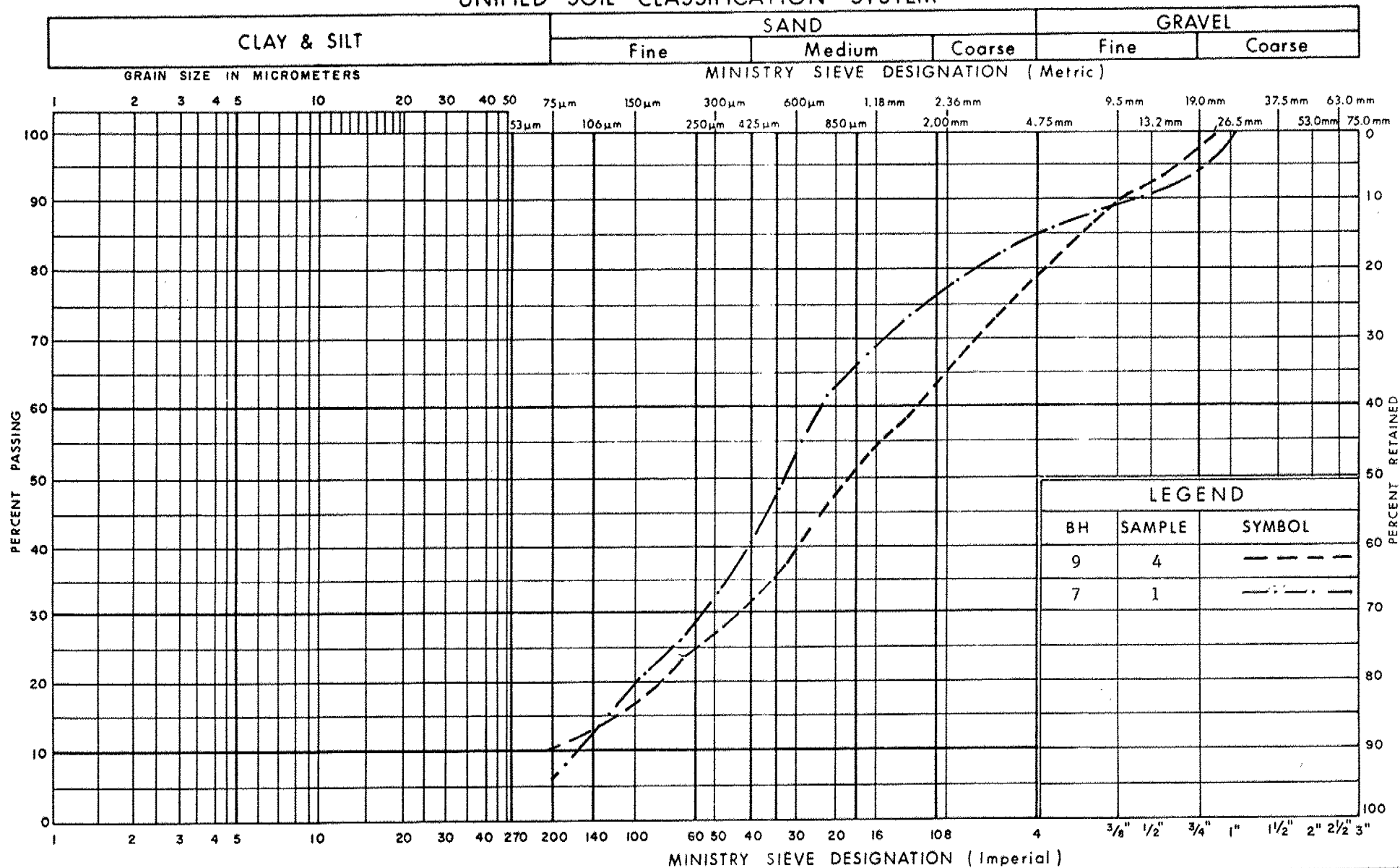
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GRAIN SIZE DISTRIBUTION
SAND
SOME SILT AND GRAVEL (FILL)

FIG No 1

W P 96-86-02

UNIFIED SOIL CLASSIFICATION SYSTEM



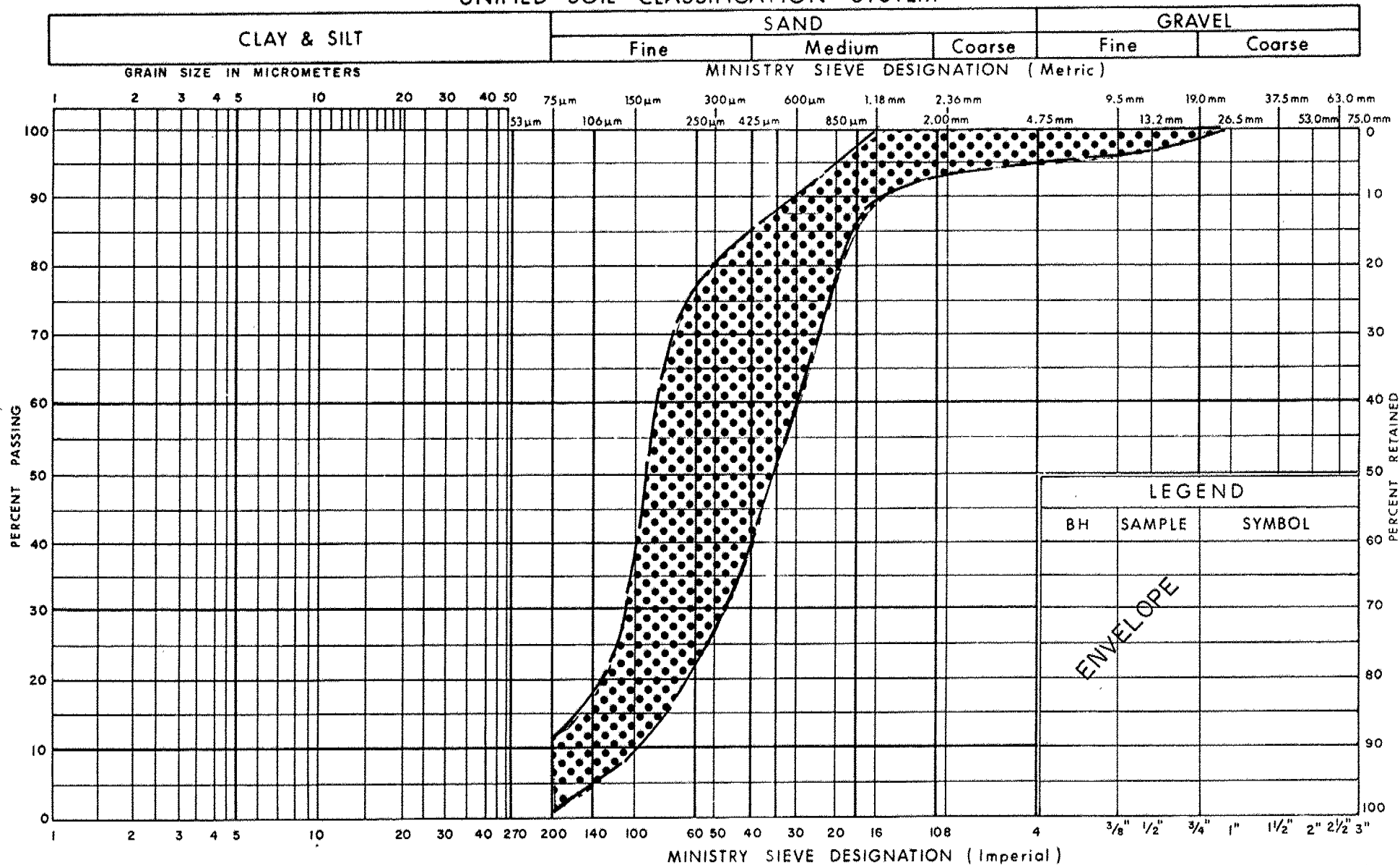
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GRAIN SIZE DISTRIBUTION
SAND
SOME GRAVEL AND TRACE OF SILT

FIG No 2

W P 96-86-02

UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION
SAND
 TRACE OF SILT AND GRAVEL

FIG No 3

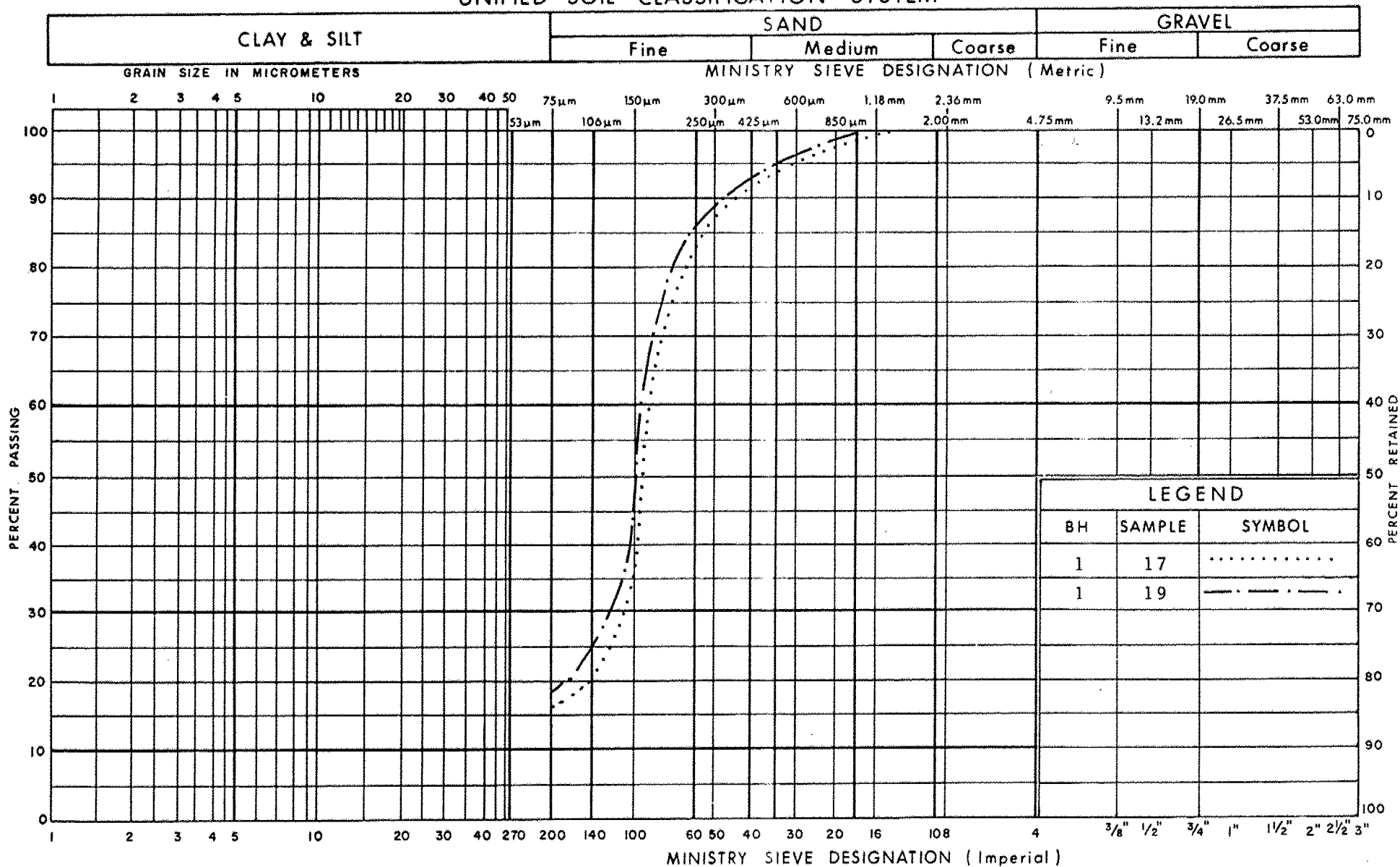
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Ontario

 Ministry of
 Transportation

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
SAND
SOME SILT

FIG No 4

W P 96-86-02

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

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

STRESS AND STRAIN

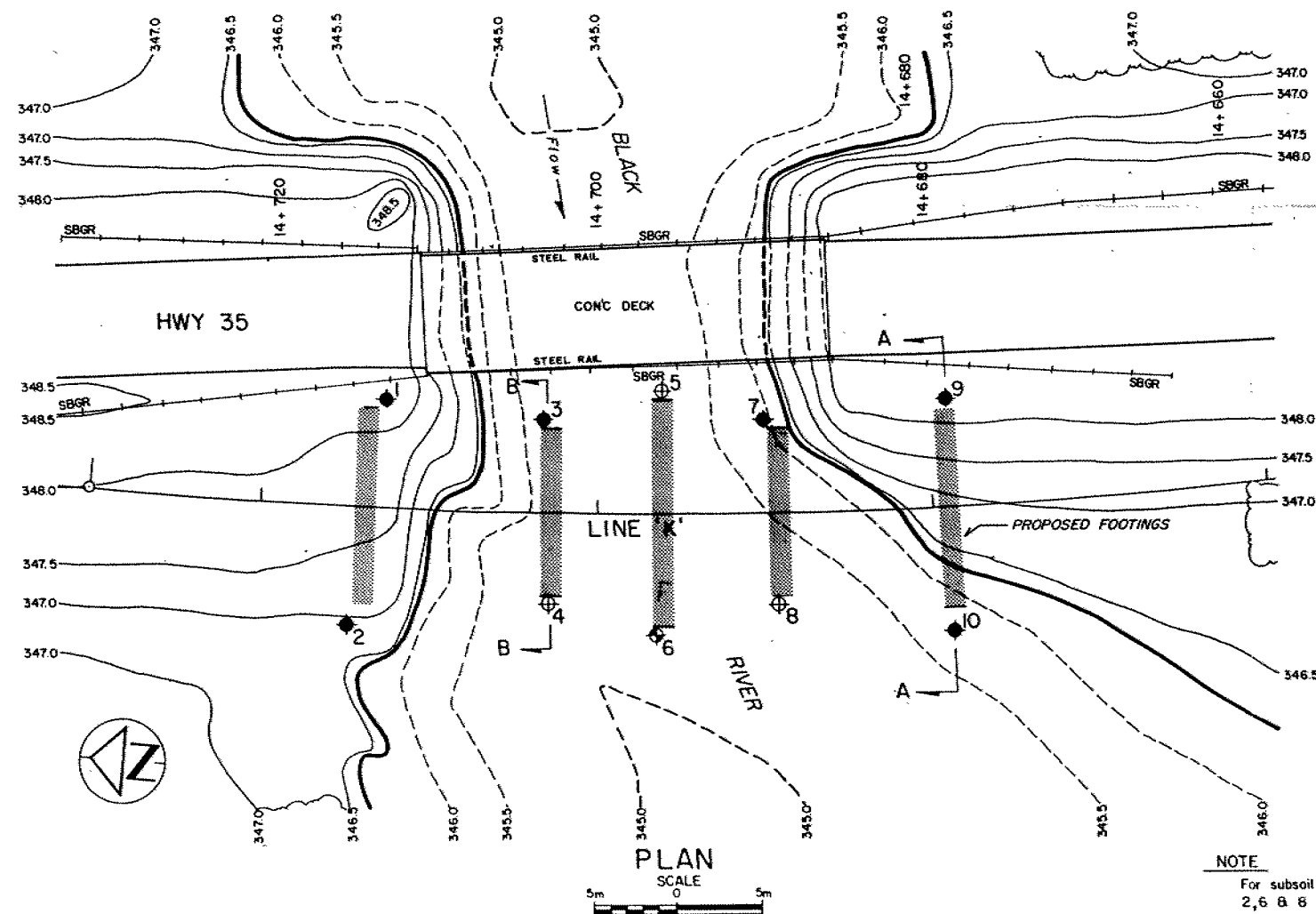
u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa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

MECHANICAL PROPERTIES OF SOIL

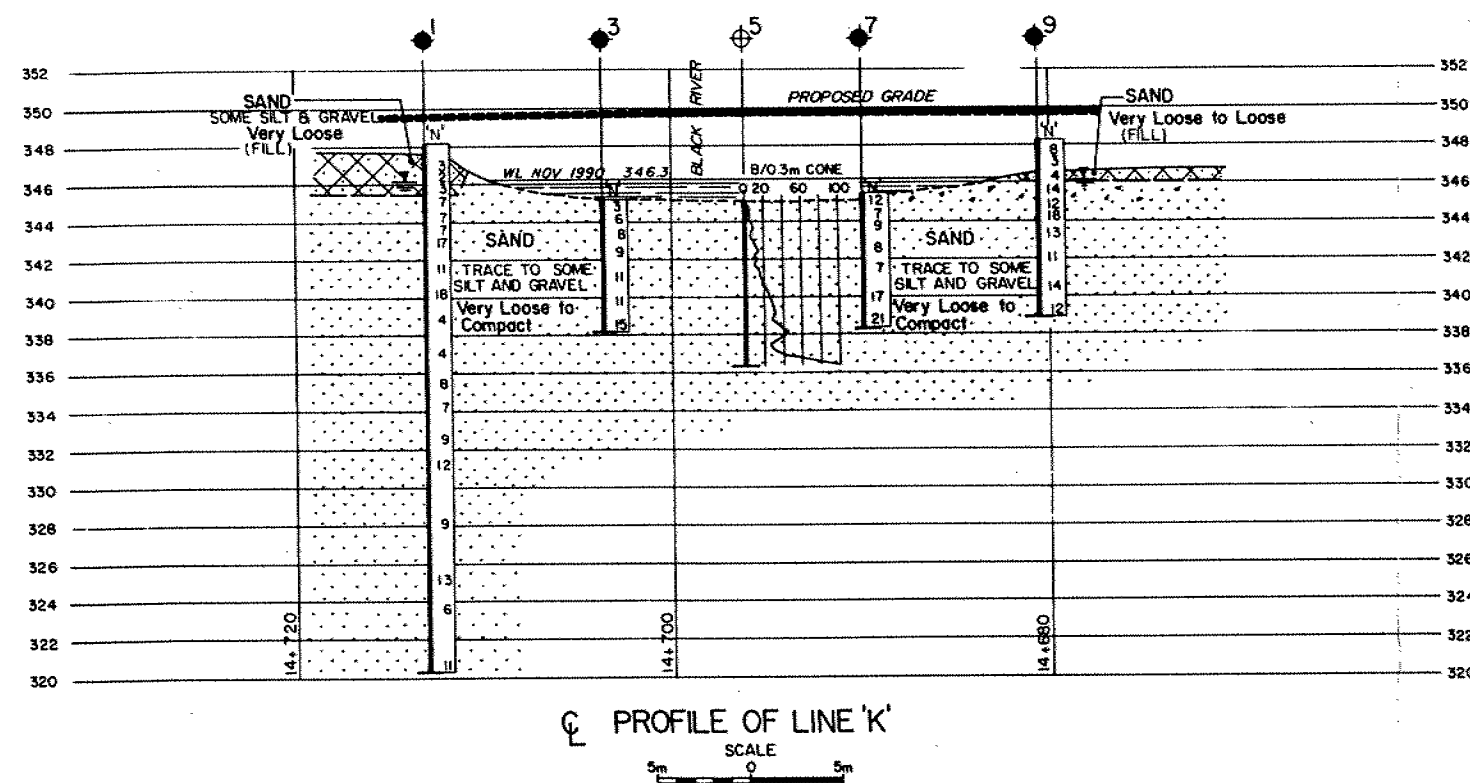
m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
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						



NOTE
For subsoil information for bore holes
2.6 & 8 refer to record of bore hole.



Q_L PROFILE OF LINE 'K'

SCALE

5m 0 5m

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

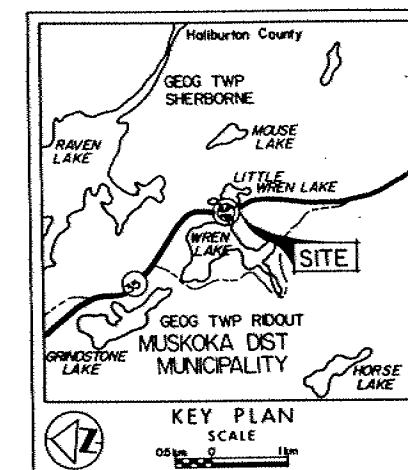
CONT No
WP No 96-86-02

BLACK RIVER AT
HWY 35
BORE HOLE LOCATIONS & SOIL STRATA







SHEET

MCCLYMONT & RAK ENGINEERS, INC



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 Nov 1990

No	ELEVATION	STATION	OFFSET
1	348.2	14+712.7	6.2 RT
2	347.2	14+714.6	7.0 LT
3	346.3	14+703.4	5.6 RT
4	346.3	14+702.6	5.5 LT
5	346.3	14+696.0	7.3 RT
6	346.3	14+696.5	7.2 LT
7	346.3	14+690.0	5.6 RT
8	346.3	14+689.0	5.5 LT
9	348.4	14+678.8	6.4 RT
10	346.3	14+679.2	7.3 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, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

REV	DATE	BY	DESCRIPTION

Geocres No 31D-332 31E-117

HWY No 35 LINE 'K'	DIST II
SUBM'D SR <input checked="" type="checkbox"/> CHECKED	SITE 40-05
DRAWN ER <input checked="" type="checkbox"/> CHECKED	DWG 968602-A
DATE DEC/90	
APPROVED	

REF No E-841-35-1; JUN 1990

RECORD OF BOREHOLE No 1

METRIC

W P 96-86-02 LOCATION Sta. 14 + 712.7, 6.2m Rt of Proposed Line 'K' ORIGINATED BY SM
 DIST 11 HWY 35 BOREHOLE TYPE Hollow stem auger and wash boring COMPILED BY SM
 DATUM Geodetic DATE November 5, 1990 CHECKED BY SB

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						
								SHEAR STRENGTH kPo						
348.2	Ground surface													
0.0	SAND some silt and gravel, trace of organic, very loose		1	CS			348							
			2	SS	3									
			3	SS	2		346							19 70 (11)
345.5	(FILL)		4	SS	3									
2.7	SAND organic specs in upper 0.6m, trace to some silt, fine to medium, very loose to compact		5	SS	7									
			6	SS	7		344							
			7	SS	7									
			8	SS	17		342							
			9	SS	11									
			10	SS	18		340							
			11	SS	4									
			12	SS	4		338							
			13	SS	8		336							0 92 (8)
			14	SS	7		334							
			15	SS	9		332							
			16	SS	12		330							
			17	SS	9		328							0 84 (16)
			18	SS	13		326							
			19	SS	6		324							0 81 (19)
			20	SS	11		322							
320.4														
27.8	End of Borehole													

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 2

METRIC

W P 96-86-02 LOCATION Sta. 14 + 714.6m, 7.0 m Lt of Proposed Line 'K' ORIGINATED BY SM
 DIST 11 HWY 35 BOREHOLE TYPE Hollow stem auger COMPILED BY SM
 DATUM Geodetic DATE November 7, 1990 CHECKED BY SB

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 kPo			WATER CONTENT (%)		
																		○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE			10 20 30		
347.2	Ground surface																						
0.0	SAND organic seam/layer from 0.9 to 1.2m, trace of silt and gravel, fine to medium, very loose to loose		1	SS	5		346								0		1 96 (3)						
2			SS	3											0								
3			SS	8											0								
4			SS	7											0								
5			SS	5						344						0							
6			SS	6												0							
7			SS	8						342						0							
8			SS	8												0							
339.2					9			SS	5		340							0			3 94 (3)		
8.0	End of Borehole																						

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 3

METRIC

W P 96-86-02 LOCATION Sta. 14 + 703.4m, 5.6m Rt of Proposed Line 'K' ORIGINATED BY SM
 DIST 11 HWY 35 BOREHOLE TYPE Wash boring COMPILED BY SM
 DATUM Geodetic DATE November 10, 1990 CHECKED BY SB

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					
346.3	Water surface																
0.0	WATER						346										
345.4	River Bottom																
0.9	SAND organic specs in upper 0.6m, trace of silt and gravel, fine to medium, very loose to compact	•••••	1	SS	3												4 84 (12)
			2	SS	6												
			3	SS	8		344										
			4	SS	9												
			5	SS	11		342										0 97 (3)
			6	SS	11		340										
			7	SS	15												
338.1																	
8.2	End of Borehole																

OFFICE REPORT ON SOIL EXPLORATION

+3, x5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 4

METRIC

W P 96-86-02 LOCATION Sta. 14 + 702.6m, 5.5m Lt of Proposed Line 'K' ORIGINATED BY SM
 DIST 11 HWY 35 BOREHOLE TYPE Cone test COMPILED BY SM
 DATUM Geodetic DATE November 8, 1990 CHECKED BY SB

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
346.3	Water surface														
0.0	WATER						346								
345.1	River Bottom														
1.2	Probable sand very loose to compact						344								
								342							
								340							
								338							
336.2															
10.1	End of Borehole														

OFFICE REPORT ON SOIL EXPLORATION

+³, x⁵: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 5

METRIC

W P 96-86-02 LOCATION Sta. 14 + 696.0m, 7.3m Rt of Proposed Line 'K' ORIGINATED BY SM
 DIST 11 HWY 35 BOREHOLE TYPE Cone test COMPILED BY SM
 DATUM Geodetic DATE November 9, 1990 CHECKED BY SB

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					
346.3	Water surface													
0.0	WATER													
345.3	River Bottom													
1.0	Probable sand very loose to compact													
336.3														
10.0	End of Borehole													

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 6

METRIC

W P 96-86-02 LOCATION Sta. 14 + 696.5m, 7.2m Lt of Proposed Line 'K' ORIGINATED BY SM
 DIST 11 HWY 35 BOREHOLE TYPE Wash boring and cone test COMPILED BY SM
 DATUM Geodetic DATE November 7 & 8, 1990 CHECKED BY SB

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 kPa					
346.3	Water surface													
0.0	WATER						346							
345.2	River Bottom													
1.1	SAND trace of silt and gravel, fine to medium, very loose to compact		1	SS	3									
			2	SS	5									
			3	SS	8									
			4	SS	11									
			5	SS	14									
			6	SS	9									
			7	SS	11									
338.3	End of Borehole						338							
336.8														
9.5	End of Cone Test						336							

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 7

METRIC

W P 96-86-02 LOCATION Sta. 14 + 690.0m, 5.6m Rt of Proposed Line 'K' ORIGINATED BY SM
 DIST 11 HWY 35 BOREHOLE TYPE Wash boring COMPILED BY SM
 DATUM Geodetic DATE November 9, 1990 CHECKED BY SB

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 kPa								10 20 30		
346.3	Water surface																	
0.0	WATER						346											
345.7	River Bottom																	
0.6	SAND trace of organics and some gravel in upper 1.0m, trace of silt and gravel, fine to medium, loose to compact		1	SS	12									16 78 (6)				
			2	SS	7													
			3		9		344											
			4		8													
			5		7		342							1 90 (9)				
			6		17		340											
			7		21													
338.3	End of Borehole																	

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 8

METRIC

W P 96-86-02 LOCATION Sta. 14 + 689.0m, 5.5m Lt of Proposed Line 'K' ORIGINATED BY SM
 DIST 11 HWY 35 BOREHOLE TYPE Cone test COMPILED BY SM
 DATUM Geodetic DATE November 8, 1990 CHECKED BY SB

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			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE						
346.3	Water surface													
0.0	WATER						346							
345.1	River Bottom													
1.2	Probable sand very loose to compact						344							
							342							
							340							
							338							
336.2														
10.1	End of Borehole													

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 9

METRIC

W P 96-86-02 LOCATION Sta. 14 + 678.8m, 6.4m Rt of Proposed Line 'K' ORIGINATED BY SM
DIST 11 HWY 35 BOREHOLE TYPE Hollow stem auger COMPILED BY SM
DATUM Geodetic DATE November 8, 1990 CHECKED BY SB

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					
348.4	Ground surface																
0.0	SAND		1	SS	8		348								0		25 66 (9)
	some silt and gravel, trace of organics, very loose to loose (FILL)		2	SS	3												
346.4			3	SS	4									0			
2.0			4	SS	14		346								0		21 69 (10)
	SAND		5	SS	12										0		
	some gravel and trace of organics in upper 1.0m, trace of silt and gravel, fine to medium, compact		6	SS	18		344								0		2 94 (4)
			7	SS	13										0		
			8	SS	11		342								0		
			9	SS	14										0		
338.8			10	SS	12		340								0		
9.6	End of Borehole																

OFFICE REPORT ON SOIL EXPLORATION

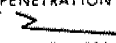
+3, x⁵: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 10

METRIC

W P 96-86-02 LOCATION Sta. 14 + 679.2m, 7.3m Lt of Proposed Line 'K' ORIGINATED BY SM
 DIST 11 HWY 35 BOREHOLE TYPE Wash boring COMPILED BY SM
 DATUM Geodetic DATE November 8 & 9, 1990 CHECKED BY SB

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		
346.3	Water surface													
0.0	WATER						346							
345.4	River Bottom													
0.9	SAND trace of organics and plants in upper 0.9m, trace of silt, fine to medium, very loose to compact		1	SS	4									
			2	SS	7									
			3	SS	9									
			4	SS	21									
			5	SS	18									
			6	SS	11									
			7	SS	23									
338.3														
8.0	End of Borehole													

OFFICE REPORT ON SOIL EXPLORATION



1595 Clark Boulevard
Brampton, Ontario
L6T 4V1

Telephone: (905) 793-9800
Facsimile: (905) 793-0641

Reference: BR11691A/G

October 20, 1997

Mr. Bob Pearson

Deep Foundations Contractors Inc.
29 Ruggles Avenue
Thornhill, Ontario
L3T 3S4

Via Facsimile:
905-881-2564

Dear Bob:

**Pile Driving Assessment
Black River Bridge at Hwy. 35
Dorset, Ontario**

As requested, Trow carried out an assessment of the pile driving at the north and south abutments at the above site. It was reported that the actual pile driving was found to be much more difficult than expected based on normal interpretation of the information presented in the borehole logs.

1. Proposed Construction

A new bridge is to be constructed over the Black River on Highway 35 approximately 8 km south of Highway 117. The bridge consists of a single span structure with seven piles supporting each of the north and south abutments.

The piles are specified to be HP 310 mm by 110 kg/m H sections driven through a sand layer to the underlying bedrock. The design load for the piles is 1155 kN (260 kips). Deep Foundation Contractors Inc. (Deep) proposed to use a 41 kN (9.2 kips) drop hammer to drive the piles. Subsequent to the findings from the driving of a three piles, a Delmag D22-23 single acting diesel hammer was used to complete the driving to the bedrock level.

At both abutments, the piles were driven from an sheet pile enclosed excavation with base elevation of 344.3m. The original ground elevations, based on the borehole log information at these areas were about 346.3 to 346.5 m.

2. Documents Available For Review

For this assessment, the following documents were obtained from Deep and reviewed by Trow:

- Report titled "Foundation Investigation Report for Black River Bridge Replacement, W.P. 96-86-02, Site40-05, Highway 35, District 52, Huntsville" prepared by Ministry of Transportation of Ontario (MTO).
- Pile Driving Records for Piles 1N to 7 N inclusive, prepared by a representative of Deep.
- Pile Driving Records for Piles 1S to 7S inclusive, prepared by a representative of Deep.

3. Subsurface Conditions

The subsoil conditions at this site were presented in a report prepared by MTO dated June, 1997 and entitled "Foundation Investigation Report for Black River Bridge Replacement, W.P. 96-86-02, Site40-05, Highway 35, District 52, Huntsville".

A total of four boreholes were drilled to obtain soil information for the proposed construction. Two of the boreholes were drilled on land using a track mounted auger machine to the north and south of the proposed abutment locations. These two boreholes were drilled to refusal on the bedrock. The bedrock was then cored for approximately 1.5 m. Two shallower boreholes were drilled using a diamond drill machine mounted on a raft in the river. These two shallower holes terminated at a depth of approximately 18.7 m below the river level.

According to the borehole information, the soil consists of a sand layer over the bedrock. The N-values for the sand was reported to range from 1 to 23 blows per 0.3 m with occasional and rare higher N-values of 41 to 45 blows per 0.3 m. The sand is described as in a "very loose to compact state" in the geotechnical report.

The sand contained 4 to 49 percent of silts and clay. The majority of the grain size analyses indicated silt and clays are less than 20 percent.

The bedrock at this site was reported to be a Hornblende-Biotite Gneiss. The core recovery was reported to be 100 percent with the exception of the third run in Borehole 11 in which only 80 percent of the cores was recovered.

A dynamic cone penetration test was conducted at the Borehole 11 location. The test was carried out from the ground surface level to a depth of approximately 20.5 m.

4. Pile Driving

The pile driving records for the seven piles installed in the north and south abutments, Piles 1N to 7 N and 1S to 7S inclusive are attached in Appendix A. The blow counts for the seven piles recorded for the north and south abutments are summarized in Table 1 and 2 respectively.

The sections of the piles which were driven with the drop hammer are shaded in the table for ease of reference.

Pile 5N was the first pile driven at this site. The record indicated that the driving with the Drop hammer using a 1.2 m drop was relatively easy to a depth of 18.9 m (62 feet), being less than or at 20 blows per 0.3m. The blow count then started to increase and reached over 200 blow per 0.3 m at 21.6 m (71 feet). The driving was discontinued for the day. On the following day, the driving was resumed and the blow count decreased to 42 blows for the first 0.3m and increased to 145 blows at 24.1 m (79 feet). The driving was terminated at this depth. The pile was restruck three days later, on October 3, 1997 and a blow count of 5 per 12 mm penetration was recorded on restrike.

Pile 2N was the second pile driven at this site. The blow counts to a depth of 15.9m (52 feet) was similar to 5N. Below this level, the blow counts increased to just over 100 blows per 0.3m at 18.9 m (62 feet) and around 60 to 105 blows down to 26.8 m (88 feet). The contractor then tried driving the pile with a 1.8 m drop height (75kJ or 55,000 foot-lb. driving energy). The blow counts using this energy ranged from 41 to 54 blows per 0.3 m to a depth of 29.7 m (97.4 feet) where the driving was terminated for the day.

The contractor then proceed to drive the first section (~ 56 feet) of the remaining piles in both abutments with the drop hammer. The remaining lower sections were then driven with a Delmag D22-23 single acting diesel hammer. The blow counts with the Delmag was noted to range from 25 to 180 blows per 0.3 m for the remaining driving to the bedrock level.

5. Assessment Of The Pile Driving Behaviour

Based on a review of the driving records, the on site observations of the driving of Pile 2N from 26.8 to 29.7 m, and the soil conditions presented in the report, the following comments can be made regarding the soil conditions and pile driving at this site.

Based on the N values and the reported soil conditions, the subsoil in general would be interpreted as a very loose to compact sand from the ground surface down to the bedrock level. Isolated N-values near Elevation 313 m at Borehole 11 and 310 m at Borehole 14, was 45 and 41 blows per 0.3 m respectively. The sand near this level (equivalent to 31.3 m or 103 feet driving depth for the piles at the north abutment) can be interpreted as being in a dense state.

However, at both the north and south abutment locations, Borehole 12 and 13 were terminated at Elevation 327.6 m or 18.7 m depth. Consequently, the information from Borehole 11 and 14 would have to be interpreted for the anticipated driving conditions of the piles at the north and

south abutments. The N-values from the ground level down to Elevation 314 m (equivalent of 99 feet driving depth) are less than 20 blows per 0.3 m with most of the N-values less than 10 blows per 0.3 m.

Based on the information presented, one would anticipate that the H piles would penetrate quite easily through the interpreted very loose to compact soil without any significant driving effort using the 50 kJ driving energy. In addition, for a very loose to compact sand, the pile driving could also liquefy the soil and make the pile driving even easier.

However, the actual pile driving proved otherwise. The first two piles reached blow counts of or in excess of 120 blows per 0.3 m near 21 m (69 feet) for Pile 5N and 18.9 m (62 feet) for Pile 2N. This would normally be considered as "practical refusal" for pile driving. The high blow counts suggest that the soil is in a very dense state and not the very loose to compact conditions as indicated in the report.

On the following day, when the pile was restruck, the blow counts dropped significantly for Pile 5N. For a drop hammer, this condition would suggest that there is a relaxation problem with the piles at this site, i.e. loss in capacity with time. Since the same hammer with the same drop height was used, the only explanation to the reduction in blow count is a reduction in the resistance from the soil. This can only be explained by the development of negative pore water pressure during pile driving which temporarily created an "apparently" stronger soil and caused the high blow counts. This further confirmed that the sand was in a dense to very dense state as negative pore water pressure can only be developed in very dense sands.

6. Conclusions

Based on the review and assessment of the pile driving conditions, the following conclusions can be made:

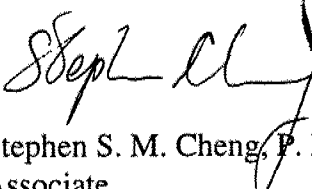
- There is no soil information below 18.7 m at the location where the piles are driven at the north and south abutments.
- Based on the overall soil description and the N-values presented in the borehole logs and soil report, the pile driving should have been fairly easy down to the bedrock level.
- From the borehole information, the pile driving is anticipated to be very easy to the bedrock level. For easy driving, a drop hammer is a suitable type of hammer for the pile driving at this site.
- The actual blow counts observed from the pile driving indicated that the sand is in a dense to very dense state below approximately Elevation 326 m (18 m depth below the bottom of excavation). There are even denser zones as evident from the diving in the south abutment where blow counts of over 100 were observed near 31 m depth.

- Relaxation was observed following the driving of Pile 5N, the first pile driven. This could only be explained by the development of negative pore water pressure which occurs only when driving piles in dense to very dense sands.
- In efficiently and effectively driven piles in the actual conditions experienced, a mechanical hammer rather than a drop hammer would be required.
- Due to the high blow counts, the pile driving would take a much longer time than originally anticipated. This was experienced at both abutments.

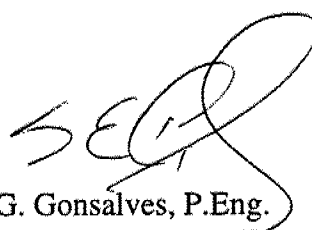
We trust that the above information is satisfactory. Should you have any queries, please do not hesitate to contact this office.

Yours truly,

Trow Consulting Engineers Ltd.


Stephen S. M. Cheng, P. Eng.
Associate




Stan G. Gonsalves, P. Eng.
Vice President, Greater Toronto Region



Enclosures: Table 1

Table 1: Pile Driving Record for North Abutment Piles

Depth (ft)	1N	2N	3N	4N	5N	6N	7N
10		8	15	0			
20			66	57		19	48
30			71	43	32	37	76
40	156	89	108	66	30	78	58
50	101	85	124	113	63	191	161
51	12	18	15	16	10	23	21
52	15	20	16	16		25	11
53	16	21	17	17	20	24	22
54	18	24	18	17		26	21
55	18/4"	25	17	17		28	20
56	-	27	17	18	20	28	22
57	21	29	25	19	20	27	23
58	20	28	29	20	20	29	26
59	18	42	29	22	20	39	28
60	19	79	28	28	20	48	30
61	24	90	31	32	20	62	31
62	30	103	30	36	20	56	33
63	31	110	31	36	24	72	35
64	34	120	30	36	38	5/8" Rebd.	37
65	35	107	31	35	45	71	37
66	31	96	30	38	50	70	38
67	33	88	29	40	55	72	38
68	38	78	30	41	83	68	36
69	39	86	32	43	145	70	33
70	42	90	31	41	171	62	37
71	46	99	1/2" Rebd.	43	210	63	39
72	39	85	31	41	208	50	41
73	38	60	30	42	42	46	43
74	1/2" Rebd.	67	33	43	46	58	45
75	-	83	21	42	143	58	45
76	40	84	30	42	166	56	44
77	43	87	34	43	137	46	5/8" Rebd.
78	46	105	33	41	140	45	50
79	40	90	30	42	145	47	51
80	39	95	30	43	70	43	50
81	41	80	29	44	70	44	49
82	43	78	27	47	69	44	48
83	44	75	29	48	54	43	49
84	40	80	31	47	61	42	54
85	42	79	1/2" Rebd.	44	1/2" Rebd.	41	50
86	40	78	-	40	-	40	53
87	39	70	32	38	59	40	50
88	41	71	31	37	64	40	51
89	43	29/6"	32	36	54	38	50

Bolded numbers represents blows per 3 m (10 feet)

Shaded numbers are blows using the 41 kN (9.2 kips) drop hammer

Table 1 (Con't): Pile Driving Record for North Abutment Piles

Depth (ft)	1N	2N	3N	4N	5N	6N	7N
90	5/8" Rebd.	48	-	35	55	40	52
91	43	54	33	36	1/2" Rebd.	41	50
92	43	53	36	37	5/8" Rebd.	39	49
93	42	45	36	36	51	37	48
94	40	42	37	38	47	37	50
95	43	41	39	40	46	35	55
96	41	45	-	42	54	36	57
97	43	27/5"	42	44	60	37	58
98	46	5/8" Rebd.	40	48	75	39	56
99	48	63	43	50	80	45	60
100	55	65	44	54	83	54	64
101	59	64	46	68	77	58	66
102	63	62	40	80	74	59	68
103	60	60	38	86	77	62	70
104	62	60	45	5/8" Rebd.	78	63	74
105	65	60	43	85	75	64	5/8" Rebd.
106	68	53	47	84	76	61	78
107	0.5" Rebd.	55	48	80	78	75	75
108	69	56	50	76	76	69	73
109	69	58	47	75	75	69	70
110	75	57	49	74	74	74	75
111	78	59	48	75	72	72	79
112	84	63	48	70	70	70	80
113	89	55	45	69	68	62	87
114	-	59	46	63	66	72	89
115	87	58	48	58	59	82	88
116	89	58	47	56	60	85	90
117	-	62	46	54	59	82	97
118	80	66	46	55	57	-	94
119	83	63	43	53	59	80	90
120	82	61	5/8" Rebd.	52	60	80	96
121	79	65	42	50	54	81	93
122	80	72	43	46	5/8" Rebd.	80	95
123	5/8" Rebd.	77	41	48	55	83	97
124	88	75	42	44	57	80	100
125	90	73	44	45	59	82	108
126	94	70	46	48	57	81	110
127	106	5/8" Rebd.	48	52	59	82	12/0"
128	103	78	50	57	58	15/0"	at 127' 8"
129	100	76	53	55	57	at 129' 4"	
130	100	78	54	58	56		
131	104	77	52	64	59		
132	106	78	53	72	101/10"		
133	105	77	81/6"	107/10"	15/0"		
134	117	86	18/0"	15/0"	at 132' 10"		
135	3/4" Rebd.	76	at 133' 6"	at 133' 10"			
136	120	85					
137	124	90					
138	140	91					
139	145	15/0"					
140	149	at 138' 7"					
141	180						
142	50/2"						
	15/0"						
	at 141' 2"						

Bolded numbers represents blows per 3 m (10 feet)
 Shaded numbers are blows using the 41 kN (9.2 kips) drop hammer

Table 2: Pile Driving Record for South Abutment Piles

Depth (ft)	1S	2S	3S	4S	5S	6S	7S
10							
20	5	4	7	6	8	15	
30	3	5	6	7	9	10	
40	8	6	6	9	13	33	47
50	15	15	21	25	25	66	73
51	4	4	5	5	4	10	10
52	6	4	5	6	3	10	10
53	9	5	5	7	4		10
54	9	5	6	6	5	11	10
55	9	6	6	6	6	10	10
56	9	6	6	6	7	10	10
57	9	10	6	15	10	15	10
58	10	15	19	17	18	20	15
59	17	19	25	21	24	24	27
60	20	30	30	27	24	29	38
61	25	35	31	31	26	35	47
62	26	36	36	33	25	41	48
63	29	39	34	34	26	40	47
64	29	38	35	32	28	44	49
65	30	50	38	34	27	46	48
66	34	54	39	30	25	43	45
67	35	57	40	38	28	42	48
68	37	55	37	39	30	45	47
69	38	56	34	36	35	42	46
70	41	54	35	38	39	45	45
71	42	50	38	40	35	41	46
72	45	49	37	42	27	43	48
73	43	47	34	40	40	45	46
74	46	44	37	41	38	46	45
75	45	42	35	42	40	41	44
76	43	40	35	46	43	42	46
77	44	43	35	43	47	44	47
78	46	47	38	47	48	43	45
79	43	49	37	43	44	40	49
80	41	44	38	40	46	45	50
81	42	43	35	48	52	47	47
82	44	46	38	45	50	42	45
83	46	45	40	47	46	43	43
84	48	45	43	46	47	41	45
85	47	47	44	44	48	40	43
86	46	45	46	43	47	39	47
87	48	44	47	50	48	43	42
88	43	47	47	53	44	44	44
89	44	42	48	56	46	45	45

Bolded numbers represents blows per 3 m (10 feet)

Shaded numbers are blows using the 41 kN (9.2 kips) drop hammer

Table 2 (Con't): Pile Driving Record for South Abutment Piles

Depth (ft)	1S	2S	3S	4S	5S	6S	7S
90	47	46	43	54	48	47	44
91	45	45	45	57	50	46	40
92	48	47	45	58	54	42	42
93	49	43	47	53	56	43	42
94	50	45	45	54	58	45	46
95	52	44	46	55	74	44	45
96	51	46	47	56	72	46	47
97	65	49	59	54	70	50	46
98	74	74	72	47	70	54	65
99	70	111	85	45	72	53	72
100	69	115	78	42	70	52	74
101	68	124	94	40	72	54	88
102	70	118	77	43	74	53	85
103	72	115	100	45	75	57	89
104	74	114	118	47	76	54	92
105	70	117	115	52	73	52	91
106	73	118	118	54	72	55	93
107	70	119	115	50	71	54	98
108	68	124	110	52	70	58	105
109	65	129	116	53	68	60	107
110	63	110	115	50	67	61	115
111	60	103	117	50	64	63	118
112	59	84	105	51	59	65	121
113	60	80	91	53	58	60	120
114	57	84	79	53	57	62	129
115	55	86	73	58	57	63	125
116	54	80	75	54	58	64	120
117	53	84	76	50	56	65	123
118	54	83	62	46	65	64	126
119	56	82	53	55	70	68	125
120	52	81	54	54	72	65	119
121	54	88	56	56	76	67	120
122	56	87	57	50	73	69	129
123	55	89	58	55	75	68	128
124	62	90	60	54	80	67	124
125	71	89	64	53	82	64	120
126	72	89	63	47	79	62	119
127	73	88	65	50	78	58	117
128	70	89	65	53	77	59	115
129	69	85	67	55	80	60	119
130	69	83	64	54	82	57	123
131	65	79	62	52	83	58	125
132	68	82	64	50	80	52	124
133	70	78	65	54	82	62	118

Table 2 (Con't): Pile Driving Record for South Abutment Piles

Depth (ft)	1S	2S	3S	4S	5S	6S	7S
135	71	80	68	68	84	61	105
136	88	87	70	73	86	64	109
137	94	83	72	69	125	65	
138	110	84	71	70	120		
139	118	85	70	74	110		
140	120	88	75	82	124		
141	124	82			110		
142	119	84					
143		85					
144		87					
145		81					
146		85					
147		89					
148		82					



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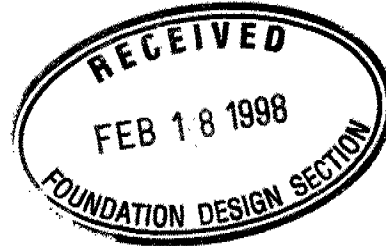
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Northern Region Construction
P.O. Box 3030, 447 McKeown Avenue
North Bay, On P1B 8L2

Telephone: (705) 497-6801 Facsimile: (705) 497-5525
email: mccauley@mto.gov.on.ca

Mr. Tae C. Kim
Sr. Foundation Engineer
Foundation Group
3rd Floor, West Building
1201 Wilson Ave.,
Downsview, ON.



**Re: Contract 97- 58 Black River Structure on Hwy 35
Piling Difficulty Issue**

Dear Sir,

I spoke to Mr. Dave Dundas on Feb. 13, 1998 on the above issue and he directed me to you for your comments. We held a meeting with the prime contractor, Fowler Const. of Bracebridge and their sub-contractor, Deep Foundation of Thornhill on Feb. 12, 1998 to discuss their piling difficulty issue as submitted to the Ministry.

I have enclosed a copy of their claim submission, a copy of our Soils Fdn. Report, Sheets 23 and 24 of the contract drawings along with the following points brought up at the meeting for your review and comments:

- 1) Was there any difficulty by the field staff in their investigation procedures in taking of the soils samples by means of a 50mm OD split spoon sampler as detailed in the specifications of the Standard Penetration Test (ASTM D 1586).
- 2) Deep Foundations claim that based on the soil type classification detailed in the Soils Foundation Report of a Sand, some silt, with a trace of gravel, very loose to compact that a diesel hammer would not fire effectively due to insufficient resistance.
- 3) Is there any relationship between the cone penetration test and the pile driving resistance one would expect to experience based on the results of the cone penetration test done at borehole # 11 on sheet 23 of the contract drawings.

4) Due to the length of pile to be driven and as noted in the soils condition, some silt ; what difficulty if any would one expect to encounter while driving the pile with the presence of silt and groundwater noted to be almost the same level as that of the existing water level of the river.

I await your response and thankyou in advance for your attention to this matter. Should you require any further information, please call.

Yours truly,

Joe McCauley
Claims Analyst
Northern Region



FOUNDATION DESIGN SECTION

**foundation
investigation and
design report**

**ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION**

WP 96-86-02 DIST 52
HWY 35 STR SITE 40-05

Black River Bridge Replacement

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GEOCRES 31E-118

DATE OCT 23 1996

FOUNDATION INVESTIGATION REPORT

For

Black River Bridge Replacement

W.P. 96-86-02, Site 40-05

Highway 35, District 52, Huntsville

INTRODUCTION

This report summarizes the results of a foundation investigation which was carried out for the construction of a new bridge over Black River at Hwy 35 (at a new alignment), about 8km south of Hwy 117 and Hwy 35 intersection.

The investigation was carried out at the request of Northern Region Structural Section. This report applies to the proposed bridge structure, approach embankment and related earthworks within 20m of the structure (Sta. 14+668 to 14+745).

SITE DESCRIPTION

The site for the proposed Black River Bridge is located about 8km south of Hwy 117 and Hwy 35 intersection. The new bridge structure will be located at the crossing of Black River and Highway 35, about 14m (centreline to centreline) east of the present structure on Highway 35, in District 52, Huntsville, Ontario.

The surrounding area at the site is covered with heavy vegetation. The water from Wren Lake flows into the Black River in the westerly direction. The river channel, at the crossing, is approximately 36m wide. At the time of the investigation, the water level in the river was at 346.3m, about 1.0m deep. Both banks are free from heavy vegetation, except for minor bushes at the south end, with a gentle to almost flat slope.

Physiographically, the site is located in the Algonquin Highlands. Although, the general area is characterized by shallow till and rock ridges, the exceptions are the sand and gravel hills and alluvial sand plains. The soils are generally shallow but thickness over the bedrock varies greatly over short distances. Many of the valleys are floored with outwash sand and gravel. As an exception several areas have deeper till and few rock outcrops, and the surface of the till is smoothed and moulded with occasional drumlins appearing (Reference: Chapman and Putnam, 'The physiography of Southern Ontario; 3rd Edition, 1984).

INVESTIGATION PROCEDURES

The field investigation for this project was conducted between 1996 05 28 and 1996 06 04. The field work for the Foundation Investigation consisted of drilling four boreholes (BH 11 through BH 14). Two boreholes (BH11 and BH 14) were drilled in the ground using a track-mounted auger machine equipped with 82mm ID hollow stem augers and BX size coring equipment. These boreholes were advanced to depths 43.7m and 46.2m respectively. Bedrock was encountered at depth 42.1m (BH 11) and 44.7m (BH 14) respectively. Two boreholes BH12 and BH13 were put down in the river using a diamond drill machine mounted on a raft. These boreholes were terminated at a depth of 18.7m below river level.

Soil samples were recovered by means of a 50mm OD split spoon sampler driven into the soil according to the specifications of the Standard Penetration Test (ASTM D 1586). Samples were retrieved at 0.75m intervals. Once practical refusal to auguring was encountered, BX-size bedrock cores were obtained from these boreholes.

Groundwater was monitored during drilling and after completion of the boreholes.

The Laboratory testing program consisted of grain size analyses and bedrock cores logging. The results of the laboratory tests are shown on the borehole logs. The result of the bedrock core logging is attached to this report. The bedrock cores were logged by D.A. Williams, Petrographer of the Soils and Aggregates Section of MTO.

The boreholes were staked out by the Pavement and Foundation Section. Ground surface elevations at the borehole locations were interpolated from an E-Plan provided to Pavements & Foundation Section (Plan E-841-35-2 dated 96 02).

SUBSURFACE CONDITIONS

The Record of Borehole Sheets in the Appendix illustrate the subsurface conditions at the borehole locations. The locations and elevations of the boreholes and test pits, along with stratigraphical profiles based on the borehole data are shown on Drawing No. 968602-A.

Overlying the bedrock which was encountered at a depth of 42.1m to 44.7m, the subsurface conditions across the site is composed of fine to medium sized sand with some silt and trace of gravel. The Standard Penetration Test result N-values within this deposit ranged from 1 blow to 23 blows/0.3m penetration which suggested that the sand deposit is in a very loose to compact state. Occasional high N-values of 41 and 45 blows/0.3m were also encountered but they were rare, and suggested that there are pockets of dense sand.

Bedrock

The bedrock was encountered only in two boreholes (BH 11 and 14) that were put down in the ground and were advanced deeper to the bedrock. The bedrock was encountered at elevations 304.4 (BH11) and 301.7 (BH 14) at depths 42.1m and 44.7m respectively. The bedrock is classified as Hornblende-Biotite Gneiss.

Bedrock cores were obtained from both locations. The length of the bedrock cores were 1.6m and 1.5m at Borehole 11 and 14 respectively. The result of the core analyses are plotted on the Borehole logs.

Groundwater Condition

Groundwater was monitored in open boreholes. Groundwater level in the boreholes was similar to water level in the river, which was at the time of investigation 346.3m. It should be noted that the groundwater is subject to fluctuation with the river level.

DISCUSSION AND RECOMMENDATIONS

General

It is proposed to shift the Hwy 35 alignment towards the east at an offset of 14m from the centreline of the existing highway. The existing bridge will be replaced with a new bridge constructed on Line 'L' at the proposed alignment. For the proposed new bridge there are two options under consideration. In the first scheme the bridge span will be 22m and in the other the span will be 32m. The actual size and the span of the bridge is not known at this time.

The existing bridge is a three span with clear spans of 6.7m and 12.2m, consisting of steel girders and a concrete deck. The central piers of the existing bridge are located within the river channel.

We understood that it is desired to construct a jointless bridge on integral abutment. For the 22m span bridge a steel girder integral abutment or concrete rigid frame bridge is proposed while for the 32m span a steel girder integral abutment bridge is recommended.

Structure Foundations

The proposed profile grade, in the vicinity of the crossing, will be at approximate elevation 349.0m. The approach fills will be approximately 2.5m to 3.5m high.

Based on the subsoil conditions, which is mainly very loose to compact sand, the most suitable structure from a cost point of view will be spread footing founded on rockfill. However, alternatives should be assessed based on cost analysis, as well as construction and environmental considerations.

Spread Footings on Rockfill

The proposed single span girder bridge can be supported on spread footing founded on rockfill. The very loose sand with organic traces will be subexcavated to elevation 346.0m. The excavation should be backfilled with rockfill. The rockfill core must have a minimum thickness of 2.0m.

The recommended bearing capacities for the footings on rockfill at elevation 348m or above, as per the OHBDC 91 are as follow:

Factored Bearing Capacity at U.L.S.	=	600 kPa
Bearing Capacity at S.L.S.	=	250 kPa

The rockfill should extend at least 2.0m beyond the underside of the footing. The side slope of the rockfill should be maintained at 1.5H:1V or flatter. The rockfill in the upper 0.6m, immediately under the footing and under the plan limit of the footing should be limited to 300mm in size. The surface of the rockfill should be covered with a 150mm thick concrete mat to provide good working base.

The settlement of the rockfill is estimated to be about 2 to 3 cm. The settlement of the underlying soils will be immediate and will occur during construction.

Deep Foundation

The structure can be supported on timber friction piles. The timber piles may be driven to any required depth but at least to elevation 335.0m. For pile size 36-treated timber with an embedded length of about 11.5m the recommended bearing capacities as per the OHBDC 91 are as follows:

Factored Axial Capacity at U.L.S.	=	340 kN/pile
Axial Capacity at S.L.S.	=	225 kN/pile
Factored Lateral Capacity at ULS	=	17 kN/pile
Lateral Capacity at S.L.S.	=	11 kN/pile

Lateral capacity may be supplemented by the horizontal component of battered piles.

Alternatively, if higher axial capacity is required or if the structure is to be constructed on integral abutments, then the structure can be founded on H-piles driven to bedrock [bedrock depth 42.1m (BH 11) and 44.7m (BH 14)]. However, this alternative should be assessed based on cost comparison. The recommended bearing capacities of H-piles founded on bedrock are as follow:

	<u>HP 310X110</u>	<u>HP 310X79</u>
Factored Axial Capacity @ ULS	1600 kN/pile	1150 kN/pile
Axial Capacity @ SLS for 25mm	1150 kN/pile	825 kN/pile
Factored Lateral Capacity at ULS	80 kN/pile	60 kN/pile
Lateral Capacity at S.L.S.	60 kN/pile	40 kN/pile

Due to long steel piles, the capacity of the pile may have to be reduced due to other structural consideration such as buckling effect etc. These aspects should be analyzed by the Structural Section.

In order to facilitate pile driving, particle sizes of any fill placed beneath the pile locations should be restricted to 75mm.

Integral Abutment

The bedrock was encountered at a depth of 42.1m (north shore, BH 11) and 44.7m (south shore BH 14) below ground surface. This corresponds to bedrock elevations of 304.4m (BH 11) and 301.7m (BH 14). This site meets all the requirements for an integral abutment i.e., length of structure, type of structure, structure geometry, abutment height and pile length etc. Therefore, an integral abutment may be considered for this site.

The integral abutment will be constructed on steel piles driven to bedrock. The piles for integral abutment should be in one row with weaker axis oriented normal to the direction of movement. The piles shall be driven through 0.5m diameter and 3m deep holes filled with uniformly graded sand. Following is the NSSP for the special sand used for integral abutment.

NSSP - Backfill to Integral Abutment-Augured Hole

The annular space between the preaugured oversize hole and the pile shall be backfilled with uniformly graded sand. The gradation for the uniformly graded sand shall be as follows:

MTO SIEVE DESIGNATION	PERCENTAGE PASSING BY MASS
2 mm (#10)	100
600 µm (#30)	80 - 100
425 µm (#40)	40 - 80
250 µm (#60)	5 - 25
150 µm (#100)	0 - 6

Alternatively, commercially available materials which meets the above gradation may be considered.

Embankment Stability

The height of embankment will be approximately 2.5m to 3.5m. Prior to placement of fill, all surficial topsoil or any organic material should be removed within the plan limits of the embankments. The embankment should be then constructed with rockfill or native sand. The embankment fill consisting of rockfill can be placed at 1.5H:1V.

The native sand is also suitable as fill material for embankment construction. If sand is used, the side slope of the embankment should be constructed at 2.5H:1V or flatter. If sand fill is used, the lower slopes must be protected from erosion which can consist of vegetation, or 600mm rock protection to the potential high water level.

There are no settlement concerns for the embankment constructed with the above mentioned materials. Settlement will be elastic in nature and should occur during construction.

No stability problems are anticipated for the proposed height of permanent embankments.

Lateral Earth Pressure

Free draining granular material such as Granular 'A' or 'B', or rockfill is recommended as appropriate backfill to abutment walls to prevent hydrostatic pressure build-up.

If rockfill is used for approaches, special care will be required to avoid damaging the abutment. It would be preferable to place a 0.3m cushion of Granular 'A' or smaller rockfill (with diameter of less than 300mm), between the structure and the main mass of rock fill. Granular material may also be used at the approaches.

For design purposes, the following properties for backfill are recommended:

Granular 'A'	$\gamma = 22.8 \text{ kN/m}^3$	$\phi = 35^\circ$
Granular 'B'	$\gamma = 21.2 \text{ kN/m}^3$	$\phi = 30^\circ$
Rockfill	$\gamma = 18.0 \text{ kN/m}^3$	$\phi = 35^\circ$

At-rest condition (K_0) may be assumed to apply for rigid and unyielding structure and active condition (K_a) will apply for yielding structure.

Resistance to Lateral Forces

For footings placed on rockfill, the sliding resistance between the concrete footing and rockfill should be computed as per OHBDC 91. For sliding within the rockfill $\phi = 35^\circ$ should be used for calculation, and $\phi = 30^\circ$ should be used for calculating sliding at the footing/rockfill interface.

Frost Protection

A soil cover of 1.8m or equivalent will be required for frost cover.

Excavation and Dewatering

The river level was at elevation 346.3m at the time of Foundation investigation. Any excavation below the river level will require dewatering. For footings on rockfill, sub-excavations to elevation 346, can be backfilled with rockfill without major dewatering.

If excavations extend below the prevailing groundwater elevation, a dewatering SP will be required. The SP should alert the Contractor that there are noncohesive soils at this site and that this soil is susceptible to disturbance under conditions of unbalanced hydrostatic head.

If excavations below the water table are required for placement of rockfill, it may be possible to excavate and backfill below water level, thus eliminating the need for dewatering.

Temporary excavations in the native overburden should be 1.5H:1V or flatter, and 2H:1V or flatter below the groundwater level.

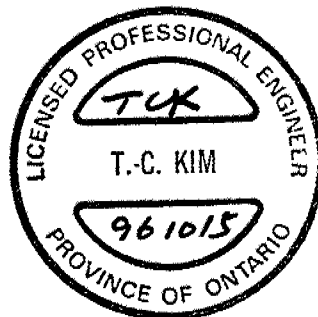
MISCELLANEOUS


The field work for this project was carried out under the supervision of K.S.Q. Ahmad, Foundation Engineer and Lizete Vieira an Engineering Student. The equipment used was owned and operated by Master Soil Investigation Ltd..

The report was written by K.S.Q. Ahmad, Foundation Engineer, reviewed and approved by T.C. Kim, Senior Foundation Engineer.




S.Q. (Ken) Ahmad, P. Eng.
Foundation Engineer




T.C. Kim, P. Eng.
Senior Foundation Engineer

APPENDIX

RECORD OF BOREHOLE No 11

1 OF 2

METRIC

W.P. 96-86-02 LOCATION Sta. 14+730.4 Centreline Line 'L', Hwy 35 ORIGINATED BY K.A.
 DIST 52 HWY 35 BOREHOLE TYPE H.S. Auger, NX Core COMPILED BY L.V.
 DATUM Geodetic DATE 1996/06/03 CHECKED BY K.A.

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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	SHEAR STRENGTH kPa					
346.5	Ground Surface												
0.0	SAND some silt trace gravel fine to medium very loose to compact		1	SS	6								2 88 (10)
			2	SS	5								
			3	SS	5								2 82 14 2
			4	SS	5								
			5	SS	5								
			6	SS	8								0 83 16 1
			7	SS	4								
			8	SS	19								
			9	SS	6								
			10	SS	4								2 69 (29)
			11	SS	12								
			12	SS	4								
			13	SS	19								
			14	SS	13								0 82 16 2
			15	SS	13								
			16	SS	6								
			17	SS	9								
			18	SS	8								0 70 (30)
			19	SS	6								
316.0													

30.5

Continued

+3, x5: Numbers refer to
Sensitivity

20
15-5 (%) STRAIN AT FAILURE
10

Continued

RECORD OF BOREHOLE No 11

2 OF 2

METRIC

W.P. 96-86-02 LOCATION Sta. 14+730.4 Centreline Line 'L', Hwy 35 ORIGINATED BY K.A.
 DIST 52 HWY 35 BOREHOLE TYPE H.S. Auger, NX Core COMPILED BY L.V.
 DATUM Geodetic DATE 1996/06/03 CHECKED BY K.A.

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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40					
316.0	Continued	[Strat Plot]	20	SS	10									
30.5														
	Sand		21	SS	45									
	Dense													
			22	SS	17									
	SAND some silt trace gravel fine to medium very loose to compact		23	SS	9									
304.4														
42.1	Bedrock	[Pattern]	24	RC	REC	100%								ROD 25%
	Hornblende-Biotite Gneiss		25	RC	REC	100%								ROD 90%
			26	RC	REC	80%								ROD 90%
302.8														
43.7	End of Borehole													

RECORD OF BOREHOLE No 12

1 OF 1

METRIC

W.P. 96-86-02 LOCATION Sta. 14+717.8 Centreline Line 'L', Hwy 35 ORIGINATED BY K.A.
DIST 52 HWY 35 BOREHOLE TYPE H.S. Auger. COMPILED BY L.V.
DATUM Geodetic DATE 1996/06/04 CHECKED BY K.A.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100	W _P W W _L	20 40 60			
346.3	Water Surface													
345.7	River Bottom						346							
0.6	SAND some silt trace gravel fine to medium very loose to compact		1	SS	2		344							1 92 6 1
			2	SS	5		342							0 94 (5)
			3	SS	5		340							
			4	SS	1		338							
			5	SS	4		336							
			6	SS	6		334							
			7	SS	11		332							
			8	SS	9		330							
			9	SS	6		328							
			10	SS	9									
327.6	End of Borehole													0 97 (3)
18.7														

RECORD OF BOREHOLE No 13

1 OF 1

METRIC

W.P. 96-86-02 LOCATION Sta. 14+693.3 Centreline Line 'L', Hwy 35 ORIGINATED BY K.A.
DIST 52 HWY 35 BOREHOLE TYPE H.S. Auger. COMPILED BY L.V.
DATUM Geodetic DATE 1996/05/29 CHECKED BY K.A.

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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
346.3	Water Surface													
0.0	River Bottom													
345.4														
0.9														
			1	SS	5									
			2	SS	7									7 91 (2)
			3	SS	4									
			4	SS	3									6 90 2 2
			5	SS	6									
			6	SS	11									0 96 (4)
			7	SS	7									
			8	SS	6									1 95 2 2
			9	SS	9									
			10	SS	22									
			11	SS	7									0 51 (49)
			12	SS	15									
			13	SS	11									
			14	SS	17									0 86 11 3
			15	SS	23									0 88 11 1
327.6														
18.7	End of Borehole													

RECORD OF BOREHOLE No 14

1 OF 2

METRIC

W.P. 96-86-02 LOCATION Sta. 14+884.6 Centreline Line 'L', Hwy 35 ORIGINATED BY K.A.
DIST 52 HWY 35 BOREHOLE TYPE H.S. Auger Rock Coring COMPILED BY L.V.
DATUM Geodetic DATE 1996/05/28 CHECKED BY K.A.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES								
346.4	Ground Surface												
0.0			1	SS	13		346						3 85 7 5
			2	SS	4								
			3	SS	4		344						0 97 1 2
			4	SS	2								2 95 2 1
			5	SS	3		342						6 91 2 1
			6	SS	7								
			7	SS	6								
			8	SS	4		340						
			9	SS	1								
			10	SS	1		338						0 98 (2)
			11	SS	7		336						
			12	SS	3		334						
			13	SS	9		332						
	SAND some silt trace gravel fine to medium very loose to compact		14	SS	4		330						1 95 (4)
			15	SS	5								
			16	SS	4		328						
			17	SS	11		326						
			18	SS	7		324						
			19	SS	18								0 63 34 3
			20	SS	18		322						
			21	SS	7		320						
			22	SS	10								0 79 (21)
			23	SS	9		318						
315.9							316						

30.5

Continued

+3, x5: Numbers refer to
Sensitivity

20
15-5 (%) STRAIN AT FAILURE
10

Continued

RECORD OF BOREHOLE No 14

2 OF 2

METRIC

W.P. 96-86-02 LOCATION Sta. 14+684.6 Centreline Line 'L', Hwy 35 ORIGINATED BY K.A.
 DIST 52 HWY 35 BOREHOLE TYPE H.S. Auger/Rock Coring COMPILED BY L.V.
 DATUM Geodetic DATE 1996/05/28 CHECKED BY K.A.

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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40					
315.9	Continued Sand Dense SAND some silt trace gravel fine to medium very loose to compact	[Strat Plot]	24	SS	9									
30.5														
			25	SS	13									
			26	SS	41									
			27	SS	15									
			28	SS	11									
301.7														
44.7	Bedrock	[Strat Plot]	29	RC	REC 100%									
300.2	Hornblende-Biotite Gneiss													
46.2	End of Borehole													

ROCK CORE DESCRIPTION
WP 96-86-02

Page 1 of 1

CORE RECOVERY					CORE DESCRIPTION	
BH#	RC#	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
11	1	42.06-42.67	100	25	42.06-43.69	HORNBLENDE-BIOTITE GNEISS , greyish black to light grey to moderate orange pink; medium to coarse grained; strong; unweathered to slightly weathered; fractures close to extremely close spaced, dipping to near vertical, undulating to planar, rough to smooth.
	2	42.67-42.92	100	90		
	3	42.92-43.69	80	37		
14	1	44.65-46.18	100	89	44.65-46.18	HORNBLENDE-BIOTITE GNEISS , greyish black to light grey to moderate orange pink; medium to coarse grained; strong; unweathered to slightly weathered; fractures moderate to close spaced, dipping, undulating to planar, smooth to rough.

*CR = CORE RECOVERY

*RQD = ROCK QUALITY DESIGNATION

Note: Depths are approximated where core recovery is less than 100%
Logged by: DAW, Soils and Aggregates Section

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 (R Q D), FOR MODIFIED RECOVERY, IS:

R Q D (%)	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

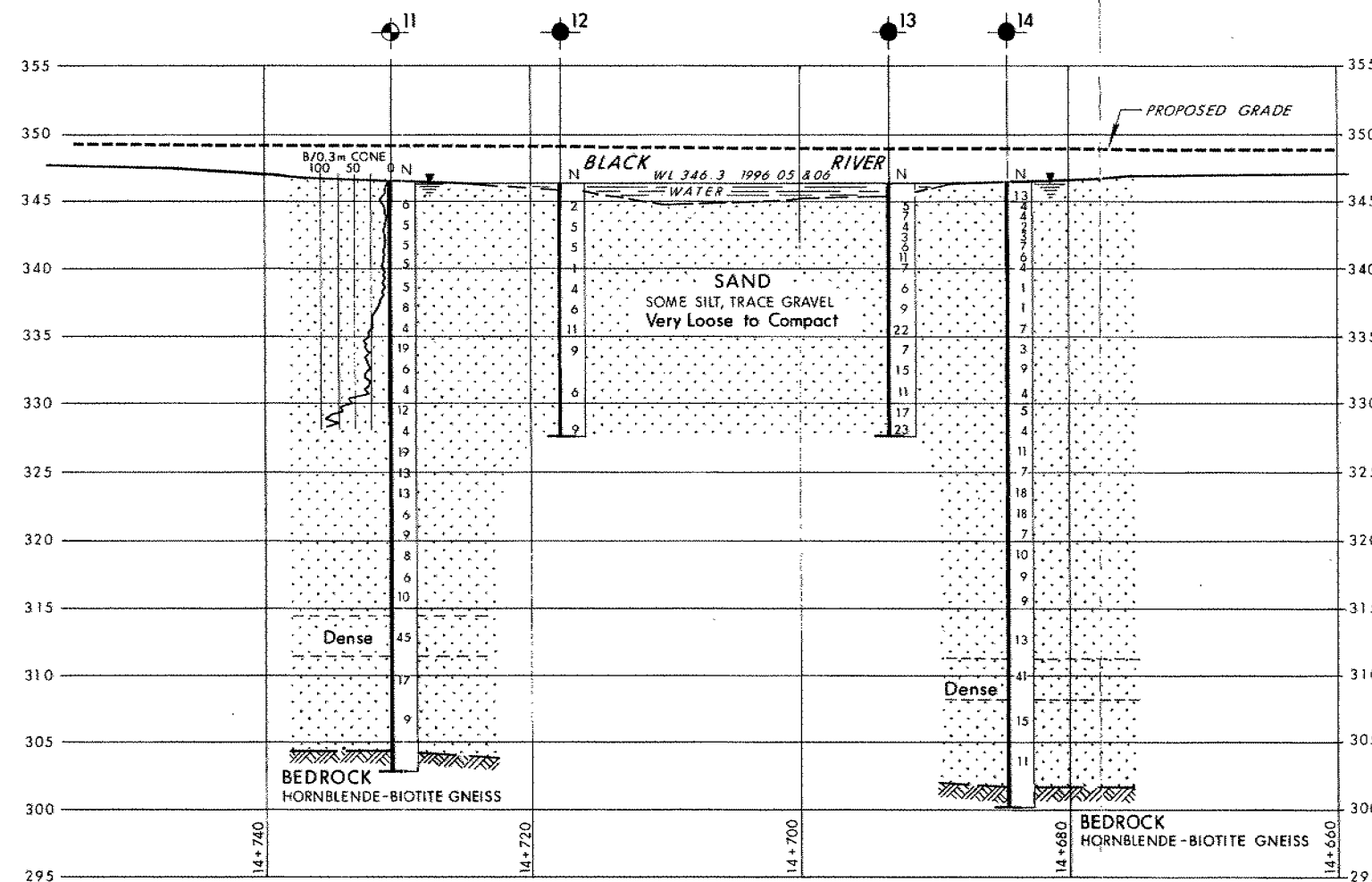
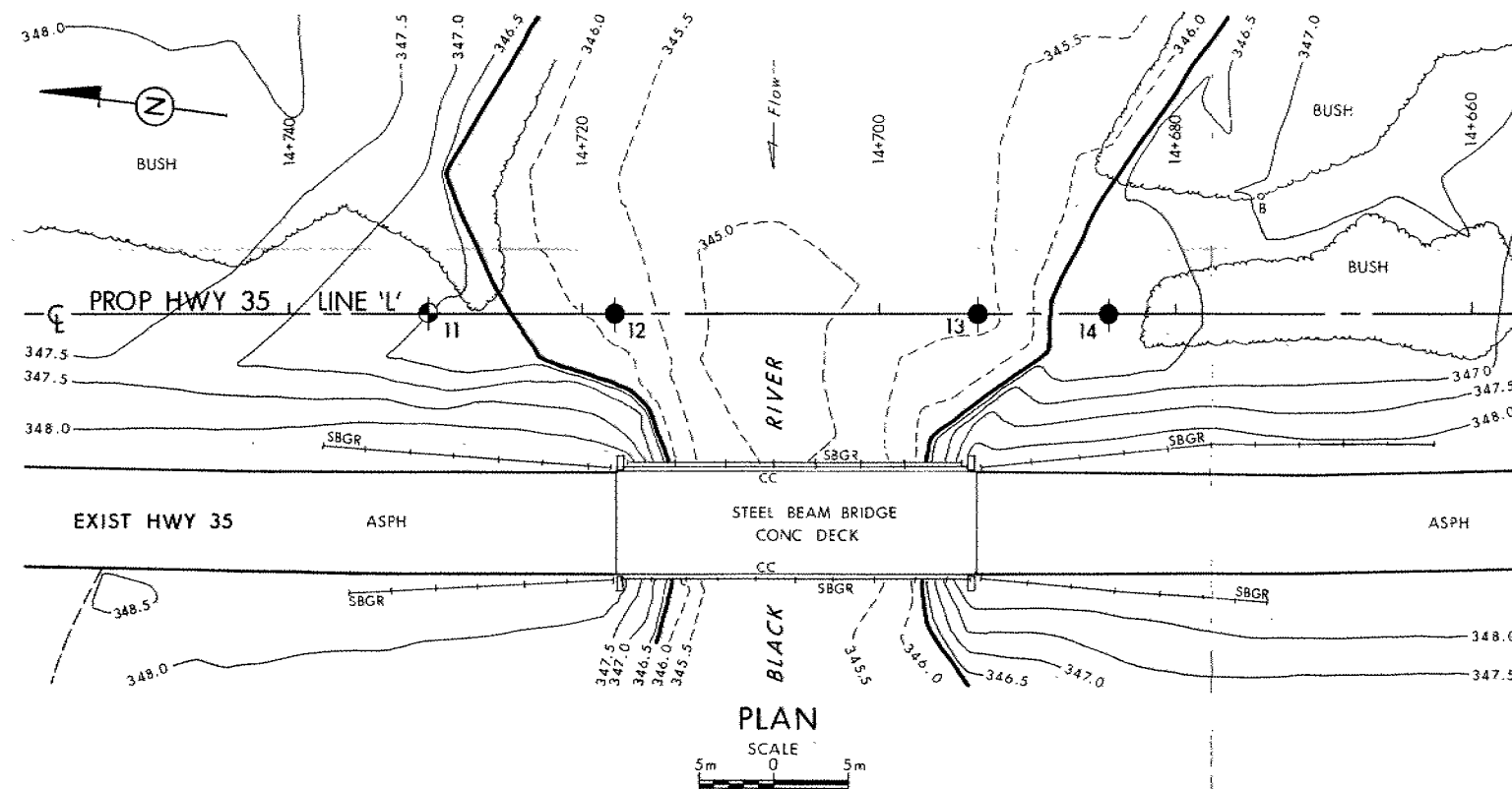
u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa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

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'_{v0}	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						



PROFILE PROPOSED HWY 35 LINE 'L'

METRIC

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

CONT No
WP No 96-86-02

BLACK RIVER

SHEET

BORE HOLE LOCATIONS & SOIL STRATA



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)
- WL at time of investigation 1996 05 and 06

No	ELEVATION	STATION	OFFSET
11	346.5	14+730.4	℄
12	346.3	14+717.8	℄
13	346.3	14+693.3	℄
14	346.4	14+684.6	℄

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 GC 2.01 of OPS Gen. Cond.

REV.	DATE	BY	DESCRIPTION
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Geocres No 31E-118

HWY No 35 LINE 'L'	DIST 52
SUBMD KA CHECKED DATE 1996 09 12	SITE 40-05
DRAWN RS CHECKED	DWG 968602-A



Black River Bridge

W.P. 96-86-02

From: Tae C. Kim
To: MTOHS2.STKITS3.Husain, MTONR.NORTHBAY.McCormiM
Subject: Cont 97-58, Black R Br, Site 40-05, Hwy 35 -Reply

Dear Mike !

Further to our telephone conversation this morning, this e-mail summarizes our discussion as follows:

- 1) If the pressure is exceeded certain level, it will cause some damage or some settlement on bridge foundations. It is therefore suggested to discuss with Structural office and monitor the possible settlement.
- 2) It is also recommended that the approach embankment should be monitored to detect any additional settlement due to the vibration, since the embankment is sitting on Loose Sand.

Extreme care should be exercised during the explosive operation.
If you need further information, please contact this office.

Thanks

Tae

FILE

>>> Mike McCormick 07/13/98 04:50pm >>>

The contractor proposes to use explosives to remove the piers of the existing structure. I will send you a fax of the blasting information but his proposal is to drill 5 - 35 mm holes per pier caisson and load each hole with 0.6 kg of explosive. This gives a peak particle velocity of 8.0 cm/s and a pressure in the water of 100 kPa at a distance of 5.94 m. The new structure is approximately 4.5 m away. Any comments/concerns?

Mike

CC: Kazmiero, Ahmad, MTONR.NORTHBAY.FurstPer, MTONR.NO...



memorandum

To: Mike McCormick, P. Eng.
Sr. Structural Engineer
Structural Section
Northern Region

1997 10 10

From: Pavements and Foundation Section
Room 223, Central Building
Downsview, Ontario

Re: Black River Bridge, Hwy 35, Str. Site 40-05
Contract 97-58, District 52

At your request, we visited the above site on October 08, 1997. We met Steve Cunningham and Brian Ridley of MTO Northern Region Construction. We also met Terry Desmarais of Fowler Construction Company (General Contractor) and Maurice Larouche of Deep Foundation Contractors (Sub Contractor). We discussed the construction problem and observed the construction in progress. At the time of our site visit, several piles were already driven through the preaugured holes. The pile driving with a diesel hammer was in progress at the proposed north abutment location. Mr. Larouche reported that the diesel hammer was a D-22. The pile driving was easy and the pile reached to the bedrock without any problem. Earlier, a drop hammer was used and there were problems as the piles could not penetrate through the dense sand deposit above the bedrock. We think, for the soil condition reported in the Foundation report and the depth to the bedrock, the Contractor should have used a diesel hammer at the first place.

At the proposed abutment locations sheet piles were used to construct the cofferdams. Excavation to the proposed underside of the abutment wall was carried out within the cofferdam. Two CSP pipes were installed with the cofferdams and sump pumps were used for dewatering. There were no signs of any heave or boiling. Within the cofferdams crushed stones were placed at the base of the excavation to act as working pads. To construct the preaugured holes an instrument was devised. The instrument consisted of a casing and a casing plug. The casing was of the same size as the proposed preaugured holes. The plug was a cylindrical shape with closed bottom. The cylinder would fit inside the casing. We understand that the plug and the casings were driven first together to the required preaugured hole depths. The plug was then pulled out and the casing was immediately filled with one metre of Ottawa Sand. The pile was then driven inside the casing. Once the pile reached to the bedrock, the casing was filled with the Ottawa sand. Another plug with closed bottom but an "H" shape opening in the bottom was also devised to keep the casing open after the pile was driven. This plug was used when it was felt that after pile driving some sand had backed up in the casing. This plug provided required opening for placing Ottawa sand inside the

casings. The contractor was planning to pull out all the casings once the piles were driven to the bedrock.

We found that the dewatering scheme was effective and no boiling was taking place in the excavation

A handwritten signature in cursive script, appearing to read "K. Ahmad".

K. Ahmad, P. Eng.
Foundation Engineer

For

T.C. Kim, P. Eng.
Senior Foundation Engineer

From: Ken Ahmad
To: MTOHS2.STKITS3.Husain
Date: 10/2/97 12:25pm
Subject: Contract 97-58, Black River Bridge, Hwy 35, District 52

Iqbal:

We received a call from Mike McCormick of Structural Section of Northern Region. They are having problem with constructing preaugured holes for integral abutments at the above site. In order to construct the hole the excavation is to take place in native sand below water table. This is difficult because the bottom of the hole would boil and the hole would not be stable. Also, we do not know how the Contractor will excavate below water level in order to construct the pile caps. This is something we have to find out.

We think that to deal with this preaugured hole problem, the Contractor should first drive sleeves to the required level (The contract drawing does not ask for sleeves). The inside of the sleeves should then be excavated while maintaining positive hydrostatic head inside the sleeves so that the boiling does not take place. The Ottawa sand (or equivalent) should then be placed inside the sleeve and then the pile should be driven through the Ottawa sand. Perhaps the use of bentonite slurry would not be suitable as the Ottawa sand would not be able to displace it.

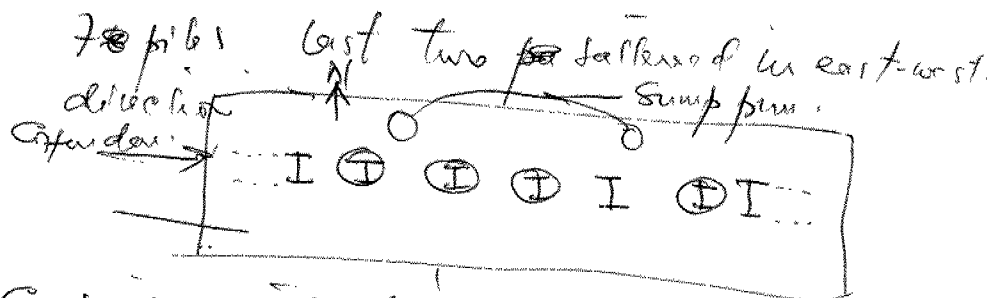
We wanted to review our proposal with you in case you may have any comment on our proposal, or you have any other idea.

P.S. Tac mentioned that perhaps there are new approach for constructing the integral abutment. In the new approach perhaps there will be two sleeves. The inner sleeve will be filled with Ottawa sand. In the Report SO-96-01 for Integral Abutment Bridges, there is no details of the new concept. If you have any revised report or some figures with such details, would you please provide me a copy?

Thanks

Ken

CC: Kim



Terry Desmarais
DESMARAIS

Fowler Construction Company

Contractor pile driving
deep foundations Contractor

~~Maurice~~
Maurice Laroche

[STEVE Cunningham,
Brian Redley,]

Memorandum

To: Gilles Truchon

From: Tim Fawcett, Quality Control Manager

Date: 25/09/97

Re: MTO 97-58 Pile Backfilling Ottawa type sand

This is the gradation of sample of sand supplied by Hutcheson Sand & Gravel of Huntsville for the Ottawa type sand specified.

Sieve	% Passing	Specification
2.00 mm	100	100
600 um	83.8	80-100
425 um	63.8	40-80
250 um	21.8	5-25
150 um	1.7	0-6

From: Ken Ahmad
To: MTONR.NORTHBAY.McCormiM
Subject: Two Questions -Reply

Mike:

There is no designated source for Ottawa sand. However, this material has been used in several projects and the Contractor should know where this material can be obtained from. If there is any difficulty in obtaining this material, then local sand that meet the gradation requirement for the Ottawa sand can be used instead of Ottawa sand.

Regarding your second question on the resistance factor, the Resistance factor shown on Table 6-6.2.2 in OHBDC 91 does not apply for the bond resistance of the dowels. You do not have to apply any resistance factor to the Factored Bond resistance value @ ULS. The factored ULS bond resistance given in our report is already reduced by applying a resistance factor.

We will send you the preliminary recommendation for Net Lake within few days.

Ken

>>> Mike McCormick 09/23/97 09:09am >>>

I have a couple of questions regarding two separate projects

1. Cont 97-58, WP 96-86-00 Black R Br- We have specified a uniformly graded sand in the pre augured holes based on an sp that you gave us. Do you have any idea of where this material (Ottawa sand) is available?

2. WP 689-93-05 Muskoka Rd 32/38 Underpass- in the Foundation Report the dowels into rock have a factored capacity of 500 kPa for the bond between the bedrock and the grout. What resistance factor as discussed in Section 6-6.2.2 of the OHBDC 91 would be used - 0.5?

CC: Kim

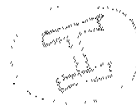
Construction
Project manager
Steve Cunningham
705-766-0573
[STEVE CUNNINGHAM
BRIAN RIDLEY]
Site office
(416)

Plug - Casing

Ken Ah

W.P. 96-86-00
Cont. 97-58

310x100



From: Ken Ahmad
To: MTONR.NORTHBAY.McCormiM
Date: 4/25/97 8:43am
Subject: WP 96-86-02, Black River Br., Site 40-05 -Reply

Mike:

The normal practice is to drive the pile first through the pre-augured hole and then the hole is filled with Ottawa sand (Uniformly graded sand). It is imperative that the sand should meet the recommended gradation requirement. If the gradation requirement is not met then the sand will compact over the time and the purpose would not be served. Regarding your second question, you are right, it is a typo, it should read micrometre rather than millimetre.

>>> Mike McCormick 04/23/97 03:39pm >>>

Regarding the piles for the integral abutment for the above noted structure please answer the following:

-what goes in first the piles or the sand in the pre-augured holes? The general consensus here is that driving the piles after the holes are filled with sand will cause the material to compact tightly and restrict the lateral movement of the piles.

-is there another sand type with a gradation similar to our regular specs for Gran B for example that could be used in place of the gradation given in the Foundation report? The concern is that the contractor will have a very hard time securing this material and it's small quantity will be very expensive. The gradation shown in the report indicates 80-100% passing for 600 mm, 40-80% passing for 425 mm, etc. I believe it should read 600 um (micrometer), etc. Please confirm.

CC: Kim

From: Ken Ahmad
To: MTONR.NORTHBAY.McCormiM
Subject: Preaugered Hole for Integral Abutment

Mike:

As discussed, I have confirmed with Tac Kim, that if the pre-augured hole that will contain Ottawa sand is below water table, there will be no adverse effect to its performance.

Ken

CC: Kim

From: Ken Ahmad
To: MTONR.NORTHBAY.McCormiM
Subject: Black R Br., Site 40-05, WP 96-86-02 -Reply

Mike:

Oslo Points, or any other types of rock points are used when the rock surface is very steep. Oslo Points then bite into the rock slope for better seating. Also, the rock points are very expensive and should be only used where necessary. At this site the rock surface is not steep, therefore driving shoes should be used to reinforce the pile tips. Please refer to the OPSD 3301.00 drawing for details of driving shoe.

Regarding other types of HP piles, please note that HP 310X110 and HP 310X79 are frequently used on MTO projects. May be because they are easily available in the market? I do not know. That is why we only provided capacities for those two common types of piles. Also, please note that for piles on bedrock the structural capacity of the piles governs the bearing capacity of the piles. The geotechnical bearing capacity of bedrock is very high as compared to the structural capacity. The capacities provided in our report are the agreed upon capacities based on the structural capacity of the piles. In fact these capacities should be increased because I understand that now piles are produced with a better grade of steel and they can take more loads. You should calculate the structural capacity of the piles and decide how much load you can put without causing enough elastic compression (that you can call settlement) in the pile members and without causing any buckling effect in the piles. Any safe load (from the structural point of view) is likely to be less than the geotechnical capacity of the bedrock.

Thanks

Ken Ahmad, P. Eng,

for

Tae Kim, P.Eng.
Sr. Foundation Engineer

CC: Kim

From: Ken Ahmad
To: MTONR.NORTHBAY.FurstPer, MTONR.NORTHBAY.McCormiM
Date: 7/3/96 11:11am
Subject: W.P. 96-86-02, Black River Bridge at Hwy 35, Spread Footing

July 03, 1996

Mike:

Further to our preliminary recommendation which we sent to you yesterday, we would like to add the following:

We have reviewed the final E-plan and understand that still the length of the proposed bridge is not finalized. Unless the abutments of the new bridge are constructed on the river bank, spread footings on rockfill placed in the river may not be feasible from the hydrological point of view.

If the rockfill is placed in the river this will reduce the channel width which may affect the velocity of the river and scour depth at that location. This is something hydrological office can comment on. We thought we just bring the matter to your notice for your structure planning.

Thanks

Ken

CC: Kim



memorandum

To: P. Furst, P. Eng.
Head, Structural Section
Northern Region
North Bay, Ontario P1B 8L2

1996 07 02

Attn: M.S. McCormick, P. Eng.
Structural Engineer

From: Pavement & Foundation Design Section
Room 315, Central Building, Downsview, Ontario

Re: Foundation Recommendations
W.P. 96-86-02, Black River Bridge Replacement
Site 40-05, Highway 35,
District 52, Huntsville

The field investigation for the above-mentioned project has been completed. The fieldwork was carried out for the proposed bridge structure and approach embankments.

This memorandum outlines the preliminary foundation recommendations that should provide sufficient information for you to carry out your structural design. Although we do not anticipate any major changes in the final recommendations, the recommendations may be slightly modified in the final report due to further analyses. The final report will be provided as soon as the drawing is ready.

Introduction

This report summarizes the results of a field investigation which was carried out for the construction of Black River Bridge at Hwy 35 (at a new alignment), about 8km south of Hwy 117.

The investigation was carried out at the request of Northern Region Structural Section. These recommendations apply to proposed bridge structure and its approaches within 20m of the structure.

Four boreholes (BH 11 through 14) were put down for the Foundation Investigation. Two boreholes (BH11 and BH 14) were drilled in the ground. These boreholes were terminated in bedrock at depths 43.7m and 46.2m respectively. Bedrock was encountered at depth 42.1m (BH 11) and 44.7m (BH 14) respectively. Two boreholes BH12 and BH13 were put down in the river. These boreholes were terminated at a depth of 18.8m below river level.

Subsurface Conditions

Overlying the bedrock which was at a depth of 42.1m to 44.7m, the subsurface conditions across the site is composed of very loose to compact, fine to medium sized sand with some silt and trace of gravel.

Groundwater Condition

Groundwater was monitored in open boreholes. Groundwater in the boreholes was similar to water level in the river, which was at the time of investigation 346.3m. It should be noted that the groundwater is subject to fluctuation with the river level.

DISCUSSION AND RECOMMENDATIONS

General

Originally, the scheme was to shift the present alignment of Hwy 35 to the west side of the existing highway, and for that plan a foundation investigation was carried out by McClymont and Rak in 1990. The plan was then changed. In the new proposal the alignment (Line 'L') will be about 14m from the centreline and on the east side of the existing highway. For the new bridge two options are under consideration. One with a bridge span of 22m and the other with a span of 32m. The final span of the bridge is not known at this time.

The existing bridge is a steel beam bridge with a concrete deck with a clear span of 12.2m and 6.7m wide.

It is understood that it is desired to construct a jointless bridge on integral abutment. For the 22m span bridge a steel girder integral abutment or concrete rigid frame bridge is proposed while for the 32m span a steel girder integral abutment bridge is recommended.

Structure Foundations

The proposed profile grade, in the vicinity of the crossing, will be at approximate elevation 349.0m. The approach fills will be approximately 2.5m to 3.5m high.

Based on the subsoil conditions, which is mainly very loose to compact sand, the most suitable structure from a cost point of view appears to be spread footing founded on rockfill. However, alternatives should be assessed based on cost, as well as construction and environmental considerations.

Spread Footings on Rockfill

The proposed single span girder bridge can be supported on spread footing founded on rockfill. The very loose sand with organic traces will be subexcavated to elevation 346.0m. The excavation should be backfilled with rockfill. The rockfill core must have a minimum thickness of 2.0m.

The recommended bearing capacities for the footings on rockfill at elevation 348m or above, as per the OHBDC 91 are as follow:

Factored Bearing Capacity at U.L.S.	=	600 kPa
Bearing Capacity at S.L.S.	=	250 kPa

The rockfill should extend out at least 2.0m beyond the footing's underside. The side slope of the rockfill should be maintained at 1.5H:1V or flatter. The rockfill in the upper 0.6m depth, immediately under the footing and under the plan limit of the footing should be limited to 300mm in size. The surface of the rockfill should be covered with a 150mm thick concrete mat to provide good working base.

The settlement of the rockfill is estimated to be about 2 to 3 cm. The settlement of the underlying soils will be immediate and will occur during construction.

Deep Foundation

The structure can be supported on timber friction piles. The timber piles may be driven to any required depth but at least to elevation 335.0m. For pile size 36-treated timber with an embedded length of about 11.5m the recommended bearing capacities as per the OHBDC 91 are as follows:

Factored Axial Capacity at U.L.S.	=	340 kN/pile
Axial Capacity at S.L.S.	=	225 kN/pile
Factored Lateral Capacity at ULS	=	17 kN/pile
Lateral Capacity at S.L.S.	=	11 kN/pile

Lateral capacity may be supplemented by the horizontal component of battered piles.

Alternatively, if higher capacity is required or an integral abutment is considered, then the structure can be founded on H-piles driven to bedrock [bedrock depth 42.1m (BH 11) and 44.7m (BH 14)]. However, this alternative should be assessed based on cost comparison. The recommended bearing capacities of H-piles founded on bedrock are as follow:

	<u>HP 310X110</u>	<u>HP 310X79</u>
Factored Axial Capacity @ ULS	1600 kN/pile	1150 kN/pile
Axial Capacity @ SLS for 25mm	1150 kN/pile	825 kN/pile
Factored Lateral Capacity at ULS	80 kN/pile	60 kN/pile
Lateral Capacity at S.L.S.	60 kN/pile	40 kN/pile

Due to long steel piles, the capacity of the pile may have to be reduced due to other structural consideration such as buckling effect etc. These aspects should be analyzed by the structural section.

In order to facilitate pile driving, particle sizes of any fill placed beneath the pile locations should be restricted to 75mm.

Integral Abutment

In the previous investigation boreholes were not advanced to the bedrock and therefore, bedrock elevation was not known. In the present investigation bedrock elevation has been determined which is 42.1m (north shore, BH 11) to 44.7m (south shore BH 14) below ground surface. This site meets all the requirements for an integral abutment i.e., length of structure, type of structure, structure geometry, abutment height and pile length etc. Therefore, an integral abutment may be considered for this site.

The integral abutment will be constructed on steel piles driven to bedrock. The piles for integral abutment should be in one row with weaker axis oriented normal to the direction of movement. The piles shall be driven through 0.5m diameter and 3m deep holes filled with uniformly graded sand. Following is the NSSP for the special sand used for integral abutment.

NSSP - Backfill to Integral Abutment-Augured Hole

The annular space between the preaugured oversize hole and the pile shall be backfilled with uniformly graded sand. The gradation for the uniformly graded sand shall be as follows:

MTO SIEVE DESIGNATION	PERCENTAGE PASSING BY MASS
2 mm (#10)	100
600 µm (#30)	80 - 100
425 µm (#40)	40 - 80
250 µm (#60)	5 - 25
150 µm (#100)	0 - 6

Alternatively, commercially available materials which meets the above gradation may be considered.

Embankment Stability

The height of embankment will be approximately 2.5m to 3.5m. Prior to placement of fill, all surficial topsoil or any organic material should be removed within the plan limits of the embankments. The embankment should be then constructed with rockfill or native sand. The embankment fill consisting of rockfill can be placed at 1.5H:1V.

The native sand is also suitable as fill material for embankment construction. If sand is used, the side slope of the embankment should be constructed at 2.5H:1V or flatter. If sand fill is used, the lower slopes must be protected from erosion which can consist of vegetation, or 600mm rock protection to the potential high water level.

There are no settlement concerns for the embankment constructed with the above mentioned materials. Settlement will be elastic in nature and should occur during construction.

No stability problems are anticipated for the proposed height of permanent embankments.

Lateral Earth Pressure

Free draining granular material such as Granular 'A' or 'B', or rockfill is recommended as appropriate backfill to abutment walls to prevent hydrostatic pressure build-up.

If rockfill is used for approaches, special care will be required to avoid damaging the abutment. It would be preferable to place a 0.3m cushion of Granular 'A' or smaller rockfill (with diameter of less than 300mm), between the structure and the main mass of rock fill. Granular material may also be used at the approaches.

For design purposes, the following properties for backfill are recommended:

Granular 'A'	$\gamma = 22.8 \text{ kN/m}^3$	$\phi = 35^\circ$
Granular 'B'	$\gamma = 21.2 \text{ kN/m}^3$	$\phi = 30^\circ$
Rockfill	$\gamma = 18.0 \text{ kN/m}^3$	$\phi = 35^\circ$

At-rest condition (K_o) may be assumed to apply for rigid and unyielding structure and active condition (K_a) will apply for yielding structure.

Resistance to Lateral Forces

For footings placed on rockfill, the sliding resistance between the concrete footing and rockfill should be computed as per OHBDC 91. For sliding within the rockfill $\phi = 35^\circ$ should be used for calculation, and $\phi = 30^\circ$ should be used for calculating sliding at the footing/rockfill interface.

Frost Protection

A soil cover of 1.8m or equivalent will be required for frost cover.

Excavation and Dewatering

The river level was at elevation 346.3m at the time of Foundation investigation. Any excavation below the river level will require dewatering. Depending on the final scheme of the bridge, sheet piling enclosure may be required to assist in dewatering. For footings on rockfill, sub-excavations to elevation 346, can be backfilled with rockfill without major dewatering.

If excavations extend below the prevailing groundwater elevation, a dewatering SP will be required.

The SP should alert the Contractor that there are noncohesive soils at this site and that this soil is susceptible to disturbance under conditions of unbalanced hydrostatic head.

It is anticipated that sheet pile cofferdams may be required for the construction of excavations in the dry. For design purposes, it may be assumed that the sheet piles should extend a minimum depth equivalent to the groundwater head below excavations. However, the dewatering scheme is the responsibility of the Contractor.

If excavations below the water table are required for placement of rockfill, it may be possible to excavate and backfill below water level, thus eliminating the need for dewatering.

Temporary excavations in the native overburden should be 1.5H:1V or flatter, and 2H:1V or flatter below the groundwater level.

If there are any questions, or any explanations are required regarding this memo, please contact this office.

S.Q. (Ken) Ahmad, P. Eng.
Foundation Engineer

For

T.C. Kim, P. Eng.
Senior Foundation Engineer

memorandum



To: P. Furst, P. Eng.
Head, Structural Section
Northern Region

Attention: M.S. McCormick, P. Eng.

From: Pavements and Foundations Section
Room 315, Central Building

Subject: Conformance of Foundation Reports to OHBDC91
Gibson River Bridge, W.P. 209-90-01
McDonald Lake Bridge, W.P. 206-90-01
Goat Island Channel Bridge, W.P. 322-85-01
Black River Bridge, W.P. 96-86-02
Musquash River Bridge, W.P. 208-90-01

Date: 95 05 29

We refer to your memorandum dated 95 03 02 concerning the conformance of the above-noted foundation reports to OHBDC (3rd edition). These reports were completed prior to the implementation of the new Bridge Code. We have reviewed the reports and related correspondence and have the following comments:

1. Gibson River Bridge (W.P. 209-90-01) -

- The foundation capacity and settlement recommendations have been revised via our memorandum dated 95 03 02, based on updated information. These recommendations are in line with OHBDC91.
- Lateral earth pressures should be computed in accordance with clause 6.7.4 of the new code.
- Sliding resistance of footings should be computed in accordance with clause 6-8.4.3 of OHBDC91. $\phi = 35^\circ$ can be used for sliding within the rockfill and $\delta = 30^\circ$ can be used for sliding at the footing/rockfill interface.

2. McDonald Lake Bridge (W.P. 206-90-01) -

- The recommended Factored Bearing Capacity at ULS of 10,000 kPa is still valid under OHBDC91.

- Sliding resistance of footings should be computed in accordance with clause 6-8.4.3 of OHBDC91. $\phi = 35^\circ$ can be used for sliding along the rock joints within the bedrock and $\delta = 30^\circ$ can be used for sliding at the footing/bedrock interface. Recommendations regarding rock dowels were given in our recent memorandum dated 95 04 27.
- Computation of lateral earth pressures should be in accordance with OHBDC91 clause 6.7.4.

3. Goat Island Channel Bridge (W.P. 322-85-01) -

- The recommended Factored Bearing Capacity on limestone bedrock of 2000 kPa is still valid under the new code.
- The Bailey bridge has been revised to be founded on rock fill rather than at the channel bottom. This was noted during previous drawing reviews.
- Computation of lateral earth pressures should be in accordance with clause 6.7.4 of OHBDC91.
- Sliding resistance of footings should be computed in accordance with clause 6-8.4.3 of OHBDC91. $\phi = 40^\circ$ can be used for sliding along the joint planes with bedrock and $\delta = 35^\circ$ can be used for sliding at the footing/bedrock interface.

4. Black River Bridge (W.P. 96-86-02) -

- The recommended bearing capacities for footings founded on rock fill is still valid under the OHBDC91. There is no SLS Type II in the new code. The capacity given for SLS Type II can be used as the SLS capacity under the OHBDC91.
- Recommendations for piles have been superseded by our memorandum dated 94 07 21. Again, there is no SLS Type II in the new code and the same capacity should be used for SLS.
- Recommendations given for lateral earth pressures are still valid.
- Sliding resistance of footings should be computed in accordance with clause 6-8.4.3 of OHBDC91. $\phi = 35^\circ$ can be used for sliding within the rockfill and $\delta = 30^\circ$ can be used for sliding at the footing/rockfill interface.

- For the design of sheetpiles, computation of lateral earth pressures should be in accordance with clause 6.7.4 of the OHBDC91.

5. Musquash River Bridge (W.P. 208-90-01) -

- It is understood that the overall design of the bridge will be revised in view of a grade raise requirement due to hydrology concerns. You have also indicated to us earlier on employing integral abutment design for this structure. We have offered our comments via the memorandum dated 95 01 04.
- In general, the foundation capacities for driven piles and caissons provided in the original report are still valid under the new code. However, for HP 310X79 piles, a SLS capacity of 900 kN/pile can be used. The corresponding ultimate capacity for Hiley Formula is 2700 kN. Please note that there is no SLS Type II in OHBDC91 and the same capacities given before in the original report for SLS Type II should be used for SLS. Downdrag forces on piles and lateral capacities, etc, will be reviewed when the revised layout of the bridge is available.



D. Kwok, P. Eng.
Project Foundation Engineer
for
T. Kim, P. Eng.
Senior Foundation Engineer

M E M O R A N D U M



P. Furst
Head, Structural Section
Northern Region

Date: July 21, 1994

Attn: M.S. McCormick, P. Eng.
Structural Engineer

From: Foundation Design Section
Room 315, Central Building
Downsview

Re: Black River Structure
Hwy 35, Site 40-05
W.P. 96-86-02
District 11, Huntsville

We have reviewed the foundation alternatives illustrated on the General Arrangement Drawings (P1A and P1B) and our comments are as follows.

- 1) The encountered soils at the site are susceptible to disturbance under conditions of unbalanced hydrostatic head.

Should the excavation be carried below the prevailing water level and dewatered, measures will be required to prevent the foundation soil from disturbance.

- 2) In view of the encountered site conditions, the most practical alternative is the deep foundation. The deep foundation units should consist of size 36 treated timber piles driven as long as possible but at least to El. 335.

The following capacities are recommended:

Factored Axial Capacities at U.L.S. = 340 kN
Axial Capacity at SLS ~~Type II~~ = 225 kN

The fill material should not contain grain sizes larger than 50 mm at locations where timber piles have to be driven. In order to eliminate the dewatering for the pile caps, one or the other procedures are feasible:

- a) the pile caps should be kept above the prevailing water level
- b) excavate under water, drive the piles and place the concrete by tremie method

memorandum



Tel: (416) 235-5652

To: Per Furst
Head, Structural Section

Date: 91 04 19

Attn: T. Pepper

Re: Black River Bridge
W.P. 96-86-02, Site 40-05
District 11 - Huntsville

I have reviewed your proposal for a new structure at this site and also discussed my findings with the Foundation Section.

While I understand that your preference is for a jointless bridge at this site, it may not be a feasible alternative.

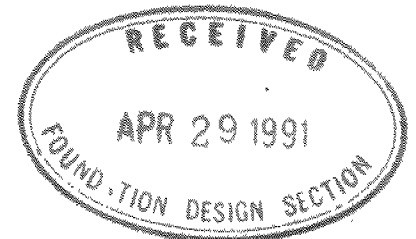
The underlying soils at this site are loose sand to a depth of 30 metres below grade, with only marginal increase in density with depth. The foundation report recommended timber friction piles with a capacity of 225kN at SLS II. According to Dave Dundas this soil is not suitable for steel H or pipe piles, because little friction can be developed. The resistance for a timber pile is gained through the taper of the pile.

While it has occasionally been done with timber piles according to literature, we have only used steel H piles for integral abutments so far.

I therefore would suggest that a bridge with expansion joints be considered for this site.

It will be more economical to build a single span, rather than a 3-span structure, construction depth, if necessary can be reduced to 1.50 to 1.60 m depending on crossfall (1100 plate girder, 50 haunch 225 + 90 slab).

Alternatively, the Foundation Section recommended that instead of the proposed rockfill, use a well compacted granular A pad with rock protection.



A SLS capacity of 250 KPa may be used under the footings. Subexcavating the very loose surficial fill with organic traces to Elev. 346 means that granular will be placed and compacted above water level.

A single span bridge founded on granular pad may be the most economical solutions for this site.



B. Farago
Design Engineer

BF/sl

c.c. D. Dundas - Foundation Section

MEMORANDUM

To: P. Furst
Head, Structural Section
Northern Region

Date: 1991 01 17

Attention: T. Pepper

From: Foundation Design Section
Room 315, Central Building

Re: Foundation Investigation Report For
Black River (Wren Lake) Structure
W.P. 96-86-02, Site 40-05.
Highway 35, District 11, Huntsville

The Foundation Design Section retained McClymont & Rak Engineers Inc., consulting geotechnical engineers, to carry out a foundation investigation for the above-noted project. The Foundation Investigation and Design Report is forwarded under cover of this memo.

After preparing the consultant agreement, this office provided technical supervision including the establishment of terms of reference and careful review of the consultant's progress at all stages of the project. Several meetings were held with the consultant during which our comments were incorporated into the report. The Foundation Investigation (factual) portion of the report was reviewed only for format, and its accuracy and completeness are the responsibility of the consultant. The Foundation Design (recommendation) portion of the report has been carefully reviewed by this office based on the subsurface information provided by the consultant.

The following comments supersede the Foundation Investigation and Design Report:

- 1) The pile capacities for Size 36 treated timber piles, driven as long as possible but at least to elev. 335 m, should be as follows:

Factored Axial Capacity at ULS = 340 kN
Axial Capacity at SLS Type II = 225 kN

Factored Lateral Capacity at ULS = 17 kN
Lateral Capacity at SLS Type II = 11 kN

Lateral capacities may be supplemented by the horizontal component of battered piles.

In order to facilitate pile driving, particle sizes of any fill placed beneath the pile locations should be restricted to 75 mm.

- 2) If excavations extend below the prevailing groundwater elevation, a dewatering SP will be required.

The SP should alert the Contractor that there are noncohesive soils at this site and that this soil is susceptible to disturbance under conditions of unbalanced hydrostatic head.

It is anticipated that sheet pile cofferdams may be required for the construction of excavations in the dry. For design purposes, it may be assumed that the sheet piles should extend a minimum depth equivalent to the groundwater head below excavations. However, the dewatering scheme is the responsibility of the Contractor and the scheme should be submitted for review a minimum of 10 working days prior to construction.

If excavations below the water table are required for placement of rock fill, it may be possible to excavate and backfill below water level, thus eliminating the need for dewatering.

- 3) At the abutments, earth approach embankments should be protected by a minimum of 0.6 m of rock protection. The rock protection should extend, vertically, from the high water level to the base of embankments and 2 m along the channel bottom, and horizontally, a minimum of 10 m on each side of abutments.
- 4) Spread footings on sand are not recommended due to the low strength of the native soil.
- 5) Temporary excavations in the native overburden should be 1.5H:1V or flatter, and 2H:1V or flatter below the groundwater level.
- 6) Regarding the rock fill recommendations on page 5, the size should be limited to 300 mm only in the upper 0.6 m under the plan limits of footings.
- 7) If rockfill is used as backfill to structures, the size should be restricted to 300 mm in the 0.6 m adjacent to structures.

If there are any questions regarding the report or during the design please contact this office.

cc: P. Furst (3)
J. McDougall
G. Todd
S. Wilson (2)
K. Bassi
S. Dunham
E. Joseph
B. Sharpe (cover only)
File

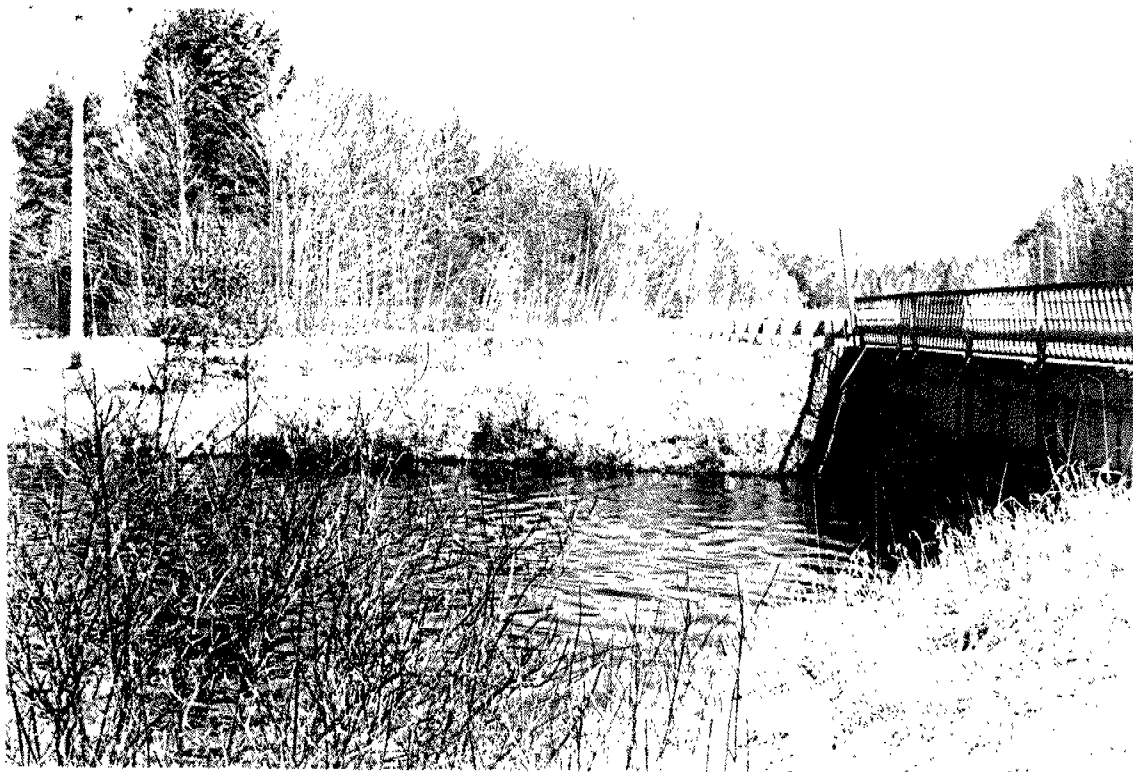
D. Dundas

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

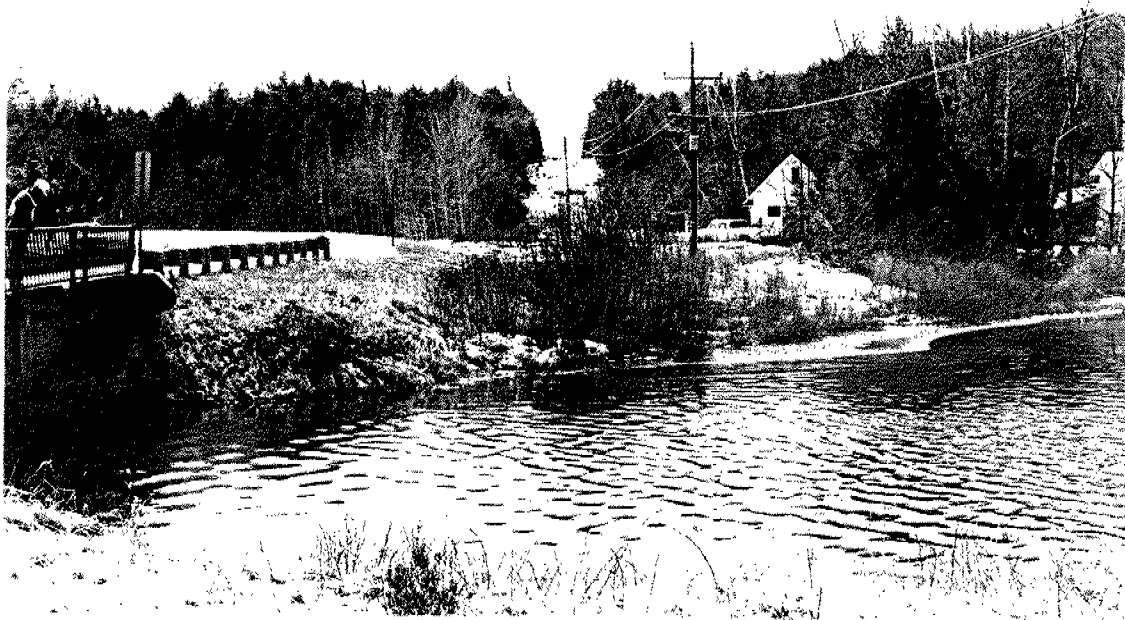
For

M. Devata, P. Eng.
Chief Foundation Engineer

40-5 BLACK RIVER
Hwy. 35



LOOKING NORTH ALONG 'K' LINE



LOOKING SOUTH ALONG 'K' LINE