DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 31C-154

DIST. 41 REGION

W.P. No. 48-94-01

CONT. No.

W. O. No.

STR. SITE No. 17-16

HWY. No. 41

LOCATION Hwy 41 & CLARE RIVER

No of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

G.I.-30 SEPT. 1976

Hwy 41 - Looking South
East
Approach


CLARE RIVER

Looking ~~[illegible]~~ South,

~~East Side~~

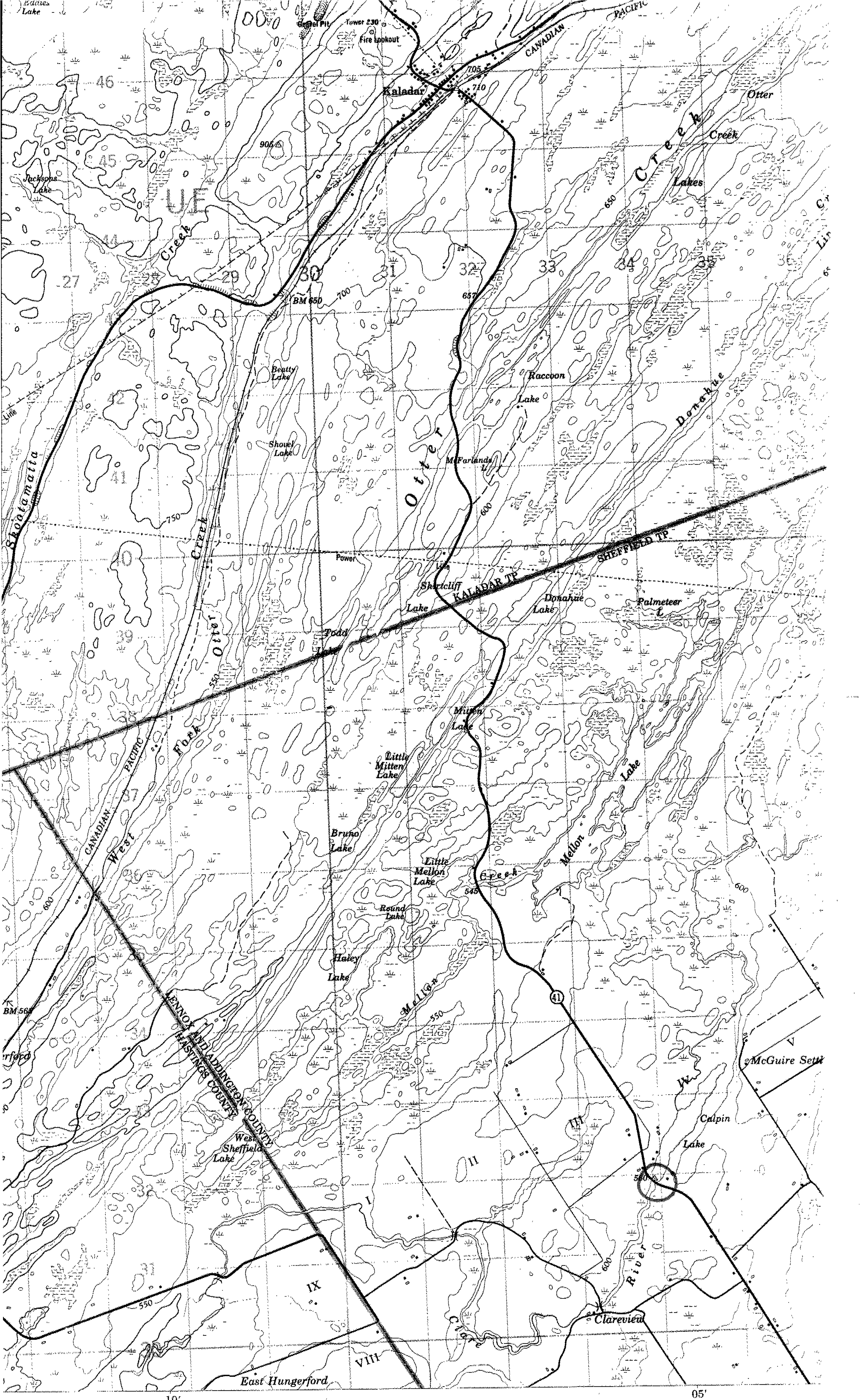
West Side

Looking North
West Side

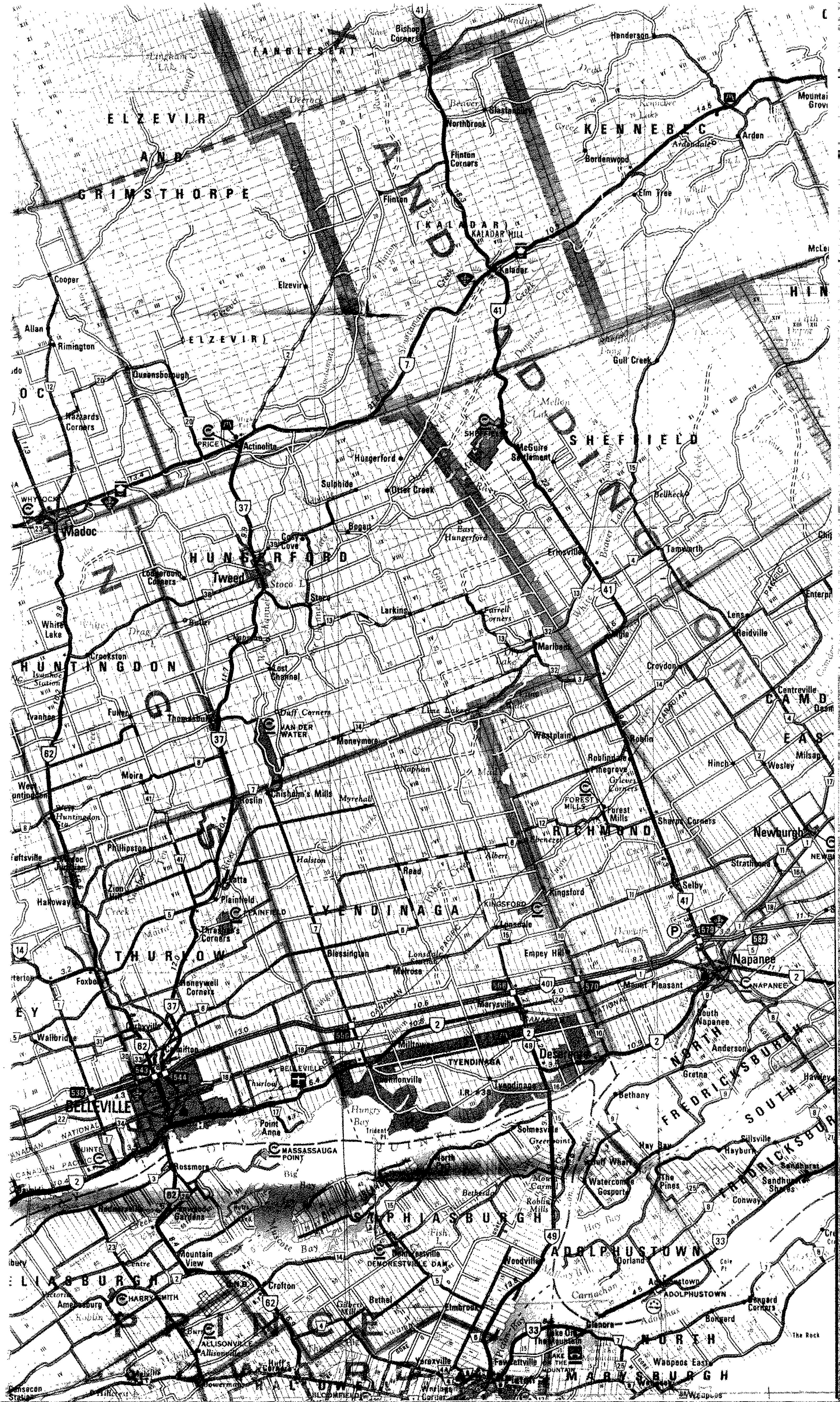
Looking North

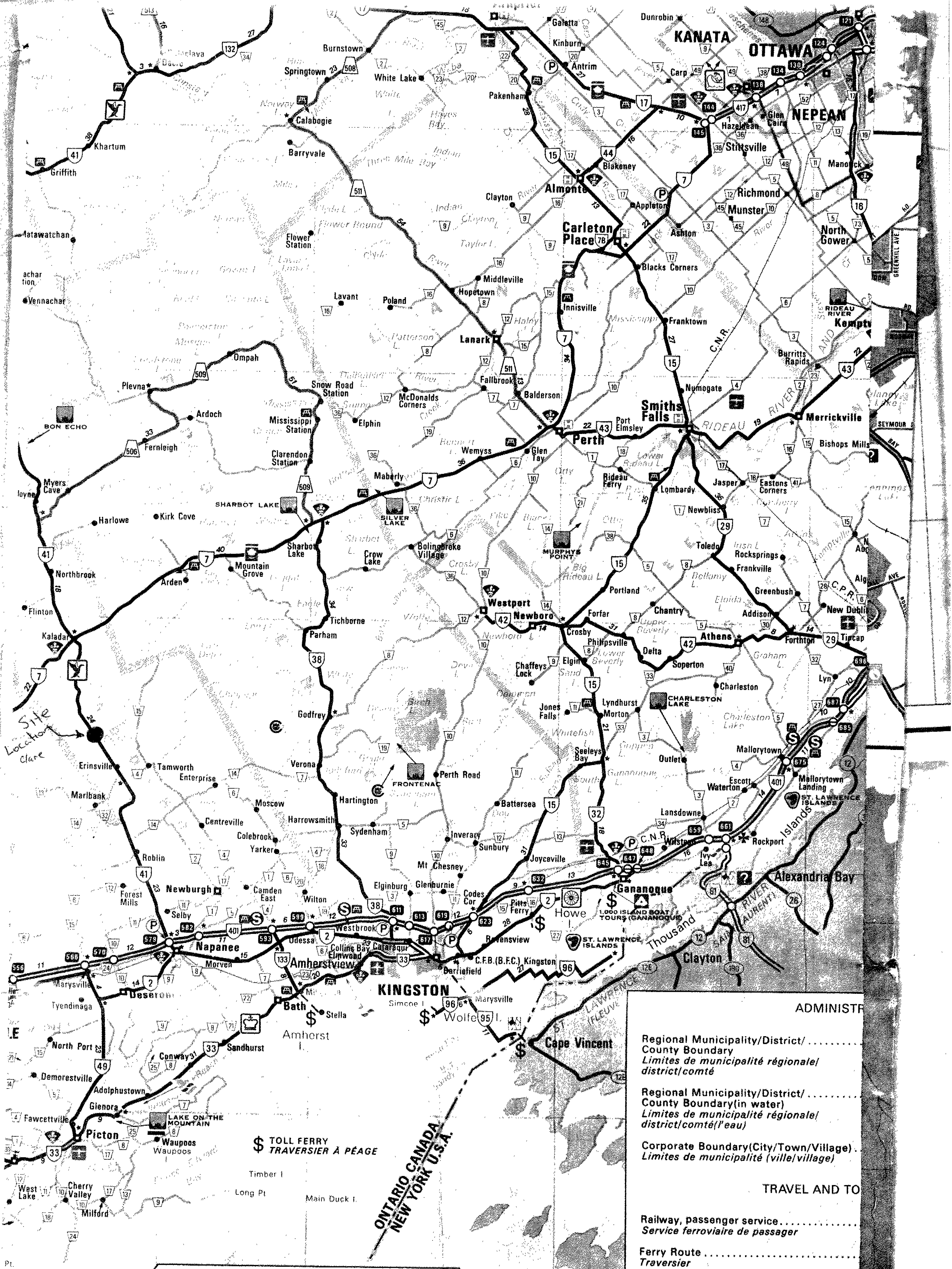
East Side

Looking North East Side



Lake
Gravel Pit
Tower 230'
Fire Lookout
CANADIAN
PACIFIC
Kaladar
705
710
46
45
905
Jacksons Lake
UF
44
Creek
Otter
Creek
Lakes
Creek
650
27
28
29
30
31
32
33
34
35
36
BM 650
700
657
43
Line
42
Beatty Lake
Raccoon
Lake
Shoolamatta
41
Shovel Lake
McFarlands L
Otter
Donahue
600
750
Creek
40
Power
Line
Skirtcliff
Lake
KALADAR TP
SHEFFIELD TP
Donahue Lake
Palmeteer L
39
Todd Lake
Otter
550
38
Fork
Mitten Lake
CANADIAN
PACIFIC
37
Little Mitten Lake
Lake
West
Bruno Lake
Mellon
36
Little Mellon Lake
Creek
545
600
Round Lake
Haley Lake
Mellon
41
550
BM 566
LENNOX AND ADDINGTON COUNTY
HASTINGS COUNTY
34
V
McGuire Settl
rford
IV
III
Calpin
Lake
West Sheffield Lake
II
560
32
I
600
River
31
IX
550
Clareview
Clare
VIII
East Hungerford
10'
05'


ANGLESEA
Bishop Corners
Henderson
Lingham Lak
ELZEVIR
AND
GRIMSTHORPE
Slate Creek
Northbrook
Flinton Corners
Flinton
KENNEBEC
Mountain Grove
Arden
Ardendale
Bordenwood
Elm Tree
KALADAR
KALADAR HILL
Kaladar
Cooper
Elzevir
Allan
Rimington
ELZEVIR
Queensborough
Hazzards Corners
PRICE
Actinolite
Hungerford
Sulphide
Otter Creek
SHEFFIELD
Sheffield
McGuire Settlement
Gull Creek
Mellon Lake
Bellrock
Tamworth
Erinsville
WHYTOCK
Madoc
Bogart
Cosy Cove
HUNGERFORD
Tweed
Stoco Lake
Stoco
East Hungerford
Larkins
Farrell Corners
Marlbank
Lennox
Reidville
Enterprise
Loaderoun Corners
White Lake
Butler
Chapman
Lost Channel
Croydon
Centreville
CAMDEN EAST
Crookston
HUNTINGDON
Ivanhoe Station
Ivanhoe
Fuller
Thomasburg
Duff Corners
VAN DER WATER
Moneymore
Westplain
Roblin
Roblindale
Pinegrove
Grieves Corners
Hinch
Wesley
Milsap
Moira
West Huntingdon
Roslin
Chisholm's Mills
Myrehall
Naphan
FOREST MILLS
Forest Mills
RICHMOND
Sharps Corners
Newburgh
West Huntingdon Sta.
Phillipston
Tuftsville
Madoc Junction
Halston
Read
Albert
Ebenezer
Strathcona
Zion Hill
Halloway
Latta
Plainfield
PLAINFIELD
Thrasher's Corners
TYENDINAGA
KINGSFORD
Kingsford
Lonsdale
Selby
THURLOW
Blessington
Lonsdale Station
Melrose
Empey Hill
Napanee
NAPANEE
Mount Pleasant
Foxboro
Honeywell Corners
Marysville
South Napanee
Wallbridge
Cannifton
Milltown
TYENDINAGA
Deseronto
Anderson
NORTH FREDERICKSBURGH
BELLEVILLE
Thurlow
Shannonville
I.R. #38
Tyendinaga
Bethany
Gretna
Hawley
BELLEVILLE
Point Anne
Hungry Bay
Trident Pt.
Solmesville
Green Point
SOUTH FREDERICKSBURGH
Sillsville
Hayburn
Sandhurst
Sandhurst Shores
Conway
QUINTE
MASSASSAUGA POINT
Rossmore
Big Bay
Hay Bay
Sniff Wharf
Watercombe
Gosport
The Pines
Mount Carmel
Roblin Mills
Bethesda
SOPHIASBURGH
Mountain View
Muscote Bay
DEMORESTVILLE DAM
Demorestville
Woodville
ADOLPHUSTOWN
Dorland
Adolphustown
ADOLPHUSTOWN
Cole Pt.
Carnachan
Bongard Corners
Bongard
AMELIASBURGH
Victoria
Ameliasburg
HARRY SMITH
Roblin
Crofton
Bethel
Gilbert Mills
Elmbrook
Glenora
The Rock
NORTH MARYSBURGH
ALLISONVILLE
Allisonville
Yerexville
Fawcettville
LAKE ON THE MOUNTAIN
Waupoos East
Waupoos
Consecon Station
Hillcrest
Bowermans
Huff's Corners
HALLOWELL
BLOOMFIELD
Wellings Corners
PRINCE EDWARD
HASTINGS
LENNOX AND ADDINGTON
CANADIAN PACIFIC
CANADIAN NATIONAL
2
7
12
14
37
41
49
62
401



INFORMATION COMPILED TO JANUARY 1, 1992
RENSEIGNMENTS RECUEILLIS AU 1er JAVIER 1992

LEGEND *LÉGENDE*

ROAD CLASSIFICATION *CLASSIFICATION DES ROUTES*

KING'S HIGHWAYS *ROUTE PRINCIPALE*

Multi-lane divided,(Interchange access only)
À voies multiples séparées,
(Accès par échangeur seulement) 401

Multi-lane divided
À voies multiples séparées ....Under construction
En construction

Multi-lane undivided ....
À voies multiples non-séparées ○ ⊖ ....Interchange(Full,Partial)
Échangeur(complet,partiel)

Two-lane,hard surface........................
Deux voies, pavées 123 ....Exit Number
Numéro de sortie

SECONDARY HIGHWAYS TER... ROADS

ADMINISTR

Regional Municipality/District/ County Boundary ....
Limites de municipalité régionale/ district/comté

Regional Municipality/District/ County Boundary(in water) ....
Limites de municipalité régionale/ district/comté(l'eau)

Corporate Boundary(City/Town/Village) .
Limites de municipalité (ville/village)

TRAVEL AND TO

Railway, passenger service..........
Service ferroviaire de passager

Ferry Route ....
Traversier

Airports(Major,Secondary,Local) ....
Aéroport(principal,secondaire,local)

kilometric distance ....
Distances en kilomètres

Trans-Canada Highway..........
Route transcanadienne

Carpool Parking ....
Parc de stationnement pour covoiturage

Ontario Provincial Police Detachment ...
Détachement de la sûreté de l'Ontario

Hospital,First Aid Station,Nursing Station
Hôpital,Poste de premier soins,
Centre de soins

Service Centre
Centre de services

Border Crossing..........

DRAFT

CANADA ENGINEERING SERVICES INC.

CONSULTING ENGINEERS

Ministry of Transportation of Ontario
Pavements and foundations Section
Room 315, Central Building
1201 Wilson Avenue
Downsview, Ontario
M3M 1J8

August 12, 1996

Attention: Mr. Dave Dundas, P. Eng.
Senior Foundation Engineer

FOUNDATION INVESTIGATION AND DESIGN REPORT
FOR
CLARE RIVER BRIDGE REHABILITATION
WP 48-94-01, SITE 17-16
HIGHWAY 41, DISTRICT 41, KINGSTON

CANADA ENGINEERING SERVICES INC.
28 Landseer Road
Scarborough, Ontario
M1K 3A7
416 757 2211

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Grain Size Analysis & Plasticity Results..Fig 1,2 & 3

Plan of Site and Borehole Locations...Dwg No.96-309-1

FOUNDATION INVESTIGATION AND DESIGN REPORT
FOR
CLARE RIVER BRIDGE REHABILITATION
WP 48-94-01, SITE 17-16
HIGHWAY 41, DISTRICT 41, KINGSTON

INTRODUCTION

Canada Engineering Services Inc., has been retained by the Foundation Design Section of the Ministry of Transportation of Ontario to carry out a foundation investigation for the Clare River Bridge Rehabilitation located across highway 41 in the Kingston Area. The project required a field investigation and design calculation to determine whether it is feasible to build a stronger and heavier deck for the bridge.

SITE DESCRIPTION

The site is the Clare River Bridge across Highway 41, north of Napanee and south of McGuire Settlement. The abutments of the bridge have been constructed on timer cribbing and rock fill. The approach embankments were also constructed on rock fill. The elevations of both approaches and the existing bridge are approximately the same. The side slopes of the embankments appear to be about 1 vertical to 4 horizontal. The bridge crosses the Clare River at a fairly narrow part of the river.

PROCEDURE

The subsurface investigation consisted of drilling 2 boreholes, one alongside each abutment. The boreholes were advanced by wash boring techniques using a truck mounted C.M.E. 55 drill rig. Visual inspection of the bridge abutments, its approaches, embankments and abutments were carried out concurrent with the drilling operations which were done on June 3 and 4, 1996.

The locations and elevations of the boreholes were established by us in the field and then the coordinates of the boreholes were established by MTO staff and provided to us. The borehole locations were plotted from these coordinates. The boreholes were put down at the locations shown on drawing number 309-96-1.

The bridge deck was cored through at borehole number 1 in front of the north abutment and the hole through the boulder fill below was advanced by wash boring with NW casing. An attempt made to obtain undisturbed shelby tube soil samples in the soft organic seam was unsuccessful and subsequently disturbed samples were obtained by means of a split-spoon sampler, in accordance with the requirements of the Standard Penetration Test, (CSA test specifications A119.1). Bedrock was encountered at a depth of 8.8 m and was cored through to 11.0 m where the borehole was terminated.

Per our original plan, borehole 2 was intended to be placed in front of the south abutment, but after the completion of borehole number 1, it was thought prudent to locate borehole 2 behind this abutment so as to obtain a better representation of the soils profile. This was done after consultation with MTO's soils engineer. Borehole 2 was advanced by auger through the first 1 m and then by NW casing through the boulder layer. Field vane tests were carried out as soon as the organic silt was encountered and this was followed by a split spoon sampler to recover sample penetrated by the vane. A shelby tube sample was taken immediately after but was unfortunately found to be disturbed upon extraction. Split spoon samples were subsequently taken to refusal. After refusal an NQ core barrel was used to core through what appeared to be either boulders or weathered bedrock. The borehole had to be terminated at a depth of 14.7 m after the casing and bit ceased below and had to be sealed in the borehole.

The samples recovered were logged in the field and they were subsequently brought to our laboratory where further visual examination, and classification were carried out. Moisture content tests, grain size analyses and Atterberg Limit and unit weight tests were carried out on selected samples. The results of these tests are shown on the borehole logs numbers 1 and 2 and on figures 1, 2 and 3. The location and elevation of the boreholes are shown on drawing number 96-309-1. A soil profile through borehole numbers 1 and 2 are also shown on this drawing.

DESCRIPTION OF SUBSURFACE CONDITIONS

Borehole number 1 was advanced through the concrete deck of the bridge. This was followed by free space down to a depth of 2.65 m, where the water level of the river was located. Below the water level the river surface was found at 3.32 m which consisted of boulder fill. The boulder fill was found to extend down to a depth of 4.9 m. Below the boulder fill was a layer of soft organic silt which extended down to a depth of 6.4 m. This layer was sandy, clayey, of high plasticity and was dark grey in colour and wet. Below this was a layer of silty sand with some clay. It was non plastic, grey stiff and wet and extended down to a depth of 7.0 m. This layer in turn was underlain by a layer of fine to medium sand, with occasional boulders, grey in colour, loose to dense and wet. This sand layer extended down to a depth of 8.8 m where a granitic gneiss bedrock was found. The bedrock was cored down to a depth of 11.0 m where the borehole was terminated. The bedrock was sound with an RQD of 95 %.

Borehole 2 was advanced through the upper 1 m of asphalt and gravel surface by a solid stem auger. Below this granitic gneiss boulder fill was encountered and the borehole was advanced by means of coring and wash boring with NW casing. At a depth of 3.35 m, timber cuttings were obtained from the wash water together with rock cuttings. This continued down to 6.86 m and thereafter only boulders were found down to a depth of 7.47 m. The timer cuttings were very likely the result of intercepting the timber cribbing and the depths that they were found are fairly consistent with the depths of the cribbing shown on the design drawings. The organic silt found in borehole 1 was also found below the boulder layer but at a depth of 7.47 m. Field vane tests were taken in this organic silt and a shear strength of 25 kPa was obtained with a sensitivity of 1.3 to 2.8.

The organic silt extended down to a depth of 8.6 m and was underlain by a layer of sandy silt with a trace of clay. This layer was non plastic, grey in colour, loose to compact and wet and it extended down to a depth of 9.75 m, and appears to be a relatively young river deposit. Below this sandy silt was a layer of fine to medium sand, with some silt and boulders and a trace of clay. It was grey, loose to compact, wet and extended down to 13.00 m below ground surface. Below this was a layer of boulders or possibly weathered bedrock. Recovery from coring 1.7 m was 500 mm. The borehole was terminated at 14.7 m. Detail soil profiles are shown on the borehole logs and grain size analyses are shown on figures 1 and 2. Plasticity index properties for the organic silt is shown on figure number 3.

SOIL PROPERTIES

The soil of primary concern at this site is the organic silt for which the following properties were noted:

BH# - SA#	Water Content (%)	Liquid Limit(%)	Plastic Limit(%)	Unit Weight(kN/cu. m)
1---1	103	88.0	23.5	---
2---1	94.7	90.0	23.0	14.3
2---2A	92.4	89.3	23.1	14.2
2---2B	75.1	-----	-----	-----

Field Vane value - 25 kPa with sensitivity of 1.8 to 2.3

The soils below this organic silt are non plastic and their natural water contents are as follows:

BH # - SA #	Natural Water Content(%)	Unit Weight(kN/cu. m)
1---2	34.6	-----
1---3	23.4	18.6
1---4	16.4	-----
2---3	31.4	-----
2---4	17.4	-----
2---5	18.9	-----

The natural water content and relative density of the non cohesive soils are not reliable because of the high water level and disturbance caused by water pressure both from the river and from the wash boring operations. It is very likely that the non cohesive sands and silts are denser than the SPT blow counts would indicate.

Liquid limit tests on the organic silt were carried on both oven dried and naturally dried samples. The oven dried liquid limits were approximately 60% which is less than 0.7 times those obtained from the naturally dried samples indicating that there is an organic content in the silt.

GROUNDWATER

As the boulder fill is permeable the ground water in both boreholes will essentially be the same as the river water level. The river water level was found at 2.6 m depth below the bridge deck surface.

ASSUMPTIONS AND ANALYTICAL PROCEDURE

The soil stratum of primary concern in terms of bearing capacity and settlement for additional loads to the bridge is the organic silt of high plasticity. This layer was found in both boreholes but was found to be relatively thin being only 1.5 m thick. The high moisture content of this soil would indicate its susceptibility to settlement. As we were unable to determine laboratory values of compression index of this organic silt, we estimated it on the basis of the moisture contents of different samples, the stress that these samples were subjected to and from estimated settlement of the bridge over the years. While these figures are not precise, they do provide a satisfactory indication of the settlement to expect from additional loads.

Basic assumptions made were that the bridge centre line surface was constructed at elevation 170.73 m as indicated on the design drawings of 1937 that were provided to us, and that the organic silt is normally consolidated. From the borehole elevations and making some allowance for a drop due to cambering we calculated a settlement of the bridge of 250 mm since 1937.

LOADS USED FOR CALCULATION

The bridge loads and abutment loads were determined from the size and material used to build the bridge. These loads together with the ramp boulder fill, were used to compute the stress transmitted in the organic silt below.

SETTLEMENT

Stresses at point locations were determined by the formulae by Davis & Poulos by the method of superimposition. It was assumed that nearly all settlement were within the organic silt layer. Stresses at the centre and top of the organic silt layer were determined at both borehole locations. The water contents of the samples obtained at these locations were then used to estimate the values of void ratios assuming a normally consolidated silt. The difference in void ratios were equated to the difference in stresses between the two boreholes. From this value the compression index of the organic silt was estimated at approximately 1. This compression index was also determined at borehole number one from the calculated settlement of the bridge as per the original design drawings from 1937. These two ways of determining the compression index produced a value of close agreement indicating that their is some validity in these estimates.

5...

Additional settlement that would result from a 20 % increase in the abutment and bridge was calculated as 10 mm. If there is also an increase in height of 300 mm of the approach fill, the settlement is expected to be 15 mm. This is also the maximum differential settlement that may occur if per chance the organic silt does not extend unto one of the abutment as would appear from the profile drawing enclosed(No. 96-309-1).

BEARING CAPACITY

The stresses generated in the organic silt layer were calculated at both borehole locations by stress distribution formulae by Davis and Poulos and also by assuming that the approach fill embankments were sloping at 2H to : 1 horizontal to vertical. The vertical stresses in the organic silt from this method were 46 kPa and 50 kPa at the front and rear walls of the abutment respectively.

The stresses were also calculated on the basis that the load from the abutment load was distributed by the boulder fill over a 1H : 2V slope and that the weight of the fill itself was distributed along slopes of 2H : 1V. A uniform stress distribution of 30 kPa was obtained at the top of the organic silt layer using this method.

The most likely failure zone would be failure of the organic silt in front of the abutment where the stress in the clay was calculated to be 46 kPa. The bearing capacity was calculated along the outer face of the abutment based on the vane test result of 25 kPa shear strength. The following formula was used

$$r_u = 5 \text{ x } \tau \mu F_c (1 + 0.2 D/B)(1 + 0.2 B/L) + {}^{\sigma' H}$$

where r_u = ultimate bearing capacity
τ = vane shear strength
μ = vane factor
F_c =Ontario Highway Bridge Code Factor
D= depth of foundation
B = width of foundation
L = length of foundation

$$r_u = 5 \text{ x } 25 \text{ x } 0.7\text{x } 0.5 (1 + 0.2 \text{ x } 1.5/8.5)(1 + 0.2 \text{ x} 8.5/16.5) + (21-10)\text{x}4.5$$
$$= 50 + 49.5 = 99.5$$

Using the stress determined in the organic silt at this depth the Factor of Safety for bearing capacity is given by

r_u /stress in bearing soil
99.5/46 = 2.2

The side slopes at this site are much flatter that the values used to calculate the stresses above. Also the boulder fill is somewhat rigid and would likely distribute the load from the abutment over a wider area on the organic silt. This means that the calculated factor of safety is a conservative value.

The increase in stress on the organic silt below by increasing the load of the abutment and bridge by 20 % is only 6 kPa resulting in a FS of 99.5/52 = 1.9.

It is obvious that the minimum FS of 3 specified by the Ontario Bridge Design Code is not available by this method. However, the bridge has been built about 60 years ago and even though a settlement of 250 mm is indicated if we are to assume that it was built according to the design drawings, then it has certainly settled fairly uniformly, as there are no visual indications of differential settlements and the face of both abutments still appear to be vertical. Probable the true load on the organic silt in front of the abutment, is somewhere between the average value calculated of 30 kPa and 46 kPa as the boulder fill is semi-rigid. The would mean that FS calculated above is conservative.

SLIDING AND OVERTURNING

Failure of the abutment against sliding was calculated to be over 3 and the eccentricity of the abutment reaction was within 1/12 of the centre of its base. This means the fill height behind the abutment can be raised another 300 mm without fair of sliding failure or overturning.

DISCUSSION AND RECOMMENDATIONS

The existing bridge has been in place for over 60 years. No significant uneven settlement or any lateral movement of the abutments are noticeable. The existing factor of safety of the bridge from a bearing capacity point appears to be lower than that permitted by the Ontario Bridge Design Code. Significant settlement (250 mm) has occurred over the years if we are to assume that the bridge was built according to the design drawings. Settlement of the bridge and its approaches have apparently been uniform and no damage has resulted in the bridge from such settlement.

However, the soils found at the sides of the two abutments from the boreholes drilled indicate that only one of the abutments sits on the soft organic silt, as shown on the profile of the boreholes extended under the abutments. If this is the case then it is unlikely that settlement would have been uniform. Possibly the soil profile is incorrect, or it could be that during the placement of the boulder fill, the fill in the cribbing punctured and displaced the organic silt layer and that both abutments are seated on the sandy silt and silty sand below or very close to it. This would mean that the bridge was not constructed according to the design drawings. If this is the cause, then the factors of safety given above would be much higher and settlement would have been much smaller.

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 1

METRIC

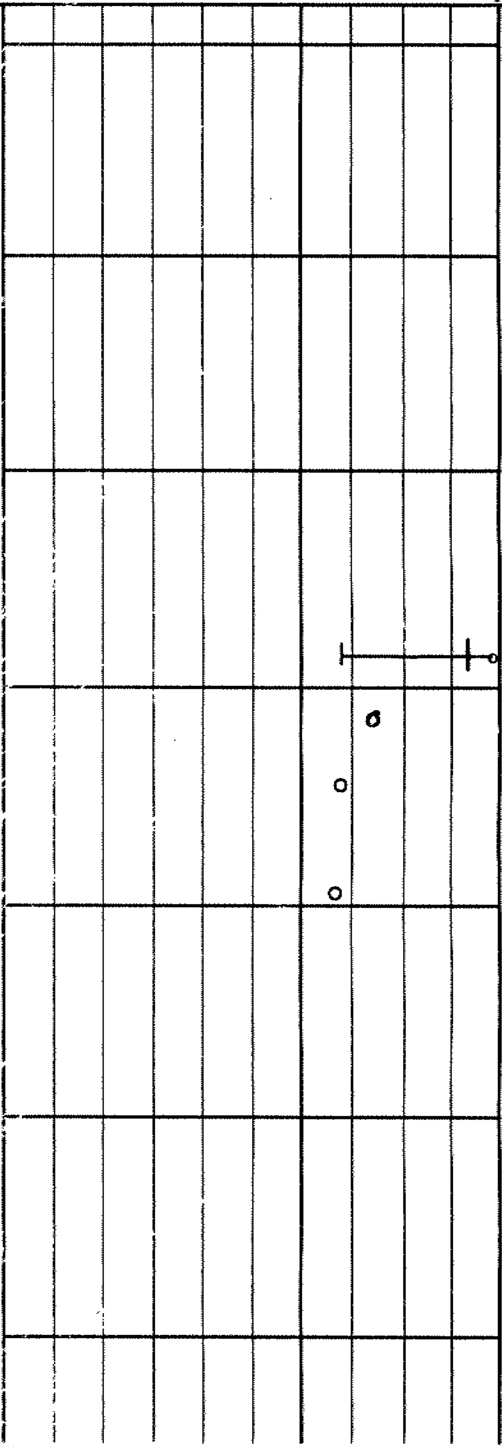
W P 48-94-01
LOCATION (Co-ords: N4,931,467.67, E 257,917.045
ORIGINATED BY R.J.
DIST. 41 HWY 41
BOREHOLE TYPE Wash Boring
COMPILED BY M.R.
DATUM Geodetic
DATE June 3/96
CHECKED BY R.J.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES 'N' VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
170.40	Bridge Deck Surface							
	ASPHALT & CONC. DECK							
0.25	Ground Surface					▽		
3..2	BOULDER FILL							
4.9	ORGANIC SILT - sandy, clayey, Of high plasticity, dark grey soft, wet		1	SS	1			
6.4	SILTY SAND- clayey, non-plastic, grey, loose, wet		2	SS	3		18.6	0 55 33 12
7.0	SAND- fine to medium, occasional boulders, grey, loose to dense, wet		3	SS	7			0 90 8 2
			4	SS	49			0 88 12 0
8.8	BEDROCK - Granite Gneiss, sound		5					
11.0	END OF BOREHOLE							

ELEVATION SCALE: 170, 168, 166, 164, 162, 160, 158

DYNAMIC CONE PENETRATION RESISTANCE PLOT
20 40 60 80 100
SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L
WATER CONTENT (%) 26 52 78



Note: liquid limit obtained from naturally dried sample; $+^3, X^3$: Numbers refer to Sensitivity

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 2

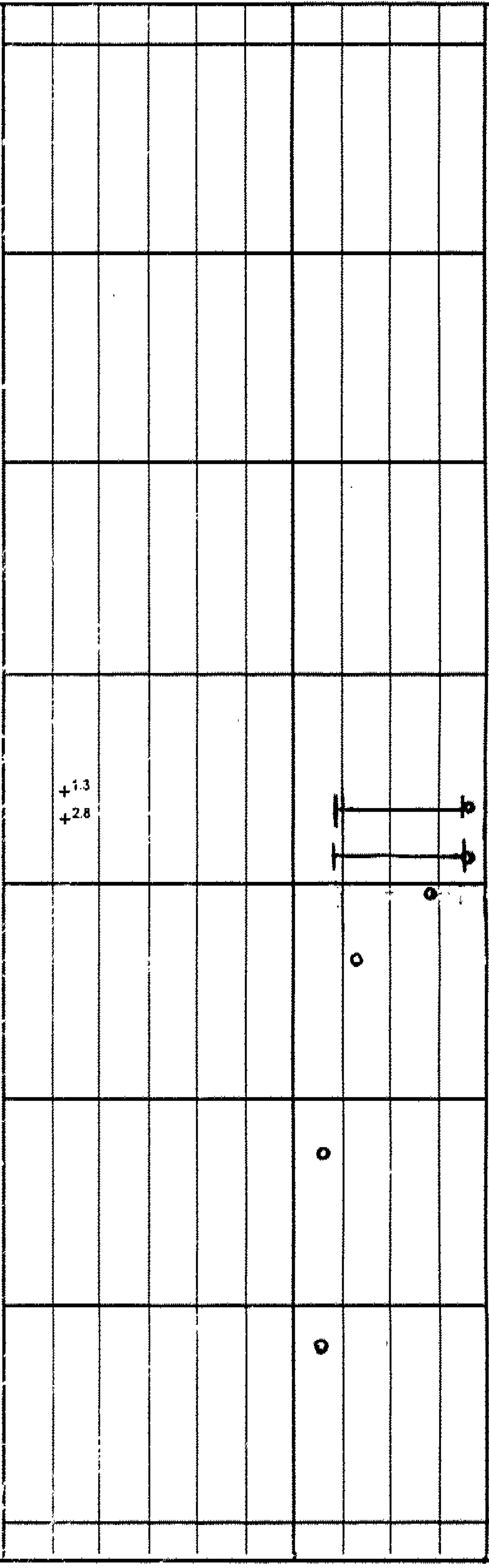
METRIC

W P 48-94-01 LOCATION Co-ords: N4,931,464.60, E 257,953.39 ORIGINATED BY R.J.
DIST 41 HWY 41 BOREHOLE TYPE Wash Boring COMPILED BY M.R.
DATUM Geodetic DATE June 4/96 CHECKED BY R.J.

SOIL PROFILE			SAMPLES						REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ	GR	SA	SI	CL
170.38	Ground Surface											
	ASPHAT & GRANULAR ROAD BASE						170					
1.0	**BOULDER FILL**					▽	168					
3..35	Pieces of wood cuttings and boulders Possible timber cribbing with Boulder fill - between 3.35 m and 6.86 m						166 164					
6.86	-End of timber cuttings END OF BOULDERS											
7.47	ORGANIC SILT - sandy, clayey, Of high plasticity, dark grey soft, wet		1	SS	2			14.2	0	48	35	17
									0	48	40	12
			2	ST	6		162	14.3	0	39	51	10
8.60	SANDY SILT, trace clay, non-plastic, grey, loose to compact wet		3	SS	9				0	13	82	5
9.75	SANDfine to medium, some silt, trace clay -with boulders, grey, loose to compact, wet						160					
			4	SS	9				0	77	15	8
							158					
			5	SS	14				3	87	8	2
13.00	BOULDERS OR WEATHERED ROCK											
14.7	END OF BOREHOLE						156					

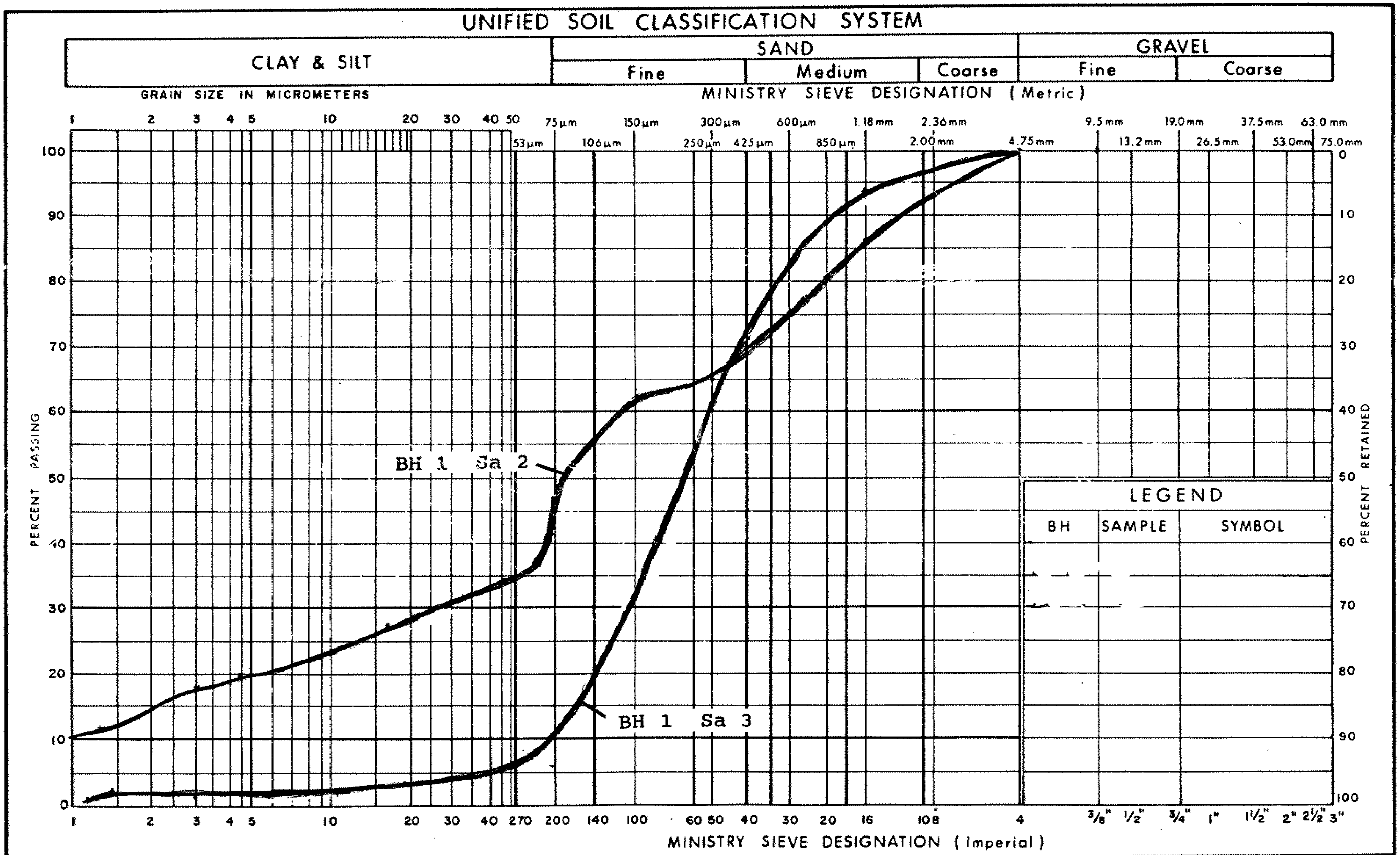
DYNAMIC CONE PENETRATION RESISTANCE PLOT
20 40 60 80 100
SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT Wp NATURAL MOISTURE CONTENT W LIQUID LIMIT WL
WATER CONTENT (%)
26 52 78



Note: liquid limit obtained from naturally dried sample; +3. ×3. Numbers refer to Sensitivity

78 12 M



Ministry of
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Ontario

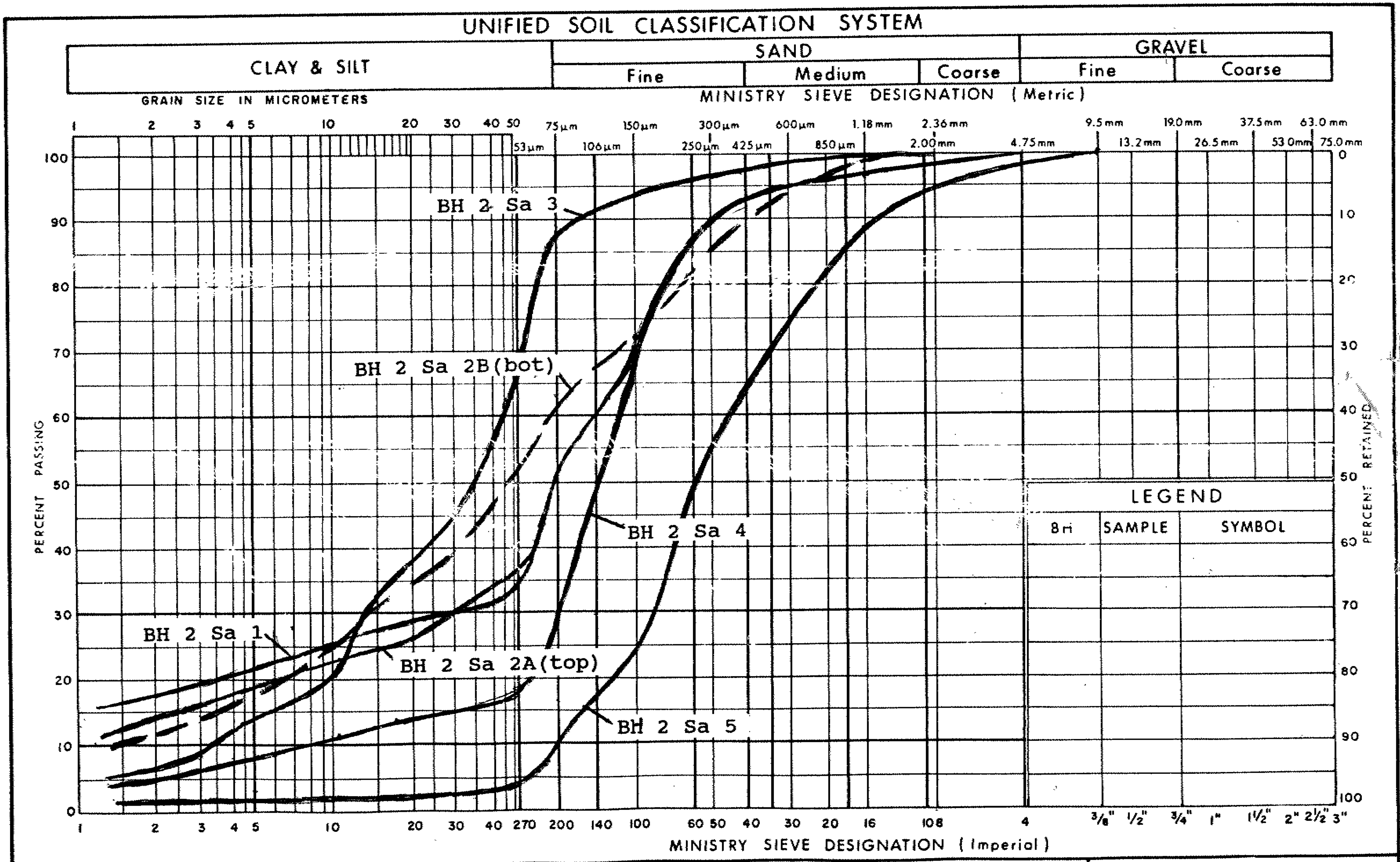
GRAIN SIZE DISTRIBUTION

SILTY SAND TO SAND

FIG No 1

W P 48 - 94 - 01

78 12 M

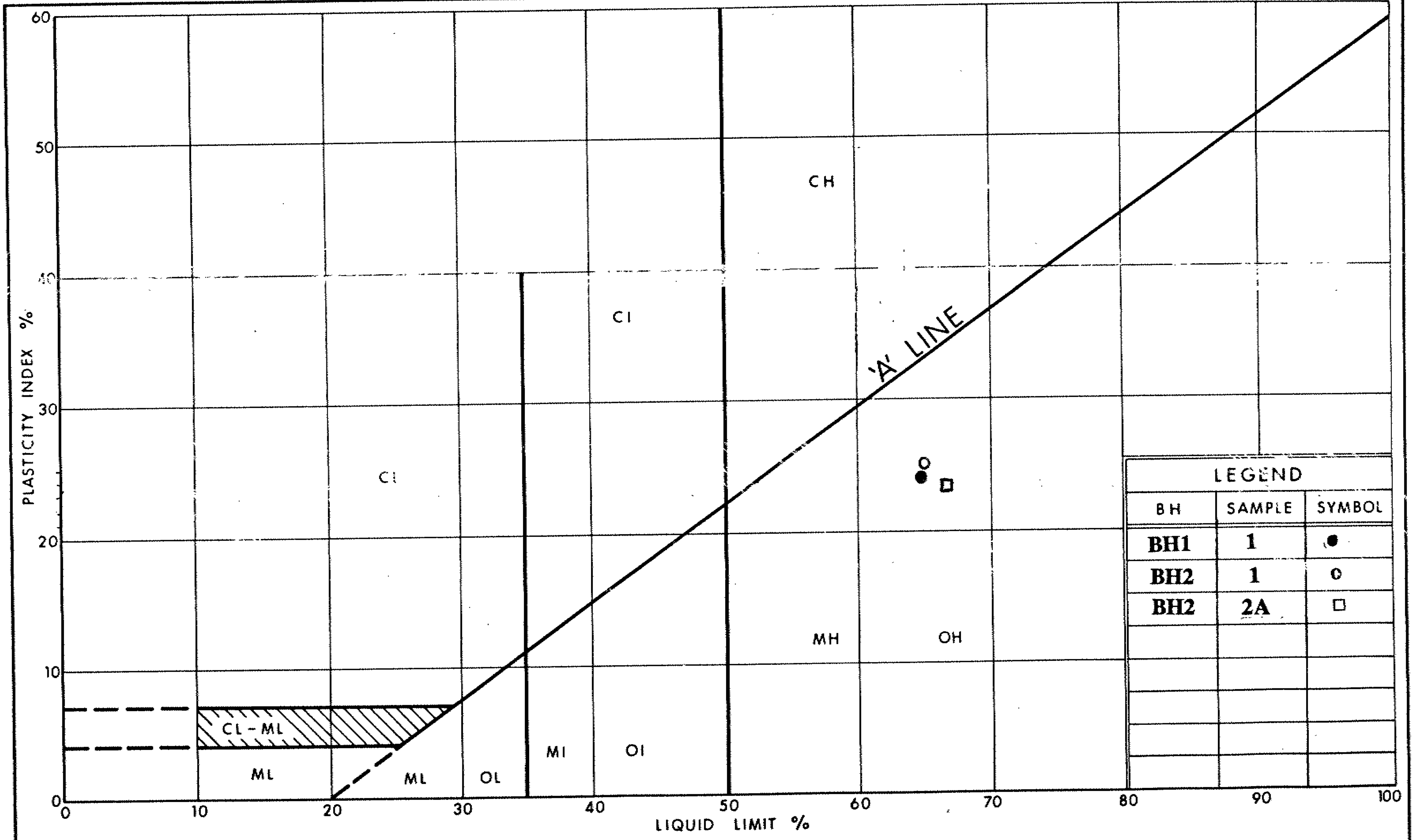


Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION
ORGANIC SILT TO SAND

FIG No 2
W P 48 - 94 - 01

Oct 75, FF-S-21



Ministry of Transportation
Ontario

PLASTICITY CHART

ORGANIC SILT

sandy, clayey

FIG No 3

W P 48 - 94 - 01

Ministry of Transportation of Ontario
Pavements and foundations Section
Room 315, Central Building
1201 Wilson Avenue
Downsview, Ontario
M3M 1J8

September 12, 1996

Attention: Mr. Dave Dundas, P. Eng.
Senior Foundation Engineer

FOUNDATION INVESTIGATION AND DESIGN REPORT FOR CLARE RIVER BRIDGE REHABILITATION
WP 48-94-01, SITE 17-16
HIGHWAY 41, DISTRICT 41, KINGSTON

GEOCRES No 31C-154

CANADA ENGINEERING SERVICES INC.
28 Landseer Road
Scarborough, Ontario
M1K 3A7
416 757 2211

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Procedure 1

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Grain Size Analysis & Plasticity Results..Fig 1,2 & 3

Plan of Site and Borehole Locations...Dwg No.96-309-1

FOUNDATION INVESTIGATION AND DESIGN REPORT
FOR
CLARE RIVER BRIDGE REHABILITATION
WP 48-94-01, SITE 17-16
HIGHWAY 41, DISTRICT 41, KINGSTON

INTRODUCTION

Canada Engineering Services Inc., has been retained by the Foundation Design Section of the Ministry of Transportation of Ontario to carry out a foundation investigation for the Clare River Bridge Rehabilitation located across highway 41 in the Kingston Area. The project required conducting a geological investigation of the soils at the site and carrying out a visual survey of the existing structure and its approach embankments to plan construction of foundation elements and the immediate approach embankments.

SITE DESCRIPTION

The site is the Clare River Bridge across Highway 41, 32.7 km north of highway 401 and south of McGuire Settlement. The existing structure was built in 1938, replacing a single lane structure. At that time the existing abutments were widened on both sides to accommodate two lanes of traffic. The original abutments were founded on timber cribbing while the extensions were placed on rock fill. The approach embankments were also constructed on rock fill. The elevations of both approaches and the existing bridge are approximately the same. The side slopes of the embankments appear to be from 1:2 to 1:4 vertical to horizontal. The bridge crosses the Clare River at a fairly narrow part of the river.

PROCEDURE

The subsurface investigation consisted of drilling 2 boreholes, one alongside each abutment. The boreholes were advanced by wash boring techniques using a truck mounted C.M.E. 55 drill rig. Visual inspection of the bridge abutments, its approaches, embankments and abutments were carried out concurrent with the drilling operations which were done on June 3 and 4, 1996.

The locations and elevations of the boreholes were established by us in the field and then the coordinates of the boreholes were established by MTO staff and provided to us. The borehole locations were plotted from these coordinates. The boreholes were put down at the locations shown on drawing number 309-96-1 attached.

The bridge deck was cored through at borehole number 1 in front of the north abutment and the borehole through the rock fill below was advanced by wash boring with NW casing. An attempt made to obtain undisturbed shelby tube soil samples in the soft organic silt seam was unsuccessful and subsequently disturbed samples were obtained by means of a split-spoon sampler, in accordance with the requirements of the Standard Penetration Test, (CSA test specifications A119.1). Split spoon samples were also taken in the Silt and Sand layers found below. Bedrock was encountered at a depth of 8.8 m and was cored through to 11.0 m where the borehole was terminated.

Originally borehole 2 was intended to be placed in front of the south abutment, but after reviewing the data obtained from borehole number one, it was decided to locate borehole 2 behind this abutment so as to obtain a better representation of the soils profile. Borehole 2 was advanced by auger through the first 1 m and then by NW casing through the rock fill layer. Field vane tests were carried out as soon as the organic silt was encountered and this was followed by a split spoon sampler to recover the sample penetrated by the vane. A shelby tube sample was taken immediately after but was unfortunately found to be disturbed upon extraction. Split spoon samples were subsequently taken to refusal. After refusal an NQ core barrel was used to core through what appeared to be either boulders or weathered bedrock. The borehole had to be terminated at a depth of 14.7 m after the casing and bit ceased below and had to be sealed in the borehole.

The samples recovered were logged in the field and they were subsequently brought to our laboratory where further visual examination, and classification were carried out. Moisture content tests, grain size analyses and Atterberg Limit and unit weight tests were carried out on selected samples. The results of these tests are shown on the borehole logs numbers 1 and 2 and on figures 1, 2 and 3. The location and elevation of the boreholes are shown on drawing number 96-309-1 attached. A soil profile through borehole numbers 1 and 2 are also shown on this drawing.

DESCRIPTION OF SUBSURFACE CONDITIONS

Borehole number 1 was advanced through the concrete deck of the bridge. This was followed by free space down to a depth of 2.65 m, where the water level of the river was located. Below the water level the river surface was found at 2.5 m which consisted of rock fill. The rock fill was found to extend down to a depth of 4.9 m. Below the rock fill was a layer of soft organic silt which extended down to a depth of 6.4 m. This layer was sandy, clayey, of high plasticity and was dark grey in colour and wet. Below this was a layer of silty sand with some clay. It was non plastic, grey stiff and wet and extended down to a depth of 7.0 m. This layer in turn was underlain by a layer of fine to medium sand, with occasional boulders, grey in colour, loose to dense and wet. This sand layer extended down to a depth of 8.8 m where a granitic gneiss bedrock was found. The bedrock was cored down to a depth of 11.0 m where the borehole was terminated. The bedrock was sound with an RQD of 95 %.

Borehole 2 was advanced through the upper 1 m of asphalt and gravel surficial fill by a solid stem auger. Below this, granitic gneiss rock fill was encountered and the borehole was advanced by means of coring as required and wash boring with NW casing. At a depth of 3.35 m, timber cuttings were obtained from the wash water together with wood cuttings. This continued down to 6.86 m and thereafter only boulders were found down to a depth of 7.47 m. The timber cuttings were very likely the result of intercepting the timber cribbing and the depths that they were found are fairly consistent with the depths of the cribbing shown on the design drawings. The organic silt found in borehole 1 was also found below the rock fill layer in borehole 2 but at a depth of 7.47 m. Field vane tests were taken in this organic silt and a shear strength of 25 kPa was obtained with a sensitivity of 1.3 to 2.8.

The organic silt extended down to a depth of 8.6 m and was underlain by a layer of sandy silt with a trace of clay. This layer was non plastic, grey in colour, loose to compact and wet and it extended down to a depth of 9.75 m. Below this sandy silt was a layer of fine to medium sand, with some silt and boulders and a trace of clay. It was grey, loose to compact, wet and extended down to 13.00 m below ground surface. Below this was a layer of boulders or possibly weathered bedrock. No recovery was obtained as the core barrel could not be retrieved. The borehole was terminated at 14.7 m. Detailed soil profiles are shown on the borehole logs and grain size analyses are shown on figures 1 and 2. Plasticity index properties for the organic silt are shown on figure number 3.

Asphalt, concrete and granular road base (Fill)

At borehole 1 there was a concrete deck 250 mm thick which appeared to be in sound condition, while at borehole 2 the surficial fill consisted of 150 mm asphalt cap over 850 mm granular base material. The granular base was in a compact state and extended down to a depth of 1.0 m or elevation 169.38 m in borehole number 2. The surface asphalt along both approaches were generally in good condition except adjacent to the north abutment where some signs of distress were observed.

Rock (Fill)

The rock fill consisted of various sizes. In borehole number 1, the rock fill was penetrated through rather easily by wash boring from 2.5 m to 4.9 m (elevations 167.9 m to 165.5 m) below the bridge deck surface. However in borehole 2 the rocks were of larger sizes, some other 600 mm in diameter and had to be cored through. Coring through the rocks were difficult and slow. The cored pieces recovered from borehole number 2 were granitic gneiss. The rock fill in borehole 2 was found between 1.0 m and 7.47 m depth below ground surface (between elevations 169.38 m and 162.91 m).

Organic Silt

The soil of primary concern at this site is the organic silt with a high sand content and some clay. The natural moisture contents of this stratum exceeded its liquid limits. From the design drawings this is the founding soil at the site. However, from the data obtained from borehole number 1, this soil was just above the timber cribbing between 4.9 m and 6.4 m below the bridge deck (between elevations 165.5 m and 164.0 m), while at borehole number 2 it was found at the base of the cribbing, between depths of 7.47 m and 8.6 m from ground surface (between elevations 162.91 m and 161.78 m). This soil was soft enough to permit the shelby tube to be hand pushed in both boreholes. Upon retrieval the shelby tube sample was lost from borehole number 1 and had to be retrieved by a split spoon while the sample recovered from borehole number 2 was later found to be disturbed upon extraction.

The following properties of this organic silt were determined in the laboratory:

BH# -	SA#	Water Content (%)	Liquid Limit(%)	Plastic Limit(%)	Unit Weight(kN/cu. m)
1	1	103	88.0	23.5	---
2	1	94.7	90.0	23.0	14.3
2	2A	92.4	89.3	23.1	14.2
2	2B	75.1	-----	-----	-----

Liquid limit tests on the organic silt were carried on both oven dried and naturally dried samples. The oven dried liquid limits were approximately 60% lower than those obtained from the naturally dried samples which indicates that there is an organic content in the silt.

Two vane tests were carried out in borehole number 2 between depths of 7.5 m and 8.0 m (elevations 162.88 m and 162.38 m) and both tests gave a shear strength of 25 kPa with sensitivities of 1.8 to 2.3.

Silty Sand

The silty sand was found to be the founding soil for borehole number 1 for the north abutment. It was clayey, non plastic, loose and was only 600 mm thick with a natural water content of 34.6 percent, unit weight of 18.6 kN/cu. m and an "N" value of 3. It was found at a depth of 6.4 m to 7.0 m below the bridge deck surface (elevation 164.0 m to 163.4 m).

Sandy Silt

The sandy silt layer was found immediately below the organic silt in borehole number 2 at a depth of 8.6 m (elevation 161.78 m) below ground surface and extended down to a depth of 9.75 m (elevation 160.63 m) Its moisture content was 31.4 percent and its "N" value was 9.

Sand

Sand was found in both borehole numbers 1 and 2 at depths of 7.0 m and 9.75 m respectively (elevations 163.4 m and 160.63 m) and extended down to depths of 8.8 m and 13.0 m (elevations 161.60 m to 157.38 m). This sand layer was fine to medium, with boulders. The natural water contents ranged from 16.4 to 23.4 percent and the "N" values ranged from 7 to 49. However, these values were likely affected by the presence of boulders and water pressure from sampling below the water table.

Bedrock

Bedrock was found in borehole number 1 at a depth of 8.8 m (elevation 161.60 m) and was cored down to a depth of 11.0 m (elevation 159.40 m). The bedrock was sound and consisted of a granitic gneiss.

Boulders/Weathered Bedrock

What appeared to be either boulders or weathered bedrock were found in borehole number 2 at a depth of 13.00 m (elevation 157.38 m). This was cored down to a depth of 14.7 m (elevation 155.68 m) where the borehole was terminated due to excessively high resistance to coring. No cores were recovered as the casing and core barrel could not be retrieved from the borehole and were eventually sealed in after several hours were spent trying to withdraw them.

GROUNDWATER

As the rock fill is permeable the ground water in both boreholes will essentially be the same as the river water level. The river water level on June 3, 1996 was found at a depth below the bridge deck of 2.6 m elevation 167.8 m) and on September 10, 1996 at a depth of 1.7 m (elevation 168.7). High water level is recorded at elevation 169.01 m or a depth of 1.4 m below the bridge deck level.

DISCUSSION AND RECOMMENDATIONS

The existing bridge has been in place for over 60 years. Per the available drawings the bridge was founded on timber cribbings and rock fill. The width of the concrete bridge abutments is 3 m while the length is 12 m. The length of the bridge between the outer faces of the concrete abutments is 18.7 m. The bridge consists of two traffic lanes and a pedestrian side walk. The deck of the bridge consists of asphalted concrete over a concrete base supported by steel beams.

The current cribbing and rock fill are presumed to be founded on the organic silt at 163.2 m. Construction of a larger heavier deck and abutments for the existing bridge are being contemplated and it is required to be known whether the existing foundations are capable of supporting additional loads. The proposed new design loads are at this time unknown.

Structure Foundations

Based on the subsoil conditions found at the boreholes drilled, additional foundation loads are feasible at this site. The major concern is the performance of the organic silt layer found in both boreholes close to the base of the rock fill and timber cribbing, between elevations 165.5 m and 164.0 m at borehole number 1 and between elevations 162.91 m and 161.78 m at borehole number 2. However, this layer is only 1.1 m to 1.5 m thick and settlement is not necessarily the most critical factor. To substantiate this we noted that the bridge deck is currently 250 mm lower than its design elevation of 170.73 m as per the design drawings provided.

Settlement

Stresses at point locations were determined by the formulae by Davis & Poulos by the method of superimposition. It was assumed that nearly all settlement were within the organic silt layer. Stresses at the centre and top of the organic silt layer were determined at both borehole locations. The water contents of the samples obtained at these locations were then used to estimate the values of void ratios assuming a normally consolidated silt. The difference in void ratios were equated to the difference in stresses between the two boreholes. From this value the compression index of the organic silt was estimated at approximately 1. This compression index was also determined at borehole number one from the calculated settlement of the bridge as per the original design drawings from 1937. These two ways of determining the compression index produced a value of close agreement and since no samples were available for consolidation testing this value of compression index above was used to estimate settlements for an increase in load of 10 percent in the abutments and bridge deck. From these calculations bearing capacities were then determined for the serviceability limit states Type II condition.

Bearing Capacities

The stresses generated in the organic silt layer were calculated at both borehole locations by stress distribution formulae by Davis and Poulos and also by assuming that the approach fill embankments were sloping at 2 : 1 horizontal to vertical.

The stresses were also calculated on the basis that the load from the abutment load was distributed by the rock fill over a 1H : 2V slope and that the weight of the fill itself was distributed along slopes of 2H : 1V.

The stabilities of the slopes in front of the abutments together with the abutments were also checked for sliding failure. Based on these calculations and the performance of the structure over the last 60 years bearing capacity values were determined.

In accordance with Ontario Highway Bridge Design Code the following design values are recommended for the footings at the base of the timber cribbing and rock fill for both abutments:

Bearing Capacity at Servicibility Limit States Type II (Kpa)	Factored Bearing Capacity At Ultimate Limit States (Kpa)	Elevation (m)
60	100	163.5 - 162.0

For the rock fill where the concrete abutments are founded the recommended design values for purposes of the O.H.B.D.C., the following design values are recommended assuming that the bases will be the same sizes and at the same locations:

Bearing Capacity at Serviceability Limit States Type II (Kpa)	Factored Bearing Capacity At Ultimate Limit States (Kpa)	Elevation (m)
140	900	168 - 167.5

It is important to note that changing the locations or sizes of the existing abutment footings will alter the design values given above as different locations and sizes of the footings will have different bearing capacities which may be controlled by the organic silt layer .

Settlement caused by additional loads to the existing foundations will be time dependent because of the compressible organic silt. Total and differential settlements based on the above values are expected to be within the tolerable limits of construction of 25 mm and 19 mm respectively, provided the organic silt is not disturbed by any activities associated with construction.

From our calculations the values provided above represents a 10 percent increase of the loads of the bridge, i.e. abutments and deck. Therefore we recommend that the existing structure be rehabilitated providing that the additional loads be kept to a maximum increase of 10 % of the current loads from the existing bridge.

Lateral Resistance

Resistance to sliding of the abutments along the base of the rock fill can be calculated based on an angle of friction of 35° while resistance to sliding or slope failure within the organic silt may be calculated based on a shear strength of 11.4 kPa in accordance with the O.H.B.D.C. at U.L.S.

Lateral Earth Pressures

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

Angle of internal friction (degrees)	35
Unit Weight (KN/cu. m.)	22

For a yielding structure, the active condition (Ka) may be assumed to apply while for rigid and unyielding structures, the at rest condition (Ko) should apply.

Approach Embankments

Per the original design drawings the existing approach embankments are about 7 m high but only about 4 m above the original river bed surface. The current side slopes of the embankments vary from 1:2 to 1:3 being steepest closest to the abutment. The slopes in front of the abutments, though are only 1:4 vertical to horizontal. Neither abutments show any signs of rotational failure or of uneven settlement.

The approaches at their current elevations appeared to have settled about an inch with respect to the abutments and there is some signs of distress (longitudinal cracks) along the centre line of the approaches. Some alligator cracking were found just to the north of the north abutment indicating some type of subgrade failure. Possibly there is some lateral displacement of the organic silt occurring from the weight of the fill along the steeper 1:2 side slopes. It is recommended that these side slopes be flattened to 1.3 if heavier loads are used for the modified bridge and particularly if the approach height is raised. In any event it is not recommended that the approach height be raised higher than 300 mm above its current height at any point. Further if the approach heights are raised they should be preloaded for a period of three months prior to new construction.

MISCELLANEOUS

The field work for this investigation was carried out under the supervision of R. Jagdat. The equipment used was owned and operated by Marathon Drilling (1994).

The laboratory work and report for this project was performed by Mahendran Raja and it was supervised and reviewed by R. Jagdat, Principal Engineer.

Submitted by
CANADA ENGINEERING SERVICES INC.

Raja Mahendran, P.Eng.

R Jagdat
Ram Jagdat, P. Eng.

LICENSED PROFESSIONAL ENGINEER
R. M. MAHENDRAN
Sept 13/96
PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER
R. JAGDAT
Sept 13/96
PROVINCE OF ONTARIO

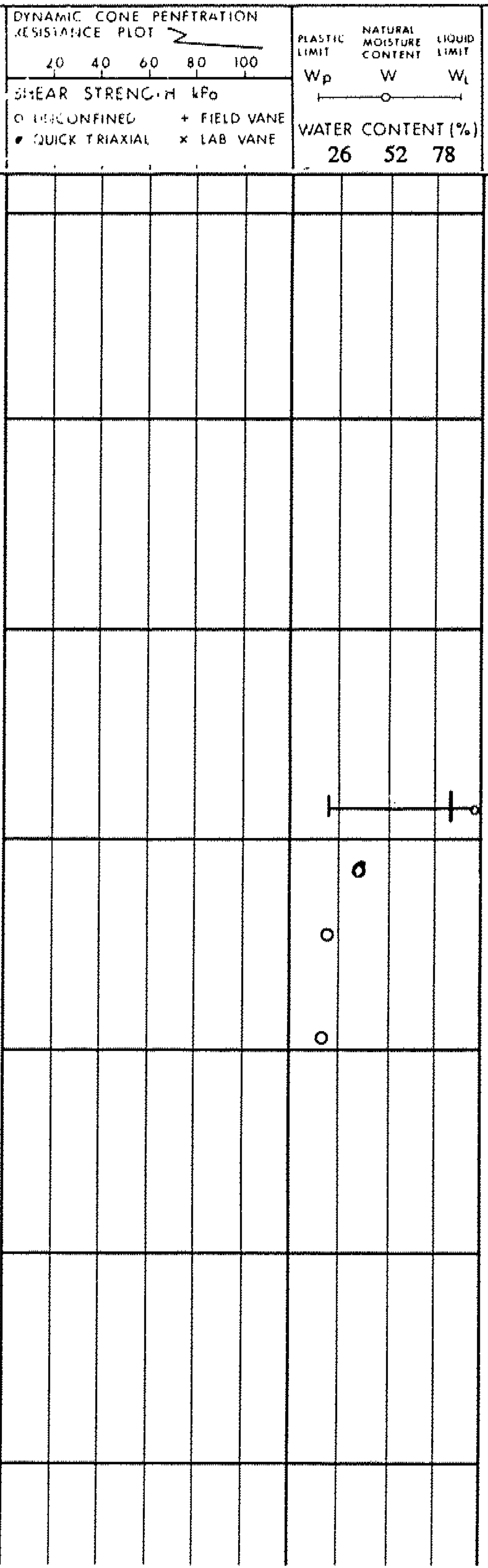
Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 1

METRIC

W P 48-94-01
LOCATION Co-ords: N4,931,467.67, E 257,917.045
ORIGINATED BY R.J.
DIST. 41 HWY 41
BOREHOLE TYPE Wash Boring
COMPILED BY M.R.
DATUM Geodetic
DATE June 3/96
CHECKED BY R.J.

SOIL PROFILE ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES 'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
170.40	Bridge Deck Surface								
	ASPHALT & CONC. DECK						170		
0.25	FREE SPACE ROCK FILL					▽	168 166		
4.9	ORGANIC SILT - sandy, clayey, Of high plasticity, dark grey soft, wet		1	SS	1		164		
6.4	SILTY SAND- clayey, non-plastic, grey, loose, wet		2	SS	3			18.6	0 55 33 12
7.0	SAND- fine to medium, occasional boulders, grey, loose to dense, wet		3	SS	7				0 90 8 2
			4	SS	49		162		0 88 12 0
8.8	BEDROCK - Granite Gneiss, sound		5				160		
11.0	END OF BOREHOLE						158		



Note: liquid limit obtained from naturally dried sample; $+^3, \times^3$: Numbers refer to Sensitivity

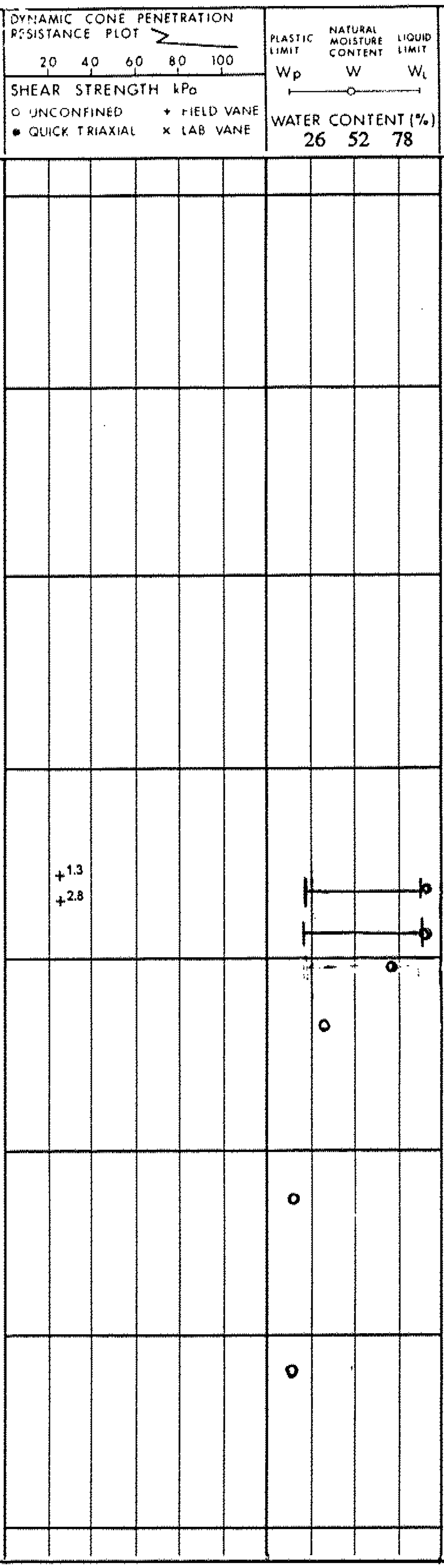
Ministry of Transportation Ontario

RECORD OF BOREHOLE No 2

METRIC

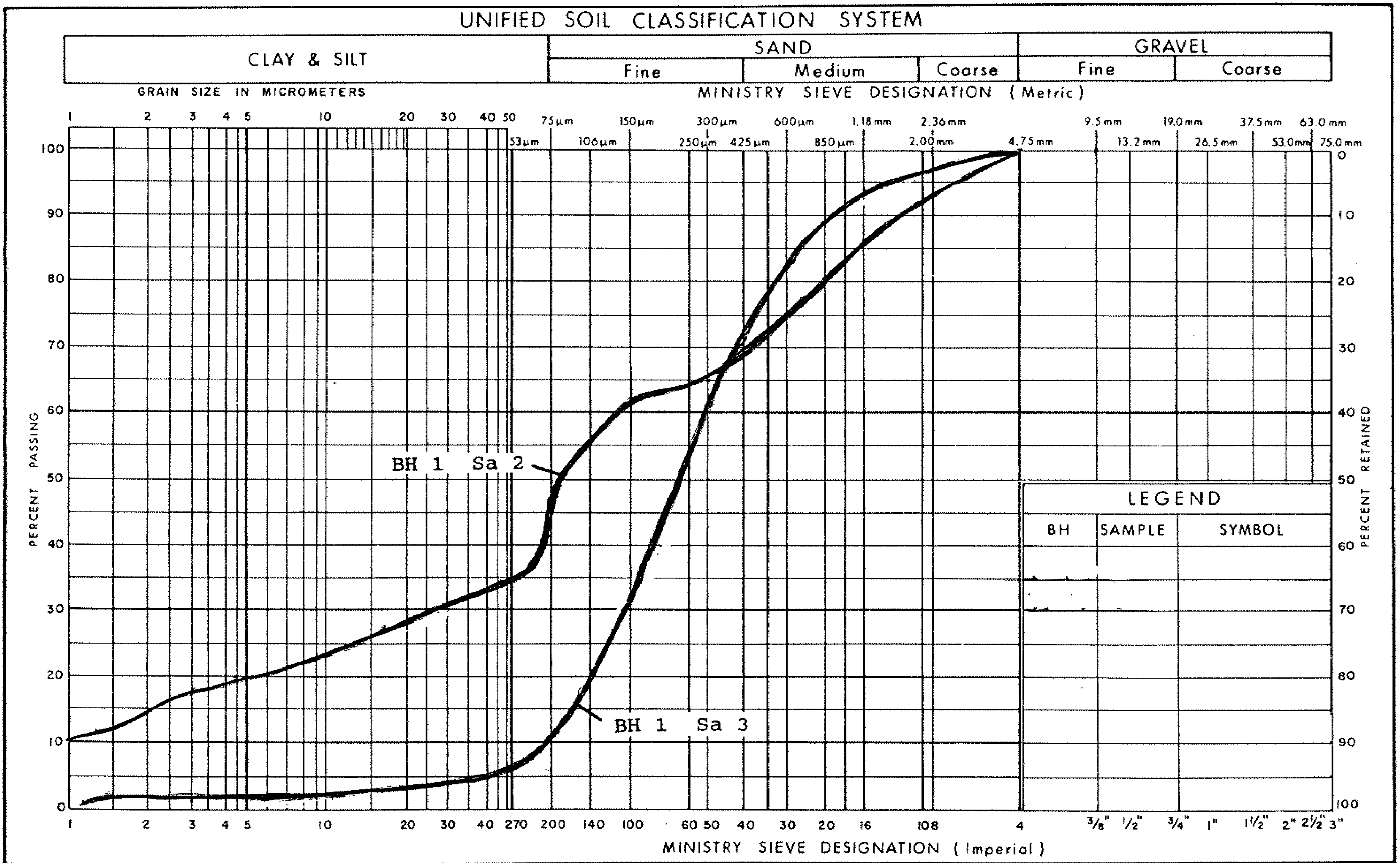
W P 48-94-01 LOCATION Co-ords: N4,931,464.60, E 257,953.39 ORIGINATED BY R.J.
DIST 41 HWY 41 BOREHOLE TYPE Wash Boring COMPILED BY M.R.
DATUM Geodetic DATE June 4/96 CHECKED BY R.J.

SOIL PROFILE ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
170.38	Ground Surface								
	ASPHAT & GRANULAR ROAD BASE						170		
1.0	ROCK FILL					▽	168		
3..35	Pieces of wood cuttings and boulders Possible timber cribbing with Boulder fill - between 3.35 m and 6.86 m						166 164		
6.86	-End of timber cuttings ROCK FILL								
7.47	ORGANIC SILT - sandy, clayey, Of high plasticity, dark grey soft, wet		1	SS	2			14.2	0 48 35 17
									0 48 40 12
			2	ST	6		162	14.3	0 39 51 10
8.60	SANDY SILT, trace clay, non-plastic, grey, loose to compact wet		3	SS	9				0 13 82 5
9.75	SANDfine to medium, some silt, trace clay -with boulders, grey, loose to compact, wet						160		
			4	SS	9				0 77 15 8
							158		
			5	SS	14				3 87 8 2
13.00	BOULDERS OR WEATHERED ROCK						156		
14.7	END OF BOREHOLE								



Note: liquid limit obtained from naturally dried sample; $+^3$, $\times^3$: Numbers refer to Sensitivity

78 12 M



Ministry of Transportation

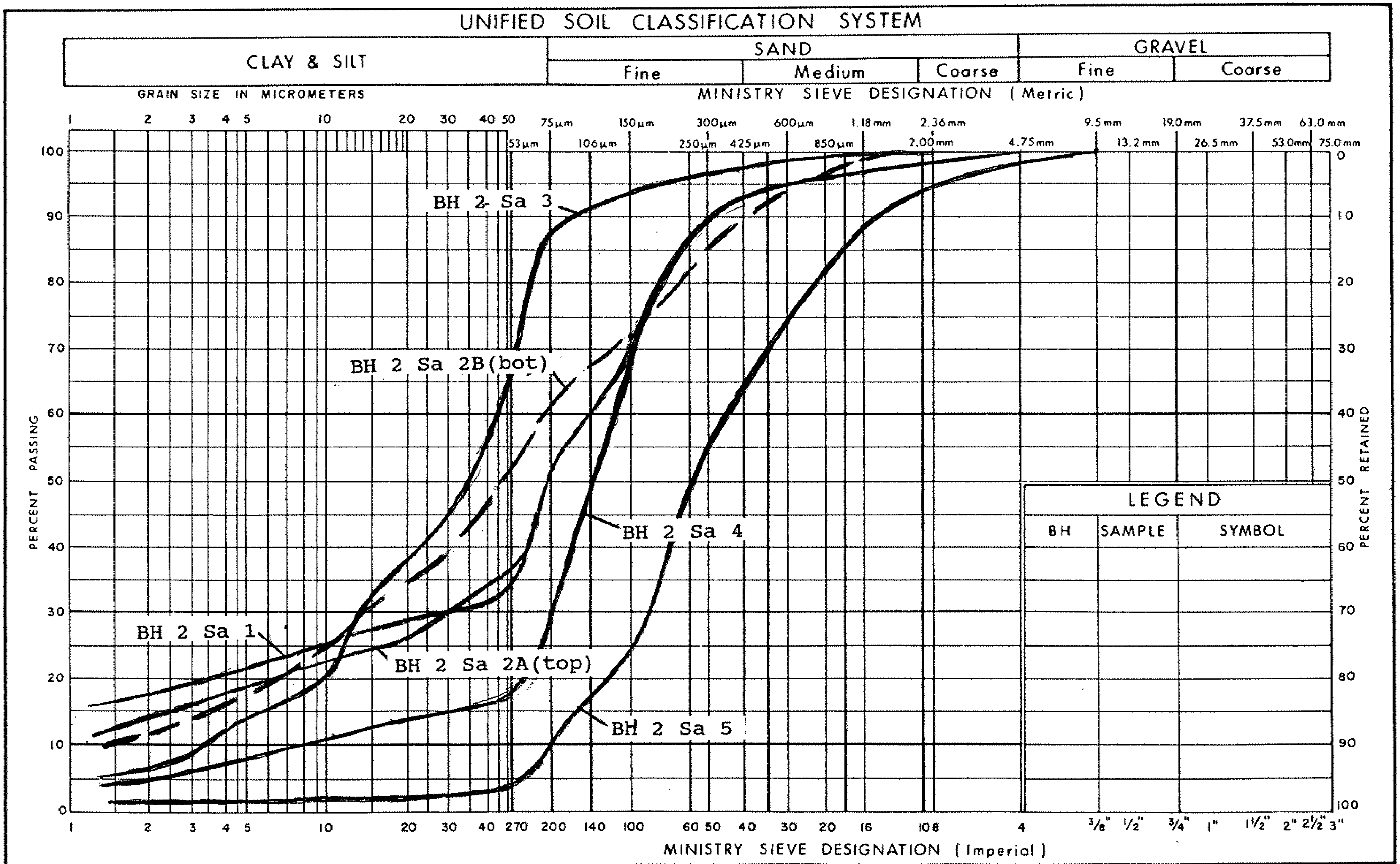
Ontario

GRAIN SIZE DISTRIBUTION

SILTY SAND TO SAND

FIG No	1
W P	48 - 94 - 01

78 12 M



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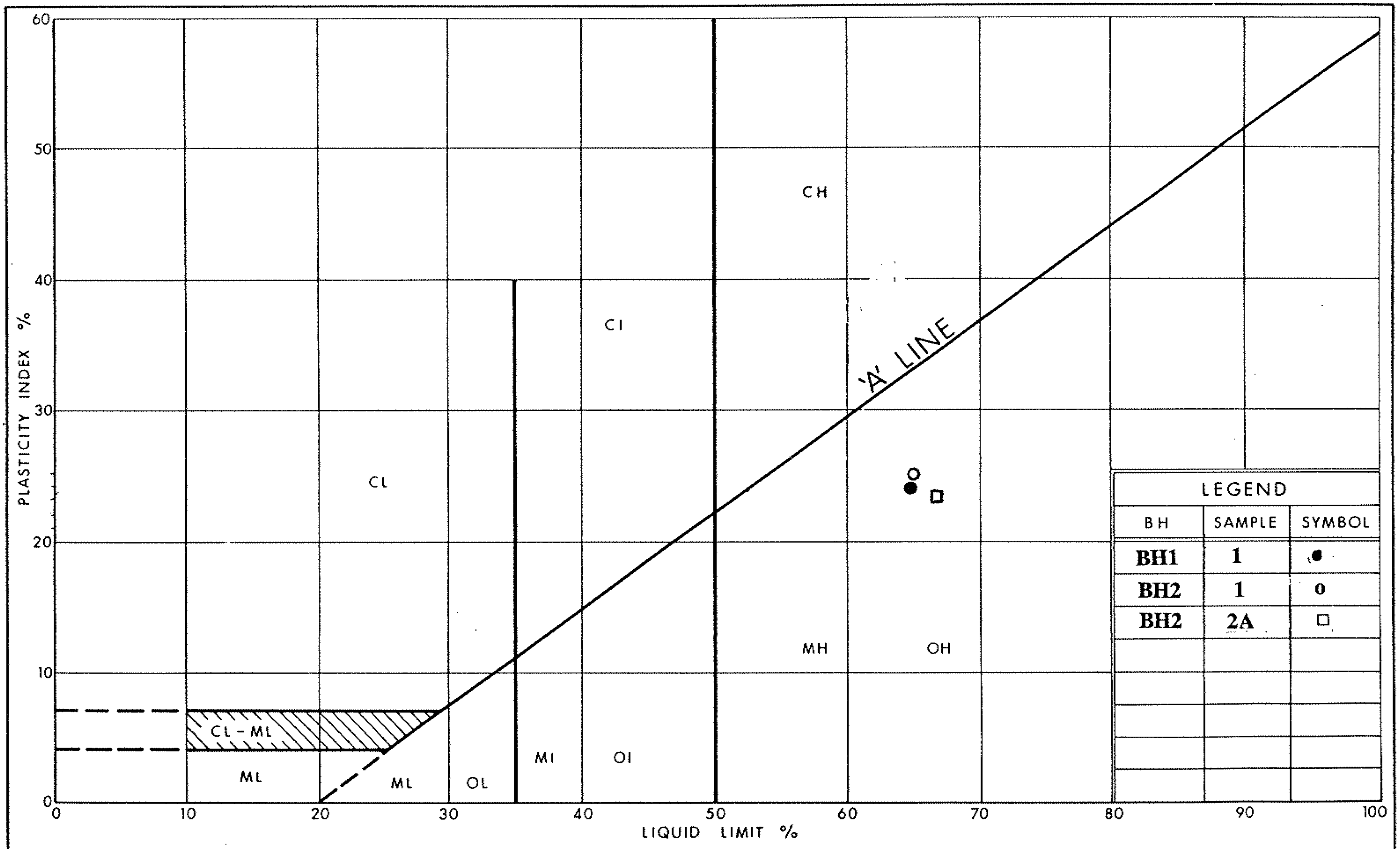
GRAIN SIZE DISTRIBUTION
ORGANIC SILT TO SAND

FIG No 2

W P 48 - 94 - 01

Oct 75, FF-S-21



BH	SAMPLE	SYMBOL
BH1	**1**	●
BH2	**1**	o
BH2	**2A**	□

Ministry of Transportation
Ontario

PLASTICITY CHART

ORGANIC SILT

sandy, clayey

FIG No **3**

W P **48 - 94 - 01**

DOCUMENT MICROFILMING IDENTIFICATION

G.I.-30 SEPT. 1976

GEOCRES No. 31C-154

DIST. 41 REGION

W.P. No. 48-94-01

CONT. No. 40-97

W. O. No.

STR. SITE No. 17-16

HWY. No. 41

LOCATION Clare River Bridge

No of PAGES - —

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

Ministry of Transportation of Ontario
Pavements and foundations Section
Room 315, Central Building
1201 Wilson Avenue
Downsview, Ontario
M3M 1J8

September 12, 1996

Attention: Mr. Dave Dundas, P. Eng.
Senior Foundation Engineer

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WP 48-94-01, SITE 17-16
HIGHWAY 41, DISTRICT 41, KINGSTON

GEOCRES No 31C-154

CANADA ENGINEERING SERVICES INC.
28 Landseer Road
Scarborough, Ontario
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416 757 2211

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Plan of Site and Borehole Locations...Dwg No.96-309-1

FOUNDATION INVESTIGATION AND DESIGN REPORT
FOR
CLARE RIVER BRIDGE REHABILITATION
WP 48-94-01, SITE 17-16
HIGHWAY 41, DISTRICT 41, KINGSTON

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The subsurface investigation consisted of drilling 2 boreholes, one alongside each abutment. The boreholes were advanced by wash boring techniques using a truck mounted C.M.E. 55 drill rig. Visual inspection of the bridge abutments, its approaches, embankments and abutments were carried out concurrent with the drilling operations which were done on June 3 and 4, 1996.

The locations and elevations of the boreholes were established by us in the field and then the coordinates of the boreholes were established by MTO staff and provided to us. The borehole locations were plotted from these coordinates. The boreholes were put down at the locations shown on drawing number 309-96-1 attached.

The bridge deck was cored through at borehole number 1 in front of the north abutment and the borehole through the rock fill below was advanced by wash boring with NW casing. An attempt made to obtain undisturbed shelby tube soil samples in the soft organic silt seam was unsuccessful and subsequently disturbed samples were obtained by means of a split-spoon sampler, in accordance with the requirements of the Standard Penetration Test, (CSA test specifications A119.1). Split spoon samples were also taken in the Silt and Sand layers found below. Bedrock was encountered at a depth of 8.8 m and was cored through to 11.0 m where the borehole was terminated.

Originally borehole 2 was intended to be placed in front of the south abutment, but after reviewing the data obtained from borehole number one, it was decided to locate borehole 2 behind this abutment so as to obtain a better representation of the soils profile. Borehole 2 was advanced by auger through the first 1 m and then by NW casing through the rock fill layer. Field vane tests were carried out as soon as the organic silt was encountered and this was followed by a split spoon sampler to recover the sample penetrated by the vane. A shelby tube sample was taken immediately after but was unfortunately found to be disturbed upon extraction. Split spoon samples were subsequently taken to refusal. After refusal an NQ core barrel was used to core through what appeared to be either boulders or weathered bedrock. The borehole had to be terminated at a depth of 14.7 m after the casing and bit ceased below and had to be sealed in the borehole.

The samples recovered were logged in the field and they were subsequently brought to our laboratory where further visual examination, and classification were carried out. Moisture content tests, grain size analyses and Atterberg Limit and unit weight tests were carried out on selected samples. The results of these tests are shown on the borehole logs numbers 1 and 2 and on figures 1, 2 and 3. The location and elevation of the boreholes are shown on drawing number 96-309-1 attached. A soil profile through borehole numbers 1 and 2 are also shown on this drawing.

DESCRIPTION OF SUBSURFACE CONDITIONS

Borehole number 1 was advanced through the concrete deck of the bridge. This was followed by free space down to a depth of 2.65 m, where the water level of the river was located. Below the water level the river surface was found at 2.5 m which consisted of rock fill. The rock fill was found to extend down to a depth of 4.9 m. Below the rock fill was a layer of soft organic silt which extended down to a depth of 6.4 m. This layer was sandy, clayey, of high plasticity and was dark grey in colour and wet. Below this was a layer of silty sand with some clay. It was non plastic, grey stiff and wet and extended down to a depth of 7.0 m. This layer in turn was underlain by a layer of fine to medium sand, with occasional boulders, grey in colour, loose to dense and wet. This sand layer extended down to a depth of 8.8 m where a granitic gneiss bedrock was found. The bedrock was cored down to a depth of 11.0 m where the borehole was terminated. The bedrock was sound with an RQD of 95 %.

Borehole 2 was advanced through the upper 1 m of asphalt and gravel surficial fill by a solid stem auger. Below this, granitic gneiss rock fill was encountered and the borehole was advanced by means of coring as required and wash boring with NW casing. At a depth of 3.35 m, timber cuttings were obtained from the wash water together with wood cuttings. This continued down to 6.86 m and thereafter only boulders were found down to a depth of 7.47 m. The timber cuttings were very likely the result of intercepting the timber cribbing and the depths that they were found are fairly consistent with the depths of the cribbing shown on the design drawings. The organic silt found in borehole 1 was also found below the rock fill layer in borehole 2 but at a depth of 7.47 m. Field vane tests were taken in this organic silt and a shear strength of 25 kPa was obtained with a sensitivity of 1.3 to 2.8.

The organic silt extended down to a depth of 8.6 m and was underlain by a layer of sandy silt with a trace of clay. This layer was non plastic, grey in colour, loose to compact and wet and it extended down to a depth of 9.75 m. Below this sandy silt was a layer of fine to medium sand, with some silt and boulders and a trace of clay. It was grey, loose to compact, wet and extended down to 13.00 m below ground surface. Below this was a layer of boulders or possibly weathered bedrock. No recovery was obtained as the core barrel could not be retrieved. The borehole was terminated at 14.7 m. Detailed soil profiles are shown on the borehole logs and grain size analyses are shown on figures 1 and 2. Plasticity index properties for the organic silt are shown on figure number 3.

Asphalt, concrete and granular road base (Fill)

At borehole 1 there was a concrete deck 250 mm thick which appeared to be in sound condition, while at borehole 2 the surficial fill consisted of 150 mm asphalt cap over 850 mm granular base material. The granular base was in a compact state and extended down to a depth of 1.0 m or elevation 169.38 m in borehole number 2. The surface asphalt along both approaches were generally in good condition except adjacent to the north abutment where some signs of distress were observed.

Rock (Fill)

The rock fill consisted of various sizes. In borehole number 1, the rock fill was penetrated through rather easily by wash boring from 2.5 m to 4.9 m (elevations 167.9 m to 165.5 m) below the bridge deck surface. However in borehole 2 the rocks were of larger sizes, some other 600 mm in diameter and had to be cored through. Coring through the rocks were difficult and slow. The cored pieces recovered from borehole number 2 were granitic gneiss. The rock fill in borehole 2 was found between 1.0 m and 7.47 m depth below ground surface (between elevations 169.38 m and 162.91 m).

Organic Silt

The soil of primary concern at this site is the organic silt with a high sand content and some clay. The natural moisture contents of this stratum exceeded its liquid limits. From the design drawings this is the founding soil at the site. However, from the data obtained from borehole number 1, this soil was just above the timber cribbing between 4.9 m and 6.4 m below the bridge deck (between elevations 165.5 m and 164.0 m), while at borehole number 2 it was found at the base of the cribbing, between depths of 7.47 m and 8.6 m from ground surface (between elevations 162.91 m and 161.78 m). This soil was soft enough to permit the shelby tube to be hand pushed in both boreholes. Upon retrieval the shelby tube sample was lost from borehole number 1 and had to be retrieved by a split spoon while the sample recovered from borehole number 2 was later found to be disturbed upon extraction.

The following properties of this organic silt were determined in the laboratory:

BH# -	SA#	Water Content (%)	Liquid Limit(%)	Plastic Limit(%)	Unit Weight(kN/cu. m)
1	1	103	88.0	23.5	---
2	1	94.7	90.0	23.0	14.3
2	2A	92.4	89.3	23.1	14.2
2	2B	75.1	-----	-----	-----

Liquid limit tests on the organic silt were carried on both oven dried and naturally dried samples. The oven dried liquid limits were approximately 60% lower than those obtained from the naturally dried samples which indicates that there is an organic content in the silt.

Two vane tests were carried out in borehole number 2 between depths of 7.5 m and 8.0 m (elevations 162.88 m and 162.38 m) and both tests gave a shear strength of 25 kPa with sensitivities of 1.8 to 2.3.

Silty Sand

The silty sand was found to be the founding soil for borehole number 1 for the north abutment. It was clayey, non plastic, loose and was only 600 mm thick with a natural water content of 34.6 percent, unit weight of 18.6 kN/cu. m and an "N" value of 3. It was found at a depth of 6.4 m to 7.0 m below the bridge deck surface (elevation 164.0 m to 163.4 m).

Sandy Silt

The sandy silt layer was found immediately below the organic silt in borehole number 2 at a depth of 8.6 m (elevation 161.78 m) below ground surface and extended down to a depth of 9.75 m (elevation 160.63 m) Its moisture content was 31.4 percent and its "N" value was 9.

Sand

Sand was found in both borehole numbers 1 and 2 at depths of 7.0 m and 9.75 m respectively (elevations 163.4 m and 160.63 m) and extended down to depths of 8.8 m and 13.0 m (elevations 161.60 m to 157.38 m). This sand layer was fine to medium, with boulders. The natural water contents ranged from 16.4 to 23.4 percent and the "N" values ranged from 7 to 49. However, these values were likely affected by the presence of boulders and water pressure from sampling below the water table.

Bedrock

Bedrock was found in borehole number 1 at a depth of 8.8 m (elevation 161.60 m) and was cored down to a depth of 11.0 m (elevation 159.40 m). The bedrock was sound and consisted of a granitic gneiss.

Boulders/Weathered Bedrock

What appeared to be either boulders or weathered bedrock were found in borehole number 2 at a depth of 13.00 m (elevation 157.38 m). This was cored down to a depth of 14.7 m (elevation 155.68 m) where the borehole was terminated due to excessively high resistance to coring. No cores were recovered as the casing and core barrel could not be retrieved from the borehole and were eventually sealed in after several hours were spent trying to withdraw them.

GROUNDWATER

As the rock fill is permeable the ground water in both boreholes will essentially be the same as the river water level. The river water level on June 3, 1996 was found at a depth below the bridge deck of 2.6 m elevation 167.8 m) and on September 10, 1996 at a depth of 1.7 m (elevation 168.7). High water level is recorded at elevation 169.01 m or a depth of 1.4 m below the bridge deck level.

DISCUSSION AND RECOMMENDATIONS

The existing bridge has been in place for over 60 years. Per the available drawings the bridge was founded on timber cribbings and rock fill. The width of the concrete bridge abutments is 3 m while the length is 12 m. The length of the bridge between the outer faces of the concrete abutments is 18.7 m. The bridge consists of two traffic lanes and a pedestrian side walk. The deck of the bridge consists of asphalted concrete over a concrete base supported by steel beams.

The current cribbing and rock fill are presumed to be founded on the organic silt at 163.2 m. Construction of a larger heavier deck and abutments for the existing bridge are being contemplated and it is required to be known whether the existing foundations are capable of supporting additional loads. The proposed new design loads are at this time unknown.

Structure Foundations

Based on the subsoil conditions found at the boreholes drilled, additional foundation loads are feasible at this site. The major concern is the performance of the organic silt layer found in both boreholes close to the base of the rock fill and timber cribbing, between elevations 165.5 m and 164.0 m at borehole number 1 and between elevations 162.91 m and 161.78 m at borehole number 2. However, this layer is only 1.1 m to 1.5 m thick and settlement is not necessarily the most critical factor. To substantiate this we noted that the bridge deck is currently 250 mm lower than its design elevation of 170.73 m as per the design drawings provided.

Settlement

Stresses at point locations were determined by the formulae by Davis & Poulos by the method of superimposition. It was assumed that nearly all settlement were within the organic silt layer. Stresses at the centre and top of the organic silt layer were determined at both borehole locations. The water contents of the samples obtained at these locations were then used to estimate the values of void ratios assuming a normally consolidated silt. The difference in void ratios were equated to the difference in stresses between the two boreholes. From this value the compression index of the organic silt was estimated at approximately 1. This compression index was also determined at borehole number one from the calculated settlement of the bridge as per the original design drawings from 1937. These two ways of determining the compression index produced a value of close agreement and since no samples were available for consolidation testing this value of compression index above was used to estimate settlements for an increase in load of 10 percent in the abutments and bridge deck. From these calculations bearing capacities were then determined for the serviceability limit states Type II condition.

Bearing Capacities

The stresses generated in the organic silt layer were calculated at both borehole locations by stress distribution formulae by Davis and Poulos and also by assuming that the approach fill embankments were sloping at 2 : 1 horizontal to vertical.

The stresses were also calculated on the basis that the load from the abutment load was distributed by the rock fill over a 1H : 2V slope and that the weight of the fill itself was distributed along slopes of 2H : 1V.

The stabilities of the slopes in front of the abutments together with the abutments were also checked for sliding failure. Based on these calculations and the performance of the structure over the last 60 years bearing capacity values were determined.

In accordance with Ontario Highway Bridge Design Code the following design values are recommended for the footings at the base of the timber cribbing and rock fill for both abutments:

Bearing Capacity at Servicibility Limit States Type II (Kpa)	Factored Bearing Capacity At Ultimate Limit States (Kpa)	Elevation (m)
60	100	163.5 - 162.0

For the rock fill where the concrete abutments are founded the recommended design values for purposes of the O.H.B.D.C., the following design values are recommended assuming that the bases will be the same sizes and at the same locations:

Bearing Capacity at Serviceability Limit States Type II (Kpa)	Factored Bearing Capacity At Ultimate Limit States (Kpa)	Elevation (m)
140	900	168 - 167.5

It is important to note that changing the locations or sizes of the existing abutment footings will alter the design values given above as different locations and sizes of the footings will have different bearing capacities which may be controlled by the organic silt layer .

Settlement caused by additional loads to the existing foundations will be time dependent because of the compressible organic silt. Total and differential settlements based on the above values are expected to be within the tolerable limits of construction of 25 mm and 19 mm respectively, provided the organic silt is not disturbed by any activities associated with construction.

From our calculations the values provided above represents a 10 percent increase of the loads of the bridge, i.e. abutments and deck. Therefore we recommend that the existing structure be rehabilitated providing that the additional loads be kept to a maximum increase of 10 % of the current loads from the existing bridge.

Lateral Resistance

Resistance to sliding of the abutments along the base of the rock fill can be calculated based on an angle of friction of 35° while resistance to sliding or slope failure within the organic silt may be calculated based on a shear strength of 11.4 kPa in accordance with the O.H.B.D.C. at U.L.S.

Lateral Earth Pressures

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

Angle of internal friction (degrees)	35
Unit Weight (KN/cu. m.)	22

For a yielding structure, the active condition (Ka) may be assumed to apply while for rigid and unyielding structures, the at rest condition (Ko) should apply.

Approach Embankments

Per the original design drawings the existing approach embankments are about 7 m high but only about 4 m above the original river bed surface. The current side slopes of the embankments vary from 1:2 to 1:3 being steepest closest to the abutment. The slopes in front of the abutments, though are only 1:4 vertical to horizontal. Neither abutments show any signs of rotational failure or of uneven settlement.

The approaches at their current elevations appeared to have settled about an inch with respect to the abutments and there is some signs of distress (longitudinal cracks) along the centre line of the approaches. Some alligator cracking were found just to the north of the north abutment indicating some type of subgrade failure. Possibly there is some lateral displacement of the organic silt occurring from the weight of the fill along the steeper 1:2 side slopes. It is recommended that these side slopes be flattened to 1.3 if heavier loads are used for the modified bridge and particularly if the approach height is raised. In any event it is not recommended that the approach height be raised higher than 300 mm above its current height at any point. Further if the approach heights are raised they should be preloaded for a period of three months prior to new construction.

MISCELLANEOUS

The field work for this investigation was carried out under the supervision of R. Jagdat. The equipment used was owned and operated by Marathon Drilling (1994).

The laboratory work and report for this project was performed by Mahendran Raja and it was supervised and reviewed by R. Jagdat, Principal Engineer.

Submitted by
CANADA ENGINEERING SERVICES INC.

Raja Mahendran, P.Eng.

RJagdat
Ram Jagdat, P. Eng.

LICENSED PROFESSIONAL ENGINEER
R. M. MAHENDRAN
Sept 13/96
PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER
R. JAGDAT
Sept 13/96
PROVINCE OF ONTARIO

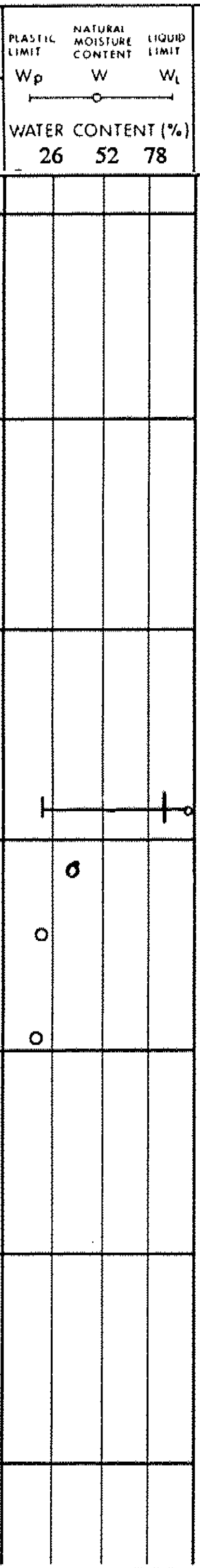
Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 1

METRIC

W P 48-94-01 LOCATION Co-ords: N4,931,467.67, E 257,917.045 ORIGINATED BY R.J.
DIST. 41 HWY 41 BOREHOLE TYPE Wash Boring COMPILED BY M.R.
DATUM Geodetic DATE June 3/96 CHECKED BY R.J.

SOIL PROFILE			SAMPLES							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
170.40	Bridge Deck Surface									
	ASPHALT & CONC. DECK						170			
0.25	FREE SPACE ROCK FILL					▽	168 166			
4.9	ORGANIC SILT - sandy, clayey, Of high plasticity, dark grey soft, wet		1	SS	1		164			
6.4	SILTY SAND- clayey, non-plastic, grey, loose, wet		2	SS	3				18.6	0 55 33 12
7.0	SAND- fine to medium, occasional boulders, grey, loose to dense, wet		3	SS	7					0 90 8 2
			4	SS	49		162			0 88 12 0
8.8	BEDROCK - Granite Gneiss, sound		5				160			
11.0	END OF BOREHOLE						158			



Note: liquid limit obtained from naturally dried sample; $+^3$, $\times^3$: Numbers refer to Sensitivity

Ministry of Transportation
Ontario

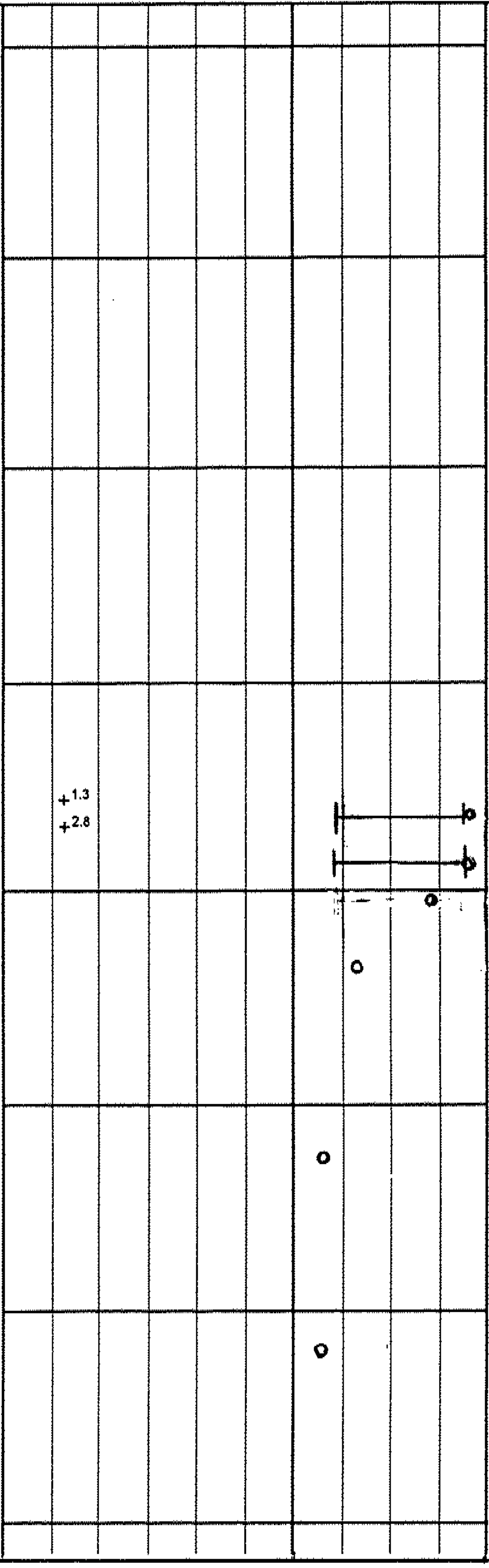
RECORD OF BOREHOLE No 2

METRIC

W P 48-94-01 LOCATION Co-ords: N4,931,464.60, E 257,953.39 ORIGINATED BY R.J.
DIST 41 HWY 41 BOREHOLE TYPE Wash Boring COMPILED BY M.R.
DATUM Geodetic DATE June 4/96 CHECKED BY R.J.

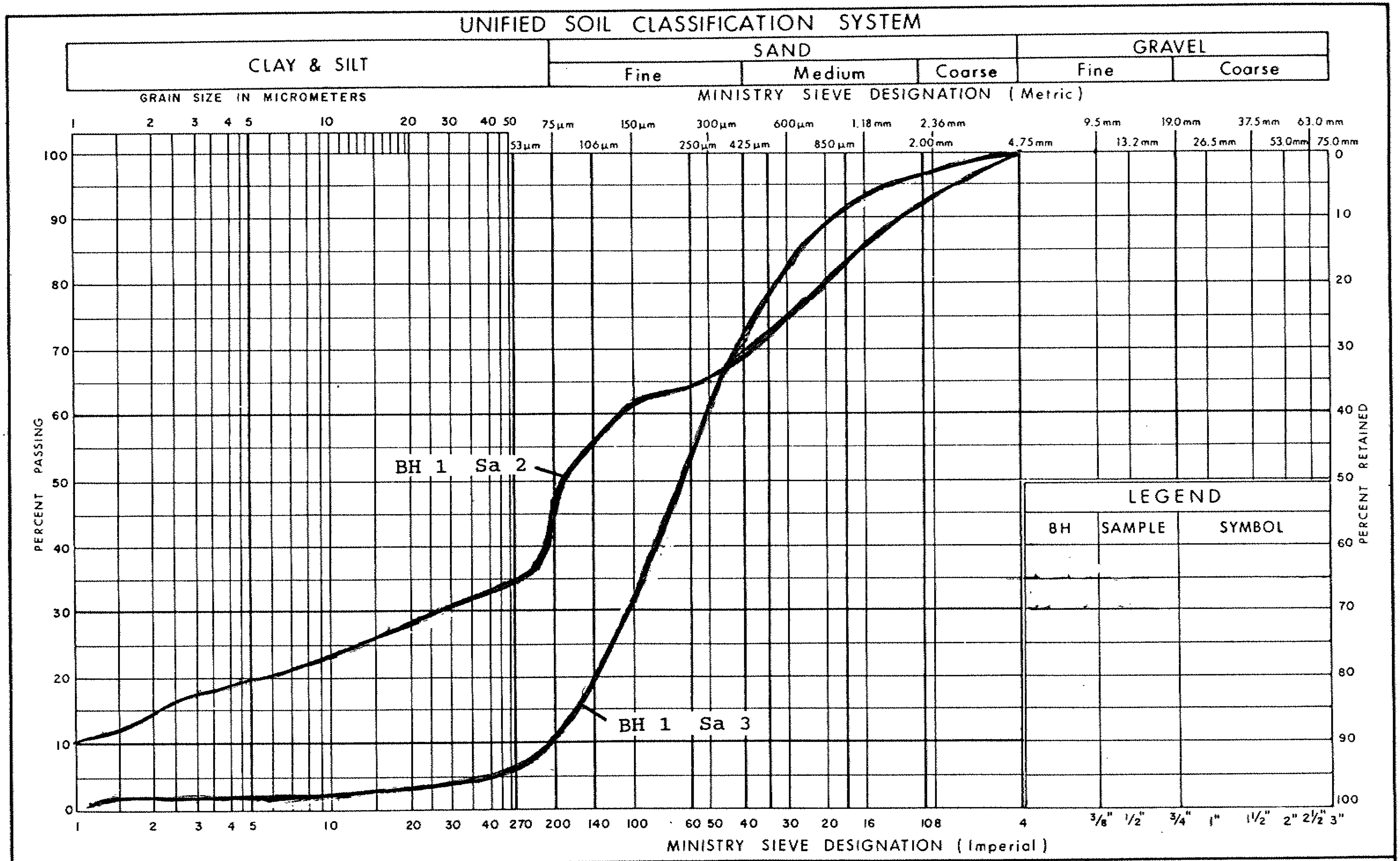
DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE
PLASTIC LIMIT Wp, NATURAL MOISTURE CONTENT W, LIQUID LIMIT WL
WATER CONTENT (%) 26 52 78

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	UNIT WEIGHT γ	GR	SA	SI	CL
170.38	Ground Surface												
	ASPHAT & GRANULAR ROAD BASE						170						
1.0	ROCK FILL					▽	168						
3..35	Pieces of wood cuttings and boulders Possible timber cribbing with Boulder fill - between 3.35 m and 6.86 m						166 164						
6.86	-End of timber cuttings ROCK FILL												
7.47	ORGANIC SILT - sandy, clayey, Of high plasticity, dark grey soft, wet		1	SS	2			+1.3 +2.8	14.2	0 0	48 48	35 40	17 12
			2	ST	6		162		14.3	0	39	51	10
8.60	SANDY SILT , trace clay, non-plastic, grey, loose to compact wet		3	SS	9					0	13	82	5
9.75	SANDfine to medium, some silt, trace clay -with boulders, grey, loose to compact, wet						160						
			4	SS	9					0	77	15	8
							158						
			5	SS	14					3	87	8	2
13.00	BOULDERS OR WEATHERED ROCK												
14.7	END OF BOREHOLE						156						



Note: liquid limit obtained from naturally dried sample; $+^3$, $\times^3$: Numbers refer to Sensitivity

78 12 M



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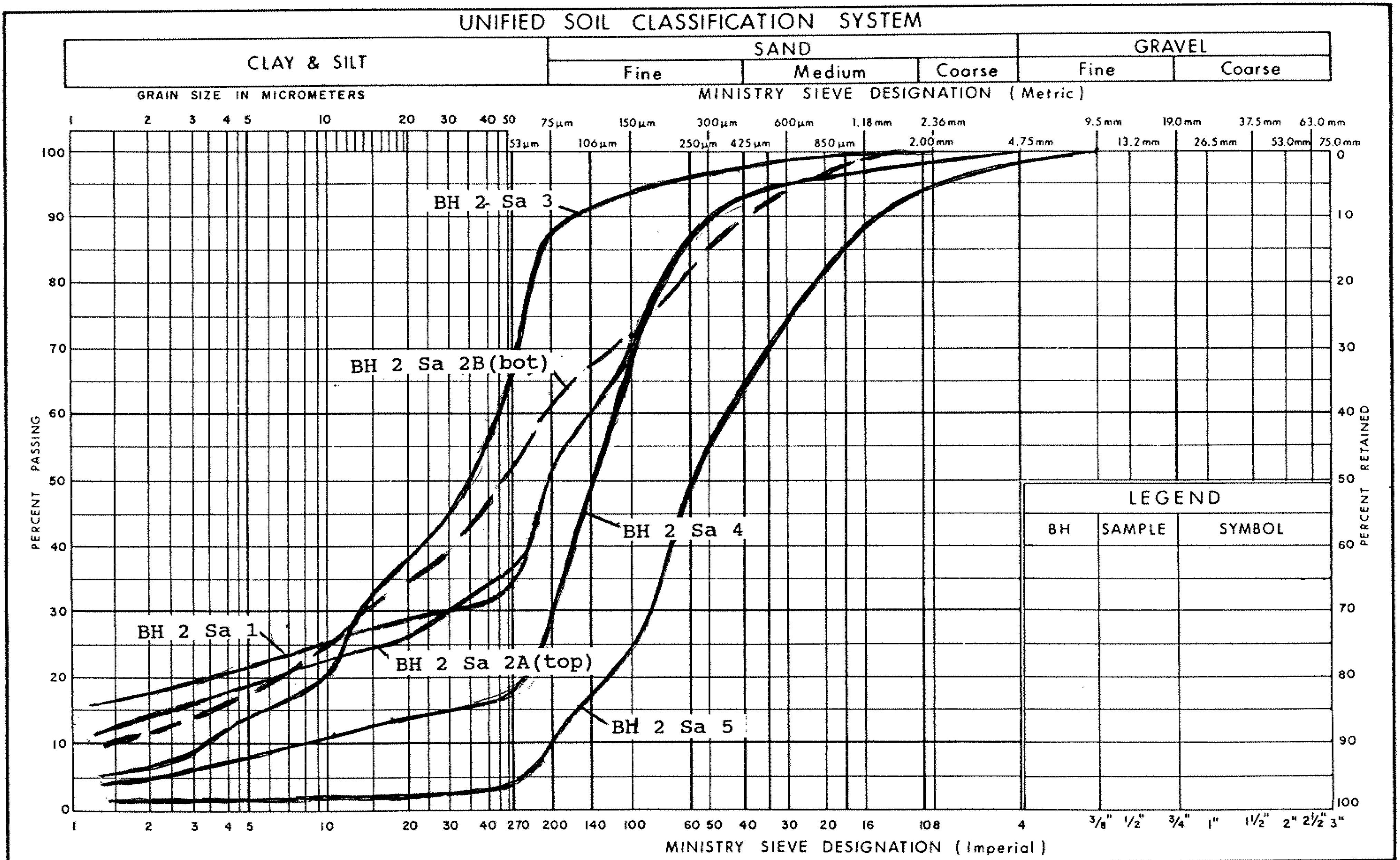
GRAIN SIZE DISTRIBUTION

SILTY SAND TO SAND

FIG No 1

W P 48 - 94 - 01

78 12 M



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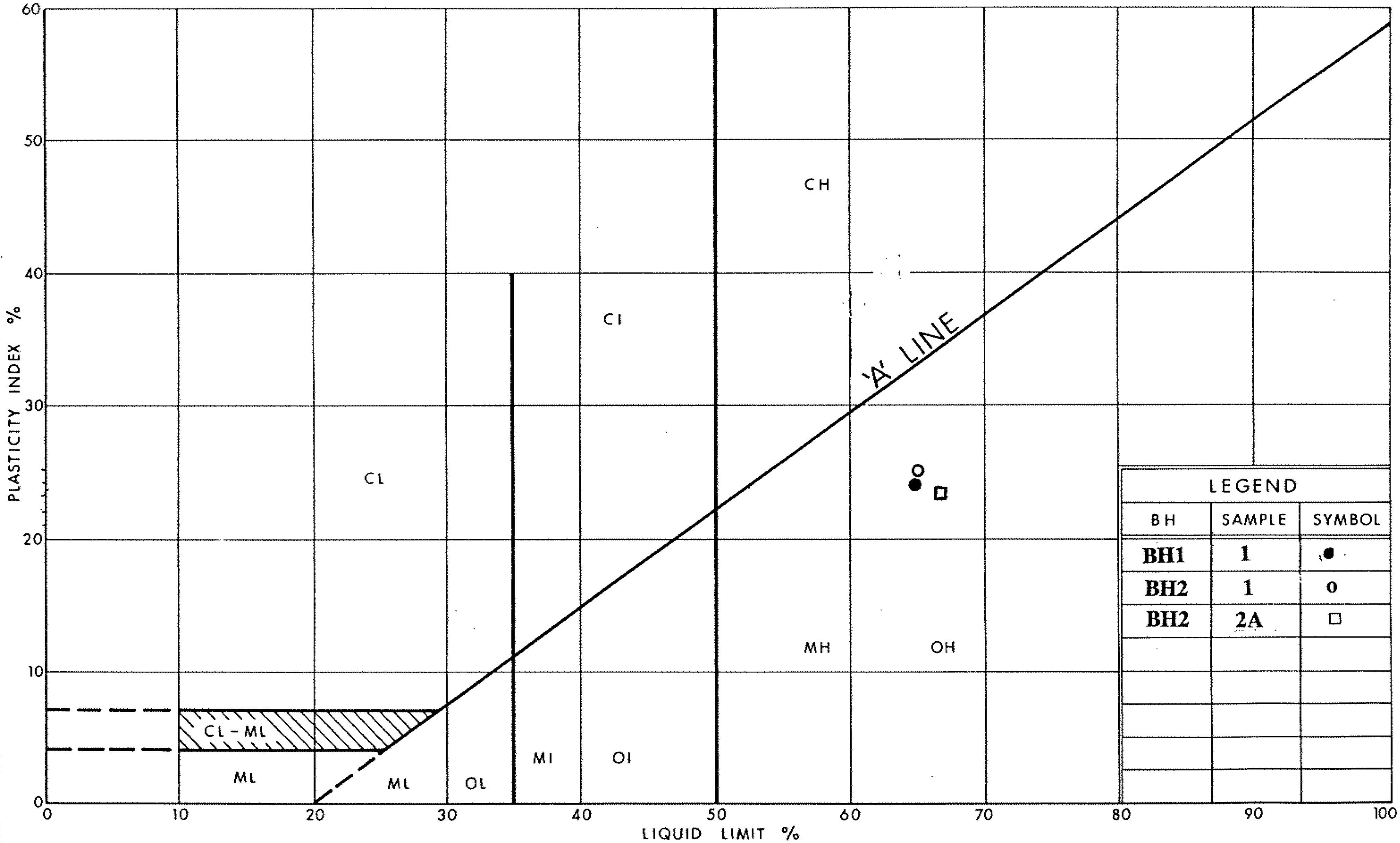
GRAIN SIZE DISTRIBUTION
ORGANIC SILT TO SAND

FIG No 2

W P 48 - 94 - 01

Oct 75, FF-S-21



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PLASTICITY CHART

ORGANIC SILT

sandy, clayey

FIG No 3

W P 48 - 94 - 01

OVERSIZE DRAWING(S)

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

PLATE No 271-41/100-0
CONT No
WP No 48-94-00

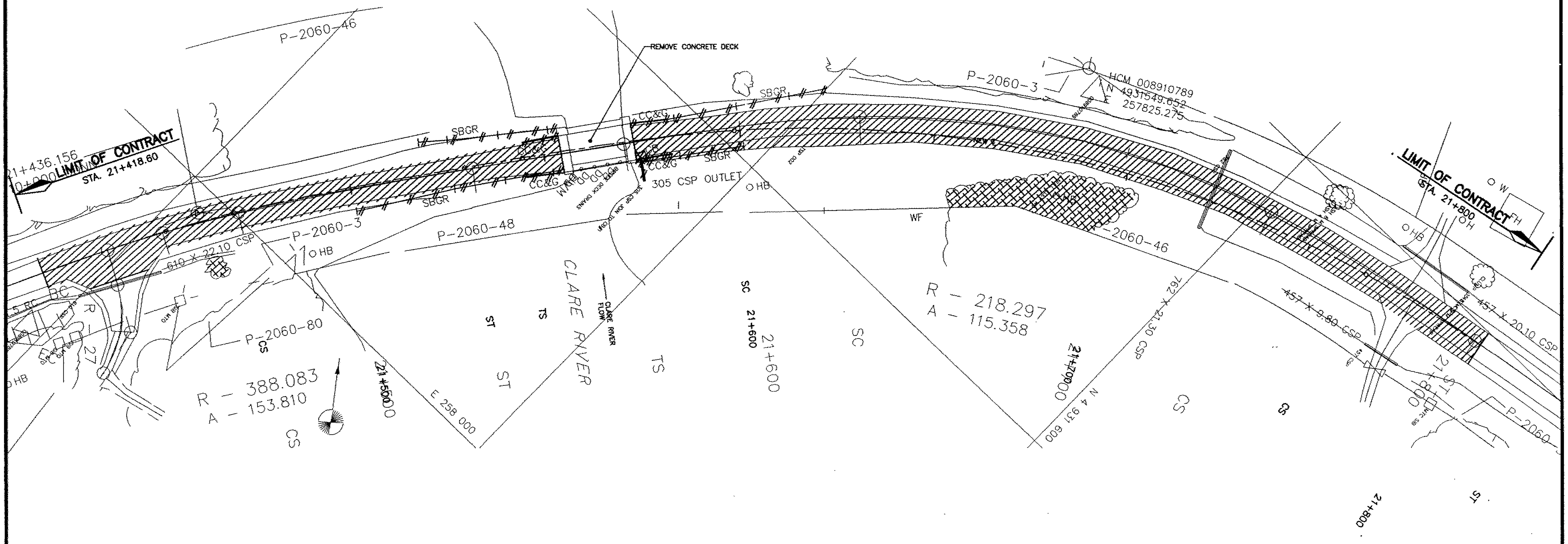
McNEELY
ENGINEERING
CONSULTANTS LTD.

REMOVALS
STA. 21+418.60 TO STA. 21+800
Survey MARCH/95 Revised

SHEET
01

SCALE
5 0 5 10m

▨ IN-PLACE FULL DEPTH RECLAMATION OF BITUMINOUS PAVEMENT AND UNDERLYING GRANULAR



PAVING DETAILS

SURFACE COURSE 50mm HL4

GRANULAR DETAILS

GRANULAR PADDING TO 400mm – GRANULAR 'A'
WIDENINGS *– 300mm GRANULAR 'A'
*– 475mm GRANULAR 'B' TYPE I
*INCREASE GRANULAR 'A' DEPTH
TO MATCH ROADWAY PADDING

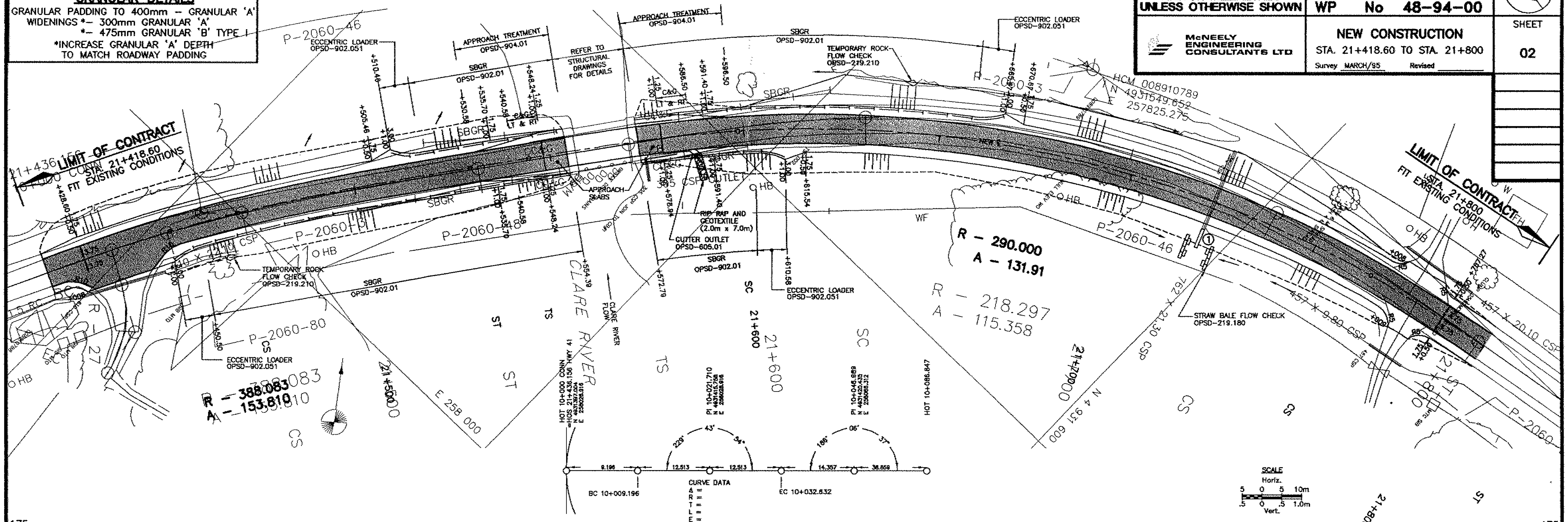
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

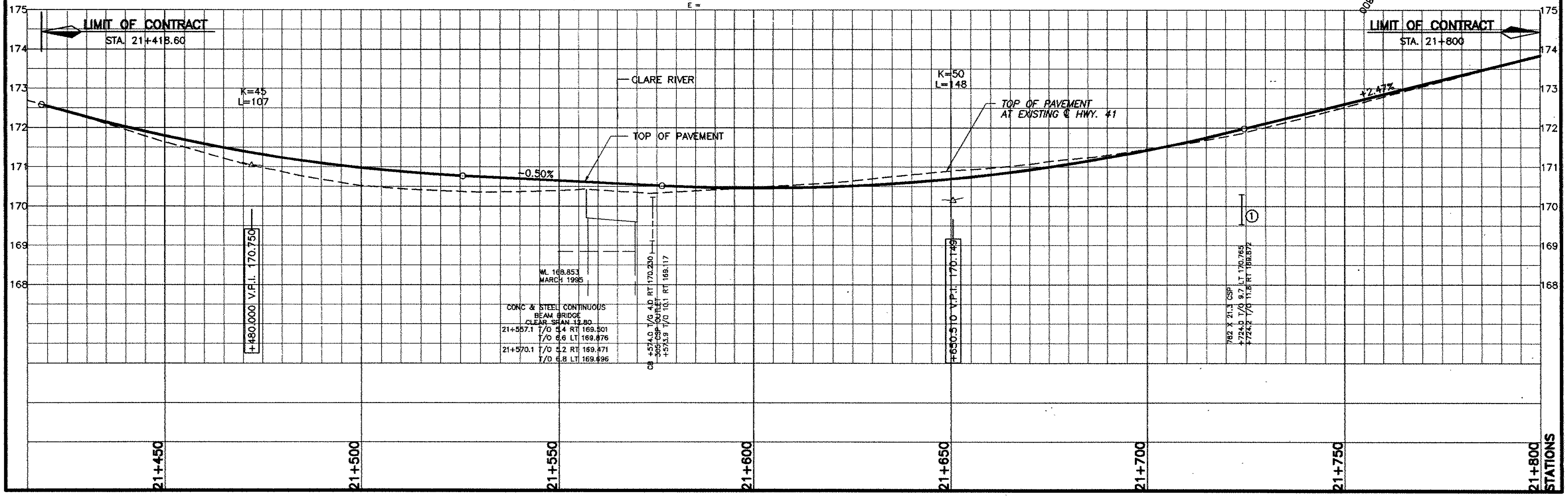
McNEELY ENGINEERING CONSULTANTS LTD

PLATE No 271-41/100-0
CONT No
WP No 48-94-00

NEW CONSTRUCTION
STA. 21+418.60 TO STA. 21+800
Survey MARCH/95 Revised

SHEET
02





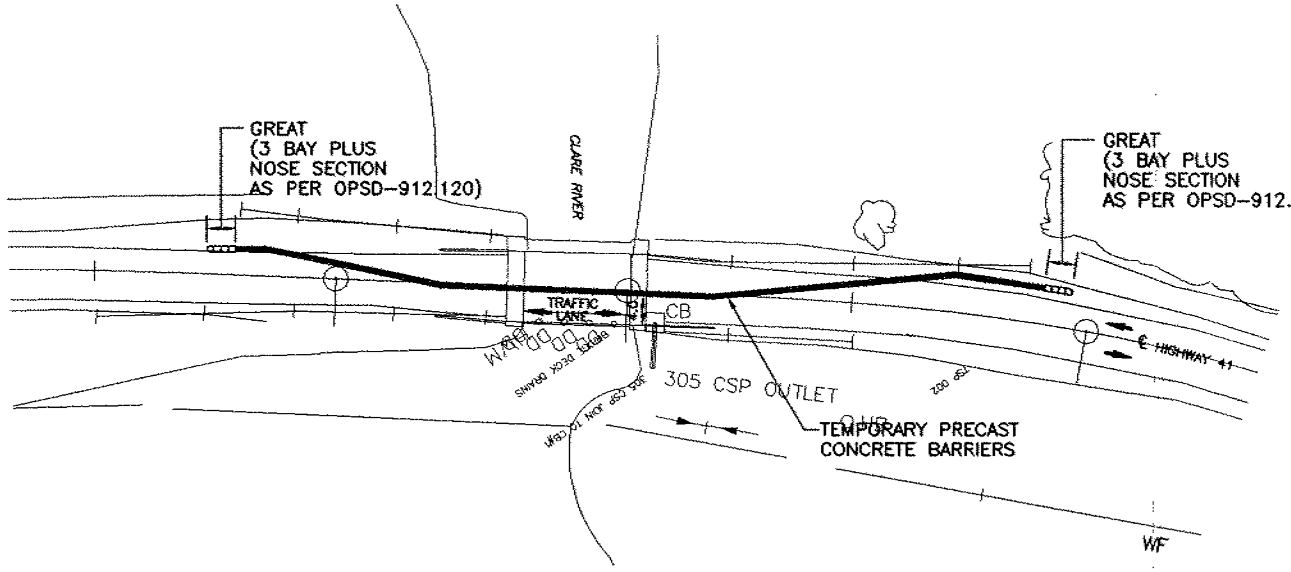
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

PLATE No
CONT No
WP No 48-94-00

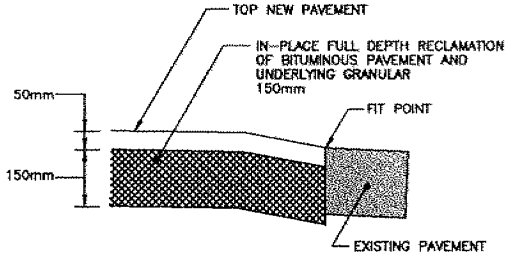
McNEELY ENGINEERING CONSULTANTS LTD.

DETOUR STAGING DRAWING AND TYPICAL SECTION
Survey ______ Revised ______

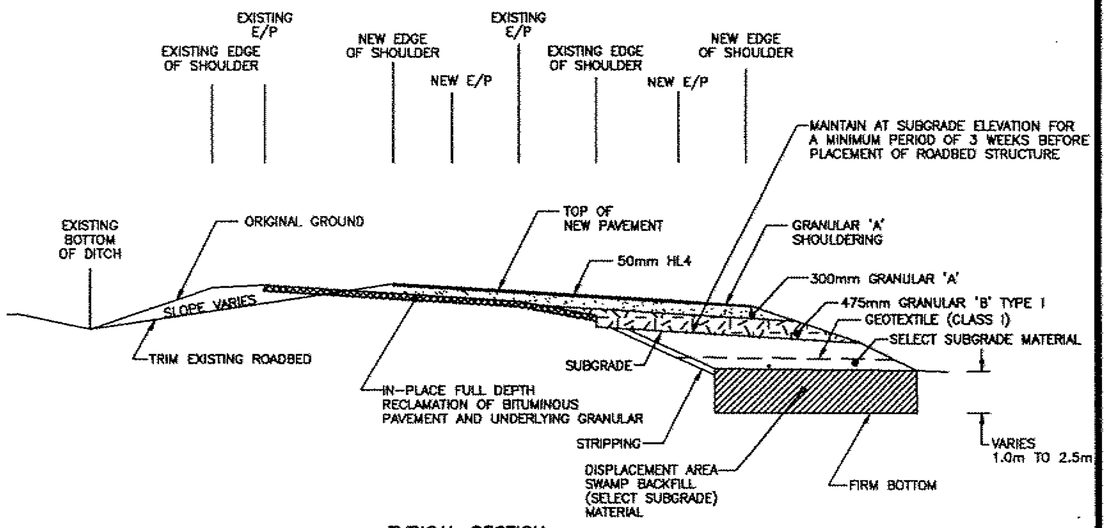
SHEET
03



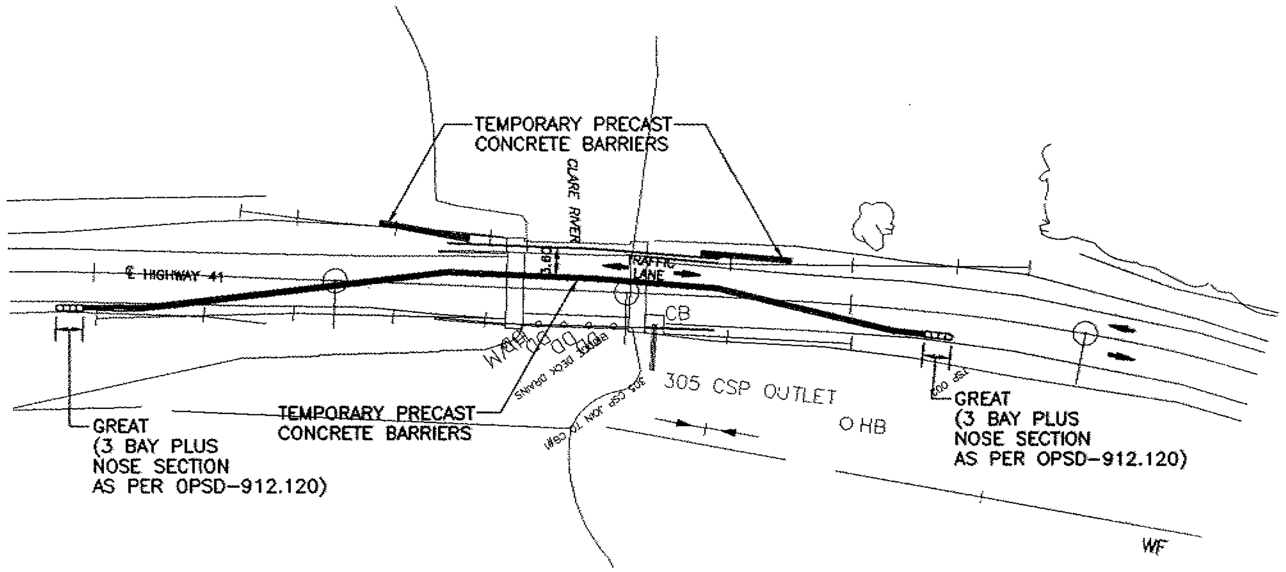
DETOUR PLAN – STAGE I
NOTE: TO BE USED IN CONJUNCTION WITH OPSD-920.02
SCALE
5 0 5 10m



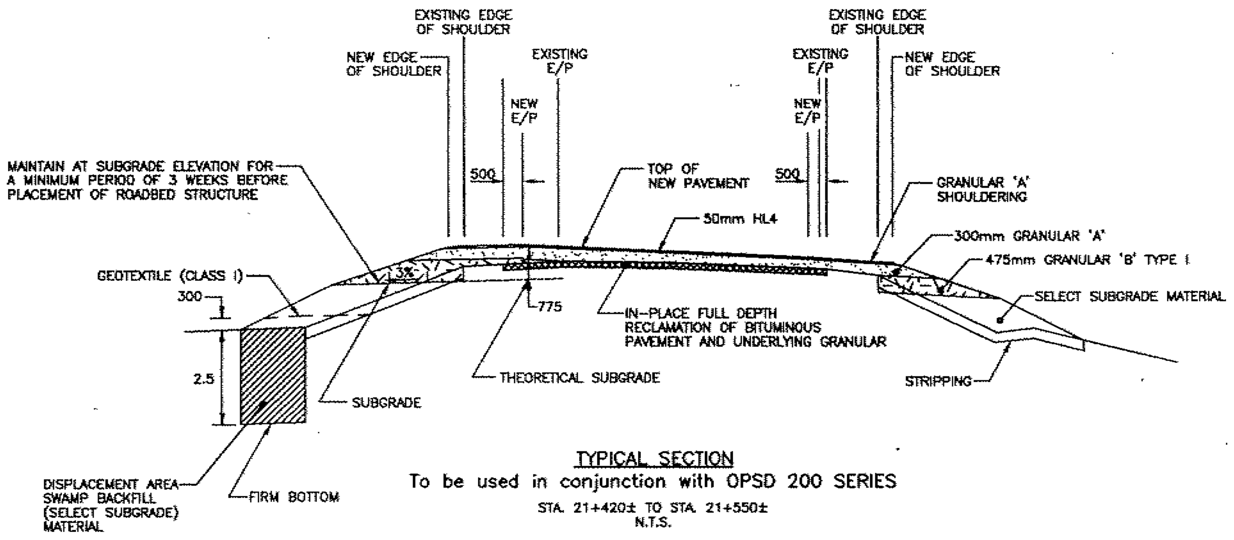
LONGITUDINAL TRANSITION
HOT MIX PAVING TO EXISTING PAVEMENT
STA. 21+418.6
STA. 21+800
N.T.S.



TYPICAL SECTION
To be used in conjunction with OPSD 200 SERIES
STA. 21+575± TO STA. 21+775±
N.T.S.



DETOUR PLAN – STAGE II
NOTE: TO BE USED IN CONJUNCTION WITH OPSD-920.02
SCALE
5 0 5 10m



TYPICAL SECTION
To be used in conjunction with OPSD 200 SERIES
STA. 21+420± TO STA. 21+550±
N.T.S.

MEMORANDUM

Ontario

To: Q.M. Islam, P. Eng..
Sr. Structural Engineer
Eastern Region

May 12, 1997

Attn: D. Kerr, P. Eng.
Structural Engineer

From: Pavements and Foundations Section
Room 315, Central Bldg.

Tel: (416) 235-5267
Fax: (416) 235-5240

Re: Foundation Investigation Report
Clare River Bridge Rehabilitation
District 41, Kingston

In response to your memoranda dated May 6, 1997 and May 9, 1997, our office has undertaken a technical review of the foundation design at the above mentioned site. Applied loads and bearing pressures submitted by McNeely Engineering Consultants have been reviewed and our comments are contained in this memorandum.

Background

The existing foundations at the site are supported by timber cribbing and rock fill that was placed on the native soil. The elevation at the interface of the base of the concrete abutment and the rock fill is approximately 167.5. The elevation at the interface of the organic silt and base of the crib wall is approximately 162.5 m.

The native soil is comprised of a surficial thickness of organic silt of thickness up to approximately 1.5 metres, underlain by a silty sand to sandy silt stratum approximately 0.6 m to 1.15 m thickness. This stratum is in turn underlain by a loose to compact sand deposit of thickness ranging from 1.8 m to 3.25 m.

The overburden at the site is underlain by granite gneiss bedrock.

Calculation of Additional Bearing Pressure on Native Soil

In their letter dated May 6, 1997, McNeely Engineering Consultants Ltd enclosed a summary of calculated stresses at the underside of the existing abutment. The calculations reveal an increase of bearing stress at the footing - rock fill crib interface equivalent to 12 kPa and 13 kPa at the SLS and ULS respectively incorporating applicable OHBDC loading factors. Based on a Boussinesq stress distribution solution, it is estimated that the additional stress imposed on the native soil will be in the order of 5 kPa.

Foundation Bearing Resistance

For purposes of the OHBDC, the following bearing resistances are provided at the concrete abutment/rock fill interface

Factored Bearing Resistance at ULS	300 kPa
Bearing Resistance at SLS	200 kPa

For the above mentioned bearing pressures, it is predicted that the additional stress imposed on the native soil will induce a total settlement of up to 25 to 30 mm. Differential settlements up to 20 mm could be expected.

We trust that the above comments are sufficient for your purposes. If you have any questions please do not hesitate to contact this office.

T. Sangiuliano, P. Eng.
Foundation Engineer

for

D. Dundas, P. Eng.
Senior Foundation Engineer

MEMORANDUM

Ontario

To: H. Kleywegt, P. Eng..
Senior Structural Engineer, Structural Section
Eastern Region

May 1, 1997

Attn: D. Kerr, P. Eng.
Structural Engineer

From: Pavements and Foundations Section
Room 315, Central Bldg.

Tel: (416) 235-5267
Fax: (416) 235-5240

Re: Foundation Design Report - *Bearing Resistance at Existing Timber Crib*
Clare River Bridge Rehabilitation
District 41, Kingston

As requested in your memorandum dated April 7, 1997, our office has coordinated the review of McNeely Engineering Consultant's comments regarding the calculated bearing pressures at the crib-native soil interface. Attached please find Canada Engineering Services Inc. addendum letter dated May 1, 1997 that clarifies the load distribution through the rockfill. Canada Engineering Services have revised their bearing capacities accordingly.

We trust the information contained in Canada Engineering's letter is sufficient for your purposes. If you have any questions , please do not hesitate to contact this office.

T. Sangiuliano, P. Eng.
Foundation Engineer

for

D. Dundas, P. Eng.
Senior Foundation Engineer

CANADA ENGINEERING SERVICES INC.
CONSULTING ENGINEERS
28 Landseer Road, Scarborough, Ontario M1K 3A7
Tel: 416 757 2211 Fax: 416 750 3926 E-mail: ces@interlog.com

May 1, 1997

Report No. 309-6

Ministry of Transportation of Ontario
Pavements and foundations Section
Room 315, Central Building
1201 Wilson Avenue
Downsview, Ontario
M3M 1J8

Attention: Mr. Dave Dundas, P. Eng.
Senior Foundation Engineer

ADDENDUM

FOUNDATION INVESTIGATION AND DESIGN REPORT FOR CLARE RIVER BRIDGE REHABILITATION
WP 48-94-01, SITE 17-16
HIGHWAY 41, DISTRICT 41, KINGSTON

Dear Mr. Dundas:

We have reviewed the letter you forwarded us from the structural consultants, McNeely Engineering Consultants Ltd., dated April 1, 1997, their calculated bearing values required for the proposed new deck for the above captioned bridge and the accompanying memorandum from the Eastern Region Structural Section dated April 7, 1997.

The structural consultants did not indicate in their letter what cribbing area the load was distributed over . In our calculations we used a significantly larger area (scaled off drawing no. 96-309-1 submitted with our original report) than the footprint of the abutment to distribute the load from the abutment. This may account for the discrepancy in the figures obtained. In any event, as we discussed on the phone, the load distribution on a 1H:2V slope is conservative and a 1H:1.4V is probably closer to reality. The required bearing values calculated by the structural consultants for a 1H:1.5V slope are only about 4 percent higher than the values that were provided in our previous report. For the foregoing reasons and assuming that the load increase of the bridge deck will remain the same as previously stipulated, we have adjusted the allowable bearing pressures to reflex this slightly higher value.

In accordance with the Ontario Highway Bridge Design Code the following modified design bearing values are recommended for the footings of the abutments:

Bearing Capacity at Serviceability Limit States (kPa)	Factored Bearing Capacity At Ultimate Limit States (kPa)	Elevation (m)
85	104	163.5 - 162.0 At the interface of the organic silt and base of crib wall
175	250	168 - 167.5 At the interface of crib wall and base of concrete abutment

Total and differential settlements based on the above values are expected to be within the limits of 25 mm and 19 mm respectively, provided the organic silt is not disturbed by any activities associated with construction.

We trust that these modified bearing values will be adequate for the proposed new deck. Please call the undersigned if there are any questions.

Yours very truly,
CANADA ENGINEERING SERVICES INC.

R Jagdat

Ram Jagdat, P. Eng.

REGISTERED PROFESSIONAL ENGINEER
R. JAGDAT
PROVINCE OF ONTARIO
May 1 97

**McNEELY
ENGINEERING
CONSULTANTS LTD.**

File No. S6721

HAND DELIVERED

April 1, 1997

MINISTRY OF TRANSPORTATION - EASTERN REGION
STRUCTURAL SECTION
REMARKS
APR 2 1997
RETENTION
STR. ENG. | STR. ENG. | STR. ENG.
STR. OFF. | STR. OFF. | STR. ENG.
CR. | CR. | STR.OFF.

Ministry of Transportation
355 Counter St., Postal Bag 4000
Kingston, Ontario
K7L 5A3

Attention: **Mr. David Kerr, Structural Engineer**
Structural Section, Eastern Region

Reference: ***Structure Rehabilitation of Clare River Bridge***
and Improvements to the Roadway Alignment
on Highway 41, District 41 - Kingston
WP 48-94-00 - MTO Site No. 17-16

Dear Sir,

Further to our meeting on March 21, 1997, I have revised the calculations for foundation bearing pressures as we discussed. I attach the calculations (pages A1-A5) for your reference when you are discussing this matter with the Foundation Design Section or with the Geotechnical Consultant.

The bearing pressures at the underside of the abutment for the proposed deck loads (and including effects of the new approach slabs) are calculated to be 169 kPa and 239 kPa at the SLS and ULS respectively. This is slightly less than the allowable values of 175 kPa and 250 kPa respectively.

The calculated bearing pressures at the silt layer are 88 kPa and 111 kPa at the SLS and ULS respectively. This is based on a distribution of vertical abutment loads on a 1H:2V slope. These pressures are calculated based on a rock fill depth of 5.5 m with a submerged weight density of 12 kN/m3 and based on neglecting the fill weight behind the abutment. As we agreed at our meeting, the calculations at the silt layer include the vertical effect of the abutment loads acting concentrically on the silt layer below, but neglect the effect of the unbalanced lateral pressure at the back face of the abutment.

815 Princess Street, Kingston, Ontario K7L 1G6
(613) 549-0500 Fax (613) 542-5082

If the distribution is taken at a slope of 1H:1.5V, then the bearing pressures are reduced to 83 kPa and 104 kPa for the SLS and ULS respectively.

Please review these results with your Geotechnical Consultant or the Foundation Design Section and confirm that the assumptions made in the calculations (ie. use of submerged weight of fill and neglecting vertical and lateral effects of abutment backfill at silt layer) are correct and that the calculated bearing pressures are acceptable.

Sincerely,

F. Pfendt

Frank Pfendt, P.Eng.
Senior Structural Engineer
FP/dp
encl.
clare.br/kerr.d04

(416) [illegible]35- 5240

memorandum

Ontario

Tel. (613) 545-4832
Fax. (613) 545-4821

TO: Dave Dundas, P.Eng. (FAX)
Sr. Foundation Engineer
Pavements and Foundations Section

DATE: January 24 1997

FROM: Structural Section
Eastern Region

RE: Clare River Bridge, Site 17-016, Foundation Design Report
Highway 41, District 41 - Eastern Region

As per your E-mail and our conversations regarding the above mentioned report I have reviewed the situation with Ted and have the following submission to offer. The present and anticipated future loading applied to the existing foundations have been reviewed and the following bearing pressures have been calculated at the elevation of 168-167.5m for SLS and ULS1 for the following options :

i) A 225 mm thick Normal density reinforced concrete deck the anticipated bearing pressure at SLS is 190 kPa and at ULS1 is 260 kPa. This option however will increase the unfactored dead load by approximately 30 %.

ii) A 225 mm thick Light Weight reinforced concrete deck the anticipated bearing pressure at SLS is 170 kPa and at ULS1 is 245 kPa. This option will increase the unfactored dead load by approximately 7% but will increase the concrete cost by approximately $7500 and introduces a concrete durability problem.

Will you please review the above information and provide us with a recommended course of action or allowable values for this structure. As I have indicated previously the RFQ has been issued and therefore a quick response to this issue is critical. If you need any further information please feel free to call.

Submitted By	Reviewed By
David K	
David Kerr, P. Eng. Structural Engineer	E.C. Lane, Head Structural Section

c.c. E.C. Lane (E-mail)
H. Kleywegt (E-mail)

memorandum

Ontario

Tel. (613) 545-4832
Fax. (613) 545-4821

TO: Dave Dundas, P.Eng. (E-mail)
Sr. Foundation Engineer
Pavements and Foundations Section

DATE: January 21, 1997

FROM: Structural Section
Eastern Region

RE: Clare River Bridge, Site 17-016, Foundation and Design Report - Addendums
Highway 41, District 41 - Eastern Region

As I discussed with yourself and Tony Sangiuliano today we have reviewed the second letter (referenced as Report 309-2) from Canada Engineering Services Inc. the Foundation Consultant for the above noted structure and are not satisfied with its recommendations.

To summarize the initial report (report 309-1) and subsequent letter from the consultant, dated January 6th, 1997, provided recommended design bearing capacities as well as a recommended allowable 10% increase in dead loads. The addendum, report 309-2, still supported a 10% increase in dead loads but modified the bearing pressure recommendations : SLS has been changed to allow an allowance above the existing loadings, and ULS bearing pressures have remained the same at the elevation 163.5-162.0m but has been reduced from 900 kPa to 200 kPa at the elevation 168-167.5m.

The latest revisions puts me in a very similar situation as I indicated in my letter of December 19, 1996 in such that the rehabilitation design, using light weight concrete, can accommodate the 10% increase in dead load but will still exceed the recommended design bearing pressures. As I indicated in my E-mail of January 8th I have calculated a SLS bearing pressure of 171 kPa at the 168-167.5 elevation for the existing structure. Now if I include the reports recommended allowance (+10 kPa at elev 168-162.5) the maximum applied bearing pressure at SLS for the rehabilitated structure would be 181 kPa which is almost the allowable loading, from the report, at ULS.

Therefore, please accept this letter as a request to have the report and its associated letters/ addendums be revisited and that a report be issued which provides appropriate recommendations for the existing foundations for this structure. Furthermore, the RFQ has been sent out to a group of consultants for the rehabilitation and roadway design employing a tight design time frame and therfore I am requesting that the appropriate speed and attention be given to this situation.

171 SLS
200 ULS

David Kerr, P. Eng.
Structural Engineer

c.c. E.C. Lane (E-mail)
H. Kleywegt (E-mail)

Tel. (613) 545-4832
Fax. (613) 545-4821

TO: Dave Dundas, P.Eng. **DATE:** December 19 1996
Sr. Foundation Engineer
Pavements and Foundations Section

FROM: Structural Section
Eastern Region

RE: Foundation and Design Report for Clare River Bridge on Highway 41, Site 17-016
District 41 - Eastern Region

As I discussed with you previously we have received the above mentioned report and are comparing our preliminary calculations for a new bridge deck on the existing foundation with the recommended values in the report. To summarize the report provides recommended design bearing capacities as well as a recommended maximum increase of 10% above existing loads.

The situation that we have encountered is that we are able to not increase the current loading above the 10% limit but the applied bearing pressures exceed the recommended design values. As well a review of the bearing pressures for the existing deck arrangement also exceeds the recommended design values.

You reminded me on December 17th that this foundation report was done in a hands off approach (the report was only reviewed by your office for format and not accuracy or completeness) and that your office would assist us in any of our concerns but that they would ultimately be the responsibility of the consultant. I therefore spoke to Ram Jagdat, co-author of the report, yesterday regarding our concerns and we discussed the recommendations and how they correlate to our calculations. Ram ultimately indicated that he could not see a problem with a 10% increase in existing loading and would be willing to issue a supplemental report or an addendum to the report to reflect this statement.

Therefore, as we discussed today, please accept this letter as a request to have the consultant provide further specific details on the recommendations. Specifically, if a 10% increase in existing loading will be allowable. With regards to time frame, I am currently preparing a consultant assignment for the rehabilitation design and plan to issue it January 10th and therefore will need a firm commitment regarding this matter prior to this date in order to avoid possible cancellation or change of the terms of reference for the assignment.

David Kerr, P. Eng.

Structural Engineer

MEMORANDUM

Ontario

To: D. Dundas, P. Eng.
Senior Foundation Engineer

September 17, 1996

From: T. Sangiuliano, P. Eng.
Foundation Engineer

Tel: (416)235-3731
Fax: (416)235-5240

Re: Review of Consultant Report
Hwy 41 & Clare River
WP 48-94-01, Site 17-16
District 41, Kingston

The Foundation Investigation Report for the above mentioned project prepared by Canada Engineering Services Inc. has been reviewed. Although most of the comments that I previously submitted in my memorandum dated August 19, 1996 have not been addressed, it is understood that with the recent MTO business initiatives, the Consultant shall be responsible for the report. I question, however, that we have accepted incomplete borehole logs.

Canada Engineering Services have made changes to the "Discussion and Recommendations" which will certainly allow the Structural Engineer to determine whether or not the existing foundations are suitable to support additional loadings. Additional comments regarding the recommendations are summarized below.

Structure Foundations

Bearing Capacities

In my opinion, the bearing capacities are generally marginally on the liberal side. In particular, the factored capacity at ULS at the rock fill/native soil interface given is **100 kPa**. At the concrete abutment/rock fill interface, the factored capacity at ULS is **900 kPa**. I expect a punching shear failure at the ULS in the Organic Silt will be realized at lower pressures.

It is not understood why the Geotechnical Engineer would comment on the magnitude of permissible structural loads as the report does at the top of page 8. We should only provide the bearing capacities and allow the Structural Engineer to determine the magnitude of additional loads allowable.

Lateral Resistance

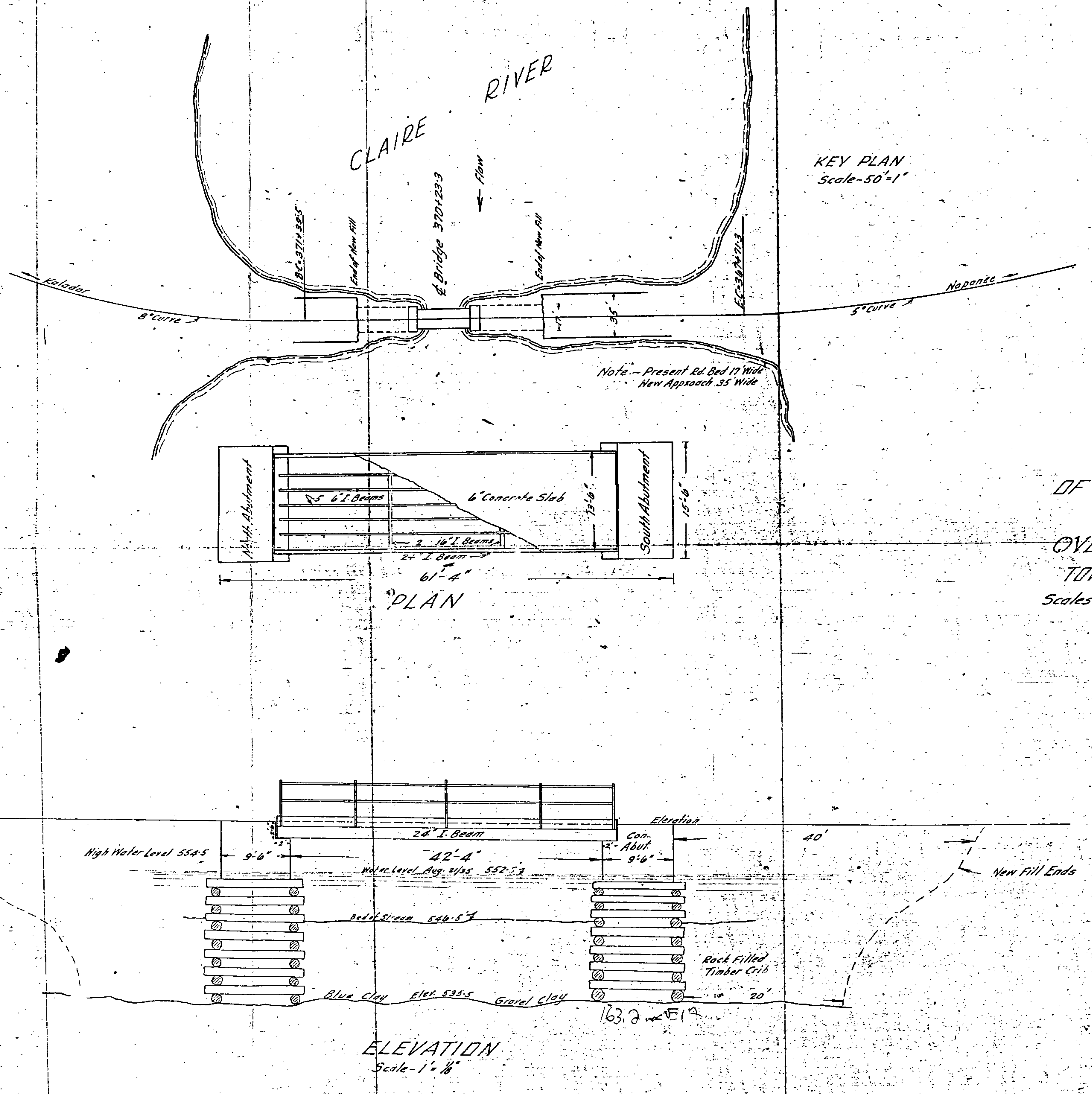
An angle of internal friction of 35° is very high for a sliding failure at the Organic Silt/Rock Fill interface.

For sliding failures occuring in the soil, we normally provide an angle of internal friction rather than a shear strength value, although the OHBDC does include an apparent cohesion as a parameter in calculating the sliding resistance of a spread footing.

Approach Embankments

I do not understand the recommendation to flatten the existing slopes from 2H:1V to 3H:1V. I do not think this is necessary.

T. Sangiuliano, P. Eng.
Foundation Engineer



1629

PLAN
OF LOCATION AND DETAIL OF
EXISTING BRIDGE
OVER CLAIRE RIVER
TOWNSHIP OF SHEFFIELD

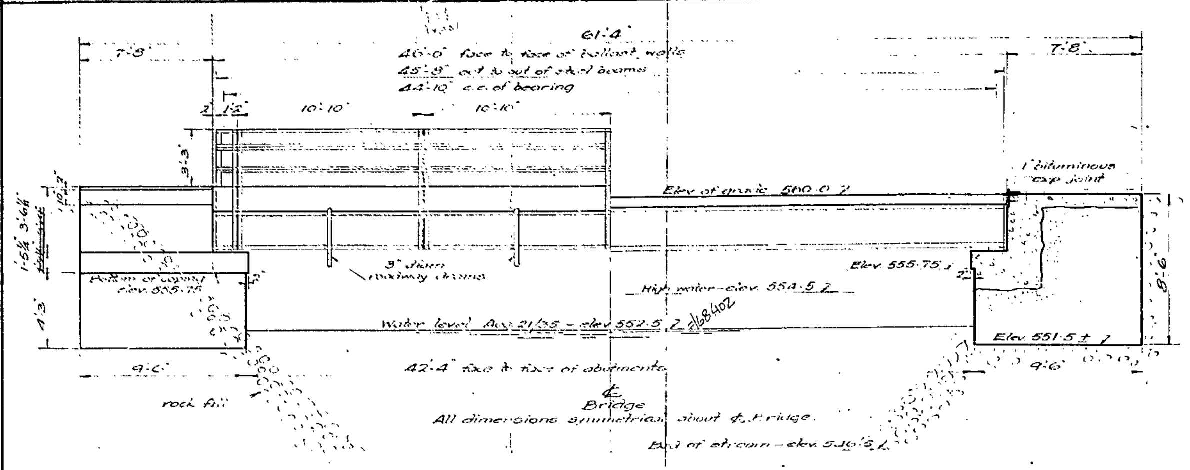
Scales as Shown Brockville, Nov. 4/35.

Notes:- Existing superstructure to be removed.
- Existing rock filled timber cribs to remain.
- Existing concrete abutments to be cut down for new beam seats as shown on drawing D-2366-1.
- Abutments to be extended on new rock fill.
- New rock fill was completed in Aug. 1930.

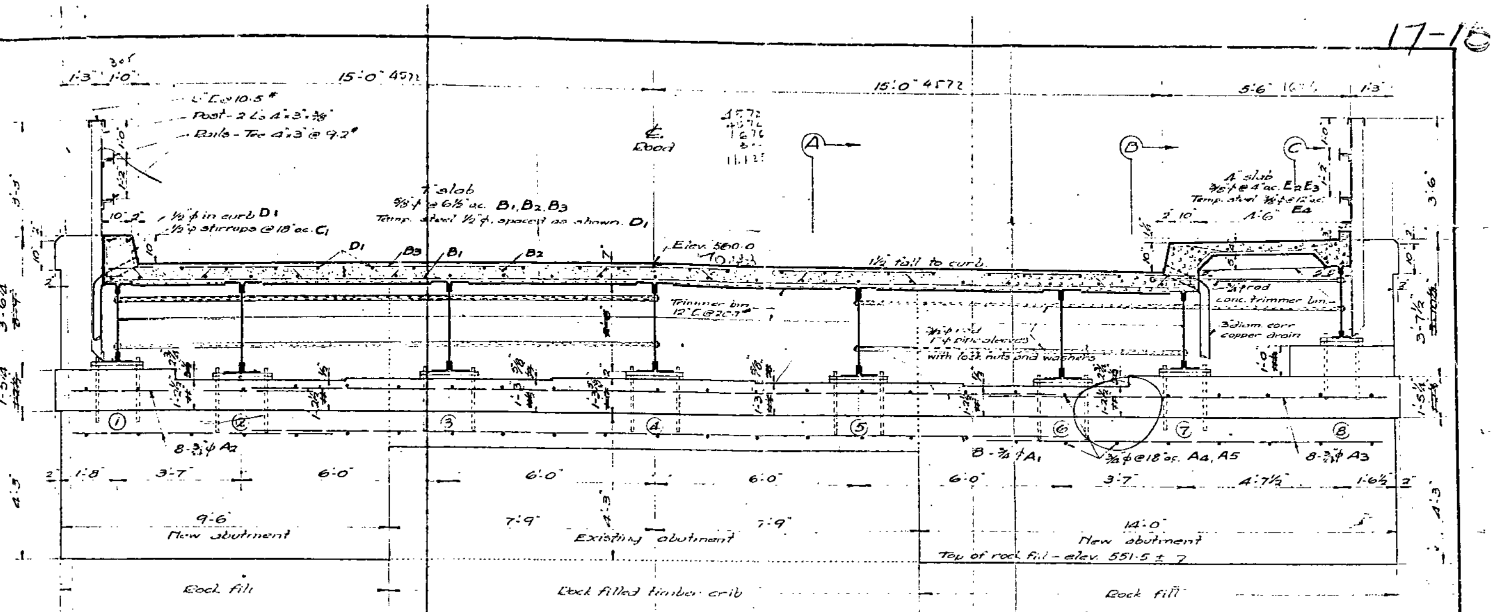
CONTRACT 37-28
DRAWING D-2366-A

17-16

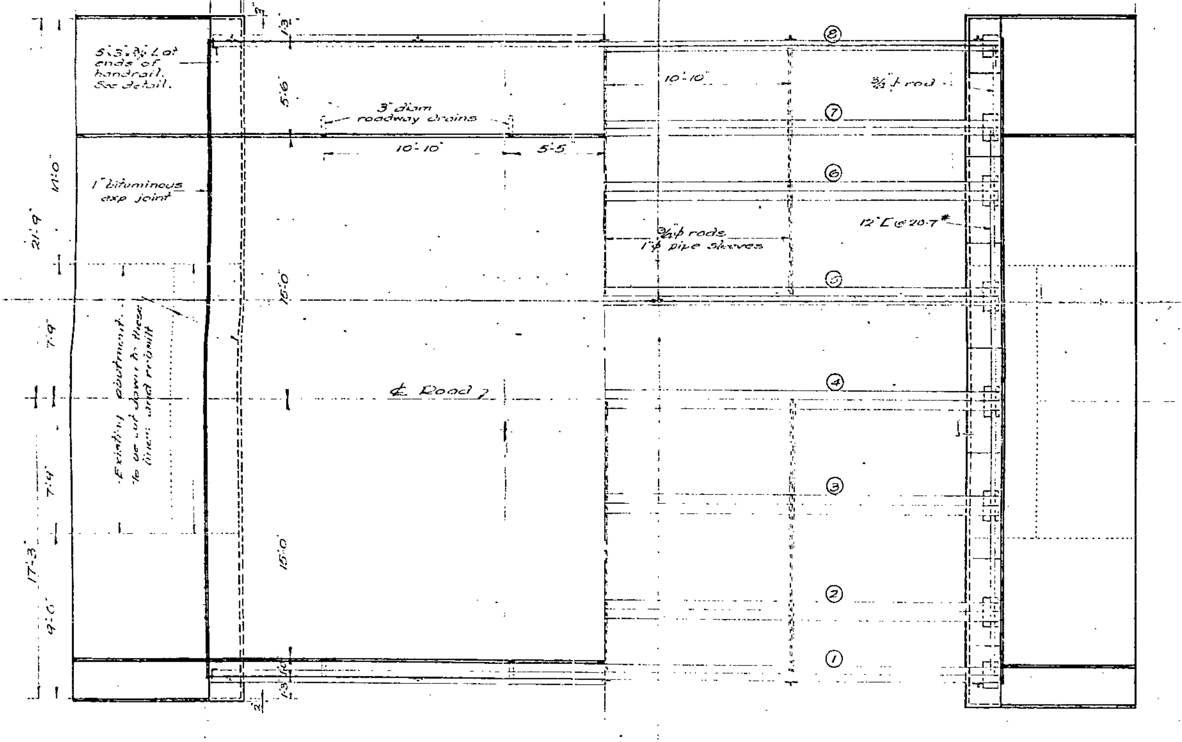
WP 48-94-01



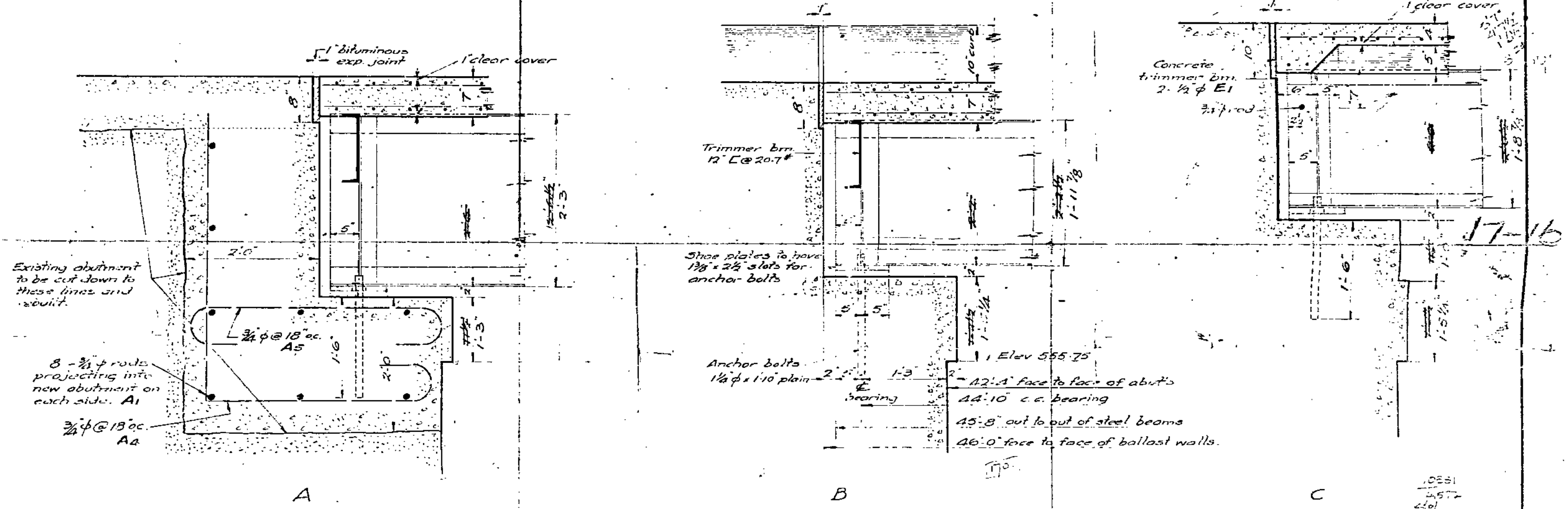
HALF ELEVATION HALF LONG'L SECTION ON ℄ ROAD
Scale ½"=1'-0"



TYPICAL CROSS SECTION AND ELEVATION OF ABUTMENT
Scale ½"=1'-0"

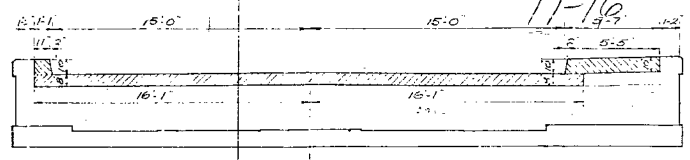


HALF PLAN OF DECK HALF PLAN OF STEEL FRAMING
Scale ⅛"=1'-0"



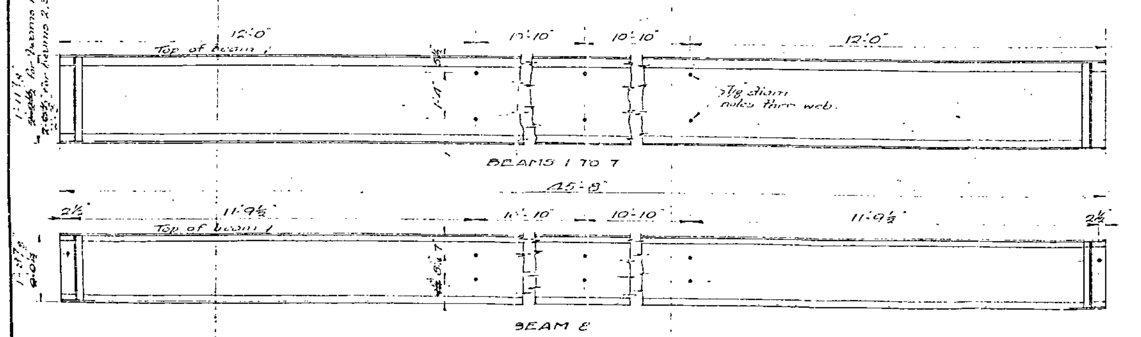
SECTIONS THRO BALLAST WALL
Scale 1"=1'-0"

17-16

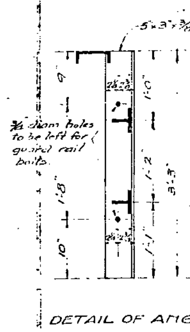


ELEVATION OF BALLAST WALL
SHOWING RECESS FOR BITUMINOUS JOINTING
Scale ¼"=1'-0"

Note:- Steel beams changed to rolled sections. See Steel Company's shop drawings.
-For anchor bolt setting see Steel Company's shop drawings.



DETAIL OF HOLES TO BE LEFT IN STEEL BEAMS FOR SEPARATOR RODS
Scale ½"=1'-0"



DETAIL OF ANGLE AT ENDS OF HANDRAIL
Scale 1"=1'-0"

DETAIL OF STEEL BEAMS

	Beam No.	Section	Stiffeners	Bearing plates	Anchor bolts
24" I @ 74#	1	~~[illegible]~~	~~[illegible]~~	10"×1"×1'-6"	1¼"φ×1'-10" plain
27" I @ 98#	2	~~[illegible]~~	~~[illegible]~~	10"×1"×1'-8"	do.
	3	do.	~~do.~~	do.	do.
	4	do.	~~do.~~	do.	do.
	5	do.	~~do.~~	do.	do.
	6	do.	~~do.~~	do.	do.
24" I @ 74#	7	~~[illegible]~~	~~[illegible]~~	10"×1"×1'-6"	do.
21" I @ 59#	8	~~[illegible]~~	~~[illegible]~~	10"×1"×1'-3"	do.

Notes:- Depth of foundations subject to revision by Engineer.
-Excavations must be cofferdammed so that concrete shall be placed in the dry.
-One man stones to be placed at all breaks between pours in mass concrete.
-Concrete in substructure is not to be poured above the elevation of the bridge seat until superstructure is built.
-Where construction joints are necessary in horizontal members they are to be made truly vertical and are to occur only over the centres of piers or at the centre of a span unless otherwise indicated.
-All exposed corners to be chamfered.
-Structure to be built in accordance with D.H.O. General Specifications for Highway Bridges 1935, and Form No. 9.

-Concrete Mix:- Substructure - 1:2¼:3¾
Superstructure - 1:1½:2½

-All Structural Steel to be Medium Grade.
-All Structural Steel to be given two field coats of paint as per Specifications.

PROPOSED BRIDGE OVER
CLAIRE RIVER
AT STATION 370+23.3

THE KING'S HIGHWAY No 41 NAPANEE TO KALADAR
COUNTY - LENNOX AND ADDINGTON STRUCTURE No
TOWNSHIP - SHEFFIELD LOT CON

APPROVED

BRIDGE ENGINEER CHIEF ENGINEER

DEPARTMENT OF HIGHWAYS, ONTARIO,
TORONTO, MARCH 10, 1937.
DEPUTY MINISTER

Steel beams changed May 25 '37
Reinforcing marks added May 25 '37

CONTRACT No 37-28

DESIGN	A.Y.F.	DRAWING	A.N.T.	STEEL	W.O.
CHECK	J.Y.M.	CHECK	J.M.	CHECK	

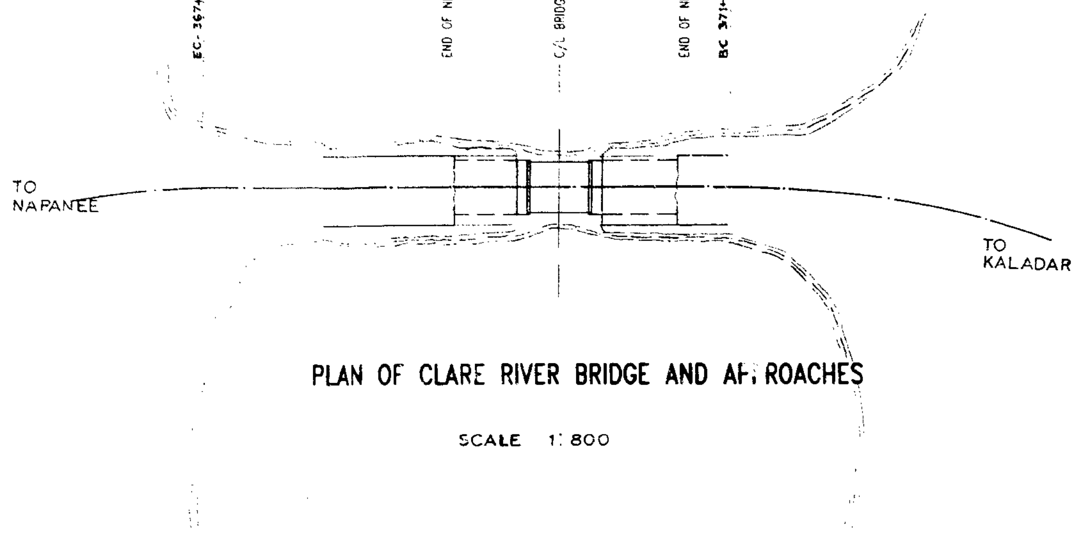
DRAWING No D-2366-1

CONT No 9590-4444-9838
WP No 48-94-01

CLARE RIVER
(Highway 41 and Clare River)
BORE HOLE LOCATIONS & SOIL STRATA

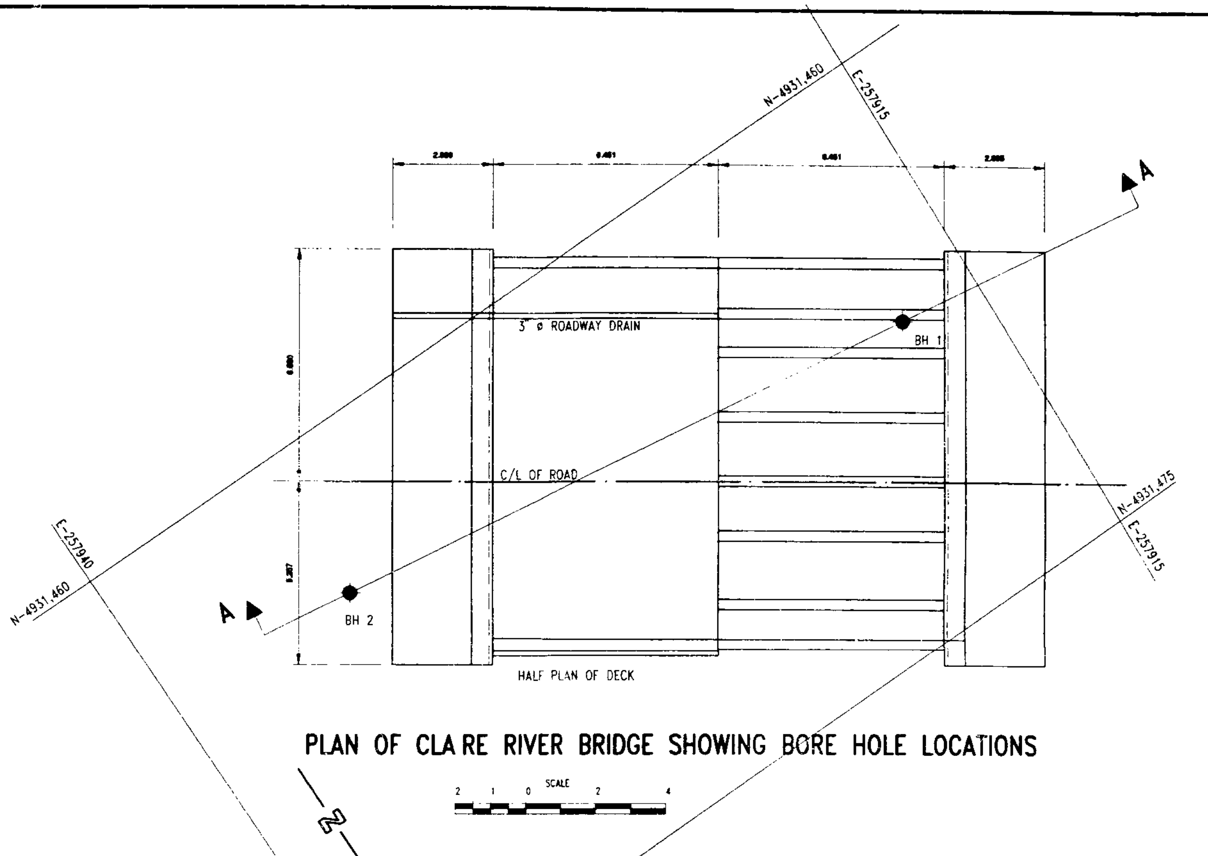
SHEET
1 of 1

CANADA ENGINEERING SERVICES INC.

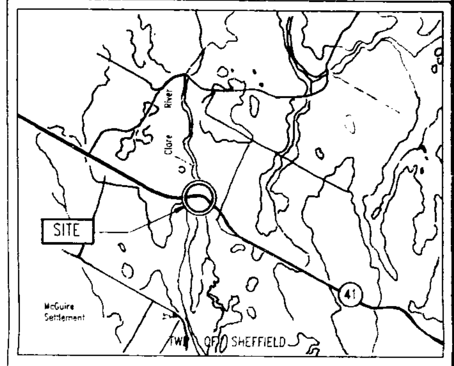


PLAN OF CLARE RIVER BRIDGE AND APPROACHES

SCALE 1: 800



PLAN OF CLARE RIVER BRIDGE SHOWING BORE HOLE LOCATIONS



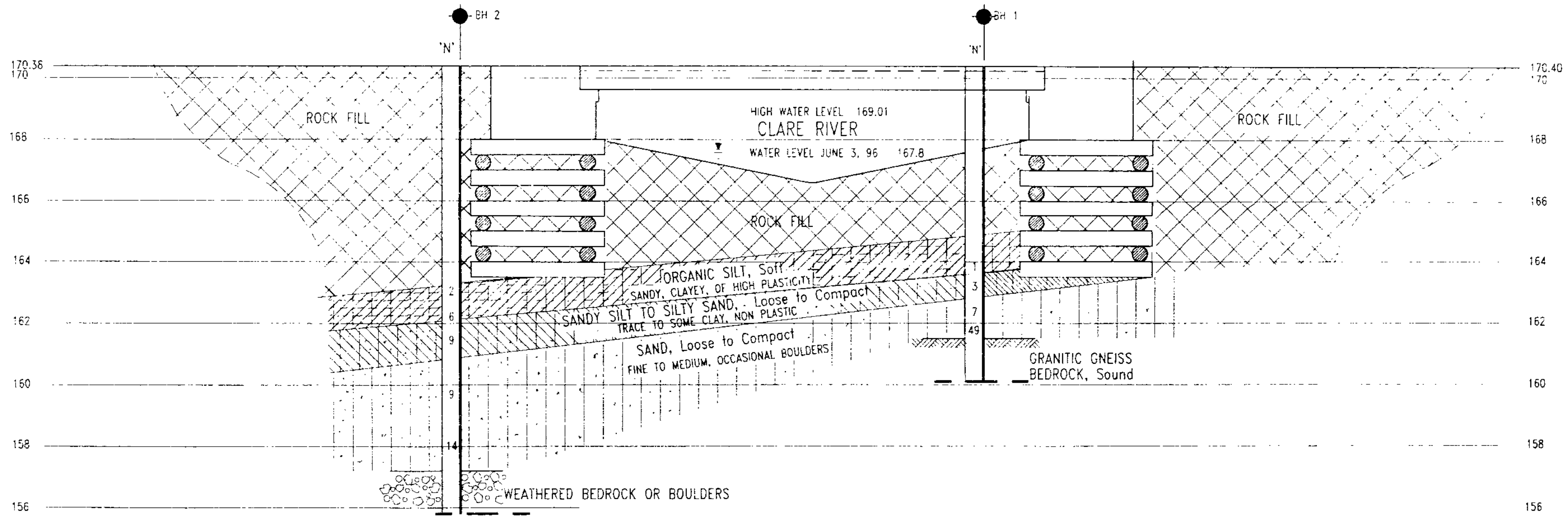
KEY PLAN

LEGEND

● Bore Hole

WL at time of investigation June 3, 1996

No	ELEVATION	CO-ORDINATES NORTH	CO-ORDINATES EAST
1	170.4	4931467.67	257917.045
2	170.38	4931464.6	257933.39



SECTION A-A

SCALE
2 1 0 2 4

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

REVISIONS DATE	BY	DESCRIPTION

GEOCRES No 31C-54

CLARE RIVER BRIDGE SOIL INVESTIGATION			DIST 41
SUBM'D R.J	CHECKED	DATE Sept. 11, 1996	SITE 17 - 16
DRAWN S.K.	CHECKED	APPROVED	DWG 96-309-1

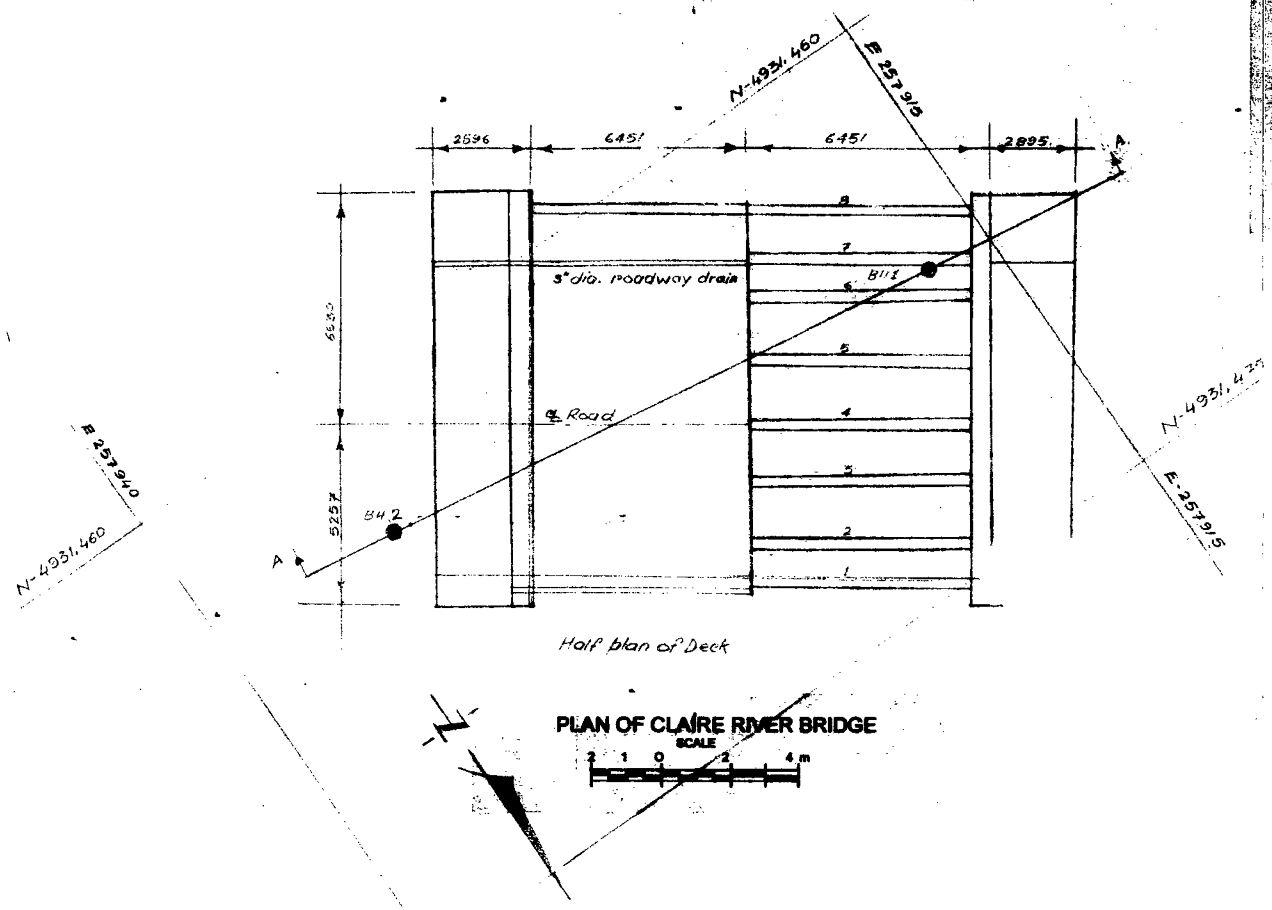
REF No D-2366-1; May 26, 1937

CONT No. [illegible]
WP No. [illegible]

CLAIRE RIVER
(Highway 41 and Claire River)
BOREHOLE LOCATIONS AND SOIL STRATA

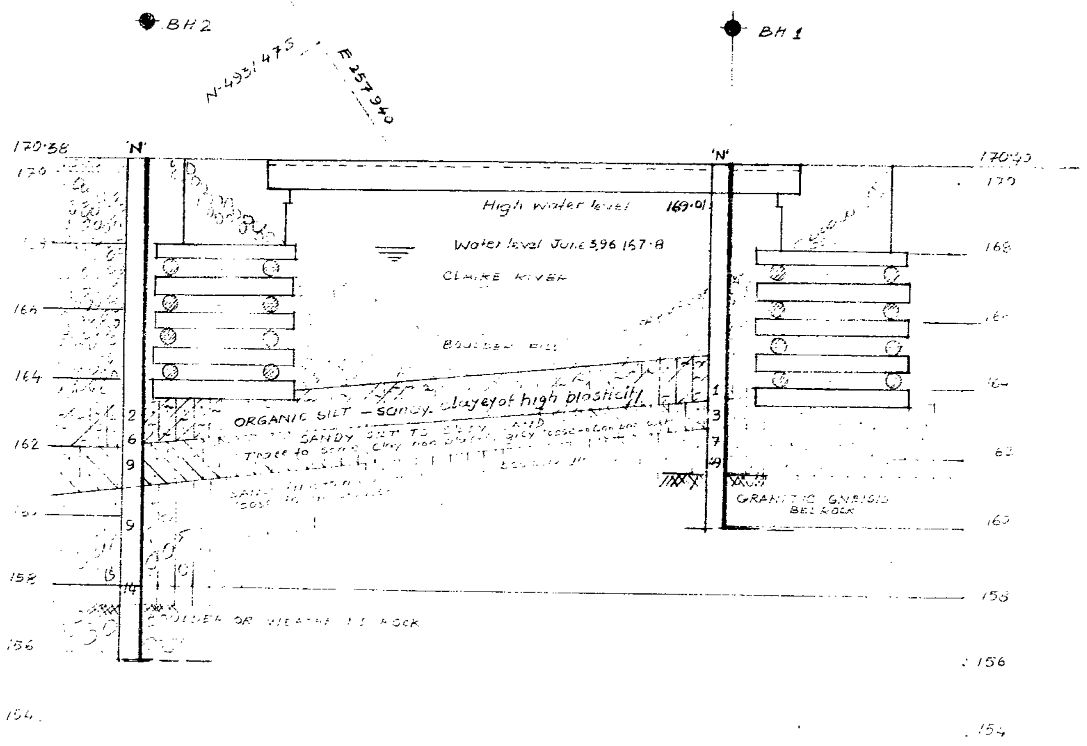
SHEET
1 OF 1

CANADA ENGINEERING SERVICES INC



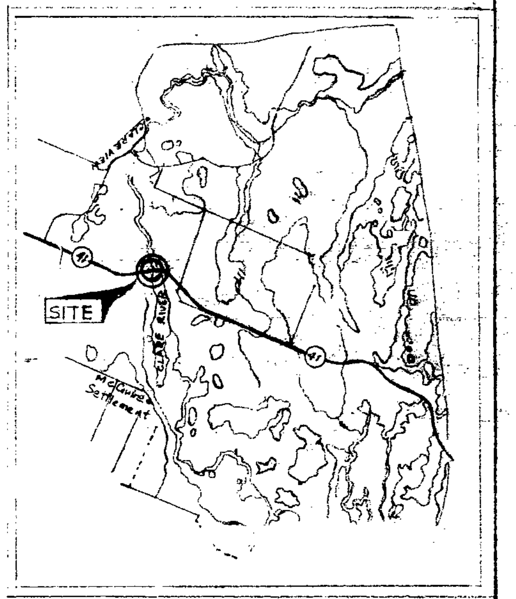
PLAN OF CLAIRE RIVER BRIDGE

SCALE
2 1 0 2 4 m



SECTION AA

SCALE
2 1 0 2 4 m



R. JAGDAT

LEGEND

- ● Bore Hole
- ▼ WL at time of Investigation Jun 3,1996

No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
1	170.4	4931467.67	257917.045
2	170.38	4931464.6	257833.30

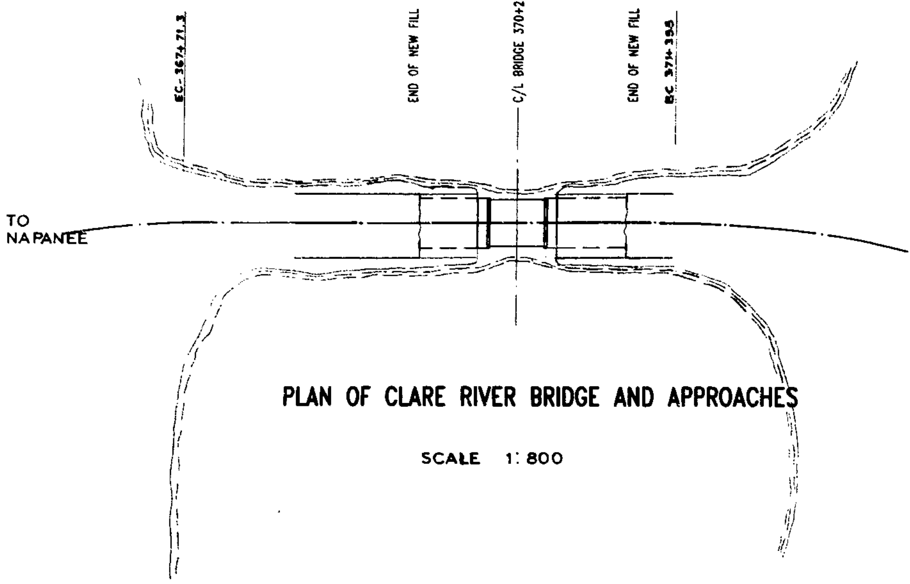
-NOTE-

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

REVISIONS		
DATE	BY	DESCRITION

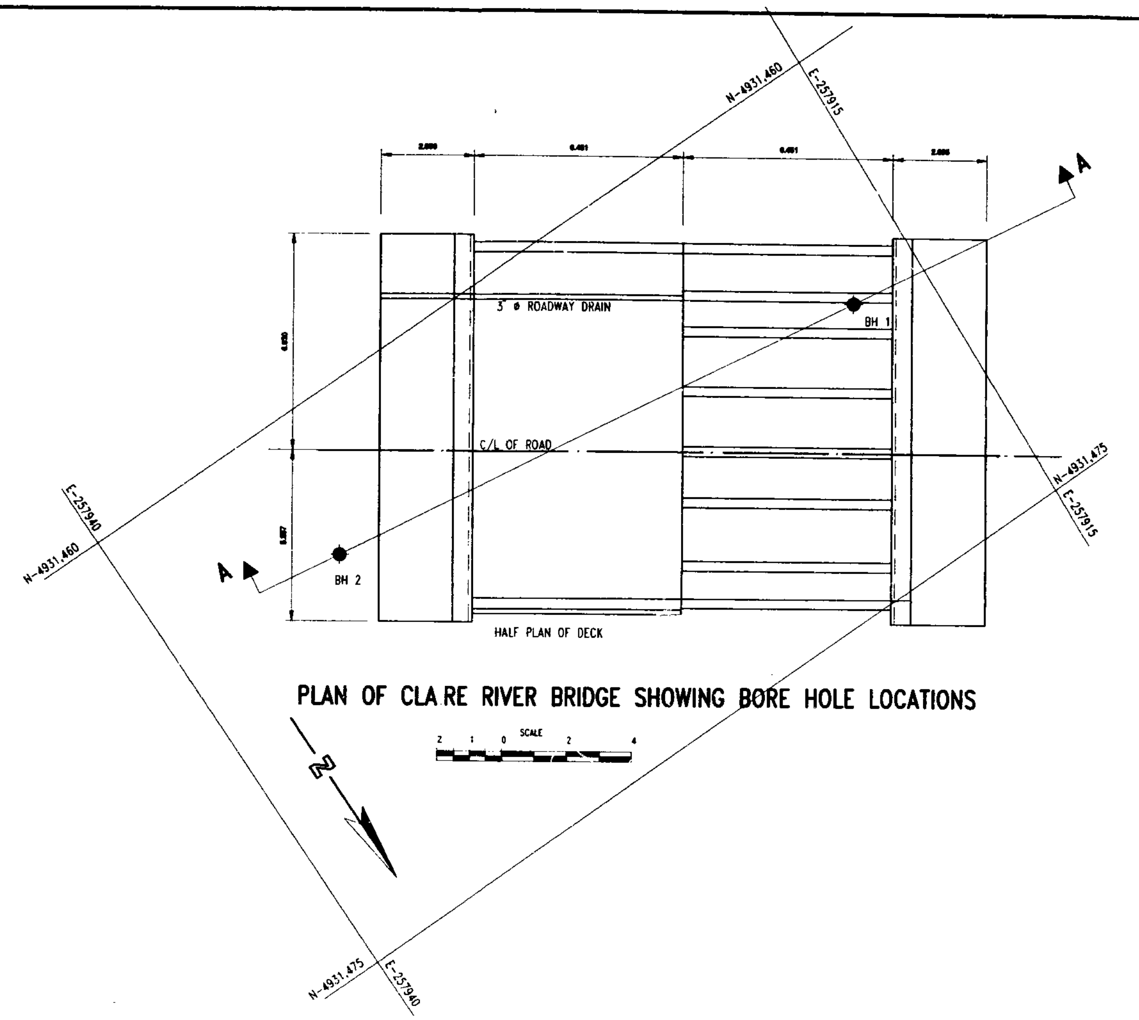
REF No D-2366-1; May 26, 1937

CLAIRE RIVER BRIDGE SOIL INVESTIGATION			DIST 41
SUBM'D RJ	CHECKED	DATE Jul 22, 1996	SITE 17 - 16
DRAWN SK	CHECKED	APPROVED	DWG 96-309-1

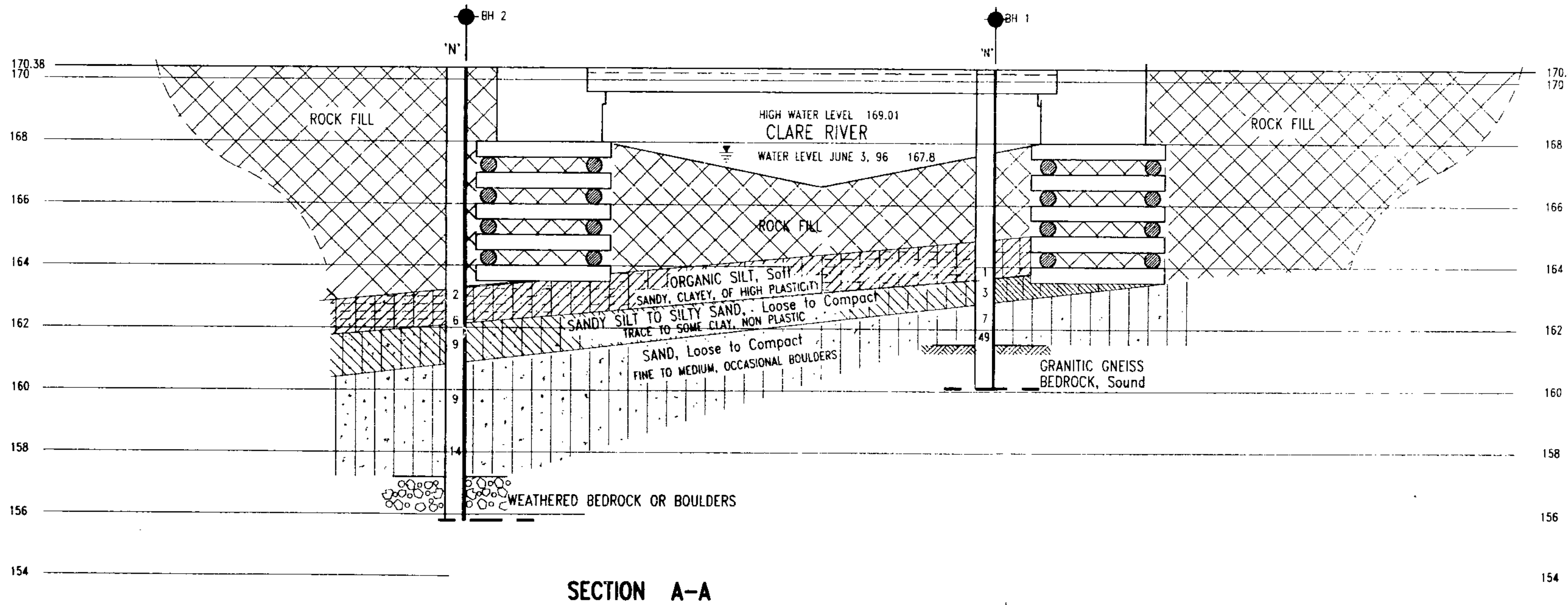


PLAN OF CLARE RIVER BRIDGE AND APPROACHES

SCALE 1: 800



PLAN OF CLARE RIVER BRIDGE SHOWING BORE HOLE LOCATIONS



SECTION A-A

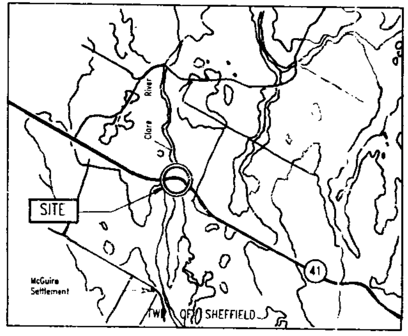
SCALE
2 1 0 2 4

CONT No 9590-4444-9838
WP No 48-94-01

CLARE RIVER
(Highway 41 and Clare River)
BORE HOLE LOCATIONS & SOIL STRATA

SHEET
1 of 1

CANADA ENGINEERING SERVICES INC.



KEY PLAN

LEGEND

● Bore Hole

▼ WL at time of investigation June 3, 1996

No	ELEVATION	CO-ORDINATES NORTH	CO-ORDINATES EAST
1	170.4	4931467.67	257917.045
2	170.38	4931464.6	257933.39

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

REVISIONS DATE	BY	DESCRIPTION

GEOCRES No 31C-154

CLARE RIVER BRIDGE SOIL INVESTIGATION			DIST 41
SUBM'D R.J	CHECKED	DATE Sept. 11, 1996	SITE 17 - 16
DRAWN S.K.	CHECKED	APPROVED	DWG 96-309-1

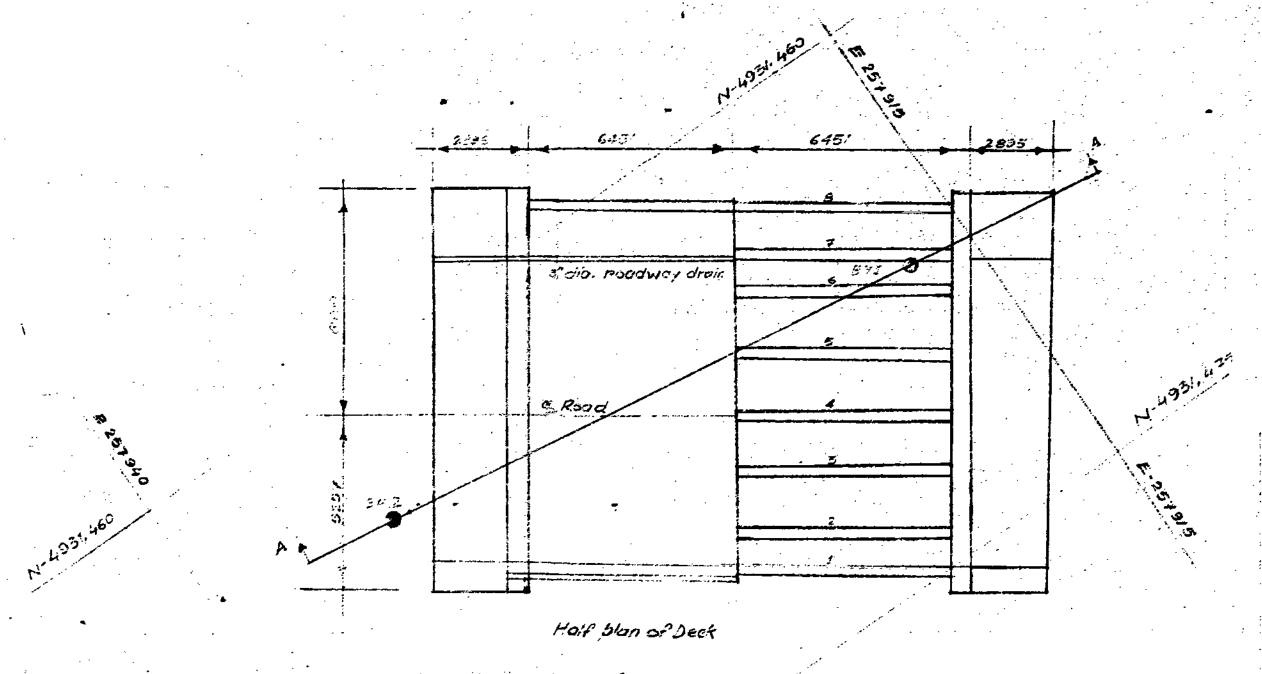
REF No D-2366-1: May 26, 1937

CONT No.	**9590-4444-9838**
WP No.	**48-94-01**

CLAIRE RIVER
(Highway 41 and Claire River)
BOREHOLE LOCATIONS AND SOIL STRATA

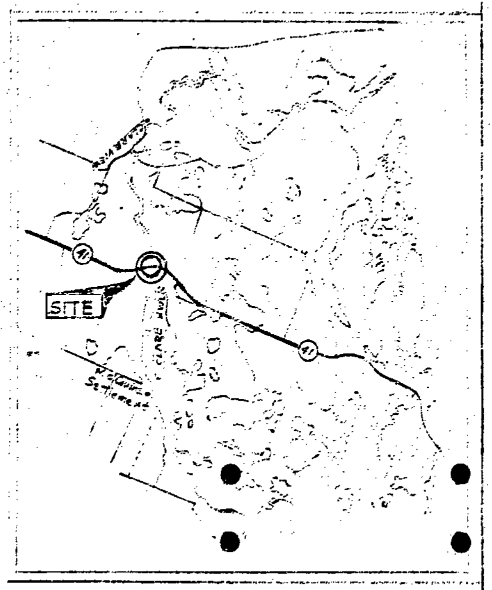
SHEET
1 OF 1

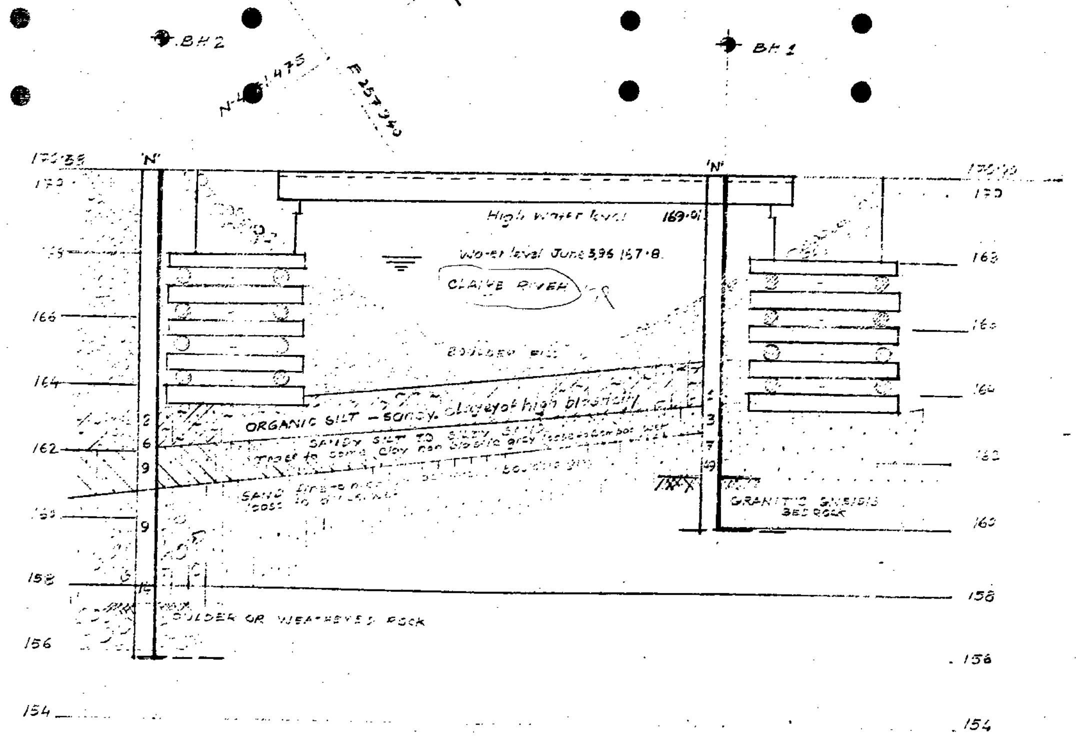
CANADA ENGINEERING SERVICES INC.



PLAN OF CLAIRE RIVER BRIDGE

SCALE
2 1 0 2 4 m





SECTION AA

SCALE
2 1 0 2 4 m

LEGEND

-●- Bore Hole

▼ WL at time of investigation Jun 3,1996

No	ELEVATION	CO-ORDINATES NORTH	CO-ORDINATES EAST
1	170.4	4931467.67	257917.045
2	170.38	4931464.6	257933.39

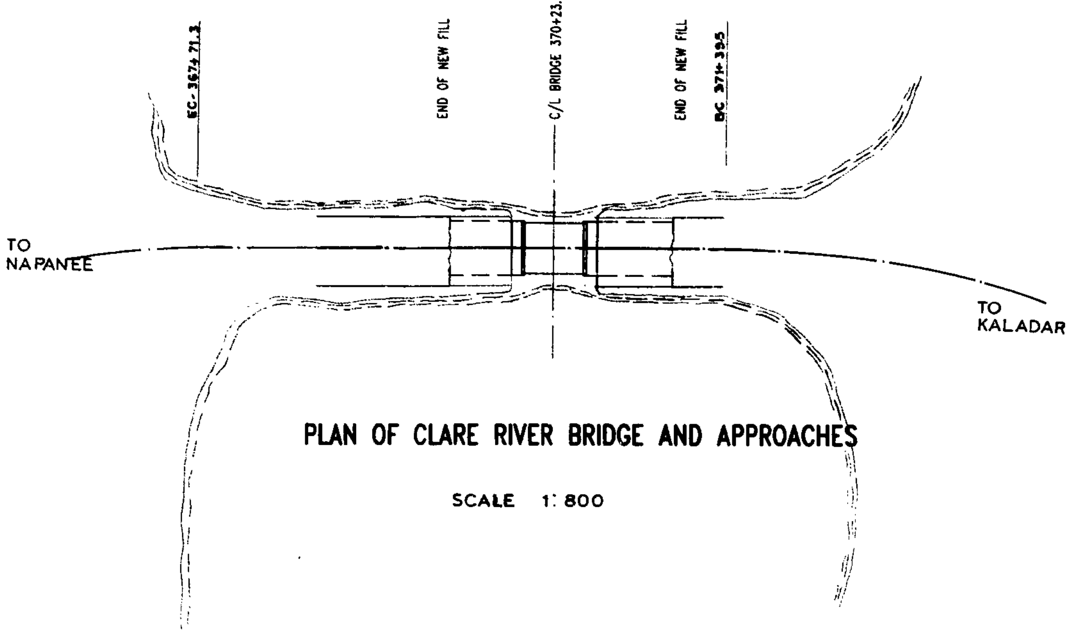
-NOTE-

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

REVISIONS	DATE	BY	DESCRIPTION

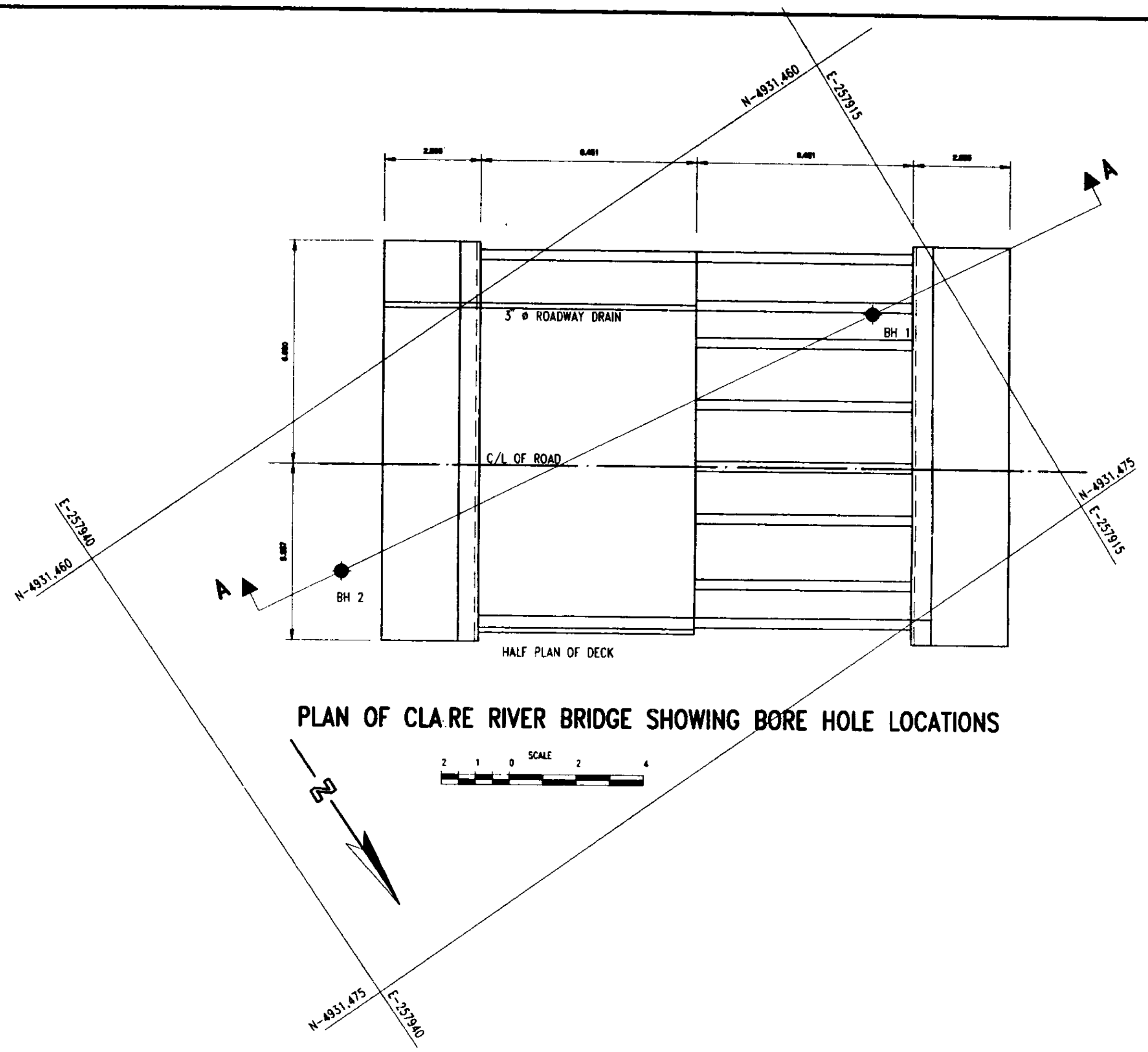
REF No D-2369-1; May 26, 1937

CLAIRE RIVER BRIDGE SOIL INVESTIGATION			DIST 41
SUBM'D RJ	CHECKED	DATE Jul 22, 1996	SITE 17 - 18
DRAWN BK	CHECKED	APPROVED	DWG 96-309-1

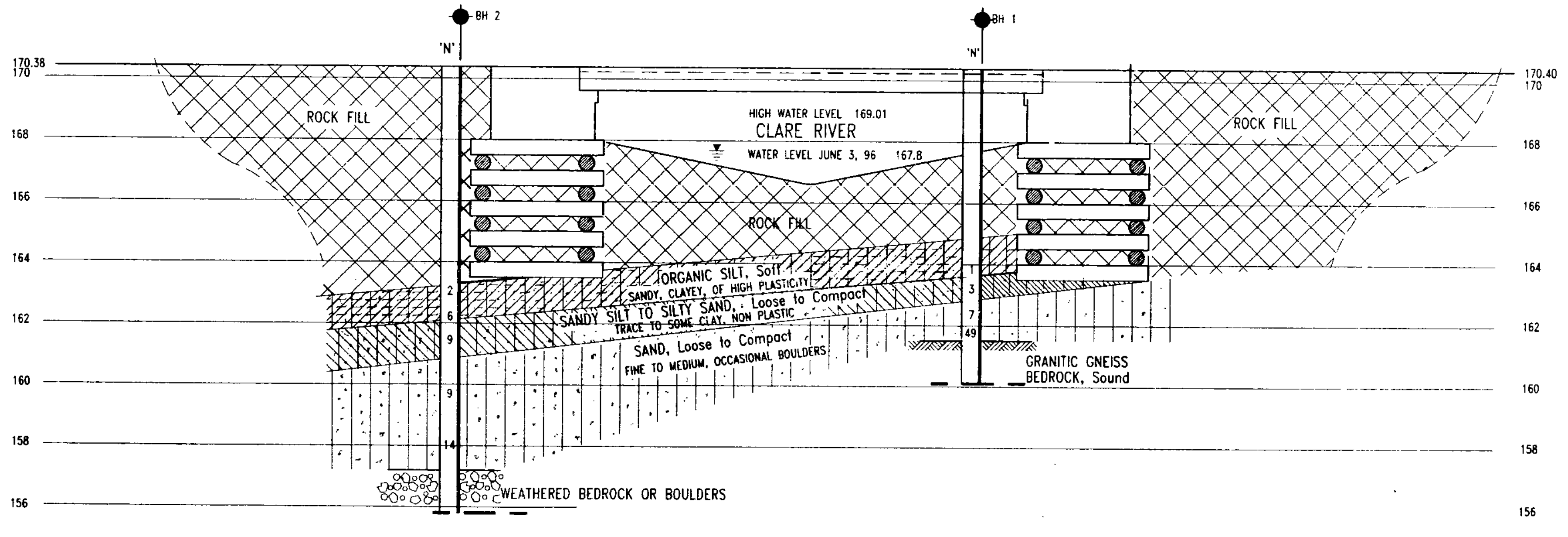


PLAN OF CLARE RIVER BRIDGE AND APPROACHES

SCALE 1: 800



PLAN OF CLARE RIVER BRIDGE SHOWING BORE HOLE LOCATIONS



SECTION A-A

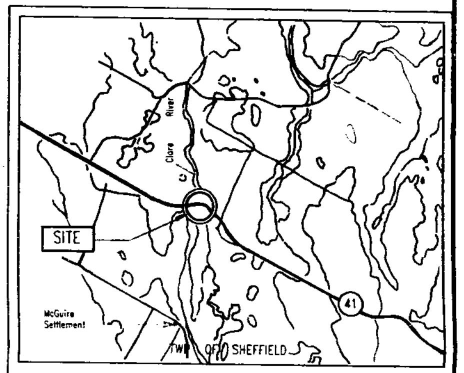
SCALE
2 1 0 2 4

CONT No 9590-4444-9838
WP No 48-94-01

CLARE RIVER
(Highway 41 and Clare River)
BORE HOLE LOCATIONS & SOIL STRATA

SHEET
1 of 1

CANADA ENGINEERING SERVICES INC.



KEY PLAN

LEGEND

● Bore Hole

▽ WL at time of investigation June 3, 1996

No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
1	170.4	4931467.67	257917.045
2	170.38	4931464.6	257933.39

R. JAGDAT

NOTE

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

REVISIONS		
DATE	BY	DESCRIPTION

GEOCRES No 31C-154

CLARE RIVER BRIDGE SOIL INVESTIGATION			DIST 41
SUBM'D R.J	CHECKED	DATE Sept. 11, 1996	SITE 17 - 16
DRAWN S.K.	CHECKED	APPROVED	DWG 96-309-1

REF No D-2366-1; May 26, 1937