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FOUNDATION DESIGN SERVICES
BAILEY BRIDGE REPLACEMENT;
LITTLE WHITE RIVER #2
WP 513-00-01
MINISTRY OF TRANSPORTATION, ONTARIO
DISTRICT 62, ALGOMA

Submitted to:

Northland Engineering (1987) Limited
Consulting Engineers & Planners
Sudbury, Ontario
P3E 3M9

GEOCRES NO. ___-___

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FOR INCLUSION IN FOUNDATION DESIGN SERVICES REPORT ONLY

As confirmed with MTO Pavements and Foundations Section, on January 10, 2004, this report contains the design memoranda as prepared for the three bailey bridge replacement sites which formed part of the Foundation Design requirements for this project. The Technical Memorandum dated January 16, 2004 forms part of the report text and follows page 5 for this report. Reference should be made to the design recommendations relevant to Little White River #2 WP 513-00-01.

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Northland Engineering Limited (Northland) on behalf of the Ministry of Transportation, Ontario (MTO) to provide Foundation Design Services for the replacement of a single lane, single span Bailey bridge carrying Secondary Highway 546 over Little White River located north of Elliott Lake, Ontario.

The terms of reference for the scope of work are outlined in Golder's letter dated October 6, 2003 and titled, "Foundation Design Services, Bailey Bridge Replacement; Dog Lake Narrows, Little White River I and II" for this project. The work was carried out in accordance with the Quality Control Plan for this project dated October 2003. A digital file of the Preliminary General Arrangement drawing showing the bridge configuration at Little White River and Secondary Highway 546 was provided to Golder by Northland by email on November 25, 2003.

2.0 SITE DESCRIPTION

The Little White River #2 site is located at the existing temporary bridge structure carrying Secondary Highway 546 over the Little White River located north of Elliot Lake, Ontario. The site is approximately 2.5 km southwest of the intersection of Highway 546 and Highway 639 (see Figure 1).

The existing bridge is a single lane, single span Bailey bridge with abutments supported on bearing plates founded on timber cribs located immediately adjacent to the edges of the river. The approach embankments are approximately 3 m high, with front sides slopes (adjacent to the timber cribs) inclined at about 31º to 36º to the horizontal.

The terrain is generally flat lying on the southwest side of the Little White River. On the northeast side, the terrain slopes gently downwards towards the river. The site consists of bush areas, forests areas, rock outcrops at ground surface, shoreline wetlands, and floodplain swamps. The ground surface within the limits of the replacement Bailey bridge and approach embankment area general lies between Elevation 100.5 m and 101 m, referenced to Local Datum.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The field work for the Little White River #2 Bailey bridge replacement was carried out on October 31, 2003 and November 1, 2003 during which time one (1) borehole, numbered LW2-1 and one (1) Dynamic Cone Penetrating Test (DCPT), numbered LW2-2 were put down at the site. The locations of the borehole and DCPT are shown in plan on Drawing 1.

The field investigation was carried out using a track-mounted CME 55 drill rig supplied and operated by Marathon Drilling Co. Ltd. of Ottawa, Ontario. The borehole was initially advanced using 108 mm inside diameter (I.D.) continuous flight hollow stem augers until refusal was reach on cobbles at a depth of about 1.8 m. Beyond this depth, the borehole was advanced using 'N'-casing and 'NQ'-size core barrel to 9.8 m. A Dynamic Cone Penetration Test (DCPT) was then carried out (through the bottom of the borehole) to a depth of 30.2 m at which point the drilling and testing was terminated on refusal to further cone advancement.

Soil samples were obtained initially at intervals of about 0.75 m to a depth of about 2.1 m followed by intervals of about 1.5 m, where possible, using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures. Between the SPT samples below 2.1 m depth, the stratigraphy was inferred from cuttings and wash samples as the N-casing and 'NQ' core barrel were advanced.

The field work was supervised throughout by a member of our technical staff, who located the borehole and DCPT, arranged for the clearance of underground service locations, supervised the drilling, sampling and testing operations, logged the boreholes, and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Mississauga geotechnical laboratory where the samples underwent further detailed visual examination.

On completion of the fieldwork, the surveying of the ground surface elevation at the locations of the borehole and DCPT was carried out by members of our technical staff and referenced to local benchmark elevations provided by Northland. The northing and easting coordinates for the borehole and DCPT were provided by Northland. The borehole location and ground surface elevations are shown on the Record of Borehole sheets and on Drawing 1.

4.0 GENERAL SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Geology

From published geologic information, the site is located in the physiographic region known as the Penokean Hills which is a subdivision of the Abitibi Uplands that form the central section of the Canadian Shield (Geology of Ontario; OGS Special Volume 4). The Abitibi Uplands form a rocky landscape, scattered with lakes and large areas which are mantled by deposits from Pleistocene glaciation consisting of the lacustrine clays and former shorelines of proglacial lakes. Landforms include outwash channels, tills and moraines. The local physiography is generally characterized by variable overburden materials including sand, gravel, cobbles and boulders and an irregular, variable bedrock surface with rock outcrops.

4.2 Subsoil Conditions

The detailed subsurface soil and groundwater conditions as encountered in the borehole advanced during this investigation are given on the attached Record of Borehole sheets following the text of this report. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests (SPTs). These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations.

In general, the subsoils at the site consist of embankment fill underlain by a loose to compact sand and gravel deposit containing trace silt and occasional cobbles. The embankment fill is composed of a loose silty sand, some gravel pavement structure overlying sand and gravel containing trace silt and cobbles. The DCPTs in borehole LW2-1 and LW2-2 were advanced to depths of 30.2 m and 6.4 m, respectively and bedrock was not encountered during drilling at this site. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Embankment Fill

Embankment fill was encountered at the existing ground surface in Borehole LW2-1. The embankment fill layer is about 3.0 m thick with the ground surface at about Elevation 100.7 m. The upper 1.4 m of the embankment fill/pavement structure consists of silty sand, some gravel, trace rootlets and occasional cobbles. The lower 1.6 m of the embankment fill consists of sand and gravel, trace silt with occasional cobbles.

The measured Standard Penetration Testing (SPT) 'N' values were between 5 and 19 blows per 0.3 m of penetration, indicating a loose to compact state of packing.

4.2.2 Sand and Gravel

A deposit of sand and gravel containing trace silt and occasional cobbles was encountered below the embankment fill. The surface of the sand and gravel deposit was encountered at about Elevation 97.7 m and the deposit was penetrated by drilling and sampling for about 6.8 m. The drilling and sampling operations were terminated within the sand and gravel at about Elevation 91.0 m and refusal to DCPT advance was achieved at about Elevation 70.5 m. In LW2-2 the refusal to DCPT advance was met at about Elevation 94.2.

The measured Standard Penetration Testing (SPT) 'N' values ranged from 7 to 22 blows per 0.3 m of penetration, indicating a loose to compact relative density.

4.2.3 Groundwater Conditions

In general, the samples taken in the overburden boreholes were noted to be moist becoming wet at depth. The water level in the open borehole was not noted at the time of drilling since the borehole was advanced using wash rotary drilling methods. However, it is expected that the groundwater level will be similar to the water level in the adjacent Little White River. On October 29, 2003 the river level was noted to be at about Elevation 97.4 m. It should be noted that groundwater levels in the area are subject to seasonal fluctuations.

GOLDER ASSOCIATES LTD.

J. Paul Dittrich, P.Eng.,
Associate

Fintan J. Heffernan, P.Eng.,
Designated MTO Contact

KG/JPD/ASP/FJH/sm
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TECHNICAL MEMORANDUM

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TO:	Northland Engineering (1987) Limited	**DATE:**	January 16, 2004
ATTN :	Mr. Stephen Ho, P.Eng.	**FAX:**	705-674-5583
FROM:	Paul Dittrich/Anne Poschmann/Fin Heffernan	**JOB NO:**	03-1111-029
EMAIL:		**# of PAGES:**	10

ABUTMENT FOUNDATION DESIGN RECOMMENDATIONS
BAILEY BRIDGE REPLACEMENT
LITTLE WHITE RIVER #1 AND #2, DOG LAKE NARROWS
WP 512-00-01, WP 513-00-01, WP 137-97-01

INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Northland Engineering Limited (Northland) to provide Foundation Design Services for the replacement of three single lane, single span Bailey bridges located on secondary highways in Northwestern Region for the Ministry of Transportation, Ontario (MTO). This memo summarizes the results of the field investigation and provides foundation design recommendations for the replacement bridge abutments. Individual Foundation Investigation Reports presenting the factual information for each of the three sites will be issued separately.

BACKGROUND AND EXISTING INFORMATION

Two of the bridges carry Highway 546 over Little White River located north of Elliot Lake, Ontario. The third bridge carries Highway 651 over Dog Lake Narrows located north of Wawa, Ontario. The following summarizes the existing conditions at the sites based on the drawings and information provided to us by Northland.

At the Little White River #1 site, the existing bridge abutments are founded on bearing plates resting directly on the approach embankments. The existing approach embankments are approximately 4 m high, with front slopes (facing the river) inclined at approximately 25^{o} to the horizontal (i.e. a profile of about 2H (horizontal):1V (vertical)). It is our understanding that at this site, the replacement bridge is to be located on a new road alignment; the maximum high water level is Elevation 98.36 m.

At the Little White River #2 site, the existing bridge abutments are supported on bearing plates founded on timber cribs that are located at the ends of the approach embankments immediately adjacent to the edge of the river. The existing approach embankments are approximately 3 m high, with front slopes (adjacent to timber cribs) inclined at approximately 31^o to 36^o to the horizontal (i.e. a profile of about 1.5H:1V). It is our understanding that at this site, the replacement bridge is to be located on the same alignment; the maximum high water level is Elevation 98.30 m.

At the Dog Lake Narrows site, the south abutment of the existing bridge is supported on bearing plates founded on a timber crib, the north abutment is founded on bearing plates resting directly on the approach embankment. The existing approach embankments are approximately 4 m to 7 m high (about 4 m above the lake water level), with the front slopes of the north approach embankment inclined at approximately 33^o to the horizontal (i.e. a profile of about 1.5H:1V). It is our understanding that at this site, the replacement bridge is to be located on the same alignment; the maximum high water level is Elevation 331.0 m.

FIELD INVESTIGATION AND SUBSURFACE CONDITIONS

The field investigation for the project was carried out between October 22 and November 1, 2003 during which time a total of six boreholes and three Dynamic Cone Penetration Tests (DCPTs) were completed at the three sites. The boreholes were advanced to depths ranging from about 5.5 m to 21.3 m and the DCPTs were carried out to depths ranging from about 6.2 m to 30.2 m.

The subsurface conditions at the two Little White River sites are similar and generally consist of loose to compact embankment fill (comprised of sand and gravel with cobbles and/or boulders) overlying loose to compact sand and gravel, trace silt and occasional cobbles. Bedrock was not encountered during drilling at these sites. At the Dog Lake Narrows site, the subsoils consist of loose sand and gravel overlying rock fill over bedrock encountered at a depth of about 5.5 m to 7 m.

FOUNDATION RECOMMENDATIONS

Based on the results of the field investigation and considering the information provided by Northland regarding the maximum high water level (HWL), length and width of replacement bridges and the factored bridge end reaction forces at the abutments, the following foundation design recommendations are provided.

It is our understanding that the new Bailey bridges will be approximately 4.12 m wide and are to be supported on shallow timber cribbing at each site. In order to provide sufficient space for the bearing plates within the cribbing, we understand that timber cribs are required to be about 6 m in length.

Axial Geotechnical Resistance

The axial geotechnical resistances for the abutment timber cribs at the Little White River and Dog Lake Narrows sites have been calculated using the method proposed by Meyerhof (1957) for assessing the ultimate bearing capacity of foundations located at or behind the crest of slopes.

Figure 1 and 2 shows the factored geotechnical resistance at Ultimate Limit States (ULS) that may be used for design of the abutment timber cribs for the Little White River #1 and #2 bridges, respectively. Figure 3 shows the factored geotechnical resistance at Ultimate Limit States (ULS) that may be used for design of the abutment timber cribs for the Dog Lake Narrows bridge. When utilizing these resistances, the following assumptions made in the analysis should be noted:

- Cribs are 2.5 m deep below final ground surface/top of approach embankment (D_{crib} = 2.5 m).
- Cribs are 6 m long (L_{crib} = 6 m).
- Cribs are either 2 m or 2.5 m wide as indicated on the plots.
- Cribs are located either at the crest of the front slope of the approaches (i.e. zero set back) or are set back a distance of 0.5 m, 1 m or 2 m from the crest of the approach embankment front slope as is indicated on the plots.
- Front slopes of the approach embankments (i.e. slopes facing the river) are inclined at the angles (β) indicated on Figure 1, 2 and 3 which approximately correspond to those existing in the field based on the information provided by Northland.

The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement for the Little White River and Dog Lake Narrows sites are estimated to be equal to or greater than the ULS values presented on Figure 1, 2 and 3 considering the composition of the existing approach embankments and underlying native soils or bedrock. For design, it is recommended that the SLS value be assumed to be equal to the ULS value.

Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the base of the timber cribs and the approach embankment fill should be calculated in accordance with Section 6.7.5 of the CHBDC. The following table lists the angle and corresponding coefficient of friction that should be assumed for the different embankment fills.

Embankment Fill (Site)	Angle of Friction (^{o})	Coefficient of Friction (^{o})
Sand and Gravel (Little White River #2)	35	0.70
Rock Fill (Little White River #1 and Dog Lake Narrows)	38	0.78

The above values represent unfactored values; in accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance.

Frost Protection

The timber cribs should be provided with a minimum of 1.8 m and 2.5 m of soil cover for frost protection at the Little White River sites and Dog Lake Narrow site, respectively.

Lateral Earth Pressures for Design

The lateral earth pressures acting on the timber cribs will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the cribs.

The following recommendations are made concerning the design of the cribs. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground surface behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope. It should also be noted that recommendations are only provided regarding the active pressures that will be acting on the cribs. The passive earth pressures acting on the cribs by the front slopes of the embankments should not be relied on considering the potential for erosion and possible frost susceptibility in these areas.

- Free draining granular material with less than 5 percent passing the 200 sieve and with maximum particle size of 0.3 m may be used as backfill behind the cribs. This fill should be placed and compacted in accordance with the latest MTO OPSS specification.

- Where the adjacent existing embankment fill is composed of rock fill, provision must be made to have a separator (either a geotextile or a graded granular filter) between the dissimilar fills to avoid loss of fines from the new crib wall backfill into existing rock fill.

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the crib, in accordance with the CHBDC Section 6.9.3 and Figure 6.9.3. Compaction equipment should be used in accordance with OPSS 501.06. Other surcharge loadings should be accounted for in the design, as required.

- Since the structure is unrestrained, active earth pressures may be used in the geotechnical design of the structure in accordance with the following (unfactored) parameters :

Soil unit weight :	20 kN/m3
Active, K_a	0.27

Erosion Protection

Erosion protection (i.e. rip-rap) of a suitable size and thickness should be placed on the front slopes of the approaches in order to protect the approach embankment and abutment crib foundations from undermining/erosion by the river water flow.

As part of the rip-rap design and installation, provision should also be made to ensure that measures are adopted to protect the loss of fines material from the underlying approach embankment fill through the erosion protection.

Timber Crib Backfill

It is recommended that the abutment cribs be backfilled using the following materials that have been selected (depending on the type of adjacent embankment fill) to produce a 'filtered' system to minimize the potential for post-construction settlements of the bearing plates due to loss of overlying material into the embankment fill:

<u>For Rock Fill Embankments (Little White River #1 and Dog Lake Narrows Sites) :</u>

Depth Below Top of Crib (m)	**Type of Backfill**
0 – 0.3	Granular B Type II
0.3 – 1.3	Crushed Rock – 0.05m to 0.10 m sizes
1.3 – bottom of crib	Rock Fill – maximum 0.3 m sizes

<u>For Earth Fill Embankments (Little White River #2 Site) :</u>

Depth Below Top of Crib (m)	**Type of Backfill**
0 – bottom of crib	Granular B Type II

The rock fill used for backfilling should be in accordance with the latest MTO OPSS specification. The upper 0.6 m layer of rock fill (with sizes less than 0.3 m) should be well chinked during placement in the crib in order to reduce the potential for loss of Granular B Type II into the underlying fill. An allowance should be made in the Contract to account for the loss of some of the Granular B Type II (by migration into the underlying fill) during construction.

Little White River #1 Approach Embankment Design

It is our understanding that the replacement bridge at the Little White River #1 site is to be located on a new alignment which will require the construction of a new approach embankment leading up to the northeast abutment. The new approach embankment could require fill up to about 4 m high.

Based on the results of the investigation carried out at this site, the embankment subgrade soils will likely consist of sand and gravel with occasional cobbles. All topsoil, vegetation/shrubs, organic matter and softened/loosened soils should be stripped from below the approach embankment area, and all subgrade soils should be inspected and/or proof-rolled prior to fill placement. It should be noted that numerous large boulders are present at the ground surface in the area of the proposed new approach embankment alignment. The presence of the boulders and the effect they may have on the access of equipment and ability to remove vegetation in this area should be identified to Contractors bidding on the works.

The embankment side slopes should be constructed with either a 2H:1V profile (for earth fill) or 1.5H:1V profile (for rock fill) in order to achieve a minimum Factor of Safety (FoS) of 1.3 against a deep-seated, global failure surface that would impact the operation of the roadway.

Based on our observations during the field investigation the existing Highway 546 embankments in the area of the bridge are composed of sand and gravel fill with cobbles and boulders. If the new approach embankment is constructed with similar, well compacted earth fill, the post-construction settlements within the fill embankment itself (and the timber crib founded on the new fill) will be minimal. It is estimated that the settlement of the new earth fill embankment (due to compression of the native subsoils) would be on the order of about 10 mm, provided that all deposits of topsoil and organic material are removed prior to new fill placement. It is expected that the majority of this settlement would occur during the construction period.

If rock fill is used in the new construction, some post-construction settlement of the embankment itself (and the timber crib founded on the new rock fill) will occur following the completion of construction. Assuming that the rock fill is not end dumped in its final position and is placed in accordance with the requirements as outlined in Special Provision, Amendment to OPSS 206 dated September 1999, the settlement of the newly placed 4 m high rock fill is expected to be approximately 40 mm in the approach area. It is anticipated that the majority (approximately 60%) of this settlement will occur in the first year following construction. It is also estimated that the timber crib could experience settlement on the order of about 15 mm (assuming a crib depth of 2.5 m within the 4 m high approach embankment), with approximately 10 mm of this occurring within the first year following construction.

In addition, at the location of the tie-in between the existing and new embankments, a provision must be made to have a separator (either a geotextile filter or a graded granular filter) between the dissimilar fills to avoid the loss of fines from the existing sand and gravel fill into the new rock fill.

We trust that the above is sufficient for your immediate requirements. If you have any questions, please do not hesitate to contact us.

JPD/ASP/FJH/jpd

Attachments : Figure 1 – Abutment Foundation Design – Little White River #1
Figure 2 – Abutment Foundation Design – Little White River #2
Figure 3 – Abutment Foundation Design – Dog Lake Narrows

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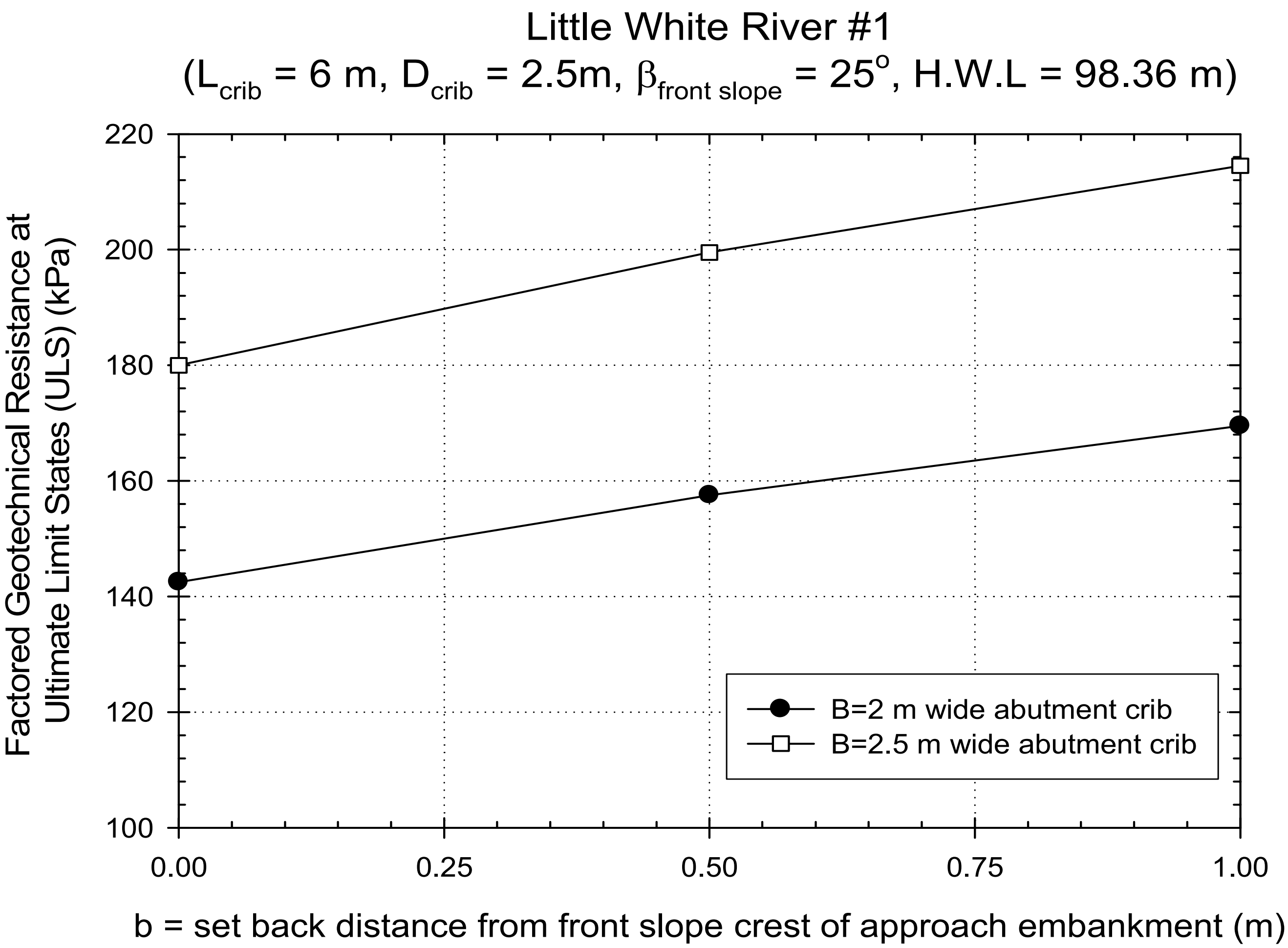
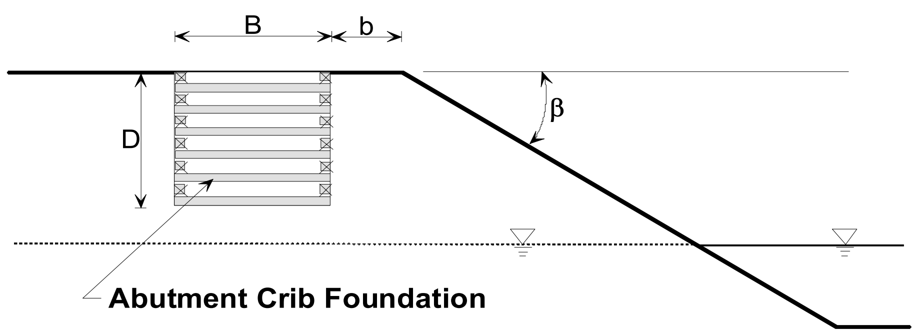
ABUTMENT FOUNDATION DESIGN
BAILEY BRIDGE REPLACEMENT
LITTLE WHITE RIVER I

Figure 1

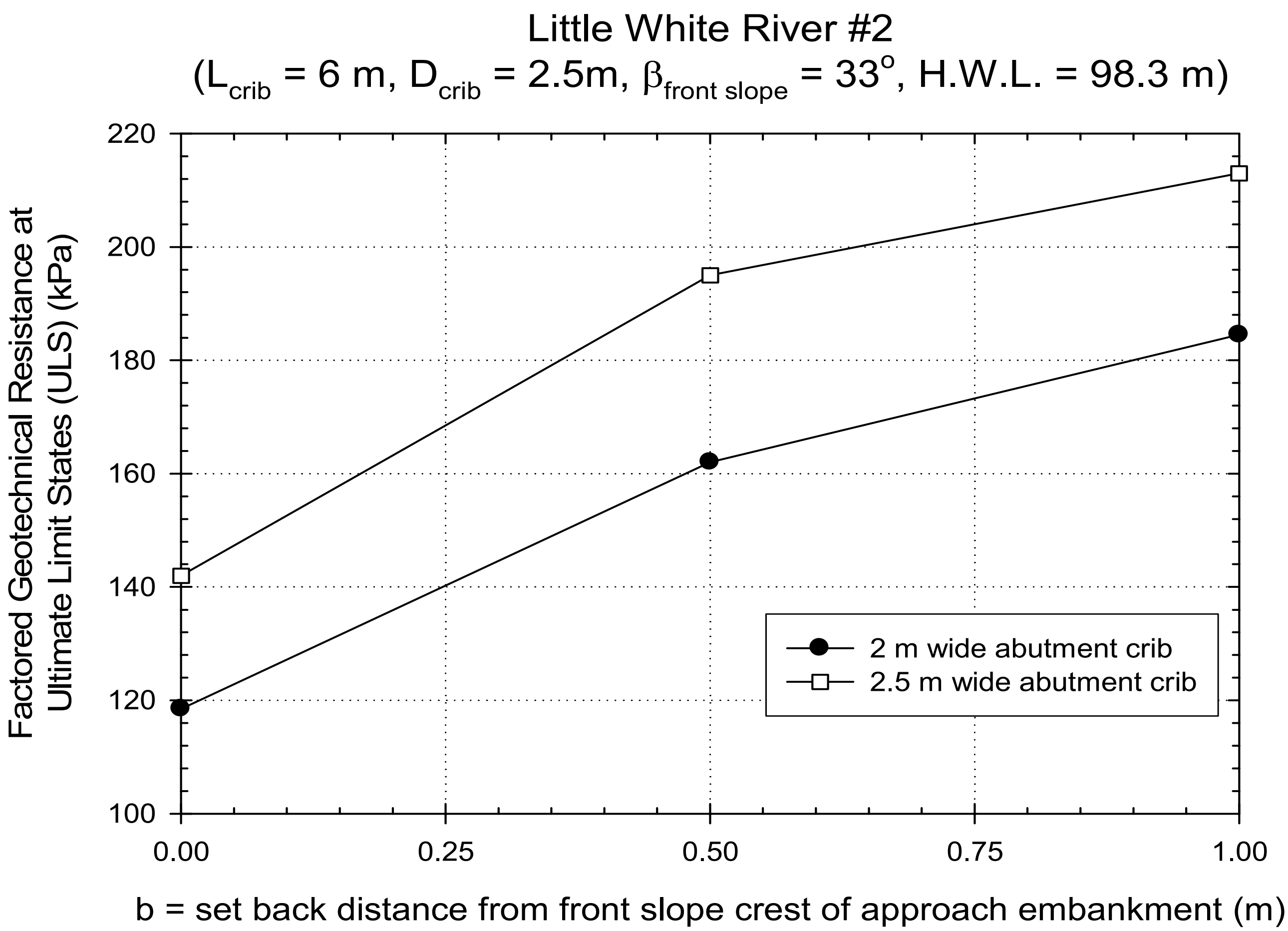


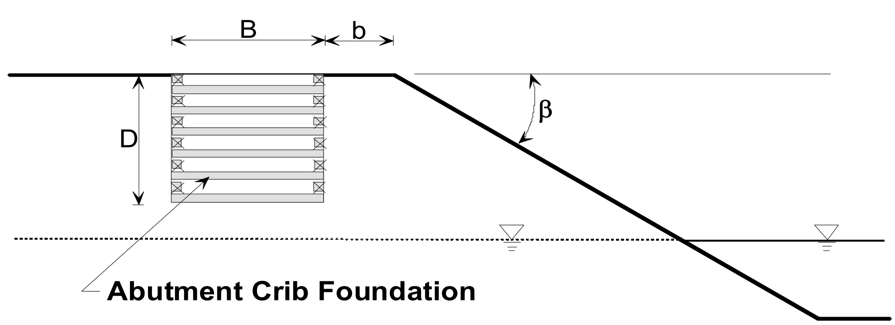


Note :
B = Width of Crib
L = Length of Crib
D = Depth of crib below ground surface / top of approach embankment
b = Set back distance from front slope crest of approach embankment
β = angle of front slope (facing river) in front of crib
HWL = High Water Level

Date: January, 2004
Project: 03-1111-029
Golder Associates
Drawn: JPD
Checked: ASP

ABUTMENT FOUNDATION DESIGN BAILEY BRIDGE REPLACEMENT LITTLE WHITE RIVER II	Figure 2





Note :

B = Width of Crib
L = Length of Crib
D = Depth of crib below ground surface / top of approach embankment
b = Set back distance from front slope crest of approach embankment
β = angle of front slope (facing river) in front of crib
HWL = High Water Level

Date:	January, 2004		Drawn:	JPD
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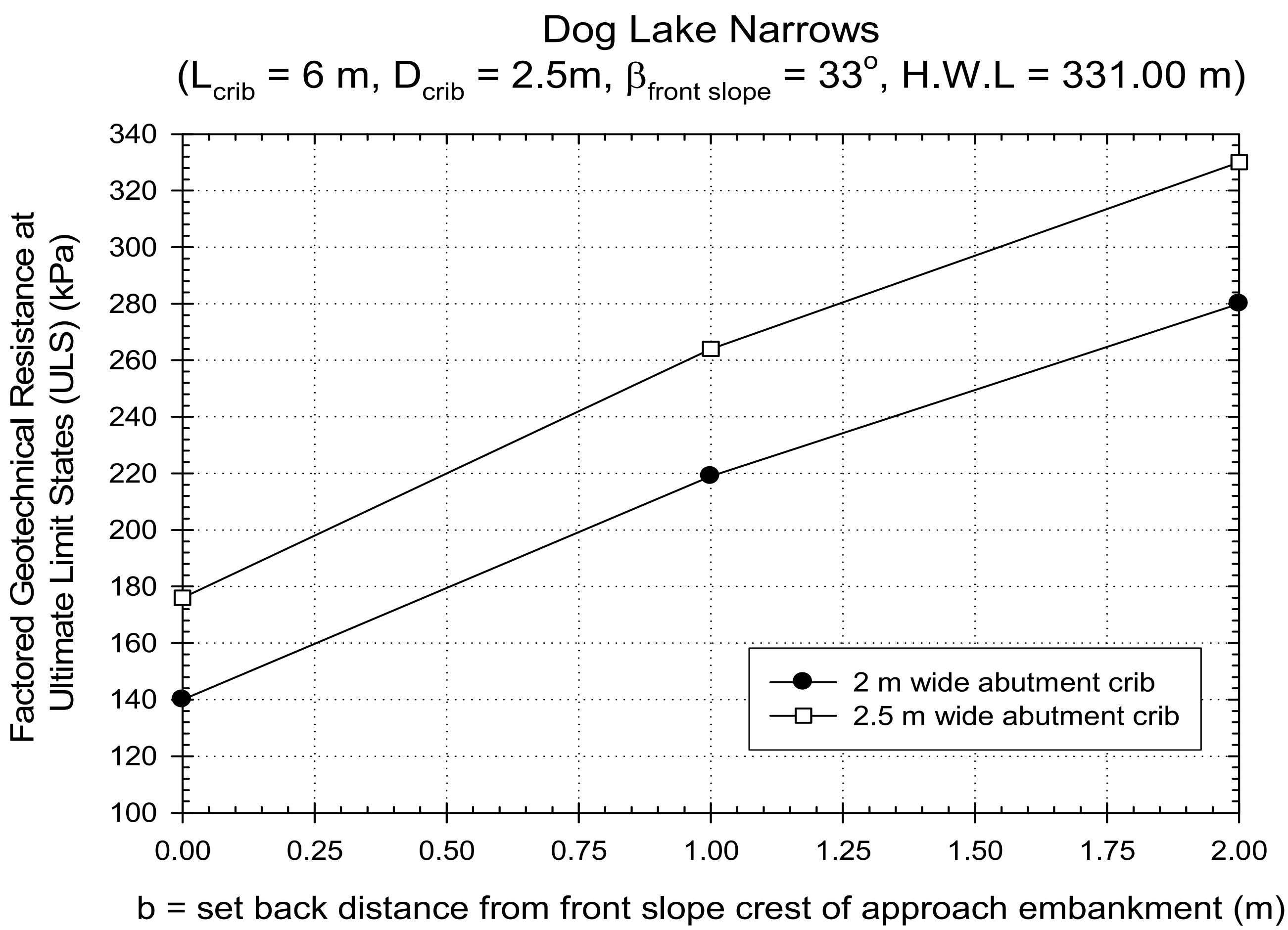
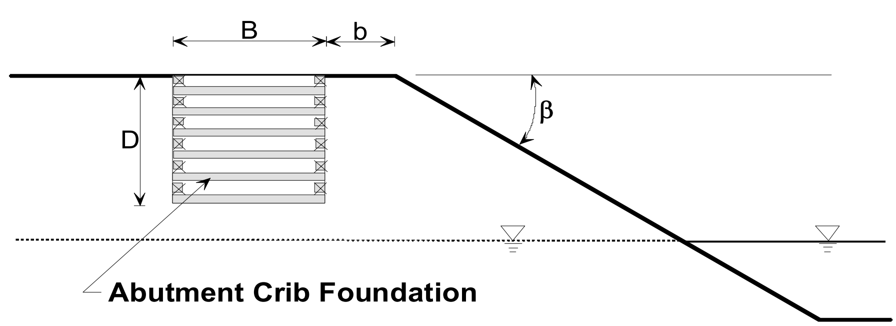
ABUTMENT FOUNDATION DESIGN
BAILEY BRIDGE REPLACEMENT
DOG LAKE NARROWS

Figure 3





Note :
- B = Width of Crib
- L = Length of Crib
- D = Depth of crib below ground surface / top of approach embankment
- b = Set back distance from front slope crest of approach embankment
- β = angle of front slope (facing river) in front of crib
- HWL = High Water Level

Date:	January, 2004		Drawn:	JPD
Project:	03-1111-029	**Golder Associates**	Checked:	ASP

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60º cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	c_u, s_u kPa	c_u, s_u psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

Note: 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. General

π	3.1416
in x,	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\ \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
v	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$-u)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1+\sigma_2+\sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_l	liquid limit
w_p	plastic limit
l_p	plasticity index $= (w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p)/I_p$
I_C	consistency index $= (w_l - w)/I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p/\sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
c'	effective cohesion
c_u, s_u	undrained shear strength (ϕ = 0 analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 + \sigma_3)$
S_t	sensitivity

Notes:
1. $\tau = c' + \sigma' \tan \phi'$
2. shear strength = (compressive strength)/2

* density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

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RECORD OF BOREHOLE No LW2-1

METRIC

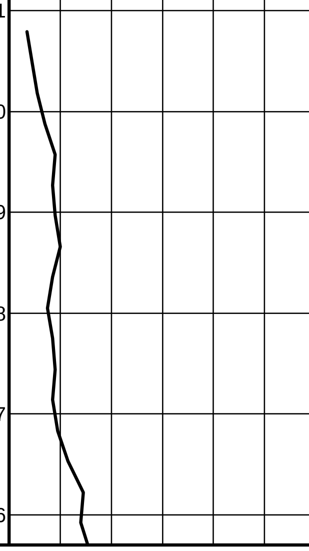
PROJECT 03-1111-029			
W.P. 513-00-01	LOCATION N 199995.9 ;E 100009.8		ORIGINATED BY GB
DIST 62	HWY 546	BOREHOLE TYPE CME 55 POWER AUGER AND WET ROTARY WITH NQ CORE BARREL	COMPILED BY KG
DATUM Local		DATE October 31, 2003	CHECKED BY JPD

Column headings: SOIL PROFILE (ELEV / DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10, 20, 30); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) (GR SA SI CL)

ELEV / DEPTH	DESCRIPTION
100.7	GROUND SURFACE
0.0	Silty Sand, some gravel and rootlets, occasional cobbles Loose Brown Moist (Fill)
99.3 / 1.4	Sand and Gravel, trace silt, occasional cobbles Compact Brown Moist (Fill)
97.7 / 3.0	Sand and Gravel, trace silt, occasional cobbles Loose to compact Brown Wet
91.0 / 9.8	End of Borehole

NUMBER	TYPE	"N" VALUES
1	SS	5
2	SS	7
3	SS	19
4	SS	11
5	SS	7
6	SS	12
7	SS	22

Elevation scale: 100, 99, 98, 97, 96, 95, 94, 93, 92, 91, 90, 89, 88, 87, 86



Notes:

1. Borehole was advanced to a depth of 1.8 m using 108 mm I.D. hollow stem augers, beyond which, further boring to 9.75 m depth was carried out using 'N' size casing and 'NQ' size core barrel.

2. Difficulties were experienced with sand blowing into the casing during drilling and sampling operations.

3. Dynamic Cone Penetration Test (DCPT) was carried out between depths of 9.75 m to 30.2 m.

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity   O $^{3\%}$ STRAIN AT FAILURE

MISS_MTO 03-1111-029-BA-MTO.GPJ ON_MOT.GDT 9/2/04

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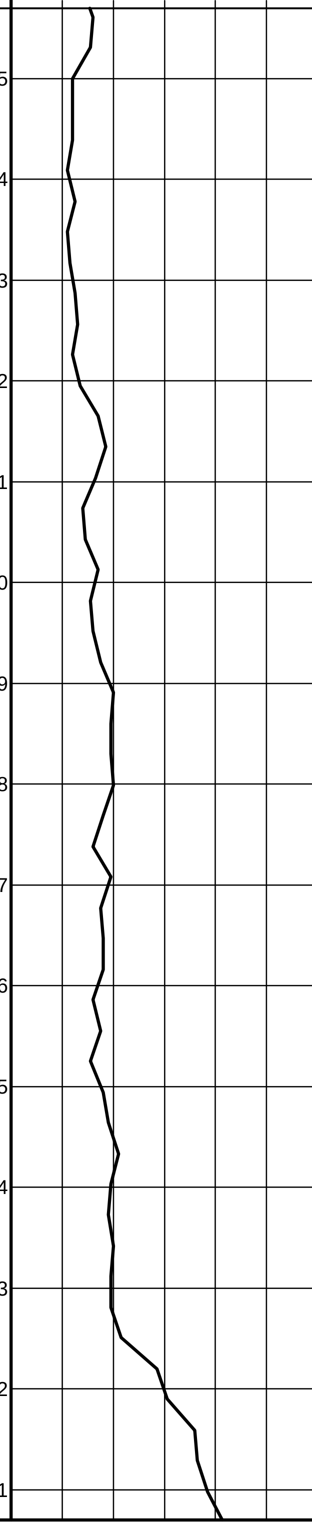
PROJECT 03-1111-029 | **RECORD OF BOREHOLE No LW2-1** | 2 OF 3 | **METRIC**

W.P. 513-00-01 | LOCATION N 199995.9 ;E 100009.8 | ORIGINATED BY GB

DIST 62 HWY 546 | BOREHOLE TYPE CME 55 POWER AUGER AND WET ROTARY WITH NQ CORE BARREL | COMPILED BY KG

DATUM Local | DATE October 31, 2003 | CHECKED BY JPD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION --- CONTINUED FROM PREVIOUS PAGE ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
							85				
							84				
							83				
							82				
							81				
							80				
							79				
							78				
							77				
							76				
							75				
							74				
							73				
							72				
							71				



Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity   O $^{3\%}$ STRAIN AT FAILURE

MISS_MTO 03-1111-029-BA-MTO.GPJ ON_MOT.GDT 9/2/04

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PROJECT 03-1111-029	RECORD OF BOREHOLE No LW2-1	3 OF 3	METRIC
W.P. 513-00-01	LOCATION N 199995.9 ;E 100009.8		ORIGINATED BY GB
DIST 62 HWY 546	BOREHOLE TYPE CME 55 POWER AUGER AND WET ROTARY WITH NQ CORE BARREL		COMPILED BY KG
DATUM Local	DATE October 31, 2003		CHECKED BY JPD

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION --- CONTINUED FROM PREVIOUS PAGE ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
							70				

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

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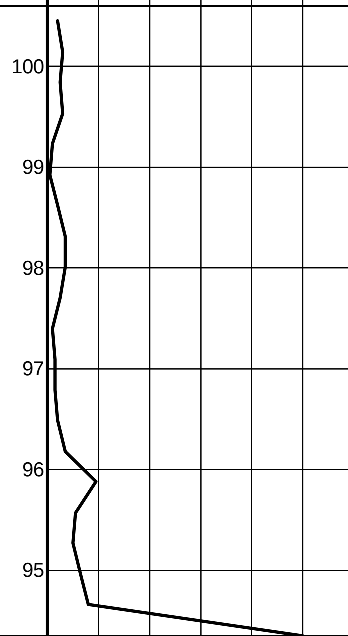
RECORD OF PENETRATION TEST No LW2-2

METRIC

PROJECT 03-1111-029			
W.P. 513-00-01	LOCATION N 200022.7 ;E 100037.1		ORIGINATED BY CS
DIST 62	HWY 546	BOREHOLE TYPE DYNAMIC CONE PENETRATION TEST	COMPILED BY KG
DATUM Local	DATE November 1, 2003		CHECKED BY JPD

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: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
100.6 0.0	GROUND SURFACE						100 99 98 97 96 95				



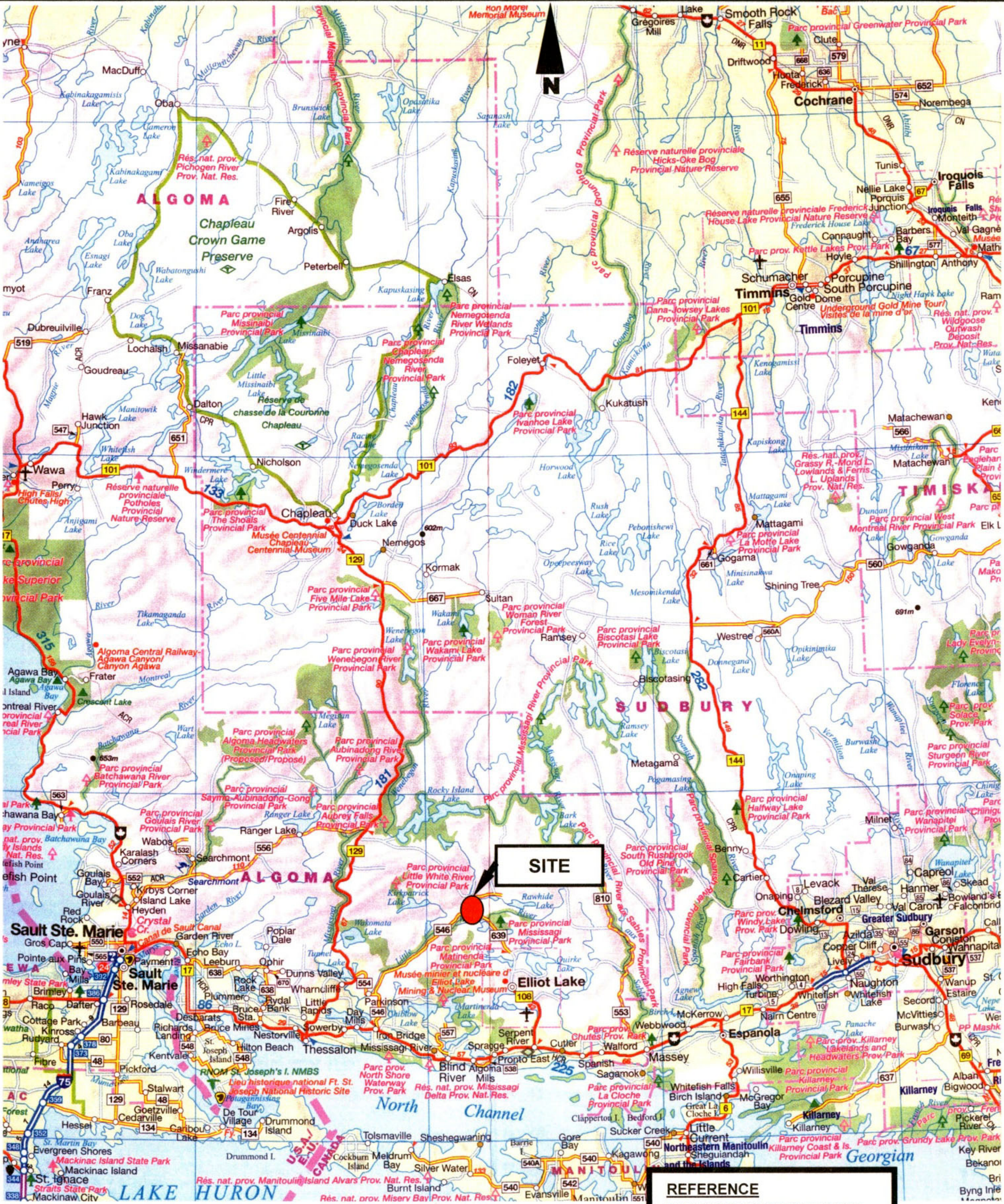
Note:

1. Dynamic Cone Penetration Test (DCPT) was carried out between depths of 0 m to 6.4 m.

$+^3$, $\times^3$: Numbers refer to Sensitivity   O $^{3\%}$ STRAIN AT FAILURE

SITE LOCATION MAP
LITTLE WHITE RIVER No. 2

FIGURE 1



REFERENCE

THIS FIGURE WAS CREATED FROM A MAP ART MAP TITLED ONTARIO, AT A SCALE OF 1:750,000 AND DATED 2003 EDITION

DATE JAN.8, 2004
PROJECT 03 1111 029

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DRAWN RBC
CHKD ....

Project: .... Drawn: TGC Reviewed: ______ Rev: .... FILE LOCATION .... 8.5x11 Fig-port.ppt

DIST. 62 HWY. 546
CONT No.
WP No.513–00–01

LITTLE WHITE RIVER#2
AND HWY 546
BOREHOLE LOCATIONS

SHEET

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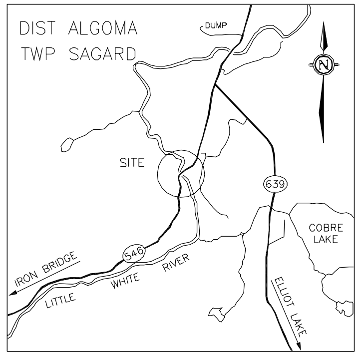
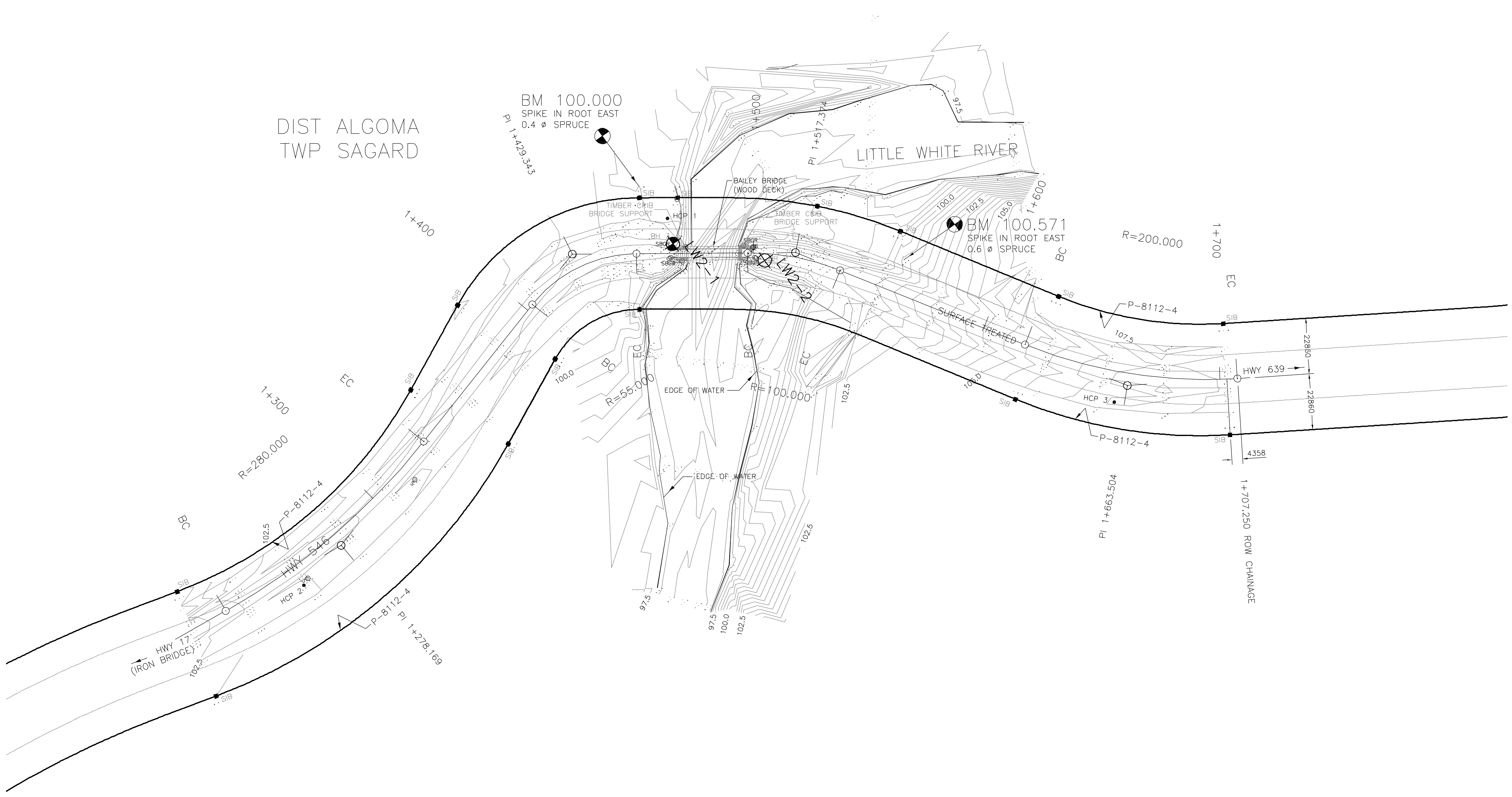
MISSISSAUGA, ONTARIO, CANADA

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN



KEY PLAN



20 0 20 40
SCALE METRES

PLAN

LEGEND

Borehole – Current Investigation
Dynamic Cone Penetration Test
Borehole and Cone

No.	ELEVATION	CO–ORDINATES	
		NORTHING [1]	EASTING [1]
LW2–1	100.7	199995.9	100009.8
LW2–2	100.6	200022.7	100037.1

NOTE

1. Northings and Eastings presented on this drawing are as presented by Northland Engineering and are not in UTM coordinate system, but are based on a local metric grid.

REFERENCE

Base plans provided in digital format by Northland Engineering (1987) limited, drawing file nos. LWR # 2 E–Plan.dwg, received November 25, 2003.

NO.	DATE	BY	REVISION

Geocres No.

HWY. 546		PROJECT NO. 03–1111–029	DIST. 62
SUBM'D.	CHKD. JPD	DATE: JAN. 2004	SITE: 38S–56
DRAWN: JDR	CHKD. FJH	APPD.	DWG. 1

MINISTRY OF TRANSPORTATION, ONTARIO

PLOT DATE: February 09, 2004

APPENDIX A

SITE PHOTOGRAPHS

PHOTO A-1: Looking North

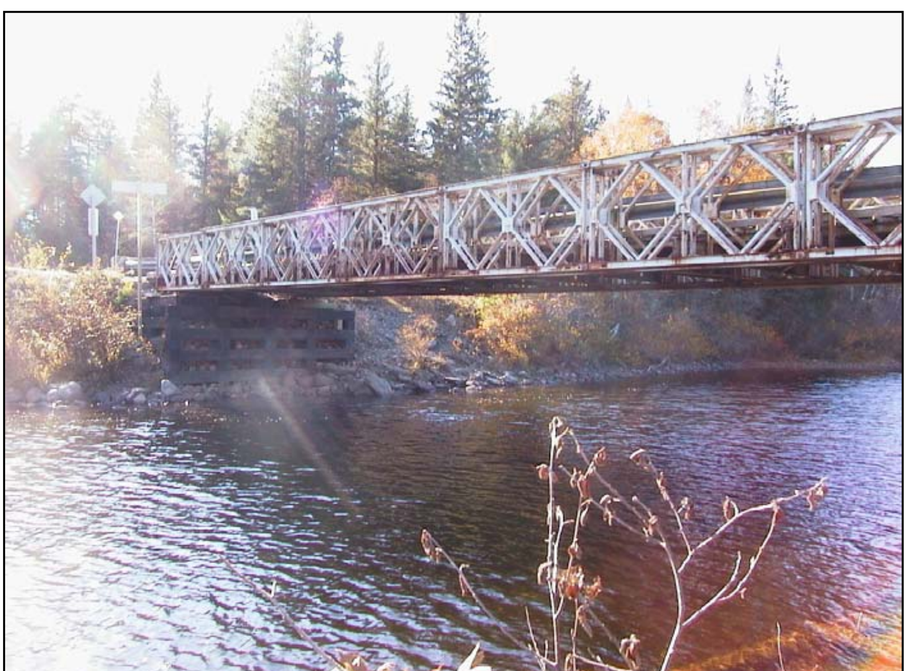


PHOTO A-2: Looking South