



**FOUNDATION DESIGN REPORT
CONCERNING PIER FOUNDATIONS
REHABILITATION/WIDENING OF SCUGOG RIVER BRIDGE
HIGHWAY 7, 0.6 KM WEST OF THE HIGHWAY 7 AND
HIGHWAY 35/KAWARTHA LAKES ROAD 15 INTERSECTION
SITE NO. 32-096
G.W.P. 4264-04-00**

**for
MORRISON HERSHFIELD LIMITED**

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Concerning Abutment Foundations, Rehabilitation/Widening of Scugog River Bridge,
Highway 7, 0.6 km West of the Highway 7 and Highway 35/Kawartha Lakes Road 15
Intersection, Site No. 32-096, G.W.P. 4264-04-00
(PML Ref.: 06HF092, dated June 28, 2007)

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FOUNDATION DESIGN REPORT
Concerning Pier Foundations
Rehabilitation/Widening of Scugog River Bridge
Highway 7, 0.6 km West of Highway 7 and
Highway 35/Kawartha Lakes Road 15 Intersection
Site No. 32-096
G.W.P. 4264-04-00

1. INTRODUCTION

This report provides Foundation Engineering comments and recommendations concerning detailed design and construction of the pier foundations to support the proposed widening of the existing bridge on Highway 7 that crosses the Scugog River about 0.6 km west of the Highway 35/Kawartha Lakes Road 15 intersection near Lindsay, Ontario.

The report was prepared for Morrison Hershfield Limited (MHL) on behalf of the Ministry of Transportation of Ontario (MTO).

Comments and recommendations concerning detailed design of the abutment foundations, based on the information documented in Geocres 31D-412 dated March 2006, were provided in Geocres No. 31D-431 dated June 28, 2007 (Abutment Report (AR); PML report reference 06HF092, dated June 28, 2007). A copy of the AR is provided in Appendix A.

The recommendations provided in the AR were reviewed following completion of the field investigation for the pier foundations; it was concluded that changes to the recommended foundation system provided in the AR in light of this new subsurface information are not required.

2. EXISTING STRUCTURE

The existing bridge is an approximate 58.3 m long three span structure supported by spread footings. The piers, situated in the Scugog River, are supported on three interconnected 2.7 by 5.5 m footings (one at each end and one at the centre of the bridge) founded near elevation 245.36, about 2.5 m below the river bottom at the borehole locations.



3. SUBSURFACE CONDITIONS

A brief description of the subsurface conditions at the foundation units is summarized in the following table.

	Water Depth (m)	Elevation (m)	Soil Description	Composition of Soil at Founding Level of existing footing
West Pier ¹	2.0	247.8 to 246.5	Soft to hard silty clay	Very dense sand and gravel with cobbles and boulders
		246.5 to 243.3	Dense to very dense sand and gravel with cobbles and boulders	
		243.3 to the termination of drilling at 235.4	Medium strength limestone, good to excellent quality (fair quality in the upper 1.5 m)	
East Pier ¹	1.9	247.9 to 242.2	Very dense sand and gravel with cobbles and boulders	Very dense sand and gravel with cobbles and boulders
		242.2 to the termination of drilling at 236.9	Medium strength limestone, excellent quality	

1. Borehole drilled from barge with platform 1.1 m above the river water level.

The river water level during the field investigation conducted in July/August of 2007 was at elevation 249.8. The 25, 50 and 100 year storm event water elevation is reported to be at elevation 250.81, 250.84 and 250.88, respectively. A design river water elevation of 250.8 was used for subsequent analysis.

The inferred piezometric level in the soil and bedrock is some 5.5 m above the founding level of the existing pier footings.

4. FOUNDATION CONSIDERATIONS

4.1 General

It is understood that the load to be supported by the pier foundations constructed to support the widened portion of the bridge is about 6900 kN. It is also understood that the design scour level is elevation 244.8.



The seismic site coefficient for the conditions at this site is 1.0 (Type I soil profile as per clause 4.4.6 of the CHBDC, CAN/CSA-S6-06, November 2006). The zonal acceleration ratio is 0.05.

The bridge is located in Seismic Performance Zone 1. The liquefaction potential of the sand and gravel at the site was assessed using the procedure suggested by Seed et al (1984) and, on this basis, it is considered that liquefaction of these soils is unlikely (clause 4.6.2 of the CHBDC).

Construction of spread footings or micro piles are considered to be feasible foundation alternatives for the piers. The preferred option will be dictated by structural design and economic considerations as well as constructability issues related to dewatering to enable construction of the foundations.

It is considered that caissons are not a suitable foundation system due to the granular subgrade material, the presence of large boulders in the granular till and the potential for basal heave due to the differential hydraulic gradient that would exist at the base of the excavation if this option is employed.

Similarly, driven piles to support the foundation loads are not suitable due to the assessed scour depth (elevation 244.8) and the limited depth of penetration that could be achieved (anticipated refusal at elevation 245.0 to 245.5).

Additional comments concerning the advantages, disadvantages, costs, risks and consequences of the micro pile and spread footing alternatives are provided in Table 1; design/construction recommendations are provided in the following paragraphs. From a foundation engineering perspective, it is recommended that micro piles are employed to support the pier loads. A copy of the Special Provisions for Protection System, Unwatering, Excavation and Tremie Concreting are provided in Appendix B.

4.2 Micro Piles

The bedrock surface elevation at the west and east piers is near elevation 243.3 and 242.2, respectively. It is noted that the granular deposit that overlies the bedrock is very dense and contains large cobbles and boulders. It will be necessary to socket the micro piles into the bedrock to enable development of sufficient resistance to support the foundation loads.



The bedrock comprised medium strength unweathered limestone of the Lindsay Formation. The core recovery during drilling of the bedrock varied from 88 to 100%, typically 100%. The Rock Quality Designation (RQD) determined from the rock core was 66% in the upper 1.5 m in Borehole 07-1 indicating a fair quality rock, and ranged from 82 to 100% below that depth indicating a good to excellent quality rock. The RQD value ranged from 91 to 100% in the rock core samples retrieved from Borehole 07-2 indicating excellent quality rock.

The unconfined compressive strength of eight core samples ranged from 28.9 to 65.8 MPa with an average of 41.8 MPa which indicates a medium strong rock. It is noted however, with the exception of the two core samples retrieved in Borehole 07-1 within 2 m of the rock surface, the unconfined compressive strength ranged from 28.9 to 42.6 MPa. The results of the five splitting tensile tests ranged from 6.0 to 9.3 MPa. The results of the rock core testing are provided in Table 3.

Resistance to the lateral loads will be provided by the passive earth pressure developed on the micro piles to a depth of six diameters of the micro pile below the scour depth as well as the horizontal bearing resistance of the soil and bedrock below this depth.

The passive resistance developed by the granular soil to a depth equal to six times the diameter of the micro pile below the scour depth should be computed using the following equation acting on an area equivalent to a width of three and depth of six diameters of the micro pile below the scour depth provided the centre to centre spacing between the micro piles is greater than five times the diameter of the micro pile (refer to following table for details).

Passive Pressure

$$P_p = K_p (\gamma' h_s + q) \quad (1)$$

where K_p = passive earth pressure coefficient

h_s = depth below design scour depth (m)

γ' = buoyant unit weight of granular material
below scour depth level ($\gamma - \gamma_w$) kN/m^3

q = surcharge load (kN/m^2)

γ_w = unit weight of water
= 9.8 kN/m^3

The factored passive resistance at ULS is $0.5 P_p$.



Elevation ⁽¹⁾	Soil Profile	Total Unit Weight (γ kN/m ³)	Effective Friction Angle (ϕ') (Degrees)	Passive Earth Pressure Coefficient (K_p)
244.8	Design Scour Level ⁽²⁾			
244.8 to 242.2	Very dense sand and gravel	23.5	42	5.0

(1) Based on BH 07-2

(2) From Morrison Hershfield Ltd.

The factored horizontal bearing resistance at ULS of the strata below the design scour elevation of 244.8 is considered to be:

Elevation ⁽¹⁾	Soil Description	Factored Horizontal Bearing Resistance at ULS / SLS (kPa)
244.8 to 242.2	Very dense sand and gravel	375 / 150
242.2	Bedrock	850 / N/A

(1) Based on BH 07-2

N/A Not applicable since load required to initiate movement to mobilize the SLS resistance exceeds the factored resistance at ULS.

The following equation should be employed to evaluate the coefficient of horizontal subgrade reaction along the micro piles:

Granular

$$k_s = n_h z/b$$

where k_s = coefficient of horizontal subgrade reaction kN/m³

n_h = coefficient related to soil density
 Refer to the following table for details

z = depth, m

b = width of micro pile, m



Elevation ⁽¹⁾	Soil Profile	Soil Density ⁽²⁾ Coefficient (n_h - kN/m ³)
244.8	Design Scour Depth ⁽³⁾	
244.8 to 242.2	Very dense sand and gravel	25,000
242.2	Bedrock	40,000

(1) Based on BH 07-2

(2) Based on micro pile width of 273 mm; values shown are applicable to 'below water' situations which is the case for the conditions at this site.

(3) From Morrison Hershfield Ltd.

The axial resistance of the micro piles will be developed by shaft friction mobilized in the very dense granular material, the bond stress developed along the portion of the micro pile socketted into bedrock and end bearing in the bedrock. Detailed comments and recommendations for design and construction of the micro piles will be provided by others.

Installation of a cofferdam will be required to enable construction of the pile cap 'in the dry'. Refer to section 5 for additional comments in this regard.

A concept for construction of the micro piles is illustrated on Figures MP1 to MP6.

The required depth of excavation will be dictated by both the thickness and elevation of the top of the pile cap as well as the tremie concrete thickness required to balance the hydraulic pressure. As noted in section 5, a 2.3 m thickness of tremie concrete is required to balance the hydraulic pressure if the excavation extends to elevation 245.36. The tremie concrete thickness could be reduced by

- Placement of heavier materials to balance the hydraulic pressure such as high density concrete and/or steel billets.
- Constructing the micro pile/tremie concrete as a structural unit. The tremie concrete must be designed to be a structural slab capable of resisting the hydraulic pressure and be of sufficient stiffness to minimize cracking when subjected to the hydraulic pressure.

Consideration could also be given to:

- Placement of 'vertical drains' in the tremie concrete which would allow water to flow vertically through the concrete. This would require:
 - i) Placement of a geotextile on the subgrade following excavation and before placement of the tremie concrete to prevent upward movement of soil particles into the cofferdam due to seepage forces.
 - ii) Placement of reinforcing steel in the tremie concrete to provide a platform for the safety of workers.
 - iii) Continuous pumping to dewater the cofferdam.

In this regard, it would be necessary to have sump pits on the tremie concrete surface in order that the water level can be depressed within the cofferdam sufficiently to enable the reinforcing steel and structural concrete to be placed 'in the dry'.

Pumping from wells installed into the underlying granular deposit through the 'vertical drains' could also be considered.

It must be noted however, that implementation of this option has considerable risk; it is recommended therefore, that this option be considered only if suggested by the successful Contractor.

4.3 Spread Footings

Footings constructed to support the piers should be founded at the same level as the existing footings (elevation 245.36). Footings bearing on the dense to very dense sand and gravel at this elevation should be designed using a factored bearing resistance at ultimate limit states (ULS) of 735 kPa and 300 kPa at serviceability limit states (SLS). The SLS value is provided since there is a potential for disturbance to the soil below water during construction and a need for minimal settlement of the bridge widening following construction. In this regard, the structural loads will be



imposed on the subgrade soil gradually as construction proceeds; the total settlement following completion of the bridge deck and application of the live loads should be in the order of 5 to 10 mm.

The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.7.4 of the CHBDC.

The horizontal force imposed on the foundations will be resisted in part by the friction force developed between the underside of the footing and the native subgrade soil. An unfactored friction factor of 0.7 is recommended.

Construction of spread footings founded at the same level as the existing pier foundation will require implementation of measures to control the inflow of water from the river as well as groundwater seepage from the soil (upward and laterally) into the work area. Refer to section 5 for additional comments in this regard.

A concept for construction of the footings is illustrated on Figures F1 to F4.

5. DEWATERING CONSIDERATIONS

5.1 Options Considered

The existing pier footings are founded at elevation 245.36, about 2.5 m below the river bottom at the pier locations. The design river water level (elevation 250.8) (and hence, the inferred piezometric level in the soil) is some 5.5 m above the founding level of the pier foundations.

The following options were considered to deal with water that enters the work area. The comments are intended to be for planning and design purposes. The tender documents should clearly state that control of water in the sheeted/excavated areas to enable construction to proceed in the dry is the Contractor's responsibility. Option 2 is preferred from a foundation engineering perspective.

Option 1

Installation of steel sheeting around the complete perimeter of the foundation to sufficient depth to provide a cut-off for groundwater seepage, heave and/or piping at the base of the excavation.



In order to provide an adequate cut-off the steel sheeting should extend to the bedrock surface. It is expected however, to meet refusal on the cobbles/boulders identified in the granular deposit near elevation 245.0 to 245.5. It is concluded therefore that this option is not feasible.

Option 2

Installation of steel sheeting around the complete perimeter of the foundation and, prior to dewatering, excavation of the soil within the sheeted area to elevation 245.36 followed by placement of a concrete plug to balance the differential hydraulic pressure.

It is anticipated that the sheeting will meet refusal at elevation 245.0 to 245.5.

In order to balance the 5.5 m differential hydraulic pressure between the design river water elevation of 250.8 and the underside of the existing footing elevation of 245.36, it will be necessary to place a 2.3 m thickness of concrete on the subgrade. The water within the limits of the cofferdam could then be pumped and the reinforcing steel placed to construct the foundation.

Refer to Appendix B for Special Provisions in this regard.

Option 3

Installation of sheeting with an embedment depth dictated by toe restraint criteria to maintain the stability of the retention structure in conjunction with closely spaced well points (or equivalent) around the perimeter of the sheeting.

Cognizant of the presence of cobbles/boulders within the granular deposit and the anticipated refusal depth of the sheeting, it is likely toe fixity for the sheeting cannot be achieved and installation of the well points would not be possible; therefore it is considered that this is not a feasible option.

It is visualized that the equipment employed to drive the sheeting will operate above the existing bridge deck where limited space exists for installation of the sheeting. Consequently, the clearance requirement should be reviewed to confirm that the sheeting can be installed.

5.2 Design Parameters

The structural analysis conducted during detailed design of the sheet pile wall enclosure structure must consider the lateral pressure imposed on the sheet pile wall by the soil and water (including short term variations due to ship traffic, natural variations in the river water level, (storm events, seasonal and long term cycles), as well as the applied horizontal (ice) and vertical surcharge loads.

The lateral active and passive earth pressures (unfactored) imposed on the sheet pile wall should be computed using the following equations:

Active Pressure

$$P_A = K_a (\gamma h_1 + \gamma' h_2 + q) + \gamma_w h_2 + C_s$$

where K_a = active earth pressure coefficient (dimensionless)

γ = unit weight of retained soil
above the design water level (kN/m^3)

γ' = buoyant unit weight of soil
below design water level ($\gamma - \gamma_w$)

h_1 = depth from ground surface (m), to design water level

h_2 = depth below design water level (m)

q = surcharge load (kN/m^2)

γ_w = unit weight of water
= 9.8 kN/m^3

C_s = earth pressure induced by seismic events, kPa (refer to clause 4.6.4 of CHBDC)
where ϕ = angle of internal friction of retained soil (40° for granular soil)
 δ = angle of friction between the soil and wall (30° for granular soil)



Passive Pressure

$$P_p = \gamma_w h_2 + K_p (\gamma h_s + q) + C_s$$

where K_p = passive earth pressure coefficient

h_s = depth below base of excavation to toe of sheeting (m)

γ_w , γ , q , h_2 and C_s were defined previously.

Elevation	Soil Composition	Total Unit Weight (γ) (kN/m ³)	Effective Friction Angle (ϕ')	Active Earth ¹ Pressure Coefficient (K_a)	Passive Earth ¹ Pressure Coefficient (K_p)
West Pier					
247.8 to 246.5	Soft to hard silty clay	18.0	25.0	0.40	2.5
246.5 to 245.6	Dense sand and gravel	22.5	40.0	0.22	4.6
245.6 to 243.3	Very dense sand and gravel	23.5	42.0	0.2	5.0
East Pier					
247.9 to 245.0	Dense sand with gravel	22.5	40.0	0.22	4.6
245.0 to 242.2	Very dense sand and gravel	23.5	42.0	0.2	5.0

1. Wall friction ignored.

The factored passive resistance at ULS is $0.5 p_p$, the active pressure is unfactored (clause 6.9.1 of the Commentary to the Code).

6. CLOSURE

The report was prepared by Mr. Dennis W. Kerr, MEng, P.Eng., Chief Foundation Engineer. Mr. Brian R. Gray, MEng, P.Eng., MTO Designated Contact, carried out an independent review of the report.

Sincerely

Peto MacCallum Ltd.

A handwritten signature in blue ink, appearing to read "D. W. Kerr".

Dennis W. Kerr, MEng, P.Eng.
Chief Foundation Engineer

A handwritten signature in blue ink, appearing to read "Brian R. Gray".

Brian R. Gray, MEng, P.Eng.
MTO Designated Principal Contact

DWK:lad

Table 1
 Comparison of Micro Pile and Spread Footing Foundation Alternatives¹

FOUNDATION TYPES	ADVANTAGES	DISADVANTAGES	RISKS/CONSEQUENCES
Spread Footings Spread footings constructed within the limits of a cofferdam and founded at elevation 245.0; cofferdam driven to refusal; water level maintained at river level during excavation to subgrade level; concrete plug placed to balance differential hydraulic pressure and prevent basal heave before pumping of water	<ul style="list-style-type: none"> • Compatible with existing foundation system • Control of water essentially limited to seepage through the cofferdam • Special construction techniques to install cofferdam in very dense granular soil not required • Basal heave/piping not a concern • Placement of the reinforcing steel will be done in the dry within the limits of the cofferdam 	<ul style="list-style-type: none"> • Excavation is done below water level • QVE requires diver to conduct site review following excavation • Top of footing may be above river level 	<ul style="list-style-type: none"> • Excavation below the water level could result in undetected deficiencies and poor performance of the footings • Possible refusal of the steel sheet piles at 245.5 some 0.5 m above the existing pier footings and the desired depth of excavation
Micro Piles	<ul style="list-style-type: none"> • Control of water essentially limited to seepage through the cofferdam • Basal heave/piping not a concern since pile cap is not supported by sand and gravel • Could be installed with barge mounted equipment • Could be socketted into bedrock and therefore, scour action would not undermine the foundation system • Compatible with existing foundation system • Resistance to lateral loads can be provided by very dense granular soil that overlies bedrock 	<ul style="list-style-type: none"> • Management of excavated material to prevent entry into the river • Potential for difficulties to be encountered during drilling through the very dense granular material that overlies bedrock 	<ul style="list-style-type: none"> • Augering difficulties that could result in construction delays and cost overrun • Undermining of pile cap by scour that could expose the mini piles

NOTES:

1. Foundation level same as existing abutments: elevation 245.0.
2. Qualitative cost comparison (least to greatest): spread footings; micro piles.

NOTES:

1. DRIVE STEEL SHEETING TO REFUSAL
ANTICIPATED NEAR EL. 245.0

RIVER WATER LEVEL EL. 249.8
JULY 31, 2007

WATER

RIVER BED EL. 247.9

- SAND AND GRAVEL WITH
COBBLE

- VERY DENSE

FOUNDING LEVEL OF
EXISTING PIER FOOTING EL. ±245.0

- WITH COBBLES AND BOULDERS
TO 380mm

LIMESTONE BEDROCK EL. 242.2

MINISTRY OF TRANSPORTATION - ONTARIO

HIGHWAY 7
REHABILITATION/WIDENING OF
SCUDOGG RIVER BRIDGE

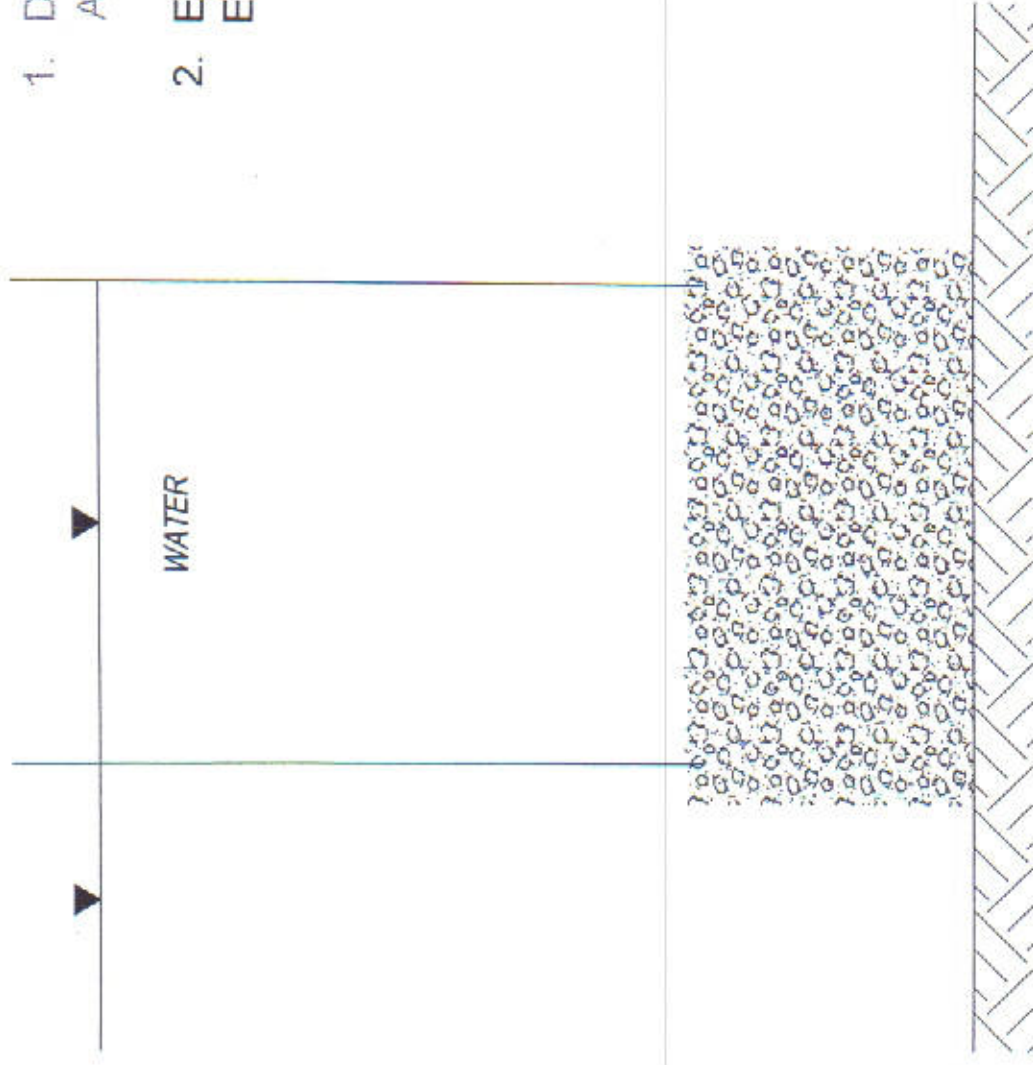
FOOTING CONSTRUCTION SEQUENCE

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CONSULTING ENGINEERS

DRAWN: N.A.	DATE: NOV. 2007	SCALE: N.T.S.	JOB NO: 06HF032B	DRAWING NO: F-1
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APPROVED: M.E.				

NOTES:

1. DRIVE STEEL SHEETING TO REFUSAL
ANTICIPATED NEAR EL. 245.0
2. EXCAVATE SAND AND GRAVEL TO
EL. 245.0



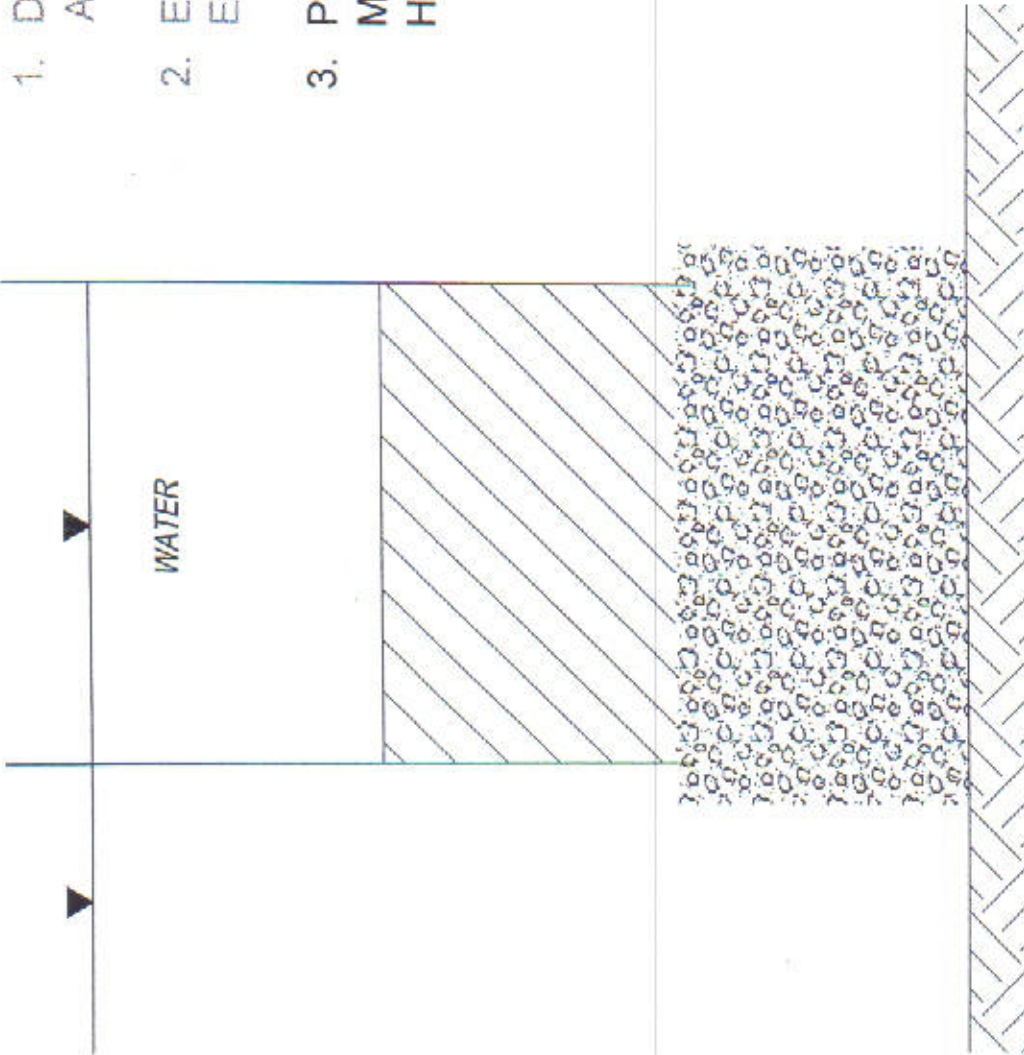
MINISTRY OF TRANSPORTATION - ONTARIO

HIGHWAY 7
REHABILITATION/WIDENING OF
SCUGOG RIVER BRIDGE

FOOTING CONSTRUCTION SEQUENCE

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CONSULTING ENGINEERS

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APPROVED: M.R.				



NOTES:

1. DRIVE STEEL SHEETING TO REFUSAL ANTICIPATED NEAR EL. 245.0
2. EXCAVATE SAND AND GRAVEL TO EL. 245.0
3. PLACE CONCRETE BY TREMIE METHOD TO RESIST VERTICAL HYDRAULIC PRESSURE.

MINISTRY OF TRANSPORTATION - ONTARIO

HIGHWAY 7
REHABILITATION/WIDENING OF
SCUGOG RIVER BRIDGE

FOOTING CONSTRUCTION SEQUENCE

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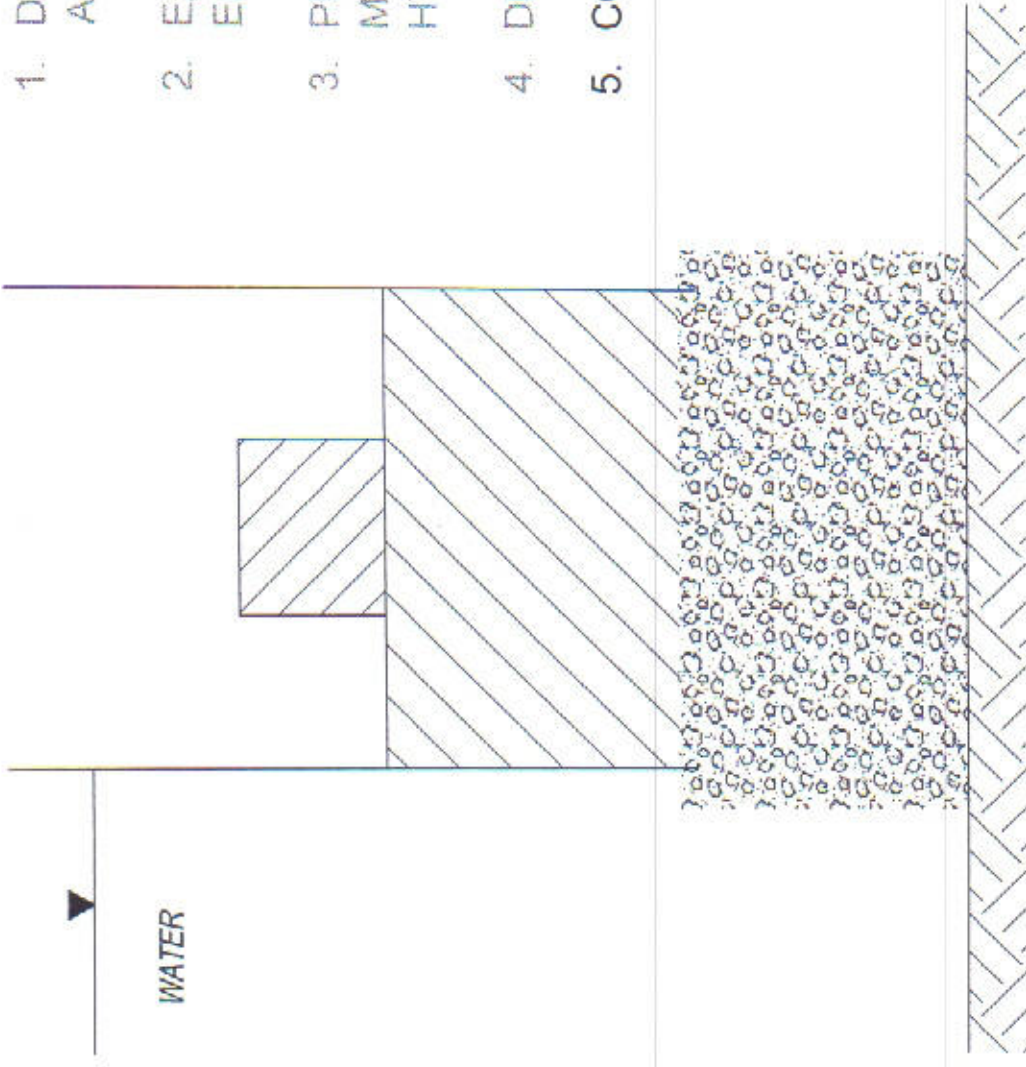
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NOTES:

1. DRIVE STEEL SHEETING TO REFUSAL ANTICIPATED NEAR EL. 245.0
2. EXCAVATE SAND AND GRAVEL TO EL. 245.0
3. PLACE CONCRETE BY TREMIE METHOD TO RESIST VERTICAL HYDRAULIC PRESSURE.
4. DEWATER THE COFFERDAM.

MINISTRY OF TRANSPORTATION - ONTARIO					
HIGHWAY 7 REHABILITATION/WIDENING OF SCUOGG RIVER BRIDGE					
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APPROVED:					



NOTES:

1. DRIVE STEEL SHEETING TO REFUSAL ANTICIPATED NEAR EL. 245.0
2. EXCAVATE SAND AND GRAVEL TO EL. 245.0
3. PLACE CONCRETE BY TREMIE METHOD TO RESIST VERTICAL HYDRAULIC PRESSURE.
4. DEWATER THE COFFERDAM.
5. CONSTRUCT STRIP FOOTING.

MINISTRY OF TRANSPORTATION - ONTARIO

HIGHWAY 7
REHABILITATION/WIDENING OF
SCU006 RIVER BRIDGE

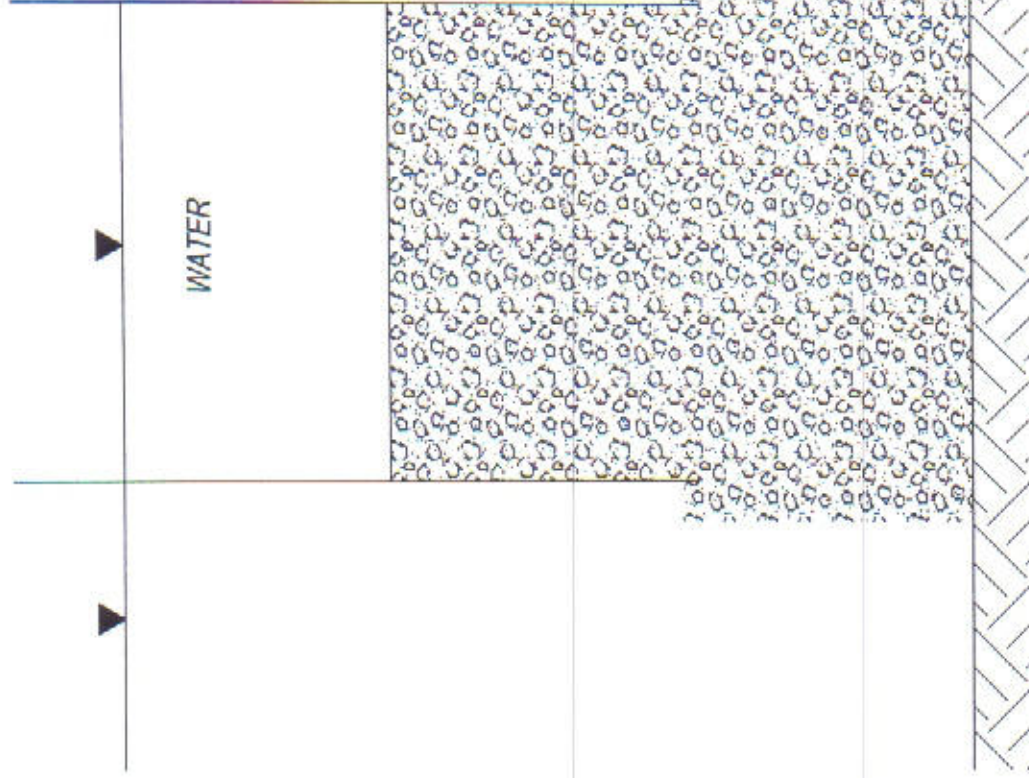
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OWNER: M.T.A.	DME	SCALE	JOB NO.	DRAWING NO.
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APPROVED:				

NOTES:

1. DRIVE STEEL SHEETING TO REFUSAL
ANTICIPATED NEAR EL. 245.0



RIVER WATER LEVEL EL. 249.8
JULY 31, 2007

RIVER BED EL. 247.9

- SAND AND GRAVEL WITH
COBBLE

- VERY DENSE

FOUNDING LEVEL OF EL. +245.0
EXISTING PIER FOOTING

- WITH COBBLES AND BOULDERS
TO 380mm

LIMESTONE BEDROCK EL. 242.2

MINISTRY OF TRANSPORTATION - ONTARIO

HIGHWAY 7
REHABILITATION/WIDENING OF
SCUGOG RIVER BRIDGE

MICROPILE CONSTRUCTION SEQUENCE

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APPROVED: M.D.C.				

NOTES:

1. DRIVE STEEL SHEETING TO REFUSAL
ANTICIPATED NEAR EL. 245.0
2. EXCAVATE SAND AND GRAVEL
TO UNDERSIDE OF TREMIE
CONCRETE



MINISTRY OF TRANSPORTATION - ONTARIO

HIGHWAY 7
REHABILITATION/WIDENING OF
SCUBOG RIVER BRIDGE

MICROPILE CONSTRUCTION SEQUENCE

PMI Peto MacCallum Ltd.
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APPROVED: R.P.R.				

NOTES:

1. DRIVE STEEL SHEETING TO REFUSAL
ANTICIPATED NEAR EL. 245.0
2. EXCAVATE SAND AND GRAVEL
TO UNDERSIDE OF TREMIE
CONCRETE
3. CONSTRUCT MICROPILES



MINISTRY OF TRANSPORTATION - ONTARIO

HIGHWAY 7
REHABILITATION/WIDENING OF
SCUGOG RIVER BRIDGE

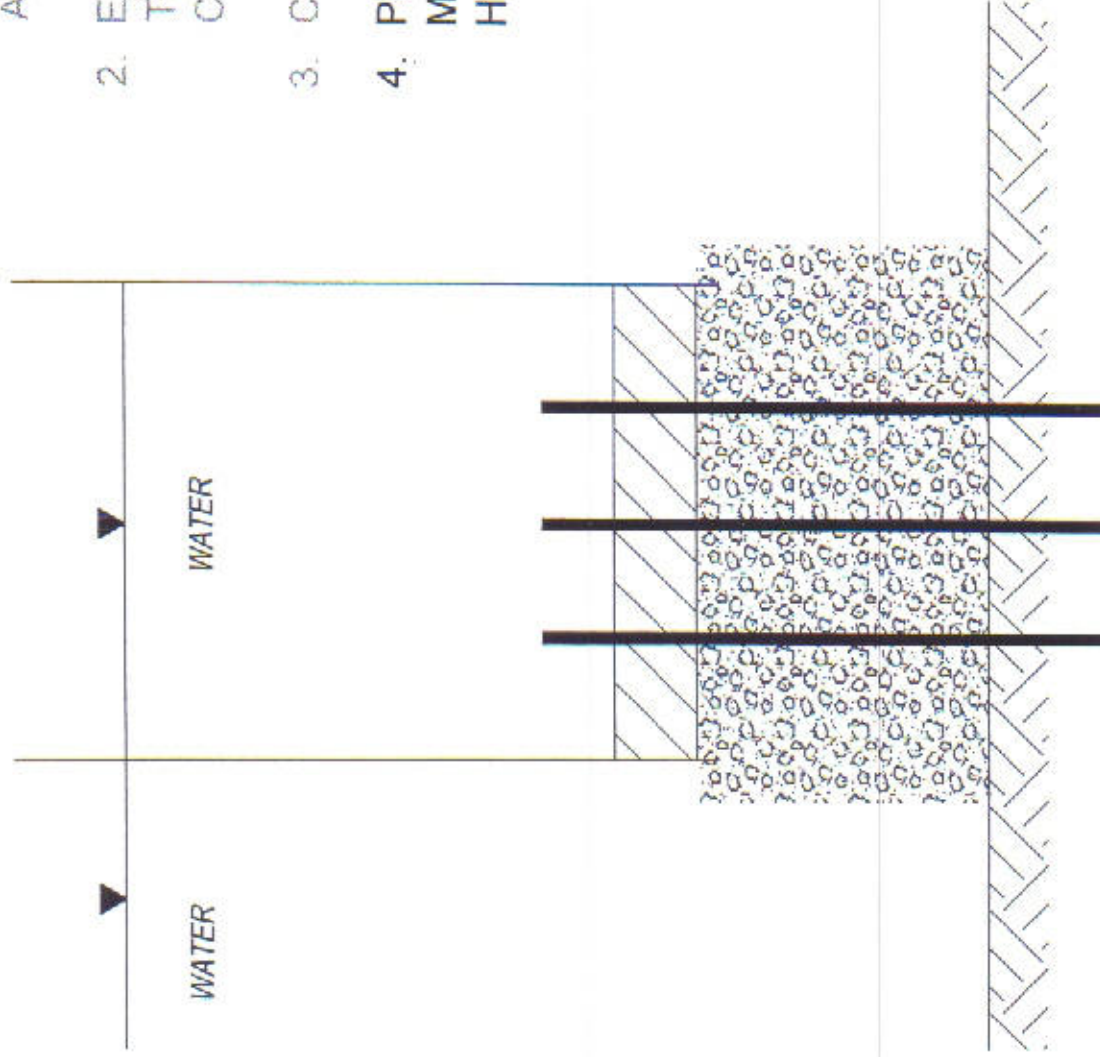
MICROPILE CONSTRUCTION SEQUENCE

PMI Peto MacCallum Ltd.
CONSULTING ENGINEERS

DRAWN: N.A.	DATE	SCALE	JOB NO.	DRAWING NO.
CHECKED: D.W.K.	NOV. 2007	N.T.S.	06HF092B	IMP-3
APPROVED: m.m.				

NOTES:

1. DRIVE STEEL SHEETING TO REFUSAL ANTICIPATED NEAR EL. 245.0
2. EXCAVATE SAND AND GRAVEL TO UNDERSIDE OF TREMIE CONCRETE
3. CONSTRUCT MICROPILES
4. PLACE CONCRETE BY TREMIE METHOD TO BALANCE THE HYDRAULIC PRESSURE



MINISTRY OF TRANSPORTATION - ONTARIO

HIGHWAY 7
REHABILITATION/WIDENING OF
SCUGOG RIVER BRIDGE

MICROPILE CONSTRUCTION SEQUENCE

PMI Peto MacCallum Ltd.
CONSULTING ENGINEERS

DRAWN: N.A.
CHECKED: D.W.K.
APPROVED: R.M.C.

DATE
NOV. 2007

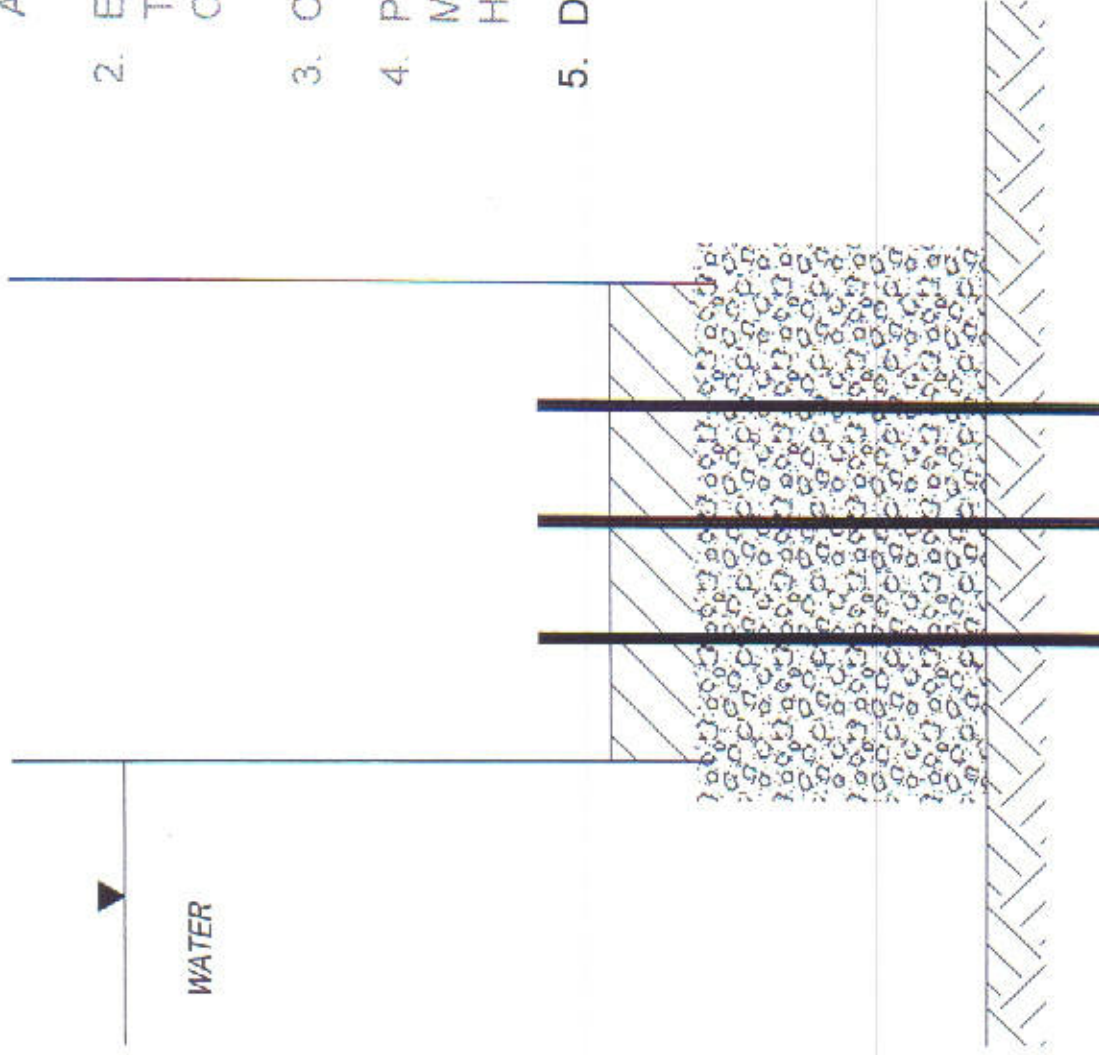
SCALE
N.T.S.

JOB NO.
08HF092B

DRAWING NO.
MP-4

NOTES:

1. DRIVE STEEL SHEETING TO REFUSAL ANTICIPATED NEAR EL. 245.0
2. EXCAVATE SAND AND GRAVEL TO UNDERSIDE OF TREMIE CONCRETE
3. CONSTRUCT MICROPILES
4. PLACE CONCRETE BY TREMIE METHOD TO BALANCE THE HYDRAULIC PRESSURE
5. DEWATER THE COFFERDAM



MINISTRY OF TRANSPORTATION - ONTARIO

HIGHWAY 7
REHABILITATION/WIDENING OF
SCUSOG RIVER BRIDGE

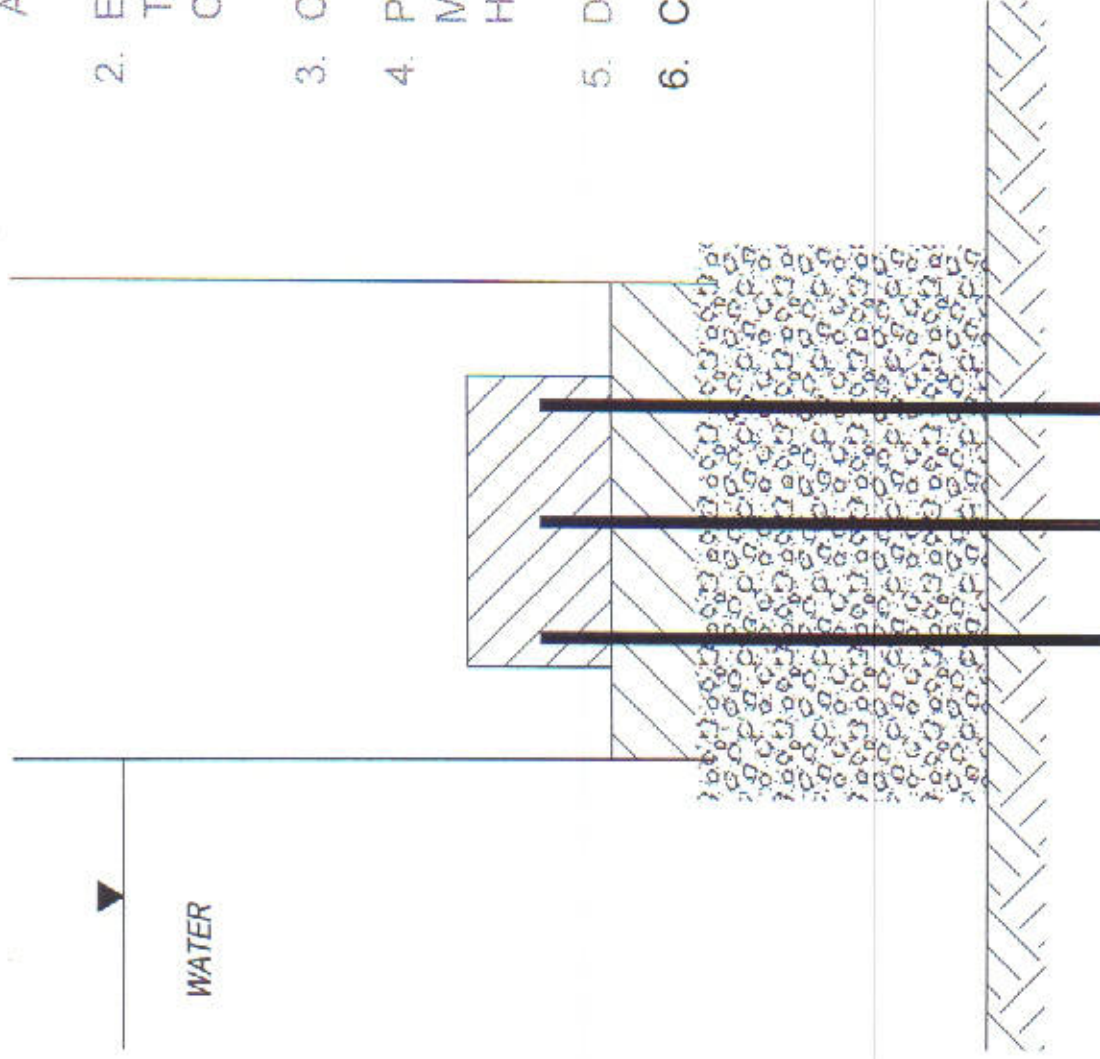
MICROPILE CONSTRUCTION SEQUENCE

PMI Peto MacCallum Ltd.
CONSULTING ENGINEERS

DRAWN: N.A.	DATE: NOV. 2007	SCALE: N.T.S.	ASB NO.: BGHF092B	DRAWING NO.: MP-5
CHECKED: D.W.K.				
APPROVED: R.D.C.				

NOTES:

1. DRIVE STEEL SHEETING TO REFUSAL ANTICIPATED NEAR EL. 245.0
2. EXCAVATE SAND AND GRAVEL TO UNDERSIDE OF TREMIE CONCRETE
3. CONSTRUCT MICROPILES
4. PLACE CONCRETE BY TREMIE METHOD TO BALANCE THE HYDRAULIC PRESSURE
5. DEWATER THE COFFERDAM
6. CONSTRUCT PILE CAP



MINISTRY OF TRANSPORTATION - ONTARIO

HIGHWAY 7
REHABILITATION/WIDENING OF
SCUGOG RIVER BRIDGE

MICROPILE CONSTRUCTION SEQUENCE

PMI Peto MacCallum Ltd.
CONSULTING ENGINEERS

DRAWN: N.A.	DATE: NOV. 2007	SCALE: N.T.S.	JOB NO.: 08HF092B	DRAWING NO.: MP-6
CHECKED: D.W.K.				
APPROVED: B.C.				



Appendix A

Geocres 31D-431 dated June 28, 2007: Foundation Design Consultations
Concerning Abutment Foundations, Rehabilitation/Widening of Scugog River
Bridge, Highway 7, 0.6 km West of the Highway 7 and Highway 35/Kawartha Lakes
Road 15 Intersection, Site No. 32-096, G.W.P. 4264-04-00

(PML Ref.: 06HF092, dated June 28, 2007)



FOUNDATION DESIGN CONSULTATIONS
CONCERNING ABUTMENT FOUNDATIONS
REHABILITATION/WIDENING OF SCUGOG RIVER BRIDGE
HIGHWAY 7, 0.6 KM WEST OF THE HIGHWAY 7 AND
HIGHWAY 35/KAWARTHA LAKES ROAD 15 INTERSECTION
SITE NO. 32-096
G.W.P. 4264-04-00

for
MORRISON HERSHFIELD LIMITED

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PML Ref.: 06HF092
Geocres No. 31D-431
Index ID017
June 28, 2007

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Table 1 – Comparison of Foundation Construction Alternatives for Spread Footings

Table 2 – Comparison of Micro Pile and Spread Footing Foundation Alternatives

**FOUNDATION DESIGN CONSULTATIONS
CONCERNING ABUTMENT FOUNDATIONS**
for
Rehabilitation/Widening of Scugog River Bridge
Highway 7, 0.6 km West of the Highway 7 and
Highway 35/Kawartha Lakes Road 15 Intersection
Site No. 32-096
G.W.P. 4264-04-00

1. INTRODUCTION

This report provides supplementary Foundation Engineering comments and recommendations concerning detailed design and construction of the abutment foundations to support the proposed widening of the existing bridge on Highway 7 that crosses the Scugog River about 0.6 km west of the Highway 35/Kawartha Lakes Road 15 intersection near Lindsay, Ontario.

The report was prepared for Morrison Hershfield Limited (MHL) on behalf of the Ministry of Transportation of Ontario (MTO).

Foundation engineering comments and recommendations concerning design and construction of both the pier and abutment foundations, as well as a description of the subsurface conditions at the existing structure were provided in our Draft Report dated April 30, 2007. The April 30, 2007 Report should be read in conjunction with this document.

2. SUBSURFACE CONDITIONS

A brief description of the subsurface conditions at the abutment foundations, documented in the Draft Foundation Report, is summarized in the following table:

	Water Depth (m)	Elevation (m)	Soil Description	Composition of Soil at Founding Level
West Abutment ¹	1.5	248.6 to 247.9	Firm silty clay	Very dense sand
		247.9 to 246.1	Compact to very dense sand	
		246.1 to termination of drilling at 244.4	Very dense sand with gravel to sandy gravel	
East Abutment ²	NE ³	250.1 to 248.2	Very soft to stiff silty clay	Very dense sand and gravel
		248.2 to 247.6	Dense silty sand	
		247.6 to termination of drilling at 244.6	Dense to very dense sand and gravel	

1. Borehole drilled from barge.
 2. Borehole drilled on land.
 3. The borehole at the east abutment was drilled from land.
- NE Not encountered



The abutment footings are about 0.8 m wide and founded near elevation 247.0, about 1.6 m below the river bottom elevation at the west abutment and 2.0 to 2.5 m at the east abutment (from Geocres No. 31D-412 and September 2006 survey data shown on Drawing No. E0070PS1 dated February 2007 provided by email dated June 18, 2007 from Morrison Hershfield Limited).

Bedrock was identified at elevation 240.0 to 241.1 in three of the six boreholes drilled prior to construction of the existing bridge in 1956, and not defined in one borehole at elevation 238.6. Three boreholes terminated on bedrock or boulders at elevation 240.8 to 242.5 and one borehole in dense sand, gravel and boulders at elevation 240.0.

The river water level noted in the Foundation Report, Geocres No. 31D-412, and other documents provided with the RFP ranged from elevation 249.6 (1956) to 250.6 (2005). It is understood the Scugog River level for the 25, 50 and 100 year storm events is elevation 250.81, 250.84 and 250.88 respectively. Accordingly, a river water level elevation of 250.8 was employed for subsequent analysis and is considered appropriate for design purposes.

3. FOUNDATION CONSIDERATIONS

3.1 General

It is understood that a load of 4800 kN from the widened portion of the bridge will be supported by the abutment foundations.

Construction of spread footings or micro piles are considered to be feasible foundation alternatives for the abutments. The preferred option will be dictated by structural design and economic considerations as well as constructability issues related to dewatering to enable construction of the footings. Refer to Table 1 for a comparison of the foundation construction options for spread footings. Use of spread footings constructed in accordance with Option 2 a) noted in Section 3.2.1 is recommended from a foundation engineering perspective.

This report provides additional detailed comments concerning design and construction of spread footings and micro piles to support the abutments.



3.2 Spread Footings

It is recommended that footings constructed to support the abutments be founded at the same level as the existing footings (elevation 247.0). Footings bearing on the dense to very dense sand/sand and gravel/sandy gravel should be designed using a factored bearing resistance at ultimate limit states (ULS) of 735 kPa and 300 kPa at serviceability limit states (SLS). The SLS value is provided since there is a potential for disturbance to the soil below water during construction and a need for minimal settlement of the bridge widening following construction. In this regard, the structural loads will be imposed on the subgrade soil gradually as construction proceeds; the total settlement following completion of the bridge deck and application of the live loads should be in the order of 5 to 10 mm.

The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.7.4 of the CHBDC.

The horizontal force imposed on the foundations will be resisted in part by the friction force developed between the underside of the footing and the native subgrade soil. An unfactored friction factor of 0.7 is recommended.

Construction of spread footings founded at the same level as the existing abutment will require implementation of measures to control the inflow of water from the river as well as groundwater seepage from the soil (upward and laterally) into the work area.

3.2.1 Construction Considerations and Dewatering

The existing footing at the west abutment is founded about 1.6 m below the river bottom; the existing footing at the east abutment is 2.0 to 2.5 m below the river bottom. The river water level (and hence, the inferred piezometric level in the soil) is some 3.6 m above the founding level of both abutment footings.

The north and west/east sides of the existing abutment footings are contiguous with the river on the east/west abutments respectively.



The following options were considered to deal with water that enters the work area. The comments are intended to be for planning and design purposes. The tender documents should clearly state that control of water in the sheeted/excavated areas to enable construction to proceed in the dry is the Contractor's responsibility.

1. Installation of steel sheeting around the complete perimeter of the footings to sufficient depth to provide a cut-off for groundwater seepage and heave and/or piping at the base of the excavation.
2. Installation of steel sheeting around the complete perimeter of the footings and excavation of the soil within the sheeted area followed by placement of a concrete plug to balance the differential hydraulic pressure.
3. Installation of steel sheeting to support the 'river side' of the footings to an embedment depth dictated by toe restraint criteria to maintain the stability of the retention structure and excavate inclined slopes on the 'land side' of the excavations in conjunction with closely spaced well points (or equivalent) around the perimeter of the excavation and sheeted area.
4. Installation of sheeting with an embedment depth dictated by toe restraint criteria to maintain the stability of the retention structure in conjunction with closely spaced well points (or equivalent) around the perimeter of the sheeting.

Refer to Table 1 for comments concerning the advantages, disadvantages, costs, risks and consequences of each option:

It is visualized that the equipment employed to drive the sheeting will operate above the existing bridge deck where limited space exists for installation of the sheeting. Consequently, the clearance requirement should be reviewed to confirm that the sheeting can be installed.

Option 1:

In order to provide an adequate cut-off to control groundwater seepage and prevent basal heave and/or piping at the base of the excavation, the steel sheeting should extend to at least



elevation 244.5. It must be noted however, that steel sheeting installed using either impact or vibratory techniques, is likely to meet refusal near elevation 245.0 to 246.0.

Therefore, it will be necessary to implement special techniques to install the sheeting to the required depth. This could involve predrilling small diameter holes (200 mm or so) at the sheeting interlocks to loosen the soil before driving. The soil in the 'web' of the sheeting would not be disturbed by drilling and therefore would provide passive resistance for toe stability and groundwater seepage into the excavation area. Preferential seepage paths would however, be created at the location of the predrilled holes that would require measures to block the seepage.

Option 2:

This option involves placement of a concrete plug following excavation to elevation 247.0 (the founding level of the existing footings) (Option 2a) as well as excavation below elevation 247.0 and placement of the concrete plug (Option 2b). A Special Provision for Unwatering the Structure Excavation and Placement of Tremie Concrete will be developed when the foundation system is selected.

The sheeting should be driven to refusal, anticipated at elevation 245.0 to 246.0, for both options.

Option 2 a):

In order to balance the 3.6 m differential hydraulic pressure, it will be necessary to place a 1.5 m thick concrete plug on the subgrade at elevation 247.0. The water within the limits of the cofferdam could then be pumped and the reinforcing steel placed to construct the footing.

Option 2 b):

It is recognized that the top of the widened footings could be higher than the existing footings. Consideration could be given to extending the depth of excavation below elevation 247.0 to lower the top of the footing. It must be noted however, that the thickness of the concrete plug must be increased by 0.4 m for each 1 m increase in the depth of excavation below elevation 247.0. Since the sheeting is likely to meet refusal to further penetration near elevation 245.0 to 246.0, extending



the depth of excavation below elevation 247.0 may require 'toe pinning' to provide fixity of the toe of the sheeting. Further comments in this regard are described in Option 4.

Cognizant of the short period required for excavation and placement of the concrete plug, and the sides of the excavation adjacent to the existing footing will be retained by the sheeting, it is unlikely that extending the depth of excavation to elevation 246.0 will have an adverse impact on the performance of the existing footings. It must be noted however, that excavation below elevation 247.0 within the confines of the sheeting is likely to be very difficult.

Option 3:

Comments concerning the anticipated refusal depth of the sheeting and measures to provide toe fixity described in Options 1 and 4 respectively, apply to Option 3. This option will require installation of wells around the perimeter of the excavations to depress the piezometric level to at least 0.5 m below the base of the excavation to prevent basal heave/piping.

A skim coat of concrete should be placed on the subgrade following completion of the excavation.

Cognizant of the high permeability of the very dense sand/sand and gravel at this site, the well points should be installed at a spacing of about 1.5 m in 300 mm auger holes backfilled with pervious sandy soil extended to elevation 243.0 around the perimeter of the abutment excavations. The upper 500 mm of the auger holes in the water should be filled with bentonite or similar material to block the flow of water from the river into the well point auger hole.

Provided groundwater seepage through the sandy soils is intercepted before it reaches the excavation slopes, excavation side slopes inclined at 1 horizontal to 1 vertical are considered to be suitable.

Option 4:

The depth of embedment of the sheeting would be dictated by the depth required to achieve toe fixity for the sheeting to resist the lateral soil and water pressure.

If sufficient penetration cannot be achieved, toe fixity could be provided by installation of the sheeting in conjunction with pipe piles (say 300 mm diameter at a centre to centre spacing of 1.5 m) equipped with an interlock to connect to the sheeting. Installation procedures for the pipes would be the same as described for Option 1. Installation of well points will be required to depress the water level to prevent basal heave/piping; additional comments in this regard are provided in Option 3.

3.2.2 Design Parameters

The structural analysis conducted during detailed design of the sheet pile wall enclosure structure must consider the lateral pressure imposed on the sheet pile wall by the soil and water (including short term variations due to ship traffic, natural variations in the river water level, (storm events, seasonal and long term cycles), as well as the applied horizontal (ice) and vertical surcharge loads.

The lateral active and passive earth pressures (unfactored) imposed on the sheet pile wall should be computed using the following equations:

Active Pressure

$$p_a = K_a (\gamma h_1 + \gamma' h_2 + q) + \gamma_w h_2 + C_s$$

where K_a = active earth pressure coefficient (dimensionless)

γ = unit weight of retained soil
above the design water level (kN/m^3)

γ' = buoyant unit weight of soil
below design water level ($\gamma - \gamma_w$)

h_1 = depth from ground surface (m), to design water level

h_2 = depth below design water level (m)

q = surcharge load (kN/m^2)

γ_w = unit weight of water
= 9.8 kN/m^3

C_s = earth pressure induced by seismic events, kPa (refer to clause 4.6.4 of CHBDC)
where ϕ = angle of internal friction of retained soil (35° for granular soil)
 δ = angle of friction between the soil and wall (26° for granular soil)

Passive Pressure

$$p_p = \gamma_w h_2 + K_p (\gamma h_s + q) + C_s$$

where K_p = passive earth pressure coefficient

h_s = depth below base of excavation to toe of sheeting (m)

γ_w , γ , q , h_2 and C_s were defined previously.

Elevation	Soil Composition	Total Unit Weight (γ) (kN/m ³)	Effective Friction Angle (ϕ°)	Active Earth ¹ Pressure Coefficient (K_a)	Passive Earth ¹ Pressure Coefficient (K_p)
West Abutment					
248.6 – 247.9	Firm silty clay	18.0	25.0	0.40	2.5
247.9 – 247.3	Compact sand	18.8	32.0	0.31	3.2
247.3 – 246.1	Very dense sand	20.5	37.0	0.27	3.7
246.1 – 244.4	Very dense sand with gravel to sandy gravel	22.5	40.0	0.22	4.6
East Abutment					
250.1 to 248.2	Very soft to stiff silty clay	18.0	25.0	0.40	2.5
248.2 – 247.6	Dense sand/sand and gravel	20.5	37.0	0.27	3.7
247.6 – 244.6	Very dense sand and gravel	22.5	40.0	0.22	4.6

1. Wall friction ignored.

The factored passive resistance at ULS is $0.5 p_p$, the active pressure is unfactored (clause 6.9.1 of the Commentary to the Code).

The seismic site coefficient for the conditions at this site is 1.0 (Type I soil profile as per clause 4.4.6 of the CHBDC, CAN/CSA-S6-06, November 2006). The zonal acceleration ratio is 0.05.



The bridge is located in Seismic Performance Zone 1. The liquefaction potential of the silts and sands at the site was assessed using the procedure suggested by Seed et al (1984) and, on this basis, it is considered that liquefaction of these soils is unlikely (clause 4.6.2 of the CHBDC).

The potential for scour to occur is a function of river hydraulics and should be established by the hydraulic engineer.

3.3 Micro Piles

Detailed comments and recommendations concerning design of micro piles were provided in the April 30, 2007 report. Pertinent comments are summarized in the following paragraphs.

As noted in Section 2 of this report, the abutment footings are founded on very dense sand/sand and gravel, about 7 m above the bedrock surface.

It will be necessary to socket the micro piles into the bedrock to enable development of sufficient resistance to economically support the foundation loads.

For preliminary planning and design purposes, we have assumed the bedrock surface is at elevation 240.0; the underside of the pile cap constructed on the micro piles shown on the General Arrangement Drawing prepared by MHL and stamped 'Preliminary June 20, 2007' is elevation 245.36.

The axial resistance of the micro piles will primarily be developed by the bond stress developed along the portion of the micro pile socketted into bedrock.

Based on our general knowledge of the engineering properties of the bedrock at this site, the factored axial resistance at ULS of micro piles socketted 6 m into bedrock is considered to be at least 850 kN. This resistance is necessarily a lower bound value due to the limited site specific data concerning the properties of the bedrock on site; subject to the results of the supplementary Foundation Investigation recommended in the April 30, 2007 Draft Report, substantially higher values may be available for detailed design.



The SLS resistance is normally based on 25 mm movement of the founding medium. Considering the bedrock to be nonyielding, and the actual magnitude of movement required to fully mobilize the bond stress of 6 to 10 mm, the design will not be governed by settlement criteria. In addition, foundation loads will be imposed gradually as the structure is constructed, hence, the total settlement following completion of the bridge deck and implementation of the live loads should be less than 5 mm.

The micro piles should be installed and monitored in accordance with the requirements of MTO OPSS 903.

Resistance to lateral loads will be provided by the horizontal bearing resistance of the soil and underlying bedrock. The factored horizontal bearing resistance at ULS of the very dense granular material that overlies the bedrock and the underlying bedrock is considered to be:

very dense granular soil	500 kPa
bedrock	500 kPa

This bedrock resistance is necessarily a lower bound value due to the limited site specific data concerning the properties of the bedrock on site; subject to the results of the supplementary Foundation Investigation recommended in the April 30, 2007 Draft Report, substantially higher values may be available for detailed design.

4. RECOMMENDATIONS

Refer to Table 2 for a comparison of the advantages, disadvantages, costs, risks and consequences of the spread footings and micro pile alternatives. It is recommended from a foundation engineering perspective that the abutment loads are supported by spread footings and Option 2 a) (Table 1) is implemented to control water during construction of the footings.


5. CLOSURE

The report was prepared by Mr. Dennis W. Kerr, MEng, P.Eng., Chief Foundation Engineer. Mr. Brian R. Gray, MEng, P.Eng., MTO Designated Contact, carried out an independent review of the report.

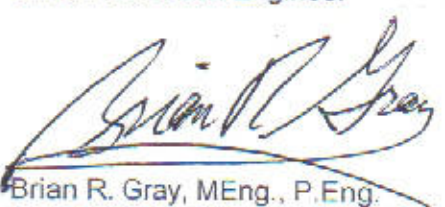
Sincerely

Peto MacCallum Ltd.




Dennis W. Kerr, MEng., P.Eng.
Chief Foundation Engineer




Brian R. Gray, MEng., P.Eng.
MTO Designated Contact

DWK:lad

Table 1

Comparison of Foundation Construction Alternatives for Spread Footings¹

FOUNDATION TYPES	ADVANTAGES	DISADVANTAGES	RISKS/CONSEQUENCES
Option 1 Spread footings ² constructed within the limits of a cofferdam; cofferdam installed to sufficient depth to control groundwater seepage and prevent basal heave; water removed from within limits of cofferdam by pumping	<ul style="list-style-type: none"> Compatible with existing foundation system 	<ul style="list-style-type: none"> Need to construct cofferdam to control both groundwater seepage and river water flow into the work area to enable construction in the dry Need to employ special construction techniques to install cofferdam to adequate depth due to the presence of very dense granular deposit Potential for basal heave and/or piping due to differential water level between the interior and exterior of the cofferdam 	<ul style="list-style-type: none"> Inability to adequately limit water ingress into cofferdam and the need for extensive pumping Inability to depress piezometric level at base of excavation due to very dense native granular soil which could result in basal heave/piping Inability to install cofferdam to sufficient depth into very dense granular soil to provide cut-off to prevent basal heave/piping Potential for damage to cofferdam during installation into very dense granular soil which would limit the effectiveness of the cofferdam
Option 2 a) Spread footings constructed within the limits of a cofferdam and founded at elevation 247.0; cofferdam driven to refusal; water level maintained at river level during excavation to subgrade level; concrete plug placed to balance differential hydraulic pressure and prevent basal heave before pumping of water	<ul style="list-style-type: none"> Compatible with existing foundation system Control of water essentially limited to seepage through the cofferdam Special construction techniques to install cofferdam in very dense granular soil not required Basal heave/piping not a concern Placement of the reinforcing steel will be done in the dry within the limits of the cofferdam 	<ul style="list-style-type: none"> Excavation is done below water level QVE requires diver to conduct site review following excavation Top of footing may be above river level 	<ul style="list-style-type: none"> Excavation below the water level could result in undetected deficiencies and poor performance of the footings

NOTES:

- Qualitative cost comparisons (least to greatest): Option 2 a), 2 b), 1, 3, 4.
- Foundation level same as existing abutments: elevation 247.0

Table 1

Comparison of Foundation Construction Alternatives for Spread Footings¹

FOUNDATION TYPES	ADVANTAGES	DISADVANTAGES	RISKS/CONSEQUENCES
Option 2 b) Spread footings constructed within the limits of a cofferdam founded below elevation 247.0; cofferdam driven to refusal; water level maintained at river level during excavation to subgrade level; concrete plug placed to balance differential hydraulic pressure and prevent basal heave before pumping of water	<ul style="list-style-type: none"> Compatible with existing foundation system Control of water essentially limited to seepage through the cofferdam Top of footing will be at same level as existing footing Special construction techniques to install cofferdam in very dense granular soil not required Basal heave/piping not a concern Placement of the reinforcing steel will be done in the dry within the limits of the cofferdam 	<ul style="list-style-type: none"> Excavation is done below water level Excavation extends below founding level of existing footings QVE requires diver to conduct site review following excavation 	<ul style="list-style-type: none"> Excavation below the water level could result in undetected deficiencies and poor performance of the footings Excavation will extend below level of existing footing which creates a risk of an adverse impact on the existing footing
Option 3 Installation of steel sheeting on the 'river side' of the excavation and inclined slopes on the 'land side' of the excavation; cofferdam driven to refusal; groundwater control measures below the subgrade implemented to prevent basal heave; groundwater control measures to deal with water seepage from excavation slopes	<ul style="list-style-type: none"> Compatible with existing foundation system Reduced length of cofferdam 	<ul style="list-style-type: none"> Need to install groundwater control measures to prevent basal heave Need to implement groundwater control measures to deal with seepage from open cut side of the excavation Potential instability of cut slopes due to groundwater seepage Potential for basal heave and/or piping due to differential water level between the interior and exterior of the cofferdam 	<ul style="list-style-type: none"> Inability to adequately limit water ingress into cofferdam and the need for extensive pumping Inability to depress piezometric level at base of excavation due to very dense native granular soil which could result in basal heave/piping Need to extend length of cofferdam and/or groundwater control measures to control groundwater seepage and/or inflow of river water due to unstable slopes

NOTES:

1. Qualitative cost comparisons (least to greatest): Option 2 a), 2 b), 1, 3, 4.
2. Foundation level same as existing abutments: elevation 247.0

Table 1

Comparison of Foundation Construction Alternatives for Spread Footings¹

FOUNDATION TYPES	ADVANTAGES	DISADVANTAGES	RISKS/CONSEQUENCES
Option 4 Spread footings ² constructed within the limits of a cofferdam; groundwater control measures below the subgrade implemented to prevent basal heave; water removed from within limits of cofferdam by pumping	<ul style="list-style-type: none"> Compatible with existing foundation system 	<ul style="list-style-type: none"> Need to implement groundwater control measures to depress piezometric level in subgrade soil within the cofferdam to prevent basal heave Potential for basal heave and/or piping due to differential water level between the interior and exterior of the cofferdam 	<ul style="list-style-type: none"> Inability to adequately limit water ingress into cofferdam and the need for extensive pumping Inability to depress piezometric level at base of excavation due to very dense native granular soil which could result in basal heave/piping

NOTES:

1. Qualitative cost comparisons (least to greatest): Option 2 a), 2 b), 1, 3, 4.
2. Foundation level same as existing abutments: elevation 247.0

Table 2
Comparison of Micro Pile and Spread Footing Foundation Alternatives¹

FOUNDATION TYPES	ADVANTAGES	DISADVANTAGES	RISKS/CONSEQUENCES
<p>Spread footings² constructed within the limits of a cofferdam OR cofferdam on 'water' side of excavation and excavation slopes on the 'land' side of the excavation; cofferdam installed to sufficient depth to control groundwater seepage and prevent basal heave; groundwater control measures below the subgrade implemented to prevent basal heave; water removed from within limits of cofferdam by pumping</p>	<ul style="list-style-type: none"> Compatible with existing foundation system 	<ul style="list-style-type: none"> Need to construct cofferdam to control both groundwater seepage and river water flow into the work area to enable construction in the dry Need to employ special construction techniques to install cofferdam to adequate depth due to the presence of very dense granular deposit Need to implement groundwater control measures to depress piezometric level in subgrade soil within the cofferdam to prevent basal heave Potential for basal heave and/or piping due to differential water level between the interior and exterior of the cofferdam Excavation may be done below water level QVE requires diver to conduct site review following excavation Top of footing may be above river level 	<ul style="list-style-type: none"> Inability to adequately limit water ingress into cofferdam and the need for extensive pumping Inability to depress piezometric level at base of excavation due to very dense native granular soil which could result in basal heave/piping Inability to install cofferdam to sufficient depth into very dense granular soil to provide cut-off to prevent basal heave/piping Potential for damage to cofferdam during installation into very dense granular soil which would limit the effectiveness of the cofferdam Excavation below the water level could result in undetected deficiencies and poor performance of the footings

NOTES:

- Foundation level same as existing abutments: elevation 247.0; refer to Table 1 for detailed comments concerning construction procedures and options to control water.
- Qualitative cost comparison (least to greatest): spread footings; micro piles.



Table 2
Comparison of Micro Pile and Spread Footing Foundation Alternatives

FOUNDATION TYPES	ADVANTAGES	DISADVANTAGES	RISKS/CONSEQUENCES
Micro Piles	<ul style="list-style-type: none"> Control of water essentially limited to seepage through the cofferdam Basal heave/piping can be controlled by placement of a concrete pad Could be installed with barge mounted equipment Could be socketted into bedrock and therefore, scour action would not undermine the foundation system Compatible with existing foundation system Resistance to lateral loads can be provided by very dense granular soil that overlies bedrock 	<ul style="list-style-type: none"> Management of excavated material to prevent entry into the river Potential for difficulties to be encountered during drilling through the very dense granular material that overlies bedrock 	<ul style="list-style-type: none"> Augering difficulties that could result in construction delays and cost overrun Undermining of pile cap by scour that could expose the mini piles

NOTES:

- Foundation level same as existing abutments: elevation 247.0; refer to Table 1 for detailed comments concerning construction procedures and options to control water.
- Qualitative cost comparison (least to greatest): spread footings; micro piles.



Appendix B

Special Provisions for:

Protection System

Unwatering Structure Excavation

Earth Excavation for Structure

Tremie Concrete; High Density Tremie Concrete

PROTECTION SYSTEM - Item No.

Special Provision

SCOPE

Work under this tender item constitutes protection systems as shown on the contract drawings, including but not limited to:

1. Roadway protection;
2. Cofferdams to facilitate construction of foundations for the bridge widening.

CONSTRUCTION

Protect Existing Foundations

The construction of temporary protection systems shall not cause settlement of the existing foundations.

The Contractor shall verify and locate the edge of existing footings prior to constructing protection system, as specified elsewhere in the Contract.

Subsurface Conditions

The Contractor is alerted that the subsurface soils include very dense sand and gravel deposits with cobbles and boulders. In addition, the Contractor is alerted that the cohesionless soils that comprise this deposit are highly susceptible to conditions of unbalanced hydrostatic head and will slough and boil under unbalanced hydrostatic head conditions. The Contractor shall implement appropriate construction procedures to prevent unbalanced hydrostatic head conditions and facilitate the construct of new foundations while preventing disturbance to existing foundations.

Removal of Protection System

Sheetpile cofferdam protection systems around foundations may be left in place; they shall be cut flush with top of tremie concrete or top of concrete as shown on the contract drawings.

Roadway protection sheetpiles may be are left in place; the top shall be removed to at least 1.2m below finished grade.

BASIS OF PAYMENT

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment and materials to do the work.

UNWATERING STRUCTURE EXCAVATION - Item No.

Special Provisions

SCOPE

Unwatering Structure Excavation applies to unwatering the cofferdam protection systems for new bridge foundations prior to constructing abutment footings and pier pile caps.

Also included under this item are sounding surveys to confirm underwater elevations at various stages of construction.

SUBMISSION AND DESIGN REQUIREMENTS

Permits and Approvals for Temporary Works

Permits and approvals have been obtained for the permanent work in the Contract and for the conceptual design of temporary work as shown on the Contract Drawings

Based on the design and the assumed method of construction, it has been determined that the unwatering rate will be less than 50,000 litres/day when stable conditions have been achieved. Accordingly, a Permit to Take Water (PTTW) has not been obtained for this project. The Contractor is alerted that without a PTTW, the pump rate shall not exceed 50,000 litres/day (including the period prior to achieving steady state).

Prior to commencing work in this Contract, the Contractor shall verify the need for a Permit to Take Water, whether the conceptual design or alternative Contractor designs are used.

The Contractor shall be solely responsible for all costs and impacts of any additional permits or approvals that may be required for temporary works that differ from the Contract Drawings and assumed construction methods.

Design Considerations

The Contractor is alerted that the tremie plug for the pier foundations is subject to an unbalanced hydrostatic head and relies on anchorage to the micropiles to resist uplift. The Contractor shall not unwater the pier foundation cofferdams before the stipulated curing period or until the stipulated minimum strength has been achieved.

CONSTRUCTION

The Contractor shall perform survey on a 2.0m by 2.0m (maximum) grid using a total station survey to confirm underwater elevations at the foundation locations at various stages of construction, including:

- a. preconstruction survey extending 2m beyond the perimeter of the foundation, submit results to Contract Administrator 2 weeks prior to installing cofferdam;
- b. preconstruction survey (which may involve probing or boring but not open excavation) to confirm limits of existing footings including top elevation and location of edge adjacent to the excavation, submit results to Contract Administrator 2 weeks prior to installing cofferdam;

- c. confirmation of excavation depth, a certificate of conformance shall be submitted by the Quality Verification Engineer to the Contract Administrator prior to commencement of subsequent activity;
- d. confirmation of tremie concrete elevation, a certificate of conformance shall be submitted by the Quality Verification Engineer to the Contract Administrator prior to commencement of subsequent activity.

The Contractor shall take measures to prevent the water removed by the unwatering operation from entering the watercourse. It shall be pumped to an on-shore settlement pond. The Contractor shall provide details of the proposed methods of preventing displaced water from entering the watercourse.

The Contractor is advised of OWRA discharge threshold.

BASIS OF PAYMENT

Payment at the contract price for the above tender item shall include full compensation for all labour, equipment, and materials to do the work.

EARTH EXCAVATION FOR STRUCTURE - Item No.

Special Provisions

SCOPE

Earth Excavation for Structure applies to excavation for the bridge structure as shown on the contract drawings. Work under this item includes but is not limited to:

1. Excavation (wet) within the cofferdams for pier and abutment foundations;
2. Excavation for new abutments;
3. Excavation behind existing abutments to accommodate partial removal/reconstruction of abutments and approach slabs.

CONSTRUCTION

Excavation and handling of excavated materials within the watercourse shall be conducted in a manner that minimizes dispersion of sediments. The contractor shall take such measures and provide such containment system or systems as required to prevent entry of any sediments from stockpiled excavation materials from entering the watercourse.

At the conclusion of the work, the control measures shall be removed.

BASIS OF PAYMENT

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment and materials to do the work.

TREMIE CONCRETE - Item No.

Special Provision

SCOPE

Tremie Concrete applies to the construction of tremie concrete plugs for the abutment widening foundations and the pier widening foundations as shown on the contract drawings.

CONSTRUCTION

Tremie concrete shall be placed in a continuous, uninterrupted operation. All the concrete required for a placement shall be on site at the start of placement.

Tremie concrete shall be overbuilt at least 75mm above the design elevation. After unwatering, the Contractor shall remove concrete to the stipulated elevation using partial depth concrete removal methods.

Operational Constraints

The Contractor shall take measures to prevent the water displaced during the placement of the concrete from entering the watercourse.

BASIS OF PAYMENT

Payment at the contract unit price for the above tender item shall include full compensation for all labour, equipment, and materials to do the work.