Peto MacCallum Ltd.
CONSULTING ENGINEERS

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

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for
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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

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

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

Cs = earth pressure induced by seismic events, kPa (refer to clause 4.6.4 of CHBDC)
where Ø = 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

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

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 Cs were defined previously.

Elevation	Soil Composition	Total Unit Weight (γ) (kN/m^3)	Effective Friction Angle (Ø')	Active Earth[1] Pressure Coefficient (Ka)	Passive Earth[1] Pressure Coefficient (Kp)
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 all (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 nonyeilding, and the actual magnitude of movement required to fully mobilize the bond stress of 6 to 10 mm, the design will not 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 m.

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.

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Chief Foundation Engineer

Brian R. Gray, MEng., P.Eng.
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Table 1

Comparison of Foundation Construction Alternatives for Spread Footings[1]

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	• Compatible with existing foundation system	• 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	• 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	• 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	• Excavation is done below water level • QVE requires diver to conduct site review following excavation • Top of footing may be above river level	• Excavation below the water level could result in undetected deficiencies and poor performance of the footings

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 Footings[1]

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	• 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	• Excavation is done below water level • Excavation extends below founding level of existing footings • QVE requires diver to conduct site review following excavation	• 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	• Compatible with existing foundation system • Reduced length of cofferdam	• 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	• 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 Footings[1]

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	• Compatible with existing foundation system	• 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	• 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 Alternatives[1]

FOUNDATION TYPES	ADVANTAGES	DISADVANTAGES	RISKS/CONSEQUENCES
Spread footings[2] 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	• Compatible with existing foundation system	• 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	• 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:
1. Foundation level same as existing abutments: elevation 247.0; refer to Table 1 for detailed comments concerning construction procedures and options to control water.
2. 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	• 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	• 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	• 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 247.0; refer to Table 1 for detailed comments concerning construction procedures and options to control water.
2. Qualitative cost comparison (least to greatest): spread footings; micro piles.