



PRELIMINARY FOUNDATION DESIGN REPORT

for

**FEASIBILITY STUDY FOR THE REHABILITATION OR REPLACEMENT
OF THE HIGHWAY 3 BRIDGE OVER THE GRAND RIVER AT CAYUGA
GWP 3501-01-00, SITE 9-43
TOWNSHIP OF NORTH CAYUGA, DISTRICT 31**

PETO MacCALLUM LTD.
45 BURFORD ROAD
HAMILTON, ONTARIO
L8E 3C6
Phone: (905) 561-2231
Fax: (905) 561-6366
Email: hamilton@petomaccallum.com

Distribution:

- 5 cc: McCormick Rankin Corporation for distribution
to Ministry of Transportation (+ digital copy)
- 1 cc: McCormick Rankin Corporation for distribution
to MTO, Pavements and Foundations Section
(+digital copy)
- 2 cc: McCormick Rankin Corporation (+digital copy)
- 1 cc: PML Hamilton
- 1 cc: PML Toronto

PML Ref.: 04HF058A
Index No. 016FDR
Geocres No: 30L13-18
July 9, 2009



TABLE OF CONTENTS

1. INTRODUCTION1

2. FOUNDATIONS.....2

 2.1 Rehabilitation/Replacement on Existing Alignment.....2

 2.2 Replacement on New Alignment4

3. CONTROL OF WATER6

4. ADDITIONAL STUDIES7

5. CLOSURE8

Table 1 – Foundation Alternative Considerations

PRELIMINARY FOUNDATION DESIGN REPORT
for
Feasibility Study for the Rehabilitation or Replacement
Of the Highway 3 Bridge over the Grand River at Cayuga
GWP 3501-01-00, Site 9-43
Highway 3, District 31

1. INTRODUCTION

This report provides preliminary foundation engineering comments and recommendations regarding the proposed rehabilitation/replacement of the existing bridge on Highway 3 that crosses over the Grand River near the west limit of Cayuga, Ontario. The study was conducted for McCormick Rankin Corporation on behalf of the Ministry of Transportation of Ontario (MTO).

The existing structure is a five span steel through truss bridge; each span is approximately 37.7 m long. The roadway is approximately 7.0 m wide and accommodates two lanes of traffic. Pedestrian traffic is accommodated on a concrete sidewalk on the north side of the structure. The bridge was originally constructed in 1924 and rehabilitated in 1976. Additional rehabilitation was undertaken in 2001 to repair corroded truss members.

The river is about 180 m wide at the location of the existing structure and the water level, although subject to seasonal/weather dependent precipitation, is normally about elevation 176, corresponding to about a 2 to 3 m water depth. The top of the bridge deck is at about elevation 182.8, typically 7 m above normal river level.

Boreholes drilled through the piers and into the underlying bedrock indicate the piers are founded on a relatively thin sand and gravel layer underlain by dolostone bedrock.



2. FOUNDATIONS

2.1 Rehabilitation/Replacement on Existing Alignment

Subject to the comments in the following paragraphs, re-use of the existing piers to support the structural loads of the rehabilitated/replacement structure is considered suitable for preliminary design and planning purposes from a foundation engineering perspective.

Except for the upper 600 mm in Pier 3, the concrete in the piers is considered to be of good quality; the unconfined compressive strength ranged from 32.9 to 62.0 MPa. However, cognizant of the age of the concrete (+80 years), it is possible that it will not meet the current CSA exposure class requirements. Therefore, a more comprehensive assessment of the condition of the concrete should be carried out prior to final design.

The sand and gravel deposit identified between the pier footing and bedrock ranges in thickness from 0.2 to 2.2 m, generally increasing from west to east. Since coring equipment was employed to advance the boreholes, in situ tests were not carried out to evaluate the density of this relatively thin deposit. Based on the information provided on the structural drawing prepared for the existing bridge in July 1923 (Drawing No. 1160, Proposed Bridge over Grand River, Talbot Street, Village of Cayuga, County of Haldimand, Station 820+31.44 to 827+67.38 revised July 24, 1923) as well as the resistance during coring, the sand and gravel was judged to be dense to very dense. The pertinent engineering properties of the sand and gravel deposit should be confirmed prior to final design.

The bedrock within the total depth of coring (2.0 to 4.8 m) is considered to be of very poor quality since:

- core typically less than 50 mm long;
- RQD typically 0;
- recovery typically 60 to 83%;
- vuggy;



- voids up to 600 mm identified;
- unconfined compressive strength of light brown to buff layers (the most predominant bedrock unit identified) is about 33 MPa; the unconfined compressive strength of the dark grey layers is about 150 MPa.

It is recommended therefore that the rock is cored to substantially greater depth during detail design to determine whether 'competent' bedrock exists at reasonable depth for construction of the bridge foundations.

We understand the stress imposed on the subgrade material below the existing pier footings is in the order of 750 kPa and there are no reports and/or evidence of settlement/poor performance of the existing foundations. It is considered therefore, that the existing footings could be employed to support the foundation loads if the bridge is constructed/rehabilitated on existing alignment provided measures, such as injecting cementitious grout to fill the voids and other discontinuities in the rock are implemented. Use of a factored bearing resistance at ultimate limit states (ULS) of 1500 kPa is considered to be appropriate for preliminary design if measures are implemented to improve the rock quality.

Since the stress imposed on the rock below the footings is currently about 750 kPa and the rock will essentially be nonyielding after measures are implemented to improve the rock quality, it is anticipated that the design will be governed by the ULS bearing resistance and the resistance at SLS will not apply. This should be assessed further during detail design.

It is visualized that micropiles could be installed through/adjacent to the existing footings to increase the bearing resistance of the composite foundation system (footing augmented with micropiles) if required to support higher loads. This would require rigorous structural analysis of the capability of the pier/foundation system to effectively transfer loads to the micropiles and could be considered during detail design if required.

For preliminary design/planning purposes, grout injected to enable re-use of the existing footings should be applied to a depth of about 4.5 m (equivalent to 1.5 times the footing width) or 3 m into bedrock whichever is greater. It is visualized that grout application points spaced on an approximate 2 m grid should be suitable and the grout can be injected from a barge.



The limits of the grout application in plan will be dictated in part by the thickness of granular material below the footing as noted in the following table.

Pier	Granular Thickness Below the Footing (m)	Plan Area of Grout Application ¹ (m)	Depth of Rock to be Grouted ² (m)	Anticipated Grout Volume (m ³)
1	0.2	5.2 x 13.2	4.3	30
2	1.2	6.2 x 14.2	3.3	30
3	2.2	7.2 x 15.2	3.0	35
4	1.6	6.6 x 14.6	3.0	30

1. Based on footing dimensions of 3 m by 11 m plus 1 m projection on all sides of footing plus 1 horizontal to 2 vertical (1H:2V) stress distribution through the granular material to the rock surface.
2. 4.5 m minus granular thickness or 3 m, whichever is greater.

If expansion of the footing area is required to accommodate the foundation loads, the plan area and depth of grouting should be increased using the criteria noted above.

The preferred means of improving the quality of the bedrock, the plan area and depth of bedrock improvement, the potential environmental impacts of grout injection on water quality, means of ensuring complete infilling of 'voids'/other discontinuities in the rock and the merits of a composite footing/micropile foundation system, as well as the volume of grout required and measures to control water entering the work area should be investigated further during detailed design.

Subject to the measures employed to improve the bedrock quality, use of a much higher ULS bearing resistance value during final design should be feasible.

2.2 Replacement on New Alignment

Construction of new footings founded on bedrock, rock socketted caissons or micropiles socketted into bedrock are considered to be feasible foundation alternatives for the piers and 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 foundations.



It is considered that driven piles are not a suitable foundation system due to the poor rock quality and the limited depth of penetration into bedrock that could be achieved.

Additional comments concerning the advantages, disadvantages, costs, risks and consequences of the footing and rock socketted caisson alternatives are provided in Table 1; design/construction recommendations are provided in the following paragraphs. From a foundation engineering perspective, it is recommended that rock socketted caissons are employed to support the foundation loads due to the poor quality rock revealed in boreholes drilled to date and the need to improve the rock quality for construction of footings. This opinion is based on the available information and should be reviewed during detail design. Micropiles are not considered in the Table and the following technical discussion since they are a proprietary product and will require additional subsurface information for a learned assessment.

Use of footings founded on bedrock to support the foundation loads of a structure constructed on a new alignment or foundations to support a complete new structure on existing alignment is considered to be suitable if the measures noted in Section 2.1 to improve the rock quality below the footings are implemented. Use of a factored bearing resistance at ULS of 1500 kPa to proportion the footings is considered to be appropriate for preliminary design. It is anticipated that the design will be governed by the ULS bearing resistance since the rock will be non yielding when measures to improve the rock quality are implemented and the resistance at SLS will not apply. This should be assessed further during detail design. Subject to the measures employed to improve the bedrock quality, use of a much higher value during detail design may be feasible. This should be assessed further during detail design.

Considering the poor quality of the rock, end bearing should be ignored and the design of rock socketted caissons based on shaft resistance only.

For preliminary design purposes, the length and diameter of the caisson socket should be computed using a factored axial resistance at ULS of 300 kPa. The factored resistance at ultimate limit states for selected caisson diameters and socket length considered to be suitable for preliminary design and planning purposes are provided in the following table. This should be investigated further during detailed design.



Caisson Diameter (m)	Factored Axial Resistance at ULS (kN)		
	Socket Length (m)		
	1.5	2.5	3.5
0.76	1075	1790	2500
0.91	1285	2140	3000

For preliminary planning and design purposes, the grout should be applied to a depth of the socket length plus two caisson diameters over an area of the socket diameter plus 2 m (4.76 and 4.91 m² for the 0.76 and 0.91 m diameter caissons, respectively).

The assessed volume of grout take for the caissons is:

Caisson Diameter (m)	Plan Area of Grout Application (m ²)	Anticipated Grout Volume (m ³)		
		Socket Length (m)		
		1.5	2.5	3.5
0.76	4.76	2.0	2.5	3.0
0.91	4.91	3.0	3.7	4.5

3. CONTROL OF WATER

Measures to control water inflow from the Grand River as well as upward through the rock will be required if the design calls for widening of the footings or other construction activities below the water level in the Grand River. It is visualized that this can be accomplished by:

- injection of grout into the rock below the work area to cut off seepage paths through the rock and prevent/minimize vertical flow into the work area.
- construction of a cofferdam around the work area (steel sheeting).
- placement of tremie concrete within the cofferdam to prevent seepage of water from the river through the joint between the sheeting and the bedrock.

Conventional sump pumping techniques from within the cofferdam should be sufficient to handle any water that enters the work area.



A large volume of water is likely to enter the excavation made in the rock for construction of caissons if employed to support the foundation loads. A liner could be installed to prevent the flow of water through the walls of the rock socket. However, the liner will not prevent ingress of water from the base of the excavation, therefore use of tremie techniques to place the concrete will be required. Alternatively, the rock could be grouted to prevent/minimize the volume of water entering the excavation. The preferred means of water control should be assessed during detailed design.

4. ADDITIONAL STUDIES

The foundation investigation carried out for detailed design should include:

1. A comprehensive assessment of the condition of the concrete, the CSA exposure class of the concrete, resistance to deterioration from freeze/thaw and wet/dry cycles as well as air void analysis if the piers are to be re-used.
2. Assessment of the pertinent engineering properties of the thin sand and gravel deposit that overlies the bedrock.
3. Detailed assessment of the bedrock to determine the extent and presence of open joints/voids in the bedrock, the hydraulic conductivity of the bedrock mass, measures to improve the rock quality and control water.
4. Assessment of the potential impacts to water quality and measures to confirm complete infilling of voids and other discontinuities in the rock if injection grouting is selected to improve the rock quality.
5. The potential for scour at the piers and abutments.
6. The foundation conditions at the east and west abutments as well as the approach embankments to the bridge.



5. CLOSURE

This report was prepared by Mr. P.A. Lyall, BEng., Mr. G.O. Degil, Ph.D., P.Eng., Senior Foundation Engineer, and reviewed by Mr. D.W. Kerr, MEng., P.Eng., Chief Foundation Engineer. Mr. B.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

PL/DWK:lad



Table 1

Foundation Alternative Considerations
 Replacement of Highway 3 Bridge over Grand River in Cayuga
 GWP 3501-01-00, Site 9-43

FOUNDATDION TYPE	ADVANTAGES	DISADVANTAGES	RISKS/CONSEQUENCES
Footings Founded on Bedrock	Proven technology at this site since foundations of existing bridge have performed satisfactorily for 85 years	Excavation below water level	Potential for environmental impacts to implement measures to improve rock quality
	Ability to resist lateral loads	Need to construct cofferdam to prevent river water from entering excavation area	Potential for undetected voids/discontinuities to exist in bedrock after measures implemented to improve rock quality
	Ease of construction following implementation of measures to improve rock quality	Need to implement measures to prevent entry of water through the base of the excavation	Inability to adequately limit water ingress into the cofferdam and the need for extensive pumping
		Need to implement measures to improve the quality of the bedrock below the footings	
		Possible need to excavate bedrock so top of footing does not impact water flow	
		QVE requires diver to do site review during construction	
		Uncertainty associated with confirmation of effectiveness of measures implemented to improve rock quality	



Table 1

Foundation Alternative Considerations
 Replacement of Highway 3 Bridge over Grand River in Cayuga
 GWP 3501-01-00, Site 9-43

FOUNDATDION TYPE	ADVANTAGES	DISADVANTAGES	RISKS/CONSEQUENCES
Rock Socketted Caissons	Control of water could be provided by casing installed to construct caisson OR installation of a cofferdam and placement of a concrete pad on the bedrock surface to allow construction of the pile cap 'in the dry'	Management of excavated material to prevent entry into the river	Drilling difficulties that could result in construction delays and additional costs
	Large surface area in contact with bedrock to minimize impacts of voids/discontinuities in rock	Potential for difficulties to be encountered drilling into the poor quality bedrock	
	Could be socketted into bedrock and therefore scour action would not undermine the foundation system	Ability to resist lateral loads	