



DETAIL FOUNDATION DESIGN REPORT

for

**REPLACEMENT OF MEADOW CREEK BRIDGE
HIGHWAY 577, SITE 39E-077
G.W.P. 181-92-00
COCHRANE DISTRICT, IROQUOIS FALLS**

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for

Replacement of Meadow Creek Bridge

Highway 577, Site 39E-077

G.W.P. 181-92-00

Cochrane District, Iroquois Falls

1. INTRODUCTION

This report provides foundation engineering comments and recommendations regarding design and construction of foundations, abutments and approach fill embankments for the proposed replacement of an existing single-lane 12-span bridge over Meadow Creek located on Highway 577 (Monteith Road) south of the Town of Iroquois Falls, Ontario. The investigation was conducted for Stantec Consulting Ltd. (Stantec) on behalf of the Ministry of Transportation of Ontario (MTO).

The proposed two-lane bridge is envisaged to be a two-span structure with a total length of 90.0 m and width of 12.2 m constructed at the location of the existing 74.2 m long bridge so that the new abutments will be about 5 m to the north and 10 m to the south of the corresponding abutments of the existing structure. Highway 577 crosses Meadow Creek at approximate Sta. 19+893, Highway 577 chainage (Ref. Preliminary General Arrangement drawing prepared by Stantec in December 2008).

The project also involves the realignment of the highway to accommodate a speed limit of 80 km/hr resulting in cut and fill slopes to the south and north of the structure within a distance of approximately 450 m south and 250 m north of the bridge. The recommendations for the deep cuts resulting from the roadway realignment are provided under a separate report prepared by Peto MacCallum Ltd. (PML) with Ref. No. 08TF009-2.



The current plans call for the centreline of the proposed two-lane bridge to be some 2.5 m to the east of the existing centreline. After one half of the new bridge is constructed immediately east of the existing structure, traffic will be diverted to it and the existing one-lane bridge will be demolished. The west half of the new structure will then be constructed.

The existing road grade on Highway 577 at the bridge location is near elevation 251.5 at the south abutment and elevation 252.0 at the north abutment based on the survey data shown on the drawing referenced above. The existing approach embankments are about 4 m high above the water level in the Meadow Creek, which is controlled by upstream and downstream dams at approximate elevation 248.

The new road grade will be at elevation 252.4 at the south abutment and 253.1 at the north abutment, thereby increasing the height of the approach embankments by about 0.9 m and 1.1 m.

In summary, the subsurface stratigraphy revealed in the boreholes drilled at the site generally included surficial fill from the roadway pavement or topsoil. The fill was mostly made up of cohesive clays and had a thickness of 7.3 and 9.1 m at the south and north abutments, respectively. Below the fill, a cohesive 14.5 to 16.8 m thick deposit of firm to stiff clayey soils extended to levels varying between elevations 225.6 and 229.4. The clayey soils were underlain by compact to very dense cohesionless soils containing cobbles and boulders. The groundwater level measured in the boreholes put down in the approaches to the structure was at elevation 243.9 to 251.7. It is worth noting that a layer of rockfill covering the bottom of the creek was detected at the pier location during the preliminary investigation.

Based on the available information, design and construction of foundations to support the replacement bridge is considered feasible at the site.



Cognisant of the relatively low bearing resistance of the native cohesive soil, the presence of cohesionless soils and cobbles / boulders below the groundwater level, it is not considered to be feasible to employ either spread footings or caissons for supporting the proposed structure foundations. Use of end-bearing piles driven into the very dense sandy soils is considered to be the preferred foundation system from a foundation engineering perspective.

The presence of existing timber piles for the existing bridge must be taken into account during design of the proposed structure. From a foundation engineering perspective, however, removal of the existing piles after the new piles are installed is unnecessary. A program of monitoring of the existing piles should be implemented as recommended in Section 2 of this report. Installation of H-piles which cause significantly lower soil displacement than other driven piles, such as tube piles, should not affect the existing piles.

The staged construction of the bridge will require the installation of a temporary roadway protection system behind each abutment.

Construction of the approach embankment widening behind the north and south abutments will be feasible, including the planned 0.9 and 1.1 m grade raises. These widenings will include placement of rock fill in the river to support the new earth fill for each widening. An advance contract to place the fills and allow for pre-loading and partial settlement of the fill is recommended in this report.

The "red flag" issues outlined in the preceding paragraphs and the recommended methods of overcoming these issues noted in the following sections of the report are intended to alert and aid the designer and the contractor. These comments and recommendations are based on the conditions revealed during the investigations and no responsibility is assumed by the consultants or the MTO for alerting the contractor to all critical issues for each foundation alternative. The requirements to deliver acceptable construction quality remain the responsibility of the contractor.



The foundation frost penetration depth at this site is 2.4 m according to OPSD-3090.100. The seismic site coefficient for the conditions at the site is 1.0 – Type I soil profile as per clause 4.4.6 of the Canadian Highway Bridge Design Code (CHBDC) 2006 Edition – for the anticipated foundation conditions.

The standard specifications referenced in this report are listed in Table 1.

2. FOUNDATIONS

The foundation types considered, their advantages and disadvantages as well as relative cost and risks/consequences are tabulated below:

Foundation Type	Advantages	Disadvantages	Relative Cost	Risks/Consequences
Spread Footings	Ease of installation	Locally weak soils would render solution not practical	Not applicable since solution is considered not practical	Would require excavation below the river level with risk of flooding into excavation
Driven H-Piles	Ease of installation, low soil displacement	Vibration induced during driving	Not applicable since solution is considered the most practical	Careful pile driving next to existing bridge, design constraints as to pile arrangements near existing bridge
Caissons	Larger bearing capacity than for other options	Presence of cobbles / boulders and bearing stratum below water table makes installation not practical	Not applicable since solution is considered not practical	Would require breaking boulders into smaller pieces to allow caisson advancement. Risk of loss of base adequacy due to water pressure/inflow.

A foundation system consisting of steel H-piles driven into the very dense sandy soils is the recommended means of supporting the abutments and pier of the replacement structure in view of the need to preserve the existing pile foundations during construction. Taking account of the anticipated foundation loads and depth to a competent bearing stratum, construction of integral abutments supported on end-bearing piles is considered to be feasible at this site.



It is anticipated that driven steel H-piles will encounter practical refusal at the depths tabulated below:

Location	Reference Boreholes	Estimated Reference Founding Levels	
		Depth, m	Elevation
South Abutment	3 and 102	24 to 31	220 to 223
Central Pier	4	23	224
North Abutment	5 and 103	26 to 33	219 to 222

- NOTES: 1. Based on information from both current and previous investigations.
2. Since the founding levels are in the very dense sandy soils, an additional penetration of 0.5 to 1.0 m should be assumed.

The H-piles should be designed using the following factored geotechnical axial resistances at ultimate limit states (ULS) for three pile sections:

Factored Geotechnical Axial Resistance at ULS, kN

HP 310x110	1600
HP 310x132	2100
HP 310x174	2900

A reduction factor of 0.8 to 0.9 has been applied to account for possible variations of the pile refusal in the bouldery soil detected in the boreholes below about elevation 229.4 at the south abutment, elevation 228.8 at the pier and elevation 223.8 at the north abutment. The selection of the pile section for the project should consider the pile length and the fact that heavier pile sections are less likely to be damaged by cobbles / boulders during installation.

The geotechnical axial resistance at serviceability limit states (SLS) normally allows for 25 mm compression of the pile and founding medium. Considering the pile length of less than 30 m, the design is not expected to be governed by settlement criteria since the loading required to produce 25 mm axial deformation of the pile is larger than the factored resistance at ULS.



The ultimate tensile geotechnical resistance of the three H-piles sections is about 350 kN. A resistance factor of 0.3 should be applied to the computed capacity to determine the ULS resistance.

The approach embankment fill as well as any fill placed below grade to deal with unsuitable/compressible soils within the limits of the pile foundation should comprise Granular A or Granular B Type II or Type III with a maximum nominal size of 75 mm to enable driving of the piles and minimise the potential for damage during pile installation. The Granular B Type II is recommended below the water table if required.

The soil adjacent to the upper portion of the abutment piles is expected to comprise granular fill over firm to stiff clayey soils. To accommodate movement of the alternative integral abutment design, it is recommended that two concentric CSPs that extend at least 3 m below the bottom of the abutment be placed around the pile to create an annular space. The inner CSP of 600 mm diameter should be filled with sand meeting the gradation requirements of Granular B Type I. Alternatively, a single CSP filled with loose uniform sand meeting the requirements given in Table 2 may be used. Refer to MTO Report SO-96-01 for further details.

Since the piles will be less than 30 m long and the soil cover generally comprises cohesive soils, it is considered, based on our extensive experience with pile driving under similar conditions, that a hammer transferring at least 40 kJ of energy to the pile should be employed to drive the piles. The rated energy of the hammer should therefore be 50 to 55 kJ depending on the type of equipment employed.

The piles should be driven to a set of about 20 blows / 25 mm and rising for each 25 mm of the last 75 mm of penetration. This should be confirmed by dynamic analysis in the process of pile installation.

The H-piles will set into the very dense sandy soils with cobbles / boulders and should be equipped with driving shoes as per OPSD-3000.100 or the Titus 'H' Bearing Pile Points, Standard model, in accordance with SP 903S01.



The piles should be installed and monitored in accordance with the requirements of SP 903S01. This should involve confirmation of the founding elevation, alignment, plumbness, uniformity of set and quality of splices, and should be done on a full-time basis by experienced geotechnical personnel.

Pile caps should be provided with at least 2.4 m of earth cover or equivalent thermal insulation as protection against frost action. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 600 mm of soil cover.

Resistance to lateral loads may be provided in part by mobilization of passive resistance along the pile. The assessed lateral resistance for the pile sections recommended is as follows:

Parameters	Firm to Stiff Clayey Soils (*)	Granular Fill
Factored Lateral Resistance at ULS, kN	140	120
Lateral Resistance at SLS, kN	50	50

(*) Includes silty clay fill

If greater resistance is required, batter piles should be installed.

The coefficient of horizontal subgrade reaction, k_s (kN/m³), for the granular backfill and the underlying cohesive deposit present at the site should be computed using the following equations to evaluate the point of contraflexure:

Cohesionless:

$$\begin{aligned}
 k_s &= n_h z/b \\
 n_h &= \text{coefficient related to soil density} \\
 &= 12 \text{ MN/m}^3 \text{ for granular fill} \\
 z &= \text{depth, m} \\
 b &= \text{pile width, m}
 \end{aligned}$$



Cohesive:

$$k_s = \frac{67c_u}{b}$$

c_u	=	undrained shear strength of cohesive material
	=	50 kPa for firm to stiff clayey soils
b	=	pile width, m

Based on the available information, the existing timber piles are founded in the clayey soils. Taking account of the proposed staged construction, consideration should be given to plan the driving of batter piles for the pier of the replacement bridge to avoid the existing piles. Since low displacement H-piles are to be employed, their installation is not expected to adversely affect the existing piles.

Since vibration is induced by pile driving operations, extreme care should be taken to maintain the existing bridge performance during construction of the new bridge. It is recommended that a NSSP be prepared to specify a monitoring system of the piles which will temporarily remain during the first stage of the bridge construction. The monitoring will essentially require a survey of the existing bridge to be carried out by an Ontario Land Surveyor with the results reviewed by the MTO Verification Engineer and Contract Administrator. The recommended items to be included in the NSSP by the structural designer are listed in the attached Appendix A.

3. ABUTMENT WALLS

The abutment walls should be designed to resist the unbalanced horizontal earth pressure imposed by the backfill adjacent to the wall. The lateral earth pressure, p (kPa), may be

computed using the equivalent fluid pressure diagrams presented in section 6.9 of the CHBDC or employing the following equation, assuming a triangular pressure distribution:



$$p = K (\gamma h + q) + C_p + C_s$$

where K = coefficient of lateral earth pressure (dimensionless)

γ = unit weight of free-draining granular material, kN/m^3

h = depth below final grade, m

q = surcharge load, kPa, if present

C_p = compaction pressure, kPa (refer to clause 6.9.3 of CHBDC)

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

where \emptyset = angle of internal friction of retained soil (35° for Granular A or Granular B Type II or Type III)

δ = angle of friction between the soil and wall (23.5° for Granular A or Granular B Type II or Type III)

Free-draining granular material should be used as backfill behind the walls. The following parameters are recommended for design:

PARAMETERS	GRANULAR A or GRANULAR B TYPE II or TYPE III
Internal Friction Angle, \emptyset (degrees)	35
Unit weight, γ (kN/m^3)	22.8
Coefficient of Active Earth Pressure, K_a	0.27
Coefficient of Earth Pressure At Rest, K_o	0.43
Coefficient of Passive Earth Pressure, K_p	3.69

Refer to MTO Report SO-96-11 for procedures to determine the earth pressure coefficient to be employed in design of integral abutments. The coefficient of earth pressure at rest should be used for design of rigid and unyielding walls, the active earth pressure coefficient for unrestrained structures.

The magnitude of the passive resistance is dependent on the actual lateral movement of the structure toward the retained soil. Refer to Figure C6.16 of the CHBDC for this computation.

A subdrain system (SP 405F03) should be installed to minimise the build-up of hydrostatic pressure behind the wall. The subdrain tiles should be surrounded by a properly designed



granular filter or geotextile to prevent migration of fines into the system. The drainage pipe should be placed on a positive grade and lead to a frost-free outlet.

Backfilling adjacent to the structure should be performed in conformance with Ontario Provincial Standards specifications for granular backfill at abutments (OPSD-3101.150). As noted earlier, Granular A should be employed within the limits of driven piles.

Operation of compaction equipment adjacent to retaining structures should be restricted to limit the compaction pressure noted in clause 6.9.3 of the CHBDC. Refer to OPSS 501 and SP 105S10 for additional information in this regard.

4. APPROACH EMBANKMENTS

The road grade will be raised by about 0.9 m at the south abutment and 1.1 m at the north abutment of the structure to respective elevations 252.4 and 253.1. The final approach embankments will be about 5 m in height. The embankments will also be widened near the structure about 2.5 m to the east (Sta. 19+938 to 19+970) and by less than 1.0 m on the west side (Sta. 19+810 to 19+847). Raising the road grade and widening the embankments will involve placement of granular fill on the existing embankment slopes and rockfill into the creek. Rockfill is envisaged to be placed on the west side at the south abutment and on the east side at the north abutment to elevation 248.5, as indicated for the embankment widening. Earth fill will be placed over the rockfill to construct the embankment platform to the final grades.

Options to construct the widening of the approach embankment include the use of earth and rock fill or the use of a layer of expanded polystyrene (EPS) to reduce the added weight on the existing fill and native soils. The EPS scheme was considered in the preliminary design with a view to reducing significant settlements that were predicted on the basis of C_c values. The consolidation tests conducted for the current investigation indicate that the preconsolidation pressure is not exceeded and therefore much smaller C_r values are of reference, with the estimated settlement of 30 to 50 mm.



The EPS scheme involves some constructability issues, such as excavation adjacent to the existing embankment and potential road performance problems with the use of EPS causing uneven frost conditions. This option is also more expensive than earth and rock fill construction. As a consequence, the use of earth and rock fill is regarded as a preferable alternative to construct the widening of the approach embankment.

In general, the embankments should be constructed following conventional MTO procedures (OPSD-200.010, 202.010 and SP 206S03). Any topsoil and other deleterious materials encountered above the creek level behind the abutment locations and along the approaches should be stripped prior to placement of the embankment fill on inorganic native soils. Embankment widenings should be generally carried out in accordance with OPSD-203.030. For placement of rockfill in the water, the procedure indicated in SP 206S03 should be followed.

The slope stability analyses performed for the most critical sections behind both abutments (approximate Sta. 19+838 and 19+965) indicate that a 5 m high embankment with slopes inclined no steeper than 2 horizontal to 1 vertical will be stable. These analyses are applicable to the sections of the embankment noted above in this section. The geotechnical parameters used in the analyses are given on the diagrams included in Appendix B. The selected values reflect the consistency of the fill in the north and south approaches, in particular the somewhat less stiff material in the south embankment.

The results of the slope stability analyses included in Appendix B carried out using the Slope/W software from Geo Slope International Inc. indicated the following factor of safety values for slope stability after grade raise and widening:

Location	Reference Boreholes	Short-Term Condition	Long-Term Condition
North Abutment, East Side	103 and 5	1.4	1.5
South Abutment, West Side	102 and 3	2.3	2.2

No bearing capacity problems due to the placement of the fills for the new embankments are anticipated.



Some settlement of the existing embankment will occur due to consolidation of the clayey soils as a result of fill placement to raise the road grade to elevations 253.1 and 252.4 at the north and south approach embankments. The approximately 20 to 40 kPa increase in pressures over the north and south approach embankments (Sta. 19+810 to 19+847 and Sta. 19+938 to 19+970) are computed to be within the pre-consolidation pressure of the deposits. Based on this conclusion, the magnitude of maximum settlement near the centreline of the existing embankment is computed to be about 30 mm at both abutments. At the toe of slope where the height of new fill is assessed to be some 2 m due to widening of the existing embankment to the east, the total maximum estimated settlement is in the order of 50 mm with 15 mm occurring shortly after construction and 35 mm post-construction. The time needed for 90% post-consolidation of the clayey soils to be completed is assessed to be up to 10 years.

It is recommended that the new fills for the widening and grade raises mentioned in this section be placed in advance of the construction of the pavements to reduce the magnitude of the long-term settlements by pre-loading in particular the differential settlements between the centreline and new east side of the roadway. The estimated reduction in long-term total settlements due to preloading during alternative six-month and 12-month periods are about 10 and 15 mm respectively, so post-construction settlements are expected to be in the order of 20 to 25 mm.

It is recommended that an embankment settlement monitoring program be implemented to verify the magnitude of the settlements prior to construction of the highway pavements. A monitoring program is included in Appendix C.

The recommended construction sequence for the approach embankments includes the following general steps:

- Advance construction – Place fill for embankment widening and grade raise at both north and south approaches. Monitor settlements
- Bridge construction – Construct bridge abutments. Continue to monitor embankment settlements



- Approach embankment construction – Proceed with final grading of approach embankments and highway pavement construction. Discontinue settlement monitoring

The backfill to the abutments should comprise granular material placed in conformance to the requirements of SP 206S03 and OPSS 501 to minimise post-construction settlements.

5. CONSTRUCTION CONSIDERATIONS

5.1 Excavation

Excavation for construction of foundations at the abutment locations (Sta. 19+847 and Sta. 19+938) is expected to extend through the embankment fill to a depth of about 3 m below existing grade, near elevation 249. Excavation of the embankment fill should be relatively straightforward since the base of these excavations is expected to be above the water level in the creek.

The fill is classified as a Type 3 soil according to Occupational Health and Safety Act (Ontario Regulation 213/91) criteria. Therefore, temporary cut slopes inclined at 45° to the horizontal should generally be safe. Flatter side slopes may be required if excessively soft/wet materials or concentrated seepage zones are encountered locally during construction.

5.2 Road Protection

The centreline of the replacement bridge is planned to be some 2.5 m to the east of the existing alignment of the road. It is anticipated, therefore, that a suitable roadway protection scheme following OPSS 539 and SP 105S19 will be necessary behind the abutments to support the walls of excavation and adjacent traffic lanes during staged construction.

Several alternative protection schemes such as sheet piling, sheeting supported by rakers or bracing, anchored soldier piles and lagging may be considered. A road protection scheme designed for performance level 2 system is recommended to prevent movement of the existing



embankment. The contractor is responsible for the selection, preparation and performance of a detailed design for the road protection scheme.

5.3 Groundwater Control

The stabilised groundwater level is expected to be consistent with the water level in Meadow Creek, near elevation 248. Taking account of the relatively impervious clayey soils at the site, it is anticipated that conventional sump pumping techniques will be sufficient to control seepage of groundwater into the excavations at the new abutment locations.

Construction of the pier foundations may require excavations below the water level in the creek. Installation of steel sheeting and placing tremie concrete will be required to effectively control seepage of water into the excavations. Use of large diameter steel tube liners instead of sheet piles is also considered suitable for construction of the pier foundations at the site.

The high water level in Meadow Creek should be assumed in design of steel sheeting if utilised. Lateral loads from boats and ice forces should be incorporated in the sheet piling design. The specifications should call for a groundwater control specialist and co-ordination with the operation of the upstream and downstream dams and clearly state that coordination of the control of water levels is the contractor's responsibility.

The sheet piles should extend to a depth equal to at least 2 times the excavation depth below the water level to minimise the potential for bottom heave.



The sheet pile design should be based on the following geotechnical parameters:

Parameter	Central Pier
Unit weight of soil above water table, γ (kN/m ³)	18.0
Unit weight of soil below water table, γ' (kN/m ³)	8.2
Active earth pressure coefficient, K_a	0.3
Passive earth pressure coefficient, K_p	3.0

All work should be carried out in accordance with the Occupational Health and Safety Act (Ontario Regulation 213/91) and with local/MTO regulations.

5.4 Pile Installation

Consideration should be given to the presence of rockfill at the pier location (identified during the preliminary investigation) and its removal prior to the driving of sheet piles or tube liners.

Taking into account the proposed staged construction of the bridge, the existing timber piles should not be damaged during installation of the new piles.

It is recommended that the layout of batter piles for the replacement bridge be chosen with a view to avoiding the existing timber piles.

6. PERMANENT CUT AND FILL SLOPES

The proposed realignment of the road to accommodate a speed limit of 80 km/hr will necessitate construction of cut slopes to the south and north of the structure within a distance of up to 450 m south and 250 m from the bridge site. Comments on this portion of the project are provided under a separate report prepared by PML with reference No. 08TF009-2.



7. CLOSURE

This report was prepared by Mr. G.O. Degil, PhD, P.Eng., Senior Foundation Engineer, and reviewed by Mr. C.M.P. Nascimento, P.Eng., Senior Project Engineer. Mr. B.R. Gray, MEng, P.Eng., MTO Designated Principal Contact, conducted an independent review of the report.

Yours very truly,

Peto MacCallum Ltd.

**NOTE: Hard copies signed
and stamped**

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