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Project No.: 0272038

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BC Ministry of Transportation and Infrastructure
#342 – 447 Columbia Street
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Dear David,

Re: Geotechnical Recommendations for Micropile Foundation Support for Eagle River Bridge, Highway 1, Three Valley Gap, BC

1.0 INTRODUCTION

BC MOTI is planning to widen the Eagle River bridge (overhead structure no. 0357, or the bridge), increasing the loads on the abutments and piers. Golder Associates (Golder) have previously provided design assessment and recommendations for the bridge abutments and piers for the increased loading (Golder, March 20, 2013; May 3, 2013, and August 1, 2013). BGC Engineering Inc. (BGC) was retained to conduct a review and revision as appropriate of the adequacy of the abutments for the increased loading, and for the micropile design to add bearing capacity to the piers. This review was required as design codes have changed since the Golder work, and because MOTI would prefer a different type of micropile than evaluated by Golder.

This letter summarizes the previous investigation and design conducted by Golder Associates (Golder), provides a review of conformance of Golder's previous recommendations for the abutments against changes in the design code, and provides recommendations for a revised micropile design for the piers from self-drilling micropiles to cased micropiles that also considers changes in the design code. The recommendations provided in this letter supersede those previously provided by Golder.

1.1. Scope of Work

BGC prepared a scope of work for geotechnical design and construction services for the deck widening and rehabilitation project for the Eagle River Bridge, as outlined in BGC's proposal dated July 18, 2018. The scope of work was accepted on July 19, 2019, and was completed under the existing time and materials As-and-When contract between BGC and MOTI, Contract Identification No. 860-CS-1491.

The scope of work documented in this letter includes:

- A site visit to evaluate constructability of cased micropiles

- Assessment if previous conclusions that the abutments were adequate for the new loads by Golder remain true for the new code requirements
- Design recommendations for cased micropiles, with consideration of changes in code requirements since earlier design by Golders.

Since the retrofit design was initially completed in 2013 the applicable design code (CSA S6-14) was updated and adopted by MOTI, and this new code requirement was used in assessment.

No additional subsurface investigations were undertaken by BGC, and all subsurface information was based on boreholes provided by Golder (Golder, March 20, 2013 and May 3, 2013).

1.2. Background

The Ministry of Transportation and Infrastructure (MOTI) plans on upgrading and renovating the existing bridge on the Trans-Canada Highway (Hwy 1) that crosses the Eagle River near the west end of Three Valley Lake (See Figure 1-1). The existing bridge is an approximately 75 m long overpass crossing a gully that contains the Eagle River outlet to Three Valley Lake, and twinned CP rail tracks adjacent to the north abutment. The north and south abutments of the bridge are located on the north and south sides of the gully, respectively. There are two piers supporting the bridge located 22 m and 52 m from the North Abutment, and the piers (north pier and south pier) are located on opposite sides of the river. The roadway comprises two-lanes (one in each direction) on a 7.9 m wide deck (Associated Engineering, Dwg. 00357-101). The north and south abutments are supported on large raft foundations, and each bridge pier is supported on two 2.4 m by 2.4 m square spread footings.

Figure 1-1. Site location of Eagle River Bridge near the west end of Three Valley Lake (NTS).

The proposed works will replace the deck of the existing bridge, widening the roadway to 10.9 m to comply with current roadway design standards (Associated Engineering, Dwg. 00357-102). No additional lanes will be added based on the Issued-for-Tender (IFT) drawings prepared by Associated Engineering (AE) (2019). The wider deck will impart additional loading on the existing bridge foundations that are required to conform to current bridge code and guidelines.

Golder previously conducted geotechnical investigations, assessments and design for retrofits to the existing bridge foundations in 2013 (Golder, March 20, 2013; May 3, 2013, and August 1, 2013). The boreholes for the north (BH13-2) and south (BH13-1) abutments were advanced from the roadway behind the abutments to depths of 18.3 m and 23.0 m, respectively (Golder, March 20, 2013). The boreholes for the north (BH13-3) and south (BH13-4) piers were completed underneath the bridge adjacent to the bridge piers to depths of 6.9 to 6.6 m, respectively (Golder, May 3, 2013). The top of footing for the north and south piers were about 0.3 and 0.8 mbgs, respectively.

Golder's March 20, 2013 report recommended the pier foundations be modified by increasing the size of the footings to avoid increasing bearing pressures, and that the abutments did not require retrofitting for the proposed increase in loading. AE recommended that micropiles be used to increase the capacity of the piers instead of widening the footings, and Golder's August 1, 2013 provided a micropile design utilizing self-drilling hollow core anchors (IBO anchors). Golder also warned about potential grout loss during anchor drilling and uncertainty about grout encapsulation of the anchors. BGC understands the MoTI was concerned about the potential for grout loss reporting to Eagle Creek, and prefers a conventional cased micropile design to minimize the potential for grout loss and provide better long-term corrosion protection and reduce the likelihood of poor micropile grout encapsulation. MoTI retained BGC to review the existing foundation design, and make recommendations for micropiles. In the interim, MoTI adopted new design codes, and part of BGC's work scope included incorporating the new code in design for the micropiles, as well as considering if Golder's original conclusion that the abutments are adequate for the higher loads is still applicable under the new code requirements.

The following details on the abutments and piers were previously provided by Golder.

1.3. North Abutment

The existing north abutment is a gravity retaining wall. The south face is terraced and exposed while the north face is vertical and buried. The south face has an approximate length of 14.9 m and height above native ground of 8 m (Golder, March 20, 2013). The wing walls extend approximately 6.8 m to both east and west sides of the abutment. The area of the existing spread footing is expected to have an approximate bearing area of 80 m^2.

Golder's March 20, 2013 report indicated that the existing bearing pressure for the north abutment is approximately 106 kPa, based on Associated Engineering's (AE) calculations. The proposed increase in loading was estimated to result in a bearing pressure increase to between 125 and 135 kPa, and it is assumed that these values have not changed. Golder concluded that the north abutment was founded on compact to dense sand and gravel till, and estimated a serviceability limit state (SLS) for the north abutment footing subgrade of 400 kPa, and an ultimate limit state (ULS) of 600 kPa, well above the proposed bearing pressure. From this, Golder recommended that the north abutment did not need to be retrofitted for the additional foundation loading.

1.4. South Abutment

Golder (March 20, 2013) states the existing south abutment is likely founded on a rectangular raft foundation supporting two columns approximately 5.2 m in height providing load transfer. The columns are approximately 1.2 m by 1.9 m at the top and 1.2 m by 4.6 m at the bottom connection to the raft foundation. Golder (March 20, 2013) further notes the columns are spaced about 5.5 m apart. The dimensions of the footing are approximately 11.0 m by 5.8 m and 1.5 m thick, giving an approximate bearing area of 64 m^2.

Golder's March 20, 2013 report indicated that the existing bearing pressure for the south abutment is approximately 106 kPa, based on Associated Engineering's (AE) calculations. The proposed increase in loading was estimated to result in a bearing pressure increase to between 125 and 135 kPa, and it is assumed that these values have not changed. Golder concluded that the south abutment was founded on compact to dense silty sand till, and estimated a SLS for the south abutment footing subgrade of 500 kPa, and an ULS of 750 kPa, well above the proposed bearing pressure. From this, Golder recommended that the south abutment did not need to be retrofitted for the additional foundation loading.

1.5. North and South Piers

Golder (March 20, 2013) states the north and south piers are the same design, which was confirmed by BGC during a site visit. There are two square columns that are joined at the top providing load transfer to two separate spread footings. The square columns have a width of approximately 0.9 m at the top of the column that increases to 1.4 m at the connection with the footing. The footings are approximately 2.4 m by 2.4 m with an unknown thickness. The base elevation for the footings is estimated to be at approximately 497.0 masl and 496.2 masl for north and south piers, respectively.

Subsurface conditions at the piers comprise 1.2 to 1.4 m of silty sand fill over cobbles and boulders interlayered with soils consisting of sand and gravel with fines (Golder, May 3, 2013). Golder interpreted that the base of the pier footings were placed on native soil, but did not provide an interpretation of the genesis of the soil (example, till or fluvial deposits). The return of drilling fluids was lost below approximately 2.0 m and 2.9 m depths, at the north and south piers, respectively, which Golder interpreted to be due to a significant presence of cobbles and boulders. No drilling water was observed escaping to the river during the investigation, although it was mentioned as a potential risk during micropile installation (Golder, August 1, 2013).

Golder’s report (March 20, 2013) indicates that the expected existing bearing pressure for the north and south piers could be as much as 422 kPa based on AE’s calculations, and this estimate allowed for the possibility of asymmetrical loading of the pad footings. AE estimated in 2012 that the proposed bridge modifications will increase the serviceability limit state (SLS) bearing pressure for the piers by approximately 141 to 228 kPa. This will result in a total SLS bearing pressure of between 563 and 650 kPa. It is understood that the existing and proposed bearing pressure values provided by AE remain unchanged.

2.0 REVIEW OF GOLDER’S ABUTMENT RECOMMENDATIONS

BGC reviewed the conformance of Golder’s previous recommendations for the abutments against changes in the design code using the estimated SLS bearing pressure increases determined by AE (Sections 1.3 and 1.4) and the estimated bearing capacity of the foundation by Golder. BGC’s review found that the proposed increased loading for the two abutments for the Eagle River Bridge satisfies the current design code (CSA-S6-14). Therefore, BGC agrees with Golder’s recommendations that the abutments will not need to be retrofitted for the additional foundation loading. This relies upon Golder’s assessment of bearing capacity of the till soils below the abutments.

3.0 MICROPILE RECOMMENDATIONS FOR PIERS

Based on the proposed bearing pressure following the bridge widening, the two piers on either side of the Eagle River will require additional bearing capacity to meet the code requirements. MoTI and AE are investigating micropiles to add additional bearing capacity to the piers.

3.1. Estimated Micropile Bond Strength and Phreatic Surface Elevation

Golder interpreted that the base of the pier footings are placed on granular native soils, but did not provide an interpretation of the origin and density of the native soil (e.g.: dense till or loose fluvial deposits). Considering that Golder concluded that both abutments are founded on till, it follows that the piers may also be underlain by till. However, standard penetration tests were not performed in the boreholes adjacent to the piers (Golder, May 3, 2013), which makes it difficult to confirm the presence of dense soils (e.g., basal till).

The existing bearing pressure at the piers is approximately 422 kPa, and BGC is not aware of any settlement issues with the bridge over the considerable life span of the bridge. From this it can be interpreted that the foundation soils have provided adequate static bearing with acceptable settlement for the piers over the current bridge life. BGC considers that adequate pier performance at this bearing pressure would require a compact to dense supporting soil. As the soils are interpreted to not be loose and assuming the granular soils continue for the full depth of the micropiles, BGC interprets that it is likely possible to achieve adequate micropile bond strength. The lack of quantitative data on the soil density and hence potential micropile bond zone strength can be somewhat mitigated by assuming a conservative grout to soil bond strength that would be appropriate for either moderately dense or loose deposits. For the purposes of calculating a bond length for the micropile design, an unconfined shear strength parameter of 125 kPa was assigned to the footing subsoils.

Groundwater conditions were inferred by Golder (March 20, 2013) to be at a similar elevation as the water level in Eagle River.

3.2. Micropile Design Recommendations

Each pier consists of two columns on individual spread footings. BGC carried out a site visit on July 26, 2019 and following the site visit considered constructability of the micropiles. There is approximately 450 mm between each pier column face and the edge of the footing. AE has indicated that the centre of the micropiles will need to be offset from the edge of the footing by 250 mm minimum. BGC contacted several local contractors to determine what the required minimum clearance from centre of the micropile to the column face is for constructability. These contractors indicated a minimum clearance of approximately 300 mm is required for the drill head. Given these constraints, it was agreed through discussion with AE and MoTI on August 14, 2019 that each footing will have four micropiles that are installed at the corners of the footings. There will be a total of 16 micropiles installed.

Associated Engineering Ltd. (AE) indicated via e-mail on August 28, 2019 that the micropiles will have the following loading:

- Serviceability Limit State (SLS) Load: 350 kN/pile
- Ultimate Limit State (ULS) Load: 420 kN/pile
- Lateral Load: 29 kN/pile.

Given the above loading, BGC recommends a double corrosion protected (DCP) 43 mm diameter (#14) 550 MPa Grade Gewi Pile made by Dwyidag (or an approved equivalent), installed in a 150 mm (6-inch) cased borehole, with centralizers placed at approximately 3 m intervals. Double corrosion protection is recommended given the anticipated service life exceeds 5 years. The micropile heads will be cast into a concrete overlay designed by AE. The design of the micropiles was completed in accordance with U.S. Department of Transportation Federal Highways Administration (FHWA) Micropile Design and Construction Reference Manual (FHWA, 2005). Lateral pile capacity was not assessed due to the small amount of lateral loading in relation to the proposed axial load.

Pile casing should be permanently installed in the upper portion of the micropile to help increase resistance against lateral loading and reduce the probability of loss of circulation and grout during grouting, including grout loss that could impact the Eagle River. The casing should have a minimum outer diameter of 150 mm and wall thickness of 9.5 mm. Use of a larger casing would need to be approved by AE given the potential effects on the structural design. The casing should extend a minimum of 3 m below the base of the existing footing. The top of the casing should be cast into the footing as directed by AE.

During the installation process, the drill hole should be cased the entire length with the casing retracted during the grouting process. Pressure grouting through the casing should be conducted as the casing is retracted. This involves additional grout being injected after the primary grout has been tremied into the borehole, with the aim to enhance grout-to-ground bond characteristics. FHWA indicates that pressure grouting is usually conducted by attaching a pressure cap to the top of the drill casing and injecting additional grout under controlled pressure. BGC recommends that the maximum applied injection pressure at the cap be 20 kPa per metre of depth to avoid ground heave and help minimize the likelihood of uncontrolled loss of grout. Grout on the exterior of the casing should not reach the base of the footing so that testing of the anchors is not influenced by grout column bond to the footing.

It is also recommended that one or more post-grouting tubes are installed in case there is an issue with the grout column not reaching the ground surface. One of these post-grout tubes should be placed at the bottom of the pile casing, which corresponds with the top of the bonded zone.

BGC recommends that the micropiles have a minimum bonded length of 15 m below the bottom of the casing. If bedrock is encountered, the bonded zone length should be 4 m (minimum) into the bedrock up to a total bonded length of 15 m. The ultimate geotechnical resistance of the bond zone for the micropiles is based on a minimum grout-to-ground bond strength of 125 kPa in the soil and 800 kPa in bedrock. The drill hole diameter of the proposed micropile is 150 mm (6-inches), which is equivalent to the outer diameter of the steel casing.

The micropiles should be grouted in place with an unsanded, non-metallic, non-shrink, Portland cement grout. The cement grout should have a minimum 28-day strength of 35 MPa. Grouting should be completed following FHWA (2005) recommendations for cased and uncased holes.

BGC should observe the drilling, grouting, and proof testing. All micropiles should be proof tested in tension to 560 kN, which is equivalent to 1.6 times the working load, as recommended by FHWA (2005). Anchor testing should be conducted in accordance with FHWA (2005). BGC recommends that AE should review the bearing frame design for the proof testing to confirm that the existing footing has enough strength and will not be adversely affected by the proof testing. Following completion of the proof testing and sign-off by BGC, the micropiles should have a nominal load applied by hand tightening hex nuts at the top of the micropile, as applying a lock off load would increase load on the footings.

The preceding are general recommendations. It is BGC's understanding that BGC will be responsible for completing the design of the micropiles, and AE will be responsible for confirming the structural design for the micropile-footing connection and any required rehabilitation on the footings or the superstructure. It is understood that BGC and AE will co-seal the general arrangement drawing of the micropiles (Dwg. 00357-103) and BGC will seal a micropile detail drawing including specifications. It is also understood that BGC will undertake micropile quality assurance for drilling, installation and proof testing to confirm the micropiles conform to the intent of the design.

In order to confirm that the design drawings and specifications meet the intent of the recommendations of this report, it is anticipated that BGC will work with AE on the drawings, and it is recommended that BGC have the opportunity to review the draft Issue-for-Tender package to confirm that the development of the micropile portions of the measurement for payment terms and schedule of prices are consistent with the intent of the micropile drawings and specifications.

4.0 UNCERTAINTIES AND RISKS

The use of micropiles to increase the bearing capacity of the piers comes with construction uncertainties and risks from the uncertain foundation conditions that AE and MoTI should be aware of and consider. The available subsurface information at the piers (drilling) only reached up to 6.9 m below the pier footings and experienced loss of drill fluid return, which both introduce construction uncertainty and risk. The actual geotechnical conditions below the investigated depths to the design length of the micropiles are unknown, and could be less favourable for micropile bond zone strength than assumed for the micropile design. The consequence could be lower than estimated bond zone shear resistance, leading to a longer bond zone or larger borehole being required. The design mitigates this risk somewhat by using a low assumed design bond zone shear strength, but some risk of longer anchors being required remains. Drill fluid loss in the investigated depth below the piers, and unknown ground conditions below that depth, increase the likelihood of grout loss. It is possible that the bond zone may not be able to be grouted due to subsurface permeability (i.e. hold a grout column to the borehole collar under gravity), or may not be able to achieve specified grouting pressures, even with the provision of post-grouting tubes. In this case, alternate measures such as repeated attempts to drill/grout/install after letting the previous grout column partially set, or the use of a grout sock, or grout additives, may be needed. Alternately, if a denser and less permeable bearing strata is found at depth during micropile drilling (e.g., dense till or bedrock), adjustment of the cased length left in the borehole to socket into that material rather than have a portion of the bond zone in potentially more permeable soil, could potentially reduce the uncertainty of installation difficulty due to grout loss. AE and MoTI may wish to consider this scenario, and if possible and desirable provide for the uncertainties about grouting and potentially longer casing lengths in the specifications and tender schedule of prices.

5.0 CLOSURE

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Yours sincerely,

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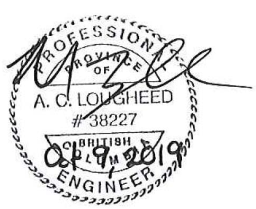

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