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File No: 8-5650-1

MMM Group Limited
600 – 1455 West Georgia Street
Vancouver, B.C.
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Attn: Mr. Jim Fowler, P.Eng.

Dear Sir,

Re: Existing Johnson Street Bridge Analysis – Geotechnical Component
Johnson Street Bridge – Victoria, B.C.

As requested, we have completed our geotechnical assessment and associated analyses of the existing Johnson Street Bridge, as such relates to the proposed seismic upgrading, as well as the proposed Pedestrian-Cyclist bridge immediately to the north.

The geotechnical scope involved only office-based activities, and consisted of review of relevant available information, geotechnical analysis of existing and conceptually proposed foundation elements, coordination /discussion with design team members, and preparation of conceptual geotechnical design recommendations.

1.0 BACKGROUND INFORMATION

We have undertaken a review of file information within our file archives, including a copy of the original railway bridge alignment and historical harbour maps, as well as various documents provided by others. The diamond drill boring logs from 1920 which were provided to us were reviewed, as were drawings detailing the abutments, wing walls and piers, and profiles depicting the bedrock and sediment surfaces. Additional reports made available to us that were utilized in our analyses and in part form the basis of our recommendations include the Stantec Consulting Ltd. report, which summarizes their geotechnical investigation undertaken for the proposed Johnson Street Bridge Replacement (Stantec 2009), along with the borehole records for 12 holes, as well as a comprehensive report by Delcan assessing the existing condition of the bridge and providing repair and rehabilitation options (Delcan n.d.).

The available information indicates the west abutment and a portion of the east abutment were founded directly on bedrock; this was corroborated by photographs in the Delcan report. The subgrade materials for the wing walls are generally unknown, however, the original drawings show the footings to be located on or near bedrock. Records indicate the rest pier was also founded on bedrock, however, this was not confirmed visually. The counterweight and main trunnion piers are supported on timber piles which the records suggest were driven to bedrock. However, it is likely the piles are actually seated on till, a very dense glacially over-ridden sediment, rather than bedrock, as till is typically dense enough to result in refusal of timber piles prior to incurring damage to the pile.

The proposed bicycle/pedestrian bridge has been conceptually designed by others, with the locations and quantities of the loads identified by the structural consultant.

2.0 Analysis Methods

Analyses of the bridge abutments and rest pier were completed using the seismic criteria put forth by the American Association of State Highway Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications. These parameters involve a design event with a 7% probability of exceedance in 75 years, which corresponds to a return period of approximately 1034 years. The peak ground accelerations (PGA) for Seismic Site Classes B, C and D were 0.36g, 0.37g, and 0.42g, respectively, which were obtained from the United States Geological Survey (USGS). The design spectra and ground motions are defined in terms of an acceleration coefficient, and accordingly, the peak horizontal ground acceleration is approximately equal to the peak horizontal ground velocity (Naumoski et al. 2000).

With respect to the coefficient of acceleration used for the analysis, typically values for the horizontal coefficient range from 1/2 PGA through to 3/2 PGA, with the lower value generally attributed to abutments and retaining walls that are free to move slightly (yielding), and the higher value attributed to abutments and walls that are fixed and unable to move (CSA S6.1 2006). The abutments in their current state are considered non-fixed, and therefore the former was used in the analysis. The horizontal and vertical coefficients of acceleration were selected as 0.28g, which is 3/4 of the PGA for Site Class B. This was considered appropriate by the structural consultant.

The abutment walls were analyzed using the Mononobe-Okabe (M-O) approach, as specified and outlined in the AASHTO LRFD Bridge Design Specifications Appendix A11 (AASHTO 2008), and the Canadian Foundation Engineering Manual (CFEM; Canadian Geotechnical Society 2006). The M-O approach is a simplified, pseudo-static analysis that is well-established and widely accepted for analysis of retaining walls under seismic loads. The method approximates dynamic loads due to seismic cycling as equivalent static loads, which are used to assess wall stability. The input factors for the analysis are the geometry of the retaining wall, the angle of internal friction and unit weight of the backfill, and the horizontal and vertical acceleration coefficients. This simplified analysis only considers one type of backfill material, and omits the effects of abutment inertia.

In addition to the M-O analysis, both AASHTO Appendix 11 and the CFEM provide equations to estimate total displacement of a retaining wall subject to an earthquake. These equations are based on the retaining wall having a yield acceleration (the maximum acceleration that can be accommodated prior to initiation of wall sliding) which is dependent upon a variety of factors including wall geometry and seismic forces.

In addition to the above theoretical calculations, the abutments and wing walls were also analyzed using finite element analysis (FEA) software, which allowed for consideration of inelastic (plastic) soil behavior, multiple soil types, and more comprehensive properties of the soils and interfaces between the wall, underlying subgrade, and backfill materials. Furthermore, the effects of abutment inertia are accounted for using FEA. Given that the abutments are gravity walls and rely upon their mass for stability, it is non-conservative to ignore these inertial forces. The rest pier was modeled using FEA analysis, however, neither the existing counterweight nor trunnion piers were modeled, as the structural consultant has advised that the piers are not structurally adequate and upgrades are required. However, the proposed pile foundations were analyzed to determine the required depth of embedment.

Appropriate model parameters were chosen based on estimated soil properties and information available in the drill logs. Within the FEA model, parameters were adjusted (back calculated) to obtain suitable results under static conditions, prior to adding seismic loads.

The required embedment length for rock anchors was determined using pullout rock cones with a 90° apex originating at the anchor tip and considering the buoyant density of rock. Interference effects within anchor groups were accounted for.

3.0 RESULTS AND RECOMMENDATIONS

The results of the analyses are presented below, and summarized in Table 1. Our recommendations are provided herein for discussion within the design team and for review by the Quantity Surveyor for costing.

3.1 MONONOBE-OKABE APPROACH

Calculations for the east abutment did not yield usable results, as initial criteria were not met for the theoretical equation to be valid. Due to the geometry of the abutment and estimated backfill parameters, the M-O equation was considered inappropriate. The only way to assess the stability of the east wall under seismic loading was to use FEA.

Calculations and analysis for the west abutment indicated failure under seismic loading by both sliding and overturning. These modes of failure are typical for retaining walls subjected to earthquakes, with basal sliding and outward rotation of the upper portion of the wall (AASHTO 2008). The calculated factors of safety were 0.76 for sliding and 0.40 for overturning. Additionally, the resultant force was not located within the middle 1/3 of the wall, indicating further instability and bearing failure. Permanent basal displacement was calculated to be in the order of 70 mm using a statistical approach, and 300 mm using a deterministic approach.

3.2 Finite Element Analysis

3.2.1 East Approach

Analyses of the east abutment indicate the retaining wall is unstable during the seismic design event, with movements up to 1 m occurring at the top of the abutment and up to 60 mm at the base, showing the wall fails through a combination of sliding and overturning. To stabilize the abutment wall, steel rock anchors were added to the model (1330 kN yield strength, minimum 10% pretensioning, 1.5 m spacing, 7 m embedment length). These reduced the wall movements to 130 mm at the top and 35 mm at the base. However, we are uncertain as to whether the presence of the bridge deck will allow for such movements as the structural consultant has indicated that the bridge should not be relied upon for lateral support of the abutments. Further complex analyses would be required, including a structural component, to assess the bridge-abutment interaction. Adding larger and/or more anchors would further reduce the wall movements if necessary.

Analyses of the east wing walls indicated both will fail under seismic loads, experiencing overturning and sliding. Modeling results indicate the north wall will move outwards and rotate mildly, resulting in horizontal movements in the range of up to 900 mm at the top of the wall and 200 mm at the base, and vertical settlement of the backfill in the order of 300 mm. The south wall may move as much as 500 mm horizontally at the top and 150 mm at the base, and experience surface settlement of the backfill up to 200 mm. These movements would affect the Johnson Street exit and the railway, however, we expect that in the event of such movement, temporary fill placement to restore the grade change could reinstate serviceability of the bridge for emergency vehicles.

3.2.2 West Approach

Analyses of the west abutment indicate that it also is unstable under seismic loading. The wall will experience sliding and overturning, with movements at the top in the order of 600 mm and 50 mm at the base. Steel anchors were also added to the model to stabilize the wall (1330 kN yield strength, minimum 10% pretensioning, 2 m spacing, 6 m embedment depth) and displacement at the top was reduced to roughly 160 mm, while the base still moved about 50 mm.

Analyses of the west wing walls indicated both will fail during the design seismic event through a combination of basal sliding and overturning. Outward rotation at the top will result in horizontal displacement of some 350 mm at the top of the wall and 120 mm at the base, and vertical settlement of 100 mm or more proximal to the wall. The roadway would likely remain passable to emergency vehicle traffic without requirement for immediate mitigative measures.

3.2.3 Rest Pier

Analysis of the pier indicates basal sliding will occur along the bedrock surface, which is shown as sloping in the original profile drawings. We understand that the pier was constructed by dredging of the soil to allow inspection of the cleaned bedrock by a diver, prior to placement of concrete underwater. Since we are uncertain of the nature of the concrete/bedrock contact (slope, roughness, cleanliness, etc.) we have considered conservative values for pertinent parameters in modeling of the pier. FEA analysis indicates basal movements upwards of 130 mm are possible. We understand that the structural consultant is considering adding large steel rock anchors vertically through the entire pier which would mitigate concern for sliding. Another option might be to add high angle anchors only through the lower portion of the pier. Analyses indicate two longitudinal rows of anchors with a yield strength of 1330 kN with a spacing of 2.5 m and embedment of 7.5 m would be sufficient to stabilize the base, reducing basal sliding to around 35 mm.

3.4 Main Trunnion and Counterweight Piers Seismic Underpinning

Following analyses by the structural consultant, we understand that the piers will be underpinned using 1.8 m diameter columns extending out of 2.4 m drilled shafts (socketed caissons). Four shafts will be positioned on each side of the piers. The factored loads at the base of the drilled shafts were provided by the structural consultant, and are as follows:

- Shear force = 4226 kN
- Bending moment = 41 352 kN-m
- Vertical force = 22 059 kN
- Torsion = 980 kN-m

Calculations were subsequently undertaken to determine the required embedment depth for the given loads, with bending moment being the controlling load. The required embedment depth was found to be 6 m in competent bedrock. The upper 1 m of bedrock was assumed to be weathered and/or substantially fractured, and therefore assumed to be unsuitable to provide satisfactory load resistance. However, the actual thickness of weathered bedrock may differ.

4.0 Bicycle-Pedestrian Bridge

Based on the conceptual design, the structural design requires four 900 mm drilled shafts (socketed caissons) at the counterweight/trunnion pier, two shafts for the rest pier, and two shafts for the eastern approach pier (pier #4). Using the vertical, shear and moment loads provided by the structural consultant and typical rock properties, we calculated the required embedment depth for the drilled shafts. These depths are summarized below, and refer to the required embedment depth in sound, intact bedrock.

- Rest Pier #2: 3.0 m embedment
- Trunnion/Counterweight Pier #3: 6.0 m embedment
- Approach Pier #4: 3.0 m embedment

Additional drilling depth may be required if the bedrock surface is fractured or weathered. We anticipate that any uplift loads can be restrained by the embedment of the drilled shafts, whereby the weight of a rock cone originating at the pile tip with a 90° apex is calculated using the buoyant density of rock. If this does not provide adequate resistance, rock anchors can be installed down the inside of the shaft.

From discussion with the structural consultant, we anticipate that it will be feasible to support the western approach pier using a cast on grade concrete pad footing. The abutments will be cast in place concrete with 3 m wing walls, all backfilled with select granular materials. An alternative option may be to utilize mechanically stabilized earth retaining walls for the abutment and wing walls, although such details are best resolved as the design progresses.

5.0 CLOSING

We have undertaken an office based geotechnical assessment and associated analyses of the existing Johnson Street Bridge with consideration for proposed seismic upgrading, as well as the proposed Pedestrian-Cyclist bridge. The results of our work indicate that a number of existing foundation elements are at risk of failure in the occurrence of a design seismic event. We have carried our preliminary geotechnical analyses to evaluate conceptual mitigative measures and foundation designs and have provided our input to the design team.

We hope the preceding is suitable for your purposes at present, however if you have any questions with respect to the above, please contact us.

Yours very truly,
C.N. Ryzuk & Associates Ltd.

S. W. Moore, P.Geo.
Geoscientist

L. G. Campbell, P.Eng
Geotechnical Engineer

Attachment – References
Quantity Estimate Table

REFERENCES

AASHTO. 2008. Appendix A11 Seismic Design of Abutments and Gravity Retaining Structures, in AASHTO LRFD Bridge Design Specifications.

Canadian Geotechnical Society. 2006. Seismic Design of Retaining Walls, in Canadian Foundation Engineering Manual, 4th Edition. Richmond, British Columbia, pp. 112-118.

CSA S6.1. 2008. Commentary on Canadian Highway Bridge Design Code, Section C4.6.4 Seismic forces on abutments and retaining walls.

Delcan. N.d. Johnson Street Bridge Condition Assessment Report. Prepared for the City of Victoria.

Naumoski, N., Cheung, M. & Foo, S. 2000. Evaluation of the seismic response coefficient introduced in the Canadian Highway Bridge Design Code. Canadian Journal of Civil Engineering, 27:1183-1191.

Stantec. 2009. Johnson Street Bridge Replacement Project: Geotechnical Investigation Report. Prepared for the City of Victoria.

Quantity Estimate

Rock Anchors	Number of anchors	Embedment length per anchor (m)	Embedded Length (m)
East abutment	16	7	112
West abutment	12	6	72
Rest Pier	20	7.5	150
Sum			334

Drilled Shafts	Number of shafts	Embedment depth per shaft (m)	Total embedded length (m)	Total length[1] (m)	
2.4 m diameter	8	6	48	56	
0.9 m diameter	4	3	12	16	**piers #2 and #4
0.9 m diameter	4	6	24	28	**pier #2

Comments:

1. Length includes 1 m allowance per shaft for weathered bedrock surface.