Memo

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Reference: Geotechnical Engineering Technical Summary Memorandum for the Proposed George Massey Crossing, Richmond/Delta, BC

Revision	Date	Revision Description
A	October 17, 2019	First draft
B	October 29, 2019	Addressed review comments from COWI
C	November 15, 2019	Addressed review comments, included bridge foundation capacities

1. INTRODUCTION

The British Columbia Ministry of Transportation and Infrastructure (MoTI) intend to carry out an Options Analysis and Comprehensive Feasibility Study for the George Massey Crossing (GMC) project (Project). The MoTI have awarded a contract to COWI/Stantec team to provide technical consulting services for the Project. This memorandum provides a summary of the Geotechnical Engineering technical input given to the COWI/Stantec team.

The subsurface soil conditions, seismic and other design considerations provided in this memorandum are preliminary. The contents of this memorandum should not be relied upon for the final design, for bidding purposes or for construction.

2. PROPOSED STRUCTURE

The proposed structure for the Project includes three options and a number of sub-grouped options. The three main options include a deep bored tunnel (DBT), an immersed tube tunnel (ITT) and a bridge.

The project site is located along the existing Highway 99 corridor and in general parallel to the existing George Massey Tunnel alignment. Available information provided by the MoTI indicates the following:

- The existing ground elevation onshore varies between 1.5 m and 2 m. Elevation of the top of the dykes at the banks of the river is estimated to be approximately 3.5 m.
- High Water Level (HWL) and Low Water Level (LWL) are El. +2.00 m and El. -1.63 m respectively.
- Top of the soil cover over the existing ITT within the river varies between approximately El. -5 m and El.-10 m, and bottom of the tunnel elevation varies approximately between El. -13.5 m and El. -19.5 m.

- Bottom of Deas Slough at the deepest location is shown to be at an approximate elevation of -3.7 m.

All elevations referred to in this memorandum are with respect to the Geodetic Datum.

3. TECHNICAL SUMMARY

This section provides a summary of the geotechnical input provided to the COWI/Stantec team for the Project.

3.1 SUBSURFACE CONDITIONS

3.1.1 Surficial Geology

The Surficial Geology Map No. 1486A, prepared by the Geological Survey of Canada identifies the subsoil conditions at this site as Fraser River Sediments – Deltaic and distributary channel fill sediments overlying and cutting estuarine sediments and overlain in much of the area by overbank sediments.

3.1.2 Geotechnical Exploration Work

Geotechnical exploration work, including Cone Penetration Testing with pore water pressure measurements (CPTu), Cone Penetration Test with shear wave velocity measurements (SCPTu), boreholes with Standard Penetration Test (SPT), soil sampling, laboratory testing and a pile load test were completed at locations along the existing Highway 99 and the tunnel alignment, at various times between 1956 and 2017. The previous work is described within the following reports:

1. Massey tunnel site investigation, dated June 04, 1991.
2. Geotechnical exploration and report, George Massey Tunnel CPT and drillhole testing 2006, dated October 18, 2006.
3. Factual Geotechnical report, Highway 99/Steveston Highway Interchange improvement project, dated February 28, 2013.
4. Field Data Report, George Massey Tunnel Replacement (GMTR) project, dated January 15 to May 29, 2014.
5. Downhole Seismic Field Data Report, GMTR project, dated February 24 and March 24, 2014.
6. Geotechnical data report, GMTR project, dated April 23, 2014.
7. Supplemental Geotechnical laboratory testing, GMTR project, dated June 23, 2014.
8. Geotechnical data report, Highway 99 and Interchanges, GMTR project, dated March 17, 2015.
9. Geotechnical data report, Test pile site, GMTR project, dated April 28, 2015.
10. Geotechnical data report, Steveston Highway Interchange and Green Slough, GMTR project, dated December 03, 2015.
11. Geotechnical data report, BC Hydro line relocation, GMTR project, dated December 11, 2015.
12. Geotechnical data report – Oak Street Bridge to Ladner Trunk Road, GMTR project, dated February 02, 2017.

Drawings showing the locations of the previously completed boreholes were provided to the COWI/Stantec team but are not included in this summary.

Brief description of the borehole drilling work, completed between 1956 and 2017 is given below:

Borehole drilling in 1956: Ten boreholes were completed along the existing tunnel alignment and the approaches. The depth of the boreholes drilled over water varied approximately between 20 m and 30 m below the river bottom.

Borehole drilling and CPT work in 1991: CPTu, SCPTu and boreholes with SPT were completed in 1991 by the B.C. Ministry of Transportation and Highways. This work included four CPTu holes over water to depths of 18.2 m to 30.6 m below the river bottom and two CPTu holes on land near the north and south entrance to the tunnel to depths of 27.2 m and 38.3 m respectively below the existing ground surface. Shear wave velocity profiles were obtained in three of the cone penetration test holes over water and in both land holes. Two over-water boreholes to depths of 9 m and 13 m below the river bottom were also completed. SPTs were completed in the two boreholes at approximately 1.5 m depth interval.

Borehole drilling and CPT work in 2006: A total of 12 CPTu holes and two boreholes were completed within the river on either side of the tunnel. The depth of the CPTu holes varied between 19.6 m and 32.1 m below the river bottom and the boreholes were 18.3 m and 26.5 m deep. ODEX drilling method was utilized at each CPT location to drill through the existing rip-rap and the gravel blanket prior to the CPT work. The drill holes were advanced into the river bottom to collect soil samples to depths varying between 3 m and 9.8 m.

Borehole drilling and CPT work in 2013: The work in 2013 was for the preliminary design of Steveston Highway Interchange, and included drilling of ten auger boreholes to 6.1 m to 15 m depth and three CPTu and one SCPTu holes to 32.3 m to 45.5 m below the ground surface.

Borehole drilling and CPT work in 2013 – 2014: Work completed in 2013 to 2104 included the following:

- Two deep boreholes near the north and south entrance to the existing tunnel, BH13-01 (335 m deep) and BH13-02 (338 m deep) respectively. The records of the boreholes indicate that the Holocene-Pleistocene Geologic boundary was at 314 m and 317 m depth below the existing ground surface near the north and south entrance to the existing tunnel respectively.
- Four 165 m to 170 m deep SCPTu holes, two holes near the north entrance to the tunnel (DEEP-SCPT14-03 and -04) and the other two near the south entrance (DEEP-SCPT14-01 and -02).
- Six 100 m to 106 m deep CPTu holes between the north bank of the river and Steveston Highway (CPT14-30 to -34 and -46).
- Seven 100 m to 110 m deep CPTu holes in Deas Island (CPT14-35 to -41).
- Four 100 m deep CPTu holes in Delta, between the south bank of Deas Slough and River Road.
- Six 75 m deep SCPTu holes between the north bank of the river and Steveston Highway (SCPT14-01 to -06).

- Two 75 m deep SCPTu holes in Deas Island, one at mid-length of the island and the other at the southern tip (SCPT14-12 and -11 respectively).
- Four 75 m deep SCPTu holes between the south bank of Deas Slough and Highway 17A (SCPT14-07 to -10).

Borehole drilling and CPT work in 2015: Work completed in 2015 included the following:

- Eight 3 m deep auger boreholes along the existing shoulders of Highway 99, from Westminster Highway to Steveston Highway.
- 24 CPTu holes to depths of 40 m to 100 m below the existing ground surface, located between Westminster Highway and Highway 17A.
- One deep borehole to 202 m depth and three CPTu holes to 100 m depth below the existing ground surface at the pile load test location, in Deas Island, near the south entrance to the existing tunnel.
- Five boreholes to depths of 11 m to 50 m and two SCPTu holes to 55 m depth below the existing ground surface, located on the north and south sides of the Fraser River and west side of the existing tunnel.

Borehole drilling and CPT work in 2017: Work in 2017 included two SCPTu holes to 105 m and 132 m depth, and one sonic borehole to 53 m depth below the existing ground surface, located between Steveston Highway Interchange and the existing tunnel.

3.1.3 Subsurface Soil Conditions

The subsurface soil conditions described in this memorandum were derived using borehole records provided by the MoTI. Table 1 provides a summary of the generalized subsurface conditions, in the order of increasing depth.

Table 1. Generalized Subsurface Soil Conditions along the Proposed GMC

Soil Unit	Description	Thickness
Post-glacial. Fraser delta and marine sediments.	Compressible clayey SILT and silty CLAY, firm to stiff near the surface and becomes soft with increasing depth.	• 2.5 to 4.5 m – onshore. • Not present in the river channel.
	Loose to compact SAND to silty SAND, intersected by clayey SILT layers.	• 23 to 30 m – onshore. • 15 to 25 m – below river bottom.
	Compressible clayey SILT to silty CLAY interlayered with SAND. Bottom of this soil layer is about 314 and 317 m deep below the existing north and south dykes respectively. *Five to ten metre thick compact sand layers were encountered between 50 m and 75 m depth below the existing ground surface. The sand layers are not continuous between boreholes.*	About 300 m
Pleistocene	Dense to very dense SAND and very stiff to hard SILT and clayey SILT	Estimated to be more than 700 m

3.1.4 Soil Parameters

Soil parameters used in geotechnical analyses for developing preliminary design considerations are discussed in this section.

The in-situ void ratio for the silts and clays was estimated using the site-specific natural moisture content (W) values reported in the borehole logs together with an assumed specific gravity (G_s) of 2.7 for silts and clays: $e = (G_S)(W)$, assuming water saturated soils. The unit weight was estimated by the equation as $(1+W)/(1+e)(G_s \times \gamma_w)$, where γ_w is the unit weight of water. Unit weight of sand was taken as 19 kN/m^3 based on previous work and published literature on Fraser River sands (e.g.: Uthayakumar and Naesgaard, 2004)[1]. Poisson's ratio was assumed as 0.3. Shear modulus, G: The maximum shear modulus was calculated G_{max} as $(\rho)(V_s)^2$, where ρ is the density of soil and V_s is the shear wave velocity as measured in $SCPT_u$ locations.

The effective shear strength parameters for the subsurface sands were obtained from published laboratory test data on Fraser River sands (e.g.: Uthayakumar and Vaid, 1998[2], Vaid and Thomas, 1995[3]). Undrained shear strength of the silts and clays were derived using the Cone Penetration Test (CPTu) data as $S_u = (Q_t - \sigma_v)/N_{kt}$, where Q_t is the cone resistance, σ_v is the total vertical stress and N_{kt} was taken as 15, based on published

[1] Uthayakumar, M. and Naesgaard, E. Ground response analysis for seismic design in Fraser River delta, British Columbia. Proceedings of the 13th World Conference on Earthquake Engineering, August 1-6, 2004, Vancouver, BC.
[2] Uthayakumar, M. and Vaid, Y. P. (1998). "Static liquefaction of sands under multi axial loading". Canadian Geotechnical Journal, Vol. 35, pp.273-283.
[3] Vaid, Y.P. and Thomas, J. (1995). "Liquefaction and post-liquefaction behavior of sand". Journal of Geotechnical Engineering; Vol. 121(2): pp. 163-173.

correlations between laboratory tests and CPTu data on Fraser Delta soils (e.g.: Creig, Campanella and Robertson, 1987[4]). Table 2 provides soil parameters for preliminary analysis and study.

Table 2. Soil Parameters for the Proposed GMC Analysis and Study

Soil Description	Thickness (m)	Unit Wt (kN/m^3)	Friction Angle, (deg.)	Undrained Shear Strength, (kPa)
clayey SILT and silty CLAY – near the ground surface onshore. Not present in the river channel.	4	17.5	0	30
Loose to compact SAND to silty SAND	15 to 30	19	35	0
clayey SILT and silty CLAY. Five to ten metre thick compact sand layers were encountered between 50 m and 75 m depth.	>100	18	0	2H

H is the depth in metres from the existing ground surface for onshore locations and from the river bottom for locations along the river channel. S_u for the deep SILT and CLAY was estimated as 0.24 times the vertical effective stress.

3.2 SEISMIC DESIGN CONSIDERATIONS

The new GMC is designated as a "Lifeline" structure. Therefore, the design will comply with the performance criteria specified in the BC Supplement to the Canadian highway bridge design code CAN/CSA-S6-14. Design earthquake motions with return periods of 475, 975 and 2475-years would have to be considered as per CAN/CSA-S6-14. We have obtained seismic design parameters as specified in CAN/CSA-S6-14 for the three design return period events.

Earthquake ground motions and its frequency content would be altered as the earthquake induced shear waves propagate through the Fraser delta and the underlying marine sediments. Preliminary seismic ground response analyses (SGRA) were completed as part of this Project. The analyses were limited to the 2475-year return period event only for this preliminary study. The objectives of the SGRA are the following:

- Evaluate the effect of the deep soil deposit on ground response during earthquake shaking.
- Develop site-specific design response spectra for structural analyses.
- Obtain acceleration time-histories at the base of the proposed structures for potential use in structural analyses.
- Calculate cyclic shear stresses for liquefaction assessment.

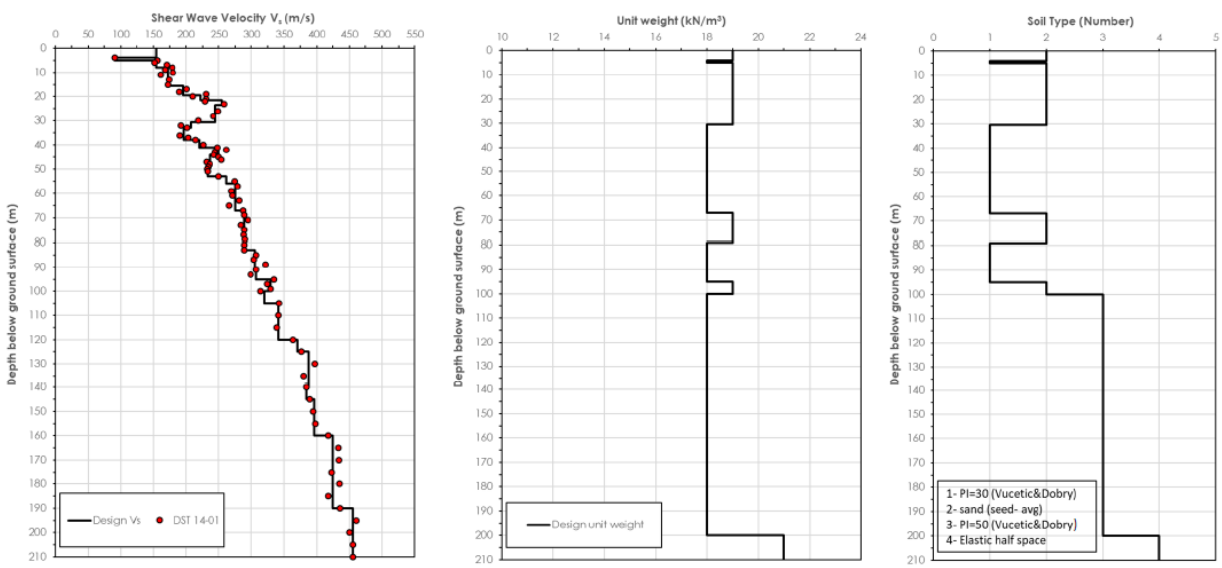
The input to the SGRA included fifteen sets of "firm-ground" time-history records, spectrally matched to the 2,475-year return period Uniform Hazard Response Spectrum (UHRS). "Firm-ground" is defined as Site Class

[4] Creig, J.W., Campanella, R.G., Robertson, P.K. (1987). "Comparison of field vane results with other in-situ test results". ASTM Proceedings, International symposium on laboratory and field vane shear strength testing, Florida., January, 1987

C soils with an average shear wave velocity of 450 m/s and the UHRS was derived in accordance with the provisions of CAN/CSA-S6-14 and the National Building Code of Canada (NBCC, 2015).

The records used in the analysis were developed earlier as part of the GMTR project in 2016. A single set of records is comprised of two orthogonal horizontal records and the relevant vertical record. The fifteen sets of time-history records included five sets of three sub-groups, representing sources from (1) shallow crustal, (2) deep in-slab, and (3) subduction interface earthquakes. All fifteen sets of time-history records were used in the SGRA.

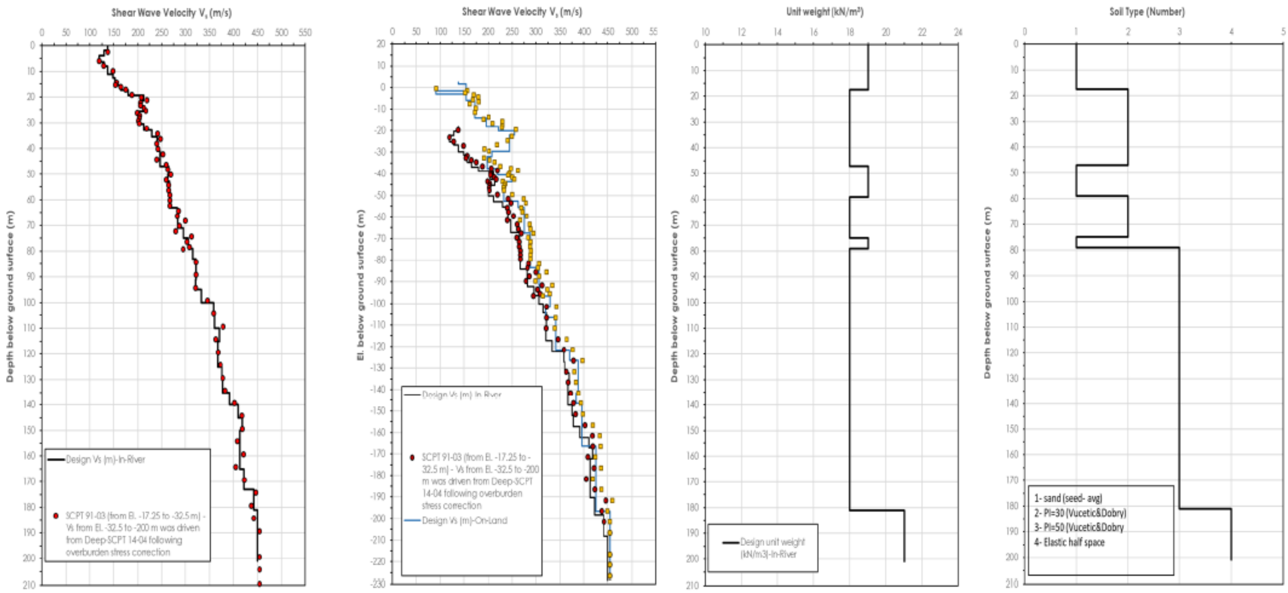
Details of the analyses, procedures and models are not provided in this memorandum due to space limitation. Graphical representation of the soil models used in the analyses are provided in Figures 1 to 3.



Borehole: BH 13-01
CPT: Deep-SCPT14-04

Figure 1. Soil Model used in Seismic Ground Response Analysis – North Bank of Fraser River

Reference: **Geotechnical Engineering Technical Summary Memorandum for the Proposed George Massey Crossing, Richmond/Delta, BC**



Data from:
1- SCPT: 91-03 (from El. -17.5 to 32.5 m)
2- Vs from El. -32.5 m to -200 m was driven from Deep-SCPT 14-04 following overburden stress correction

Figure 2. Soil Model used in Seismic Ground Response Analysis – In-River

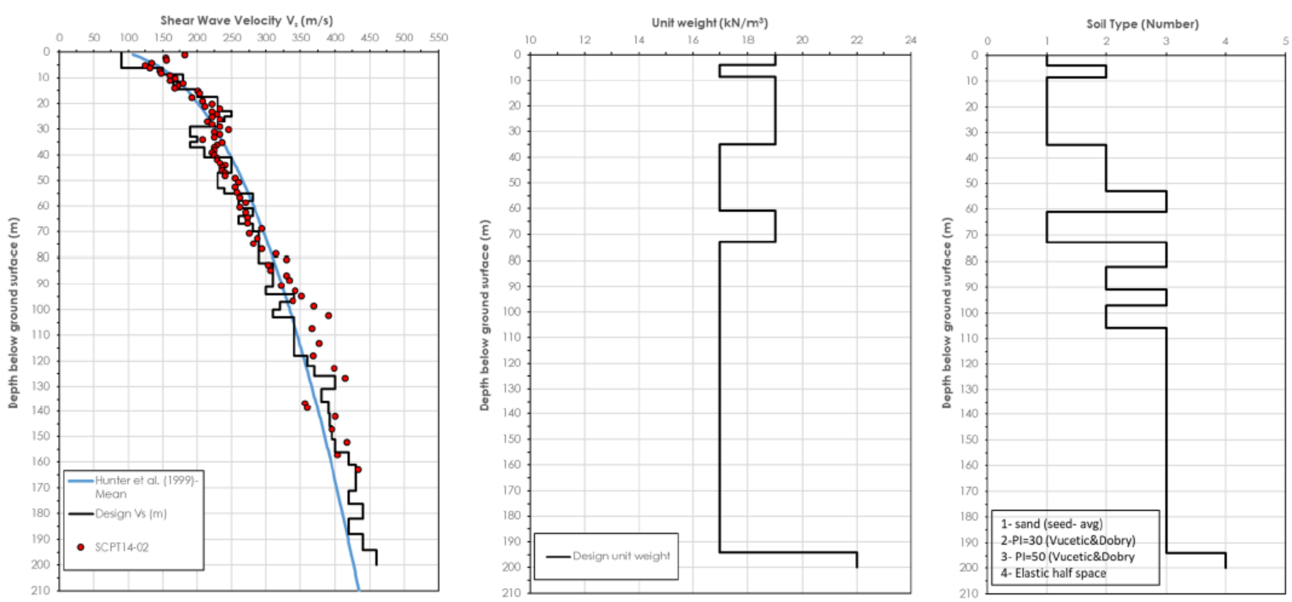


Figure 3. Soil Model used in Seismic Ground Response Analysis – South Bank of Fraser River

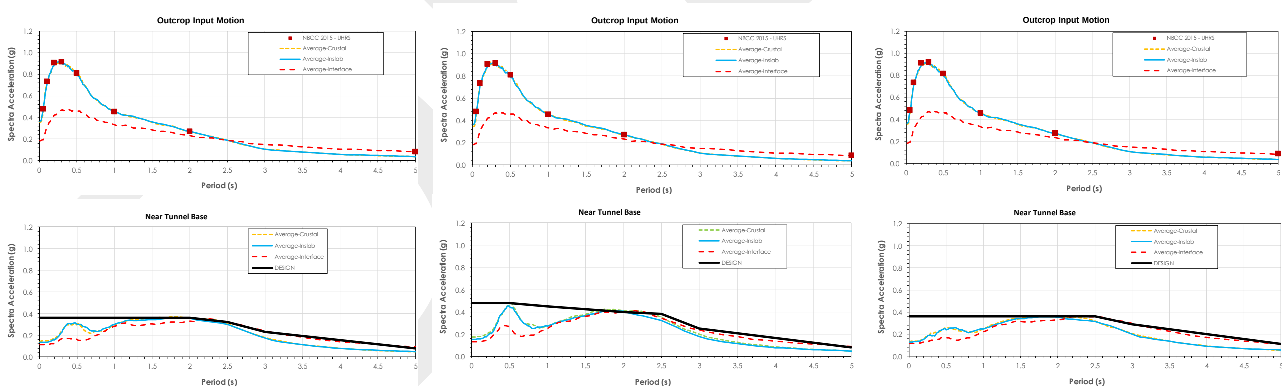
Reference: Geotechnical Engineering Technical Summary Memorandum for the Proposed George Massey Crossing, Richmond/Delta, BC

3.2.1 Seismic Design Response Spectrum and Acceleration Time Histories

Seismic design response spectra were developed using ground motions from the SGRA at various depths, including at the base of the proposed immersed tube tunnel (ITT), bridge pile caps and at the ground surface. The SGRA analyses were repeated at three representative locations along the proposed GMC alignment – the north and south banks of the river and in-river locations. The design response spectra were then provided to the design team as requested. Similarly, representative acceleration time histories were provided at the base of the proposed ITT. Each of the time histories included two horizontal orthogonal components and the corresponding vertical component, taken at the base of the tunnel at three locations, the north bank, in-river and the south bank.

Figures 4a, 4b and 4c present the acceleration response spectra from the SGRA analysis for north bank, in-river and for the south bank of the Fraser River respectively at the GMC.

A representative set of acceleration time histories, one of horizontal crustal and one of horizontal interface time history are shown in Figures 5a and 5b respectively for illustration.



(a) (b) (c)

Figure 4. Acceleration response spectra for (a) north bank (b) in-river and (c) south bank of the Fraser River at the GMC

Reference: Geotechnical Engineering Technical Summary Memorandum for the Proposed George Massey Crossing, Richmond/Delta, BC

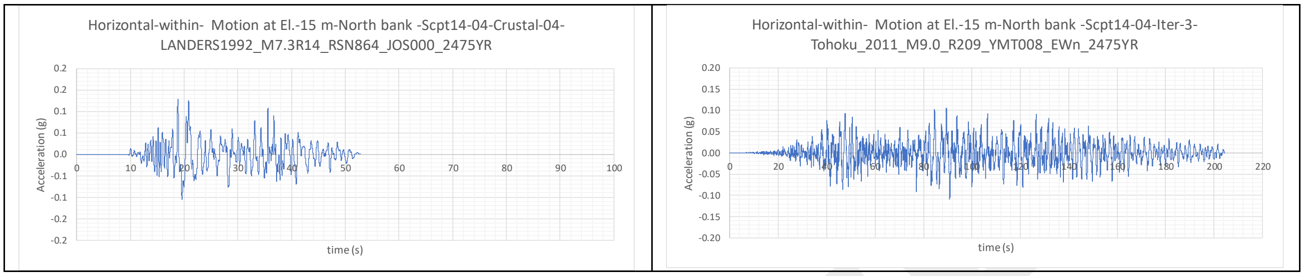


Figure 5. Acceleration time history at the base of the tunnel at the north bank of the Fraser River at the GMC (a) one horizontal component from Crustal Earthquake (b) one horizontal component from Interface Earthquake

3.2.2 Liquefaction of Soils

Liquefaction assessment of the subsurface soils were carried out using ground motions developed from the SGRA and the method developed by Boulanger and Idriss (2014). Details of the assessment are not included in this memorandum. Preliminary liquefaction assessment shows the following for the 2475-year return period seismic event:

- The existing sand to silty sand layer is liquefiable. Thickness of this liquefiable sand layer is assessed to be about 23 m on the northside of the Fraser River, about 30 m on the south side of the river and about 15 m to 30 m within the river channel.
- The five to ten metre thick sand layers within the deep SILT/CLAY layer are also considered to be liquefiable for the design 2475-year event. The sand layer is not continuous, occurs at different depths at different locations along the alignment. For preliminary design work, we have assumed about 40% of the five to ten metre thick sand layers below El. -25 m could potentially liquefy during the design 2475-year seismic event.

3.2.3 Consequence of Liquefaction

Consequences of liquefaction would include ground settlement, down drag load on piles foundations, reduction in pile capacity, pile settlement, large settlement or failure of shallow foundations, deformation of embankments, slopes and foundations, uplift of buried structures including tunnels, breakage of tunnel joints and water in-fill due to differential movement in horizontal and vertical directions. Post-earthquake settlement, assuming no ground improvement, in the order of 40 mm to 760 mm is estimated at onshore locations, varying with the thickness and density of the soils. Post-earthquake settlement within the river channel is estimated to be in the range of 200 mm to 1000 mm. Very large deformation of the ground, in the form of flow-slide is estimated with the existing dykes deforming towards the river. Large horizontal deformation due to flow slide would also result in large vertical deformation.

3.2.4 Ground improvement

Ground improvement would be required to minimize the effects of liquefaction and to reduce the risk of flow-slide of the dykes. Ground improvement would be required as discussed below:

- Deep Bored Tunnel: Ground improvement would be required around the portals, and at each end of the tunnel as it passes through the liquefiable sandy soil layer.
- ITT: Ground improvement would be needed along the entire length of the tunnel to limit the deformation and uplift of the ITT elements.
- Bridge Structure: Ground improvement would be required around select pier foundations to maintain axial and lateral capacities of the structure.

The commonly used ground improvement methods to mitigate the effects of liquefaction include vibro-replacement stone columns, vibro-compaction, dynamic compaction, driving of compaction piles, compaction grouting, jet grouting and soil-cement mixing.

Considering the required depth of ground improvement, cost, environmental impacts and local experience, the vibro-replacement stone column method is considered most suitable for this project for work onshore. For ground improvement within the river channel, either, stone columns or driven compaction piles may be considered. Timber piles or precast concrete piles with a nominal 350 mm diameter may be considered as compaction piles for ground improvement.

Stone columns are typically installed at 2.7 m to 3 m equilateral triangular pattern in plan. Timber and precast concrete compaction piles are typically installed at 1.3 to 1.5 m square grid pattern. For preliminary consideration we have assumed the following depths and width of ground improvement:

- **Deep Bored Tunnel Option:** Ground improvement to El. -30 m. Improvement below the footprint of the twin tunnels, between the tunnels and 20 m wide area on either side of the tunnels.
- **ITT Option:** Ground improvement to El. -30 m onshore and to El. -35 m in-river. Improvement below the footprint of the tunnel and 20 m wide area on either side of the tunnel.
- **Bridge Option:** The proposed bridge piers and abutments are all located onshore and/or in Deas Slough and are assumed to be supported on deep pile foundations. The proposed ground improvement area is rectangular in plan, the footprint of the pile caps and extending outwards from all sides of the outer perimeter of foundation pile groups. For preliminary design, the area of improvement can be taken as the footprint of the pile cap plus 10 m wide envelope, measured from the outer edge of the pile cap. A window, free of stone columns would have to be provided for the installation of each of the foundation piles. For preliminary considerations, the width of the window can be considered as 1.0 m plus the pile diameter. Ground improvement to elevation up to El. -30 m would be required.

3.3 STIFFNESS COEFFICIENTS

Dynamic stiffness coefficients for the proposed ITT were developed as requested by the GMC ITT Lead. The coefficients were developed using the procedure of Gazetas, G. (1991)[5]. The coefficients were developed using soil parameters described previously and in the X (longitudinal), Y (transverse) and Z (vertical) directions for

[5] Gazetas, G. (1991). Foundation Vibrations. Foundation Engineering Handbook, edited by Fang, H.Y., Van Nostrand Reinhold, New York.

the proposed tunnel elements, each assumed to be 103 m x 46 m x 10.2 m (length x width x height). The estimated dynamic stiffness coefficients are provided in Table 3.

Table 3. Dynamic Stiffness Coefficients for the Proposed ITT Elements 103 m x 46 m x 10.2 m

Deformation Direction	(a) For ITT Elements North of Sta. 117+50	(b) For ITT Elements from Sta. 117+50 to 122+50	(c) For ITT Elements South of Sta. 122+50
Vertical, K_z	5,200 MN/m	2,900 MN/m	5,200 MN/m
Horizontal Transverse, K_y	6,100 MN/m	3,200 MN/m	6,100 MN/m
Horizontal Longitudinal, K_x	6,100 MN/m	3,200 MN/m	6,100 MN/m

3.4 EXCAVATION

The proposed ITT and DBT options require deep excavations onshore for the approach structures. In addition, excavation of the river bottom would be required for the construction of the ITT tunnel elements. Further, large area of excavation would be required for the construction of a dry dock to fabricate the ITT tunnel elements and then to float them out to the final location of the alignment. Excavation using the tunnel boring machine for the DBT option is not discussed here.

Preliminary analysis shows that the excavation of the trench at the river bottom for the ITT can be sloped at 2H:1V or flatter. Slope stability analysis with 2H:1V underwater slope in sands shows a factor of safety of 1.3 against slope failure for this temporary excavation work. Stability of the existing tunnel and the tunnel protection, including rip-rap, gravel and concrete mattress were considered in selecting a clear distance of 42 m between the existing and the new ITT.

3.4.1 Onshore Excavation Shoring

Excavation as deep as to elevation -37 m is required onshore for the construction of the tunnel elements and approach structures for the ITT and DBT options. Slope cutting the excavations has been reviewed and ruled out as the site is constrained with the existing highway, adjacent properties, and available space for construction traffic and laydown area. Excavation with vertical shoring wall options were then reviewed. The excavation shoring options reviewed included steel sheet pile walls, reinforced soil-cement mixed walls, jet-grouted reinforced walls, and cement-slurry walls. Shotcrete walls with soil anchor option was assessed to be not suitable for the soil conditions at the site.

The shoring walls would have to be braced either, internally within the excavation or externally. The external support could be another row of buried walls acting as deadman or using soil anchors.

Installation of the ITT sections located outside the existing riverbanks could be carried out by excavating trenches and placing the tunnel sections underwater.

For the approach structures of both, ITT and BT, an important consideration of the excavation shoring works is groundwater cut-off. A dry working surface is required within the excavated areas. Presence of shallow groundwater table, and the presence of the existing tunnel, settlement sensitive buildings, utilities, adjacent infrastructure and agricultural lands require pumping of groundwater from the excavation areas to be limited to as low as practically possible. Otherwise, unacceptable settlement and deformation of structures, road embankments and buried utility lines and drying out of the agricultural lands may occur. The excavation shoring, including the length of the shoring walls, would have to be designed to minimize the volume of groundwater pumping.

The bottom of the sand layer in the boreholes drilled at the Site extended to as deep as to EL. -30 m. To cut-off groundwater seepage the shoring walls could be extended two to three metres below the bottom of the sand layer.

3.4.2 Lateral Earth Pressure Parameters

Lateral earth pressure for the design of the excavation shoring may be estimated using the parameters provided in this subsection. In providing the parameters it is assumed the walls would deflect sufficiently to mobilize active earth pressure state. This would require horizontal movement in the order of 0.004 times the height at the top of the wall. Water pressure behind the walls would have be included. Further, loading from any construction vehicle and equipment behind the walls should be included as surcharge.

The following parameters may be used for estimating lateral pressure on the shoring walls:

- Above the groundwater level, soil pressure: $p_a = \gamma\ H\ K_a$
- Below the groundwater level, soil pressure: $p_a = \gamma\ D_W\ K_a + (\gamma - \gamma_w)(H\text{-}D_W)K_a$

where:

- H is the height of the wall below the ground surface
- D_W is the groundwater depth (assume a groundwater elevation of El. +2 m)
- γ is the unit weight of soil (assume 19 kN/m^3)
- γ_w is unit weight of water (9.8 kN/m^3)
- K_a is the "active" earth pressure coefficient (assume 0.3).

Increase in earth pressure due to seismic loading is not included for the design of the temporary excavation shoring works.

3.5 BRIDGE FOUNDATIONS

As the subsurface soils consist of compressible silts and clays, and loose to compact liquefiable sands, the bridge piers and abutments would have to be supported on deep pile foundations. The following pile types have been used in previous major highway bridge projects in Vancouver and the surrounding areas:

- Driven steel pipe piles with diameters up to 2000 mm
- Bored, cast-in-place concrete piles with diameters up to 2800 mm

3.5.1 Pile Load Test Program at the Project Site on Driven Steel Pipe Pile

A static pile load test program on a 2 m diameter driven steel pipe pile was completed at the Project site in August 2016. The test pile was driven to approximately 68 m depth below the existing ground surface near the south entrance to the tunnel in Deas Island. The pile was instrumented to obtain friction capacity distribution along the shaft and end bearing at the toe of the pile. The test data indicates a shaft capacity of about 17 to 18 MN and a toe capacity of about 2 to 3 MN for a total capacity of about 20 MN.

3.5.2 Pile Load Test for the Golden Ears Bridge Project on Bored Cast-in-Place Concrete Pile

Pile load test using “O-cell” method on a nominal 2.5 m diameter 74.5 m long bored pile was completed. The test pile was constructed by vibrating a permanent 2.5 m diameter steel casing to a depth of 21 m and then excavating with a spherical grab to 74.5 m depth. Polymer slurry with a positive head of about 7.5 m above the water table was used to help maintain stability of the hole. From sonar caliper tests, an average shaft diameter below the steel casing was determined to be of 2.6 m (actual diameter of the uncased section varied between 2.5 m and 2.8 m). The subsurface soil conditions at the test- pile location included 17 m of loose to compact silty sand to sand overlying 21 m of compact to dense sand overlying stiff silty clay with intermittent thin silty sandy layers to depth beyond 100 m. The “O-Cell” test data shows a shaft capacity of 58 MN and a toe capacity of 4 MN to a total capacity of 62 MN. The “stiff silty clay with intermittent thin silty sandy layers” at the Golden Ears Bridge site are stiffer (stronger) than the silt/clay found at this Project site.

3.5.3 Preliminary Axial Pile Capacity of Driven Steel Pipe Pile Foundations

Axial capacities of piles have been calculated using the following methods for driven steel pipe piles:

- Using the results of the static pile load test completed at the Project site
- CPT method (LCPC - Bustamante and Gianeselli 1982[6], Canadian Foundation Engineering Manual, 2006[7])
- Effective stress (beta) method (Canadian Foundation Engineering Manual, 2006).

Estimated unfactored ultimate axial capacities for driven steel pipe piles, 2500 mm in diameter are provided in Table 4. A minimum pile spacing of 2.5 times to 3 times the pile diameter, centre-to-centre, is typically used for the design of piles in groups. It is assumed the piles would be driven open-ended, then the inside of the piles would be cleaned out and filled with concrete. The clean out and concrete in-fill depth is assumed to be in the range of 6 to 10 times the pile diameter.

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[6] Bustamante, M. and Gianeselli, L. (1982). "Pile bearing capacity prediction by means of static penetrometer CPT"; Proceedings of the second European symposium on penetration testing, May, 1982.
[7] Canadian Foundation Engineering Manual, 4th Edition (2006). Published by the Canadian Geotechnical Society, c/o BiTech Publishers Ltd., 173 – 11860 Hammersmith Way, Richmond, BC, V7A 5G1.

Reference: **Geotechnical Engineering Technical Summary Memorandum for the Proposed George Massey Crossing, Richmond/Delta, BC**

Table 4. Pile Tip Elevation, Embedment Depth and Capacity of 2500 mm Diameter Driven Steel Pipe Piles

Location Along Hwy 99 Corridor	Pile Tip Elevation (m)	Approximate Embedment depth (m)	Ultimate Shaft (Friction) Capacity (MN)	Ultimate Toe Capacity (MN)	Ultimate Capacity (Shaft + Toe) (MN)
North Approach	-81 to -84	83 to 88	22	3	25
Near the North Bank of the River	-76	78	22	3	25
Near the South Bank of the River	-66	68 to 70	22	3	25
South Approach Near the Deas Slough/Hwy 99	-85	90	22	3	25
Near River Road/Hwy 99	-76	80	22	3	25

3.5.4 Preliminary Axial Pile Capacity of Bored Cast-in-Place Concrete Pile Foundations

Axial capacities of bored cast-in-place concrete piles (bored piles) have been calculated using the following methods:

- Review of test data from the “O-cell” test for the Golden Ears bridge, in consideration of the difference in subsurface soil and geology between the two project sites
- CPT method (LCPC - Bustamante and Gianeselli 1982[8], Canadian Foundation Engineering Manual, 2006[9])
- Total stress (alpha) method (Canadian Foundation Engineering Manual, 2006).

Estimated unfactored ultimate axial capacities for the bored piles, 2500 mm in diameter are provided in Table 5. A minimum pile spacing of 2.5 times to 3 times the pile diameter, centre-to-centre, is typically used for the design of piles in groups.

Table 5. Pile Tip Elevation, Embedment Depth and Capacity of 2500 mm Diameter Bored Piles

[8] Bustamante, M. and Gianeselli, L. (1982). "Pile bearing capacity prediction by means of static penetrometer CPT"; Proceedings of the second European symposium on penetration testing, May, 1982.
[9] Canadian Foundation Engineering Manual, 4th Edition (2006). Published by the Canadian Geotechnical Society, c/o BiTech Publishers Ltd., 173 – 11860 Hammersmith Way, Richmond, BC, V7A 5G1.

Reference: Geotechnical Engineering Technical Summary Memorandum for the Proposed George Massey Crossing, Richmond/Delta, BC

Location Along Hwy 99 Corridor	Pile Tip Elevation (m)	Approximate Embedment depth (m)	Ultimate Shaft (Friction) Capacity (MN)	Ultimate Toe Capacity (MN)	Ultimate Capacity (Shaft + Toe) (MN)
North Approach	-81 to -84	83 to 88	34 to 40	2 to 4	36 to 42
Near the North Bank of the River	-76	78	35 to 36	2 to 5	38 to 40
Near the South Bank of the River	-66	68 to 70	28 to 32	2 to 4	32 to 34
	-70	72 to 74	29 to 34	2 to 4	33 to 36
	-75	77 to 79	31 to 39	1 to 4	35 to 40
South Approach Near the Deas Slough/Hwy 99	-85	90	38 to 43	1 to 5	41 to 44
Near River Road/Hwy 99	-76	80	35 to 38	2 to 5	40

3.6 INSTRUMENTATION AND MONITORING

Geotechnical instrumentation would be required to measure groundwater levels, surface and subsurface movement, movement of existing tunnel and other infrastructure during construction. Performance requirements and tolerable deformation of the existing tunnel could be established. The objective is for the construction contractor to implement construction means and methods to avoid or limit movements or deformation of the existing tunnel and other structures to values lower than the threshold values. Instrumentation that could be used includes the following:

- Inclinometers
- Ground Deformation Monitoring Points (GDMP): consist of a capped steel pipe installed in the ground to monitor vertical deformation (settlement or heave) of the ground at or near the ground surface.
- Surface Monitoring Points (SMP): consist of PK survey nails driven into the pavement to monitor vertical deformation (settlement or heave) of the pavement at the ground surface. SMPs could be installed in an array across Highway 99 approach to the existing tunnel. Since pavement can “bridge” underlying soil displacements (i.e., ground settlements do not immediately show at the pavement surface), it would be worthwhile to occasionally pair an GDMP with an SMP array to monitor soil displacements below the pavement.

Reference: Geotechnical Engineering Technical Summary Memorandum for the Proposed George Massey Crossing, Richmond/Delta, BC

- Survey Monuments: Install survey monuments on the concrete base slab or concrete foundations of the existing tunnel or adjacent structures.
- Utility Monitoring Point (UMP): used for monitoring vertical deformation (settlement or heave) of an existing underground utility. UMPs consist of a steel pipe installed in borehole over an existing subsurface utility of concern, such that its bottom rests on top of the utility.
- Instrumentation to measure the existing tunnel joints and approach wall structures.

A monitoring and reporting program for the instruments could be established. This would include monitoring and reporting during installation of the shoring walls, excavation, construction of the GMC and until backfilling the excavation and removal of the shoring walls and support structures.

4. CLOSURE

This report was prepared for the exclusive use of the MoTI and its agents for specific application to the GMC Options Analysis and Comprehensive Feasibility Study project. Any use of this report or the material contained herein by third parties, or for other than the intended purpose, should first be approved in writing by Stantec. Use of this report is subject to the General Conditions enclosed.

We trust that the information contained herein meets your current requirements. If you have any questions, or if we can be of further assistance, please do not hesitate to contact the undersigned.

Regards,

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