Technical Memorandum

To:	Darryl Matson, P.Eng. GMC Technical Lead	Project:	George Massey Crossing
From:	Marco Moccichino, P.Eng GMC Bored Tunnel Lead	cc:	Anne-Marie Langlois, Doug Grimes, Fred Marquis
Date:	November 15, 2019	Doc No.:	58362_005_MO_BT_A_Technical_Summary
Subject:	DRAFT –Bored Tunnel Technical Summary		

Revision Log

Revision No.	Date	Revision Description
A	Nov 15, 2019	Issued for review

1.0 Introduction

In July 2019, the COWI-Stantec Team (CST) was awarded a contract by the Ministry of Transportation and Infrastructure (MoTI) for technical services to study crossing options for the George Massey Crossing project (GMC). McMillen Jacobs Associates (MJA) is a subconsultant to COWI providing specialist technical input for bored tunnel options.

This technical summary provides the key design assumptions and findings for the bored tunnel concepts and for the support of excavation structures required at the entry and exit points of the tunnel.

The bored tunnel concept developed by MJA for the Independent Technical Review Report (dated September 2018) by Westmar Advisors (led by Stan Cowdell), was used as starting point. The 2019 work involved gathering new information and establishing design assumptions during the Team interactions and meetings with MoTI. The key assumptions of the concept are divided in four categories:

1. Site Conditions (ground and groundwater conditions);
2. Relevant General Design Requirements;
3. Traffic Requirements; and
4. Technology Requirements.

Additional details on the assumptions are provided below.

1.1 Site Conditions

The subsurface soil conditions consist of post-glacial Fraser River delta and marine sediments. Up to 25 m below the river bottom, the soil consists of loose to compact sand to silty sand, with clayey silt layers (Fraser River Sands). Beneath this soil, is marine sediments consisting of compressible clayey silt to silty clay interlayered with sand. Below this, a glacial/Pleistocene layer is encountered some 700 m below grade. Onshore, the water table is considered close to grade.

The shallow sand to silty sand layer is liquefiable during an earthquake. The thickness of this liquefiable sand layer is considered to be from the surface down to approximately -30 m elevation (geodetic) below the river channel. In addition, sand layers (up to 10 m of thickness) within the marine silt/clay are also considered to be potentially liquefiable in an earthquake.

1.2 Relevant General Design Requirements

The design life for the new structure is 120 years. The new structure is designated as a “lifeline” and it therefore needs to resist the 2,475 year design seismic event. The bored tunnel shall also be flood resilient to a grade of elevation 4.38 m on both shores to protect both the local communities and the tunnel itself from flooding (this represents the 500 year flood level, plus 1 m allowance for future sea level rise, plus 0.6m freeboard). It is possible that the Provincial Dike Authority will require additional height for the flood resiliency, however this is readily accommodated within the options presented.

The new crossing shall connect the existing interchanges (with improvements, if needed) with Highway 17A to the south and Steveston Highway to the north. The maximum slope of the roadway is 5%.

The Fire Life Safety (FLS) provisions shall conform to National Fire Protection Association (NFPA) and specifically to NFPA 502 and NFPA 130.

1.3 Traffic Requirements

MoTI initially proposed 18 options for the crossing comprising different numbers of lanes, configurations and use of the existing tunnel. In July 2019, 12 options were eliminated, resulting a shortlist of 6 options, all with 6 General Purpose (GP) traffic lanes and two dedicated bus lanes. Each of the three technologies (bored tunnel, immerse tube tunnel, and bridge) have two options – one option with 6 GP lanes on the new structure and the bus lanes in the existing tunnel, and one option with all 8 lanes on the new structure. The bored tunnel concept focused on the following:

a) new twin bored tunnels with 6 GP lanes, plus multi-use path (MUP) and 2 transit lanes in the existing tunnel (to be retrofitted)
b) new twin bored tunnels with 6 GP lanes and 2 transit only lanes, and MUP in the existing tunnel (to be retrofitted)

MoTI requirements were used for the roadway geometry: 3.7 m wide transit lane; 3.7 m wide GP lanes; 1 m inside shoulders; and 2.5 m wide outside shoulders. The vertical clearance is 5 m with additional 500 mm provision for overhead road signs.

1.4 Technology Requirements

A bored tunnel is excavated with a tunnel boring machine (TBM). A TBM is a purpose built machine that sequentially excavates and installs a permanent concrete lining of a discrete length of the tunnel. The bored tunnel has fixed inside and outside diameters (ID and OD, respectively) and the typical cross section must be finalized before the manufacturing the TBM. Due to the number of lanes required at GMC, a stacked configuration of traffic lanes and two tubes are required to make efficient use of the space available within the bore diameter.

Bored tunnels, being under the water table, tend to be buoyant. Ground cover is necessary to assure a standard factor of safety (industry standard is a factor of safety not less than 1.3). Underground structures located in liquefiable soil are susceptible to floatation following a major earthquake event. Such failure phenomenon generally occurs when the soil liquefies and loses its shear resistance against the uplift force from the buoyancy of the underground structure. Liquefiable soil not only does not provide a dead load contribution to buoyancy, but aggravates the equilibrium by increasing the uplift forces.

Lastly, bored tunnel technology requires the excavation of a launching pit or portal with sufficient ground cover where the TBM starts tunnelling. The launching pit and the surrounding laydown area serve as a tunnel yard throughout the duration of the excavation. Tunnel yard features of note are: considerable temporary power requirements (in the order of tens of megawatts); and incoming (tunnel lining) and outgoing (excavated material) truck traffic that may impact the nearby road network during tunnel boring hours which are typically 24 hours per day, five to seven days per week.

2.0 Results

2.1 Tunnel Depth

The current characterization of the depth of liquefiable soil and the prevalence of liquefiable materials within the clay layer below the Fraser River Sands drives the requirement for a deep and long tunnel (see drawing GEN-BT011 in Appendix A).

The tunnel crown is set at elevation -61 m geodetic (approximately 46 m below the river bed) to provide adequate resistance against uplift forces when accounting for the behavior of liquefiable soil. This represents a ground cover of approximately 2.5 times the tunnel ID.

Scour is typically a significant design issue to bored tunnels, as any scouring of the river bottom reduces the soil cover on top of the tunnel. However, since CST determined that the best solution for the alignment was to go deep enough to not have to rely on the layer of liquefiable sands for buoyancy resistance, the effects of the scour (diminution of the ground cover) does not influence the vertical alignment, and the concepts do not require scour mitigation measures.

MJA drafted a memorandum that presents the comparison between the 2018 and 2019 concepts of the bored tunnel (included in Appendix B). In addition, COWI performed an independent review of MJA work to confirm the depth of the bored tunnel (included in Appendix C).

MJA considered a shallower tunnel option which would have required the liquefiable sand layer to be densified over the tunnel to ensure buoyancy resistance could be provided by this layer. This resulted in the need for ground cover of approximately 1.5 times the tunnel ID, and therefore reduced the depth of the 8 lane option by about 18 m (and reduced the length by approximately 700 m). However, this option would require a broader ground improvement programme over the entire length of the crossing to include additional 2,800 m of densification across the full width of river. In addition, it would require extensive scour protection in the river over the tunnels for the full width of the river. Both of these requirements, especially the impacts in the river, were deemed to be significant enough to justify the deeper option being selected for the purpose of this project.

2.2 Tunnel Length

The depth of the tunnel under the river and Deas Slough, combined with the roadway design criteria, establish an approximate length of 3,500 m for each of the two drives, with additional transition portals approximately 450 m long at either end.

In plan, the new structures are located east of Highway 99 and upstream of the existing tunnel on the Richmond side of the river. They then cross over Deas Slough and eventually move to the west of Highway 99 on the Delta side of the river. The total length of the new structures (approximately 4.4 km) conflicts with both the existing interchanges (Steveston Highway and Highway 17A) and the concept requires them to be completely reconfigured.

2.3 Tunnel Geometry

The inside diameter of the tunnel, that conforms with the MoTI traffic requirements, is 17 m for the new 8 lanes option or 16 m for the new 6 lanes option (see drawing GEN-BT012 in Appendix A). The thickness of the segmental lining is 600 mm, as a minimum. These dimensions drive the size of the TBM beyond the current leading edge of the technology, with the 8 lane option requiring an excavation diameter of 19 m. To provide context, the recently completed Alaskan Way Tunnel (SR99) in Seattle, WA, adopted a TBM (Bertha) with 17.5 m excavation diameter, which was the largest diameter bored tunnel in the world as of 2014. An increment of 1.5 m in this upper range of excavation diameter is considered a challenge. No study has been performed regarding the expected type of TBM (earth pressure balance TBM or slurry TBM), but the use of pressured face TBM is required.

In term of FLS requirements, the tunnel section maintains the necessary ventilation with fans or longitudinal ducts mounted over head in each traffic cell. As such, no counterflow traffic can be accommodated, either in a single cell or in each tube. Emergency egresses are provided via refuge corridors that run longitudinally on both upper and lower decks with access through fire doors. Spacing of ventilation fans and fire doors are still to be determined at the time of drafting of this summary. For the 6 lane option, there is additional space for emergency egress corridors vs the 8 lane option.

2.4 Tunnel Portals

The bored tunnel concept requires a portal structure with head wall for launching and receiving the TBM. These portal structures are built using open cut methods and are expected to be approximately 450 m long in order to accommodate the tunnel horizontal and vertical alignment and traffic and roadway design criteria. At the portal head wall, the base of the excavation will be approximately 34 m below grade, 65 m wide and 110 m long (see Figure 1 below) and will serve as the main site for tunnel construction. The portal structures will need to accommodate a high ground water table with unlimited recharge. It is assumed that the portal wall will be a slurry (diaphragm) or secant pile wall with tiebacks. If the bored tunnel option is carried forward, this concept could be further optimized to mitigate the challenges of tiebacks in the Fraser River sand.

Away from the portal head wall, the excavation is sized to accommodate the double-deck traffic structure, leaving an unexcavated core in the middle. At this location, the support of excavation is expected to include a combination of slurry or secant pile walls with internal steel bracing. This excavation then transitions into a single large shallower bottom-up excavation structure (U-section) which will allow the transition for the double-decker traffic structure and tie-in with the at-grade roadway.

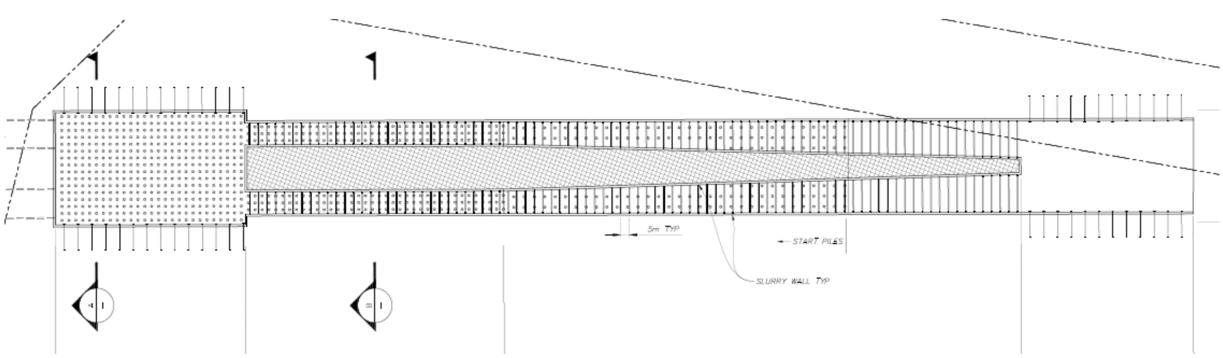


Figure 1: Portal Support of Excavation - Plan

2.5 Ground Improvement

When the tunnel profile rises above elevation -61 m, ground improvement is required, likely via densification with stone columns. The extent of ground improvement linearly increases with the reduction of cover. In addition, a massive ground improvement programme is necessary at the portal locations to contribute to the support of the excavation system.

In addition, ground improvement methods (most likely jet-grouting) will be required to seal the bottom of excavation and provide a water-tight structure. Because of the site conditions and the high hydrostatic uplift pressures, drilled piles are also required where the depth of excavation exceeds 10 m. The structures would then be excavated, and a base slab (working slab) would be cast on top of the jet grout slab. The required ground improvement for the onshore structures could be used also for the TBM break-out/break-in and to reduce the ground cover of the TBM at the portal walls.

2.6 Sinkhole Risk

Large diameter bored tunnels have an increased risk of sinkholes. Due to the risk, particularly if a sinkhole occurred under the river, MJA presented the risks of sink holes associated with the bored tunnel in a separated memorandum that is included in Appendix D. COWI also performed an independent study of the potential of sink holes for this concept (included in Appendix E).

Both of these documents outline that the risk of a sinkhole is fairly significant and would need to be planned for if the bored tunnel option is carried forward.

Appendix A – Drawings

COWI-Stantec - *Massey Crossing* - *Bored Tunnel -General Arrangement* – drawing GEN-BT011

COWI-Stantec - *Massey Crossing* - *Bored* - *Typical Cross Sections* – drawing GEN-BT012

COWI-Stantec - *Massey Crossing* - *Bored – Concept Traffic Tie-ins* – drawing GEN-BT013

Appendix B – 2018 vs 2019 Memorandum

McMillen Jacobs Associates. August 15, 2019. DRAFT – 2018 and 2019 Bored Tunnel Concepts Comparison.

Appendix C – Bored Tunnel Depth Memorandum

COWI. September 03, 2019. George Massey Crossing Project – Preliminary Assessment of Bored Tunnel Depth. Revision 0.1 DRAFT

Appendix D – Sink Holes Risks Memorandum

McMillen Jacobs Associates. October 07, 2019. DRAFT – Bored Tunnel – Sink Hole Considerations.

Appendix E – Potential of Sink Holes Memorandum

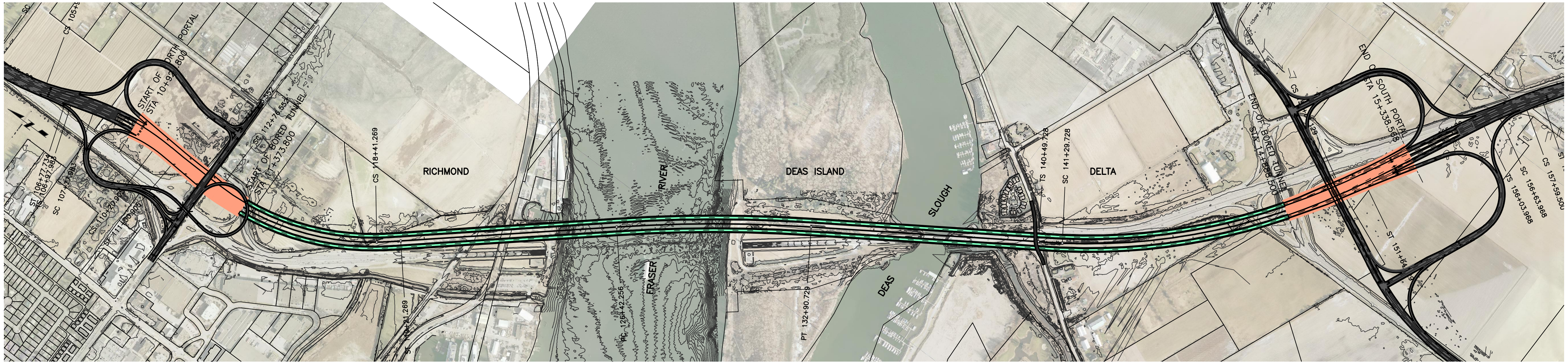
COWI. October 21, 2019. George Massey Crossing Project – Risk associated with TBM tunneling in the Fraser River area. Revision 1.

Appendix A – Drawings

COWI-Stantec - Massey Crossing - Bored Tunnel -General Arrangement – drawing GEN-BT011

COWI-Stantec - Massey Crossing - Bored - Typical Cross Sections – drawing GEN-BT012

COWI-Stantec - Massey Crossing - Bored – Concept Traffic Tie-ins – drawing GEN-BT013



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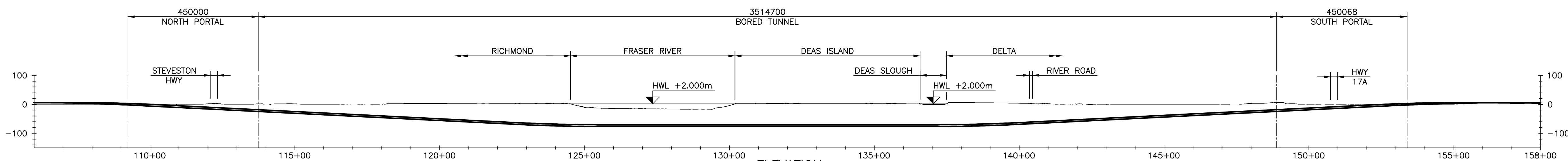

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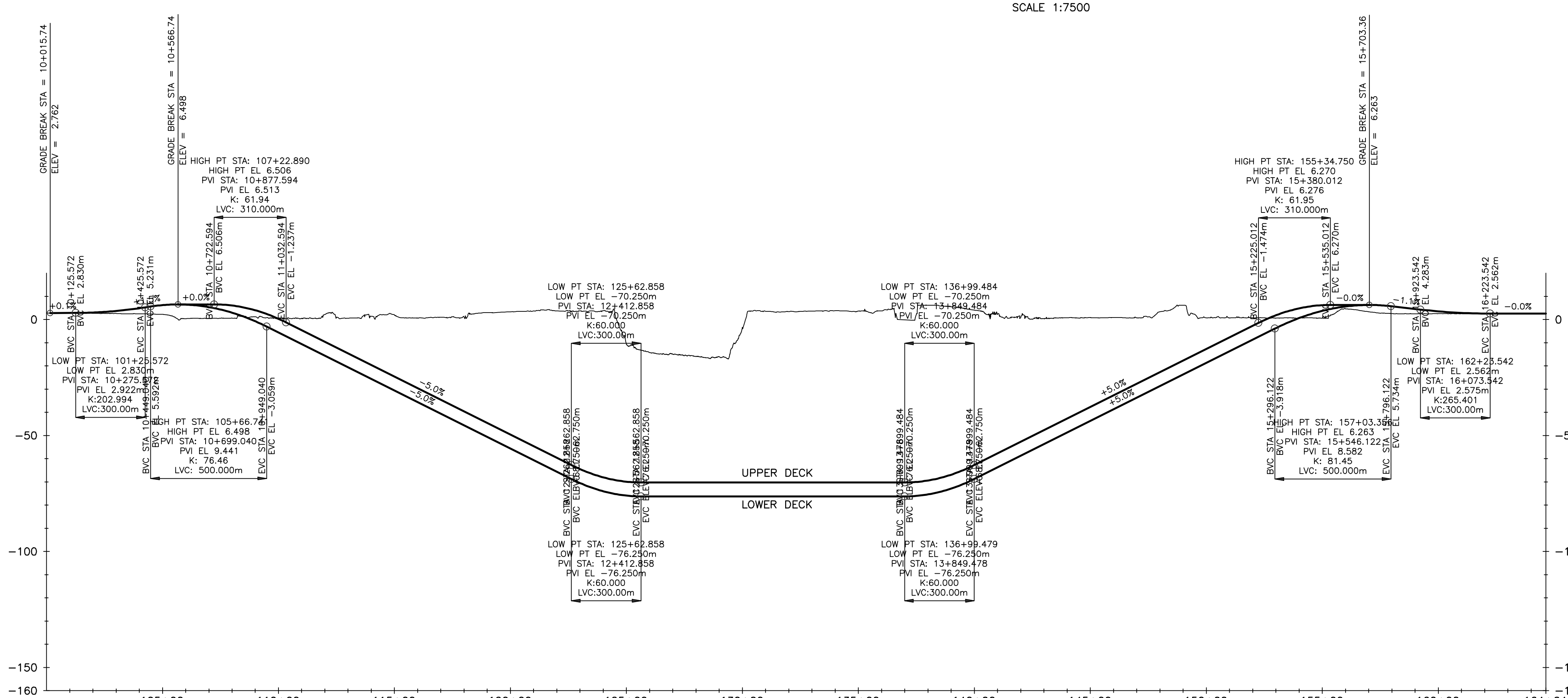

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DRAFT

2019 NOV 15

Consultant Logo

COWI | McMILLEN JACOBS ASSOCIATES | Stantec

Rev	Date	Description	Init
A	2019–11–15		

REVISIONS

BRITISH COLUMBIA

Ministry of Transportation & Infrastructure
South Coast Region

LOWER MAINLAND HIGHWAY DISTRICT
HIGHWAY 99

MASSEY CROSSING – BORED TUNNEL
GENERAL ARRANGEMENT

PREPARED UNDER THE DIRECTION OF

ENGINEER OF RECORD

DATE

DESIGNED MM DATE 2019–11–15
CHECKED DATE
DRAWN CRLS DATE 2019–11–15
SCALE AS SHOWN
NEGATIVE No.

FILE No.	PROJECT No.	REG.	DRAWING No.	
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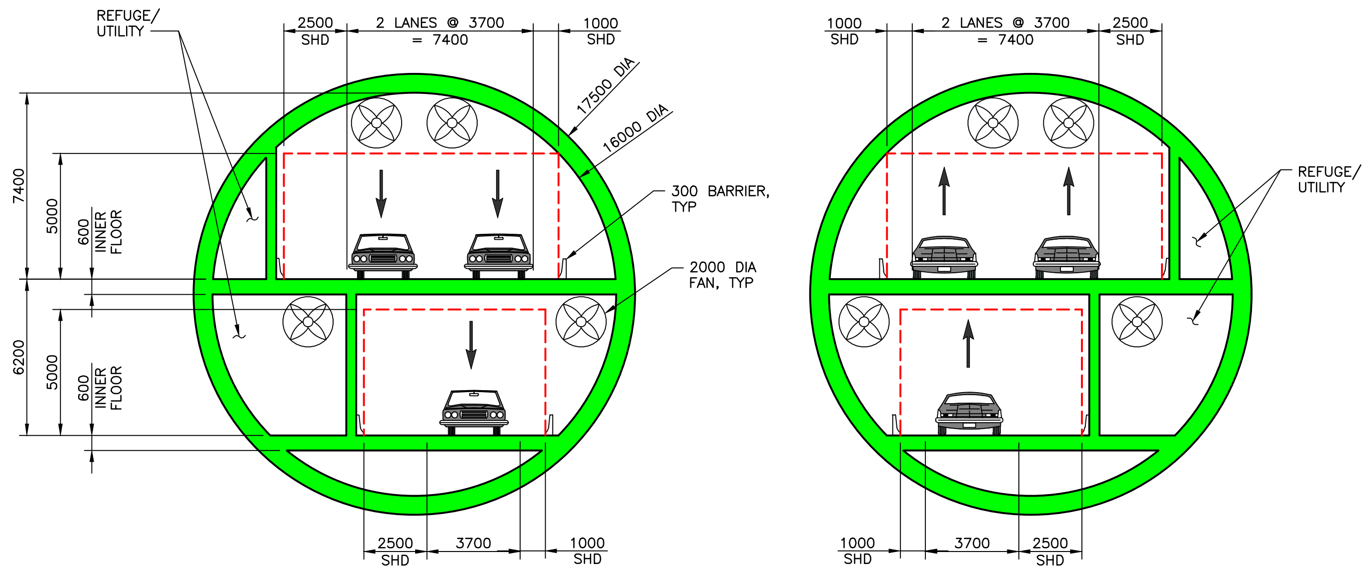
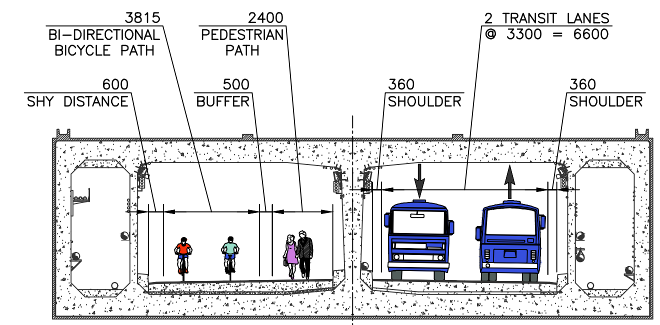
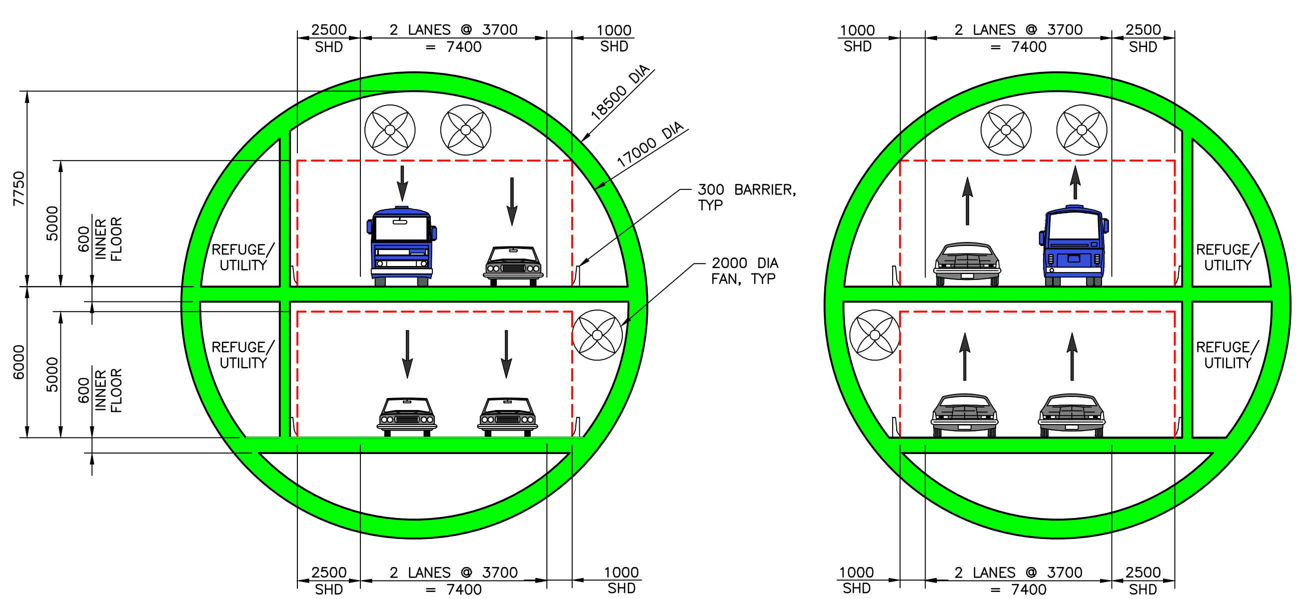
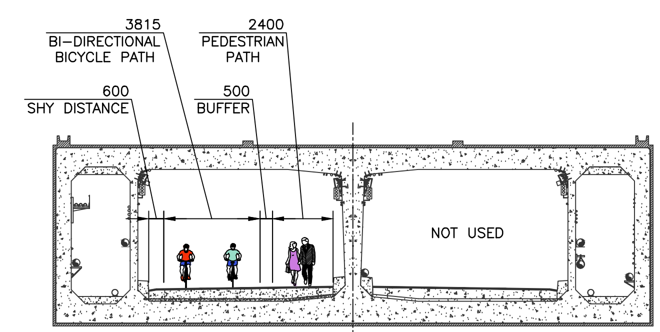




EXISTING TUNNEL CONFIGURATION

6–LANE OPTION
6 GP LANES IN NEW BORED TUNNELS W/ 2 LOCAL BUS LANES AND MUP IN EXISTING TUNNEL
SCALE 1:150





EXISTING TUNNEL CONFIGURATION

8–LANE OPTION
6 GP LANES AND 2 BUS LANES IN NEW BORED TUNNELS W/ MUP IN EXISTING TUNNEL
SCALE 1:150

DRAFT 2019 NOV 15

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Rev	Date	Description	Init
A	2019–11–15		

REVISIONS

BRITISH COLUMBIA
Ministry of Transportation & Infrastructure
South Coast Region

LOWER MAINLAND HIGHWAY DISTRICT
HIGHWAY 99
MASSEY CROSSING – BORED TUNNEL
TYPICAL CROSS SECTIONS

PREPARED UNDER THE DIRECTION OF
ENGINEER OF RECORD
DATE

DESIGNED MM DATE 2019–11–15
CHECKED DATE
DRAWN MACM DATE 2019–11–15
SCALE AS SHOWN
NEGATIVE No.

FILE No.	PROJECT No.	REG.	DRAWING No.	
1070–20	15CS1036	1	BT012	A

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RICHMOND TIE-INS

CONCEPT TO GET TRAFFIC INTO/OUT OF DOUBLE DECK TUNNEL

DELTA TIE-INS

CONCEPT TO GET TRAFFIC INTO/OUT OF DOUBLE DECK TUNNEL

NOTES:

1. INTERSECTION CONFIGURATION SHOWN ON DWG GEN-BT011 AND SHOWN HERE IN GRAY RESULT IN SIGNIFICANT TRAFFIC OPERATION ISSUES AS A RESULT OF THE STACKED SEPARATED CONFIGURATION ROADWAY IN THE BORED TUNNEL. THE RED INTERSECTION CONFIGURATION SHOWS CONCEPTUALLY WHAT IS NEEDED TO ALLOW THE TRAFFIC SUFFICIENT WEAVE DISTANCE TO BALANCE THE TRAFFIC IN EACH LANE.

DRAFT 2019 NOV 15

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Rev	Date	Description	Init
A	2019-11-15		

REVISIONS

BRITISH COLUMBIA — Ministry of Transportation & Infrastructure, South Coast Region

LOWER MAINLAND HIGHWAY DISTRICT

HIGHWAY 99

MASSEY CROSSING – BORED TUNNEL

CONCEPT TRAFFIC TIE-INS

PREPARED UNDER THE DIRECTION OF

ENGINEER OF RECORD

DATE

DESIGNED D. GRAHAM DATE 2019-11-15

CHECKED DATE

DRAWN MACM DATE 2019-11-15

SCALE AS SHOWN

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Appendix B – 2018 vs 2019 Memorandum

McMillen Jacobs Associates. August 15, 2019. DRAFT – 2018 and 2019 Bored Tunnel Concepts Comparison.

Memorandum

To:	Darryl Matson, COWI	Project: George Massey Crossing
From:	Marco Moccichino	cc. Doug Grimes, Dan Adams, Derek Drummond, Don Bergman
Date:	Aug 15, 2019	Doc. No.: 58362_001_MO_BT_A_Tunnel_Concepts_Comparison
Subject:	DRAFT – 2018 and 2019 Bored Tunnel Concepts Comparison	

Revision Log

Rev. No.	Date	Revision Description
A	Aug 15, 2019	Issued for review

1.0 Introduction

In 2018, BGC Engineering retained McMillen Jacobs Associates (MJA) to complete a conceptual design study for a bored tunnel as an alternative crossing technology for the George Massey Tunnel Replacement project. The summary memo of the MJA study was included as appendix in the Independent Technical Review Report (dated September 2018) by Westmar Advisors (led by Stan Cowdell).

In July 2019, the joint venture COWI-Stantec (JV) was awarded a contract by the Ministry of Transportation and Infrastructure (MoTI) as the provider of technical services for the George Massey Crossing project. MJA is a subconsultant of the JV as specialist of the bored tunnel technology.

The scope of the technical services of the JV includes shortlisting 6 out of 18 options (as previously identified by MoTI) of the crossing of the Fraser River and Deas Slough. The shortlisted options include two bored tunnel concepts with either 6 or 8 new lanes of traffic. MJA is advancing the design definition of the bored tunnel concept and the introduction of new assumptions suggests a solution with substantial differences from the 2018 study. The purpose of this memo is to outline the rationale for these differences.

2.0 2018 Concept

The 2018 study was conceptual in nature and considered appropriate for development of a Class 5 cost estimate only. The study was not intended as the basis for selection of the bored tunnel option, rather it was intended to provide a basis for inclusion of the bored tunnel approach in future alternatives analyses. The study was based on the following key design inputs:

(a) Life line structure

(b) Traffic: new 8 lanes and existing tunnel decommissioned or 4 new lanes plus 4 lanes in the existing tunnel (retrofitted);

(c) Traffic: general purpose lane and shoulders widths and vertical clearances were not provided. MJA assumed the widths and clearances of the typical cross section of the Alaska Way Tunnel (SR99) in Seattle, WA as it presents substantial similarities with this project, it was recent and MJA had intimate knowledge of the design;

(d) Site Conditions: liquefiable soil down to elevation -27 m from the Geodetic Datum (source: Independent Technical Review Workshop presentation held on May 3 and 4 2018)

(e) Hydraulic: information on future scour depth and flood protection of tunnel were not considered;

(f) Highway configuration: no maximum grade slope was provided. MJA adopted a geometry that accommodates a gentle transition and connection south of Steveston Hwy and north of Hwy 17A, respectively.

(g) Project Land and ROW: the impact of the construction areas and the new highway layout was not been considered

The study produced a bored tunnel concept with the following main features:

1. Twin tube, with scenarios for two sizes developed
2. Tunnel internal diameter of 16 m (new 8 lanes, stacked configuration) or 11 m (new 4 lanes)
3. Maximum TBM diameter that resides within the upper boundary of the technology, but few tunnel excavations have been already completed.
4. Tunnel crown elevation -27 m
5. Maximum slope for approach ramps 8%
6. Tunnel length approximately 1,700 m
7. Portal length approximately 400 m
8. Class 5 estimate: $1.2B (new 4 lanes) and $1.8B (new 8 lanes)

3.0 2019 Concept

The outcome of the 2018 study has been considered as a basis for the 2019 development. Nevertheless, the technical group (mainly COWI, Stantec and MJA) has identified changes in the 2018 key design inputs as follows:

(a) Traffic: new 8 lanes and multiple users path (MUP) in the existing tunnel or new 6 lanes plus with MUP and 2 transit lanes in the existing tunnel (retrofitted);

(b) Traffic: general purpose lane 3.7 m wide, inside shoulders 1 m wide, outside shoulder 2.5 m and 5 m of vertical clearance;

(c) Site Conditions: liquefiable soil up to elevation -30 m. Presence of liquefiable sand layers and sand pockets in the clay layer below -30 m elevation constituting up to 40% of the volume;

(d) Hydraulic: future scour depth to reduce the cover within the liquefiable layer.

(e) Hydraulic: tunnel to be flood resilient and to grade at elevation 5.5 m;

(f) Highway configuration: maximum preferred slope 4%, maximum allowable slope 5%.

These design input changes have a significant impact and lead to a different bored tunnel concept with the following main features:

1. Tunnel internal diameter of 17 m (new 8 lanes, stacked configuration) or 16 m (new 6 lanes, stacked configuration)
2. Maximum TBM diameter (approximately 19 m) that resides outside the upper boundary of the technology, i.e. world record equipment.
3. Tunnel crown elevation -61 m
4. Maximum slope 4%
5. Tunnel length approximately 3,500 m
6. Portal length approximately 650 m
7. Ground improvement (densification with stone columns up to elevation -30 m) requirements for all the on-shore structures, including the tunnel
8. The size and depth of the support of excavation at the tunnel portals are significant and not comparable with any other excavation performed to date in the Fraser River sands
9. Class 5 estimate: still to be completed, but scaled up costs could top $4B to $5B

4.0 Conclusions

The design input changes introduced in the 2019 study are substantial and their nature is predominantly geotechnical. The different characterization of depth of liquefiable soil and, mainly, the prevalence of liquefiable materials within the clay layer below the Fraser River Sands drive a deeper / longer tunnel with significant ground improvement requirements. In addition, the reduced maximum ramp slope has increased the extent of the already massive excavation at the tunnel portals and has amplified the challenges in the support of excavation. Lastly, the necessity of the tunnel cross section to conform with the MoTI traffic requirements drives a requirement for a TBM size that is beyond the current leading edge of the technology.

Appendix C – Bored Tunnel Depth Memorandum

Cowi. September 03, 2019. George Massey Crossing Project – Preliminary Assessment of Bored Tunnel Depth. Revision 0.1 DRAFT

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MEMO

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TITLE George Massey Crossing Project - Preliminary Assessment of Bored Tunnel Depth

DATE 03 September 2019

TO Darryl Matson

COPY Mike Wong

FROM RYWO/CACA/HISK/SZKO

PROJECT NO A127810-006

Revision Log

Revision	**Date**	**Revision Description**
0.1	03 September 2019	First Draft

Contents

1 Introduction and objective

This preliminary assessment is for an independent review of the twin bored tunnels option for the proposed Massey Tunnel Replacement connecting Richmond and Delta through Deas Island, British Columbia. This memo primarily focuses on the geological conditions for tunnel drive, tunnel alignment design and minimum feasible depth below the Fraser River.

The objective of this high-level review of ground conditions along the proposed tunnel alignment is to provide an overview of the main geological conditions, key elements that will impact the design and construction methods, and the effect of these on construction and ground risk management.

2 Bored Tunnel Option (Large Diameter TBM)

Based on the Draft Technical Memorandum for Bored Tunnel Concept issued for review, dated 15 August 2019 (McMillen Jacobs Associates, 2019), it is understood that one of the options considered for the crossing consists of 2 bored tunnels for up to 4 traffic lanes each in stacked configuration. The maximum tunnel internal diameter will be in the order of 16 m. This assessment assumes a TBM diameter of 18.5 m, and the segmental lining thickness will be about 4-5% of the inner diameter based on the experience from similar size tunnel projects. Tunnel length will be approximately 3.5 km with portals and depressed roads extending to about 650 m at both ends with an assumption of tunnel crown ground cover of 46 m below the river bed.

This assessment reviews the configurations of the preliminary alignment design presented in McMillen Jacobs Associates (2019) in terms of geological conditions, technical feasibility, earthquake design and TBM types.

3 Subsurface Conditions

3.1 Geology

A review of geological publications and ground investigation data shows that the site is overlain by both Fraser River Sand Deposits and Post Glacial Marine Deposit at the proposed tunnel level. The Fraser River Sand Deposits consist mostly sand with intercalated SILT and SILTY SAND layers. The Fraser River Sand Deposits attain a maximum depth of around 32 m. The Post Glacial Marine Deposits consist of intercalated SAND, SILTY SAND, SILT and CLAYEY SILT layers. The Post Glacial Marine Deposits extend to a depth of greater than 300 m below ground.

3.2 Soil parameters

Soil parameters are mainly derived from empirical correlation using CPT data. As there is only limited laboratory testing data, no correlation with laboratory testing data have been developed for this site. The derivations are primarily based on CPT soil behavior type I_C and normalized CPT data.

The laboratory test results, including particle size distribution (PSD) and natural water content, indicate that the Fraser River Sand Deposit consist of mainly sand-size particles with maximum fines content of up to about 20%. The Post Glacial Marine Deposits show a mixture of SAND, SILT and CLAY particles. The fines content of the Post Glacial Marine Deposits ranges between 20% to 90%.

In general, the natural water content of the Fraser River Sand Deposits and Post Glacial Marine Deposits predominately ranges from 20% to 40%. As Fraser River Sand Deposits have a low fines content and a high natural water content relative to the Liquid Limit, these deposits are susceptible to liquefaction under seismic loading. The soil index testing results from BH13-01 and BH13-02 samples also show that the Liquid Limit and Plasticity Index of Post Glacial Marine Deposits are below 40% and 12% respectively. The low level of plasticity of these SILT and CLAYEY SILT indicates that these deposits may comprise a low content of CLAY minerals that otherwise reduces the liquefaction susceptibility. Indeed, the presence of clay-size fine quartz particles in these soils will trigger a cohesionless response during seismic loading, i.e. the Post Glacial Marine Deposits are susceptible to liquefaction as well. Section 4 of this memo presents the seismic design considerations.

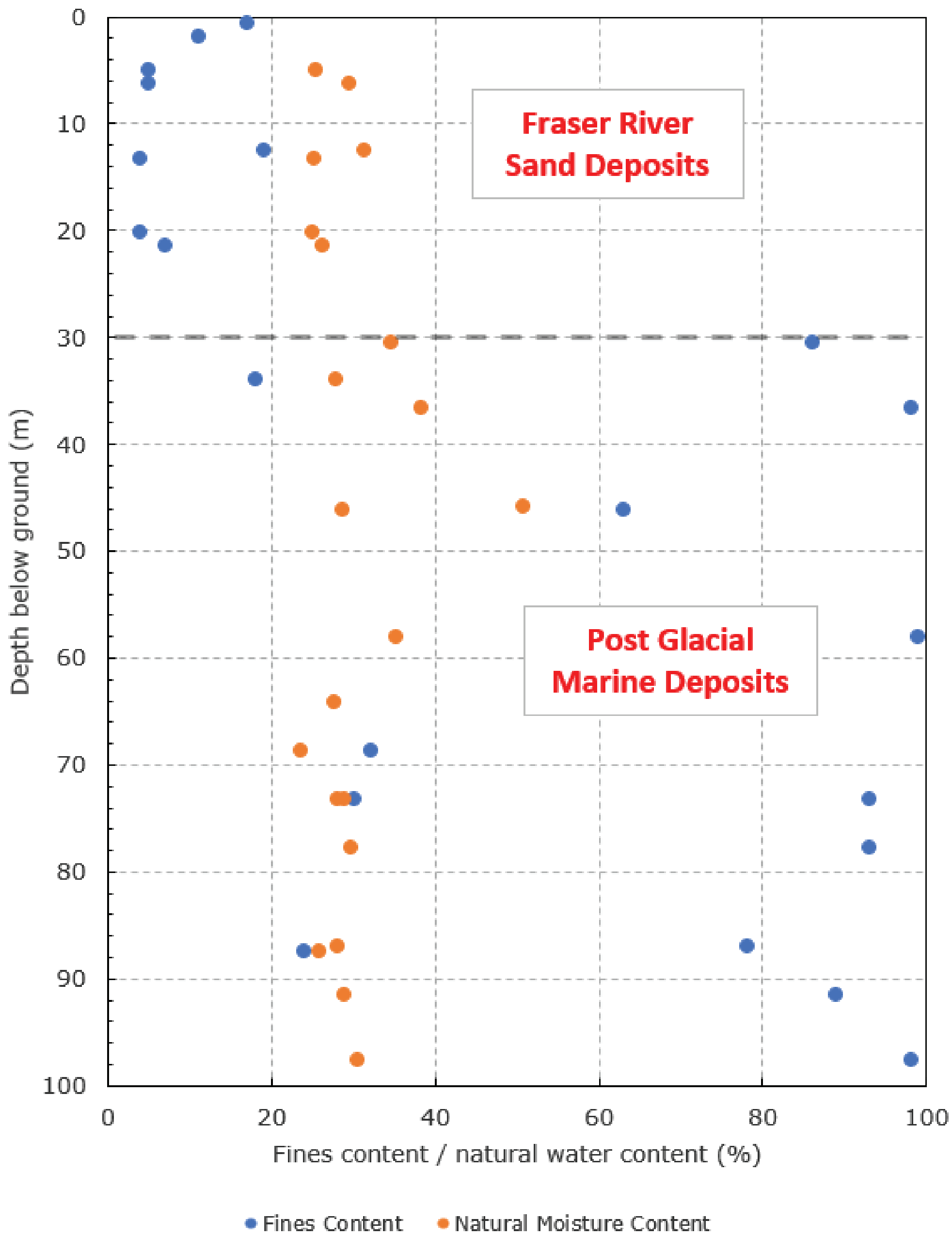

Figure 3.1. Fines content and natural water content.

Relative Density D_r of sand strata has been derived from CPT correlation. For Fraser River Sand Deposits, D_r mostly ranges between 35% to 55%, which indicates a medium dense sand. For the Post Glacial Marine Deposits, D_r range is between 20% to 40%, which indicate a loose to medium dense sand.

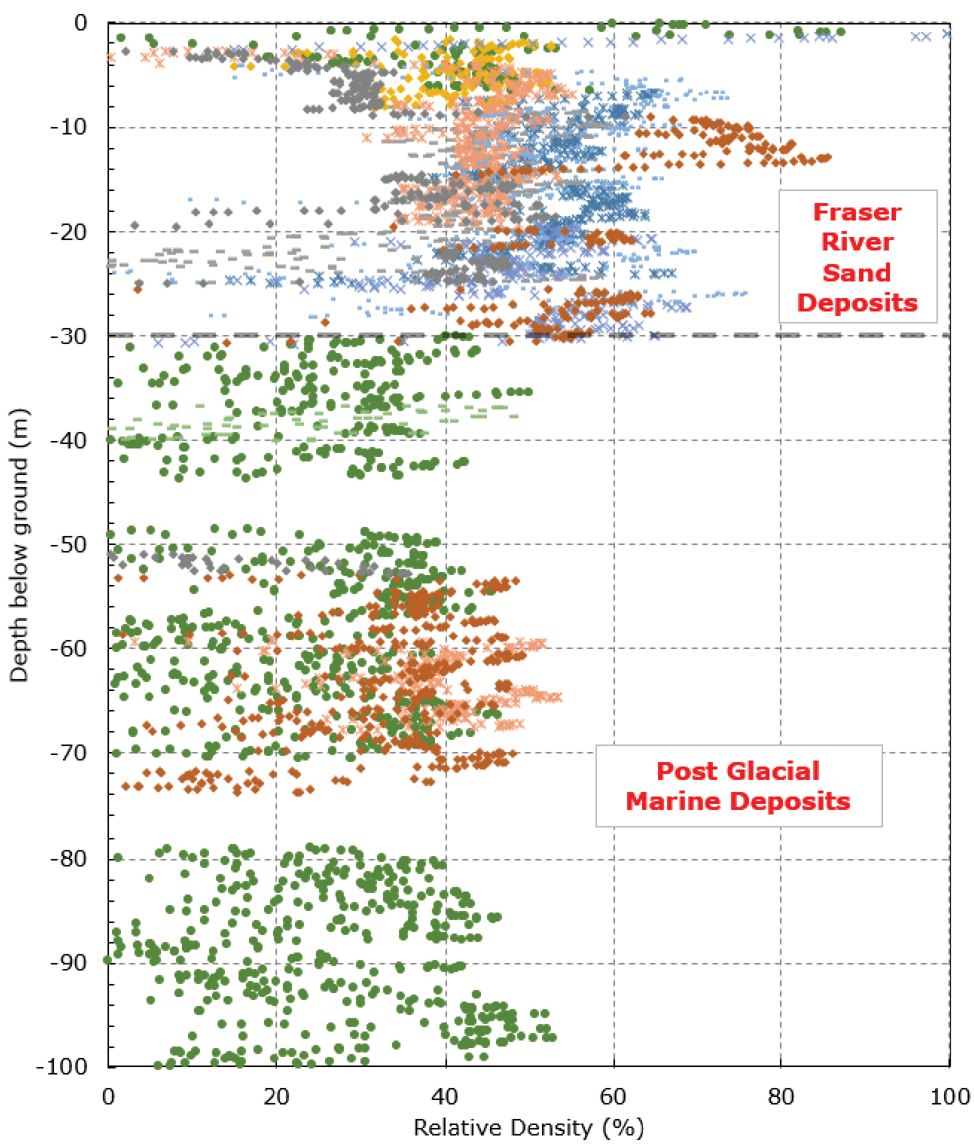

Figure 3.2. Relative density correlated based on CTP data.

For cohesionless strata, the peak effective friction angle has been derived from CPT data. For Fraser River Sand Deposits, the average friction angle is in the order of 35° to 37°. While for the Post Glacial Marine Deposits the friction angle ranges between 32° and 36°.

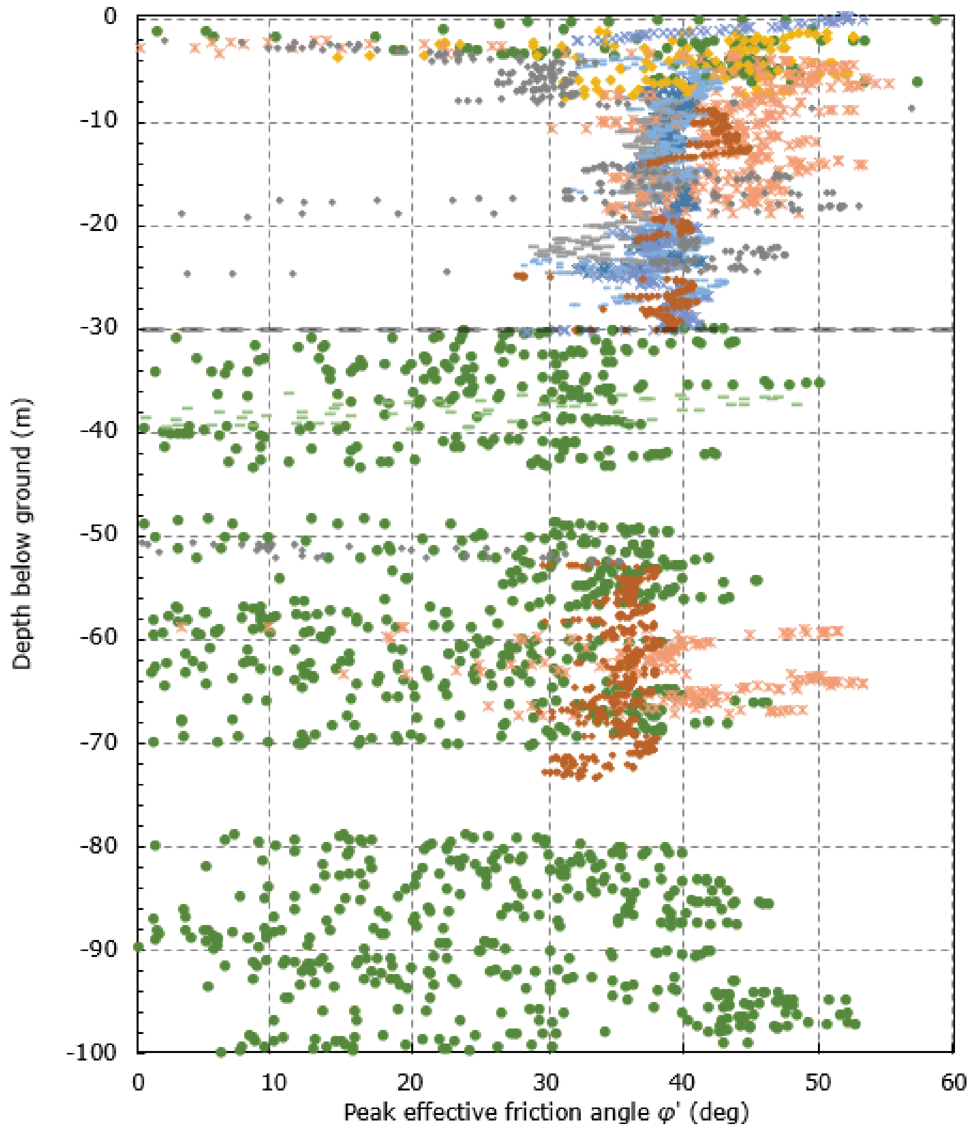

Figure 3.3. Peak effective friction angle correlated based on CPT data.

The undrained shear strength for cohesive strata has been derived from the CPT results. The result indicates that the undrained shear strength consistently increases with depth.

The correlation is approximately equal to $s_u = 2z$, where z is meters below ground.

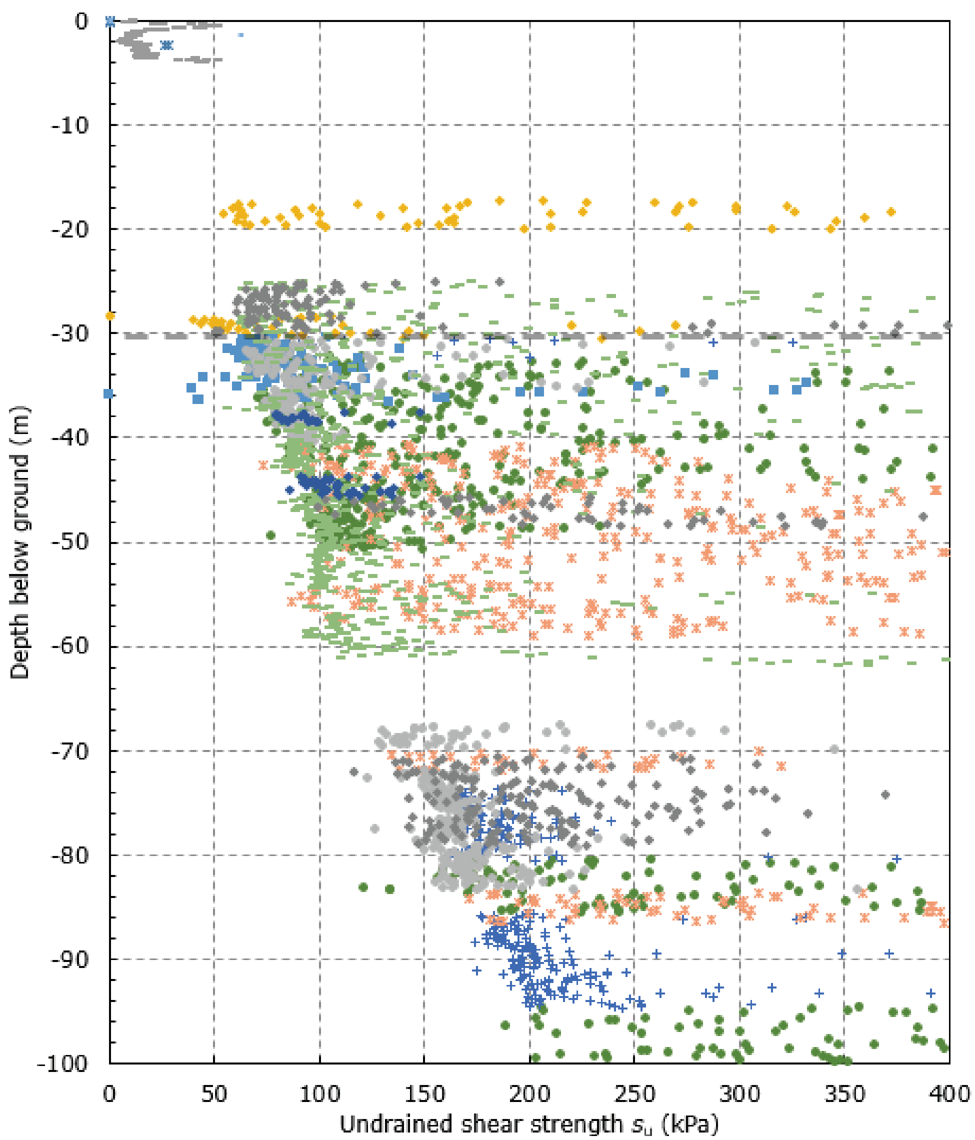

Figure 3.4. Undrained shear strength correlated based on CPT data.

4 Seismic Design Consideration

In general sense, seismic design for tunnel can be grouped into two categories, namely ground failure and ground shaking. Ground failure refers to various types of ground instability such as liquefaction, subsidence, faulting and landslide etc. Ground shaking refers to the vibrations of the ground produced by seismic waves propagation and the induced ground deformation imposed on tunnels and underground structures. This preliminary assessment considers the seismically induced ground failure only, as this defines the feasibility of the bored tunnel specific depth, alignment design and site constraints. While ground shaking can be dealt with by designing a stronger but more flexible structure, thus it is not discussed here.

The primary focus of this preliminary seismic assessment is the feasibility of the twin-bored tunnel option driving underneath Fraser River subject to cyclic liquefaction. It is expected that the bored tunnel section underneath Fraser River is the critical controlling factor of this option. The results available from three CPT CP06, CP09 and CP29 (Golder Associates Ltd., 2014) by Fraser River have been considered in this study to evaluate the impact of seismic activity on the proposed tunnel alignment.

4.1 Liquefaction of soils

The liquefaction assessment for the area underneath Fraser River has considered the first 60 m soil layers below the ground surface as summarized in Table 4.1. It is important to note that a 60-m deep soft ground bored tunnel with high groundwater table is considered to be at the current upper boundaries of the TBM technology and Occupational Health & Safety limit for compressed air working in case of cutter head intervention for a large diameter TBM in soft ground.

Table 4.1. Simplified strata for preliminary liquefaction assessment.

Stratum	Symbol	From (m)	To (m)	γ (kN/m³)
Silt / Clayey Silt	M/CM	0	5	17.5
Sand / Silty Sand	S/SM	5	30	19
Silt/ Silty Clay	MC/CM	30	>100	18

As the river bed elevation is about 10 – 15 m below the existing ground surface, the top SILT / CLAYEY SILT layer has not been considered in this liquefaction assessment. Section 4.1.1 and 4.1.2 present the liquefaction assessment for the SAND to SILTY SAND Fraser River Sand Deposits with typically less than 10% fines content and the SILT to CLAYEY SILT with over 35% of fines.

4.1.1 Liquefaction of Fraser River Sand Deposits

The assessment for the liquefaction potential of Fraser River Sand Deposits below Fraser River has been based on the CRR estimated using the corrected CPT tip resistance $q_{c1N,cs}$ (Robertson & Wride, 1997). The respective Peak Ground Acceleration (PGA) magnitudes of 0.1g, 0.15g, 0.2g and 0.25g, considered in the preliminary liquefaction assessment, cover the full range of average crustal, inslab and interface PGA of an earthquake event with a 2475-year return period according to the performance criteria of the Canadian Highway Bridge Design Code CAN/CSA-S6-14 (Figure 4.1).

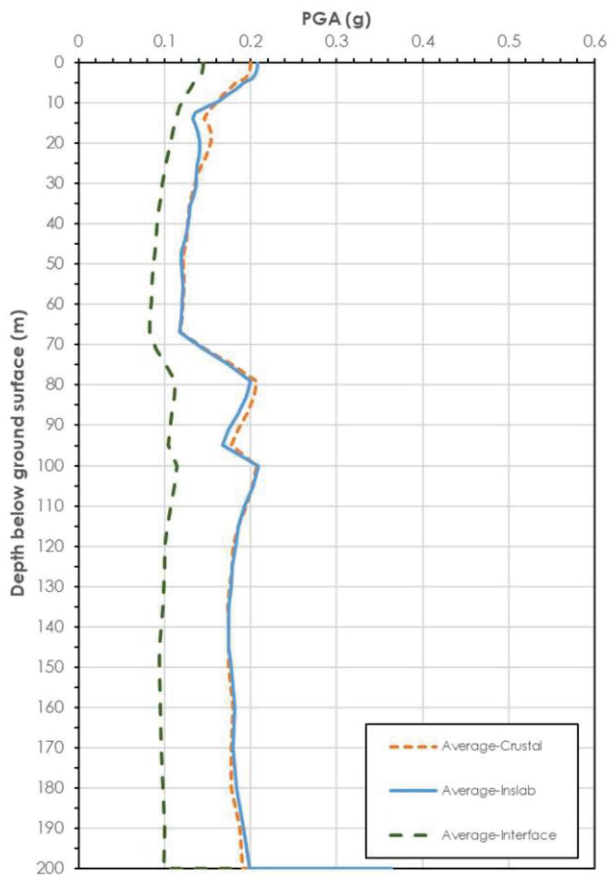

Figure 4.1. Peak ground acceleration of 2475 year return period.

As suggested in Seed et al. (2003) and Robertson & Wride (1997), this preliminary liquefaction assessment compares the CPT-based Cyclic Resistance Ratio (CRR) to Cyclic Stress Ratio (CSR) at various levels of PGA. The CSR results show that the Fraser River Sand Deposits lying within the first 30 m depth from the ground surface are susceptible to liquefaction. Figure 4.2 to Figure 4.5 present the liquefaction susceptibility of the Fraser River Sand Deposits at various depths and levels of PGA. It is expected that SAND and SILTY SAND within the Fraser River Sand Deposits will require extensive densification and ground improvement to mitigate the risk of liquefaction.

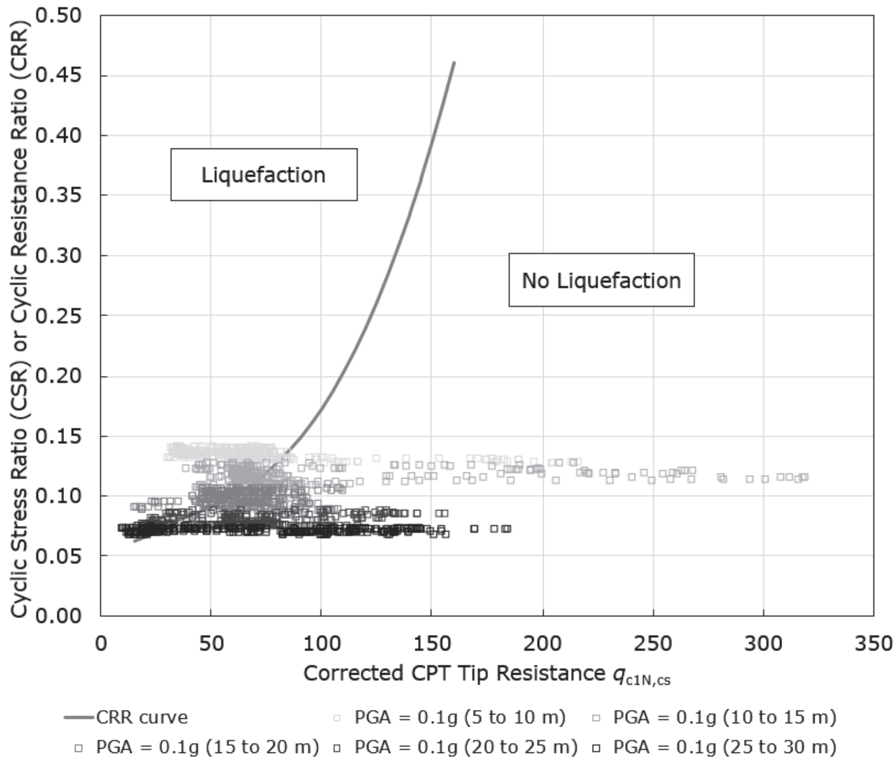

Figure 4.2. CRR curve and CPT determined CRS data at PGA = 0.10g.

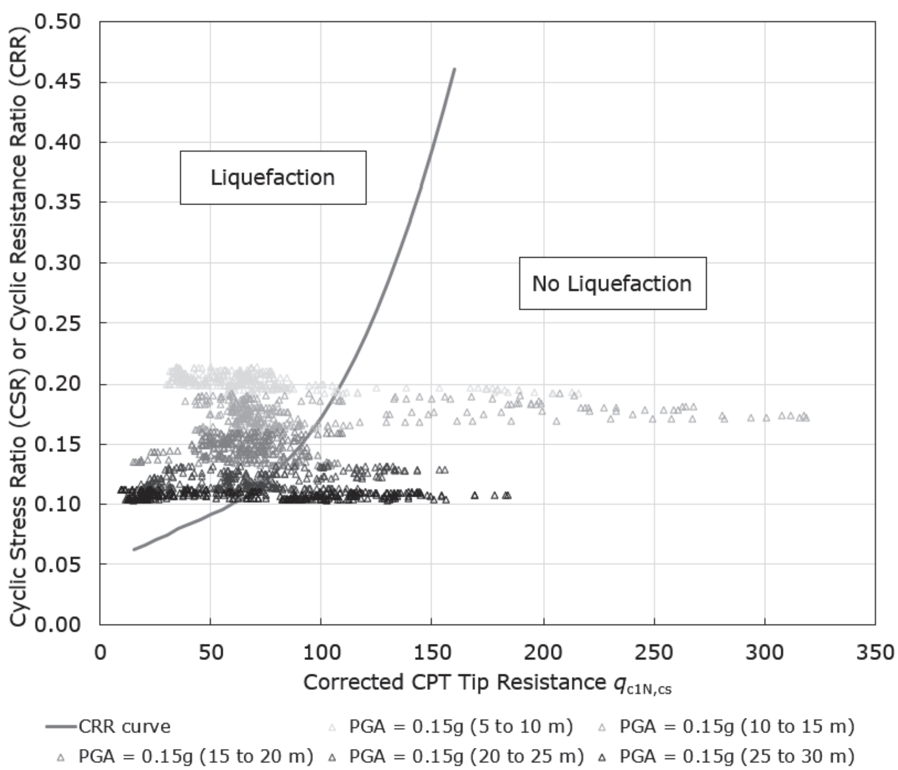

Figure 4.3. CRR curve and CPT determined CRS data at PGA = 0.15g.

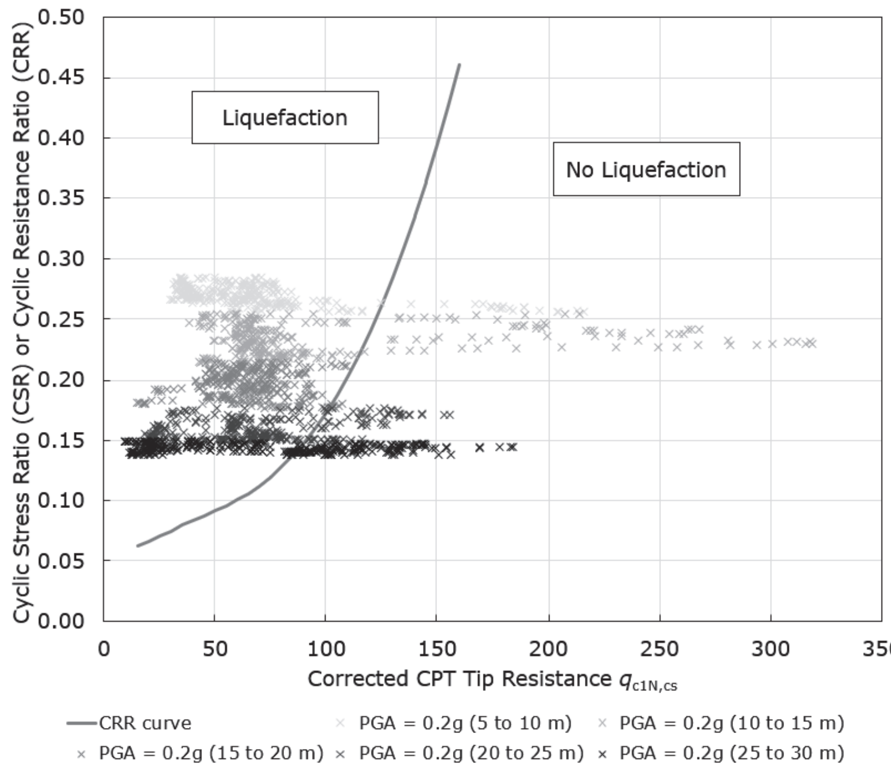

Figure 4.4. CRR curve and CPT determined CRS data at PGA = 0.20g.

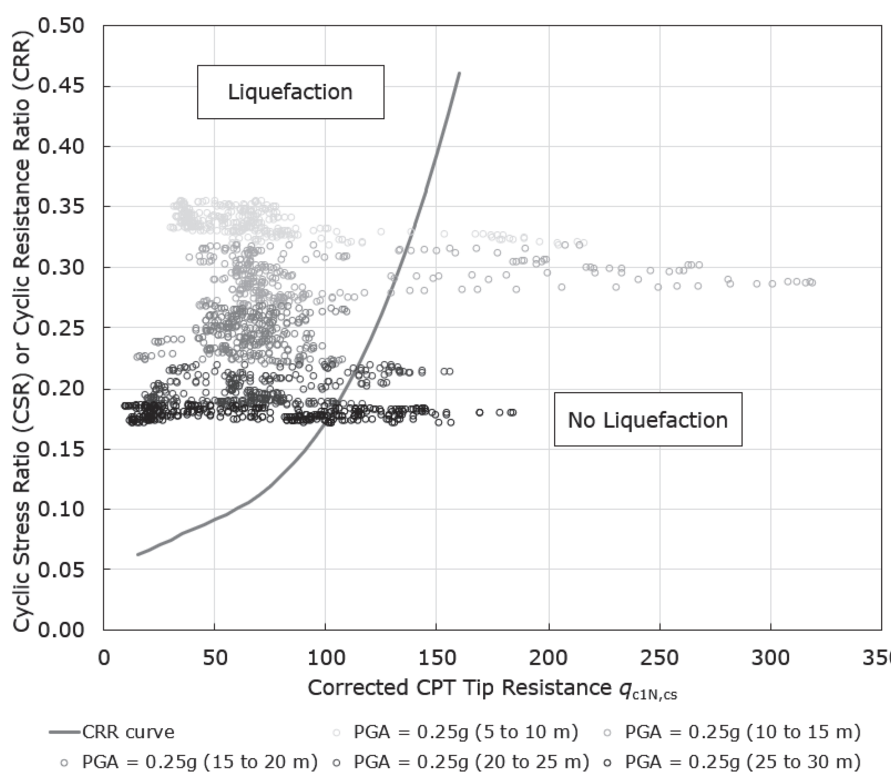

Figure 4.5. CRR curve and CPT determined CRS data at PGA = 0.25g.

Table 4.2. Percentage ground subject to liquefaction at different depth from ground as per CPT results.

Depth from ground surface	PGA = 0.10g, CRR < CSR	PGA = 0.15g, CRR < CSR	PGA = 0.20g, CRR < CSR	PGA = 0.25g, CRR < CSR
5 to 10 m	86.0%	91.7%	93.7%	94.0%
10 to 15 m	50.9%	73.4%	77.2%	79.2%
15 to 20 m	35.3%	87.3%	97.3%	100.0%
20 to 25 m	17.2%	60.9%	78.8%	87.2%
25 to 30 m	25.8%	45.5%	55.9%	68.2%

Table 4.2 summarizes the estimated percentages of ground that is susceptible to liquefaction under PGA of 0.10g, 0.15g, 0.20g and 0.25g, i.e., CRR < CSR. It shows that Fraser River Deposits have a high risk of liquefaction, where a PGA magnitude in the order of 0.10g could trigger a liquefaction event in case of an earthquake near this site.

4.1.2 Liquefaction of Post-glacial Marine Deposits (considered to depths of 60 m below ground surface)

As Post-glacial Marine Deposits have a typical fines content of greater than 35% and plasticity index of lower than 12%. As discussed in Section 4.1.1 for SAND to SILTY SAND, such soils are in many cases not well suited to evaluation method using a conventional in-situ penetration-based liquefaction hazard assessment methods (Seed, et al., 2003). In light of this, the SILT to CLAYEY SILT layers intercalated with SILTY SAND from 30 to 60 m below the ground surface have been studied based on the Atterberg limits as suggested by Seed et al. (2003). All the sample data points from 30 m to 60 m depth of test hole BH13-01 in the geotechnical data report (Golder Associates Ltd., 2014) are summarized in Table 4.3 and Figure 4.6.

Table 4.3. Atterberg limits and water content of test samples from 30 - 60 m of depth.

Sample depth (m)	Water content (%)	Liquid limit (%)	Plasticity index (%)
37.19	38.3	31	4
46.33	50.6	35	11
58.52	35.1	37	12
77.72	29.6	31	7

Figure 4.6 shows that the soils within the Post-glacial Marine Deposits on this site are potentially liquefiable. All the samples in Table 4.3 have water contents higher than 80% of the corresponding liquid limits. Despite some of the given water contents being higher than the liquid limits, the samples generally show a high degree of soil saturation, which indicates that these soils also have a high risk of liquefaction. Further investigation and sample testing should be carried out to evaluate the level of seismic load necessary to trigger liquefaction in these soils (Seed, et al., 2003).

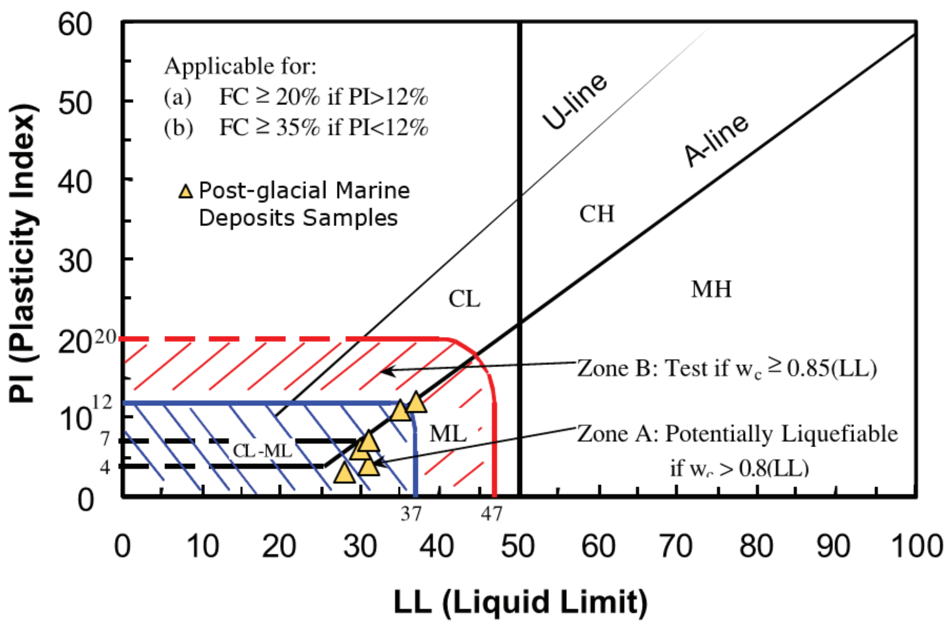

Figure 4.6. Liquefiable soil types recommendations (after Seed, et al., 2003)

4.2 Consequence of liquefaction

Liquefaction reduces uplift resistance above the tunnel and could cause significant differential movement along the tunnel alignment. With no ground improvement, the Fraser River Sand Deposits and upper Post Glacial Marine Deposits are liquefiable, and the minimum depth of the tunnel should be considered below these layers.

The required depth of the tunnel beneath the Fraser River Sand Deposits depends on the assessment of liquefaction of these lower deposits. Limited data is available to challenge the current estimate of over 40% of the silt and sand layers being liquefiable. Hence, the tunnel depth is unlikely to be lessened without further investigation or ground improvement measures.

5 Ground Improvement

Ground improvement and stabilization measures might be adopted to prevent or reduce the risk of liquefaction of the Fraser River area for the bored tunnel construction. This section discusses the parameters that control liquefaction and possible ground improvement methods. This section does not consider the cost or environmental constraints of the Fraser River Site, and further studies regarding these will be required as necessary.

5.1 Controlling parameters of liquefaction susceptibility

The susceptibility of cyclic liquefaction of soils can be reduced by preventing a sudden increase in excess pore pressure exude during a seismic event without being dissipated rapidly, thus increasing CRR and reducing CSR. The controlling parameters include but not limited to the following:

- **Relative density:** When ground vibration compacts soils, it generates excess pore water pressure. Soils with more voids exhibit a higher degree of compaction and generate higher excess pore water pressures under vibration. Increasing relative density lowers the total stress-to-effective stress ratio component of CSR, which reduces CSR.

- **Plastic fines content:** Soils with higher fines contents do not generally exude excess pore water pressures rapidly. A higher clay minerals content (not "clay-size" particles) increases the plasticity rage and improves the ability of soil to hold water. Improving plasticity reduces the risk of rapid pore water pressures generation.

- **Drainage channel:** Providing drainage channels for rapid dissipation of cyclically generated excess pore water pressures.

- **Shear strength:** Increasing shear strength to improve Cyclic Resistance Ratio.

5.2 Ground improvement methods

Ground improvement methods can be used to mitigate liquefaction hazard by improving the controlling parameters given in Section 5.1. Table 5.1 shows a brief list of ground improvement methods available today. The critical considerations for selecting and implementing ground improvement methods are applicability, effectiveness, the ability to verify the ground improvement performance, cost and environmental impacts and regulations, etc.

Table 5.1. Ground improvement methods for soil liquefaction hazard mitigation.

Category	Ground improvement method	Improved parameter
I. In-situ ground densification	1 Compaction with vibratory probes 2 Installation of stone/gravel columns 3 Dynamic consolidation (heavy tamping) 4 Compaction piles/drill shafts 5 Compaction grouting	› Relative density › Shear strength
II. Ground treatment	6 Permeation grouting 7 Chemical grouting 8 Deep cement mixing 9 Jet grouting	› Relative density › Shear strength
III. Drainage provision	10 Installation of stone/gravel/sand columns 11 Installation of pre-fabricated strip drains	› Drainage channels › Relative density
IV. Other types	12 Surcharge pre-loading 13 Structural fill 14 Lime soil treatment 15 Bentonite mixing	› Relative density › Plastic fines content › Shear strength

6 Discussion on Proposed Bored Tunnel Concept

6.1 Tunnel alignment

This section discusses the specific depth and flotation criteria of the proposed tunnel alignment design under the best and worst-case scenarios based on the geological and seismic assessments discussed in Sections 3 to 5 of this memo.

6.1.1 Flotation criteria

The flotation assessment considers a tunnel internal diameter of 16 m according to a previous space proofing study (McMillen Jacobs Associates, 2019). This assessment assumes a lining thickness of 4.5% based on experience of similar size tunnel, i.e., 720 mm thick lining. Only self-weight of the lining structure (without internal structure or stage 1 concrete) and soil mass above the proposed tunnel have been considered in the flotation with an equilibrium stability safety factor of 1.10 used.

- Tunnel internal diameter: 16,000 mm
- Tunnel lining thickness: 720 mm
- Concrete unit weight: 24 kN/m³
- Average soil unit weight: 18 kN/m³
- Water unit weight: 10 kN/m³

The required minimum depth from the lowest point of non-liquefiable ground surface to the tunnel crown is approximately 13 m, which is in the order of one tunnel internal diameter of embedded depth.

6.1.2 Best case alignment design (with ground improvement)

The specific tunnel depth of the best-case scenario assumes that there will be ground improvement measures within the Post Glacial Marine Deposits layer. Therefore, the soil within Post Glacial Marine Deposits will not be susceptible to liquefaction. With the bottommost point of Fraser River bed located at approximately 16 m below ground surface, as indicated in the geological profile, and the bottom of the liquefiable Fraser River Sand Deposits being at 30 m below ground surface, the ‘best case’ upper bound tunnel alignment will have a minimum river bed-to-tunnel crown distance of 30 m, i.e. 46 m below ground surface.

If ground improvement measures will be carried out extensively and to a depth of 50 m below the ground surface, the tunnel crown could be as shallow as one tunnel diameter below the lowest point of the river bed. However, many studies have shown that the cost-to-benefit ratio of improving liquefiable ground is typically greater than 1.0, i.e., the cost for ground improvement will be higher than the cost saving of a particular construction method.

6.1.3 Worst case alignment design (without ground improvement)

It is expected that the tunnel alignment for the 'worst-case' scenario is the same as the assessment presented by McMillen Jacobs Associates (McMillen Jacobs Associates, 2019) where the tunnel crown will be located approximately 51 m below the river bed.

6.2 Opportunity of optimizing Space proofing

A review of large diameter tunnel projects in various countries has been carried out and summarized in Section 10. This indicates there is an opportunity to further optimize the preliminary space proofing design. The review shows that some tunnels with stacked four-lane configuration in the United States and China have tunnel internal diameter ranging from 13 m to 15.5 m. Further studies should be carried out to investigate the possibilities of reducing the proposed 16 m internal diameter, especially for tunneling in difficult ground below Fraser River.

6.3 Opportunity of Increasing risk tolerance

The current design considers an earthquake event with a return period of 2475 years. This return period is equivalent to a 4.7% and 4.0% probability of at least one event occuring within a 120-year and 100-year design life respectively (see Table 6.1). There is an opportunity of optimizing the tunnel design by relaxing the risk tolerance level. Hence, a lower PGA magnitude could be considered in the design. Adopting a lower PGA magnitude can significantly reduce the need for ground treatment. For example, AASHTO and United States Federal Highway Administration considers the upper-level design with a ground motion level at an occurrence of 7% exceedance in 75 years.

Table 6.1. Earthquake occurrence probability of various design lives (Poisson distribution).

Return period	2475	2475	975	975	475	475	173	145
Design life (year)	120	100	120	100	120	100	120	100
Exceedance probability $P(X \geq 1)$	4.7%	4.0%	11.6%	9.7%	22.3%	19.0%	50.0%	50.0%

7 Risk Associated with TBM Tunnelling

There are risks of sinkholes formation and blowouts during driving a large diameter TBM tunnel below Fraser River as well as in the greater study area. It is challenging to control face stability of a large TBM of over 15 m in diameter driven in intercalated soil layers and pockets of SILT, CLAYEY SILT, SILTY SAND and SAND. This is particularily difficult when TBM face encounters multiple layers and pockets of intercalated soil in the order of 5 m to 10 m thick. The soil particle size distribution curves presented in Figure 7.1 to Figure 7.3 show that the medium to fine SAND and SILTY SAND are uniformly graded. The coefficient of uniformity of these soils is approximately 2.5. With the presence of CLAYEY SILT, neither a Slurry TBM or an Earth Pressure Balance Machine (EPBM) alone would operate at its optimal application soil ranges, as shown in Figure 7.4 and Figure 7.5. These difficult mixed ground conditions encountered at the face could lead to loss of face stability during tunnelling. It is recommended that a consideration should be given to Variable Density TBM to be used in driving bored tunnel in such ground conditions to enhance face control and minimize risk of face instability.

Also, given the relatively low relative density of the soil layers, using a large diameter TBM will likely induce a greater volume loss of 1% to 2%. It is expected that for such large TBM, tunnelling will induce fairly significant ground settlements that could impact the existing George Massey immersed tunnel and other structures sensitivity to tunel indiuded ground moveemnts along the proposed alignment. Further study and risk assessment will be required to better understand the potential tunnel construction impacts.

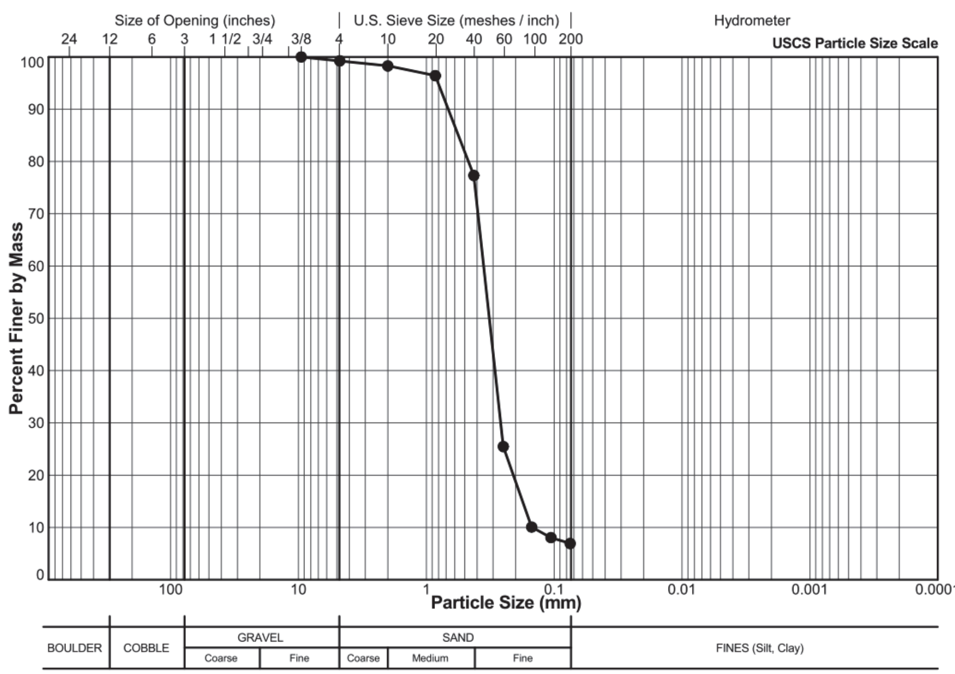

Figure 7.1. Particle size distribution of Fraser River Sand Deposits at a depth of 21.34 m to 21.95 m

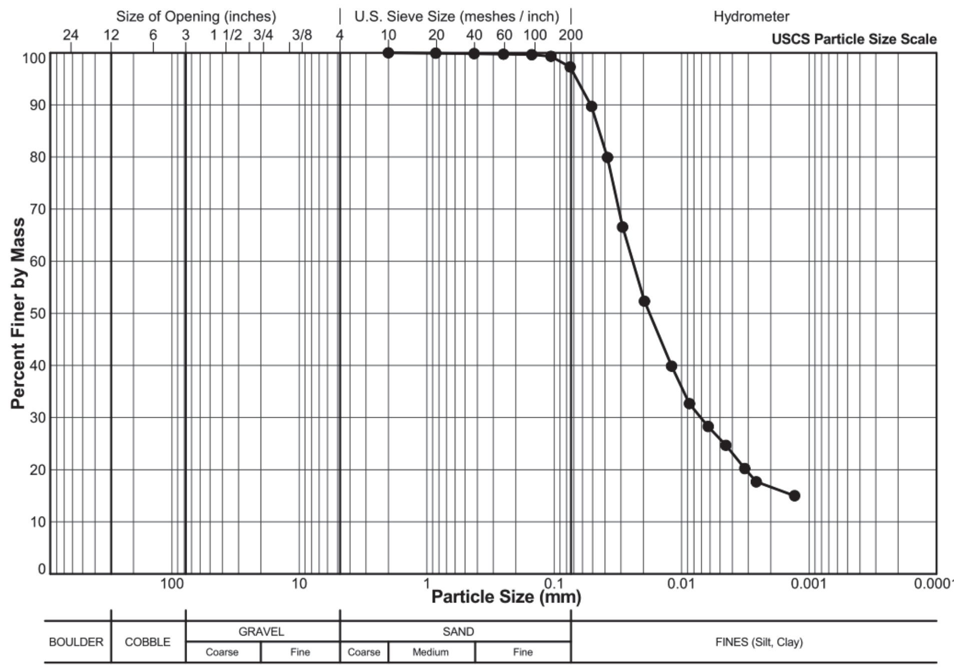

Figure 7.2. Particle size distribution of Post Glacial Marine Deposits at a depth of 36.58 m to 37.19 m.

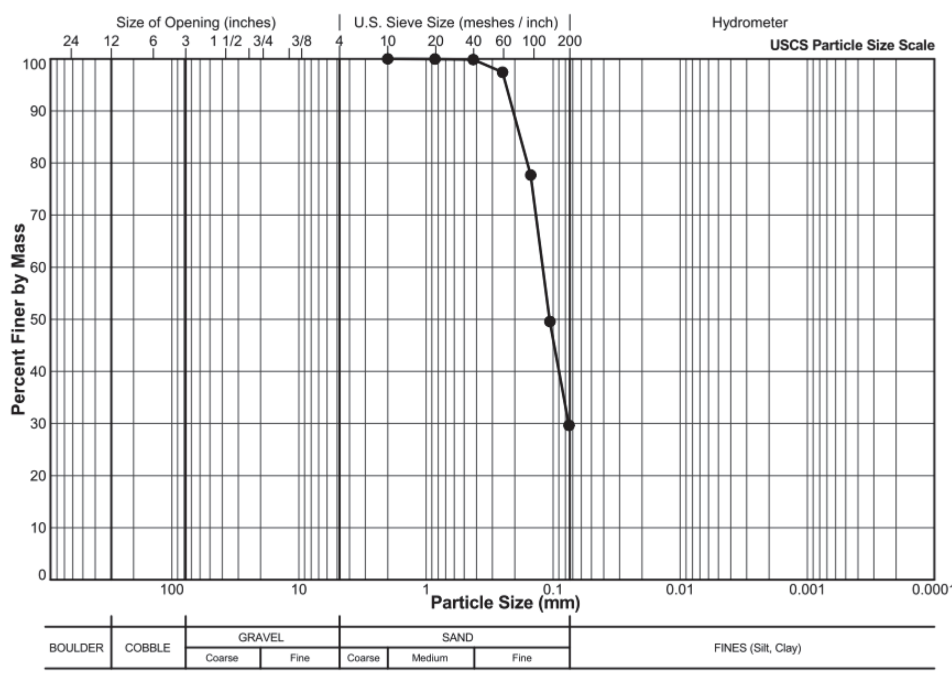

Figure 7.3. Particle size distribution of Post Glacial Marine Deposits at a depth of 73.15 m to 73.76 m

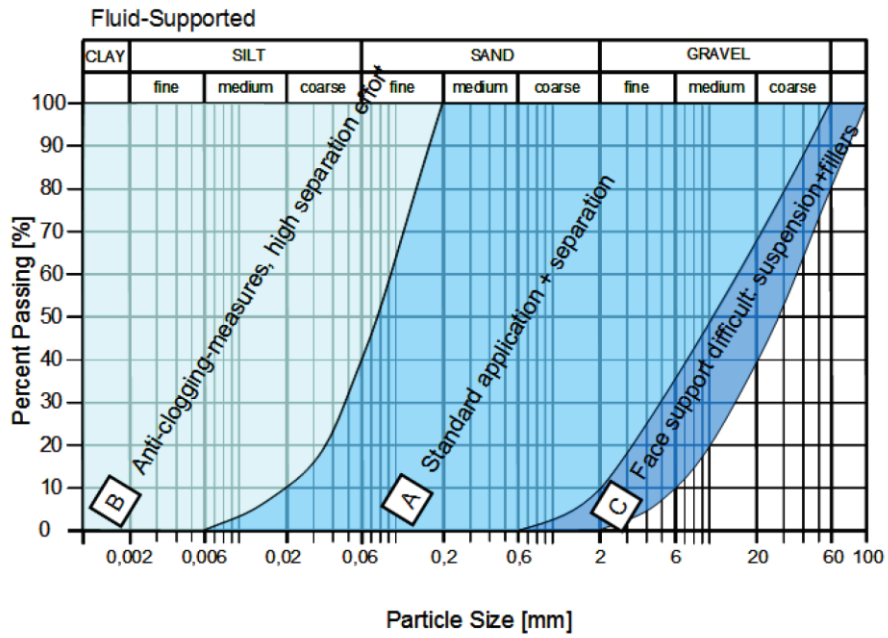

Figure 7.4. Application ranges of slurry TBMs (German Tunnelling Committee, 2016).

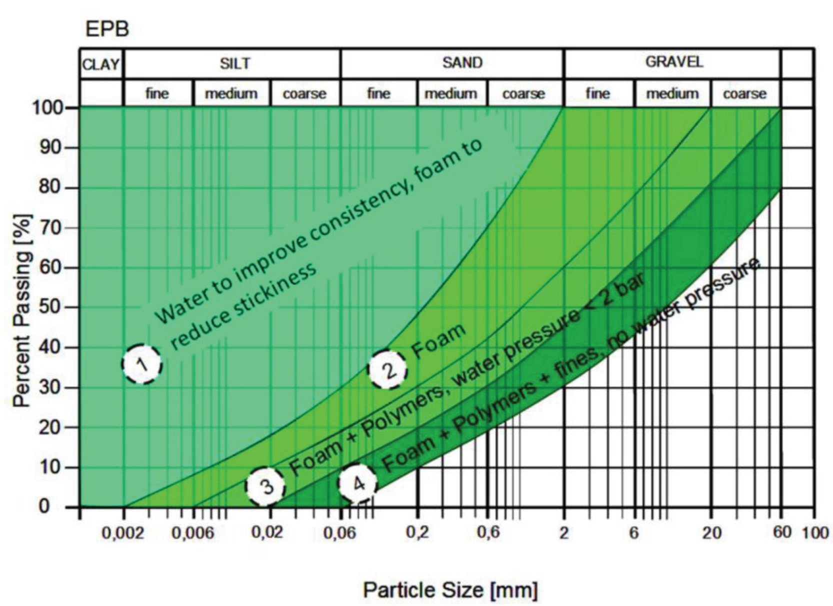

Figure 7.5. Application ranges of EPB TBMs (German Tunnelling Committee, 2016).

8 Summary

The feasibility assessment of the specific depth of the bored tunnel option presented in this memo shows that the first 60 m of soil layers below ground surface are susceptible to liquefaction under seismic events.

This study indicates that driving bored tunnel in Fraser River area within these ground conditions will either require extensive ground treatment or driving tunnel deeper below the river bed. There are possible value-engineering opportunities for this bored tunnel option and further studies will be required to explore these opportunities.

9 References

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10 Annex: Table of Tunnel Project References

Large Diameter TBM Tur

max 15.60 17.56
min 12.40 13.95

Key technical criteria (Boring period)

No.	Start / Launch Year	Tunnel type	Project	Country	Arrangement	Tunnel ID (m)	TBM dia. (m)	TBM type.	Geology	Locality	Grd motion	Liquafaction	Environment	Construction Window (years)	Navigation	Proximity to Structures	Tunnel Width	Link
1	2019	Road	Melbourne West Gate Tunnel*	Australia	3 lanes	14.10	15.55	EPB	Clay, silty clay, sandy clay	River + Urban								https://www.tunneltalk.com/Australia-06Mar2019-Mega-TBM-arrives-in-Melbourne.php
2	2018	Road	Nanjing MeiZiZhou Tunnel	China		13.72	15.43	Slurry										
3	2018	Road	Shanghai Zhou Jia Zui Road River Crossing Motorway	China		13.24	14.90	Slurry							x	x		
4	2017	Road	Shantou Su'Ai Sub-sea Tunnel	China	3 lanes	13.30	14.96	Slurry	Clay, sand, granite	Urban								https://www.herrenknecht.com/en/references/referencesdetail/shantou-suai-sub-sea-tunnel-east/
5	2017	Road	Suai highway tunnel, Shantou	China		13.60	15.30	Slurry										
6	2017	Road	Shenzhen highway tunnel*	China	4 lanes (stacked)	14.04	15.80	Slurry		River + Urban						Metro + Buildings +infra		https://www.tunneltalk.com/China-19Sep2017-Mega-TBM-for-double-deck-Shenzhen-highway-tunnel.php
7	2017	Road	Tokyo Outer Ring Road Kan-etsu to Tomei*	Japan	3 lanes	14.50	16.10	4 x EPB	Clay, sand, gravel, cohesive soil	Greenery + Ponds	x	x						https://www.tunneltalk.com/Japan-25Apr2017-Mega-TBMs-for-Tokyo-ring-road.php http://tokyo-gaikan-project.com/library/pdf/pamphlet02_e.pdf
8	2017	Road	Shanghai Zhuguang Road Tunnel	China	4 lanes (stacked)	12.81	14.41	EPB										
9	2016	Road	Shanghai Yanjiang A30 Motorway	China			15.43	2 x Slurry										
10	2016	Road	Shanghai Bei Heng Motorway	China	4 lanes (stacked)	13.80	15.53	Slurry		River						Metro		http://vmt-gmbh.de/en/keeping-shanghais-beiheng-passage-tunnel-on-track/
11	2016	Road	Zhuhai Hengqin Tunnel	China	3 lanes	13.24	14.90	Slurry										
12	2016	Road	Santa Lucia Highway Tunnel, A1 near Firenze*	Italy		14.11	15.87	EPB		Mountain								https://www.tunneltalk.com/TBM-Recorder-07Sep2016-Largest-ever-TBM-for-European-project.php
13	2015	Road	TMCLK	Hong Kong	2 lanes	12.40	13.95	Slurry	CDG, alluvium, granite, marine deposits	Sub-sea tunnel						x	x	
14	2015	Road	TMCLK*	Hong Kong	3 lanes	15.60	17.56	Slurry	CDG, alluvium, granite, marine deposits	Sub-sea tunnel						x	x	https://www.tunneltalk.com/Hong-Kong-15Sep2014-Tuen-Mun-Check-Lap-Kok-Link-world-record-TBM.php
15	2015	Road	Lung Shan Tunnel on Liantang Highway Project*	Hong Kong		12.60	14.10	EPB		Hills								https://www.tunneltalk.com/China-Hong-Kong-16Aug2017-Lung-Shan-road-tunnel-TBM-turn-around.php
16	2015	Road + Metro	Wuhan Metro road/metro river crossing*	China	3 lanes + metro (stacked)	14.01	15.76	2 x Slurry	Sand, clay, mudstone									https://www.tunneltalk.com/China-08May2014-Mega-TBM-for-Wuhan-Metro-Line-7-Yangtze-River-crossing.php
17	2013		Shouxhiou Lake Highway Tunnel	China		13.27	14.93	Slurry										
18	2013	Road	Seattle Alaskan Way SR 99*	USA	4 lanes (stacked)	15.54	17.48	EPB		Urban								https://www.tunneltalk.com/Alaskan-Way-Jul11-Hitachi-Zosen-to-build-mega-Seattle-TBM.php
19	2013	Road	Caltanissetta highway tunnel, Sicily*	Italy	2 lanes	13.45	15.08	EPB	clay, marl deposits, faults									https://www.tunneltalk.com/Italy-Nov2017-Caltanissetta-mega-TBM-performance.php
20	2013	Road	Auckland Waterview Tunnel*	New Zealand	3 lanes	12.81	14.41	EPB	Sandstone, siltstone	Urban								https://www.tunneltalk.com/New-Zealand-Aug11-Award-of-the-Waterview-Project-in-Auckland.php
21	2012	Road	Shanghai Hong Mei Road Tunnel	China	3 lanes	13.27	14.93	Slurry		River								
22	2011	Road	A1 Sparvo Highway Tunnel*	Italy		13.82	15.55	EPB		Rural								https://www.tunneltalk.com/Sparvo-project-Italy-29Jul2013-Final-breakthrough-proves-mega-TBM-method-alternative.php
23	2011	Road	Shanghai West Changjiang Yangtze River Road Tunnel	China		13.72	15.43	Slurry										
24	2011	Road	Galleria Sparvo	Italy	3 lanes	13.82	15.55	EPB	Clay, mudstone, sandstone	Farms / Rural								https://www.herrenknecht.com/en/references/referencesdetail/galleria-sparvo/
25	2011	Road	Weisan Road Tunnel, Nanjing*	China	4 lanes (stacked)	13.30	14.93	2 x Slurry	Clay, muddy soil, coarse to abrasive sand, gravel, boulders	River								https://www.tunneltalk.com/TBM-Recorder-Aug11-Mega-TBM-for-second-river-crossing-in-Nanjing.php
26	2010	Road	Seville SE-40 Highway Tunnels*	Spain	3 lanes	12.44	14.00	2 x EPB	Sand, Gravel and Marl									https://www.tunneltalk.com/TBM-Recorder-Apr10-NFM-TBMs-for-Spanish-river-crossing.php
27	2010	Road	Hangzhou Qianjiang Under River Tunnel*	China	3 lanes	13.72	15.43	Slurry			ü							https://www.researchgate.net/publication/258517773_The_Reasonable_Layout_of_Cross_Passages_for_Qianjiang_River_Tunnel_Based_on_Seismic_Analysis
28	2009	Road	Yingbinsan Road Tunnel, Shanghai	China		12.68	14.27	EPB										
29	2008	Road	Nanjing Yangtze River Crossing*	China	3 lanes	13.30	14.93	Slurry	Silt, sand, clay, gravel (heterogeneous)	River								https://www.tunneltalk.com/TBM-Recorder-Herrenknecht-Sept09-Yangtze-River-drives-complete.php
30	2007		Bund Tunnel, Shanghai	China		12.68	14.27											
31	2006		Shanghai Jungong Road Subaqueous Tunnel	China		13.22	14.87											
32	2006	Road	Shanghai Changjiang Under River Tunnel	China	3 lanes	13.72	15.43	Slurry	Silt, sand clay, rubble (heterogeneous)	River								
33	2006		Niagara Water Diversion Tunnel*	Canada			14.40											https://www.tunneltalk.com/Niagara-May11-Robbins-mega-TBM-breaks-through.php
34	2005	Road	Madrid M-30 By-Pass Sur Tunel Norte	Spain	3 lanes	13.51	15.20	EPB	Penuela, gypsum (soft ground)	Urban								
35	2004	Road + Metro	Moscow Silver Forest Tunnel	Russia	3 lanes + metro (stacked)	12.62	14.20	Slurry	Sand, clay, rock (heterogeneous)	Forest								
36	2004		Shangzhong Road Subacqueous Tunnel, Shanghai	China		13.22	14.87											
37	2004		Tokyo Metro	Japan		12.60	14.18											
38	2001	Road	Moscow Lefortovo*	Russia		12.62	14.20	Slurry	Fine to coarse sand, clay, limestone	River + Urban								https://www.tunneltalk.com/Russia-Oct2002-Lofortovo-TBM-highway-tunnel-project-Moscow.php
39	2000		Groenehart double-track rail tunnel	The Netherlands		13.22	14.87											
40	1997	Road	Hamburg 4th Elbe River Highway Tunnel*	Germany		12.62	14.20	Slurry	Boulder clay, silt, gravel, erratic blocks	River								https://www.tunneltalk.com/Germany-Jan2000-Fourth-Elbe-Tunnel-applies-cutting-edge-slurry-TBM-technology.php
41	1994		Trans Tokyo Bay Highway Tunnel*	Japan		12.57	14.14											https://www.tunneltalk.com/Japan-Aug1994-Trans-Tokyo-Bay-Highway.php

Means info difficult to obtain

Means approximate ID

Appendix D – Sink Holes Risks Memorandum

McMillen Jacobs Associates. October 07, 2019. DRAFT – Bored Tunnel – Sink Hole Considerations.

Memorandum

To:	Darryl Matson, COWI	Project: George Massey Crossing
From:	Marco Moccichino	cc. Anne-Marie Langlois, Doug Grimes, Fred Marquis
Date:	October 07, 2019	Doc. No.: 58362_003_MO_BT_B_Sink Hole_Considerations
Subject:	DRAFT - Bored Tunnel – Sink Hole Considerations	

Revision Log

Rev. No.	Date	Revision Description
A	Sept 09, 2019	Draft for review
B	Oct 07, 2019	Revised to address COWI and Charter PDI comments

1.0 Introduction

In July 2019, the COWI-Stantec Team was awarded a contract by the Ministry of Transportation and Infrastructure (MoTI) for technical services to study crossing options for the George Massey Crossing project (GMC). MJA is a subconsultant to the COWI team providing specialist advice for bored tunnel options for the crossing.

This memo addresses a request from MoTI to review the consequences, mitigations and impacts associated with sink holes, should they happen during tunnelling under the Fraser River.

2.1 Sink Holes during Tunnelling

Bored tunnels are excavated with tunnel boring machines (TBM). In general, a TBM is a shield with excavation, ground control, steering, lining assembly and propulsion equipment, including trailing gear and support equipment, required for performing tunnel excavation. This is a general term comprising the class of tunnelling machines that are fully shielded, use a full-diameter rotating cutterhead equipped with cutting tools, and advance using propulsion cylinders that thrust against an initial tunnel lining erected as a ring of segments within the shield tail. In addition, if a TBM excavates through non self-standing ground such as those at GMC, an active support pressure is applied to the tunnel face by a bentonite-water or bentonite-water- polymer slurry for slurry TBM's, or by highly viscous conditioned soil formed by the excavated material mixed with conditioners for Earth Pressure Balance TBM's.

The active pressure is instrumental to maintaining the stability of the excavation face in a tunnelling operation and to minimize the ground deformation that the excavation generates. When the applied pressure is less than that required to maintain a stable face, over excavation occurs. In the case of over excavation along the proposed alignment for the GMC bored tunnel and especially underneath the Fraser River, three scenarios can be considered with the associated consequences:

- Scenario A: the over excavation generates excessive settlement of the river bed without creating a sink hole;

- Scenario B: the over excavation generates excessive settlement of the river bed creating a sink hole but without flooding the tunnel or TBM; and
- Scenario C: the over excavation generates excessive settlement of the river bed creating a sink hole and flooding the TBM and tunnel.

The fourth scenario where the TBM and tunnel are abandoned would result in a complete halt to the project. This scenario has not been considered because it is extremely unlikely.

Mitigations and impacts can be foreseen for each scenario, as follows.

- Scenario A: tunnelling would continue while an intervention is implemented. The intervention would include grout injection from within the tunnel and possibly grouting of the river bed carried out from instream, barge-mounted equipment. This scenario would produce a delay in construction schedule on the order of a few months. The required grouting campaign would be substantial and costly, depending on the severity of the over excavation.
- Scenario B: tunnelling would be halted while an intervention is implemented. The intervention would require placing of concrete or equivalent engineered backfill in the river in addition to grout injection from within the tunnel and possibly grouting of the river bed carried out from instream, barge-mounted equipment. Placing concrete or backfill in the river may require installing a cofferdam. A cofferdam could pose an impediment to shipping for up to a year, depending on its size and location. If this scenario occurred in the vicinity of the existing tunnel, the stability of the existing tunnel could be affected. This scenario would delay the construction schedule by 6 months to 1 year per intervention. The anticipated intervention would require a substantial amount of instream works and be costly.
- Scenario C: tunnelling would be halted while an intervention is implemented. The intervention would require installation of a cofferdam in the river to stop water from flowing into the tunnel from the river. This cofferdam would create an impediment to shipping for up to 2 years, depending on its location. If this scenario occurred in the vicinity of the existing tunnel, the stability of the existing tunnel could be affected. Once the cofferdam is installed, water would be pumped out from the tunnel and concrete or equivalent engineered backfill would be placed within the cofferdam to seal the sink hole. In addition, the intervention would include grout injection from within the tunnel and possibly grouting of the river bed carried out from instream, barge-mounted equipment. Ultimately, damage to the TBM and other tunneling equipment could occur. This scenario would delay the construction schedule by 1 to 2 years and the cost of the intervention would be extremely high.

3.0 Likelihood of Sink Holes

Recent experience with large diameter TBMs in British Columbia and Washington State illustrate that sink holes during tunneling are real concerns (e.g. 4 occurrences during the Evergreen Line in Coquitlam and 1 during the SR99 Alaskan Way in Seattle). It is important to point out that the TBM diameter plays a fundamental role in the likelihood and consequence of sinkhole risks. For the GMC bored tunnel solution, a TBM diameter of more than 17.5 metres would be required and as shown in Figure 1, this would rank amongst the world's largest TBM's. To-date, MJA has not found a project with similar diameters, lengths or soil conditions as would be required for GMC. Unlike with the typical "metro" size TBMs (from 5 to 7 metres in diameter) with which industry has 20+ years and hundreds of projects worth of experience, experience is limited with tunnels with diameters similar to that anticipated for GMC and correspondingly, risks associated with this size tunnel are higher than normal.

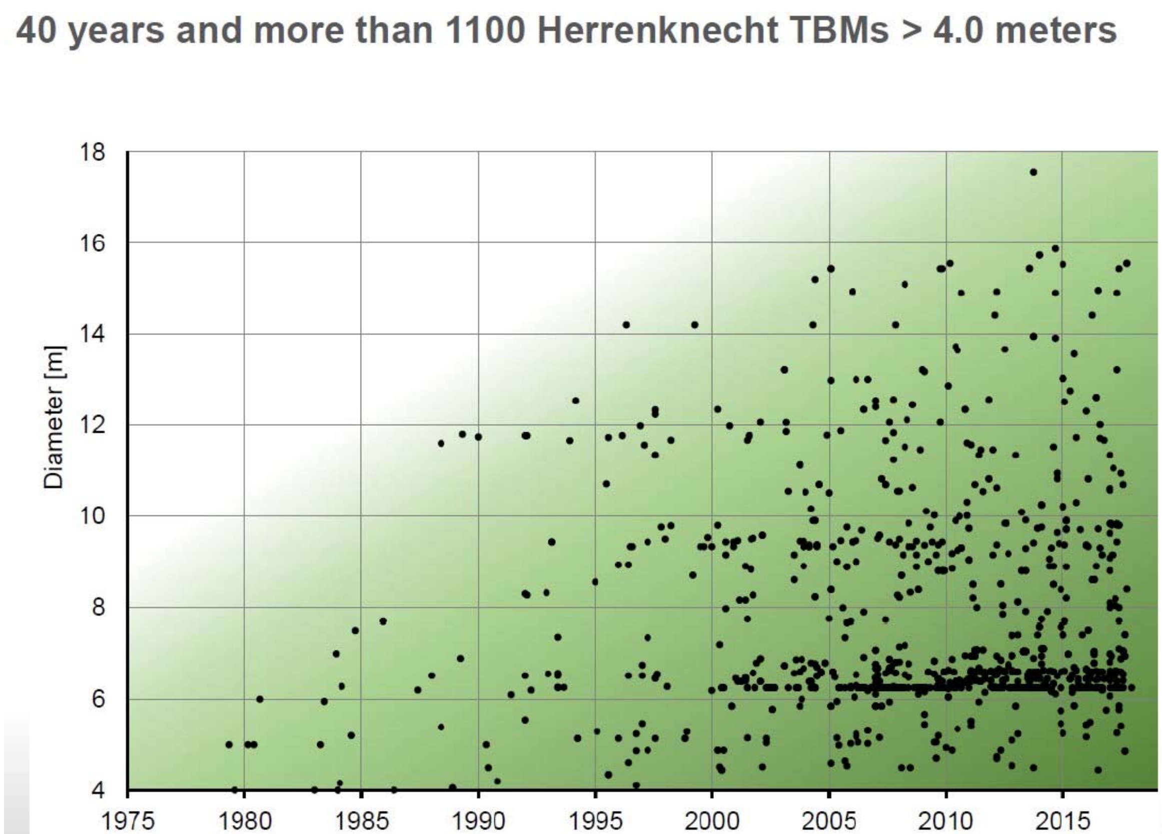

Figure 1 – TBMs Manufactured by Herrenknecht (source Herrenknecht AG)

Appendix E – Potential of Sink Holes Memorandum

Cowi. October 21, 2019. George Massey Crossing Project – Risk associated with TBM tunneling in the Fraser River area. Revision 1.

MEMO

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TITLE	George Massey Crossing Project – Risk associated with TBM tunneling in the Fraser River area
DATE	21 October 2019
TO	Darryl Matson
COPY	Mike Wong
FROM	RYWO/SZKO/DAMR
PROJECT NO	A127810-006

Revision Log

Revision	**Date**	**Revision Description**
1	21 October 2019	Final

Contents

1 Introduction and objective

This assessment is for the twin bored tunnels option for the proposed George Massey Crossing Project (GMC) connecting Richmond and Delta through Deas Island, British Columbia. This review is provided under a technical services contract awarded by the Ministry of Transportation and Infrastructure (MoTI) in July 2019. The potential for sinkholes has been identified as a critical risk while excavating a bored tunnel.

This memo:

- Provides an overview of a proposed bored tunnel configurations for the GMC and geology of the Fraser River area;
- Briefly reviews some historical cases of recorded sinkholes in tunnel boring machine (TBM) tunnel projects;
- Analyzes the use of TBM types and the potential for sinkholes; and
- Describes remedial measures in the event of sinkholes occurring.

2 Bored Tunnel Option

According to the Draft Technical Memorandum for Bored Tunnel Concept issued for review, dated 15 August 2019 (McMillen Jacobs Associates, 2019), one of the options considered for the crossing consists of 2 bored tunnels for up to 4 traffic lanes in each direction in stacked configurations. This concept assumes a TBM diameter of 18.5 m crossing the Fraser River with a total length of approximately 3.5 km for each tunnel, where the excavation takes place in the strata of Fraser River Sand Deposits and Post Glacial Marine Deposits. The assessment in the following sections focuses on the TBM operational risks specific to soft ground tunneling in the Fraser River area. For the envisaged ground conditions, TBM selection should consider the capability of machine operation in closed-face mode, which would be either a Slurry TBM or an Earth Pressure Balance TBM (EPBM).

An initial review of the proposed alignment indicates the following potential consequences of sinkholes or excessive settlements, a tunnel geological profile is presented in Annex A of this memo:

North Portal to Ch 117 – this area has mainly greenfield conditions with the tunnel increasing in depth from initial launch to approximately 25 m to tunnel crown. Should a sinkhole develop here the impact would be minimal in terms of damage to existing infrastructure. However, there would be a significant delay to the project. The TBM launching pit is often an area that can lead to high settlement and sinkholes as it is difficult to control the face pressure of the TBM at this stage, and there is generally very shallow cover. Ground treatment in these areas is often undertaken to help mitigate this risk.

Ch 117 to 119.5 – this area is adjacent to Highway 99, and sinkhole development in this area would have an impact on the highway, lead to possible road closures and introduce corresponding safety related concerns with regard to the travelling public.

Ch 119.5 to 124.5 – this area is predominately greenfield with an access road (Rice Mill Road). The depth of the tunnel increases to approximately 45 m cover above the tunnel crown. The implications of a sinkhole here would be low in terms of the impact on infrastructure and property but would cause a significant delay to the project.

Ch 124.5 to 130 – the tunnel is beneath the main stem of the Fraser River at this point with a depth to tunnel crown of 51 m. The tunnel is also in the zone of influence of the existing George Massey tunnel. The implications of a sinkhole under the river could be difficult to rectify and require substantial intervention works in the river to permit tunneling to continue. Such work would potentially require construction of cofferdam works that would alter aquatic ecology and river hydrology upstream and downstream, affecting water quality, quantity and aquatic environment. This sinkhole and environmental approvals and implementation of necessary measures would also cause a significant delay to the project.

Ch 130 – 134 – this section is predominately greenfield with access roads.

Ch 134 – 145 – for this section, the proposed tunnel is adjacent to or below the existing highway with the depth ranging from around 50 m to around 30 m to the tunnel crown. Sinkholes in this area would present a significant risk to the highway and safety of the travelling public. This area is also below the southern branch of the Fraser River. Therefore, the impact of any sinkhole/settlement will both occur in proximity by the highway and the river. The concerns similar to above given for tunneling below Fraser River should be considered when tunneling under Deas Slough also with alike implications.

Ch 145 – 149 – this is a combination of greenfield and highway, which a sinkhole formation would not lead to any significant impact to the public. It is common for similar projects that strict monitoring regime and controls are in place for early problem detection. Any subsidence within the highway area would require temporary diversions and remedial works with some impacts to the public expected.

3 Geology

A review of geological publications and ground investigation data for the GMC site shows that the site is overlain by both Fraser River Sand Deposits and Post Glacial Marine Deposit at the proposed tunnel level. The Fraser River Sand Deposits consist predominantly of uniformly graded SAND, interlayered SILT and SILTY SAND layers. The Fraser River Sand Deposits attain a maximum depth of around 32 m. The Post Glacial Marine Deposits consist of interlayered SAND, SILTY SAND, SILT and CLAYEY SILT layers. The Post Glacial Marine Deposits extend to a depth of greater than 300 m below ground.

The details of geological conditions and soil parameters at the site are presented in the memo of George Massey Crossing Project - Preliminary Assessment of Bored Tunnel Depth (COWI, 2019) and are not repeated here.

4 Historical large settlement/sinkhole incidents

This section summarizes case histories of tunneling projects that have experienced significant incidents that caused excessive settlement or sink holes. The British Tunneling Society and Institution of Civil Engineers (2005) has presented, see Table 4.1, the data of over 100 international TBM tunneling incidents for projects carried out in Japan, China, Hong Kong, Singapore, the United States, Canada, the UK, Germany, France and other European countries. The risk of an incident is dependent on the type of TBM used.

In general, and for TBM selection and application in similar ground conditions to expected for the proposed GMC, there are two types of TBM to be considered: a slurry TBM and an earth pressure balance TBM (EPBM). As is discussed below, an EPBM is possibly appropriate for the GMC site given the sites geotechnical conditions. Based on the research done as presented in Table 4.1 (BTS & ICE, 2005), the risk of incidents is typically higher with an EPBM than with a slurry TBM. However, the total number of incidents indicated in Table 4.1, was caused by multiple factors simultaneously and as such could be linked to multiple variables including misinterpretation of ground conditions, experience of operators and ongoing technological development of both TBM types, etc.

A smaller reported number of slurry TBM incidents does not necessarily indicate that slurry TBM is safer than EPBM. However, based on the tunnel date surveyed in this study, it shows that incidents are more frequent when using an EPBM. The number of EPBM incidents due to inappropriate technical decisions is significantly higher than that of slurry TBM incidents. The study indicates that decisions were made to reduce the face pressure to assist in spoil conditioning or to reduce cutterhead wear, which lead to these EPBM incidents (BTS & ICE, 2005).

Table 4.1. Summary of incidents experienced in 100 surveyed tunnel (BTS & ICE, 2005)

Incident features	Slurry TBM incidents	EPBM incidents
Total number of incidents	14	>47 multiple similar incidents
Problem during maintenance or TBM problems	4	8 series of incidents
Ground obstructions	1	4
Over-excavation	8	16
System obstructions/other unknown causes	None	10
Mixed-face ground conditions	5	13
Human error	4	15
Inappropriate technical decisions	None	13
Exit/entry to launch/reception shafts	None	9

In addition to the review of the tunneling projects presented in Table 4.1, an analysis of the major EPBM projects presented in Table 4.2 indicates that, on average, settlement and sinkholes for seven major projects involving EPBMs show that there is one incident of settlement larger than 150 mm for every 1.35 km of EPBM driven tunnel (Shirlaw & Boone, The risk of very large settlements due to EPB tunnelling, 2005). Based on this average, 5 incidents of excessive settlement and/or sinkholes could be expected for the GMC project if an EPBM is used.

Table 4.2. Cases of large settlement or sinkholes over driven tunnels (modified from Shirlaw & Boone, The risk of very large settlements due to EPB tunnelling, 2005).

Tunnel	Country	Year	Length (km)	Incidents	No./km
MTR Phase 2	Singapore	1986-1987	1.6	0	0
St. Clair River	Canada	1993-1994	1.9	3	1.6
Allen Sewer	Canada	1994	1.1	1	0.9
Sheppard Line	Canada	1997-1998	6.4	15	2.3
Changi Line	Singapore	1998-1999	7	1	0.1
North East Line	Singapore	1998-2000	20	16	0.8
Tunnels in general (to October 2003)	Singapore	1999-2005	38.9	21	0.5
Alaskan Way SR99	USA	2013-2017	3.5	1	0.3

5 Risks of substantial settlement/sinkholes

Given the complex and challenging ground conditions, including the risk of liquefaction, the risk for tunnel construction with a large diameter TBM is high. Also, given the relatively low relative density of the soil layers, using a large diameter TBM may increase the risk of a higher than normal volume loss of >1%, which could lead to significant settlement on the surface. Due to these complex ground conditions there is a significant risk of that sinkholes or excessive settlement will be encountered when driving a large diameter TBM tunnel below Fraser River. This risk is a result of the following governing factors:

- Challenges with controlling tunnel face stability;
- Groundwater conditions and soil permeability;
- Overcutting/over-excavation due to TBM steering;
- Mechanical breakdowns;
- Compressed air interventions;
- Unexpected subsurface conditions; and
- Unexpected voids.

These factors are discussed in the following:

5.1 Controlling of tunnel face stability

The design and operation of the TBM as a means of controlling occurrence of excessive settlement and sinkholes is the crucial element of controlling of the tunnel face stability. Ensuring that the TBM is adequately designed will minimize the risk.

This determines the type of TBM used for the predicted ground conditions. The decision of using an EPBM, a slurry or a variable density TBM will be the key factor over the control of the ground stability and minimizing ground loss. Alongside the selection of TBM type, others factors include:

- TBM cutterhead design and selection of the appropriate tools for the head;
- Tail void grouting system; and
- Muck removal system design.

It is particularly difficult to control the face pressure when a 18.5 m diameter TBM face encounters inter-layered soils and pockets such as the sand lenses present at the GMC site.

The soil particle size distribution from the GMC geotechnical data report shows that the medium to fine SAND and SILTY SAND are uniformly graded. The coefficient of uniformity of these soils is approximately 2.5. With the presence of CLAYEY SILT, neither a Slurry TBM nor an EPBM alone are optimal solutions for the GMC project. The difficult ground conditions expected to be encountered at the GMC site will require careful operation and control of the TBM to maintain face stability and thus reduce the risk of adverse ground movement and formation of sinkholes.

Figure 5.1 shows that when a slurry TBM encounters the SILT and CLAYEY SILY layers within the Post Glacial Marine Deposits that are present at the GMC site (i.e. when soil gradation falling within shaded areas B of Figure 5.1), significant effort will be required to separate the fines from the slurry and will require a sophisticated slurry separation plant. This will impact on the cycle times and therefore impact cost and program (BTS & ICE, 2005). However, a slurry TBM will also operate within the optimal range when encounters the SAND layers within the Fraser River Sand Deposits and Post Glacial Marine Deposits, i.e. when soil gradation falling with in shaded areas A of Figure 5.1.

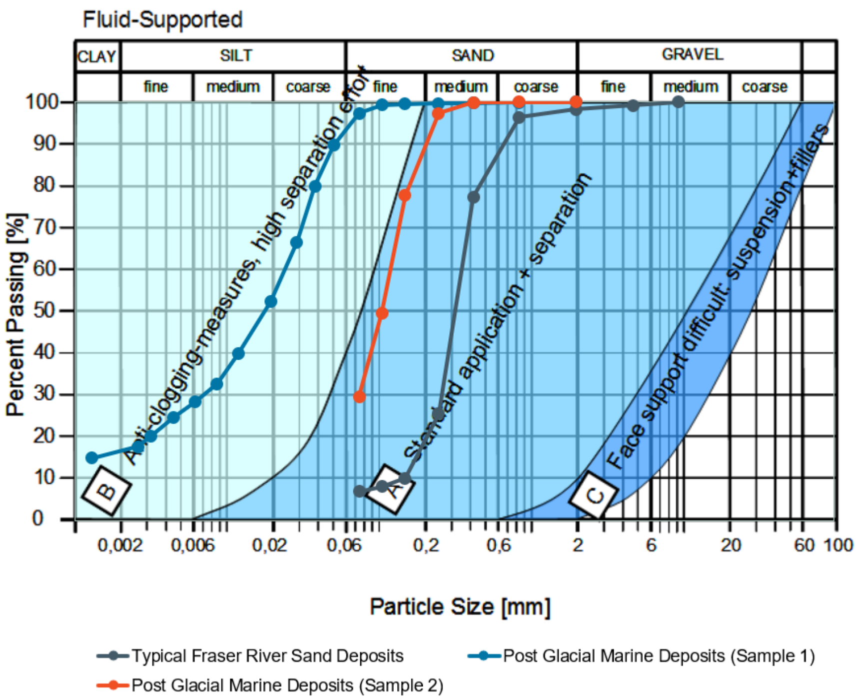

Figure 5.1. Application ranges of slurry TBMs (German Tunnelling Committee, 2016) with soil particle distributions curve of samples from Fraser River (Golder Associates Ltd., 2014).

Figure 5.2 shows where an EPBM intersects the SAND and SILTY SAND layers of the Fraser River Sand Deposits and Post Glacial Maine Deposits, i.e. when the gradation cures intersects shaded areas 3 and 4 of Figure 5.2. With the high groundwater pressures present under the Fraser River combined with the highly permeable sands an EPBM is also not optimal but likely a better choice than a slurry TBM. Notwithstanding, an EPBM it would be difficult to the control - both in terms of the steering and the face support. This could lead to loss of ground at the face leading to excessive settlements and/or sinkholes. However, it is desirable for EPBM excavation in the CLAYEY SILT and SILT layers within the Post Glacial Marine Deposits, i.e., Sample 1, of Post Glacial Marine Deposits shown in Figure 5.2 falling within shaded area 1.

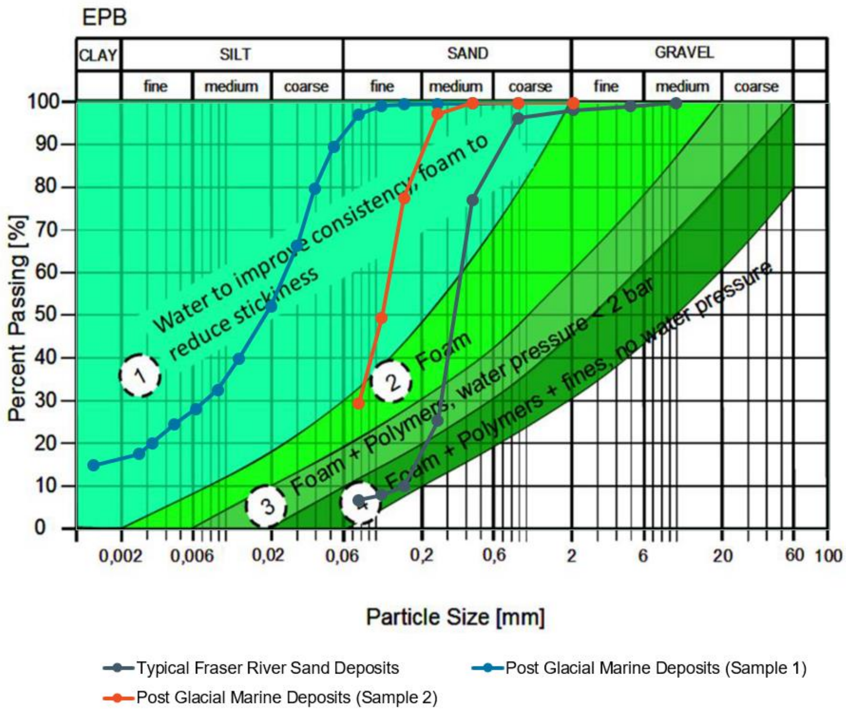

Figure 5.2. Application ranges of EPBMs (German Tunnelling Committee, 2016) with soil particle distributions curve of samples from Fraser River (Golder Associates Ltd., 2014).

5.2 Groundwater and soil permeability

The permeability and hydrostatic head also have a significant influence on TBM's effectiveness of maintaining a stable excavation face. Based on a review of the permeability of the sols at the GMC site, neither a slurry TBM nor an EPBM alone would be able to mitigate the risk of sinkholes formation or adverse settlement entirely because of the challenges of maintaining face stability in such complex geology.

As such, consideration may need to be given to driving the bored tunnels using a Variable Density TBM (multi-model TBM with Slurry and EPB modes). There is limited experience with this technology, and this could introduce currently unforeseen risks.

5.3 Overcutting due to TBM operations and steering

TBM shields are commonly designed such that the cutterhead has a slightly larger diameter than the tail shield to allow a degree of overcutting and thereby minimize the shield skin friction. The effect of overcutting creates voids in loose sand that are liable to close before tail void grouting. A case study (Shirlaw, 2006) presented in Table 5.1 shows the potential volume loss due to TBM overcutting and negotiation through curves. The combination of the factors in Table 5.1 could lead to a cumulative volume loss of over 3% and significant surface settlement. As a result, one should carefully select the geometry and controlling features of a TBM to minimize the risk of inducing large ground movements leading to undesirable settlements.

Table 5.1. Case history of a 6.46 m TBM and causes of voids around TBM shield skin.

Potential cause of void around TBM shield skin	Potential volume loss
Difference between minimum cut diameter and skin diameter	0.6%
Extending over-cutters by 70 mm	5%
200 m turning curve	0.6%

5.4 Mechanical breakdown

Mechanical breakdown of TBMs could also lead to large settlements, particularly where the TBM becomes stuck or stops for maintenance for a considerable period. Therefore, the TBM design should consider the ground, site conditions such that potential obstacles expected along the alignment can be accommodated and therefore stoppages minimized. For example, a slurry TBM should have a crusher in the slurry chamber if boulders are present to prevent clogging the slurry pipes. Also, the contractor should routinely inspect and maintain the TBM to minimize the risk of mechanical breakdown and set up contingency protocols for circumstances in case of TBM breakdown.

5.5 Compressed air interventions

Compressed air cutterhead interventions is a high risk activity when tunneling using TBMs. Historical cases have shown that sinkhole occurred during this type of interventions (see Section 6.1.5). The locations and frequency of interventions should be planned ahead of the tunnel excavations to minimize the risk of excessive settlement and sinkholes as well as the occupational health and safety of the workers. The contractor should plan to avoid compressed air invention in difficult ground conditions or at the deepest area of the tunnel. A well-planned TBM tunnel construction will prevent putting workers' lives at stake and minimize the risk of excessive settlement.

5.6 Unexpected underground conditions

Unexpected underground cavities are a common cause of losing slurry pressure suddenly, which causes excessive settlements and sinkholes. This usually is a situation in karstic rocks and not likely an issue of the Fraser River area. However, sufficient site investigation should be carried out to ensure that there is no open path in the soil layers or any unknown man-made wells or pipes intersecting the tunnel alignment. This is very important in minimizing the risk of excessive settlement and sinkholes.

6 Remedial measures

This section discusses some possible remedial measures in case of excessive settlement or sinkhole. The first part of this section reviews some notable tunnel failure case histories, and the second part describes potential remedial measures that can be taken if a sinkhole occurs while the TBM is under the Fraser River.

6.1 Case histories of notable tunnel failure cases

6.1.1 Green Park Tunnel, London, United Kingdom, 1964

Background: A segmental lined tunnel from Green Park to Victoria was driven through London Clay with low soil cover. The London Clay layer was overlain by water-bearing sands and gravels. The crown of the shield penetrated through the top of the London Clay layer into the water-bearing sand and gravel and caused a face collapse burying most of the shield.

Remedial measures: The contractor sunk a shaft from the surface to remove the collapse materials and treat the ground to allow for further tunneling.

6.1.2 Munich Underground, Germany, 1980

Background: Two 6 m diameter tunnels were excavated in the flinty marl layer using the New Austrian Tunneling Method (NATM). Due to the unexpected local variation in marl thickness over the tunnel elevation and tunnel face, the ground overstressed the sprayed concrete lining and caused a momentous and massive flow of soft clay into the tunnels. A 10 m wide and 14 m deep sinkhole occurred.

Remedial measures: The contractor backfilled the void with crushed rock and cement and treated the ground by pressure grouting.

6.1.3 Moda Collector Tunnel, Istanbul Sewerage Scheme, Turkey, 1989

Background: A TBM tunnel was driven in mixed ground conditions from very fine mud to rocks. TBM intersected an unexpected layer of soft clay and caused a sinkhole on the road surface 5 m above. The broken rocks jammed the TBM.

Remedial measures: The contractor sunk a shaft to rescue the TBM from the collapsed face.

6.1.4 Heathrow Express Tunnel, London, United Kingdom, 1994

Background: Two NATM tunnels were driven for a new express train line connecting the Heathrow Airport to London. A series of design and management errors combined with poor workmanship and quality control, which led to the tunnel collapse.

Remedial measures: The contractor backfilled the shaft and tunnel excavations with 13,000 m^3 of concrete.

6.1.5 Kowloon Southern Link, Salisbury Road, Hong Kong, 2007

Background: Two railway tunnels in the downtown area of Hong Kong were driven in highly and moderately weathered granite overlain by marine sands using slurry TBMs. A sudden loss of compressed air pressure during intervention through the interface between weathered granite and marine sand resulted in a face collapse and a 2 m by 3 m sinkhole on the road surface.

Remedial measures: The contractor backfilled the sinkhole with granular fill.

6.1.6 Langstaff Road Trunk Sewer, Toronto, Canada, 2008

Background: A TBM tunnel was driven by an EPBM through highly saturated sands and silts under a high water table. The damaged wire brushes at the tail shield of the EPBM initiated approximated 1,800 m^3 mudflow into the TBM over 48 hours burying the TBM. A deep sinkhole occurred on the ground surface and caused a significant delay to the project.

Remedial measures: The contractor filled the sinkhole with non-shrink low strength concrete and built a bulkhead at about 300 m behind the TBM face to control ground inflow as emergency measures. The continuous subsidence in the area was stabilized with sand infill.

6.1.7 Ottawa Light Rail Transit, Ottawa, Canada, 2014

Background: 2.5 km single-tube double-track running tunnels were excavated using roadheader in limestone with notable pockets of clay and sandy clay deposits. Soft clay and sandy clay flew into the tunnel face from the heading and caused a sinkhole of 8 m wide by 12 m deep.

Remedial measures: The contractor built a bulkhead behind the tunnel face to stabilize the ground before backfilling the sinkhole with over 620 m^3 of concrete.

6.2 Options for remedial measures

Remedial measures for excessive settlement and sinkholes generally comprise:

- backfilling with concrete or grout (Figure 6.1);
- backfilling with soil or crushed rocks (Figure 6.2);
- building a bulkhead to stabilize tunnel face; and
- sinking shafts to rescue the tunnel.

Given the geological conditions and site constraints of the Fraser river area, if a sinkhole occurs while the TBM is under the river, there are at least two possible scenarios as discussed in Section 6.2.1 and 6.2.2.

Figure 6.1. Backfilling sinkhole with concrete.

Figure 6.2. Backfilling sinkhole with crushed rocks.

6.2.1 Scenario 1: A sinkhole occurs without damaging the TBM

One of the appropriate solutions for the situation that a sinkhole occurs without damaging the TBM could be building a cofferdam surrounding the sinkhole with sheet piles to cut off the water flow. Then, one could backfill the sinkhole with crushed rocks or granular fills to stabilize the ground. Given the size of a large diameter TBM, the cofferdam would need to be sufficiently large and deep to cut-off the TBM shield from the surrounding ground. According to COWI's tunnel depth technical memo (COWI, 2019), the maximum feasible tunnel depth would be approximately 50 m with the current TBM technology, where it would be costly if the sinkhole is large and extend to the tunnel level.

6.2.2 Scenario 2: A sinkhole occurs with soil flowing into the TBM

One of the possible solutions for the situation that sinkhole occurs with soil flowing into the TBM is similar to case 1 in Section 6.2.1. Apart from that, a bulkhead should be built within the tunnel to stabilize the tunnel face before backfilling the sinkhole. The collapsed ground material should be treated as necessary.

The cost and consequence analysis has been covered in a separate assessment (McMillen Jacobs Associates, 2019), which is not repeated here.

7 Summary

This assessment reviews the risk of excessive settlement and sinkholes with a deep bored tunnel. Risk mitigation measures and appropriate remedial works are also discussed.

Based on the review of numerous tunneling projects around the world, incidents that could lead to excessive settlement and/or sinkholes are not unusual. Based on the review of several EPBM projects, it was found that, on average, one sinkhole occurs for every 1.35 kilometer of bored tunnel.

For a TBM with the diameter proposed for GMC, tunneling would induce a significant amount of ground settlement. This potentially excessive settlement could impact various assets along the proposed route including Highway 99, local access roads; local buildings, exiting immersed tube tunnel. Further study and risk assessment would be required to fully understand the potential tunnel construction impacts.

Remedial work to repair sinkholes vary from filling the sinkhole with gravel or concrete to installing a cofferdam to isolate the TBM and allow the sinkhole to be repaired. If a sinkhole were to develop in the Fraser River, installation of a cofferdam would be required. Such a cofferdam would be expensive and would result in a significant delay to the project.

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ANNEX 1: Tunnel geological profile (McMillen Jacobs Associates, 2019)

PROFILE VIEW

COWI Stantec

GEORGE MASSEY CROSSING PROJECT

SCALE	0 20 1:2500 100m	CAD FILENAME CAD FILE	PLOT DATE 8/15/2019

REV	DATE	REVISIONS	NAME

BRITISH COLUMBIA

MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE
SOUTH COAST REGION
HIGHWAY ENGINEERING AND GEOMATICS

SENIOR DESIGNER
DATE YYYY-MM-DD

DESIGNED Name 1 DATE YYYY-MM-DD
QUALITY CONTROL Name 2 DATE YYYY-MM-DD
QUALITY ASSURANCE Name 3 DATE YYYY-MM-DD
DRAWN Name 4 DATE YYYY-MM-DD

PLAN
GEORGE MASSEY
CROSSING PROJECT
BORED TUNNEL OPTION
STA. 100+00.000 to STA. 127+00.000

FILE NUMBER	PROJECT NUMBER	REG	DRAWING NUMBER	REV
File #	Proj #	1	CONCEPT #?	-