GEORGE MASSEY CROSSING: PRELIMINARY CFD MODELLING RESULTS

DRAFT REPORT

Prepared for:

COWI North America Ltd.
North Vancouver, British Columbia

On behalf of:

Ministry of Transportation and Infrastructure
British Columbia

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DISCLAIMER

CREDITS AND ACKNOWLEDGEMENTS

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The following NHC staff participated in the project, under the roles indicated in parenthesis:

- Cass Zhao (CAD Specialist)
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- Ashley Dudill (Numerical Modeller)
- Jose Vasquez (Principal Numerical Modeller)
- Matt Gellis (Principal Hydrotechnical Engineer and Project Manager)

A large portion of the bathymetric data used in this report was kindly provided to NHC by Public Services and Procurement Canada whose regular multibeam surveys of the lower Fraser River provide an invaluable source of data for the river.

EXECUTIVE SUMMARY

Northwest Hydraulic Consultants Ltd. (NHC) was retained by COWI North America Ltd. (COWI) to undertake preliminary computational fluid dynamics (CFD) analysis for the proposed George Massey Crossing (GMC) Project, specifically in support of the Immersed Tube Tunnel (ITT) options. Modelling was undertaken using the CFD software FLOW-3D. Modelling commenced on August 23, 2019 and was completed on September 3, 2019.

The preliminary CFD simulations were undertaken to:

- Assess current (baseline) conditions related to velocities and flow patterns in the reach of river surrounding the existing GMC
- Analyse potential changes to velocities and flow patterns due to the GMC project
- Infer sedimentation and erosion responses to the project from the changes in velocities and flow patterns.

Preliminary simplified modelling has been undertaken. The results presented herein are preliminary (the limitations are clearly stated in the report) and should be confirmed with more detailed analysis if the ITT options are carried forward.

The CFD model extends approximately 1.9 km upstream and downstream of the existing GMC, including Ladner Reach, Deas Slough and Deas Dock. Six model geometries were developed as part of the assignment, with each geometry run assuming two different flow conditions. Models A and D represent baseline tests, with the existing tunnel only; A represents anticipated bed conditions in 2070 (2 m of bed lowering compared with current conditions) and D represents current bed conditions. Model B represents the existing tunnel and the proposed immersed tube tunnel (ITT) under anticipated bed conditions in 2070 (Models B-1 and B-2 represent different riprap configurations for the proposed ITT). Model C represents the existing tunnel and the proposed excavation (Models C-1 and C-2 represent different excavation configurations), under current bed conditions.

The main findings from Models A and B (2070 anticipated bed conditions) are:

- Riverbed degradation, the long-term lowering of the bed, has been historically occurring in the lower Fraser River. This degradation has been estimated at 2 m by 2070. This anticipated reduction in bed levels needs to be accounted for in the design of a new ITT as well as in the design of the scour protection for the existing tunnel.
- Assuming 2 m bed degradation, the existing tunnel will protrude further above the river bottom, generating turbulence within the flow field. Due to the proximity, the proposed ITT will be subjected to this turbulence, which will have scouring implications that will need to be considered during the design of the ITT and scour protection.

The main findings from Models C (construction phase) and D (baseline conditions) are as follows:

- The flow will decelerate within the construction trench, due to the increase in flow area. The reduction in velocities will cause sediment to deposit within the trench.
- A sloping excavation results in a more gradual change in flow, which is advantageous from a hydraulics perspective when compared to an underwater sheet pile retaining wall.

In addition to presenting and discussing the CFD model and results, this report summarises critical information related to hydraulics and sediment transport at the project site, which was provided to the COWI-Stantec Team by NHC during the conceptual design of the proposed ITT.

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1 INTRODUCTION

1.1 Scope of Work

Northwest Hydraulic Consultants Ltd. (NHC) was retained by COWI North America Ltd. (COWI) to undertake preliminary computational fluid dynamics (CFD) simulations and provide hydrotechnical assistance during the immersed tube tunnel (ITT) design development for the George Massey Crossing (GMC) upgrade project. NHC's scope of work included:

1. Development of a hydrotechnical scope
2. Review of background information
3. Assessment of existing hydrotechnical conditions, including an overview assessment of existing river morphology, scour and deposition patterns, and the effects of the tunnel and the nearby Lulu Island-Delta Main.
4. Development of a CFD model of existing conditions as a baseline for comparison with the crossing options
5. Development of four (4) CFD models to assess the hydraulic and morphologic effects of various new Immersed Tube Tunnel (ITT) crossing options defined by COWI
6. Produce a report detailing the analyses and results, providing conclusions regarding the hydrotechnical aspects of the crossing options.

The assignment commenced on August 2, 2019. Following collaboration between COWI and Stantec, along with hydrotechnical and geomorphological input from NHC, concept designs for the ITT were defined in mid-to-late August. CFD modelling was undertaken between August 23 and September 3, 2019. The preliminary results contained herein were presented to COWI on Wednesday September 4, 2019.

1.2 Purpose

Preliminary CFD simulations were undertaken to:

- Assess current (baseline) conditions related to velocities and flow patterns in the reach of river surrounding the existing GMC
- Analyse potential changes to velocities and flow patterns due to the GMC upgrade project
- Infer sedimentation and erosion responses to the upgrade project from the changes in velocities and flow patterns.

In addition to presenting and discussing the CFD model and results, this report summarises in Section 2.0 critical information related to hydraulics and sediment transport at the project site, which was provided to the project team by NHC during the conceptual design of the proposed ITT.

1.3 Previous Work

In 2014 the British Columbia Ministry of Transportation and Infrastructure proposed a project to replace the existing George Massey Tunnel. The project consisted of removing the existing tunnel and replacing it with a new 10-lane bridge which spanned the Fraser River. NHC was retained by MMM Group Limited to undertake a hydrotechnical analysis (NHC, 2015b) as part of this project.

The purpose of NHC's 2014 study was to assess the potential changes in the Fraser River hydraulics and morphology due to the removal of the existing tunnel. The study reviewed the potential changes to water levels, velocities, flow patterns and their influence on sedimentation and erosion within the Fraser River. At the time of the project, a clear-span bridge was proposed to replace the existing tunnel. As such the potential changes due to the crossing replacement were not investigated.

2 PROJECT BACKGROUND: FRASER RIVER

This Section provides background information on the Fraser River relevant for the existing tunnel and proposed new GMC, including hydrology, riverbed degradation and salt wedge. Past observations in the Fraser River suggest the existing tunnel could behave in the future as a submerged sill, therefore historic cases of submerged sills on the lower Fraser are briefly presented and discussed in Section 2.3.

2.1 Hydrology

The Fraser River has a snowmelt-dominated flow regime, with discharges typically rising in April, peaking between May and July, and then receding during the autumn and winter months. Discharges on the lower Fraser River are commonly referenced to Hope, which is upstream of the tidal influence, due to presence of a Water Survey of Canada (WSC) gauge which has been active since 1912. Figure 1 illustrates the typical annual hydrograph shape and the range in measured discharge magnitudes at Hope.

A WSC gauge also exists at Mission, located approximately 75 km downstream of Hope. Flows at Mission are typically between 10 % and 15 % larger than at Hope due to inflows from the Harrison River and the Chilliwack River. Table 1 summarises the estimated peak discharges at the Hope and Mission gauges for a range of return periods (NHC, 2008). Inflows downstream of Mission are relatively small; the drainage area below Mission represents less than 2 % of the total basin. Approximately 50 km downstream of Mission, the Fraser River trifurcates into the North Arm, South Arm and Annacis Channel. Annacis Channel rejoins the South Arm approximately 7.5 km farther downstream. The North Arm accounts for approximately 10 % of the flow, with the remaining 90 % flowing through the South Arm. The existing GMC is located on the South Arm of the Fraser, 16 km downstream of the trifurcation.

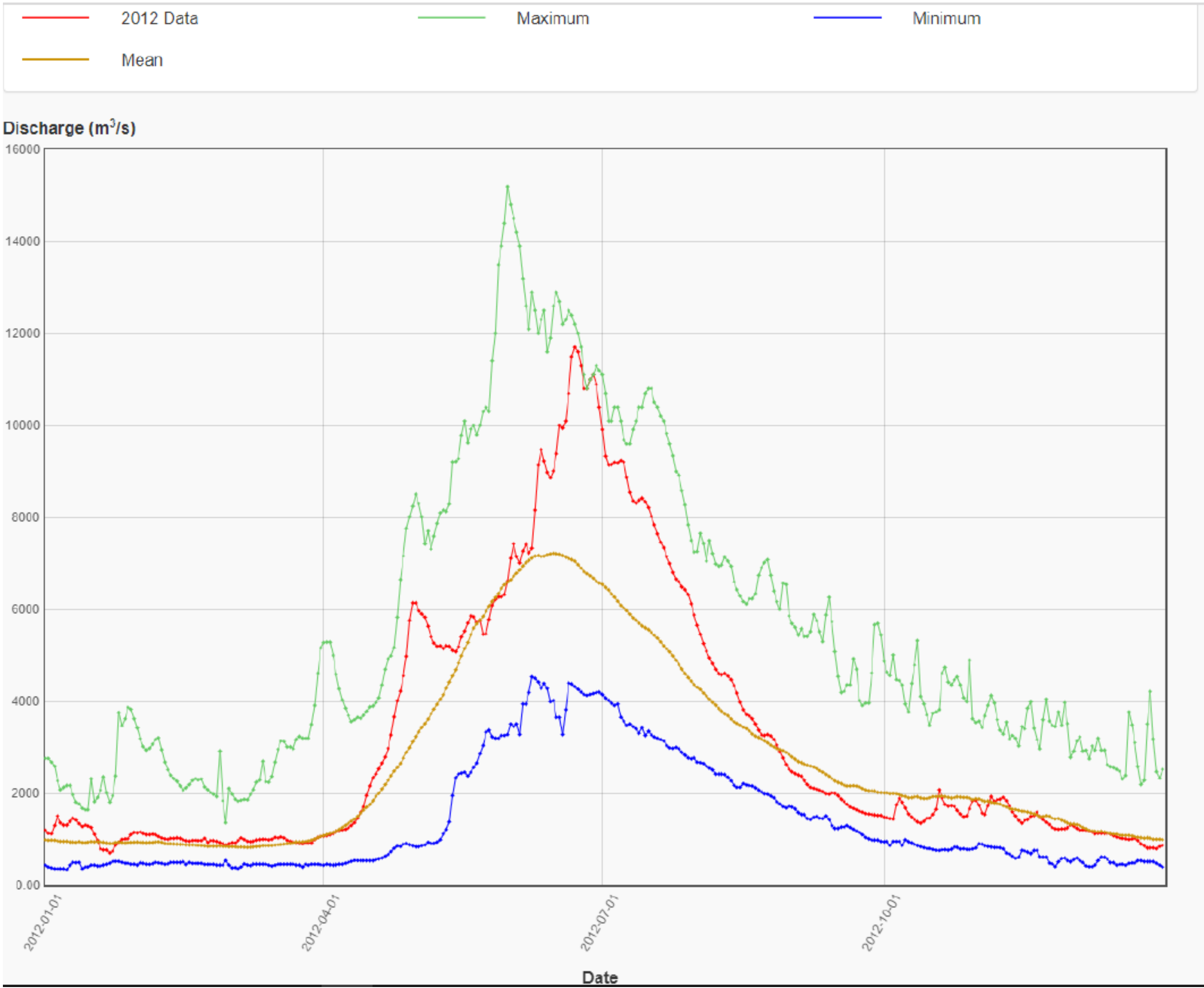


Figure 1 **Maximum, mean and minimum recorded discharges and 2012 discharges on the Fraser River at Hope (08MF005). Image reproduce from the Water Survey of Canada website (https://wateroffice.ec.gc.ca/report/historical_e.html?stn=08MF005&dataType=Daily&parameterType=Flow&year=2012&mode=Graph&y1Max=1&y1Min=1&mean1=1&scale=normal).**

Table 1 Flood frequency analysis at Hope and Mission

Return Period (Years)	Annual Exceedance Probability (%)	Discharge (m^3/s)	
		Hope	Mission
10	10	11,100	12,500
20	5	12,100	13,700
50	2	13,300	15,000
100	1	14,200	16,000
200	0.5	15,300	17,300
500	0.2	17,000	19,000

2.2 Sediment Transport and Riverbed Degradation

Sediment transport, erosion and scour processes are often strongly related to extreme flood events. A major flood occurred on the Fraser River in 1948, having a return period of approximately 100 years at Hope. More recently, and since the construction of the existing GMC, relatively high flows have occurred in 1972 (~ 40-50 year return period), 2012 (~ 20 year return period) and 2018 (~ 10 year return period). As such, the existing tunnel has not been subjected to flood event greater than a ~ 40-50 year return period.

Riverbed degradation is the long-term lowering of the bed, at a reach scale, and is generally caused by changes in base level, sediment supply or hydrologic regime, and changes which may occur as a result of anthropogenic change. Average riverbed levels in the lower Fraser River have historically been degrading at a rate of 2-8 cm/year, with the highest degradation rates associated with periods of intensified dredging (McLean and Tassone, 1990; NHC, 2017a). NHC (2017a) undertook a detailed scour assessment in the lower Fraser River using two different approaches to estimate future bed degradation rates. Firstly, a morphodynamic model was used to estimate future bed degradation rates at New Westminster if the navigational draft in the Fraser was to be increased from 11.5 m to 13.5 m. The morphodynamic model spanned 90 km of the Fraser River, between Sumas Mountain and the Strait of Georgia. For the current navigational draft of 11.5 m, the model predicted an average bed lowering rate of ~ 2 cm/year, whereas if the navigational draft was to be increased to 13.5 m, the bed lowering rate would increase to ~ 8 cm/year. Observations at select sites along the lower Fraser River have shown, however, the minimum bed levels (i.e. scour holes) are lowering at a faster rate than general (average) bed levels (e.g. NHC, 2015a). In the morphodynamic model, scour holes were found to degrade at 12 cm/year for the current navigational draft and 20 cm/year for a 13.5 m draft. These rates were suggested for a 50-year time span and need to be considered as part of the scour protection design for the ITT options for the GMC.

Ongoing climate change over the next several decades is expected to cause changes to the hydrological regime of the Fraser River and to induce sea level rise. However, there is considerable uncertainty in all of the projections, including the consequences for peak flows and sea level rise. Increases in peak flows could potentially increase rates of scour in the long-term. However, an increase in sea level would induce backwater at the site, resulting in higher water levels and lower velocities. The net effect of these changes is uncertain.

2.3 Scouring around Submerged Sills

A submerged sill is a hard structure that protrudes from the riverbed, partially obstructing the flow, resulting in local scouring in erodible materials (see Figure 2 for a conceptual diagram), which could be a potential issue for the existing tunnel in the future. Submerged sills are observed at a number of locations on the lower Fraser River, due a variety of types of infrastructure built on the riverbed, for example:

1. The trifurcation training wall: see Figure 3
2. South Surrey Interceptor (riprap-covered pipeline crossing): see Figure 4
3. Lulu Island-Delta water main (riprap-covered pipeline crossing): see Figure 5

In all of these examples, the infrastructure was built below or flush with the riverbed, not protruding into the flow. However, due to the gradual bed level lowering in the Fraser River (i.e. bed degradation described in Section 2.2), the infrastructure emerged above the riverbed and became a submerged sill. The increased projection of a sill results in stronger diving currents (downwards flows), thereby leading to deeper local scour.

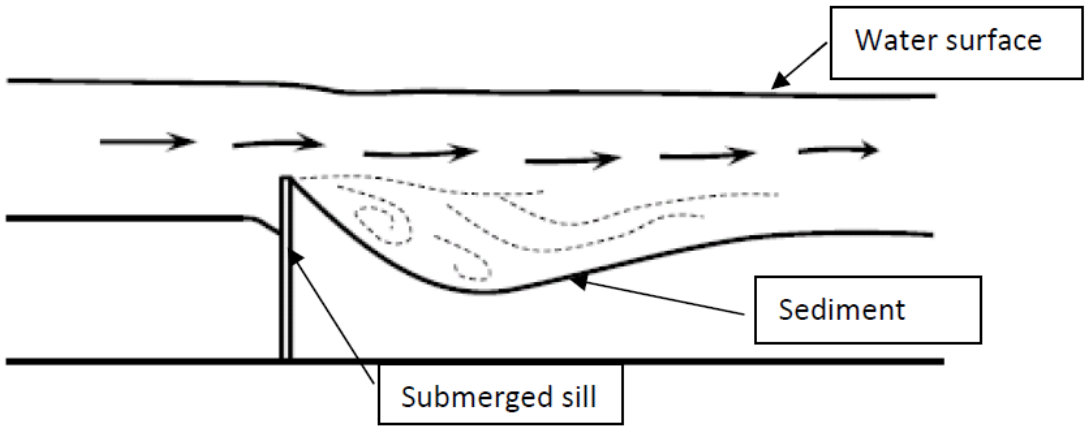


Figure 2 **Conceptual diagram of a submerged sill taken from Melville (2014) with annotations added.**

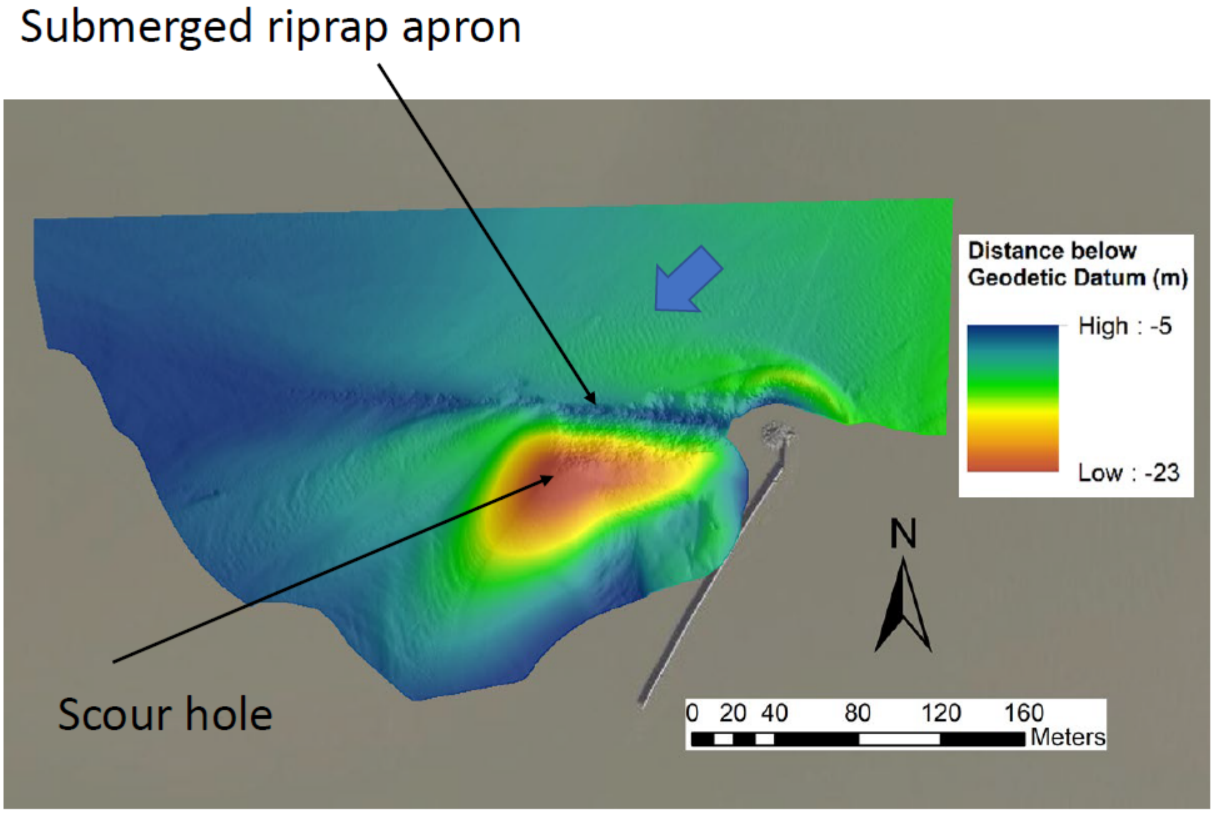


Figure 3 **Bathymetric data from Public Services and Procurement Canada at the Trifurcation Wall. During an ebb (falling tide) flow is from top to bottom.**

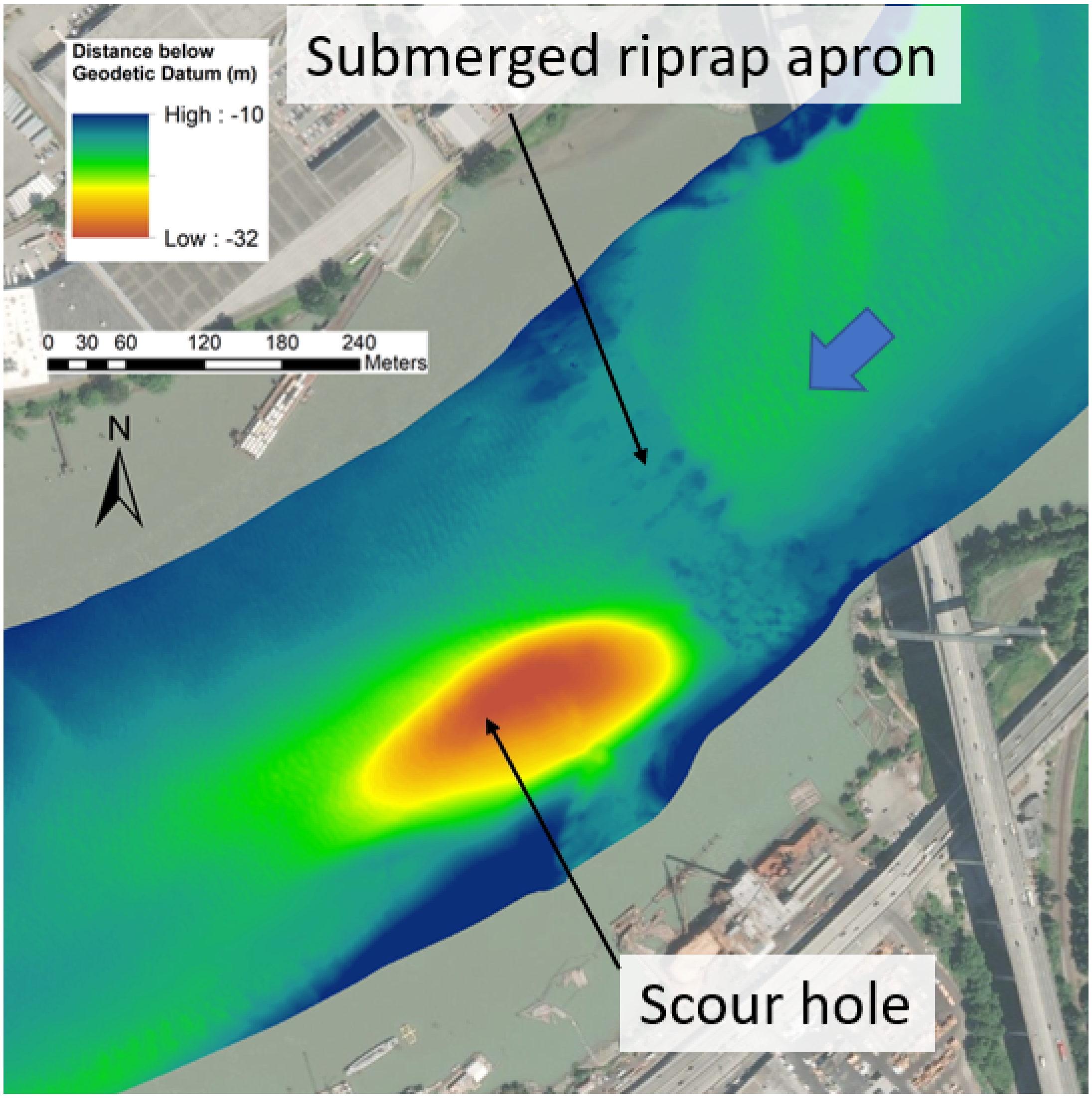


Figure 4 **Bathymetric data from Public Services and Procurement Canada at the South Surrey Interceptor. During an ebb (falling tide) flow is from top right to bottom left.**

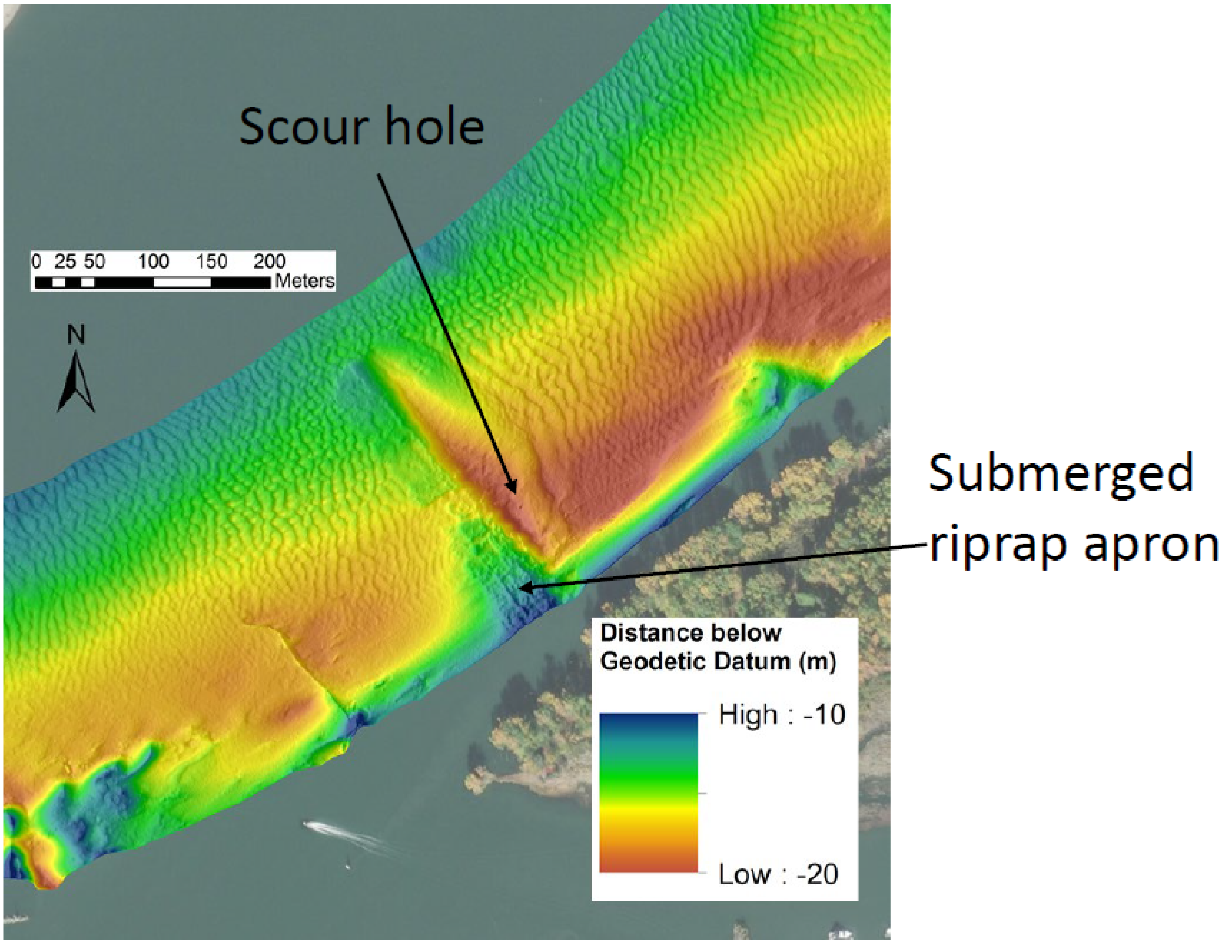


Figure 5 Bathymetric data from Public Services and Procurement Canada at the Lulu Island-Delta Water Main. During an ebb (falling tide) flow is from top right to bottom left.

2.4 Salt Wedge

Flows and sediment transport in the lower Fraser River are influenced by the presence of a salt wedge, which is common in estuarine environments. As saline water from the ocean is denser than riverine freshwater, the salt wedge remains near the riverbed below the freshwater layer. The position of the salt wedge tip moves as a function of both river discharge and variations in tide heights. The salt wedge migrates upstream in the river during flood (rising) tides, causing rapid deposition of suspended bed material as a result of reduced turbulence in the upper non-saline layer and the de-coupling of the of the river flow from the bed which prevents entrainment (Kostaschuk and Luternauer, 1989). Conversely, during ebb (falling) tides, the salt wedge recedes, moving seaward, and bed material re-suspension commences as the tip of the salt wedge passes (due to enhanced turbulence) and is sustained due to accelerating downstream flows (Kostaschuk and Luternauer, 1989).

3 SITE CONDITIONS

The existing tunnel is located on the South Arm of the Fraser River, between km 18 and km 19 (measured upstream from Sand Heads); see Figure 6. Flows at the existing tunnel are tidally affected. Full reversal of flow occurs during the winter months when tidal amplitude is larger and Fraser River flows are small. NHC (2015b) analysed the specific conductivity (function of salinity) in 2012 at the Environment and Climate Change Canada water quality buoy (BC08MH0453), which is ~ 4km upstream of the GMC, to infer the position of the salt wedge. The analysis revealed that limited saline water was detected at the buoy when flows were greater than 4,500 m^3/s at Hope.

The construction of the existing tunnel, which was completed in 1959, involved floating concrete sections into position and sinking them into a trench. According to Drawing 1540-R-04 (Appendix A), the Tunnel is 23.8 m wide and 7.3 m high. The Tunnel is protected from scour and erosion with riprap and flexible concrete mattresses. A rock apron composed of 680-kg (1,500 lb) rock covers the Tunnel. Beneath that is a layer of 227 kg (500 lb) rock. A 3.8 cm (1½”) flexible concrete mattress is keyed into the rock protection and extends upstream and downstream from the Tunnel with 227 kg (500 lb) rock overburden.

Figure 7 and Figure 8 show the bathymetry at the GMC in 2017 collected by PSPC in 2017, the bathymetry illustrates that the GMC scour protection protrudes above the bed. The protrusion of the existing tunnel above the riverbed varies over the channel width, but it is generally in the range of one to three meters, forming a submerged sill (note that the elevation of the top of the scour protection is not moving with time, and therefore this has not impacted the navigation clearance over the existing tunnel). If the riverbed in this location continues to degrade over time, the projection of the existing GMC, which causes a flow obstruction, will increase.

The Lulu Island-Delta Water Main is located approximately 600 m downstream of the GMC. Bathymetric survey data for the Lulu Island-Delta Water Main is shown in Figure 5 and partially in Figure 7; the data shows that scour occurs upstream of the Main, primarily on the left (south) side of the channel.

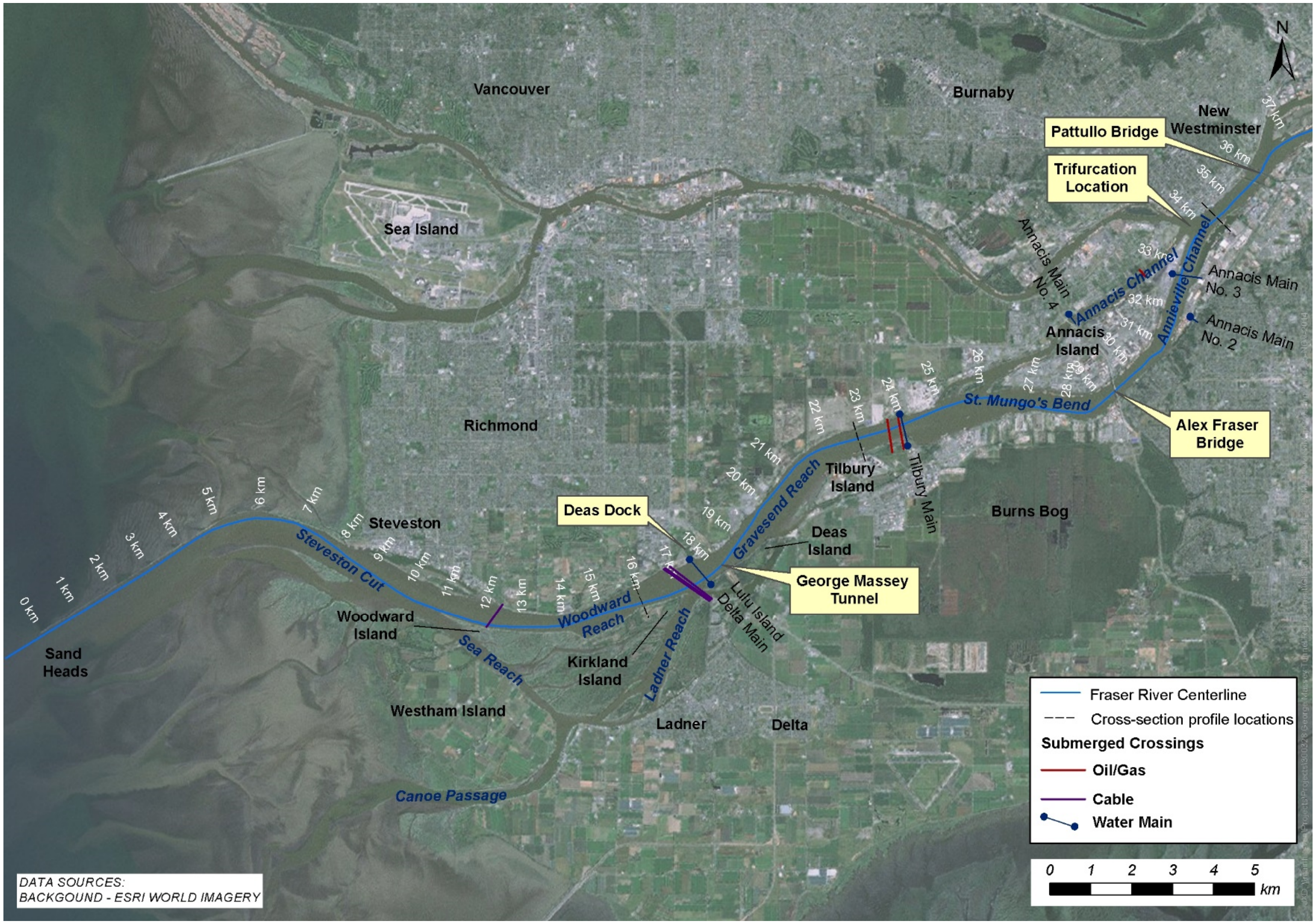


Figure 6 **Overview map for the GMC project. Map reproduced from NHC (2015b). During an ebb (falling tide) flow is from top right to bottom left.**

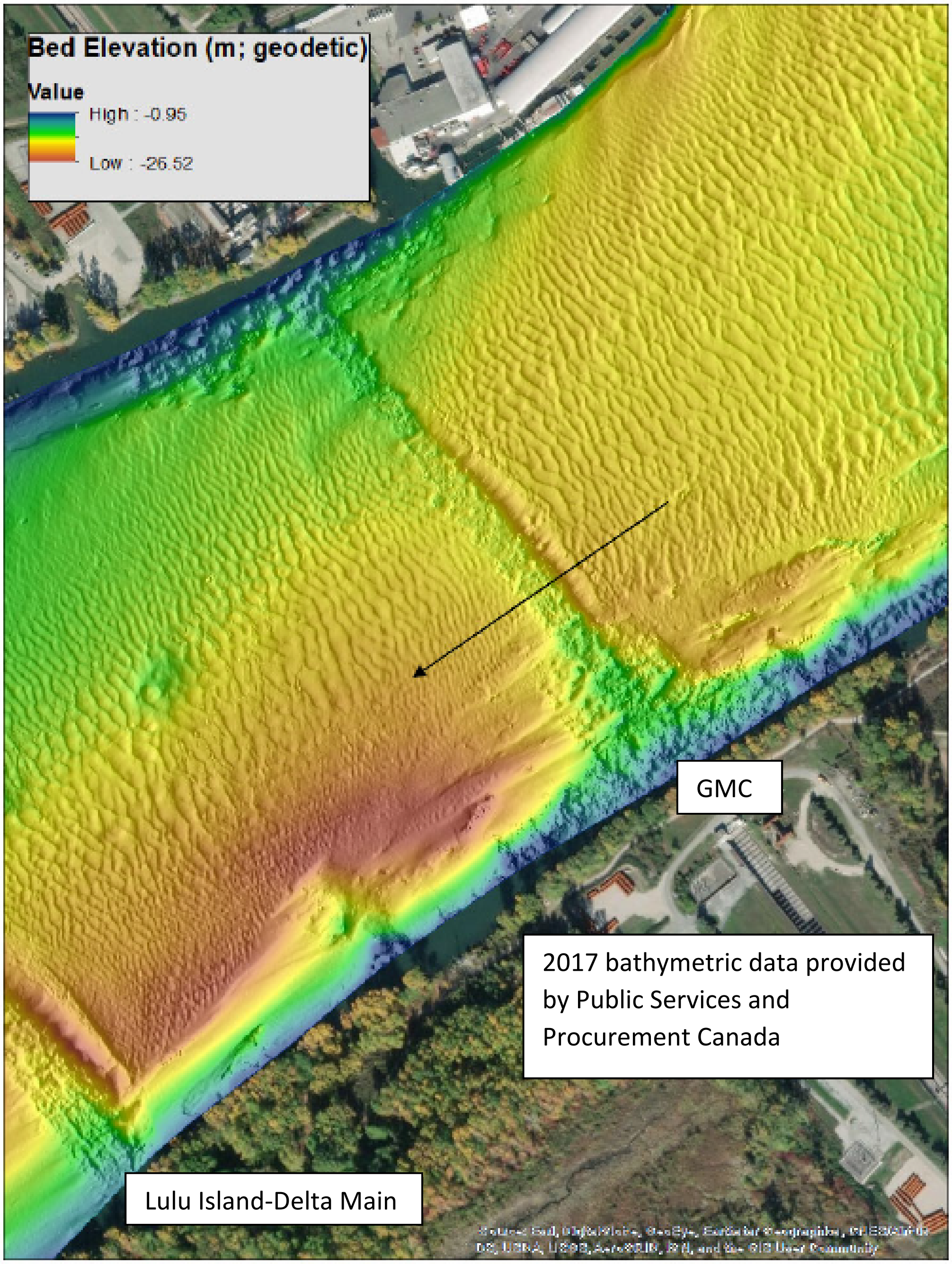


Figure 7 **Bathymetry at the George Massey Crossing, collected by Public Services and Procurement Canada in 2017. Black arrow shows location of profile cut presented in Figure 8. During an ebb (falling tide) flow is top right to bottom left.**

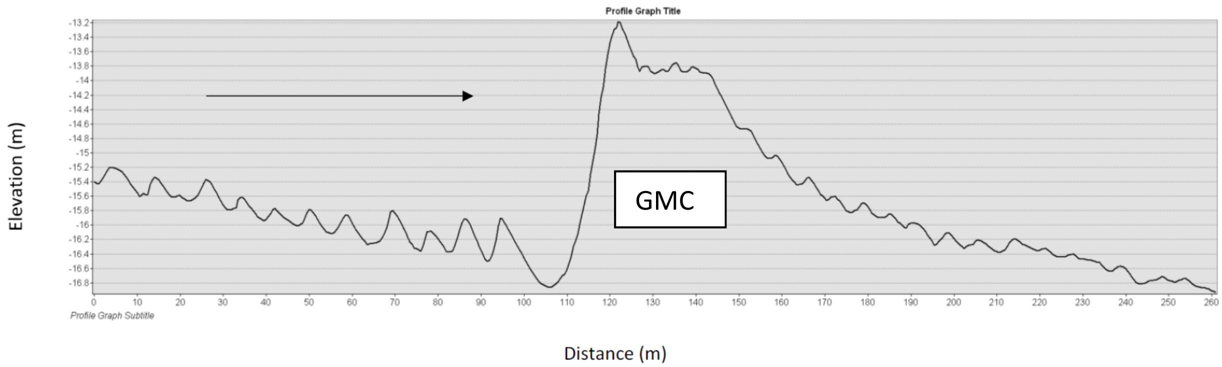


Figure 8 **Bathymetric profile over the existing George Massey Crossing. Location of cut is shown in Figure 7. During an ebb (falling tide) flow is from left to right.**

4 CFD MODELLING

CFD modelling was undertaken using FLOW-3D version 11.2 (www.flow3d.com) using the Renormalized Group (RNG) turbulence model. FLOW-3D is a three-dimensional (3D) CFD model with special capabilities for modelling complex geometries and free surface flows. NHC has considerable experience with FLOW-3D and has used it successfully in similar projects on the lower Fraser River, including the South Surrey Interceptor pipeline crossing (NHC, 2017b), the Pattullo Bridge Replacement Project and the New Westminster Rail Bridge (NHC, 2019).

4.1 Model Limitations

The following limitations of the modelling should be acknowledged:

- Future changes including sea level rise, changes to hydrograph timing and shape, sediment supply and changes to the river morphology (e.g. deepening and land reclamation) will influence hydraulics and morphology on the lower Fraser River in ways that cannot be predicted at present.
- The potential effects of riverbed degradation on future water levels was not considered as part of the preliminary analysis. Bed degradation may result in lower water levels and more severe scour conditions; these conditions should be investigated in the preliminary design phase.
- Mesh independence testing, which examines whether the computation mesh is sufficiently fine, such that it is not influencing the model results, has not been undertaken. Mesh independence was not undertaken as a result of the time constraints associated with project. Mesh independence testing should be undertaken in future phases of the project.
- Salinity was not incorporated into the CFD model, consequently, the potential effects of the salt wedge are not represented.
- Model validation has not been undertaken using field measurements. In future phases of the project, mesh validation using velocity measurements taken in the field should be undertaken.

- FLOW-3D can simulate highly unsteady flows and hence can simulate tidal flows (Vasquez and Walsh, 2009). However, in order to minimise runtime all of the simulations for this study were performed assuming steady flow with constant discharge and water level.
- Bed scour and sediment transport is not modelled.

4.2 Model Domain (Extent)

An overview of the CFD model domain (extents) can be seen in Figure 9. The model extends approximately 1.9 km upstream and downstream of the existing GMC, including Ladner Reach, Deas Slough and Deas Dock.

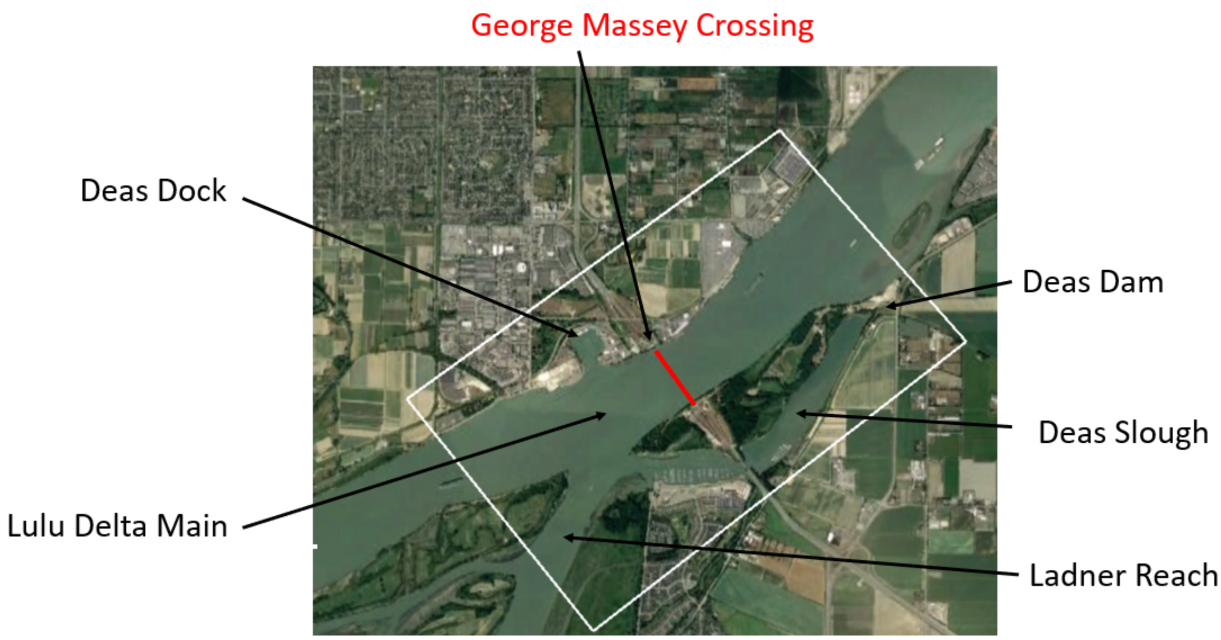


Figure 9 Model domain shown in white outline. Imagery from Google Earth. During an ebb (falling tide) flow is from top right to bottom left.

4.3 Simulated Conditions

Six model geometries were developed as part of the project, with each geometry run assuming two different flow conditions (see Table 2). Models A and D represent baseline tests, with the existing tunnel only; A represents anticipated bed conditions in 2070 and D represents existing bed conditions. Based on NHC's previous work (2017a; described in Section 2.1.2) and discussions with COWI, it was determined that 2 m of bed lowering (degradation) be assumed to represent anticipated conditions in 2070. Model B represents the existing tunnel and the proposed ITT (models B-1 and B-2 represent different riprap configurations for the proposed ITT), under anticipated bed conditions in 2070. Model C represents the existing tunnel and the proposed excavation during construction (models C-1 and C-2 represent different excavation configurations), under current bed conditions.

Table 2 Simulated Modelled Conditions

ID	Tunnel Configuration	Bed Conditions	Flow conditions
A	Existing tunnel only	50 years in future (2 m bed degradation)	500-year return period event including moderate climate change and no sea level rise
			December 2012 king tide
B-1	Existing tunnel and proposed ITT (with merged riprap)	50 years in future (2 m bed degradation)	500-year return period event including moderate climate change and no sea level rise
			December 2012 king tide
B-2	Existing tunnel and proposed ITT (without merged riprap)	50 years in future (2 m bed degradation)	500-year return period event including moderate climate change and no sea level rise
			December 2012 king tide
C-1	Existing tunnel with vertical combi-wall excavation	Existing bed conditions	August 2012 flows
			December 2012 king tide
C-2	Existing tunnel with sloped excavation	Existing bed conditions	August 2012 flows
			December 2012 king tide
D	Existing tunnel only	Existing bed conditions	August 2012 flows
			December 2012 king tide

4.3.1 Boundary Conditions

Boundary conditions (BCs) are the user-defined flow conditions imposed at the external or internal boundaries of the model. Given these imposed BCs, the CFD model computes the flow velocity and depth within the enclosed region limited by such boundaries. Typical BCs in rivers are flow discharges upstream (at the inflow) and water level downstream (at the outflow). Since the BCs are imposed by the user – and not computed by the model itself – it is recommended to locate them far from the area of interest to minimise the risk of artificially influencing the results.

Three different flow conditions were simulated in the preliminary CFD simulations (see Table 2):

1. 500-year return period event including moderate climate change and no sea level rise
2. August 2012 flows
3. December 2012 king tide

The boundary conditions associated with each of the flow conditions were exported from NHC's MIKE11 one-dimensional flow model at a 15-minute resolution (see Table 3). For flow conditions (1) and (2), discharges were imposed at the north-east boundary and water levels at the south-west boundary (see Figure 9 for boundary locations). For the December 2012 king tide simulation, discharge was imposed at the south-west boundaries as flows were travelling upstream (this is indicated by the negative sign in discharge shown in Table 3), and water level was imposed at the north-east boundary. It is critical to note that for this preliminary analysis, due to time constraints, it was assumed that water levels will remain the same over time; i.e. the potential effect of riverbed degradation on future water levels was not considered.

Table 3 Model boundary conditions

Flow Condition	Boundary Conditions	
	Discharge (m^3/s)	Water Level (m; geodetic)
500-year flow including moderate climate change and no sea level rise	21,055	0.8
August 2012 flows	11,256	-0.9
December 2012 king tide	-8,861	2.0

4.4 Model Geometry

4.4.1 Model A and D: Existing Tunnel

An overview of the Model A geometry, with the existing crossing, assuming 2 m of bed degradation is shown in Figure 10; a close-up view of the existing GMC is shown in Figure 11. In Figure 10 and Figure 11, the ground and bank topography is coloured in green, the bathymetry in brown, the GMC in yellow and the 2H:1V GMC apron upstream and downstream of the crossing in blue. The geometry of Model D is identical to that in Model A, with the exception of the bathymetric (brown) component; Model D bathymetry represents current conditions, whereas in Model A, the bathymetric component was lowered by 2 m.

The data sources for each of the aforementioned model components are detailed in Table 4.

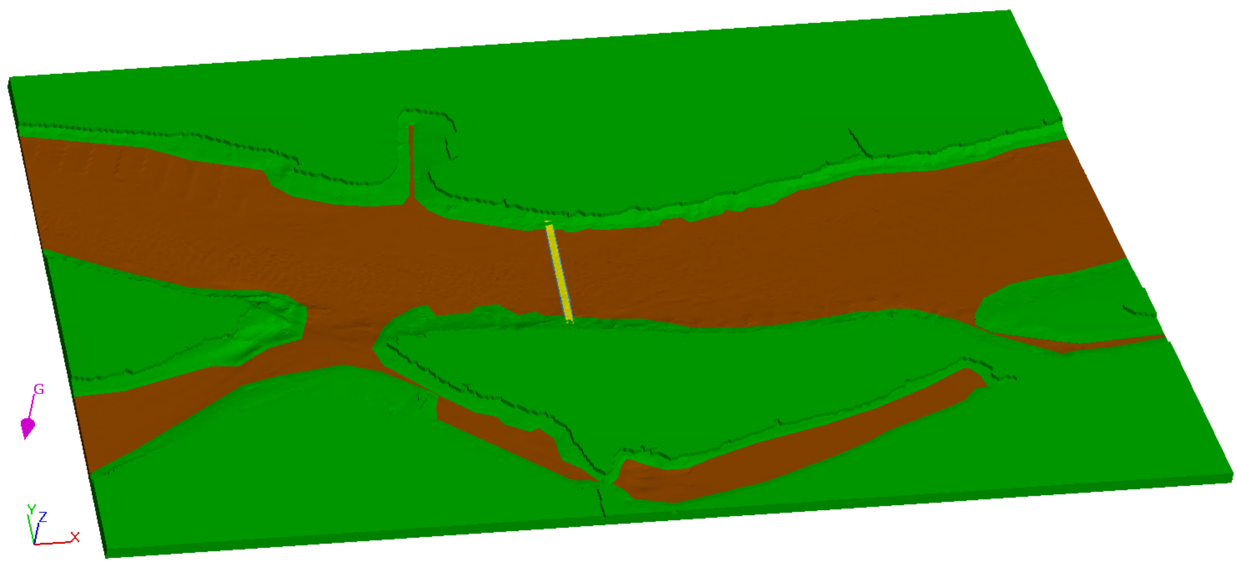


Figure 10 Overview of the Model A geometry. During an ebb (falling tide) flow is from right to left.

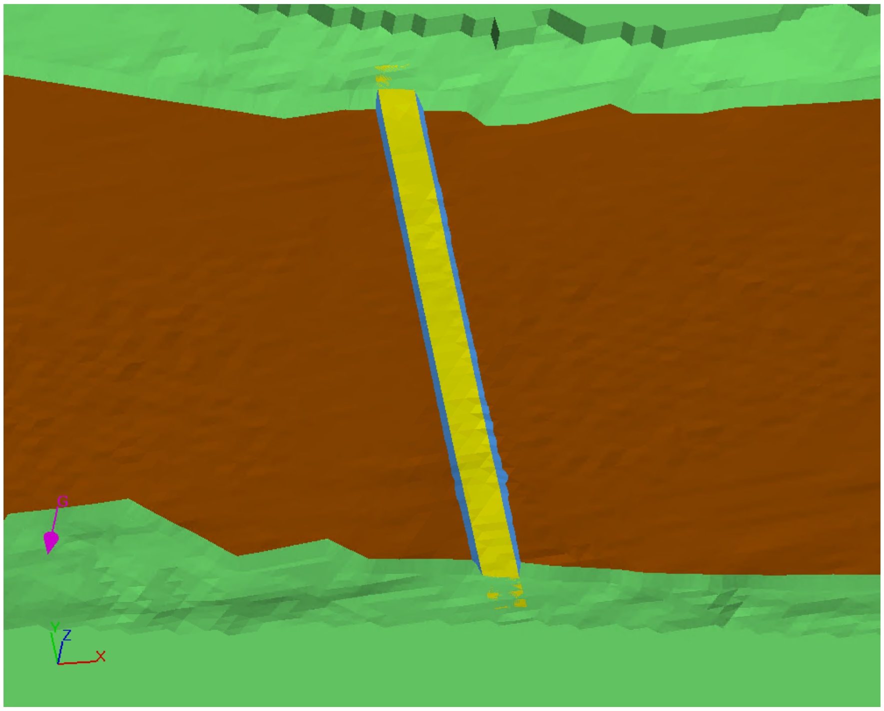


Figure 11 Existing GMC geometry in Model A. During an ebb (falling tide) flow is from right to left.

Table 4 Data sources for each of the Model A and D components

Component	Data Sources
Ground and bank topography (*green*)	Digital Elevation Model (DEM) created during NHC's 2014 GMC study (NHC, 2015b): includes data from 2004 and 2005 bathymetric surveys by Public Works and Government Services (PWGSC).
Bathymetry (*brown*)	
Existing GMC (*yellow*)	Buckland and Taylor Ltd. Drawing No. 1540-R-04 (Appendix A).
GMC 2H:1V apron (*blue*)	Public Services and Procurement Canada bathymetry collected in 2017 (Figure 7).

The DEM developed during the previous GMC project (NHC, 2015b) was used to build the model geometry for the current project; this DEM was developed using bathymetric data collected in 2004 and 2005 (as stated in Table 4). This decision was deemed to be reasonable following an assessment of the differences in bathymetry between 2017 and 2004; see Figure 12, which shows the 2004 bed levels subtracted from the 2017 bed levels.

Massey Tunnel Elevation Change (2017-2004)

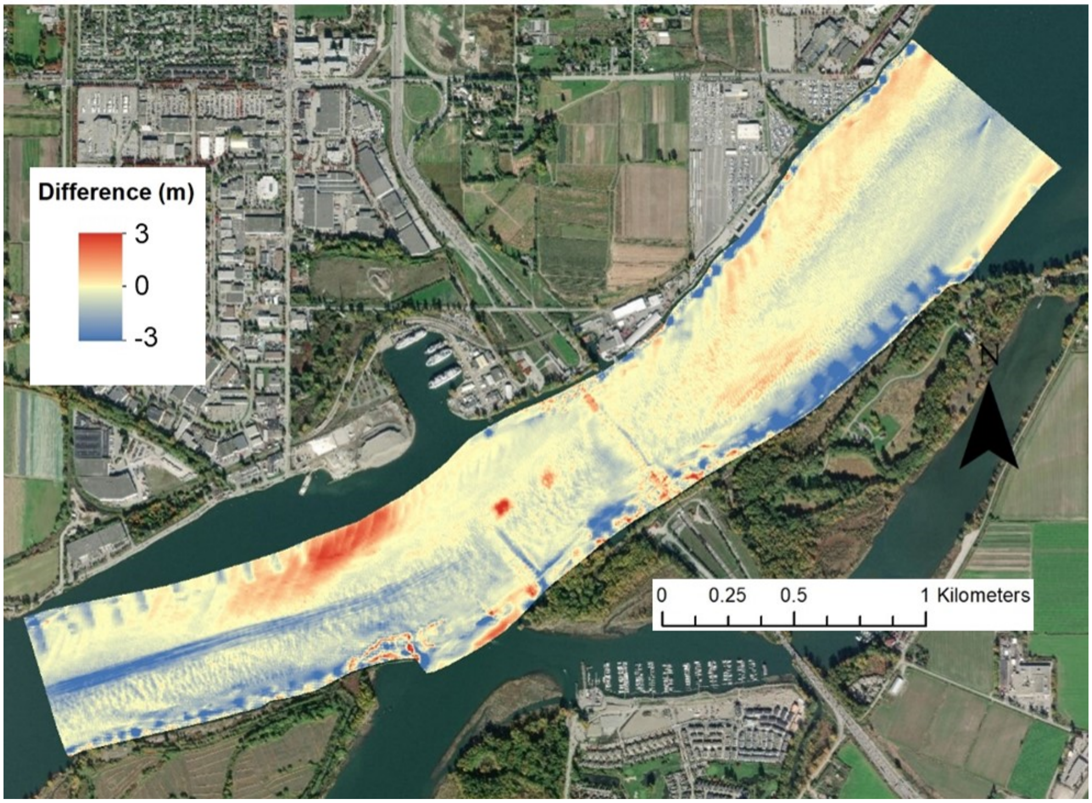


Figure 12 Difference in bed elevation between 2017 (data collected by PSPC, formerly PWGSC) and 2004 (data collected by PWGSC). During an ebb (falling tide) flow is from top right to bottom left.

The same roughness (k_s) value of 0.4 was assigned to all of the model components. A value of 0.4 m was selected based on previous calibration of a two-dimensional tidal flow model (NHC, 2007).

4.4.2 Model C: Excavation

The model geometries for the excavation simulations (Model C) were developed based on .xml files provided to NHC by Stantec. Two excavation configurations were tested:

1. Model C-1: Vertical combi-wall excavation - .xml provided to NHC on August 29, 2019
2. Model C-2: Sloped-wall excavation – xml provided to NHC on September 3, 2019

Figure 13 and Figure 15 show have been reproduced from drawings provided by Stantec, and Figure 14 and Figure 16 show the excavations as seen in the model.

Figure 13 Drawing of vertical combi-wall excavation provided by Stantec (received by NHC on August 29, 2019). During an ebb (falling tide) flow is from left to right.

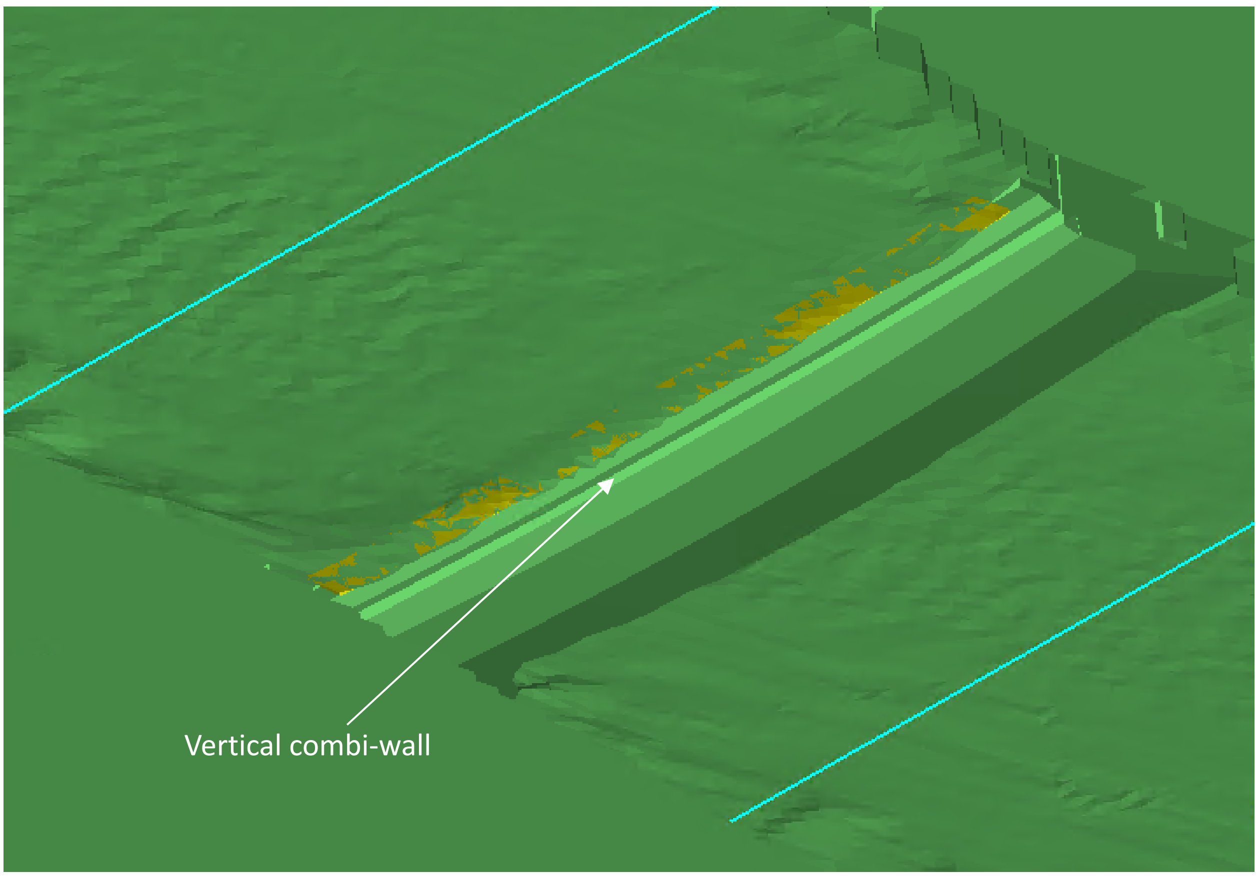


Figure 14 Model C-1: Vertical combi-wall excavation in the model. During an ebb (falling tide) flow is from bottom right to top left.

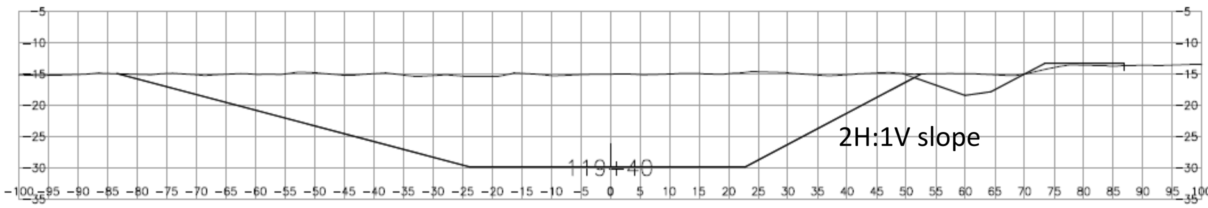


Figure 15 Drawing of 2H:1V sloping excavation provided by Stantec (received by NHC on September 3, 2019). During an ebb (falling tide) flow is from left to right.

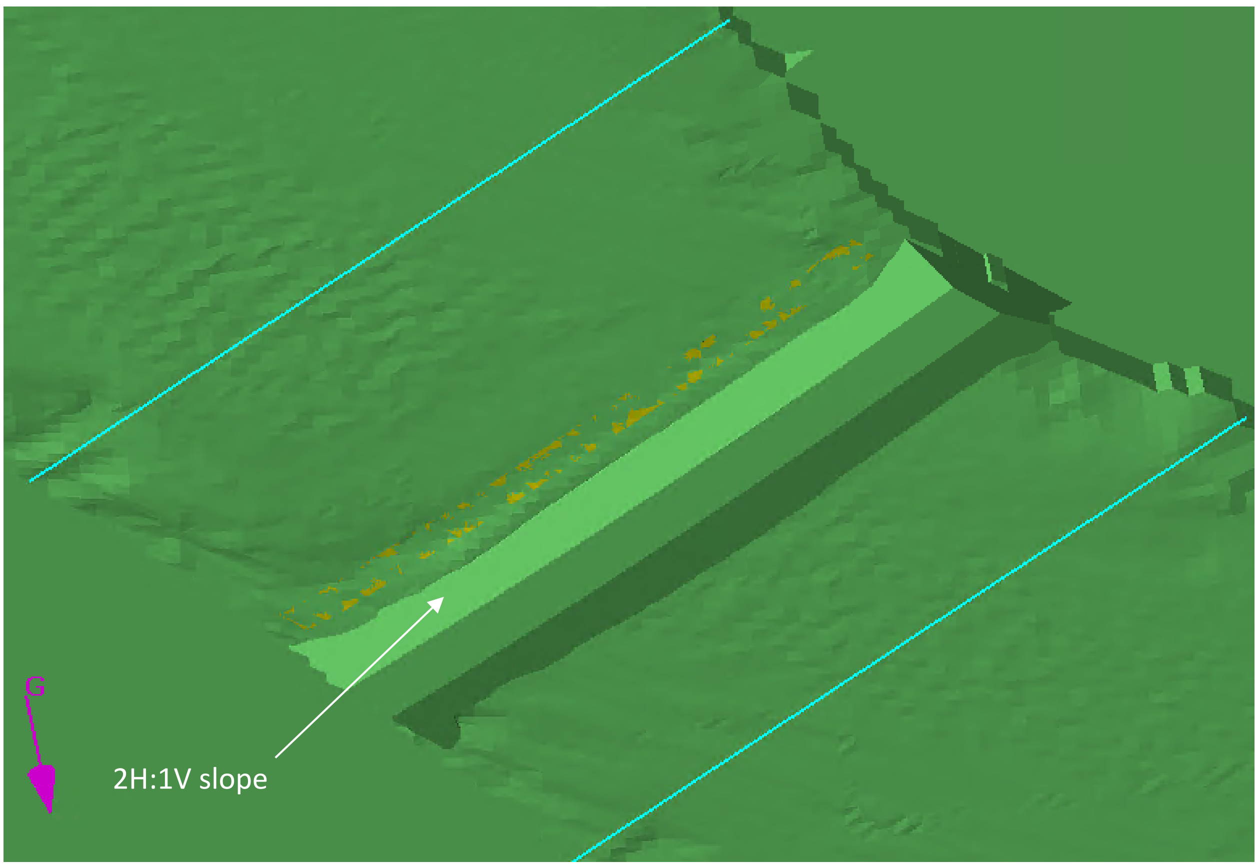


Figure 16 Model C-2: 2H:1V sloping excavation in the model. During an ebb (falling tide) flow is from bottom right to top left

4.4.3 Model B: Proposed ITT

Figure 17 and Figure 18 illustrate the model geometry for Models B-1 and B-2, respectively, representing the proposed ITT. The proposed ITT is shown in purple. The geometry of the proposed ITT was built based on the vertical combi-wall excavation xml file provided to NHC by Stantec on August 29, 2019, along with conceptual sketches of the proposed tunnel provided to NHC by COWI on August 30th, 2019. At the time of the model development, it was unknown whether the riprap would be merged between the proposed and existing tunnels; as such both configurations were tested.

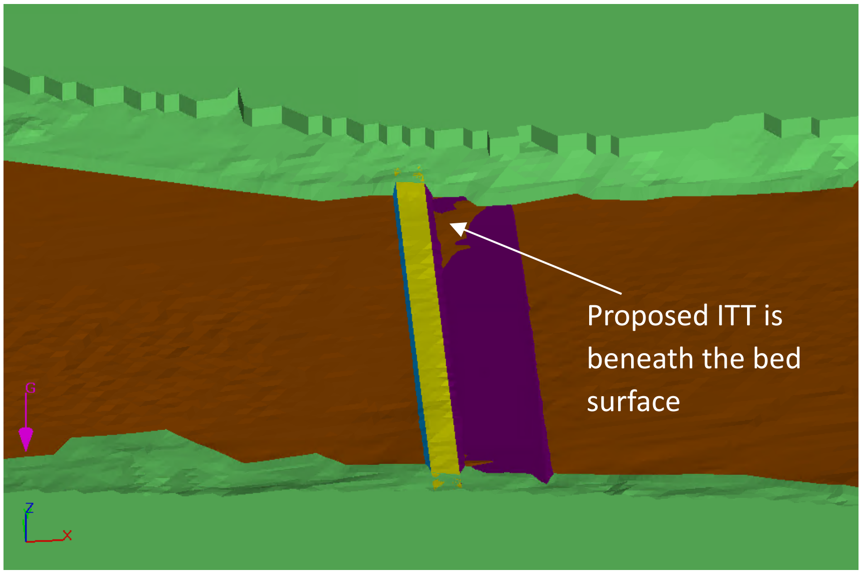


Figure 17 Model B-1 geometry: Existing tunnel and proposed ITT (with merged riprap). During an ebb (falling tide) flow is from right to left.

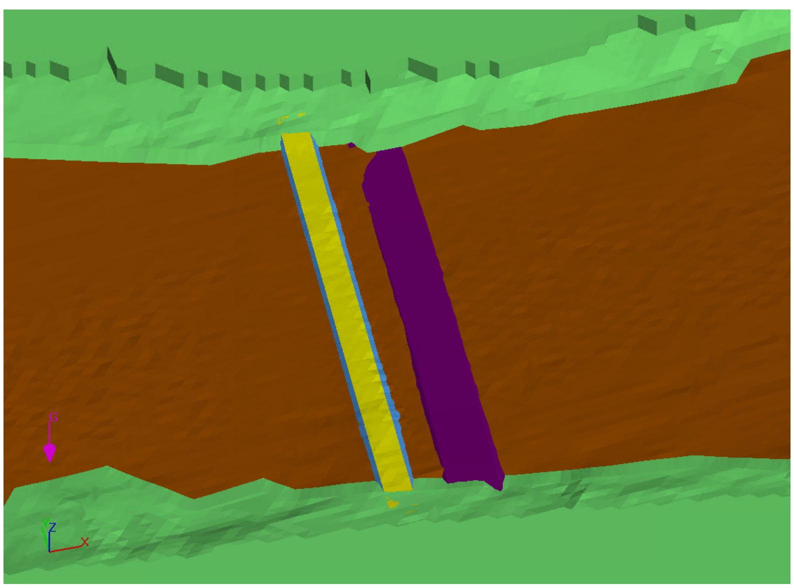


Figure 18 Model B-2 geometry: Existing tunnel and proposed ITT (without merged riprap). During an ebb (falling tide) flow is from right to left.

4.5 Model Mesh

The FLOW-3D computational mesh varied slightly between simulations but was generally formed of three mesh blocks (MBs) of varying size and resolution. Figure 19 illustrates the mesh blocks, including the cell size within each mesh block. The cells in the model are cubic; the cell size is identical in the x, y and z direction. The total number of cells exceeds 16.8 million in Models A, B and D and 23.5 million in Model C due to excavation depth.

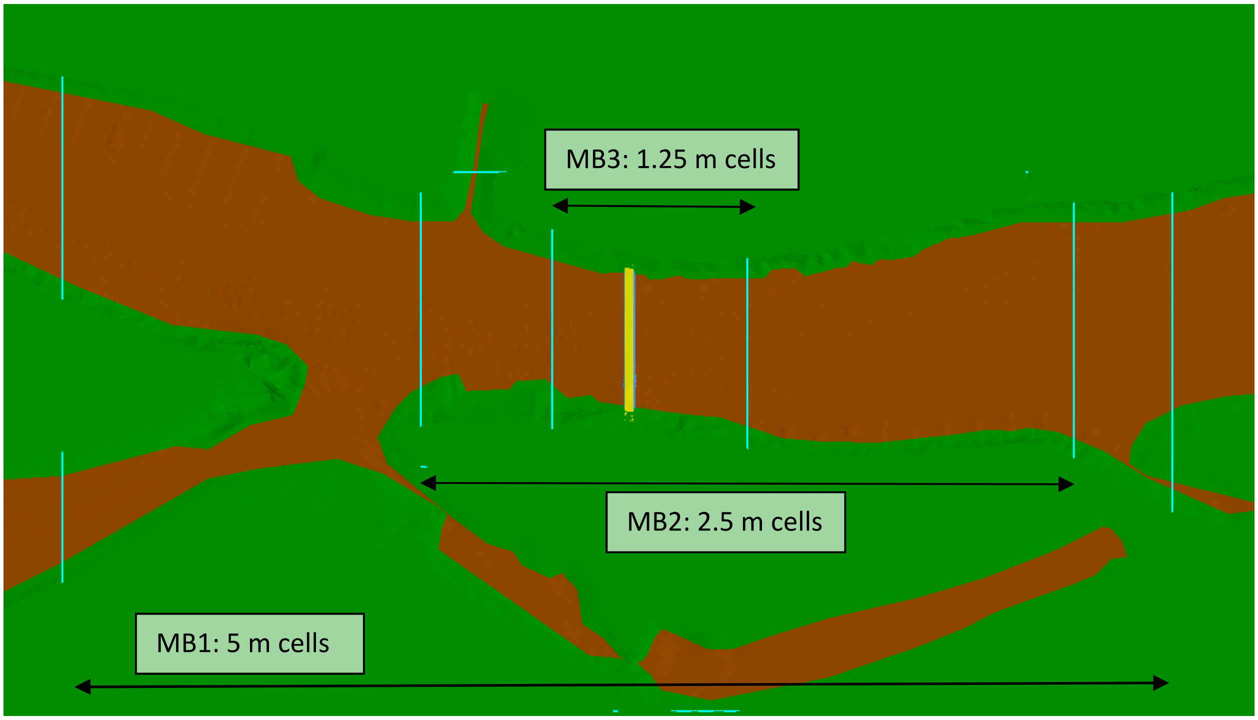


Figure 19 Mesh blocks (MBs) within the GMC CFD model, including the cell size. During an ebb (falling tide) flow is from right to left.

4.6 Results

4.6.1 Future Condition with 50 Years Bed Degradation (Models A and B)

2012 King Tide (Upstream) Flow Condition

The velocity magnitude simulated in the Model A for the 2012 King Tide flow condition is shown across the entire model domain in Figure 20; this simulation represents velocity magnitude predicted for future conditions (2 m of bed degradation) with only the existing tunnel. Figure 21 shows a 2D cross-section of the velocity magnitude in the XZ plane, at approximately the channel mid-width, simulated in Models A, B-1 and B-2. Figure 21 enables a comparison of the velocity magnitude between Models A, B-1 and B-2, thereby showing the impact upon velocity magnitude of introducing the new ITT.

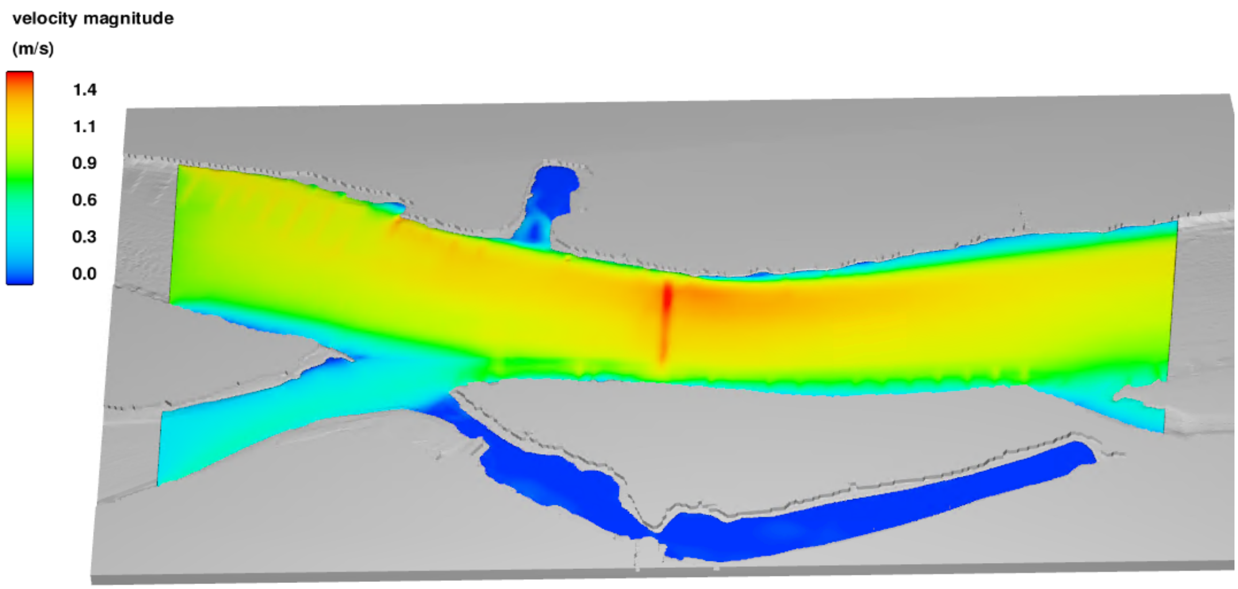


Figure 20 **Velocity magnitude simulated in Model A (existing tunnel with bed degradation) for the 2012 King Tide simulation. Flow is from left to right.**

500-Year Return Period Event Including Moderate Climate Change and No Sea Level Rise

Figure 22 shows a 2D cross-section of the velocity magnitude in a longitudinal vertical plane, at approximately the channel mid-width, simulated in Models A, B-1 and B-2 for the 500 year return period event including moderate climate change and no sea level rise flow condition. Figure 22 enables a comparison of the velocity magnitude between Models A, B-1 and B-2, thereby showing the impact upon velocity magnitude of introducing the new ITT.

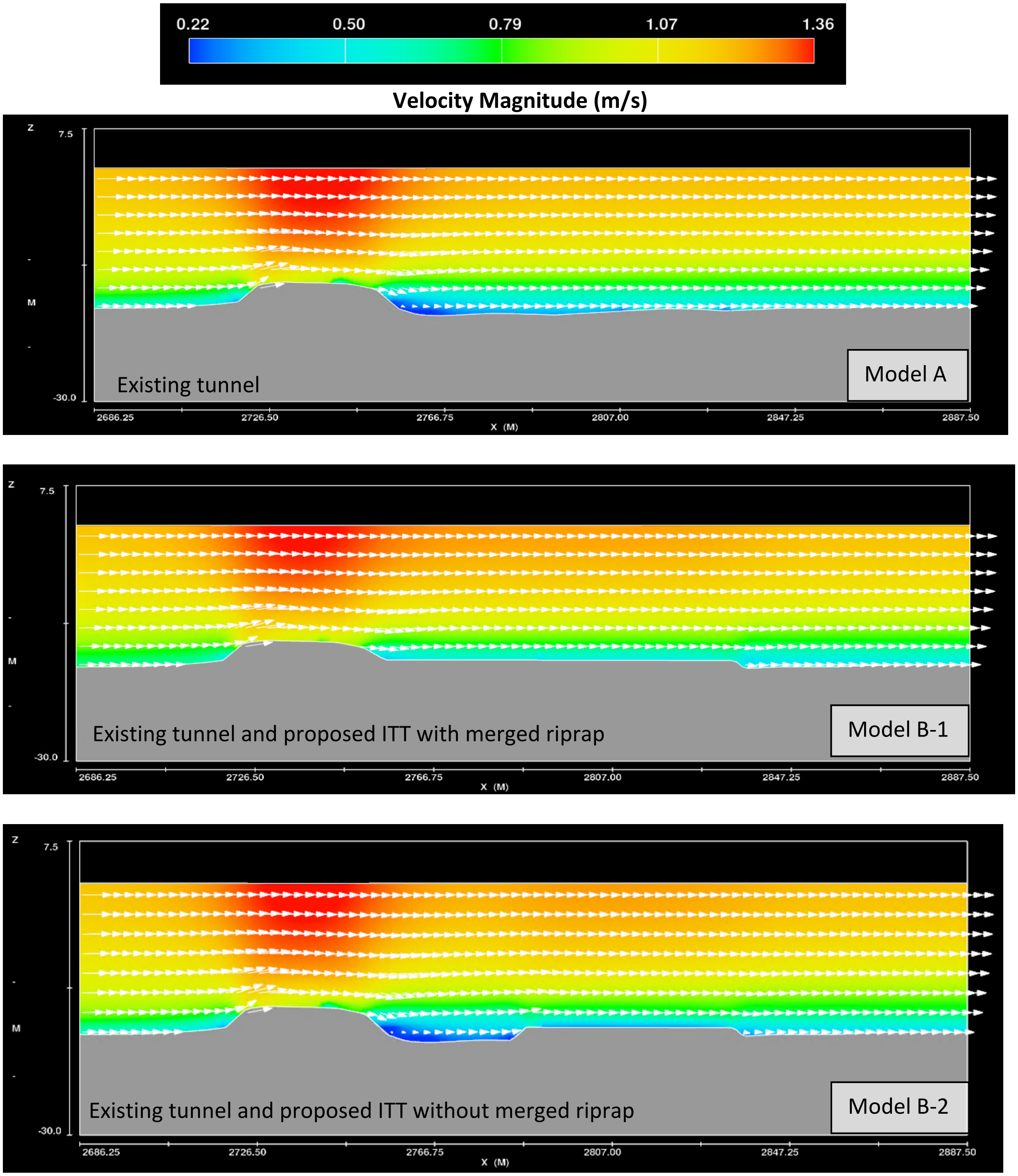


Figure 21 Velocity magnitude (m/s) simulated for the 2012 King Tide flow condition. Flow is from left to right.

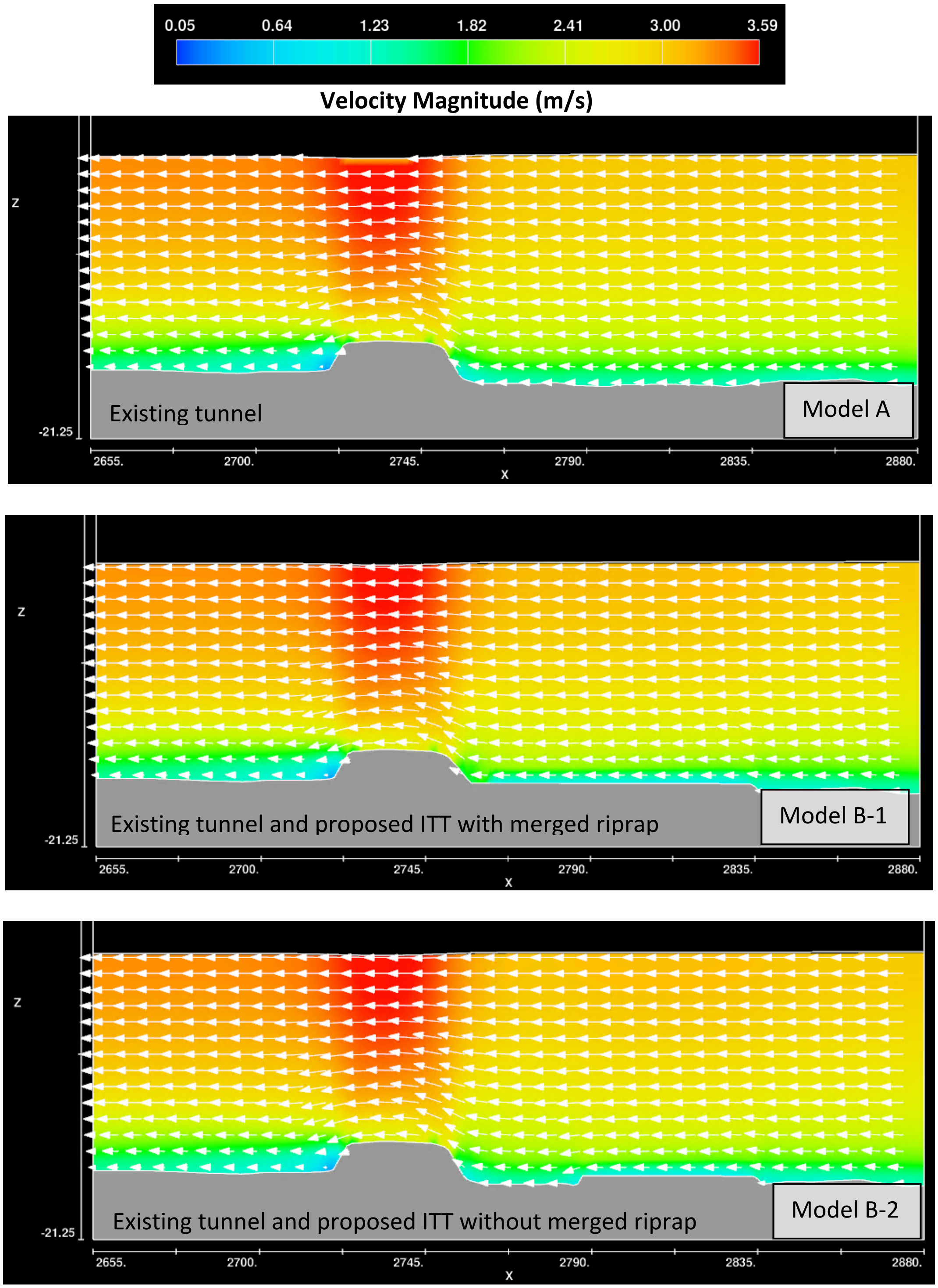


Figure 22 Velocity magnitude (m/s) simulated for the 500-year return period event including moderate climate change and no sea level rise flow condition. Flow is from right to left.

Conclusions

Based on Figure 21 and Figure 22, the following conclusions have been drawn:

- Assuming 2 m of degradation (compared to current conditions), which is realistic for bed levels in 2070, the existing tunnel will protrude above the bed surface (in excess of 4 m in some locations), creating a flow obstruction. This protrusion will lead to flow acceleration over the existing crossing (as seen in Figure 21 and Figure 22), due to the reduction in cross-sectional flow area.

- The protrusion of the existing tunnel will generate high levels of turbulence within the flow field. Due to the proximity, the proposed ITT will be subjected to this high turbulence, especially during upstream flows (rising tides), which occur regularly at this location. This high turbulence must be considered during the design of the scour protection for the proposed ITT. Larger scour protection will be required for the proposed ITT in the configuration modelled in B-1 and B-2, compared with if the existing tunnel was to be removed. Upgrades to the scour protection on the existing tunnel may also be necessary; further investigation would be needed to confirm this requirement.

- The CFD simulations have been undertaken assuming a fixed bed (i.e. erosion and deposition is not modelled). As such, the local scouring around the increasingly exposed existing GMC has not been considered within the model geometry. If general bed levels are to lower by ~ 2 m over the next 50 years, then a much more substantial reduction in bed levels should be expected locally due to the protrusion of the existing GMC (as demonstrated in Section 2.3). Methods to predict local scouring are currently under development by NHC; further investigations will enable quantitative estimations of local scour depths under future conditions at the existing GMC. This change in bed levels must be taken into account when considering the elevation of the proposed ITT and the potential for launching of the scour protection.

- Merging the riprap between the proposed ITT and the existing GMC (as modelled in B-1) results in a smoother flow profile, which is advantageous from a hydraulic perspective. Merging the riprap, however, will require a large volume of material, which will increase costs and environmental impacts.

4.6.2 Excavation (Models C and D)

2012 King Tide Flow Condition

Figure 23 and Figure 24 present the depth-averaged velocity and the surface velocity, respectively, in plan simulated in Models D (baseline), C-1 and C-2 (excavation). Please note the difference in scale on the colour bar between Figure 23 and Figure 24; the surface velocities are higher than the depth-averaged velocities. Figure 25 shows a 2D longitudinal profile of the velocity magnitude in the vertical plane at approximately the channel mid-width, simulated in Models C-1 and C-2.

August 2012 Flow Condition

Figure 26 shows a 2D cross-section of the velocity magnitude in the vertical plane, at approximately channel mid-width, simulated in Models C-1 and C-2.

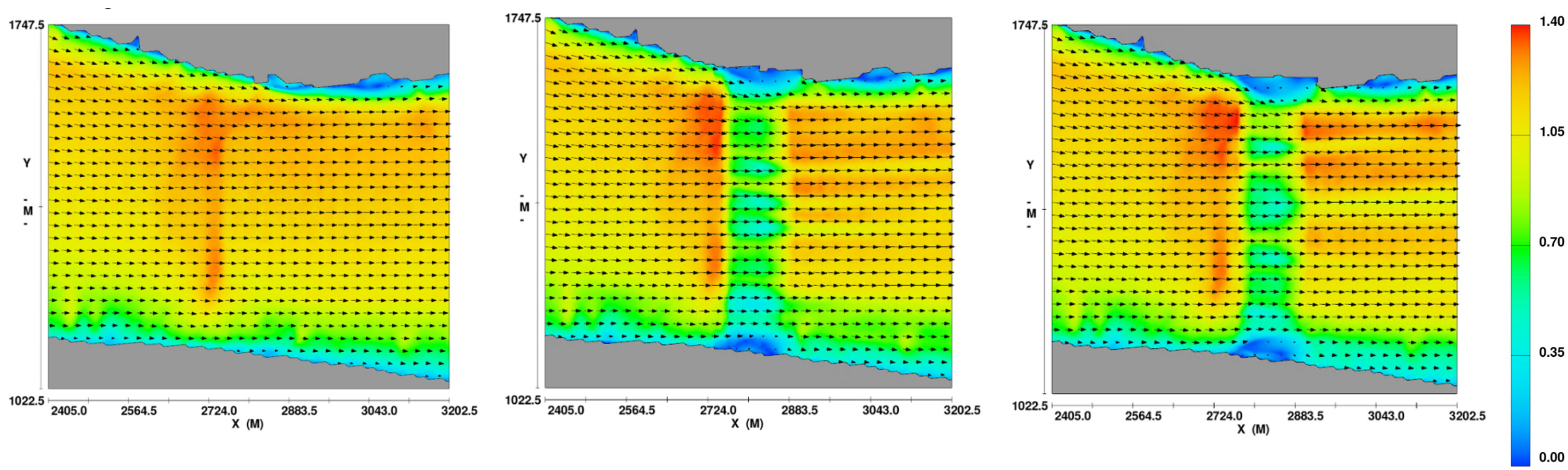
Conclusions

Compared with baseline conditions (Model D), a trench excavation (both Models C-1 and C-2) will result in:

- Flow deceleration over the deeper trench region due to the increase in flow depth and area (Figure 23). The reduction in velocities will cause sediment to deposit within the trench. Consequently, the trench will act as a sediment trap.
- A reduction in surface velocities (Figure 24), which may have consequences for navigation.
- Changes in velocity distributions around the trench (Figure 23 and Figure 24).
- The formation of back eddies at the edge of the channel within the trench, which will result in sediment deposition.
- The trench excavations result in more complex flow patterns (see velocity vectors in Figure 25 and Figure 26). These complex flow patterns will generate turbulence, which may have consequences for the scour protection over the existing crossing. Upgrades to the scour protection on the existing tunnel may be necessary; further investigation would be needed to confirm this requirement.

Comparing Model C-1 (vertical combi-wall excavation) with Model C-2 (sloping excavation), it can be seen that a sloping excavation results in a more gradual change in flow (Figure 25 and Figure 26), which is advantageous from a hydraulics perspective.

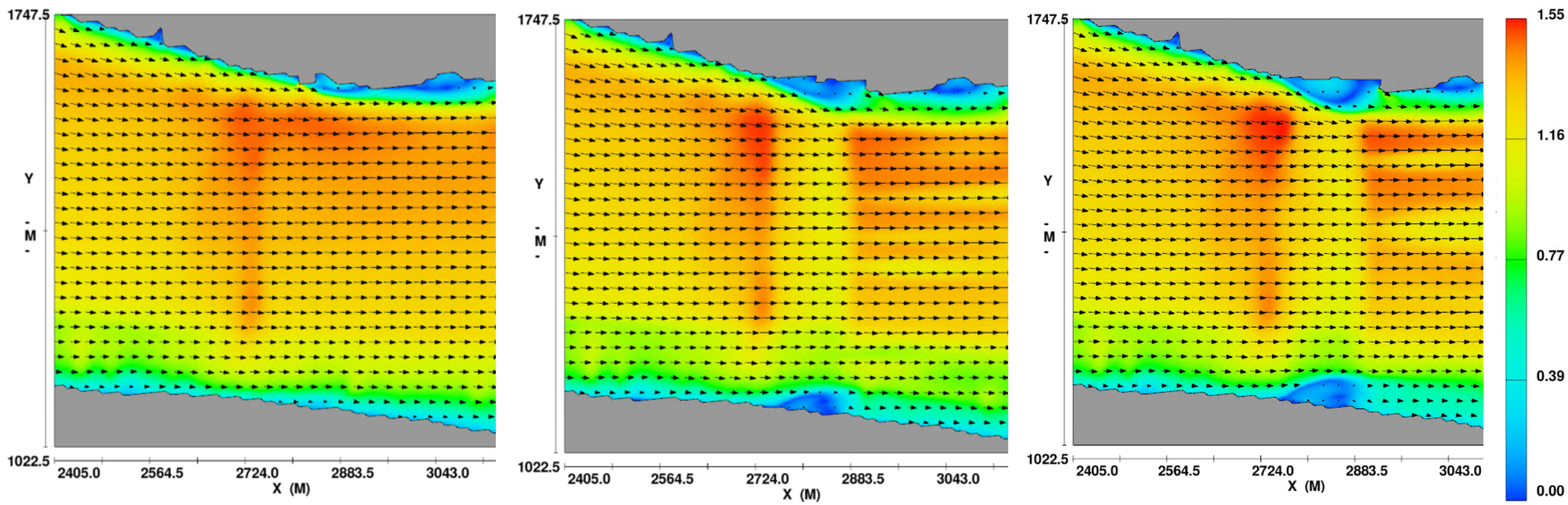
Depth-Averaged Velocity (m/s)



Model D *Model C-1: Vertical combi-wall excavation* *Model C-2: Sloping excavation*

Figure 23 **Plan View - Depth-averaged velocity (m/s) simulated during the 2012 King Tide Simulations in Models D (baseline), C-1 and C-2. Flow is from left to right.**

Surface Velocity Magnitude (m/s)



Model D *Model C-1: Vertical combi-wall excavation* *Model C-2: Sloping excavation*

Figure 24 Plan View - Surface velocity (m/s) simulated during the 2012 King Tide Simulations in Models D (baseline), C-1 and C-2. Flow is from left to right.

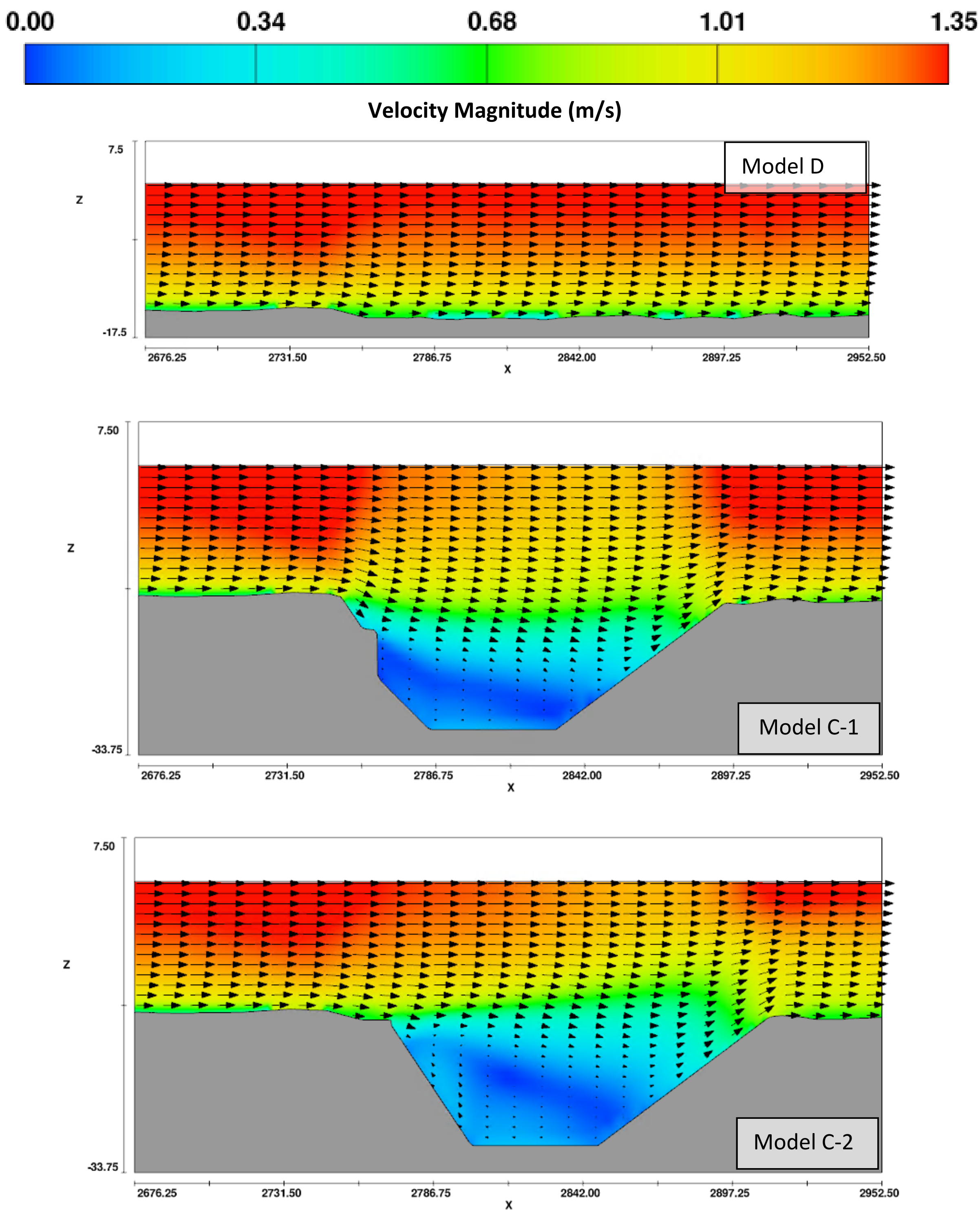


Figure 25 Longitudinal profile in vertical plane – Velocity magnitude (m/s) simulated during the 2012 King Tide Simulations in Models C-1 and C-2. Flow is from left to right.

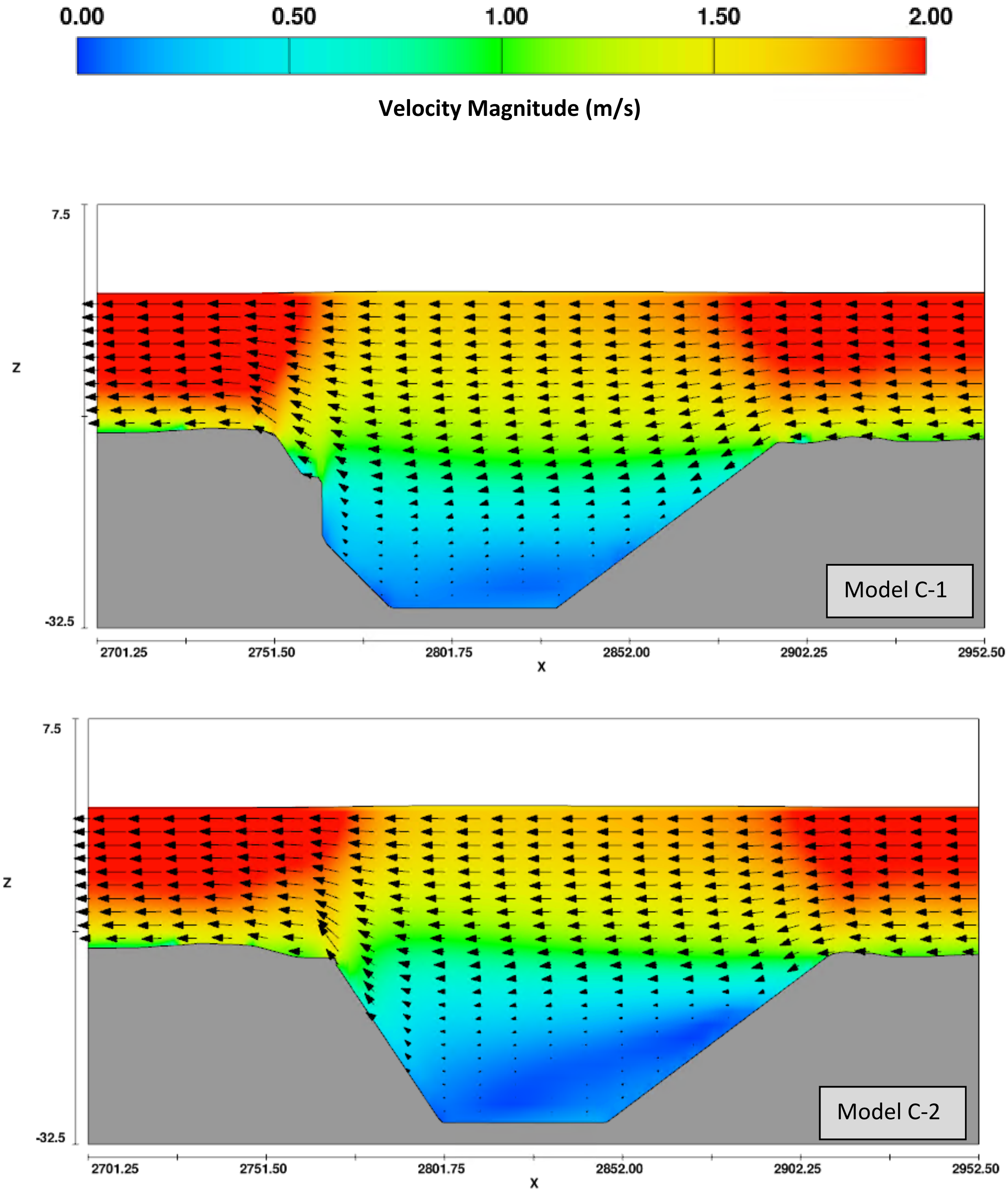


Figure 26 Longitudinal profile in vertical plane – Velocity magnitude (m/s) simulated during the 2012 August Flow Simulations in Models C-1 and C-2. Flow is from right to left.

5 DISCUSSION

5.1 Trench Migration

As part of NHC's 2014 study (2015b), numerical modelling was undertaken to examine the consequences of removing the existing GMC. The numerical modelling revealed that, as the flow decelerates over the deep trench, sediment is deposited, resulting in infilling. At the downstream edge of the trench, the model predicted that sediment would be entrained by the accelerating flow, resulting in trench migration downstream. Figure 27 has been reproduced from NHC's 2014 study and shows the downstream migration of the trench in the numerical model. When designing the construction sequencing for the proposed ITT, the propensity for trench migration should be considered.

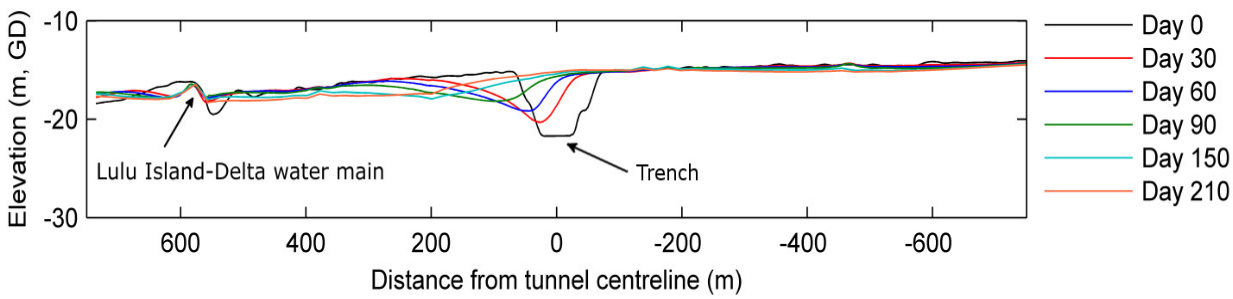


Figure 27 Downstream trench migration, predicted by NHC's (2015b) numerical model, following the removal of the existing GMC. During an ebb (falling tide) flow is from right to left.

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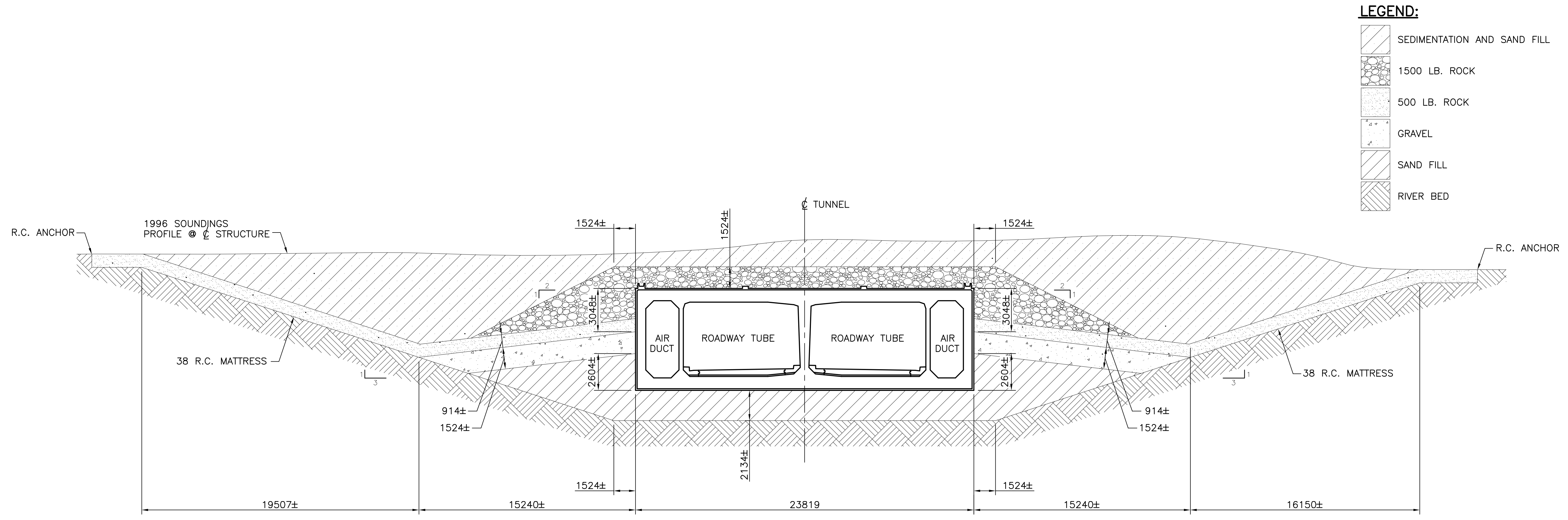
Appendix A: Drawing 1540-R-04



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MINISTRY OF TRANSPORTATION AND HIGHWAYS
SOUTH COAST REGION

LOWER MAINLAND HIGHWAY DISTRICT
GEORGE MASSEY TUNNEL NO. 1509
SEISMIC SAFETY RETROFIT AND REHABILITATION – ASSESSMENT PHASE

TUNNEL CROSS SECTION

PREPARED UNDER THE DIRECTION OF

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PROJECT No.	NEGATIVE No.	REG.	DRAWING No.
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