Approach Embankment Design Report (15%)

Causeway Pedestrian & Cyclist Bridges Project

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Causeway Link Alliance

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REV.	DATE	SECTION	DESCRIPTION
A1	08-08-2022	0000	15% Design Internal Review
A	11-08-2022	0000	15% External Design Review

	NAME	TITLE	SIGNATURE	DATE
PREPARED BY	Su Kwong Tan	Geotechnical Design Lead		08-08-2022
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Contents

1. INTRODUCTION

1.1 Project Background

The Causeway Pedestrian and Cyclist Bridge Project is an opportunity to deliver a landmark pedestrian and cyclist connection across the Swan River that responds to the unique cultural and historic significance of the area, integrates with existing landscape and urban design, and provides an attractive link for both tourists and the wider community.

The existing causeway bridge is one of only four pedestrian and cyclist crossings of the Swan River, being one of the busiest carrying approximately 1,400 cyclists and 1,900 pedestrians per day, with peak hour volumes of over 150 cyclists and 200 pedestrians. The need to improve this connection has been identified for some time, with concerns about existing shared path width, surface condition and mix of user groups generally causing safety concerns.

The new bridge will have a 3.5 m wide cycle path and a 2.5 m wide pedestrian walkway provided for separated and safer access across the Swan River for both cyclists and pedestrians independent of the road traffic. Located 80-90m downstream of the existing Causeway, this alignment was considered appropriate in terms of its ability to improve pedestrian/cyclist amenity, maintain directness and minimise impacts on flora and fauna, as well as the Swan River itself. Consisting of two cable stay bridges, the proposed option limited the number of river piers to just three, acknowledging the spiritual and cultural importance of the Swan River (Derbal Yerrigan) to Perth’s first nations peoples.

1.2 Project Location

The project is located between East Perth and Victoria Park, located within the local government authority of the City of Perth and the Town of Victoria Park.

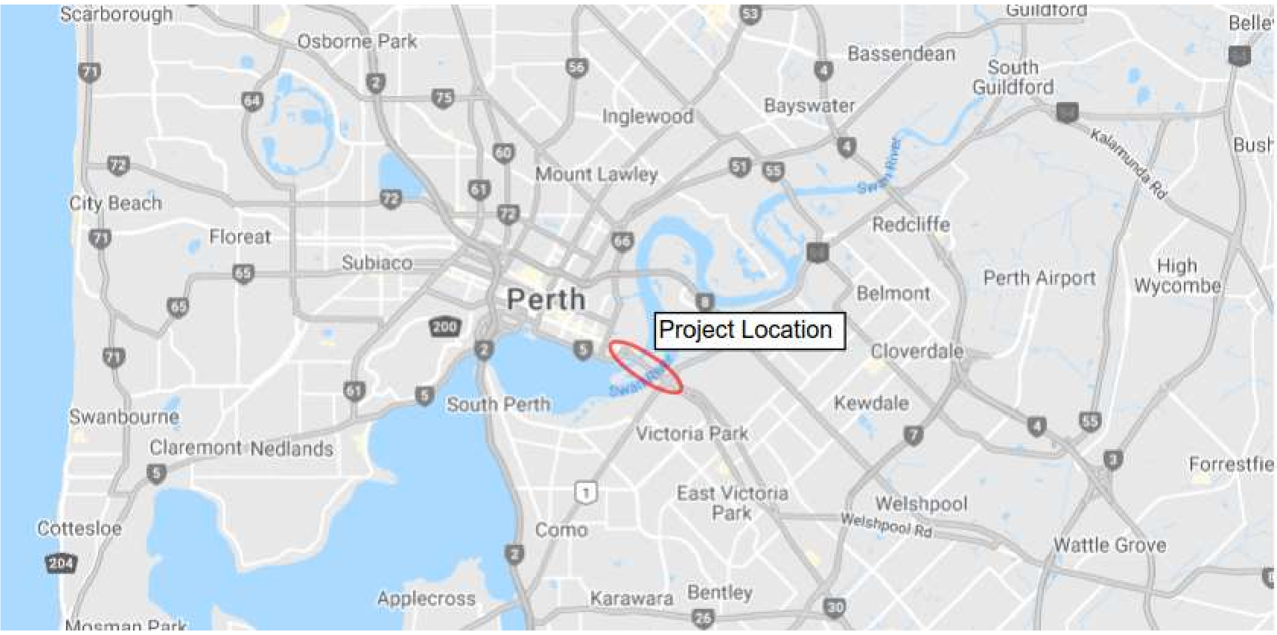


Figure 1: Project location

1.3 Purpose

The purpose of this report is to document the parameters adopted in the design, information and relevant standards used, design assumptions that may have been made, and design discussions and agreements between this consortium and the stakeholders.

This report is prepared to discuss specifically the potential options for ground improvement for the approach embankments at Point Fraser, Heirisson Island and McCallum Park. Additional design details will be included once an option is confirmed and taken forward.

It is assumed that the recipients of this Design Report have an understanding of the Project, the BDC, the SWTC and other relevant referenced documents, prior to reading this document. Therefore, this Design Report is intended to highlight design constraints, assumptions, issues and exclusions and not reiterate all information outlined within the BDC and SWTC.

2. DESIGN REQUIREMENTS

2.1 Design Input

The documents and data listed in Table 1 were relied upon during the development of this geotechnical design package.

Table 1: Design Input Documents and Data

Reference No.	Document Title	Revision
BGE (2021)	RN 1098 - Preliminary Waterways Assessment. Ref: P0181-REP-W-0001 dated 28/01/2021	0
CLA (2022)	Additional Geotechnical Investigation for Causeway Pedestrian and Cyclist Bridge – Factual Report. Ref. C301-CLA-0000-GE-REP-0003	In progress
Golder (2019)	Waterbank Development Geotechnical Interpretive Report. Ref: 137642103-017-R-Rev1	1
Golder (2021)	Compilation of Historic Geotechnical Field Investigation Data. New Causeway Pedestrian Bridges, Perth. Ref: 20391097-001-R-Rev0	0
Gordon (2003)	Sea Level Change and Paleochannels in the Perth Area. Australian Geomechanics Vol 38(4). The Engineering Geology of Perth, Part 2.	NA
Gozzard (1986)	Perth Sheet 2034 II and part 2034 III and 2134 III. Perth Metropolitan Region Environmental Geology Series. Geological Survey of Western Australia GSWA)	NA
Gozzard (2007)	The Guildford Formation Re-Evaluated. Australian Geomechanics Vol 42, pp.59-80.	NA
WSP (2021a)	Causeway Pedestrian and Cyclist Bridge Geotechnical Factual Report. Ref: PS124806-GTT-REP-001	0
WSP (2021b)	Causeway Pedestrian and Cyclist Bridge Geotechnical Interpretive Report. Ref: PS124806-GTT-REP-002	0
Youd et al (2001)	Liquefaction resistance of soils: Summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils. Journal of Geotechnical and Geoenvironmental Engineering, 127(10), 817-833	NA

2.2 Codes, Reference Documents and Regulations

The geotechnical design information presented in this report is in accordance with the Scope of Works and Technical Criteria (SWTC) and Basis of Design and Construction (BDC).

Compliance with clauses from the SWTC and BDC are presented in Table 2 below.

Table 2: Summary of Relevant Clauses from SWTC and BDC

<table>
<tr><th>Reference No.</th><th>Document Title</th><th>Compliant</th></tr>
<tr><td>AS</td><td>Australian Standard AS 5100.3 (2017) Bridge Design – Foundations and Soil Supporting Structures.</td><td>Yes</td></tr>
<tr><td>AS</td><td>Australian Standard AS 2159 (2009) – Design and Installation.</td><td>Yes</td></tr>
<tr><td>AS</td><td>Australian Standard AS 1170.4 (2007) _R(2018) – Structural Design Actions – Part 4: Earthquake Actions in Australia</td><td>Yes</td></tr>
<tr><td>BDC Cl 4.2.1</td><td>Settlement and movement limits of paths must be as in the following table. Paths must be designed and constructed so that gradients take into account expected settlement and movement without exceeding grade limits during the life of paths.
<table>
<tr><th>Area</th><th>Time Period After Practical Completion</th><th>Vertical Settlement (mm)</th><th>Horizontal Displacement (mm)</th><th>Differential Settlement over any Distance</th></tr>
<tr><td rowspan="2">Paths</td><td>5 Years</td><td>200</td><td>100</td><td>1:100</td></tr>
<tr><td>40 Years</td><td>400</td><td>200</td><td>1:100</td></tr>
</table>
New loading near the existing abutment must be eliminated or limited to a nominal amount to avoid damaging the existing bridge and/or abutment.</td><td>Yes</td></tr>
<tr><td>BDC Cl 5.14</td><td>Steel tube piles for the footbridges, if used, must be completely filled with reinforced concrete.</td><td>Yes</td></tr>
<tr><td>BDC Cl 9.3</td><td>Pile driving activities must comply with LGA and statutory requirements and environmental and development approval conditions. [Refer clause 9.4(j)].</td><td>Yes</td></tr>
<tr><td>BDC Cl 9.5</td><td>Causeways will only be permitted if approved by the relevant statutory authorities.
The participants will be responsible for gaining all approvals for the placement of temporary piers or supporting structures or scaffolds in the Swan River.
Any river sediments excavated and brought to the surface must not be replaced in the river floodplain or disposed of in the river. This includes any dredging material and any material from mucking out of piles. River sediments may be treated appropriately and used as fill material (subject to geotechnical requirements) outside of the river floodplain or they must be disposed of appropriately to a landfill facility.</td><td>Yes</td></tr>
<tr><td>SWTC Cl 3.2</td><td>The various components of the Project Works must have a minimum design life in accordance with Table 3.1.
<table>
<tr><th>Component</th><th>Minimum design life</th></tr>
<tr><td>bridge works (including underpasses)</td><td>100 years</td></tr>
<tr><td>retaining walls</td><td>100 years</td></tr>
</table></td><td>Yes</td></tr>
<tr><td>SWTC Clause 3.8 (a) (i)</td><td>During construction, the Contractor must carry out sufficient monitoring to ensure that settlement and movement criteria are in accordance with the measures adopted for the construction of the Project Works.</td><td>Yes</td></tr>
<tr><td>SWTC Clause 3.8 (c)</td><td>The maximum settlement of any new structure must be limited to the amount specified by the designer in accordance with Bridge Code Part 3.</td><td>Yes</td></tr>
<tr><td>SWTC Clause 4.4 (d) (viii)</td><td>Bridge Code 2 – 22 Construction Forces and Effects
For the design of incrementally launched prestressed concrete bridges, the following standard must apply during the construction stage:
(E) differential settlement – as specified by designer (must be monitored and controlled during construction) but in no circumstances more than 25 mm</td><td>Yes</td></tr>
</table>

SWTC Clause 4.4 (e) (i)	Bridge Code 3 - 6 Piled Foundations Continuous Flight Auger (CFA) piles will not be permitted. (Note: only under special circumstances will CFA piles be allowed, and only with the approval of PE(Structures) or SES. If so, the following clauses must apply otherwise they must be deleted.) Continuous Flight Auger (CFA) piles will not be permitted: (A) at Bridge No (list bridges – allow lesser bridges to have CFAs) (B) in cohesive soils, silts or soil profiles with layers of coarse gravels or larger particles, except where excavation of an uncased hole near vibration sensitive Services is not possible; (C) where the concrete exposure classification is more severe than B1 in accordance with Section 5 of the Bridge Code; (D) where a socket in rock of a better quality than highly weathered is required; (E) where a rock socket longer than 300 mm is required; (F) where raked piles are required; (G) where the soil profile is complex with hard layers over soft layers; or (H) for end bearing piles, where the bearing stratum is on a slope steeper than one vertical to four horizontal.	Yes
SWTC Clause 4.4 (e) (ii)	CFA piles must be constructed in accordance with Main Roads' Specification 814 Continuous Flight Auger Piles.	NA
SWTC Clause 9.4 (k) (ii)	The Contractor must limit ground vibrations in adjoining properties by ensuring that the ground particle velocities from any necessary operation of vibratory compaction or percussion equipment cause minimum nuisance and do not exceed any such limit that could result in damage to property, and at most 5 mm/s	Yes in general, but 5 mm/s may be exceeded over short intervals during dynamic pile load testing (only at some pile locations).
SWTC Clause 9.4 (k) (iii)	The Contractor must seek to minimise the effects of vibrations in adjoining properties through the use of non-vibrating or lower vibrating construction methodologies or by operating plant as far away as practicable from those properties	Yes
SWTC Clause 9.4 (m) (iv)	All costs associated with damage caused to existing roadway surfaces, structures, Services, buildings and other surface and sub-surface features as a result of any construction activity must be met by the Contractor.	Yes

3. DESIGN DEVELOPMENT

3.1 Changes from Previous Revision

This section will be updated with each successive submission to demonstrate design progress and how any issues, or stakeholder requirements have been addressed and resolved.

3.1.1 Design Development – Tender to 15%

No significant changes have been made since the tender design phase. It is however noted that the chainage system has been updated. For the purpose of this report, the chainage presented in this report refers to the chainage system as per tender design. The updated chainage system will be adopted in the 85% design stage.

3.1.2 Design Development – 15% to 85%

This section will be developed following completion of the 85% design development.

3.1.3 Design Development – 85% to 100%

This section will be developed following completion of the 100% design development

3.2 Issues, Risks and Non-compliances

3.2.1 Design Criteria Non-compliances

None identified at this stage.

3.2.2 Outstanding Design Issues

The following issues will be addressed in the next phase of the design:

- Results of additional site investigation will be incorporated.
- DBCA's requests:
  - To address how the bridge abutments/landings will integrate with the adjoining foreshore reserves at Point Fraser, Heirisson Island and McCallum Park and
  - To consider piled foundations instead of earth batters/berms for a better visual amenity outcome and use of the foreshore area. DBCA has requested both options be evaluated from a cost-based perspective and if the proposal to have piled foundations for all the bridge landings cannot be done due to cost limitations, can a combination of both methods be considered.
  - Any earth batters for the bridge landings will need to be vegetated (as outlined in the preliminary drawings) to stabilise the batters and assist with improving the visual amenity of the bridge structure.
  - Investigations to confirm there will not be any disruption to groundwater movement or or expression of groundwater from pre loading or the primary path's embankment by the geotechnical team. Any mobilised groundwater will need to be treated.
- Value engineering using lightweight Expanded Polystyrene (EPS) in lieu of Controlled Modulus Columns (CMCs) design is being carried out for the approach embankment Point Fraser Abutment 2 and McCallum Park Bridge Abutments 1 and 2. Confirmation of final ground improvement solutions for approach embankments will be provided in the next design stage.
- Potential impacts on existing infrastructure and proposed paths due to heave from embankment settlement.
- Impact of embankment settlement on the following structures and services:
  - City of Perth Irrigation at Point Fraser
  - Existing Causeway Bridge at Point Fraser
  - Existing structure (About Bike Hire) at Point Fraser
  - Water main at Point Fraser
  - Town of Victoria Park Irrigation at McCallum Park
  - Telstra lines at McCallum Park.

4. DESIGN INTEGRATION

4.1 Alignment

Alignment design will be documented in the 15% Civil and Drainage Design Report C301-CLA-CI-REP-00001.

4.2 Structures

Details of the bridge structural design are provided in the relevant 15% structural reports listed below:

- C301-CLA-1000-ST-REP-00001 Lot STR01 – McCallum Park Bridge Design Report
- C301-CLA-2000-ST-REP-00002 Lot STR02 – Point Fraser Bridge Design Report

5. SITE INVESTIGATION

A previous site investigation (Phase 1) was carried out adjacent to and along the proposed bridge alignment with results detailed in the reports summarised in WSP (2021a). An additional (Phase 2) geotechnical investigation was carried out between 23 June 2022 and 15 July 2022 to inform the detailed design of the CPCB project and fill in gaps in the previous data. The scope of work included investigation of geotechnical and hydrogeological conditions along the alignment of the proposed CPCB with the details presented in the Geotechnical Factual Report (C301-CLA-0000-GE-00003), which is currently under preparation.

The interpretation of the latest results will be incorporated in the next revision of this report once available.

The locations of previous and additional site investigations are shown in Appendix A.

6. SITE CONDITIONS

6.1 Site History

The proposed CPCB will be situated on reclaimed land at Point Fraser, McCallum Park and Heirisson Island which have been modified and reclaimed through the following activities at various times:

- Point Fraser: Reclamation by placement of uncontrolled fill, sometime between 1953 and 1961.
- Heirisson Island: Reclamation of parts of the river by placement of uncontrolled fill including industrial and domestic landfill and clean sand, sometime before 1953.
- McCallum Park: Reclamation by placement of uncontrolled fill, sometime before 1953.

6.2 Geological Setting

This section of the report describes the topography along the CPCB alignment and prevailing geological conditions in both a regional and local context. The character, distribution and depositional history of the geological units outlined in Section **Error! Reference source not found.** are described in detail in G ordon (2003), Gozzard (2007) and Gordon (2012).

6.2.1 Topography

Based on available Landgate ground contour information, the CPCB alignment is typically characterised as flat lying. Elevations along the alignment vary relative to the Australian Height Datum (AHD) as follows:

- Point Fraser and McCallum Park: elevations typically vary between RL 0.4 m and 2.0 m AHD.
- Heirisson Island: elevations typically vary between RL 0.4 m and 3.0 m AHD.

6.2.2 Geomorphology

The Geological Survey of Western Australia (GSWA) Perth Sheet 2034 II, part of 2034 III and 2134 III (Gozzard, 1986), 1:50,000 scale environmental geology series map indicates that the CPCB alignment is situated within a geomorphological domain that is characterised by a gently undulating surfaces associated with a river floodplain and undifferentiated river terraces with slope angles that range between 0° and 3°.

6.2.3 Regional Geology

The Swan Coastal Plain is the surface expression of the Perth Basin, which contains sedimentary rocks and soils of Mesozoic Age (Jurassic to Early Cretaceous Age) which are widespread and represent filling of the basin. Erosion of the Cretaceous Age sediments during the Late Tertiary (Pliocene Age) created a planar unconformity surface on which Pliocene Age to recent superficial deposits were laid down in marine, alluvial and aeolian environments.

Rivers that cross the Swan Coastal Plain have formed a network of channels in the Pliocene and recent superficial formations and older underlying Mesozoic sedimentary rocks, filling the channels with a complex suite of granular and cohesive soils. These channels are referred to in the published literature as paleochannels.

At the Causeway site, Gordon, (2012) suggests that two paleochannels are present in a broad valley that was created by erosion of the underlying Kings Park and Osborne Formations (Mullaloo Sandstone and Kardinya Shale Members respectively) by an ancestral Swan River. The paleochannels are assigned names i.e., Channel 2 (Last Glacial) and Channel 6 (Penultimate Glacial), which correspond to two separate drops in global sea levels that are believed to have occurred sometime around 20,000 and 150,000 years ago respectively. Gordon, (2012) also suggests that Channel 2, situated below the East Perth side of the Swan River, is infilled with superficial deposits that have been assigned to the Swan River Formation. The thalweg for Channel 2 (i.e., line of lowest elevation within a channel) is believed to be located at a depth of around 26 m, however this remains speculative at present.

Similarly, Gordon, (2012) suggests that Channel 6, located below the Victoria Park side of the Swan River, is infilled with superficial deposits that are now assigned to the Perth Formation. The thalweg for Channel 6 is believed to be located at a depth of around 35 m, however this remains speculative at present.

6.2.4 Site Geology

Historical records and information from historic and recent ground investigations indicates that a superficial layer of uncontrolled fill, derived from landfill and reclamation activities is present at the Causeway site i.e., at Point Fraser, Heirisson Island and McCallum Park. Available GI information indicates that the thickness of fill varies along the project alignment e.g., at Heirisson Island the fill is up to 6 m thick. At Point Fraser and McCallum Park, fill is up to 4.5 and 1.5 m in thickness respectively.

Gozzard, (1986) indicates that the superficial layer of uncontrolled fill is underlain by geological Unit C1 (unit terminology adopted in Gozzard, 1986) which is described as "*mid to dark grey, soft, saturated CLAY of alluvial origin, with prominent 0.2 m thick oyster shell bed near the surface*." Geological Unit C1 is inferred to be the Swan River Alluvium (SRA), which is referred to as Sulphurous Silt and Holocene Alluvium in Gordon, (2003) and Gozzard, (2007) respectively.

Available GI information (WSP, 2021a and 2021b) and information derived from this GI indicates that the SRA (Unit 2 of this study) is around 15 m thick at Point Fraser and decreases in thickness along the CPCB project alignment to the southern end of Heirisson Island, where it appears to be absent. The SRA is believed to be underlain by Pleistocene Alluvium, i.e., soils that are now assigned to the Swan River Formation (formerly categorised as the Guildford Formation in the literature). The Swan River Formation (Unit 3 of this study) may also be underlain by soils belonging to the Perth Formation (Unit 4 of this study); however the thickness and spatial distribution of both the Swan River and Perth Formations along the CPCB alignment is uncertain at present. Soils belonging to the Perth Formation may be restricted in occurrence to the Victoria Park channel of the Swan River.

The unconsolidated soils of Holocene and Pleistocene Age (Units 2 to 4 of this study) are in turn underlain by unconsolidated and partly lithified sediments assigned to the Palaeocene-Eocene Age Mullaloo Sandstone Member (Unit 5 of this study) of the Kings Park Formation. Information from boreholes undertaken at McCallum Park suggests that the fill and superficial deposits at McCallum Park may be underlain by the Kardinya Shale Member (Unit 6 of this study), which is assigned to the Osborne Formation.

A summary of the project specific geological units and lithological descriptions developed from published geological information and available GI information adopted for the CPCB project are presented in **Error! Reference source not found.**. Reference should be made to WSP, (2021b) for a d etailed description of the thicknesses, distribution, and extent of geological units along the alignment of the CPCB.

Table 3: Summary of Project Specific Geological Units

Unit Name	Unit ID	Lithological Description	Depositional Environment
Uncontrolled Fill	Unit 1 Fill	Designated as uncontrolled fill (Sand fill, possible domestic waste). (Filling records unavailable).	Deposited / placed in recent history
Swan River Alluvium	Unit 2 SRA	Typically comprises SAND and CLAY: mid to dark grey, black, blue, soft, saturated, prominent 0.2 m thick oyster shell bed near surface of alluvial origin. (Described in Gozzard, (2007) as Holocene Alluvium and as Black Sulphurous Silt in Baker, (1954).	Fluvial / Estuarine
Swan River Formation	Unit 3 SRF (Formerly Guildford Formation)	Typically comprises clean, coarse, grey sands and conglomerates to red, brown, yellow, and black clays and occasional shell beds. (Unit 3 may unconformably overlie the Kings Park Formation and may also be restricted in occurrence to the East Perth side of Swan River Channel)	Fluvial / Estuarine
Perth Formation	Unit 4 PF (Formerly Guildford Formation)	Typically comprises medium dense sands, interbedded with stiff clay lenses and some gravel layers. (Unit 4 may unconformably overlie the Kings Park Formation and may also be restricted in occurrence to the Victoria Park side of Swan River Channel).	Fluvial / Estuarine
Mullaloo Sandstone Member (Kings Park Formation)	Unit 5 MSM (KPF) (Sub-units 5a – 5c)	Typically comprises unconsolidated and partly lithified soils and rocks. Mullaloo Sandstone Member: Poorly sorted, fine to very coarse grained, pale brownish green, slightly glauconitic and clayey sand. (Unit 5 believed to be incised into the siltstones and shales of the Kings Park Formation)	Fluvial / Estuarine / Marine
Kardinya Shale Member (Osborne Formation)	Unit 6 KSM	Typically comprises Interbedded siltstones and shales. (Unit 6 may be restricted in occurrence to McCallum Park)	Fluvial / Estuarine / Marine

6.3 Groundwater

6.3.1 Aquifer

The superficial formations (comprising the sedimentary materials above the Kings Park Formation) are generally considered to be a single aquifer – the Superficial Aquifer. However, the low permeability layers (SRA and clayey lenses within the PF (PFc) and SRF (SRFc)) typically act as confining layers which restrict the movement of groundwater between the different sandy units, resulting in several different sub-aquifers within the Superficial Aquifer.

Error! Reference source not found. summarises typical sequence of aquifers and aquitards in the a rea.

Table 4: Summary of Hydrogeological Conditions

Geological Unit	Hydraulic Unit	Comments
FILL	Upper Aquifer	Main groundwater unit below site, connected to the Swan River. Where the top of the SRA is higher than the Swan River level, perched groundwater may occur.
SRA, PFc, SRFc	Aquitards	Clay units with low permeability
SRF, PF, MSM	Lower Aquifers	The aquifers are indirectly connected to the Swan River
KSM	Aquitard	At this site considered an aquitard and lower boundary of the Superficial Aquifer.

The Upper Aquifer is recharged by direct rainfall, stormwater runoff at the site as well as the Swan River. The main factor that will affect the groundwater level behavior and the Design Groundwater Level (DGWL) for the project is the Swan River levels.

6.3.2 Groundwater Levels

The Perth Groundwater Atlas, which shows the inferred historical maximum groundwater level (1m contour intervals) does not show contours over the site but indicates that the historical maximum groundwater level is less than RL 2.0 m AHD. The 1997 Groundwater Atlas also indicates that the estimated river flood level at Heirisson Island is RL 1.5 m AHD (unknown Annual Exceedance Probability (AEP). BGE 2021 has estimated that the surface water level at the proposed bridges for a 1 in 100 Yr AEP is RL 2.3 m AHD (BGE, 2021)

Due to the proximity of the site to the Swan River, the groundwater levels in the Upper Aquifer would mainly be governed by the Swan River and thereby affected by flood events, seasonal fluctuations and tidal fluctuations. The tidal influence will depend on the distance to the river, which would need to be further investigated through groundwater level monitoring, if required. Based on our experience in the area we would expect the groundwater level fluctuations in the Upper Aquifer could be up to 1 m.

The groundwater levels during geotechnical drilling during the investigation phases in June and July 2022 indicated groundwater levels (one-off measurement) ranged between approximately RL 0 m AHD and RL 0.85 m AHD. It is noted that the observed measurements are one-off measurements, which may not have fully stabilised and that range in levels could reflect geographical locations of boreholes and tidal effects. Long-term groundwater level monitoring would be required across the site to better understand the groundwater behaviour and changes (daily and seasonal).

Further assessment of the flood level and design groundwater level will be carried out during subsequent design stages, which needs to consider potential sea level rise.

6.4 Preliminary Design Subsurface Profiles

Sub-surface profiles at the proposed abutment and pylon locations are based on the available CPT and geotechnical borehole data within the vicinity of each structure location. These profiles are presented in Table 5 to

Table 10. The design subsurface profiles will be reviewed and updated accordingly in the next revision taking into consideration the results of additional geotechnical investigation.

Table 5: Design Sub-surface Profile for Point Fraser Northern Approach Embankment (CH 45 to CH 126)

Elevation (m AHD)		Profile based on CPB-CPT01, CPB-CPT03, CPB-CPT05, CPB-BH01
From	To	
2.5	-3.5	Unit 1 Fill

-3.5	-7	Unit 2 SRA (above RL -7 m AHD)
-7	-17.5	Unit 2 SRA (below RL -7 m AHD)
-17.5	-28	Unit 3 SRF/Unit 4 PF (clay), Stiff to Very Stiff
Below -28		Unit 5c KPF Rock

Table 6: Design Sub-surface Profile for Point Fraser Southern Approach Embankment (CH 375 to CH 425)

Elevation (m AHD)		Profile based on CPB-CPT11, CPB-CPT12, CPB-BH06
From	To	
1.5	-2	Unit 1 Fill
-2	-7	Unit 2 SRA (above RL -7 m AHD)
-7	-7.5	Unit 2 SRA (below RL -7 m AHD)
-7.5	-24.5	Unit 3 SRF/Unit 4 PF (Sand), Loose to Medium Dense
Below -24.5		Unit 5b KPF Marine

Table 7: Design Sub-surface Profile for Point Fraser Southern Approach Embankment (CH 425 to CH 475)

Elevation (m AHD)		Profile based on CPB-CPT13, CPB-CPT14
From	To	
2	-1.5	Unit 1 Fill
-1.5	-7	Unit 2 SRA (above RL -7 m AHD)
-7	-24	Unit 3 SRF/Unit 4 PF (Sand), Loose to Medium Dense
Below -24		Unit 5b KPF Marine

Table 8: Design Sub-surface Profile for Point Fraser Southern Approach Embankment (CH 475 to CH 570)

Elevation (m AHD)		Profile based on CPB-CPT15, CPB-CPT18, CPB-CPT19, CPB-CPT20, CPB-CPT21, CPB-BH07
From	To	
3	-1.5	Unit 1 Fill
-1.5	-7	Unit 2 SRA (above RL -7 m AHD)
-7	-9.5	Unit 2 SRA (below RL -7 m AHD)
-9.5	-26.5	Unit 3 SRF/Unit 4 PF (Sand), Loose to Medium Dense
Below -26.5		Unit 5b KPF Marine

Table 9: Design Sub-surface Profile for McCallum Park Northern Approach Embankment (CH 570 to CH 585)

Elevation (m AHD)		Profile based on CPB-CPT22, CPB-BH08
From	To	
1.5	-0.5	UNIT 1 FILL
-0.5	-7.0	Unit 2 SRA (above RL -7 m AHD)

Elevation (m AHD)		Profile based on CPB-CPT22, CPB-BH08
From	To	
-7.0	-8.0	Unit 2 SRA (below RL -7 m AHD)
-8.0	-13.0	Unit 3 SRF/Unit 4 PF (Sand), Loose to Medium Dense
-13.0	-22.0	Unit 3 SRF/Unit 4 PF (Clay), Very Stiff
-22.0	-23.5	Unit 3 SRF/Unit 4 PF (Sand), Dense
-23.5	-25.0	Unit 3 SRF/Unit 4 PF (Clay), Very Stiff
-25.0	-27.0	Unit 3 SRF/Unit 4 PF (Sand), Dense
Below -27.0		Unit 5b KPF Marine

Table 10: Design Sub-surface Profile for McCallum Park Southern Approach Embankment (CH 930 to CH 975)

Elevation (m AHD)		Profile based on CPB-CPT24, CPB-CPT28, CPB-BH13, CPB-BH14
From	To	
1.5	0.5	Unit 1 FILL (UF)
0.5	-3.5	Unit 2 SRA (above RL -7 m AHD)
-3.5	-7.5	Unit 3 SRF/Unit 4 PF (Clay), Stiff to Very Stiff
-7.5	-9.5	Unit 3 SRF/Unit 4 PF (Sand), Medium Dense
-9.5	-27	Unit 3 SRF/Unit 4 PF (Clay), Very Stiff
-27	-32	Unit 3 SRF/Unit 4 PF (Sand), Dense
Below -32		Unit 5b KPF Terrestrial

Table 11: Design Sub-surface Profile for McCallum Park Southern Approach Embankment (CH 975 to CH 1000)

Elevation (m AHD)		Profile based on CPB-CPT27, CPB-CPT28, CPB-CPT29
From	To	
1.5	0.5	Unit 1 FILL (UF)
0.5	-0.5	Unit 2 SRA (above RL -7 m AHD)
-0.5	-4.0	Unit 3 SRF/Unit 4 PF (Sand), Medium Dense
-4.0	-18.0	Unit 3 SRF/Unit 4 PF (Clay), Very Stiff to Hard
-18.0	-33.0	Unit 3 SRF/Unit 4 PF (Silt), Stiff to Very Stiff
Below -33		Unit 5b KPF Terrestrial

6.5 Geotechnical Design Parameters

Preliminary geotechnical design parameters based on available test results from the Phase 1 geotechnical investigation are summarised in Table 12 and Table 13.

Table 12: Preliminary Geotechnical Design Parameters

Soil Type	γ' (kN/m^3)	S_u (kPa)	Φ'(°)	E' (MPa)	υ'
Unit 1 Fill	18	-	32	20	0.25
Unit 2 SRA	15	20 to 30	-	Refer to Table 13 for consolidation parameters	-
Unit 3 SRF / Unit 4 PF (Sand)	17 - 19	-	33 to 36	30 to 90	0.25
Unit 3 SRF / Unit 4 PF (Clay)	18 - 19	50 to 200	-	20 to 80	0.25
Unit 5	19 - 21	-	40	100 to 300	0.25

Note: γ' = Bulk Unit Weight, S_u = Undrained Shear Strength, f' = Effective Friction Angle, E' = Drained Young's Modulus, v' = Poisson's Ratio

Table 13: Preliminary SRA Consolidation Parameters

Soil Type	γ' (kN/m^3)	e_0	C_c	C_r	C_α	OCR	c_v (m^2/year)
Upper SRA (above RL -7 m AHD)	15	2.4	1.4	0.17	0.0756	1.18	5
Lower SRA (below RL -7 m AHD)	15	1.8	1.0	0.17	0.0756	1.08	0.5

Note: SRA = Swan River Alluvium, γ: unit weight; e_0: initial void ratio; C_c: Compression Index; C_r: Recompression/Swelling Index; C_α: Secondary compression index; k_v: coefficient of vertical permeability

6.6 Liquefaction Potential

6.6.1 Hazard Factor and Site Sub-Soil Class

From the recommendations provided in AS1170.4-2007 (R2018), the design earthquake event is defined by the following characteristics:

- Site classification in accordance with Table 4.1 of AS1170.4-2007 (R2018):
  - E_e (very soft soil) for PFB Abutment 1 area.
  - D_e (soft soil) for PFB Pylon 1, PFB Abutment 2 and MPB Abutment 1 areas.
  - C_e (soft soil) for MPB Pylons 1 and 2 and MPB Abutment 2 areas.
- Site hazard design factor (Z) of 0.09 based on Figure 3.2(C).
- From Table 6.4 of AS1170.4-2007 (R2018),
  - a spectral shape factor $C_h(T)$ of 1.1 has been adopted based on a period of zero seconds and a site sub-soil class of D_e and E_e.
  - a spectral shape factor $C_h(T)$ of 1.3 has been adopted based on a period of zero seconds and a site sub-soil class of C_e.
- Importance factor:
  - k_p = 1.0 (AS1170.4-2007) with an annual probability of exceedance (AEP) of 1 in 500 for approach embankments.
  - k_p = 1.7 (AS1170.4-2007) with an annual probability of exceedance (AEP) of 1 in 2,000 for the abutments and pylons.

6.6.2 Earthquake Parameters for Geotechnical Design

The use of AS1170.4-2007 parameters that are developed for structural design could lead to a conservative estimate of the peak ground acceleration for geotechnical assessment such as global stability of embankment or liquefaction assessment.

Based on the information provided in Section 6.6.1,

- a design horizontal peak ground acceleration a_h = 0.12g and a_h = 0.1g (z × Ch(T=0s) × k_p) is defined for Class C_e and D_e/E_e respectively for approach embankments (AEP of 1 in 500).
- a design horizontal peak ground acceleration a_h = 0.20g and a_h = 0.017g (z × Ch(T=0s) × k_p) is defined for Class C_e and D_e/E_e respectively for the abutments (AEP of 1 in 2,000).

The 2018 National Seismic Hazard Assessment (NSHA) for Australia (Geoscience Australia 2018, accessible at www.ga.gov.au) indicates a maximum probabilistic hazard value of about 0.028g for Perth for a 1/500 AEP earthquake (in comparison to the hazard factor (z) of 0.09 in AS1170.4-2007). The 2018 NSHA states that “The 2018 update takes advantage of recent developments in earthquake-based research and ensures that the hazard modes use the best available, evidence based science.” In the 2018 NHSA, a value of k_p of 1.0 and 2.27 is appropriate for a 500-year and 2,000-year return period respectively.

Therefore, based on information provided in the 2018 NSHA,

- a design horizontal peak ground acceleration a_h = 0.036g (0.028 × 1.3 × 1.0) and a_h = 0.031g would be defined for a 500 year return period event for Class C_e and D_e/E_e respectively.
- a design horizontal peak ground acceleration a_h = 0.083g (0.028 × 1.3 × 2.27) and a_h = 0.07g would be defined for a 2,000 year return period event for Class C_e and D_e/E_e respectively

Based on the above, there would be benefit in applying the NSHA findings in situations where AS1170.4 does not apply.

For liquefaction potential assessment, the following parameters have been adopted:

- Pseudo-static horizontal coefficient of acceleration based on 2018 NSHA hazard value for Perth.
- Moment magnitude of 6.0.

6.6.3 Liquefaction

Liquefaction is one of the principal geotechnical hazards associated with earthquakes. The term “liquefaction” is widely used to describe ground damage caused by earthquake shaking even though several different phenomena may cause such damage.

Cyclic behaviour of saturated soils during strong earthquakes is characterised by development of excess pore water pressures and consequent reduction in the effective stress. In the extreme case, the effective stress may drop to zero (pore pressure equal to the total stress) and the soil would liquefy.

Liquefaction is associated with significant loss of stiffness and strength in the liquefied soil and consequent large ground deformation. Particularly damaging for engineering structures are cyclic ground movements during the period of shaking and excessive residual deformations such as settlements of the ground and lateral spreads.

Youd et al. (2001) indicates that liquefaction is generally a risk in very loose to loose granular soils with poor drainage, such as silty sands, but can occur in very soft clays. The subsurface conditions along parts of the footprint of SP3 comprise very loose to loose silty sand layers and very soft clay lays. The groundwater table in these areas is at shallow depths.

The liquefaction analysis was carried out along the alignment using Cliq software and focused on areas where CPT data indicated very loose to loose granular soils and very soft clays. An earthquake magnitude of 6.0 and a maximum acceleration of 0.42g as stated in Section 6.6.1 was used in the liquefaction analysis with conservative groundwater levels.

Preliminary analysis indicates that proposed works along the alignment are unlikely to be susceptible to liquefaction. This will be further assessed and confirmed during detailed design.

7. GROUND IMPROVEMENT DESIGN

7.1 Embankment Height Profile

The embankment height profile from the tender stage design earthwork model is summarised in Table 14.

Table 14: Maximum Fill Thickness for Current Design

Load Case	Chainage		Approx. Maximum Embankment Height (m)
	From	To	
PFB Northern Approach	45	75	1.0
	75	126	2.5
PFB Southern Approach	336.5	375	5.5
	375	425	4.0
	425	475	2.5
	475	570	1.0
MPB Northern Approach	570	585	2.5
	585	603.33	4.0
MPB Southern Approach	878.33	930	4.3
	930	975	2.5
	975	1000	1.7

7.2 Ground Improvement Options

In compliance with SWTC requirements, a range of potential solutions have been considered and summarised below:

- Preloading or surcharging without Prefabricated Vertical Drain (PVD)
- Surcharging with Prefabricated Vertical Drains (PVD)
- Rigid inclusions such as Controlled Modulus Columns
- Dry vibro replacement stone columns with surcharge
- Additional back spans supported on piled foundations
- Lightweight Fill using Expanded Polystyrene (EPS)
- Steeper embankment slope using either reinforced soil or retaining wall

Further details of the above ground improvement options are provided in Appendix B, which includes commentary on the relative time and cost, benefits and risks.

Based on cost, schedule, and construction considerations, the following ground improvement options have been considered in the preliminary design and are discussed in the following sub-sections:

- Preloading or surcharging without Prefabricated Vertical Drains (PVD)
- Surcharging with Prefabricated Vertical Drains (PVD)
- Rigid inclusions using Controlled Modulus Columns

7.3 Surcharging/Preloading with/without PVD

7.3.1 Design Methodology

The following methodology was adopted in the preliminary assessment of the long-term post construction settlement along the approach embankments for the proposed Point Fraser and McCallum Park Bridges:

- Model existing ground surface profile.
- The following cross sections have been selected for the assessment taking into considerations embankment heights and inferred ground conditions:
  - Point Fraser Northern Approach
    - CH 70
    - CH 126
  - Point Fraser Southern Approach
    - CH 425
    - CH 475
    - CH 570
  - McCallum Park Norther Approach
    - CH 585

- McCallum Park Southern Approach
  - CH 975
  - CH 1000
- Model the following construction stages for the above cross sections in Settle3 software:

  Surcharging/Preloading

  - Stage 1: Model construction stages to construct proposed embankment profile.
  - Stage 2: Surcharging or preloading.
  - Stage 3: Remove surcharging.
  - Stage 4: Construct pavement at the finished design level. Apply 5 kPa surface load.
  - Stage 5: Long-term settlement over 40 years.
  - Stage 6: Carry out surcharge design in Stages 2 and 3 where necessary to determine the extent of the required surcharge height and duration to achieve the SWTC requirements.

  Surcharging/Preloading with PVD

  - Stage 1: Install PVD
  - Stage 2: Model construction stages to construct proposed embankment profile.
  - Stage 3: Surcharging or preloading.
  - Stage 4: Remove surcharging.
  - Stage 5: Construct pavement at the finished design level. Apply 5 kPa surface load.
  - Stage 6: Long-term settlement over 40 years.
  - Stage 7: carry out surcharge design in Stages 3 and 4 where necessary to determine the extent of the required surcharge height and duration to achieve the SWTC requirements.

7.3.2 Design Assumptions

The following design assumptions and definitions were made in the assessment:

- Preloading refers to a period of sustained loading prior to final construction works, where the loading is equal to the permanent load. Fill during the preload period is to top of finished pavement level.
- Surcharging refers to a period of sustained loading prior to final construction works, where the loading is greater than the permanent load. Surcharge height is measured from the finished pavement level.
- Bulk density of new fill and surcharge = 20 kN/m^3.
- Rate of fill placement or removal is assumed to be 7 days/m vertically.
- Long-term settlement post-surcharging or post-preloading was assessed as the duration between post-surcharging/post-preloading and commencement of pavement construction is not known.
- Surcharge height refers to the thickness of fill placed above the final design level.

- Normal traffic load is modelled as 2 kPa surcharge under static condition. For the purpose of long-term settlement assessment, a factor of 0.4 could be applied to 5 kPa to account for long-term transient live load (i.e. 2 kPa) in accordance with AS 1170.0 Section 4.3 and Table 4.1, which provides live load combination factors for long-term conditions for serviceability assessment.

7.3.3 Results

Results of calculated long term post-construction settlement are presented in Table 15 which include the estimated settlement during preloading/surcharging. Various options of preloading and surcharging have been assessed for cost and construction schedule considerations.

7.4 Controlled Modulus Columns (CMCs)

7.4.1 Design Principle

The design principle of CMCs is the transfer of load to a stronger and stiffer stratum below the soft, compressible soils. In the preliminary design, the embedment length into the stronger/stiffer stratum is determined such that the load above the neutral plane is similar to the resistance below the neutral plane with the resultant long term post-construction settlement to be in the order of 150 to 200 mm.

CMCs could be constructed to support either a structural slab, or more economically a granular mattress reinforced with high strength tensile geofabrics. Enlarged heads on the CMC are often used to maximise the CMC spacing. The tensile geofabric serves the purpose of load transfer to the pile caps via catenary action and prevents lateral spread of the CMC at the edges of the embankment, which is essential to maintain embankment stability.

The advantage of CMCs is that the embankment can be constructed immediately to the final surface level without the need for foundation strength gain over time. However, the cost and schedule for installation of CMC is a factor to be considered in selection.

7.4.2 Preliminary CMC Details

Indicative spacing, lengths and stabilisation measures for CMCs are presented in Table 16.

It is recommended to carry out load testing on 2% of the total number of columns with a minimum of 5 tests.

7.4.3 CMC Installation Risks

Potential risks during CMC installation need to be managed. These risks include:

- Incompetent crane working platform due to soft ground
- Excessive lateral ground displacement which may damage adjacent CMC

Subject to additional geotechnical investigation findings and further assessments, the risks will be addressed at the next design submission.

Table 15: Summary of Preliminary Ground Improvement Details using Surcharging/Preloading with/without PVD along Approach Embankments

Location	CPT Ref.	Chainage		Existing Fill Thickness (m)	SRA Thickness (m)	Approx. Maximum Embankment Height (m)	Surcharge Thickness (m)	Surcharge/Preload Rest Period (months)	PVD Details (100 mm wide x 3 mm thick)			Estimated Settlement at End of Surcharging (mm)	Estimated Post Construction Settlement (mm)		Comment
		from	To						Length (m)	Spacing (m)	Spacing Type		5 years Allowable = 200 mm	40 years Allowable = 400mm	
		45	75	6	15	1	0.5	12	22	1.5	Triangular	230	40	60	PVD + Surcharging
PFB Northern Approach	CPT01/03/05	75	126	6	14	2.5	1.5	9	21	1	Triangular	810	50	150	PVD + Surcharging
								9	21	1.25	Triangular	760	80	150	
								9	21	1.5	Triangular	720	100	150	
								12	21	1.5	Triangular	750	80	150	
PFB Southern Approach	CPT11/12 BH06	375	425	3.5	9	4	2	9	13.5	1.25	Triangular	1375	195	345	PVD + Surcharging + Basal reinforcement using PET600 geogrid with long term tensile strength of 362 kN/m (or equivalent).
		375	425	3.5	9	4	2	12	13.5	1.25	Triangular	1405	165	315	
		375	425	3.5	9	4	2	9	-	-	-	975	240	375	Surcharging+ Basal reinforcement using PET600 geogrid with long term tensile strength of 362 kN/m (or equivalent).
		375	425	3.5	9	4	2	12	-	-	-	1000	210	335	
	CPT13/14	425	475	3.5	5.5	2.5	1.5	6	10	1.5	Triangular	780	60	85	PVD + Surcharging.
		425	475	3.5	5.5	2.5	1.5	3	-	-	-	560	125	130	Surcharging
		425	475	3.5	5.5	2.5	1.5	6	-	-	-	630	70	80	Surcharging
	CPT15/18/19/20/21 BH07	475	570	4.5	8	1	0.5	6	13.5	1.25	Triangular	250	20	40	PVD + Surcharging.
		475	570	4.5	8	1	0.5	6	13.5	1.5	Triangular	230	25	50	PVD + Surcharging.
		475	570	4.5	8	1	0.5	3	13.5	1.5	Triangular	200	40	60	PVD + Surcharging.
		475	570	4.5	8	1	0	3	13.5	1.5	Triangular	110	140	200	PVD + Preloading.
		475	570	4.5	8	1	0.5	6	-	-	-	140	40	70	Surcharging
		475	570	4.5	8	1	0.5	9	-	-	-	150	30	65	Surcharging
		475	570	4.5	8	1	0.5	12	-	-	-	160	25	60	Surcharging
		475	570	4.5	8	1	0	3	-	-	-	75	110	325	Preloading[3]
		475	570	4.5	8	1	0	6	-	-	-	80	105	315	Preloading[3]
		475	570	4.5	8	1	0	9	-	-	-	85	100	310	Preloading[3]
		475	570	4.5	8	1	0	12	-	-	-	90	95	305	Preloading[3]
MPB Northern Approach	CPT22 BH08	570	585	2	8	2.5	1.5	6	11	1.25	Triangular	1205	200	275	PVD + Surcharging
		570	585	2	8	2.5	1.5	6	-	-	-	845	230	260	Surcharging
		570	585	2	8	2.5	1.5	9	-	-	-	900	185	215	Surcharging
MPB Southern Approach	CPT24/28 BH13/14	930	975	1	4	2.5	1	6	6	1.25	Triangular	975	80	140	PVD + Surcharging
		930	975	1	4	2.5	1	6	6	1.5	Triangular	960	80	140	PVD + Surcharging
		930	975	1	4	2.5	1	3	6	1.5	Triangular	920	130	140	PVD + Surcharging
		930	975	1	4	2.5	1	6	-	-	-	880	80	150	Surcharging
		930	975	1	4	2.5	1	9	-	-	-	910	75	150	Surcharging
		930	975	1	4	2.5	1	12	-	-	-	930	75	150	Surcharging
	CPT27/28/29	975	1000	1	1	1.7	0	3	-	-	-	240	90	205	Preloading[3]

Notes:

1. Wick drains to extend from current ground level to nominally 1 m below the base of the Swan River Alluvium. Interpreted wick drain lengths are shown in the attached table.
2. Preliminary design based on a maximum allowable post-construction settlement of 200 mm and 400 mm over a 5 year and 40 year period respectively.
3. Allows for additional 50 mm post construction settlement at 5 years (number in red) and additional 150 mm post construction settlement at 40 years (number in red) due to potential ongoing creep settlement.
4. There is a higher uncertainty (and hence risk) for the case where preloading/surcharging is adopted without PVD as the rest period is highly dependent on the in-situ permeability of the SRA which is known to be variable across the site. A higher safety factor/margin on the rest period would need to be allowed for in the construction schedule for this risk. An allowance of 3 to 6 months is recommended to be added to the proposed surcharge/preloading rest period for the area where surcharge/preloading without PVD is adopted.
5. Geotechnical monitoring of the approach embankment and adjacent infrastructure will need to be carried out.
6. The potential impact on the Causeway Bridge and existing structure (About Bike Hire) at Point Fraser and other undeground utilities will need to be monitored.
7. Vertical alignment is based on civil alignment revision "x-civ-des-cpcb-501-290921" dated 29 September 2021.

Table 16: Summary of Preliminary Ground Improvement Details using CMCs along Approach Embankments

Location	CPT Ref.	Approx. Chainage		Approximate Current Ground Level	Approx. Existing Fill Thickness (m)	Approx. SRA Thickness (m)	Approx. Maximum Embankment Height above existing ground level (m)	Approx. GWL Depth (m)	CMC Diameter (m)	CMC Spacing (m)	CMC Embedment Length below Base of SRA [2] (m)	Total CMC Length [4] (m)	Toe of CMC (RL m AHD)	Geotextile
		from	To	RL m AHD										
PFB Southern Approach	CPT10 [3] BH05	337	375	1.5	3.5	8	5.5	1.1	0.45	2	2	13.5	-12	2 layers (PET800-50 or equivalent) in both longitudinal and transverse direction of the approach embankment alignment. Geotextile to be placed just above CMC heads, at base of embankment.
MPB Northern Approach	CPT21/22 BH08	585	603	1.1	2	8	4.0	0.8	0.45	2.3	2	12	-10.9	
MPB Southern Approach	BH13/BH14 CPT24/28	878	930	1.4	1	4	4.3	1.2	0.45	2.5	2	7	-5.6	

Notes:

1. Load transfer platform details and extent of CMCs across the embankment width will be provided in the next design stage.
2. Embedment depth below SRA is an initial estimate intended to limit post construction settlement at finished level to about 150-200 mm.
3. Soil profiles have been extrapolated below the CPT termination depth.
4. Length below the top of the CMC head. An allowance on CMC pile length of up to about 3 m is recommended to account for the variability in the ground conditions, especially SRA thickness.
5. Assumes a minimum concrete grade of 32 MPa.
6. Preliminary CMC arrangement is presented above. Refinement of the arrangement to optimise CMC spacing, diameter and potential use of enlarged heads can be undertaken in a subsequent
7. Geotechnical monitoring of the approach embankment and adjacent infrastructure will need to be carried out.
8. The potential impact on the Causeway Bridge and existing structure (About Bike Hire) at Point Fraser and other undeground utilities will need to be monitored.
9. Vertical alignment is based on civil alignment revision "x-civ-des-cpcb-501-290921" dated 29 September 2021.
10. It is recommended that an **allowance is made for a nominal reinforcement bar (say N32) to be inserted into the 4 CMC columns rows (same length as the CMC)** behind both abutments at Heirisson Island and at McCallum Park to provide additional lateral shear resistance.

7.5 Global Stability Assessment

7.5.1 Design Methodology

The global stability assessment of approach embankment was carried out using the commercially available software program SLIDE with the General Limit Equilibrium/Morgenstern-Price method using unfactored soil properties and loads.

Based on the approach proposed for the global stability analysis, the acceptance criteria adopted for the embankment stability are summarised in Table 17**Error! Reference source not found.**.

Table 17: Adopted Minimum Factor of Safety for Embankment Stability Analysis

Load Case	Description	Required Factor of Safety	Comment
LC1	Temporary static load case	≥ 1.25	End of embankment construction (including surcharge), embankment stability based undrained shear strength for soft clay. Rapid drawdown where relevant. 10 kPa live load is considered.
LC2	Permanent static load case	≥ 1.35	Embankment stability based on increased shear strength of the soft clay sediment following consolidation (normally consolidated undrained shear strength based on $s_u/\sigma'_v = 0.25$) 5 kPa live load is considered.
LC3	Pseudo-static earthquake load case	≥1.1	

The following considerations were made in modelling the undrained shear strength:

- Net stress at the top of SF due to new fill placement and surcharging on top of the existing embankment fill was used in assessing the gain in strength.
- 90% consolidation was assumed to have been achieved at the end of surcharging.
- Gain in strength vertically within the SF was assessed in accordance with the approach described in Table 17**Error! Reference source not found.**.

7.5.2 Stability Results

Selected preliminary slope stability outputs are presented in Appendix C. The FOS of the approach embankments in both temporary and permanent cases meets the acceptance criteria.

7.6 Preliminary Ground Improvement Layout

Based on results presented in Table 15 and Table 16, preliminary ground improvement layout is proposed and presented in Appendix D.

8. INSTRUMENTATION AND MONITORING

Preliminary instrumentation for monitoring of ground improvements comprises the following:

- Settlement plates to monitor embankment settlements
- Vibrating wire piezometers to monitor excess pore pressures in the SRA layer
- Inclinometers to monitor lateral displacement of embankments on soft ground

- Laser survey embankment surface level for signs of ‘mushroom’ or ‘egg carton’ effect
- Fibre-optic sensors embedded in geosynthetic (i.e. GeoDetect), for measuring strain in LTP or selected levels within the fill
- Earth pressure cells to monitor embankment filling progress with time
- Magnetic extensometer to determine the settlement at each geological unit

The quantities and locations of the instrumentation will be included in the next design submission.

9. IMPACT ON EXISTING AND NEW STRUCTURES

The potential of the ground improvement and piling works will need to be considered in the detailed design. Preliminary comments are as follows:

- Existing heritage-listed Causeway Bridge
  - Due to relatively close proximity, ground vibration limit of 5 mm/s may be exceeded over short intervals during dynamic pile testing on bored piles at Point Fraser Bridge (PFB) Abutments/Piers (only at some pile locations) and steel tube driven piles at PFB Pylon, depending on the hammer energy to be adopted.
- Bike Shed
  - The closest distance between “About Bike Hire” building and PFB Abutment piles is about 50 m. The vibration induced from the dynamic testing on bored piles at PFB Abutment is expected to be within the transient limit of 25 mm/s based on a maximum hammer energy of 120 kJ.
- New Abutment Piles
  - Impact of the temporary works on the proposed abutment piles has not been currently assessed. However, it is expected that a minimum offset of at least 10 m will be required to avoid/minimise potential impact on the new abutment piles, although this may need to be increased following detailed assessment.
- CMC Ground Improvement Areas
  - In the absence of a detailed assessment, cranes are not permitted to track over the CMC ground improvement area.
  - However, the Load Transfer Platform (LTP) may be used as a piling platform on the proviso that spreader mats that are of sufficient width are used to spread the pressure to be lower than that imposed by the rig used for CMC installation. Detailed assessment will be required during detailed design to assess the capacity of the ground improvement area against the imposed pressure from the piling rig.

10. SAFETY IN DESIGN

Under the Work Health and Safety Act 2020the Designer has a responsibility to undertake the design such that as much as practicable that people who maintain or construct the works are not exposed to hazards in doing so. In completing the 15% design, this obligation has been adhered to as practicable as possible for a preliminary stage design.

‘Safety in Design’ reviews are scheduled to take place for all packages and consider all the following phases:

- Construction;
- Operation;
- Maintenance; and

- Decommissioning

The reviews will take the form of a peer review and a checklist or “what if” review.

The first Safety in Design (SiD) workshop took place just before the submission of this 15% design report, in August 2021. The outcomes of the workshop and hazard identification risk register are presented in Appendix E. The workshop was combined with the Asset Managers workshop, so that all issues effecting both the design and specific operations and maintenance hazards that are to be mitigated for in the design of this bridge will be documented in the 85% design report.

The second Safety in Design review will take place with the Alliance in the detailed design stage and the results of this review will be documented and updated in the subsequent revision of the report for this package. Any residual risks or unresolved issues remaining at the completion of the design will be transferred to the Construction Risk Register for appropriate consideration during construction process planning.

11. DESIGN REVIEW AND VERIFICATION

11.1 WSP Internal Verification

This report has been internally verified in accordance with WSP Quality Assurance procedures prior to its issue. All comments have been closed out

11.2 Alliance Integrated Review

This report has been reviewed internally by the design and construction team prior to issue, including inter-disciplinary review. All comments have been closed out to enable the report to be issued.

11.3 External Review and Verification

This report will be reviewed and verified by the following:

- Main Roads WA
- Independent Verifier (IV)

The responses raised and their close out comments raised at the successive design stages will be included in the subsequent revisions of this report.

APPENDIX A: FIGURES

APPENDIX B: SUMMARY OF GROUND IMPROVEMENT OPTIONS

Ground Improvement Options	Relative Cost	Relative Time[a]	Benefits	Disadvantages/Risks
A1 – Preloading or Surcharging **without** Prefabricated Vertical Drains Preload = place fill to slightly higher than design level and allow settlement to occur. Surcharge = place fill to higher than design level, allow settlement to occur and then trim off excess fill (typically > 1 m excess) to over-consolidate soft soil.	Low	Slow to Medium	■ May be adopted in areas of low fill thickness and deeper soft soils. Can be used as part of a whole-life pavement strategy allowing relatively large post construction settlements and requiring ongoing pavement maintenance to rectify damage from ongoing settlement. ■ Potentially relatively low cost of raw fill material and placement and compaction.	■ Variations in rate of settlement needs to be carefully considered. ■ Long rest period is likely to be required which may not suit the construction schedule. ■ Rest period may be found to be longer than expected, leading to construction delays. Induced significant vertical and lateral movement which may impact the foundations of existing adjacent heritage listed Causeway Bridge or on new CPCB foundations.
A2 – Surcharging **with** Prefabricated Vertical Drain (S+PVD) Install PVD at typically 1 to 1.5 m grid spacing through full thickness of soft soil. Place and compact temporary fill up to higher than the design level and leave for a period sufficiently long for over-consolidation of the soft soil to occur.	Medium	Slow to Medium	■ Monitoring can be installed prior to construction of the surcharge which can be used to verify ground improvement. ■ Potentially relatively low cost of raw fill material and placement and compaction. ■	■ The surcharge will require a similar but potentially wider construction footprint to the final embankment (i.e. higher fill but possibly temporarily steeper side slopes). ■ Induced significant vertical and lateral movement which may impact the foundations of existing adjacent heritage listed Causeway Bridge or on new CPCB foundations. ■ Risk of encountering obstructions which may impede the installation of PVD at the nominated spacing. Predrilling/punching/excavation may be required.
B – Rigid Inclusions such as Concrete Injected Columns (CICs) Install concrete columns on a typically 1.5-2.5 m grid through full thickness of soft soil and into founding layer below. Place high-strength geotextile layers and then place and compact fill material to design level.	High	Medium to Rapid	■ Significantly reduces settlement and increases slope stability of embankments ■ Can be constructed relatively quickly compared to surcharging and PVD option ■ Can incrementally reduce rigid inclusion length to transition settlements away from the structure location ■ Lesser movement impact (i.e. lower risk) on the foundations of existing adjacent heritage listed Causeway Bridge ■	■ CICs may have difficulty penetrating through the obstructions (allowance for CIC trial or predrilling/punching/excavation may be required) ■ If columns require reinforcement, then durability issues can increase the cost of grout mix design. ■
C - Dry Vibro replacement stone columns with surcharge (SC)	Medium to High	Medium	■ Suited to high embankments and soft soils ranging up to 10 to 15 m depth. Typically reduces settlement by around 50% compared with no improvement case and enhances embankment stability. ■ Dry method would likely be required due to environmental considerations. ■ Lesser movement impact (i.e. lower risk) on the foundations of existing adjacent heritage listed Causeway Bridge. ■	■ Risk of encountering obstructions which may impede the installation of stone columns at the nominated spacing. Predrilling/punching/excavation may be required. ■
D – Additional back spans supported on piled foundations (Backspan)	Medium	Rapid	■ Reduces the amount of embankment fill and ground improvement requirements. Surcharging without PVD may potentially be feasible for low fill height subject to further assessment on the compliance with BDC requirements. ■ Lesser movement impact (i.e. lower risk) on the foundations of existing adjacent heritage listed Causeway Bridge.	■ Risk of encountering obstructions which may impede the installation of piles. Predrilling/excavation may be required. ■
E – Lightweight Fill using Expanded Polystyrene (EPS)	Medium	Rapid	■ Reduce embankment fill weight and resulting long term settlement and post construction maintenance. ■ Lesser movement impact (i.e. lower risk) on the foundations of existing adjacent heritage listed Causeway Bridge or CPCB foundations. ■	■ Anchorage to mitigate flotation of EPS ■ Buoyancy issues may limit extent of applicability. ■ Relatively costly fill material ■ Environmentally less desirable (e.g. ISCA rating) ■ Likely to require protective slab/membrane. ■
F – Steeper embankment slope using either reinforced soil or retaining wall	Medium	Rapid to Medium	■ Reduce embankment footprint	■ Availability of geotextile. Long lead time may be required.

Note:

a. Rapid (than 3 months), Medium (3 to 6 months), Slow (minimum 6 months or longer)
b. The above ground improvement options require more detailed investigation and analysis.

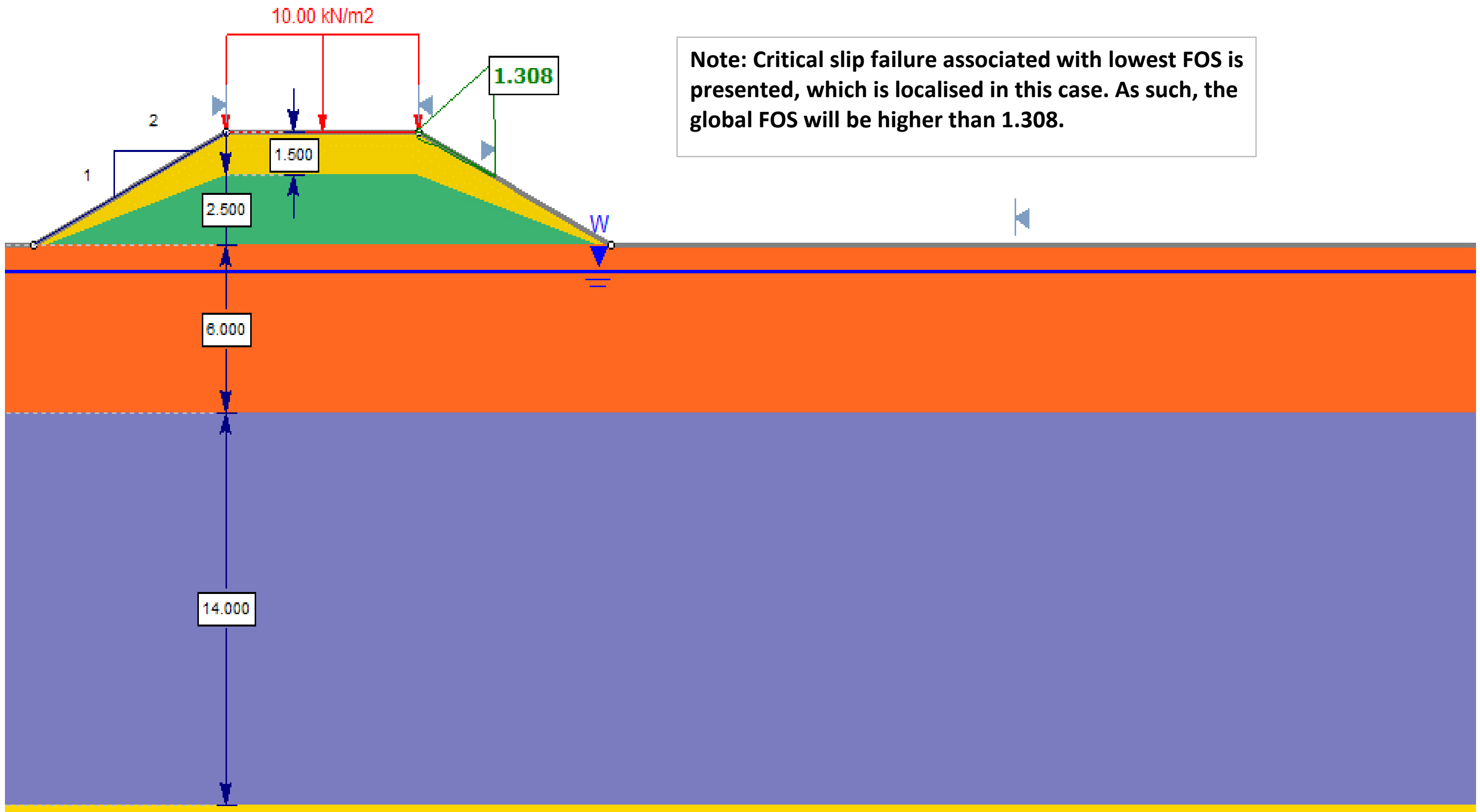
APPENDIX C: SLOPE STABILITY OUTPUTS

APPENDIX D: PRELIMINARY GROUND IMPROVEMENT LAYOUT

APPENDIX E: SAFETY IN DESIGN

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Allow Sliding	Water Surface	Hu Type	Hu
Surcharge		20	Mohr-Coulomb	0.1	35			Water Surface	Custom	1
Embankment		20	Mohr-Coulomb	0.1	35			Water Surface	Custom	1
FILL		18	Mohr-Coulomb	0.1	32			Water Surface	Custom	1
SRA 1		15	Undrained	20		Constant		Water Surface	Custom	1
PF SAND		18	Mohr-Coulomb	0.1	34			Water Surface	Custom	1
MS - SANDSTONE		20	Infinite strength				Yes	Water Surface	Custom	1

Note: Critical slip failure associated with lowest FOS is presented, which is localised in this case. As such, the global FOS will be higher than 1.308.



CLIENT
Causeway Link Alliance

PROJECT
Causeway Pedestrian and Cyclist Bridge

CONSULTANT

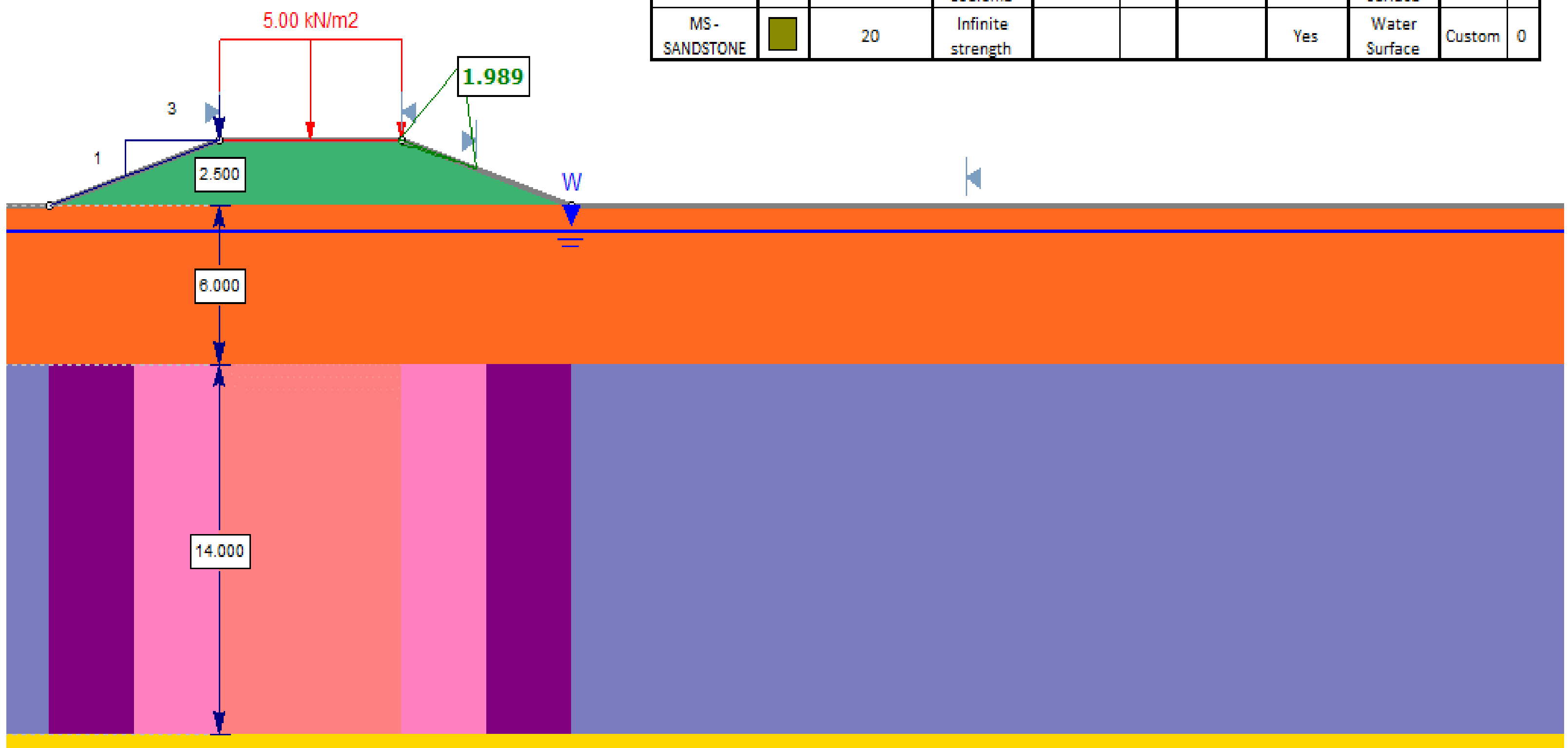
DATE	8/10/2022
PREPARED	JK
DESIGN	JK
REVIEW	SK
APPROVED	SK

TITLE
Global Stability Assessment - Point Fraser Northern Abutment (Chainage 75 to 125 m)
Short Term Case

PROJECT No.	DOC. No.	Rev.	FIG No
C301-CLA	0000-GE-REP-00001	A	C1

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Allow Sliding	Water Surface	Hu Type	Hu
Embankment		20	Mohr-Coulomb	0.1	35			Water Surface	Custom	1
FILL		18	Mohr-Coulomb	0.1	32			Water Surface	Custom	1
SRA 1		15	Undrained	20		Constant		Water Surface	Custom	0
SRA 2		15	Undrained	21		Constant		Water Surface	Custom	0
SRA 3		15	Undrained	23		Constant		Water Surface	Custom	0
SRA 4		15	Undrained	25		Constant		Water Surface	Custom	0
PF SAND		18	Mohr-Coulomb	0.1	34			Water Surface	Custom	1
MS - SANDSTONE		20	Infinite strength				Yes	Water Surface	Custom	0

Note: Critical slip failure associated with lowest FOS is presented, which is localised in this case. As such, the global FOS will be higher than 1.989.



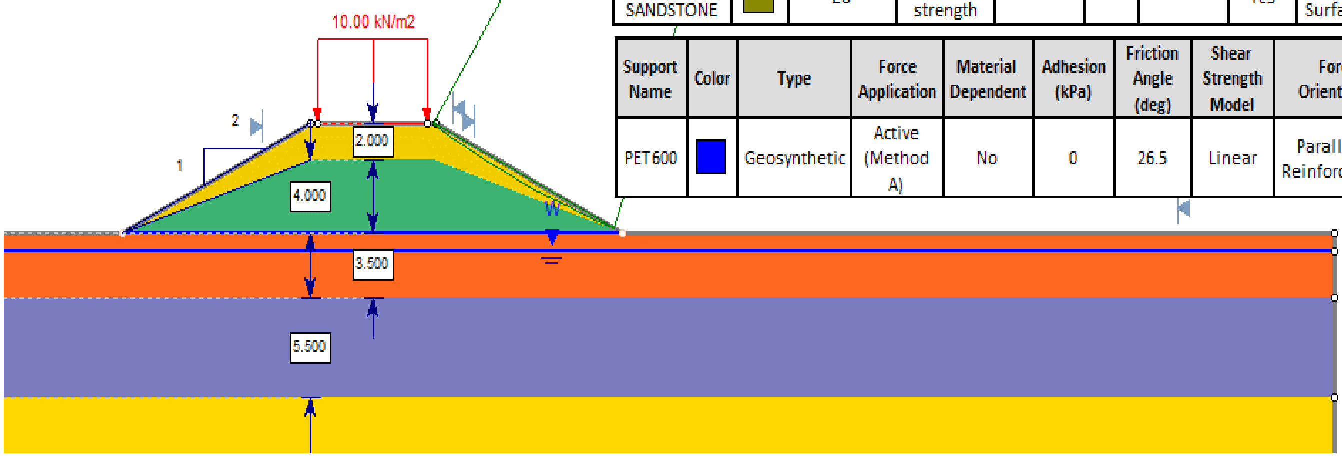
CLIENT	Causeway Link Alliance
PROJECT	Causeway Pedestrian and Cyclist Bridge
CONSULTANT	
DATE	8/10/2022
PREPARED	JK
DESIGN	JK
REVIEW	SK
APPROVED	SK
TITLE	**Global Stability Assessment - Point Fraser Northern Abutment (Chainage 75 to 125 m) Long Term Case**
PROJECT No.	C301-CLA
DOC. No.	0000-GE-REP-00001
Rev.	A
FIG No	C2

1.455

Note: Critical slip failure associated with lowest FOS is presented, which is localised in this case. As such, the global FOS will be higher than 1.455.

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Allow Sliding	Water Surface	Hu Type	Hu
Surcharge		20	Mohr-Coulomb	0.1	35			Water Surface	Custom	1
Embankment		20	Mohr-Coulomb	0.1	35			Water Surface	Custom	1
FILL		18	Mohr-Coulomb	0.1	32			Water Surface	Custom	1
In-situ SRA (Su=20 kPa)		15	Undrained	20		Constant		Water Surface	Custom	0
PF SAND		18	Mohr-Coulomb	0.1	33			Water Surface	Custom	0
MS - SANDSTONE		20	Infinite strength				Yes	Water Surface	Custom	1

Support Name	Color	Type	Force Application	Material Dependent	Adhesion (kPa)	Friction Angle (deg)	Shear Strength Model	Force Orientation	Anchorage	Strip Coverage (%)	Tensile Strength (kN/m)
PET600		Geosynthetic	Active (Method A)	No	0	26.5	Linear	Parallel to Reinforcement	None	100	361.947



CLIENT
Causeway Link Alliance

PROJECT
Causeway Pedestrian and Cyclist Bridge

CONSULTANT

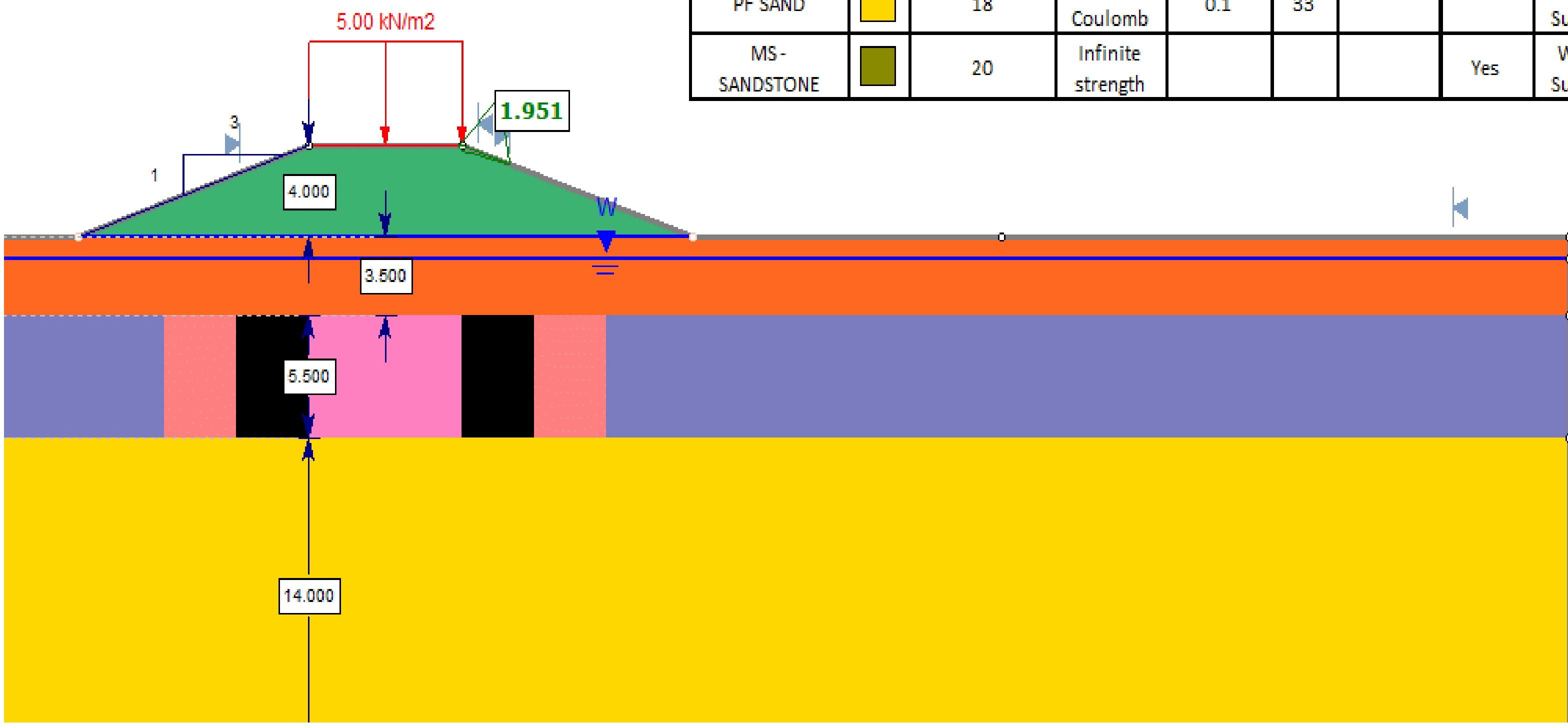
DATE	8/10/2022
PREPARED	JK
DESIGN	JK
REVIEW	SK
APPROVED	SK

TITLE
Global Stability Assessment - Point Fraser Southern Abutment
(Chainage 375 to 425 m)
Option 2: Geotextile Embankment - Short Term Case

PROJECT No.	DOC. No.	Rev.	FIG No
C301-CLA	0000-GE-REP-00001	A	C3

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Allow Sliding	Water Surface	Hu Type	Hu
Embankment		20	Mohr-Coulomb	0.1	35			Water Surface	Custom	1
FILL		18	Mohr-Coulomb	0.1	32			Water Surface	Custom	1
In-situ SRA (Su=20 kPa)		15	Undrained	20		Constant		Water Surface	Custom	1
SRA below embankment		15	Undrained	45		Constant		Water Surface	Custom	1
SRA (Su = 25 kPa)		15	Undrained	25		Constant		Water Surface	Custom	1
SRA Su = 30 kPa)		15	Undrained	30		Constant		Water Surface	Custom	1
PF SAND		18	Mohr-Coulomb	0.1	33			Water Surface	Custom	1
MS - SANDSTONE		20	Infinite strength				Yes	Water Surface	Custom	0

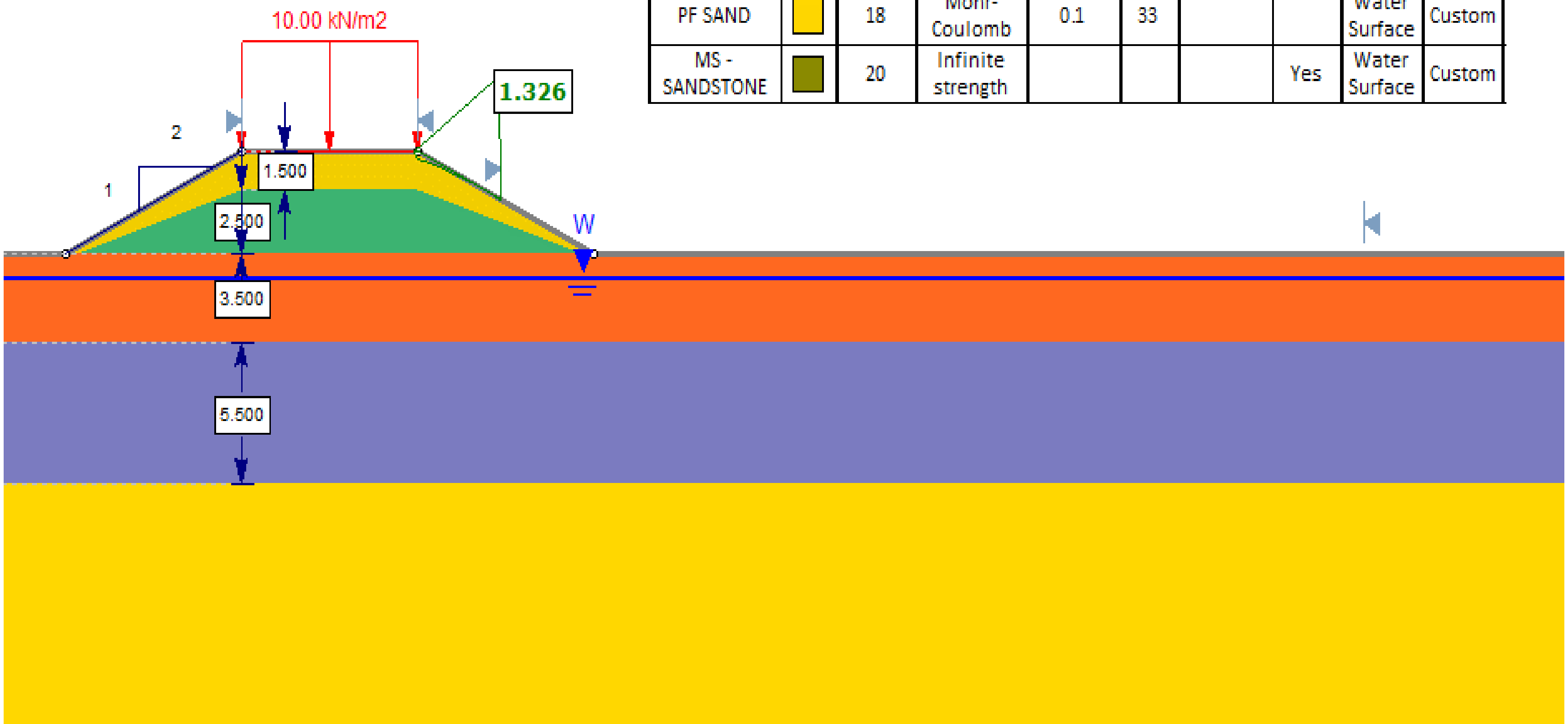
Note: Critical slip failure associated with lowest FOS is presented, which is localised in this case. As such, the global FOS will be higher than 1.951.



CLIENT: Causeway Link Alliance	PROJECT: Causeway Pedestrian and Cyclist Bridge
CONSULTANT	TITLE: **Global Stability Assessment - Point Fraser Southern Abutment (Chainage 375 to 425 m) Option 2: Geotextile Embankment - Long Term Case**
DATE 8/10/2022	PROJECT No. C301-CLA
PREPARED JK	DOC. No. 0000-GE-REP-00001
DESIGN JK	Rev. A
REVIEW SK	FIG No C4
APPROVED SK	

Material Name	Color	Unit Weight (kN/ m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Allow Sliding	Water Surface	Hu Type
Surcharge		20	Mohr-Coulomb	0.1	35			Water Surface	Custom
Embankment		20	Mohr-Coulomb	0.1	35			Water Surface	Custom
FILL		18	Mohr-Coulomb	0.1	32			Water Surface	Custom
SRA 1		15	Undrained	20		Constant		Water Surface	Custom
PF SAND		18	Mohr-Coulomb	0.1	33			Water Surface	Custom
MS - SANDSTONE		20	Infinite strength				Yes	Water Surface	Custom

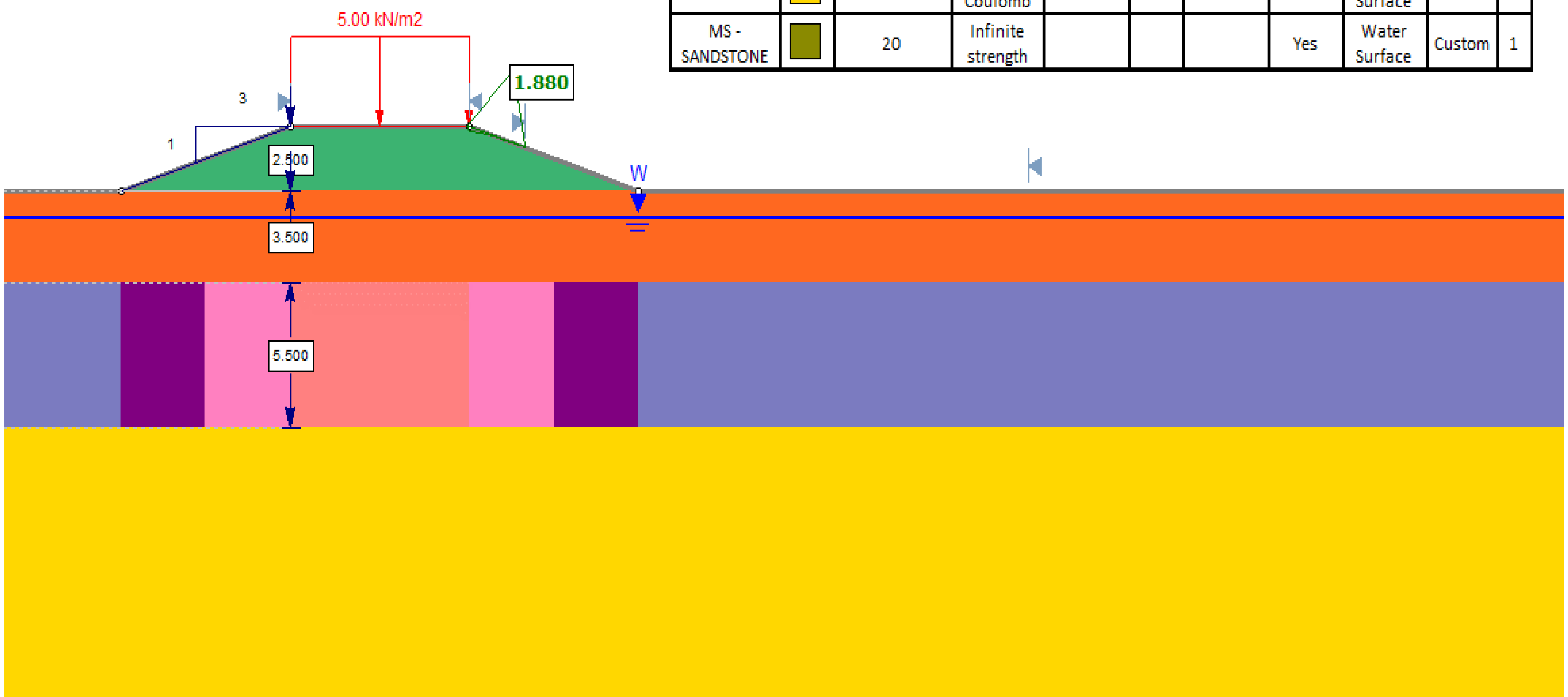
Note: Critical slip failure associated with lowest FOS is presented, which is localised in this case. As such, the global FOS will be higher than 1.326.



CLIENT	Causeway Link Alliance	PROJECT	Causeway Pedestrian and Cyclist Bridge
CONSULTANT	DATE 8/10/2022; PREPARED JK; DESIGN JK; REVIEW SK; APPROVED SK	TITLE	Global Stability Assessment - Point Fraser Southern Abutment (Chainage 425 to 475 m) Short Term Case
		PROJECT No. C301-CLA; DOC. No. 0000-GE-REP-00001; Rev. A	FIG No C5

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Allow Sliding	Water Surface	Hu Type	Hu
Embankment		20	Mohr-Coulomb	0.1	35			Water Surface	Custom	1
FILL		18	Mohr-Coulomb	0.1	32			Water Surface	Custom	1
SRA 1		15	Undrained	20		Constant		Water Surface	Custom	1
SRA 2		15	Undrained	22		Constant		Water Surface	Custom	1
SRA 3		15	Undrained	25		Constant		Water Surface	Custom	1
SRA 4		15	Undrained	28		Constant		Water Surface	Custom	1
PF SAND		18	Mohr-Coulomb	0.1	33			Water Surface	Custom	1
MS - SANDSTONE		20	Infinite strength				Yes	Water Surface	Custom	1

Note: Critical slip failure associated with lowest FOS is presented, which is localised in this case. As such, the global FOS will be higher than 1.880.



CLIENT
Causeway Link Alliance

PROJECT
Causeway Pedestrian and Cyclist Bridge

CONSULTANT

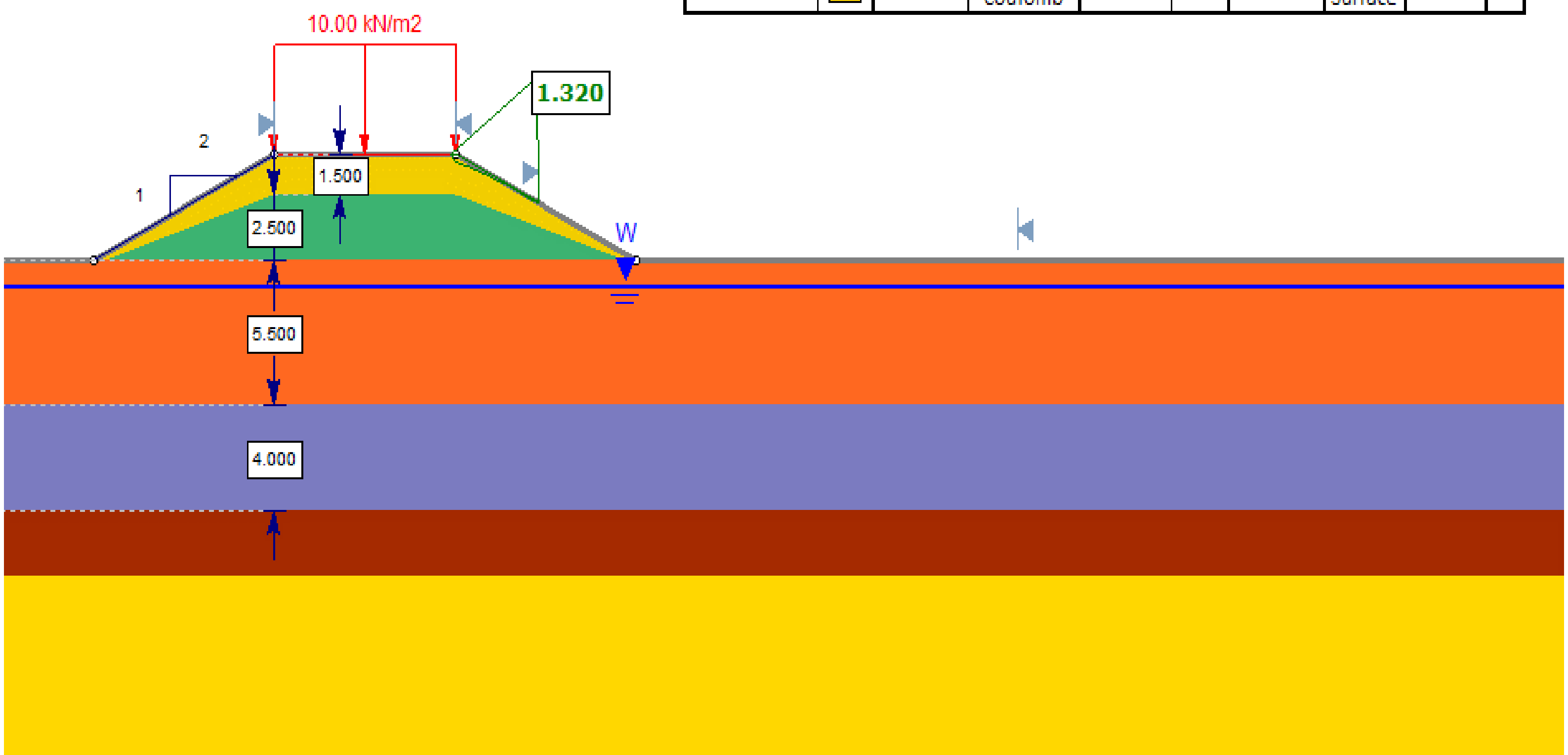
DATE	8/10/2022
PREPARED	JK
DESIGN	JK
REVIEW	SK
APPROVED	SK

TITLE
Global Stability Assessment - Point Fraser Southern Abutment (Chainage 425 to 475 m)
Long Term Case

PROJECT No.	DOC. No.	Rev.	FIG No
C301-CLA	0000-GE-REP-00001	A	C6

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Water Surface	Hu Type	Hu
Surcharge		20	Mohr-Coulomb	0.1	35		Water Surface	Custom	1
Embankment		20	Mohr-Coulomb	0.1	35		Water Surface	Custom	1
FILL		18	Mohr-Coulomb	0.1	32		Water Surface	Custom	1
SRA 1		15	Undrained	20		Constant	Water Surface	Custom	1
PF CLAY		18	Undrained	80		Constant	Water Surface	Custom	1
PF SAND		20	Mohr-Coulomb	0.1	36		Water Surface	Custom	1

Note: Critical slip failure associated with lowest FOS is presented, which is localised in this case. As such, the global FOS will be higher than 1.320.



CLIENT
Causeway Link Alliance

PROJECT
Causeway Pedestrian and Cyclist Bridge

CONSULTANT

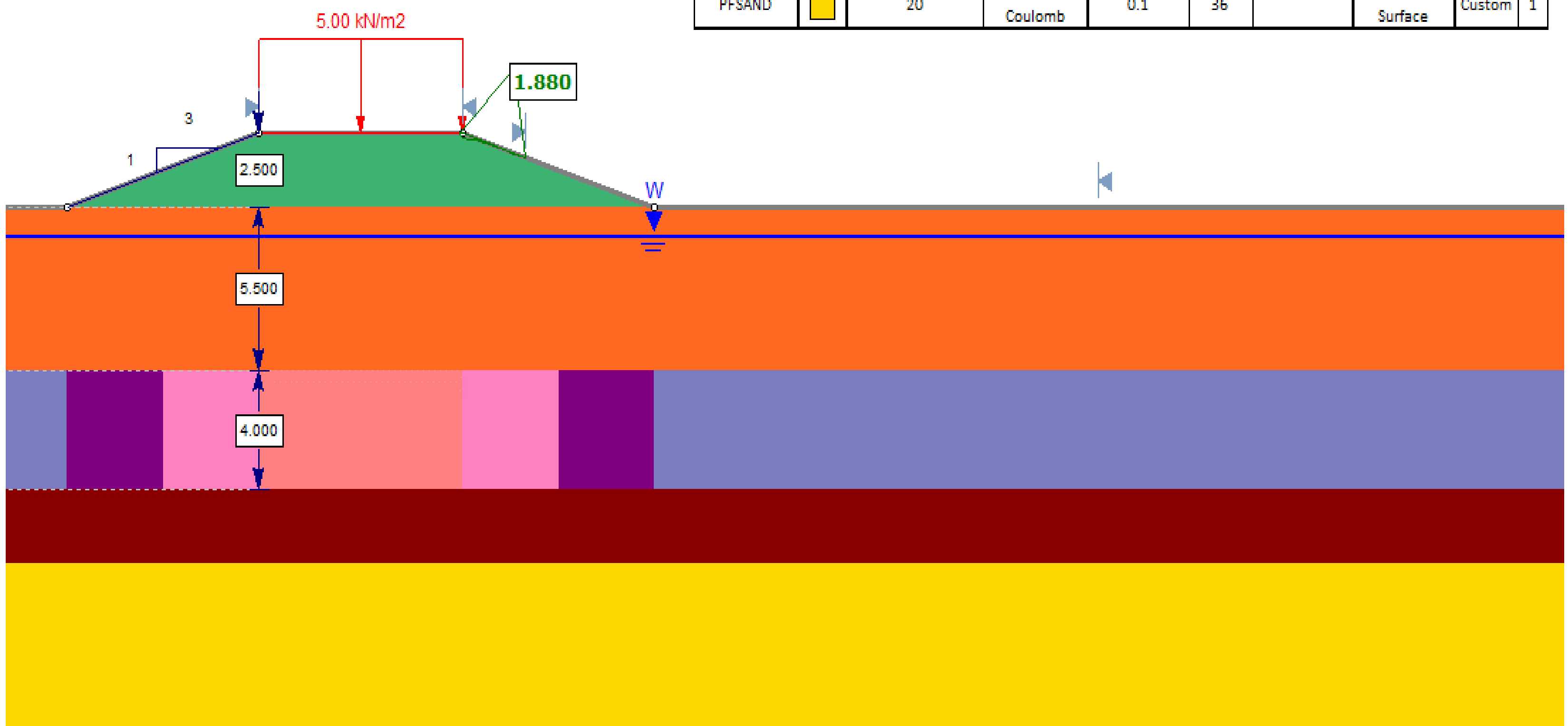
DATE	8/10/2022
PREPARED	JK
DESIGN	JK
REVIEW	SK
APPROVED	SK

TITLE
Global Stability Assessment - McCallum Park Northern Abutment (Chainage 570 to 585 m)
Short Term Case

PROJECT No.	DOC. No.	Rev.	FIG No
C301-CLA	0000-GE-REP-00001	A	C7

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Water Surface	Hu Type	Hu
Embankment		20	Mohr-Coulomb	0.1	35		Water Surface	Custom	1
FILL		18	Mohr-Coulomb	0.1	32		Water Surface	Custom	1
SRA 1		15	Undrained	20		Constant	Water Surface	Custom	1
SRA 2		15	Undrained	21		Constant	Water Surface	Custom	1
SRA 3		15	Undrained	23		Constant	Water Surface	Custom	1
SRA 4		15	Undrained	25		Constant	Water Surface	Custom	1
PF CLAY		18	Undrained	80		Constant	Water Surface	Custom	1
PF CLay (su=40)		18	Undrained	40		Constant	Water Surface	Custom	1
PFSAND		20	Mohr-Coulomb	0.1	36		Water Surface	Custom	1

Note: Critical slip failure associated with lowest FOS is presented, which is localised in this case. As such, the global FOS will be higher than 1.880.



CLIENT
Causeway Link Alliance

PROJECT
Causeway Pedestrian and Cyclist Bridge

CONSULTANT

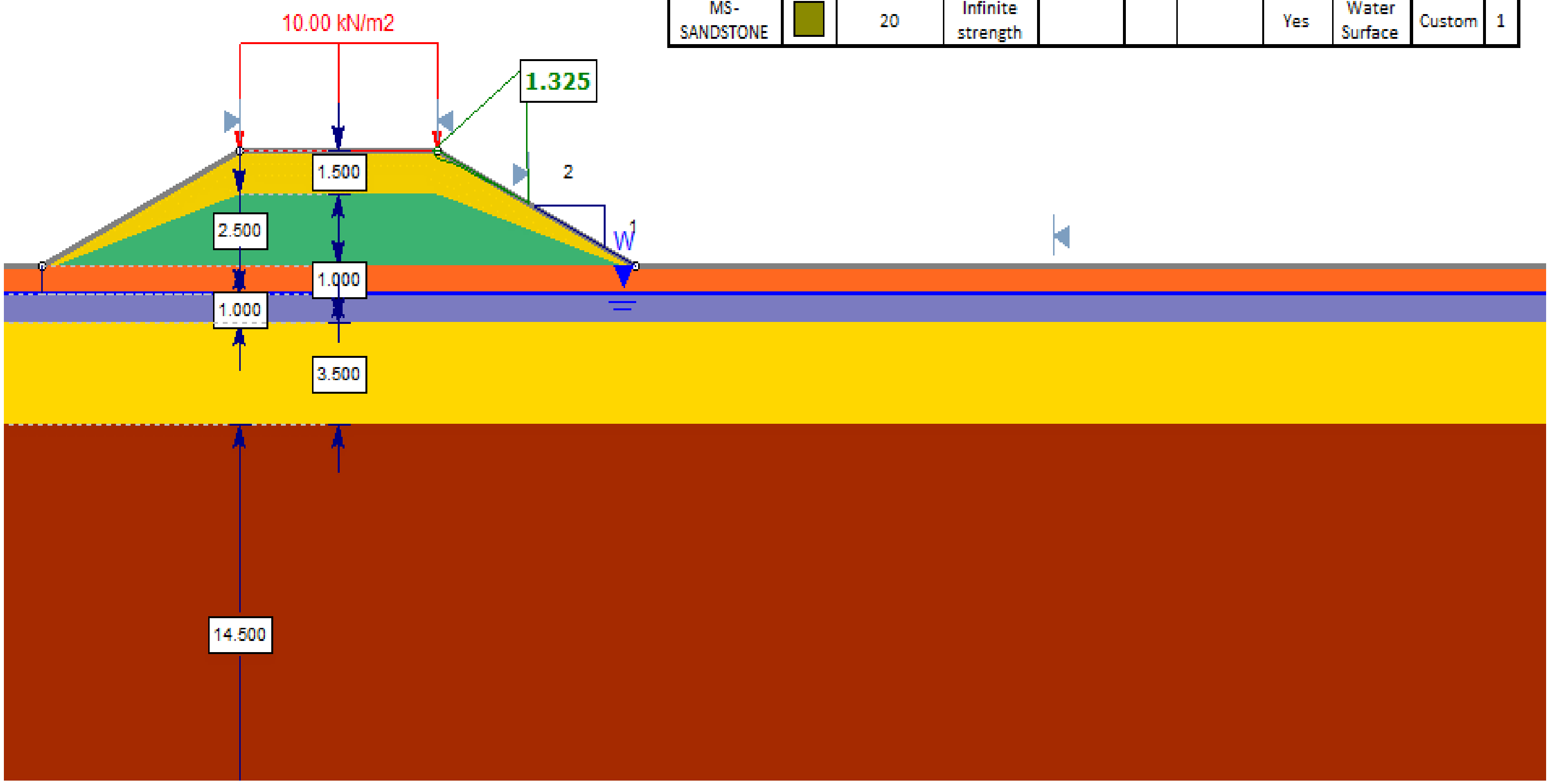
DATE	8/10/2022
PREPARED	JK
DESIGN	JK
REVIEW	SK
APPROVED	SK

TITLE
Global Stability Assessment - McCallum Park Northern Abutment (Chainage 570 to 585 m)
Long Term Case

PROJECT No.	DOC. No.	Rev.	FIG No
C301-CLA	0000-GE-REP-00001	A	C8

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Allow Sliding	Water Surface	Hu Type	Hu
Surcharge		20	Mohr-Coulomb	0.1	35			Water Surface	Custom	1
Embankment		20	Mohr-Coulomb	0.1	35			Water Surface	Custom	1
FILL		18	Mohr-Coulomb	0.1	32			Water Surface	Custom	1
SRA 1		15	Undrained	20		Constant		Water Surface	Custom	1
PF CLAY		18	Undrained	80		Constant		Water Surface	Custom	1
PF SAND		18	Mohr-Coulomb	0.1	34			Water Surface	Custom	1
MS-SANDSTONE		20	Infinite strength				Yes	Water Surface	Custom	1

Note: Critical slip failure associated with lowest FOS is presented, which is localised in this case. As such, the global FOS will be higher than 1.325.



CLIENT
Causeway Link Alliance

PROJECT
Causeway Pedestrian and Cyclist Bridge

CONSULTANT

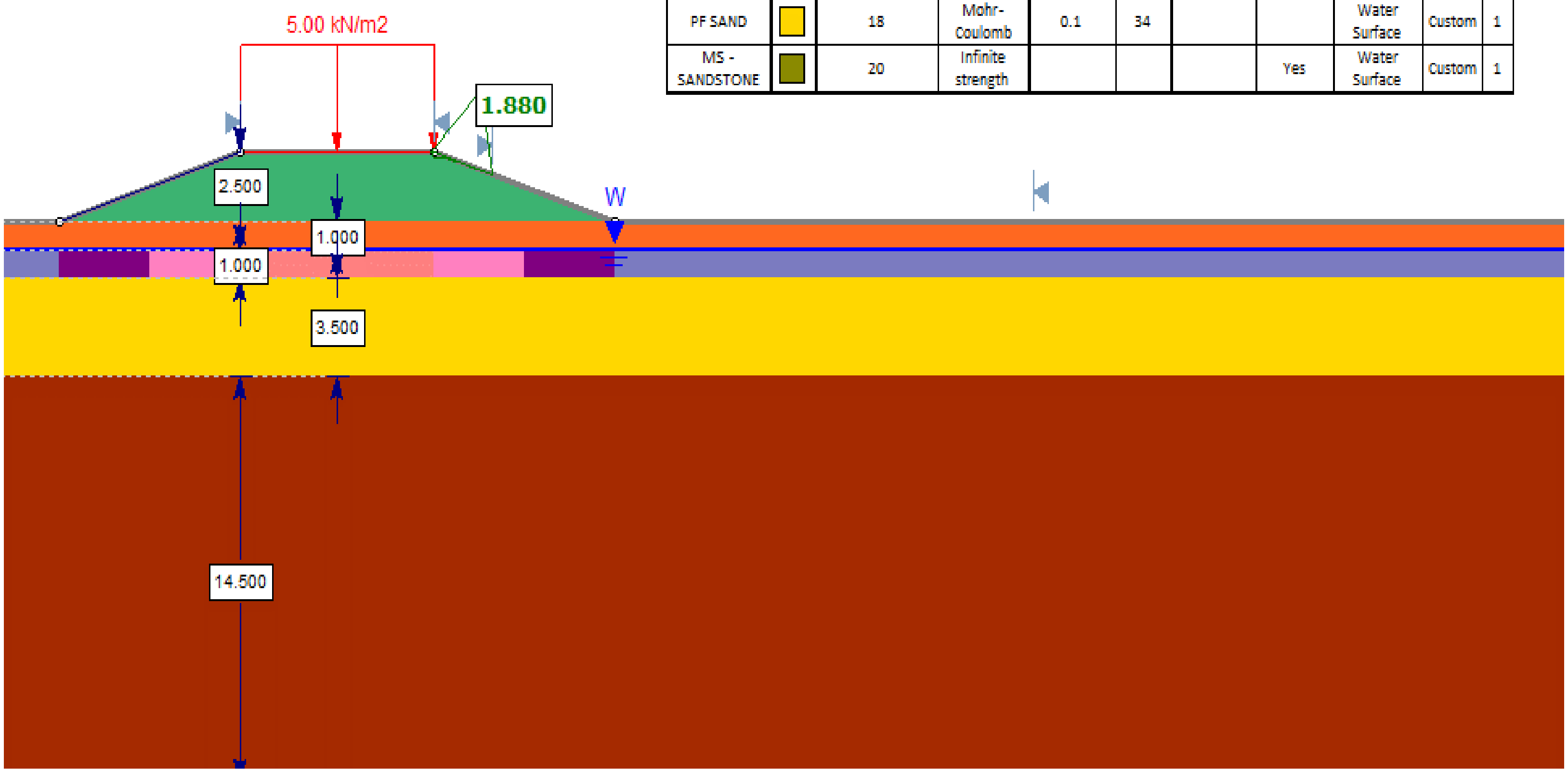
DATE	8/10/2022
PREPARED	JK
DESIGN	JK
REVIEW	SK
APPROVED	SK

TITLE
Global Stability Assessment - McCallum Park Southern Abutment (Chainage 925 to 970 m)
Short Term Case

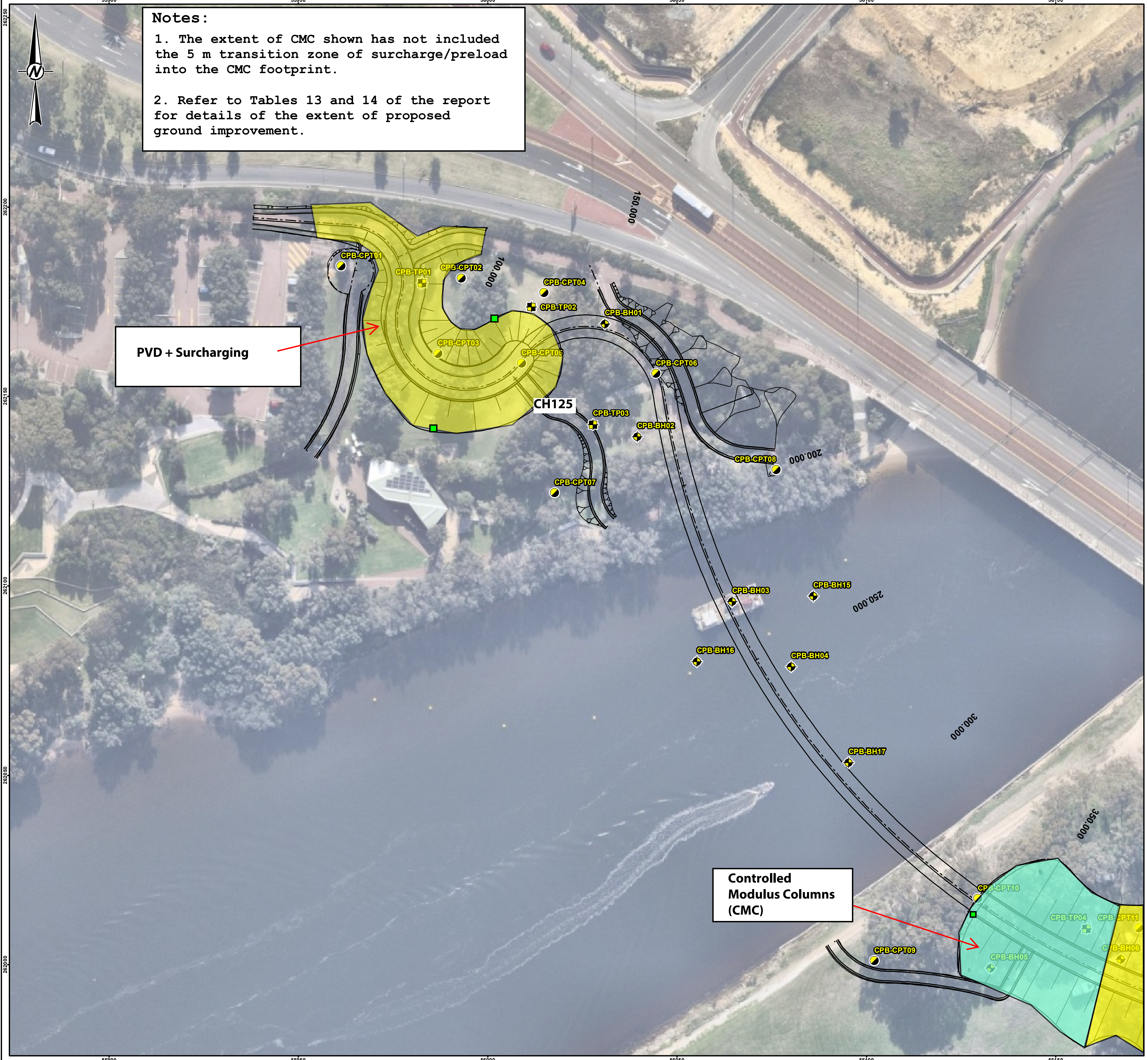
PROJECT No.	DOC. No.	Rev.	FIG No
C301-CLA	0000-GE-REP-00001	A	C9

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Allow Sliding	Water Surface	Hu Type	Hu
Embankment		20	Mohr-Coulomb	0.1	35			Water Surface	Custom	1
FILL		18	Mohr-Coulomb	0.1	32			Water Surface	Custom	1
SRA 1		15	Undrained	20		Constant		Water Surface	Custom	1
SRA 2		15	Undrained	24		Constant		Water Surface	Custom	1
SRA 3		15	Undrained	28		Constant		Water Surface	Custom	1
SRA 4		15	Undrained	32		Constant		Water Surface	Custom	1
PF CLAY		18	Undrained	80		Constant		Water Surface	Custom	1
PF SAND		18	Mohr-Coulomb	0.1	34			Water Surface	Custom	1
MS - SANDSTONE		20	Infinite strength				Yes	Water Surface	Custom	1

Note: Critical slip failure associated with lowest FOS is presented, which is localised in this case. As such, the global FOS will be higher than 1.880.

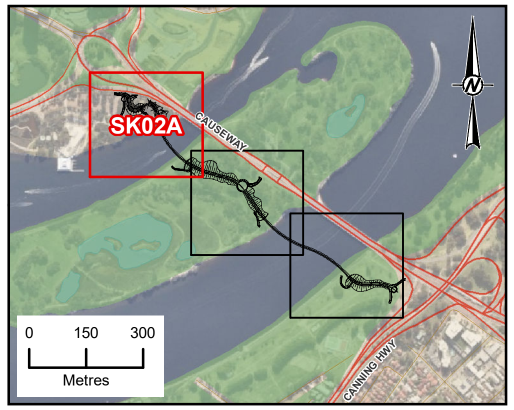


CLIENT	Causeway Link Alliance	PROJECT	Causeway Pedestrian and Cyclist Bridge
CONSULTANT	DATE 8/10/2022; PREPARED JK; DESIGN JK; REVIEW SK; APPROVED SK	TITLE	Global Stability Assessment - McCallum Park Southern Abutment (Chainage 925 to 970 m) Long Term Case
PROJECT No. C301-CLA	DOC. No. 0000-GE-REP-00001	Rev. A	FIG No C10



LEGEND

- TEST PIT LOCATION
- BOREHOLE LOCATION
- CONE PENETRATION TEST LOCATION



0 25 50

1:1,000 METRES

NOT FOR CONSTRUCTION
DRAFT

NOTE:
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)

REFERENCES:
1. SITE LAYOUT OVERLAY PROVIDED BY CLIENT. DRAWING FILE: *X-CIV-DES-CPCB-501-290921_NATIVE.DWG
2. CADASTRE BASED ON INFORMATION PROVIDED BY AND WITH THE PERMISSION OF THE WESTERN AUSTRALIAN LAND INFORMATION AUTHORITY TRADING AS LANDGATE (2021).
3. AERIAL IMAGERY SOURCED FROM NEARMAP DATED JULY 2021.

CLIENT
CAUSEWAY LINK ALLIANCE

PROJECT
CAUSEWAY PEDESTRIAN AND CYCLIST BRIDGE

TITLE
PROPOSED GROUND IMPROVEMENT LAYOUT

CLIENT

Causeway Link Alliance

YYYY-MM-DD	2021-09-29
DESIGNED	SKT
PREPARED	JRP
REVIEWED	
APPROVED	

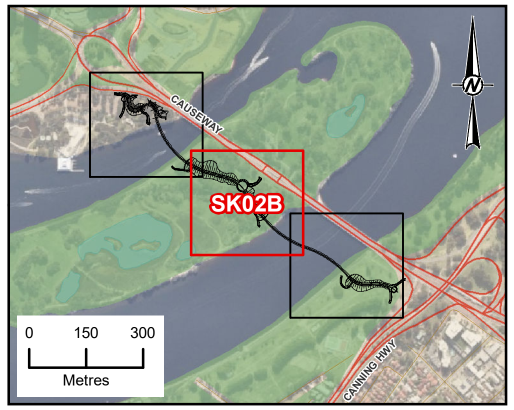
PROJECT NO.	REV.	SKETCH
C301-CLA-0000-GE-REP-00001	A	SK02A

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A3

25mm 0

LEGEND

- TEST PIT LOCATION
- BOREHOLE LOCATION
- CONE PENETRATION TEST LOCATION



0 25 50

1:1,000 METRES

NOT FOR CONSTRUCTION
DRAFT

NOTE:
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)

REFERENCES:
1. SITE LAYOUT OVERLAY PROVIDED BY CLIENT. DRAWING FILE: *X-CIV-DES-CPCB-501-290921_NATIVE.DWG
2. CADASTRE BASED ON INFORMATION PROVIDED BY AND WITH THE PERMISSION OF THE WESTERN AUSTRALIAN LAND INFORMATION AUTHORITY TRADING AS LANDGATE (2021).
3. AERIAL IMAGERY SOURCED FROM NEARMAP DATED JULY 2021.

CLIENT
CAUSEWAY LINK ALLIANCE

PROJECT
CAUSEWAY PEDESTRIAN AND CYCLIST BRIDGE

TITLE
PROPOSED GROUND IMPROVEMENT AND INSTRUMENTATION MONITORING LAYOUT

CLIENT
Causeway Link Alliance

YYYY-MM-DD	2021-09-29
DESIGNED	SKT
PREPARED	JRP
REVIEWED	
APPROVED	

PROJECT NO. C301-CLA-0000-GE-REP-00001 REV. A SKETCH SK02B

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A3

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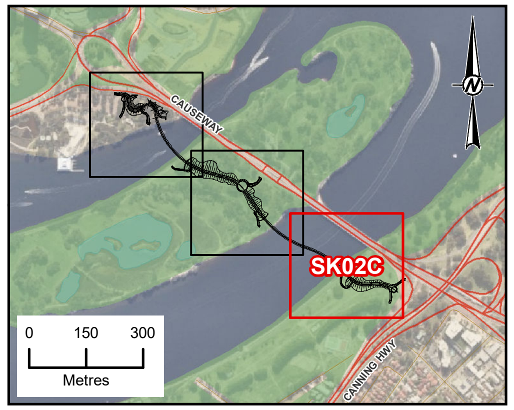
LEGEND

- TEST PIT LOCATION
- BOREHOLE LOCATION
- CONE PENETRATION TEST LOCATION



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1:1,000 METRES

NOT FOR CONSTRUCTION

DRAFT

NOTE:

1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)

REFERENCES:

1. SITE LAYOUT OVERLAY PROVIDED BY CLIENT. DRAWING FILE: *X-CIV-DES-CPCB-501-290921_NATIVE.DWG
2. CADASTRE BASED ON INFORMATION PROVIDED BY AND WITH THE PERMISSION OF THE WESTERN AUSTRALIAN LAND INFORMATION AUTHORITY TRADING AS LANDGATE (2021).
3. AERIAL IMAGERY SOURCED FROM NEARMAP DATED JULY 2021.

CLIENT
CAUSEWAY LINK ALLIANCE

PROJECT
CAUSEWAY PEDESTRIAN AND CYCLIST BRIDGE

TITLE
PROPOSED GROUND IMPROVEMENT AND INSTRUMENTATION MONITORING LAYOUT

CLIENT

Causeway Link Alliance

YYYY-MM-DD	2021-09-29
DESIGNED	SKT
PREPARED	JRP
REVIEWED	
APPROVED	

PROJECT NO. C301-CLA-0000-GE-REP-00001 | REV. A | SKETCH SK02C

Risk Reference Number	Project Risk	Project Stage	Causes	Resulting In	Priority	Consequence Category	RISK REGISTER		
							Risk Rating		
							Consequence Rating	Likelihood Rating	Level of Risk
	Example: Design Route -Route meets current and future customer requirements	Design, Procurement, Construction, Operations	Poor evaluation of current customer needs, lack of integrated long term planning for Perth CBD, insufficient stakeholder consultation.	Impact on project objectives, reputation damage	High, Medium, Low	Reputation & Trust	Moderate (3)	Possible (3)	Medium 9
43	Damage to fauna and flora during construction	Construction	construction practices, poor planning, lack of approvals and review	damage to flora and fauna, negative public image, impact on project objectives			Moderate (3)	Likely (4)	High 12
44	Poor water quality - Swan River	Construction	spills in river	impact to project objectives, impact on flora and fauna, potential health and safety risks			Moderate (3)	Likely (4)	High 12
31	Fire risk with large source of mulch.	Construction	Storage of mulch prior to distribution	Fire to surrounding area	Medium	Health & Safety	Major (4)	Rare (1)	Low 4
21	dust from high embankment due to high wind	Construction	weather conditions, construction		Medium	Health & Safety	Minor (2)	Possible (3)	Low 6
14	Electrocution from electrcial underground services / clash	Construction	poor planning and communication	death or injury, reputation damage			Major (4)	Unlikely (2)	Medium 8
16	Hitting underground services	Construction	poor planning and communication	death or injury, reputation damage			Major (4)	Unlikely (2)	Medium 8
46	Hitting services - live or abandoned	Construction	Unidentified services or historic abandoned services	Personnel injury, loss of critical services	Medium	Health & Safety	Major (4)	Unlikely (2)	Medium 8
3	Unauthorised access to constrcuction site	Construction	Illegal access - criminal activity, theft, purposeful damage to equipment, angry stakeholders Access has not been properly restricted or monitored (CCTV, security patrols, fencing…etc)	injury, damage to precinct, dangerous environment, reputation damage, damage to structure			Moderate (3)	Possible (3)	Medium 9
6	Injury or property damage in worksite	Construction	People wishing to pass through the site or get from one side to another	injury, damage to precinct, dangerous environment, reputation damage, damage to structure			Moderate (3)	Possible (3)	Medium 9
51	Lighting and emergency systems power outage.	Construction	weather conditions, fault, fault with provider, maintenance work	injury, damage to precinct, dangerous environment, reputation damage			Moderate (3)	Possible (3)	Medium 9
60	lack of access to the bridge and precinct for those with dissabilities	Design	lack of wayfinding and poor design	injury, impact on project objectives, reputation damage by not providing a safe and inclusive environment for all.			Minor (2)	Almost Certain (5)	High 10
37	Access to top of pylon (aircraft lights)	Design	Maintenace access requirements not taken into account.	Infrastructure failure due to lack of maintenance / creation of maintenance risks		Health & Safety	Catastrophic (5)	Unlikely (2)	High 10
53	Bridge becomes unstable	Design	Cables vibration under wind / rain condition	injury, damage to precinct, dangerous environment, reputation damage		Reputation & Trust	Major (4)	Possible (3)	High 12
33	Access for bearing maintenance	Design	Maintenace access requirements not taken into account.	Infrastructure failure due to lack of maintenance / creation of maintenance risks		Health & Safety	Moderate (3)	Likely (4)	High 12
4	People jump off bridge	Design	Suicide attempts, adrenaline - jumpng to swim, unsupervised children	injury, damage to precinct, dangerous environment, reputation damage			Moderate (3)	Likely (4)	High 12
48	Flood	Design	weather conditions, environmental factors, construction impact	impacting temporary piers; permanent piers with potential debris floatting at high velocity			Major (4)	Possible (3)	High 12
7	Pedestrians hit by vehicles accessing the site	Design	Limited site access	injury, damage to precinct, dangerous environment, reputation damage			Moderate (3)	Almost Certain (5)	High 15
12	High wind loads on structure without cables	Design	Pylon installed without cables tying it down	Impact on project objectives, reputation damage	High	Health & Safety	Catastrophic (5)	Possible (3)	High 15
11	Surrounding structures (utilities, Causeway bridge, etc) affected by settlement	Design	Settlement more than anticipated and affecting surrounding structures (utilities, Causeway bridge, etc)	damage to the structure or surrounding utilities			Moderate (3)	Almost Certain (5)	High 15
41	erosion and runoff to swan river	Design	weather conditions, extensive clearing	polution, environmental damage	Medium	Environmental	Catastrophic (5)	Possible (3)	High 15
18	Bridge maximum load exceeded	Design	event and no crowd control, lack of cctv monitoring of people on bridge	damage to structure, injury of crowd, possible structure collapse?			Insignificant (1)	Rare (1)	Low 1
40	Non-compliant bridge height for water traffic clearance	Design	Excessive deflection of the bridge impacting the required navigation clearance	River traffic blockage or damage to boats	Medium	Legal & Compliance	Insignificant (1)	Rare (1)	Low 1
35	Lack of traffic control at Point Fraser	Design	Changes to traffic management and flow around Point Fraser	Impact to access of Causeway Bridge			Insignificant (1)	Unlikely (2)	Low 2
1	Unlawful public access to laydown area	Design	Illegal access - criminal activity, theft, purposeful damage to equipment, angry stakeholders Access has not been properly restricted or monitored (CCTV, security patrols, fencing…etc)	injury, damage to precinct, dangerous environment, reputation damage, damage to structure			Insignificant (1)	Possible (3)	Low 3
34	Replacement of critical elements : bearing / cable etc	Design	Maintenace access requirements not taken into account.	Infrastructure failure due to lack of maintenance / creation of maintenance risks		Health & Safety	Minor (2)	Unlikely (2)	Low 4
27	Inaccessible call point on bridge to summon help	Design	poor design, not enough access points				Minor (2)	Unlikely (2)	Low 4
9	Damage or impact to pad requirements	Design	Change of crane requirements	injury, damage to precinct, dangerous environment, reputation damage			Minor (2)	Possible (3)	Low 6
13	fall from high embankment areas and securing batters	Design	easy access	potential injury and envirnmental damage	High	Health & Safety	Minor (2)	Possible (3)	Low 6
56	lack of CCTV clarity or signage recognition	Design	insufficient lighting	injury, damage to precinct, dangerous environment, reputation damage			Moderate (3)	Unlikely (2)	Low 6
36	Access to water conduit inside box girder ?	Design	Maintenace access requirements not taken into account.	Infrastructure failure due to lack of maintenance / creation of maintenance risks		Health & Safety	Moderate (3)	Unlikely (2)	Low 6
58	Pedestrian Safety risk - attack or injury between bridges	Design	Not enough lighting on Heirisson Island between bridges	attack or injury between bridges	High	Health & Safety	Minor (2)	Possible (3)	Low 6
47	Injury from thrown objects	Design	People throwing objects off the bridge	injury to people			Moderate (3)	Unlikely (2)	Low 6
39	access to maintain water for taps on bridge	Design	poor planning and design	no access to water taps for maintenance	Low	Legal & Compliance	Minor (2)	Possible (3)	Low 6

							RISK REGISTER		
							Risk Rating		
Risk Reference Number	Project Risk	Project Stage	Causes	Resulting In	Priority	Consequence Category	Consequence Rating	Likelihood Rating	Level of Risk
54	environment causes reduced life of the asset	Design	UV / Heat deteriorating structural components over time	reduced life of the asset, impact on project objectives			Moderate (3)	Unlikely (2)	Low 6
19	risk of the slipping off the slope	Design	wet weather, incorect surface treatment, incorrect incline	Impact on project objectives, reputation damage, injury	High	Health & Safety	Minor (2)	Possible (3)	Low 6
30	event occurs that requires injured people to be evaInjured people need to be evacuation from the structure	Design	accident / medical	if not able to exit in time or safely more harm may be done to injured person. May result in litigation.	High	Health & Safety	Catastrophic (5)	Rare (1)	Medium 7
2	Car accesses and drives across bridge	Design	bridge access not restricted enough - poor design, malfunctioning bollards	injury, damage to precinct, dangerous environment, reputation damage, damage to structure			Catastrophic (5)	Rare (1)	Medium 7
17	People creating excessive vibrations on purpose on the bridge (crowd)	Design	poor design	unenjoyable experience, avoidance of the bridge, injury	High	Health & Safety	Catastrophic (5)	Rare (1)	Medium 7
24	Access to lighting, handrail and feature	Design	Maintenace access requirements not taken into account.	Infrastructure failure due to lack of maintenance / creation of maintenance risks		Health & Safety	Minor (2)	Likely (4)	Medium 8
29	Emergency access to bridge restricted	Design	poor design, not enough access points	emergency situations may have have a lag in response times - results in injury, death, reputation or structural damage			Major (4)	Unlikely (2)	Medium 8
42	Limited or no access for landscaping and maintenance vehicles	Design	poor design, not enough access points	limited maintenance activities can be performed - may damage precinct over time			Minor (2)	Likely (4)	Medium 8
49	shared path clearance under the bridge decks when the sea level rises by 0.9m?	Design	climate change / rising water levels	low clearance height would not meet standards - may impact safety or call for re-design and project modification - $$$	Medium	Environmental	Moderate (3)	Possible (3)	Medium 9
57	injury / harrassment of visitors to the preceinct	Design	lack of adequate lighting	impact on project objectives, loss of visitors to the precinct, reputation damage			Moderate (3)	Possible (3)	Medium 9
23	Access to CCTV / Electrical infrastructure	Design	Maintenace access requirements not taken into account.	Infrastructure failure due to lack of maintenance / creation of maintenance risks		Health & Safety	Moderate (3)	Possible (3)	Medium 9
38	Boating incidents (crashes and near misses)	Design	poor communiaction with local authorities and lack of wardens or signage	collisions and injury			Moderate (3)	Possible (3)	Medium 9
22	Access to in bridge services is restricted	Design	poor design, not enough access points	limited maintenance activities can be performed - may damage precinct and structure over time			Moderate (3)	Possible (3)	Medium 9
8	Cyclist travelling at high speed collide with pedestrians	Design	Poor design, lack of signage, poor wayfinding, public behaviour (not following rules, overtaking)	death or injury, reputation damage			Major (4)	Likely (4)	Very High 16
10	Damage or impact to crane or bridge during and after construction	Design	Extreme weather - wind	injury, damage to precinct, dangerous environment, reputation damage			Catastrophic (5)	Likely (4)	Very High 20
61	Poorly marked bike paths	Design	poor design and incorect wayfinding/signage	injury (bike collision), impact on project objectives, reputation damage			Major (4)	Almost Certain (5)	Very High 20
55	Pedestrians slip on pathways	Design	rain or weather conditions, incorrect material selection	injury to pedestrians			Major (4)	Almost Certain (5)	Very High 20
28	Lightning hitting pylon, deck or cables -> Risk of fire / damage to structure	Design	weather conditions, design of structure, materials	Risk of fire / damage to structure	High	Business Operations	Major (4)	Almost Certain (5)	Very High 20
32	Failure of 1 cable	Operations	manufacturing defect	quality issue	Low	Business Operations	Insignificant (1)	Rare (1)	Low 1
15	Risk of fire - on bridge	Operations	Having a fire for a BBQ at the pause points	percieved poor planning, percieved lack of security and safety	Low	Health & Safety	Moderate (3)	Rare (1)	Low 3
45	throwing rocks from landscaping on to tourist boats or shared path below	Operations	poor security / too much access	injury and litigation	Low	Legal & Compliance	Major (4)	Rare (1)	Low 4
20	Terrorism, blowing a cable or bearing or support etc	Operations	opportunity, access, unhappy stakeholders	potential injury, sturctural damage, environmental damage, lowered community confidence	High	Reputation & Trust	Catastrophic (5)	Rare (1)	Medium 7
25	Risk of fire - bushfire near embankments	Operations	natural casues or arson	impact on safety of environment and community	Medium	Health & Safety	Major (4)	Unlikely (2)	Medium 8
	Incident occuring from colllission or disruption of pedestrian activities by electric quad bikes wrongly accessing the bridge	Operations	Quad bikes can be hired from local hire shop	injury, damage to precinct, dangerous environment, reputation damage			Moderate (3)	Possible (3)	Medium 9