Sensitivity: General

REPORT

Harania Flood Resilience Works - Tennessee Bridge

Coastal and fluvial geomorphic effects assessment

Prepared for
Auckland Council

Prepared by
Tonkin & Taylor Ltd

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Glossary

Terminology	Description
TRB	True Right Bank (looking downstream).
TL	True Left Bank (looking downstream).
MALF	Mean Annual Low Flow. Describes the average amount of water in a river during times of low flow.
ARI	Average Recurrence Interval.
AEP	Annual Exceedance Probability.

Executive summary

Tonkin & Taylor Ltd (T+T) has been engaged by Auckland Council's Healthy Waters to undertake a Coastal and fluvial geomorphic effects assessment related to the proposed Tennessee Bridge upgrade works (the Project) and this has been prepared to accompany a resource consent application for the Tennessee Bridge project under the Severe Weather Emergency Recovery (Auckland Flood Resilience Works) Order 2024.

The Tennessee Bridge project involves removing the current embankment which carries the existing Eastern Interceptor (EI), an approximately 2.6 m diameter reinforced concrete wastewater pipe. The replacement will comprise a new pipe and pipe bridge in the coastal marine area (CMA) to open up the waterway capacity to allow increased flood conveyance.

The coastal hazards and effects assessment concluded that the proposed works will have a negligible impact on coastal hazards. The coastal effects assessed include the occupation of the CMA, the coastal water levels and wave conditions, tidal velocities and tidal channel alignment. The coastal hazards that were assessed are coastal flooding and erosion.

It was analysed that the present-day tidal flows are not obstructed by the presence of the embankment, as the discharge capacity of the two culverts connecting both sides of the embankment is sufficient even for a 100-year ARI coastal event. Coastal flooding extents are therefore not expected to change because of the proposed works. The long-term coastal erosion rate in the project area is expected to be small, because of the sheltered coastal environment of the project area. The coastal erosion rate is dependent on local soil properties and future sea level rise. As these are not expected to change due to the proposed works, the effects on coastal erosion of the proposed works are expected to be minimal.

The existing fluvial geomorphic condition suggests that the reaches directly above and below the Tennessee Ave embankment remain relatively stable, likely due to the dominance of the cohesive marine clays in the bed and lower banks.

As a result of the assessment of geomorphic effects (using T+T modelling outputs) and potential mitigation options, it is deemed that any likely adverse geomorphic effects as a result of the proposed activity can be managed. The geomorphic effects assessed include the likelihood and consequence of bed and bank erosion, scouring and/or undercutting. These effects were then considered with respect to the upstream and downstream environments.

There is no residual geomorphic risk in the upper reaches of the catchment, and a low likelihood for geomorphic effects in the downstream end of the catchment. In general, lateral adjustment is unlikely to result in any adverse effects, except for a small area around the bridge piers, where localised scour may occur. As per the stream plan drawings, rock rip rap has been included in the stream design. Therefore, any erosion risk at the bridge piers is considered to be appropriately mitigated. Effects management actions are not required in the upper reaches of the catchment. At the downstream end of the catchment monitoring for geomorphic effects particularly after flood events in excess of 50-year ARI is considered appropriate.

1 Introduction

1.1 Background

The January 2023 floods, followed closely by Cyclone Gabrielle, marked a period of unprecedented weather challenges for Auckland. Auckland Council is carrying out flood resilience projects with the aim of mitigating flood risk to property through a series of blue-green networks, addressing critical flood-prone areas with sustainable stormwater solutions. The Harania catchment was one of the worst affect areas of Auckland following the January 2023 floods. Healthy Waters identified significant flooding, causing risk to life, and widespread flood damage to homes. This occurred due to poor flood conveyance at the location of the current Tennessee Avenue embankment dam.

1.2 Project Description

A detailed description of the full project works can be found in the Assessment of Effects on the Environment (AEE) report[1].

The Tennessee Bridge project involves removing the current embankment which carries the existing Eastern Interceptor (EI), an approximately 2.6 m diameter reinforced concrete wastewater pipe. The replacement will comprise a new pipe and pipe bridge in the coastal marine area (CMA) to open up the waterway capacity to allow increased flood conveyance. Diversion chambers are required at either end of the new pipe, connecting it to the existing pipe to facilitate the change over from the old pipe to the new pipe bridge diversion.

1.3 Scope of works

Tonkin & Taylor Ltd (T+T) has been engaged by Auckland Council's Healthy Waters to undertake a Coastal and fluvial geomorphic effects assessment related to the proposed Tennessee Bridge upgrade works (the Project) and this has been prepared to accompany a resource consent application for the Tennessee Bridge project under the Severe Weather Emergency Recovery (Auckland Flood Resilience Works) Order 2024. The purpose of this Coastal and fluvial geomorphic effects assessment is outlined in Section 1.3.1 and 1.3.2.

1.3.1 Coastal assessment

The primary focus of this work is to undertake a coastal assessment of the study area, focused on the part of Harania Creek that is located within the CMA. The coastal assessment is comprised of the following:

- Analysis of environmental conditions that are relevant for the coastal assessment, including tidal water levels, wave climate, tidal flows and tidal channel alignment;
- Analysis of coastal inundation and erosion hazards; and
- Assessment of potential coastal effects within the CMA boundary, resulting from the Tennessee Bridge works.

1.3.2 Fluvial geomorphic assessment

The primary scope of this work is to undertake a geomorphic assessment of the study area as shown in Figure 3.1. The preliminary geomorphic assessment comprises the following:

[1] *Harania Flood Resilience Works – Tennessee Bridge Assessment of Effects on the Environment*, Beca Limited, November 2024.

- Historic aerial analysis of geomorphic and landscape evolution of Harania Creek;
- Assessment of current geomorphic condition of Harania Creek, including erosion susceptibility;
- Assessment of potential geomorphic effects of the proposed Tennessee Bridge works;
- Comment on the potential for cumulative geomorphic impacts of the proposed Harania Creek embankment upgrade on Tennessee Bridge works; and
- Provide recommendations to manage the potential geomorphic effects, following the effects management hierarchy.

The primary purpose of this geomorphic assessment is to assess the potential effects of the proposed upgrades on geomorphic processes in the immediate vicinity of the Tennessee Avenue dam embankment. However, streams are longitudinally connected, so secondary geomorphic impacts (such as bank erosion triggered by bed lowering) as a result of any potential effects from the works will also be assessed.

The geomorphic assessment is focused on the creek environments outside of the CMA boundary (as mapped in Auckland Council GeoMaps) and assed potential fluvial geomorphic effects within the CMA. Coastal effects within the CMA are considered in the coastal assessment section of this report, as identified in Section 1.2.1 above.

This assessment has been informed principally by policy direction of the AUP, and the direction set out in the objectives and policies of the National Policy Statement for Freshwater Management 2020 (NPS-FM) and the NES F. The following key matters have been considered:

- Hydrological and hydraulic effects including retention of sufficient creek flow conveyance capacity;
- Bed and bank erosion, scouring or undercutting, and land instability effects;
- The effects on downstream stream or wetland environments;
- Any effects arising from any permanent modification in creek state or function.

2 Methodology

2.1 Terminology

This report describes the results of both a fluvial geomorphic and a coastal assessment. In fluvial assessments, we often use the terms upstream and downstream to indicate locations or directions in the area under consideration. However, these terms are more difficult to interpret from a coastal point of view, as the meaning of these terms will depend on the direction of the tide, which flows in both directions.

To avoid confusion, we will consistently use the terms upstream and downstream throughout the report. Note that these terms are related to the direction of fluvial flows and not to the direction of tidal flows.

2.2 Data sources

The geomorphic assessment was primarily a desktop-based analysis, supported by a single field visit to assess the existing condition of the Harania Creek. The site visit was undertaken on 12 July 2024 by two T+T fluvial geomorphologists. A summary of data sources and their purpose is provided in Table 2.1.

Table 2.1: Data used to undertake the preliminary geomorphic assessment

Data source	Layer accessed	Purpose
Auckland Council Geomaps	• Overland flow paths. • Catchment delineation. • Stormwater infrastructure. • Historic imagery.	Baseline catchment data.
Auckland Council/T+T Flood Modelling	• Discharges, water levels and velocities for a 100-year flood event for the existing condition and post Tennessee Bridge project upgrade. • 2016 LiDAR derived digital elevation model (DEM) (Auckland Council), referenced to NZVD-16.	High level identification of areas where there may be increases in velocities to help inform potential risk to creek stability. This is only to be used as an indicator, as the flood model was not intended for this use. Estimated peak flood flows for a range of flood events that are able to generate geomorphic change.
NIWA River Maps	• Low and mean flow statistics. • Upstream catchment area.	Catchment and reach scale geomorphic, and ecological parameters.
GNS Surface Geology 250k	• Surface geology for the catchment and survey reach.	Underlying geology of the catchment and the survey reach which has the potential to exacerbate or moderate geomorphic rates of change.
Retrolens[2]	Historic aerial images to assess the effect of the construction of the embankment on the tidal	Historic stream character and planform to inform potential future stream change.

[2] Retrolens, 2024

Data source	Layer accessed	Purpose
	channel alignment, channel planform and landscape evolution. The following datasets were used: • 1940 aerial image. • 1958 aerial image. • 1972 aerial image. • 1988 aerial image.	
Preliminary geotechnical assessments	• Cross-sections. • Soil profiles and soil properties for catchment.	Generalised description and estimated depth of potential materials forming banks and stream bed.
Land Information New Zealand (LINZ)	• Digital Elevation Model (DEM). • Catchment delineation.	Identify catchment and reach scale geomorphic drivers of change.

Note: geospatial data uses AVD-46 datum.

2.3 Erosion susceptibility

Bed and bank erosion processes are assessed separately to differentiate between reaches that are likely to experience on-going incision, or those reaches that are more likely to experience bank erosion, independent of bed erosion.

The intention of differentiating between bed and bank erosion is to help guide management actions, as those reaches displaying a high bed erosion potential may require different interventions than those reaches that are displaying bank erosion independent of bed processes. This should help to get the best outcomes for geomorphic effects management from the outset of the project.

Erosion susceptibility values have been grouped into ranges and then characterised as “Low” erosion susceptibility, “Moderate” erosion susceptibility, “High” erosion susceptibility or “Extreme” erosion susceptibility. As the erosion susceptibility assessment method does not solely rely on active erosion, this rating helps to identify those reaches that may experience severe and ongoing erosion in the future if catchment conditions change, rather than waiting for that erosion to occur before management actions are initiated. In addition to the erosion susceptibility values, stream power[3] has also been calculated and used to inform the assessment of effects from predevelopment to post development.

[3] Note that a unit stream power of 300 W/m is the threshold for catastrophic modification.

3 Environmental conditions

3.1 Site and catchment context

The Harania Creek catchment is a sub catchment (Figure 3.1) of the Mangere Inlet catchment. It drains a catchment area of 4.4 km^2 and flows for approximately 1,800 m before discharging into the Harania Creek Tidal Flats[4]. The Tennessee Creek is a principal tributary of the Harania Creek joining the Harania Creek approximately 280 m above the embankment (see reaches and indicative catchment outline on Figure 3.1).

[4] NZ River Maps (niwa.co.nz)

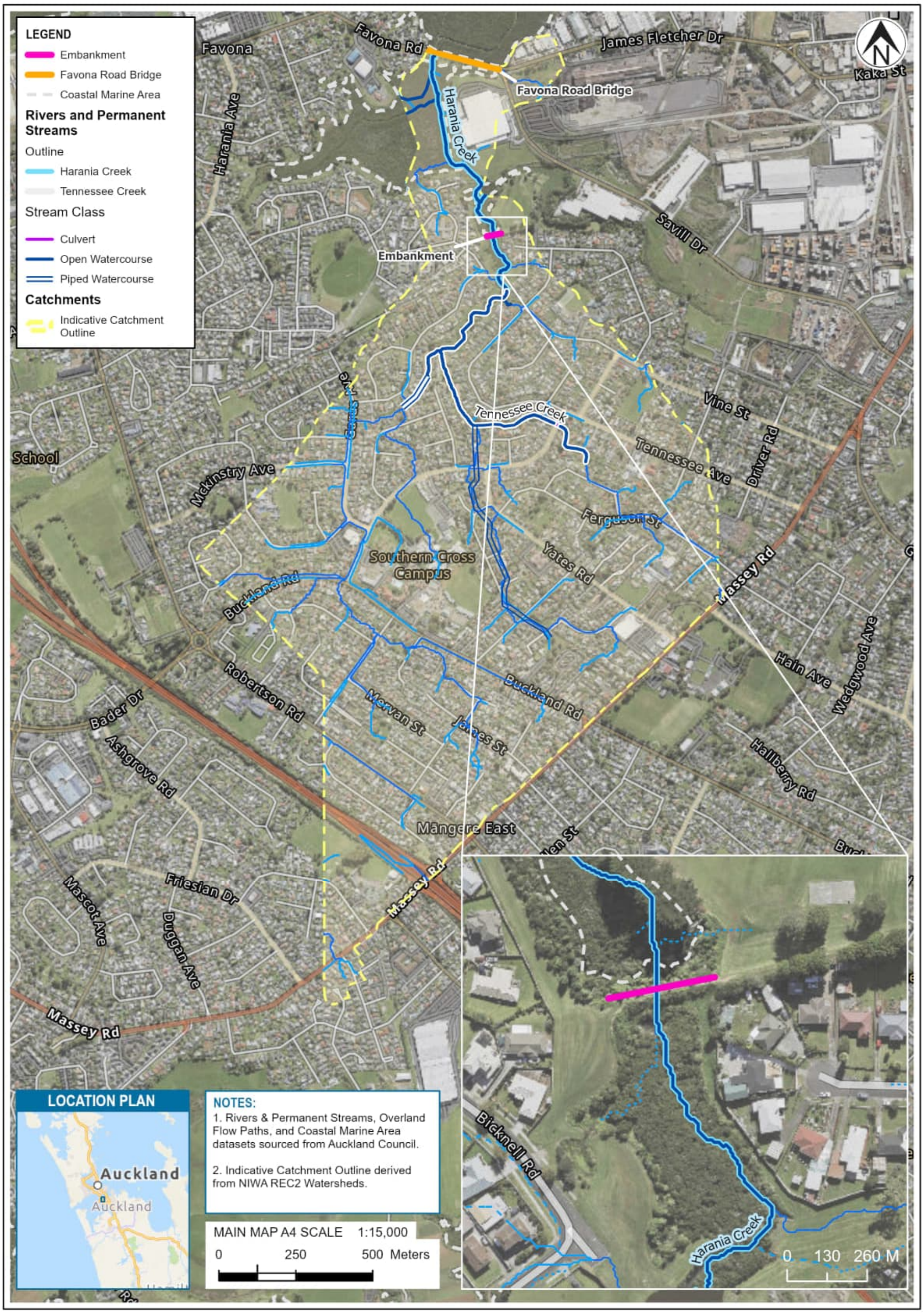

Figure 3.1: Coastal Marine Area (white dotted line), Harania Stream catchment (yellow dotted line). The inset indicates the immediate vicinity of the current embankment (pink line).

The proposed Tennessee Bridge works location is between Blake Road Reserve to the east and Bicknell Road to the west (Figure 3.1). The catchment has experienced rapid urban development from 1940 to 2001, which has increased the catchment imperviousness. Discharge and streamflow velocities in Tennessee Creek, Harania Creek and their tributaries would likely have increased in response to the catchment changes.

Currently, there is a constructed embankment that traverses Harania Creek (i.e., connecting the eastern and western banks of the creek). Two large culverts of ~1 m diameter and ~20 m length each are located at the bottom of the embankment, allowing water to flow through the embankment (Figure 3.1). The embankment, which is proposed to be removed and replaced with a pipe bridge, runs perpendicular to Harania Creek and supports the Eastern Interceptor wastewater line.

The lower part of Harania Creek is located within the CMA. Within this part of Harania Creek, the tidal flow direction reverses with the tide; ebb flows through the creek are directed towards the north (downstream) and flood flows are directed towards the south (upstream). Following heavy rainfall events, storm flows move downstream through the stormwater network and open sections of channel and into the Manukau Harbour. The width of Harania Creek varies along its length but increases towards the Manukau Harbour. Below the embankment and within the CMA, the Harania Creek becomes a clear tidal channel that meanders over the width of the valley bottom. During a field visit on 12 July 2024, the surface bed material at both sides of the embankment was observed to consist of silty sand with cohesive marine clays approximately 0.1 m below the fine silty sand layer.

There are no measures in place to stop the culverts from blocking from time to time. During the field visit of the project area, debris was observed in the culvert opening on the upstream side. Photographs of the debris observed in the reach are included in Appendix A.

Before January 2023, mangrove trees were abundantly present on both sides of the tidal channel, both upstream and downstream of the embankment. At the downstream side of the embankment, mangroves were partly removed during the emergency works following the floods in January 2023. Mangrove growth was observed to be continuous on the upstream side of the embankment with a tidal channel bisecting the mangrove growth. Mangrove trees are not present in the tidal channel itself (see Figure 3.2). Further pictures of the project site can be found in Appendix A.

Figure 3.2: Mangroves at the southern site of the embankment in Harania Creek. The active channel width is in white.

3.2 Geology

River behaviour is directly influenced by imposed catchment boundary conditions, i.e. valley shape, slope and topography which are dictated by the regional geology. Geologic and lithologic properties underlying a catchment, determine the erodibility of a system and in turn drives rates and patterns of river change.

The Harania Creek catchment is underlain by mud, sand, silt and clay (Pliocene to Holocene Takanini Formation) with a dense sand layer (approximately 4 m thick) at about 8 m below the existing 100-year floodplain surface (Figure 3.3). The surface geology is then underlain by East Coast Bays Formation (ECBF) at about 18-20 m below the existing 100-year floodplain surface. Detailed borehole logs for the true right bank from the 100-year floodplain terrace surface to a depth of 25 m are provided in the Harania Flood Resilience Works – Tennessee Bridge Geotechnical Resource Consent Assessment Report (October 2024. T+T Ref 1017033.2002).

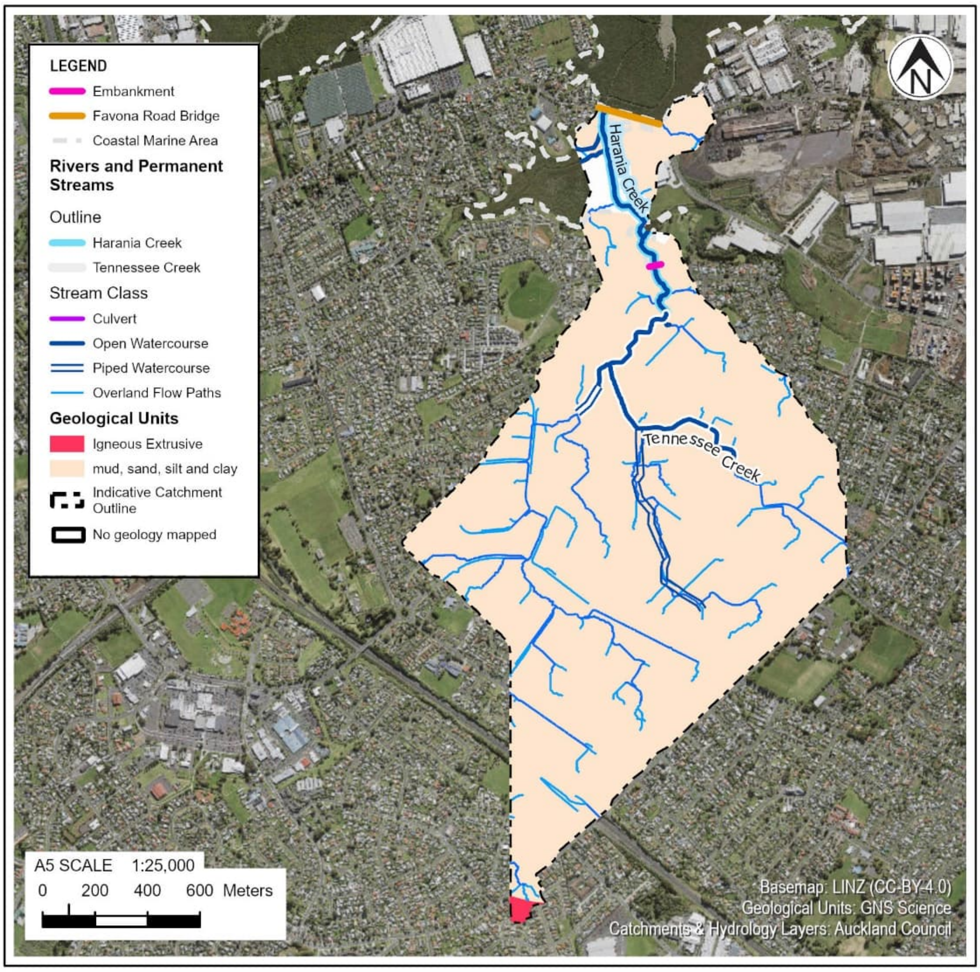

Figure 3.3: Geology map of the Mangere Catchment with the indicative catchment outline of the Harania Stream in black.

3.3 Historical context

Harania Creek is a low gradient stream which has undergone significant agricultural and urban modification from the 1940s (Figure 3.4).

In 1940, the lower end of the Harania Creek catchment was predominantly tidal (i.e., upstream of the 1964 eastern interceptor embankment). By 1958, residential development increased to the east of the project area. Construction of the Eastern Interceptor (i.e. embankment) was completed in 1964. As a result of the embankment limiting storm flows, the reach upstream of the embankment began to aggrade. Downstream of the embankment, flood flows would have been significantly attenuated, with prolonged high velocity flows from the culvert. This is reflected in the 1972 aerial where there is widening of the channel above of the embankment but contraction of the channel downstream of the embankment.

In 1988, an area directly to the south of Favona Road Bridge was reclaimed, contracting the lower end of the channel further and resulting in a lateral channel shift to the west. The channel upstream of the embankment was straightened in 1988, potentially to facilitate increased urban development. As a result of the increased imperviousness of the catchment, and the straightened channel, it is probable that there was an increase in the frequency and magnitude of flood events (see 2024 Harania Creek location in pink line in Figure 3.1).

Despite the likely increase in run-off associated with an urbanising catchment, the channel upstream of the embankment appears to have contracted between 1977 and 2024. This is likely a result of the embankment induced ponding and aggradation. Additionally, the colonisation of mangroves across the channel floor likely promoted aggradation and aided in channel contraction.

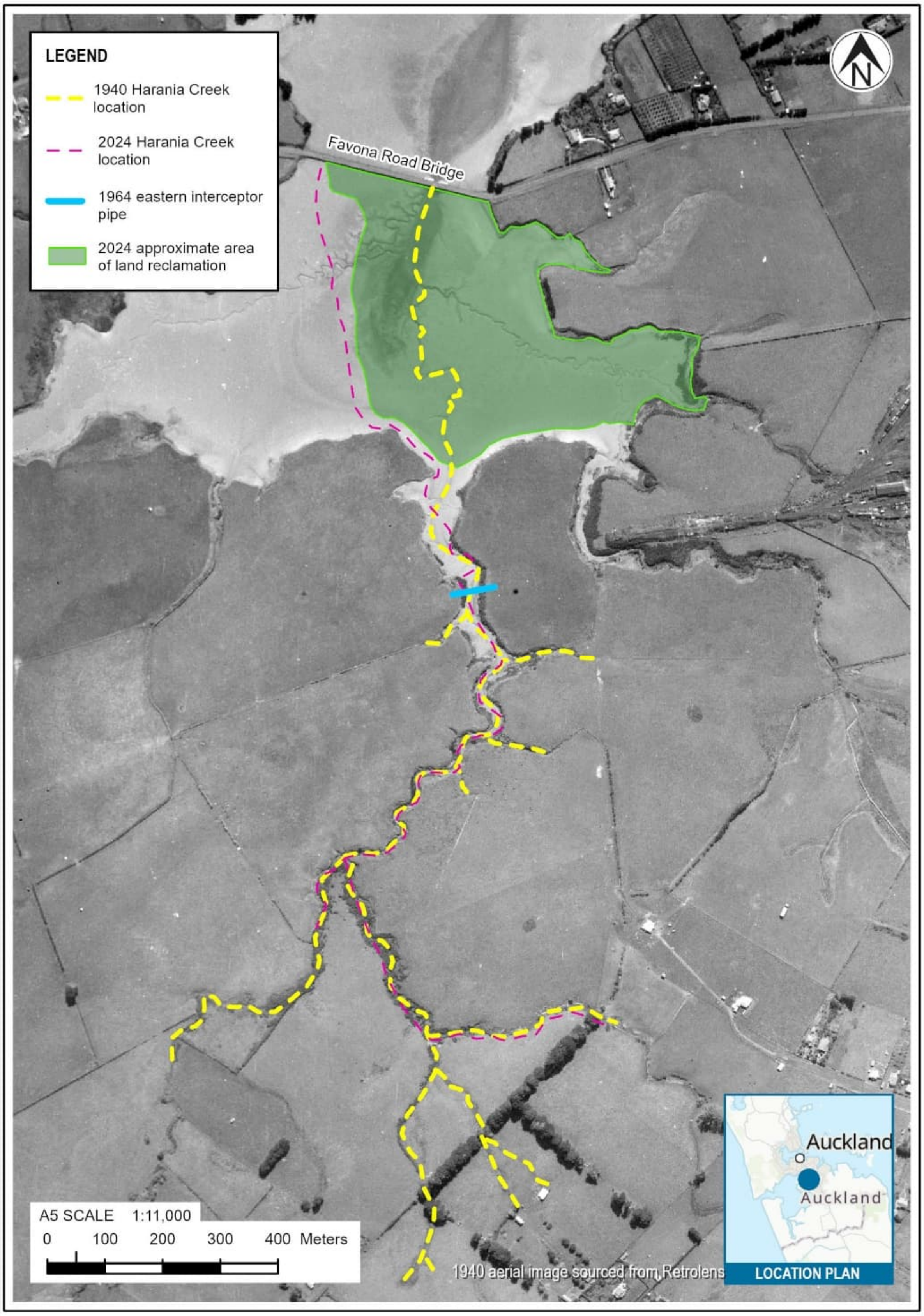

Figure 3.4: Historical catchment evolution of Harania Creek. Base aerial is 1940. Source: Retrolens.

In the direct vicinity of the embankment (both on the upstream and downstream side), the tidal channel alignment in 1958 (red line in Figure 3.6) has shifted to the west compared to the channel alignment in 1940 (blue line Figure 3.6). This lateral adjustment is likely a result of the construction of the embankment. From the figure, it can be seen that the effect of the embankment on the location of the tidal channel was mostly local; the further away from the embankment (both on the upstream and downstream side), the smaller the lateral change in tidal channel alignment. Furthermore, small water pools have formed at both sides of the embankment, where the water pool at the downstream side is significantly larger than at the upstream side. During the field visit on July 12th, it was observed that the bottom level of both culverts is located slightly higher than the bottom of these water pools.

In Figure 3.7, the 2016 LiDAR of the project area is shown. On top of the 2016 LiDAR, the 1958 tidal alignment is plotted as a pink line. From this Figure, it can be seen that after the construction of the embankment, the tidal channel alignment has remained very stable and has hardly changed over ~60 years.

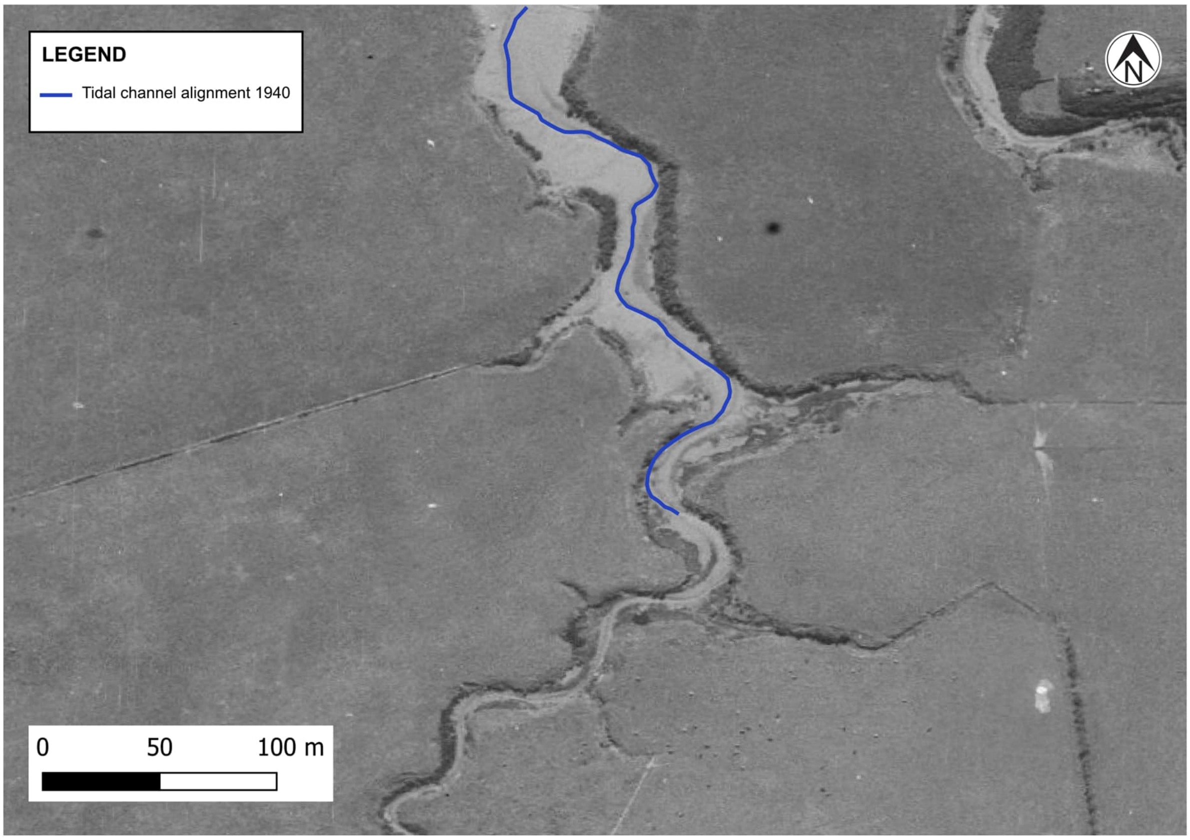

Figure 3.5: Historical aerial image from 1940, indicating the tidal channel alignment in blue. (Aerial image is sourced from (Retrolens, 2024)).

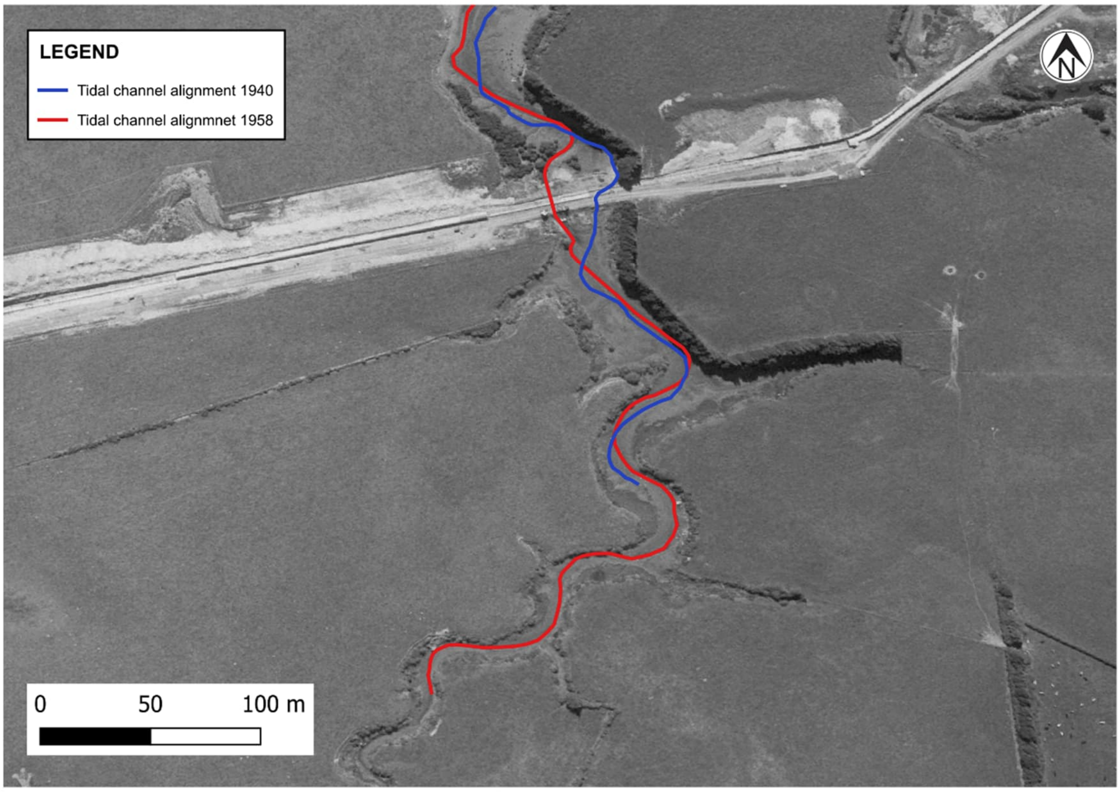

Figure 3.6: Historical aerial image from 1958, indicating the tidal channel alignment in both 1940 (blue) and 1958 (red). (Aerial image is sourced from (Retrolens, 2024)).

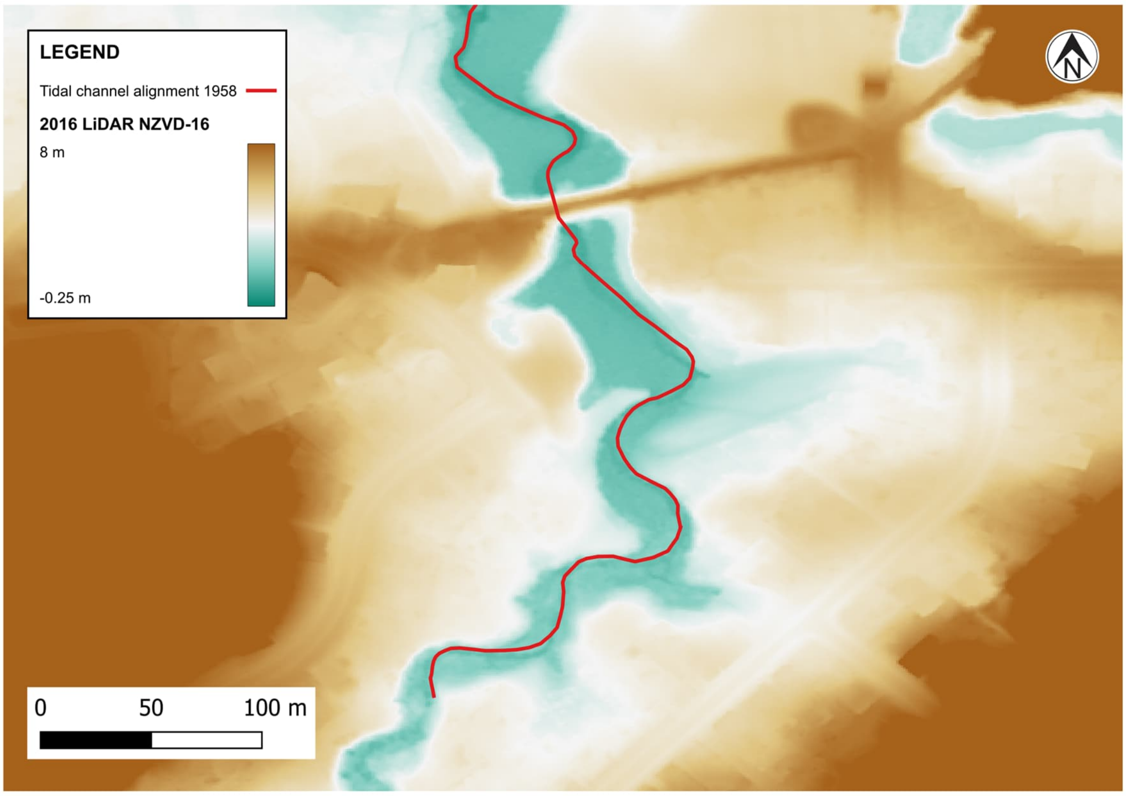

Figure 3.7: 2016 LiDAR map from the project area. The red line indicates the tidal channel alignment from 1958 (after construction of the embankment).

3.4 Topography and bathymetry

Figure 3.8 shows the topography of the surrounding area inferred from 2016 LiDAR data. This LiDAR dataset was corrected for the presence of vegetation. However, since there are no reference points available from a local topographic survey, the vertical accuracy of the dataset is unknown. Furthermore, cross-sections at both sides of the current embankment are depicted in Figure 3.9 and Figure 3.10, showing that the western side of Harania Creek is higher than the eastern side. The elevation difference is approximately 0.5-1.5 m.

From the cross-sections in Figure 3.9 and Figure 3.10, it can be seen that the invert level of the tidal channel in the creek is located at an elevation of ~1.25 m NZVD-16 at both sides of the embankment.

Finally, the area within the Mean High-Water Springs (MHWS) line was plotted over the elevation map, indicating that the coastal extent would naturally extend past the embankment and into the lower catchment of Harania Creek.

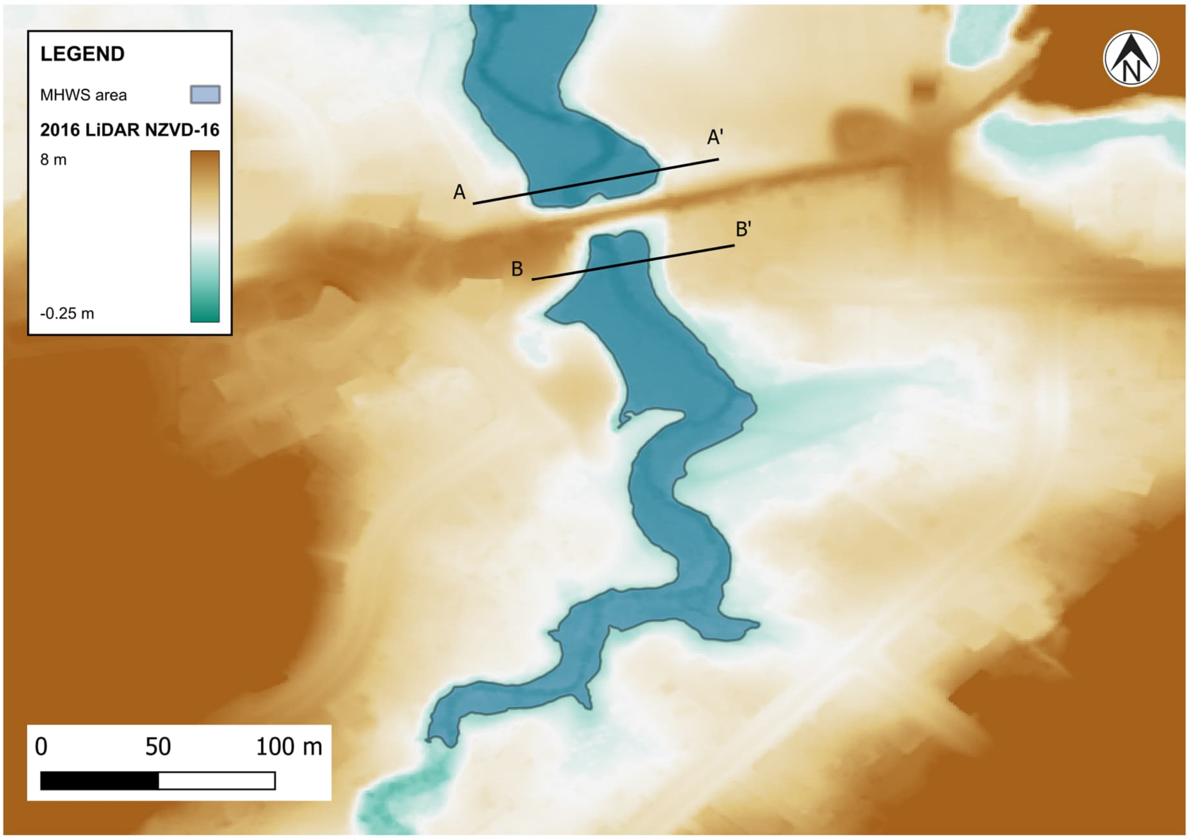

Figure 3.8: Topography of the Harania Creek, the blue area represents the MHWS-line.

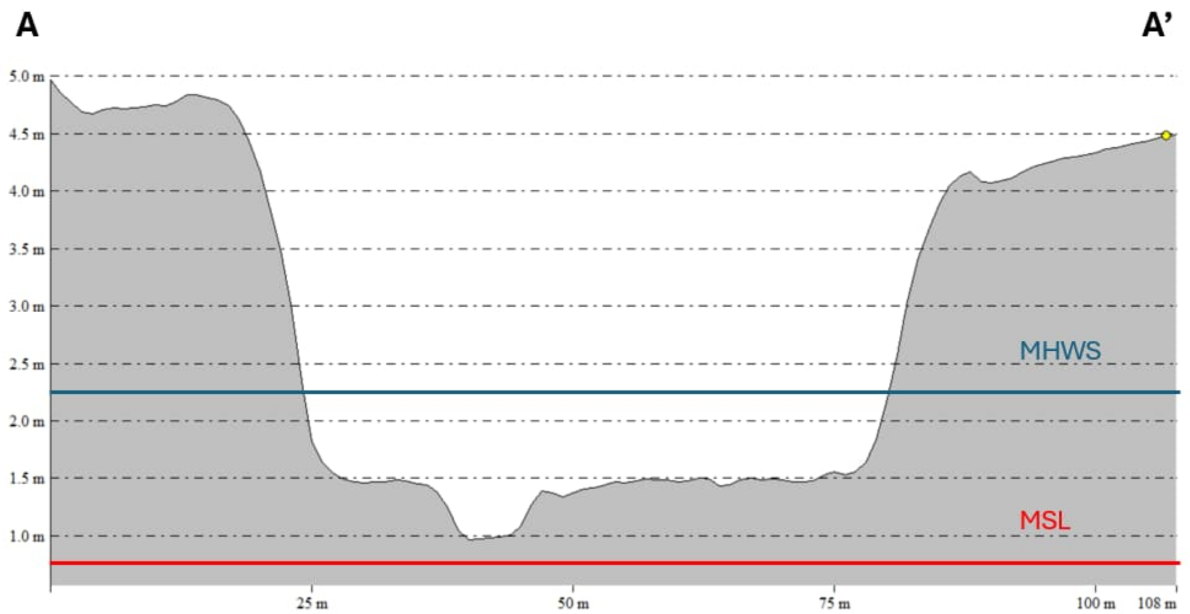

Figure 3.9: Cross-section AA'. Vertical axis shows elevation in metres NZVD-16.

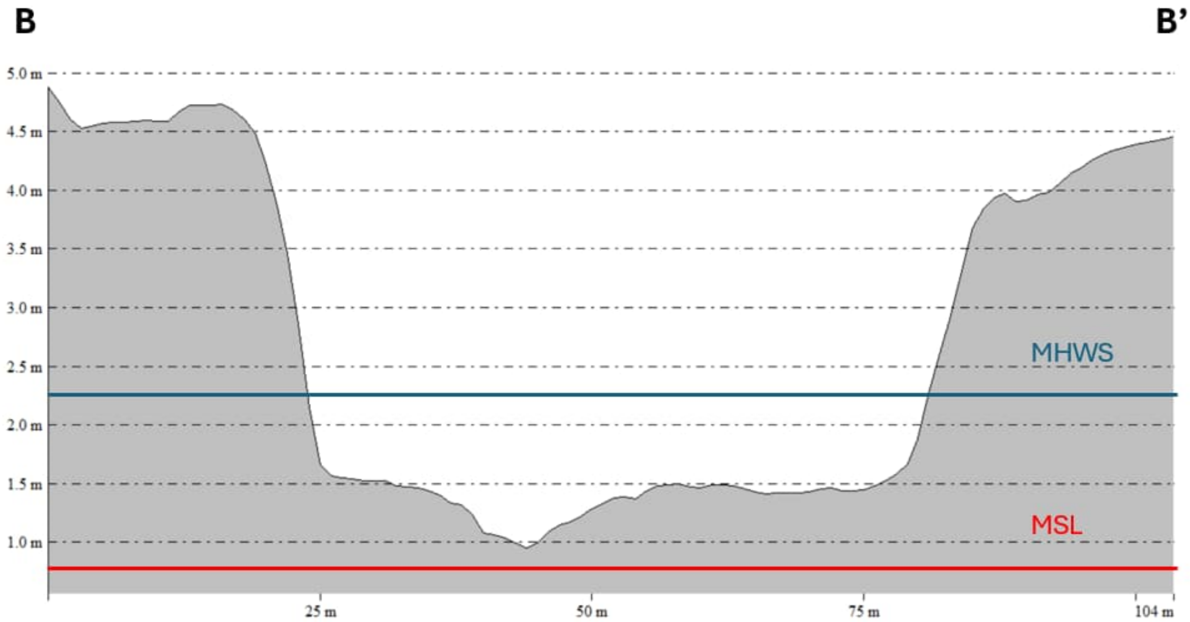

Figure 3.10: Cross-section BB'. Vertical axis shows elevation in metres NZVD-16.

3.5 Water levels

Water levels at Harania Creek are influenced by the astronomical tide, local storm surge, future sea level rise, overland flows and stormwater inputs. Each of these components will be elaborated on in the following sections.

3.5.1 Astronomical tides

Harania Creek is subject to a diurnal tide with a spring tide range of approximately 3.6 m and a neap tide range of 1.9 m. The tidal levels at Harania Creek with respect to both CD and NZVD-16 are listed in Table 3.1[5].

Table 3.1: Tidal levels Harania Creek (LINZ, 2014)

Tide	Tidal level (m CD)	Tidal level (m NZVD-16)
Highest astronomical Tide (HAT)	4.54	2.62
Mean High Water Spring (MHWS)	4.18	2.26
Mean High Water Neap (MHWN)	3.33	1.41
Mean Sea Level (MSL)	2.43	0.51
Mean Low Water Neap (MLWN)	1.45	-0.47
Mean Low Water Spring (MLWS)	0.56	-1.36
Lowest Astronomical Tide (LAT)	0.12	-1.80

3.5.2 Storm surge

Regional weather patterns that influence wind and atmospheric pressure can cause a storm surge phenomenon that raises water levels above the astronomical tide levels. These storm surge levels were assessed by (Stantec, 2020). Tidal and extreme water levels for Harania Creek with an Annual Exceedance Probability (AEP) of 1%, 2%, 5% and 20% (equivalent to an Annual Recurrence Interval of 100, 50, 20 and 5 years respectively) are listed in Table 3.2.

Table 3.2: Extreme water levels at Harania Creek (Stantec, 2020)

Recurrence Interval	Tidal level (m NZVD-16)
100-year ARI extreme water level	3.30
50-year ARI extreme water level	3.19
20-year ARI extreme water level	3.04
5-year ARI extreme water level	2.86

3.5.3 Long-term sea level rise

Due to climate change, mean sea levels are expected to rise in the future. At the same time, land positions may move up or down vertically (i.e. Vertical Land Movement (VLM), depending on the location of interest. These processes together account for the Relative Sea Level Rise (RSLR) at a given location. Both the SLR and RSLR values corresponding to different climate change scenarios taken from (NZSeaRise, 2024) are listed in Table 3.3. Expected RSLR values for 2130 (assuming a 100-year design life) are 0.76, 0.87, 1.08, 1.35, and 1.52 m for climate scenarios SSP1-1.9, SSP1-2.6, SSP2-4.5, SSP3-7.0, and SSP5-8.5 respectively.

Table 3.3: SLR and RSLR under different climate scenarios for the period 2030-2130 for Harania Creek

	2030	2050	2060	2080	2130

[5] For the Manukau Harbour, Chart Datum at the port of Onehunga is 2.201 m below AVD-46 (Stantec, 2020)

Climate scenario	SLR	RSLR	SLR	RSLR	SLR	RSLR	SLR	RSLR	SLR	RSLR
SSP1-1.9	0.02	0.04	0.09	0.17	0.12	0.24	0.20	0.38	0.42	0.76
SSP1-2.6	0.02	0.04	0.11	0.19	0.16	0.27	0.26	0.44	0.53	0.87
SSP2-4.5	0.03	0.05	0.13	0.22	0.20	0.31	0.33	0.51	0.73	1.08
SSP3-7.0	0.03	0.05	0.15	0.24	0.23	0.34	0.40	0.59	0.95	1.35
SSP5-8.5	0.03	0.05	0.17	0.25	0.26	0.38	0.47	0.66	1.17	1.52

Note: Values obtained from (NZSeaRise, 2024) at ID point 3280. The values listed in the table correspond to p50. The values in NZSeaRise are relative to 2005; the values in this this table are corrected such that they are relative to present day water levels.

3.6 Waves

Due to the sheltered location of the project site, the shallow water depths and the presence of abundance vegetation, waves do not penetrate as far inland as the project site. There is no wave climate at the project site, and waves will not be considered in the remainder of this coastal effects assessment.

3.7 Coastal hydrodynamics

During a typical tidal cycle, tidal flows will occur in both directions in Harania Creek. Flood flows will occur towards the south, and ebb flows towards the north of the creek. In Figure 3.11, the water level record from Onehunga Port is shown for the year 2024 (Auckland Council, 2024). From this Figure, it can be seen that the elevation in Harania Creek just upstream of the embankment is higher than ebb tide water levels. Therefore, only the top of the tide will reach as far inland as the embankment in Harania Creek.

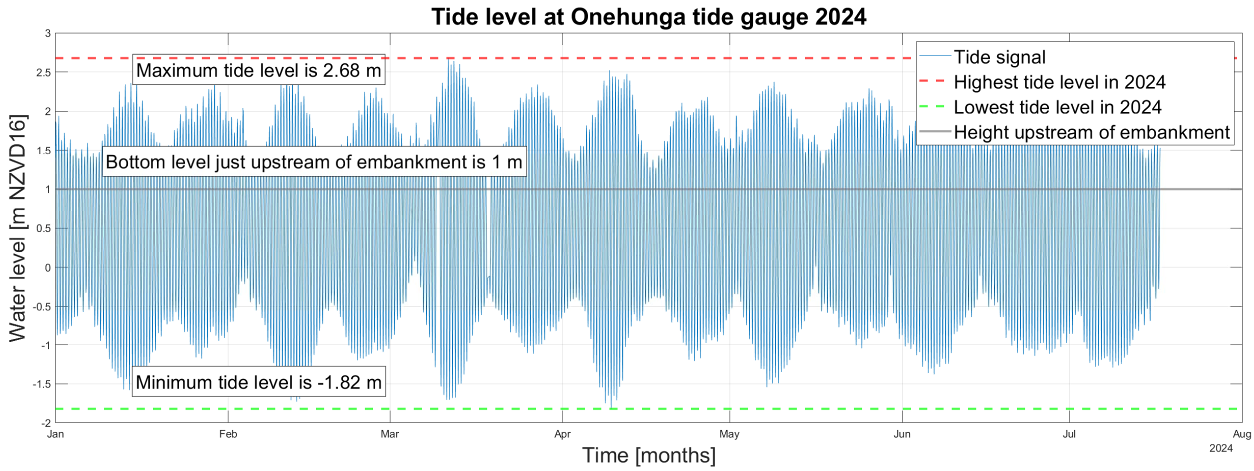

Figure 3.11: Water level record at Onehunga Harbour (Auckland Council, 2024).

To see whether the present-day tidal flows are obstructed by the embankment in Harania Creek, the tidal flows were compared to the discharge capacity of the culverts underneath the embankment. This analysis considers the situation at a present-day 100-year ARI storm tide level of 3.30 m NZVD-16.

This water level would correspond to a volume of ~14,000 m^3 that would flow through the culverts under the embankment (based on the 2016 LiDAR for Auckland). This volume was calculated by calculating the total volume of water in the area upstream of the embankment, assuming a water level of 3.30 m NZVD-16 everywhere. Assuming a 2.5-hour period till high tide peak at the location of the embankment, this results in an (average) expected flow of ~1.6 m^3/s through the culverts. Next, the discharge capacity of the culverts was calculated using Mannings' equation for culvert flow:

$$Q = \frac{1.486}{n} AR^{2/3}S^{1/2}$$

Where:

Q = Discharge [m^3/s]

m = Manning's coefficient [$s/m^{1/3}$]

A = Cross-sectional area of flow [m^2]

R = Hydraulic radius [m]

S = Slope [m/m]

Applying this equation results in a discharge capacity in the range of 3.5 – 12 m^3/s (depending on the roughness of the culverts and the degree of blockage). Since the expected tidal flow under the 100-year ARI storm tide level is much lower than the capacity of the culverts, it can be concluded that currently, the tidal flows through Harania Creek are not obstructed by the presence of the embankment.

3.8 Geomorphic existing condition

On 12 July 2024, two T+T fluvial geomorphologists carried out a site visit to understand the existing geomorphic condition of the Harania Creek directly upstream the embankment (HS_US1) and directly downstream of the embankment (HS_DS1) during low tide at approximately 0830 hr. Previous investigations upstream of these two locations were completed in 2022 and 2023 (Tenn_05 and Tenn_06) Figure 3.12. This section summarises the existing geomorphic condition of the Harania Creek.

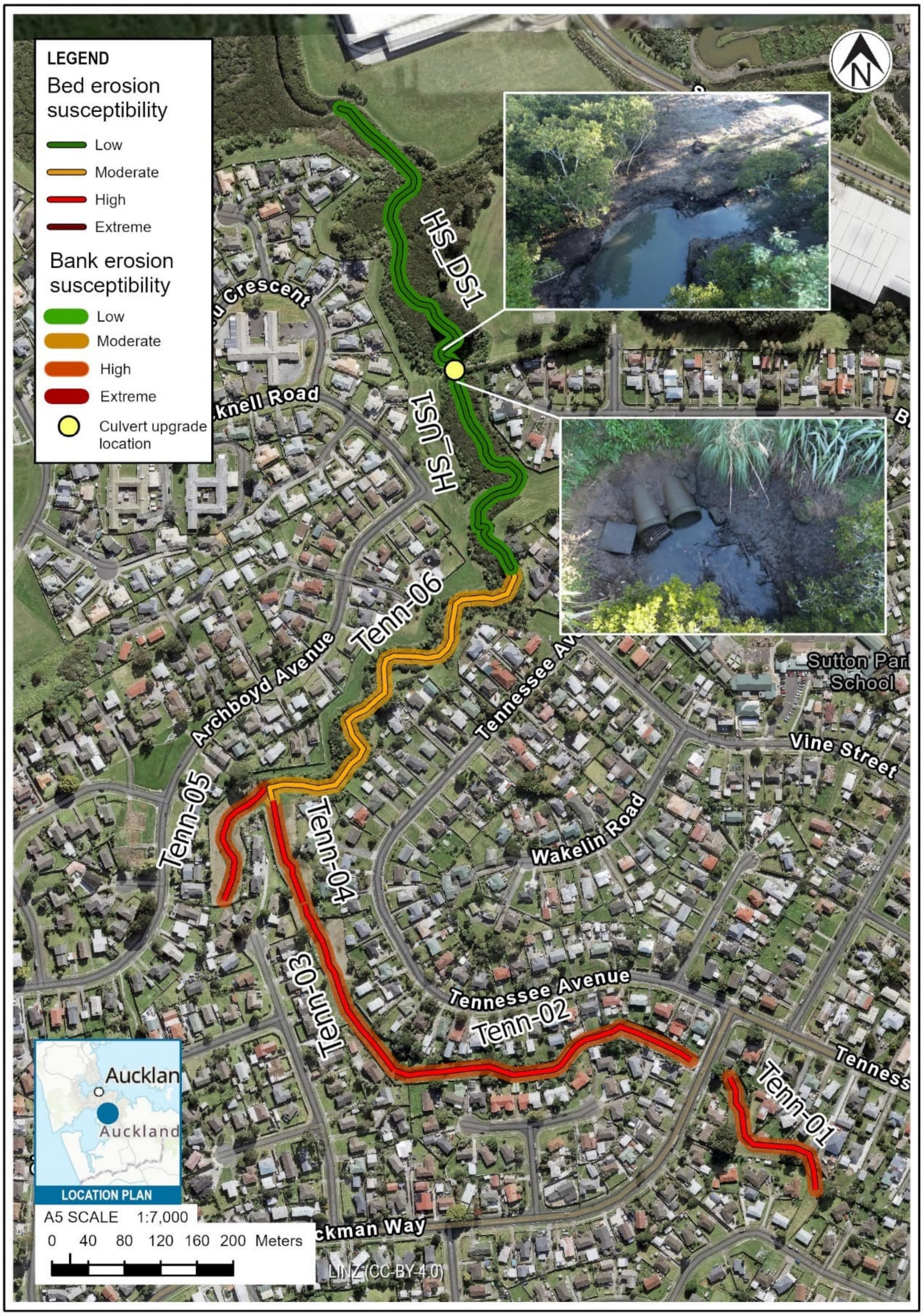

Figure 3.12: Erosion susceptibility of reach HS_US1 and HS_DS1 including the Tenn01 to 05 reach data collected in 2022 and 2023. Drone images taken on 17 July 2024 during low tide.

3.8.1 Summary of previous investigations

The LEAD Alliance carried out work upstream of the proposed embankment upgrade location in 2022[6] and 2023[7]. See Figure 3.12 for Tenn_01 to Tenn_06. The key findings observed in the 2022 and 2023 investigations for the relevant sections of Harania and Tennessee Creek are described in the following sub-sections.

3.8.1.1 Tenn_05

Tenn_05 flows for approximately 150 m from a stormwater outfall to the meeting of the Tennessee Creek tributary (upstream extent of Tenn_06). The channel is partly confined between terraces, is a widened u-shape and has a relatively gentle slope. Bank full bank height is approximately 2 m and bank full width is 13 m, the low flow width is approximately 4 m and low flow bank height is 0.5 m. Bed and bank material is predominantly sand and silt. Using NIWA Mean Annual Flood (MAF) discharge of 2.8 m^3/s and assuming that flows remain confined to the bankfull channel, Tenn-05 has an average specific unit stream power of 3 W/m^2 (Figure 3.13). At a 100-year ARI flood flow velocity of approximately 1.37 m/s, the Hjulström curve indicates that the reach is likely to undergo erosion and transport of clay and silt sized sediments. Indicative stream power threshold for movement of gravels is between 1.5-16 (W/m^2)[8]. This suggests that the unit stream power may be slightly over-estimated, and / or that as downstream ponding increases, sediment transport may reduce.

Minimal erosion features were observed in reach Tenn_05 during the 2022 site visit, although some undercutting was observed on both TRB and TLB. This is consistent with the stormwater outfall upstream of the reach conveying high velocity flows combined with the low width to depth ratio of the active channel. The low width to depth ratio suggests that the reach has widened and infilled in the past but may have reached some form of equilibrium. Because of this, the bed and banks were characterised as having high bed and bank erosion susceptibility, but the reach was not actively eroding in 2022. This reach was not reassessed after the January 2023 flood event.

As a result of these geomorphic observations, the bed and banks were characterised as high bed and bank erosion susceptibility.


Photograph 3.1: LH: Looking upstream towards the stormwater outfall. Note low width to depth ratio of the channel. RH: Looking downstream from the stormater outfall. Photos taken 30 March 2022.

[6] LEAD Alliance. (2022). DTOC38 Mangere East, Tennessee Stormwater Catchment Plan: Ecological technical input. T+T Ref: 1007708.2105.
[7] LEAD Alliance. (2023). Tennessee Catchment Stormwater Study. T+T Ref: 1007708.2105.
[8] Indicative stream power thresholds from Fryirs and Brierley (2012). *Geomorphic analysis of river systems: an approach to reading the landscape*. John Wiley & Sons.

3.8.1.2 Tenn_06

Tenn_06 is approximately 460 m long and begins at the confluence of Tenn_05 and Tenn_04. This reach has a moderate sinuosity and a partly confined valley setting. Bank full height is 4 m, bank full width is 12 m, low flow bank height is approximately 0.5 m and low flow width is 5 m.

Under the existing condition[9] scenario, the undersized culverts at the Harania creek embankment (HS_US1 and HS_DS1) create a backwater effect[10] upstream of the culverts which extends into Tenn_06. It is likely that the backwatering has created a depositional environment which could have resulted in blankets of unconsolidated silts forming on the bed and lower banks. This was unable to be confirmed during the site visit in 2022 due to the presence of standing turbid water.

Using NIWA MAF discharge of 2.8 m^3/s and assuming that flows remain confined to the bank full channel, Tenn-06 has an average specific unit stream power of 3 W/m^2 (Figure 3.12). Indicative stream power threshold for movement of sediment up to the size of gravels is between 1.5-16 (W/m^2)[11]. The 100-year ARI flood flow velocity is approximately 0.22 m/s, which is only just at the threshold of the erosion silt sized sediments based on the Hjulström curve and confirms the backwater effect occurring in this reach during high magnitude low frequency flood events. This suggests that the MAF sized flood events potentially have a higher erosive power than the high magnitude, lower frequency events.

In the 2013 Watercourse Assessment Report[10] 20-40% of the reach was identified as having active erosion, but only some minor undercutting on the TRB and TLB was observed during the 2022 site visit. This reach was not reassessed after the January 2023 flood event. Regardless, this suggests that localised erosion may occur in Tenn_06 in response to particular environmental conditions (e.g. certain flood flow and tailwater conditions, and possible vegetation condition).

The combination of the reduction in observed active erosion in the reach and possible blankets of unconsolidated silts on the bed and lower banks, suggests that Tenn_06 is a low energy environment, but may experience higher energy conditions during flows less than the MAF. Therefore, the bed and banks were characterised as moderate erosion susceptibility.

Photograph 3.2: LH: Looking across the stream at the undercut banks. RH: Looking downstream, note undecut banks and inset benches. Photos taken 30 March 2022.

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[9] BASE and TB01 scenario from T+T model version: run conducted in August 2024 titled *HaraniaBGN 202408_v05.*

[10] Young, D., Hartnett, C., Jackson, S., and Meijer, K. (2013) Māngere Inlet Watercourse Management Plan. Prepared by Morphum Environmental Ltd for Auckland Council.

3.8.2 HS_US1

Reach HS_US1, flows for approximately 300 m with an active bed slope of approximately 0.1% (0.0015 m/m). The reach is located in a partly confined valley setting[11]. The near vertical banks (Approximately 0.9 m high) and gentle bed slope are predominantly composed of silts and cohesive clays of marine origin. The reach is characterised by a 1 m wide u-shaped low flow channel inset into a broader flood channel (approximately 12 m wide bankfull extent). The broader flood channel is colonised by dense mangroves (*Avicennia marina*). The presence of mangroves has likely had a stabilising effect and indicates a tidally influenced environment.

Using NIWA MAF discharge of 2.8 m^3/s and assuming that flows remain confined to the bankfull channel, HS_US1 has an average specific unit stream power of 4 W/m^2 (Figure 3.12). The 100-year ARI flood flow average velocity is approximately 0.13 m/s, which is below the erosion threshold for most sized sediment. Indicative stream power threshold for movement of sediment up to the size of gravels is between 1.5-16 (W/m^2)[11]. This shows that MAF sized flows are likely to freely pass through the culverts and have sufficient energy to erode and transport the sediment present. However, once the flood flows start ponding behind the embankment, erosion and sediment transport processes become a little more complex and are directly linked to the culvert hydraulics.

Up to 9.2 m^3/s of water is able to flow downstream through the two parallel culverts[12]. When discharge exceeds 9.2 m^3/s, approximately 11 hours after the start of the 100-year ARI event, floodwaters will pond upstream of the embankment (Appendix B). This generates high velocities during mean low water springs up to approximately 8.4 m/s[12] which will likely determine the vertical scour at the downstream end of the culvert and the lateral scour of the point bar on the TLB. If debris blocks the culvert, smaller flows can backwater or lead to prolonged ponding at the base of the culvert.

The expected 100-year ARI water level considering tidal and freshwater flows is 5.3 m AVD-46 (Section 3.5). Bankfull height is approximately 4 m which compared with the 5.3 m AVD-46 water level, indicates that the bank full extent will be inundated.

In-stream geomorphic units observed were predominantly runs and pools (i.e. directly upstream of the embankment see Photograph 3.2). During low tide, freshwater baseflows are still observed in HS_US1 (Photograph 3.3). Upstream of the embankment, a scour hole was observed[13], likely as a result of the rapid drawdown associated with the transition from ponded to free flow states. Despite this, the longitudinal profile (Figure 3.13) indicates that HS_US1, specifically at the embankment is approximately 1.0 m from base level (assumed 0 m is sea level) and so further vertical and/or lateral adjustment is unlikely, given the highly cohesive marine clays that are colonised by mangroves.

Figure 3.10 identifies the bed and bank erosion susceptibility as low. This is likely a reflection of the highly cohesive clays forming the bed and banks.

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11 Indicative stream power thresholds from Fryirs and Brierley (2013). *Geomorphic analysis of river systems: an approach to reading the landscape*. John Wiley & Sons.

12 See T+T model version: BASE run conducted in August 2024 titled *HaraniaBGN 202408_v05*.

13 For safety reasons, the water depth of the scour pool could not be physically measured during the site visit on 17 July 2024.

Photograph 3.3: LH: Drone photograph showing the pooling of water and accumulation of sediment, woody debris and household litter directly upstream of the embankment. RH: active channel is u-shaped channel with active bank height of 0.9 m. Presence of mangroves during low tide and base flow conditions. Photo taken at low tide on 17 July 2024.

3.8.3 HS_DS1

Reach HS_DS1, flows for approximately 400 m with an active bed slope of approximately 0.2% (0.0028 m/m), entering the CMA downstream of the culvert. The active bank height is approximately 1 m and the bank full height is 1.8 m. The active channel has a low to moderate sinuosity which has been modified since the 1950's (Section 3.3). At the time of the embankment construction (1952) the active channel moved to the true left and tightly curves to the right to the original channel location approximately 50 m downstream of the embankment. During the 1980's, the eastern portion of the tidal inlet was reclaimed approximately 300 m downstream from the embankment, contracting the 100-year floodplain terrace.

Both the bank and bed material consist of cohesive, dark-brown coloured clays. However, the material of the TR margin associated with the reclaimed land is not known.

The channel cross-section is typically asymmetrical with an active channel of approximately 3.5 m wide. There is ample accommodation space for higher flow events within the 100-year floodplain terrace as bankfull width is 15 m. In-stream geomorphic units observed were predominantly runs and bar features. The mid channel bar observed at the bend directly below the culvert had a high proportion of gravels (< 40 mm) which likely demarcates the point at which the water discharging from the culverts starts to lose velocity, depositing coarser grained sediment into the active channel. Another large depositional point bar feature was observed 20 m directly downstream from the culvert on the TLB. The graded point bar had a flat top, and a 0.1 m fine silt layer on top of cohesive marine clays. The bankfull extent has early mangrove (*Avicennia marina*) growth on the TLB (Photograph 3.4).

Up to 9.2 m^3/s of water is able to flow downstream through two parallel culverts[12]. When discharge exceeds 9.2 m^3/s, floodwaters will pond upstream of the embankment (Appendix B). This generates high velocity flows, particularly during low tailwater conditions (mean low water springs)which has created a large scour pool on the downstream side of the culverts.

The average stream power is approximately 8 W/m^2, based the NIWA MAF discharge 3.1 m^3/s. This is still within the indicative entrainment threshold for most sediments up to small sized gravels[8]. The average velocity for the 100-year ARI event is 0.5 m/s[14] which is above the indicative erosion threshold for silts and sands based on the Hjulström curve. Together, this suggests that both low

[14] Channel velocity in Section B of Scenario A under T+T model version: run conducted in August 2024 titled *HaraniaBGN 202408_v05.*

magnitude high frequency events, and the lower frequency large flood events have sufficient energy to entrain and transport the unconsolidated sediments present within the reach but are unlikely to initiate scour of the heavy clay material.

Despite this, three erosional features were identified in the 2024 site walkover. First a 3 m long lateral scour feature, secondly minor stripping of fine silts and sands from the point bar was observed on the TLB, and scour was observed along the TR terrace bank. All of these erosional features are likely to be associated with the very high velocity flows from the existing culverts. In general, the reach remains relatively stable despite the observed erosional features, and this is likely due to the dominance of the cohesive marine clays in the bed and lower banks. Therefore, the bed and bank erosion susceptibility are considered to be low.

Photograph 3.4: LH photo shows minor scour on TRB and point bar that has been cleared of mangrove vegetation on TLB. RH scour of the TLB and the very cohesive marine clays overlain by silts and organics. The mid channel bar indicates the point at which sediments are deposited as the flows exiting the culvert lose velocity. Photo taken during low tide on 17 July 2024.

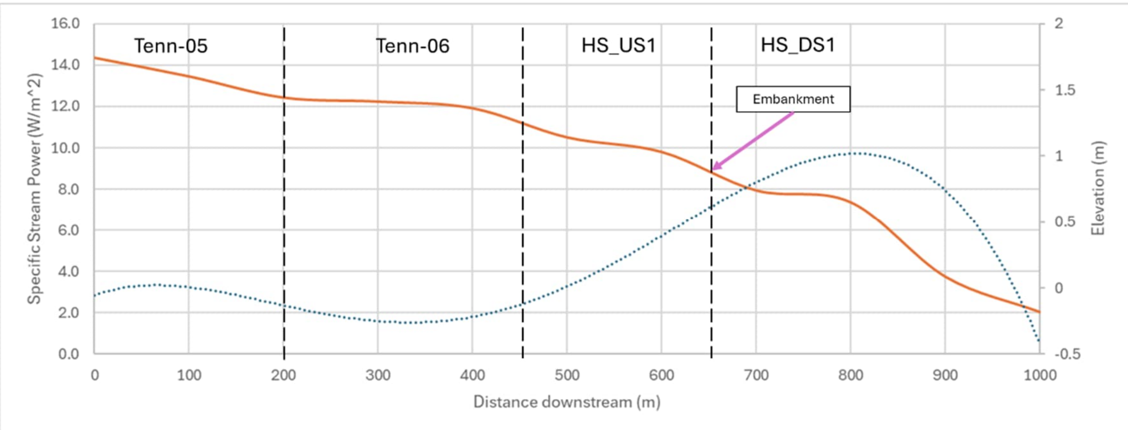

Figure 3.13: The Harania long profile (orange) with stream power (dotted blue). The stream power line has been smoothed to represent general trends across the Harania Creek catchment. Note: the stream power was derived from NIWA MAF discharge and corrected for the actual catchment extent.

3.8.4 Geomorphic condition summary

In 2022 and 2023, the reaches of Harania Creek upstream from the project area (Tenn_05 and Tenn_06) showed signs of moderate to high bed and bank erosion susceptibility. Under existing flow conditions, these upstream reaches are likely impacted by concentrated stormwater outflows during low magnitude high frequency storm events, but ponding/backwater effects arising from the existing embankment have likely limited the occurrence of erosion in events with a discharge larger than 9 m^3/s.

Reach HS_US1 is in a partly confined valley setting with a bed gradient of approximately 0.1 %. Flows under 9 m^3/s free-flow through the embankment culverts, but flows above 9 m^3/s pond behind the embankment. This means high frequency, but reasonably low magnitude events likely have enough energy to do geomorphic work (erosion and scour). High magnitude low frequency events create an opposing erosion deposition scenario where the larger low frequency floods will create a primarily depositional environment in the upper half of the reach whilst the area adjacent to the culvert experiences erosion as a result of the rapid drawdown associated with the transition from ponded to free flow states through the culvert. The presence of mangroves and cohesive clays in the bed and low banks have a stabilising effect on the channel, and therefore lateral or vertical adjustment of the reach is unlikely.

In HS_DS1, flows have been significantly attenuated by the embankment and culverts, with a maximum discharge of 9.2 m^3/s able to pass through the culverts. This will have had an influence on river form and function. A scour pool downstream of the culvert in Reach HS_DS1, is likely a result of high velocity flows exiting the culverts and scouring adjacent areas. Approximately 15 m downstream of the culvert there is a reduction in flow velocity. This has resulted in the deposition of coarser grained sediment in the bed of HS_DS1. Three other erosional features were identified in HS_DS1 all of which are likely to be associated with the very high velocity flows from the existing culverts. In general, the reach remains relatively stable likely due to the dominance of the cohesive marine clays in the bed and lower banks.

The specific unit stream powers for MAF events are between 2 and 10 W/m^2 for all reaches which is within the indicative threshold (1.5 – 16 W/m^2) for the entrainment of sands, silts and small gravels (< 40 mm) but below the entrainment threshold for coarse gravels. However, velocities for the 100-year ARI are currently below all erosion thresholds in HS_US1 and above the erosion threshold for silts and sands (but not clays) in HS_DS1. This suggests that high frequency, but low magnitude flow events are the ones that are able to do the most geomorphic work, with the larger floods more likely to be associated with erosion outside of the active channel, or only in localised areas.

Cardno[15] undertook an assessment of bank erosion susceptibility in the Auckland Region, looking at bank material composition, hydraulic resistance (critical shear stress in Pascals (Pa)) and bank shear strength (expressed as cohesion (kPa) and friction angle (degrees)). Results suggest that higher sand content in banks may make them more prone to erosion, especially under specific conditions such as increased peak flow discharges, poor vegetation cover, or incised channels. Therefore, the geological mapping suggests that the bank material along the Harania Creek, may have localised areas, or layers, that are susceptible to erosion. Based on the results from the Omaru Creek Catchment[16], critical bank height was 1.7 m for saturated banks at 90-degree bank angles. In comparison, while near vertical, the active channel bank heights in the Harania Creek catchment were generally below 1 m which suggests that the banks are not close to unsupported critical bank height, reflecting low to moderate bank erosion susceptibility[17]. Where bankfull height was recorded above 1.7 m in the Harania Creek catchment, the bank angle was typically less than 80 degrees.

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[15] Resistance and Critical Height of Streambanks in Selected Catchments of the Auckland Region. Prepared for Auckland Council by Cardno. 11 March, 2020.

[16] Cardno identified critical bank height from a similar sub-catchment, the Omaru Creek catchment. Omaru Creek was selected as a similar catchment because of the likeness in geology, tidal influence and catchment land use to the Harania Creek catchment.

[17] Critical bank height has been based on a 90 ˚ bank angle. Omaru creek 20 ˚ to 90 ˚ bank angle ranges between 9.5 to 1.7 m saturated critical bank height.

Therefore, bed and bank erosion susceptibly in the two assessment reaches (HS_US1 and HS_DS1) is considered to be low. However, mangroves are considered critical to the ongoing stability of the channel, and there maybe localised areas of higher erosion risk, where there are sand lenses in the bed or banks.

4 Coastal hazards

4.1 Coastal inundation

In Figure 4.1, the coastal inundation map for the project area is shown. This coastal inundation map was created by plotting the 100-year ARI extreme water levels as listed in Table 3.2 over the 2016 LiDAR map for the area.

Furthermore, for the expected 100-year design life of the proposed works, the coastal inundation extent including RSLR of 1 m is evaluated. Therefore, Figure 4.1 additionally shows the expected coastal inundation extent for a 100-year ARI event with a RSLR of 1 m. As illustrated in Figure 4.1, for a 100-year ARI event with a RSLR of 1 m, the coastal inundation extends through a number of adjacent residential properties abutting both sides of the creek. To the north, where Harania creek flows into Manukau Harbour, extensive flooding of plots on the western side of this creek could occur in this situation.

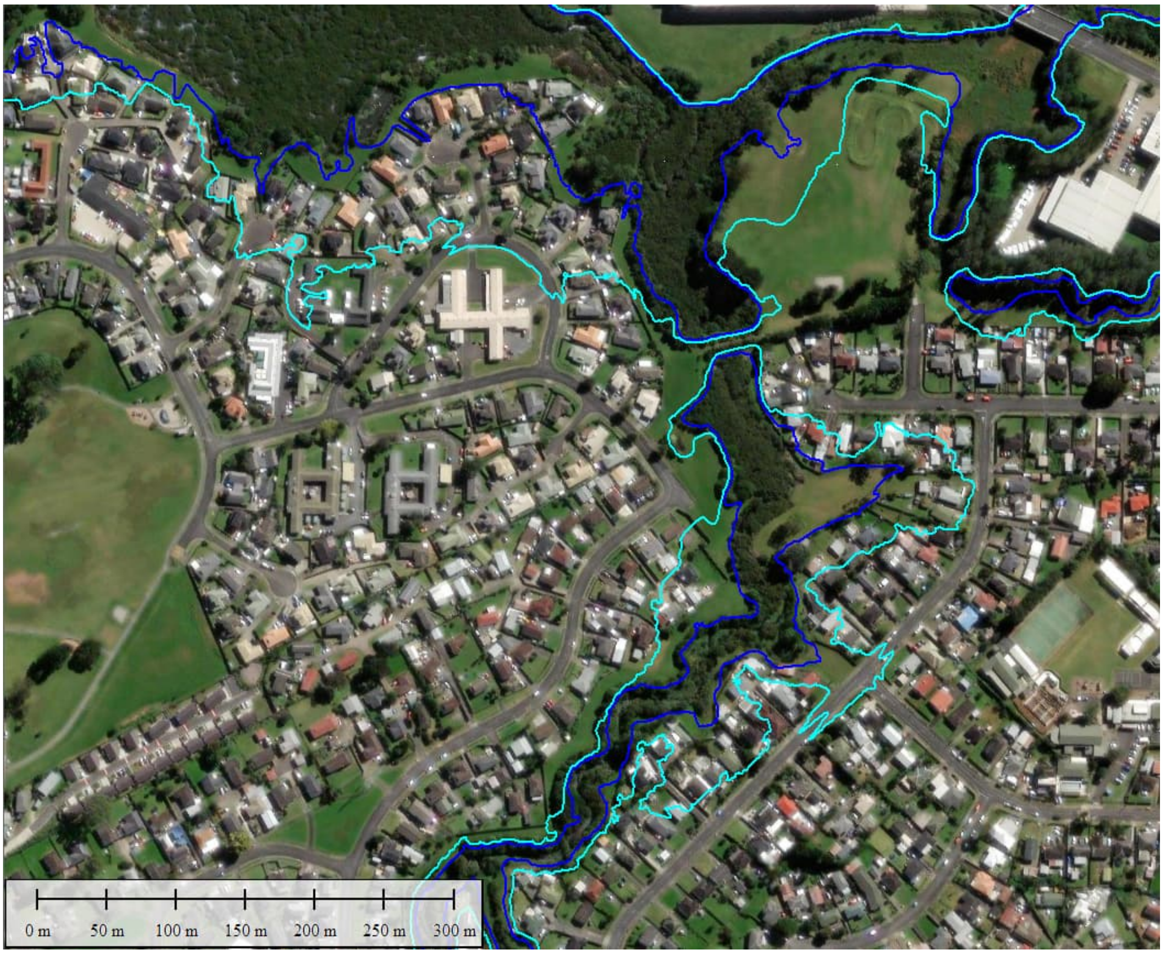

Figure 4.1: Present day coastal inundation extent for a 100-year ARI water level indicated with the dark blue line, coastal inundation extent for a 100-year ARI water level with 1 m of RSLR in 2130 indicated with the cyan line.

4.2 Coastal erosion

A regional erosion hazard assessment was undertaken in 2021 (Tonkin & Taylor Ltd., 2021). Figure 4.2 shows erosion lines at the project site corresponding to different time horizons under the RCP8.5 climate scenario. The dark blue line indicates the expected erosion in 2050, the cyan line in 2080 and the light green line in 2130.

Figure 4.2: Erosion lines at the project site corresponding to different time horizons under the RCP8.5 climate scenario. The dark blue line indicates the expected erosion in 2050, the cyan line in 2080 and the light green line in 2130.

4.2.1 Site specific erosion hazard assessment

Regional erosion hazard information above is not generally considered of sufficient detail for site-specific assessment. This section describes our site-specific assessment for this area, specifically the Area Susceptible to Coastal Instability and/or Erosion (ASCIE).

We regard this shoreline as being a cliffed coastline, being influenced by erosion of the cliff toe caused by marine and biological processes, weathering and slumping of the over steepened cliff face. Present day coastal erosion hazard zones for cliffs are determined by assessing the effect of slope instability and depend on cliff height (Tonkin+Taylor Ltd, 2011), future hazard zones for cliffs then include cliff toe erosion as a result of changes in sea level. This relationship is outlined below in Equation 1 and illustrated in Figure 4.3.

$$\mathrm{ASCIE} = \underbrace{\left(\frac{H_c}{\tan(\alpha)}\right)}_{\text{Cliff crest}} + \underbrace{LT_H \cdot T \cdot \left(\frac{SLR_F}{SLR_H}\right)^m}_{\text{Cliff toe}} \tag{1}$$

Where:

H_C	=	Cliff height (m)
α	=	Stable cliff stable (degrees)
LT_H	=	Historic long-term retreat (m/year)
T	=	Planning time frame (years)
SLR_H	=	Historical sea level rise (mm/year)
SLR_F	=	Future sea level rise (mm/year)
m	=	Sea level rise factor

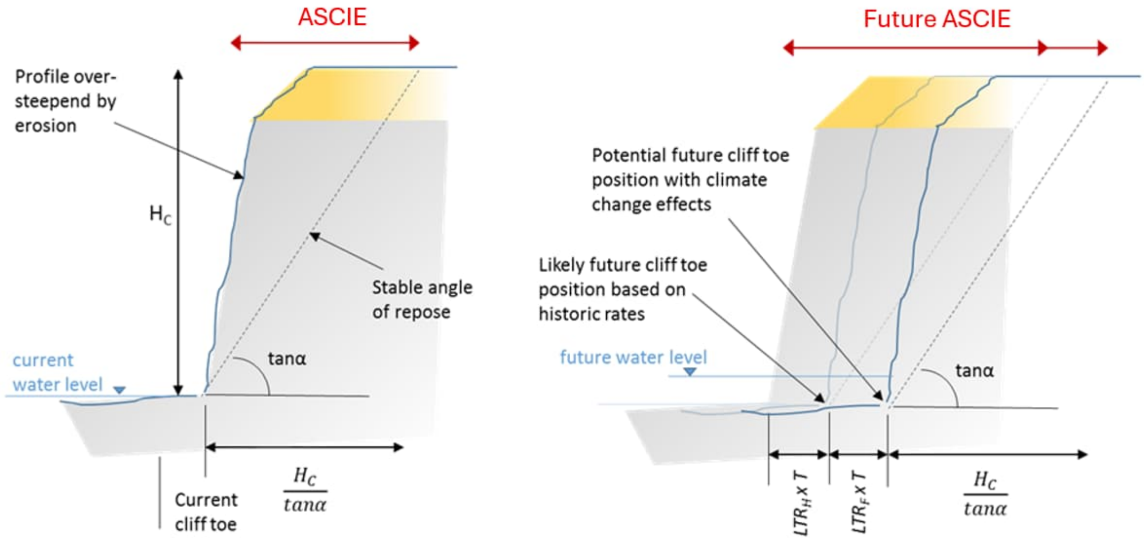

Figure 4.3: Definition sketch for cliffed shore ASCIE for the present day (left) and future (right).

4.2.2 Component derivation

4.2.2.1 Cliff height (H_C)

LiDAR information indicates cliff heights between 2 and 4 m. An average cliff height of 3.0 m has been adopted.

4.2.2.2 Stable cliff angle (**α**)

Geotechnical review of available borehole information in conjunction with site assessment by a geotechnical engineer (see 'Geotechnical Resource Consent Assessment Report for Harania Flood Resilience works – Tennessee Bridge') included slope stability modelling, including a *Silt (with clay)* with an effective friction angle of 28 degrees and an effective cohesion of 3 kPa. Land abutting the

ASCIE area comprises parks and reserve areas, where we consider a minimum factor of safety of 1.5 applies. Back analysis using the same soil parameters above indicates a *stable angle* of 28°.

4.2.2.3 Historic long-term retreat (LT_H)

In the regional erosion hazard assessment, a historic long-term retreat rate of 0.03 m/year was assumed for the project site. For site-specific erosion hazard assessments, this rate is usually determined based on an analysis of historical aerial imagery. However, for this specific project site, the presence of dense vegetation has prevented such an analysis.

From historical aerial images for the year 1940 and 1958 (Retrolens, 2024), a length of coastal edge free of vegetation was located approximately 1 km north of the project area. This area is considered to have similar geology and cliff height. By digitizing the crest of this area, long term erosion rates were estimated in the order of 0.05 m/year. This value is slightly higher compared to the values used in the regional erosion hazard assessment. However, the area for which this value was estimated is more exposed that the project site. Therefore, the lower value of 0.03 m/year was adopted in the analysis for the less exposed project site.

We consider this value consistent with our understanding of surface weathering in sheltered estuarine environments (such as the project site).

4.2.2.4 Planning time frame (T)

For site-specific erosion hazard analysis, we have analysed the erosion impacts for the following planning horizons:

- 2024 Coastal Erosion Hazard Zone (present-day);
- 2080 Coastal Erosion Hazard Zone (approx. 50 years); and
- 2130 Coastal Erosion Hazard Zone (approx. 100 years).

4.2.2.5 Sea level rise effects (SLR_H, SLR_F)

An historic sea-level rise rate (SLR_H) for Auckland of 1.7 mm/year (Hannah and Bell, 2012) has been adopted for this assessment.

A range of future sea level rise rates (SLR_F) have been adopted based on the four RSLR scenarios; SSP1-2.6, SSP2-4.5, SSP3-7.0 and SSP5-8.5. For each climate scenario, the relative sea level rise (RSLR) value has been considered, which includes both the effects of sea level rise (SLR) and vertical land movement (VLM) at the project site. These values, with respect to 2024, have been presented in Table 4.1.

Table 4.1: Relative sea level rise values and rates under different climate scenarios and time horizons

Climate scenario	2080		2130	
	RSLR	Rate [mm/year]	RSLR	Rate [mm/year]
SSP1-2.6	0.44	6.79	0.87	8.21
SSP2-4.5	0.51	7.86	1.08	10.19
SSP3-7.0	0.59	10.54	1.35	12.74
SSP5-8.5	0.66	11.79	1.52	14.34

4.2.2.6 Sea level rise coefficient (m)

(Ashton, et al., 2011)proposed a generalised expression for future recession rates of cliff coastlines where a coefficient ‘m’ is determined by the response system. No feedback (m → 0) indicates that the cliff is insensitive to sea level rise effects and future recession will occur at historic rates. An instantaneous response (m = 1) indicates that the future rate of recession will increase proportional to the increase in SLR. A negative/damped feedback system (0 < m < 1) occurs where rates of recession are slowed by development of a shore platform or fronting beach.

There is limited guidance on selection of appropriate coefficients for increased recession under SLR. (DEFRA, 2002) suggested that for soft cliffs an instantaneous response (m = 1) should be assumed. (Walkden, et al.) found that for soft cliffs in the UK (recession rates of 0.8 – 1 m/year) a factor of m = 0.5 could be assumed over the long term. Although material strength is likely comparable to this site, the sheltered estuarine embayment means sea level rise will only have a minor-to-moderate effect. Therefore, we propose m = 0.3 is adopted.

4.2.2.7 Overview of parameters

In Table 4.2, and overview of the parameters from Equation 1 that were used in the site-specific erosion hazard assessment is given.

Table 4.2: Parameter values for the calculation of erosion lines at the project site

Parameters	Description	Value in regional assessment	Remarks on use for site-specific assessment
LT_H	Historic long-term horizontal coastline movement.	0.03 m/year	This value was adopted from the regional erosion hazard assessment and corresponds to a medium-exposed soft cliff environment. Since the project site is a very sheltered, low-exposure environment, this is considered to be conservative. However, available historical aerial images for the project site were not suitable for the determination of a more precise value.
T	Time horizon.	2080 & 2130	2130 as per the required 100-year design life. Erosion rates were also calculated for 2080.
α	The stability allowance delineates the area landward of a coastal cliff associated with a global factor of safety ~1.5.	28°	Geotechnical review of available borehole information in conjunction with site assessment by a geotechnical engineer (see ‘Geotechnical Resource Consent Assessment Report for Harania Flood Resilience Works – Tennessee Bridge’), indicates a *stable angle* of 28° as being generally appropriate.
H_c	Cliff height.	3 m	LiDAR information indicates cliff heights between 2 and 4 m.
SLR_H	Historic sea level rise.	1.7 mm/year	The historic sea level rise rate at the project site is assumed to be 1.7 mm/year following (Hannah and Bell, 2012).
SLR_F	Allowance for future sea level rise.	6.8 – 14.3 mm/year	The rate of future sea level rise varies between 6.8 – 14.3 mm/year, depending on the climate scenario and the time horizon.

Parameters	Description	Value in regional assessment	Remarks on use for site-specific assessment
m	Sea level rise factor.	0.3	Value adopted from the regional erosion hazard assessment for the project site.

4.2.3 Coastal erosion assessment results

Table 4.3 presents the future erosion hazard extending landward of the cliff toe baseline across the range of potential scenarios and planning time frames. For a 100-year planning horizon, the erosion hazard setback ranges from -10.7 to -11.7 m. The erosion hazard setback ranges from -9.5 to -10.5 m in the regional assessment. These values are slightly lower compared to the site-specific erosion hazard assessment, since in the regional assessment a larger stable angle of 34° was used (instead of 28°).

In Figure 4.2, the erosion lines corresponding to different time horizons for the project site are depicted. Since the erosion lines for the different climate scenarios for both the 2080- and 2130-time horizon were almost identical, these different lines could not be distinguished individually within a figure. Therefore, in Figure 4.4, only the erosion lines corresponding to the SSP5-8.5 climate scenario were visualized for both time horizons.

Table 4.3: Future erosion hazard extending landward of the cliff crest baseline

SLR Scenario	2080	2130
SSP1-2.6	-8.3	-10.7
SSP2-4.5	-8.4	-11.1
SSP3-7.0	-8.5	-11.5
SSP5-8.5	-8.6	-11.7

Note: negative values denote distances landward of the existing cliff toe (i.e. indicate erosion).

Figure 4.4: Coastal toe erosion lines for the CMA at Harania Creek.

5 Coastal effects assessment

This section considers the potential coastal process effects as a result of the proposed Tennessee Bridge works. This section does not include the effects of fluvial flows, which are covered in Section 6.

5.1 Occupation of the Coastal Marine Area (CMA)

The embankment that is currently present in the tidal channel will be removed and replaced by a construction with a lower footprint. The occupation of the CMA will therefore decrease.

We assume that the CMA extends landwards of the current culvert location. We have conservatively included areas inland of the culverts in the CMA and therefore the coastal effects assessment.

5.2 Water levels

As was described in Section 2.7, the discharge capacity of the two culverts connecting both sides of the embankment is higher than the expected discharge related to 100-year ARI storm tide level of 3.30 m NZVD-16. It can therefore be concluded that currently tidal flows through Harania creek are not obstructed by the presence of the embankment. This means that the changes in water levels in and around the project areas due to the removal of the embankment are expected to be negligible. Effects on fluvial flows are discussed in Section 3.8.

Figure 5.1: Culverts under current embankment in tidal channel.

5.3 Waves

As was described in Section 3.8, waves will not penetrate as far as the project location due to the limited water depth and high friction under the influence of vegetation. Therefore, the effect of the removal of the embankment on the local wave climate is negligible.

5.4 Tidal flow velocity

Under normal conditions, the tidal flow through Harania Creek is not obstructed by the presence of the embankment. The cross-sectional flow area of the culverts, and therefore the tidal velocity, is determined by the shape and size of the culverts. Removal of the culverts and reshaping of the tidal channel around the former embankment location might change the cross-sectional area of the tidal channel. Since the tidal discharges are expected to remain constant, this might locally affect the flow velocity. However, these local changes in tidal velocity are expected to be small.

5.5 Sediment processes and tidal channel alignment

Following the local changes in tidal flow velocity around the proposed works, as described in the previous section, sediment processes might change slightly in the area around the current embankment. However, since the changes in tidal velocities are expected to be small, the local changes in sediment processes are expected to be small as well.

Based on historic tidal channel evolution as described in Section 3.3, the location of the tidal channel is unlikely to significantly change under the current tidal flows. Some minimal realignment of the tidal channel caused by tidal flows may occur following the proposed works, as the location of the tidal channel is no longer fixed by the location of the culverts and the presence of mangrove trees.

5.6 Coastal flooding

As was described in Section 3.7, currently the tidal flows through the culverts caused by a 100-year ARI storm tide level in Manukau harbour are not obstructed. Therefore, the coastal flooding extent in the area upstream of the two existing culverts is not influenced by the presence of the embankment. The effects on coastal flooding of the proposed works (removal of the embankment and culverts) are therefore expected to be negligible.

5.7 Coastal erosion

As was described in Section 4.2, a long-term erosion rate of 0.03 m/year is expected to occur at the project site in the future. The coastal erosion rate is dependent on local soil properties and future sea level rise. As these factors will not be affected by the proposed works, exposure, the effects of the proposed works on the local erosion rate are expected to be negligible.

5.8 Risk screening

A risk screening based on the risk matrix in Table 5.1 has been undertaken and summarised in Table 5.2. For each of the coastal risks, the likelihood, consequence and associated level of risk are listed.

Table 5.1: Risk matrix

		CONSEQUENCE		
		Low	Medium	High
LIKELIHOOD	Unlikely	Low	Moderate	High
	Likely	Low	Moderate	High
	Very likely	Moderate	High	High

Table 5.2: Coastal risk screening

Environmental effect description	Risk screening assessment				Risk mitigation or assessment
	Consequence	Likelihood	Risk level	Risk (mitigated)	
Tidal water levels	Low	Unlikely	Low	No mitigation	The change in water levels due to removal of the embankment and construction of the bridge is negligible, since tidal flow is not obstructed in the current situation under normal conditions.
Waves	Low	Unlikely	Low	No mitigation	The change in wave climate due to removal of the embankment is negligible, as waves do not penetrate this far into the tidal channel, due to the inland location, the shallow water depths and established vegetation.
Tidal flow velocity	Low	Unlikely	Low	No mitigation	The tidal flow velocities are not obstructed by the current presence of the embankment. Therefore, the change in tidal flow velocities following the proposed works is expected to be negligible.
Sediment processes within tidal channels and tidal channel alignment	Low	Likely	Low	No mitigation	Following the proposed works, the position of the tidal channel is no longer fixed by the location of the culverts and the presence of mangrove trees. We consider natural realignment of the tidal channel is likely. Localised erosion and sedimentation associated with this change could occur.
Coastal flood risks	Low	Unlikely	Low	No mitigation	Within the existing CMA-area (i.e. downstream of the embankment), removal of the embankment is not expected to exacerbate coastal flood risk. Coastal flood risk in the area upstream of the embankment (i.e. not part of the current CMA), will experience the same water levels as the downstream side.
Coastal erosion	Low	Unlikely	Low	No mitigation	The proposed works are expected to have a negligible effect on the existing long-term coastal erosion rate in the area.

6 Fluvial geomorphic effects assessment

The assessment of geomorphic effects of the Tennessee Bridge works on erosion and deposition processes in the Harania Creek catchment is not intended to be a standalone assessment and should be read in conjunction with the resource consent application for the Tennessee Bridge project under the Severe Weather Emergency Recovery (Auckland Flood Resilience Works) Order 2024. This is especially relevant when considering the objectives and policies of the AUP, particularly chapter E3 which is associated with Lakes, rivers, streams and wetlands. In preparing this assessment the following key matters informed by the AUP have been considered:

- Hydrological and hydraulic effects including retention of sufficient creek flow conveyance capacity;
- Bed and bank erosion, scouring or undercutting, and land instability effects;
- The effects on downstream stream or wetland environments; and
- Any effects arising from any permanent modification in creek state or function.

In addition to the AUP, the NPS-FM (2020) has also been considered, in particular Policy 7 of the NPS-FM which is associated with "Loss of river extent and values". Loss of river value in the NPS-FM specifically relates to:

- Ecosystem health;
- Indigenous biodiversity;
- Hydrological functioning;
- Māori freshwater values; and
- Amenity.

Sediment transport processes including erosion, contribute to several of these values. Thus, this report also informs the effects assessments prepared by other technical specialists for the Tennessee Bridge resource consent application.

To assess the above matters, the Tennessee bridge project upgrade was modelled under the 'worst case scenario'[18]. Limitations of the flood model meant that the 2 yr ARI and 10 yr ARI outputs were not available to inform the fluvial geomorphic assessment. The 'worst-case scenario' model results were one tool and were used in conjunction with field evidence collected during the site assessment, as well as the historical channel evolution and geomorphic reasoning to inform the actual and potential geomorphic effects.

The T+T model version conducted in August 2024 titled *HaraniaBGN 202408_v05*[19] showed that for the 'worst case scenario', stream velocities are expected to increase within the Harania Creek Catchment (Figure 6.2).`

The Hjulström Curve is used to make estimates of sediment transport processes including erosion, transport and deposition (Figure 6.1). Figure 6.1 identifies that the amount of sediment mobilized is likely to be higher under the Tennessee Avenue Dam Embankment Upgrade compared to the existing conditions (Section 3.8).

There are two areas that have the potential for geomorphic effects. The locations of these areas are in reaches HS_US1 and HS_DS1 at the Tennessee Bridge and the reclaimed land approximately 180 m downstream from the Tennessee Bridge on the true right bank (HS_DS1). It is important to note

[18] Worst case scenario included 1 in 100-year ARI, 3.8 degrees of climate change, maximum probable development in the catchment and Mean Low Water Springs.
[19] Results discussed in this section relate to T+T model versions "BASE (Existing scenario)_202408_v05" and "TB01 (Tennessee Bridge option)_202408_v05".

that fluvial systems are inherently dynamic, and there may be other localised areas which could be subject to adverse geomorphic effects but are not immediately apparent using our current methodology. Refer to Appendix B for the methodology and Table 5.1 for risk categorisation matrix using a likelihood and consequence approach.

The water levels throughout the catchment are also expected to be lower after the Tennessee Bridge works (Figure Appendix B.7), as a result of less restricted flows at the embankment/bridge. The net effect of this change for fluvial processes is the concentration of flood flows to predominantly within the 100-year floodplain terrace, whereas in the existing condition scenario flood flows overtop the 100-year floodplain terrace and inundate the surrounding landscape. As a result, the system is expected to go from the majority of flows being dispersed over a wider area with slower velocities to flows confined to a narrow corridor at higher velocities (refer to Cross Sections Appendix C).

A more detailed assessment of each reach is provided in the sections below.

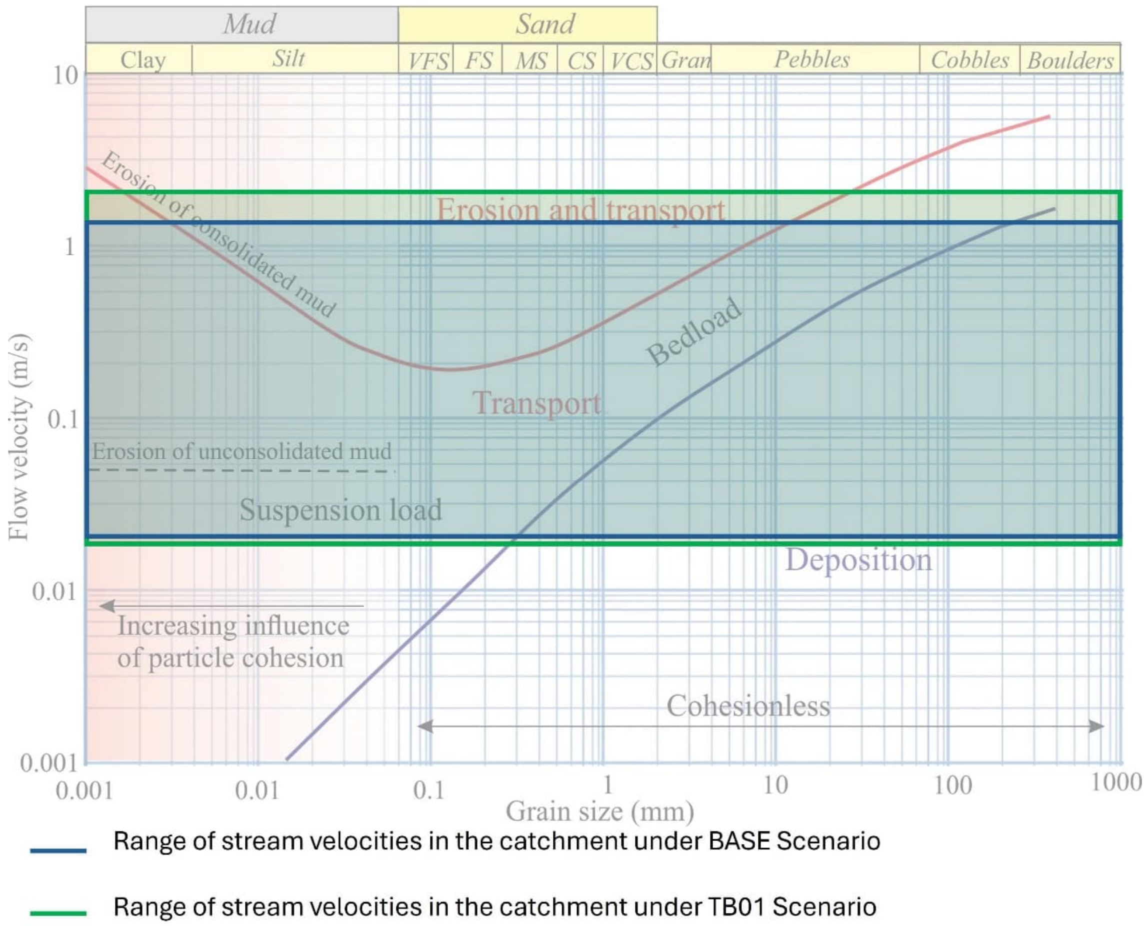

Figure 6.1: Envelope of modelled stream velocities under BASE and TB01 Scenarios and the associated effect on erosion, transport and deposition. Note that the Hjulström curve is predominantly for unconsolidated sediment, freely available for transport (i.e. the curve does not consider additional cohesion factors).

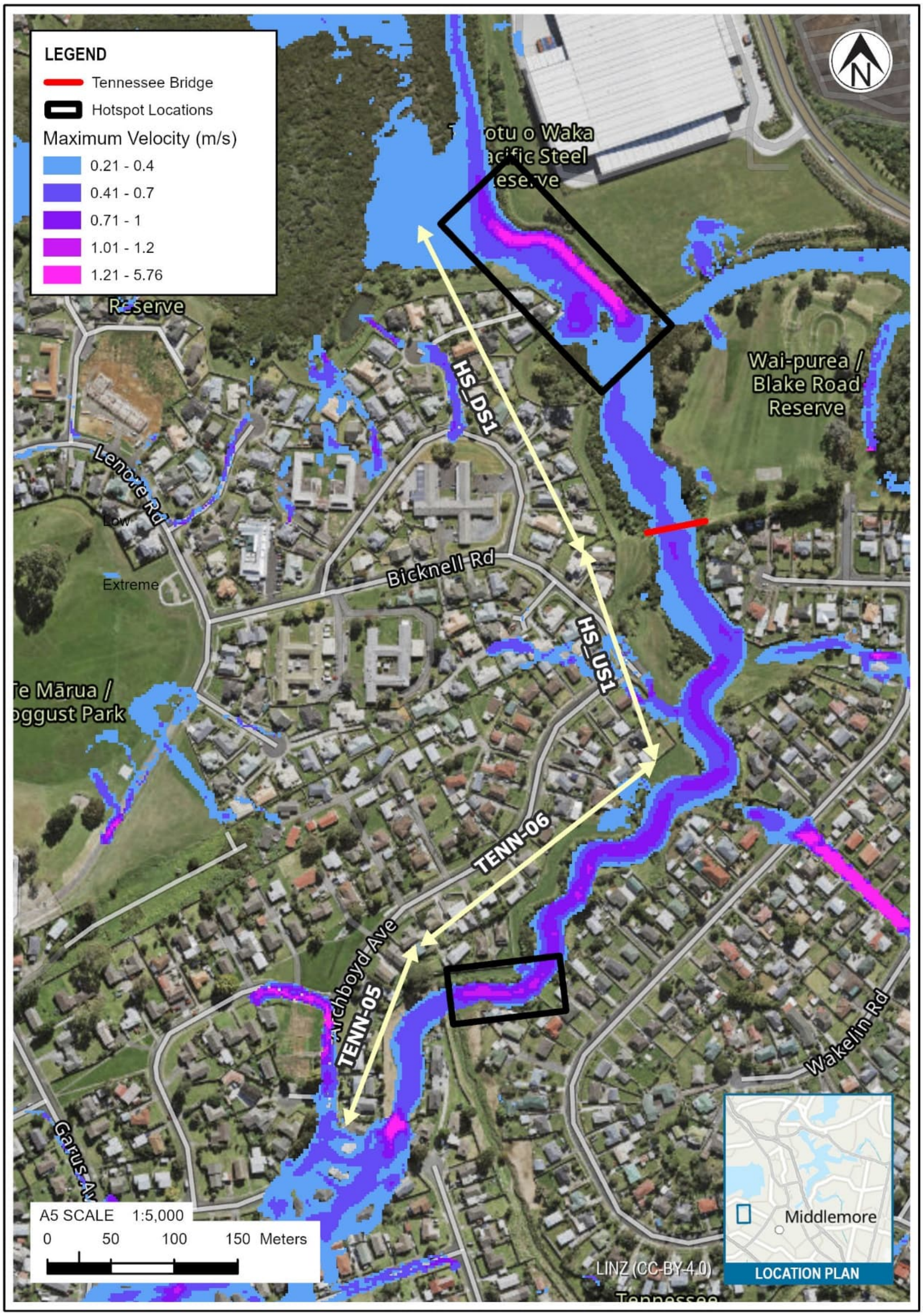

Figure 6.2: Map showing modelled velocities above indicative erosion thresholds for the post construction 100-year ARI flood event (TB01 scenario). The black rectangles indicate areas of very high velocity which have a higher likelihood of initiating erosion during a 100-year ARI flood event.

6.1.1 Tenn-05

Minor bank undercutting was observed in reach Tenn_05 during the 2022 and 2023 site visits. This is likely due to the presence of the stormwater outfall upstream of the reach, and the low width to depth ratio of the active channel. The low width to depth ratio suggests the reach has widened and infilled in the past but may have reached some form of equilibrium. As a result, the bed and banks were characterised as having a high bed and bank erosion susceptibility.

Based on the modelling results, the proposed activity is unlikely to change water levels significantly in a 100-year ARI event despite the discharge increasing 8.9 m^3/s under existing conditions to 10.2 m^3/s following the removal of the embankment and construction of the bridge. As a result of the increased discharge, the 100 year specific unit stream power is modelled to increase from approximately 2.5 W/m^2 under BASE conditions to 5 W/m^2 under TB01[20] (Figure 6.3) The modelled velocities for Tenn-05 are also predicted to increase following the proposed activity. The velocity changes are presented in Table 6.1.

Table 6.1: Summary of velocity changes for the proposed activity in Reach Tenn-05

TL 100-year floodplain terrace		TL bank full extent		Active Channel		TR bank full extent		TR 100-year floodplain terrace	
Velocity difference (m/s)	Difference (%)	Velocity difference (m/s)	Difference (%)	Velocity difference (m/s)	Difference (%)	Velocity difference (m/s)	Difference (%)	Velocity difference (m/s)	Difference (%)
0.22	96	0.84	145	0.53	34	0.68	110	0.3	150

Note: high percentage changes are likely a result of an area going from ponding to free-flow state once the embankment is removed. It does not necessarily indicate that large scale erosion will occur as erosion thresholds may not be exceeded.

During a 100-year ARI event, the increase in velocities could result in lateral widening of the active channel and potential scour of areas up to the 100-year floodplain terrace. As the modelled velocities and stream power are higher once the Tennessee Bridge is in place compared to the existing condition (Table 6.1), it is probable that erosion thresholds will be exceeded. However, this assumption does not consider additional cohesion factors, such as cohesive clays and vegetation, both of which were prominent along reach Tenn-05. These additional cohesion factors will increase stream resilience and aid in reducing the likelihood of geomorphic change. Based on the existing condition in the reach (Section 3.8.1), scour and erosion potential are unlikely to change in response to changes in specific unit stream power and velocity as the changes are unlikely to be high enough to cause erosion or alluvial bank instability in the active channel (Fryirs and Brierley, 2012).

The 100-year floodplain margin is potentially medium to low plasticity silt with some traces of clay (based on the geotechnical investigations around the embankment). Therefore, these surfaces may be erosion prone, particularly in areas where there is no substantial riparian vegetation.

Considering the current geomorphic condition and low likelihood of bank failure, there is unlikely to be any observable change in geomorphic processes within the active channel as a result of the proposed activity, with a low likelihood of erosion of the 100-year floodplain margin.

6.1.2 Tenn-06

Under the existing condition, only 9 m^3/s is able to pass through the culverts at the embankment creating a backwater effect in flows larger than 9m^3/s. It is therefore probable that under most flood

[20] BASE refers to the current embankment being retained onsite and TB01 refers to the Tennessee Avenue Bridge upgrade. Refer to Appendix B for further details.

events greater than the MAF, Tenn_06 experiences some ponding on the inset benches. The combination of no active erosion observed in the reach and the depositional environment suggests that Tenn_06 is currently a low energy depositional environment with potentially erodible blankets of silt on the bed and banks. Therefore, the bed and banks were characterised as moderate erosion susceptibility.

The proposed activity is likely to change water levels in a 100-year ARI event (Figure Appendix B.7) as the embankment is removed and water is no longer ponded in the larger flood events. Modelling results (Figure Appendix B.11) show a reduction of inundation outside the 100-year ARI floodplain terrace. As a result of the water level dropping and predominantly occupying the area between the 100-year ARI floodplain terraces, larger and more frequent flows are confined within a narrower area. This means that under the 100-year ARI event, flow velocities and specific unit stream power are likely to increase as flows are confined to the 100-year floodplain terrace (Figure 6.2). There is an area of modelled high velocity immediately downstream of the confluence between Tenn-05 and Tenn-04 reaches, which suggests 100-year ARI flows from Tenn-04 and Tenn_05 are more freely able to pass into Tenn_06 , removing the backwater effect. As a result of the confinement of flows, 100-year specific unit stream power is modelled to increase from approximately 2.5 W/m^2 under BASE conditions to 4 W/m^2 under TB01 (Figure 6.3).

Table 6.2: Summary of velocity changes for the proposed activity in Reach Tenn-06

TL 100-year floodplain terrace		TL bank full extent		Active Channel		TR bank full extent		TR 100-year floodplain terrace	
Velocity difference (m/s)	Difference (%)	Velocity difference (m/s)	Difference (%)	Velocity difference (m/s)	Difference (%)	Velocity difference (m/s)	Difference (%)	Velocity difference (m/s)	Difference (%)
0.22	220	0.36	113	0.53	147	0.46	192	0.2	500

Note: high percentage changes are likely a result of an area going from no/low flow to an area of high flow comparatively. It does not necessarily indicate that large scale erosion will occur as erosion thresholds may not be exceeded.

During a 100-year ARI event, the increase in velocities could result in lateral widening of the active channel and potential scour of areas up to the 100-year floodplain terrace. As the modelled velocities (Table 6.2) and specific unit stream power are higher once the Tennessee Bridge is in place compared to the existing condition, it is probable that erosion thresholds are exceeded. However, this assumption does not take in consideration additional cohesion factors, such as vegetation. There may be cohesive clays on the bed and banks of the active channel, however these are likely overlain with an uncohesive silt blanket which may be prone to fluvial erosion. Therefore, there may be some localised scour and erosion, but the presence of vegetation and underlying cohesive clays will likely limit the extent and severity of erosion.

Based on the existing condition in the reach (Section 3.8.1.2), scour and erosion potential are unlikely to change in response to changes in specific unit stream power and velocity as the changes are unlikely to be high enough to cause erosion or alluvial bank instability in the active channel (Fryirs and Brierley, 2012).

The 100-year floodplain margin is potentially medium to low plasticity silt with some traces of clay (based on the geotechnical investigations around the embankment). Therefore, these surfaces may be erosion prone, particularly in areas where there is no substantial riparian vegetation.

Considering the current geomorphic condition and low likelihood of bank failure, there is unlikely to be any observable change in geomorphic processes within the active channel as a result of the proposed activity. There may be some localised erosion, but it is unlikely to be extensive or severe. The likelihood of erosion of the 100-year floodplain margin is also low.

6.1.3 HS_US1

HS_US1 is approximately 0.7-1.0 m above sea level, comprising of highly cohesive marine clays colonised by mangroves. Therefore, vertical and lateral adjustment is possible but very unlikely. Upstream of the embankment, a scour hole was observed, likely a result of freshwater flows rapidly drawing down through the culvert. Due to the cohesivity of the active channel (clays and vegetation), the bed and bank erosion susceptibility is considered to be low.

The proposed activity is likely to reduce water levels in a 100-year ARI event as the constraint created by the embankment has been removed. Modelling results (Figure Appendix B.7) show a reduction of water inundating the areas outside the 100-year floodplain terrace. As a result of the water level dropping and predominantly occupying the area between the 100-year ARI floodplain terraces, larger and more frequent flows are confined within a narrower area. This means that under the 100-year ARI event, flow velocities and stream power are likely to increase as flows are confined to the 100-year floodplain terrace (Table 6.3). In HS_US1 in particular, there is a substantial increase in velocity for the pre and post construction scenarios, largely due to the reach going from a backwater/ponded environment to a free-flowing environment in the 100-year ARI event. Despite this, post construction velocities are relatively low and only cross the indicative erosion threshold for coarse silts and sands (Figure 6.1). Similarly, 100-year specific unit stream power is modelled to increase from approximately 4 W/m^2 under BASE conditions to 10 W/m^2 under TB01 (Figure 6.3).

Table 6.3: Summary of velocity changes for the proposed activity in Reach HS-US1

TL 100-year floodplain terrace		TL bank full extent		Active Channel		TR bank full extent		TR 100-year floodplain terrace	
Velocity difference (m/s)	Difference (%)	Velocity difference (m/s)	Difference (%)	Velocity difference (m/s)	Difference (%)	Velocity difference (m/s)	Difference (%)	Velocity difference (m/s)	Difference (%)
0.21	300	0.42	300	0.47	168	0.32	168	0.26	1300

Note: high percentage changes are likely a result of an area going from no/low flow to an area of high flow comparatively. It does not necessarily indicate that large scale erosion will occur as erosion thresholds may not be exceeded.

There is a low potential for the active channel to adjust laterally within the wider flood channel following the removal of the culverts. The vegetation removal plan[21] indicates that up to 1000 m^2 mangrove removal will occur in a localised area upstream and downstream of the culvert. The removal of vegetation is not expected to influence the magnitude of adjustment, if it occurs, adjustment is likely to be low in the active channel due to the stabilising effects of the marine clays in the short term, in the longer term the establishment of vegetation will further stabilise the area. In general, lateral adjustment is unlikely to result in any adverse effects, except for a small area around the bridge piers, where localised scour may occur . As per the stream plan drawings, rock rip rap has been included in the stream design. Therefore, any erosion risk at the bridge piers is considered to be appropriately mitigated. Regardless, the cohesive marine clays and presence of stabilising mangroves within the channel suggests that active channel adjustment is unlikely.

The existing scour hole upstream of the existing culverts is likely to infill relatively quickly. Once the culverts are removed, the scour hole will have localised lower velocities, and silt mobilised from further upstream under low magnitude but frequent flow events, will likely be readily deposited in the bed of the pool. There may be some flushing of the pool time to time, but it is expected that the silts will consolidate and become more cohesive (and therefore more resistant to erosion) through time.

[21] The Tree Consultancy, 16 September 2024. Blake Road Reserve Tennessee Pipe Bridge Vegetation Removal Plan. Project 3053. Drawing 002 Revision A.

Considering the current geomorphic condition and erosion susceptibility in HS_US1, there may be minor channel adjustments, and some localised erosion where coarse silts and sand are present (and vegetation is absent). But this is unlikely to result in a noticeable shift away from baseline conditions.

6.1.4 HS_DS1

HS_DS1 has had significant flow attenuation since the construction of the embankment, with a maximum discharge of approximately 9 m^3/s able to pass through the culverts. The reach is located at sea level, therefore vertical adjustment is not likely. Lateral adjustment is possible but unlikely, given the reach comprises of highly cohesive marine clays and is beginning to be colonised by mangroves. Downstream of the embankment, three existing erosional features were observed:

- A scour hole at the base of the culverts;
- An approximately 3 m long lateral scour feature on the TLB; and
- An approximately 15 m long lateral scour feature along the TR terrace bank.

These features are an indication of increased velocities and shear stress on the bed and bank as a result of the existing culverts. But generally, the reach is relatively stable. Due to the cohesivity of the active channel (clays and vegetation), the bed and bank erosion susceptibility is considered to be low.

As the flow attenuation caused by the embankment is removed the 100-year ARI flood event discharge increases from 9.3 m^3/s to 28.5 m^3/s. This increases water depth and lateral inundation extent, but flow velocities are likely to decrease from the existing condition, as flows are dispersed over a wider area (Table 6.4). The increased discharge also results in an increase in 100-year specific unit stream power. It is modelled to increase from approximately 8 W/m^2 under BASE conditions to 19 W/m^2 under TB01 (Figure 6.5).

Table 6.4: Summary of velocity changes for the proposed activity in Reach HS-DS1

TL 100-year floodplain terrace		TL bank full extent		Active Channel		TR bank full extent		TR 100-year floodplain terrace	
Velocity difference (m/s)	Difference (%)	Velocity difference (m/s)	Difference (%)	Velocity difference (m/s)	Difference (%)	Velocity difference (m/s)	Difference (%)	Velocity difference (m/s)	Difference (%)
0.158	718	-0.03	-7	-0.23	-30	-0.13	-20	0.242	275

Note: high percentage changes are likely a result of an area going from no/low flow to an area of high flow comparatively. It does not necessarily indicate that large scale erosion will occur as erosion thresholds may not be exceeded.

The proposed activity is likely to increase discharge and stream power but decrease velocity for much of the channel cross section as the larger discharge is conveyed over a wider area. There is a low potential for the active channel to adjust laterally within the wider flood channel following the removal of the culverts. The vegetation removal plan[24] indicates that up to 1000 m^2 mangrove removal will occur in a localised area upstream and downstream of the culvert. The removal of vegetation is not expected to influence the magnitude of adjustment, if it occurs, adjustment is likely to be low in the active channel due to the stabilising effects of the marine clays in the short term, in the longer term the establishment of vegetation will further stabilise the area. If lateral adjustment did occur it is unlikely to result in any adverse effects. The decrease in velocities in the active channel may allow the remnant scour holes to infill.

Conversely, there is an increase in velocities (From 0.022 m/s to 0.33 m/s) at the floodplain terrace margin in a 100-year ARI event. The increase is not expected to cause a shift in erosional processes as the absolute velocities (0.16 m/s and 0.24 m/s) are below any indicative erosion thresholds (Figure 6.1). However, a specific stream power of 19 W/m^2 is just above the entrainment and mobilisation threshold for larger sediment sizes (around 64 mm). This may be an over estimation but does suggest that higher magnitude low frequency events that are confined to channel may have sufficient energy to do geomorphic work, resulting in localised erosion of the true right terrace margin.

Considering the current geomorphic condition and low likelihood of bank failure, there is unlikely to be any observable change in geomorphic processes within the active channel as a result of the proposed activity. There may be some localised erosion, but it is unlikely to be extensive or severe.

On the true left bank, immediately downstream of the Tennessee Avenue Bridge Upgrade, the likelihood of erosion of the 100-year floodplain margin is likely, but only in high magnitude low frequency events (such as the 100-year ARI) that are contained within the terrace margins. Therefore, monitoring of the extent and frequency of erosion at the terrace margin should be sufficient.

6.1.5 Reclaimed land

There is a portion of HS-DS1 approximately 180 m downstream from the Tennessee Bridge that has been assessed separately. This is because the modelling results (Figure 6.3) showed an area of proportionally higher velocities in the vicinity of the reclaimed land at the north end of the catchment on TRB (see Photograph 6.1).

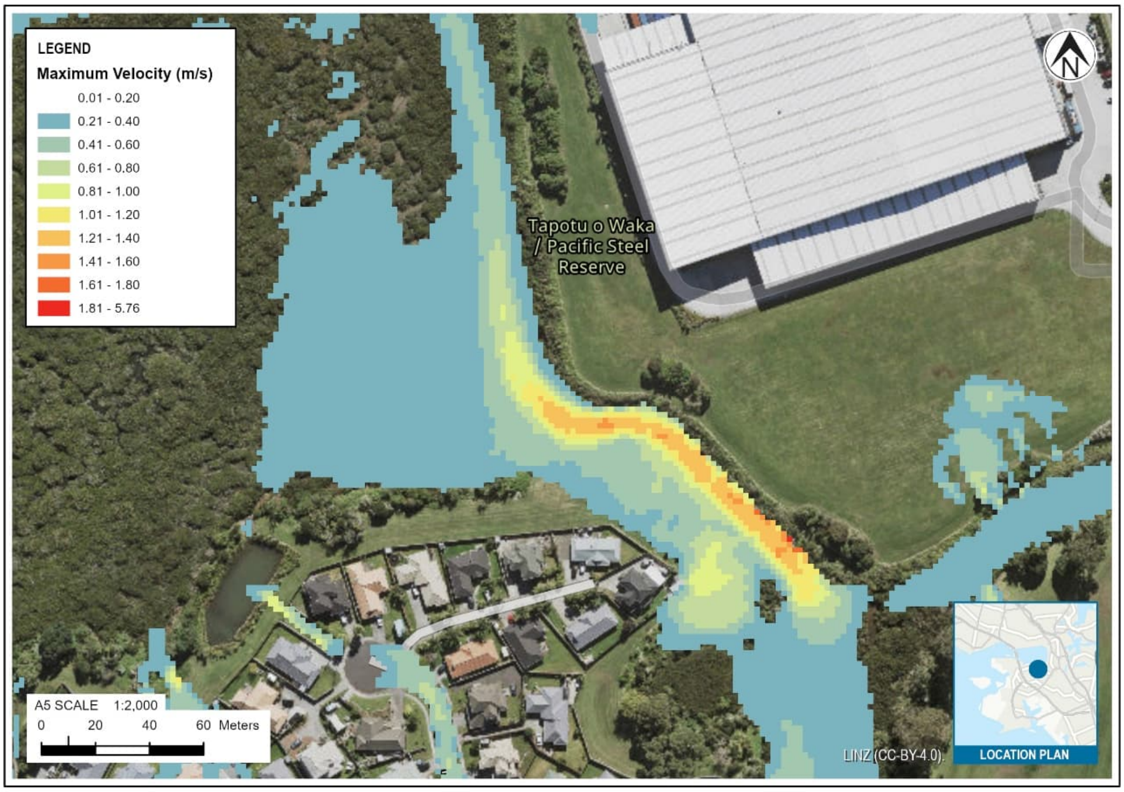

Figure 6.3: Modelled velocities of the area adjacent to the reclaimed land (Tapotu o Waka/Pacific Steel Reserve), showing areas of higher velocity and therefore potential erosion risk.

Photograph 6.1: Photograph showing the TRB and the mixed vegetation cover.

The proposed Tennessee Bridge is likely to increase velocities, discharge and stream power in a 100-year ARI event as the contraction created by the embankment has been removed, facilitating free-flow in HS_DS1 in all flood events. The modelling results show the maximum possible discharge through HS_DS1 at the reclaimed land increasing from 16 m^3/s to 34.6 m^3/s. The increase in discharge is also associated with an increase in 100-year specific unit stream power, modelled to increase from approximately 10 W/m^2 under BASE conditions to 23 W/m^2 under TB01 (Figure 6.5) . This has increased approximately 4 W/m^2 , compared to Reach HS_DS1 at the Tennessee Avenue Bridge Upgrade as it includes the contributions of the Blake Road Tributary. Figure 6.2 shows that the flow velocities are modelled to increase at the 100 yr floodplain terrace margin with the reclaimed land area under the worst-case scenario (Table 6.5).

Table 6.5: Summary of velocity changes for the proposed activity at the reclaimed land

TL 100-year floodplain terrace		TL bank full extent		Active Channel		TR bank full extent		TR 100-year floodplain terrace	
Velocity difference (m/s)	Difference (%)	Velocity difference (m/s)	Difference (%)	Velocity difference (m/s)	Difference (%)	Velocity difference (m/s)	Difference (%)	Velocity difference (m/s)	Difference (%)
0.25	208	0.38	146	0.63	50	0.39	43	-0.01	-2

The initial analysis of modelled results suggest that for the 100-year ARI flood event, velocities may exceed the erosion threshold for silts and sands (Figure 6.1). Similarly, the increase in stream power is above the entrainment and transport thresholds for large grain sizes (approximately 64 mm), suggesting an elevated likelihood of erosion.

As the bank material comprising the TRB is unknown there is an uncertain erosion risk to this area. Further analysis was undertaken to understand the magnitude and period of events which have the potential to exceed erosion thresholds given the current vegetation cover of the TRB. The TRB is covered in a mixture of long grasses, shrubs and trees (Figure 6.4). Review of erosion control documents [22] [23] [24] suggest erosion risk of grassed areas increases when flow velocities exceed between 1 m/s to 1.2 m/s [25] . The erosion risk increases if the duration of flows above this velocity thresholds are in excess of one hour.

Figure 6.5 provides a comparison of water levels and flow velocities under high tailwater (high tide) and low tailwater (low tide) scenarios over the period of the high flow event, with an erosion threshold of 1 m/s. Under the low tailwater scenario, the erosion threshold (1m/s) is reached 12 hours into the 100 year ARI flood event, near the peak of the flood, suggesting the erosion threshold is only reached in large infrequent flood events. The duration of flows above the erosion threshold is approximately 2 hours, suggesting erosion would probably occur during the flood peak. During High tailwater scenario, the erosion threshold is not reached at all. This suggested that if the 100 year flood peak coincided with high tide (or thereabouts) erosion would be highly unlikely to occur.

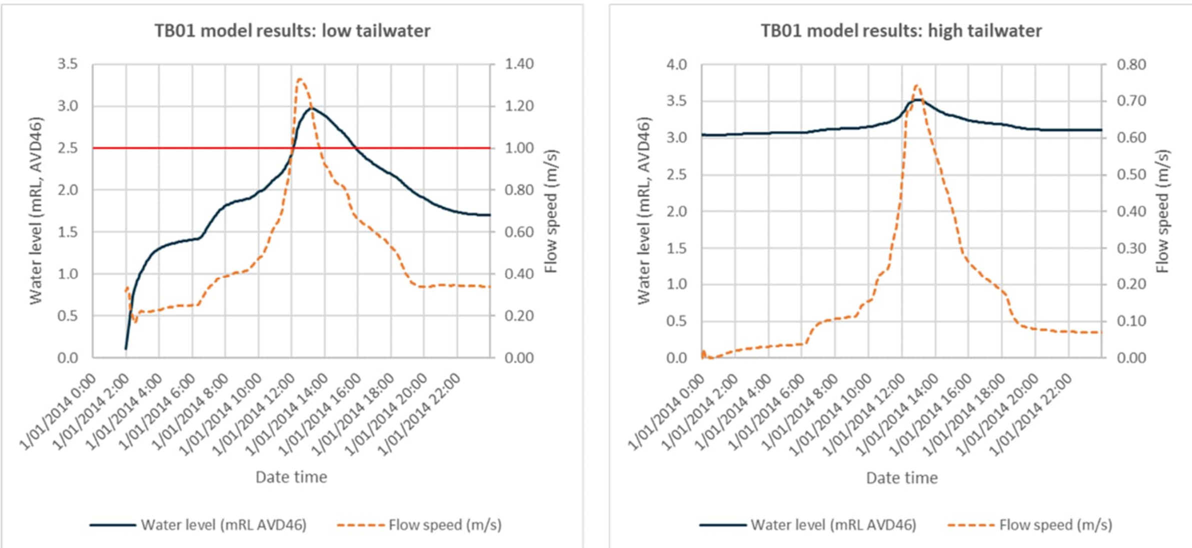

Figure 6.4: Comparison of water level and velocity under low tailwater and high tailwater scenarios. The red line on the low tailwater scenario graph represents the threshold for noticeable erosion risk , the threshold is not marked on the high tailwater scenario graph as flow velocities reach a maximum of 0.8 m/s.

Based on the above modelling results and the collation of potential erosion thresholds for grassed banks [24] [25] [26] (1 m/s) there is the potential for lateral adjustment of the active channel and bankfull extent (terrace margin toe), the effects are likely to be noticeable but unlikely to be significant under most flow conditions and only occur over short time periods. The erosion hotspot adjacent to the reclaimed land aligns with Table 4.3 and Figure 4.4 which shows the indicative average cliff toe retreat of up to 10 m by 2130 under SSP5-8.5 climate scenario.

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[22] Brisbane City Council - Stormwater Outlets in Parks and Waterways (2003, Version 2) cited in Buchanan K, Clarke C, Voyde E, (2013). Hydraulic energy management: inlet and outlet design for treatment devices. Prepared by Morphum Environmental Limited for Auckland Council. Auckland Council technical report, TR2013/018.

[23] United States Department of Agriculture Soil Conservation Service, 1954. Handbook of Channel Design For Soil And Water Conservation.

[24] Fischenich, Craig. (2001). Stability Thresholds for Stream Restoration Materials. 11.

[25] The thresholds assume that the bank material of the channel margins are moderately erodible and the banks are colonised by there is long grasses instead of vegetation (trees and shrubs).

HS_DS1 in general has a low bed and bank erosion susceptibility. However, in the modelled worst case scenario, there is an elevated erosion risk to the reclaimed land at the downstream end of HS_D1 in high magnitude but low frequency events with the removal of the embankment. With the Tennessee Bridge in place, and under the worst-case scenario, the reach is considered to have a moderate erosion risk. This moderate erosion risk can likely be monitored through an adaptive management approach, which identifies changes in erosion processes and outlines possible management responses.

6.1.6 Geomorphic effects assessment summary

The effects discussed in sections 6.1.1 to 6.1.5 are based on a 100-year ARI flood event and therefore present a worst-case scenario for geomorphic effects. The model outputs are indicative and provide guidance about areas of potential change because of the channelisation of flows and the increased velocities and specific unit stream power. Figure 6.3 shows the increases in stream power with reference to the long profile and individual reaches.

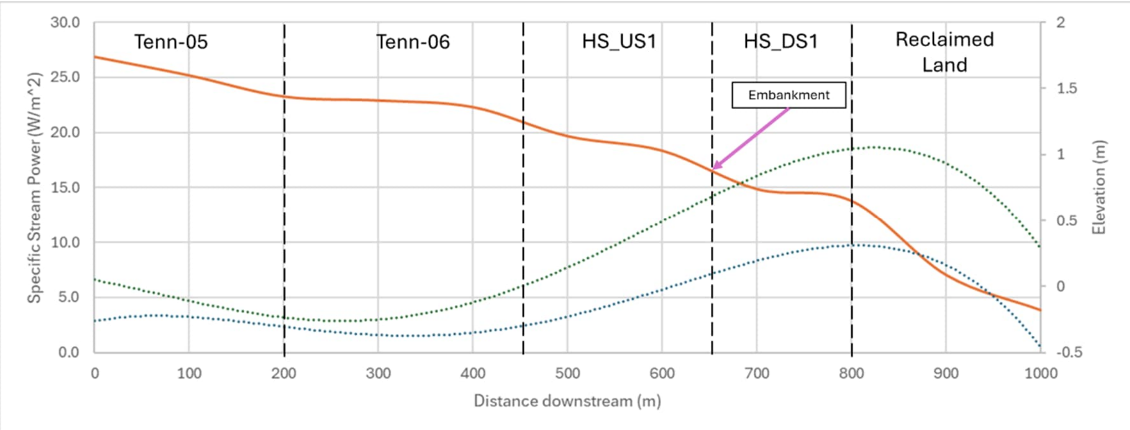

Figure 6.5: The Harania long profile (orange) with existing condition stream power (dotted blue) and predicted stream power following to development of Tennessee Bridge (dotted green). The stream power line has been smoothed to represent general trends across the Harania Creek catchment. Note: the stream power was derived from modelled discrage values.

Considering the individual effects of each reach in Sections 6.1.1 to 6.1.5, it is likely that removal of the Tennessee Ave Embankment and replacement with Tennessee bridge might have short term geomorphic effects, with an initial period of erosion and deposition in the vicinity of the bridge as the bed level adjusts to the change in flow regime and channel cross-sectional area.

In the long term, all reaches will undergo periodic but mainly localised erosion in response to the increased velocities associated with free-flow conditions (in events greater the 9 m^3/s). The active channel may have some minor adjustment to accommodate the increased flows, however in general erosional processes are unlikely to change from the existing conditions. The scour potential at 100-year floodplain terrace and bank full extent is increased as more frequent flood flows are concentrated between the 100-year floodplain terraces. The catchment scale controls, soils (predominantly cohesive marine clay) and vegetation cover (mangroves) will likely buffer these effects in the immediate vicinity of the upgrades and therefore the adverse geomorphic effect to the bank full extent and 100-year floodplain terrace will be minimal.

Under the worst-case scenario, the risk to the reclaimed land at the northern end of HS_DS1, is potentially of highest importance (Figure 6.2). The erosion of the reclaimed land may increase exposure to the contents within the reclaimed land, mobilising possible reclaimed land debris. Based on the information collected to date the risk is moderate. The potential risk of erosion occurs for a short duration (2 hours), and only in a small window of opportunity when the low tailwater condition coincides with the peak discharge of the high flow event. Considering the uncertainty of the above erosional scenario and the noted potential for cliff toe retreat in the coastal effects sections, a monitoring approach to understand the system response to the Tennessee Avenue Bridge Upgrade is recommended.

From a geomorphic perspective, based on the above modelling results and existing condition assessments the effects associated with the proposed activity can be separated into two levels of effect: one relating to reaches Tenn-05 to the upstream portion of HS_DS1, and one to the downstream end of reach HS_DS1 (Reclaimed Land).

For reaches Tenn-05 to the upstream portion of HS_DS1, the potential geomorphic effects are unlikely to result in noticeable changes to river form and function, and there is a low likelihood of the following geomorphic effects being worsened:

- Bed and bank erosion in reaches Tenn-05 and Tenn-06;
- Bank scour at bank full extent and potentially valley margins in reach HS-US1; and
- Scour at bridge piers in reach HS_DS1.

Scour in the meanders in the tidal flat of reach HS_DS1. Therefore, effects mitigation measures are not considered necessary.

For the downstream end of reach HS_DS1 (Reclaimed Land) there is a high risk of bed and bank erosion, however given the small window of opportunity for the effects to occur they are considered to have a low likelihood. Therefore, monitoring of the geomorphic effects particularly following flood events larger than a 50 year ARI is recommended.

6.2 Proposed Management options

The fluvial geomorphic assessment has assessed the potential effects of the proposed upgrades on geomorphic processes in the immediate vicinity of the Tennessee Ave Dam embankment Bridge works. There is no residual geomorphic risk in Reaches Tenn-05 to the upstream portion of HS_DS1, and a low likelihood for geomorphic effects in the downstream end of HS_DS1. Therefore, effects management recommendations are not required, and monitoring for geomorphic effects at the reclaimed land is considered appropriate.

Table 5.1 outlines the risk matrix for assessing the level of geomorphic effect. Table 6.6 outlines the level of potential geomorphic effect for each reach and effects management recommendations.

Table 6.6: Summary of potential geomorphic constraints for each reach, and recommended management options

Reach	Geomorphic effect	Likelihood	Consequence	Geomorphic effect	Effects management options
Tenn-05 & Tenn-06	Potential for bed/bank erosion during storm events as result of increased velocity within confined margins concentrating flow in bank full and terrace margin extent.	Likely	Low	Low	None required.
HS_US1	Bed and Bank erosion as a result of increased velocity.	Likely	Low	Low	None required.
	Scour around bridge piers.	Unlikely	Medium	Moderate	Mitigation of erosional scour around bridge piers in the form of rock rip rap has been included in the stream design.
	Bank instability leading to the undermining of the wastewater infrastructure.	Likely	Medium	Moderate	None required.
HS_DS1	Scour in the meanders in the tidal flat.	Unlikely	Low	Low	None required.
	Erosion of the terrace margin in high magnitude low frequency events (such as the 100-year ARI) that are contained within the terrace margins.	Likely	Medium	Moderate	Monitoring for geomorphic effects particularly after flood events in excess of 50-year ARI is considered appropriate.

Reach	Geomorphic effect	Likelihood	Consequence	Geomorphic effect	Effects management options
	Bed and bank erosion near the reclaimed land on TRB.	Likely	Medium	Moderate	Monitoring for geomorphic effects.
HS_US1 & HS_DS1	Low flow channel erosion effects.	Likely	Low	Low	Mitigation of potential low flow channel erosion effects through the vegetation planting plan.

7 Conclusion

As a result of the assessment of coastal hazards and effects, it can be concluded that the proposed works will have a negligible impact on coastal hazards. The coastal effects assessed include the occupation of the CMA, the coastal water levels and wave conditions, tidal velocities, tidal channel alignment, and coastal flooding and erosion.

The fluvial geomorphic assessment has assessed the potential effects of the proposed upgrades on geomorphic processes in the immediate vicinity of the Tennessee Ave Dam embankment Bridge works. There is no residual geomorphic risk in Reaches Tenn-05 to the upstream portion of HS_DS1, and a low likelihood for geomorphic effects in the downstream end of HS_DS1. Effects management actions are not required between Tenn-05 to the upstream portion of HS_DS1. There is potential for erosion scour at the bridge piers however, as per the stream design drawings, rock rip rap will be installed in these areas. Therefore, any risk is considered to be appropriately mitigated. At the downstream end of HS-DS1 (reclaimed land) monitoring for geomorphic effects particularly after flood events in excess of 50-year ARI is considered appropriate.

MART/ALTI/ELLS

8 Applicability

This report has been prepared for the exclusive use of our client Auckland Council - Healthy Waters, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

We understand and agree that our client will submit this report as part of an application for resource consent and that Auckland Council as the consenting authority will use this report for the purpose of assessing that application.

Tonkin & Taylor Ltd
Environmental and Engineering Consultants

Report prepared by:	Authorised for Tonkin & Taylor Ltd by:
....	....
Selene Conn Senior Fluvial Geomorphologist	Chris Bauld Project Director
Peter Quilter Senior Coastal Engineer	

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Appendix A Field visit observations

In this appendix, the photos that were taken during the field visit are listed. In Figure Appendix A.1 a schematic overview of the project area is depicted. This overview is not drawn to scale. In the map overview of the project area, the locations where the different photos were taken are indicated. The photos are depicted in Figure Appendix A.2 till Figure Appendix A.8.

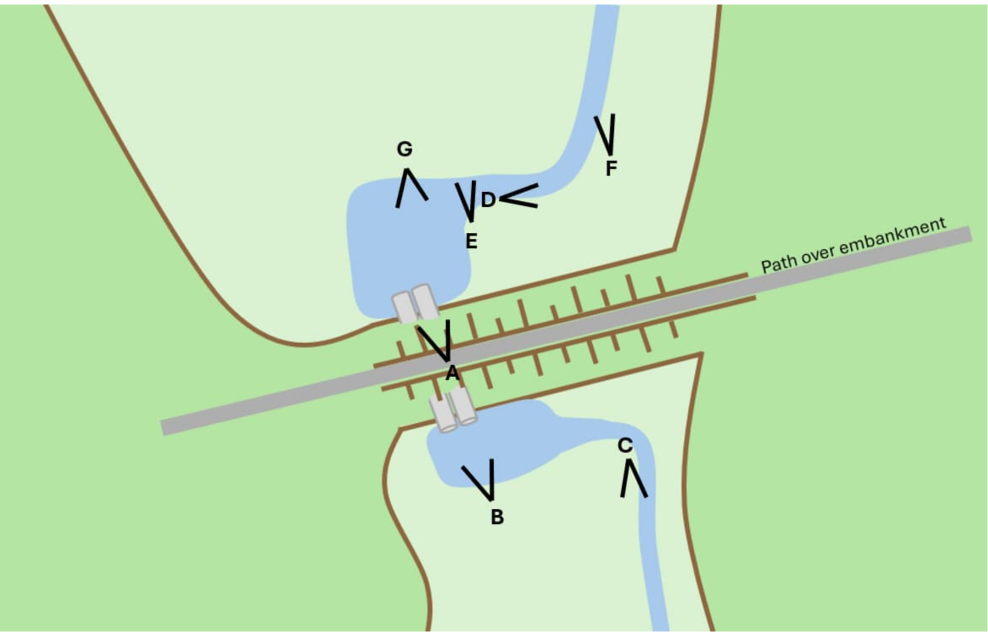

Figure Appendix A.1: Schematic overview of project site (not to scale). In the overview, locations of the different photos that were taken are indicated.

Figure Appendix A.2: Photo A.

Figure Appendix A.3: Photo B.

Figure Appendix A.4: Photo C.

Figure Appendix A.5: Photo D.

Figure Appendix A.6: Photo E.

Figure Appendix A.7: Photo F.

Figure Appendix A.8: Photo G.

Appendix B Fluvial Geomorphology Methodology

In this section the effects caused by fluvial processes are described in the existing condition and post Tennessee Bridge culvert upgrade using the 'worst case' modelled scenario.

B1 T+T model version: HaraniaBGN 202408_v05 scenarios and results

The T+T model version: run conducted in August 2024 titled *HaraniaBGN 202408_v05*[26] involved boundary conditions which considered annual recurrence interval (ARI), degrees of climate change (°C), maximum probable development (MPD) and tidal stage (Mean Low Water Springs[27] (MLWS)). The boundary conditions were then modelled against existing conditions of the flows through the culvert in the embankment and post development of the Tennessee Bridge. The pre-existing condition refers to the current embankment being retained onsite (herein referred to as "BASE"), and the post development construction involves the Tennessee Bridge culvert upgrade (herein referred to as "TB01").

The TB01 scenario is deemed to be the worst case and therefore effects under a different climate or tidal cycle are not considered. The boundary conditions in this scenario are summarised in Table Appendix B.1 below.

The velocity outputs from the worst-case scenario are referred to in relation to five specific locations along the stream profile. Figure Appendix B.1presents the locations within the catchment with the associated naming convention and Appendix C details the side profile of the cross sections and expected water levels under T+T model version: *HaraniaBGN* 202408_v05.

[26] Results discussed in this section relate to T+T model versions "BASE (Existing scenario)_202408_v05" and "TB01 (Tennessee Bridge option)_202408_v05".

[27] Mean Low Water Spring (MLWS) - The average of the levels of each pair of successive low waters, during that period of about 24 hours in each semi-lunation (approximately every 14 days), when the range of the tide is greatest (spring range).

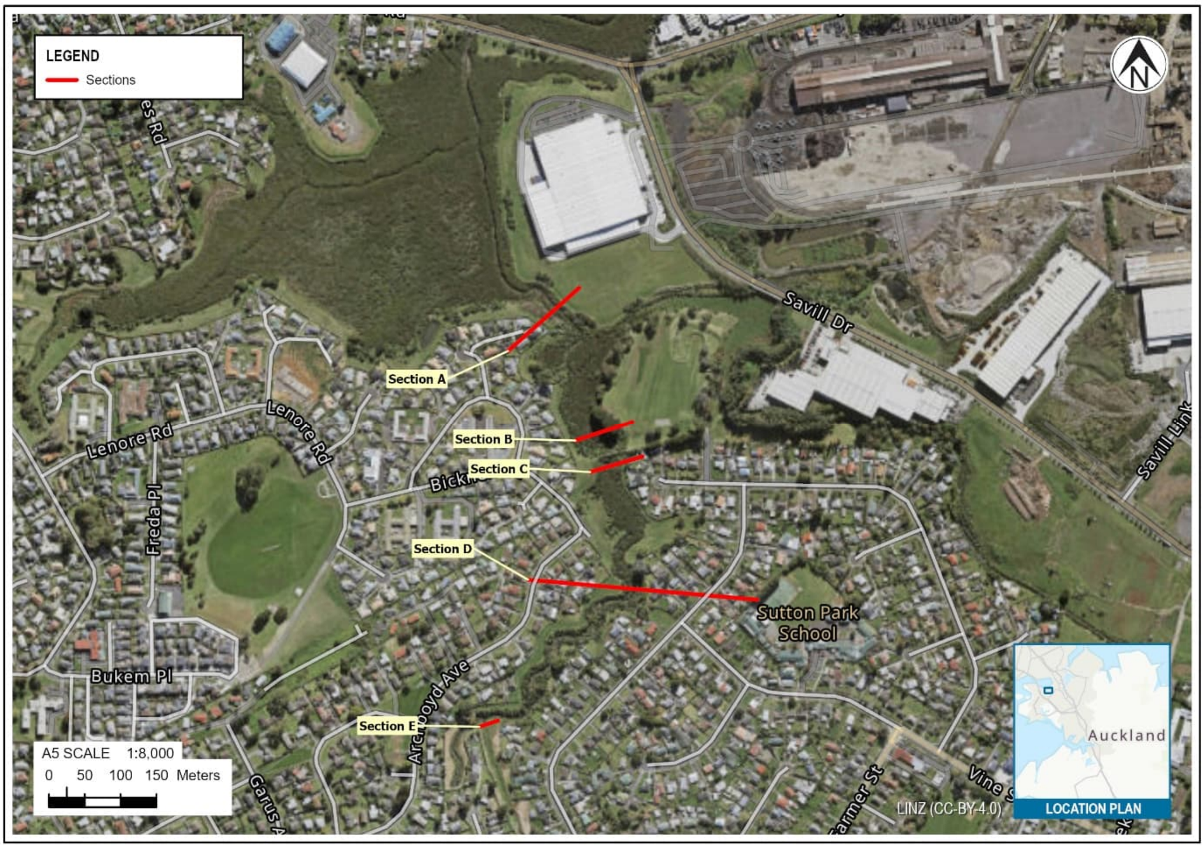

Figure Appendix B.1: Cross section locations within Harania Creek catchment

Table Appendix B.1: Summary of boundary conditions modelled in Scenario C

Recurrence interval	Degrees of climate change	Tidal stage
1% AEP	3.8°C	MLWS 'low' tailwater condition of -0.78 m Auckland Vertical Datum (AVD46).

Model outputs are considered in a spatial form (raster) and in tabulated form (graphs and tables of data). When considering modelling values and the differences between them it is important to recognise that peak values can be determined two ways. Values determined from raster layers are targeted to areas of high values, which are spatially inconsistent or heterogeneous whereas the data presented in the graphs or as tabulated data from the model are derived from set modelling ‘sample point’ to allow for the identification of changes between modelling scenarios form the same spatial location. A summary of data used in the effects assessment is provided in Table Appendix B.2 below.

Table Appendix B.2: Summary of data used in the effects assessment

Metric	Data type
Water Depth	Raster
Max Velocity	Tabulated
	Raster
Discharge	Tabulated
Water Level	Raster

	Cross Section
Water Depth Difference (between BASE and TB01 Development)	Raster
Max Velocity Difference (between BASE and TB01 Development)	Raster

Note:

Raster – a continuous layer of data.

Tabulated – a table of values.

Cross section – a 2-dimensional drawing of the valley shape with estimated water levels plotted against the drawing.

B2 BASE model results for discharge and velocity

Figure Appendix B.2 graphs the discharge relationship for BASE. The stream discharge reaches a peak discharge of 16.2 m^3s^{-1} at the midpoint of the catchment at Vine Street. The peak is reached 12.6 hours after the start of the event. Stream discharges at Section A range from 4.38 m^3s^{-1} at the downstream reach of the embankment at the reclaimed land to 0.1 m^3s^{-1} in the upper reaches at section E, 24 hours after the start of the flow event.

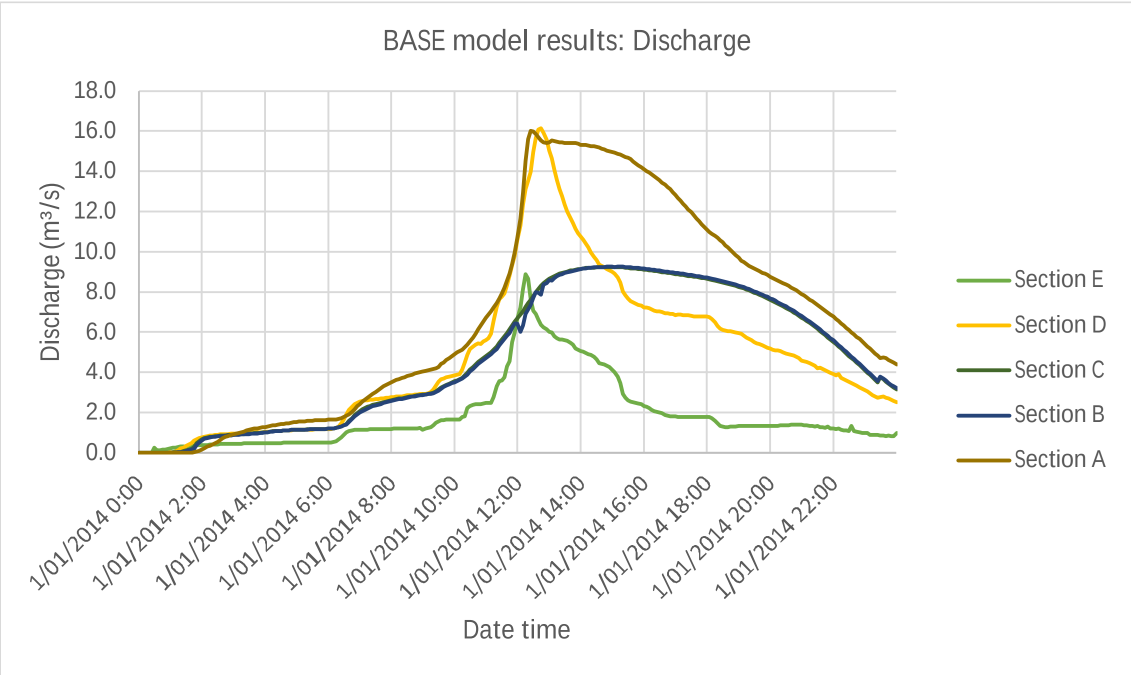

Figure Appendix B.2: Peak discharge with current embankment in place, under the BASE development scenario.

Figure Appendix B.3 is initially similar to a standard flood hydrograph, whereby discharge steadily increases for 10 hours, before sharply rising to its peak 12 hours after the start of the event. After the peak discharge is reached, it recedes at a steady rate. The net effect of the phenomena is that the area upstream of the embankment remains inundated for a prolonged period.

Stream velocities similarly increase to the peak of the event before slowly receding. The peak velocity for the catchment in this scenario is 0.3 m/s at the downstream point of the embankment. Based on the modelling data, peak velocities are reached approximately 18 hours after the start of the event. Stream velocities reduce to 0.168 m/s upstream of the bridge and 0.144 m/s below the bridge 24 hours after the start of the weather event. Figure Appendix B.3 represents velocities immediately upstream and downstream of the embankment.

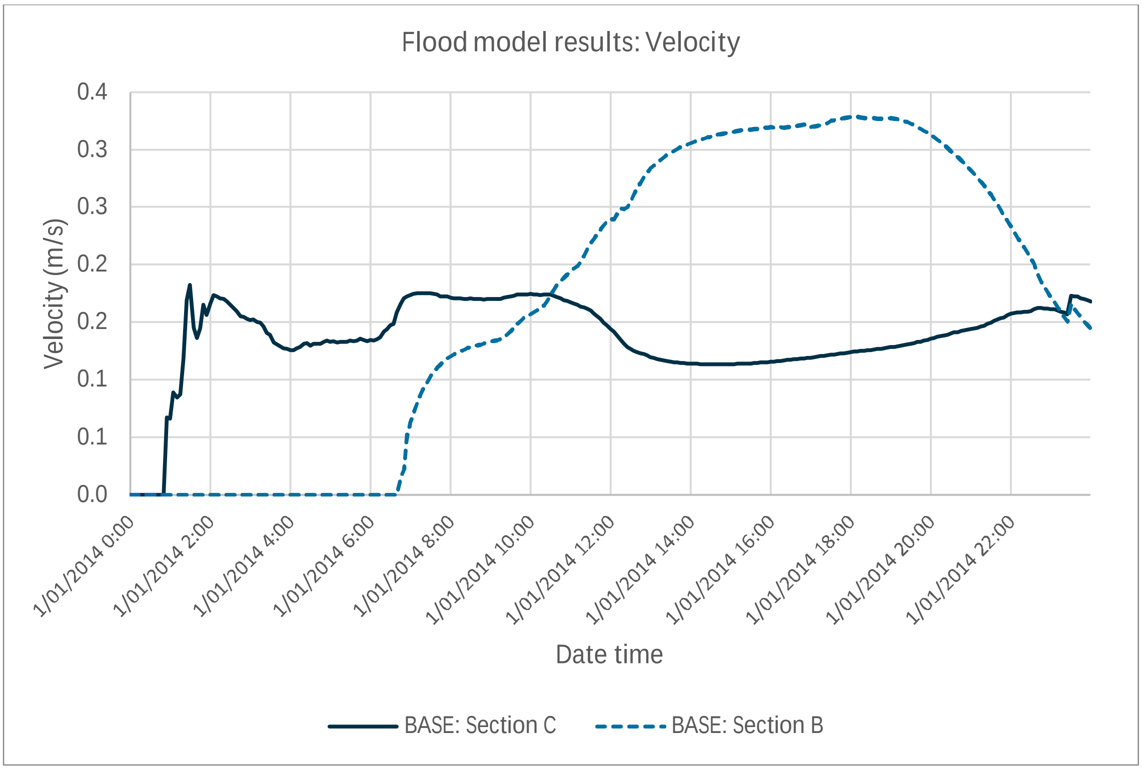

Figure Appendix B.3: Peak stream velocities with current embankment in place, under the BASE development scenario

In addition to the modelling data, the velocity raster, which presents velocities spatially, indicates peak velocities in the catchment range from 0.02 m/s to 1.55 m/s. Table Appendix B.3 presents the peak velocities at each cross section, with respect to the velocities across the cross section (Figure Appendix B.1).

Table Appendix B.3 : Summary of stream velocities at each cross section for the BASE Scenario

Cross-section	True left (TL) flood extent (ms-1)	TL bank full extent (ms-1)	Channel (ms-1)	True right (TR) bank full extent (ms-1)	TR flood extent (ms-1)
Section A	0.12	0.26	1.27	0.91	0.57
Section B	0.022	0.46	8.4	1.01	0.088
Section C	0.07	0.14	0.28	0.19	0.02
Section D	0.1	0.32	0.36	0.24	0.04
Section E	0.23	0.58	1.55	0.62	0.2

Table Appendix B.3 highlights how the BASE – Scenario has a high variability of stream velocities depending on where measurements are taken from. This has implications for potential effects which are addressed in Section 6.

Under the BASE scenario the water level is high and inundates a wide area within the catchment (Figure Appendix B.4). The inundation is a result of low flow conveyance below the embankment.

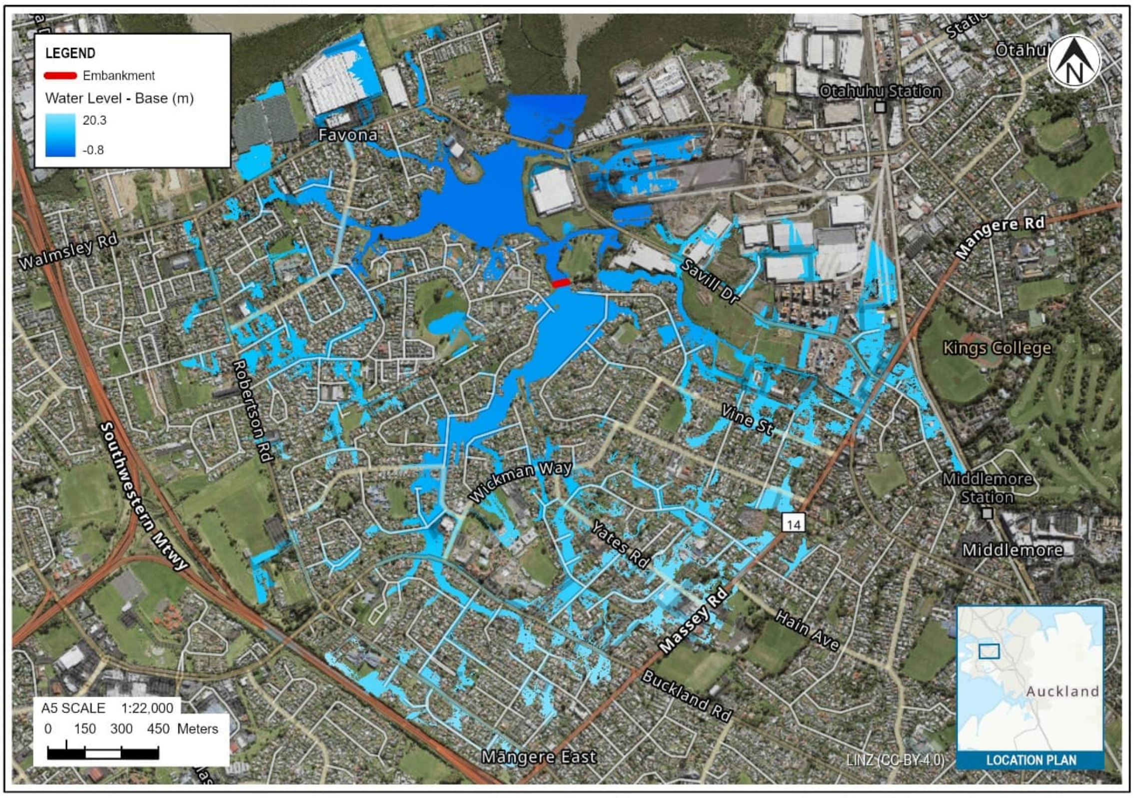

Figure Appendix B.4: Water level in the catchment under the BASE development scenario

B3 TB01 – model results for discharge and velocity

Stream discharge under this scenario reaches a peak discharge of 35.7 $m^3 s^{-1}$ at the downstream point of the catchment where the reclaimed land is noted and the confluence with Blake Road Tributary meet Harania Creek. The peak discharge is reached approximately 13 hours after the start of the event. Stream discharges range from 2.67 $m^3 s^{-1}$ at the downstream reach of the embankment to 0.84 $m^3 s^{-1}$ in the upper reaches 24 hours after the start of the weather event. Figure Appendix B.6 graphs the discharge relationship in this scenario.

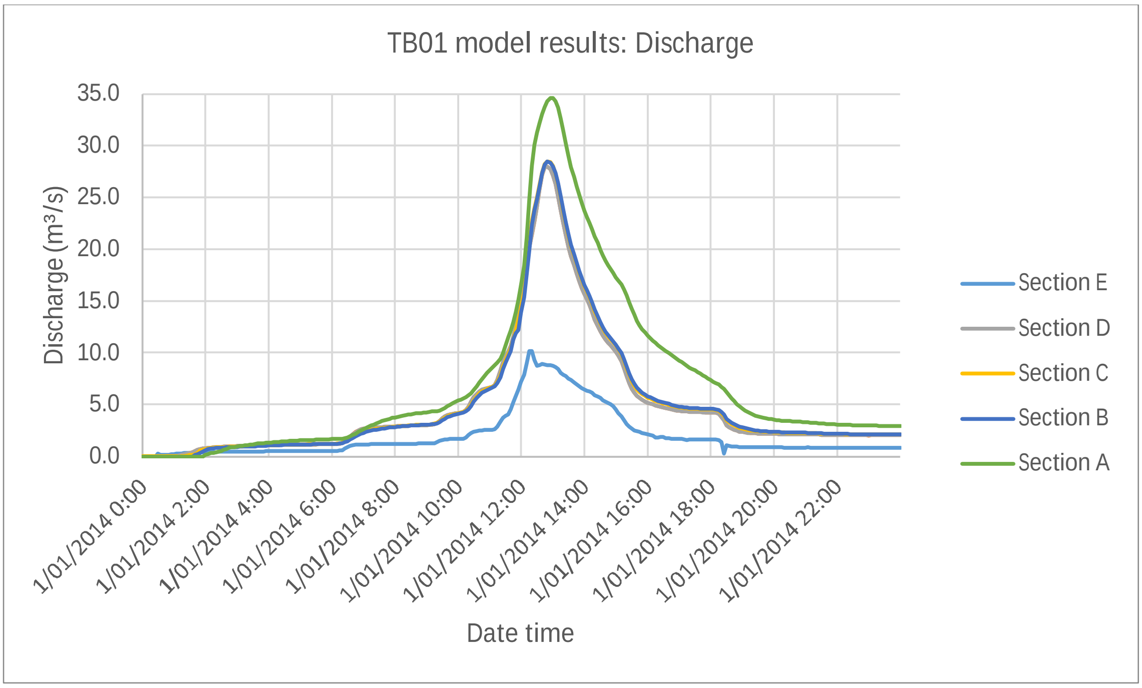

Figure Appendix B.5: Peak discharge with the proposed bridge (TB01) in place (100-year ARI, climate change 3.8°, MPD land use, MHWS+1m tailwater).

Figure Appendix B.6 closely follows a standard flood hydrograph, whereby discharge steadily increases for 10 hours after the start of the event, before sharply rising to its peak 12 hours after the start of the event. After the peak discharge is reached the discharge responds as expected and rapidly decreases in response to a decreasing input of rainfall and therefore freshwater flow upstream. The net effect of the phenomena is that the area upstream of the embankment inundated for a short period.

Stream velocities similarly increase as expected to the peak of the event before quickly decreasing. The peak velocity for the catchment in this scenario is 0.7 m/s at the downstream point of the embankment. Based on the modelling data, peak velocities are reached approximately 13 hours after the start of the event. Stream velocities reduce to 0.051 m/s for scenario TB01 upstream of the bridge and 0.034 m/s downstream of the bridge 24 hours after the start of the weather event. Figure Appendix B.6 represents velocities immediately above and below the embankment.

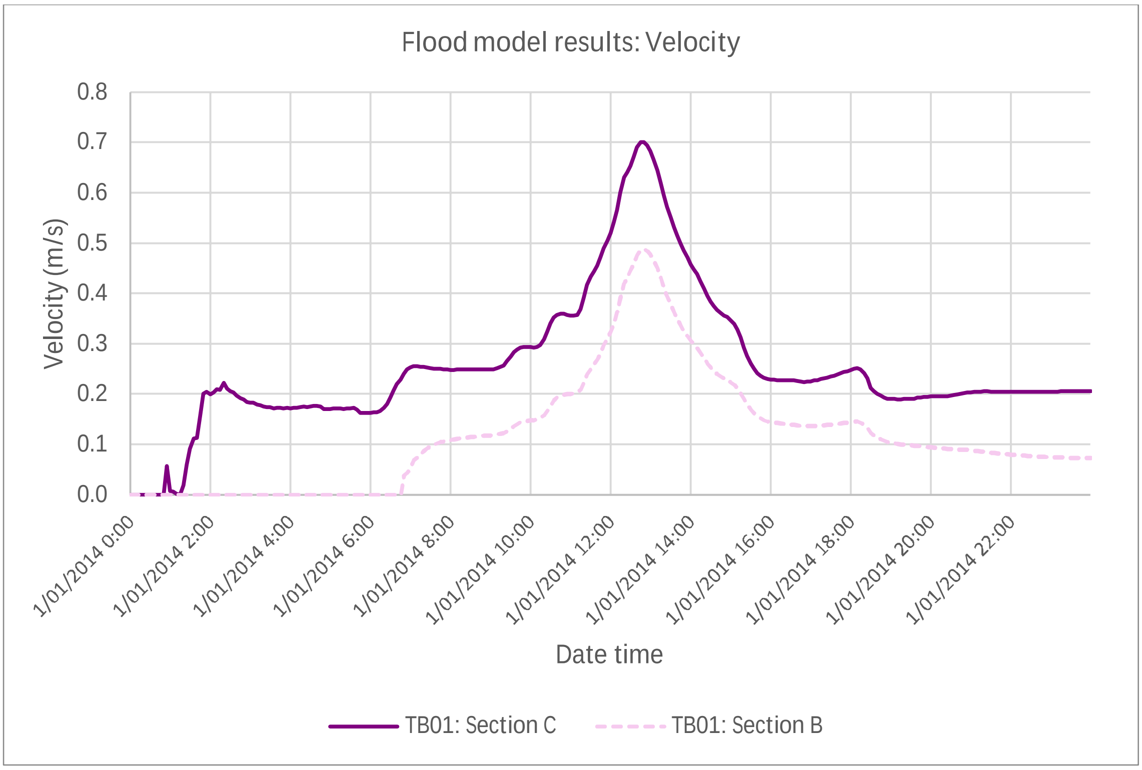

Figure Appendix B.6: Peak stream velocities with current embankment in place (100-year ARI, climate change 3.8°, MPD land use, MLWS tailwater).

In addition to the modelling data, the velocity raster indicates peak velocities in the catchment range from 0.18 ms^{-1} to 2.08 ms^{-1}. Table Appendix B.4 the peak velocities at each cross section under TB01 scenario, with respect to the velocities across the cross section.

Table Appendix B.4 : Summary of stream velocities at each cross section for TB01

Cross-section	True left (TL) flood extent (ms-1)	TL bank full extent (ms-1)	Channel (ms-1)	True right (TR) bank full extent (ms-1)	TR flood extent (ms-1)
Section A	0.37	0.64	1.9	0.7	0.56
Section B	0.18	0.43	0.76	0.52	0.33
Section C	0.28	0.56	0.75	0.51	0.28
Section D	0.32	0.68	0.89	0.7	0.24
Section E	0.3	1.42	2.08	1.3	0.7

Table Appendix B.4 highlights how the TB01 scenario has a high variability of stream velocities depending on where measurements are taken from. This has implications for potential effects which are addressed in Section 6.

Under the TB01 scenario the water level inundates a smaller area within the catchment, appearing to occupy areas within the 100-year floodplain terrace. Figure Appendix B.7 shows the extent of inundation with the proposed bridge in place. The inundation is a result of high flow conveyance below the embankment.

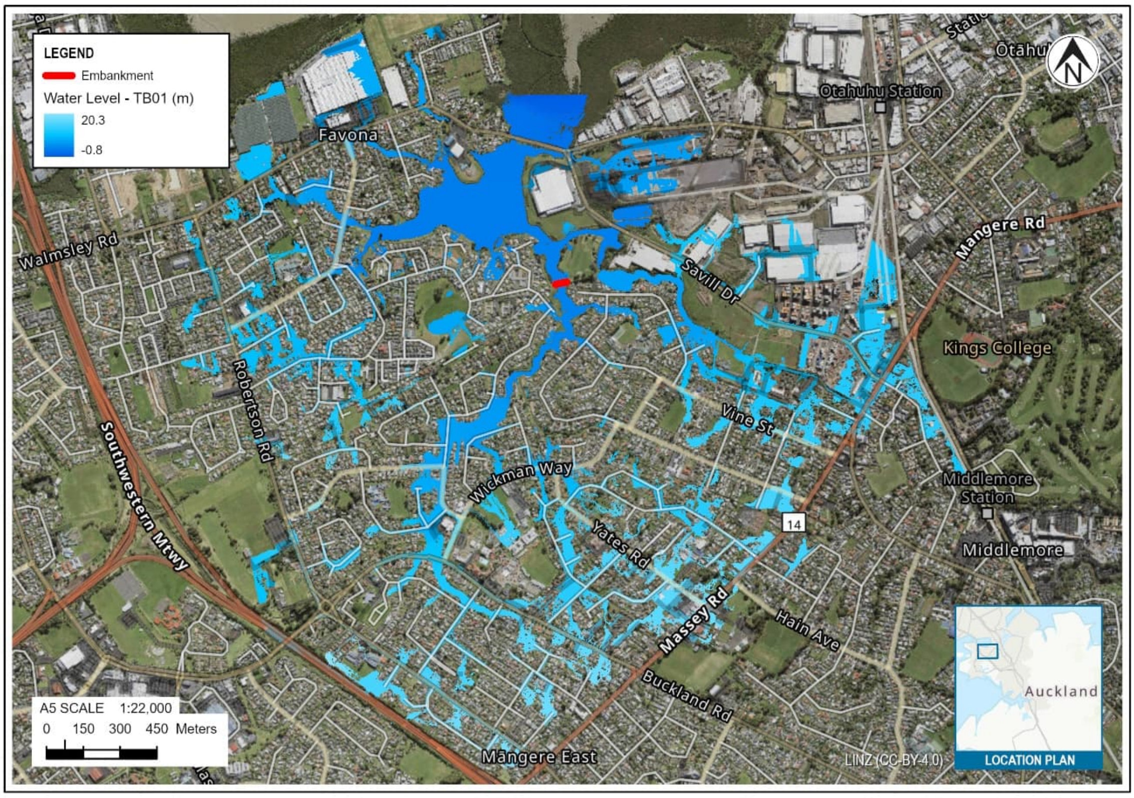

Figure Appendix B.7: Water level in the catchment under the TB01 development scenario.

B4 Comparison of scenarios

The key difference between the scenarios is the reduction in inundated space as indicated in Figure Appendix B.4 and Figure Appendix B.7. The comparison of these figures shows the more homogenised stream velocities throughout the stream long profile with higher velocities upstream of the proposed bridge (scenario TB01), as velocity is unrestricted by the culvert.

Figure Appendix B.8 shows a prolonged discharge scenario under BASE compared to TB01. However, TB01 reaches almost 3 times greater discharge compared to BASE. Figure Appendix B.8 plots discharge of both development scenarios together to show the higher peak discharge and prolonged discharge.

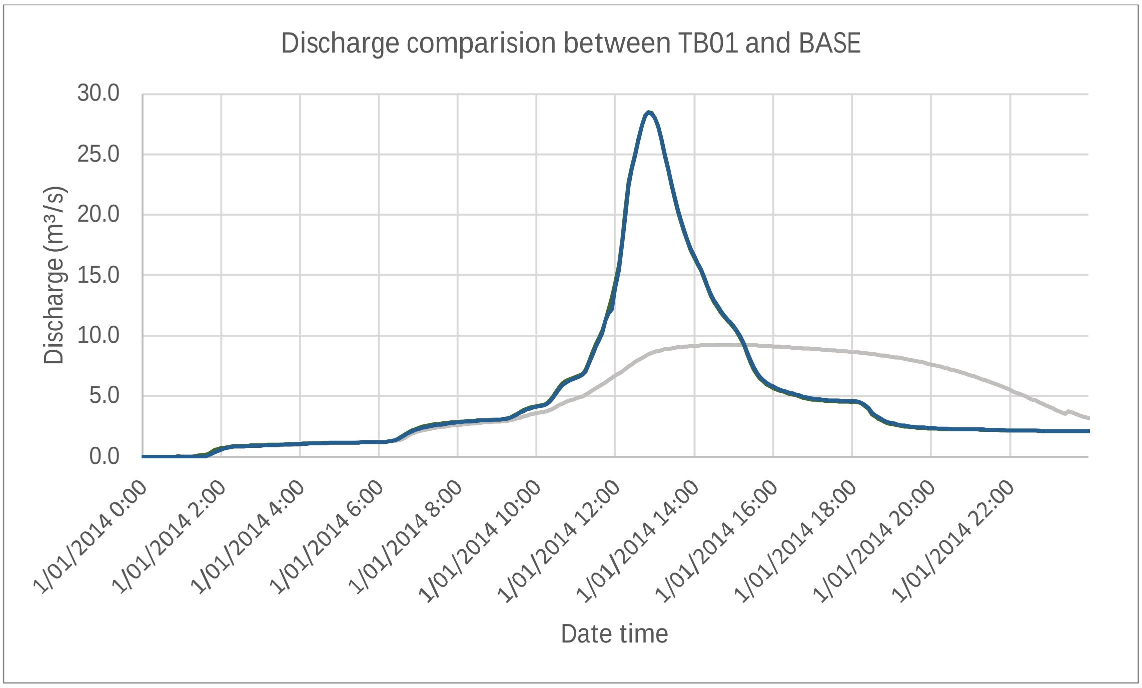

Figure Appendix B.8: Peak stream discharges with both development scenarios in place. BASE (grey Line) and TB01 (blue line).

Velocities under TB01 are faster on average across the long profile however peak velocities are lower compared to areas of concentrated and high velocity under base scenarios. Figure Appendix B.11 shows the key areas of high velocity, low velocity and areas of high change. Table Appendix B.5 shows the change in velocity at the cross sections from the raster layer (Figure Appendix B.1). Whilst Figure Appendix B.9 shows the difference in velocities in the development area (upstream and downstream of the current embankment).

Table Appendix B.5: Summary of the difference in stream velocities between BASE scenario and TB01 scenario at each cross section

Cross Sections	TL 100-year floodplain terrace		TL bank full extent		Active Channel		TR bank full extent		TR 100-year floodplain terrace	
	Velocity difference (m/s)	Difference (%)	Velocity difference (m/s)	Difference (%)	Velocity difference (m/s)	Difference (%)	Velocity difference (m/s)	Difference (%)	Velocity difference (m/s)	Difference (%)
Section A	0.25	208	0.38	146	0.63	50	0.39	43	-0.01	-2
Section B	0.158	718	-0.03	-7	-0.23	-30	-0.13	-20	0.242	275
Section C	0.21	300	0.42	300	0.47	168	0.32	168	0.26	1300
Section D	0.22	220	0.36	113	0.53	147	0.46	192	0.2	500
Section E	0.22	96	0.84	145	0.53	34	0.68	110	0.3	150

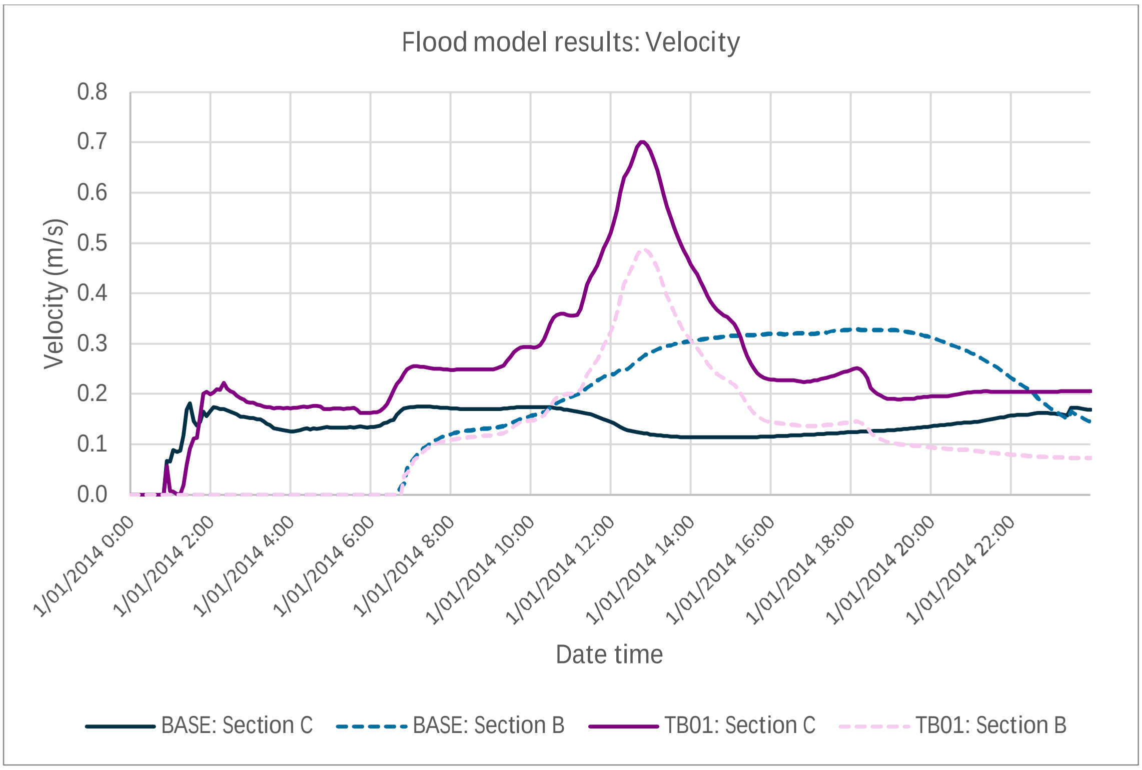

Figure Appendix B.9: Peak stream velocities with both development scenarios (BASE and TB01) in place.

Water levels are reduced under TB01 as flood flows are conveyed into the tidal zone between the catchment and Savill Drive Bridge. Figure Appendix B.10 shows the change in water level and Figure Appendix B.11 shows comparison of TB01 and BASE water depth and velocity.

Figure Appendix B.10: Comparison of water level under the two development scenarios (BASE, left and TB01, right).

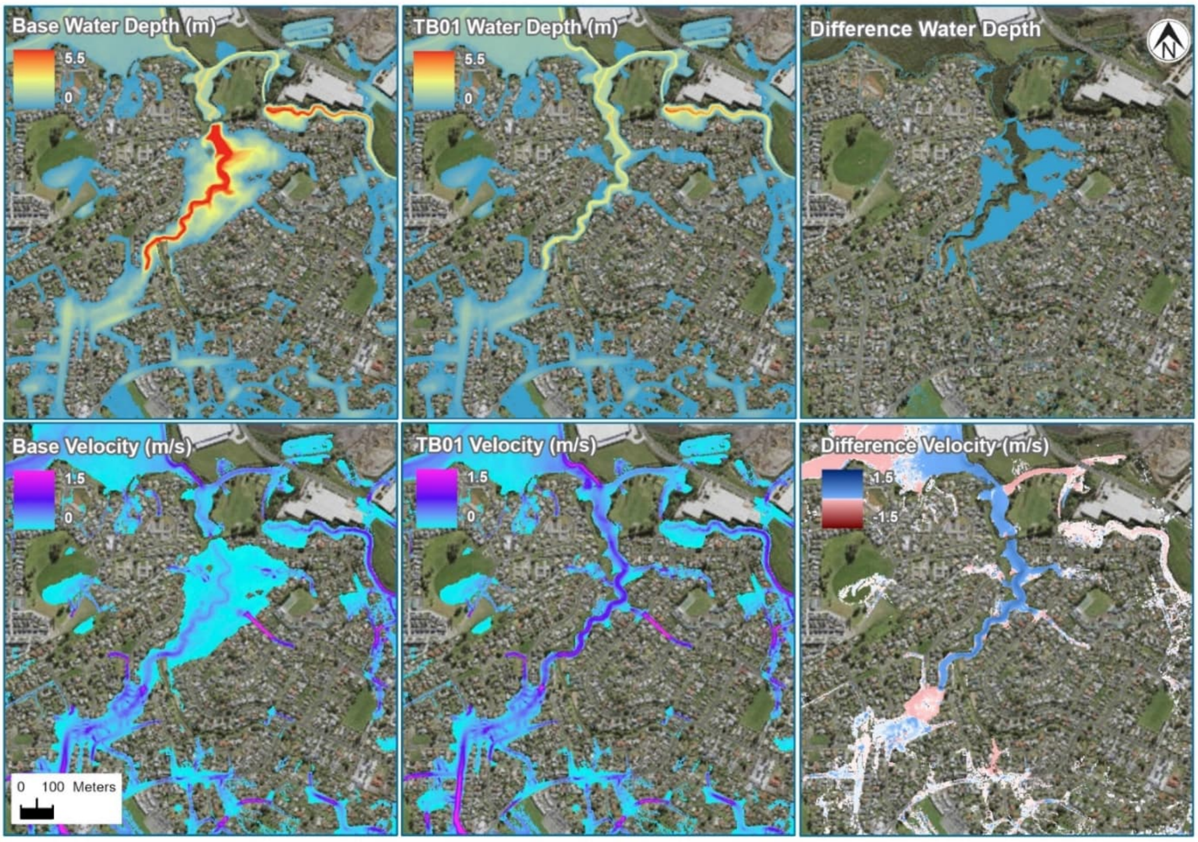

Figure Appendix B.11: Comparison of GIS raster layers between development scenarios.

Appendix C Cross Sections

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Figure Appendix C.1: Cross section overview map. Detailed cross sections A to E are referenced below with comparison of water levels (WL) between BASE and TB01.

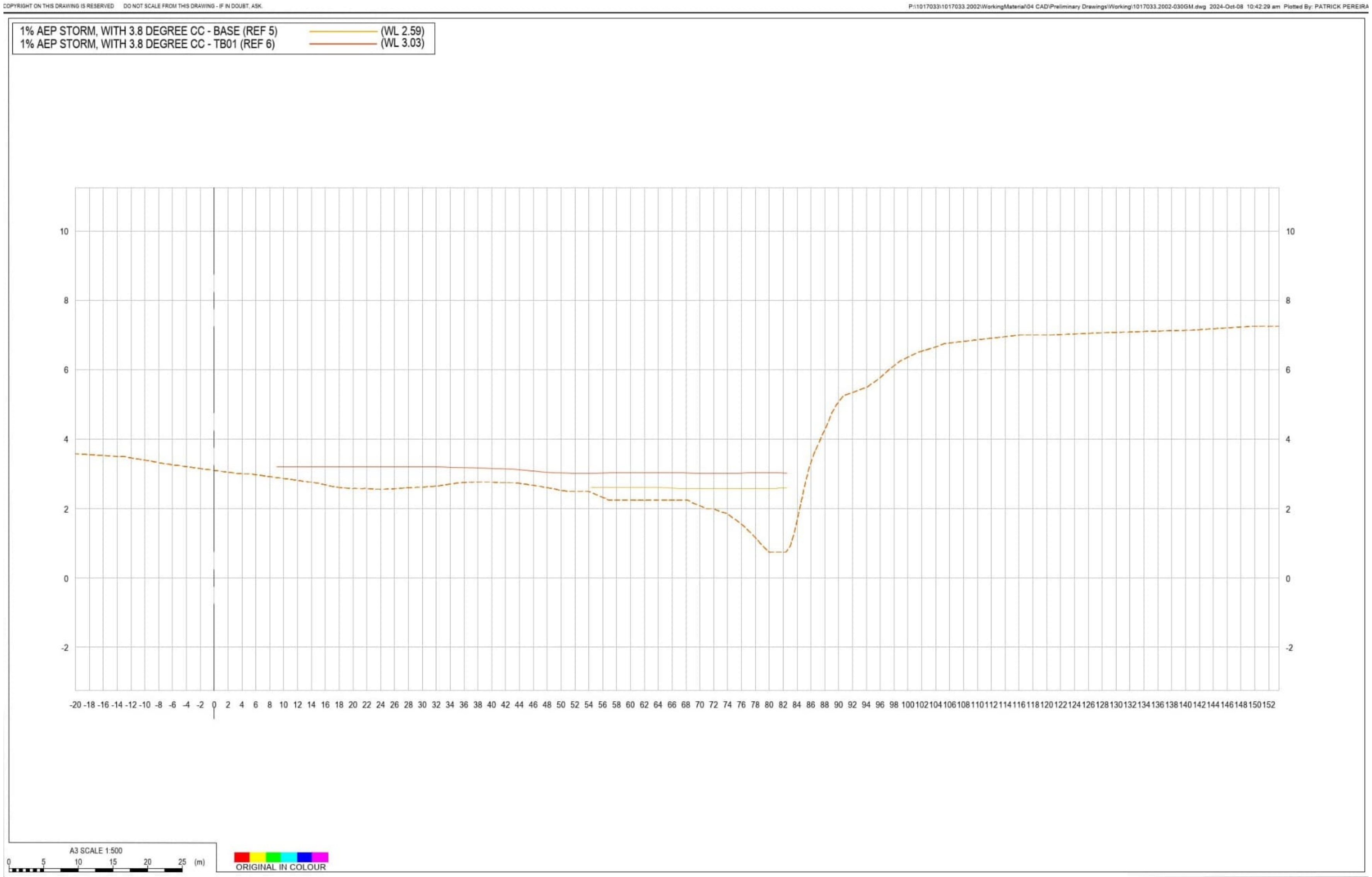

Figure Appendix C.2: Cross Section A.

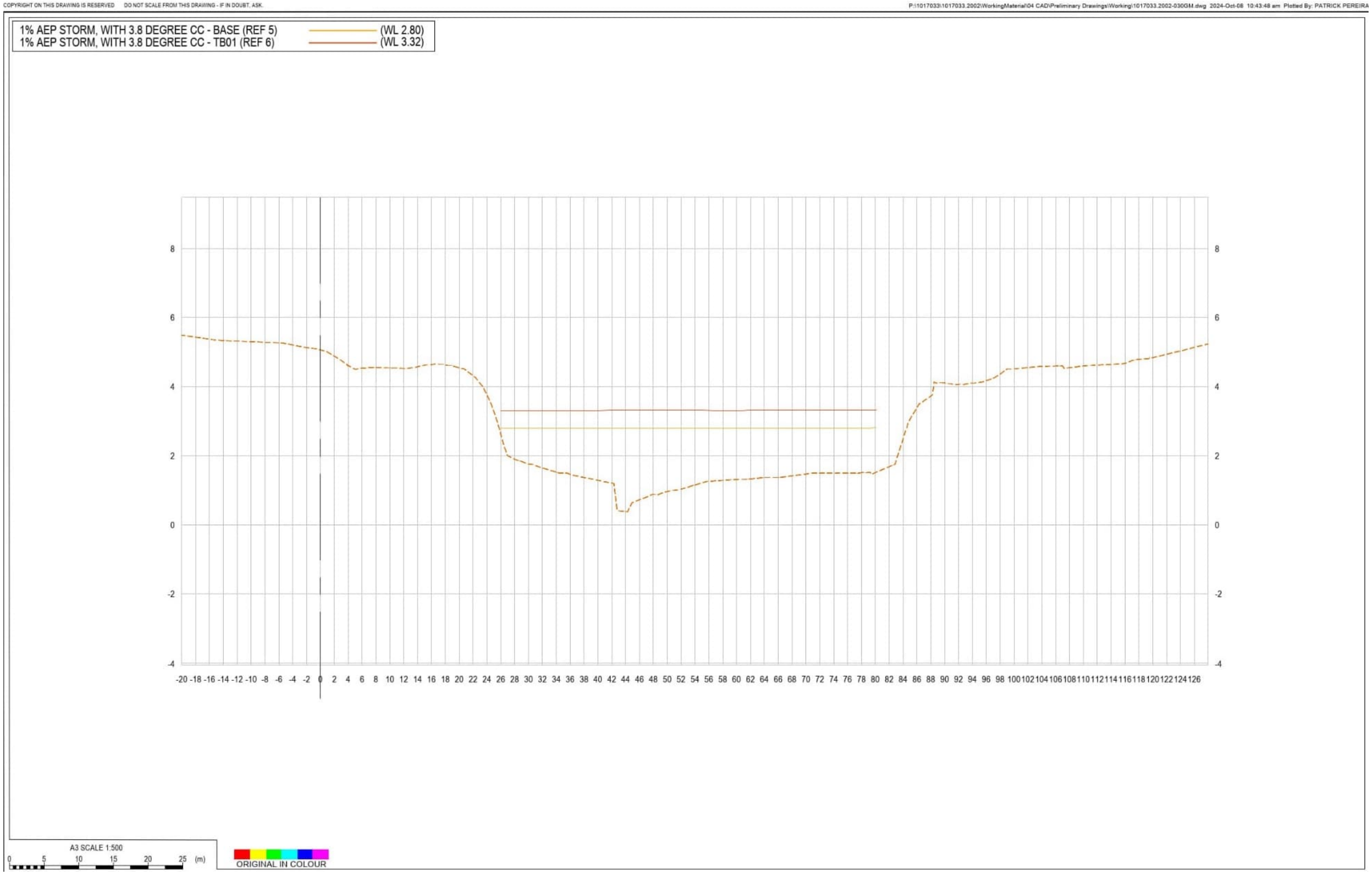

Figure Appendix C.3: Cross Section B.

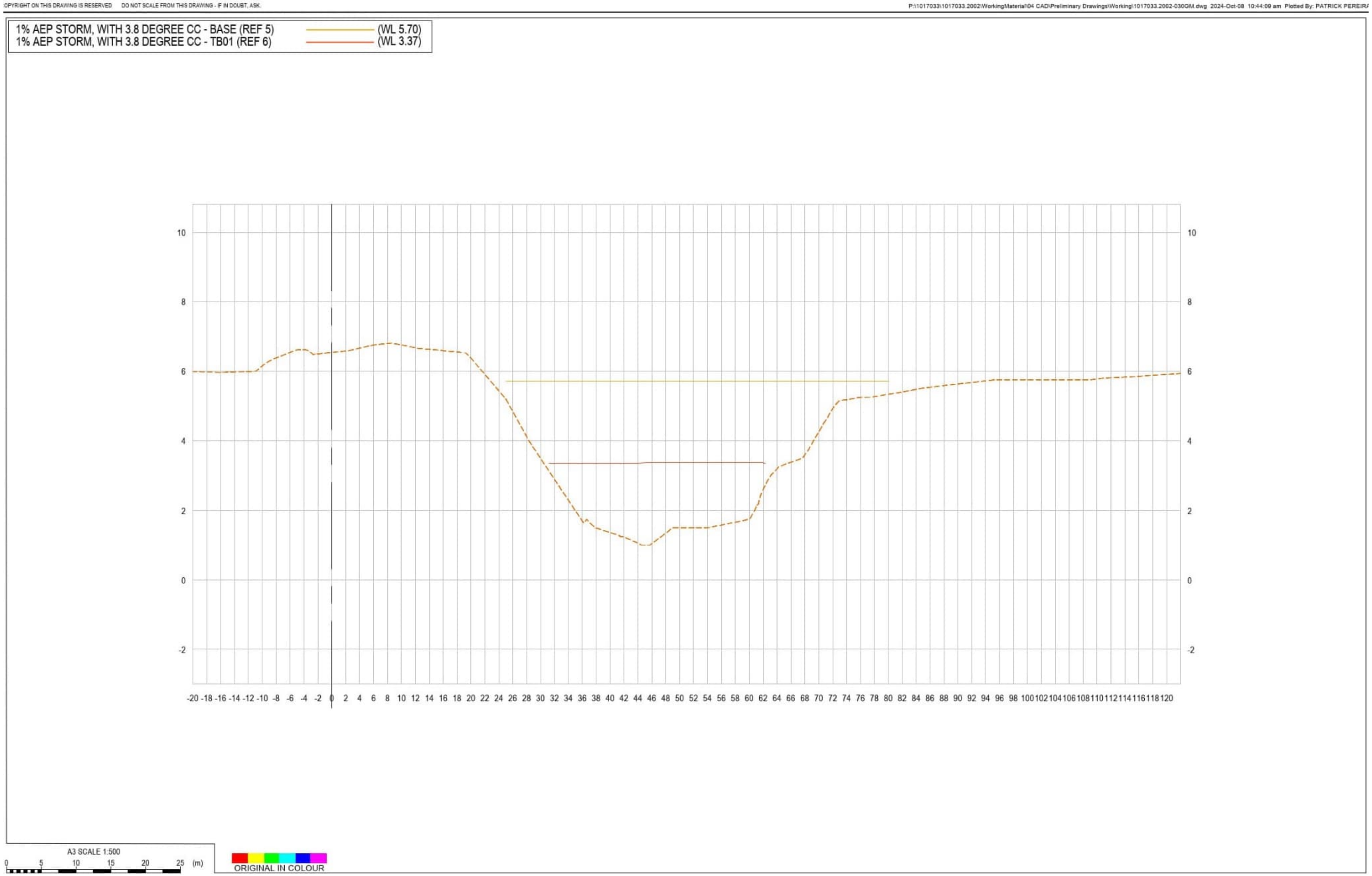

Figure Appendix C.4: Cross Section C.

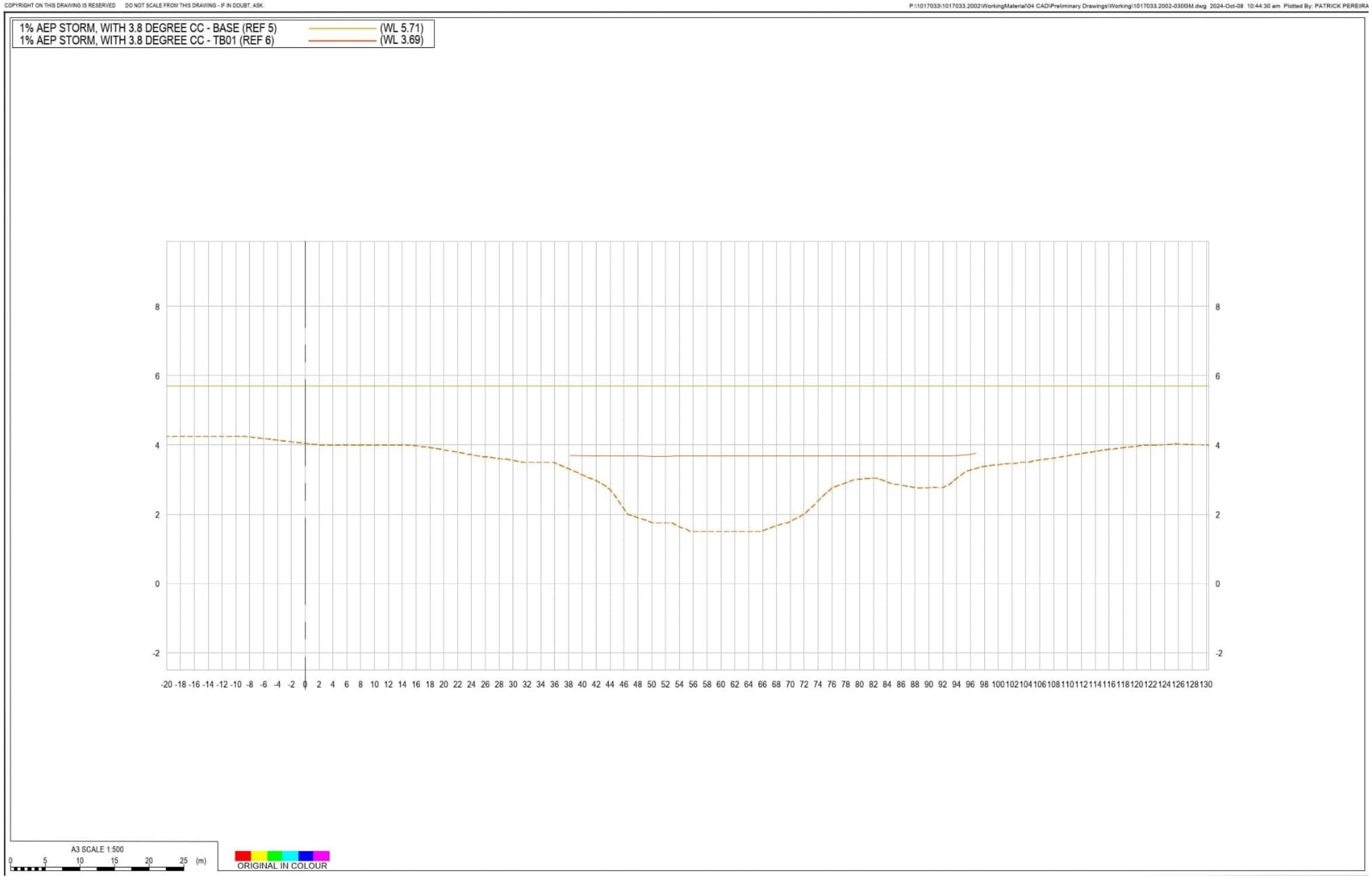

Figure Appendix C.5: Cross Section D.

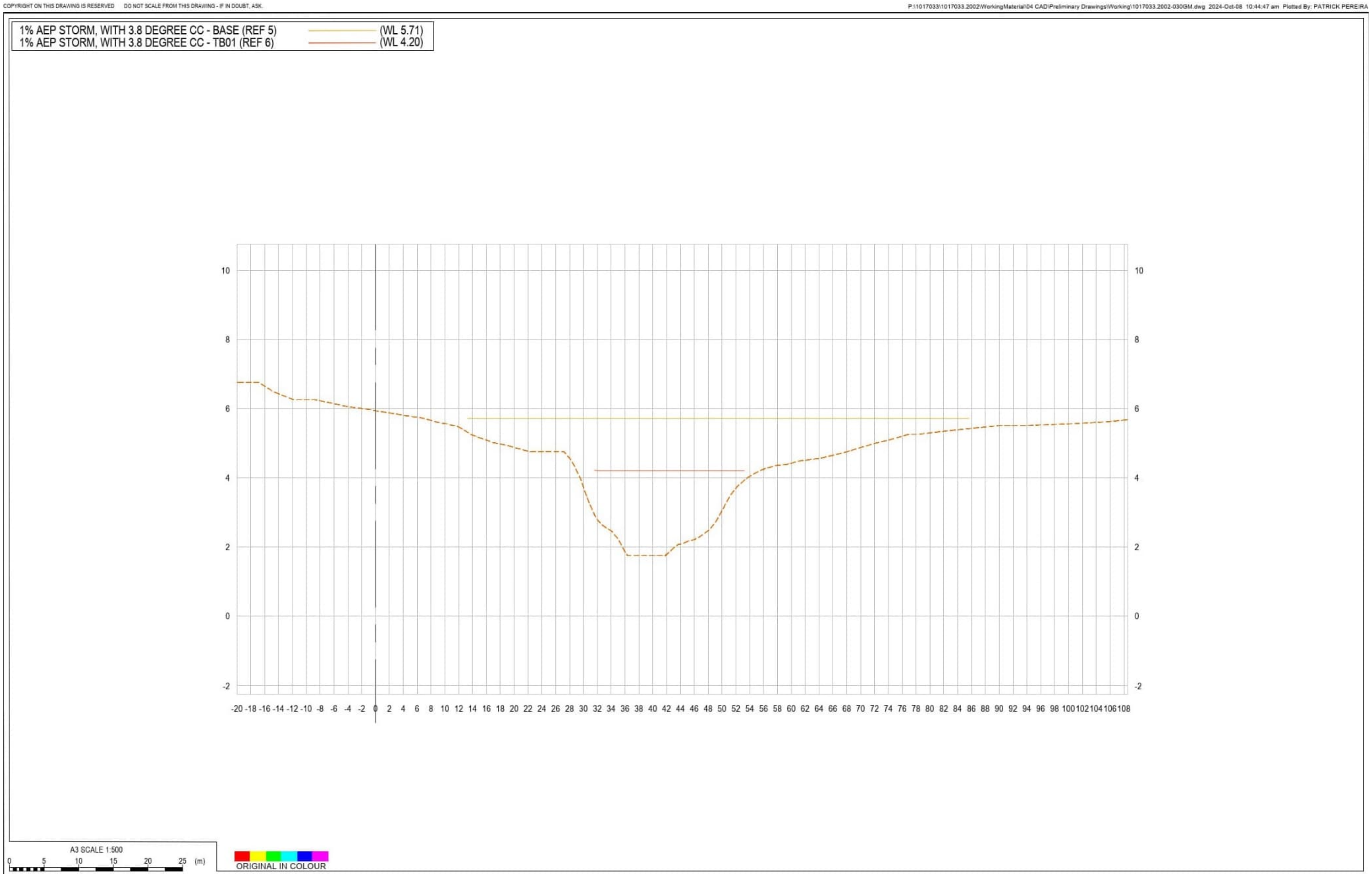

Figure Appendix C.6: Cross Section E