REPORT

Coastal Protection Feasibility Study for the Coromandel Peninsula - A Summary

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1 Introduction

1.1 Background

This report provides a summary of a feasibility study undertaken into possible coastal protection for eight locations in the Coromandel Peninsula at imminent or extreme risk of coastal inundation (see **Figure 1**). The study has been undertaken by Royal HaskoningDHV (RHDHV) for Thames Coromandel District Council (TCDC) as part of the Thames Coromandel Shoreline Management Plan (SMP) project.

It is important to note that the possible defence options developed as part of this work are concepts only. They are not proposed designs; they are not full or outline designs for implementation. They have been developed for the purpose of identifying constraints, assessing feasibility and developing high-level costs estimates.

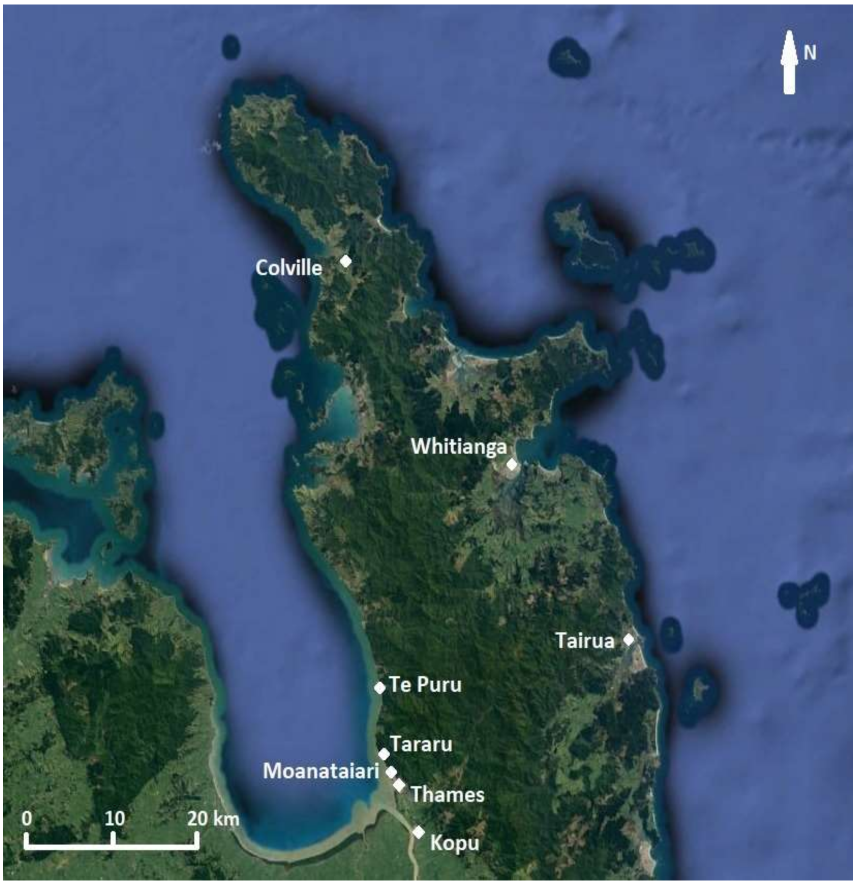

Figure 1: *Communities on Coromandel Peninsula assessed in this study*

In line with the Ministry for the Environment's *Coastal Hazards and Climate Change Guidance for Local Government* (2017) the SMP project has undertaken hazard, vulnerability and risk assessments (see **Figure 2**) and identified those locations where the risks associated with coastal erosion and inundation in particular are imminent and/or high to extreme (as well as those locations where they are not). These locations typically face a stark choice between defending (with significant infrastructure) or retreating. Therefore, to inform the process of adaptation option and pathway identification and evaluation in these locations (option selection; see Steps 5 and 6 in **Figure 2**), the feasibility of 'defending' has been examined.

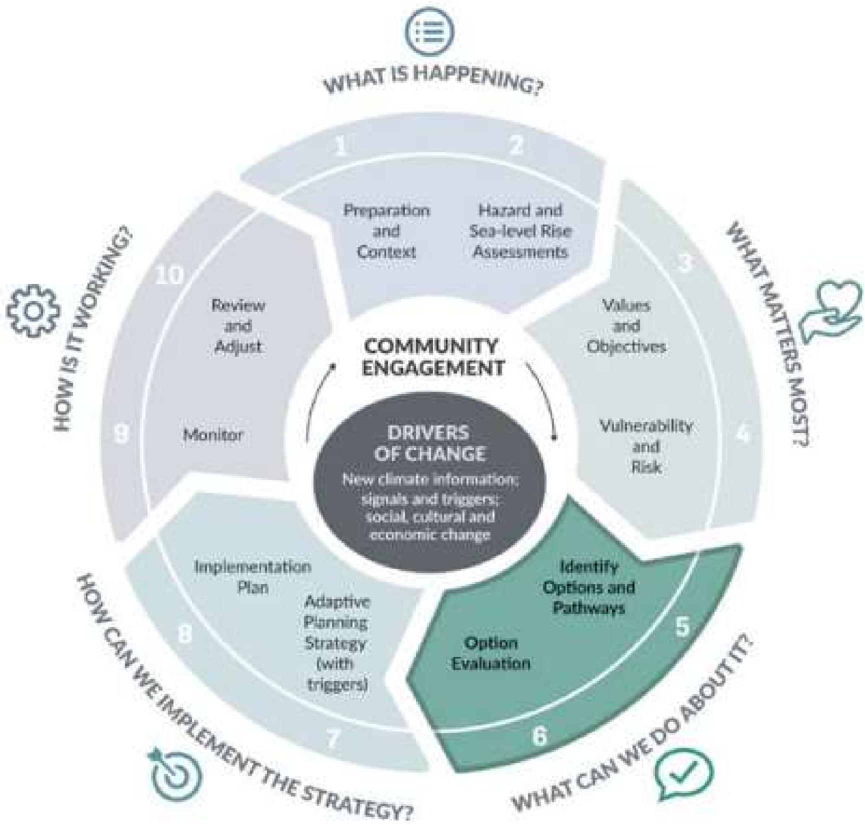

***Figure 2:** MfE's Coastal hazards decision cycle*

1.2 Scope

The scope of the investigation comprised:

- High-level assessment of predicted sea levels and the wave climate during a 100 year annual return period (ARI) storm in 100 years' time (2120).
- Investigation of possible coastal defence options with a primary focus on earth embankment seawalls and their required crest level, and vertical walls where there were space constraints.
- Protection of the proposed structures from coastal erosion.
- Consideration of requirements for pumping the 'internal' catchment, including where elevated ocean water levels prevent gravity drainage.
- Preliminary high-level cost analysis.
- Commentary on constraints and feasibility.

The assessment excluded:

- Staging scenarios.
- Detailed hydrologic and hydraulic modelling of associated creeks.
- Sizing of internal drainage structures, e.g., stormwater pipes, overflow routes and culverts.
- Detailed numerical modelling beyond that already undertaken for the SMP.
- Outline or detailed design of coastal defence and flood prevention structures.
- Fluvial flooding and groundwater (drainage and pumping) assessment.
- Cost assessments by a quantity surveyor.

2 Datums

Across the Coromandel Peninsula several vertical datums are in use. For simplicity and consistency, this report has adopted the New Zealand Vertical Datum 2016 (NZVD2016).

The surface elevation models that RHDHV have utilised in the preparation of this report, primarily use the Auckland Vertical Datum 1946 (AVD-46), the Moturiki Vertical Datum 1953 (MVD-53) or NZVD2016. To ensure all elevations extracted are consistent, the conversion factors presented in **Table 1** have been applied. A general conversion factor is not available, as the offsets between the different vertical datums vary spatially[1].

***Table 1:** Spatially Varied Offset Factors to NZVD2016*

Location	AVD-49 offset to NZVD2016	MVD-53 offset to NZVD2016
Tairua	0.306	0.286
Whitianga	0.303	0.291
Colville	0.294	0.293
Te Puru	0.313	0.307
Tararu	0.327	0.314
Moanataiari	0.336	0.315
Thames	0.336	0.315
Kopu	0.332	0.315

3 Methodology

3.1 Basis of Design

For this study, the 'Basis of Design' (or objective) for the defence concepts was to protect the eight townships against coastal inundation (as well as stormwater/river flooding) 100 years into the future (up until 2120), considering the joint occurrence of a 1% Annual Exceedance Probability (AEP) coastal storm coinciding with a 5% AEP rainfall runoff event. The design concepts were to be of sufficient elevation to limit the majority of wave overtopping during a 1% AEP coastal storm. That is, they are "belt and braces" options. In addition, the 1% AEP rainfall runoff event was tested against Mean High Water Springs (MHWS) to assess gravity drainage with the walls in place.

Within this Basis of Design, several flood mechanisms (or components) were considered, including:

- Storm tide – a storm surge (driven by wind and low barometric pressure raising water levels) plus the tide creates a storm tide.
- Wave set up (raised water levels in the near shore environment as a result of wave break) occurs on top of a storm tide (see **Figure 3**), and these components were integrated in the inundation modelling.
- Wave run up (which is affected by exposure and so considered separately) and overtopping.
- Sea level rise – conservatively assumed to be 1.4m by 2120 (based on a future with continued high emissions).

Further, the threat from Tsunami was reviewed using the New Zealand Tsunami Hazard assessment, completed by GNS Science in 2014 and a cursory note is made in places where appropriate.

[1] A shapefile (from which the offset values were derived), containing points on a grid demonstrating the spatial variance of the offset factors is publicly available on the LINZ data portal.

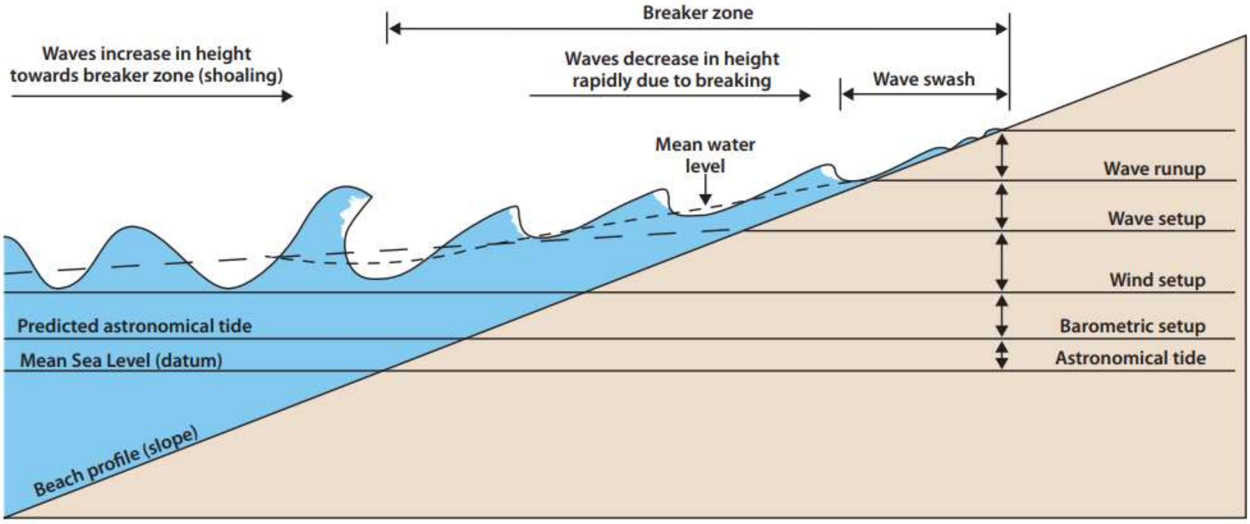

Figure 3: *Storm tide schematic*

The following were not considered:

- Tsunami – however there is a cursory note in places where appropriate. In addition, protection measures for coastal inundation would mitigate tsunami effects.
- Groundwater – based on a sea level rise of 1.4 m over the next 100 years and the topography of the towns in question, it is reasonable to assume that groundwater will be an issue in the future that would require management. This is already apparent in Moanataiari and Thames.
- Subsidence – whilst subsidence can affect relative sea level rise, it was not considered here.
- Changes in storm frequency or the intensity of storms – as these predictions are not definitive.

3.2 Validation of Coastal Inundation Modelling

The design water levels adopted for this study have been derived from the coastal inundation modelling undertaken for the SMP project which has been peer reviewed by TCDC, Waikato Regional Council (WRC) and modelling experts at Auckland University. The storm tide and wave climate outputs broadly align with the outputs from WRC's Inundation Tool and have been validated against a range of past events and topographic features.

3.2.1 January 2018 storm (Thames coast)

For the West coast of the Coromandel Peninsula there have been several recorded storm events that provide insight. The 5 January 2018 storm had an ARI of approximately 100 years, roughly equating to an 1% AEP event in some locations (e.g., Te Puru, but not Thames). This event, and its photographic records, has been used for validation of the SMP inundation modelling against current West coast conditions. Its severity was due to the combination of a tide near the highest astronomical tide (HAT) combined with strong northerly winds and low barometric pressure driving the storm surge. Further, the strong winds were driving significant waves into the shore where the shallow water induced wave breaking and wave setup.

3.2.2 Storm tides – Sugar Loaf Wharf (Coromandel)

Sugar Loaf Wharf has a deck level at approximately 0.3 m above HAT (see **Figure 4**). Conditions at the Wharf were captured during the January 2018 event, when it was completely under water (see **Figure 5**). The site had a storm tide water level of approximately 2.3 m NZVD2016 during this event, corresponding to a recurrence interval of approximately 100 years (1% AEP). This water level compares to the outputs of the coastal inundation modelling and the design levels used for this study. From this, it is clear that the design levels (derived from the model outputs) are consistent with the current 1% AEP storm tide levels for Colville (to the North) at 1.9 m NZVD2016 and Te Puru (to the South) at 2.5 m NZVD2016. It is not possible to compare the wave climates due to the sheltered conditions within Coromandel Harbour.

Figure 4: *Sugar Loaf Wharf operating during a near HAT event*

Figure 5: *Storm tide on 5 January 2018 – water level approximately 2.6 m MVD53 (2.3 m NZVD2016)*

3.2.3 Overtopping assessment (Moanataiari)

The 5 January 2018 event in Moanataiari resulted in overtopping of the existing coastal defence structures. The overtopping was captured in the photos presented in **Figure 6**. The observed conditions during this event have been determined to be close to the 1% AEP design event used herein. That is:

- The storm tide level was 2.86 m MVD53 = 2.55 m NZVD2016 (compared to the SMP inundation modelling and design level of 2.77 m NZD2016).
- Significant wave hight was ~ 1.5 m (compared to 1.43 m).

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- Wave period was ~ 4.4 s.
- Overtopping caused some flooding in low lying areas, but no damage to the embankment (i.e., was < 5 l/m/s).

Figure 6: *Waves overtopping Moanataiari defences on the 5 January 2018 (Note levels indicted are m MVD53)*

Based on the SMP modelling inundation outputs, the calculated values are:

- The overtopping rate = 4 l/m/s.
- Wave runup = 5.1 m MVD53 = 4.8 m NZVD2016.

Hence the calculated conditions (here) reflect the observed conditions (above) and are good evidence that the adopted methodology is robust.

3.3 Commentary on Feasibility

In considering the feasibility of protection from inundation and erosion at each site the following key factors have been addressed:

1. Engineering feasibility – relating to how difficult it would be to build the conceptual defences.
2. Cost relative to the assets threatened – broadly weighing the cost of construction against the value of the assets under threat.
3. Planning and community considerations – safety issues related to catastrophic failure and possible adverse reactions by the community to large barriers between them and the sea.

Given the limited scope of this study, there are several items that have not been considered which have the potential to impact the feasibility of the possible protection options over the next 100 years, including:

- Geotechnical ground improvements for the embankments/walls[2] and potential borrow sources for construction materials.
- Potential groundwater impacts.
- Larger events, the impact of overtopping, and the potential human cost given the likely sense of security provided by possible defences.
- Suggested design alterations to reduce costs (e.g., culvert and penstock gates that could be utilized at stream outlets rather than returning walls up the streams).

4 Possible Protection Options

4.1 Overview

Several protection measures have been considered as possible options for the purposes of developing appropriate concepts (for the purpose of assessing feasibility). The possible approaches to protection adopted for this study were:

- Embankments (bunds) – earth embankments and rock protection.
- Vertical walls.
- Managing fluvial flows – stormwater / river pumping.

Where scour risk is sufficiently low, or negligible, unprotected earth embankments can be considered. However, this was not considered to be applicable here, in most cases.

4.2 Embankments

Earth bunds were adopted as the base case possible protection measure for this study. They are robust and more affordable than alternatives. Examples are presented in **Figure 7** and **Figure 8**.

Figure 7: *Example of an armoured embankment face. Example from Moanataiari*

[2] *Which could increase costs but, equally, a review of likely loadings and detailed design of the T-Walls may reduce piling and concrete costs.*

Figure 8: Turf covered rear face of a high embankment

Key considerations in determining the feasibility of earth embankments include:

- Space to build the bund: this is a major constraint in many places. The bund will occupy a wide strip of land. In urban settings this land requirement potentially will clash with assets, private property, parkland or intertidal ecosystems.
- Bearing capacity of the foundation material: for this assessment it is assumed that the foundation material has sufficient bearing capacity although, prior to any design work, RHDHV recommend detailed geotechnical investigation and testing be undertaken. Additional ground preparation measures may be required (e.g., over-excavation and backfill with bridging material, pre-loading or dynamic ground improvement techniques).
- Seepage: it has been assumed that the foundation material is reasonably impermeable thus only a 2 m deep cut off key has been adopted for the earth embankment. For areas where permeability is higher (assumed 20% of total length), a 12 m sheet pile cut off wall has been assumed. Further geotechnical investigation are required to determine cut off requirements.
- Borrow source of embankment fill / rock armour: haulage can be a significant cost, thus relatively local sources have been assumed.
- Potentially damaging overtopping could occur during extreme events (larger than the 100 year ARI design): to limit scour of the landward face, as a minimum, good turf cover is required. A more robust landside protection, including rock armour or concrete in combination with catch drains and pumping, should be considered in the design phase should any of these concepts be progressed.

Where the embankment is located on a coastal foreshore or riverbank that is exposed to serious erosion, a more substantial solution is required than earth embankments. In these locations an embankment with a front batter solution with more substantial armouring and a deeper toe has been assumed. The extent and size of the additional armouring will need be determined in the design phase should any of these concepts be progressed.

4.3 Vertical walls

Reinforced concrete T-Walls have been proposed as options where there is insufficient room for an earth embankment or where poor foundation materials are likely. The assumed crest height of the seaward facing T-Walls is the same as that adopted for the earth embankments. Such structures can be stand alone or as the crest detail on an earth embankment (or a rock revetment) to reduce the base width. Examples of a coastal defences that includes concrete walls are presented in **Figure 9** and **Figure 10**.

As for the earth embankments, where the T–Wall is located on a foreshore or riverbank that is exposed to serious erosion threats a more substantial solution is required. Hence a front scour protection solution with more substantial armouring and a deeper toe has been proposed for the purposes of this study.

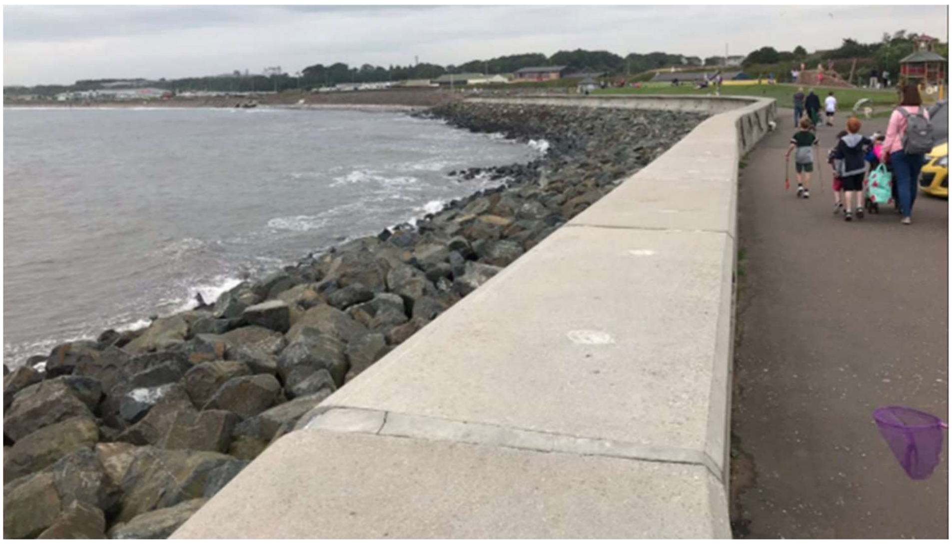

Figure 9: Example of concrete wall flood defence

Figure 10: Example of vertical concrete walls lining a stream

4.4 Managing Fluvial Flows

The construction of such possible coastal protection solutions would have potential impacts on stormwater and fluvial flooding. Sea defences have the potential to effectively dam a catchment, stopping rainfall runoff from freely discharging to the ocean. In addition, predicted sea level rise could affect gravity drainage.

For this study, to assess and determine the volume of rainfall runoff generated by the dammed catchments a simple hydrological model was developed in DRAINS for each site[3]. The stage storage

[3] *Informed by Section 6 of the* High Intensity Rainfall Design System, Version 4, 2018 *prepared by the National Institute of Water and Atmospheric Research Ltd.*

relationship was determined through analysis of the surface elevation model in GIS. With this information, the height of the water level behind the embankments was able to be calculated.

The initial flooding assessment considered landside elevations and compared them to predicted sea levels 100 years into the future. The aim of this assessment was to determine if passive drainage of the catchments was possible without flooding any residential or commercial areas during a MHWS tide. As can be seen from **Table 2** gravity drainage of the dammed catchments during a MHWS tide (including sea level rise to 2120) is not possible, as the majority of the townships have dwellings situated below MHWS. As a MHWS tide is a relatively frequent event and any rainfall that occurred during this tide would be unable to drain, the risk of flooding would be unacceptable and pumping of river and stormwater discharge required if sea defences are constructed and predicted sea level rise eventuates.

***Table 2**: Comparison of 2120 MHWS tide levels to floor levels in low lying areas.*

Site	MHWS 2120	Floor Level (in low lying areas)
Tairua	2.095	1.1 – 1.9
Whitianga	2.207	1.2 – 2.2
Colville	2.617	1.7 – 2.0
Te Puru	2.853	1.4 – 1.9
Tararu	2.876	1.5 – 2.9
Moanataiari	2.875	1.4
Thames	2.875	1.6 – 2.8
Kopu	2.884	1.2 – 2.5

Note: All levels in m NZVD2016

To size pumps, it must first be determined what the tolerable level of inundation behind the defences is. For this assessment, it was assumed that no residential or commercial property in the low-lying areas is to experience above floor level flooding during events up to and including a 1% AEP event. Pump size is therefore determined by the peak flows from the catchments and water levels within the corresponding low points. It should be noted that the sizing of pumps has not accounted for future changes to rainfall intensity and durations as result of climate change. However, a +10% allowance has been made in the pricing of the pumps to account for possible increases in the required pump rates.

It is assumed that these pumps stations typically will be located immediately landward of the coastal defences and at the lowest point along the alignment. For them to be effective, rerouting of the existing stormwater network and construction of significant sump(s) or detention basin(s) would be required.

5 Possible Options Considered - Site by Site

5.1 Tairua

5.1.1 Overview

A possible protection option for Tairua is an earth embankment. The potential extent of such an embankment is presented in **Figure 11**, totalling 3,072 m. The bulk of the defences, some 2,463 m, would face foreshores with potential erosion risks, as shown in blue. The remaining 609 m of embankment, as shown in purple, would be in fields with a low risk of erosion, and therefore the scour protection could be reduced.

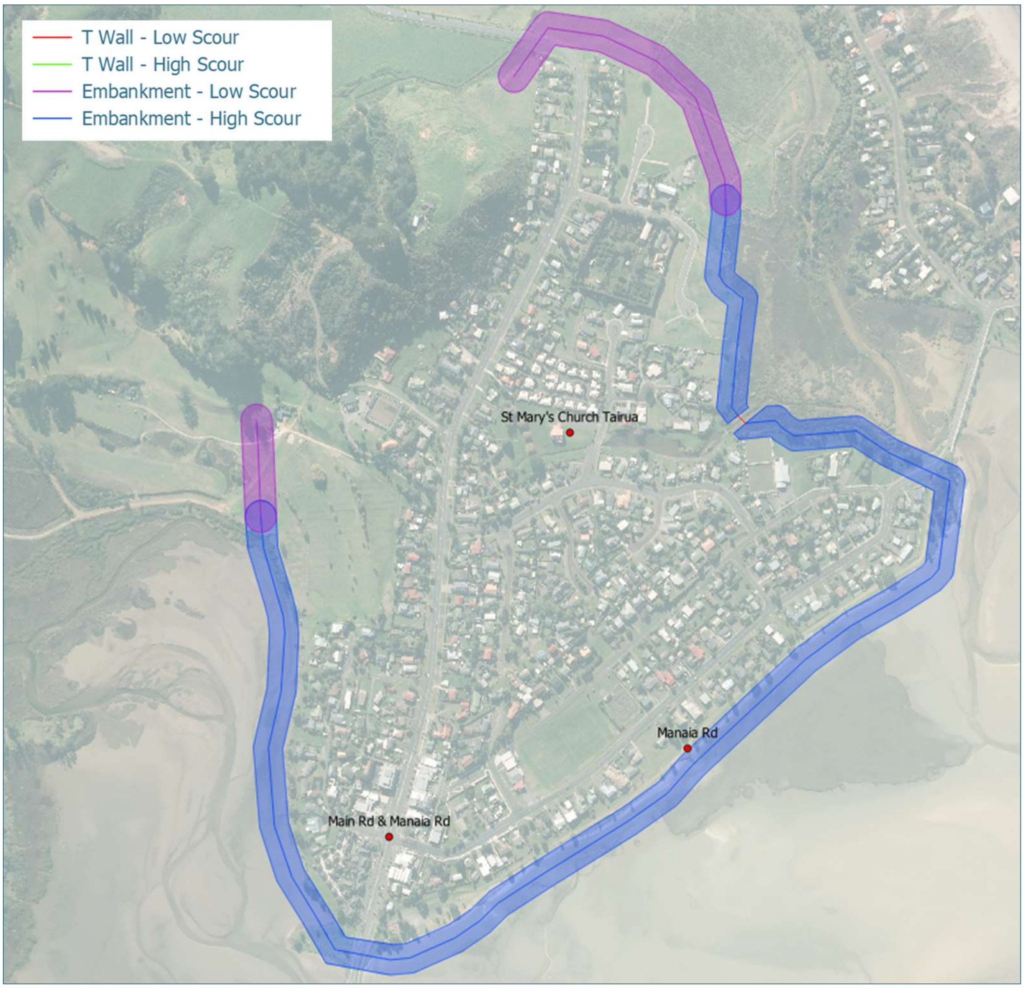

Figure 11: *Possible extent of the Tairua coastal defences*

Such defences in Tairua would not be exposed to significant ocean waves, as such the possible crest level has been defined as the critical water level plus a 0.5 m freeboard; that is, 4.7m NZVD2016. This level relative to key landside elevations and oceanic levels is presented in **Figure 12**, which provides a schematic illustration of the form the defence could take. Note that the potential moderating impact of the estuary, protected by high coastal dunes has not been considered in this study. This may impact design levels should a design be progressed in the future.

An intertidal channel (estuary) exists east of Saint Mary's Church and the crossing of this channel by the proposed defences would need to be addressed in any design. To preserve the environmental values of this estuary a penstock or tidal gate that could be closed during extreme events would be required. This type of system would require ongoing maintenance and operation. An alternative to a penstock / tidal gate could be the implementation of a one-way flow valve, but this would effectively prevent tidal ingress, changing the estuary into a fluvial channel/drain.

Several major roads, including State Highway 25 and two bridge abutments, are crossed by the proposed alignment of the defences. These locations would require additional works to create suitable approaches and manage services.

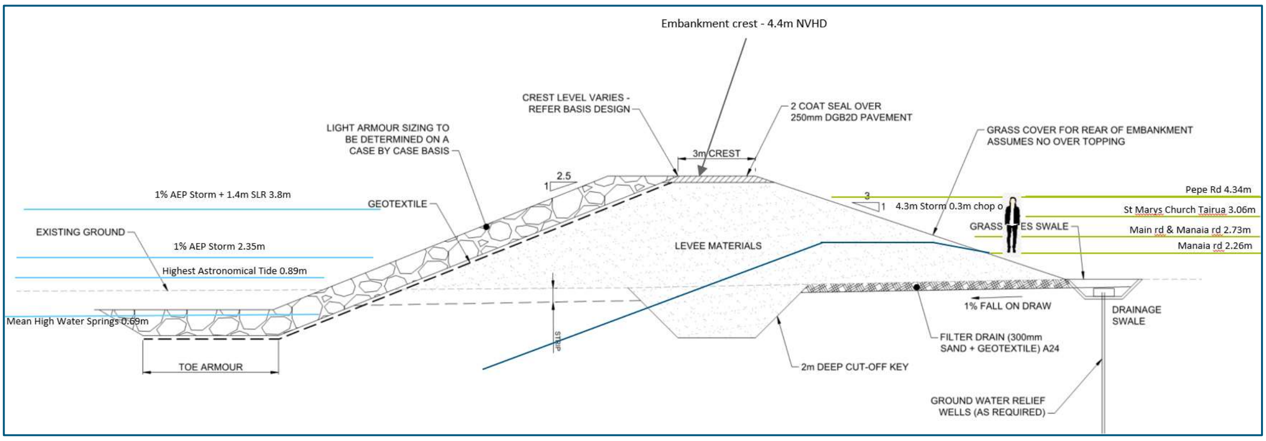

Figure 12: *Tairua possible defence crest levels compared to oceanic and landside elevations*

5.1.2 Feasibility

Tairua does not experience direct exposure to the ocean, thus the flooding threats are associated with storm tides. There is sufficient space to allow earth embankments to be built, which offer a more robust, amenable, and affordable solution than concrete walls. The possible works are generally constructable with limited additional complexities.

The height of the defences at 4.7 m NZVD2016 is typically 2 to 2.5 m above backing land levels. The embankment would block some vistas, particularly for residents along the comparatively low-lying foreshore. Staging any works would be a way to develop a solution that is more affordable and commensurate with current threat levels.

The nature of the threat means that catastrophic overtopping when static water levels breach the crest of the defences (during events that exceed the 1% AEP event plus a free board threshold) represents an additional hazard. If coast protection is progressed in this location, the management of this hazard needs to be considered and addressed. It is recommended that more detailed studies for a suite of events and scenarios are undertaken as part of any design investigations to assess the risks faced by the community from a catastrophic inundation event and how the defences may impact the community's readiness and willingness to respond to evacuation warnings. This is also the case for all of assessments that follow.

5.2 Whitianga

5.2.1 Overview

The selected possible protection option for Whitianga is rock-protected earth embankments (seawalls), where space permits, and concrete walls where there are limitations on the achievable footprint. The possible extent of the different defence options is presented in **Figure 13**, where there is approximately 4,650 m of embankment seawall (shown in blue and purple) and 3,838 m of concrete wall (shown in green and red). The bulk of the defences would face foreshores where scour and erosion are significant risks, however, the nature of the erosion threat differs significantly between the ocean facing eastern foreshores and the estuary facing southern and western foreshores. There is a total length of 3,022 m of foreshore that faces the ocean and 3,536 m of foreshore that faces the estuary. For each of these lengths, high scour defences would be required. The remaining 1,930 m of defensive structures could have reduced scour protection measures, due to these sections not being exposed to significant wave action or flood flows.

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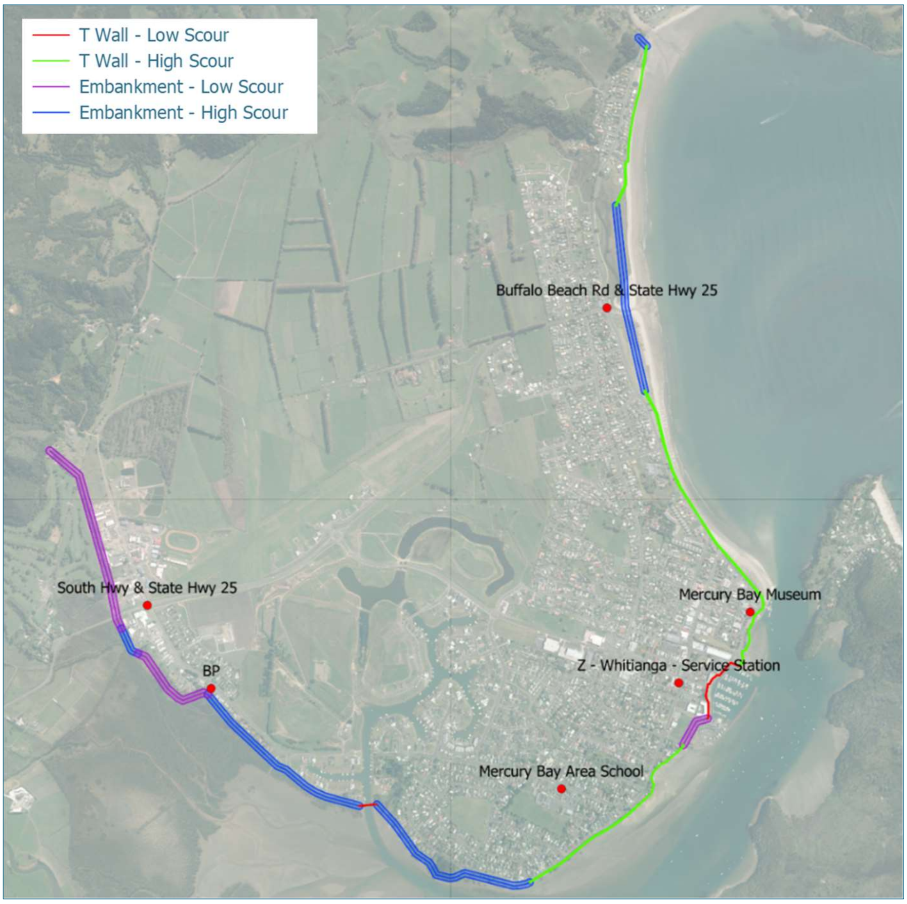

Figure 13: *Possible extent of the Whitianga coastal defences*

Whitianga is exposed to both a severe Tsunami hazard and heavily attenuated ocean swell on its eastern foreshore. Defences exposed to the ocean would require a crest level of 6.4 m NZVD2016, based on a 5 l/m/s limit on overtopping for the design storm event and an anticipated 1.4 m of sea level rise. These levels relative to key landside elevations and oceanic levels are presented in **Figure 14**, which provides a schematic illustration of the form the defence could take.

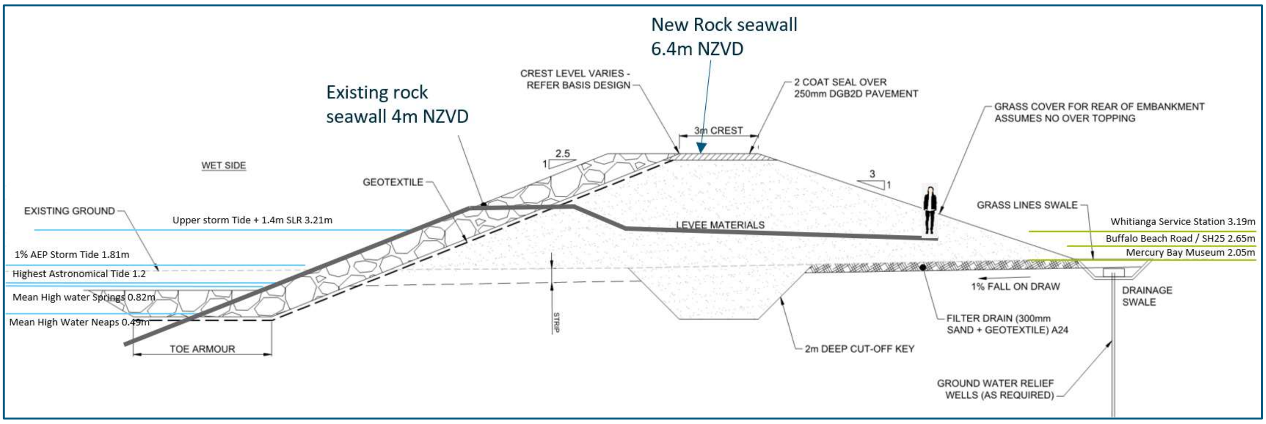

Figure 14: *Whitianga possible coastal defence crest levels compared to oceanic and landside elevations*

Unfortunately, due to the highly developed nature of the Whitianga foreshore the construction of a coastal defence would be highly constrained. Some examples of such constraints are presented below in **Figure 15**, **Figure 16** and **Figure 17**. Further, the proposed defences cross State Highway 25 at both ends of the site and it is likely that additional engineering and construction effort would be required to facilitate the approaches and adjust the bridges accordingly. The cost of these works has been excluded from this assessment. Additionally, development has extended in many areas right up to the foreshore, as a result there is very little space for the construction of sea defences, adding complexity and cost; however, utilisation of materials present in the existing defences may potentially offer some saving provided this rock is suitably sized.

Figure 15: *Ocean facing armoured foreshore (Mercury Bay)*

Figure 16: *Ocean facing foreshore (Mercury Bay), looking south*

Figure 17: *Estuary facing foreshore, note space constraints and power lines*

To add further complexity to options for defending Whitianga, the waterways would require either a lock (refer to **Figure 18**) or a barrage gate (a single gate) to seal the canal system and protect the estate and the surrounding low-lying areas during extreme events. The advantage of a lock is that it permits control of water levels, allowing for simple foreshore details and maximum land use. The advantage of a single barrage is that it is cheaper to construct, and tidal ingress can be maintained with related environmental and water quality advantages.

Figure 18: *Example of a canal estate lock*

In addition, a small estuary is located to the east of the waterways entrance. If the entrance to waterways is managed with a lock, the estuary would cease to be tidal (with implications for the upstream catchment and susceptibility to flooding).

5.2.2 Feasibility

The Whitianga foreshore experiences a mix of ocean exposure (Mercury Bay) with heavily attenuated swell and estuarine exposure. Located on the east coast of the peninsula tsunami hazards are significant. Much of the foreshore at present is armoured against erosion threats, but flooding threats are not mitigated. Further, along much of the foreshore community assets are hard up against or very close to the existing defences (rock armour). This increases the complexity of any possible defences and would require that they are pushed seawards. The works are feasible, but the engineering would be difficult due to significant work needing to occur either in tidal areas or below water. The need to work around existing services and infrastructure would also increase the complexity of delivery. These issues would be reflected in increased costs.

Further to the constraints around constructability, defences constructed seaward of existing defences would significantly impact beach amenity.

The design crest levels of 6.2 to 6.4 m NZVD2016, are approximately 3 to 4 m higher than the existing foreshore and would have a major impact on the amenity of this coastal community. This issue would be exacerbated by the fact that most community assets are located above normal tidal ranges, even with 1.4 m sea level rise. A staged approach, with modest levels of defence initially may provide a pathway to delivering defences for this community.

Based on the forecast tsunami hazard for this community, there is an existing risk of catastrophic flooding. This risk is mitigated by early warning systems for evacuation and a hazard aware population. If coastal defences were built, the residual risk associated flooding by overtopping or failure of defences during a tsunami or extreme storm event remains (although the community's perception of the hazard and the need to act on warnings may change.

The overall viability of constructing coastal defences around Whitianga is not clear cut. Despite the significant value of the assets that would be defended, the impact on community amenity and the residual risk of catastrophic failure are significant issues. However, if a staged approach is adopted, such a project could be feasible.

5.3 Colville

5.3.1 Overview

The selected possible protection option for Colville is primarily embankments, although a concrete T-Wall with a protected toe is proposed adjacent to the stream on the eastern side of the township, due to space limitations. The extent of the different defence options is presented in **Figure 19**, where there is approximately 1,352 m of earth embankment (shown in blue and purple) and 181 m of the concrete T-Wall (shown in green). The erosion threatened sections are on the north-western side of the school, facing the sea, and the concrete wall adjacent to the stream.

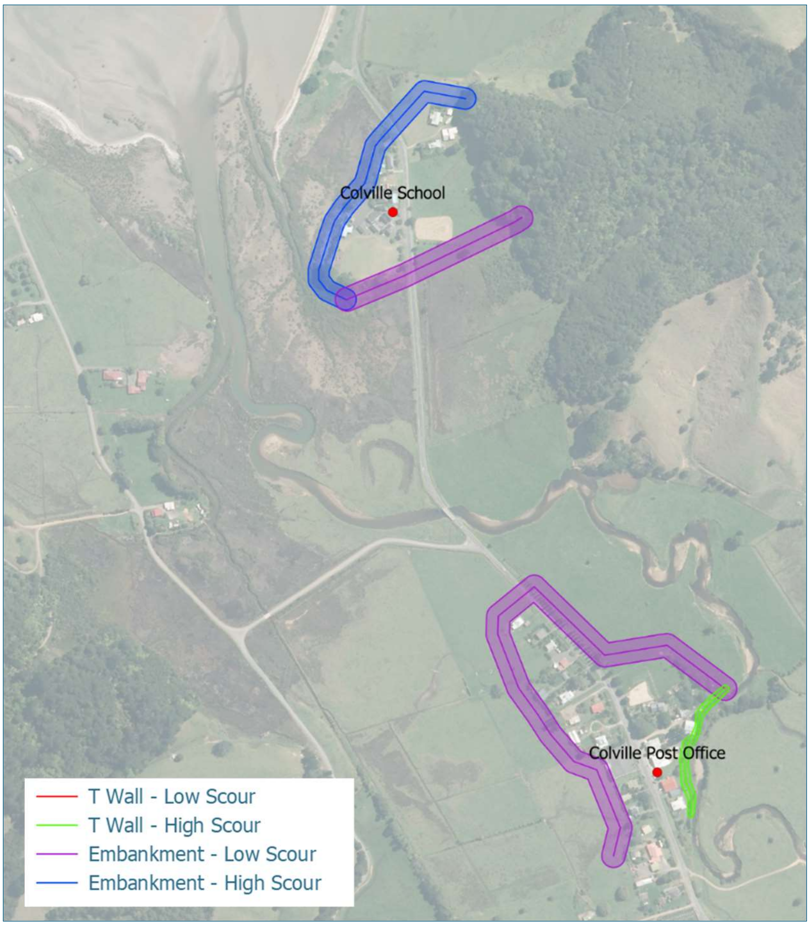

Figure 19: *Possible extent of the Colville coastal defences*

Despite fronting the bay, the crest level of the defences at the Colville School site are not defined by the waves reaching the foreshore, as they are likely to be so attenuated that wave runup is insignificant, thus overtopping does not define the crest level. Rather the crest levels of the defences are defined by the tsunami level, with 0.5 m freeboard. Any defences, therefore, would require a crest level of 4.6 m

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NZVD2016. A depiction of this relative to key landside elevations and oceanic levels is presented in **Figure 20**, which provides a schematic illustration of the form the defence could take.

One of the main constraints to the construction of these defences is the fact that the proposed earth embankments cross Colville Road in three locations. Construction of approaches and adjustments to the road would add significantly to the engineering effort required for these relatively short lengths of bund, a cost which has been excluded from this assessment. In addition, the management of stream flooding would be a key consideration and this was raised by the community (via consultation events) as a major constraint.

5.3.2 Feasibility

The scale of the possible option for Colville is modest in comparison with other communities, in line with the modest size of this community. With rural land surrounding the community, constructing the defences is feasible. There are space constraints associated with the close proximity of the stream but access to construct a concrete T-Wall is considered to be adequate.

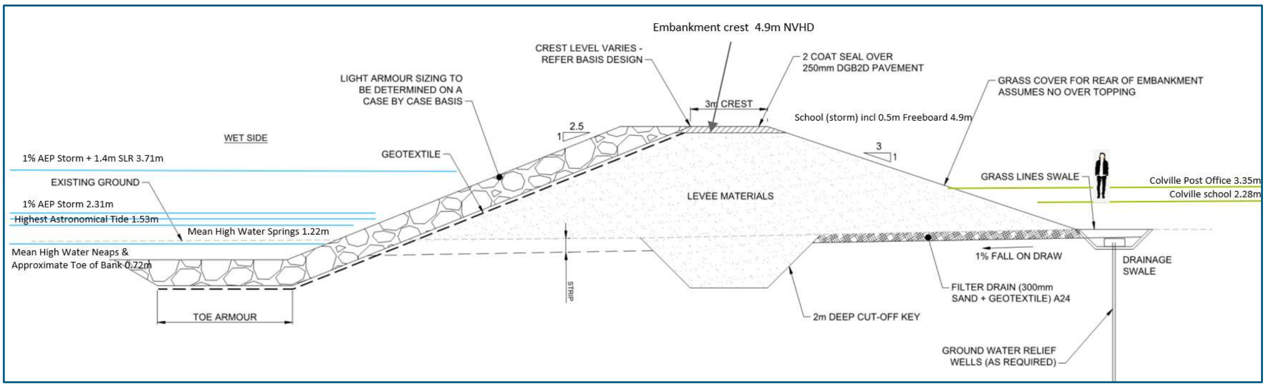

__Figure 20:__ Colville possible defence crest levels compared to oceanic and landside elevations

The threat to Colville from the sea, though serious, impacts relatively few assets. For this community a cost of around $56M for defence (see Section 6) would be difficult to justify. Based on a simple cost comparison assessment, relocation of the community would be significantly less expensive.

Furthermore, the hazard represented by the coastal inundation mapping is considered to underestimate the risk, due to the frequent influence of rainfall (fluvial flooding) in conjunction with coastal flooding in this location.

The construction of defences in this location would also introduce a residual risk associated with overtopping or total failure during an event greater than the event (1% AEP) considered as part of this assessment. If a defence option is to be progressed for Colville, a detailed study should be undertaken for a suite of events and scenarios, that includes initial geotechnical investigations, full hydrodynamic modelling, and joint probability analysis of coincident coastal and fluvial flooding events.

5.4 Te Puru

5.4.1 Overview

The selected possible protection option for Te Puru is a mixture of embankment seawalls and concrete walls in constrained areas. The extent of the different defence solutions is presented in **Figure 21**, with approximately 1,043 m of embankment (shown in blue) and 1,496 m of concrete wall (shown in green and red). The erosion threat in this location is assessed as high. For the stream, the existing bank protection is considered adequate. The stream scour risk would need to be assessed in the design process should a defence option be progressed, however, due to space limitations, it is likely that any scour protection measures along the stream would need to be limited to avoid excessive constriction of flows.

***Figure 21** Possible extent of the Te Puru coastal defences*

The majority of the possible defences in Te Puru would be exposed to the ocean and wave action. As such, crest elevations along the foreshore are defined by wave runup during the design storm event and an acceptable overtopping rate of less than 5 l/m/s. The crest level of any defences required here would need to be 5.1 m NZVD2016. In areas not exposed to wave action, such as within the stream, the crest level is defined by the storm tide plus a 0.5 m free board, resulting in a level of 4.4 m NZVD2016. A depiction of these levels relative to key landside elevations and oceanic levels is presented in **Figure 22**, which provides a schematic illustration of the form the defence could take.

The Te Puru community is situated on a sediment fan (fan delta), backed by rugged hills, which would allow any defences to tie back into high ground. The foreshore is flat and wide, though space constraints exist in some locations, as seen in **Figure 23** and **Figure 24**. Two road crossings exist at the extreme ends of the community and any works around the stream (see **Figure 25**) and boat ramp would require additional construction and engineering effort. In addition, construction of coastal protection, if avoiding private property, may encroach into existing beach amenity areas.

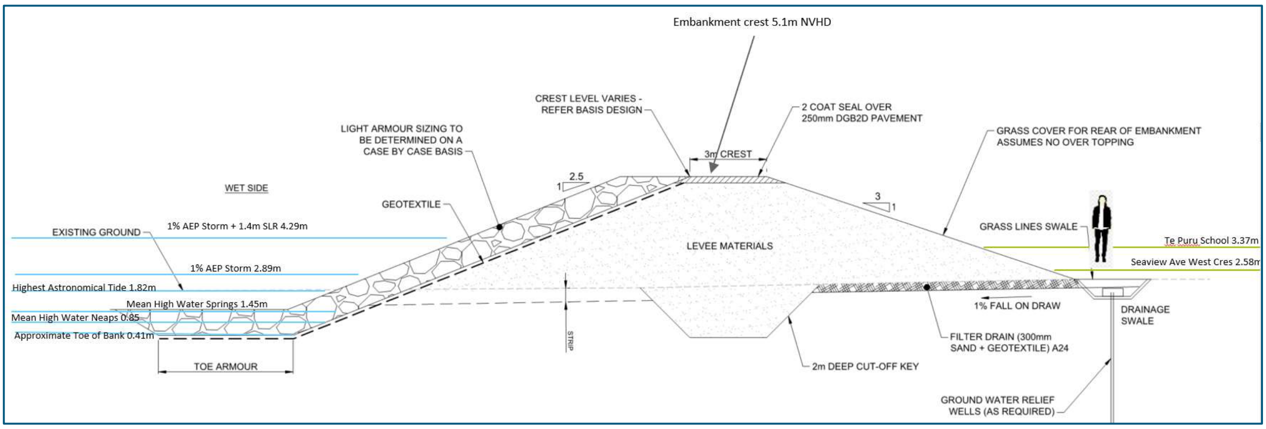

Figure 22 *Te Puru possible defence crest levels compared to oceanic and landside elevations*

Figure 23: *Te Puru foreshore with a low natural berm behind the beach*

Figure 24: *Southern foreshore, noting space constraints*

Figure 25: Te Puru Creek looking downstream from Pacific Coast Highway

5.4.2 Feasibility

As above, Te Puru community is located on a fan delta that has developed around the local creek. The foreshore is sandy with a natural berm formed under extreme marine conditions (storm tides). House occupy the limited flat land available and the adjacent land is steep. Constructing a defensive barrier approximately 2 m higher than the existing berm level along this foreshore (5.1 m NZVD2016) would be viable, with good access along the proposed route. However, this would affect the amenity of residents. These concerns could be mitigated by staging the works to be more in line with the level of threat faced by the community.

Because the extent of the threat to Te Puru in 2120 is significant. The cost of defending (at around $115M, see Section 6) would be large compared to the scale of the community.

If defence is pursued, the residual risk associated with overtopping or total failure during an event greater than the design event, must also be considered.

5.5 Tararu

5.5.1 Overview

The flooding threat and geological setting in Tararu are very similar to Te Puru, as such the possible protection option is similar. The possible embankments and concrete walls in constrained areas are presented in **Figure 26**. There is approximately 1,219 m of embankment (shown in blue) and 935 m of concrete wall (shown in green and red). For the exposed foreshores, the erosion threat is assessed as high for the entire length of the possible defences. For the creek frontage the erosion threat is also high, but it is assumed existing stream bank protection is adequate. If erosion protection is determined to be required, then the scale of works would need to be limited to avoid constriction of flows.

The majority of any defences within Tararu would be exposed to the ocean and wave action. As such, crest elevations along the foreshore are defined by wave runup during the design storm event and an acceptable overtopping rate of less than 5 l/m/s; i.e., 5.1 m NZVD2016. In areas not exposed to wave action, such as within the creek, the required crest level would be 4.4 m NZVD2016. A depiction of these levels relative to key landside elevations and oceanic levels is presented in **Figure 27**, which provides a schematic illustration of the form the defence could take.

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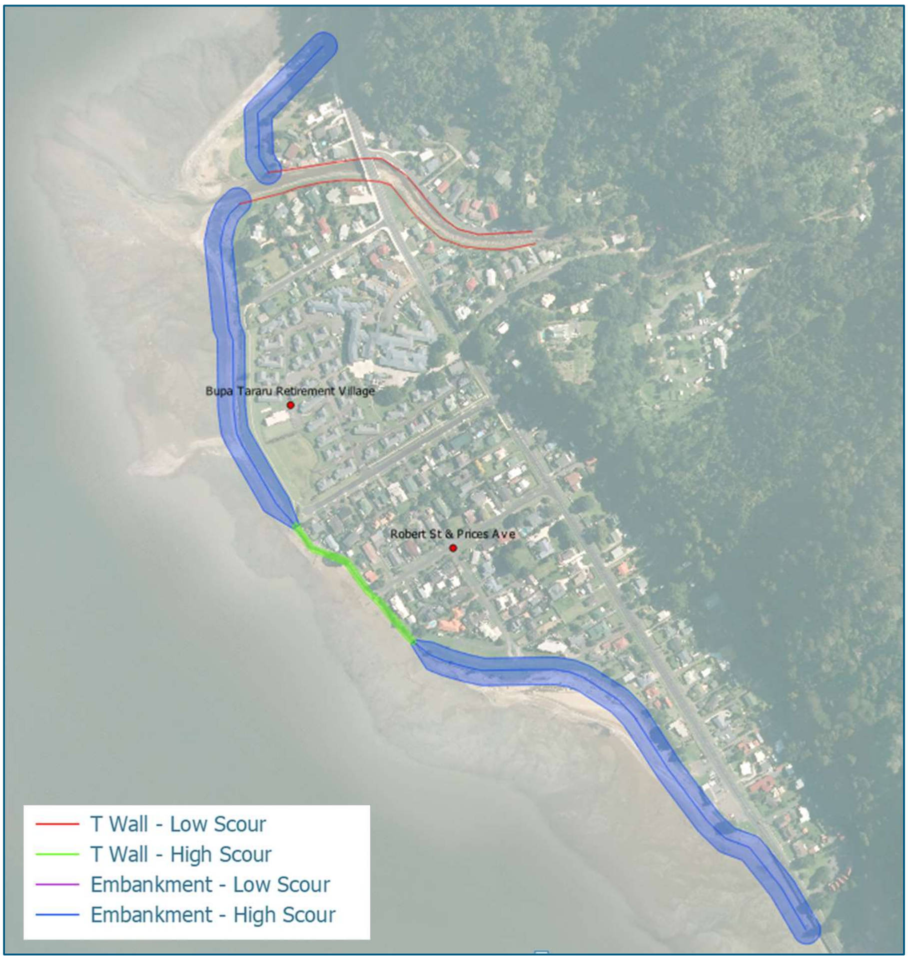

Figure 26: *Possible extent of Tararu coastal defences*

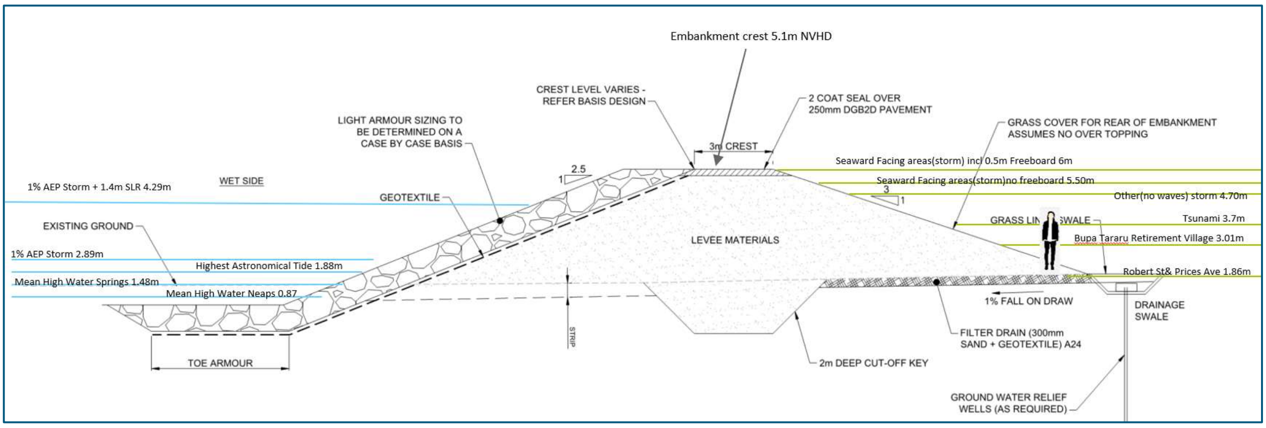

Figure 27: *Tararu possible defence crest levels compared to oceanic and landside elevations*

Like Te Puru, Tararu is also situated on a sediment fan that has developed around the local stream, backed by rugged hills. This would allow any defences to tie back into high ground. The foreshore is flat and wide, though an approximately 200 m long section at the end of Robert Street has some space constraints. There are some existing low-level defences that may provide some cost savings, as seen in **Figure 28**; however, there are four locations where any defences would need to cross Tararu Road. Integration of the defences in such a way that the serviceability of the road is not impacted would require significant engineering input and adds complexity to this project, due to the highly constrained site as seen in **Figure 29** and **Figure 30**.

Figure 28: Low berm defences at the Bupa Tararu Care Home

Figure 29: *Tararu Creek looking upstream, note various edge treatments and proximity to the road*

Figure 30: Tararu Creek looking downstream, note the proximity of dwellings to the top of bank

5.5.2 Feasibility

Tararu benefits from a wide sandy foreshore that, in general, would provide space for the construction of defences. However, this community is more intensely developed and has limited low level flood defences already in places. Overall, the feasibility of constructing defences in this location is reasonable, with good access, though works around the creek would be impacted by the limited space. The construction of defences would also impact beach amenity in the future.

As for other foreshore-focused communities, the construction of defences with a crest level significantly higher than the existing levels could cause concern. The possible defences, in due course (for 1.4 m of sea level rise), would require a height of 5.0 m NZVD2016, approximately 2 to 2.5 m above existing levels.

The extent of the threat faced in Tararu is significant, with almost the entire community vulnerable. The cost of any defences, at around $91M (**Section 6**), would be high for the number/value of the affected assets. The cost benefit of defending the community would need to be tested.

If defence is pursued as an option, the residual risk associated with overtopping or total failure during an event greater than the design event considered as part of this assessment, must also be considered.

5.6 Moanataiari

5.6.1 Overview

Moanataiari is currently defended by an armoured earth embankment (3.1 m NZVD2016) with a timber wall situated on its crest (at 4.0 m NZVD2016). In its current state, the level of service provided by the embankment and wall is to defend up to the present day 1% AEP coastal storm, although some overtopping is experienced; refer to **Figure 6** and **Figure 7**. To defend Moanataiari in the future it is considered that approximately 974 m of earth embankment would require construction and/or augmentation, as presented in **Figure 31**. A concrete T-Wall would also be required adjacent to Moanataiari Creek although, for the purposes of this investigation, this section of the wall is costed as part of the works to defend Thames (refer to **Section 5.7**).

The erosion threat is assessed as high for the entire length of the possible raised defences. With increased water levels it is anticipated that the current foreshore protection will be inadequate and that it would either need to be replaced or significantly supplemented if a defence option is to be pursued.

Within the stream/drain, it is assumed that the existing bank protection measures are adequate. If during any design phase it is assessed that bank protection in the stream/drain is required, then the nature of the works would need to be constrained to avoid excessive constriction of flows.
At the southern end of the Firth of Thames the storm tides and wave climate are more severe than further to the north and, as a result, the required crest levels for any defences would be slightly greater. As elsewhere, along the foreshore, crest elevations are defined by wave runup during the design storm event and an acceptable overtopping rate of less than 5 l/m/s. That is, a crest level of 5.2 m NZVD2016 would be required. For defences adjacent to Moanataiari Creek, however, a lower crest level of 4.7 m NZVD2016 would be required, as this area is sheltered from wave action. A depiction of these levels relative to key landside elevations and oceanic levels is presented in **Figure 32**, which provides a schematic illustration of the form the defence could take.

Drainage is a significant issue for this site, as demonstrated by the ponded water behind the embankment in **Figure 6**. With minimal storage capacity available, the flooding risk for low lying housing is significant and already a point of concern given that (even in the current climate) the catchment requires pumping, see **Figure 33**. These pumps would require upgrading as part of any augmentation works.

Other issues for Moanataiari include groundwater (and rising groundwater), concerns relating to contamination of the groundwater and subsidence.

5.6.2 Feasibility

Unlike most of the Thames township, this community already has significant coastal defences in place. These barriers (and pump station) are required due to ground levels in the foreshore areas being very low (below high tide levels) and vulnerable to severe flooding.

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Figure 31: Possible extent of new Moanataiari coastal defences

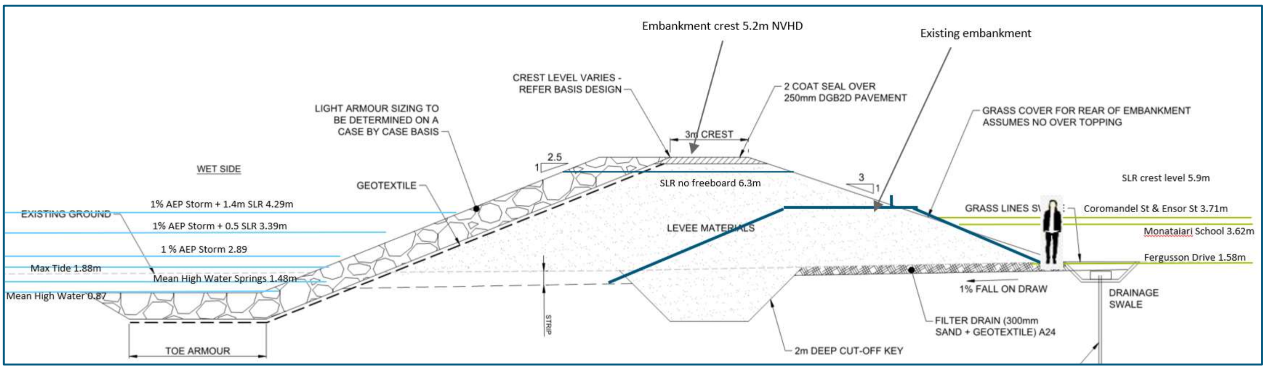

Figure 32 Moanataiari possible defence crest levels compared to oceanic and landside elevations

Figure 33 Pump station on Fergusson Drive behind embankment

To convert the existing defences into a suitable structure in 2120 the crest of the defences would need to be raised from 4.0 m (top of timber wall) or 3.1 m (top of embankment) to 5.9 m NZVD2016. The engineering logistics of increasing the height of the defences by 1.9 m would feasible, with the existing access corridors facilitating construction. Although the community is used to having a significant barrier on the foreshore, raising this barrier by such a large amount may be difficult for the community to accept without a clear and present threat. Should defence be progressed as an option for the future, it is likely that a staged raising program, with levels raised incrementally as threats increase, would be more acceptable.

The very low ground levels behind the existing defences make planning for future conditions more complicated. These defences are at risk of catastrophic overtopping during significant events. During an event where static water breached the crest, flood levels would rapidly increase (by several meters over a short period) and pose serious risks to assets and lives. This threat would increase as sea levels rise and defences are raised and needs to be considered in determining the appropriate response. If a defence option is progressed, more detailed studies for a suite of events and scenarios would need to be undertaken, including initial geotechnical investigations, full hydrodynamic modelling of flooding events, combined modelling of flooding, subsidence and groundwater intrusion, and an assessment of emergency evacuation routes to high ground at short notice.

Further, the cost benefit assessment for Moanataiari needs to take account of the age/value of the assets and the combined influence of coastal, fluvial and surface water flooding (due to groundwater intrusion).

5.7 Thames

5.7.1 Overview

The possible defences for Thames include a combination of earth embankments, where space permits, and concrete T-Walls, where there are limitations to the footprint. The extent of the possible different defence solutions is presented in **Figure 34**, with approximately 2,962 m of embankment (shown in blue and purple) and 4,200 m of concrete wall (shown in green and red). Some sections of the foreshore already have "defences" in place, such as the existing low armoured bund that protects areas with ground levels below HAT at the end of Albert Street (refer to **Figure 35**). The areas that face the open sea, with a westerly aspect, have been assessed as being at significant risk of erosion and scour due to wave attack and will need additional protection. However, the channels and sheltered banks of the Kauaeranga River are deemed to have lower erosion and scour risks and, subsequently, less robust measures would be required.

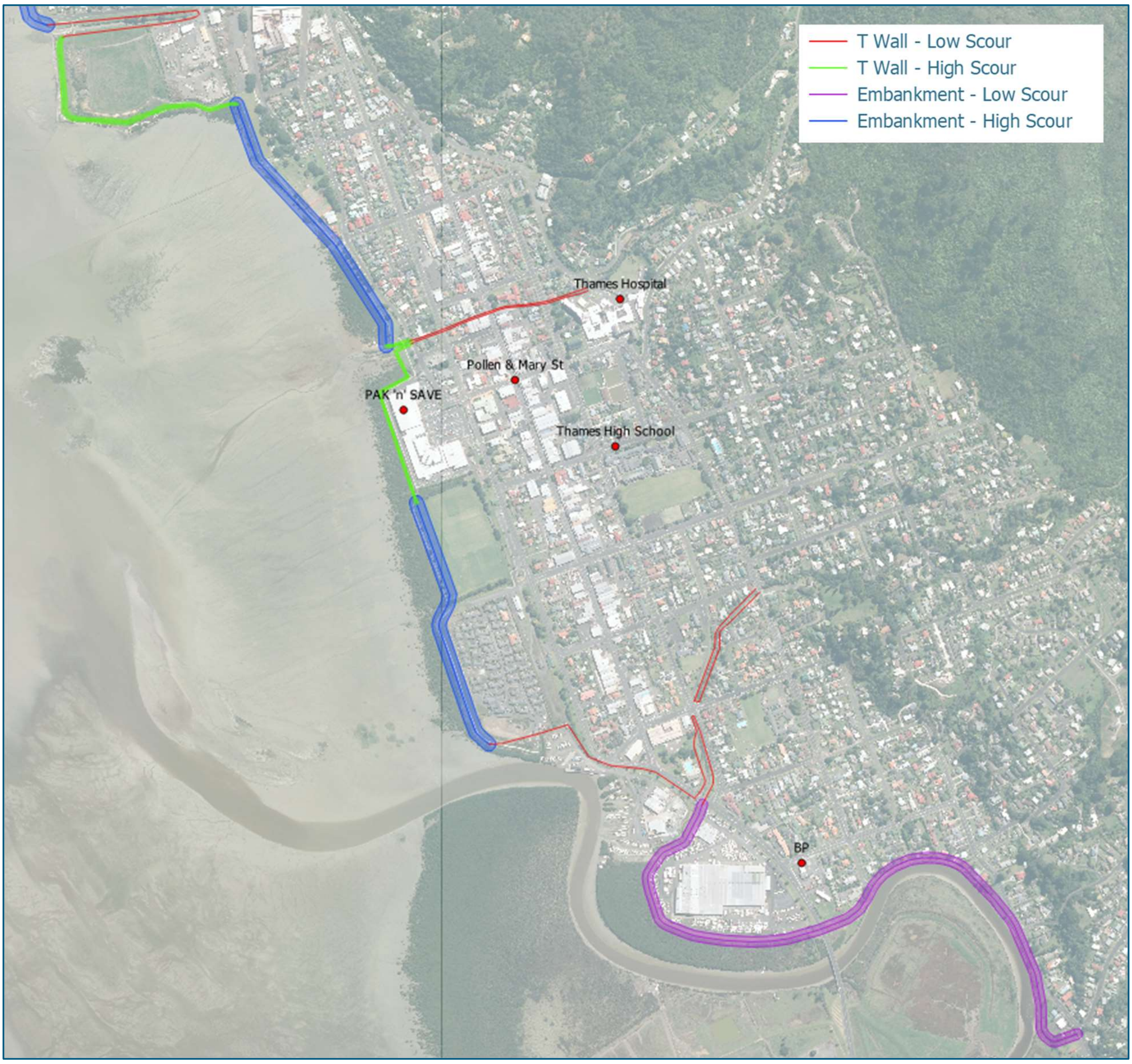

***Figure 34:** Possible extent of coastal defences for Thames*

As for Moanataiari, the outputs from the SMP inundation modelling indicates that storm tides and exposure to waves is more severe at the southern end of the Firth of Thames than to the north, hence a higher crest level would be required for any defences in Thames. Along the foreshore, a crest level of 5.2 m NZVD2016 would be required. For areas sheltered from waves, e.g., the creeks through Thames and the Kauaeranga River, the required crest level is predicted to be 4.7 m NZVD2016. A depiction of these levels relative to key landside elevations and oceanic levels is presented in **Figure 36**, which provides a schematic illustration of the form the defence could take.

Figure 35: Low armoured bund protecting northern foreshore (at the end of Albert Street)

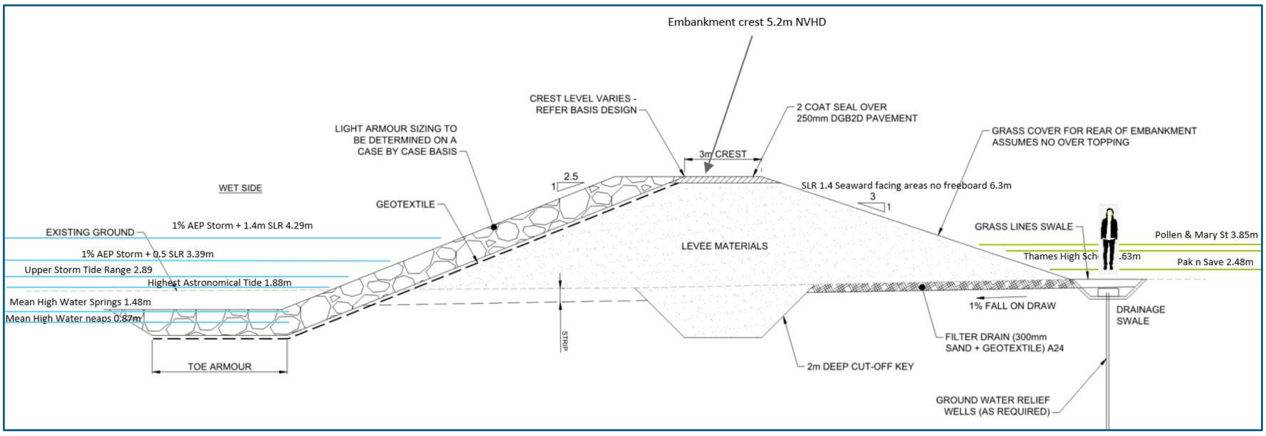

Figure 36: Thames possible coastal defence crest levels compared to oceanic and landside elevations

In several locations there would be significant difficulties in constructing any possible defences, due to space limitations because of encroached development. This is a major constraint and is particularly evident along the lined channels through Thames where a number of roads, road crossings and dwellings have been constructed within 1-2 m of the top of bank; refer to **Figure 37**, **Figure 38** and **Figure 39**. Given the inherent difficulties in retrospectively constructing these defences, investigation into alternative solutions is likely to be required. One such solution may be the installation of one-way flow valves or tide gates at or close to the foreshore, although this could have significant ecological impacts.

5.7.2 Feasibility

From this preliminary assessment, although considered possible, it would be very challenging from both an engineering and planning perspective to protect the Thames township against coastal inundation for a 1% AEP storm over the next 100 years. The estimated costs are not insignificant and need to be refined and detailed prior to any final decision making. In addition, the cost of protection needs to be compared to other options[4].

The main challenge associated with protecting Thames is the required scale of any defensive structures, as many locations are low lying, with elevations typically around 2 – 4 m NZVD2016. The required crest level of any defensive is estimated at being 5.9 m NZVD2016, that is, typically 2 – 4 m high and up to 4.5 m above existing ground elevations. This would affect the outlook and amenity of residents.

[4] As part of a separate study, a Real Options Analysis, which considers the cost benefit of defence against retreat, has been undertaken for Thames and shows that value of the assets in Thames is significantly greater than the cost of defending.

Figure 37: *Head of exposed drain - limited space and erosion prone with the culverts and drains requiring one-way flow valves*

Figure 38: *An example of the highly constrained site adjacent to a lined channel*

Figure 39: *Intersection of MacKay and Grey Streets with the crossroads bridging the channel (bottom left to top right)*

Another significant issue is the lack of space for any works. Thames has significant established development and minimal existing defences. As a result, along much of the possible line for the works there is no readily available corridor for construction without significantly impacting on existing assets or natural systems. This has implications for both costs and community acceptance.

The cost of the possible solution is predicted to be in the order of $283M (**Section 6**), which can be justified based on the number and value of threatened assets. This cost could be managed with a staged approach to construction.

As elsewhere, if a defence option is pursued, it is strongly recommended that a detailed study is undertaken that considers a suite of events and scenarios and includes initial geotechnical investigations, full hydrodynamic modelling, and joint probability analysis of coincident coastal and fluvial flooding events.

5.8 Kopu

5.8.1 Overview

The Kopu area is currently defended from ocean and river inundation by a series of stopbanks. This flood mitigation infrastructure is operated by WRC. The options considered here for future coastal defence, for consistency with the other locations considered, have been assessed (in part) as distinct from the existing infrastructure. However, it is likely that the cost of future improvements to the level of service provided by the existing defences would be partly met by ongoing maintenance funds.

With relatively more space available around Kopu, all possible defences considered as part of this study are earth embankments, except for one location beneath the Waihao River Bridge where a concrete T-Wall would be likely to be required to tie into the bridge abutment. The extent of the defences considered are presented in **Figure 40**, where there is approximately 7,099 m of earth embankment (shown in blue and purple) and 76 m of concrete wall (shown in green). As Kopu is situated on the Waihao River, those lengths of embankment that run parallel to the river are considered to have a higher erosion and scour risk due to the likely severity of flood flows and potential propagation of waves up the river.

Although wave propagation up the river is a possibility, overtopping does not define the crest level, as the waves would most likely be sufficiently attenuated to make runup insignificant. Rather, the crest levels of the defences have been defined by the storm tide level with 0.5 m freeboard. For the defence of Kopu, the required 2120 crest level would be 4.7 m NZVD2016. A depiction if this relative to key landside elevations and oceanic levels is presented in **Figure 41**, which provides a schematic illustration of the form the defence could take.

Given the existing defences and easements in place, a significant cost saving would be afforded in the implementation of a future defence solution at Kopu, when compared to other locations on the Coromandel Peninsula. Examples of these existing defences are presented in **Figure 42**, **Figure 43** and **Figure 44**. As can be seen from the figures, the existing and proposed embankments are crossed by several roads and access points (e.g., the boat ramp). Construction of approaches, adjustments to the roads as needed and tying into the newly constructed Waihao River Bridge would add to the engineering effort required for these works.

5.8.2 Feasibility

Kopu has existing bunds that provide protection from flooding. The general alignment of the works considered here for possible future defence largely follows the alignment of existing works. There is typically good access and generally sufficient space for any works. As such, engineering future bunds would be feasible within the existing defence corridors and on rural land.

The existing and future defences are extensive nature and reflect the extent of high value assets being defended. The predicted cost of the works, at around $109M, is significant but is justifiable for the urban community of Kopu. The value of defending the extensive areas of rural land in this compartment could be considered further, with the potential for large savings by reducing the extent of any works to focus on urban assets only.

As for elsewhere, it is recommended that more detailed study of a suite of events and scenarios is undertaken, including initial geotechnical investigations, full hydrodynamic modelling, and joint probability analysis of coincident coastal and fluvial flooding events.

Figure 40: *Possible extent of Kopu coastal defences*

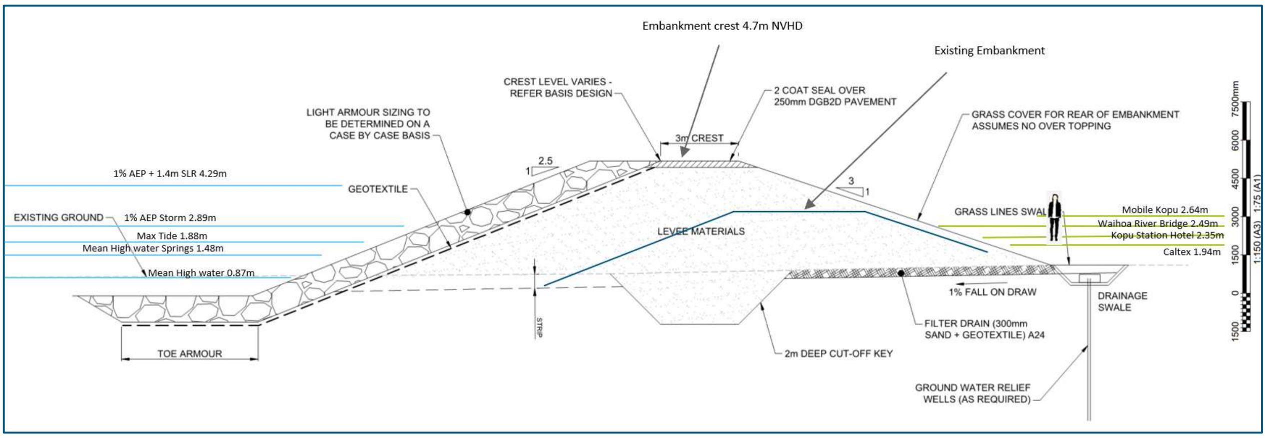

Figure 41: *Possible future coastal defence crest level for Kopu compared to oceanic and landside elevations*

Figure 42: *Existing bund protecting Kopu facing the Waihou River (looking south from boat ramp towards Highway 25 bridges)*

Figure 43: *Existing bunds either side of the drain with foot bridge crossing (looking east from highway)*

Figure 44: Existing bund facing Kauaeranga River (looking north from Maramarahi Road)

6 Preliminary Cost Estimates

Preliminary cost estimates for the possible defence solutions considered for each location are provided in **Table 3**. These costs estimates are high level, indicative and are based on providing protection from a 1% AEP coastal inundation event in 2120, based on 1.4m of sea level rise (i.e., a 100-year event in 100 years' time). Benefits have not been considered in any detail at this stage (e.g., the value of what is being protected based on damage-cost curves). However, an indication of the assets at risk that would be protected (and, where data is available, their present-day or estimated 'value') is provided.

The estimates presented primarily comprise costing of the major components of each possible defensive strategy. For other elements likely to be necessary, limited general allowances have been made. Construction of specific high value elements linked to the performance of the defence, such as pumps, penstocks, locks, and one-way flow valves are included. Other associated infrastructure such as road/bridge adjustments or service relocation are not included.

Table 3: Cost estimates for the possible coastal defences

Location	Defence Length (m)	Pump Stations	Cost ($M, NZD)	Benefits (estimates of assets at risk in 2120 from a 1% AEP event, assuming 1.2m of SLR)
Tairua	3,072	2	87	230 dwellings and 36 other buildings
Whitianga	8,488	6	342	1400 dwellings and 350 other buildings
Colville	1,533	2	52	83 dwellings
Te Puru	2,539	2	115	275 dwellings and 114 other buildings
Tararu	2,154	3	91	261 dwellings and 23 other buildings
Moanataiari	974	1	25	354 dwellings and 80 other buildings
Thames	7,098	4	283	996 dwellings and 374 other buildings (where the 'value' at risk, as estimated for the Thames Spatial Plan project, is $974M)
Kopu	7,943	3	109	210 dwellings and 382 other buildings (inc. the industrial precinct / business park)

As these estimates are preliminary only, no allowance has been made for:

- Staging the works – costs could be spread across a long timeframe.
- Fees and charges.

- Property acquisitions.
- Cost escalation beyond November 2021.
- Additional stormwater infrastructure improvements.
- Dewatering of excavations and groundwater management.
- Temporary barriers / structures for construction.
- Stakeholder negotiation.
- Tendering costs.
- Testing, treatment and management of contaminated material / buried rubbish.
- Adjustments to existing utilities.
- Major construction-related infrastructure, such as haul roads, piling pads etc.
- Unknown ground conditions and geotechnical design / improvement works.

If the defence option is to be progressed in any of the locations considered, further individual possible implementation strategies will need to be developed and incorporate an assessment of cost-effectiveness or benefit-cost analysis. A cost-effectiveness analysis considers a range of options for achieving a particular outcome and shows which is the most cost-effective.

The coastal defence implementation strategy for those locations where defence forms part of the preferred adaptation pathway will need to consider in detail how this could best be achieved, in terms of the approach to the engineering, the level of service to be provided (i.e., what degree of overtopping might be acceptable) and the staging (i.e., progressive development) of a defence over time. In short, it will consider a range of options (for example, for a greater or lesser level of service / risk acceptance) and this will allow for the assessment of opportunity costs.

Work on funding strategies (i.e., assessment of the communities' ability to pay) also needs to be progressed. These additional studies will inform feasibility.

7 Summary

As part of the SMP project, coastal inundation modelling / storm tide assessments have been undertaken for communities around the Coromandel Peninsula. This study assessed coastal flooding risks from storm tides, wave runup and overtopping, as well as tsunamis, for eight locations. The methodology used to determine these inputs was validated against observed events and geomorphological features. Based on the hazard exposure and physical constraints in each location, possible solutions (embankments and T-walls) have been developed for protection from up to a 1% AEP coastal inundation event in 2120, based on a sea level rise allowance of 1.4 m. Recognising the impact of the coastal flood and raised tailwater levels on runoff, fluvial events were also assessed, and pump station solutions developed. The cost and feasibility of each of the possible solutions was assessed to provide guidance on possible responses to the threat of rising sea levels.

Reference (typical) concepts were prepared and the possible solutions tailored to the space available at each site. Crest levels for the protection works were assessed based on storm tide or tsunami levels plus 0.5 m freeboard for areas not exposed to waves or limiting overtopping to less than 5 l/m/s for areas exposed to waves. Further, the threat from erosion was considered, with variations to the defence concept for areas exposed to erosion threats.

With raised tailwater levels due to sea level rise and barriers in place, drainage under gravity would be impacted. The fluvial flood risk was assessed for local catchments and the need for pump solutions assessed. Based on the affected area and internal drainage paths, pumps were sized and it was determined that every community would require at least one pump.

Based on the reference concepts, unit costs for the different defences were calculated. Costs for other ‘big ticket items’ such as pumps and penstocks and pumps were captured. Ancillary costs such as upgrades to roads or bridges or relocating services were not captured.

The assessment identified several constraints, which differ in magnitude from site to site. Typically, constraints for consideration include, but are not limited to:

- Land availability (space).
- Land ownership and potential for property acquisition.
- Existing infrastructure and services.
- Geotechnical ground conditions.
- Groundwater and seepage.
- Material availability (borrow source).
- Loss of habitat or amenity (i.e., the beach).
- Construction access.
- Cost.

It is important to note that the possible defence options developed as part of this work are concepts only. They are not proposed designs; they are not full or outline designs for implementation. They have been developed for the purpose of identifying constraints, assessing feasibility, and developing high-level costs estimates. If a defence option is to be progressed in any of the locations assessed, further detailed investigations should be progressed through specific coastal defence strategies.

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