BEFORE A BOARD OF INQUIRY
EAST WEST LINK PROJECT

UNDER

the Resource Management Act 1991 (the RMA)

AND

IN THE MATTER OF

Notices of requirement for designation and resource consent applications by the NEW ZEALAND TRANSPORT AGENCY for the East West Link Project

STATEMENT OF PRIMARY EVIDENCE OF STEPHEN JOHN PRIESTLEY ON BEHALF OF THE NEW ZEALAND TRANSPORT AGENCY

Coastal Processes

Dated: 12 April 2017
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1. **SUMMARY OF EVIDENCE**

1.1 My name is Stephen Priestley and I am a Senior Technical Director with Beca Limited. My evidence addresses the potential effects on coastal processes.

1.2 Coastal processes relate to the coastal hydrodynamics (the movement of fluid such as tides and wave action), sedimentation (the supply, transport, erosion and deposition of sediment) and morphology (the natural form) of the study areas.

1.3 For the **Foreshore Works** Assessment, the study area is the Māngere Inlet (the *Inlet*) and adjoining coastal marine areas (*CMA*) which is the area most likely to be affected by the East West Link Project (the *Project*). For the **Ōtāhuhu Creek Bridge** assessment, the study area is the upper part of the Tāmaki Estuary. In addition I have addressed both the short and long-term effects related to the **Dredging and Mudcrete Operation** within the Inlet.

1.4 The overall aim of my evidence is to describe the coastal processes and to assess the environmental effects of the Project on the study areas. My general approach for assessing the effects of reclamation was to gain an understanding of the existing environment, and the changes and effects that have occurred with historical reclamations. As there has been significant reclamation within the Inlet and encroachment into the Inlet channel, the effects of these human related activities provided a gauge as to potential future effects. I have also relied on the results of the numerical modelling by NIWA (**Appendix A**) of the existing environment and the Project.

**Foreshore Works**

1.5 The Foreshore Works involves creating reclamation of some 18.4ha along the northern coastline of the Inlet. It also includes the coastal structures and revetments and infrastructure within the reclamation such as the proposed wetlands and boardwalks.

1.6 Māngere Inlet and Onehunga Bay have been subject to significant change since the mid 1800s. Up to the old Māngere Bridge, the original Inlet had CMA of 7.5km$^2$ but this has been reduced to 5.7km$^2$ through reclamation, resulting in a loss of 1.8km$^2$ (24%) of the CMA.

1.7 Observed historical changes of the coastal processes have been more pronounced with narrowing of the tidal Inlet channel than with reclamation. Narrowing of the tidal Inlet channel has created a coastal inlet whereas it was originally part of the wider...
harbour environment. This has resulted in a deepening of the main tidal channel and a reduction of wave energy entering the Inlet.

1.8 Potential environmental effects are related to changes to the tidal regime, the sedimentation regime and the coastal morphology. Although changes will occur with the implementation of the Foreshore Works, the tidal current circulation and the sedimentation patterns remain similar as they are today.

1.9 I consider that adverse effects associated with coastal processes for the Foreshore Works alone will be minor. The recreated coastline which is part of the Foreshore Works will have a beneficial effect such that the coastal processes are part of the overall processes to enhance natural character. The Project design has allowed for sea level rise and tsunami for at least the next 100 years.

**Ōtāhuhu Creek Bridge**

1.10 Although the existing culverts under SH1 at Ōtāhuhu Creek perform their hydraulic function, the Transport Agency has agreed to replace the culverts with a new bridge.

1.11 The proposed bridge will span the original creek to effectively declaim the area. On the northern side, the abutment more-or-less follows the original landform. On the southern side the new landform will declaim about 0.55ha of land. A new tidal channel will be formed to be located in a similar alignment to the original channel at a mid-span location.

1.12 I consider that the new bridge structure and landform will have a beneficial effect. This effect is mainly as a result of re-introducing the coastal processes that relate to natural character.

**Dredging and Mudcrete Operation**

1.13 It is proposed that ‘mudcrete’ be used to build parts of the Foreshore Works although other alternative material could also be used. Mudcrete is a material made from mixing dredged material with ordinary Portland cement. The dredged area will be sub-tidal and occupy an area of 15ha within Māngere Inlet and some 200,000m$^3$ of material will be removed.

1.14 The Manukau Harbour is known for having high natural levels of total suspended solids and sediment deposition. In my opinion, the sediment plumes from the dredging and mudcrete operation will have a minor adverse effect. It will be temporary, persisting for a period of about 1 year.
2. INTRODUCTION

2.1 My full name is Stephen John Priestley. I am a Senior Technical Director with Beca Limited in Auckland.

Qualifications and Experience

2.2 I have a Bachelor of Engineering with First Class Honours in Civil Engineering from the University of Auckland and a Masters of Engineering Science from the University of New South Wales. I am a member of the New Zealand Hydrological Society and the New Zealand Coastal Society. I am a Chartered Professional Engineer and a Fellow of the Institution of Professional Engineers, New Zealand. I have worked on major infrastructural projects for over 35 years in New Zealand, Australia, the United Kingdom, Central America and the Asia-Pacific region.

2.3 The main focus of my project experience has been in port and coastal projects, water management and land development. My relevant experience includes:

(a) Managing many of Ports of Auckland Limited’s (POAL) resource consent applications and projects including the Fergusson Container Terminal extension, the Wiri Inland Container Terminal development, the deepening of the Rangitoto Channel shipping lane and other dredging and disposal projects, wharf repairs and upgrades and stormwater management.

(b) The assessment of infrastructure and contamination issues for Development Auckland – Panuku’s Western Reclamation and later engineering input and construction monitoring of its Wynyard Quarter development.

(c) Resource consent applications and the design and construction of marine works for Fisherman’s Wharf and the Viaduct Basin (formerly known as the America’s Cup Harbour).

(d) Engineering studies and the design of marine projects including:

(i) Chelsea Wharf;

(ii) The Port of Napier;

(iii) The Northport berth extensions;

(iv) Wharf repairs at the Port of Timaru;

(v) Dredging at the Port of Lyttelton;
(vi) The Northern Regional Boat Ramp for the Western Bay of Plenty District Council;

(vii) Kings Wharf in Suva, Fiji; and

(viii) The Victoria Desalination Plant in Melbourne, Australia.

(e) The design of marinas (e.g. Westhaven, Pine Harbour, Gulf Harbour, Bayswater, Whitianga, Mercury Island and the Silo Park superyacht marina).

(f) Creation of beaches and renourishment projects for Westshore in Hawkes Bay, Sulphur Point in Tauranga, Mission Bay, Kohimaramara and St. Heliers Beaches in Auckland’s eastern suburbs. Also creation of a mudcrete coastal walkway in the Ōtāhuhu Estuary at Pakuranga.

(g) Declamation associated with the removal of the oxidation ponds at Mangere and 5km of coastal restoration within the Manukau Harbour.

(h) Review on behalf of the Auckland Regional Council the coastal processes associated with the 2nd Manukau Harbour Crossing (MHX).


2.4 In addition to the above project experience, I have acted as an Independent Commissioner and the Minister of Conservation's representative at resource consent application and plan change hearings. I have previously presented expert evidence to Commission hearings, arbitrations, the Environment Court and the High Court.

2.5 I have been engaged by the New Zealand Transport Agency (the Transport Agency) to undertake (amongst other things) an assessment of the coastal processes effects of the Project, for which the Notices of Requirement (NORs) and resource consent applications have been lodged with the Environmental Protection Authority (EPA). My evidence describes this assessment and its findings.

3. CODE OF CONDUCT

3.1 I confirm that I have read the Code of Conduct for Expert Witnesses contained in the Environment Court Practice Note and that I agree to comply with it. I confirm that I have considered all material facts that I am aware of that might alter or detract from the opinions that I express, and that this evidence is within my area of expertise, except where I state that I am relying on the evidence of another person.
4. **SCOPE OF EVIDENCE**

4.1 The purpose of my evidence is to provide the rationale and conclusions of the assessment of effects for the coastal processes related to the Foreshore Works and the Ōtāhuhu Creek Bridge. In addition, my evidence will address the effects of dredging, the use of the dredged material as mudcrete and the construction methodology for the proposed reclamation.

4.2 This evidence highlights the key points from my Technical Report No 15 – Coastal Processes Assessment (*Technical Report 15*), which formed part of the Assessment of Effects on the Environment (*AEE*). Where appropriate in response to design changes to respond to submissions, this evidence updates my original assessment and recommendations. With the exception of any updates noted in this evidence I confirm the content and accuracy of that report.

4.3 My evidence addresses the following matters:

(a) Project description and my role in the Project;

(b) Assessment methodology;

(c) A description of the existing environment relevant to my assessment;

(d) Assessment of effects and proposed mitigation;

(e) Construction methodology for reclamation;

(f) Response to section 149G report / other reports;

(g) Response to submissions; and

(h) Overall conclusions.

4.4 As part of my assessment, I commissioned the National Institute of Water and Atmospheric Research (*NIWA*) to numerically model a number of Foreshore Works scenarios and report on the likely changes in tidal currents and long term sedimentation. NIWA also modelled the far field dispersion of dredged material in the water column and reported the changes in suspended solids levels and sediment deposition throughout the upper part of the Manukau Harbour. I have relied on the outcome of NIWA’s modelling in determining the likely effects on coastal processes and their report is attached to my evidence as Appendix A.
4.5 As part of this Project, site specific studies of sediment quality were undertaken in order to characterise the level of sediment contamination. This work was undertaken by specialist sediment contamination scientists who were part of the Project team. I have relied on that information in assessing the effect on sediment quality.

5. **OVERVIEW OF THE PROJECT AND MY ROLE**

5.1 The Project has been described in Section 6 and 7 of the AEE and in the evidence of Mr Noel Nancekivell.

5.2 A number of design changes to the proposal are proposed to respond to submissions. The only change which affects coastal processes is a relocation of the proposed boardwalk between landform 2 and landform 3 so that it is located 15-25m from the edge of the embankment rather than 25-40m.

5.3 My main role in the Project has been to manage the coastal process assessment and be the principal author of Technical Report 15 – Coastal Processes Assessment.

5.4 Other aspects of the Project that I have been involved in were:

(a) Providing design parameters and input into options for the Foreshore Works as summarised in Appendix B;

(b) Part of the multi-criteria-assessments (MCAs) for the Foreshore Works and the Ōtāhuhu Creek Bridge; and

(c) Presentations to Mana Whenua and the EPA on the coastal process assessments.

5.5 With regard to design aspects of the Project, I worked with the ecologists, landscape architects, urban designers, stormwater engineers and planners to develop the overall coastal enhancement programme. As part of the Project Team we were mindful that a key theme of the New Zealand Coastal Policy Statement (NZCPS) and associated planning documents is the need to avoid reclamation where it cannot be justified. This aspect is described in detail in the AEE document. Evaluation of options and providing mitigation of adverse effects have been key considerations in the development of the proposed works.

5.6 I note that prior to the assessment of effects for the Foreshore Works some of the potentially more significant effects on the coastal processes were reduced by the design of the proposal, including:
(a) Minimising intrusive reclamation in the area near the Inlet entrance (by Galway Street up to Albert St). From a review of the effect of historical changes, intrusion into the Inlet entrance has resulted in more noticeable effects;

(b) Avoiding reclaiming into the tidal channels. This would alter the morphology of the channels and result in a different distribution of tidal flows and sedimentation regime. The exception was at the eastern end where a secondary tidal channel that feeds into Anns Creek is located close to the northern coastline. In this location a new tidal channel has been included in the design with the same dimensions as the existing channel;

(c) Discharging stormwater from the proposed treatment system through the headland structures into the tidal channels; and

(d) Incorporating coastal features such as headland structures and shingle beaches into the recreated coastline.

6. ASSESSMENT METHODOLOGY

Foreshore Works Assessment

6.1 The initial phase of the study was to assess the existing environment. As there has been significant reclamation within the Inlet and encroachment into the Inlet channel, the effects of these human related activities provided a gauge as to potential future effects.

6.2 This part of the study included; site visits, condition assessment of existing coastal protection structures, meeting with Mana Whenua, organising a bathymetric survey of the Inlet, literature search for previous investigations, assessing the historical coastal process changes (and effects) within the study area due to natural processes and human intervention, and developing a numerical model to simulate the hydrodynamic and sedimentation processes (by NIWA).

6.3 This methodology is outlined in Section 3.0 of Technical Report 15.

6.4 The second phase of the study was to assess the effects of the Project on the existing environment. The Project includes reclamation, coastal enhancement and protection works, dredging and stormwater discharges to the CMA.

6.5 I directed modelling (by NIWA) of the Project to gain an understanding of the likely changes in the hydrodynamic and sedimentation processes, including the dredging operations. In addition, I assessed any morphological changes that relate to any
encroachment into the Inlet channel and other tidal channels; quantified the extent of dredging and likely short and long term changes; and interpreted the changes from the numerical model, morphology study, dredging assessment, and coastal enhancement programme as either adverse or beneficial effects.

**Ōtāhuhu Creek Assessment**

6.6 The initial phase of the study was to assess the existing environment. Due to installation of culverts, reclamation and encroachment into the estuarine channel associated with the original SH1 motorway, the effects of these human related activities provide a gauge as to potential future beneficial effects of building a new bridge.

6.7 The second phase of the study was to assess the effects of the Project on the existing environment. The Project will include removal of the culverts and declamation.

**Dredging and Mudcrete Operation**

6.8 As part of considering the design options for the Foreshore Works, the Project Team identified the benefits for the road embankment and outer bunds to be relatively impermeable so as to form a barrier for leachate migration and tidal ingress. The Project Team decided to form it with mudcrete although alternative material could also be used. Mudcrete is a material made from mixing dredged material with ordinary Portland cement.

6.9 After assessing the quantity of mudcrete required, it was then a case of finding an area where material could be dredged. As the old Māngere Bridge forms a navigation barrier for the ready transit of vessels, I concluded that the dredged area needed to be in the Māngere Inlet. Following discussions with the Project ecologists, we concluded that the dredged area needed to be in the sub-tidal zone as the inter-tidal zone is an important bird habitat. The 15ha selected area also contains an area of invasive Asian date mussels which would be removed with the dredging operation.

6.10 The next phase of the study was to assess the effects of the short-term dredging operation on water quality (suspended solids) and sedimentation around the Māngere Inlet.

**7. EXISTING ENVIRONMENT**

**Foreshore Works Assessment**

7.1 Māngere Inlet is a semi-enclosed basin comprising of shallow tidal creeks, mangroves and large expanses of intertidal mud flats. From the LiDAR data (2013) it is estimated
that the estuary surface area is 5.7km$^2$ east of the old Māngere Bridge. Within the Inlet the spring intertidal area is estimated to be 5.0km$^2$ and the neap intertidal area is 4.2km$^2$. Given that the overall area of the Inlet is 5.7km$^2$, this illustrates how shallow the Inlet is. The contributing catchment is mostly urbanised catchment with an area of about 34.5km$^2$. It is predominantly an intertidal estuary with low freshwater inflows.

The northern coastline has undergone extensive urban development and reclamation since the 1940s. Reclamation along the eastern shore of Māngere Inlet was also carried out in the 1960s with the development of the Westfield rail yards. Overall it has been estimated that 1.8km$^2$ (24% of the Inlet surface area) of land has been reclaimed within the bay since 1940 as shown in Figure 1.

Figure 1: Changes in the northern shorelines between 1853 (yellow outline) and 2010

The Manukau Harbour is characterised as a predominantly semi-diurnal (twice daily) tide. Tidal velocities are relatively high (1m/s during spring tides and up to 0.5m/s during neap tides). At these velocities the tidal flow are capable of mobilising and transporting sediment.

The tidal wave within the Manukau Harbour is amplified from a spring tidal range of 2.9m at Manukau Heads to 3.8m at the Port of Onehunga. Mean high water springs (MHWS) is 4.25m above Chart datum or 2.05m above the Auckland Vertical datum (AVD which is the Project datum and 0.0m AVD or 0.0m RL is close to mean sea level).
7.4 Extreme sea levels represent a storm tide, based on a high tide plus storm surge. The 100 year ARI (average recurrence interval) is estimated to be 3.0m AVD.

7.5 As it lies at the north-eastern end of Manukau Harbour, Māngere Inlet is sheltered from higher energy wave climate of the main body of the harbour. Typically ambient wave conditions are generally less than 0.3m for the majority of the time and could reach a 0.6m significant wave height during extreme events.

7.6 Tide and wind generated currents in combination with locally generated wind waves act together to suspend and transport fine sediments from the exposed intertidal banks, leading to enhanced turbidity levels. Measured suspended sediment concentrations in the Manukau Harbour are relatively high with a median of 26g/m³ with a long term range between 10 to 150g/m³.

7.7 I concluded that Māngere Inlet is a sediment sink and experiences a significant amount of sediment movement, particularly during windy conditions. Sediment in the Māngere Inlet is derived predominantly from redistribution around the harbour and Inlet rather than from local catchment sources. The increase in mangrove distribution throughout the Inlet over the past 50 years has probably increased sedimentation levels. Overall I assessed that the average sedimentation rate was 10mm/yr which equates to a total annual deposition volume of 57,000m³ or 43,000 tonnes. Areas of mangroves and exposed intertidal flats had similar rates with an average sedimentation rate of 17mm/yr while in tidal channels and around islands the average erosion rate was 26mm/yr. In general the northern coastline had higher sedimentation rates than the southern coastline.

7.8 From Project specific sediment quality investigations, it was found that metal concentrations were consistent with that of concentrations measured in sediments in other parts of the Auckland region, and likely to be generally representative of background concentrations. The exception to this was arsenic where some concentrations were elevated above natural concentrations, and above ANZECC low acceptance criteria at approximately half the locations. High levels of ammonical nitrogen were found in porewater along the northern coastline, probably due to diffuse discharges from the adjacent landfills.

7.9 Historical human intervention to the Māngere Inlet was compared with local hydrodynamic and morphological changes to gain a better understanding of how it will react to the Project. Observed historical changes have been more pronounced with

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narrowing of the tidal Inlet channel than with reclamation. Changes to the Inlet due to reclamation have tended to be relatively benign, as reclamation did not encroach into the main tidal channels. It has probably been masked by the effects of narrowing the tidal Inlet channel and the increase in mangrove coverage (now occupy 20% of the inlet over the past 60 years). Narrowing of the tidal Inlet channel has created a coastal inlet whereas it was originally part of the wider harbour environment. This has resulted in deepening of the Inlet channel and a reduction of wave energy entering the Inlet.

**Ōtāhuhu Creek Bridge**

7.10 Ōtāhuhu Creek is a tidal creek which flows east to northeast into the Tāmaki Estuary. See Figure 2. Currently the approximate CMA of the creek to the west of SH1 is 5ha, 95% of which is covered with mangroves. There has been extensive urban development surrounding the creek since 1940 and has a contributing area of 144ha.

7.11 In the late 1950s, triple culverts were installed under SH1. The culverts comprise three 2.1x2.1m box culverts, 33m long, with their invert at about 0.5m above mean sea level.

**Figure 2: Ōtāhuhu Creek Aerial Photo (LINZ)**

7.12 Mean high water spring tidal levels is 1.64m AVD and the 100 year ARI storm surge event is 2.42m AVD.

7.13 Ōtāhuhu Creek is a shallow tidal creek with extensive mangroves covering the majority of the creek. Waves would be wind generated and most likely be dissipated by the
mangroves. The combination of a small wind fetch length and large mangrove area does not promote any significant wave action.

7.14 As part of the Project the area around Ōtāhuhu Creek was surveyed. The survey confirmed that the downstream level of the culverts is about 0.5m above mean sea level. Upstream and downstream seabed levels in the main tidal channel are lower. This indicates that the tidal flows are sufficient to maintain a formed channel rather than for it to be in filled.

7.15 Based on the as-built drawings it appears that the area was reclaimed with the SH1 construction. The reclamation area was in the order of 0.5ha or about 10% of the Ōtāhuhu Creek CMA.

8. ASSESSMENT OF EFFECTS AND PROPOSED MITIGATION

Foreshore Works

8.1 The layout of the Project Foreshore Works is shown in Figure 3.
8.2 The Project has a reclamation area (above MHWS) of 18.4ha and a reclamation footprint\(^2\) of 24.27ha, compared to the existing CMA surface area of the Inlet of 5.7km\(^2\). The new reclamation represents a change of 3.5% in area. (It is noted that the reclamation area due to the road embankment of the Project is 5.6ha whereas the wetlands and recreated coastline is 12.2ha.) The Project also reduces the tidal prism by 3.5%, from 11.9 million m\(^3\) to 11.5 million m\(^3\). (The tidal prism is the volume of seawater exchanged between MHWS and MLWS upstream of a reference point, e.g. old Māngere Bridge).

8.3 The Manukau Harbour CMA surface area is estimated at 368km\(^2\) with a historical area of harbour reclamation of 8.5km\(^2\). This Project will therefore increase the reclamation area from 2.3% of the harbour area to 2.4%.

8.4 For the structures in the CMA through Anns Creek, there are 21 No. 2.1m diameter piles which will occupy an area of about 73m\(^2\) or 0.001% of the Inlet area.

8.5 Reclamation reduces the footprint of the CMA and correspondingly reduces the tidal prism. Reducing the tidal prism reduces the tidal currents. For the Inlet, where the flood tidal currents are stronger than the ebb tidal currents it could be expected that the transport of sediment into the Inlet could reduce but this is more likely to depend on the extent of wind driven currents and waves which tend to transport sediment into the Inlet. As these wind driven currents and waves, which originate outside of the Inlet, are unlikely to change the average level of sedimentation is not expected to change significantly. Continued sedimentation in the Inlet is likely to lead to more favourable conditions for mangrove coverage.

8.6 The entrance is also likely to respond to the change in the tidal prism. With a lesser tidal prism the cross-sectional area of its entrance will reduce to reach a new equilibrium condition. Some accretion could therefore be expected at the seabed of the entrance, probably in the order of 35m\(^2\) cross-sectional area or 0.25m depth. Historically the seabed has been more elevated. The entrance will continue to limit the amount of wave energy entering the Inlet.

8.7 NIWA has modelled the Project in terms of tidal currents and sedimentation (as contained in Appendix A of my evidence) which demonstrates the differences in maximum tidal currents and sediment deposition between the existing environment and the environment after the implementation of the Project. A summary is given below:

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\(^2\) The reclamation footprint is the reclamation area above MHWS as well as the reclamation toe area which is below the MHWS.
(a) The maximum reduction in tidal current occurs within the new embayments along the northern coastline with change of 0.1m/s in a spring tide. The maximum increase in tidal current occurs offshore of the new headlands with change of 0.1m/s. Away from these locations, the maximum change in tidal currents is less than 0.05m/s during a spring tide;

(b) The general circulation and extent of tidal currents is the same as the existing regime;

(c) Overall there is a slight increase in average sediment deposition within the Inlet from 9.8mm to 10.5mm (7% change); and

(d) There is an increase in deposition within the new embayments along the recreated coastline with an increase of 5mm/year to a new level of up to 30mm/year. Mud deposits could therefore be expected in these locations as would have occurred with the original and existing environment.

8.8 Sedimentation will naturally continue within the Inlet at a rate of about 10mm/year. This has the potential to affect the discharge of stormwater as the intertidal areas in front of stormwater pipes silt up. The recreated coastline design, however, incorporates discharging the stormwater into the tidal channels which have a tendency to erode rather than accrete. This effect is therefore minimised.

8.9 The tidal channels have more-or-less the same level of erosion as the existing situation. These channels should therefore remain in a morphological stable condition.

8.10 Within the area of the structure over Anns Creek the peak tidal velocities are less than 0.2m/s. As this is less than the velocity required to mobilise marine mud, no scouring of sediment around the piles is expected.

8.11 My assessment of the coastal processes assumes that this Project along with other probable activities should be assessed on the environment as it exists. Other probable activities include other coastal works which have resource consents and natural processes such as climate change and tsunami. I understand that the only coastal project which has resource consents but which have not been implemented is the replacement of the old Māngere Bridge. The proposed replacement bridge will not include any reclamation and will be a piled structure. That new structure is not expected to add any additional adverse effects associated with this Project.

8.12 I concluded that the cumulative historical developments have probably had a significant adverse effect on the original environment, mainly due to the narrowing of the Inlet
entrance and intrusion into the main tidal channel by the old Māngere Bridge. But the changes related to those effects have now become part of the existing environment for which this Project is assessed against.

8.13 In my opinion this Project will have the following effects:

(a) The Inlet will remain a depositional environment with minimal erosion risk to the coastline;

(b) Potential erosion risk of the coastline associated with this Project is low as it will be protected from inundation and wave action;

(c) Adverse effects associated with coastal processes within the Inlet for the Foreshore Works are minor. The Project design needs to carefully consider the design of any new tidal channel works at the eastern end of the Project so as to closely mimic the geometry of the existing channel; and

(d) Adverse effects associated with coastal processes within Anns Creek for the elevated structures are negligible.

8.14 I can confirm that the design change to relocate the location of the proposed boardwalks does not alter any of my assessments or conclusions.

Management of natural hazards

8.15 The NZCPS (Policies 24 and 25) requires hazards to be identified and development to consider these hazards over at least the next 100 years. Hazards that have been identified and that I have had regard to include:

(a) Sea level rise (SLR - 1.0m as required by the Unitary Plan (2016));

(b) Tides and storm surge, for example, the 100 year ARI event will have a still water level of RL 4.0m, allowing for SLR (i.e. about 1.95m above MHWS);

(c) Wave climate (10% increase as per MfE, 2008 in extreme winds which affect wave generation); and

(d) Tsunami (wave heights have been as 1.5m for the 100 year ARI event, based GNS (2013) for a 50% confidence level, for the Manukau Harbour).

8.16 The effect of sea level rise, storm tide, and wave run-up has dictated that the minimum height of the road embankment be RL 4.5m.
Natural character

8.17 While the reclamation associated with the recreated coastline does have minor adverse effects on the existing environment, the recreation of coastal features such as headlands and shingle beaches is beneficial in terms of the coastal processes related to natural character. As shown in Figures 4a and 4b, the original coastline in 1949 was a mixture of headlands and embayments, delineating a non-linear coastline.

Figure 4: Māngere Inlet Northern Foreshore (1949)
8.18 I recommend that proposed monitoring and mitigation include:

(a) Any tidal channel for which the Project covers over, a new channel be created (with similar geometry) to minimise morphological changes;

(b) Peer review of the detailed design to ensure the effects of climate change and tsunami have been adequately allowed for; and

(c) Detailed design of the Foreshore Works needs to recognise that elements will be subject to seawater spray and corrosion.

8.19 In Technical Report 15 I had recommended that options for declamation in the Manukau Harbour be investigated to evaluate the adverse effects of reclamation. I
understand that the ecology and planning members of the Project team have undertaken such an investigation in the context of offsetting the effects of reclamation of foraging habitat. However, no suitable areas were identified.

Ōtāhuhu Creek Bridge

8.20 Although the existing culverts perform their hydraulic function, the Transport Agency has agreed to replace the culverts with a new bridge as part of this Project. The proposed bridge is shown in Figure 5 and comprises five 22.5m spans to give a total bridge length of 112.5m and width of 50m. There are proposed to be 19 No. 900mm piles which will occupy an area of about 10m² or 0.02% of the upstream CMA.

Figure 5: Ōtāhuhu Creek Bridge

8.21 The proposed bridge will span the original creek to effectively declaim the area. On the northern side, the abutment more-or-less follows the original landform. On the southern side the new landform will declaim about 0.5ha of land. Complete declamation was not feasible as it would interfere with private property. Some 20,000m³ of material will need to be removed to form a new landscape similar to the original landform.
8.22 From a coastal processes perspective, I note the following:

(a) The area has a low energy wave climate, particularly with the presence of the mangroves and this will continue;

(b) The tidal currents, without the culverts, will be relatively low.

(c) Flood flows could be in the order of $30 m^3/s$ for the 100 year ARI. This is easily accommodated within the new bridge opening;

(d) Although the culverts were installed with an invert of 0.8m RL at the upstream end, an upstream channel has been maintained deeper than 0.8m RL. Therefore it has probably reached some equilibrium where the extent of upstream sedimentation is limited. This could be attributed to the occasional flushing of flood flows from the local urban catchment;

(e) The tidal channel alignment has changed with the installation of the culverts, probably because it was installed off-line from the original channel. I recommend that the channel be realigned close to its 1940’s alignment; and

(f) Given the extent of the mangrove forest, erosion of the flat intertidal area would be limited. Some erosion of the tidal channel could be expected as part of the readjustment and would be an on-going process.

8.23 The Project can accommodate inundation levels associated with a 100 year ARI tsunami and a 1m sea level rise.

8.24 I recommend that detailed design of the Ōtāhuhu Creek Bridge recognises that elements will be subject to seawater spray and corrosion and that the Construction Management Plan include for removal of the existing culverts address the demolition and removal process, particularly with regard to discharges to the CMA.

**Dredging and Mudcrete Operation**

8.25 It is proposed to use ‘mudcrete’ to build parts of the Foreshore Works. Mudcrete is a material made from mixing dredged material with ordinary Portland cement. The dredged area will be sub-tidal and occupy an area of 15ha within Māngere Inlet as shown in Figure 6.
8.26 Dredging production rates will be a maximum of 750m$^3$/day. Required quantities of mudcrete include:

(a) 36,000m$^3$ of in-situ material under the outer bunds to form a stable foundation;

(b) 200,000m$^3$ which needs to be won externally, and of which some 120,000m$^3$ would form the outer bund to contain the wetlands. The remaining quantity of 80,000m$^3$ would be used as a liner on the internal bunds; and

(c) Material won from within the wetland footprint can used to form the seal or liner for the wetlands and to create the main bund for the Project road, but that operation would not be exposed to the tide.

8.27 Sources of sediment from the dredging/placing operation that could cause a sediment plume which would be dispersed around the Inlet/harbour are from:

(a) A dredger bucket in the subtidal area,

(b) Overflow from the receiving barge;

(c) A small dredger bucket in the intertidal area; and
8.28 Based on the sediment plume modelling by NIWA, the maximum sediment concentration away from the Project area and from the dredging operation at 200m is estimated to be 31 g/m³ for the dredging and mudcrete operations at the eastern end of the Foreshore Works. Based on my experience with other dredging projects, this is probably a conservative estimate as it allows for the settling velocity of a single particle rather than for coalescing of particles which encourages higher settling rates. This in turn allows for the sediment to be deposited closer to the dredging operation.

8.29 For comparison the ambient median sediment concentration in the Manukau Harbour is 26g/m³ with a wide variation (10 to 150g/m³). Average sediment concentration in the Māngere Inlet is about 30g/m³. These values are representative of median/average concentrations which would persist over the whole tidal cycle.

8.30 It is estimated that the average release of sediment (and mudcrete) will be 9 tonnes/tide. This compares to the average natural flux of sediment into and out of the Inlet of 700 tonne on a spring tide and 350 tonne on a neap tide. The sediment release is therefore some 1.5 to 2.5% of the natural sediment flux. The dredged material is the native material, not an introduced source, so the water quality associated with sediment release will be similar to the native material when re-suspended.

8.31 The fate of the sediment plumes will be predominantly into the Inlet as naturally occurs. The total release of sediment for the Project is about 6,300 tonne. It is estimated that the maximum deposition away from the mixing zones will be 6mm with an average of 4mm. This compares to an average deposition within the inlet of 10mm/year (i.e. 43,000 tonne/year) with 25mm/year at the northern coastline and 10mm in the Anns Creek area.

8.32 The Manukau Harbour is known for having high natural levels of total suspended solids and sediment deposition. In my opinion, the sediment plumes from the dredging and mudcrete operation will have a minor adverse effect. It will be temporary, persisting for a period of about 1 year.

8.33 On completion of the Project the dredged area will cover an area of about 15ha, with the removal of about 200,000m³ of sediment at an average depth below existing seabed levels of 1.5m. As part of the dredging operation the area of Asian date mussels will be removed.
I consider that the long term adverse effects from the use of mudcrete and potential leaching of contaminants will be negligible.

9. METHODOLOGY FOR RECLAMATION

9.1 The reclamation for the Foreshore Works will be constructed from variety of materials. These materials will include mudcrete for the outer bunds and liners, quarry products for bulk filling, and rock and shingle for coastal protection works and beaches. Finishing works will include measures such as soil and grassing, boardwalks and footpaths and formations such as wetlands, roads and landscaping. The specific construction process for the reclamation is discussed in the evidence of Mr Nancekivell.

9.2 This construction methodology includes the use of ‘mudcrete’ to build parts of the Foreshore Works, although this does not exclude the use of alternative material for this purpose. The main benefits of using mudcrete are that dredged material is locally available, it can be placed in all weather conditions and requires no compaction. Dredged material will be won from within the Foreshore Works footprint and from a dedicated dredging area as discussed in para. 8.25 of my evidence.

9.3 It is intended that the mixing plant for the mudcrete, probably a pug mill, will be located near the Waikaraka Park construction site. In order to have an efficient operation, a navigation channel would be dredged to -3.5m RL from the dredging area to the mixing plant. This is outlined in Figure 6 of my evidence. This channel would enable the dredging operation to provide material continuously to the mixing plant without having to work around a tidal window.

9.4 The dredging operation will be undertaken by a mechanical dredger such as a bucket excavator on a barge. This type of operation minimises the amount of water taken into the dredged material which is important for the mudcreting operation. The dredger will excavate the seabed material and place it in a barge which will then transport it to the mixing plant. This operation will be undertaken during daylight hours but could also be done at night time.

9.5 From the mixing plant mudcrete would be transported to the working area via either a conveyor system or vehicles. The mudcrete would initially be placed at existing seabed levels then gradually built up for later reshaping. Prior to this operation the seabed would be stabilised with mudcrete to form the foundation for the bund which could be achieved a number of ways. It could be stabilised in-situ with specialised equipment; or it could be dredged, mixed in an adjacent basin or barge, and placed back into the
dredged area; or it could be dredged, conveyed to the mixing plant (at Waikaraka Park), mixed and conveyed back into the dredged area.

9.6 It is intended that the outer bund will be completed as the first stage. This would encapsulate the Project area, thereby limiting the potential for sediment plumes from the placing of mudcrete material and potentially allowing the remainder of the Project to be completed in the dry.

9.7 The bulk of the reclamation fill will be sourced from quarry material which will comprise rock and weak rock. Benefits of using this material are that it can be placed in the wet, requires low levels of compaction, and produces minimal sediment plumes. It will be transported to the Project site via truck which will deposit it at its final location where it will be bladed out and compacted.

9.8 Rock material will form the lining of the embankment and headlands. It will comprise basalt boulders, similar to the in-situ lava flow material. Rock will be delivered to the site via trucks which will dump the material over a rock underlayer or cushion layer. The rock will then be individually placed with an excavator to form a smooth even finish. Some part may also be randomly placed such as at the headlands to mimic the original landform.

9.9 Rock shingle will form the new pocket beaches. This material will be sourced from quarries, transported to the site via trucks and be deposited at its final location. From there it will be bladed out to form the beach.

10. RESPONSE TO AUCKLAND COUNCIL'S 149 ISSUES REPORT/OTHER REPORTS

10.1 The Key Issues report raises only two issues of relevance to coastal processes:

(a) At paragraph 50(b), "whether the scale of the proposed reclamation is appropriate and could be reduced"; and

(b) At paragraph 60(c), "whether it is appropriate to undertake dredging in the Māngere Inlet, particularly with regard to the resuspension of contaminants into coastal waters"; and

10.2 I have addressed these issues within Technical Report 15, the assessment of effects outlined above, and my response to submissions below.
11. RESPONSE TO SUBMISSIONS

11.1 I have read and considered the submissions addressing the potential impact of the Project on coastal processes, and my responses are set out below.

Submission by Auckland Council (Submission No 126336)

11.2 Section 5.1 Reclamation and Boardwalks: Council has a desire to minimise the extent of reclamation area and coastal occupation. As I have set out under Section 5 of my evidence, the Project Team was mindful of minimising the extent of the reclamation and carried this through in designing the layout of the road embankment, stormwater treatment areas and landscape features. In terms of addressing the various planning documents that is covered in the evidence of Ms Andrea Rickard.

11.3 Section 5.1 Reclamation and Boardwalks: In para 7 Council states that there is no evidence to support the proposed configuration of the headlands and embayments. Attached as Appendix B is a Design Note which I issued to the Project Team in August 2016 which addresses the philosophy is designing the coastal features. For the beach creation:

*It is intended that as part of the naturalisation of the coastline a number of shingle type beaches will be created. These beaches will be located between headland structures. They should be aligned parallel with the offshore bathymetry contours with some local concave curvature as the beach approaches the headland structures.*

*The inlet has a low energy wave climate. It is recommended that the shingle beach should have the following features:*

- Shingle size 20-100mm (D50=50mm)
- Berm level of RL 2.5m (i.e. about 0.5m above MHWS)
- Berm width of 5m

11.4 I address a related concern of the Council regarding alternative designs in paragraph 11.6 below.

11.5 Section 5.1 Reclamation and Boardwalks: In para 8 Council states that modelling predicts significant (five-fold) increase in sediment deposition… increase from 5mm/yr to 30mm/yr. I consider there has been a misinterpretation of my assessment. Technical Report 15: Section 7.3 states “*There is an increase in deposition within the new embayments along the recreated coastline with an increase of 5mm/year to a new level of up to 30mm/yr.*” (NIWA report pg. 33 – Appendix A states an increase of 3-5mm/yr). This reflects an increase of 12-20% not 500% as stated by Council.
11.6 Section 5.1 Reclamation and Boardwalks: In para 9 Council states that the alternative configurations for the coastline shape should be considered. The shaping of the proposed naturalised coastline was influenced by the shape of the original coastline which effectively was a terrestrial formed landscape from lava flows. This aspect is covered in the evidence of Mr Gavin Lister. The beaches show in the new coastline are generally within headland sections. These locations will be more exposed with wave focussing due to refraction from the more energetic south/south west quarter. But it needs to be recognised that the whole area has a low energy wave environment, is depositional and contains mud deposits. These areas are currently muddy and will remain so, particularly in leeward locations.

11.7 Section 5.1 Reclamation and Boardwalks: In para 10 Council states increased sedimentation will result in increased mangrove colonisation. Given that the increase in sedimentation rates will not increase to the degree stated by Council, the extent of mangrove colonisation should be less than they have anticipated. It is my understanding that the predominant mangrove species germinates down to mean sea level as it can only tolerate intermittent submergence. The area seaward of the proposed coastline is generally at or below mean sea level. Therefore mangroves should be quite sparse through this area as can be observed along the existing northern coastline, closer to Onehunga. Mangrove coverage, however, may well increase with increasing sedimentation, particularly in areas above mean sea level. This occurs now and will continue. Conversely it may decrease with sea level rise. Council also state that erosion will increase within the central area of the Inlet. In my opinion erosion in the central area of the Inlet (around the tidal channels) pre and post-development is similar (see Figures 5-12 and 5-15 of NIWA report – Appendix A).

11.8 Section 5.1 Reclamation and Boardwalks: In para 11 Council considers that the outer coastal bund level be increased in height. This is responded to in the evidence of Ms Dale Paice.

11.9 Section 5.2 Mudcrete: In para 1 Council states that the application of mudcrete is limited and small scale. Mudcrete has been applied to large projects, eg Fergusson reclamation at about 1,000,000m$^3$ and Viaduct Harbour at about 150,000m$^3$. These are not small scale applications. Other applications include Wynyard Quarter, Waitemata Upper Harbour Bridge, Kohimarama and St Heliers Beaches, Bayswater Marina, Fishermans Wharf, and Pakuranga walkway. The decision to use mudcrete was based on being able to form an outer containment bund in the wet. Other options were to use a clay core, or plastic liner (both of which would need to be constructed in the dry), or some form of sheetpiling. The Project has not ruled out other options (as per AEE), but
Technical Report 15 Appendix F has addressed the effects of dredging and placement of mudcrete if it were to be used.

11.10 Section 5.2 Mudcrete: In responding to Council’s query about whether the mudcrete will be covered, I confirm that it will be covered in the long term.

11.11 Section 5.3 Dredging: In paragraph 1 Council has stated a preference to use other materials for fill to avoid dredging. As stated above the Project has not ruled out other options (as per AEE), but Technical Report 15 Appendix F and my evidence has addressed the effects of dredging and placement of mudcrete if it were to be used.

11.12 Section 5.3 Dredging: In para 3 Council has stated that the assessment has not taken into consideration the effects of disturbing contaminated materials. I disagree with this statement. The Project Team sampled and tested the sediment for a variety of contaminants within the Inlet. See Section 5.2.6 and Appendix E of Technical Report 15 for a description of contaminated sediments. Appendix E – Figure 2 has some diagrams, illustrating the change in contaminant concentration with depth, particularly heavy metals. The overall conclusion is that the contaminants do not change significantly with depth. Therefore the distribution contaminants associated with dredging will have a similar distribution with natural redistribution of sediment through tidal and wave motion.

11.13 Section 5.3 Dredging: In para 3 Council has stated that the dredging assessment should be updated to reflect Council’s proposed increase in bund height as presumably Council has assumed that more mudcrete material will be needed. That is, however, not the case. It is proposed that mudcrete be provided up to a level of RL 2.5m (about 0.5m above mean high water springs) in order to enable construction works inside the bund in the dry. If the bund were to be higher, then other material (e.g. land based clay) can be used to form the impermeable core of the more elevated part of the bund. On this basis, the mudcrete volume will remain at about 200,000 m³, and the dredging assessment remains valid.

Other Submissions

11.14 Onehunga Business Association (Submission No. 126230, para 22), the Re-think East West Link Society Incorporated (Submission No. 126318, paras 1i and 1k), the Manukau Harbour Restoration Society (Submission No. 126319, para 1f), and the Onehunga Enhancement Society (Submission No. 126335, para 1h and 1j) have all submitted a similar concern about coastal processes. They state that the Project would have adverse effects on sediment transfer and build up in the harbour. In particular the
reduction in tidal area of the harbour will reduce the hydrological flushing effect on the enclosed harbour inlet which will increase silting. That is what Technical Report 15 and my evidence addresses, with the overall conclusion that the adverse effects are minor.

12. CONCLUSION

12.1 Based on my experience with other coastal processes studies, and with other port, marine and dredging projects, I have formed the following conclusions;

(a) Adverse effects associated with coastal processes for the Foreshore Works alone will be minor;

(b) The recreated coastline which is part of the Foreshore Works will have a beneficial effect in that the coastal processes are part of the overall processes to enhance natural character;

(c) Effects associated with coastal processes for the replacement of the existing culverts on Ōtāhuhu Creek with a new bridge will be beneficial; and

(d) The Project design has allowed for sea level rise and tsunami for at least the next 100 years.

Stephen Priestley

12 April 2017
APPENDIX A :
NIWA REPORT
Modelling the effects of coastal reclamation on tidal currents and sedimentation within Mangere Inlet

*Prepared for The East-West Link Alliance*

*October 2016*
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Executive summary

The East West Link Alliance (NZTA, Beca, GHD, and Buddle Finlay) has been formed to prepare preliminary design, and supporting technical reports and assessment of environmental effects for resource application for a new road link. The road aims to improve freight efficiency and vehicle travel times in the Onehunga, Penrose, Mt Wellington, Mangere, Otahuhu and East Tamaki areas of Auckland. Accommodation of the East West link will require coastal reclamation work along the north coastline of Mangere Inlet.

NIWA was contracted to develop a hydrodynamic and sediment transport model of Mangere Inlet, to model the relative change in tidal flow and sediment deposition following coastal reclamation along the inlet’s north shore.

This technical report presents details on the model development and calibration plus model results from a series of tide, wind and wave driven scenarios designed to establish the effects of coastal reclamation on tidal circulation, and sediment transport inside Mangere Inlet. The model scenarios are aimed to assess relative changes inside the inlet (compared to pre-reclamation) in coastal physical processes (hydrodynamic, wave and sediment transport) in the entrance and inside Mangere Inlet.

The model was calibrated and validated using available water level and current measurements. Skill levels (statistical agreement) were high between the observed and modelled water levels. The modelled currents were consistent with the gauging observations, reproducing the shape of the flood-ebb current hysteresis curve through the inlet.

The calibrated hydrodynamic model was run for spring and neap tidal ranges for pre- and post-reclamation coastlines and bathymetries. The main differences in tidal circulation caused by coastal reclamation were predicted adjacent to the reclaimed coastline. The series of new headlands, coves and embayments along the northern stretch of the new coastline act to reduce tidal current speeds and therefore, encouraged slightly more sediment deposition inside the inlet.

The model predicted the inlet to be flood-dominant (infilling). The estimated annual sedimentation rate, averaged over the whole of the existing inlet, was 9.8 mm/yr, which compares closely to an analysis using LIDAR data that estimated an annual sedimentation rate of 10 mm/yr.

The modelled mean annual sedimentation rate after reclamation was similar, being 10.5 mm/yr. The model predicted relative increases in sediment deposition along most of the northern stretch of reclaimed coastline, due to lower current speeds which have less sediment-carrying capacity. Further east along the reclamation there was more relative erosion that resulted from a localised change in the tidal circulation. Analysis of volumetric tidal fluxes in the east section on the inlet showed there was no overall change in the tidal exchange due to the coastal reclamation.

The effects of sea level rise were incorporated into modelling by assuming 1 m increase in sea-level and 0.5 m increase in seabed height due to sedimentation. Results from the modelling predicted a 10.4 mm/yr annual sedimentation rate.

Modelling that investigated the effect of dredging and removing approximately 300,000 m³ of bed material from the west of Mangere Inlet predicted a decrease in tidal current speeds through the entrance to the inlet. The modelling predicted decreases in tidal current speeds would increase sediment deposition to the north and south of the dredged area. The predicted annual sedimentation rate with both coastal reclamation and dredged area was 10.4 mm/yr.
The model was used to investigate the dispersion of sediment plumes discharged from three point sources during construction and coastal reclamation work in Mangere Inlet. The results from the modelling showed that sediment settled out of the water column almost immediately once a source was turned off. Most of the sediment deposited along the northern and eastern coast of the inlet.

The report includes a table of the expected frequency and magnitude of extreme wave heights in Mangere Inlet, in water depths of 0.5, 1.0 and 2.0 m.
1 Introduction

A new east west connection road link project aims to improve freight efficiency and vehicle travel times in the Onehunga, Penrose, Mt Wellington, Mangere, Otahuhu and East Tamaki areas of Auckland. The East West Link Alliance (NZTA, Beca, GHD, and Buddle Finlay) awarded NIWA the contract to develop a hydrodynamic and sediment transport model of Mangere Inlet where there will be coastal reclamation work along the north shore of the inlet (see Figure 1-1) required to accommodate the new road connection and coastal edge.

The numerical model was used to estimate changes in present day tidal circulation and sedimentation rates caused by the proposed coastal reclamation work inside Mangere Inlet.

An extreme-value analysis on wave height inside Mangere Inlet was performed for shallow water regions adjacent to the coast.

Figure 1-1: Google earth image of study area showing the geographical limits of the model, tidal boundary, model calibration sites and area of coastal reclamation. Image courtesy of Google Earth®
2 Model Applications

The hydrodynamic and sediment transport models developed for this project were used to predict:

1. Tidal flows in the inlet for existing bathymetry during neap tides.
2. Tidal flows in the inlet for existing bathymetry during spring tides.
3. Peak tidal flows in the inlet with coastal reclamation and dredging in place during neap tides.
4. Peak tidal flows in the inlet with coastal reclamation and dredging in place during spring tides.
5. Sedimentation rates over a single mean tide during calm, SW winds and NE winds for existing bathymetry.
6. Sedimentation rates over a single mean tide during calm, SW winds and NE winds with reclamation and dredging in place.
7. Sedimentation rates over a single mean tide during calm, SW winds and NE winds with reclamation and dredging in place and incorporating 100 year sea-level rise.
8. The dispersion of sediment plumes that are discharged from 3 single-point sources.

The study assumes interference of tidal flows around the piles/piers of Old Mangere and the SH20 bridges have no impact on wider scale sediment transport processes in Mangere Inlet. The piles/piers are localised features, and therefore flow (small) headloss near to the bridges only has a minor amount of scour around the piles/piers.

The pre- and post-reclamation model predictions of inlet tidal flows and sedimentation were then used to estimate relative changes in:

1. Inlet peak flood and ebb current speeds on a mean spring tidal range (3.80 m at Onehunga Wharf, LINZ CHART 4315).
2. Inlet peak flood and ebb current speeds on a mean neap tidal range (2.11 m at Onehunga Wharf, LINZ CHART 4315).
3. Inlet sedimentation rates over a single mean semidiurnal tide period during calm, SW winds and NE winds for existing bathymetry.
4. Inlet sedimentation rates over a single mean semidiurnal tide period during calm, SW winds and NE winds with reclamation in place.
5. Inlet sedimentation rates over a single mean semidiurnal tide period during calm, SW winds and NE winds with reclamation in place and 100 year sea-level rise.
6. Inlet sedimentation rates over a single mean semidiurnal tide period during calm, SW winds and NE winds with reclamation in place and mudcrete dredge volume removed.

The pre- and post-reclamation tidal period sedimentation rates were weighted and scaled up to provide estimates of annual sedimentation rates. These results were used to provide an estimate on the relative changes in sedimentation rates caused by the coastal reclamation.
3 Hydrodynamic, wave and sediment transport model development

3.1 Delft3D modelling Suite

Tidal dynamics and sediment transport in Mangere Inlet were modelled using the Deltares Delft3D hydrodynamic, wave and sediment transport modelling suite.

The curvi-linear, 2-dimensional (depth averaged) or 3-dimensional sigma coordinate (multi-layer) semi-implicit model suite finds numerical solutions for 2 and/or 3-dimensional flows (Delft3D-FLOW), wave–current interaction (Delft3D-WAVE), non-cohesive/cohesive sediment transport (Delft3D-SED) which can incorporate morphological evolution by scaling the hydrodynamic and sediment transport predictions (Delft3D-MOR) (see Appendix A).

Past studies by NIWA in Manukau Harbour have shown that the harbour is vertically well mixed. Therefore, for the present application in Mangere Inlet, Delft3D-FLOW was set up in 2-dimensional depth-averaged model and run in barotropic mode. This assumes that the inlets waters are generally vertically well-mixed. The model was used to investigate shallow water tidal dynamics and wave/current interaction and sediment transport in shallow water.

The model grid was developed from bathymetry surveys, navigation charts and LIDAR data. The model was calibrated using archive water level, tidal gauging and comparisons with an existing model of Manukau Harbour. The sedimentation rates predicted by the sediment transport model were compared to the patterns in sedimentation and bulk estimates of annual sedimentation rate estimated from LIDAR surveys of the inlet.

3.2 Model grid

The curvi-linear Delft-3D x-y and bathymetric grid (z) of Mangere Inlet was developed for pre-reclamation coastline in ArcGIS using data sourced from:

- Google Earth (coast line).
- LINZ Chart 4315 – Approaches to Onehunga.
- Port of Auckland survey data of inner Mangere Inlet (supplied by East-West Alliance).
- LIDAR – NIWA archive data originally supplied by Auckland City Council.

The grid-cell size in the model was about 64 m² within the inlet where most of reclamation will take place. The model grid was slightly coarser in the wider inlet and the approaches to Onehunga Bay. The coarser grid away from the main areas of earthworks and reclamation optimises model run times by allocating more computer time to the higher grid resolutions in the inner inlet where detail was required. The model grid was set up relative NZGD 2000 datum, and used Mount Eden Circuit 2000 horizontal coordinates. The model grid has 82,703 grid cells with highest density of cells along the north shore where the future reclamation work will be taking place (see Figure 3-1 and Figure 3-2).

The model’s open tidal boundary was set to the west in Manukau Harbour from Wattle Bay to the north and Puketutu Island to the south (see Figure 1-1 and Figure 3-1). The location of the open boundary enabled a full spring-tidal excursion and volume exchange to be simulated between inner Mangere Inlet and outer Onehunga Bay.
A high resolution digital terrain model (DTM) was developed in ArcGIS. This was imported into Deltares modelling suite bathymetry generator and interpolated onto the model grid. A bathymetric contour plot of the inlet is shown in Figure 3-3. This bathymetry and model grid was to be used to investigate pre-reclamation tidal circulation and sediment transport inside Mangere inlet. Model bathymetry was set to Onehunga (Auckland) vertical datum plus the 2016 mean sea-level offset.

A second modified bathymetry (Figure 3-4) and modified shoreline (provided by The East West Alliance) along the north shore of Mangere Inlet that incorporates the proposed reclamation was incorporated into the original model grid. This modified bathymetry was used in a series of simulations to investigate the relative changes (relative to pre-reclamation) in tidal circulation and sediment transport caused by the proposed shoreline reclamation work.

Figure 3-1: Model computational grid. Model is forced along the western most boundary with time series of tidal elevations or astronomical constants.

Figure 3-2: High resolution model grid developed for region of inlet where coastal reclamation and changes in coastline will be taking place.
The pre-reclamation area of Mangere Inlet is approximately 5.7 km². With reclamation in place this reduces to approximately 5.5 km², or a reduction in total area of 3.5%. Assuming a spring tidal range between 2.05 m to -1.75 m, Mangere Inlet has a pre-reclamation tidal prism volume of approximately $11.9 \times 10^6$ m³. The tidal prism with the proposed reclamation is reduced by approximately $4 \times 10^5$ m³ equating to a new tidal prism volume of approximately $11.5 \times 10^6$ m³.

Figure 3-3: Pre-reclamation model coastline and bathymetry developed from Ports of Auckland survey data, LINZ charts and LIDAR data.

Figure 3-4: Post-reclamation and bathymetric layout and coastline on the north shore of Mangere Inlet for reclamation version V04.
3.3 Tidal elevations, currents and boundary conditions

Tidal elevations and currents used in the model calibration/validation and pre- and post-reclamation scenarios were based on archive tidal data recorded at:

- A NIWA “S4” mooring site in Wairopa Channel in the approaches to Onehunga (Bell et al. 1998).
- Onehunga tide gauge records (Ports of Auckland).
- A tidal gauging site located on Old Mangere Bridge (Green and Bell, 1995).
- Numerical model results extracted from a larger calibrated DHI MIKE3-FM regional model of Manukau Harbour (Reeve and Pritchard, 2010).

The tidal boundary water levels for the model were synthesised from least-squares harmonic analysis of Onehunga water-level data because this was the most reliable and best quality controlled data set. However, because of the distance between the model boundary and Onehunga wharf, the phase of each tidal constituent had to be adjusted to compensate for the phase lag between the two sites. Calculations based on shallow water wave speed suggested there is an approximate 8 min phase lag at Onehunga as compared to the phase at the model tidal boundary. This was further verified by comparing phase values extracted from archive MIKE3-FM model runs with the phases at Onehunga wharf and phases predicted at the S4 site by the Delft3D model. The MIKE3-FM model is now not in use at NIWA, so could not be run for other time periods.

Analysis of tidal currents showed that the S4 mooring currents were poor quality and unsuitable for model calibration. However, sea-level elevations from the mooring were used for model calibration. The model was also calibrated against tidal gauging data from Green and Bell (1995). These data included measured current speed and direction plus tidal elevations at Old Mangere bridge over a full M2 (12.42 hr) tidal cycle.

3.4 Winds

Wind speeds were used in the model to generate both wind driven currents and local fetch-limited wind waves. Figure 3-5 shows a wind rose that summaries the time series of wind speed and direction recorded at Auckland Airport between 1980 and 2011. The rose shows the dominance of the prevailing SW wind and also the presence of NE tropical lows that pass through the region.

3.4.1 Effects of wind waves on sediment transport

Previous studies have shown that fetch-limited (locally generated) wind waves have an important impact on sediment transport processes in Manukau Harbour and Mangere Inlet (Green et al. 2000). Wind wave generated orbital velocities that ‘feel’ the bed in shallow water increase the total bed shear stress (through non-linear interaction with tidal currents) and consequently can increase the rate of sediment erosion/resuspension at the bed. This process is prevalent at high water on intertidal flats and on the edge of tidal channels where the waves tend to erode and ‘stir up’ the bed deposited sediments into suspension. This is often observed as ‘turbid fringe’ that on the ebb tide drains from the flats into the main sub-tidal channels and moves seaward (Green et al. 1997; Green et al. 2000). On the next flood tide the sediments sourced from the flats and now in suspension in the channels is transported and dispersed in a landward direction by the flood tidal currents and the cycle repeats.
We used a representative wind climate to create three wind scenarios, to simulate wind waves and their effect on long-term (annual) sedimentation rates. The winds were superimposed on the tides. Based on Figure 3-5 we assumed a bi-directional wind climate and modelled south westerly, north easterly and calm states. Winds blowing from the south west sector (125°–315°) and north east sector (315°–125°) were assigned either a south west (225°) or north east (45°) angle respectively.

Previous NIWA studies and shallow water sediment transport models developed for Kaipara Harbour and Hauraki Gulf (Pritchard et al., 2015) found that wind speeds below 7.5 m/s had little effect over and above that of tidal currents on sediment transport. When wind speeds exceeded 7.5 m/s waves become important. Therefore, for this study we assumed that wind speeds below 7.5 m/s are classed (in context of sediment transport) calm and above this wind speed, waves are then generated. The resulting percentage trend in the idealised wind wave climate for sediment transport is then:

- Calm – 80%
- SW – 15%
- NE – 5%

Wind speeds in the model were ramped up from zero to 7.5 m/s, sustained at 7.5 m/s for 72 hours and then ramped down to zero for all scenario simulations. Winds stronger than 7.5 m/s were not simulated – the assumption was made that 7.5 m/s represents the wave impacts for all wind speeds ≥ 7.5 m/s. This assumption appears reasonable given the apparent dominance of tidal flows on sediment transport and the good sediment transport calibration that was achieved, as shown in Section 5.2.1.
3.5 Sediments and sedimentation rates

The historical and present day sedimentation rates in Mangere Inlet have a range of reported values in the literature. These values vary from 0.01 mm/yr to 49 mm/yr (Croucher, 2005; Wilcock and Northcott, 1995). No present day coring data was available to verify any of the published rates. Therefore, in an attempt to gauge the recent sedimentation levels inside Mangere Inlet, a comparison was made of the intertidal seabed levels between the 2006 and 2013 LIDAR information. Inherent inaccuracies will lie within the LIDAR data and consequently the comparisons. Therefore to obtain some confidence in results, areas of the inlet where a comparison was possible were block averaged. From these estimates the overall average sedimentation rate throughout the entire Inlet was approximately 10mm/yr. Areas of mangroves and exposed intertidal flats had similar rates with an average sedimentation rate of 17mm/yr. In areas near tidal channels, in tidal channels and near islands the average erosion rate was 26 mm/yr. In general the northern coastline had higher sedimentation rates than the southern coastline (Stephen Priestley, BECA, pers. comm.).

Figure 3-5: Auckland Airport wind rose for the period 1980 to 2011.
3.6 Sediment transport model set up

3.6.1 Sediment transport model

The coupled hydrodynamic and sediment transport model was run with a mean tide (M2) boundary condition, with waves and without waves for a period of 30 days.

The cohesive sediment transport model was set up with a sediment particle settling velocity \( W_s \) of 0.7 m/hr (See Appendix E). The critical sediment deposition threshold \( \tau_{cd} \) and critical sediment erosion threshold \( \tau_{ce} \) were set at 0.125 N/m² and 0.25 N/m² based on literature values (e.g., Whitehouse et. al, 2000). The sediment and specific and dry bed densities were set at 2650 kg/m³ and 700 kg/m³ respectively.

Previous work has shown that Mangere Inlet is flood dominated (Green and Bell, 1995). Therefore, the inlet acts as sink for sediment imported from the Manukau Harbour. Based on these observations we initialised the model domain with no deposits of bed sediment. Else if the model was started with an ad hoc bed sediment distribution it could later bias the estimates of the relative changes in present day sediment deposition rate inside the inlet caused by the reclamation work.

Therefore, suspended sediments were introduced into the model domain from a series of 18 point sources in the main Wairopa Channel (see Figure 1-1). The sources mimic the sediment transport flux off the intertidal flats and into the main sub-tidal Wairopia Channel as observed by Green et al. (1997).

The model simulations that matched a modelled suspended sediment concentration (SSC) to a measured SSC of 40 mg/m³ at Mangere Bridge (Green and Bell, 1995) required a sediment input flux of 0.3 kg/s at each point source plus a source of legacy bed sediment from Onehunga Bay. To incorporate both legacy and instantaneous fluxes into simulations, sediment inputs into the model were started 2 tidal cycles after the model start up to eradicate any effects of hydrodynamic instability on the transport of the sediment in the model domain. The model was then allowed to run for a further 5-days to build up the supply of legacy bed sediment. The new sediments that were introduced into the model ‘evolve the bed’ through a morphological feedback into Delft3D-Flow and Delft3D-Wave by the model by Delft3D-MOR model.

No model wind climate scenarios were started in a model run until 6 days after the start up. This delay allows a build-up of legacy bed sediments that can be eroded when the scenario begins. This gives the model a physical reality where the existing bed deposition footprints inside the inlet have resulted from transport by tidal currents not an arbitrary allocated bed deposition layer. This gives better confidence in the assessments of relative changes in bed levels later performed using weighted model scenario predictions.

3.6.2 Prediction of inlet sedimentation rates for pre- and post-reclamation coastline and bathymetry

Annual sedimentation rate (ASR) inside Mangere Inlet for pre- and post-reclamation bathymetry were predicted from the relative change (+/-mm) in bed level that occurs between the start and the end of 1 complete M2 (12.42 hr) tidal cycle. The 30-day model simulation was setup for periods (after the 6-day start up period) for calm (tides only), SW and NE wind and wave conditions. The relative change in the bed level over a M2 tidal period for calm, SW and NE wind were then extracted from model results. The M2 tidal sedimentation rate was then scaled up to an approximate ASR through (1):

\[
\text{Annual sedimentation rate (ASR)} = \frac{\text{Relative change in bed level}}{\text{1 complete M2 tidal cycle}} \times \text{number of M2 tidal cycles in 30 days}
\]
ASR = Tidal sedimentation rate × 1.93 tides/day × 365 days \hspace{1cm} (1)

A composite (\(\text{ASR}_{\text{WDCLI}}\)) in the inlet based on annual wind climate is then estimated using the modelled ASR. Each separate modelled ASR for calm and winds was weighted by its percentage annual occurrence (see 3.4) and then all values summed together to produce a single wind climate composite ASR for the inlet. This can be summarised as equation (2):

\[
\text{ASR}_{\text{WDCLI}} = \text{ASR}_{\text{CALM}} 0.8 + \text{ASR}_{\text{SW}} 0.15 + \text{ASR}_{\text{NE}} 0.05
\]

(2)

The changes in ASR caused by reclamation inside the inlet were investigated through application of equation (2) where \(\Delta \text{ASR}_{\text{WDCLI}}\) gives a measure of the relative change in annual sedimentation (more or less erosion or deposition) due to coastal reclamation as compared to pre-reclamation \(\text{ASR}_{\text{WDCLI}}\). This can be expressed as equation (3):

\[
\Delta \text{ASR}_{\text{WDCLI}} = \text{ASR}_{\text{WDCLI-PRE}} - \text{ASR}_{\text{WDCLI-REC}}
\]

(3)
4 Model calibration and validation

The model was calibrated for:

- 2006 water levels observed at Onehunga and water levels predicted at the S4 site for same period during 2006.
- Tidal currents for the period that coincided with 1994 Old Mangere Bridge gauging experiment.
- Validated with DHI model modelled tidal elevations extracted from archived model results for the period during 1995.
- Regions of sedimentation and erosion were compared to the LIDAR estimates of erosion and sedimentation inside Mangere Inlet.

4.1 Calibration procedure

The Delft3d hydrodynamic model was calibrated by adjusting the two main calibration parameters:

- Horizontal Eddy Viscosity (Kx, Ky) – A time constant (but can be varied in space) parameter used to close the 2-d momentum equations that can be used to damp unexplained variance in modelled flows. Raising the values of Kx and Ky causes the modelled amplitude of maximum flows and tidal elevations to reduce.
- Chezy bed roughness formulation – A hydraulic bed roughness parameter used to formulate bottom boundary closure by parameterising energy dissipation at the seabed. This is often used to finely tune and calibrate current and tidal phase. However, correct specification of the bathymetry is the main determent of correct model phase.

A measure of ‘model skill’ or ‘best fit’ between the model predictions and observations and/or model predictions and predictions from tidal harmonic analyses of data was then determined through a series of basic statistical measures and tests. These are:

- RMSE – Root Mean Square Error - A measure of the difference in the variance between the observed and predicted signal.
- BIAS - The averaged signed residual offset between two time series. The bias indicates a positive/negative offset i.e., central tendencies in predicted and observed time series data.
- SKILL - where values span 1 (high) to 0 (poor) skill decreases towards zero as described by Warner et al. (2005) and Haidvogel et al. (2008).
- Rxy - Cross-correlation analysis – where 1 = 100 % correlation at zero phase lag is perfect correlation between two time series.

See Appendix B for the complete mathematical description of the above tests.

Model calibration parameters were iteratively changed until the best possible skill measures (indicating fit between observations and model predictions) were obtained. This process fine-tunes the model’s performance to minimise the error between the predictions and the observations.
4.2 Hydrodynamic model calibration and validation results

4.2.1 2006 Water levels

Modelled time series of tidal elevations were extracted from model domain at the locations of the S4 mooring site and Onehunga Wharf tide gauge. Figure C-1 (Appendix C) shows the results from the inter-comparison between modelled-observed and modelled-predicted. Results show close agreement and this is further verified through model skill test results presented in Table 4-1.

Table 4-1: Skill test results between observed 2006 water levels at Onehunga tide gauge and S4 mooring and Delft3D model predictions for the coincident 2006 period.

<table>
<thead>
<tr>
<th>Location</th>
<th>Skill score (%)</th>
<th>RMSE (m)</th>
<th>Bias (m)</th>
<th>Rxy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Onehunga Tide Gauge</td>
<td>0.99</td>
<td>0.08</td>
<td>0</td>
<td>0.98</td>
</tr>
<tr>
<td>S4</td>
<td>0.99</td>
<td>0.09</td>
<td>-0.03</td>
<td>0.99</td>
</tr>
</tbody>
</table>

4.2.2 1994 tidal gauging

The modelled trend in flood-ebb dominance of tidal currents is an important factor for the transport of sediment. This flood-ebb dominance is illustrated in more detail by Figure 4-1 that shows a comparison between results from the 1994 old Mangere Bridge tidal gauging experiment (Green and Bell, 1995) and the Delft-3D model predictions made at the same location for the same 1994 tide. Figure 4-1 shows the observed and modelled tidal hysteresis (tidal height vs tidal current speed) measured over a single spring-tidal cycle (approximately 12.42 hrs). The modelled currents are consistent with the gauging observations to the degree that can be expected given expected bathymetry differences between the model grid and the harbour and inlet channel shape in 1995.

![Figure 4-1: Observed (Green and Bell, 1995) and Delft3D modelled tidal hysteresis at Mangere Bridge during a tidal gauging experiment over a complete spring tidal cycle during 1994.](image-url)
Following Figure 4-1 in a clockwise direction both observations and model show that the highest current speeds are on the flood tide. Furthermore, the hysteresis curve on the flood cycle is more ‘peaky’ and distorted than hysteresis on the ebb. This suggests there are two processes here that can influence sediment transport:

1. Higher current speeds on the flood than on the ebb implies flood-biased tidal asymmetry. Therefore, this bias has the potential to promote a higher suspended sediment flux by the tide into the inlet than out of the inlet.

2. The peaky flood hysteresis shows that tidal currents accelerate faster on the flood than the ebb. Therefore, because sediment transport is proportional to the cube of velocity, this is an important factor in the erosion, resuspension and transport of bed sediments. Observations and model would suggest a flood tide bias in the transport of sediments.

4.2.3 1995 Model – model tidal height and current validation

Modelled time series of tidal elevations and currents were extracted from an archive MIKE3-FM Manukau Harbour model run for a period during 1995 at the locations of the S4 mooring site and the Onehunga Wharf tide gauge. The Delft3D model was run for this same 1995 period and results compared to the DHI model. High skill test scores on water levels and currents at the two sites shown in Table 4-2 show that the Delft3D model closely matched the MIKE3-FM model.

The skill test results shown in Table 4-3 indicate the Delft3D modelled water levels were a better match with MIKE3-FM than the modelled currents. This is a possible result from that current speed can vary over a very small scale due to localised bathymetric irregularities, such as reef outcrops and channel meanders, sea-surface elevations depend more on volume, which is less sensitive to small-scale irregularities in the bathymetry.

<table>
<thead>
<tr>
<th>Location</th>
<th>Skill score (%)</th>
<th>RMSE (m)</th>
<th>Bias (m)</th>
<th>Rxy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Onehunga tide gauge</td>
<td>0.99</td>
<td>0.03</td>
<td>0.01</td>
<td>0.98</td>
</tr>
<tr>
<td>S4</td>
<td>0.99</td>
<td>0.06</td>
<td>0.0</td>
<td>0.95</td>
</tr>
</tbody>
</table>

Table 4-2: Skill test results between DHI model 1995 water levels at Onehunga tide gauge and S4 site and Delft3D model predictions for the coincident 1995 period.
Table 4-3: Skill test results between DHI model 1995 currents at Old Mangere Bridge (OMB) and S4 site and Delft3D model predictions for the coincident 1995 period.

<table>
<thead>
<tr>
<th>Location</th>
<th>Cartesian component of velocity</th>
<th>Skill score (%)</th>
<th>RMSE (m/s)</th>
<th>Bias (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>OMB</td>
<td>u</td>
<td>0.99</td>
<td>0.07</td>
<td>0.01</td>
</tr>
<tr>
<td>OMB</td>
<td>v</td>
<td>0.89</td>
<td>0.02</td>
<td>-0.01</td>
</tr>
<tr>
<td>S4</td>
<td>u</td>
<td>0.99</td>
<td>0.11</td>
<td>0.01</td>
</tr>
<tr>
<td>S4</td>
<td>v</td>
<td>0.98</td>
<td>0.01</td>
<td>0</td>
</tr>
</tbody>
</table>
5  Model results

5.1  Tidal current flows

5.1.1  Pre-reclamation tidal circulation

The simulated peak flood and ebb current speeds and directions for a mean spring tide and mean neap tide ranges for the approaches to Onehunga and inside Mangere inlet are presented in Figure 5-1 to Figure 5-4. Peak flood and ebb currents occurred two-hours before local high and low water (respectively) at Onehunga Wharf.

At local low water (LW), currents are slack but as the tide begins to flood current speeds increase and flow towards the east. The tidal currents continue to accelerate through to the mid-flood and, 2 hours before high water, reach peak speeds of over 1 m/s (2 knots) in the main flood channels of Onehunga Bay and through the Mangere inlet tidal channel. Fast currents occur as the water squeezes through the narrow inlet entrance. Inside Mangere inlet, the currents decelerate as the inlet widens out. After local HW, the tide turns and begins to flow towards the west and current speeds peak 2 hours before low water. Notably the model predicts that tidal current speeds in the entrance to Mangere Inlet are slightly slower on the ebb than on the flood. Similar circulation patterns occur on neap tides, although neap current speeds are approximately half the speed of spring tidal current speeds.

Figure 5-1: Model predicted peak flood current speed and direction on a spring tidal cycle in Mangere Inlet. HW-2 = 2-hours before high water.
Figure 5-2: Model predicted peak ebb current speed and direction on a spring tidal cycle in Mangere Inlet. LW-2 = 2-hours before low water.

Figure 5-3: Model predicted peak flood current speed and direction on a neap tidal cycle in Mangere Inlet. HW-2 = 2-hours before high water.
5.1.2 Predicted changes in tidal circulation due to reclamation

The predicted difference in tidal current speed due to the coastal reclamation is shown for mid-flood and mid-ebb flows for both spring and neap tidal ranges in Figure 5-5 to Figure 5-8. The four plots highlight where there is a predicted increase (yellow to red) or decrease (light blue to dark blue) in peak current speeds due to coastal reclamation. The analysis shows the same general regional changes in peak current speeds for both spring and neap tidal ranges, and for both ebb and flood currents. Due to reclamation there is a small decrease in peak current speeds in the inlet entrance and along the north shore where the reclaimed coastline slows the alongshore tidal flows. The largest decreases in peak current speed are inside the new coves and small embayments of the reclamation. The most striking feature shown in this analysis is decrease in current speeds adjacent the reclamation (in the north of the inlet), and an increase in current speeds south of the reclamation. This is a result of diverting tidal flow from an existing tidal channel (that is infilled during reclamation, see Figure 3-3 and Figure 3-4) to the south into a newly dredged channel that will link the NE corner of the inlet to the main tidal channel.
Figure 5-5: Difference in peak flood current speed due to proposed coastal reclamation in Mangere Inlet during a spring tide.

Figure 5-6: Difference in peak ebb current speed due to proposed coastal reclamation in Mangere Inlet during a spring tide.
Figure 5-7: Difference in peak flood current speed due to proposed coastal reclamation in Mangere inlet during a neap tide.

Figure 5-8: Difference in peak ebb current speed due to proposed coastal reclamation in Mangere inlet during a neap tide.
5.2 Sediment transport: pre- and post-reclamation

The modelled physical and sediment transport processes over a tidal cycle in the main tidal channel at Old Mangere Bridge is shown in Figure 5-9. Bed shear stress ($\tau$) was computed using equation (4):

$$\tau = C_D \rho U^2$$

Where $C_D = 0.0025$ and $\rho = 1026 \text{ kg/m}^3$ and $U$ is current speed. Figure 5-9 shows that the magnitude of $\tau$ is nearly double on the flood than that predicted on the ebb. This flood-ebb asymmetry is also shown in predicted SSC at the site which, on the flood are the same order of magnitude as those measured by Auckland Council surveys (Stephen Priestley, BECA, pers. comm.) The modelled peak in SSC was lower than that observed at the site during highly turbid conditions (Green and Bell, 1995; Williamson et al. 1996), but had the same relative flood-ebb trend in SSC. The SSC predictions also show a higher secondary peak on ebb when the effects of wind waves are incorporated into the simulations. This results from higher rates of sediment resuspension and reworking inside the inlet due to higher bed shear stresses due to wave-current interaction – the reworked sediments are then transported through the tidal channel. Predicted sediment deposition and resuspension at the Old Mangere Bridge site is higher during wave events than during calm conditions.

![Figure 5-9: Time series of modelled water levels ($\eta$), bed shear stress ($\tau$), SSC and sediment deposition at the Old Mangere Bridge site for calm, SW and NE wind wave conditions.](image-url)
5.2.1 Predicted inlet sedimentation rates for pre-reclamation bathymetry

Figure 5-10 to Figure 5-12 illustrate the simulated annual sedimentation rate (ASR) from equation (1) inside Mangere Inlet for pre-reclamation bathymetry during calm (tides only), SW and NE wind and wave conditions.

Figure 5-10 shows that the Mangere inlet is mostly depositional during calm conditions, with exception of the scoured (eroded) fan at the entrance and on the flanks of the inlet. Inclusion of wind and waves into the simulations tends to increase downwind scour (erosion) of sediment especially on the NE and SW edges of the inlet. Furthermore, the wind/waves increased the deposition rate in the tidal channels (Figure 5-11 and Figure 5-12) because wave-scoured suspended sediment off the intertidal flats drains on the ebb into the tidal channel. These channel sediment deposits are ultimately reworked by tidal currents.

The result composite ASR\text{WDCLI} from equation (2) (Figure 5-13) is similar to the calm (tide only) simulation with subtle changes only.

![Figure 5-10: Predicted ASR\text{CALM} (mm/yr) in Mangere Inlet for calm (tide only) conditions for pre-reclamation bathymetry using equation (1).](image)

3.0 Modelling the effects of coastal reclamation on tidal currents and sedimentation within Mangere Inlet
Figure 5-11: Predicted ASR$_{sw}$ (mm/yr) in Mangere Inlet for SW wind (tide and waves) conditions for pre-reclamation bathymetry using equation (1).
32 Modelling the effects of coastal reclamation on tidal currents and sedimentation within Mangere Inlet

Figure 5-12: Predicted ASR_{NE} (mm/yr) in Mangere Inlet for NE wind (tide and waves) conditions for pre-reclamation bathymetry using equation (1).

Figure 5-13: Predicted wind climate composite ASR_{WDCLI} (mm/yr) in Mangere Inlet from the weighted wind climate using equation (2) for pre-reclamation bathymetry.

Figure 5-14: Predicted ASR rate (mm/yr) in Mangere Inlet from LIDAR data analysis (see Section 3.5).
The comparison of modelled ASR\textsubscript{WDCLI} shown in Figure 5-13 with the results from analysis of the LIDAR data (Figure 5-14) shows the model makes a good job of predicting morphological features and ASR throughout the inlet. Most of the inlets shallows (i.e., on the north shore where the proposed reclamation will take place) and inter-tidal regions were predicted as depositional. In contrast in deeper water through the inlet entrance, a bifurcated fan shaped scoured region extends down to the south east and to east where it then couples with a network of sub-tidal channels. The channels are erosional being scoured out by mainly tidal currents (see Figure 5-10).

5.2.2 Predicted sedimentation rates due to coastal reclamation

The Mangere Inlet sediment transport model was run with same boundary, sediment source and wind conditions as the pre-reclamation scenarios but with the modified bathymetry and coastline as shown in Figure 3-4.

The changes in ASR caused by reclamation inside the inlet were investigated through application of equation (2) and equation (3) where $\Delta$ASR\textsubscript{WDCLI} gives a measure of the relative change in annual sedimentation.

**Figure 5-15:** Predicted wind climate composite ASR\textsubscript{WDCLI} (mm/yr) in Mangere Inlet from the weighted wind climate using equation (2) for V04 post-reclamation coastline and bathymetry.

Figure 5-15 and Figure 5-16 displays results from equation (2) and equation (3) respectively. The results show that reclamation promotes an increase in sedimentation rates along most of the coastline where coastal reclamation will take place. The new embayments, coves and headlands along this coast act as sediment traps and decrease tidal flows (see Figure 5-5 to Figure 5-8). This causes a localised increase in ASR of approximately 3-5 mm/yr. The sharp divergence between an increase in accretion (+5mm/yr) and erosion (-5mm/yr) on the east of the reclamation is caused by changes in tidal flows due to the effects of reclamation and the infilling of a sub-tidal channel.
The implication of the reclamation work on tidal volumetric transport in the eastern end of the inlet was investigated by extracting transport rates through the 2 cross-sections shown in Figure 5-16.

The resultant cumulative volume (V) and transport rate (Q) for pre- and post-reclamation bathymetric layouts and coastlines over NE (Figure 5-17) and N2S (Figure 5-18) cross-sections shows that there were no notable changes in tidal exchange due to coastal reclamation.
Figure 5-17: Predicted tidal cumulative volumetric transport (V) and instantaneous volumetric flux (Q) over x-section N2S (see Figure 5-16) for present day (V0) and reclamation (V4) coastlines.
5.3 Predicted area mean ASR in Mangere Inlet pre- and post-coastal reclamation

The mean ASR over the area of Mangere Inlet as shown in Figure 5-13 was estimated for pre-reclamation conditions as $9.8 \text{ mm/yr}$ and coastal with reclamation in place $10.5 \text{ mm/yr}$. This is an increase of $0.7 \text{ mm/yr}$.
6 Predicted inlet sedimentation rates and sea level rise

The effect of global sea-level rise (SLR) on Mangere Inlet’s sedimentation rates with the coastal reclamation in place were investigated by modelling a 1 m increase in mean sea level and 0.5 m of sediment deposition inside the inlet (scenario agreed with Stephen Priestley of BECA). A SLR of 1 m over the next 100 years is a commonly-applied SLR allowance in New Zealand, and was applied in the Proposed Auckland Unitary Plan. 0.5 m of seabed deposition is an approximation based on 5 mm/year over a 100-year period. The 0.5 m deposition was applied uniformly over the existing bathymetry.

The simulations used in this investigation were set up with coastal reclamation in place, with and without SLR. The series of sediment transport simulations were run, and equation (2) was used to estimate ASR rates inside the inlet. Figure 6-1 shows the predicted ASR\textsubscript{WDCLI} (equation (3)) that results due to SLR.

The effect of SLR and reclamation (Figure 6-1) compared to reclamation only (Figure 5-16) predicts the centre of the inlet becomes more erosional and the edges of the inlet more depositional. These changes occur because of the increase in water depth due to SLR. SLR exposes pre-SLR intertidal regions of the central inlet to permanent submergence or more frequent tidal inundation. Therefore, the central inlet becomes more susceptible to erosion and ‘off flat’ transport by the tidal currents and waves. Similarly, greater water depths in the south of the inlet increases tidal inundation time and tidal excursion higher up onto the intertidal flats. This encourages more ‘on flat’ sediment transport in the south of the inlet.

![Figure 6-1: Predicted changes in wind climate composite ASR\textsubscript{WDCLI} (mm/yr) caused by proposed coastal reclamation in Mangere Inlet using equation (3) and sea-level rise. Differences in modelled pre- and post-reclamation ΔASR\textsubscript{WDCLI} ± 0.5 mm/yr are blanked out.](image)

Modelling the effects of coastal reclamation on tidal currents and sedimentation within Mangere Inlet
6.1 Predicted mean ASR in Mangere Inlet with reclamation and SLR

The predicted mean ASR for Mangere Inlet with coastal reclamation and SLR is 10.4 mm/yr, compared to 10.5 mm/yr without SLR.
7 Effects of mudcrete dredging in entrance channel to Mangere Inlet

The coastal reclamation work in Mangere Inlet will require approximately 300,000 m³ of dredged mud to produce the mudcrete (a mix of sand and concrete/cement and marine mud) required in the construction of the new coastline. It is proposed that the mud will be dredged from the area shown in Figure 7-1. This area was selected on the basis that it avoids ecologically sensitive areas to the east of the dredge area but also removes an area of the invasive Asian date mussels inside the area of the dredge area.

The simulations used in this investigation were set up with coastal reclamation in place, with and without the dredged volume. The series of sediment transport simulations were run using the equation (2) wind climatology weighting methodology to estimate ASR rates inside the inlet. Figure 7-2 shows the predicted $\Delta ASR_{WDU}$ (equation (3)) that results due to the dredged volume removed.

Removal of the dredged volume has most impact on ASR in the entrance to the inlet. Figure 7-3 and Figure 7-4 show that the dredge area reduces current speed both on the flood and ebb tide. However, the current speeds and bed shear stress $\tau$ do not drop below the critical bed erosion threshold $\tau_r$ in the subtidal channel, therefore, sediment is still scoured in the tidal channel and in the main area of the dredged area. On the shallower flanks of the dredge where there are already lower current speeds, the further reduction in current speed caused by the dredge is enough to cause $\tau$ to drop below $\tau_r$ and increase the rate of sediment deposition as shown in Figure 7-2.

It is expected that over time the sediment deposition (and bed level) will eventually reach an equilibrium depth where the bed becomes susceptible to erosion by waves on the flood tide. The sediment will then be then transported off the flanks on the falling tide (as a turbid fringe) and back into the dredged area (e.g., Green and Bell, 1995).

Figure 7-1: Proposed area of the dredging (black broken line) of bed material to be used for mudcrete in the coastal reclamation. Three locations of point sources of sediment plumes are also shown (see Section 8).
Figure 7-2: Predicted changes in wind climate composite $\text{ASR}_{\text{wocu}}$ (mm/yr) caused by dredging operations in Mangere Inlet with proposed coastal reclamation in place based equation (3). Differences in modelled pre- and post-dredging $\text{ASR}_{\text{wocu}} \pm 0.5$ mm/yr are blanked out.
7.1 Predicted mean ASR in Mangere Inlet with coastal reclamation and mudcrete dredge

The predicted mean ASR for Mangere Inlet with coastal reclamation and mudcrete dredge in place is **10.4 mm/yr**.
8  Point source discharge of suspended sediment plumes

During the coastal reclamation work inside Mangere Inlet, dredging and earthworks will generate suspended sediment plumes discharged from point sources. These plumes will be dispersed by the tide and wind and will ultimately deposit sediment at locations around the inlet.

The aim of this investigation is to model a point source input of 1kg/s of native sediment (50 percentile settling velocity of about 0.7m/hr) for a 10 hour period every 24 hours. The results from the model simulations will be used to determine:

- The rate SSC decays away with distance from each point source.
- The location of sediment deposits discharged from each source.

8.1  Suspended sediment plume modelling – model set up

The location, water depth and discharge rate from 3 point source discharges inside the inlet are shown in Table 8-1 and on Figure 7-1.

<table>
<thead>
<tr>
<th>Source</th>
<th>Easting (m)</th>
<th>Northing (m)</th>
<th>Water depth (m)</th>
<th>Discharge rate (kg/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>402591</td>
<td>794029</td>
<td>3.3</td>
<td>1</td>
</tr>
<tr>
<td>S2</td>
<td>402723</td>
<td>794194</td>
<td>1.7</td>
<td>1</td>
</tr>
<tr>
<td>S3</td>
<td>405023</td>
<td>794330</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

The approach to plume modelling assumes each source will separately discharge a suspended fine sediment flux at a rate of 1 kg/s for a 10 hour period in a day. This equates to a total of 36 tonnes of sediment. The water in the sediment from each source is assumed saline and the receiving water column throughout the inlet is also saline and well mixed. Each model scenario was started at local low water and ran with a mean (M2) tide and a 10 hour discharge of suspended sediment. Simulations were run separately for each of calm, and 7.5 m/s SW and NE wind conditions simulations.

The predicted SSC at sites 100 m and 200 m east and west of the 3 sources were extracted from the model and plotted as a 24 hour time series. The corresponding daily depositional rate for each wind scenario was also extracted from the model 24 hrs after the initial start of discharge i.e., 14 hours after the discharge is turned off, and spatially mapped inside the inlet.
Figure 8-1: Predicted water column SSC at Source 1 (3.3 m) and 100 m and 200 m east and west of source location during calm, SW and NE winds.

Figure 8-1 shows the predicted SSC time series during calm, SW and NE wind conditions for a Source 1 plume discharge (see Figure 7-1). The highest SSC values were predicted at the location of the source input and the SSC quickly decayed with distance from the source for all wind conditions. The highest overall SSC was predicted during SW winds reaching SSC of 180 g/m³ because of the SW wind forcing on an eastward flowing flood tide. Once the discharge was terminated, SSC at all locations dropped to zero.

Figure 8-2 shows the predicted locality and daily rate of sediment deposition for a Source 1 sediment plume that is discharged during calm, SW and NE wind conditions. The predicted highest rate of bed deposition was always close to the source but the extent of the plume spreading depended on the wind. During calm and NE winds, plume deposits end up on the intertidal flats to the north east of the inlet and also to the west in the throat of inlet entrance. During a SW wind wave action reduces sedimentation on the intertidal flats and so most sediment is deposited close to the source and in the deeper sub-tidal channels on the north east of the inlet.
Figure 8-2: Source 1 suspended sediment plumes daily rate of bed deposition.

Modelling the effects of coastal reclamation on tidal currents and sedimentation within Mangere Inlet
8.1.2 Suspended sediment plume – Source 2

Figure 8-3 shows the predicted SSC time series during calm, SW and NE wind conditions for a Source 2 plume discharge. The peaks in the SSC time series largely reflect the effects to tidal advection that is transporting sediments up and down the main inlet channel. The highest single peak of approximately 240 g/m³ is close to the discharge source and results from the effects of ‘wind on tide’ at the start of the sediment discharge. After the initial release peaks in SSC drop down to approximately 50 g/m³ for all conditions. Once sediment Source 2 is terminated SSC decreases towards zero.

Figure 8-4 shows the predicted locality and daily rate of sediment deposition for a Source 2 sediment plume that is discharged during calm, SW and NE wind conditions. The highest rate of bed deposition was predicted close to the source for all wind conditions. Low levels of sediment deposition (<20 um) were predicted all along the northern coast of the inlet, on the north eastern intertidal flats and some small deposits in the inlet entrance. During a SW wind sediment also deposited along the northern coast of the inlet but the deposits to the north east were mainly confined to the subtidal channels.
Figure 8-4: Source 2 suspended sediment plumes daily rate of bed deposition.
8.1.3 Suspended sediment plume – Source 3

Figure 8-5: Predicted water column SSC at Source 2 (1.0 m) and 100 m and 200 m east and west of source location during calm, SW and NE winds.

Figure 8-5 shows the predicted SSC time series during calm, SW and NE wind conditions for a Source 3 plume discharge. High levels of SSC that approach 1000g/m³ were predicted to the west of the source for all wind conditions. In contrast, close to the source and east of the source SSC levels were relatively low.

Figure 8-6 shows through the predicted locality and daily rate of sediment deposition that there is relatively thick sediment deposition to the east of the source on the north eastern intertidal flat during calm and NE winds. Sediment that deposits is trapped on the flat and does not resuspend as reflected by the SSC (Figure 8-5). The high rates of bed deposition on the north eastern intertidal flats coincide with calm and NE winds when there is either no wind (and no waves) or wave height is fetch limited (NE). During a SW wind, less sediment deposits on the north eastern flat because of wave driven resuspension – consequently most of the sediment deposits in the tidal channels only.
Figure 8-6: Source 3 suspended sediment plumes daily rate of bed deposition.
Wave climate and extreme wave analysis inside Mangere inlet

The wind wave climatology (1980-2011) inside Mangere Inlet was estimated for nearshore water depths (assuming near to the area of coast where reclamation will take place) of 50 cm, 1 m and 2 m. Significant wave heights (Hs) at these water depth were estimated from the wind data using the empirical formula of Young and Verhagen (1996). The formula estimates the growth of fetch limited waves in water of finite depth from measurements of wind speed and fetch length.

The wave height computations were made using a time series of wind speeds approaching from either the southwest or northeast direction, as described in Section 3.4. Winds blowing from the southwest sector (125–315 °T) and northeast sector (315–125 °T) were assigned either a southwest (225 °T) or northeast (45 °T) angle respectively. A fetch distance of 3 km was used, which approximates the maximum span across the basin in the northeast–southwest direction. The results were insensitive to the relatively short reduction in fetch length caused by the coastline reclamation work.

9.1 Extreme value analysis

An extreme value model commonly applied to analyse extreme wave heights is the generalised Pareto distribution (GPD). This is fitted to independent data peaks that exceed a given high threshold (known as peaks over threshold, or POT).

9.1.1 GPD fitting to significant wave heights (H_{sig}) over a threshold

The GPD/POT method was used to estimate the extreme wave height frequency–magnitude distribution from the predictions of Hs at each water depth. The POT method was applied to the hourly Hs data, and involved separating the time series into independent wave peaks that were at least 3 days apart and above a specified threshold. A generalised Pareto distribution (GPD) was then fitted to the peaks, and used to estimate the frequency–magnitude distribution.

In summary the POT approach is:

1. With various wave height thresholds, calculate the shape and scale parameters of a GPD fitted to wave heights above that threshold. Plot the variability of the shape and scale parameters against varied threshold.

2. At each site, choose the lowest threshold for which the shape parameter approximately stabilises. The scale parameter should have an approximately linear relationship with wave height at this threshold.

3. Using a 3 day window to separate the wave peaks into independent events, the peak wave heights above the chosen threshold were extracted from each window.

4. The number of observations above the threshold per year (S_y) were determined, where S_y=total number of peaks/record length (years).

5. Fit a GPD to the wave height peaks, and determine the annual exceedance probability (AEP) of the projected significant wave heights.
9.2 Results of extreme value analysis of $H_{\text{sig}}$

The results from the extreme analysis of $H_{\text{sig}}$ at 3 different water depth are presented in Table 9-1 and Figure 9-1. The results show the typical increase in $H_s$ with an increasing recurrence interval and an increasing $H_{\text{sig}}$ with increasing water depth.

Table 9-1: Maximum predicted significant wave height ($H_{\text{sig}}$) for a range of water depth and ARI (years).

<table>
<thead>
<tr>
<th>ARI (yr)</th>
<th>Water depth (m)</th>
<th>2</th>
<th>5</th>
<th>10</th>
<th>20</th>
<th>50</th>
<th>100</th>
<th>200</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>0.5</td>
<td>0.27</td>
<td>0.29</td>
<td>0.29</td>
<td>0.30</td>
<td>0.30</td>
<td>0.31</td>
<td>0.31</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>0.36</td>
<td>0.37</td>
<td>0.38</td>
<td>0.39</td>
<td>0.40</td>
<td>0.40</td>
<td>0.41</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.41</td>
<td>0.43</td>
<td>0.44</td>
<td>0.45</td>
<td>0.46</td>
<td>0.46</td>
<td>0.41</td>
</tr>
</tbody>
</table>

Figure 9-1: Frequency–magnitude distribution of significant wave height ($H_{\text{sig}}$) at 3 nearshore water depths inside Mangere Inlet.
10 Conclusions

This study developed and applied a hydrodynamic, wave and sediment transport model of Mangere Inlet, to model the relative change in tidal flow and sediment deposition following coastal reclamation along the inlet’s north shore.

The main differences in tidal circulation caused by coastal reclamation were predicted adjacent to the reclaimed coastline. The series of new coves and embayments along the western stretch of the new coastline act to reduce tidal current speeds. The most notable change in relative current speeds was predicted on the eastern side on the reclamation where the reclamation will infill a tidal channel.

The estimated inlet mean annual sedimentation rate, averaged over the existing inlet, was 9.8 mm/yr. The mean annual sedimentation rate after reclamation was similar, being 10.5 mm/yr. This is an increase in sedimentation of 0.7 mm/yr. The model predicted relative increases in sediment deposition along most of the western stretch of reclaimed coastline, due to the lower current speeds which have less sediment-carrying capacity. Further east along the reclamation there was more relative erosion that resulted from a change in the tidal current speeds and circulation.

The effects of sea-level rise were incorporated into modelling by assuming 1 m increase in sea-level and 0.5 m increase in seabed height due to sedimentation. Results from the modelling predicted a 10.4 mm/yr annual sedimentation rate after sea-level rise.

Modelling that investigated the effect of dredging and removing approximately 300,000 m³ of bed material from the west of Mangere Inlet predicted a decrease in tidal current speeds through the entrance to the inlet. The modelling predicted decreases in tidal current speeds would increase sediment deposition to the north and south of the dredged area. The predicted annual sedimentation rate with both coastal reclamation and dredged area was 10.4 mm/yr.

The model was used to investigate the dispersion of sediment plumes discharged from three point sources during construction and coastal reclamation work in Mangere Inlet. The results from the modelling showed that sediment settled out of the water column almost immediately once a source was turned off. Most of the sediment deposited along the northern and eastern coast of the inlet.

The report includes a table of the expected frequency and magnitude of extreme wave heights in Mangere Inlet, in water depths of 0.5, 1.0 and 2.0 m.

11 Acknowledgements

East-west Alliance provided bathymetric survey data, new coastline projections and physical characteristics and properties of the sediments. Archived current-meter and tidal gauging data were supplied by NIWA (via Rob Bell). Ports of Auckland provided tidal height records at Onehunga Wharf.
12 References


Appendix A  Deltares Delft3D modelling suite

Delft3D-FLOW
Delft3D-FLOW solves the Navier-Stokes equations for momentum whilst conserving mass through the principle of continuity (Deltares, 2011). Physical processes in the model can be parameterised and simulated through specifying for example, eddy scales, turbulent-closure schemes, surface and bottom boundary conditions, surface winds and pressure fields, wave-current interaction, surface heating, salinity & temperature structure and the earth’s rotational effects.

The Delft3D-FLOW model can be forced at open and source input boundaries by oceanic tides, freshwater and heat sources. These forcing mechanisms produce the essential boundary physics required to simulate barotropic (surface-pressure gradients) and baroclinic (internal pressure gradients driven by horizontal and vertical water-density gradients) in the model domain which allow variation in seawater density to be included in model solutions. The superimposed effect of currents and waves on the bed shear stress is taken into account by means of the current-wave interaction model of Fredsøe (1984).

Delft3D-WAVE
Within the Delft3D modelling suite, the wave module Delft3D-WAVE simulates the evolution of random, short-crested wind-generated waves in estuaries and tidal inlets and based on the third-generation Simulating WAves Nearshore or SWAN model (see Booij et al. 1999; Ris et al. 1999).

The SWAN model is a spectral wave model intended for shallow water applications in coastal and estuarine environments (Booij et al. 1999; Ris et al. 1999). It computes the evolution of the wave energy spectrum in position (x,y) and time (t), explicitly taking into account the various physical processes acting on waves in shallow water. These include the effects of refraction by currents and bottom variation, and the processes of wind generation, white-capping, bottom friction, quadruplet wave-wave interactions, triad wave-wave interactions and depth-induced breaking. The model can incorporate boundary conditions representing waves arriving from outside the model domain.

For all model simulations of sediment transport predictions, Delft3D-FLOW was ‘online’ coupled with Delft3D-WAVE which has a two way wave-current interaction i.e., the effect of flow on the waves (via set-up, current refraction and enhanced bottom friction) and the effect of waves on current (via forcing, enhanced turbulence and enhanced bed shear stress).

Delft3D-SED
The transport of scalars i.e., salinity and temperature (density), and sediments, is solved using a coupled three-dimensional advection-diffusion equation. Horizontal and vertical variations in density in the model caused by, for example, freshwater from rivers, are initially transported and mixed by the coupled advection and diffusion (A-D) equation. The model solution assumes the Boussinesq approximation where the variations in the density are only dynamically accounted for in the pressure terms of the Navier-Stokes equation. In the Delft3d vertical sigma co-ordinate system, this reduces the immediate effects of buoyancy on vertical flows by assuming this is taken into account through the effects of the horizontal pressure gradient and vertical turbulent closure scheme (Deltares, 2011).
Cohesive suspended sediments are transported by the same A-D transport methodology where suspensions are advected by the flow and diffused vertically by turbulent mixing. In addition, sediment fractions have a specified settling velocity (related to particle size via the Stokes settling equation), bed power of erosion, a critical bed erosion threshold and critical deposition threshold (related to bed shear stresses at the bottom boundary layer) that may be varied through the model domain. The velocities used in these computations are provided by the flow fields predicted by Delft3D flow and if coupled, Delft3D-WAVE.

Sediment sample analysis by BECA showed the majority of bed sediments in Mangere Inlet were in the size fraction of muds/ fine silts. Therefore, all sediment transport simulations in this study used the Delft3d cohesive sediment transport model.

**Delft3D-MOR**

The Delft3D–MOR module applies sediment transport formulae (both suspended and/or bed total load) to estimate the morphological change through the model domain (Lesser et al. 2004). Throughout a simulation, elevation of the sea-bed is dynamically updated at each computational time-step by computing the change in the mass of bed material that has occurred as a result of the sediment sink and source terms and the sediment-transport gradients. The mass is then translated into a bed level change based on the dry-bed densities of the sediment fraction (mud in the case of Mangere Inlet). This means that subsequent hydrodynamic calculations are always carried out using the continually-updated bathymetry as the sea-bed erodes or accretes.
Appendix B  
Skill tests

Bias

\[ Bias = \frac{1}{n} \sum_{i=1}^{n} (y_i - x_i) \]

Where: \( x_i \) is the \( i^{th} \) modelled value, \( y_i \) the \( i^{th} \) measured value, and \( n \) the number of values being compared.

Root Mean Square Error (RMSE)

\[ RMSE = \sqrt{\frac{1}{n} \sum_{i=1}^{n} (y_i - x_i)^2} \]

Where: \( x_i \) is the \( i^{th} \) prediction and \( y_i \) the \( i^{th} \) true value and \( n \) the number of values being compared.

Cross Correlation Function (\( R_{xy} \))

Cross correlation function (\( R_{xy} \)) is computed from the cross-covariance function:

\[ C_{xy}(\tau) = E[ (y(t) - \mu_y)(x(t+\tau) - \mu_x) ] \]

Where: \( C_{xy} \) is the cross-covariance function, \( E \) is the expected value, \( x(t) \) and \( y(t) \) are discrete variables at time \( t \), \( \mu_y \) and \( \mu_x \) are means of the two time series, and \( \tau \) is the time lag between them.

The cross correlation function (\( R_{xy} \)) is a non-dimensional summary of this analysis which ranges from 0 to 1, where 1 infers a strong in-phase agreement between the two signals.

\[ R_{xy} \equiv \frac{C_{xy}(\tau)}{\sigma_x \sigma_y} \]

Where: \( \sigma_x \), \( \sigma_y \) are the standard deviations of each time series.

Measures of model performance were bias (\( BIAS \)), root mean square error (\( RMSE \)) and model skill (\( SKILL \)).

\( SKILL \) is defined as:

\[ SKILL = 1 - \frac{||X_m - X_o||^2}{\sum_{i=1}^{N}(|X_{mi} - \bar{X}_m| + |X_{oi} - \bar{X}_o|)^2} \]  \hspace{1cm} (1)

(Warner et al. 2005; Haidvogel et al. 2008) where \( X \) is the variable (in our case, either water level, current speed or current direction), \( \bar{X} \) is the time average of \( X \), the subscripts \( m \) and \( o \) denote model and observed values respectively, \( i \) is \( i^{th} \) value, and \( 0 \leq SKILL \leq 1 \).
Appendix C  Tidal Calibration

Figure C-1: Water level calibration at Onehunga Wharf and the NIWA S4 mooring site for 2006.
Figure C-2: Model current velocity validation at Onehunga site for 1995 period. The comparison is between the present model (Delft), and the DHI model applied by Reeve and Pritchard (2010).
Figure C-3: Model current velocity validation at NIWA S4 mooring site for 1995 period. The comparison is between the present model (Delft), and the DHI model applied by Reeve and Pritchard (2010).
Figure C-4: Model water level validation for locations at Onehunga and NIWA S4 mooring site. The comparison is between the present model (Delft), and the DHI model applied by Reeve and Pritchard (2010).
Appendix D  Reclamation Scenarios

The East-West Alliance provided 5 versions coastal reclamation to NIWA for modelling. Through this iterative process, the Alliance selected version 4 (V04) as the preferred new coastaline.

Reclamation Version V01

Figure D-1: Difference in peak flood current speed due to coastal reclamation design V01 in Mangere Inlet during a mean tide.

Figure D-2: Difference in peak ebb current speed due to coastal reclamation design V01 in Mangere Inlet during a mean tide.
Figure D-3: Predicted changes in wind climate composite $ASR_{WDCU}$ (mm/yr) caused by coastal reclamation design V01 in Mangere Inlet using equation (3). Differences in modelled pre- and post-reclamation $ASR_{WDCU}$ ± 0.5 mm/yr are blanked out.
Modelling the effects of coastal reclamation on tidal currents and sedimentation within Mangere Inlet

Reclamation Version V02

Figure D-4: Difference in peak flood current speed due to coastal reclamation design V02 in Mangere Inlet during a mean tide.

Figure D-5: Difference in peak ebb current speed due to coastal reclamation design V02 in Mangere Inlet during a mean tide.
Figure D-6: Predicted changes in wind climate composite ASR_{WDCL} (mm/yr) caused by coastal reclamation design V02 in Mangere Inlet using equation (3). Differences in modelled pre- and post-reclamation ASR_{WDCL} ± 0.5 mm/yr are blanked out.
Reclamation Version V03

Figure D-7: Difference in peak flood current speed due to coastal reclamation design V03 in Mangere Inlet during a mean tide.

Figure D-8: Difference in peak ebb current speed due to coastal reclamation design V03 in Mangere Inlet during a mean tide.
Figure D-9: Predicted changes in wind climate composite $ASR_{WDCU}$ (mm/yr) caused by coastal reclamation design V03 in Mangere Inlet using equation (3). Differences in modelled pre- and post-reclamation $ASR_{WDCU}$ ± 0.5 mm/yr are blanked out.
Reclamation Version V04

Figure D-10: Difference in peak flood current speed due to coastal reclamation design V04 in Mangere Inlet during a mean tide.

Figure D-11: Difference in peak ebb current speed due to coastal reclamation design V04 in Mangere Inlet during a mean tide.
Figure D-12: Predicted changes in wind climate composite ASR_{WDCL} (mm/yr) caused by coastal reclamation design V04 in Mangere Inlet using equation (3). Differences in modelled pre- and post-reclamation ASR_{WDCL} ± 0.5 mm/yr are blanked out.
Reclamation Version V05

Figure D-13: Difference in peak flood current speed due to coastal reclamation design V05 in Mangere Inlet during a mean tide.

Figure D-14: Difference in peak ebb current speed due to coastal reclamation design V05 in Mangere Inlet during a mean tide.
Figure D-15: Predicted changes in wind climate composite $ASR_{WDCU}$ (mm/yr) caused by coastal reclamation design V05 in Mangere Inlet using equation (3). Differences in modelled pre- and post-reclamation $ASR_{WDCU}$ ± 0.5 mm/yr are blanked out.
Appendix E  Sediment settling velocity experiments

Results of Manukau Harbour Marine Sediment Settling Velocity Testing - May 2016

Beca Ltd trading as Envirolab was commissioned by the NZ Transport Agency as part of the East West Link Alliance to undertake determination of particle settling velocity on marine sediment samples using a Bottom Withdrawal Tube Method. The testing was undertaken on Marine sediment samples from the Manukau Harbour, this report presents the testing results.

This report relates only to the samples as tested, sampling was undertaken by others.

Bottom Withdrawal Tube Method Summary

Detailed Test Procedure for Bottom Withdrawal Tube: Measurement and Analysis of Sediment Loads in Streams, Report 7: St Paul U.S. Engineer District Sub-Office Hydraulic Laboratory, University of Iowa; 1943.

The device is a glass tube of 100cm in length, graduated with volumetric scale and a quick-acting tap outlet at the bottom of the tube. A sample is uniformly dispersed in the tube. The tube is then placed in an upright position and a series of samples of known volumes are drawn from the bottom at known time intervals (increasingly spaced). The sediment weight of each sample fraction is determined. Based on Stokes Principle that a particle of 1mm will fall 90cm in six seconds and a 62 micron particle will take 5 minutes to fall the same height, the particle size distribution can be calculated with the aid of an Oden curve.

As consecutive samples are taken, the fall height reduces and allows for the calculated sediment load to be extrapolated for longer sampling periods. Hence, a sediment load of fine silt and clay material, which may normally take over 24 hours to settle, can be reduced to within a 2 hour sampling programme due to the falling head height. This method can be applied to lower sediment loads (<10g/L) unlike the particle size determination method using an hydrometer, which requires a high sediment load (>300g/L)

Deviation from Method

The method was investigated by the St Paul U.S. Engineer District Sub-Office Hydraulic Laboratory for the use of sediment loads in streams. We have undertaken the testing on solid sediment material suspended in seawater.
Sample Description

Detailed in Table 1 are the samples presented for testing.

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<tr>
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Sample Preparation

Materials were received as moist sediments. A sub-sample of approximately 30 – 40 g was taken and mixed with seawater in a container. The containers of mixed material were placed on a rolling table for a number of hours to aid the breakup of the aggregate to allow dispersal of the material. The resulting slurry was passed through a 2mm sieve to remove large material (e.g. shell fragments) and transferred into the testing tube with the aid of additional seawater until the tube was filled to a fixed mark (0.67 litres).

The tube was then mixed as per the procedure, by repeatedly inverting until the material was dispersed uniformly, at which point the tube was placed upright in a stand and a timer started. At each specific time interval the bottom fraction (of 55mL) was sampled from the tube.

The timed fractions were taken at 10 & 30 seconds, 1, 3, 7, 10, 16, 40, 80, 100, 120 minutes.

Each sampled fraction was tested for total suspended solid content and calculated to represent the % suspended, depth factor and time to settle 100cm. Data was plotted to form the Oden curve from which a tangent line is drawn to the Y-intercept to determine the % solid in suspension. From this, the particle distribution can be determined as it relates to the settling velocity of particles.
Results

The results of the settling velocity, Oden curve and particle distributions are detailed in Tables 2 to 4 and Plots 2 to 4.

The Oden curve is presented with two time scales, one showing 24 hours (1,440 minutes), the other 2 hours (120 minutes).

Classification of the fraction size is based on the Wentworth Scale, 1922 which is included in Appendix A.

Table 2: Sample EWL-S04

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Out Ref: 42182010
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Plot 2a: Sample EWL S04: % Fraction Range

% Fraction Range, Sample Number EWL S04

Plot 2b: Sample EWL S04: Settling Velocity

Settling Velocity, Sample Number EWL S04
Modelling the effects of coastal reclamation on tidal currents and sedimentation within Mangere Inlet
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Plot 3a: Sample EWL S07: % Fraction Range

% Fraction Range, Sample Number EWL S07

Plot 3b: Sample EWL S07: Settling Velocity

Settling Velocity, Sample Number EWL S07

Our Ref: 4216210
NZ1/69031384 0.4
Modelling the effects of coastal reclamation on tidal currents and sedimentation within Mangere Inlet
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Qtr Ref 4215290
NZI:U2623132#4 D.4
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Modelling the effects of coastal reclamation on tidal currents and sedimentation within Mangere Inlet
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Yours sincerely,
David Steiner
Laboratory Assistant

P.P.

on behalf of
Beca Ltd
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Email: david.steiner@beca.com
### Appendix A: Values used from the Wentworth Scale

<table>
<thead>
<tr>
<th>Fraction</th>
<th>Size Term</th>
<th>Size [mm] (upper boundary)</th>
<th>Size [micron] (upper boundary)</th>
<th>Settling Velocity [cm/s]</th>
<th>Time to Fall 100cm [minutes]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>Very Coarse</td>
<td>2</td>
<td>2000</td>
<td>2</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>Coarse</td>
<td>1</td>
<td>1000</td>
<td>16</td>
<td>0.11</td>
</tr>
<tr>
<td></td>
<td>Medium</td>
<td>0.5</td>
<td>500</td>
<td>8</td>
<td>0.21</td>
</tr>
<tr>
<td></td>
<td>Fine</td>
<td>0.25</td>
<td>250</td>
<td>3</td>
<td>0.56</td>
</tr>
<tr>
<td></td>
<td>Very Fine</td>
<td>0.125</td>
<td>125</td>
<td>1.2</td>
<td>1.39</td>
</tr>
<tr>
<td>Silt</td>
<td>Coarse</td>
<td>0.062</td>
<td>62</td>
<td>0.320</td>
<td>5.1</td>
</tr>
<tr>
<td></td>
<td>Medium</td>
<td>0.031</td>
<td>31</td>
<td>0.095</td>
<td>10.6</td>
</tr>
<tr>
<td></td>
<td>Fine</td>
<td>0.016</td>
<td>10</td>
<td>0.023</td>
<td>72</td>
</tr>
<tr>
<td></td>
<td>Very Fine</td>
<td>0.008</td>
<td>8</td>
<td>0.0057</td>
<td>292</td>
</tr>
<tr>
<td>Clay</td>
<td>Clay</td>
<td>0.004</td>
<td>4</td>
<td>0.0014</td>
<td>1190</td>
</tr>
<tr>
<td></td>
<td>Clay Mineral</td>
<td>0.002</td>
<td>2</td>
<td>0.00036</td>
<td>4762</td>
</tr>
</tbody>
</table>
COASTAL ENGINEERING

1 SCOPE

Objectives for the coastal engineering elements of the project include:

- Understanding the coastal processes within the Māngere Inlet. (This is covered in a separate report No 15 – a brief description is given in Section 2).
- Establishing an embankment level that has a low probability of being overtopped in the 100 year ARI sea storm event, allowing for sea level rise.
- Providing coastal protection works (eg rock armour revetments) for the embankment that have a low level of damage for the 100 year ARI sea storm event. Work with the urban and landscape designers for the final naturalisation of the coastline. Provide advice on beach shape and slopes, rock sizing for headlands and protection from the tidal channels.
- Investigating the feasibility of using the adjacent seabed material as fill for the embankment using mudcrete. (This is covered in a separate report as an Appendix to Technical Report 15)

Figure 1: Mangere Inlet
2 COASTAL PROCESSES

Mangere Inlet (the Inlet) and Onehunga Bay have been subject to significant change since the mid 1800s and been the location of several large scale industrial developments that has resulted in reduction of the Inlet surface area and Inlet cross-sectional area. On-going development has resulted in reclamation of the foreshore along the northern coastline of the Inlet and to the west of Gloucester Park. Up to the old Mangere Bridge, the original Inlet had coastal marine area (CMA) of 7.5 km² but has been reduced to 5.7 km² through reclamation, resulting in a loss of 1.8 km² (24%) of the CMA.

The coastal edge is protected by a variety of coastal structures including tipped rock, rock revetments and vertical seawalls offering varying degrees of coastal erosion protection. There is no evidence of any significant erosion along the northern coastal edge.

Sediments within the inlet consist of mud and fine grained sand. Historical sounding surveys and core sampling indicated that sediment texture has been muddy since pre-human times. Measured mass fluxes of suspended sediment in Māngere Inlet were greater during flood than ebb tide, indicating that the Inlet acts as a sediment and contaminant sink.

Māngere Inlet experiences a significant amount of sediment movement, particularly during windy conditions. Sediment is predominately from redistribution around the harbour and Inlet rather than from catchment sources. Overall, it is assessed that the average present day sedimentation rate is 10 mm/yr whereas the pre-1950 rate was probably around 5 mm/yr. Over the period 1950-1990, sedimentation rates were probably higher than present day rates.

Historical changes over time have been analysed because the construction of a new reclamation has the potential to change the natural character and coastal processes within the Inlet. Observed historical changes have been more pronounced with narrowing of the tidal Inlet channel than with reclamation. Changes to the Inlet due to reclamation have tended to be relatively benign and probably masked by the effects of narrowing the tidal Inlet channel and the increase in mangrove coverage (now occupy 20% of the inlet over the past 60 years). Narrowing of the tidal Inlet channel has resulted in a deepening of the channel and a reduction of wave energy entering the Inlet.

3 DESCRIPTION OF PHYSICAL COASTAL CHARACTERISTICS

3.1 Tidal Levels and Currents

Onehunga Bay is characterised as a predominantly semi-diurnal tide. At Onehunga Wharf the peak-flood velocity is only 1.75 hours before high water whereas peak ebb occurs 2.5 hours after high water. (Bell et al, 1998).

Tidal velocity information is available within the main channel adjacent to the Onehunga Wharf (NZ4314, LINZ). The data suggests peak flood tide velocities at the neck of Māngere Inlet reach 1 m/s during spring’s tides and up to 0.5 m/s during neap tides (Bell et al. 1998).

The tidal wave within the Manukau Harbour is amplified from a spring tidal range of 2.9m at its entrance to 3.8m at the Port of Onehunga. The tidal characteristic for the Port of Onehunga, based on recorded tide levels since 1925, are given in Table 3.1. These tidal levels will be adopted for the project in terms of the Auckland Vertical Datum (AVD, 1946).
Table 3.1  Tidal Levels for project in terms of Auckland Vertical Datum 1946

<table>
<thead>
<tr>
<th>Tidal levels</th>
<th>Chart Datum (m) for 2016</th>
<th>Auckland Vertical Datum (1946 –m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Highest Recorded Tide (1965)</td>
<td>5.24</td>
<td>3.04</td>
</tr>
<tr>
<td>Highest Astronomical Tide (HAT)</td>
<td>4.54</td>
<td>2.34</td>
</tr>
<tr>
<td>Mean High Water Springs (MHWS)</td>
<td>4.25</td>
<td>2.05</td>
</tr>
<tr>
<td>Mean High Water Neaps (MHWN)</td>
<td>3.44</td>
<td>1.24</td>
</tr>
<tr>
<td>Mean Sea Level (MSL)</td>
<td>2.42</td>
<td>0.22</td>
</tr>
<tr>
<td>Mean Low Water Neaps (MLWN)</td>
<td>1.33</td>
<td>-0.87</td>
</tr>
<tr>
<td>Mean Low Water Springs (MLWS)</td>
<td>0.45</td>
<td>-1.75</td>
</tr>
<tr>
<td>Lowest Astronomical Tide (LAT)</td>
<td>0.10</td>
<td>-2.10</td>
</tr>
<tr>
<td>Lowest recorded tide</td>
<td>-0.48</td>
<td>-2.68</td>
</tr>
</tbody>
</table>

The highest recorded storm-tide to date since measurements started at the Port of Onehunga was RL 2.75 which occurred on 21 June 1947 but an event on 31 July 1965 is believed to have reached RL 3.04 (as the gauge had already reached its upper limit).

Extreme sea levels represent a storm tide, based on a high tide plus storm surge. Extreme sea levels have been modelled for Auckland City at the site and probability based levels reported in NIWA (2013). Along the Māngere Inlet storm tide levels are given in Table 3.2.

Table 3.2  Extreme sea-level events

<table>
<thead>
<tr>
<th>Frequency (Annual Recurrence Interval - yr.)</th>
<th>Still Water Level (AVD –m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>2.56</td>
</tr>
<tr>
<td>5</td>
<td>2.64</td>
</tr>
<tr>
<td>10</td>
<td>2.72</td>
</tr>
<tr>
<td>20</td>
<td>2.80</td>
</tr>
<tr>
<td>50</td>
<td>2.92</td>
</tr>
<tr>
<td>100</td>
<td>3.00</td>
</tr>
<tr>
<td>200</td>
<td>3.09</td>
</tr>
</tbody>
</table>

3.2 Wind
The prevailing surface wind direction is predominantly from the south-west (26%), west (15%) and from the north to north-east (15%).
Wind generated currents are a frequent feature within Māngere Inlet, with currents reaching about 2% of the wind speed. For wind speeds less than 7m/s, wind generated currents predominate whereas for wind speeds greater than 7 m/s, wave conditions predominate.

### 3.3 Wave Climate

Māngere Inlet has a maximum fetch of 2.2 km at MHWS from the south west, 1.6 km from the south, and 3.2 km from the south east. As it lies at the north-eastern end of Manukau Harbour it is sheltered from higher energy wave action from the main body of the harbour.

Typical significant wave heights (Hs - m), peak wave periods (Tp - s) and the percentage exceedance per year (%/yr) at MHWS are presented in Tables 3.3, based on hindcast wave techniques (CUR, 2007).

**Table 3.3 Predicted Wave climate within Māngere Inlet**

<table>
<thead>
<tr>
<th>Wind Speed</th>
<th>3 m/s</th>
<th>7 m/s</th>
<th>10 m/s</th>
<th>15 m/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>Direction</td>
<td>Hs (m)</td>
<td>Tp</td>
<td>%/y</td>
<td>Hs</td>
</tr>
<tr>
<td>S</td>
<td>0.1</td>
<td>1.00</td>
<td>7.2</td>
<td>0.15</td>
</tr>
<tr>
<td>SW</td>
<td>0.1</td>
<td>1.00</td>
<td>23</td>
<td>0.2</td>
</tr>
<tr>
<td>SE</td>
<td>0.1</td>
<td>1.10</td>
<td>4.4</td>
<td>0.2</td>
</tr>
</tbody>
</table>

According to Table 6.3 ambient wave conditions are generally less than 0.3 m within Māngere Inlet for the majority of the times. It is a low energy environment.

Extreme significant wave heights (Hs - m), peak wave periods (Tp - s) and sustained wind speed (W - m/s) at MHWS are presented in Tables 3.4. An average water depth of 2.5m was assessed at MHWS.

**Table 3.4 Predicted Wave climate within Mangere Inlet**

<table>
<thead>
<tr>
<th>Event</th>
<th>2 year ARI</th>
<th>10 year ARI</th>
<th>50 year ARI</th>
<th>100 year ARI</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Hs</td>
<td>Tp</td>
<td>W</td>
<td>Hs</td>
</tr>
<tr>
<td>S</td>
<td>0.3</td>
<td>2.2</td>
<td>16</td>
<td>0.4</td>
</tr>
<tr>
<td>SW</td>
<td>0.4</td>
<td>2.6</td>
<td>19</td>
<td>0.5</td>
</tr>
<tr>
<td>SE</td>
<td>0.4</td>
<td>2.6</td>
<td>16</td>
<td>0.5</td>
</tr>
</tbody>
</table>

### 3.4 Effects of Climate Change and Tsunami

The NZCPS (Policies 24 and 25) requires hazards to be identified and development to consider these hazards over at least the next 100 years, having regard to:

- Sea level rise (1.0m as required by the Unitary Plan (2016))
- Tides storm surge (see Table 3.2)
Wave climate (10% increase (MfE, 2008) in extreme winds which affect wave generation)

Tsunami (1.5m for 100 year ARI event and 2.6m for 500 year ARI event (GNS, 2013))

The effect of sea level rise, tides and storm surge will be to increase water levels given in Table 3.2 by 1.0m. For example, the 100 year ARI event will have a still water level of RL 4.0m.

The effect of increased windiness by 10% will increase wave heights. Information on significant wave heights (Hs - m), peak wave periods (Tp - s) and sustained wind speed (W - m/s) at MHWS are presented Table 3.5 for this increase in wind speed. For example a 100yr SE wave increases in height from 0.6m to 0.7m. As it approaches the northern coastline, however, it will refract and the wave energy will be spread out, resulting in a wave height of 0.6m perpendicular to the coastline.

<table>
<thead>
<tr>
<th>Event</th>
<th>Direction</th>
<th>2 year ARI</th>
<th>10 year ARI</th>
<th>50 year ARI</th>
<th>100 year ARI</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Hs</td>
<td>Tp</td>
<td>W</td>
<td>Hs</td>
<td>Tp</td>
</tr>
<tr>
<td>S</td>
<td>0.3</td>
<td>2.3</td>
<td>17</td>
<td>0.4</td>
<td>2.5</td>
</tr>
<tr>
<td>SW</td>
<td>0.4</td>
<td>2.7</td>
<td>20</td>
<td>0.5</td>
<td>2.8</td>
</tr>
<tr>
<td>SE</td>
<td>0.4</td>
<td>2.7</td>
<td>17</td>
<td>0.5</td>
<td>3.0</td>
</tr>
</tbody>
</table>

The effect of tsunami on the Project will be dependent on the tide level. At an average tide level (MSL) the tsunami water level could reach RL 1.72m or RL 2.72m with sea level rise for 100 year ARI event. At mean high water springs the tsunami water level could reach RL 3.77m or RL 4.77m, but this would be a more extreme event than the 100 year ARI.

4 COASTAL ENGINEERING DESIGN

4.1 Minimum Embankment Levels

The project minimum road level has been determined as follows:

- The edge road embankment level has been set at RL 4.50m based on:
  - 100 year ARI tide + storm surge  RL 3.0m
  - Sea level rise                  1.0m
  - Wave run-up allowance           0.5m
  - Total                           RL 4.5m

The wave run-up allowance is based on a nonbreaking significant wave height (H_s) of 0.6m, a mean wave period of 2.8s and a rock armour slope of 1(V):2(H). Based on CUR (2007) this will result in an overflow rate of <10L/s/m at the road edge and <0.02 L/s/m, 3.0m from the edge. This is considered acceptable for protection of the embankment, vehicles and pedestrians (more than 3m from the edge).

The recommended edge level of the embankment at RL 4.5m, compares with the existing level of the Onehunga Port wharves at RL3.9m.

At the location of the trench near Galway Street, waves will approach the trench vertical wall which will have relatively high wave reflection. Based on a maximum wave height of 1.5 H_s for shallow water, a 90% reflecting wave and sea level rise, a wall height of RL 5.7m is required to
avoid wave overflows into the trench. Incorporating an embankment seaward of the trench wall would reduce the height of the trench wall.

For the naturalised coastline, the outer bund level of RL 2.8 to 3.0m is based on the existing walkway levels (for amenity and flood protection purposes) but the foundations will allow for a gradual increase in height up to 1.0m.

### 4.2 Rock Sizes

There will be a variety of coastal protection works along the northern coastline. The following armour and underlayer rock is recommended based on the following information:

- Significant wave height of 0.6m
- Mean wave period of 2.8s
- Rock density of 2.6 tonne/m³
- For 2 layer system use Van der Meer’s (CUR, 2007) equation for shallow water with Damage level parameter \( S_d \) =2 (low damage), permeability factor \( P \) =0.1, and No of waves \( N \) =3000.
- For headlands increase rock weight by 50% using Van der Meer’s equation.
- For single layer or isolated rocks on a firm surface increase rock weight by 100% using Van der Meer’s equation for 1: 5 slope. Only to be used on 1:5 slopes or less.
- A geotextile should be placed on the underside of the underlayer.
- If founded in soft material, the structure should include a 1.5m long toe. If founded in hard material or mudcrete the out rock layer should be trenched into the hard layer by 300mm.
- A 1.2m horizontal crest should be incorporated into the structure.

### Table 4.1 – Recommended Rock Sizes

<table>
<thead>
<tr>
<th>Case</th>
<th>Armour rock</th>
<th>Underlayer rock</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( D_{50} ) (mm)</td>
<td>( W_{50} ) (kg)</td>
</tr>
<tr>
<td>Slope 1(V):1.5(H) 2 layer</td>
<td>400</td>
<td>100</td>
</tr>
<tr>
<td>All other slopes 2 layer</td>
<td>350</td>
<td>65</td>
</tr>
<tr>
<td>Slope 1(V):1.5(H) 2 layer</td>
<td>450</td>
<td>150</td>
</tr>
<tr>
<td>Headland</td>
<td></td>
<td></td>
</tr>
<tr>
<td>All other slopes 2 layer</td>
<td>400</td>
<td>100</td>
</tr>
<tr>
<td>Headland</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Single layer or isolated</td>
<td>500</td>
<td>200</td>
</tr>
<tr>
<td>rocks on 1:5 slopes or less</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### 4.3 Beach Design

It is intended that as part of the naturalisation of the coastline a number of shingle type beaches will be created. These beaches will be located between headland structures. They should be aligned parallel with the offshore bathymetry contours with some local concave curvature as the beach approaches the headland structures.

- The inlet has a low energy wave climate. It is recommended that the shingle beach should have the following features:
- Shingle size 20-100mm (\( D_{50} \)=50mm)
- Berm level of RL 2.5m (i.e. about 0.5m above MHWS)
- Berm width of 5m

In locations where a headland encroaches to within 20m of a tidal channel, the foundation of the headland should extend down to the invert of the tidal channel for a thickness of 3m.