MAHANGA COASTAL HAZARD ASSESSMENT

in the vicinity of the proposed Williams, Mexted and Van Breda Malherbe subdivision

A report prepared for Mahanga E Tu Incorporated

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Client Report 2012-03 CRep 17 April, 2012

EXECUTIVE SUMMARY

The final section of this report (Section 6: Summary and Conclusions) has been structured and written so as to suffice as the Executive Summary

TABLE OF CONTENTS

1.0 BACKGROUND

1.1 Terms of Reference

In November 2010, Coastal Systems Ltd (CSL) was contacted by Mrs Jenny Krzanich of Happy Jacks Road, Mahanga, with regard to representing the community group *Mahanga E Tu* (MET) as a coastal hazard witness in a forthcoming Environment Court appeal against a recent decision by the Hawke's Baty Regional Council (HBRC) and the Wairoa District Council (WDC) to grant subdivision consents to Williams, Mexted and Malherbe for Lots 1 and 2 DP28759 at Mahanga Beach. Figures 1.1 is a location map of Mahanga Beach and the proposed subdivision, and Figure 1.2 shows hazard planning details.

A site visit was carried out on 18 January, 2011. The proposed subdivision is located on the (western) side of an inlet connecting the Mahanga (Waituna) Stream with the ocean. In general inlets comprise one of, if not the, most dynamic and hazard prone parts of coastal systems. This inlet seemed to be no exception, with bounding topography and low elevation making the site vulnerable to tsunami and storm inundation, and fresh scarping on the western bank and a spit on the eastern side indicating possible erosion issues. At that stage it appeared an error may have been made by the councils' decisions to permit such a subdivision and I considered we had a professional ethical duty to at least investigate the extent/severity of potential hazards. After an exploratory study it became clear to us that the site was particularly vulnerable to tsunami, storm inundation and extensive systematic erosion during the mandatory assessment period of at least 100 yrs and that such hazards presented a serious risk to personal safety and property. CLS were subsequently engaged by MET to (1) prepare a full and detailed coastal hazard assessment, (2) compare our results with past assessments for this area, (3) assess the earlier studies against requirements of the New Zealand Coastal Policy Statement (NZCPS 2010) which became operational on 3 December 2010, (4) assess the viability of hazard risk mitigation measures, and (5) comment on hazard zoning.

1.2 Past assessments and hazard zoning

Several coastal hazard assessments have been carried out for the area in question between 2002 and 2008 as part of subdivision consent applications. Quantitative details from these assessments are provided later in Sections 4 and 5. The following is based on Gibb 2002, 2005, 2007, 2008, Tonkin and Taylor 2004 and 2008, HBRC 2007 (HBRCEP Hearing Report 500 series Coastal Hazards), and Paul Thomas's EC evidence dated 24-2-2012 evidence.

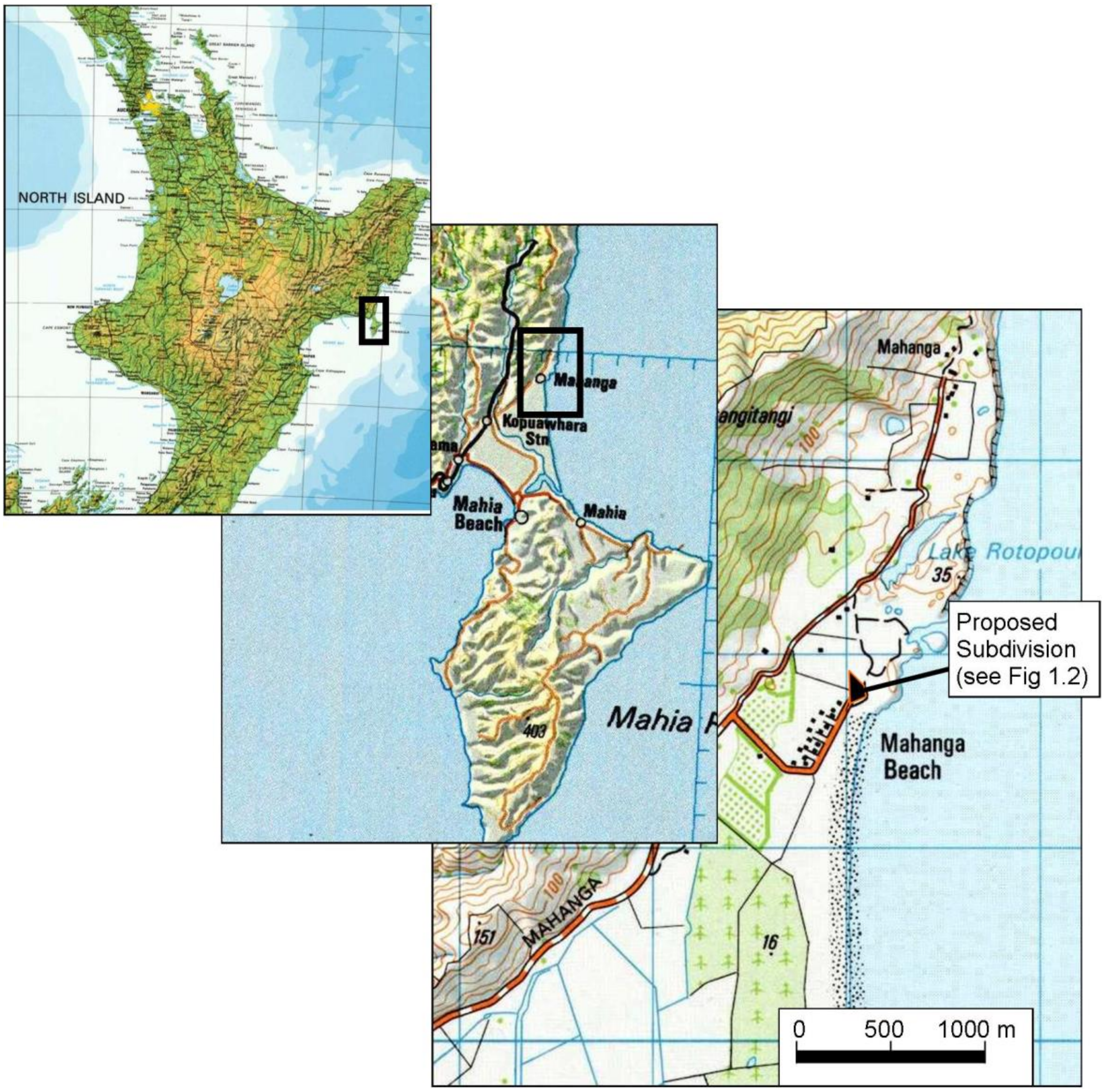


Figure 1.1 Location maps for Mahanga Beach and proposed subdivision

The first hazard assessment was carried out by Dr Jeremy Gibb in 2002 for Mahanga Beach Ltd. as part of a subdivision (Pukenui Drive, Lot 7) application to the Wairoa District Council (WDC). The shoreline analysis showed all of Mahanga Beach to be accreting.

In 2004, Tonkin and Taylor Ltd (T&T'04) carried out a coastal hazard assessment at Mahanga as part of their "regional" assessment for the Hawke's Bay Regional Council (HBRC). A regional assessment is an assessment based on "spatially broad" hazard component values and is necessarily conservative. That assessment was based on estimating several components typically used for inundation and erosion assessments (see summary Tables 4.2 and 5.3 in the present report). The T&T'04 assessment used the long-term erosion result from Gibb 2002, i.e. all of Mahanga Beach is accreting. T&T'04 produced three hazard lines with the seawardmost being based on the sum of all erosion components excluding the shoreline response to projected sea-level rise

(SLR); they recommended no-building seaward of this line. The adjacent landward area potentially affected by SLR response and inundation (2% AEP) allowed only temporary/removable buildings with clearly defined design criteria. It is noted that closer to the inlet, i.e. the area of the proposed subdivision, their hazard lines were offset landward, apparently as a precautionary measure in recognition that inlets typically have unstable shorelines.

In 2005, Dr Gibb carried out a "site specific" coastal hazard assessment for the present owners in the vicinity of the proposed subdivision (Lots 1 and 2), and concluded *structures would likely be subject to material damage by erosion and inundation.* The 100 yr erosion hazard line within the property (Gibb 2005, Fig 6) plotted close to the T&T'04 line.

In 2007 Dr Gibb was commissioned by the owners to apply his 2005 assessment to an alternative hazard risk management zonation. In particular, the long-term erosion component was excluded from the no-build zone. This approach reduced the proposed T&T'04 no-build distance some 66%, from 42 m to 14 m (assessment values summarized in Table 5.3 of the present report), this resulting from the 2005 site specific assessment having a short-term value of 9 m compared to T&T'04's conservative value of 42 m, and the Gibb NON-ZERO long-term erosion values (0.22 m/yr in Gibb 2005 and 0.12 m/yr in Gibb 2007, see Section 5.4)) was delegated to the adjacent landward lower hazard risk zone where building may be permitted. This treatment of long-term erosion is surprising, but was subsequently adopted by the HBRC and later by the Wairoa District Council (WDC) and the current hazard zoning is depicted in Figure 1.2. The secondary hazard management zone (CHZ 2) is based on the combined effect of long-term erosion and the response to SLR (plus inundation criteria) and permits building as restricted discretionary activities. The proposed subdivision application was approved by a joint council hearing decision in 2009 with a condition that required the building of a seawall to protect against possible systematic erosion. This condition is also surprising given that the then operative NZCPS 1994, Policy 3.4.5 required any **new subdivision** to be so located as **to avoid** the need for protection works.

1.3 Coastal hazards and their assessment

Coastal hazards consist of erosion, inundation and landslide and these are driven by tides, tsunami, storm surge, waves, currents, sediment supply and sea-level change. The seaward dip of the alternating mudstone/sandstone layers of sedimentary rock making up the coastal ranges, facilitates slope failure, and it is shown in Section 2 that the Mahanga coastal environment was formed by a catastrophic slope failure (the Mahanga Landslide) some 800 yrs ago. The hillslope immediately south of the landside could produce landslides in line with the Mahanga Beach settlement. However, basal protection from wave action by depositional material from the Mahanga Landslide, along with lagoon infill (the present wetland), greatly reduce the likelihood of such an

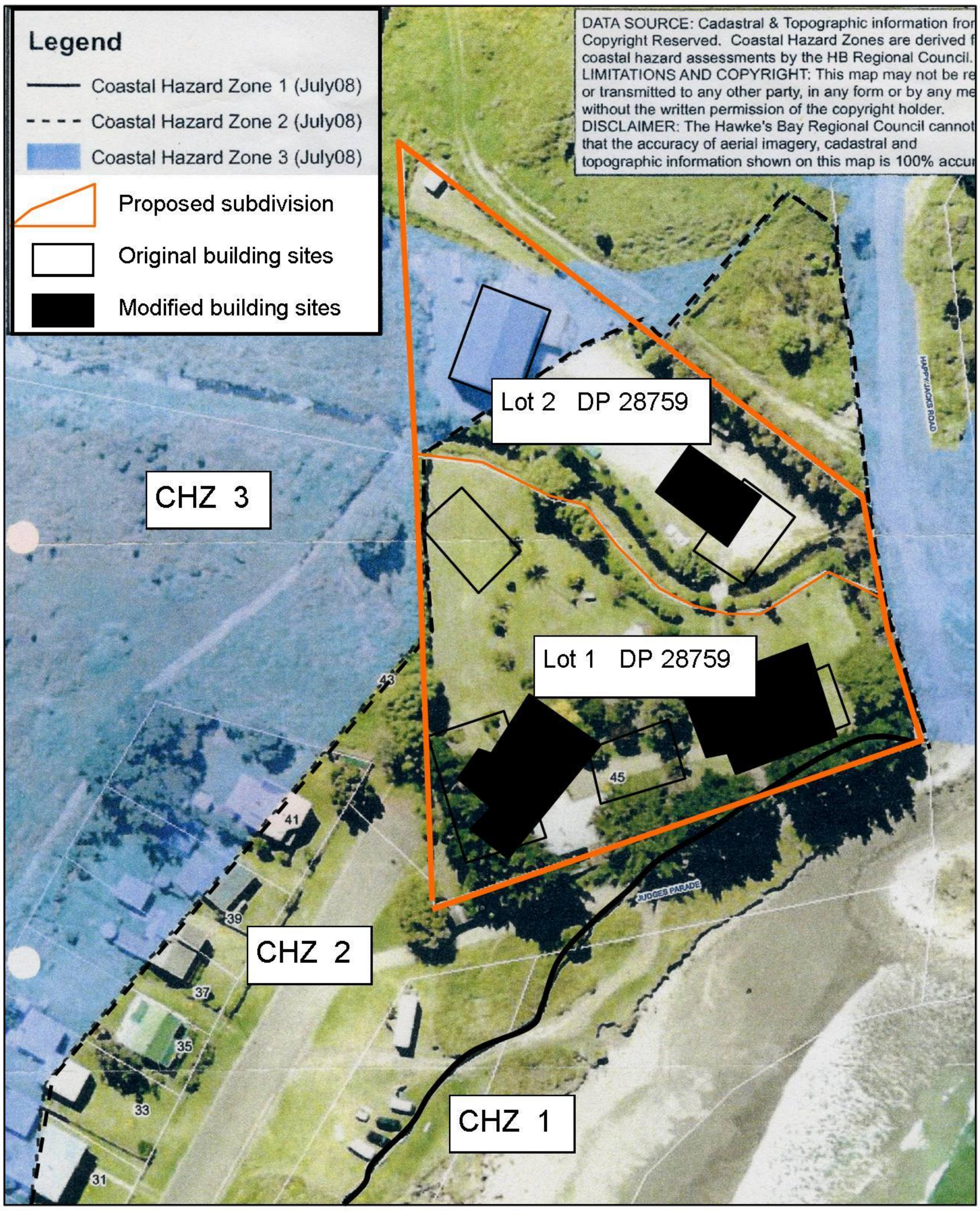


Figure 1.2 Hawke's Bay Regional Council (HBRC) coastal hazard zones in the vicinity of the proposed subdivision. Coastal hazard zone 1 (CHZ 1) covers land subject to erosion from short-term shoreline fluctuations and dune instability hazard. Building is not permitted. CHZ 2 covers land potentially at risk of erosion up to 2100 form long-term shoreline change and retreat associated with sea-level rise. CHZ 2 permits building as restricted discretionary activity. CHZ 3 covers areas potentially at risk from sea water inundation in a 1 in 50 yr event.

Sources: compiled from materials from the HBRC and the Applicant.

event within the current assessment period, although predicted SLR and coastal erosion beyond the assessment period may change this situation. Coastal hazards assessed in the present investigation will be restricted to tsunami inundation, storm inundation and erosion.

The methods used to assess coastal hazards have been developed over the past 30 years as scientific understanding of coastal processes developed and as methods of data acquisition and analysis improved. Hazard assessments consist of defining several components and then appropriately combining their assigned values such that the result applies to a designated assessment period and storm magnitude. While there has evolved general agreement over the main sub-components to be assessed, the techniques for their assessment can varied considerably (e.g. ARC 2000) depending on the type of coastal environment and the qualifications and experience of the practitioner. Over the last few years there has been an increase in official guidance and in December 2010 the revised New Zealand Coastal Policy Statement became operative and this contained a dedicated policy (24) on matters that need to be considered when carrying out a coastal hazard assessment. While this will bring a welcome consistency to hazard assessment, it should be noted that in the past, thorough assessments already accommodated the matters raised in the NZCSP 2010.

As well as practitioners determining the extent of erosion over the assessment period, and the level of inundation from an event of a particular (critical) magnitude, a hazard is only a hazard if the event actually negatively affects persons or property. Hazard sensitivity analyses (HSA) have often been a first step in the hazard assessment process, whereby areas likely to suffer systematic erosion or inundation and where there will be consequence (personal safety/property impacts) are generally identified and these then undergo subsequent prioritized assessment. This raises the issue of hazard “risk” which is the combination of probability of an event (inundation or erosion) and the consequences of impact, and the NZCPS 2010 gives particular attention to this concept, especially with regard to hazard management (Policies 25 and 27), such that new development in hazard-prone areas must **avoid increasing the risk**.

1.4 New Zealand Coastal Policy Statement 2010, Policy 24.

Policy 24 consists of two sentences. The first sentence requires areas of the NZ coastal environment *potentially affected by coastal hazards “(including tsunami*)” to be identified, giving priority of areas of high risk of being affected. This is in keeping with the EHS approach outlined in the previous paragraph. Inclusion of tsunami as an inundation driver is significant as while tsunami had often been described in New Zealand assessments it had not been included in the output due to the lack of definitive information (as was the case in the earlier Mahanga assessments). Note that further directive now requiring the inclusion of tsunami in assessments is provided in Policy 24 (d).

The second sentence relates to assessing the extent of erosion or inundation over a prediction period "*of at least 100 yrs". This is another significant directive as the only previous time-span guide was the (indirect) 50 yrs specified in the Building Act Guidelines, although use of 100 yrs was often used in local government hazard assessments, especially where subdivision was involved. Case law documented in MFE (2008)shows the Environment Court has, especially more recently, generally required 100 year timeframes (assessment period or prediction period). However, there is no directive as to the critical inundation event (magnitude/frequency) and this is discussed below in Section 1.5.

The second sentence goes on to list *8 matters* an assessment is *to have regard to*, with the 8th relating to climate change and has *3 subpoints*, the first of which requires reassessment of the previous seven matters in term of climate change effects. This direct inclusion of climate change and the detail with which is to be incorporated is a clear indication that associated hazards cannot be assigned lesser significance in hazard zoning as has occurred in the past with local government, including the 2008 HBRC zoning at Mahanga. These 8 matters are now described and implications noted.

Policy 24a *Have regard to the physical drivers and processes that cause coastal change including sea-level rise.*

This essentially requires the practitioner to compile a conceptual geomorphological model (as quantitative as possible), that will explain the past morphological behaviour of the coast in the area of interest and indicate future change. Policy 24a is a matter of fundamental importance and some of the subsequent matters in the list relate to the provision of information to define the geomorphological system. The Gibb reports broadly defined the very large-scale, very long-term system, but failed to accurately identify the more localized smaller scale system operating at a time-scale of ± 100 to 150 yrs in the vicinity of the subdivision, and this has significant implications for the hazard assessment.

Policy 24b *Have regard to short-term and long-term natural dynamic fluctuations of erosion and accretion*

Longer-term (LT) and shorter-term (ST) shoreline behaviour comprise two important components in conventional coastal erosion hazard assessments, the others being retreat associated with sea-level rise (RSLR), dune erosion scarp stability adjustment (DS) and a combined uncertainty estimate (CU)). Identification of erosion/accretion patterns also plays an important role in quantitatively defining the morphodynamic system. However, to provide reliable assessment at the now mandatory 100^{+} yr prediction period requires obtaining and analyzing the longest possible historical data, preferably well in excess of 100 yrs.

Dr Gibb's previous assessments used data spanning 1942 to 2002-8. By contrast, the present assessment extends the data-base back to 1899 and also included more intermediate samples. The previous assessments also tended to use less comprehensive methods of time-series analysis. Some significantly contrasting analysis results were obtained

Policy 24c *Have regard to the geomorphological character*

This essentially refers to understanding the land forms and their formative processes, and provides primary information for defining the geomorphological systems. Also included in most hazard assessments is as allowance for post erosion adjustment of the steep dune scarp to a stable slope. While this parameter can be significant where the foredunes are several metres high, it was relatively insignificant in the Mahanga assessments with T&T'04 ignoring it altogether.

Policy 24d *Have regard to potential sources, inundation pathways and overland extent*

This relates to all forms of seaward driven inundation (including tsunami) and how the pulse or wave behaves as it propagates landward. This matter requires an understanding of wave type (including tsunami) and wave climate, extreme value analysis, and marine and terrestrial wave transformation across marine and terrestrial relief. The earlier assessments relied on historical evidence (which is important but limited), earlier and somewhat less sophisticated wave data, and minimal consideration of pathways and propagation behaviour.

Policy 24e *Have regard to the cumulative effects of sea-level rise, storm surge and wave height under storm conditions*

This relates to the storm-driven inundation components: sea-level rise (SLR), storm surge (SS), and wave processes of runup (RU) and set-up (SU). Note that, in addition, storm inundation is affected by tide, lower frequency sea-level cycles (seasonal, El Nino/La Nina, and the Inter-Pacific Oscillation), with catchment run-off and the factors noted above in Policy 24d having additional effects. Most importantly, Policy 24e refers to their combination and this is a critical consideration. Unlike the erosion hazard components which are simply added together, storm components may be inter-related so care must be taken not to over-estimate their combined effect. To achieve this, extensive data sets and specific statistical analyses are required. These are matters of fundamental importance in storm inundation assessment and are considered further in Section 1.5 below. As the required data sets have only recently begun to become available, the earlier assessments were not able to incorporate such detail so used more conservative approaches and necessarily arrived at higher inundation values.

Policy 24f *Have regard to the influences humans have had or are having on the coast.*

Shoreline behaviour can be affected by anthropogenic activities which can control hydrodynamics and sediment supply. Such activities include coastal protection and rivermouth control structures, river controls such as dams, large-scale pioneer clearance of hinterland, and more recent land catchment stabilization work. The latter may influence sediment supply to the Mahanga coast and require the use of the longest possible shoreline data sets coupled with appropriately precautionary uncertainty values. As noted earlier, the existing assessments used substantially shorter records than the present assessment.

Policy 24g *Have regard to the extent and permanence of built structures*

The assessment also needs to consider the effect existing structures have on erosion and inundation process, and how this situation may change in the future. The limited present buildings on the site are considered ineffectual in influencing erosional or inundation processes. However, a seawall is an integral part of the subdivision proposal and this will have significant environmental impacts not considered in previous assessments (Gibb, 2006, Cardno 2008). In addition, the solid platforms planned for the proposed dwellings can be expected to affect inundation by channeling and redirecting flows, increasing local flow velocities and impounding inundation, and these situations were not addressed in previous assessments.

Policy 24h *Have regard to the effects of climate change on:*

i) *matters a to g above,*
ii) *storm frequency, intensity and surges, and*
iii) *coastal sediment dynamics*

The original 1994 New Zealand Coastal Policy Statement (NZCPS 1994) only required that the <u>possibility of sea level rise on coastal hazards needed to be recognised</u> (Policy 3.4.2 and 3.4.4), and consequently Local Government often relegated climate change effects a low priority in hazard assessment and/or implementation into hazard zones (as appears to have been the case at Mahanga - NB Section 1.2). However, over the past 10 yrs or so, scientific understanding of climate science and coastal impacts have greatly increased (e.g. IPCC 2007, MFE 2008, RSNZ 2010, Shand and Manning 2010), and NZCPS 2010, Policy 24 a, e and h make it clear that such effects must be accorded fundamental importance in hazard assessment and implementation.

In particular, Policy 24h, subpoint (i) directs the assessor to address climate effects with respect to the previous 7 matters (a to g). It is noted that matters (ii) and (iii) appear to have been included for emphasis as in a thorough assessment they would already be covered when addressing subpoint (i).

Previous assessments incorporated SLR into erosion and inundation calculations, based on the official guidance at those times. However, there was no consideration of further impacts of climate change on hazard drivers.

Information-base. Policy 24 finishes with a clause that assessments are *to take into account national guidance and the best available information* on the likely effects of climate change. There has been a considerable increase in available information over the past few years as can be appreciated when perusing the Reference Section of the current report.

1.5 Inundation terminology and definitions

Inundation assessments must be based on an event of a pre-defined magnitude and this is described in terms of the return period or the annual excedence probability (AEP). The return period refers to the average length of time in which an event of a particular size will occur. The AEP refers to the probability of an event which exceeds a particular magnitude, occurring in any given year), with each approximating the reciprocal of the other. So a 100 yr return period event refers to an event that occurs, on average, once in 100 yrs. The AEP equivalent is an event a 1% or 0.01 probability of occurring in any particular year. Of course this assumes no systematic change occurs in the driver's future climate, and allowance must be made for climate change.

What return period to use for coastal hazard assessments? A critical return period has not been directly stated in the NZCPS 2010 or other guidance. The only NZ legal directive is the Building Act 2002 (Building Regulations 1992, Schedule 1 Building Code clause E1.3.2) that surface water level from a 50 yr return period event shall not enter buildings. T&T'04 and MFE (2008) note that some councils have adopted a 100 yr return period for housing and new subdivision. However, the NZCPS 2010 Policy 24 and Policy 25 use the wording "potentially affected by coastal hazards over at least 100 years", and this indicates that only a very low likelihood of coastal hazard impacts can be ignored by decision makers. Therefore, larger events (for example, a 200 yr return period event with a 40% chance of occurring within the next 100 years, such as examined in the tsunami inundation assessment in paragraph 3.2 below) warrant serious consideration by decision makers. In the present assessment, a 100 yr return period is used with an indication of the 50 yr event/impact included. However, in view of the comment above, these could be viewed as conservative (undersized) extreme events relevant for a 100+ yr assessment period.

Other important terms/concepts are:

- The planning horizon or prediction period or assessment period, this being the length of time the hazard assessment is required to span (broadly equivalent to the design life of the type of allowable structure), and
- The recurrence interval, which is the likelihood that an event of a particular size will occur within the stipulated planning horizon.

To illustrate these concepts by example: a 50 yr return period event, which is equivalent to a 0.02 or 2% AEP, has a 86% chance of occurring within the mandatory minimum 100 yr hazard assessment period. By contrast, a 100 yr return period event (0.01 or 1% AEP) has a 63% chance of occurring within the 100 yr planning horizon, a 200 yr return period event has a 40% chance of occurrence, a 500 yr return period event has and 18% chance, and a 1000 yr return period event has a 10 % chance of occurrence.

Deriving component values is not straightforward and until the recent advent of extended data sets, component values were based on observation with maximum values over a typical period of 50 to 100 yrs often being used. In addition, combining component values requires consideration of the likelihood of their combined occurrence or joint probability as if the variables are independent, e.g. tide and storm surge, then arithmetical addition of their individual values result in a significant overestimation. These situations are considered in some detail in Section 4 and Appendix A.

1.6 Report Structure and Review

Section 2 described the *Geomorphological System*, this being of fundamental importance in coastal hazard assessments and this is now clear in the NZCPS 2010. The definition of this system is based on: published geological and geomorphological information; information provided by the HBRC including 2005 LIDAR from which a detailed Digital Terrain Model (DTM) was constructed; results from the present assessment, for example the shoreline analysis in Section 5; and lastly some material provided by colleagues at Massey University who carried out extensive research during the late 1980s-1990s. Section 3 assesses the tsunami inundation hazard, and its impact across the proposed subdivision is illustrated using the LIDAR-based DTM. Section 4 assesses the storm inundation hazard and its impact is also illustrated with the LIDAR-based DTM. Section 5 assesses the erosion hazard using over 100 yrs of historical shorelines, and both linear and nonlinear-regression analysis. With the aid of the DTM, a realistic future evolution of the Mahanga Inlet could be predicted and its impact on the proposed subdivision site identified. The erosion components values are set out for comparison with earlier assessment values in a comparison table (5.3). The feasibility of hazard risk mitigation is considered at the end of each of the assessment sections.

To achieve the most thorough review possible, each section of the report was considered by a different practitioner with specialist knowledge of current hazard assessment techniques and associated matters related to that particular section. Reviewer’s comments were subsequently incorporated within the report to maximize accuracy and ensure best available methodology.

Mr Mike Jacobson reviewed parts of Sections 1, 3, 4 and 5 regarding the interpretation and application of coastal hazard policies from the NZCPS 1994 and NZCPS 2010. Mr Jacobson [BSc, BE] is a coastal management consultant and has been working in the field of coastal hazard management for the past 24 years, at times with the Department of Conservation, the Kapiti Coast District Council and as a private consultant. He is presently commissioned by the Department of Conservation to prepare guidance notes on the coastal hazard policies in the New Zealand Coastal Policy Statement 2010. That work is part of delivering web-based guidance notes on the NZCPS 2010 to promote implementation of that national policy statement by local government and others, as part of the Department of Conservation's National Implementation Plan.

Dr Mike Shepherd reviewed Section 2 and part Section 5 as pertains to the Geomorphological System. Dr Shepherd [PhD] was a geomorphology lecturer and researcher at Massey University for 33 yrs (now retired) and specialized in coastal evolution. Dr Shepherd has undertaken coastal research throughout New Zealand and Australia and spent several years in the late 1980s and 1990s supervising both under-graduate and post-graduate student research projects on the Mahia Peninsula Tombolo.

Professor James Goff reviewed Section 3 (Tsunami Inundation Assessment). Professor Goff [PhD] is presently Director of the Australia-Pacific Tsunami Research Centre and Natural Hazards Research Laboratory, University of New South Wales. Earlier appointments being with GNS (Senior Scientist), DOC (Principal Regional Scientist) and NIWA (Group Manager). Professor Goff visited the Mahanga site in 2007 as part of NIWA's general tsunami hazard assessment for the Hawke's Bay Regional Council.

Mr James Carley reviewed Section 4 (Storm Inundation Assessment). Mr Carley [Master of Engineering Science] is a Senior Project Engineer at the Water Research Laboratory, University of New South Wales, Sydney. For the past 20 yrs Mr Carley has undertaken hazard assessment and structure design including field-based analytical, numerical and physical modelling, at sites throughout Australia, the South Pacific, South-East Asia and the Middle East. He has authored more that 100 Water Research Laboratory reports, many of which involved developing guidance for coastal practitioners.

Mr Jim Dahm reviewed Section 5 (Erosion Assessment). Mr Dahm [MSc] is Principal Director of Eco Nomos Ltd., a consultancy specializing in coastal hazard assessment and management. Mr Dahm has over 30 yrs experience in applied coastal science and management with previous appointments including the Ministry of Works and Development and Environment Waikato. For 22 yrs he has carried out coastal hazard assessments and Mr Dahm also runs hazard education courses in association with NIWA and a range of other organizations.

2.0 THE GEOMORPHOLOGICAL SYSTEM

There is more uncertainty with respect to future coastal behaviour near the proposed subdivision than elsewhere at Mahia.

Comment by reviewer Dr Mike Shepherd

2.1 Introduction

A *geomorphological system* is here defined to comprise the present landform(s), associated energy-sediment processes and pathways, the evolutionary sequences to attain this state, and thus indicative directions of future change. This section provides the conceptual overview of the system with supplementary support material contained in the following sections.

2.2 Geological setting

The large-scale landscape of this region is a product of interactions between the *Pacific and Australian tectonic plates*, with the former sliding beneath the latter. This process has resulted in distinct geomorphic expressions extending from the Hikurangi Trough some 20 km east of the Mahia Peninsula across to the "Taupo Volcanic Zone" in the west (Cole and Lewis, 1981). Between these extremes are a series of thrust faults giving rise to topographic highs (*anticlines*) and intervening lows (*synclines*) which become older and more pronounced from east to west.

During the last one million years, the seabed immediately east of the Mahia Peninsula has been uplifting to form the most recent anticline, the *Lachlan Ridge* (Mazengarb et al., 2000), and this has resulted in incremental uplift of the peninsula itself and the formation of a series marine terraces along its eastern and northern coasts with associated fluvial terraces within stream valleys (Berryman, 1993). Uplift rates diminish from east to west (see Section 2.3). Note fluvial terraces are important as their evidence of relatively recent uplift often endures whereas marine terraces along an eroding coast will likely have been destroyed.

Growth of the Lachlan Ridge appears to have been associated with the development of the adjacent (westward) Mahia Syncline (stratagraphic low), causing the lattters NNE-SSW aligned axis to translate some 5 km to the northwest (Ota et al., 1989) where it is some 0.5 km seaward of Mahanga Beach settlement. The syncline location gives the Mahanga area a tectonic disposition to subside, and this is supported by the reducing uplift rate across the northern side of the Mahia Peninsula (2.5 mm/yr to 0.7 m/yr), and the lack of terraces on the adjacent mainland coast and within river and stream valleys (LIDAR inspection, this study). Berryman et al. (2008) shows this area as having a subsidence rate ranging between 0 and 0.1 m/100 yrs. As such subsidence is episodic, prediction of potential future subsidence has not been incorporated in this assessment. It is noted several million years ago the syncline was located about 5 km further offshore (SE) with the same orientation as the current axis. At that time the Mahanga area may have experienced uplift associated with the anticlines further west.

2.3 Holocene Geomorphology

During the current warm period (~10 ka) referred to as the Holocene, sea-level reached its present level at ~6,500 BP (after rising some 120 m over the preceding 20 ka). At this time Mahia Peninsular was an island separated from the mainland by a strait some 5 km wide. Dateable volcanic airfall deposits on ancient foredunes show that by ~5,000 yrs BP a sandy foreland had started to extend from the mainland in the vicinity of the Kopauwhara Stream. This foreland was fed by littoral drift arriving from southwest (Hawke Bay), the northern mainland coast (Mahanga and beyond), and also the sediment from the Kopauwhara stream, so by ~3300 yrs BP it had extended some 2 km toward the peninsula. By ~1800 yrs BP the sand-based foreland had joined with the peninsular to form a *tombolo,* with the final connection likely associated with the dramatic Taupo Eruption which occurred about that time. The 5 km long tombolo continued to grow in size and is now 3 km across at the peninsular end and 6 km wide at the mainland. This landform is the largest of its type in New Zealand and is registered in the New Zealand Geopreservation Inventory (www.geomarine.org.nz/NZGI/).

Mahanga is located where the northernmost shoreline of the tombolo joins the mainland and the general geomorphology of this area is depicted in Figure 1.1 (reproduced from Dr Gibb's 2005 initial hazard assessment report for the proposed subdivision). This map shows an "early shoreline" which his accompanying text describes as a 3-6 m high shore-parallel sand escarpment some 250 to 400 m landward of the present beach. Such an extensive erosional shoreline indicates a severe sediment deficit which may have been caused by the decline and eventual cessation of sediment flows from Hawkes Bay associated with the development of the tombolo. The sea-level maximum which occurred between 1100 and 1200 BP in association with the Medieval Warm Period (Grindsted et al., 2009), which is known to have affected New Zealand (Boswijk et al,. 2006), may also have contributed to this early erosional shoreline . The present predictions of sea-level rise could lead to a significant increase in mass failure along the landslide-prone New Zealand East Coast, with an increased risk of burial and local tsunami.

Gibb (2002) notes that landward of the escarpment, sand dunes occur some 8-10.5 m above MSL and these are 3-4 m higher than more recently formed dunes closer to the shoreline. He interprets this as being evidence for ongoing tectonic uplift - which conflicts with my earlier argument for subsidence/stability. While variation in dune height can be indicative of tectonic effects, such evidence requires a cross-shore profile displaying a series of increasing crest heights (for uplift) or decreasing heights (for subsidence), to rule out the many other dune height controls which include variation in: sediment supply; shoreline behaviour, or climate change influencing the wind regime, wave regime or temperature/rainfall (vegetative) regime.

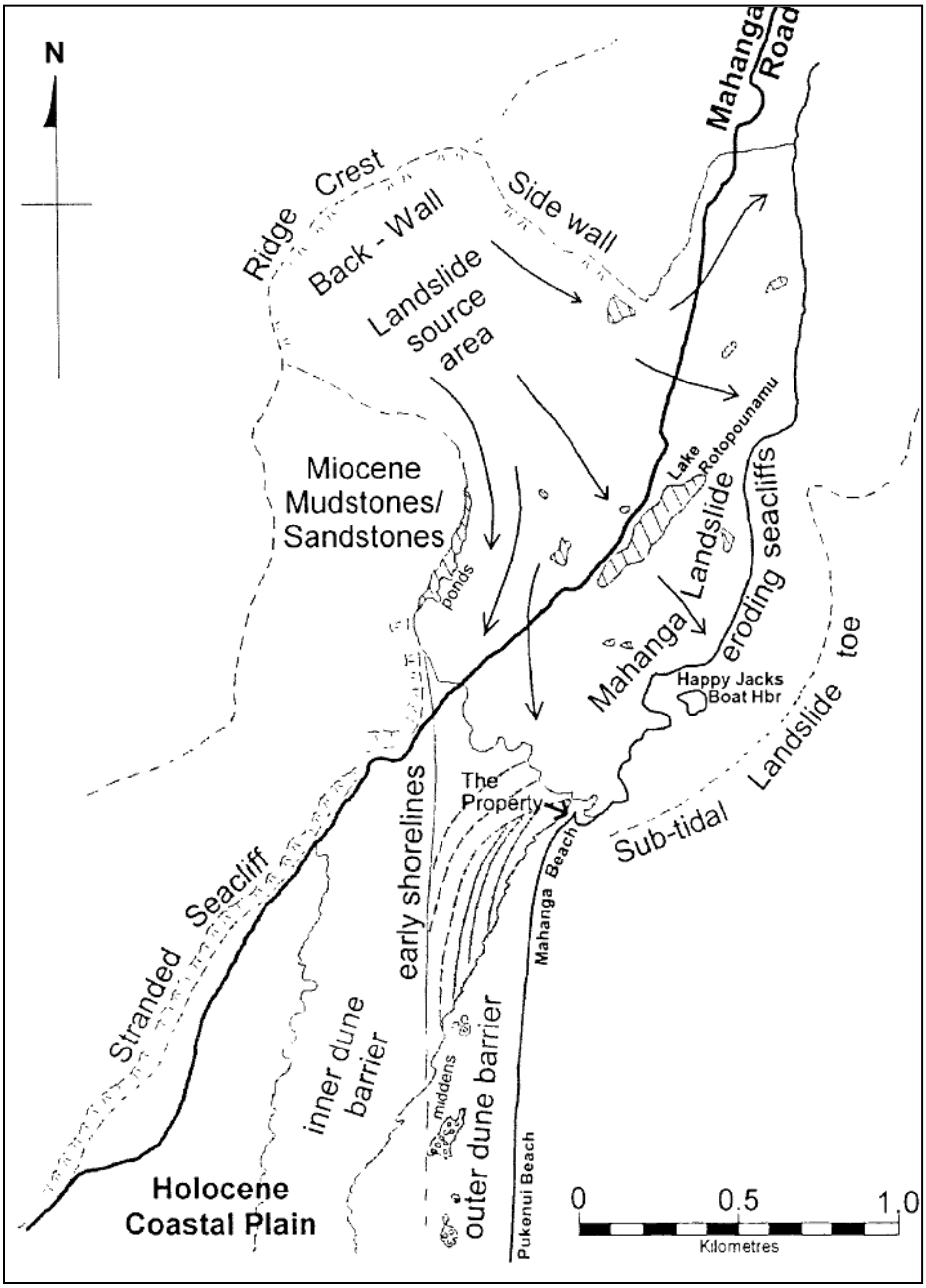


Figure 2.1 Map of geomorphological features of the wider Mahanga area prepared by Dr Jeremy Gibb (2005). The proposed subdivision is marked as "The Property". The "Holocene Coastal Plain" is the northernmost part of a sand tombolo which joined the Mahia Peninsula to the mainland some 2000 yrs ago. The Mahanga landslide occurred about 800 yrs ago, likely when the "early shoreline" was the actual shoreline. The subsequent seaward progradation of the shoreline (forming the outer barrier in the diagram) and depicted erosion of the landslide is indicative of a large-scale geomorphological system working toward achieving a state of equilibrium. The current focus of adjustment is in the vicinity of the Mahanga Inlet.

Accompanying the development a sediment deficit-driven "early shoreline" would have been significant cliff erosion of the hillside where the tombolo joined the mainland, and it appears that it was the loss of basal support that lead to the catastrophic "Mahanga Landslide" which is well depicted in Figure 2.1. Gibb (2002) provides a range of evidence to argue that the landslide occurred some 800 yrs ago and would have originally extended some 200 m further seaward. The landslide's footprint extends upward from a lower depositional lobe near sea level to the back scarp that reaches the top of the 270 m high hillside (slope distance ~700 m and width 450 m). The depositional lobe would have been some 700 to 800 m wide and 2.5 km around its seaward margin. It is noted that the hillslopes immediately north and south of the landslide also bear erosion scars and depositional features indicative of mass failure, indicating the high susceptibility of this coast to such events.

The landslide's depositional lobe formed a hummocky landscape with (pressure) ridges, lakes and shoreline recesses. The landslide material is colluvium, a weak mixture of fine sediments and sandstone boulders. It is relatively easily eroded by wave action which has resulted in the cliff and boulder beach topography evident today. The entire lobe is eroding and thus the cliff and fronting boulder beach are retreating landward. At the southern end of the landslide's coastal margin (Mahanga) a boulder spit has been migrating landward toward the tombolo's eastern ocean beach for at least the past 100 yrs.

Analysis of historical aerial photos indicates the tombolo's beach response to the landslide was the seaward adjustment of the shoreline, forming a barrier beach that partially impounded terrestrial runoff to form a lagoon between the landslide and advancing beach. This is a typical coastal sequence following rapid shoreline progradation. Wind-blown sand accumulated upon the stable prograded beach to form dunes, while ongoing channel migration and bank erosion within the inlet (typical inlet behaviour) ensured minimal dune development/preservation occurred adjacent to the inlet, although early aerial photo analysis indicates it was dune migration that caused the channel dogleg some 35 m above the ford. The landward lagoon would have progressively infilled to become a marsh/wetland and ultimately the low lying drained land evident today (~2 m above MSL). The inlet, that area where fresh and marine processes interact, will have reduced in size as the lagoon reduced in size and its throat is now approximately in the location of the ford.

2.4 Present and Future Geomorphology

The recent HBRC's *Hawke's Bay Wave Climate* study (MSL, 2011) shows Mahanga has a "moderate" oceanic wave regime with mean significant wave height (H_{sig}) = 1.1 m and maximum yearly significant wave height at the 5 m bathymetric contour = 3.5 m. That investigation did not include assessment of the longshore current regime which is an indicator of sediment transport potential and thus areas of likely erosion and deposition. With HBRC approval, we obtained the raw wave time-series data from MetOceans and calculated nearshore longshore sediment transport potential using the Kamphuis (2002) model which has been found to be in good agreement with physical and field studies without the need for extensive parameter calibration (Smith *et al*., 2003). Computation details are provided in Appendix B. Transport potential seaward of the Mahanga inlet (using porosity = 0.32 and D50 = 0.2 mm) gave north to south transport = 51,628 m^3/yr and south to north = 24,445 m^3/yr. A net southward drift potential of 27,183 m3/yr (i.e. 2:1 ratio) therefore exists in the vicinity of the proposed subdivision. This net transport drops to zero between 500 m to 1 km south of the inlet. This result indicates the finer sediment fraction from cliff erosion, has been, and will continue to be, a major sediment source for the beach to the south of the settlement.

The present inlet (area of marine-fluvial interaction) is here defined as extending from about the ford (landward throat) up to 200 m along Mahanga Beach based on shoreline analysis in Section 5 (Figure 5.3), and to the end of the boulder spit on the north/eastern side (<100 m). As is typical of inlets, the banks are not stable and, in the case of the Mahanga Inlet, are undergoing systematic (longer-term) migration (as detailed in Section 5 and depicted in Figure 2.2). In particular, the boulder spit is migrating landward (westward toward the proposed subdivision) at a linear rate of 54 m/100 yrs, and this increases to 119 m/100 yrs using a (statistically significant) nonlinear analysis (Figure 5.4). This is forcing the stream channel westward at 41 m per 100 yrs, and again the non-linear distance is substantially greater. While the western bank (adjacent to the proposed subdivision) has only been eroding at 24 m/100 yrs, this lag results from the system being "squeezed" over the past 72 yrs due to there having been a greater distance between the spit and western shoreline (compare the 1938 shoreline with the 2010 shoreline in Figure 5.2). This capacity is now all but gone so an enhanced erosion rate can be expected for the inlet's western embankment.

The erosion hazard analysis assumes the spit system (including the stream channel and western inlet shoreline) will continue to translate landward in the same manner as in the historical past. The viability of this assumption is addressed in Section 5.3.1.

The location of the proposed subdivision leaves it prone to inundation. Its low elevation and topographic containment (the elevated landslide to the north and dunes to the south) make it a floodway for terrestrial storm runoff flowing to the sea and also a pathway for storm and tsunami inundation traveling inland (conceptually illustrated in Figure 2.2). The following hazard assessment will show critical storm surges and tsunami can easily overtop the inlet bank fronting the subdivision (Sections 3 and 4) and propagate inland.

In summary, the local geomorphological system (summarized in Figure 1.2) is dominated by a dynamic inlet with one side controlled by an onshore-migrating boulder spit which, coupled with predicted climate change effects, is likely to drive future landward displacement of the inlet. In addition, the inlet is located at the mouth of an infilled lagoon with sides constrained by elevated topography (the landslide hummocks and sand dunes), forming a natural inundation pathway. This is the setting for the present hazard assessment .

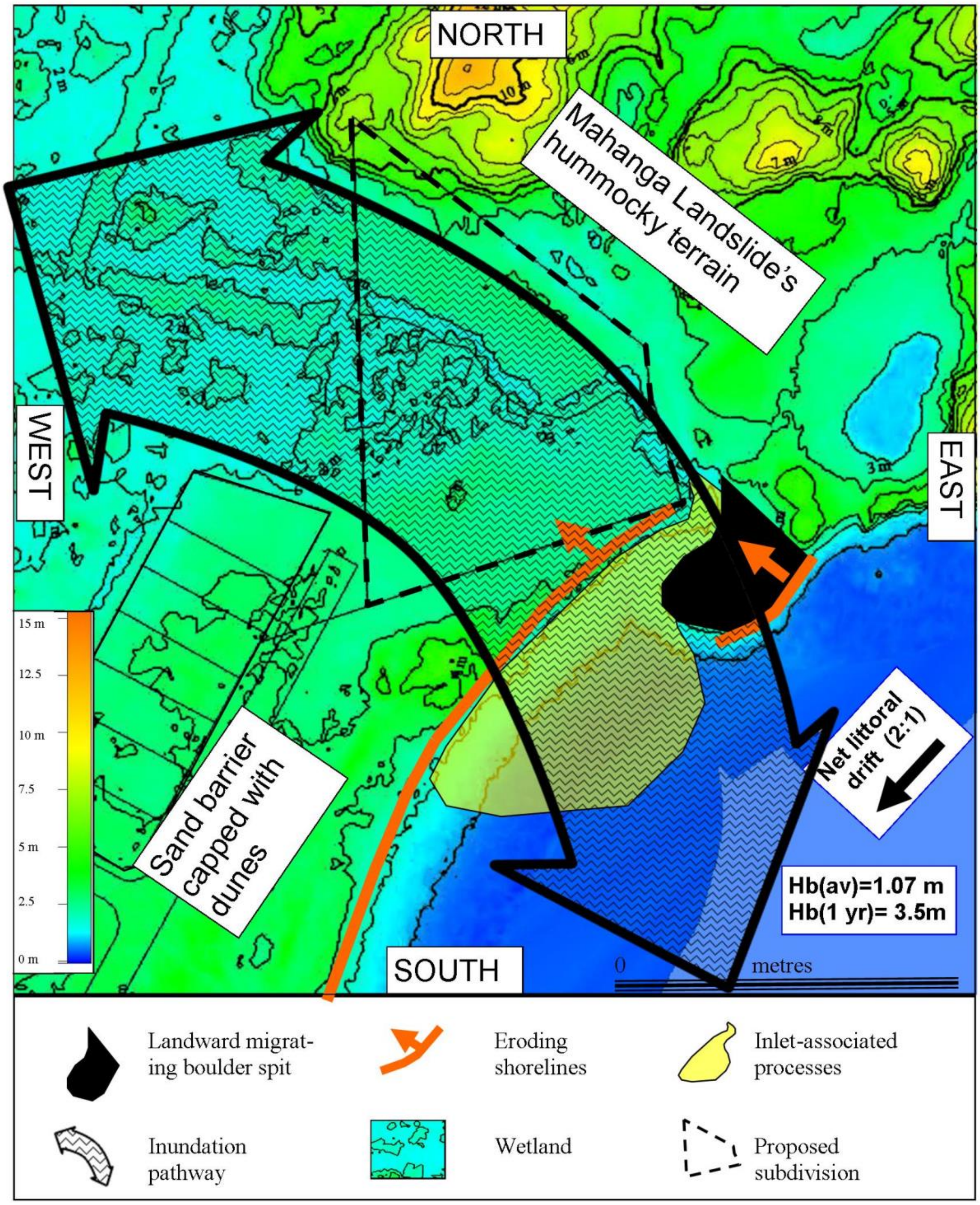


Figure 2.2 Geomorphological system diagram in the vicinity of the proposed subdivision, as depicted upon LIDAR-derived topography. Shown are the major geomophological units (described in Section 2), wave parameter values (Section 4), where Hb is breaking wave height, sediment transport parameters (Section 2), the location and direction of present erosion (Section 5 for detail) and inundation pathways (Sections 2, 3 and 4).

Table 2.1 Landform sensitivity to changes in climate change drivers. Source MFE (2008).

Landform type	Climate change sensitivity				
	Sea-level rise	Storm surge	Precipitation	Wave height	Wave direction
Simple cliff	High	Moderate	Moderate	High	Low
Simple landslide	High	Low	High	High	Low
Composite cliff	Moderate	Low	Moderate	High	Low
Complex cliff	Moderate	Low	High	High	Low
Relict cliff	High	Low	High	High	Low
Embryonic dunes	High	High	Low	High	Low
Foredunes	High	High	Moderate	High	Low
Climbing dunes	Moderate	Moderate	Moderate	Moderate	Low
Relict dunes	Low	Low	Moderate	Low	Low
Parabolic dunes	Moderate	High	Low	High	Low
Transgressive dunes	Moderate	Moderate	Low	Moderate	Low
River delta	High	High	Moderate	High	Moderate
Tide dominate delta	High	High	Low	High	Moderate
Wave dominated delta	High	High	Low	High	Low
Shore platform	High	Moderate	Low	High	Low
Sandflats	High	High	Low	High	Low
Mudflats	High	High	Low	High	Moderate
Pioneer saltmarsh	High	High	Moderate	High	Low
Saltmarsh	High	High	Moderate	High	Low
Sand beach	Moderate	Moderate	Low	Moderate	High
Gravel beach	Moderate	Moderate	Low	High	Moderate
Mixed beach	Moderate	Moderate	Low	High	Moderate
Composite beach	Moderate	Moderate	Low	High	Moderate
Boulder beach	Low	Low	Low	Moderate	Low
Barrier island	High	High	Low	High	High
Barrier beach	High	High	Low	High	High
Spit	High	High	Low	High	High
Cuspate foreland	Low	Low	Low	High	Low

3.0 TSUNAMI INUNDATION ASSESSMENT

3.1 Background

A tsunami is a very long period wave, or series of waves, generated when a large volume of sea (or lake) water is rapidly displaced. Tsunami are mainly generated by large earthquakes, landslides (submarine or terrestrial) or volcanic eruptions. The former two generators are very characteristic of the East Coast region (Mahia to East Cape) due to its tectonic setting in the Hikurangi subduction zone, its geological makeup of faulted and folded mudstone, and geomorpholocial processes of wave erosion removing basal support along the coastal hill country. The East Coast region is also exposed to distant tsunami generated in the Pacific. GNS (2006) shows this particular region is the most tsunami hazard-prone area in New Zealand. In addition, topographic constrictions further enhance the wave height and volume as tsunami waves near or reach the coast and propagate inland. Embayments and inlets such as at Mahanga can thus act to focus and increase wave dimension and hazard risk.

Following generation, tsunami waves radiate outward and occupy the whole ocean depth. While they have low amplitude within the ocean, they have long wavelengths (several km to over 400 km) and periods (minutes to hours), and travel at high speeds (500 km/hr in the open ocean). This speed and volume result in significant height gain at the coast, for example a half metre high tsunami wave in the ocean can increase to 10 m at the shore, and travel inland at up to 20 m/s (if topographically-controlled surging occurs), often with little loss of energy. If the wave encounters steeper terrain such as the sides of a river valley, runup will further elevate the water surface. Sand dunes were found to be particularly effective in dissipating tsunami waves in the 2004 Boxing Day event in the Indian Ocean.

While distant generated tsunami waves are tracked and adequate warning times given to coastal communities, locally generated waves, which are more likely to affect the Mahanga area, can reach the coast within minutes and with little or no warning (de Lange, 2003). While inundation itself may be hazardous to personal safety and property, it is the momentum of these waves that can be devastating as they flood inland. A comparison of predicted tsunami wave forces with minimum strength for houses prescribed in NZS3604:1999, shows that the strength of a well build house is likely to be exceeded by the forces exerted by even a 1 m deep tsunami (GSN, 2006). Recent results on human and vehicle stability in flood flows (Engineers Australia, 2010a and b) show that maximum depth * velocity values for persons to retain stability in uniform flow is between 0.6 and 0.7 m/s (0.4 for children) with maximum depth regardless of flow = 1.2m, and maximum flow regardless of depth = 3m/s. For vehicles, maximum depth x velocity values range from 0.3 to 0.6 with maximum depth regardless of flow = 0.3 to 0.5m, and maximum flow regardless of depth = 3m/s.

In the past, lack of tsunami-knowledge meant that this phenomenon was often described but not accounted for within hazard assessments and management, and this occurred in the previous Mahanga hazard assessments. However, following the 2004 Boxing Day tsunami the Institute of Geological and Nuclear Sciences (GNS) produced a detailed report (GSN, 2006) that summarized the current state of knowledge of tsunami and used current modelling techniques to determine return period probabilities and the associated level of risk at a national and regional level. NZCPS 2010 requires tsunami to be assessed as part of a hazard analysis and the GSN (2006) guidance forms the basis of the present assessment.

However, reviewer Professor James Goff comments that:
"A paper by Satake and Atwater in 2007 – one that has been endorsed by many researchers, points out that we are always under-estimating the size of subduction zone events, so the GNS work is also under-estimating".

3.2 Assessment

The present assessment uses both 50 and 100 yr return period events with even higher return period heights also be listed. GSN (2006) provides current wave height return periods for Gisborne and these are indicative of likely values for use in the tsunami hazard assessment for Mahanga. In particular, the 50 yr return period mean estimate = 2.9 m (+ 1 standard deviation = 4.4 m), and the 100 yr return period = 4.2 m (+ 1 standard deviation = 6.2 m). As the inlet topography could locally enhance tsunami wave height and momentum, and as the level of uncertainty associated with the GNS modelling procedures is relatively high, addition of the one standard deviation should be seriously considered as likely.

Professor Goff notes:
"I think this (mean plus one standard deviation) is reasonable. The area is more exposed than others in NZ to tsunamis from both local (subduction zone, submarine landslide, and other fault ruptures) and distant – South American in particular, tsunamis".

The resulting (GNS 2006) values of 4.4 m and 6.2 m for the 50 and 100 yr return period events are reasonably consistent with the most significant historical event to affect this area which occurred on 28th March, 1947. At that time a seemingly minor earthquake generated a tsunami which, 30 minutes later resulted in a 4.5 to 6 m wave at Mahanga that flooded inland for ~200 m (Gisborne Herald, 29-31st March, 1947).

Given that the 100 yr return period value has a 63% chance of occurring within a 100 yr period (the required minimum hazard assessment period under the NZCPS 2010), and that the difference between a 50 yr and 100 yr event is substantial (approx 30%), higher return period tsunami estimates should also be considered. Mean to one standard deviation tsunami height ranges for a 200 yr return period event (40% chance of occurring within the next 100 yrs) = 5.7 to 8.3 m, a 500 yr event (18% chance) = 8.0 to 11.6 m and a 1000 yr event (10% chance) = 9.9 to 14.5 m.

Tsunami inundation depths and velocities across the proposed subdivision for 100 yr return period waves were assessed using a LIDAR-based DTM and the results depicted in Figure 3.1. No account has been taken of reflection-dissipation by the foredune fronting the south of the subdivision as this feature may well be removed by predicted erosion (Section 5).

Two propagation models were used to derive these propagation curves which are considered to depict a credible inundation envelop. The lower surface (depths **2.4 to 3.6 m**) ignores the influence of ground topography so is considered a lower estimate, while the upper surface (depths **5.3 to 5.8 m**) is topographically adjusted and thus makes some allowance for local runup and other topographic focusing. Wave speeds of ~**11 m/s and 15 m/s** respectively were calculated using equation $V = 2(g*D)^{0.5}$ where: V = inundation velocity; D = inundation depth; G = acceleration due to gravity 1 (GNS 2006).

Professor Goff notes:
"I suspect the best estimate is probably somewhere between the bathtub (horizontal line) and the topo superimposed line".

3.3 Tsunami risk management

The above inundation impacts present significant risk to the safety of future inhabitants and property for tsunami with magnitudes as low as a 50 yr return period event. Given that the site of the proposed subdivision lies within an inundation pathway and that there will be little, if any, warning of the type of tsunami most likely to have higher impact, pedestrian escape as a mitigatory measure may well not be viable. The increasing vulnerability to greater events which have a reasonable likelihood of occurring within the assessment period is of even greater concern. The feasibility of designing tsunami resistant buildings, even for the lower return period events, appears highly questionable, and the 2011 Japanese experience found structures can magnify impacts by channeling flows, increasing the inundation reach, and providing tsunami debris materials (Gomez et al., 2012).

Professor Goff's concluding comment:
"Why deliberately put people in harm's way?"

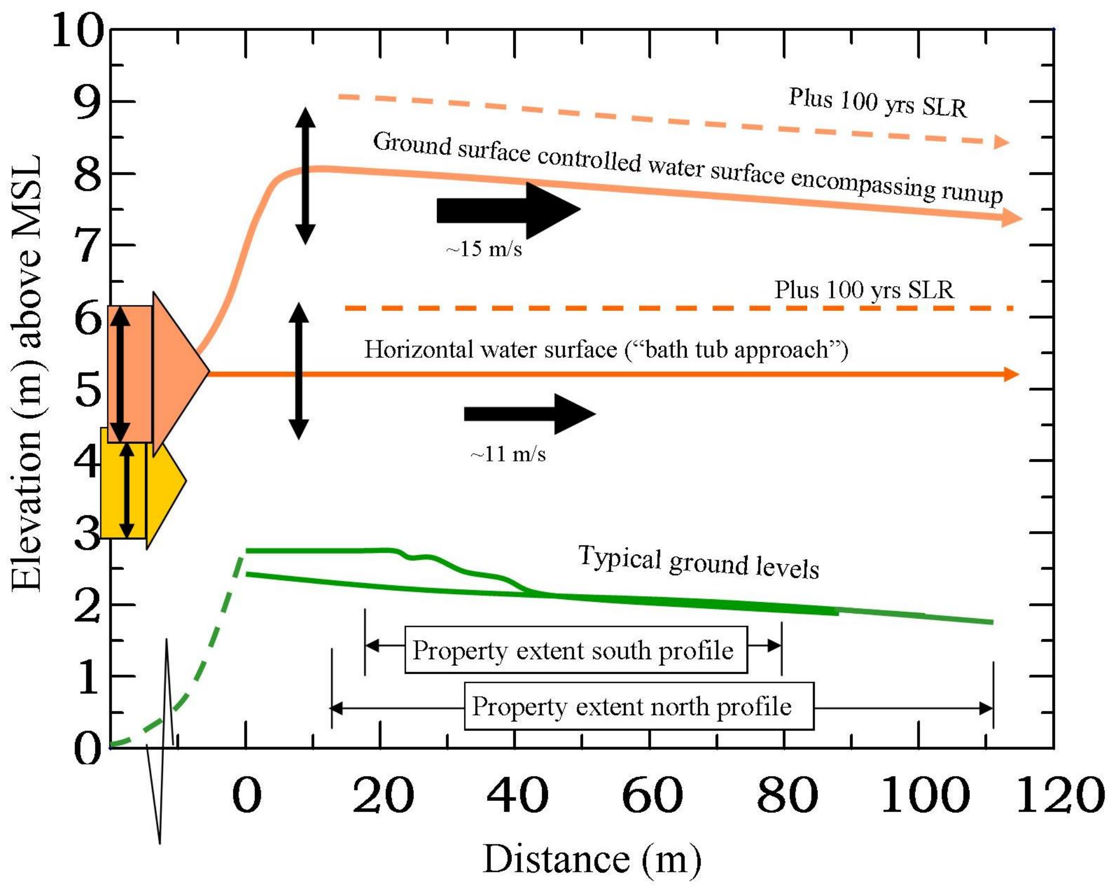


Figure 3.1 Bracketing tsunami inundation levels and speeds across the proposed subdivision for 100 yr return period event with bracketing inundation models (see text). Levels incorporating 100 yrs of SLR depicted by dashed lines. Fifty year return period waves marked on left (yellow)

4.0 STORM INUNDATION ASSESSMENT

4.1 Background

Under storm conditions, several atmospheric and marine processes typically elevate sea level at the coast and thus increase the risk of inundation hazard on otherwise dry land. The parameters which, in combination, define the inundation level at the coastal margin comprise the following which are described in WRL (2010):

- Astronomical tides (predicted tides);
- Longer-term sea-level fluctuations (seasonal, ENSO, IPO);
- Storm surge (wind setup, barometric pressure, coastal trapped waves and long waves from distant weather systems), and
- Waves effects (wave set-up and wave runup).

In addition, the extent of inundation is affected by catchment runoff (which is often somewhat elevated during storm events), and other factors such as driver sources, inundation pathways and overland extent which were not given adequate consideration in past hazard assessments and which are now mentioned in the NZCPS 2010 Policy 24(c) and 24(d).

Of relevance is the wave spectum (frequency mix) coupling with local topography/bathymetry which can result in storm inundation occurring as distinct and potentially destructive pulses sweeping into an inlet at about 2 to 10 minute intervals. To determine how this may apply at the Mahanga inlet, we carried out "time-stack" analysis of video data collected at the Mahanga inlet during a moderate storm on 30th March 2011. Such an approach is described in Shand and Bailey (1995), and Shand et. al. (2011). The resulting images constructed from dual cameras located on the Mahanga headland and in the bay, are shown in Figure 4.1. Low frequency runup pulses are evident at 2 to 6 minute intervals with these pulses propagating inland beyond the line of mean runup for durations of about 40 seconds.

While such lower frequency pulses incorporate the volume of several incident waves, they can still decay as they travel inland - in marked contrast to tsunami. The hazard risk thus decreases with increasing distance from the shoreline and this is defined within the present hazard assessment (Section 4.2.9).

It is particularly important in a storm inundation assessment to incorporate the effects climate change on the various inundation drivers (NZCPS 2010, Policy 24e). While basic guidance is now available on the potential effects of climate change on storm inundation, (e.g. MFE, 2008), there is still considerable uncertainty as the nature and timing of such effects, and NZCPS 1994, Policy 3.3.1 and NZCPS 2010, Policy 3 accordingly require adequate precaution be taken when dealing with processes affecting the coastal environment.

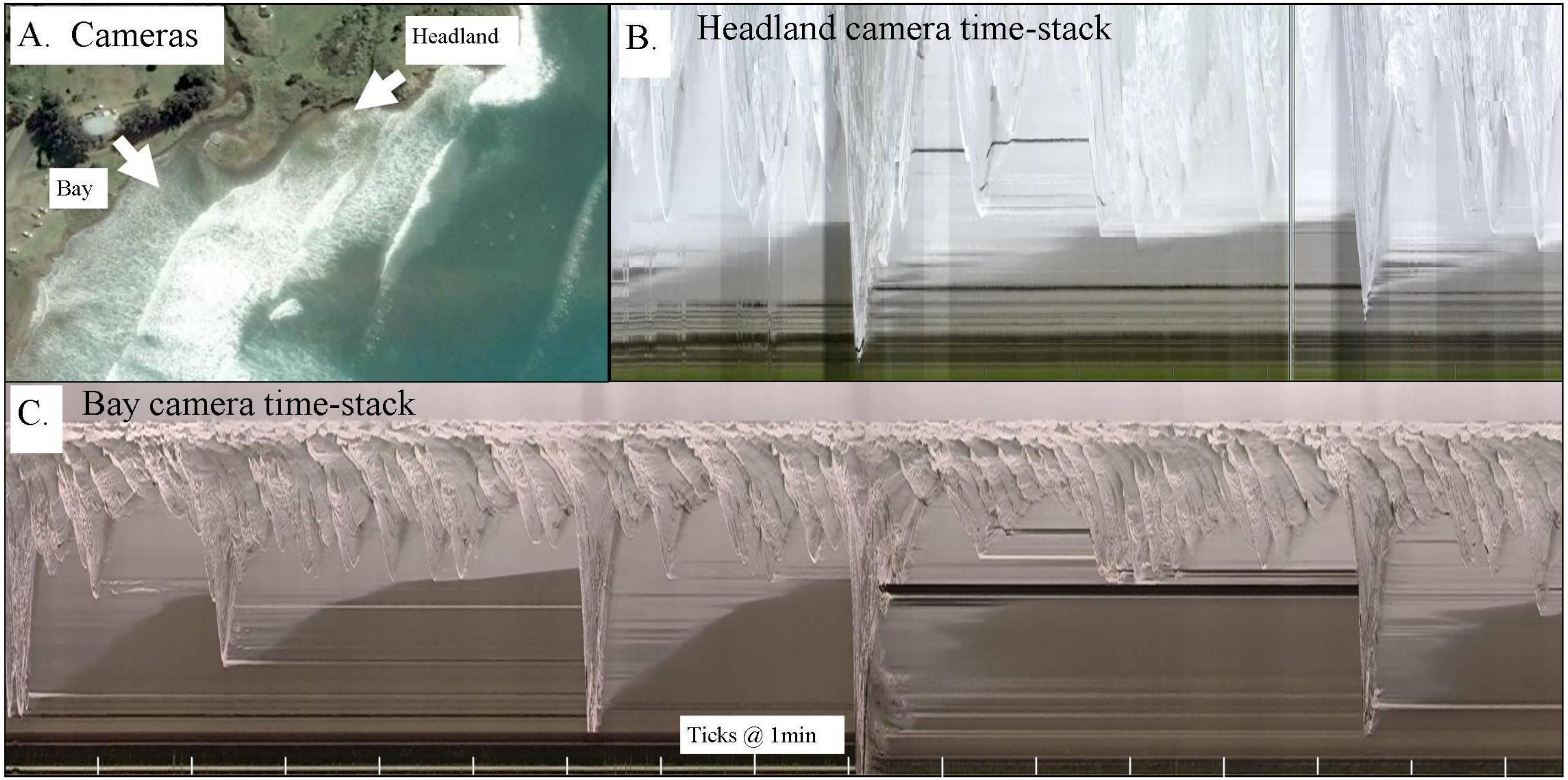


Figure 4.1 Synchronised "time-stack" images constructed from video tapes with cameras located on headland and in bay (orientations in A). Time-stack cross-shore distances on vertical axes non-linear (B and C). Time-stacking involves the extraction of a single line of pixels from sequential images and chronological placement along the horizontal axis to provide a record of change over time which can then be analysed to extract spatially and temporally varying processes information. Video tapes provided by members of MET under CSL instruction at high tide during a moderate storm on 30 March 2011. Storm wave conditions: Hsig=3.0m, Hmax=5.5 Tp = 11.5s source MetOceans Ltd: Tide = 0.45 m (neap). Wave pulses running up dune face and reaching landward extent of inlet.

Deriving storm inundation component values is not straightforward and has been evolving as available data improves. The advent of continuous data collected over several years and eventually over decades, is enabling ever more accurate derivation of extreme values (return period magnitudes) and, of particular importance, the combination of parameter values to give the correct combined return period for a storm event. A brief history of this process is provided in Appendix A

In the present assessment, derived parameter values are based on recently available continuous water-level data and hindcast wave records using return period ratios given in CIRIA (1996), such that combinations provide a dependency-adjusted total shoreline runup for a 100 yr return period event. Note that the 50 yr return period values were derived by down-scaling the 100 yr values such that this combined 50 yr probability was achieved. Also note the return period and AEP reciprocal relationship detailed in Section 1.5, for example a 100 yr return period event is equivalent to a 1% or 0.01 AEP (Annual excedence probability) event. Details of the storm parameter value derivations are given below (Section 4.2) and results summarized in Table 4.1. Also listed in Table 4.1 are the values used in the previous Mahanga assessments which tended to use maximum or estimated 50 yr and/or 100 yr return period component values which when added together gave combined inundation levels in excess of 50 or 100 return periods (see Appendix A).

Storm inundation analysis output may be categorized by equation 4.1 to 4.3 and Table 4.1 has been set out accordingly:

- *Static Level = tide + storm surge + longer-term sea-level fluctuations + wave setup* *(4.1)*

- *Total present runup = static level + wave runup*[1] *(4.2)*
  *(*This gives the time-varying inundation level at the present time)

- *Total future runup = total present runup + SLR* *(4,3)*

It should be noted that to allow for climate change uncertainty, parameter values for the present sea level scenarios (Table 4.1A) incorporate predicted climate change driver changes for wind and waves (from MFE, 2008), as these will occur throughout the assessment period.

TABLE 4.1 Storm wave inundation parameters values and combination for stipulated AEP for the present CSL assessment, and also for previous Mahanga assessments. The current sea level scenario is in A, with addition of assessment period SLR scenario in B. Note for CLS assessments, the present sea-level estimates (A) include climate change driver adjustments as these can occur during the assessment period.

A. Present sea level

	T&T'04	Gibb'07	Cardno'08	CSL'11	CSL'11
Combined AEP (Return Period)	2% (50 yr)	2% (50 yr)	? ?	2% (50 yr)	1% (100 yr)
MSL (HBVD+10m)	0	0	0	0	0
Tide: MHWS	1.0	0.7	0.7	0.86	0.9
Sea-level fluctuations (m)	0.2	0.2	0	0.15	0.15
Storm surge (m)	0.9	0.9	0.9	0.45	0.47
Wave setup (m)	1.4	1.4	0.6	1.06	1.11
Static level	3.5	3.2	2.2	2.52	2.63
Wave runup (m)	2.3	0	0	2.55	2.60
Total present runup (m)	5.8	3.2	2.2	5.1	5.2

Note Gibb 2007 included 0.5 m for contemporaneous freshwater flooding (see text).

Field evidence	Gibb 2007	CSL 2012
	4.8 m NthPovBay	4.8-5.2 m Otaki Beach
	4-5 m SthPovBay	H1 = 4.6 m, Tp=9.8, Upper foreshore (UFS) = 0.044
	5 m Mahanga	c.f Mahanga Beach statistic
		H1 = 4.2 m, Tp=10.5, UFS = 0.043

B. Plus sea-level rise at end of assessment period

	T&T'04	Gibb'07	Cardno'08	CSL'11	CSL'11
Combined AEP (Return Period)	2% (50 yr)	2% (50 yr)	? ?	2% (50 yr)	1% (100 yr)
Prediction period	2100	2100	2100	2112	2112
Plus SLR (m)	0.5	0.8	0.5	1.0	1.0
Static Level	4.0	4.0	2.7	3.52	3.63
Plus Wave runup	2.3	0	0	2.55	2.6
Total future runup	6.3	4.8	2.7	6.1	6.2

4.2 Assessment

4.2.1 Datum

Based on IPCC (2007), MFE (2008) advise that inundation including SLR are to be measured relative the mean level of the sea between 1980 and 1999. Using 1997 harmonic constituent values for the Port of Napier, this value is 2 cm below the Hawke Bay vertical datum's MSL = 10 m.

4.2.2 Tide Level

Spring tide level (MHWS) is the favoured level for use in a 100 yr storm inundation assessment (MFE, 2008). About 18% of tides exceed this value which has a return period of 0.0075 yrs. The current LINZ value for MHWS at Napier (the nearest standard port) is 0.9 m (MSL). The adjusted value for use in the 50 yr storm inundation assessment is 0.86 m (MSL).

Previous Mahanga hazard assessments used MHWS values which conflict with my values; the T&T'04 value of 1.0 m apparently includes a rounding adjustment, while the Gibb and Cardno assessment values (0.7 m) apparently come from an earlier Nautical Almanac.

4.2.3 Longer-term sea-level fluctuations

Mean sea level may fluctuate over months to decades due to several longer-term processes such as seasonal weather variation in temperature and windiness, ENSO-based climatic oscillations and IPO shifts. While the limited long-term, open coast sea-level records available suggest inter-annual elevation changes of up to 0.2 m could occur (Bell et al., 2000), a lesser value (0.15 m) will suffice for a joint probability-based combination given the relative independence of these processes. Previous Mahanga hazard assessments either used a longer-term sea-level fluctuation of 0 or 0.2 m.

4.2.4 Storm Surge

Storm-surges are temporary increases in ocean water level (peaking from say 12 to 24+ hours) primarily associated with low barometric pressure which allows the water surface to rise, and onshore-alongshore blowing winds which cause water to pile up at-along the coast. In addition, embayed coastal configurations can further enhance the storm surge level by constricting the flow. This latter aspect may well be relevant at the proposed subdivision given firstly that it backs a small inlet and secondly, at a larger scale, the northern Mahia Peninsula-tombolo-mainland coast also presents an embayment orientated to the northeast..

Previous Mahanga hazard assessments all used a storm surge value of 0.9 m based on observed levels from around the New Zealand coast (see Appendix A). However, recent analysis of new sea-level data show this is higher than the actual value for an overall 50 or 100 yr event. While no NIWA long-term sea-level data is available for the Poverty Bay region, available records from other North Island sites (Anawata by Piha, Kapiti Island

and Moturiki Island in the Bay of Plenty) show broad similarity so the maximum computed value for different return periods was used to provide a conservative value for the Mahanga coast. An additional 10% was added to allow for any local topographic/regional climate effects ("best professional judgment", which is conservative as the value could in fact be a reduction), plus a further10% was added to allow for changes in climate change drivers in keeping with the recommendations in MFE (2008). I consider these values are suitably conservative to account for future uncertainty.

A storm surge return period of 2.5 yrs (0.47 m) combines with MHWS (plus yearly waves - see below) to yield a combined 100 yr return period. The storm surge value for the 50 yr return period combination is 0.45 m.

4.2.5 Wave Effects

Waves affect inundation in two different ways:

- wave setup refers to water becoming elevated to balance the onshore directed momentum flux which occurs following wave breaking, and is a static water level component (NB equation 4.1) , and

- wave runup refers to the propagating wave form directed forward and upward across the beach, into inlets, and subsequent bank overtopping (NB equation 4.2).

Wave data.

Wave effects were assessed for the coast fronting the proposed subdivision by applying a range of models to a hindcast wave data set recently supplied to the HBRC by MetOcean Solution Ltd (MetOceans, 2011). This data set comprises 12 yrs of 3 hourly wave and wind data provided by the National Oceanic and Atmospheric Association (NOAA) which were transformed to the 5 and 10 m contours by MetOceans using the SWAN numerical model, and then extreme values for different return periods were calculated. The wave parameters are summarized in Table 4.2. While the numerical data have not been instrument verified, they are within 9 to 14% of satellite-derived data (MetOceans, 2011) so an extra 10% was added for values used in the hazard assessment. An additional 10 % was added for climate change as recommended in MFE (2008). Note that yearly waves are used in the hazard analysis to produce a combined 100 yr return period event (using joint probability combination), while 0.75 yr waves are used for the combined 50 yr return period event. These values are 4.2 and 4.0 m respectively (including the 20% uncertainty addition) at the -5 m contour. Note under extreme storm conditions waves can be expected to break at -5 m, so these are $H_b$sig values.

Table 4.2 Wave height values at -5 m contour (return periods ~$H_b$sig) off Mahanga Beach.
Source MetOceans (2011).

	mean	95%	6 month	9 month	1 yr	20 yr	100 yrs
Wave heights (m)	1.07	2.27	3.25	3.35	3.5	4.02	4.12
Wave periods (sec)	10.5	16.9					

Wave setup

Wave setup was derived using equation ll-4-25 in USACE (2003), this being an industry-wide guiding reference. Using the adjusted yearly wave heights and a surfzone slope of 0.01 (from the Mahia Bathymetric Chart 1 :200,000 Coastal Series, 2nd edition), the computed setup values were 1.11 m and 1.06 m for use in the 100 and 50 yr combined return period inundation assessments

Previous Mahanga inundation assessment set-up values used by T&T'04 of 1.4 m was about 0.3 m higher than that derived in the present assessment. T&T'04 used the same equation but an earlier hindcast wave data set and transferred results from a different location (the northern Hawke Bay coast). Gibb 2007 used the same T&T'04 value. Cardno (2008) used 0.6 m, a reduction of almost 60%, and appears to have based this on an assumption that the Mahanga coast has some sheltering from the larger southerly quarter waves used to calculate the T&T'04 value. Their argument ha some merit; however, the hindcast data now available shows the an over-estimate to be only about 20%.

Wave runup

Runup (including setup) was derived using the model of Mase (1989) which was found in a recent comparative study (Shand et al., 2011) to be the most accurate empirical model (of the several available) for extreme runup on open coasts, such as at Mahanga The MetOceans yearly wave heights at the -5 m contour were deshoaled to deep water using the Delft Coastal and River Engineering Software System (CRESS) to give the required $H_o$sig parameter values. The average upper beach slope is 0.043 as derived from 8 yrs of beach profile data surveyed by the HBRC. The resulting runup values (less set-up) were 2.55 m and 2.6 m for use in the 50 and 100 yr combined return period inundation assessments.

The previous Mahanga inundation assessment runup value used by T&T'04 was 2.3 m which is about 10% under the value derived in the present assessment. This is still a satisfactory result considering T&T'04 simply used a co-efficient-based approach. Gibbs 2007 and Cardno 2010 did not include the runup component in their assessments and gave no explanation. While runup (and overtopping then inland propogation/dispersion) is not part of static inundation, it is a particularly destructive process facilitated by the static water-level elevation and is included in coastal hazard assessment literature and guidance manuals.

4.2.6 Sea-Level Rise (SLR)

The NZCPS 2010 clearly directs hazard assessment to be carried out for at least 100 yrs. The SLR projection must therefore be until at least 2112. The official guidance for predicted SLR over 100 yrs is contained in the MEF (2008) manual and for locations where rising sea-level will have consequence, a value of 1.0 m for the period 2110 to 2120 is given. While 2112 is earlier in the 2110 to 2120 decade, more recent research (Shand

and Manning, 2010; RSNZ, 2011) indicate a value of at least 1 m should be used for subdivisions out to 2110, let alone 2120.

The previous Mahanga hazard assessments used projections to 2100, which is now less than 90 yrs from the present time. Gibb (2007) used the value of 0.8 m appearing in MFE (2008) after contacting the authors during the preparation of that manual. T&T'04 used a value of 0.5 m which was recommended in official guidance at that time. Cardno (2008) used a value of 0.5 m based on the argument that it was unlikely maximum impact would occur as SLR would not reached this value for 100 yrs. This argument has merit in terms of event combination probabilities. However, in coastal hazard assessment there is emphasis on higher impact/lower probability situations coupled with high uncertainty regarding SLR projections, so it has become convention to include the 100 yr projection value within inundation assessments, and hence the MFE (2008) guidance values In some cases councils relax this requirement for hazard zones containing existing buildings; however, this does not apply for new subdivision.

4.2.7 Freshwater flooding

Gibb 2007 included 0.5 m for stream flood flow overtopping the channel at the same time marine-driven extreme inundation occurred. By contrast, the other assessments including the present one, have not included a value for this parameter. In the Mahanga case, we consider the likelihood of significant freshwater induced inundation above the channel banks coinciding with short-lived extreme oceanic-induced inundation to be low. However, water ponding on the subdivision reduces surface roughness and thus enhances overland penetration and such allowance was incorporated in dispersal modelling described below in Section 4.2.9.

4.2.8 Total runup

The values in Table 4.1A show total runup ranging from 2.2 (Cardno) to 5.8 (T&T), with our estimate being 5.2. The lower values stem from reduced estimates of wave effects and the higher value from an increased storm surge estimate. Incorporating SLR lifts the range to 2.7 (Cardno) to 6.3 (T&T) with our estimate of 6.2 m approximating the higher value due to the higher SLR value now required. This is a typical pattern we have observed in other hazard assessment comparisons, i.e. earlier investigations having relatively higher storm surge values and relatively lower SLR values, but summing to broad equivalence.

4.2.9 Landward inundation dispersion

Static inundation depth is horizontal across the proposed subdivision for both inundation scenarios in Table 4.2A and B and this is illustrated by the horizontal dashed lines in Figure 4.2. Under present sea level most of the proposed subdivision will be covered with about **0.5 m** of water with the most serious inundation occurring to the lower lying northern portion of the subdivision. Sea-level rise increases the inundation depth to about **1.5 m** in 100 yrs time by which time there will be minimal ground resistance to wave propagation across the property.

By contrast, total runup pulses reduce in height as they propagate landward across the site. Such dispersion was modelled by applying FEMA (2005) equation D.3.4-39 to the total runup output. This equation is a derivation of the general dissipation equation in Cox and Machemehl (1986) with the addition of a friction-based calibration coefficient suited to local conditions. Note that the co-efficient reduces by an order of magnitude from a wave encountering vegetation to one encountering a water surface. Using total runup height above ground level at the front of the property, the wave pulse period (as measured in the storm video time-stack described earlier), and the designated friction coefficients for vegetation, surface water etc, bore height was calculated across a (LIDAR based) representative northern and southern ground profile and the result is depicted in Figure 4.2. Estimates of surge speeds have also been included, these being based on wave bores crossing shore platforms (Shand et al., 2009).

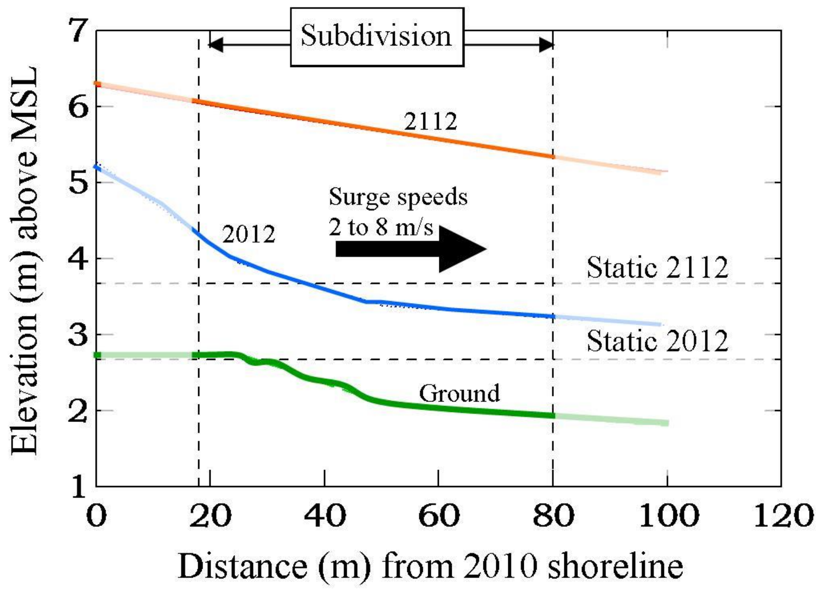
Under present sea-level (Table 4.2A values) distinct differences are evident between the northern and southern sectors, with the increased ground elevation to the south causing rapid reduction in surge (bore) height of 50% (**2.6 to 1.3** m) between the shoreline and rear of the subdivision. However, the lower northern sector has higher initial surge (bore) height (3m) and this drops by only 25% (**3 to 2.3** m) at the rear of the subdivision. With the increase in sea level of 1 m (the 2112 scenario), the dissipation advantage of the southern sector has gone and the relative depth reduction for the whole site is only about 10% with the average depth being **3.5 m**.

4.3 Storm inundation risk management

Under present sea level, inundation depths present a significant hazard to persons in particular (1.3 to 2.3 m), especially when coupled with lower-end velocity estimates of 2-5 m/s. However, inundation levels with the inclusion of SLR reduce cross-shore dissipation and the average depth increases to 3.5 m. Mr Carley noted in his review that the propagating bore crest determines the design floor level. The building would therefore need to be at least 3.5 m above ground level, say 4 m including free board. When coupled with upper-end wave surge speeds (5-8 m/s), this scenario presents a serious hazard to both persons and property.

As noted with tsunami, structures can further magnify environmental impacts by channeling or impounding flows within the inundation pathway, and providing debris which can cause additional damage and destruction.

A. Southern Profile through subdivision section 4 to 2



B. Northern Profile through subdivision section 3 to 1 to 2

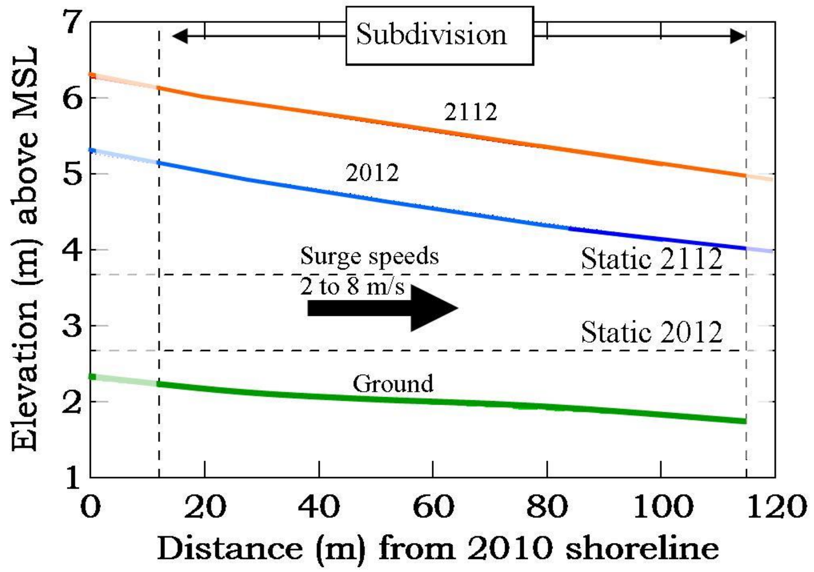


Figure 4.2 One hundred year return period storm inundation levels at northern and southern sectors across the proposed subdivision for 2012 and 2112 sea-levels. Wave pulse speeds also shown.

5.0 EROSION ASSESSMENT

5.1 Background

While the previous erosion assessments carried out for Mahanga (T&T'04 and Gibb 2002, 2005, 2007 and 2008 tend to demonstrate increasing robustness in terms of components derivation, they nonetheless have important deficiencies.

Pre-NZCPS 2010 approaches to determining coastal erosion hazard were often based on summing the following components such that the width of the coastal erosion hazard zone is represented by equation 5.1

$$CEHD = LT + ST + RSLR + DS + CU \tag{5.1}$$

Where:

CEHD= *coastal erosion hazard distance*

LT = *longer-term historic shoreline change, typically defined by a trend.*

ST = *Shorter-term shoreline change, typically defined by fluctuations*

RSLR = *Shoreline retreat associated with sea-level rise (SLR)*

DS = *Dune stability - a post-erosion stability adjustment.*

CU = *combined uncertainly, typically a multiplicative factor of safety*

The NZCPS 2010 includes these components but now for an assessment period of at least 100 yrs. It also has additional requirements, notably an increased understanding of the underlying geomorphological processes/sediment dynamics including longer-term shoreline change and human influences, and greater regard to the associated effects of climate change (see discussion in Section 1.4). These matters are particularly significant in the vicinity of inlets (river and stream exits) which are typically the most unstable and dynamic coastal features.

5.2 Shoreline data

Section 2 has already described key aspects of the underlying morphology and processes as they relate to coastal erosion at this site. Fundamental to quantifying the erosion associated with these processes is a shoreline analysis using the longest and most comprehensive data set available. The list of source data used in the present assessment, including that used in earlier assessments, is set out in Table 5.1. By using survey plans, satellite imagery and additional historical aerial photos, the data base for my analysis was extended back in time some 45 yrs. It also included 10 additional intermediate samples which enabled a much improved understanding of the coastal processes operating at this site and more reliable quantification of erosion parameters. To confidently predict over a 100+ yr period, a data set of at least that duration is desirable and closer sampling enables mid to shorter-term behaviour to be better defined.

Table 5.1 Details of data used in the present and earlier erosion hazard assessments at Mahanga

Year	Data Type	Reference	Source	Georefn Acc (±m)	Application
1899	Survey Plan	ML 204	LINZ	2 to 5 m	*
1814	Survey Plan	ML 1077	LINZ	2 to 5 m	*
1938	Aerial Photos	SN 77	NZAM	1 to 1.5 m	*
1942	Aerial Photos	SN 226	NZAM	0.5 to 1 m	* # ## <>
1962	Aerial Photos	SN 1455	NZAM	0.5 m	* # ##
1970	Aerial Photos	SN 3299	NZAM	0.5 m	* # ##
1971	Survey Plan	DP 12744	LINZ	0.3 to 0.5 m	*
1973	Aerial Photos	SN 2637	NZAM	0.5 m	*
1979	Aerial Photos	SN 5325	NZAM	0.5 m	* # ##
1983	Aerial Photos	SN 8260	NZAM	0.5 to 1 m	*
1990	Aerial Photos	SN 9101	NZAM	0.5 m	* ##
1995	Aerial Photos	SN 9452	NZAM	0.5 to 1 m	*
2002	Aerial Photos	SN 50149C	NZAM/WDC	0.125 m	* # ## <>
2003	LIDAR	81016801NHB	GeoScan	<0.55 m (z=0.15 m)	* ##
2004	Satellite Image	1010010002D52D0I	DigitalGlobe	2.5 m	*
2005	Satellite Image	1010010004A09C0A	DigitalGlobe	2.5 m	*
2006	Satellite Image	BH43	Kiwi Image	0.6 m	*
2008	Aerial Photos	SN 50635D 2132-46	NZAM	0.15 m	##
2010	Aerial Photos	SN 50864C	NZAM	0.1 m	*

Notes

Gibb (2005); ##Gibb (2008); <>T&T (2004); * CSL (2012)
T&T (2004) also used beach profile data from Waimarama

All these spatial data sets were analytically georeferenced to the same co-ordinate system thereby enabling precise overlay to quantify geomorphological and shoreline change. Shoreline indicators were the cliff-top, the vegetation-front and the mean high water line (MHWL). Georeferencing was carried out in house by CSL. While accuracy in rural locations can be hampered by lack of control points, this was not the case at Mahanga as there is ample spatial survey information associated with the settlement which was evident on the earliest plans, and for aerial photos, some fencing was common to all historical images.

The historical shorelines are shown in Figure 5.1. Figure 5.1A depicts samples derived only from aerial photos (1938 to 2010), and used the clifftop along the headland as the shoreline indicator, and the vegetation-front for defining the spit, inlet and beach shoreline. Figure 5.1B depicts shorelines from the inlet along Mahanga Beach based on the MHWL and obtained from the early survey maps (1899 and 1914) and the aerial photo series (1938 to 2010). An experienced practitioner can identify the MHWL from sand texture/tone contrast landward (as at higher elevation) of the very distinct water saturation line about MSL which is visible on lower tides. While the vegetation-front is the preferred shoreline, as its higher beach elevation results in less noise from marine processes, the longer MHWL-based data set must also be considered when defining the long-term trend.

Shoreline analysis was carried out by measuring shoreline distances along several carefully selected transects (see Figure 5.2) using the initial shoreline as the reference datum for the measurements. The 1938 and 2010 shorelines are also depicted in Figure 5.2 and it can be seen that change in the location of the boulder spit is particularly notable. It is clear from Figure 5.1A and Figure 5.2 that the spit is systematically migrating landward towards the inlet shoreline fronting the proposed subdivision. Active spits are key indicators of morphological change and it is particularly important in erosion hazard assessment to define their behaviour as accurately as the available data permits. The spit behaviour was emphasized earlier in Section 2 when describing the geomorphological system.

5.3 Assessment

5.3.1 Longer-term erosion trend

The shoreline data sets for each transect were subject to linear regression analysis to determine longer-term rates of change and shorter-term behaviour. Longer-term cliff-top and vegetation-front rates are presented in Figure 5.3. Also plotted in Figure 5.3 are results repord in earlier assessments (T&T'04, Gibb 2005 and Gibb 2008), with Gibb 2007 applying the Gibb 2005 rates. The statistical regression uses all (shoreline) data points from a particular transect to establish the rate of change (trend) over the sampling period. This approach involves fitting linear (and nonlinear) statistical models to each data-set to define any underlying trend. Such an approach has the added advantage of enabling the significance of such trends to be tested. In addition, the residuals (differences between measured data points and the corresponding modelled values) can provide an estimate of short-term shoreline fluctuations about the trend. These terms and concepts are described in statistical textbooks such as Shaw and Wheeler (1985).

A direct comparative analysis indicates close agreement between rates from Gibb 2008 and a CSL 2012 analysis using only data for the years used in Gibb 2008 (see Figure 5.3). This similarity gives confidence in the data abstraction approaches used by the different practitioners.

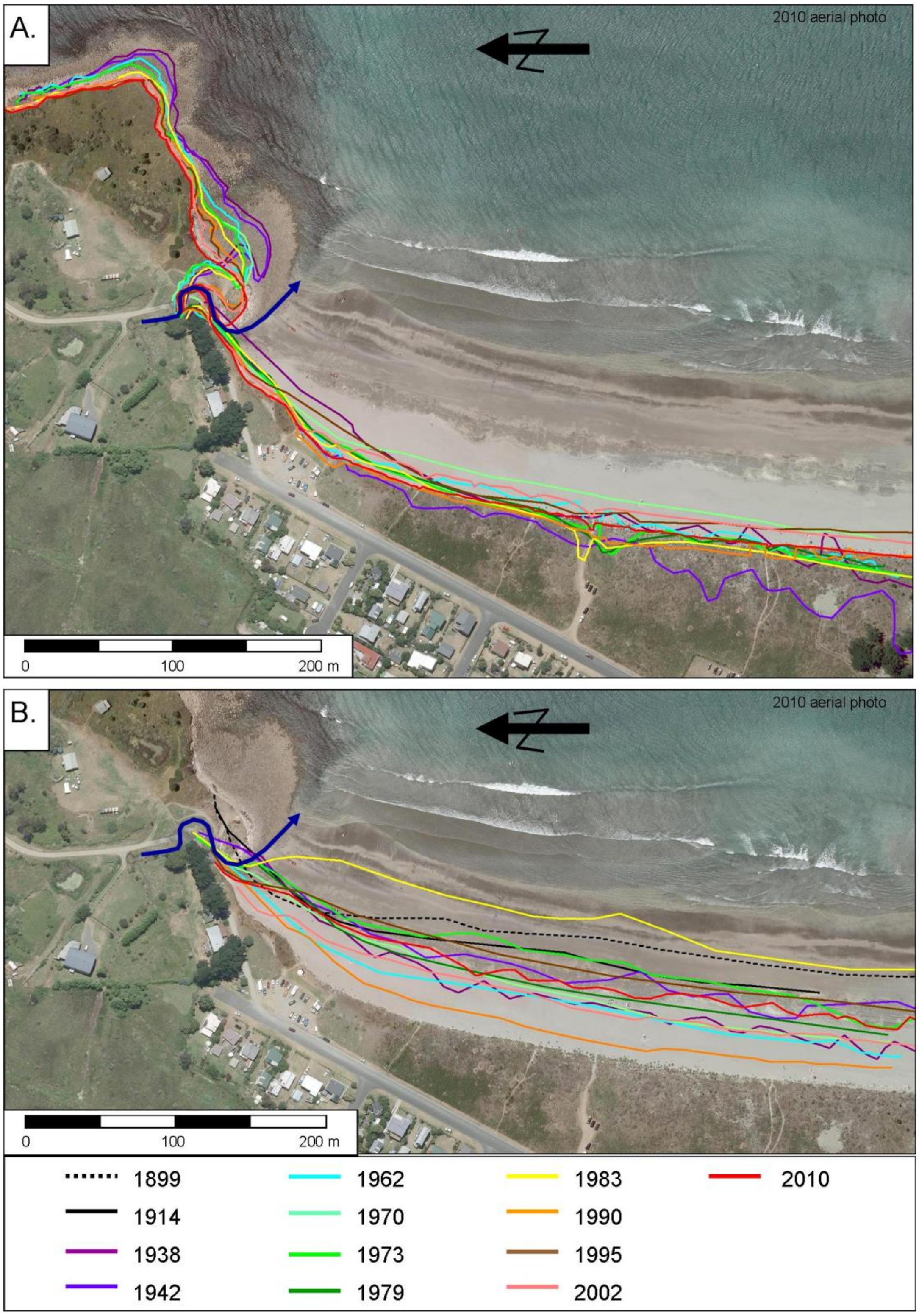


Figure 5.1 Shoreline overlays based on clifftop indicator for headland and vegetation-front indicator for spit, lagoon and beach in A, and mean high water line indicator (see text) in B.

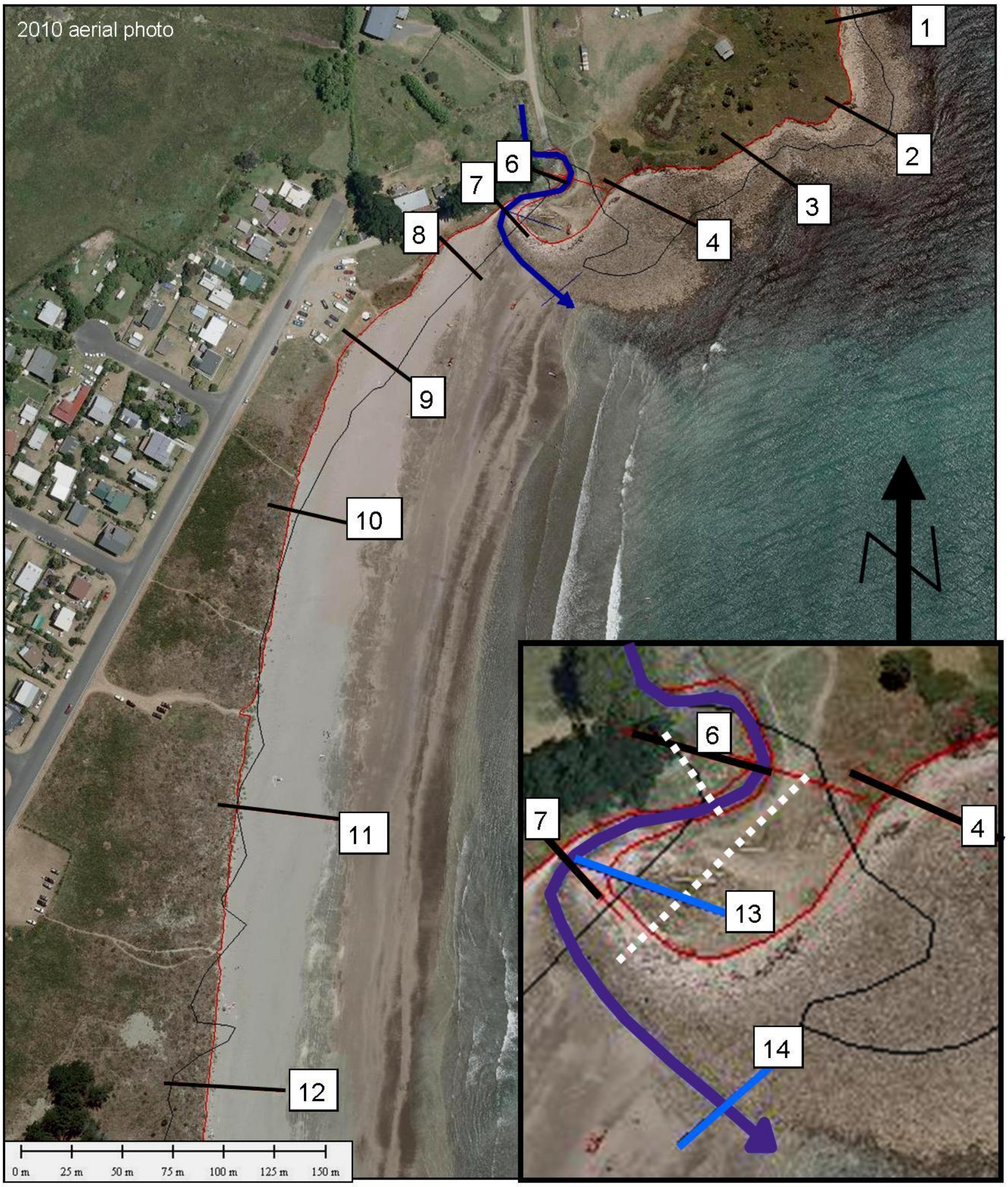


Figure 5.2 Present assessment transect locations (1-14): black straight lines for shoreline measurement transects and blue lines for channel measurement transects. Longer dashed white line (insert) is Gibb (2008) transect 19. Short dashed white line (inset) is Gibb (2005) transect 5. Continuous black line is 1938 vegetation-based beach and clifftop shoreline, and red line is present vegetation/clifftop shoreline. Stream channel (2010) depicted by blue arrow.

Of significance in Figure 5.3 is the difference between the Gibb 2005 rates and the Gibb 2008 rates along the south coast, with the latter being some 2 to 4 times lower than the former, i.e. the shoreline is indicated to be prograding at a much lower rate in the later Gibb study than in the earlier survey. This variation appears to be due in part to a longer data set and also to the earlier assessment using only the end-point difference (1942 and 2002) to estimate the average rate of change.

The CSL results show erosion occurring about twice as far along the southern coast as the earlier assessments (200 m c.f. 100 m), this being associated with the more comprehensive data set (more samples and longer coverage) as well as the regression-based approach (compared with Gibb 2005).

Of particular note are differences in the vicinity of transects 6 and 14. The former result primarily from our use of a different transect orientation (to Gibb 2005, transect 5) to better define channel meander development where the stream exits the Applicant's property (for transect orientations see Figure 5.2 inset). This positive rate (seaward directed) is likely being influenced by fluvial processes.

Transect 14 (T14) is used to define alongshore behaviour of the spit, with the result showing southward extension at 0.15 m/yr, much less than the rate of 0.61 m/yr as assessed by Gibb's 2008 transect 19 (for transect location and orientations see Figure 5.2 inset). Gibb's analysis used only the 1942 and 2008 shorelines and as can be seen, his transect 19 failed to intersect with the early shoreline. Gibb thus concluded the spit was a recent development undergoing rapid extension. By comparison, our analysis shows quite different spit behaviour. Results from our transects 4, 13 and 14 (along with the shoreline overlays in Figures 5.1A and 5.2), define the spit as being primarily an onshore migrating feature with large onshore directed component (T13=0.44 m/yr and T4=0.54 m/yr) and, as noted above, only a small alongshore (N-S) component (T14=0.15 m/yr). This behaviour has been consistent and systematic throughout the 1938 to 2010 record. The longer MHWL data set analysis below also indicates that this trend of onshore migration was occurring as far back as records began (1899). The failure of earlier analyses to detect this systematic onshore migration of the spit means they missed the critical process driving long-term shoreline erosion near the stream entrance. Correct assessment of erosion requires careful assessment of such underlying morphodynamic processes.

Finally, it is noted that Gibb's data for the headland (cliffs) as well as the spit, also only used bracketing (earliest and latest) data, and they used the cliff-base as the shoreline indicator rather than the more clearly defined clifftop as used in the present study. These differences likely explain other deviations in Figure 5.3.

To maximise understanding of the morphological behaviour of the spit and inlet shoreline adjacent to the proposed subdivision, shoreline data from transects 4 (seaward site of spit), transect 7 (fronting the subdivision) and transect 13 (channel between spit and subdivision) were also subject to nonlinear regression modelling. Results in Figure 5.4 show that for both linear (A) and non-linear (B) modelling, erosion rates are greater further seaward. Seaward morphodynamic control of the landward system, and hence the lagging of landward change, is typical of coastal behaviour as the driving energy arrives from offshore (e.g. Shand et al., 2001; Shand, 2007). The inlet's shoreline behaviour landward of the spit (fronting the proposed subdivision) can thus be expected to lag. The stream channel (transect 13) appears to be responding to the seaward forcing and is now squeezing against the south bank which can in turn be expected to undergo enhanced erosion in the future.

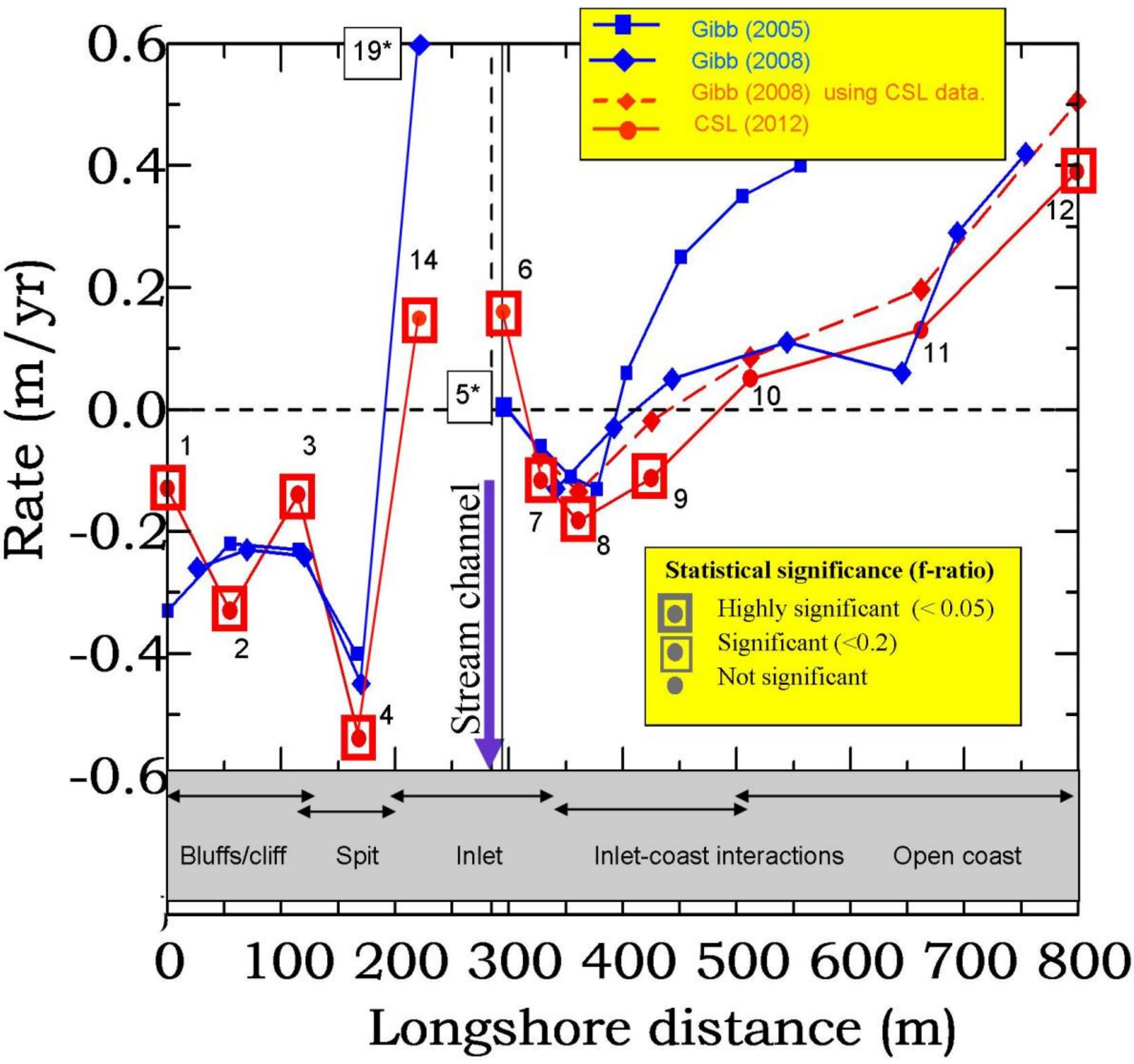
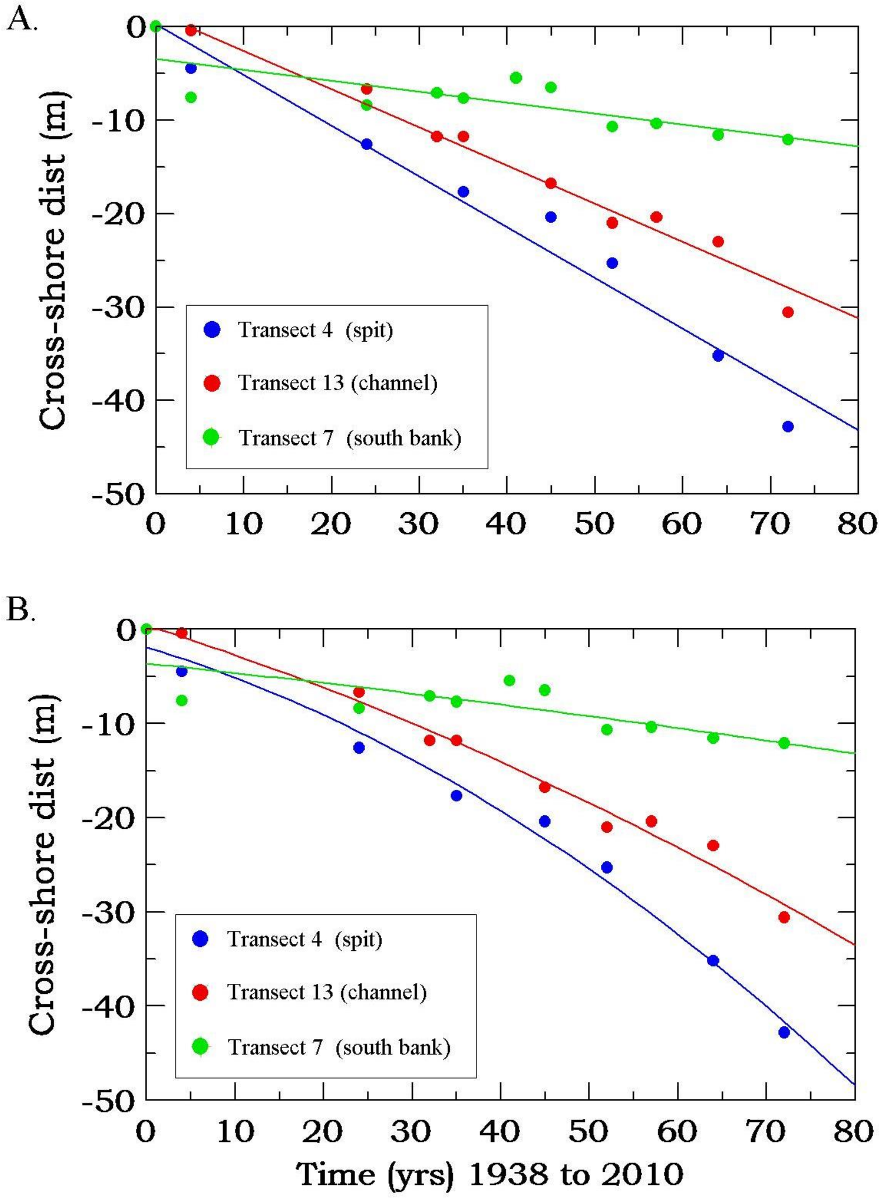


Figure 5.3 Shoreline change analysis (vegetation-base from aerial photographs) for CSL transects 1, 2, 3, 4, 6, 7, 8, 9, 10, 11, 12 and 14 as located in Figure 5.2, and comparable output from earlier hazard assessments. Note 5* refers to transect 5 in Gibb (2005), and 19* refers to transect 19 in Gibb (2008). Statistical significance of the CSL output is depicted by symbols in box. Arrow ranges and text in the grey rectangle at base of figure define the differing coastal environments and their associated alongshore extents as interpreted from the analysis output.



C.

Transect reference	Type of regression	Regression Model T = time, D = distance	R^2	Standard deviation of residuals
4 (spit)	Linear	D = -0.542T + 0.254	0.964	2.55 m
	Non-linear	$D = -0.0037T^2 - 0.2868T - 1.918$	0.989	1.56 m
13 (channel)	Linear	D = -0.408T + 1.458	0.976	1.47 m
	Non-linear	$D = -0.0015T^2 - 0.3002T + 0.4135$	0.985	1.23 m
7 (bank)	Linear	D = -0.117T-3.440	0.576	2.10 m
	Non-linear	$D = -0.0003T^2 - 0.099T - 3.6231$	0.620	2.10 m

Figure 5.4 Linear (A) and nonlinear (B) regression analysis for shoreline data from transects 4 (spit), channel (13) and beach-bank fronting the property (7). Model equations and fit parameters are listed in C where R^2 refers to the co-efficient of determination which can be used to provide a relative measure of spread about the regression line, with values closer to 1 indicating better fit of model to observation. The standard deviation of the residuals provides an actual measure of spread about the regression line.

If there were no stream and hence no inlet, the boulder spit would weld/merge with the existing shoreline fronting the proposed subdivision to form a steep profile. However, as we are dealing with an inlet, the stream channel provides a pathway not only for the seaward exit of freshwater but also for incoming storm wave surges or pulses (Figure 4.1). Such processes act to maintain inlet plan shape and allow the system to translate landward. Inlet system translation is discussed further below.

The results in Figure 5.4 show non-linear models better fit these data and predict higher shoreline recessions than the corresponding linear model, with the rate of non-linear erosion increasing over time (see footnote below). For transect 4 on the seaward side of the spit, the non-linear model predicts an increase in recession of 120% over the next 100 yrs (2012 to 2112), or a 74% increase from the time datum of 1938. For transect 13 (channel) the non-linear model increase in recession over the linear model is 62% over the next 100 yrs (2012 to 2112), or 39% from the time datum of 1938. For transect 4 on the landward side of the inlet, the non-linear model predicts a 46% increase over the linear model over the next 100 yrs (2012 to 2112), or a 25% increase from the time datum of 1938. However, the shoreline modelling for transect 7 in particular is likely to underestimate the predicted recessional behaviour as the expected squeeze-effect from the spit and channel migration takes effect.

When determining a shoreline retreat value for future prediction, linear rates are used. While non-linear models may better fit the data, they may also lead to increased inaccuracy if the system is not particularly well understood (Fenster et al., 1993). In the case of the Mahanga inlet system (channel, banks and spit) a value weighted to the spit's seaward shoreline is appropriate as the inlet system is controlled by seaward drivers, (i.e. landward migration rate of the spit will eventually occur at the shoreline further landward as described earlier). A value of **0.5 m/yr** is thus selected for use in morphodynamic modelling and hazard assessment. However, it should be kept in mind that the non-linear model provides a considerably better fit for the seaward shoreline data so the 0.5 m/yr erosion should be considered a **minimum rate** for use in morphodynamic modelling of the inlet system, i.e. that area covered by transects 4, 5, 6, 7, 8, 13 and 14 in Figure 5.2.

Spit morphological evolution was modelled by translating the system landward along an axis defined by analyzing the morphological behaviour of the *spit tip* from 1938. In particular, a linear regression model was fitted to the vegetation-based tip locations for the aerial photo samples. This axis is depicted by the red line in Figure 5.5. The translation rate was 0.5 m/yr as determined earlier. Predicted locations of spit shoreline and channel for both 50 yr and 100 yr time spans are depicted in Figure 5.5, these being translated 25 and 50 m respectively from present locations.

The reason for the increasing rate over time was resolved in a later site visit described in Appendix C. Briefly the internal structure of the spit had increased its volume of sand vs cobbles during its landward migration and thus its susceptibility to erosion

The question of how far such an inlet system can translate is now considered further. As noted above, inlet processes tend to preserve form, but, all things being equal, we could expect a limit as to how far the process of landward inlet translation will go before a major channel change and a system reset occurs, such as drainage impoundment resulting in the stream taking on an entirely different course. However, all things are not equal in this case, and Mahanga Beach shoreline is likely to systematically erode in the future under both longer-term erosion (see Figure 5.6 and text below), and shoreline response to sea-level rise (Section 5.3.3). In addition, the effects of climate change are expected to include increases in spit overwash, and in the frequency and magnitude of present inlet process (see Table 2.1). Given the high level of uncertainty in the future status of system drivers, it would be precautionary to assume that future continuation of the spit's previous landward migration, and future landward translation of the inlet system, will occur throughout the assessment period

While the above analyses define spit behaviour for projection purposes, how the inlet merges with the open coast to the south over time periods in excess of 100 yrs requires consideration of the longest possible shoreline data set. As noted above in Section 5.2, MHWL shoreline data was available back to 1899. These data were not used in previous assessments. Such a marker is typically offset several metres seaward from the dune vegetation front, so these two types of data cannot readily be reconciled. However, as noted earlier, the MHWL location can be identified by sand texture/tone/contrast on aerial photographs, so a full MHWL-based shoreline set was abstracted (Figure 5.1B). The linear regression analysis (trend) output rates are shown in Figure 5.6. For comparison, the CSL 2012 vegetation-based shoreline from Figure 5.3 is also included in Figure 5.6.

Most notable in Figure 5.6 is the contrast in alongshore erosion pattern with the longer data set showing less erosion closer to the inlet and increased erosion further alongshore. Overall, the inclusion of the three earlier samples not used in previous assessments (1899, 1914 and 1938) enabled a trend of **longshore erosion** along Mahanga Beach to be defined, whereas exclusion of these samples has resulted in the identification of **long-term accretion**.

While the survey plans did not map the spit, the reduced erosion along the inlet's western bank (proposed subdivision side) infers less channel influence from the spit, and this would be consistent with the spit being further seaward, a conclusion that indicates the spit migration process has been occurring for at least 100 yrs.

The reduced statistical significance of the MHWL analysis compared with the vegetation-based analysis reflects the expected greater scatter in the tide-based indicator and this is clearly evident in Figures 5.1A and B. However, as the bounding transect results show increased statistical significance, and as a precautionary approach must be adopted in such a situation (NZCPS 1994 Policy 3.3 and NZCPS 2010, Policy 3), the indicated longer-term erosional trend along the open coast is incorporated within the present erosion hazard analysis.

Before selecting a representative rate of change for the open coast (transects 10 to 12), it is noted that the assumed shoreline indicator for the 1899 survey plan was MHWL as this was not marked on the plan and the original field book could not be located. LINZ staff report that the field book may have been destroyed in the 1931 Napier Earthquake). The MHWL assumption was made as this is the usual indicator appearing on such early survey plans. Other sandy beach shoreline indicators used by early surveyors were the spring high tide line or dune toe. If we had used either of these alternative shoreline indicators in our analysis, the assessed erosion rate would further increase. For example, if the indicator was the spring tide line and we take the actual MHWL as being 10 m seaward, then the erosion rate for transect 12 increases from 0.22 to 0.27 m/yr and the level of statistical significance of the rate of change also increases. This assessment uses a long-term erosion rate of **0.2 m/yr for the open coast** (transects 10-12) and this is considered the **minimum rate** for use in hazard assessment.

The long-term erosion curve for the shoreline (back) on the western side of the inlet, i.e. fronting the proposed subdivision (transects 6 and 7) was derived from Figure 5.5. The long-term erosion curve between the subdivision shoreline and the open coast was interpolated using curve fitting based on the existing shoreline shape. The resulting long-term erosion lines for 50 and 100 yr prediction periods are depicted in Figure 5.7.

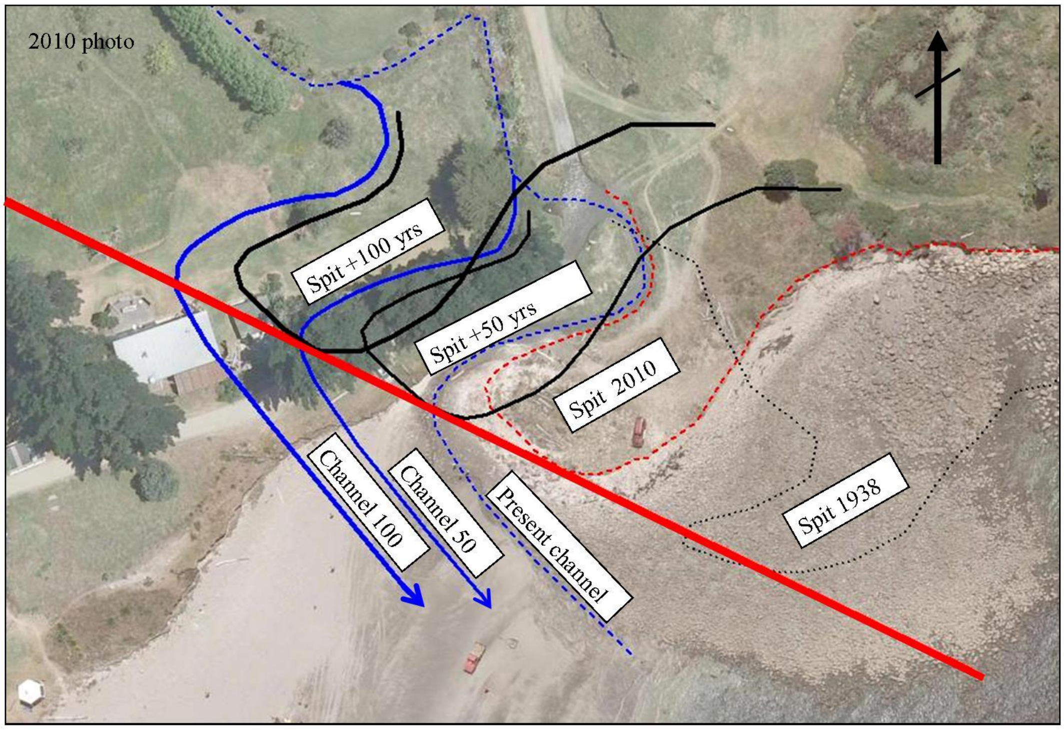


Figure 5.5 Fifty year and 100 year modelled spit shoreline and inlet channel. Bold red line is translation axis. Note 1938 spit location (black dotted line) is included for comparison.

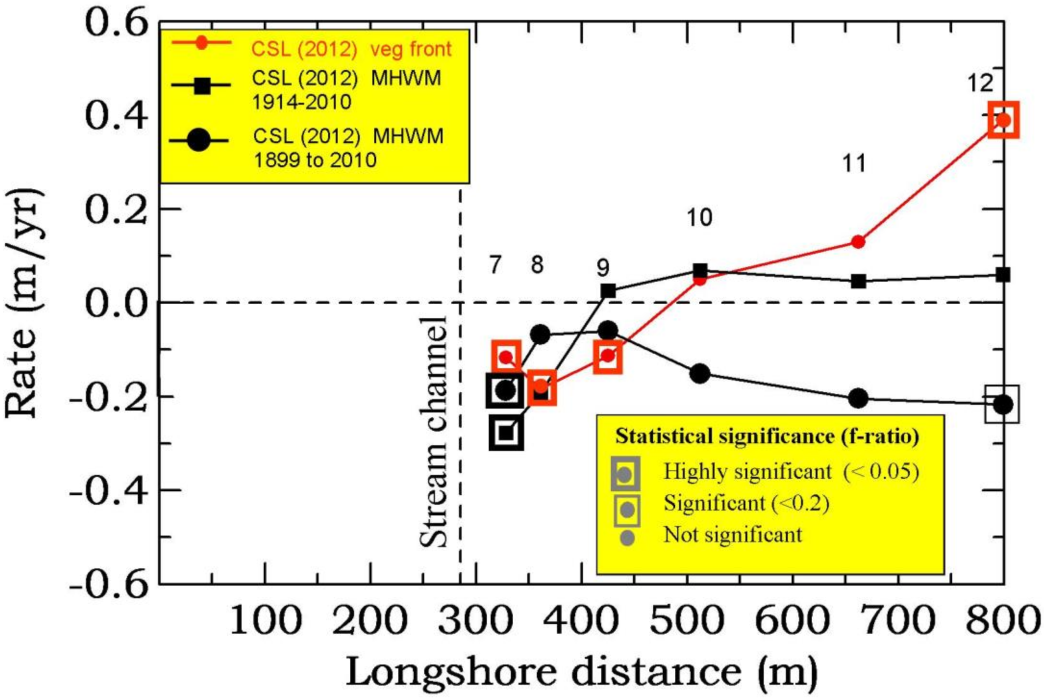


Figure 5.6 Shoreline change along Mahanga Beach 1899 to 2010. The shoreline indicator is the mean high water line at the time of survey. The CSL 2012 vegetation-based shoreline from Figure 5.3 is included for comparison. Level of statistical significance marked. Data sourced from survey plans 1899, 1914 and aerial photos 1938-2010.

5.3.2 Shorter-term erosion fluctuations

Shorter-term erosion refers to cross-shore fluctuation in shoreline position superimposed upon any longer-term trend. Such fluctuation may result from storm response (weeks) to climate fluctuation (e.g. ENSO cycles) over several years, to variation in sediment supply (decades). An appropriate and robust technique to quantify such variation can be carried out using the *standard error of estimate (SEE)*, a similar parameter to the *standard deviation of the residuals* used in Figure 5.4.

It can be shown, e.g. see Shaw and Wheeler (1985), that a fluctuation defined by 2 x SEE on each side of the regression line, i.e. ± (2 x SEE), will encompasses 95% of population values. In other words, we can be 95% certain that this interval will encompass the range of possible shorelines. Alternatively, ± 3 x SEE will encompasses 99% of population values. As the shorter-term fluctuation is a particularly significant component in erosion hazard analysis, there is merit using 3 x SEE for new development. However, I have used the **2 x SEE** option for this assessment and stress that this must be viewed as a **minimum value**.

It is noted that the regression-residual based approach assumes that the residuals are normally distributed, i.e. they fit a bell-shaped (normal) frequency distribution. To test this requirement, residuals were plotted against expected values for a normal distribution and in

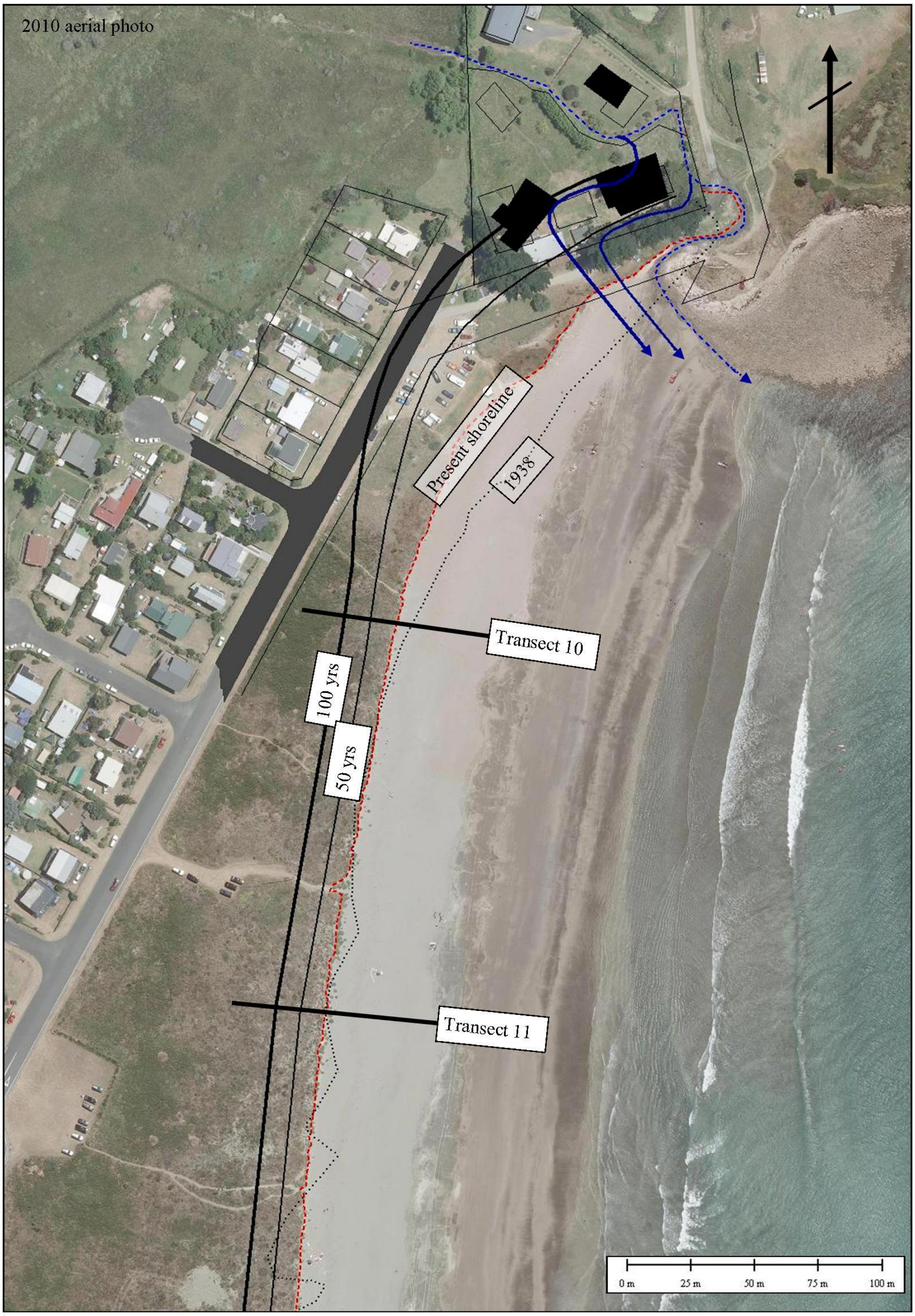


Figure 5.7 Fifty and 100 yr modelled long-term erosion shorelines along Mahanga Beach. The 1938 and 2010 shorelines, along with the present and predicted stream channel are marked. Road and property boundaries are shown, as are the proposed subdivision building outlines with infilled blocks representing October 2011 modified proposal.

all cases the result approximated a straight line, thus indicating the residuals are normally distributed (Wilkinson, 1998).

The representative short-term erosion mean value for the open coast (transects 10, 11, 12) is 19.2 m, while the value for the inlet is 8.6 m.

5.3.3 Erosion (retreat) from sea-level rise (RSLR)

There are two aspects to estimating shoreline response to sea-level rise: the actual amount of sea-level rise (SLR) that will occur during the assessment period (at least 100 yrs), and the response model used. The Storm Inundation Assessment (Section 4), used a minimum SLR value of 1 m for the assessment period prediction period based on the best information presently available. The choice of shoreline response model is a matter of ongoing debate amongst coastal scientists. Profile translation models are most widely used in erosion hazard assessments. Most common is the Brunn Rule (equation 5.2) which is based on the principle that as sea level rises, sediment is eroded from the upper beach and deposited offshore raising the bed level (Bruun, 1983). The profile thus translates between the offshore limit of sediment transport (closure depth) to the crest of the foredune by that amount required to fit the predicted SLR.

$$R = S(L/[B + d]) \qquad (5.2)$$

where R is the profile shift in the landward direction, S is the predicted rise in sea-level, L is the cross-shore length of the profile, B is the height of the beach-berm or dune above initial MSL, and d is the depth below initial MSL beyond which significant sediment exchange is not considered to occur (*closure depth*). The rule is governed by simple, two-dimensional conservation of mass principles and is subject to the following limitation in its application:

1) There is no consensus as to the existence of closure depth (Pilkey et al., 1993), and the range of methods used in its estimation have produced results that vary by a factor of 3 having been reported (Shand 2008);

2) The rule assumes no offshore or onshore sediment losses, and no alongshore flux in sediment transport;

3) The rule assumes an equilibrium beach profile (a statistically average profile) exists through the prediction period, and

4) The rule does not accommodate variations in sediment properties across the profile, and

5) The rules assumes there is no profile control by hard structures such as substrate geology or adjacent headlands or engineered structures.

Numerous researchers have tested the Bruun Rule against a variety of field and laboratory data. Full reviews of these comparisons are provided within SCOR (1991) and Ranasinghe et al. (2007). In general, while the overall principles of the Bruun model have been demonstrated (i.e. an increase in sea level results in an upward and landward shift in the profile), the quantitative accuracy of the Bruun Rule has not been convincingly verified. Predictions for specific sites have varied from measured rates by factors of 2 to 5 (both over- and under-prediction) with greatest variation occurring where the assumptions are least fulfilled (Everts, 1985; Zhang et al., 2004).

The proposed subdivision is part-fronted by a rock controlled headland and reef with the present study showing modelled alongshore sediment transport changing significantly in the longshore direction. Critical Bruun Rule assumptions are thus not met.

An alternative empirical-based model to estimate shoreline retreat driven by SLR is to substitute dune-closure profile in the Brunn Rule by the beach-face profile. Note that the beach-face approximates the inter-tidal beach. This version of the model is often referred to as the Komar Model and has its origins in the derivation of an empirical model to determine storm erosion during periods of elevated water level on the United States West Coast (Komar et al., 1999). The Komar Model can be expressed by the equation 5.3.

$$R = S/\tan\beta \qquad (5.3)$$

where R is the profile shift in the landward direction, S is the predicted rise in sea-level, and $\tan\beta$ is the average inter-tidal slope. Typically, our comparisons between application of the Bruun Rule and Komar Model found the latter method predicted about half the SLR-induced retreat of the former method. In the present assessment of the Mahanga coast we consider the Komar Model will provide a more realistic estimate of SLR-induced shoreline retreat, but once again our estimate for this parameter is considered to be **a minimum**.

Applying equation 5.3 using $\tan\beta = 0.03$ and SLR values of $\geq$1 m and 0.43 m for the prediction period and 50 yr period respectively, results in R = $\geq$33.3 m and 14.3 m respectively.

Note that the inter-tidal slope was based on our analysis of 10 yrs of HBRC profile data for Mahanga, which consisted of 12 samples taken over 7 yrs. We consider these data produce a reliable average value for this parameter. Also note that the SLR value of 0.43 m for 50 yrs comes from the MFE (2008) recommendation of 0.45 less 0.02 m from adjusting to datum (see Section 4.2.1).

5.3.4 Dune stability adjustment (DS)

This parameter refers to the retreat distance of the scarp-top following storm erosion and this distance may be defined using equation 5.4.

$$DS = h/2*\tan \propto \qquad (5.4)$$

Where h = dune height and $\propto = 33^0$, the approximate angle of repose for dry dune sand.

Using the HBRC LIDAR to determine current dune height, the average retreat value along the proposed subdivision = 1.03 m, and for the open coast = 2.8 m.

5.3.5 Combined uncertainty

Uncertainly can be determined either by assessing each component and combining these errors, taking care to account for variable dependency, or using a Factor of Safety (FOS) approach which involves adding a multiple of the combined hazard components. For example a FOS of 1.5 relates to 50% of the hazard distance.

The previous Mahanga assessments used the FOS approach with Gibb 2002 using a value of 1.5, T&T'04 using 1.25 (for certain components only), Gibb 2005, 2007, 2008 using 1.3. Our own research elsewhere involving a particularly accurate assessment (Shand, 2008) and an individual component error approach, equated to a FOS of about 1.15. The current assessment uses a value of 1.3; however, as lower-end estimates were used for LT and ST erosion and retreat to sea-level rise, an FOS value of 1.5 was also computed for comparison in the vicinity of the proposed subdivision.

5.4 Coastal erosion hazard distances (CEHD) and lines (CEHL)

The CSL 2012 average coastal erosion hazard distances (CEHDs) for the inlet shoreline fronting the proposed subdivision (transects 6-8) and for the open coast (transects 10 to 12) using the parameter values determined in the this analysis are listed in Table 5.2. The CEHD for ≥100 yr assessment period is at least 98 m along the open coast and at least 121 m fronting the inlet. Of note is the LT-ST relativity reversal between the inlet and open coast and also how the FOS increase of 1.3 to 1.5 increases the inlet CEHD by an additional 10 m for a 50 yr period 17 m for 100 yrs.

The CSL 2012 coastal erosion hazard lines (CEHLs) derived from the values in Table 5.2 are depicted in Figure 5.8 (bold red line = 100 yrs, bold yellow line = 50 yrs). Interpolation within the inlet-open coast transition zone follows the form of the long-term erosion curves in Figure 5.7. The 50 yrs of predicted erosion is likely to impact on the seaward sections in the proposed subdivision, while 100 yrs will impact on all but the landward-most section line (with the existing dwelling). Given that the assessment period must exceed 100 yrs and that key components within the present assessment are likely to be conservative (low), the entire subdivision must be considered vulnerable to coastal erosion with the risk likely to increase over time.

Table 5.2 Coastal erosion hazard distances (CEHD) for the inlet (fronting the proposed subdivision) and open coast for different assessment periods (AP) and uncertainty (FOS).

Location	Transects	AP	LT	ST	RSLR	DS	FOS	CEHD
Inlet	6-8	**50**	25	8.6	14.3	1.0	1.3	63
Inlet	6-8	**50**	25	8.6	14.3	1.0	**1.5**	73
Inlet	6-8	**100**	50	8.6	33.3	1.0	1.3	121
Inlet	6-8	**100**	50	8.6	33.3	1.0	**1.5**	138
Open Coast	10-12	**50**	10	19.2	14.3	2.8	1.3	60
Open Coast	10-12	**100**	20	19.2	33.3	2.8	1.3	98

Where: AP = assessment period, LT = longer-term retreat (m), ST = shorter term landward fluctuation (m), RSLR = retreat from sea-level rise (m), DS = post-erosion dune scarp adjustment (m) and FOS is the factor of safety (includes 1.5 fronting the proposed subdivision for comparison)

The CSL erosion hazard component values and CEHDs are also compared with those used in the previous assessments in Table 5.3. In particular:

1. **Longer-term (LT) erosion values range between 0 and 50 m.** The T&T'04 long-term rate = 0 comes from the Gibb 2002 assessment which indicated an accreting shoreline. Many practitioners set positive rates to zero to reflect the uncertainty in future shoreline behaviour. Gibb's 2005 long-term value included a 100% allowance for uncertainty in future spit behaviour, but this was removed in the Gibb 2007 assessment ! The CSL 2012 assessment's erosion values are higher due to the increased understanding of spit dynamics and Mahanga shoreline behaviour arising from this investigation.

2. **Shorter-term (ST) erosion values range between 8.6 and 42 m.** The very high T&T'04 shorter-term value (42 m) was based on cross-shore fluctuation identified in beach profile data from survey sites on the exposed sandy Waimarama coast south of Hawke Bay. As noted earlier in Section 5.3.1 and by comparing Figure 5.1A with 5.1B), beach fluctuation is substantially greater than dune fluctuation so 3* such standard deviations could yield particularly high values. However, given the uncertainty of transferring data from another site, such conservatism is acceptable in a regional hazard assessment.

3. **Retreat from sea-level rise (RSLR) values range between 17.1 and 59.5 m.** The lower values came from the earliest assessments where official SLR values were lower than at the present time. The highest estimate was from Gibb 2007 when the revised IPCC (2007) SLR value of 0.8m to 2100 was first used. It is noted that

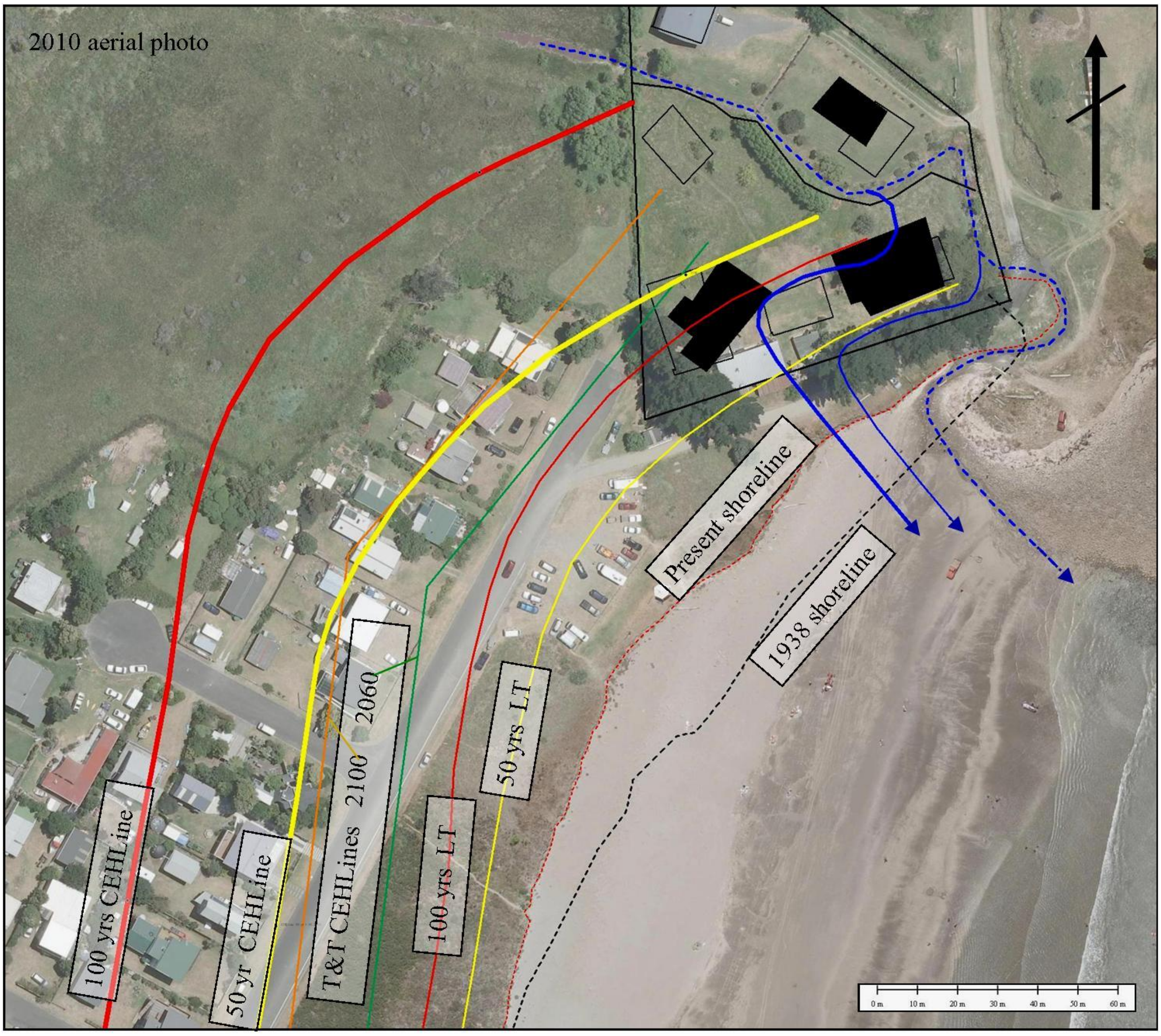


Figure 5.8 CSL coastal erosion hazard lines (CEHLs) for 50 yr (bold yellow) and 100 yr (bold red) prediction periods were derived using hazard component values listed in Table 5.2. The T&T'04-based CEHLs are depicted in green for 2060 and orange for 2100. CSL long-term erosion lines (NB Figure 5.7) are depicted by the thin yellow and red lines. Existing and predicted (50 and 100 yr) stream channel is marked by blue arrows and define future spit migration. Proposed building sites are shown with October 2011 modifications infilled.

Gibb 2007 subtracted the historic SLR value of 1.6 mm/yr on the grounds that present beach processes have already adjusted for this amount. While this argument has some merit, due to the range of uncertainty associated with climate change, many practitioners use the full SLR value when determining the RSLR and this was the case in the present CSL assessment. The CSL 2112 estimate of 33.3 m out to 2112 is substantially lower than the others despite using the highest SLR value (1.0 m) and this reflects the fundamental difference between the Bruun Rule and the Komar model.

Table 5.3 Erosion hazard parameter values for present (right columns) and previous erosion hazard assessments carried out in the vicinity of the proposed subdivision.

	T&T'04	Gibb (2005)	Gibb (2007)	CSL (2012)#	
Assessment period (yr)	96	95	93	50	100
Longer-term (m)	0	-20.9	-11.2	-25	-50.0
Shorter-term (m)	-42.0	-9.0	-9.0	-8.6	-8.6
Dune adjust -ment (m)	-	-2.0	-2.0	-1.0	-1.0
Retreat (m) SLR [SLR values (m)]	-20.2 [0.3]	-17.1 [0.2]	-59.5 [0.64]	-14.3 [0.43]	-33.3 [1.0]
Uncertainty (FOS)	*1.25##	*1.3	*1.3	*1.3	*1.3
TOTAL (m)	-67	-64	-106	- 63	-121

Inlet values in Table 5.2 (averaged over transects 6, 7, 8).
Already applied to shorter-term value

Note: Gibb (2008) provided shoreline analysis output in the vicinity of the subdivision (NB Figure 5.3), but only derived CEHLs for the Pukenui Drive area.

The final CEHD results in Table 5.3 show the CSL 2012 total hazard distance value to be about 50% above the early assessments and about 15% above the Gibb 2007 value. While the CSL and Gibb 2007 CEHDs are similar, the significance of the present assessment lies with the apportionment of the component values, in particular the increase in systematic shoreline erosion from 11.2 to >50 m for a 100^{+} yr assessment period. *This result makes the exclusion of long-term erosion from the primary no-build hazard management zone, as used in Gibb 2007 and adopted by the HBRC, completely inappropriate.* Note that T&T'04 (Figure 5.1) recommended long-term erosion be included within the primary (no build) hazard zone.

5.5 Erosion risk management

Both NZCPs 1994, Policy 3.4.5 and NZCPS 2010, Policy 25 state that new subdivision should avoid erosion-prone areas, with the latter being framed in terms of avoiding increasing the hazard risk where risk is defined as the combination of (i) likelihood of particular magnitude event and (ii) impact consequences of that event. The CSL hazard assessment has demonstrated that virtually the entire site is likely to be affected by erosion the next 100 yrs - the minimum required by the NZCPS 2010. The creation of additional lots, and the construction of additional dwellings, will only increase the consequences and hence the hazard risk.

As noted in Section 1, the 2010 joint council decision to grant subdivision consent included the need for a seawall near the seaward boundary of the property. The Applicant's current proposal shows the wall running along the boundary and the westernmost portion only being constructed when erosion reaches a predetermined trigger line.

However, granting that consent was based on the understanding of processes and erosion hazard as described in Gibb 2007. The present assessment has identified process variations and a different erosion potential with the onshore translating inlet system likely to induce systematic shoreline retreat in excess of 50 m during the assessment period.

Hard protection has become increasingly unacceptable to decision makers with the NZCPS 1994 Policy 3.4.5 stating that "new subdivision, use and development should be so located and designed that the need for hazard protection works is avoided", and the NZCPS 2010 Policy 25 requiring that decision makers discourage hard protection structures and promote the use of alternatives to them. This official attitude has come about in recognition of the range of negative impacts such structures can have on the coastal environment including:

1) Serious beach loss will inevitably occur in front of a seawall where there is a high rate of net shoreline retreat. While the seawall protects land behind the wall, it does not reduce erosion of the fronting beach, and under some conditions erosion can be exacerbated.

2) The need for ongoing maintenance and strengthening as water depths increase seaward of the structure (due to the fact that the natural beach profile is effectively located ever further landward). This results in the structure foundations becoming increasingly undermined as well as the structure being subject to increasingly more frequent and stronger wave attack - due to the deeper water to seaward. Climate change is likely to further exacerbate these situations.

3) The potential impact on coastal processes and thus effect on the beach, dunes and other property. Seawalls invariably modify wave and current patterns such that erosion embayments form at the end(s) of the wall; this is referred to as flanking erosion or end (effect) erosion (see Figure 5.9). A seawall fronting the proposed subdivision will almost certainly result in such erosion, particularly at the exposed southern terminus. Erosive intensity and embayment dimensions depend on structure length, profile location (which will vary as systematic erosion proceeds), and the energy and sediment transport environment. An empirical model recently developed by Shand (2010) for moderate oceanic environments, relates embayment length to structure length by equation 5.5. Laboratory results (McDougal et. al., 1987) found the erosion depth to erosion length ratio is 1:7.

$$Le = 6.089Ls^{0.5357} \quad (5.5)$$

Where Ls = length of structure and Le = length of adjacent erosion.

4) Characteristics of the local geomorphology can exacerbate environmental impacts associated with a protection structure. In the present situation, the landward migrating spit will increasingly force the stream against the seawall and reduce channel capacity. Ongoing excavation will be required if upstream drainage is to be maintained and flooding avoided.

The proposed seawall will significantly and permanently alter the natural sediment transport processes and geomorphological form and behaviour of the inlet system. It should be kept in mind that the spit and inlet are the present day focus of geomorphological activity associated with the 800 year old Mahanga Landslide as it continues its adjustment toward equilibrium with the 2000 yr old Mahia Tombolo (Section 2). These processes and the evolving landscape are nationally unique and have particularly high scientific value. In addition, such change will alter stream hydrology and impact upon the special ecology further inland (see Appellant's Evidence). Cultural values may also be compromised (Appellant's Evidence). Such impacts contravene aspects of NZCPS 2010 such as Policy 2: The Treaty of Waitangi, tangata whenua and Maori heritage, Policy 13: Preservation of

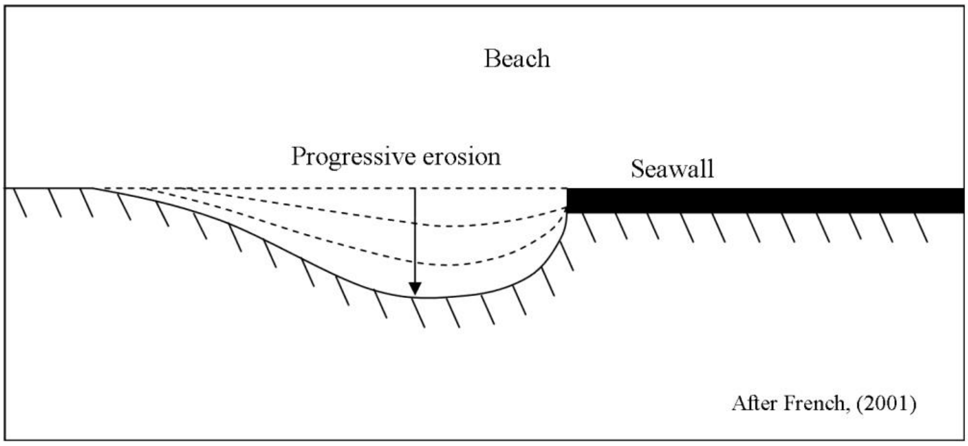


Figure 5.9 Typical shoreline response (embayment) pattern associated with seawall end-effect erosion.

natural character, and Policy 15: Natural features and landscapes. Use of structural protection as proposed by the Applicant would contravene the very purpose of the RMA 1991 'to promote the sustainable management of natural and physical resources".

It is noted that beach nourishment, a form of "soft protection" which allows the natural sediment transport processes to prevail, is also not sustainable for sites undergoing long-term erosion. Once the nourishment has been removed by wave action and currents, it must be replaced if the onset of systematic erosion is not to recommence. And just as in the seawall case, as the beach systematically lowers the required level of maintenance (in this case the quantity of replacement sediment) must increase though time.

Regarding the joint council's 2010 decision to grant the subdivision consent for valuable new development in a hazard-prone area on the basis that future removal or relocation of the development's buildings within their design lives was an available and feasible way of mitigating likely coastal hazard impacts. It is accepted that providing for removal of existing or replacement buildings from a threatened site is both a recognized and preferred method of mitigating coastal erosion hazard for existing development and associated use rights (e.g. NZCPS 2010, Policies 25(c) and 27 (1)(a). However, it is not consistent with sustainable resource management to use this approach to justify increasing the risk of adverse effects from coastal hazards, i.e. by creating additional lots with separate titles and thus providing for additional new development in hazard-prone areas (NZCPS 2010, Policies 25(a) and (b)). While Policy 25(c) encourages design for relocatability or recoverability from hazard events, as one possible way to reduce hazard risks, when read along with Policies 25(a) and 25(b) this clearly relates to redevelopment or new development on exiting properties, rather than to new subdivision with all its associated new development that can only increase overall hazard risk. In addition, managed retreat by relocation or removal of these particular proposed developments could be especially fraught with problems – including new sites having to be distant from the subdivision, whether such relocation is feasible, and the understandable reluctance of owners (particularly new future owners) to comply and undertake such a difficult exercise.

6.0 SUMMARY and CONCLUSIONS

1) **Coastal Systems Ltd (CSL) agreed in February 2011** to assist Mahanga E Tu in their Environment Court challenge to the Hawke's Bay Regional Council (HBRC) and Wairoa District Council (WDC) consenting of the Williams, Mexted and Van Breda Malherbe proposed subdivision at Mahanga Beach, by providing the court with independent hazard evidence. We considered we had an ethical duty to the public (including the Applicants) to do this following a site visit on 18th January 2011, when we became particularly concerned at the site's apparent vulnerability to coastal hazards, with high risk of future property damage and to personal safety.

2) **Comprehensive inundation and erosion hazard assessments** were carried out in the vicinity of the proposed subdivision using:

- More extensive process and morphological data bases than used in previous relevant assessments (Gibb 2000, Tonkin and Taylor 2004, Gibb 2005, Gibb 2007 and Gibb 2008),
- More robust analyses than used in most of the earlier assessments.
- A range of new hazard assessment references and guidance materials (including the New Zealand Coastal Policy Statement 2010 which contains a dedicated policy [24]), and
- Peer review by several practitioners with expertise in different aspects of coastal hazards.

3) **The Geomorphological System** was identified and this included previously unrecognized characteristics which are critical to achieving accurate hazard assessment. In particular:

- The site of the proposed subdivision lies within a particularly dynamic region at the margin between two nationally significant and interacting geomorphological features: the c.2000 yr old Mahia Peninsula Tombolo (the sand plain that joins the Mahia Peninsula to the mainland) and the 800 yr old Mahanga Landslide where the north coast of the tombolo joins the mainland. These two features meet in the vicinity of the proposed subdivision and they are evolving toward an equilibrium state. That evolution is very likely to continue throughout the present hazard assessment period.
- The proposed subdivision lies within a low-lying area, 2 to 2.5 m above MSL, that was until relatively recently a lagoon entrance. This low area is bounded on each side by elevated terrain and as such lies within an inundation pathway for both storm and tsunami waves.

- The proposed subdivision is fronted to seaward by a boulder spit which is migrating onshore and in the future will likely cause enhanced erosion rates at the shoreline fronting the proposed subdivision.

4) **Hazard assessment results**:

- Empirical-based storm inundation modelling indicated water depths in excess of 3.5 m at pulse speeds of 5-8 m/s across the proposed subdivision during the assessment period (at least 100 yrs). Earlier assessments did not quantify these hazards across the proposed subdivision
- Empirical-based tsunami inundation modelling showed the site to be even more vulnerable, with inundation depths in excess of 5.5 m at speeds of 11 to 15 m/s possible across the subdivision during the assessment period. Earlier assessments did not quantify these hazards across the proposed subdivision.
- Empirical modelling of shoreline behaviour showed considerably greater systematic retreat of the shoreline fronting the subdivision than predicted by earlier assessments (≥0.5 m/yr cf. 0 m to 0.22 m/yr). With incorporation of the other erosion hazard components (e.g. retreat associated with predicted sea-level rise) virtually the entire proposed subdivision could be affected by erosion during the assessment period.

5) **Climate change effects**, including sea-level rise (SLR), are likely to enhance and prolong the interaction process between both the larger-scale morphological units (the landslide and tombolo) and also the smaller-scale units (the boulder spit and inlet channel/shorelines). Climate change, including SLR, will directly increase hazard levels and risk during the assessment period and this has been factored into the assessment.

6) **Hazard mitigation** to reduce the risk is not considered viable:

- Tsunami (even for 50 yr return periods) appear to be too powerful for mitigation by construction design. The viability of emergency evacuation is particularly questionable given the likelihood of little or no warning time and the location of the site within an inundation pathway.

- Storm inundation would challenge the viability of building design and the environmental impact and associated hazard risk would be very high.

- Systematic longer-term erosion control using a hard structure is not viable as the structure would modify processes both in front of, and alongshore of, the proposed seawall. Firstly, the structure itself will be increasingly undermined as the fronting beach lowers and will need ongoing repair and strengthening. Secondly, the landward migrating boulder spit (see Point 3 above) will increasingly force the stream against the structure thereby impeding terrestrial drainage and induce flooding unless ongoing channel excavation occurs. It is noted that important hydrological, ecological, recreational (loss of beach) and cultural values, as detailed in evidence by Appellant witnesses, will likely be negatively impacted by such spit-seawall interaction and associated maintenance. Thirdly, the structure is likely to modify processes at and beyond the ends of the seawall and induce (end-effect) erosion once ongoing background erosion on the adjacent beach exposes the structure's end(s) to wave action.

7) **Existing HBRC and WDC hazard zones** for this site are based on Gibb's 2007 proposal in which the time-based erosion component was removed from the primary (seawardmost) no-build zone. The previously proposed Tonkin and Taylor 2004 zoning included this component with their value of 0 m being based on an earlier preliminary assessment by Gibb 2002. Comparison of findings from the present assessment with Gibb 2007 (see Table 5.3 1 in the present report) indicates that the latter significantly underestimated the longer-term erosion component (11.2 m c.f. ≥50 m), although it should be noted that the Gibb 2005 assessment used a value of 0.22 m/yr which (justifiably) included a 100% allowance for uncertainty in future spit behaviour.

Furthermore, under official Ministry for the Environment (2008) guidance and the New Zealand Coastal Policy Statement, 2010, it is required that enhanced climate change and associated impact prediction are included within hazard risk assessment and management. Some allowance for climate change within a no-build zone must be made in hazard zoning.

Finally, the present assessment has demonstrated severe potential impact of both tsunami and storm inundation across the site to the extent that these processes cannot be assigned secondary-tertiary status as is the case in the current hazard zoning.

The existing local government hazard zoning should not used as a management guide for the proposed subdivision.

8) **In conclusion,** the present assessment demonstrates that:

- The proposed subdivision is highly vulnerable to coastal erosion and inundation processes,
- Effective hazard risk mitigation is not viable,
- There is a very high risk that, at times within the assessment period, property will be exposed to severe damage and life and safety threatened

In my professional opinion, this subdivision proposal should be declined in its entirety.

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RD Shand

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Dr Roger Shand

ACKNOWLEDGMENTS

Firstly I wish to thank the five reviewers: Dr Mike Shepherd, Mr Jim Dahm, Mr Mike Jacobson, Mr James Carley and Professor James Goff who willingly and at short notice provided well considered feedback. I also thank Dr Tom Shand (WRL) for carrying out the sediment transport analysis (Appendix B), Dr Peter McComb (MetOcean Solutions Ltd) for providing the raw wave time-series data, Mr Lawrie Cairns (Aerial photography), Mr David Napier and Mr Chris Parkin (NZAM) for providing aerial photographs, and Hawke's Bay Regional Council staff for providing a range of council data.

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APPENDIX A

The derivation of storm inundation parameter values: a brief history

Prior to the late 1990s, inundation hazard assessments were largely based on defining extreme component values from historical observation and summing these extreme values, i.e. it was assumed that all terms were dependent. This later point is particularly important as any two components may be independent (e.g. tide and tsunami), or be dependent (to varying levels) such as waves and storm surge, which may well be driven by the same weather system. For example, if two components are independent, then a 100 yr return period event would occur if one component had a 0.5 yr return period and the other a 0.6 yr return period. By contrast, if the components were moderately dependent (related) then a combined 100 yr return period event would occur if the 0.5 yr component co-incided with a 6 yr component. The nature of the components and their joint occurrence thus become critical.

By the late 1990s some account of independency was being made (NIWA 2000); however, this was really just a guestimate. In 2004, MFE (2004) was advocating a combination of MHWS with a storm surge of 0.9 m (see footnote 1) to provide a realistic tide/storm surge event. We now know this gives a combined return period in excess 1300 yrs. At that time both NIWA (2000) and MFE (2004) avoided recommending appropriate wave parameters values for determining wave setup and wave runup components. The typical outcome was for the practitioner to use an extreme wave estimate based on output from limited wave buoy analysis (e.g. Pickrill and Mitchell, 1979), and this resulted in further over-estimation of a design combined return period. Components used in past assessments for the Mahanga inlet area by T&T'04, Gibb 2007, and Cardio 2010 are listed in Table 4.1; and all, to a greater or lesser extent, adopted this "exploratory" approach.

Over the past decade or so, NIWA have established a network of sea-level recorders around the New Zealand coast, the output of which can decomposed to provide storm surge and other sea-level fluctuations (e.g. temperature-based seasons). In addition, numerical wave models driven by the NOAA's global atmospheric models, provide hourly wave data

1. *The origins of using a storm surge value of 0.9 m for hazard and design assessment comes from observed levels around the New Zealand coast. An extensive record of individual storms has been documented by Heath (1979), Hay (1991) and Bell et al. (2000). Hay (1991) studied 153 storms and found the largest storm surge to be 0.76 m, the second largest to be 0.49 m and 119 were less than 0.35 m. The second largest recorded storm surge since 1890 was for Cyclone Giselle (1969) where 0.88 m was recorded in Tauranga Harbour (NIWA, 2000); an event which had a return period of at least 450 yrs (de Lange, 1996). Based on extrapolation of available storm data, storm surges in NZ have an upper limit of ~1m (Bell et al., 2000) and this consistent for all NZ (Goring, 1995). It seems it was from this setting that the 0.9 m value became established.*

for most of the world's sea surfaces. Verification with marine instruments show these data can provide a reliable record which spans from 1997. Having continuous data records for several years provides a distribution of values from which statistical extreme value analysis can derive return periods for both the individual components and joint occurrences. This more "robust" approach was formally "introduced" to the wider New Zealand professional coastal community by NIWA scientists Ramsey and Stephens (2006).

In the present assessment for the proposed subdivision site I have derived parameter values based on recently available continuous water level and hindcast wave records using ratios given in CIRIA (1996), and carried out analyses such that their combination provides a dependency-adjusted total shoreline runup for a 100 yr return period event. Note that the 50 yr return period values were derived by down-scaling the 100 yr values such that the joint probability total was maintained.

Appendix B Sediment Transport Modelling at Mahanga

Calculated by Kamphius formula and using wave data supplied by MetOceans Ltd.

Sediment Transport expressed in m^3/year calculated according to Kamphius (2002)

$$Q_s = \frac{7.3}{3600} H_b^2 . T_p^{1.5} . m_b^{0.75} . D_{50}^{-0.25} . \sin^{0.6}(2\alpha_b) \quad (1)$$

With parameters given according to below table

Table 1: Sediment Transport Parameters

Parameter	Physical Description	Value
ρ_w	Density of sea water	1025 [kg/m^3]
ρ_s	Density of sand	2650 [kg/m^3]
γ	Breaker index	0.65 [-]
n	porosity	0.32 [-]
H_b	Significant wave height in -8m depth	Varies [m]
α_b	Wave angle at break point	Varies [°]
T_p	Peak wave period	Varies [s]
D_{50}	median grain size	0.2 [mm]

This formula calculates the sediment transport rate for the entire surfzone based on several physical parameters including nearshore wave height, period and angle, sand grain size, surfzone slope etc. The Kamphuis Model has been found in good agreement with physical and field studies without extensive parameter calibration (Smith *et al*. 2003).

Sediment transport calculated for every 1 hour time step over the period January 1998 to January 2010. Wave height given by Metocean output in 8 m water depth. Wave breaking angle calculated according to linear theory and Goda (2007) method for calculating break point.

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19 July 2011

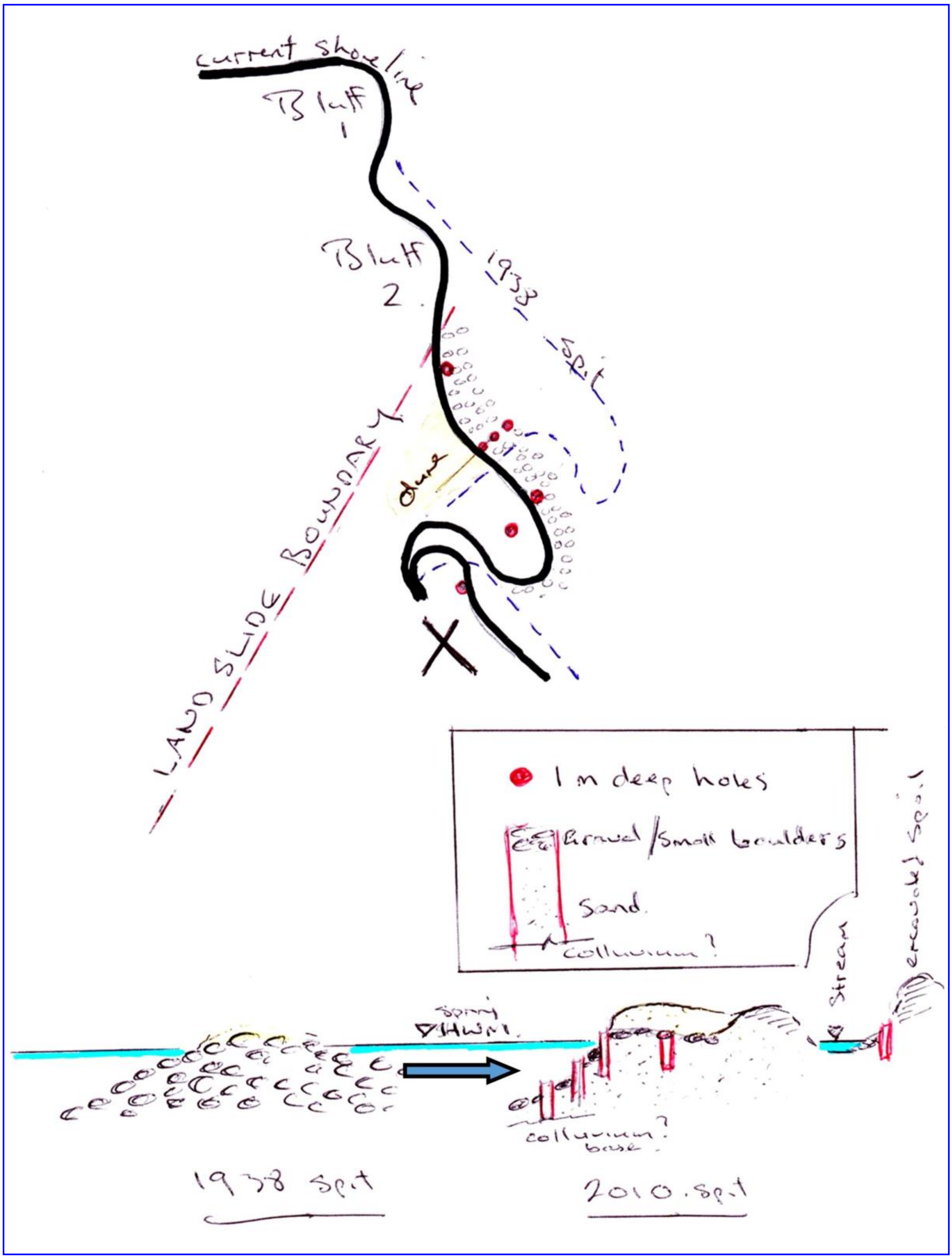
Appendix C Field inspection 10 February 2014

Accelerating spit erosion is evident in data presented in Figure 5.4. This is important as it indicates the system is becoming increasingly unstable and this makes long-term prediction increasingly uncertain

During a later site visit a series of holes (7) were dug across the spit face and crest including the Transect 4 area which is backed by a low sand dune (see sketch).

In each case a veneer some 0.2 to 0.3 m thick of gravel and cobbles with the occasional small boulder covered 0.7 to 0.8 m of sand. The final excavation depth was 1 m.

This result affords the following explanation of the spit's increasing erosion rate. As depicted in the sketch, the 1938 spit lay well seaward of the present spit and we can assume was primarily composed on boulders from the eroding headland. However, as the spit migrated landward, the intervening area to the stream and shoreline reduced and shallowed and increasing wave sheltering enabled sand to settle. This resulted in the core of the migrating spit comprising an increasing amount of sand, and the stone component reducing to the veneer identified today. As the sand portion increased, the structure becomes increasingly erodible resulting in the increasing erosion rate.


current shoreline
Bluff 1
Bluff 2
1938
Spit
LANDSLIDE BOUNDARY
dune
X
1m deep holes
Gravel/Small boulders
Sand
colluvium?
Stream
encroached Spit
spring
HWM
colluvium? base
1938 Spit
2010 Spit