Coastal Flood Hazard Assessment

108C Whanga Road, Whale Bay, Raglan

A report prepared for Mr J. Hemi

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Executive Summary

In March 2006, Coastal Systems (NZ) Ltd were commissioned by Mr J Hemi to carry out a thorough flood hazard assessment (from the sea) for his property at 108C Whanga Road, Whale Bay, Raglan. The assessment was to include a review of the methodology previously used to assess the predicted flood hazard elevation (as presented to the Waikato District Council resource consent hearing in December 2005), acquire the necessary and available data to accurately determine the flood hazard level for a 100 yr return period event (this being the best practice value used in flood hazard assessment in New Zealand), and accurately determine return periods for the storms in April, 1999 and September, 2005 which caused some flooding at the margins of the property, and which would be used to verify the predicted flood hazard level.

Review of the previous assessment identified significant error with mean sea level (MSL), and also in the determination of the design elevations for the proposed dwelling. These errors amounted to 1.3 m. The corrected MSL is 93.5 m relative to local datum.

The flood assessment was based on sea-level, barometric pressure and wind data obtained from the New Zealand National Institute of Water and Atmospheric Research (NIWA), deepwater wave and wind data obtained from the American National Oceanographic and Atmospheric Administration's (NOAA) global models, plus site-specific wave and run-up data collected by Hamilton surveyors Skyworks Waikato Ltd. In addition, detailed contour mapping was carried out by Skyworks. Data analysis was carried out by NIWA, Coastal Systems (NZ) Ltd and Tonkin and Taylor Ltd. During the investigations, discussions were held with Mr Jim Dahm, the Waikato District Council's coastal consultant, to ensure acceptable methodologies and assumptions were being used. It is also noted that considerable effort went into assessing the effect of waves on flooding because of the unique site characteristics – low relief and exposure to high wave energy. For this reason, Tonkin and Taylor Ltd were commissioned to provide a report on boulder bank run-up and overtopping implications for the proposed house site.

Part A of this report deals with the predicted flood hazard level. Standard statistical procedures were used to derive the hazard component values (storm-surge, tide, wave effect, intra and inter-annual sea-level variation and sea-level rise associated with global warming). Two combinations of waves and storm surge were used: monthly waves with a 0.9 m storm surge, and yearly waves with a 0.5 m storm surge. These combinations, when coupled with the other hazard components give return periods well in excess of 100 yrs and the resulting hazard levels are thus conservative. The resulting predicted inundation levels at the Hemi building site are 3.93 and 4.07 m above MSL (97.43 and 97.57 m local datum). These values are 0.13 to 0.27 m higher than the value of 3.8 m used earlier at the December 2005 resource consent hearing.

Part B of the report deals with assigning inundation return periods to the storm events of 17 April, 1999 and 18 September, 2005. As continuous records of sea-level and associated wave conditions are generally not available in NZ, return periods have to be derived by using the components' return periods and applying appropriate methods of

combination which depend on the likelihood of joint occurrence. Such an approach was used in this investigation.

The 1999 event was found to have an inundation level at Whale Bay of ~3.0 m above MSL (96.5 m local datum) and a return period less than 100 yrs. By comparison, the 2005 event had an inundation level of 3.36 m above MSL (96.86 m local datum) and a return period equal to or greater than 100 yrs. These values are consistent with the predicted flood hazard levels given above if 100 yrs of sea-level rise (0.45 m) is excluded. Those values then become 3.48 to 3.62 m above MSL (96.98 to 97.12 m local datum) and, as noted above, the predicted flood levels have a return period of at least 100 yrs.

A comparison of the predicted inundation levels (3.93 to 4.07 m above MSL) with the critical assessment level of 4.2 m used at the 2005 hearing, this being the level of the basement floor, shows that flooding during a storm equivalent to the hazard design event will fail to reach this level by 0.13 to 0.27 m. While this may appear to be a relatively small margin of safety, it is significant that the predicted inundation hazard level return periods are well in excess of 100 yrs. Further confidence is provided by the site being 0.84 m above the flood level associated with the 2005 storm, an event found to have a return period of at least 100 yrs.

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Introduction

An application by J Hemi for a Land Use Consent to construct a (second) dwelling at 108C Whanga Road (Fig 1) was heard by the Waikato District Council during 19-21 October, 2005 and 9 December, 2005. The hazard assessment presented by the applicant's coastal consultant, Dr Roger Shand of Coastal Systems (NZ) Ltd, at the December hearing, was based on a standard inundation analysis using, where possible, component values generally applicable to the NZ coast. Some site-specific professional judgment for wave impacts was included. Verification of the predicted hazard level was attempted by assigning return periods to storms that were known to have caused flooding in the Whale Bay area in 1999 and 2005. However, the comprehensive data required to accurately assess these storms was not obtainable within the available preparation time, so they were qualitatively assessed using their physical characteristics, the extent and nature of their environmental impacts and testimony by local residents. The council's coastal consultant (Mr Jim Dahm of Eco Nomos), and Dr Shand estimated the return periods ranged between 10 yrs and 30 yrs. The 100 yr return period inundation level (including a component for sea-level rise associated with global warming) was determined to be 3.8 m above MSL, or 0.8 m above the floor of the basement, by Dr Shand. Such occasional inundation was acceptable to the applicant as the dwelling had been designed such that the floor level would be some 1.8 m above this predicted 100 flood level. While the council accepted the inundation value (3.8 m above MSL), the application was declined, primarily on the basis that the council considered it contravened NZCPS Policy 3.4.5 which states that "New subdivision, use and development should be so located and designed that the need for hazard protection works is avoided".

J Hemi subsequently lodged an appeal with the Environment Court and in March, 2006 and instructed Coastal Systems to carry out a thorough flood hazard assessment. This work was to include a review of the methodology previously used to assess the predicted flood hazard elevation, acquire the necessary and available data to accurately determine the flood level, and determine accurate return periods for the 2 storms which would be used for verification.

The methods review found significant sources of error in the derivation of MSL, and also in its application to determine the building's design elevations. It is noted that time constraints meant that both consultants (Coastal Systems and Economos) had to assume materials provided to them prior to the 2005 hearings were accurate. These errors subsequently resulted in the flood depths presented to the 2005 hearings being some 1.3 m too high! This unfortunate situation was explained to council staff at a progress meeting held on 8 June, 2006. The means by which MSL was accurately determined are explained in Appendix A, these materials being presented to Mr Dahm and council officers at the 8 June meeting.

Mean sea level was fixed at 93.5 m relative to the local site datum; this compares with the value of 94.4 m used earlier. The value of 93.5 m was selected on the basis of two precise measurement exercises on the 23 March, 2006 and 21 April, 2006 which derived values of 93.50 and 93.46 m. It is noted that a value of 93.6 m was tentatively used at the 8 June meeting and is presented in Appendix A, Table 3. However, this value was based on the inclusion of two additional measurements (93.50 m and 93.53 m) derived during exploratory field work on 22 February and 1 March, 2006 (as explained in Appendix A). When these less reliable measurements are discarded, the value of 93.5 m becomes appropriate and this was used as a basis for all subsequent measurement and analysis.

The corrections resulted in the 100 yr hazard level (3.8 m) now being 0.4 m below the basement level compared with being 0.8 m above it as derived in the 2005 assessment. However, it also meant that the 2005 storm flood level increased from 2.6 to ~3.4 m relative to MSL. That storm flood level had previously been assigned a return period of up to 30 yrs by the consultants based best professional judgment and the earlier MSL of 94.4 m. This new value now compares very closely with 3.35 m for the predicted 100 yr hazard level (this being 3.8 m less 0.45 m for 100 yrs of sea-level rise, see Appendix A, Table 3). This situation meant that either the assigned return period (up to 30 yrs) for the September 2005 storm was too low, or else the predicted 100 yr flood level was too low. This inconsistency was to be resolved during the forthcoming detailed assessment.

By December, 2006, the investigation was well advanced and discussions were held with Mr Dahm at a meeting in Hamilton. The purpose of that meeting was to:

- Familiarize Mr Dahm with the research,
- Identify issues Mr Dahm considered needed further attention, and
- Reach agreement on a range of parameter/value options.

The final analysis and write-up of the full report was then undertaken.

The report contained herein consists of two parts: Part A derives the hazard inundation level(s), while Part B assigns return periods to the 1999 and 2005 storms. Particular attention has been paid to the acquisition and use of data collected by credible agencies and subcontractors and to the collection and use of quality data from the site itself, to the use of accepted statistical procedures to derive hazard components, their return periods, and component return period combinations, and finally to ensuring caution when selecting representative values. The results are summarized and conclusions drawn in the final section of the report.

Part A. Inundation assessment

A1 Definition of inundation level

In New Zealand, it is considered best practice to use the following flood hazard prediction formula:

Inundation level = storm surge + tide level + sea-level variation + sea-level rise + wave effects (1)

Where the components consist of:
- storm surge = inverted barometric pressure + wind set-up;
- tide level measured relative to MSL;
- sea-level variation related to monthly to multi-decadal variation;
- sea-level rise (SLR) resulting from the effect of global warming;
- wave effects = wave run-up (swash plus wave set-up) plus overtopping considerations.

As noted earlier, general values are available for some components. However, for other components it is desirable, and in some cases mandatory, to derive site-specific values and this has been an major objective of the present exercise.

A2 Inundation recurrence

The agreed flood recurrence value is to be the 1% AEP (annual excedence probability), i.e. 100 yr return period (RP). However, in the interests of caution, the derived flood level exceeds this value.

Care has been taken to combine components using appropriate techniques based on variable' dependency-independency status and the probability of joint occurrence. These concepts are considered in greater detail in section B3. However, it is noted here that different return period combinations of two components can produce a 100 yr output. For example a (relatively high) 10 yr tide occurring with a (relatively low) 0.03 yr (11 days) wave gives a 100 yr return period event, while a relatively low (0.03) tide combined with a (relatively high) 10 yr wave also gives 100 yrs, as will numerous intermediate combinations. The relative importance of water level and wave conditions depend on the particular coastal response in question. Stability of engineering structures is more dependent on wave height, while beach response tends to be a function of both water level and waves. In the case of flooding by overtopping, it is water level that is more important as wave effects will usually be ameliorated by depth limitation. It will be shown in this report that water level is the predominant control at the Whale Bay site and greater emphasis is made in the flood level computation process of relatively longer water level return periods compared with those for waves for this reason.

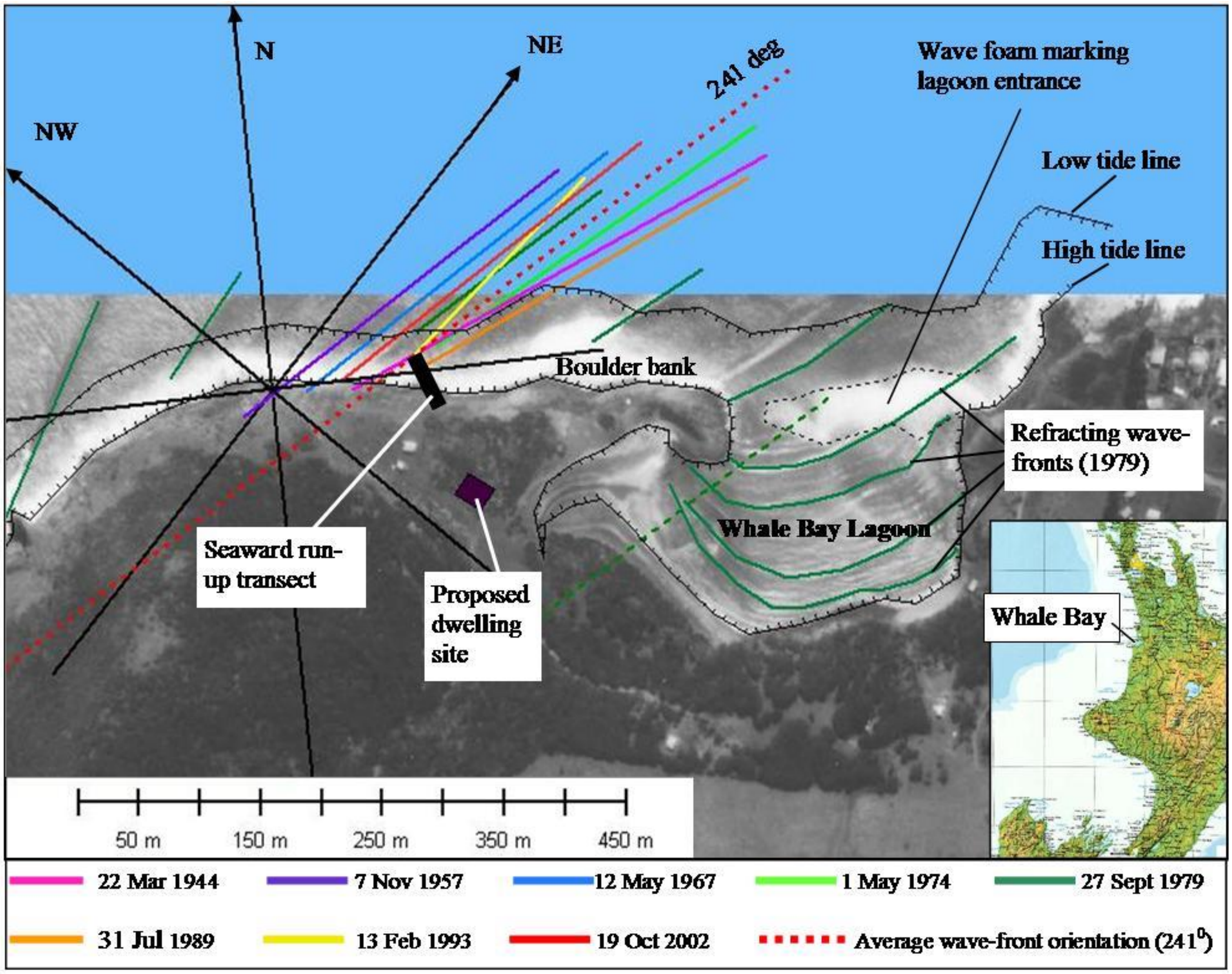


Figure 1 Location maps of the study site. Crest alignments are marked as waves approach the seaward run-up site (black bar). This location was used for the Tonkin and Taylor run-up/ overtopping study (see Appendix B) as waves overtopping at this location are in line with the building site. The wave crests were identified from geo-rectified aerial photos (see key). The 1979 photo is depicted and the full set of its associated wave fronts have been marked to illustrate the intense wave refraction-effect that occurs between the seaward run-up site and that innermost part of the lagoon fronting the building site. The dashed green line ······ depicts the wave crest line from the seaward run-up site overlaid upon the fully refracted crest line within the inner lagoon to illustrate subsequent refraction in the order of 90°.

A3 Storm surge

A3.1 Sea-level data

No sea-level data are available for the Raglan/Whale Bay area. The closest monitored sites are at Anawhata (NIWA) and New Plymouth (Westgate), which are approximately 100 km and 150 km to the north and south respectively.

As the Anawhata sea-level record is the longer (7 yrs between 1998 to 2006), and as with the Westgate record is also fragmented, NIWA were commissioned to undertake an extreme value analysis (EVA) on the former. The raw sea-level measurements were first corrected for missing or corrupt data and then each month of data was normalized by its monthly average; this process was carried out to remove lower frequency variation in MSL. Storm surge values were next obtained from the sea-level values by subtracting corresponding inverted baromentric pressure values. NIWA's extreme sea level analysis software - EXTLEV was then used to identify the largest number of independent extreme events per year (r-Largest Method), usually 5, and carry out a Gumbel fit. The resulting extreme values are summarized in Table 1 and show the 100 yr return period (RP) storm surge = 0.52 m. It is further noted that the difference between 10 yr and 100 yr RPs is 0.1 m.

Table 1 EVA of Anawhata storm-surge levels

Return Period (yr)	5	10	20	50	100
Surge height* (m)	0.39	0.42	0.45	0.49	0.52

* relative to the mean level of the sea (MLOS)
Source: NIWA

While these results should only be considered *broadly indicative* of Whale Bay because of the relatively short length of record and the contrast in coastal exposure between the two sites (discussed later), some long-term extrapolation confidence is afforded for the following reasons. The r-largest methodology used by NIWA fully utilizes the data but traditional EVA only uses the highest value per year. In addition, confidence is increased by the similarity of the barometric extreme values for 50 and 100 yr using the long-term (41 yr) record from Auckland Airport and the more recent 7 yr record from Anawhata. In particular, the 50 yr values are identical at 971.7 hPa, while the 100 yr are very close at 968.8 hPa c.f. 968.4 hPa. These results suggest that for this area the more severe events have happened during the more recent period and the Anawhata data is more representative of the longer-term than would otherwise be expected.

A3.2 NZ-wide storm-surge record

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). However, the level corresponding to the 100 yr return period would be considerably lower (NIWA, 2000).

A3.3 Assessment values

Based on the above results, it was agreed (Dahm-Shand meeting 5 December 2006) that an upper and lower storm surge level would be assigned for the Whale Bay inundation calculation. In particular, the lower value would be 0.5 m and the upper value 0.9 m. Note that a value of 0.9 m was used for the December 2005 assessment. It is acknowledged that the upper value will greatly exceed 100 yrs when combined with other inundation components.

A4 Tide level

Tide levels and the associated EVA for Whale Bay were obtained from NIWA. These data were obtained using their tidal model at 15 minute intervals over a 100 yr period. The levels and return periods are given in Table 2. As tides and storm surge are independent, their combined return period can be obtained by multiplying probabilities (see section B3.2). Combining the mean high water spring (MHWS exceeded by 18.5% of high tides) return period of 0.0075 yrs, with the 0.5 m storms surge's return period of 20 to 50 yrs, and also the 0.9 m surge of > 100 yrs, gives return periods of 55 to 137 yrs and >274 yrs respectively. As the combined return periods will greatly exceed 100 yrs in all cases once wave effects are incorporated (see section A7.1 and section B3.2 Table 12), a lower tide level could be chosen. Indeed, even using the mean high water level's (MHW exceeded by 50% of high tides) RP of 1.17 m, the 100 yr threshold would be exceeded once waves were incorporated. However, the more conservative tidal value of 1.4 m (exceeded by 25% of high tides) as used in the 2005 assessment will continue to be used in the inundation computation. Note that this gives combined surge plus tide return periods of 40 to 100 yrs for the 0.5 m surge and >200 yrs for the 0.9 m surge.

A5 Lower frequency sea-level fluctuations

Mean sea level may fluctuate over months to decades due to several longer-term factors such as seasonal weather changes in temperature and windiness, ENSO-based climatic oscillations and IPO shifts; the limited long-term open coast sea-level records suggest inter-annual elevation changes of up to 0.2 m could occur (Bell et al., 2000). The storm-surge values based on the individual storms (section A3.2) will have included such variation. By contrast, the Anawhata storm-surge extreme values will be less influenced as the input data had had at least a portion of this variation removed during preprocessing as described earlier in section A3.1. This could certainly help explain the lower RP values derived from the Anawhata sea-level data.

As the storm surge values selected for use in the inundation computation were higher values based more on the individual storm data, and in recognition of the likelihood of 'double dipping' should a separate lower frequency sea-level variation component be included in the calculation, it was agreed (Dahm-Shand meeting of 5 December, 2006) not to include any such a component.

Table 2 Calculated high tide levels (above MSL) and return periods for Whale Bay

Level (m)	Descriptor	RP (days)	RP (yrs)
1.17	Mean = 50% of all high tides	1	0.0027
1.47	Spring = 18.6% all high tides	2.25	0.0075
1.56	Pragmatical = 12% all high tides	4	0.011
1.69	MHWPS = 4.8% all high tides	10	0.027
1.775		30	0.082
1.81		61	0.167
1.826		92	0.252
1.849		178	0.488
1.863		267	0.732
1.868		365	1
1.884		730	2
1.891		1095	3
1.894		1460	4
1.896		1825	5
1.901		2737.5	7.5
1.903		3650	10
1.906		5475	15
1.907		7300	20
1.909		10950	30
1.911		14600	40
1.912	HAT Highest astronomical tide	18250	50

Source NIWA

A6 Sea-Level Rise

Hazard assessment requires a component be included to account for the projected rise in sea-level over the coming 50 to 100 years. It is current best-practice to use the mid-range projection for 100 yrs which has been given as 0.45 m by NIWA (2000), or 0.3 to 0.5 m by MFE (2004). The value of 0.45 m, which was also used in the December 2005 assessment, will continue to be used in the present hazard calculation.

A7 Wave effects

Waves could affect the inundation levels at the building site either via overwash resulting from run-up and overtopping of the seaward boulder bank, or via wave action within the lagoon. Accounting for wave effects in the 2005 assessments was the main difficulty encountered by consultants. It was therefore necessary to thoroughly investigate this matter using site-specific data for calibration-verification. Boulder bank overtopping has been addressed in a theory-empirical study by Tonkin and Taylor Ltd., whilst lagoon effects have been addressed by a theory-empirical study carried out by Coastal Systems (NZ) Ltd. However, before describing these studies and results, the deepwater wave height to be used in the flood assessment will be determined.

A7.1 Assessment wave heights

Joint probability analysis was undertaken to determine the combined return period of sea level with waves. This was carried out using the methods detailed in CIRIA (1996) which are detailed later in section B3.2(A) when considering the return period of the April 1999 and the September 2005 storms. The lower sea level estimate of 40 yrs in section A4 requires a wave height return period of 0.08 yrs to give a combined return period of 100 yrs. The remaining sea-level estimates of 100 to 200 yrs in section 4 would require no more than about average wave conditions. In keeping with a conservative approach it was decided to combine monthly return period waves with the 0.9 m storm surge, and yearly return period waves with the 0.5 m storm surge (Dahm-Shand meeting, 5 December, 2006).

Wave parameters are generated at three hourly intervals by the National Oceanographic and Atmospheric Administration's (NOAA) NewWaveWatch3 (NWW3) wave model, a third generation ocean wave propagation model which is the world standard. NWW3 uses wind fields to solve the spectral action density balance equation for wave number-direction spectra which can be applied to virtually any location on the planet. Significant wave heights, periods and directions for a site 3.5 km offshore and at 20 m depth were extracted for all 9 yrs of available wave data using the MetOcean Data Interface (MDI). These data were separated into 45 degree bins centred on NW, W, and SW. An EVA was performed using the MatLab suit which applied standard statistical procedures such as Weibull and Fisher-Tippet distributions to the 25560 sets of data. The best-fit solution was then selected to represent the return periods (see Table 3A).

The EVA output in Table 3A shows similar results for southwesterly and westerly waves, with slightly higher waves from the southwest corresponding to return periods up to 1 yr, and higher waves from the west corresponding to the longer return periods. It is noted that a breakdown into smaller bins found that WSW waves actually had the highest values.

Before deciding which wave direction to use when selecting the assessment values, several additional factors must be considered. Firstly, deepwater waves undergo shoaling and varying levels of refraction before reaching breakpoint at the site. These processes alter wave heights so their effects must be quantified.

To determine the average refraction angles that apply to waves from the directional different bins during deepwater to breakpoint transformation, wave crests from available vertical geo-rectified aerial photographs (1944, 1957, 1967, 1974, 1979, 1989, 1993, 2002) were plotted in the vicinity of the boulder bank run-up transect (section A7.2). There should be a range of deepwater wave directions within a sample of this size. The average orientation of the 8 crest lines (Fig 1) was 241 deg (see dashed red line). There was a narrow range of angles (232 to 248 degrees) indicating all incident wave directions are able to refract to reach a similar final breaking orientation.

Table 3 EVA output for deepwater waves and wind speed

A. Wave Analysis

Return Period (yrs)	Northwest bin	West bin	Southwest bin
fortnightly 0.042	3.2	4.56	4.73
monthly 0.083	3.62	5.04	5.17
3 monthly 0.25	4.23	5.76	5.85
6 monthly 0.5	4.6	6.18	6.27
9 monthly 0.75	4.8	6.42	6.51
1	4.9	6.65	6.48
5	5.65	7.56	7.35
10	5.95	8.06	7.71
25	6.34	8.6	8.17
50	6.62	8.99	8.52
100	6.89	9.38	8.86

B. Wind Analysis

Return Period (yrs)	Northwest bin	West bin	Southwest bin
1	19.09	19.61	20.06
5	21.68	22.11	22.47
10	22.72	23.24	23.45
25	24.05	24.56	24.69
50	25.02	26.53	25.6
100	25.96	26.47	26.48

Data source: NOAA (see text)

These data indicate NW waves refract 16 deg to reach the run-up transect, W waves refract 61 deg and SW waves refract 106 deg. As noted above, the largest waves in the W bin (and hence those controlling the EVA output) are tending WSW, so they will undergo in excess of 61 deg refraction.

From the wave-crests depicted in the 1979 photo underlying Fig 1, it can be seen how the waves refract along the boulder bank and on into the lagoon until facing the building site. At this point a wave has refracted about 90 deg beyond the orientation at the run-up transect on the boulder bank. Note that the 1979 photo was the only one taken at high tide and accompanied by large waves. Fortuitously, in this sample the crest line at the run-up monitoring transect had an orientation of 241 deg which was the same as the average value for all 8 samples.

Determining the wave height transformation from deep water to breakpoint during refraction and shoaling was carried out using the DELFT Coastal and River Engineering Software System (CRESS). Deep and shallow water wave characteristics are determined using first and second order theory respectively, and shoaling and refraction determined using standard equations. Monthly and annual return period wave height for the west and northwest bins (Table 3) were transformed for the angles (Fi) noted above. The average wave period (T) was determined for these waves and a slope-based coefficient

(γ) was also used as input. The modelling input and output values are summarised in Table 4. Note that southwest bin waves were not been included as the deepwater wave heights approximated westerly wave values and after the additional refraction, the transformed heights were significantly lower.

The results in Table 4 show that the transformed westerly waves approximate the transformed northwesterly wave heights. However, at noted above, the extreme westerly waves actually have a more southerly direction and thus undergo greater refraction. For example, another 10 degrees refraction (71 degrees) results in an additional 15 % height loss. The results also show that the transformation affected the waves of each bin differently with westerly waves reducing by approximately 22% at the breakpoint, while northwesterly wave heights remained approximately the same. These results indicate that for westerly waves the refractive loss dominated attenuation gain, while the opposite effect occurred for NW waves. Overall, the results in Table 4 support the use of monthly and yearly return period waves from the northwest for use in the inundation calculations.

Table 4 Shoaling and refraction transformations for westerly and northwesterly waves

	Westerly waves		Northwesterly waves	
	Monthly	Yearly	Monthly	Yearly
Ho	5.25	6.65	4.11	4.9
T	11	11	10	10
γ	0.57	0.57	0.57	0.57
Fi	61	61	16	16
Hb	4.1	5.1	4.2	4.9

Input data sources: NOAA (waves), Raglan 1:200,000 Bathymetric Chart, Fig 1 (angles)

The use of NW waves is also appropriate as maximum storm surge (inverted barometric pressure (iBP) and wind set-up) will accompany depressions originating in the north Tasman Sea and then tracking SE to cross the west coast to the south of Raglan. This allows the site to be affected by the maximum possible iBP and the maximum wind set-up entrapment within the Mt Karioi embayment. This issue is discussed further in the later Part B when describing the 1999 and 2005 storms.

A7.2 Wave effects from the sea

The likelihood of flooding at the proposed dwelling site via wave overtopping of the boulder bank was the subject of an investigation by Tonkin and Taylor Ltd and their report (Tonkin and Taylor, 2006) is attached as Appendix B. Briefly, this study consisted of a run-up analysis followed by an overtopping analysis. The former was based on the model used by Mr Dahm in his September, 2005 report to the Waikato District Council (Eco Nomos, 2005). While the methodologies and numerical calculations were found to be sound, several assumptions and parameter values were based on either incorrect approximations or outdated information. Using site-specific run-up data collected under a range of wave conditions, it was possible to calibrate the model to fit the characteristics of the Whale Bay environment. Run-up elevations were produced for the range of water levels and wave heights likely to be experienced at the site. Results indicated that although overtopping would not be as severe as predicted in the Eco Nomos report, some overtopping is likely during certain water level and wave combinations.

An assessment of overtopping discharges was then carried out. The applicability of the overtopping discharge model was verified by satisfactorily reproducing the 2005 storm overwash limit. Model output for the parameter values being used in the present assessment (sea-level = 95.85 to 96.25 [tide = 1.40, storm surge = 0.5 to 0.9 m, sea-level rise = 0.45 m, MSL = 93.5m], and wave height = 4.1 to 4.9 m, plus an order of magnitude safety factor), shows overwash would extend <30 m landward of the crest of the boulder bank and this compares with ~80 m to the building site (see Appendix B, Fig 3 for SWL = 96.0 m). Note that the 95.85 m sea-level goes with the 4.9 m waves. The Tonkin and Taylor Report essentially shows that wave overtopping of the boulder bank will not present a flood hazard at the dwelling site.

A7.3 Wave effects from the lagoon

This study applied the standard run-up model for sandy shorelines (CEM, 2002) by using site-specific parameter values. The results were then compared with run-up levels and lagoon water levels observed during relatively high sea-levels and wave conditions. This approach enabled a lagoon run-up value to be defined which was normalized with respect to deep water wave height (NW bin); this coefficient could then be applied to assessment wave heights to identify the lagoon-based wave effect.

A7.3.1 Theoretically-based run-up

Extreme run-up on a smooth sloping beach is typically determined using the equation stemming from the works of Hunt (1959) and Battjes (1974) and the CEM (2002) uses the following form of their equations:

$$R_{2\%} = H_o 1.86 \xi_o^{0.71} \qquad (2)$$

where $R_{2\%}$ is the wave run-up height above average SWL that only 2% of waves exceed, and ξ_o is the deep water dimensionless 'surf similarity number' or 'Iribarren number' which is defined as:

$$\xi_0 = S/(H_o/L_o)^{0.5} \qquad (3)$$

where S = average slope (tan β), and L_o = the deepwater wave length defined by:

$$L_o = (g2\pi)T^2 \qquad (4)$$

where T is wave period.

The formula applies to irregular waves and has been confirmed under a range of incident wave conditions (Guza and Thornton, 1981; Holman and Sallenger, 1985). It should be noted that such extreme run-up formulations measure total run-up which includes wave set-up and swash run-up.

While these formulae were developed for slopes >0.01, application of equation (1) to lower slope situations is generally made on the grounds that it over-predicts run-up from field data (Holman, 1986) by a factor of two. In addition, for the Whale Bay lagoon situation, further application assurance is given by reduced wave energy reaching the shore due to storm waves having already undergone breaking at the mouth of the lagoon (see Fig 1) and reformation before travelling across the lagoon, refracting, and finally re-breaking prior to run-up. Note that the lagoon entrance is only about 1 m below MSL, so under extreme water levels (MSL + 2 to 2.5 m) waves higher than about 1.8 to 2.1 m are depth limited. Considerable wave energy is lost during this initial breaking process with field measurements showing reformed waves rarely contained more than 20-40% of their incident value (Carter and Balsille, 1983). This argument assumes that the controlling rocky morphological configuration of the spit and lagoon will remain the same during the next 50 to 100 yrs. This assumption is reasonable based on no change being evident when inspecting the superimposed aerial photos; in fact individual boulders and clusters of boulders could be matched.

Using T = 10s, tan β = 0.012, $H_{monthly}$ = 4.1 m and and H_{yearly} = 4.9:

- L_o = 156 m, (equation 4);
- ξ_0 = 0.074 for $H_{monthly}$ and 0.068 for H_{yearly} (equation 3), and
- $R_{2\%}$ = 1.20 m for $H_{monthly}$ and 1.35 m for H_{yearly} (equation 2), which results in wave-normalized values of 0.292 and 0.275 respectively. These normalized run-up values (coefficients) appear in Table 5.

Table 5 Theoretical and empirical lagoon run-up values normalized by wave height*

Run-up method	$H_{monthly}$ = 4.1 m	H_{yearly} = 4.9 m	$H_b$sig
$R_{2\%}$ (equation 2)	0.292	0.275	
Water level (10.7.06)			0.28
Run-up (11.10.05)			0.258

*Normalization for equation 2 output was with respect to NW extreme wave height values. Normalization for empirically determined values was $H_b$sig (which was measured concurrently at the boulder bank run-up transect, see Tonkin and Taylor (2006), Table 2), transformed to H_o for the NW sector.

A7.3.2 Empirically determined run-up

Lagoon run-up under higher wave and tide conditions was carried out using water level data and run-up data.

(i) Water-level approach

This approach was based on the analysis of a water-level record that was obtained using a float arrangement (Fig 2) that was tracked by video camera. The apparatus was located within the lagoon in a region observed to have maximum surface fluctuation (Fig 2). The survey was carried out over the 8.40 am high tide on 10 July, 2006 when a run-up survey on the boulder bank was also underway, H_b data were thus available for use in the analysis. Environmental conditions were: tide = 1.05 m; storm surge (average of Anawhata and Westgate) = -0.1 m; $H_b$sig = 4.8 m; T = 16.5 s. Using the CRESS wave transformation model, equivalent Ho from the NW sector = 3.95 m.

The water-level data were abstracted using an image processing algorithm written by Professor Donald Bailey of the Institute of Information Sciences and Technology at Massey University. A sample of the record is depicted in Fig 3A while the full 40 minute record is represented by the histogram in Fig 3B. Note that the lower frequency wave motions evident in Fig 3A are at ~3 min intervals. This value was similar to the wave group periodicity observed during the seaward run-up survey and was expected given the low slope in the lagoon. The maximum reformed wave height is estimated at 1.1 m. Note that 0.1 m has been added to the signal to compensate for truncation of the lower values as explained in the caption for Fig 3.

While exact conversion of the maximum wave height to run-up is not possible, it is noted that on sandy beaches run-up is usually much less than the height of waves seaward of the breakpoint, this reduction being due primarily to friction and turbulence. However, where lower frequencies predominate and topography influences wave behaviour, maximum run-up could conceivably equal wave height. In this situation the normalized run-up value would be 1.1/3.95 = 0.280 m and this result has been included in Table 5.

(ii) Lagoon run-up measurements

Run-up elevations were obtained from a video record taken during the 13.15 hr high tide of 11 October, 2006 when the final run-up survey was being conducted on the boulder bank. Environmental conditions were as follows: tide = 1.44 m, storm surge (average of Anawhata and Westgate) = 0.04, H_b sig = 4.5 m and T = 15 s. Using the CRESS wave transformation model, equivalent Ho from the NW sector = 3.8 m.

Elevations corresponding to the three highest run-ups were 95.96 m 95.90 and 95.86 m. A frame depicting the highest run-up appears as Fig 4. Subtracting tide, storm surge and mean sea level from the highest elevation (95.96 m) gives a run-up value of 0.98m. Normalizing this value with respect to NW sector deepwater wave height gives 0.98/3.8 = 0.258. This value has been added to Table 5.

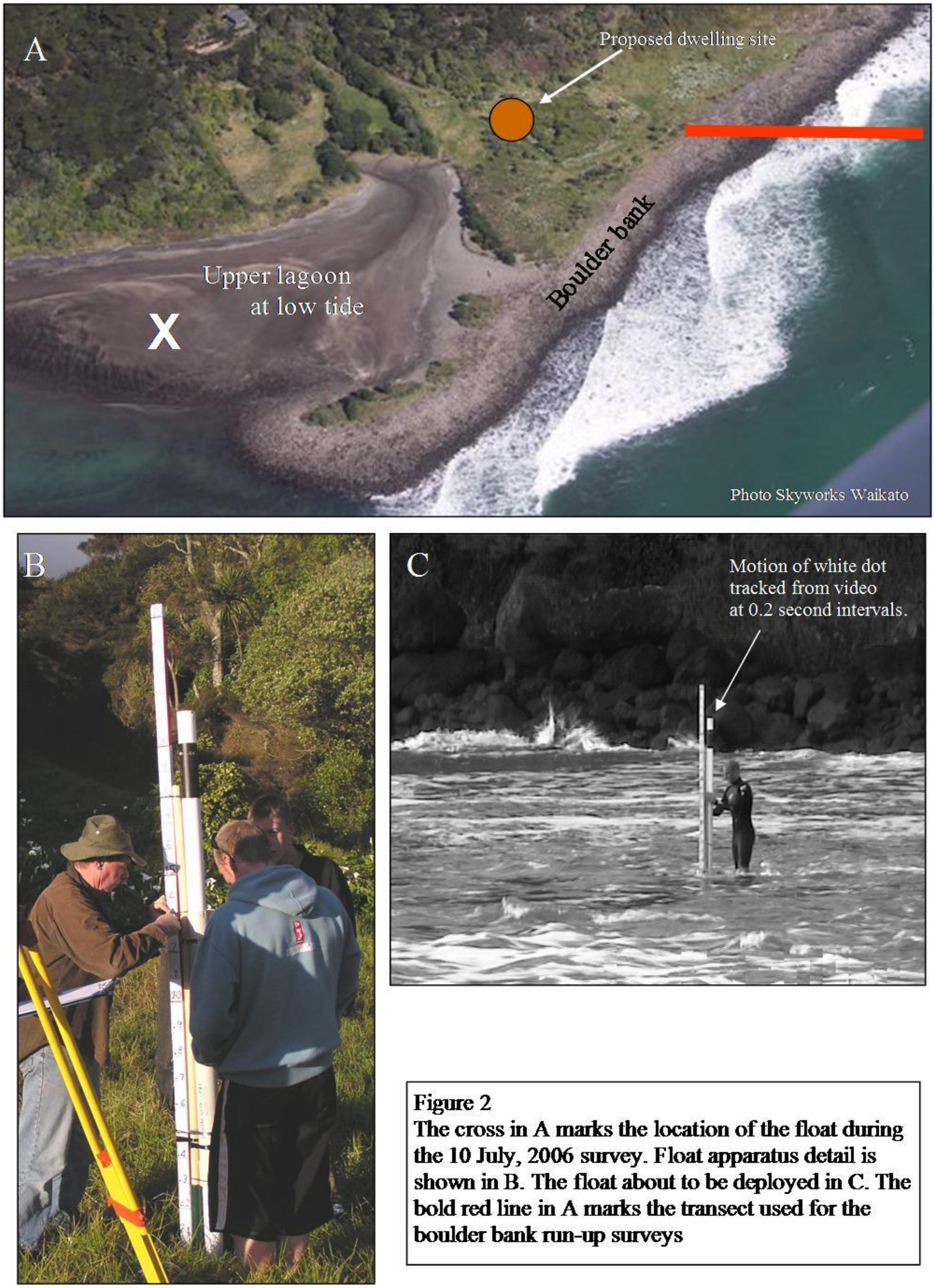


Figure 2
The cross in A marks the location of the float during the 10 July, 2006 survey. Float apparatus detail is shown in B. The float about to be deployed in C. The bold red line in A marks the transect used for the boulder bank run-up surveys

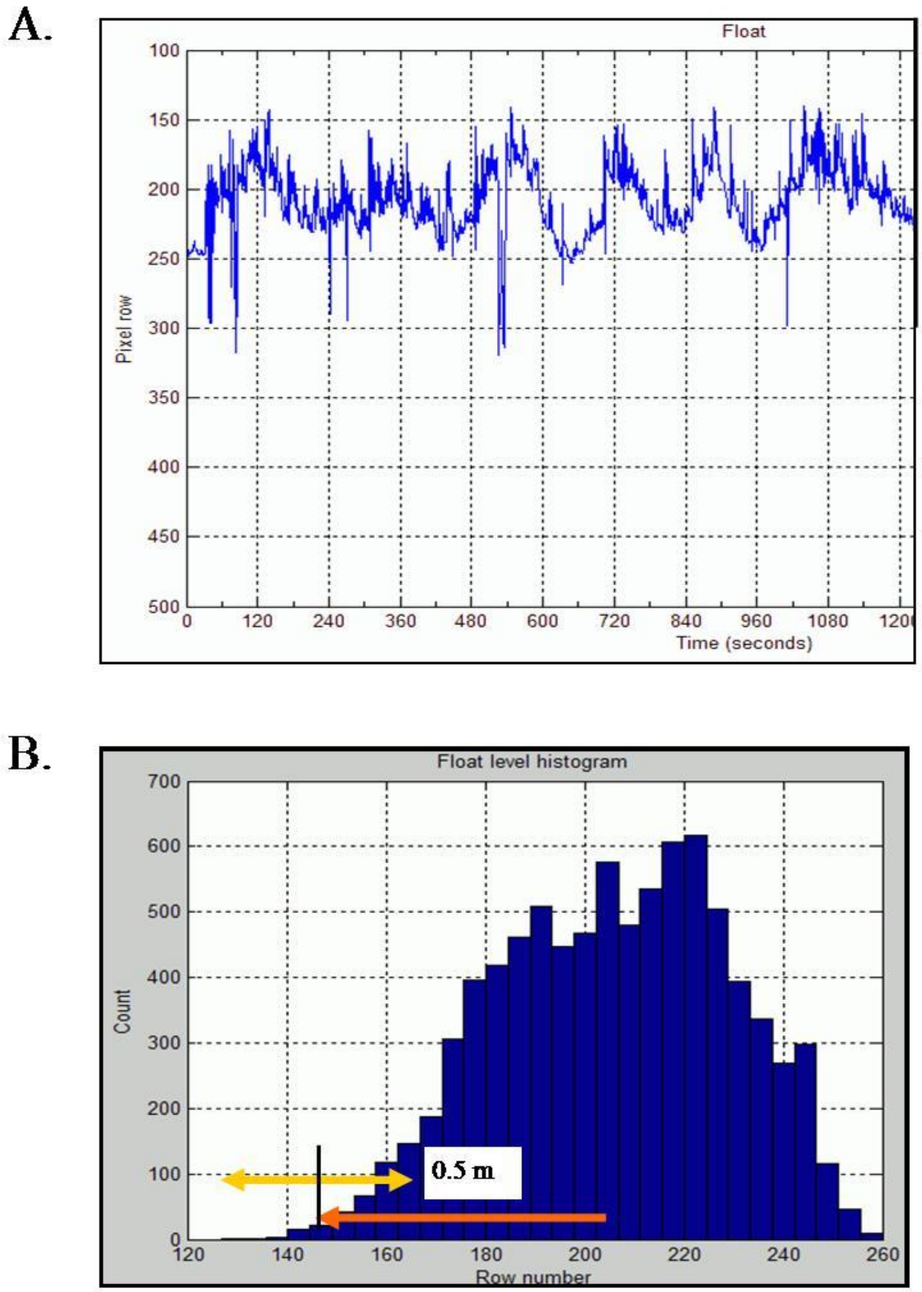


Figure 3 Water level output (A) for 20 mins of record. Elevation scale is 50 pixels = 0.44 m . The downward spikes relate to 'instrument wave shock'. These spikes were cropped and this resulted in the truncated tail on the right hand side of the histogram in B. The large red arrow in B depicts the maximum elevation above SWL (from a centre of mass at 206 pixels). The yellow double arrow separates out the higher frequency noise evident in A—which observation showed related to 'bobbing' of the float when stuck by wavesby up to 0.1 m

Figure 4 A video frame depicting the maximum run-up from the high tide of 11 October, 2006. The ebb flow/back-rush has just begun affecting the main water body (large arrow), while the final run-up is still occurring in the foreground (small arrow). The camera was located at the top of the embankment backing the lagoon beach and in line with the proposed dwelling site.

A7.3.3 Discussion on lagoon wave effect

The normalized theoretical and empirical run-up values (coefficients) in Table 5 are remarkably close, ranging between 0.258 and 0.292, and give confidence to the methodologies. It is also helpful that the most reliable estimate, the actual run-up measurement made under higher wave and tide conditions (0.258), happens to be the lowest value. A representative value for the lagoon wave effect coefficient of 0.323 was selected for use in the inundation calculation; as this value provides a 25% safety margin over the most reliable value of 0.258.

A8 Discussion of Part A (inundation assessment)

Component values derived in the preceding sections have been summarized in Table 6. The predicted inundation levels at the Hemi building site, using established and/or agreed to criteria, range between 3.93 and 4.07 m above MSL (97.43 and 97.57 m above local datum). These values are 0.13 to 0.27 m higher than the value (3.8 m) proposed at the December 2005 hearing,

Table 6 Summary of inundation values

WAVES:	$H_{monthly}$=4.1 m	H_{yearly} = 4.9 m
Storm surge	0.9	0.5
Tide	1.4	1.4
Sea level variation	0	0
Wave effect_sea	0	0
Waves effect_lagoon: 0.323*H	1.32	1.58
SLR	0.45	0.45
Totals (MSL datum)	4.07	3.93
MSL	93.5	93.5
Totals (local datum)	97.57	97.43

Part B. Return Periods for two recent storms events

B1 Introduction

Two particularly energetic storms recently caused flooding in the vicinity of the proposed dwelling, so assigning return periods to these events provides a means of validating the design inundation levels. The storms are particularly interesting as they were caused by contrasting weather systems. The first on the 17 April, 1999, was driven by a depression which crossed the lower South Island, while the second on the 18 September, 2005, had a more northward origin and crossed the North Island near New Plymouth (see Fig 5). The characteristics of the two events will now be described in some detail; firstly based on observed damage and then based on storm parameter values derived from meteorological and oceanographic data. The return periods of these parameter values will then be derived using extreme value analysis and combined to provide event inundation return periods.

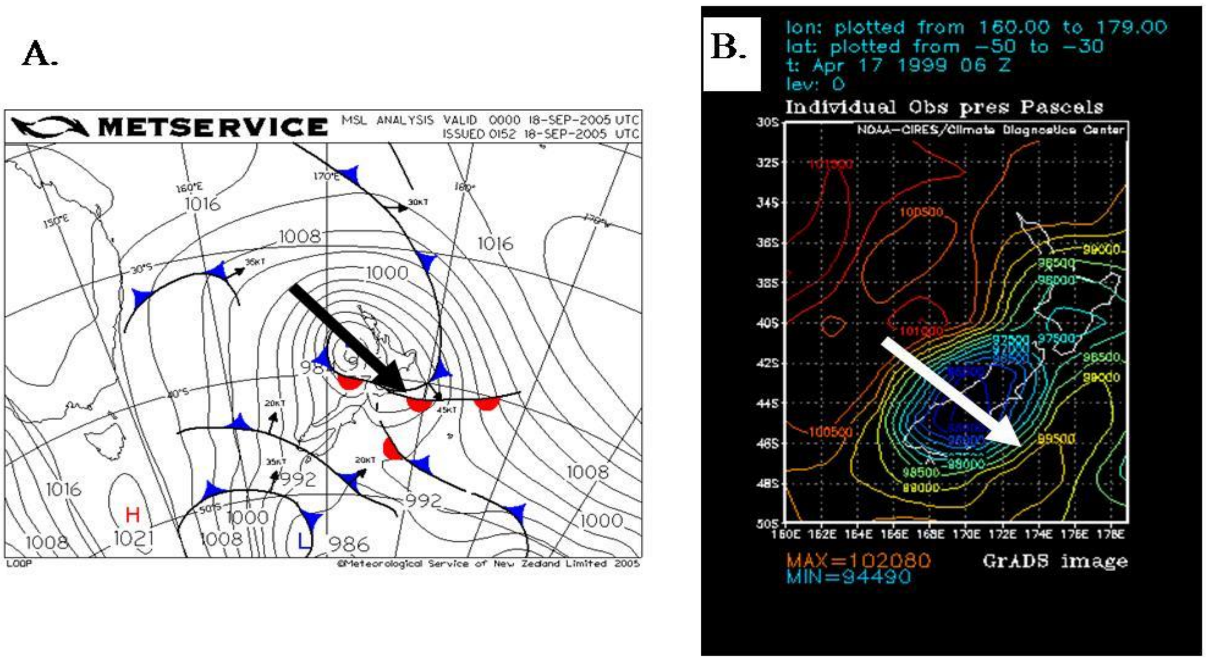


Figure 5 Pressure maps from the 18 September, 2005 storm (A), and the 17 April, 1999 storm (B). The bold arrows depict the approximate storm tracks.

B2 Observed flood Impacts

B2.1 Affidavit of Mr James (Tex) Edwin Lancelot Rickard

Mr Rickard moved to Raglan 1945 to work at the Post Office. He has lived and farmed about Te Whaanga and developed a particular awareness of storms and coastal processes through decades of metrological reporting and organizing coastal erosion conservation programmes. Mr Rickard’s observational credibility must be respected. His full affidavit attached as Appendix C. Of particular relevance are paragraphs 5 and 6 (reproduced below) which relate to flooding in general and to the September 2005 storm in particular.

> *5. I understand that Mr Hemi applied for a resource consent to build a home on his ancestral land but his application was declined by the Waikati District Council and is now subject to appeal. The council seem to think the sea will flood this land. I have never seen this area flood or heard of it flooding in all my time in Raglan.*
>
> *6. I used to spend a lot of time at Te Whaanga planting native trees, fencing, gardening, cleaning up around my wife's bach (which my youngest son has inherited) and generally relaxing with our extended family. From the bach window I can see the remains of last year's September storm. Logs and rocks are still lying on the edge of the land, beside the pohutukawa tree. The land didn't flood, but the sea overtopped the edges in places*

B2.2 Debris, overwash and rock fracture

The extent of the September 2005 event has been well recorded in photos and surveys. The site was visited by Mr Dahm on 19 September 2005 while the storm was still subsiding and a comprehensive set of photographs taken and reported (Eco Nomos, 2005). The site was also visited by the author on 12 October with follow up visits in June, 2006 and October, 2006. During these visits photographs and measurements were taken. Photos were also taken by Professor Richie.

Figure 6 contains 2 photos depicting the overwash extent along the boulder bank with wave-thrown boulders, logs and general debris having been marked.

Figure 7 contains several photos detailing the extent of flooding from the lagoon.

Figure 8 locates the maximum flood elevation (96.86 m) as determined by Skyworks Waikato based on the debris-front and vegetation damage. The flooding extended some 16 m landward of the lagoon embankment and reached some 25 m from the proposed dwelling site. As noted in the Introductory section, MSL at Whale Bay has been fixed at 93.5 m so the flood elevation reached 3.36 m above MSL.

Figure 9 shows the location and nature of rock fracture via the process of wave shock on the lava bluffs and promontories bounding the eastern side of the lagoon entrance. Such fracturing only occurred on the NE side of the lava flows, this being consistent with the NW waves which characterized this storm. They also occurred on top of the flows as would be expected during elevated water levels when the waves could reach these areas. The light coloured scars where lava blocks were removed were still clearly discernable over a year after the event. The author has not observed such fracturing before, indicating the rarity of such an event.

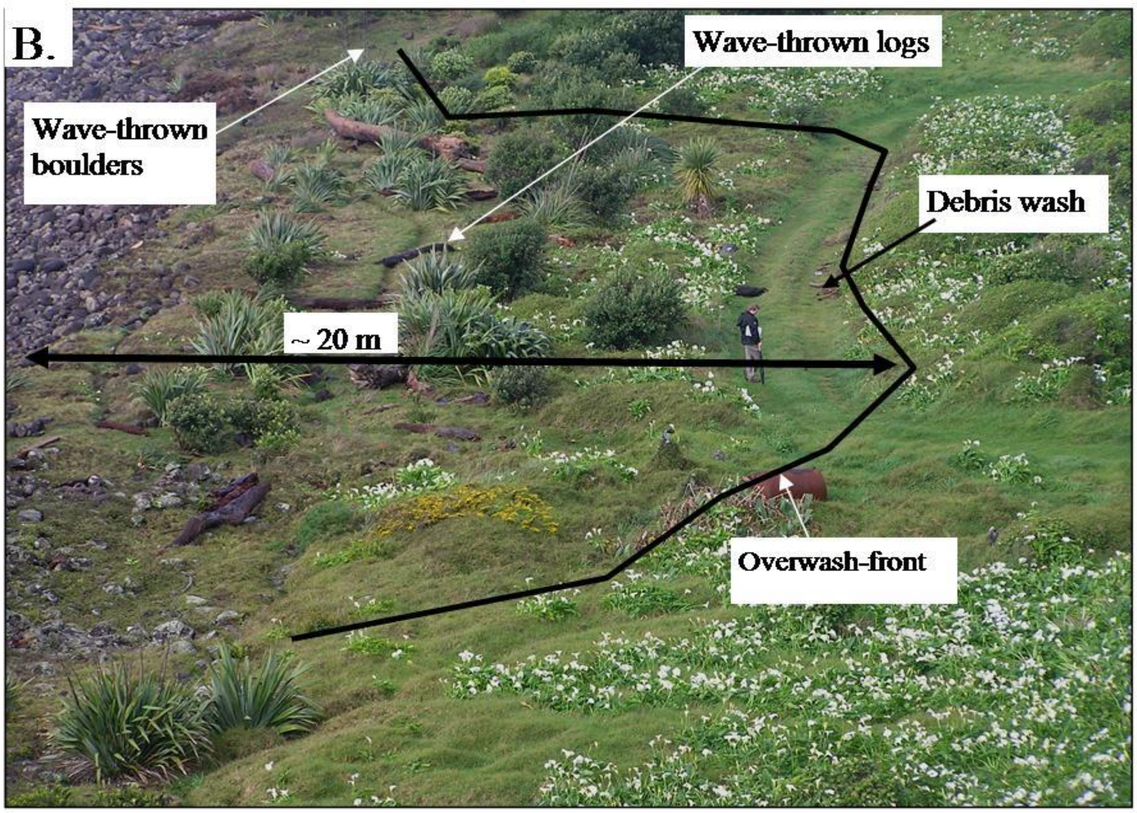


Figure 6 Extent of overtopping along boulder bank on 18 September, 2005. Photo A (Richie) with overwash extent as marked by Richie. Photo B (Dahm) with overwash extent marked by Shand. Wave thrown logs, boulders and debris wash marked in B

A.

B.

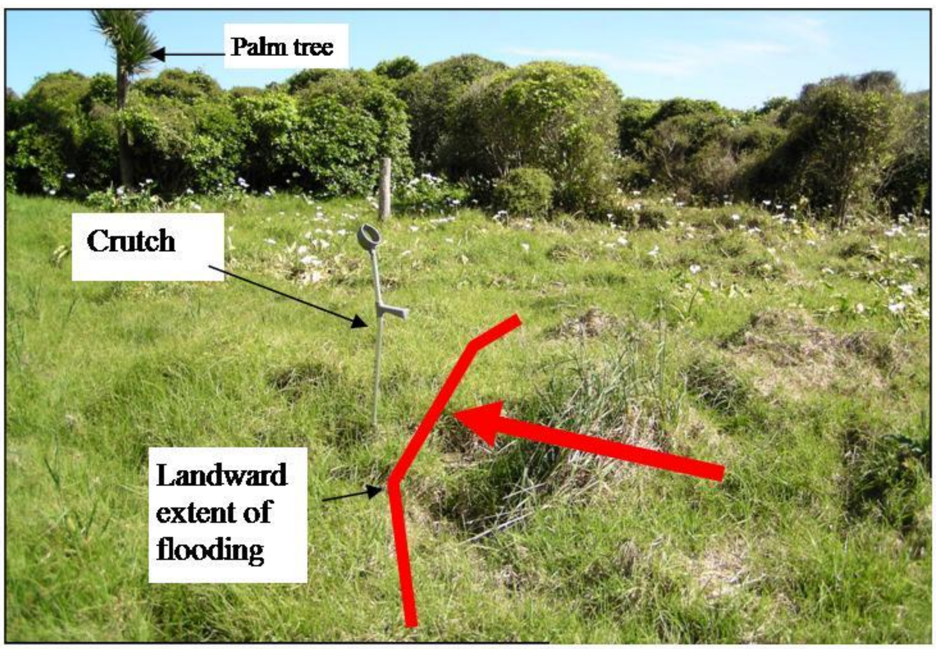


Figure 7A and B Photos depicting extent (red line) of flooding from the lagoon during the September 2005 event. Photo A (Dahm) showing the effect of flooding as viewed landward with the lagoon-track step in foreground; B (Shand) showing landward extent of flooding (lagoon to right of photo). The red arrows denote direction of flood wave and red lines denotes landward margin of flooding.

C.

D

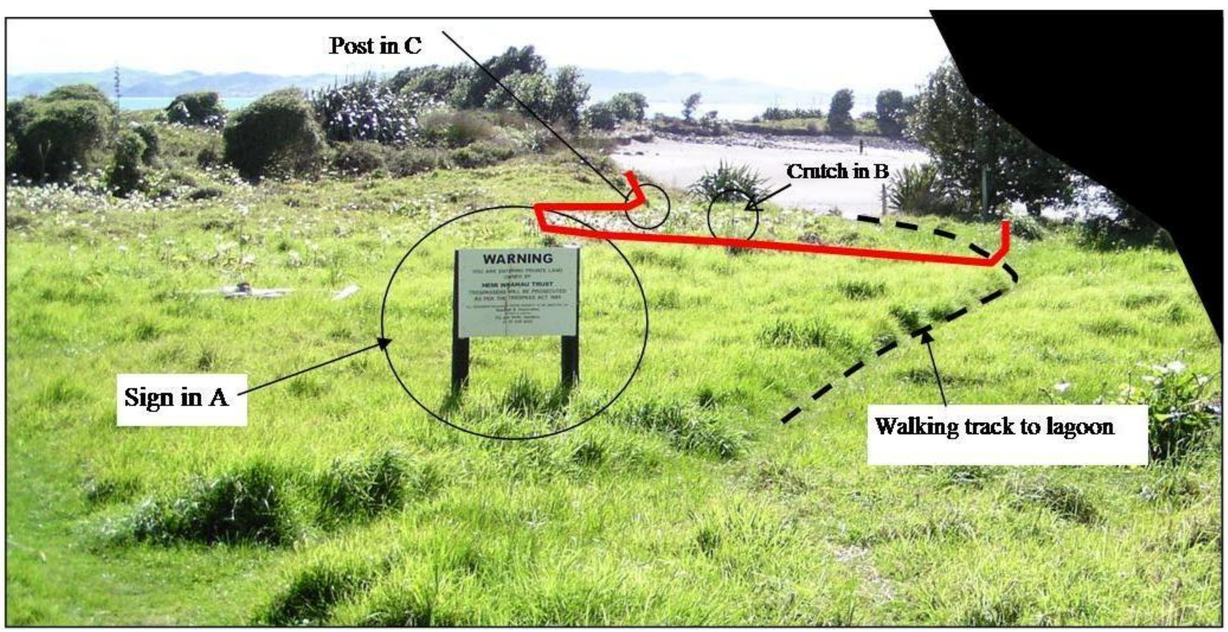


Figure 7C and D Additional photos depicting extent (red line) of flooding from the lagoon during the September 2005 event. Photo C (Richie) is a panorama of 2 photos showing flooding along overwash margins extending from the lagoon, and D (Shand) is an overview photo showing extent of flooding based on the boundaries defined in other photos.

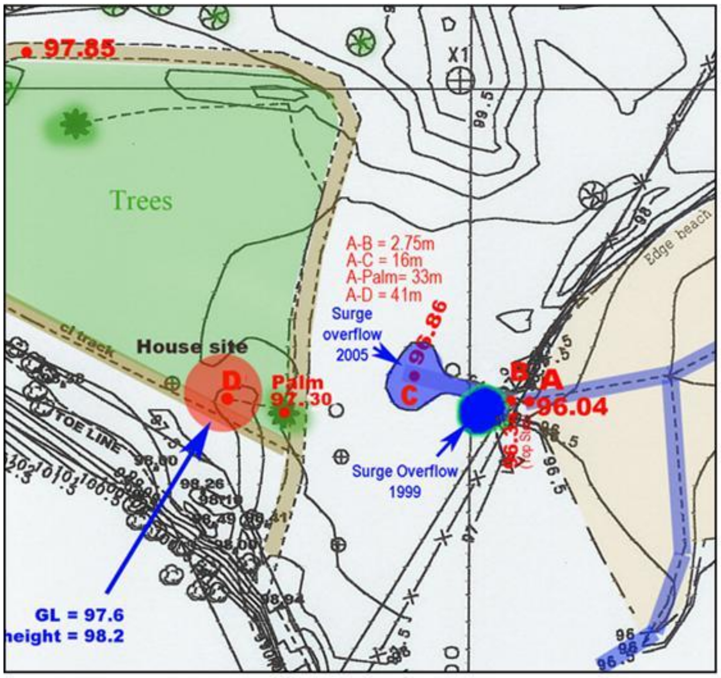


Figure 8 Contour plan of the upper lagoon and dwelling site with landward extent and elevations of the 1999 and 2005 flooding depicted (Skyworks Waikato).

By comparison, there is much less evidence as to the impact of the April 1999 event at the site. Some photos were presented at the October 2005 hearing by Professor Richie. One showed wave action at the lagoon step and this has allowed a tentative elevation to be reconstructed and marked on the contour map (Fig 8) at 96.5 m (3.0 m above MSL). This level, together with the 2005 level of 96.86 m (3/36 m above MSL) are included in Table 7 which summarizes the different flood levels at different locations.

Table 7 Flood levels (m)* for 1999 and 2005 storms at different locations

Event	Whale Bay	Raglan Harbour	Anawhata
1999	~3.00	2.85	2.16
2005	3.36	3.02	1.76

* Whale Bay and Raglan Harbour levels based on Moturiki datum.
Note local datum at Whale Bay = 93.5 m
Anawhata datum is mean level of the sea (MLOS) over the recording period.

Another of the Richie photos showed a wave-thrown log. This evidence, together with comment by other local residents that relatively little environmental damage accompanied that event, indicates the 1999 storm had less local impact than the 2005 event.

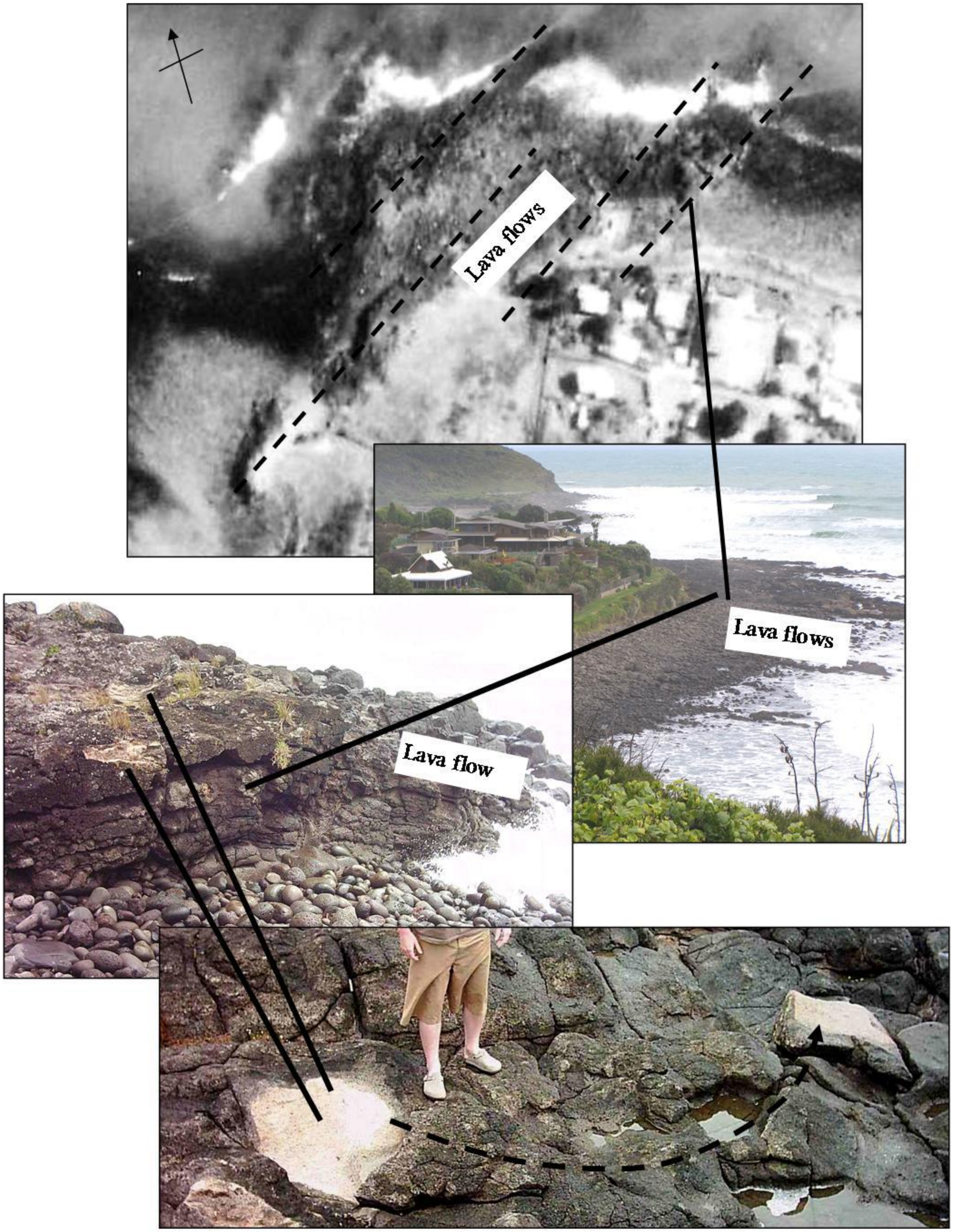


Figure 9. Block-fracture by wave-shock during the September 2005 event. Fracture occurred a) on the eastern face of the NNE-SSW orientated flows—consistent with wave approach from the NW, and b) on the upper surface of the flows consistent with elevated sea-level

B2.3 Water level comparisons

A reliable water level from Raglan Harbour (3.02 m above MSL) was obtained by leveling a well defined debris line from a photograph taken by Waikato District Council staff following the September, 2005 flood (see Fig 10). The site is in the vicinity of Aroaro Bay on the northern side of the township and was thus well sheltered from the direct effect of oceanic incident waves. This 2005 level compares with 2.85 m for the 1999 event in the same location as measured by Waikato District Council staff (see APPENDIX D). These flood levels are included in Table 7. The Raglan results qualitatively support the observation at Whale Bay that the 2005 flood level exceeded the 1999 level.

B2.4 Anawhata sea-level data

The NIWA sea-level recorder at Anawhata just north of Piha on the Auckland west coast also provided comparative data for the 1999 and 2005 events and this result has also been included in Table 7. The values are significantly lower than the corresponding Whale Bay and Raglan Harbour values and this is to be expected as wave effects have been filtered from the Anawhata output, there is no freshwater contamination, and a different elevation datum applies. However, what is particularly noteworthy is the relative elevation reversal with the 1999 value being substantially greater than the 2005 level. An explanation of this difference is particularly important because it would offer insight into why such significant flooding occurred in September, 2005 at Whale Bay/Raglan while such flooding was absent on much of the west coast.

The location of the Anawhata recorder and the Raglan sites are depicted in Fig 11A-D. The two areas have contrasting geographic orientation with Anawhata (A and B) being exposed to the southwest and Raglan (C and D) to the northwest. Anawhata could be expected to undergo wind and wave set-up during southwest conditions, while probably undergoing wind set-down under northwest conditions. These effects would be reversed at Raglan, and magnified due to the size of the Mt Karioi headland. The former process (wind set-up) is enhanced by topographic 'entrapment' of surface waters, while the latter (set-down) occurs when surface water being blown seaward faster than it can be replaced by upwelling. Such processes are well documented and Fig 12 is included as an example. It is noted that at Raglan, winds from WNW (292.5) appear to be at the southern limit of entrapment.

Different hydrodynamic conditions will therefore have differing process-responses at the two sites. As will be seen in the following section, contrasting conditions occurred during the 1999 and 2005 events with northwest domination during the latter resulted in enhanced elevations at Raglan, while southwest domination during the former resulted in relatively higher levels at Anawhata and less flooding at Raglan.

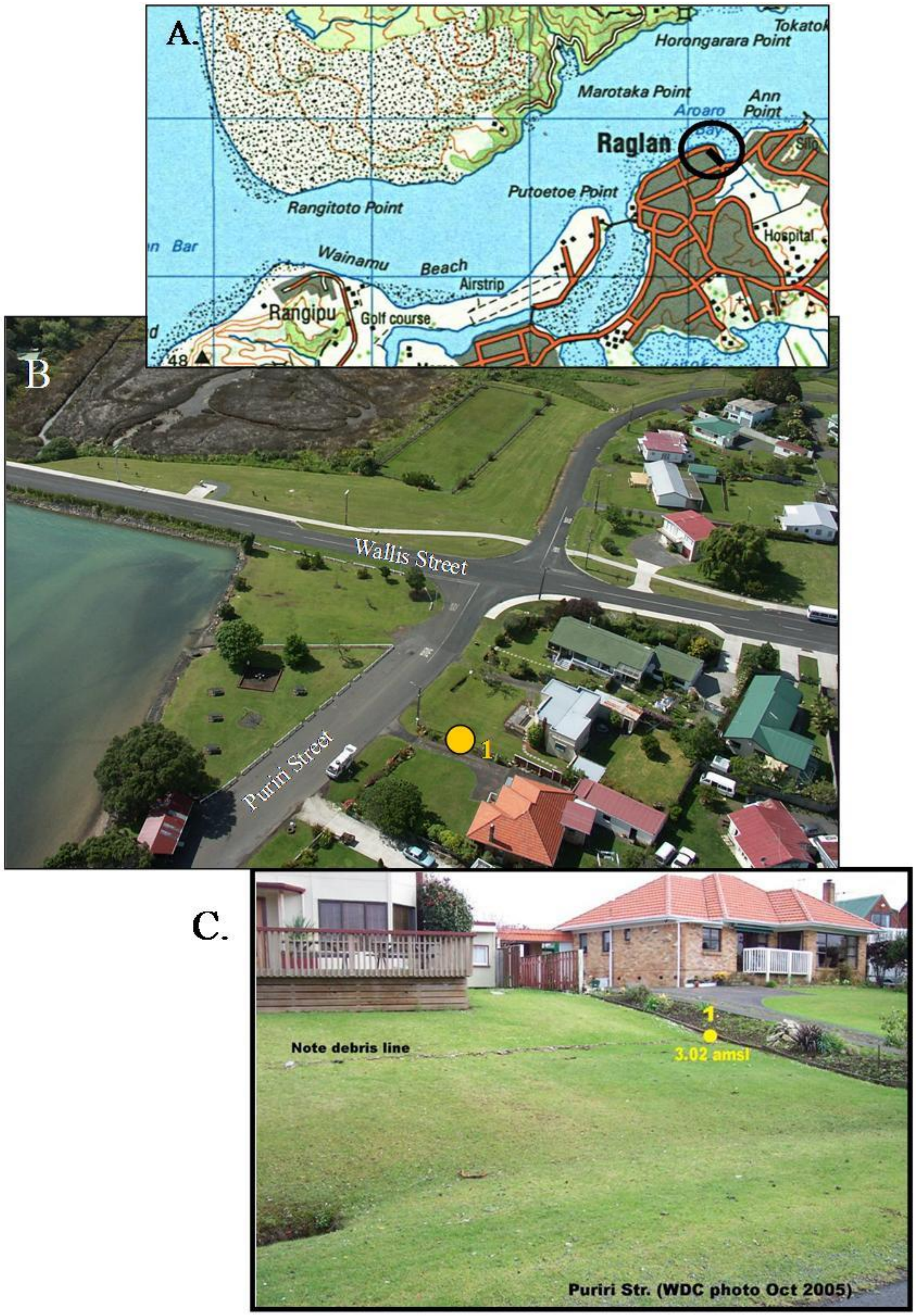


Figure 10 Location map (A) and aerial photo (B) showing the location of the debris line evident in C. Photo C was taken by Mr Steve Soanes of the Waikato District Coucnil following the 2005 flooding. The elevation of the debris line was surveyed by Skyworks Waikato on 7 June, 2006 at 3.02 m above MSL relative to Moturiki bench mark number AF96/C (CB 11)

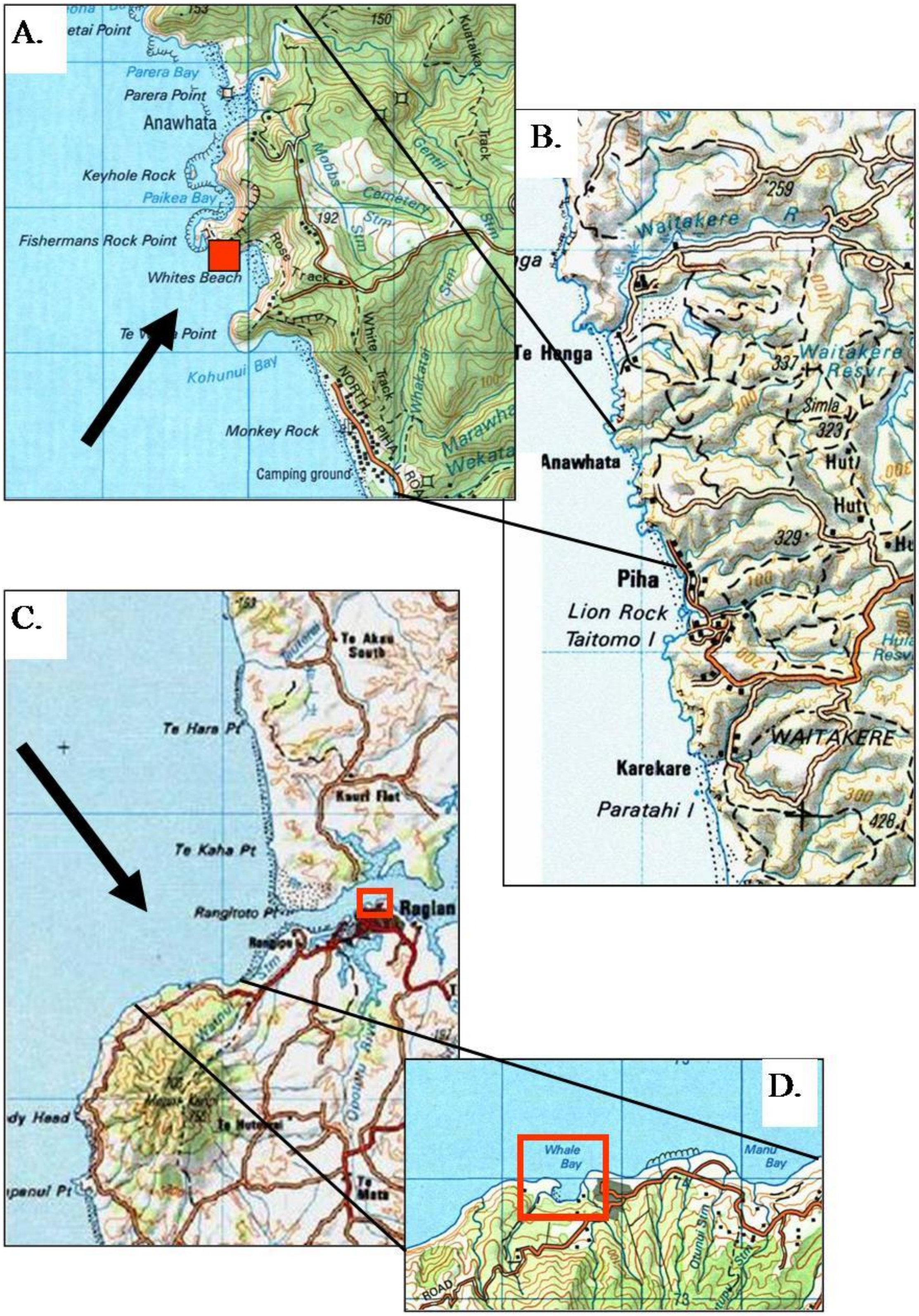


Figure 11 **Anawhata water level recorder site in A and B, and Raglan Harbour/ Whale Bay sites in C and D. Arrows depict direction of maximum wind and wave exposure and 'entrapment' effect.**

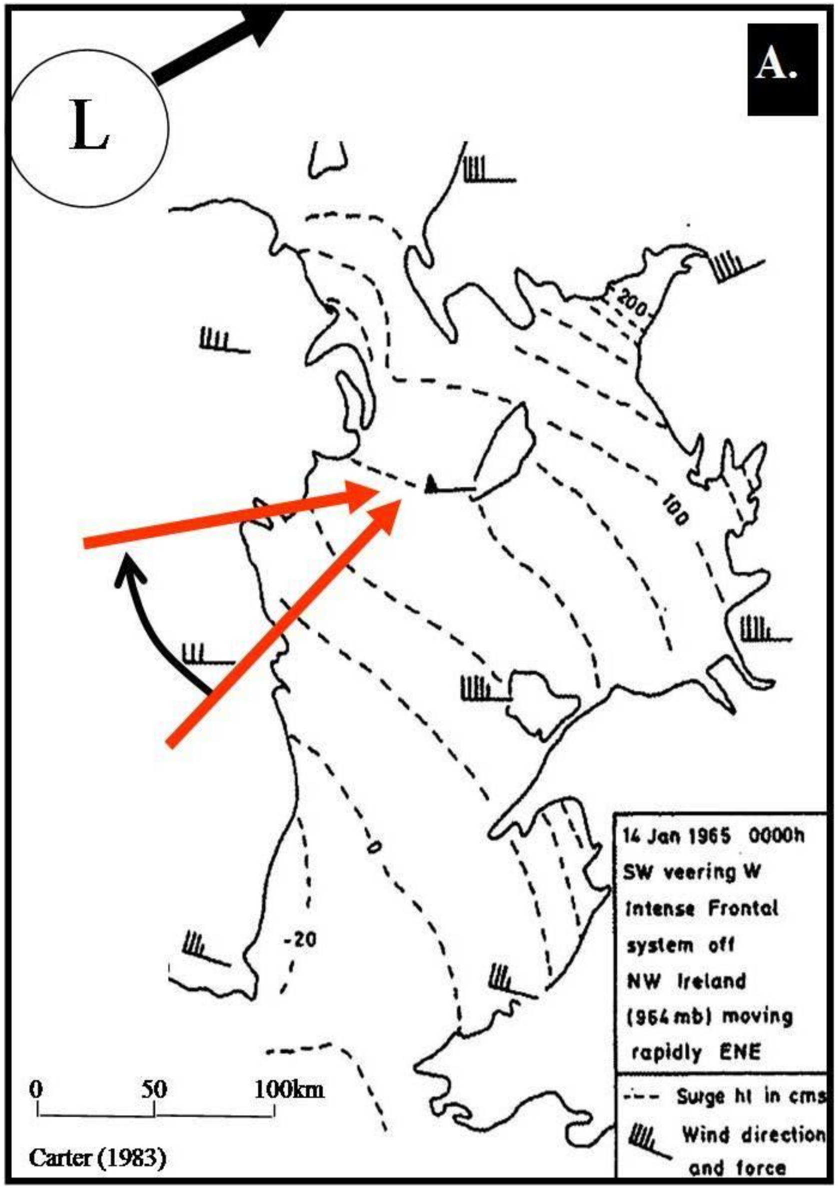
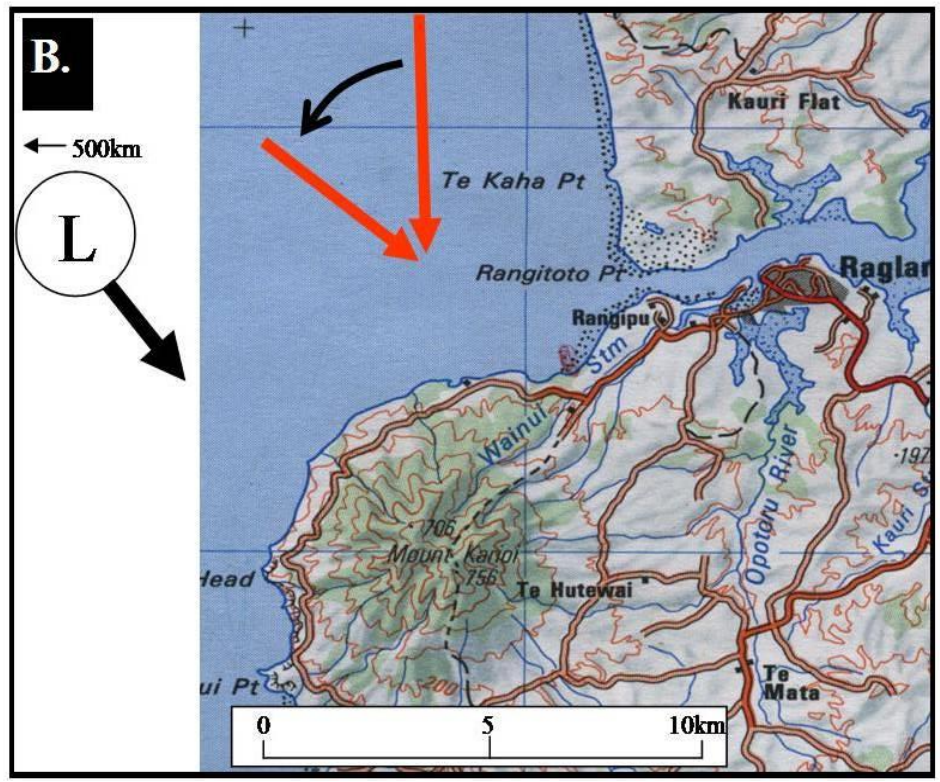


Figure 12 Effect of coastal orientation and wind-wave direction on water elevation.

Wind set-up and set-down in the Irish Sea (A) associated with a depression tracking ENE. In particular
1. Wind set-down (surface drag) in the lee of SE Ireland;
2. Wind set-up (surface drag) across the Irish Sea;
3. Wind set-up (entrapment) within southwest-facing embayments.

The September 2005 event in NZ (B) had a very similar setting in terms of depression track, energy conditions and coastal orientation , and the associated flooding was consistent with the wind set-up patterns evident in the Irish case.



B3 Storm inundation parameters

B3.1 April 1999 storm

The 1999 event occurred on the 17th April during the 10.18 hr (am) high tide; this was an exceptionally high spring tide exceeded only 6 times per year. The magnitude of the inundation components are summarized in Table 8A. Note that associated return periods are also listed and these values, together with their derivation, will be considered further in section B4. The wind direction record from Hamilton Airport (Fig 13) shows winds were essentially south of the entrapment threshold for 5 hours preceding the high tide and remained so thereafter. NOAA Global Forecast System (GFS) wind data for a site 3.5 km offshore (Fig 14 left column) show maximum speeds occurred just prior to the high tide. These data have been used in preference to the Hamilton Airport wind speed record because of significant topographic interference between the ocean and airport. In the lee of the Raglan headland the strong WSW wind may well have resulted in sea surface set-down. NOAA NWW3 wave data (Fig 14 left column) show waves during this time were rapidly increasing in size and period, and arriving from the WSW. Barometric pressures from Hamilton Airport and NOAA are shown in Fig 15A. The NOAA predicted minimum value is slightly lower and earlier than the observed value at Hamilton Airport. The airport data is used in Table 8 with a 1 hr offset to account for the temporal lag between the open coast and airport.

Table 8 Magnitudes and return periods of inundation forcing components for the storms of April, 1999 and September, 2005.

A. 17 April, 1999. 1018 hr high tide

Component	Magnitude	Return Period (yrs)
Tide (m)	1.81	0.16 (58 days = 6x /yr)
iBP (m) (990.5)	0.24	0.023 (8.5 days = 43x /yr)
Wind (m/s)	19.1 WSW (250^o)	0.49 (179 days = 2x /yr)
Waves (m)	<6.63[a] SW (245^o) @ 12.5s	<1.28 (467 days)

1. see text

B. 18 September, 2005. 2148 hr high tide

Component	Magnitude	Return Period (yrs)
Tide (m)	1.78	0.08 (29 days = 12.5x /yr)
iBP (m) (980.5)	0.34	0.58 (212 days = 1.7x /yr)
Wind (m/s)	17.6 WSW (250^o)	0.165 (60 days = 6x /yr)
Waves (m)	4.11 [4.5 to ~6.5][b] NW (305^o) @ 11.7s	0.162 [0.4 to 30]

a,b see text

Data sources described in text

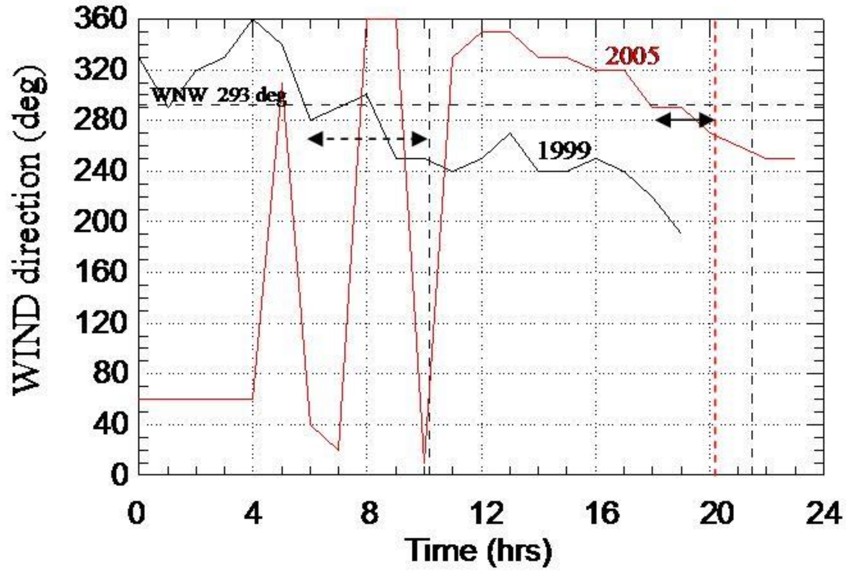


Figure 13 Wind direction from Hamilton Airport for 17 April, 1999 and 18 September, 2005. Arrows denote time period before the high tide for which wind was outside (south of) the entrapment threshold.

B3.2 September 2005 storm

The 2005 event occurred on the 18th September during the 21.48 hr spring high tide. Note that significant overtopping of the port breakwaters occurred at Port Taranaki on this tide and significant erosion happened at Muriwai Beach; both locations having northwesterly exposure similar to Raglan. This high tide level (Table 8B) was only slightly less than the 1999 level (1.78 c.f. 1.81 m). The wind direction record from Hamilton Airport (Fig 13) shows winds were north of the entrapment threshold for the whole day (from 60 deg around through 360 deg and on down to the threshold of 293 deg) leading up to the evening tide and only crossed the threshold an hour or so before the tidal maximum. It is therefore likely that wind set-up would still have been contributing to the storm surge during the high tide. NOAA Global Forecast System (GFS) wind data show maximum speeds were occurring during the high tide just prior to the high tide (Fig 14 right column). The NOAA NWW3 wave data (Fig 14 right column) show waves during this time were from the NW and building both in size and period. While deepwater height magnitudes were less than the 1999 levels (4.11 m c.f. 6.63 m), the more northerly approach (305 deg c.f. 245 deg) ensured the 2005 waves underwent less subsequent refractive energy loss. While minimum pressure (maximum inverted barometer) occurred four hours prior to the high tide and the pressure had risen some 7.5 hPa, the level was still ~10 hPa lower than during the 1999 high tide (see Fig 15). Of particular significance in Fig 15B is the temporal variation between the NOAA predicted data and the observed Hamilton Airport data (used in Table 8). This situation will now be discussed further.

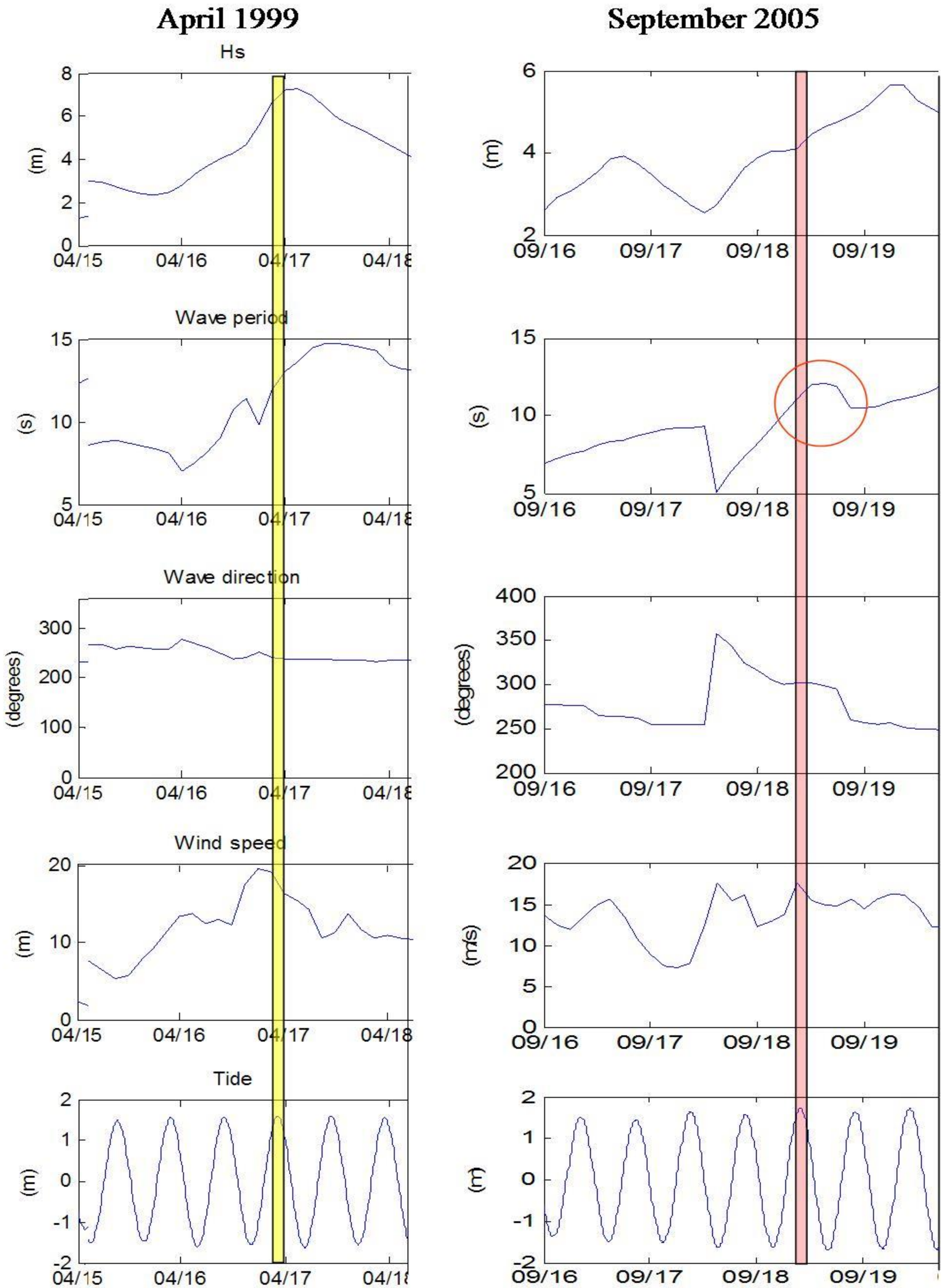


Figure 14 Wave parameters and wind speed for the period 15 to 18 April 1999 on left, and 16 to 20 September, 2005 on right. Vertical bars denote critical high tides when flooding occurred. Wave data are from NOAA NWW3, wind from NOAA GFS and tides from NIWA's EEZ tidal model. Red circle denotes possible superimposed wave trains (see text).

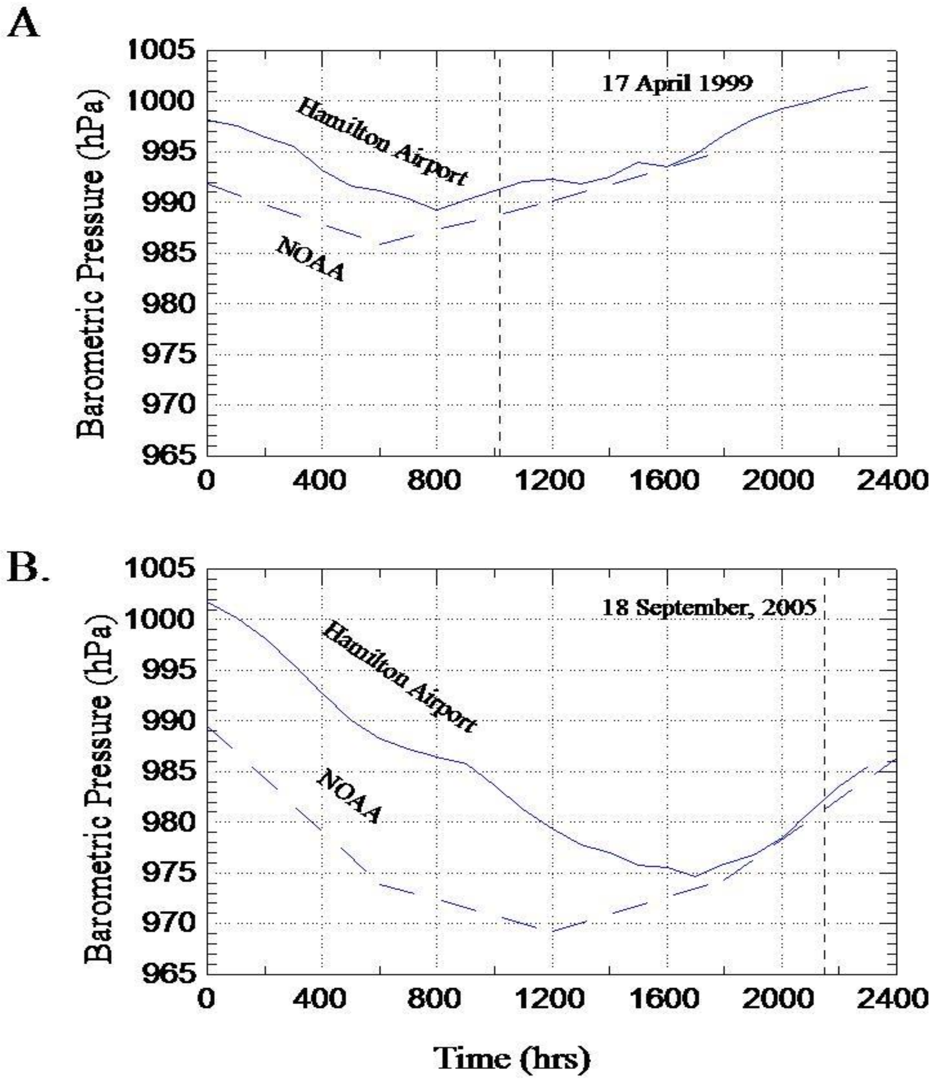


Figure 15 Barometric pressure from Hamilton Airport on 17 April 1999 (A) and 18 September 2005 (B) using NOAA predictions and Hamilton Airport measurements. Dashed vertical lines denote the time of high tide when flooding occurred.

B3.3 Limitations of NOAA data for describing individual storms

While NOAA produces forecast data which is subject to continual update, it is nonetheless based on wind-field prediction, and weather system behaviour in the final hours may differ from modelling expectation. Indeed, this appears to have happened during the 2005 event in particular, as indicated by comparing the observed (Hamilton Airport) and predicted (NOAA) barometric pressure graphs in Fig 15B. While observed pressures were ~ 5 hPa higher than predicted, thus reducing the potential storm surge, the depression itself appears to have also moved slower than expected. The difference between the 1999 predicted and observed data (Fig 15A) is much less and probably within variation associated with offset (airport to coast) and sampling resolution. The 4-5 hours difference between the 2005 data sets, however, could have significant implications for wind and wave effects. Firstly, the delay would have allowed the development of larger waves, and possibly also of stronger winds, than those predicted by NOAA, and secondly, the earlier wind and wave approach direction (NW) would have persisted longer than predicted. The values in Figure 14 right column and Table 8B may therefore underestimate the actual values. The effect such storm delay would have on the wave data will be assessed using the wave growth model of Goda (2003). However, before this is carried out, another weakness of the NOAA model as pertains to describing individual storms, c.f. describing wave climate, will be described, as this can also be assessed by the Goda wave growth model.

Prior to 2006, NOAA NWW3 wave data was ‘broadly directional’ in that once a new wave train entered a model grid point, it was combined with the existing train to give an ‘average’ height, period and direction. NOAA now produce a spectrum of directional output. While this earlier methodology is not expected to have much effect on wave climate output, it does have implications when wave trains are changing - as occurred during the 2005 event. In particular, the magnitude or direction of waves from the dominant train could be undervalued by the combination process. This situation not only has implications for the wave height, but it may change the period and directional bin – the latter resulting in the assignment of a different return period. In the 2005 situation the earlier and higher impact northwest waves could be contaminated by subsequent west to southwest waves and hence be assigned a lesser return period.

B3.4 Wave Growth Modelling

Input parameters for the Goda wave growth model consisted of wind speed, fetch (F) and duration. Parameter values were derived from 3 to 6 hourly situation maps obtained from NIWA. Overlaying the situation maps (6 hourly samples only) showed isobars directed toward the North Island from when the depression appeared off the Australian coast (Queensland-NSW border) on the 0000hr map of 17th September. However, the directional spread increased until the map of 1800 hrs (17th Sept) when isobars became more uniformly directed at the mid North Island west coast, and speeds rose to be in excess of 20 m/s thereafter. A value of 20 m/s was selected as the input value. Fetch on individual maps ranged between ~300 and 600 km and the ‘system-translation fetch’ essentially extended from the initial map. All output was found to be fetch unlimited. Storm durations from 9 hrs to 50 hrs were used. Resulting H_0sig and T_0sig output, together with the differences between sequential H_0sig values, are shown in Table 9.

Table 9 Wave generation output based on the model Goda (2003)

Wind duration (hrs)	9	12	15	18	21	24	30	40	50
H_0sig (m)	4.14	4.76	5.27	5.7	6.08	6.41	6.96	7.67	8.2
H_{diff}	-	0.62	0.51	0.43	0.37	0.33	0.55	0.71	0.53
T_0sig	7.4	8.1	8.66	9.13	9.54	9.89	10.48	11.25	11.83

Firstly, it is noted that the longest wave periods in Table 9 correspond to NOAA's 11.7s value appearing in Table 8B. However, this situation would require full wind contribution from the time the depression first appeared and would have generated waves heights of ~8 m. It seems more plausible that wind contribution occurred during the final 24 hours (mid -21 hr and -27 hr maps, see Fig 16) as indicated by the isobars (~wind direction) on the situation map of 1800 hr (-21 hrs) becoming more directed toward the mid North Island and remaining so thereafter. The longer period NOAA waves would then be an artifact of the spectral combination method described earlier. Indeed, the hump on the wave period time series (circled in Fig 14) suggests super-positioning of 2 wave trains. In this situation of 24 hrs of wave growth, the wave height of 6.41 m is still well in excess of NOAA's value of 4.11 m (Table 8B). Once again, this is demonstrably an artifact of the spectral combination method!

Secondly, if the storm delay of 3-6 hours indicated in Fig 15B is taken into account, then NOAA would have been modelling based on 18 to 21 hours instead of 24 hours. The delay would have resulted in an underestimate of wave growth by ~ 0.3 to 0.7 m (say 0.5 m). NOAA's value of 4.11 m should thus be increased to 4.41 to 4.81 m.

B3.5 Wave height conclusions

The results of the wave analysis show that waves accompanying the 2148 hr tide on 18th September, 2005 would have been in the range 4.5 to 6.5 m rather than the 4.11 m predicted by NOAA.

In addition, when the increase in wave height for the 2005 event is coupled with the greater refractive energy loss experienced by the 1999 event's 6.63 m deepwater waves from the southwest, the waves affecting shoreline run-up on the 18 September, 2005 would very likely have been larger.

B4 Return period analysis

Determining the return period for a flood-inducing storm often requires knowledge of the probability of occurrence of individual component variables. Component return periods are considered in section B4.1, while combining the component return periods is dealt with in section B4.2.

B4.1 Component return periods

Return periods were calculated using extreme value analysis (EVA) applied to the maximum value per year from the available data-set, or in some cases the several highest independent values per year were used.

As noted already, this study has used wind and deepwater wave data obtained from NOAA. In particular, all 9 years of available data (1997 to 2006) were obtained for a location 3.5 km offshore. Several different distribution shapes were used in the analysis and the best fit selected to derive the extreme values and associated return periods. While a longer sampling period would of course be preferable, several years are considered

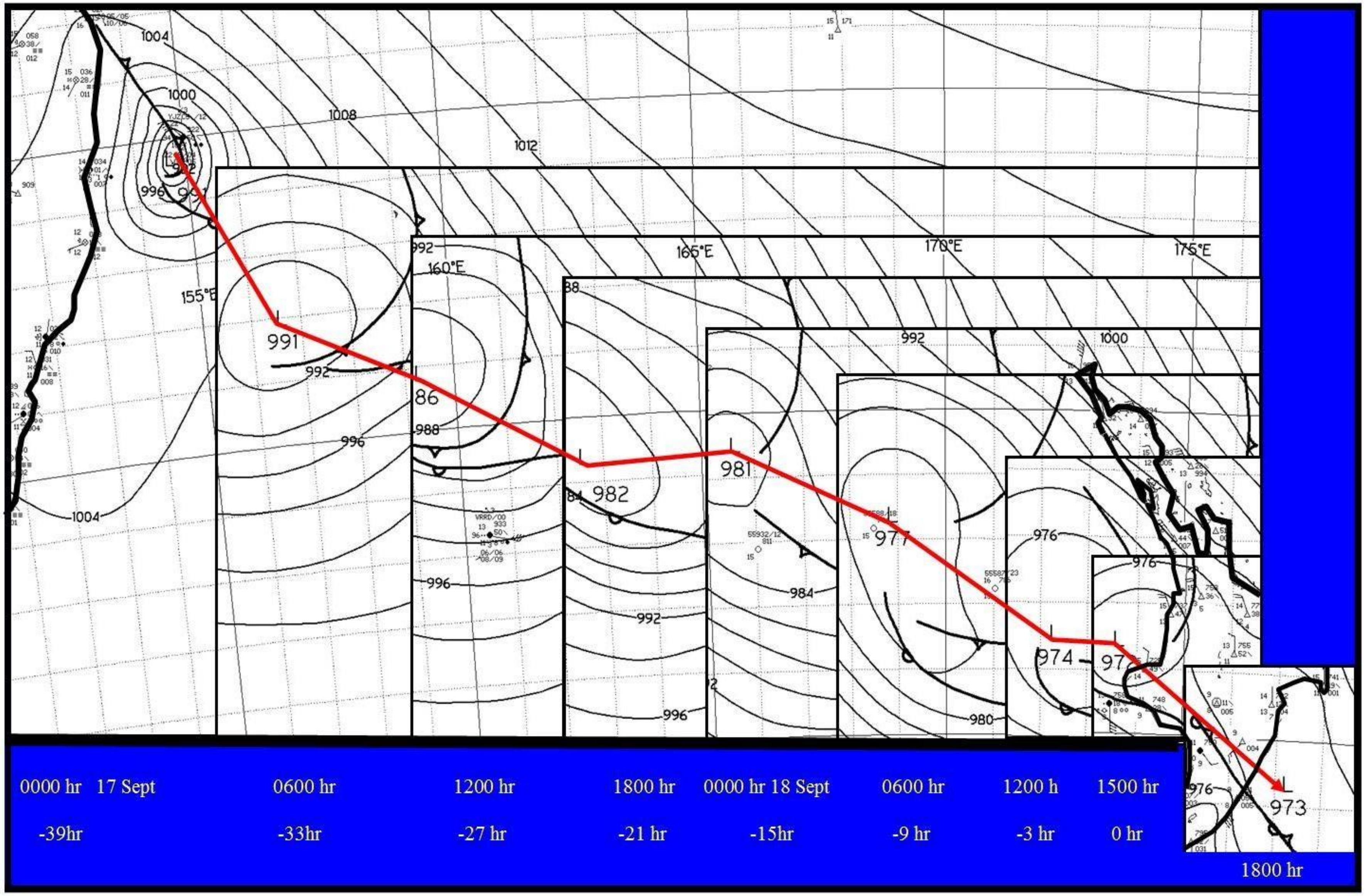


Figure 16 Situation maps at 6 hr intervals depicting storm passage from the Australian coast at 0000 hrs on 17th September, 2005, to landfall on the NZ west coast ~40 hrs later at ~ 1800 hrs on 18th September. Three hourly maps were obtained from the 18th (not shown). To see full isopleths, click on map of interest in electronic version.

acceptable (CIRIA, 1996). The return periods for wave heights from different directions were summarized earlier in Table 3A, and the wind speed return periods for different directions were summarized in Table 3B. Note that wind speed and direction is used to indicate relative levels of wind-setup or set-down.

The barometric pressure EVA was carried out by NIWA using the r-largest method (described earlier in section A3.2) applied to the longest available record for this region: 41 yrs from Auckland Airport (1965 to 2006). The output appears in Table 10.

Table 10 Auckland Airport barometric pressure return periods

Return Period (yrs)	Barometric Pressure (hPa)
0.08 (1 monthly)	987.1
0.18 (2 monthly)	984.5
0.25 (3 monthly)	983.1
0.50 (6 monthly)	980.8
0.75 (9 monthly)	979.7
1	978.8
2	977
5	974.7
10	973.6
20	972.7
50	971.7
100	968.9

Source: NIWA

The tidal EVA was based on 100 yrs of simulated tidal data and carried out by NIWA (NB section A4). The resulting values and associated return periods were listed earlier in Table 2.

Return periods corresponding to the tide, pressure, wind and wave values from the April 1999 and September, 2005 storm events are summarized in Table 8.

The sea-level EVA for the Anawhata record was carried out by NIWA (described earlier in section A3.1) and the results are listed in Table 11. While these values are not directly applicable to Whale Bay, for reasons already discussed, they nonetheless provide indicative information on the general sea-level-return period relationship, and also allow return periods to be assigned to the Anawhata results for the 1999 and 2005 storms as listed in Table 7.

As noted in section A3.1, Anawaha return periods were derived using data normalized by their averaged monthly MLOS, thereby removing much of the contamination caused by lower frequency sea-level variation. To use the values in Table 11 to assign a return period for an individual event, the sea-level for that event must first be adjusted by its particular monthly value. During April 1999, MLOS was 2.455 m. Maximum sea-level on 17 April was 4.522 m, so the adjusted value is 2.067 m which has a return period of ~60 yrs. This value compares with 2.156 m for the long-term MLOS adjusted value given in Table 8. By contrast, the September 2005 MLSO was 4.128 m and the monthly average was 2.360 m giving an adjusted value of 1.768 m which has a return period of ~0.4 yrs. This value compares with 1.762 m for the long-term MLOS adjusted value in Table 8.

Table 11 Anawhata sea-level return periods

Return Period (yr)	Sea-level (m)
0.08	1.61
0.25	1.73
0.5	1.78
0.75	1.81
1	1.83
2	1.87
5	1.93
10	1.98
20	2.02
50	2.06
100	2.09

Source: NIWA

B4.2 Computation methods for combined component return period

The joint probability (P) of two variables (X and Y) is given by the likelihood that:

$$0 \leq P(X \text{ and } Y) \leq 1 \qquad (5)$$

Two trivial cases of joint probability are complete *dependence* and complete *independence*. Two variables are dependent if one always occurs at the same time as the other and the return period for each variable would be equal. In this case it can be shown that:

$$P(X \text{ and } Y) = P(X) = P(Y) \qquad (6)$$

For example, storm surge components of barometric pressure and wind set-up are usually highly correlated as they are driven by the same weather system.

Alternatively, two variables are independent if there is no correlation of occurrence between them. In this case the joint probability is the product of the two marginal probabilities and the individual return periods will vary. It can be shown that:

$$P(X \text{ and } Y) = P(A) * P(Y) \qquad (7)$$

For example, tides and storm surge are independent as tides are driven by astronomical phenomena while storm surge is driven by local weather systems.

While dependent and independent cases are simple to calculate, inundation combinations contain some level of dependency that is best resolved by carrying out a probability analysis of several years of concurrently sampled data. Unfortunately, such data-sets are rarely available in NZ and certainly not for Whale Bay-Raglan. In such situations alternative approaches are required and these will now be described and applied.

(i) Where sea-level data is available but simultaneous wave data is not available

This situation applies to Anawhata[1]. The approach, detailed in CIRIA (1996), provides combinations of sea-level and wave conditions (each specified in terms of their marginal return period) which give a joint probability of 100 yrs. In addition, the level of dependency between the two variables, i.e. the degree of correlation, is provided by practitioner judgment. The correlation is in terms of an assigned *correlation factor*. A value of 2 indicates approximate independency. ("without detailed analysis it is considered rather risky to assert complete independency and assign a value of 1"). A value of 20 represents a modest level of dependency ("…appropriate if some correlation is expected even if there is no particular evidence for it"). Finally a value of 100 represents well correlated conditions ("…such as where a strong wind moves along a narrowing sea area thereby producing both high surge and waves"). For the Whale Bay-Raglan setting, it was agreed (Dahm-Shand December 2006 meeting) that a value of 20 would be appropriate. The relevant marginal return periods for a correlation factor of 20 are given in Table 12. Note that if these variables were completely independent then the combined return periods would be ~1000 yrs rather than 100 yrs.

Table 12 Combinations of water level and wave condition marginal return periods with a 100 yr joint probability and a correlation factor of 20

Water-level return period (yrs)	Wave condition return period (yrs)
0.02	100
0.05	57
0.1	28
0.2	14
0.5	6
1	2.8
2	1.4
5	0.6
10	0.28
20	0.14
50	0.06

Source: CIRIA (1996)

1. In actual fact, available NOAA wave data could have been matched with the sea-level record but NIWA were not able to either carry out the analysis themselves or release the Anawhata sea-level record so we could carry out the analysis. However, the CIRIA approach was adequate.

This method is applied to the April 1999 and September 2005 Anawhata sea-level and wave values and the results summarized in Table 13.

Anawhata: April 1999:

Sea-level return period ~60 yrs.
Required wave height return period for 100 yr combination is <0.06 ys (Table 12).
But this is << than the observed 1.28 yrs (Table 8A)*
Therefore the storm's inundation return period was **>> 100 yrs**.

Anawhata: September, 2005:

Sea-level return period ~0.4 yr.
Required wave height return period for 100 yr combination is >6 yrs (Table 12)
But this is (i) >> than the observed 0.162 yrs for the NOAA value in Table 8B*,
(ii) mid range for the computed return period of 0.4 to 35 yrs.
Therefore the storm's inundation return period was **<<100 yrs** based on the NOAA wave data, and probably <**100 yrs** based on the wave generation model results (Table 9).

* Assumes NOAA data for the deepwater Raglan grid point approximates Anawhata.

(ii) Where sea-level data is not available but the other component data is available

This situation applies to Whale Bay and is described as follows. As sea-level is based on tide level, inverted barometric pressure and wind set-up, it is possible to estimate the inundation return period by using the component return periods. This approach utilizes the independence between tide and either one of the remaining two sea-level components, and then making a qualitatively adjustment for the remaining (dependent) component. Sea-level return period is thus given in terms of an inequality. This value is then combined with waves using the CIRIA (1996) method described above. This method is now applied to the April 1999 and September 2005 sea-level and wave values and the results summarized in Table 13.

Whale Bay: April 1999:

Tide = 0.16 yrs.
Barometric pressure = 0.023 yr
Wind: in section B2.1 it was argued that the WSW wind could have a set-down (negative) influence on sea level.
Combining tide with barometric pressure (independent components) gives 1.3yr, or <1.3 yrs with wind
Required wave height return period for 100 yr combination is >2.3 yrs (Table 12).
But this is > than the observed <1.28 yrs in Table 8A.
Therefore the storm's inundation return period was **< 100 yrs**.

Whale Bay: September, 2005:

Tide = 0.08 yrs.
Barometric Pressure = 0.58 yrs
Wind: in section B2.2 it was argued that the NW wind set-up was probably still operative, so a + inequality will be assigned.
Combining tide with barometric pressure (independent terms) gives 17 yrs, or >17 yrs with wind.
Required wave height return period for 100 yr combination is <0.18 yr (Table 12).
But this (i) approximates the observed 0.162 for the NOAA value, or is (ii)< than the computed return period range (0.4 to 35 yrs) in Table 8B.
Therefore the storm's inundation potential was **~100 yrs** based on the NOAA wave data but **>100 yrs** based on the wave generation model results (Table 9)

Table 13 Summary of inundation return periods using different methods

Apr-99		
Method (section B4.2)	Whale Bay Return Period	Anawhata Return Period
(i)	N/A	>> 100 yr
(ii)	< 100 yr[a]	N/A

Sep-05		
Method (section B4.2)	Whale Bay Return Period	Anawhata Return Period
(i)	N/A	<< 100 [~100 yrs][b]
(ii)	~100 [>> 100 yrs][b]	N/A
a,b Table 8		

The return period results in Table 13 show that the 1999 storm inundation was well in excess of 100 yrs at Anawhata, but below 100 yrs at Whale Bay. By contrast, the 2005 event had a return period near or below 100 yrs at Anawhata, but approximated or exceeded 100 yrs at Whale Bay.

The Whale Bay results for 1999 of 96.5 m (3.0 m above MSL in Table 7) at <100 yrs, and for 2005 of 96.86 m (3.36 m above MSL in Table 7) at ≥100 yrs, are consistent with the predicted inundation hazard levels (excluding 0.45 for SLR), of 96.98 to 97.12 m from Table 6 (i.e. 3.48 to 3.62 m above MSL) at >>100 yrs.

This analysis shows that the storm return period range of 10 to 30 yrs proposed at the 2005 hearings for the two storms was too low

Discussion and Conclusion

Contour levels of key features along a transect between the building site and the lagoon (base of step), are summarized in Table 14A. The critical assessment level used at the 2005 hearing was the basement floor level, some 97.7 m above local datum (4.2 m above MSL). Note that the basement level made allowance for a 200 mm concrete pad. As noted earlier, a 0.7 m pad is now proposed and this option has also been included in Table 14A.

The critical flood hazard assessment details have been highlighted in Table 14B; these are based on at least a 100 yr return period flood level, together with a time span of 100 yrs for sea-level rise. Details of flood levels from the 1999 and 2005 storms also appear in Table 14B.

Table 14 Summary of elevations related to features and inundation levels

A. Distances and elevations of key features.

	Distance (m) Datum = step	Elevation (m) Local datum (93.5 m)	Elevation (m) Datum = MSL
Base of step	0	96.04	2.54
Top of step	0	96.34	2.84
Palm tree	33	97.3	3.8
Site: natural ground level	41	97.5	4
Site: initial basement level	41	97.7	4.2
Site: present basement level	41	98.2	4.7

B. Inundation characteritics

Predicted (1% AEP) inundation	37 to 41	97.43 to 97.57	3.93 to 4.07
1999 event inundation	~5	~96.5	~3.0
2005 event inundation	16	96.86	3.36

Note that some of the ground-based levels vary slightly from those presented in earlier evidence, e.g. the September 2005 flood level is 3.36 m above MSL c.f. 3.4 m as used in Appendix A. This is due to subsequent more accurate surveys.

Comparing the critical highlighted values in Table 14, shows that the site is above the predicted inundation level (4.2 m above MSL) by 0.13 to 0.27 m. While this may appear to be a relatively small margin of safety, it is significant that these inundation hazard assessment levels have return periods well in excess of 100 yrs. Further confidence is provided by the site being 0.84 m above the flood level associated with the 2005 storm, an event found to have a return period of at least 100 yrs.

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APPENDIX A Notes presentation at the Hemi-WDC meeting, 8 June, 2006, by Dr Roger Shand

1.0 Establishing Mean Sea Level at the Whale Bay site

Mean sea level used for evidence at the December 2005 hearing was taken as 94.4m relative to the arbitrary datum at the site.

Potential sources of error
The 94.4m value was provided to consultants during the lead-up to the December hearing. Subsequent investigation found that its derivation was based on averaging an observed high tide and low tide elevation. Measurements were made on the seaward side of boulder bank, and therefore contained significant contamination by wave run-up and set-up. Other environmental influences such as the barometric pressure (BP) effect, would also have contaminated the result.

Exploratory sea-level surveys
Two surveys were carried out under low energy conditions within Whale Bay, on 22 February 2006 and 1 March 2006 by Hamilton-based surveyors Skyworks Waikato. Note that Skyworks produced detailed reports on all surveys and these are available upon request.

No adjustments were made for BP and seiching (waves with periods ranging from a few minutes to about an hour). A correlation with Moturiki datum (true MSL) was attempted on 22 February by comparing the high tide level reached in Whale Bay with the approximate high tide level in Raglan Harbour – that was subsequently related to Moturiki Datum.

The results presented in Table 1 indicate the previously used MSL of 94.4 was too high.

Table 1 Results of surveys 1 and 2

Survey 1: 22 Feb	MSL = 93.50 m
	MSL – high water = 0.72 m at site c.f 0.76 m in town
Survey 2: 1 March	MSL = 93.53 m

Two more detailed surveys were subsequently carried out.
Instructions from Coastal Systems for carrying out these surveys are attached. These surveys were adjusted for BP, and any mid-term sea-level fluctuation (e.g. temperature, IPO, ENSO can cause variations of ± 0.2 m) was accounted for by adjusting to the Moturiki-based MSL using contemporaneous measurements in Raglan Harbour. Rainfall data for several days prior to each survey was obtained from NIWA as freshwater inflow could cause water level in harbour to be higher than at Whale Bay. Finally, mathematical

filtering was used to remove any seiche-effect or measurement error. While seiche amplitudes of up to 4 cm were detected, averaging of the high and low tide levels to determine MSL reduced this effect to 1 cm.

Table 2 Results from surveys 3 and 4

Survey 3: 23 March	MSL(excl filtering) = 93.51	MSL (incl filtering) = 93.50
Survey 4: 21 April	MSL (excl filtering) = 93.47	MSL (incl filtering) = 93.46

The 0.04 m (4 cm) difference in MSL values could, in part, relate to 12 mm of rain which occurred 3 days prior to survey 4, while no rainfall occurred for several days prior to survey 3. A value of 93.5 was adopted as MSL for the site.

Comparisons between this new MSL datum and the previous MSL datum for tide parameters are shown in Table 3A (p3), for predicted hazard levels and observed flood levels in Table 3B, and for natural features in Table 3C.

2.0 Double dipping on sea-level rise

The ground contours and building design in Drawing 4B used a further variation in MSL datum: 94.8 m. This extra 0.4 m was added by the design team in keeping with early Waikato District Council advice that 0.3 to 0.5 m would need to be taken into account for predicted sea-level rise (SLR) from global warming. This was before a separate hazard assessment was contemplated. When the hazard assessment was subsequently carried out, SLR was accounted for again. This situation of inadvertently incorporating a hazard component twice is referred to as ‘double dipping’.

A value of 0.4 m (in addition to the 0.8 m MSL adjustment) must therefore be added to building design levels, which were, in total, 1.2 m too low relative to actual MSL. These adjustments are shown in Table 3D.

Table 3 Comparison of critical levels using different MSL datum

Feature	Dec 2005				June 2006
	Relative to arbitrary site datum				
MSL	**94.40 m**				**93.60 m**
A. TIDE					
Mean low water spring (MSL-1.45m)	92.95				92.15
Mean low water (MSL-1.15m)	93.25				92.45
Mean high water (MSL+1.15m)	95.55				94.75
Mean high water spring (MSL+1.45)	95.85				95.05
B. FLOOD	**Relative to MSL**				
Hazard design flood level*	3.35 m	no change			3.35 m
Hazards design flood level including SLR#	**3.55 to 3.8**	no change			3.55 to 3.8
Highest observed flood level (Sept 05)	2.60	+ 0.8 m		=	3.40
Highest observed flood level + SLR#	2.80 to 3.05	+ 0.8 m		=	3.60 to 3.85
C. NATURAL FEATURES					
Crest of boulder bank at site (98.2)	3.80 m	+ 0.8 m		=	4.60 m
Natural ground level	3.20	+ 0.8 m		=	4.00
D. DESIGN					
Basement (Dec 05 design)	**3.00**	+0.8 m	+0.4 m	=	4.20
Basement (May 06 design◙)	NA				4.70
Ground floor level	5.60	+0.8 m	+0.4 m	=	6.80
Road access	NA				4.50

* 50 to 100 yr return period (relevant for storm events),

\# 50 yr = 0.2 m, 100 yr = 0.45 m

◙ plus 0.5 m

Bold values denote key levels in December 2005 evidence

3.0 Implications for inundation hazard

In my evidence presented at the December, 2005 hearing, I concluded that *extreme flood levels of 3.55 to 3.8 m above MSL could be experienced during one or more extreme events during the next 50 to 100 yrs, and as the proposed basement level was 3.0 m above*

MSL, this would result in basement flooding. The predicted depth of basement flooding was therefore between 0.55 to 0.8 m, and this was 'rounded out' to 0.5 to 1.0 m.
The critical hazard values of 3.55 to 3.8 m and the basement level of 3.0 m have been highlighted in Table 3D.

Adjusting for the new MSL datum (+0.8 m) and double dipping (+0.4m), results in the basement now being 4.2 m above MSL (Table 3D, row 1). This is at least 0.4 m above the hazard prediction values of 3.55 to 3.8 m (Table 3B, row 2).

(Attachment)

Water Level Survey Instructions from Coastal Systems
for determining MSL at the Hemi house site at Whale Bay, Raglan

A) Survey day

A day must be chosen with minimal weather and sea conditions. This eliminates numerous environmental influences on sea level.

B) Environmental information

During the surveys environmental information is needed to show there were benign conditions at this time – thus minimal influence on sea-level. In addition, the BP reading will let us remove its sea-level effect.

i) Information available from web:

*Use Raglan Weather http://www.raglanweather.co.nz/ for local BP and wind.

*Official high and low tides time are available from http://www.niwascience.co.nz/services/tides

*Met office situation map will confirm the Raglan Weather's barometric pressure http://www.metservice.co.nz/default/index.php?alias=mapsandobservations

* Wave measurements…. The following address will give you a 7 day prediction. These prediction tables will help your planning BUT the values will alter a bit as each day approaches so keep checking. IDEALLY, THE FIRST COLUMN, i.e. "Surf (ft)" SHOULD READ "0-1". Run the programme EACH DAY and SAVE the output table. http://www.buoyweather.com/wxnav1.jsp?region=NZ&program=nww3surfTable&grb=nww3&latitude=-38.0&longitude=173.75&zone=12&units=e

ii) Site observations:

Describe wind (direction and speed) and sea conditions (wave height) and also take photos of the sea/waves from a good vantage point every couple of hours through the

survey period. These site measurements provide a check on the official measurements which may be for distant locations or are predictions.
Note that wind speed can be estimated using the following (BEAUFORT) scale:

Calm	< 1 km/h	cannot feel any air movement on face
Light	1-5 km/h	smoke drifts, wind not felt on face
Slight	6-11 km/h	leaves move, wind felt on face
Gentle	12-19 km/hr	leaves in constant motion, wind extends flag
Moderate	20-28 km/hr	small branches move, wind raises dust.
Fresh	29-38 km/hr	small trees sway.

C) Water level measurements

i) Tide range:
It will be better to survey during neap tides rather than spring tides because the lagoon wont dry out and you will be able to set up a permanent staff (ie a single staff location for whole survey period) within this sheltered area. The same applies to the harbour site.

ii) Measurement details
You need to take readings thoughout both the high tide and low tide, at the house site and the Raglan harbour site. This is to enable MSL for each to be calculated (as you have been doing for the house site). It also lets me filter out any seiching that may be present. Seiching is a low frequency wave motion with periods upward of a few minutes.
To filter it I will need:
*measuring to be done for a couple of hours each side of the high and low tide.
*a new measurement to be done every 15 minutes.
*each measurement to involve visually averaging out wavelet effects…. Say 20 seconds, or until you decide upon a stable value.

iii) Contemporaneous measurements
An observer is required at each site at the same time. They could use binoculars to inspect the staff and the staff should be related to the local benchmark (datum) at a later/earlier time.

D) Data Processing
Adjust the water level readings for barometric pressure, reduce the results to local datum and then send these data to me for filtering.

Roger Shand
22.3.06

APPENDIX B Report on extreme run-up and overtopping of the boulder bank by Tonkin and Taylor Ltd

APPENDIX C Affitdavit of Mr James Rickard

BEFORE THE ENVIRONMENT COURT

ENVA2006/

IN THE MATTER	of **THE RESOURCE MANAGEMENT ACT 1991**
BETWEEN	**J HEMI**
	Appellant
AND	**WAIKATO DISTRICT COUNCIL**
	Respondent

AFFIDAVIT OF JAMES EDWIN LANCELOT RICKARD

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-Hemi

NRC

AFFIDAVIT OF JAMES EDWIN LANCELOT RICKARD

I, JAMES EDWIN LANCELOT RICKARD of Te Kopua Road, Raglan SWEAR:

1. That my name is James Rickard, but I am known in Raglan as Tex Rickard. I am of Ngati Porou descent and came to Raglan in 1945 to work in the Post Office where I stayed for 20 years before moving to manage Trust Bank Waikato (now Westpac) and later Hartstones Ltd which I managed until I retired.

2. I farmed 50 acres at Te Kopua from 1952 until the mid-70s. In the 1960's I was responsible for reporting Raglan's daily weather to the Meteorological office in Auckland. I became competent at accurately predicting weather patterns based on observation e.g. the position and types of clouds above the mountain etc.

3. From 1983-1996 as manager of Whaingaroa Kite Whenua Trust I was responsible for managing several training schemes which were aimed at educating youth and preparing them for employment. Many of our programmes had an environmental focus and several of our students carried out a number of surveys such as monitoring the sewage ponds, outfall and rubbish dump, reclaiming coastal land, vegetating sand dunes, planting native trees and grasses and monitoring the effects of their efforts. Our Conservation Corps received an award from the Waikato Regional Council for their environmental work within the Whaingaroa Harbour
area.

4. Over the last 40 years I have lived through many storms in Raglan and have gained experience in observing and working with environmental change. I am familiar with the whanau, land, and rocky shore area between Te Whaanga and Te Rekereke as up until the 70's I frequently gathered sea food from this area when the moons were right and the tides were calm. I was lucky enough to go fishing, with my wife's cousin Kawharu Kereopa who taught me about the tikanga in the Whaingaroa area that had to be observed when gathering food from the sea.

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5. Sam Kereopa, my late wife's elder brother, was given land at Te Whaanga by his mother when he returned from the Second World War He farmed the place and grazed his horses and cows down by the lagoon right up until Roro Puke built his house there in the late 60's. Sam then moved his stock to the top paddocks to stop surfers and other people chasing them. Occasionally the cows would escape and end up in the bush or back down the bottom. Since then the land has changed hands. Jojo Hemi, who is of Ngati Koata descent purchased the land from the Te Puke family a few years ago.

6. I understand that Mr Hemi applied for resource consent to build a home on his ancestral land but his application was declined by Waikato District Council and is now subject to appeal. The Council seem to think the sea will flood his land. I have never seen this area flood or heard of it flooding during all of my years in Raglan.

7. I used to spend a lot of time at Te Whaanga planting native trees, fencing, gardening, cleaning up around my wife's bach (which my youngest son has inherited) and generally relaxing with our extended family. From the bach window I can see the remains of last year's September storm. Logs and rocks are still lying on the edge of the land, beside the pohutukawa tree. The land didn't flood, but the sea overtopped the edges in places.

8. For over thirty years I have studied erosion on almost a daily basis. My focus has been mainly on sandy beaches along the southern shores of Whaingaroa harbour. I understand causes, effects and possible remedies of coastal erosion in the Raglan area. A substantial amount of land began to erode between 2002 and 2003 so I and several hapu members and residents took immediate action to prevent the loss of land. The winds, waves, materials and labour all played a role in rapidly restoring dunes and stopping erosion. Once a suitable profile had been shaped by the wind and the sea, the planting of pingao and spinafex took place and is still ongoing. When fully grown these plants trap sand as it builds up providing a permeable but protective barrier to combat the erosion of coastal lands.

9. I am 82 years old this year and am currently working with school children who are learning to plant food according to the Maori calendar which relies on the moon's cycles. They have planted and are now monitoring spinafex plots on the newly formed dunes at Te Kopua.

10. Today the restored and revegetated dunes provide a buffer between the land and the sea. Waikato District Councils response to the coastal hazard at the time was to remove their building and let nature take its course. Given that most of the land lost at Te Kopua is Maori land, leaving the sea to take the rest of the land was not an option.

11. While visiting Te Whaanga recently, I noticed a number of changes including some erosion on the west of the lagoon. In a couple of areas, sand is already being trapped by coastal plants so I am confident that the erosion that has occurred can be mitigated. The Te Whaanga site unlike Raglan beaches is in an enclosed area which has a running stream feeding into it. The replenishing of sand will take place when enough sand is deposited by the wind and sea and captured by vegetation so it is not lost on the next high tide.

12. In my opinion one of the biggest contributors to erosion on the west coast that I have observed is human activity. The building of hard structures into the sea and the interfering with the natural wave action at Manu Bay created recurring erosion of the land near the boat ramp and beyond. Removal of rocks or trampling down of coastal vegetation on beaches is the first step to letting the sea come in to erode the land. Replanting the area gradually restores the balance.

13. The descendants of Riria and Honehone Kereopa, my wife's family have the longest association of any Raglan residents with Te Whaanga. Members of the Kereopa whanau have collectively lived on their land continually for hundreds of years. They know the stories associated with this place.

14. In conclusion, I can say that I have walked over the house site and that the proposed Hemi home will not affect the sea, or cause erosion as it is some distance from the beach behind some vegetation. Neither Environment Waikato nor the Department

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of Conservation who are in charge of coastal matters produced any evidence at the hearing last year that there was a risk of flooding or hazards in this area. In fact, they didn't object so I am surprised that this matter has become an issue for the Waikato District Council.

SWORN at Raglan by)
JAMES EDWIN LANCELOT)
RICKARD this 18th day of November 2006)
before me:-)

JELRickard

NR

A Solicitor of the High Court of New Zealand
N R Coddington

APPENDIX D McGowan Memo re 17 April, 1999 flood levels in Raglan Township

WAIKATO DISTRICT COUNCIL

MEMORANDUM

TO:	Hazard information holders	**FILE:**	67 02 12
FROM:	Bryce McGowan	**DATE:**	1 July 1999
SUBJECT:	Raglan costal flooding		

The event consisted of a low pressure storm along with strong easterly winds and a spring tide.

The event also caused civil defence emergencies in Pukehoe due to extreme localised rainfalls and around Dargaville due to costal inundation.

The event was monitored by Steve Soanes of the Raglan office. There were some reports of 20 foot waves on the ocean beaches however no accurate record of this was taken. Photos around the Raglan harbour however showed the high point of the storm.

The high point was recorded as being at Wallis street at the tip of the Aroaro Estuary where the flap gate is.

This level was recorded as during a surge having passed over the crest of the road which is **2.8490m above Motoriki Datum**.

As such this should be recorded as the highest known flood event for inside this harbour.

Any development to occur in the vicinity of the sea level should ensure that they are at least 300mm above this although Jim Dahn of EW believes that this was only about a 30-40 year return event.

An evaluation of this event suggests that a conservative approach could be taken and round this event up to a 1% AEP event by going from 2.849m to **3m**

This rounding up fits roughly within the analysis provided by Dr Willen De Lange in his paper to the 1995 ARC sponsored workshop on natural hazards. This suggests that for a storm surge the difference between a 30 year return period event and 100 year return event will be about 100mm difference in the height of the storm surge. However a 100 year event is likely to be associated with higher winds and therefore larger inner harbour waves so the analysis is probably a little underestimating. This aspect should be dealt with by the 300mm freeboard that is required in setting floor levels.

The final aspect that needs to be assessed is sea level rise this aspect is required to be taken account of by the district plan. The international group on climate change (recognised as the world authority on climate change) has stated that they believe sea level rise will be between 0.3 and 0.5m over the next 100 years as such using the precautionary approach the 0.5m level has been used.

Therefore the Design Event for costal flooding in the inner harbour area in Raglan which is assessed as the best estimate for the 1% AEP event is 3.5 meters above Motoriki Datum.