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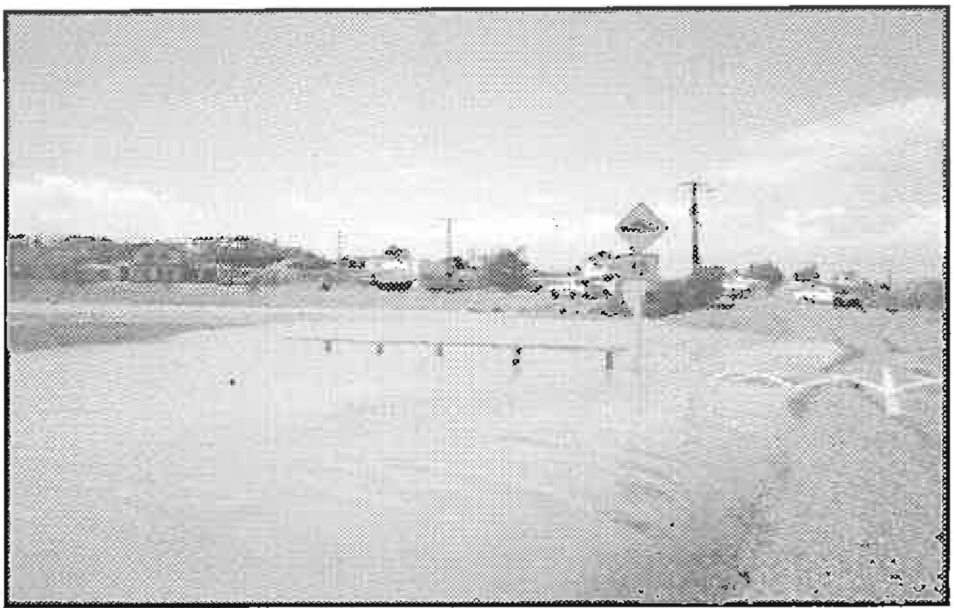
Storm Surge and Wave Run-up Design Levels for Foxton Beach

An Assessment of Flood Risks and Mitigation Options



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Front Cover Photos
Flooding at Holben Parade, 22 October 1998

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Executive Summary

CONTENTS

Executive Summary i

Contents iii

1. Introduction 1

- 1.1 Objective 1
- 1.2 Scope of Work 1
- 1.3 Datums 2

2. Components of Storm Surge 3

- 2.1 Astronomical Tide 3
- 2.2 Barometric Set-Up 4
- 2.3 Wind Set-Up 5
- 2.4 Estuary Effects 5
- 2.5 Total 'Stillwater' Level 6
- 2.6 Wave Set-Up 6
- 2.7 Wave Run-Up 6

3. Available Data 7

- 3.1 Extreme Water Levels at Port Taranaki Recorder 7
- 3.2 Recorded Inundation Levels Within the Study Area 8
- 3.3 Wave Characteristics 9

4. Model Development 11

- 4.1 Methodology 11
- 4.2 Model Layout and Boundary Conditions 11
- 4.3 Extreme Sea Level analysis 12
- 4.4 Wave Run-up and Wave Set-up 13

5. Results 15

- 5.1 Calibration 15
- 5.2 Design "Stillwater Sea Levels" 16
- 5.3 Design Wave Run-Up Levels 16

6. Flood Risk Assessment 17

- 6.1 Flood Threats From Manawatu River 17
- 6.2 Flood Threats From the Open Coast 18

7. Flood Mitigation Options 19

8. Flood Mitigation Options Cost Estimate 20

References 21

1. Introduction

1.1 Objective

On 22 October 1998 significant flooding occurred at Foxton Beach. Some remedial works were installed. Horizons Regional Council has undertaken to comprehensively assess the threats of flooding to Foxton Beach and to propose further remedial options to provide a 1% Annual Exceedance Probability (AEP) level of flood protection..

There are four main flood threats to Foxton Beach:

1. Direct flooding from the Manawatu River.
2. Wave run-up inundation either over the fore-dunes, or through an eroded dune.
3. Localised stormwater ponding.
4. Groundwater flows through the pervious sand terrain.

MIKE11 hydraulic modelling of the Manawatu River (G.S.Doull, Senior Design Engineer) shows that the direct flooding from the Manawatu River is almost entirely influenced by extreme sea levels (rather than river flows and likely coincidence thereof). This report therefore conducts an assessment of storm surge levels and storm surge wave run-up elevations, with the purpose of determining design levels for a 1% AEP storm, otherwise known as the 1 in 100 year storm. These estimates include the mid-range 2000 IPCC (Intergovernmental Panel on Climate Change) mid-range forecast of sea level rise to the year 2100.

This analysis identifies flood threats I and II (above).

This report does not deal with localised stormwater ponding (from rainfalls directly onto the local catchments) or groundwater flows. In regard to the latter, the local residents have reported that groundwater levels vary in response to tidal levels. These matters are more likely under the jurisdiction of the Horowhenua District Council. The information contained in the report on design storm surge levels is relevant to determining mitigation options.

This report complies with key policies within the New Zealand Coastal Policy Statement and the Horizons Regional Council, Regional Coastal Plan (Changes 1 and 2).

1.2 Scope of Work

This report presents design water level estimates for the Foxton Beach location. Design storm surge estimates are also presented for the coast at the Whanganui River mouth (to provide information for a conjunctive investigation of flood risk and mitigation options for the Lower Whanganui River).

There is very little calibration data from storms. However, the information that is available has been used.

Tsunami risks are not included in the study scope.

1.3 Datums

All design levels presented are in terms of Moturiki Datum. In some cases the data evaluated is in terms of Wellington Datum 1953 (for example the Manawatu River cross-section data and no doubt the peak wave run-up levels recorded in the 1976 storm). However, there is only a small (and insignificant) difference between the two datums. Data presented in terms of the Taranaki Vertical Datum has been translated to Moturiki Datum by subtraction of 0.132m - after in-depth analysis by the Horizons Regional Council survey team (Ron Estall pers comm). Similarly, the Port Taranaki Chart Datum is -1.947m in terms of Moturiki Datum.

2. Components of Storm Surge

Storm surges are caused by coastal storms raising sea levels through a combination of the drop in atmospheric pressure and wind driving the sea onto land. The adjustment of sea level in response to pressure changes is called barometric set-up (otherwise known as the inverse barometer effect). The adjustment of water levels due to wind is called wind set-up. Note: wind set-up can be of two types, being that caused on the open coast and differential effects across an estuary.

The astronomical tide, barometric set-up and wind set-up combination create an equivalent "stillwater" level.

A third component is the near-shore increase in water levels as the waves approach land, called wave set-up.

For the open coast a significant component of the peak water level reached on land is wave run-up. Wave run-up causes waves to swash above the "stillwater" level, and may cause inundation landward of the dunes (or other coastal defences) if the wave run-up overtops the dune crest.

Typically storm surge persists for 1.5 to 15 days in New Zealand waters.

2.1 Astronomical Tide

The astronomical tide is caused by the gravitational attraction of the sun and moon on Earth's waters. The astronomical tide levels are estimated based on around 600 components and presented in the New Zealand Nautical Almanac and other publications. The almanac presents detailed data on the Port Taranaki site and further data on the Secondary Ports site at Manawatu River. This data shows the following high tide levels:

Table 2.1. Astronomical Tide Levels (Chart Datum)

Astronomical Tidal Levels	Port Taranaki (m)	Manawatu River (m)
Mean High Water Springs (MHWS)	3.57	2.4
Mean High Water Neaps (MHWN)	2.72	1.8
Mean Sea Level (MSL)	1.97	1.3
Mean Low Water Neaps (MLWN)	1.20	0.9
Mean Low Water Springs (MLWS)	0.37	0.2

To convert these figures to Moturiki Datum subtract 1.947 m. Note this gives some anomalous numbers at the Manawatu River mouth, with some other datum correction possibly required.

The Web-based NIWA tide forecaster gives peak levels (relative to MSL). An assessment of these for the week of 18-24 May 2007 (File Note "Whanganui River MIKEFLOOD Modelling Design Sea Levels, dated 18 May 2007, Horizons Regional Council files DE 0101, RE 0270, PP 0403) gives the following peak levels:

Port Taranaki: Peak of 1.69 m on 18 May declining to 0.76 m on 24 May.
Whanganui: Peak of 1.31 m on 18 May declining to 0.48 m on 24 May.
Manawatu: Peak of 1.14 m on 18 May declining to 0.40 m on 24 May.

The above information shows there is a big variation in the tidal range, with it decreasing southwards. The Port Taranaki peak of 1.69 m on the afternoon of 18 May is at the spring tide chart datum value of 3.6 m (chart datum) in the New Zealand Nautical Almanac. It is thus exactly at spring tide.

Consequently, it appears that spring tide levels at the Whanganui River mouth may be around 0.4 m lower than at Port Taranaki and 0.55 m lower at the Manawatu mouth (note the MHW values will be closer together).

Summarised Data provided by Marianne Watson includes 74 pages of relevant information. Particularly relevant are very detailed analyses by NIWA staff Derek Goring and subsequently by Michelle Wild – the latter analyses incorporated data from the short term Foxton Beach offshore recorder and Kapiti Island recorder. The concluded levels over the period 1/1/1985 to 26/2/2005 are presented in Table 2.2.

Table 2.2: Design Tidal Levels - NIWA

Level (mm)	Ex Derek Goring (Hydrometric Archive)	Ex Michelle Wild (Hydro Tasmania File)
Minimum	-1634	-1393
Maximum	1668	1392
Mean	0.0715	0.0136
Standard Deviation	637	598
Lower Quartile	-489	-486
Median	-49.7	0.0938
Upper Quartile	481	486

Careful examination of the Wild analysis shows that MHW is around 0.25 m below the maximum level and so the MHW values adopted are:

Manawatu: 1.15 m
Whanganui 1.25 m

These values would be within ±0.1m of Moturiki Datum.

2.2 Barometric Set-Up

Depressed atmospheric pressures cause a rise in water levels. This is sometimes referred to as a bulge of water following the centre of the depression. The general increase in water levels is one centimetre per hectapascal (or millibar) drop. There is also a corresponding drop in sea level associated with rises in air pressure.

The standard equation for calculating the barometric set-up is:

$$\Delta H_{BAR} = z(1014 - pa)$$

where

ΔH_{BAR} = surface elevation change (metres)
z = barometric factor with standard value of 0.01
pa = atmospheric pressure at sea level (HPa or mb)

The value of Z varies from place to place slightly, but the given factor is a good guide. When a depression moves quickly the change in water level is superimposed as a surge on prevailing water levels. The surge behaves as a long wave with a wave length approximately equal to the width of the depression. Along the open coast these long waves may increase in amplitude as a result of shoaling on the near-shore bathymetry, while within harbours amplitudes may increase or decrease depending on the harbour geometry and conveyance capability through the mouth. The response of sea level to pressure changes occurs relatively slowly, taking in the range 2 to 12 hours, and affects large areas in an approximately uniform matter (Tonkin & Taylor, 2002).

Barometric set-up can also be harmonically enhanced by the speed of travel of the storm. This occurred in central parts of New Zealand in Cyclone Giselle (the "Wahine" storm) in April 1968 (reported on by Heath, R..(reference to be inserted)).

2.3 Wind Set-Up

The surface shear stress caused by the wind (called the geostrophic wind) travelling over the sea surface drives water from the prevailing wind direction. An on-shore wind thus drives a "wedge" of water against land. The magnitude of the height of this wedge component is called the wind set-up.

There are complex equations available to calculate the wind set-up, dependent on several factors including:

- Intensity, duration and direction of high winds;
- Coastline bathymetry (the wind set-up is greater in shallower waters); and
- Coastline geometry. The concave shape of the Manawatu coastline is minorly conducive to enhancing surge levels. (This is a much-documented phenomenon during hurricanes, with significantly elevated levels on the Gulf of Mexico coastline in the Caribbean Sea.)

2.4 Estuary Effects

There are two main components in estuary effects, being:

- Hydraulic effects relating to influx of freshwater, mouth outlet controls, depth and channelling of water; and
- Spatial differences in water levels due to the differential effects of wind stress; with increases in water levels on the downwind shoreline and decreases on the lee shoreline.

This report does not deal further with estuary effects as they are minor due to a limited fetch length – although there are waves generated within the Foxton Estuary which would wash against stopbank defences, with potentially occasional minor overtopping.

2.5 Total 'Stillwater' Level

The total "stillwater" level is the combined total of the astronomical tide, barometric set-up, wind set-up and estuary effects. These may well not act with peak magnitude simultaneously. For example the peak set-ups may occur at low tide (although generally there would not be a significant attenuation in set-ups by the preceding or succeeding high tide); similarly the surge on various parts of the New Zealand coast caused by both Cyclones Giselle (1968) and Fergus (1996) occurred on neap tides.

2.6 Wave Set-Up

Wave set-up is a super-elevation of the water surface over normal surge elevation due to onshore mass transport of water by wave action alone. It is caused by energy dissipation due to the shoaling of incoming waves, and is more pronounced in environments with steeper beach slopes and hence the depth of breaking is closer to the shoreline.

It is smaller in restricted fetch environments, low beach slopes and shallow water depths (Tonkin and Taylor, 1999 and 2002).

2.7 Wave Run-Up

The wave run-up component is the elevation above the combined level from the other components reached by the wave swash. It is dependent upon breaking wave height and period, beach slope, and the resistance characteristics of the beach sediment. It increases with wave height and period and beach slope and decreases with coarser sediment.

It is also affected by degree of sheltering from headlands, islands or reefs, and wave angle with the shoreline.

3. Available Data

Documented data that is available on water levels and wave characteristics is very sparse. Some anecdotal information may be available from longstanding locals and the tangata whenua. This should be included in future design reviews.

3.1 Extreme Water Levels at Port Taranaki Recorder

The Port Taranaki water level recorder has been in operation since 1 December 1985.

An extreme sea level analysis was undertaken by NIWA in 1997 to estimate the annual exceedance probabilities of extreme sea levels using a revised joint probability method.

This analysis was not just based on an annual series of peak water levels. Instead it allowed for the interaction between the astronomical tide and storm surge components.

The estimates for the frequency domain 20 to 0.5 percent are presented in Table 3.1 (estimates are also presented for 0.2 to 0.01 percent).

Table 3.1: Exceedance Probabilities of Extreme Sea Levels For Port Taranaki (1986-90) - NIWA

Exceedance Probability (%)	Return Period (Years)	Extreme Sea Level (m)	Standard Error (m)	Storm Surge Component (m)
20	5	2.34	0.10	1.14
10	10	2.58	0.13	1.38
5	20	2.81	0.16	1.61
2	50	3.10	0.19	1.90
1	100	3.32	0.22	2.12
0.5	200	3.54	0.25	2.34

Note: The storm surge component has been based relative to MHW at Port Taranaki of 3.15 m, which is 1.20 m in Moturiki Datum.

Unfortunately the analyses is only based on the five years of continuous sea level data from 1986 to 1990 (inclusive) and the estimates presented give much higher storm surge components than are generally accepted (or even observed) in New Zealand. For example estimates of the 1% AEP storm surge generally fall between 1.0 and 1.5 m on the New Zealand coastline. This unusual result is no doubt due to the very short length of record. Consequently, the estimates presented in Table 3.1 cannot be regarded as being reliable.

Horizons Regional Council staff have extended this analysis, May 2007, to incorporate an additional seven years of record at the Port Taranaki Site, through to 1997 (P.L. Blackwood, L-Moments Analysis). The additional data was of poor quality and has been carefully corrected by Jeff and Marianne Watson. Data since this date is available, but not in suitably corrected form. The estimates are presented in Table 3.2.

Table 3.2: Exceedance Probabilities of Extreme Sea Levels For Port Taranaki (1986-97) - Blackwood

Exceedance Probability (%)	Return Period (Years)	Extreme Sea Level (m)	Storm Surge Component (m)
20	5	2.075	0.875
10	10	2.165	0.965
5	20	2.26	1.06
2	50	2.405	1.205
1	100	2.54	1.34
0.5	200	2.705	1.505

Note: The storm surge component has been based relative to MHW at Port Taranaki of 3.15 m, which is 1.20 m in Moturiki Datum.

These values seem reasonably realistic and suitable for both the Foxton Beach and Whanganui River modelling.

3.2 Recorded Inundation Levels Within the Study Area

Details on peak water levels around Foxton are scarce. The best available information is contained on a series of photographs taken during the 22 October 1998 event. These show flooding of Linklater Street, Shortt Street, Warren Street, Watchorn Street and the large playground north of Holben Parade. Careful examination of the photographs shows that peak levels were between 1.5 m and 2.0 m in the playground, around 2.0 m in Linklater and Shortt Street and slightly higher on the corner of Warren and Watchorn Streets (although flooding here is surface ponding from a local catchment, possibly aggravated by high tailwater levels downstream).

Gibb presents on a large 1976 Storm Event. Dr J.Gibb recorded a mean storm surge level of 0.72 m in Pauatahanui Inlet, somewhat south of the area. Brougham and Gestro report that the same storm produced a total wave run-up level of 5.8 m at Otaki. The swash from the wave run-up came over Ocean Beach Road, with similar conditions experienced in 1955. It seems reasonable to conclude that the wave run-ups from the 1955 and 1976 storms were around a 5% AEP magnitude.

During 27 December 1947, Foxton experienced a Westerly gale which caused substantial lifting of sea level. This event was observed by Mr S.G.W. Long, an engineer with the Manawatu Catchment Board. Mr Long's observations of this event are contained in Manawatu Catchment Board Level Book No. 16.

Mr Long notes that high water occurred at 10:58 am and that he levelled the high water mark at the Surf club at about 4:30 pm.

A number of high water levels are noted by Mr Long as he surveyed from the old Signal Station (it is believed that this was located near the existing yacht club) and the beach.

The levels obtained along the surveyed line show some variation buy may be summarised as:

Location	Reduced Level (ft)	Comment
SURF CLUB		
	11.39	Datum is Low Water Ordinary Spring Tide (LWOST) Foxton
	11.55	Average of readings between Surf and old Signal Station.
CLUB		
Signal Station	11.99	
Mabey's Store	12.74	

The above observations provide an average maximum sea level resulting from this storm event of 11.92 ft (3.633 m) LWOST.

A reference point used by Mr Long has subsequently been located and levelled to Lands and Survey Wellington Datum. This survey resulted in an average maximum water level of 2.545 m, MSL Wellington, for this 1947 event.

3.3 Wave Characteristics

Wave characteristics that have been recorded are key inputs to model validation and design. Again there is little information. The DSIR Hydrology Centre Report (1988) presents frequency estimates for the significant wave heights at Wanganui and these are presented in Table 3.3.

Table 3.3: Exceedance Probabilities Significant Wave Heights at Wanganui - DSIR

Exceedance Probability (%)	Return Period (Years)	Significant Wave Height (m)
10	10	6.9
5	20	7.4
2	50	7.9
1	100	8.3

These design significant wave heights seem low for the high energy environment in the Taranaki Bight. The frequency estimates were based on deployment of one buoy 9 km off the Whanganui River mouth, in 30 m of water, for just over a year (11 January 1986 to 25 February 1987) and a second buoy 3 km off Himitangi Beach, also in 30 m of water, for a three-month period (18 November 1986 to 25 February 1987).

This very short period of record is insufficient for reliable estimates of significant wave heights of the lower frequency events.

Frequency estimates of wave heights and periods for the Maui area are also presented in Table 4 of the Shell Todd Oil Services Civil/Structural Design Manual. Selections of these are shown in Table 3.4. Note the wave height value for the 10, 5 and 2% AEP storms are an extrapolated value in proportion to the Reduced Y Variate (the estimates are clearly based on an Extreme Value Type 1 frequency distribution).

Table 3.4: Exceedance Probabilities Significant Wave Heights at Maui – Shell Todd

Exceedance Probability (%)	Return Period (Years)	Wave Height (m)	Wave Period (s)
10	10	16.9	13.35
5	20	18.0	13.7
2	50	19.4	14.2
1	100	20.45	14.5
0.33	300	22.1	15.1
0.2	500	22.86	15.4
0.1	1000	23.88	15.7
0.02	5000	26.27	16.5
0.01	10000	27.3	16.8

These design wave heights appear to be the maximum (one in one hundred, or 1%) height, somewhat larger than the significant wave height. Note, the one in one hundred waves in a particular storm are not to be confused with the 1% AEP storm significant wave height. The 1% waves are calculated as 1.53 times the significant wave height (Holman, 1986 and Healy, 1993)

For comparison details on wave characteristics recorded in Cyclone Ivy (approximately 10% AEP) at the Environment Bay of Plenty wave rider buoy located off Otamarakau, including levels right along the eastern Bay of Plenty coastline were:

H_s 6.8 m
H_{Max} 10.3 m
T_p 20.0s (peak period)
T_s 16.4 (significant wave period)

Hindcast estimates of the wave characteristics in Cyclone Giselle in the Bay of Plenty (approximately 2% AEP, being rank 2 or 3 in 110 years of recorded storms, with the 1936 storm the largest) are presented in Frisby and Goldberg (1981).

H_s 9.9 m
T_p 15s

The wave climate on western New Zealand coasts is described as "*a fairly high wave energy zone*" and north-eastern North Island (East Cape to North Cape) "*a low-energy lee shore*" although at this location "*highest waves occur* *during extra-tropical cyclones, or as swell that is generated by Pacific cyclones well out to the north-east of the North Island"* (Ministry for the Environment, May 2004).

Consequently, design wave heights are expected to be higher on the Manawatu coastline than those recorded and estimated in the Bay of Plenty.

4. Model Development

4.1 Methodology

The following methodology is used to determine the design 1% AEP storm surge wave run-up elevations.

i. Review historic inundation information (refer section 3.2).
ii. Review previous design estimates.
iii. Derive the "stillwater" levels applicable along the coastline (these include the conjunctive impacts of astronomical tides, barometric set-up and wind set-up) for the calibration storms.
iv. Derive the "stillwater" levels for the design 1% AEP event.
v. Create the wave run-up and set-up computer model. Derive the wave run-up components for the calibration storms and calibrate the relevant parameters in the model.
vi. Derive the final 1% AEP storm surge wave run-up level by summing the components. Note: the conjunctive summing does not overstate the levels expected in the design storm, as there will be a strong correlation between the "stillwater" and run-up components. Any out-of-phase or other effects will be minor as the "stillwater" effect generally persists for some time. In any case this assumption is inherent in the calibration process (that is, any out-of-phase or other effects would understate the resultant parameters).

The calculated levels are thus determined by summing the following components:

- 1% AEP "stillwater" level (conjunctive astronomical tide, barometric set-up and wind set-up)
- Wave set-up and run-up
- Sea level rise to 2100 of 0.43 m – being the mid-range 2000 Intergovernmental Panel on Climate Change (IPCC) estimate.
- Freeboard of 0.5 m

4.2 Model Layout and Boundary Conditions

The design wave characteristics for the 1% AEP storm are based largely on those for the Maui area, but with a 10% reduction in wave heights to reflect a lesser exposure in the Taranaki Bight; with the information from the other storms corroborating, these are reasonable estimates, as follows (refer section 3.3):

H_s	12.0 m
$H_{1\%}$	18.36 m
T_p	14.5s

4.3 Extreme Sea Level analysis

Different values are needed for the following two cases:

- Design river flood plus likely coincident storm surge – it is common in New Zealand to use a combination of 1% AEP flood with 5% AEP storm surge unless the correlation is weak (e.g. a thunder pump on a small coastal stream often has little or no associated storm surge);
- Design storm surge plus likely coincident river flood. The frequencies as above but opposite.

However, after examining the available data (including over-plots of coastal and inland recorder sites on both the Whanganui and Manawatu Rivers supplied by Jeff Watson, it is evident that the coincidence of storm surge with river flooding is well below normal for both the Whanganui and Manawatu Rivers. This is because the catchments of both rivers contain a significant area well away from direct west coast maritime influences – particularly the Manawatu, where the catchment extends eastwards of the Ruahine Range. It is concluded that the most likely (and possibly still slightly conservative) combination of events is 1% AEP river flood with 20% AEP storm surge and vice versa.

It should be stressed however, that this would not apply to coastal catchments (e.g.Ohau-Manakau) where the same meteorological drivers causing flooding could well cause storm surges.

On this coastline an array of design sea levels have been determined and justified for use in conjunction with the design flood (usually 1% AEP – the design sea levels usually being for a less intense storm, as the correlation between events is not 100%). The levels are either in Wellington Datum 1953 or Moturiki Datum.

These designs have included:

Waikanae River

Peak level 2.0 m; range (0.2 m, 2.0 m) based on a spring tidal range and 0.9 m storm surge - adopted by Wellington Regional Council.

Otaki River

Peak level of 2.5 m; range (0.7 m, 2.5 m) based on a spring tide range and 1.4 m surge. This surge was 0.4 m greater than a 5% AEP, however sea levels were observed to reasonably frequently reach 2.5 m (possibly including a misunderstood wave run-up component) – adopted by Wellington Regional Council, reconfirming the Manawatu Catchment Board figure.

Manawatu River

Peak level of 2.0 m; range (-1.0 m, 2.0 m). This was adopted by Graham Doull after careful consideration of the available information. It is based on a ±1.5 m large spring tide cycle plus 0.5 m storm surge. Graham Doull also applied a storm surge component of 0.9 m, giving a peak level of 2.4 m for a 1% AEP storm surge; combining that with a lesser flood peak.

This surge component is in keeping with advice contained in Ministry for the Environment (May 2004) which states (section A2.1.3, p.127) "*Though tides are well described, storm-surge measurements around New Zealand are limited, which makes it difficult to carry out a rigorous return-period analysis of the likelihood of coastal inundation from storm tides around the New Zealand coast. In the interim, the equivalent MHWS or MHWPS high tide level and a default value of 0.9 m storm surge, along with estimates of wave set-up and wave run-up, will provide a realistic severe storm-tide 'event'.*"

Whanganui River

Peak level of 1.45 m for the 1% AEP flood (as at the 2001 climate) and 2.2 m (as the "future" climate) – adopted by Colin Hovey, Wanganui District Council.

4.4 Wave Run-up and Wave Set-up

It is extremely difficult to measure the impacts of wave set-up and wave run-up separately. Calibration processes of these two separate processes would not be reliable. Instead the two components are treated together by formulae that incorporate both effects. Two such formulae are applied being those of Holman (1986) and Ruggiero et al (2001) as modified by Tonkin and Taylor (2002, App G, pp 13-14). The formulae are:

Holman:

$R_{Max} = C\{g/2\pi\}^{0.5}\beta H_B T$

where the parameters are:

R_{Max} = Maximum wave run-up above the "stillwater" elevation (m)
C = A coefficient that varies from 0.83 (rocky slope) to 1.5 (smooth slope), a value of 1.07 was found by Tonkin & Taylor (1999) to fit fine sandy beaches.
β = Beach slope. This was principally obtained through ground survey, with supplementary cross-sections derived from the LiDAR survey.
H_B = Breaking wave height (m), based on a transformation of $H_{1\%}$
T = Wave period (s)

Ruggiero et al:

$R_{Max} = 0.27C(\beta H_B L)^{0.5}$

and where the additional parameters are:

C = A coefficient that was found by Tonkin & Taylor (2002) to be 0.9 for several eastern Bay of Plenty beaches
L = Wave Length (m) = $\{g/2\pi\}T^2$

Great care is required in applying these formulae. The biggest difficulty is the common problem where a beach has more than one characteristic slope – particularly in the fore-dune zone.

On occasions the calculated wave run-up elevation exceeds the dune crest, often with a much flatter slope behind. In these instances a "composite slope" analysis is required where the residual wave height above the cusp of the fore-dune is redistributed to the flatter slope behind the dune and this gives very good calibration results. The Ruggiero method gave the best predictions for this situation.

Where the land falls away behind the dune, all that can be done is an assessment of the levels taking account of the available flood storage volumes (and any possible egress points). These areas are identified in the hazard zones, but of course the peak level reached landward of the fore-dunes may be less than that of the fore-dune crest.

5. Results

5.1 Calibration

The wave run-up model was calibrated to produce levels close to those recorded at Otaki Beach in the 1976 storm. Surveyed beach profiles recorded over the period 1983 to 2005 were examined. It was clear that there was little variation in beach slope over time (some variation in fore-dune position though) and the surveyed profiles were suitable for estimating wave run-up.

The following parameters were applied:

- Astronomical tide: 1.0 m
  (Datum adjustment included)
- Static storm surge: 0.7 m
  (Based on Gibb's recordings at Pauatahanui Inlet)
- Beach slope: 0.044
- Beach slope (incl. dune): 0.055
- Run-up formulae: Ruggiero
- Significant Wave Height: 10.59 m
  (Maui 5% AEP with 10% exposure reduction)
- 1% Wave Height: 16.20 m
- Wave Period: 13.7 seconds
  (Maui 5% AEP)

This yielded the following outcome:

Calculated Inundation Level:	5.63m
Recorded Inundation Level:	5.8m
Error:	0.17m (2.9%)

No calibration model is expected to exactly reproduce the recorded data. Indeed, to produce an exact mathematical match may result in the development of an unrealistic model, with consequent errors when extrapolating to design extreme events. It is important that the parameters applied in the model are appropriate for the physical nature of the beach and wave interaction.

The result of the calibration is favourable and this shows that the model can be used with a reasonable degree of confidence to produce design 1% AEP inundation levels. A freeboard component of 0.50 m is an appropriate and well-justified quantum for application of the model.

However, considerably more data is required before the methodology could be applied to large scale subdivisional developments with high confidence. This includes recorded peak wave run-up levels, together with measurement of design wave heights and periods and estimation of frequency data.

5.2 Design "Stillwater" Sea Levels

The concluded levels are based on the normal tidal range and storm surge components. This is the least biased approach. Whilst there is a reasonable chance of a surge occurring during a spring tide, this would require full coincidence to meet potential levels. In any case the normal freeboard allowance accommodates this possibility.

The design levels are presented in Table 5.1.

Table 5.1: Concluded Design "Stillwater" Sea Levels at Foxton

Component	Extreme Sea Level (m)	
	20% AEP	1% AEP
Tide (MHW)	1.15	1.15
Storm Surge	0.875	1.34
Datum Correction	<0.10	<0.10
Design Level	**2.1**	**2.6**

The tidal range has an amplitude of 2.3 m.

Therefore, for the 2%, 1% and 0.5% AEP floods the seaward boundary condition has a range of (-0.2 m, 2.1 m), varying sinusoidally. For the 1% AEP storm surge the seaward boundary condition has a range of (0.3 m, 2.6 m).

5.3 Design Wave Run-Up Levelsphi

The wave run-up model was applied to calculate 1% AEP wave run-up levels. The following parameters were applied

- Design "Stillwater" Level: 2.6 m
- Beach Slope: 0.057
- Beach Slope (incl. dune): 0.105
- Run-up Formulae; Ruggiero
- Significant Wave Height: 12.0 m (Maui 1% AEP with 10% exposure reduction)
- 1% Wave Height: 18.36 m
- Wave Period: 14.5 seconds (Maui 1% AEP)

This yields a very high 1% AEP storm surge wave run-up level of 9.65 m, based on the average slope of the combined beach and fore-dune. The beach slope alone would generate a wave run-up level of 7.48 m. The actual level will be between these two figures and vary from place to place.

The recommended design 1% AEP storm surge wave run-up level is 9.0 m (Moturiki Datum)

6. Flood Risk Assessment

6.1 Flood Threats From Manawatu River

Clearly the critical flood risks to the town from the Manawatu River occurs in the 1% AEP storm surge case. MIKE11 hydraulic modelling of the Manawatu River (G.S. Doull, Senior Design Engineer) shows that the river rises approximately 0.03 m from the mouth to Holben Parade in a severe storm surge, thus attaining a level of 2.63 m at that point. Around the boat ramp the river has attained a level of around 2.65 m.

These levels are just above the average level recorded in the major storm on 27 December 1947 of 2.545 m. They also tie up well with the average level between the surf and old Signel Station (near existing yacht club) recorded in that storm of 2.43 m, MSL Wellington.

There are two areas of evident flood risk as follows:

Overflows at Holben Parade

The design level at this point is 3.13 m inclusive of 500 mm freeboard. Freeboard is a provision for the estimate imprecision (generally regarded as being ±0.30 m), construction tolerances and phenomenon not explicitly included in the computer models. These phenomenon include waves and wind effects, debris effects, energy losses (at bends, contractions and expansions), super-elevation (water levels are higher on the outside of a bend) and aggradation.

The Holben Parade embankment at the flood gate is just over 2.5 m in elevation (based on LiDAR contours accurate to ± 0.15 m). Therefore, minor overtopping can occur there. However, the greater concern is a 25 m lower section of embankment immediately to the west of the flood gate. Here levels drop to around 2.0m, possibly lower. The photography of the 22 October 1998 flood suggests this area possibly overtopped, although the flood gate may well not have shut properly.

Allowing for attenuation of the peak level a long-duration storm surge could reach levels of 3.0 m for up to 1k m north of Holben Parade to as far as Cousins Avenue West, with a potential inundation area of around 40 ha.

Overflows at Hartley Street

The design level at this point is 3.15 m inclusive of 500 mm freeboard. Water can overflow from the Manawatu River into adjoining streets over a length of about 400 m west of Dawson Street. Roore Street, Linklater Street and Shortt Street would also flood, with a total potential inundation area of around 10 ha.

Houses at the east end of Hartley Street appear to be above the design flood level, but detailed survey is required to confirm this.

6.2 Flood Threats From the Open Coast

All current houses are protected from storm surge wave run-up by dunes of at least 9.0 m in elevation. The surf club may experience some flooding around the building in a 1% AEP storm (the ground level is around 7.5 m in elevation at this location).

However, as the fore-dune is only 30 m wide in places, the potential exists for a dune blow-out in a severe storm, exposing the houses behind. The risk may be low, but requires confirmation.

7. Flood Mitigation Options

Overflows at Holben Parade

These are easily remedied by construction of a 25 m long stopbank, approximately one metre high. The embankment may require minor raising.

Inspection of this flood gate suggests it requires some upgrading to ensure sound operation. It is understood that the pump station was upgraded following the 1998 flood and this may well be adequate.

Remedying the overflows would be relatively cheap in comparison to potential flood damages.

Individual house raising would be impractically expensive. This option also leaves a high residual risk to residents' access and egress during flooding.

Overflows at Hartley Street

A 400 m long stopbank, approximately one metre high, is required extending eastwards from Dawson Street into higher ground at its western end. It would be preferable to site this slightly landward of the existing ad hoc stopbank at the edge of the Manawatu River. A speed hump or stop log structure is required to return the stopbank into higher ground at Dawson street – this will be a small but complex structure.

A pump is likely to be required to remove the stormwater from the catchment on the landward side of the stopbank. It appears possible that this stormwater was a contributor to the 22 October 1998 flooding, no doubt caused by an inability to discharge to high river levels.

Again individual house raising would be impractically expensive and leave a high residual risk to residents' access and egress during flooding.

Flood Threats From the Open Coast

A coastal hazards report on the erosion risk to the fore-dunes should be commissioned. This will address risks to existing houses in the first instance.

No new development in the fore-dune area or hinterland should be permitted until a coastal hazards report confirms that risks of dune failure are suitably remote.

8. Flood Mitigation Options Cost Estimate

Holben Parade Stopbank Extension	4,000	
Holben Parade Embankment Raising	10,000	(subject to survey)
Holben Parade Flood Gate Upgrade	20,000	
Holben Parade Pump Upgrade	0	(assumed to be complete)
Hartley Street Stopbank	60,000	
Hartley Street Stoplogs	50,000	
Boat Ramp Pump Station	100,000	(possibly less)
Sub-total	244,000	
Add Contingency (15%)	36,600	
Rough Order of Cost:	**$250,000 - $300,000**	

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Lacock, Graeme Wanganui CO & AO

From: Craig Davey [Craig.Davey@horizons.govt.nz]
Sent: Wednesday, 31 October 2007 5:42 p.m.
To: glacock@doc.govt.nz
Subject: FW: Coastal modelling of Tsunami threats
Attachments: Storm Surge Wave Run-up Design Levels for Foxton Beach - May 2007.doc

fyi

From: Peter Blackwood
Sent: Wednesday, 31 October 2007 3:55 p.m.
To: Craig Davey
Cc: Shane Bayley; Allan Cook
Subject: RE: Coastal modelling of Tsunami threats

Craig,

Attached is my Foxton report. That covers storm surge and storm surge wave run-up (open coast).

Tsunami is covered by Shane Bayley - so I've copied this to Shane.

Pete

<<Storm Surge Wave Run-up Design Levels for Foxton Beach - May 2007.doc>>

From: Craig Davey
Sent: Monday, 29 October 2007 9:19 a.m.
To: Peter Blackwood
Subject: Coastal modelling of Tsunami threats

Hi Peter, thanks for considering supplying information regarding your coastal work. The request from the CDVN organiser is as follows;

'has Horizons done any modelling of tsunami threats for Wanganui or this coast, or do you know of such work? I figured it would fit in well with a session on the Castlecliff beach grooming.' - this could also be expanded to include forcasts for climate change affects on sea levels.

The CDVN (Coastal Dune Vegetation Network) is a body of people that are involved with coastal processes and usually dune stablisation and enhancement to protect production, infrasturcutre and beaches for recreation etc. They are holding their annual get together in Wanganui March 5-7 2008.

The seminar would involve a mix of field trips and lecture sessions that highlight the local issues.

Craig Davey

Environmental Coordinator
Horizons Regional Council

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