Effects on Flood Levels from the Proposed Whakatane Marina

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Executive Summary

This report describes the investigation into potential effects on design flood levels from the proposed Whakatane Marina. Computer simulations using the HecRas river analysis computer program were used to estimate the upstream backwater effects due to Schemes C and D. These simulations assumed some compensatory dredging of the left bank. Scheme D was also modelled without this compensatory dredging. The computer model was calibrated against the July 2, 1998 flood event and verified against the November 1999 flood. The modelling approach centred on the use of a synthetic cross section for each of the proposed schemes investigated, and the selection of appropriate transition-type energy loss coefficients. The results showed the magnitude of the effect of the Scheme C proposal to be a rise in 1% AEP flood levels of 0.12m at river chainage 2.195 km (immediately upstream of the marina) and 0.072m at the Landing Road Bridge. The effect due to Scheme C gradually diminishes in the upstream direction to 0.013m at chainage 5.950km. The effect of the Scheme D proposal is a predicted rise in 1% AEP flood levels of 0.086m at river chainage 2.195 km and 0.050 m at the Landing Road Bridge. Upstream of the bridge the effect due to Scheme D is insignificant at less than 0.010m. Building the Scheme D proposal without compensatory dredging would result in a backwater effect of 0.285m at river chainage 2.195 km.

As a final note, the design 1% AEP flood estimate of 2820 m^3/s may be increased due to the 18 July 2004 flood event post analysis. That may result in minor increases to the backwater effects presented above. The values presented are considered suitable for the current project evaluation.

Contents

Tables & Figures

Chapter 1: Introduction

Environment Bay of Plenty was commissioned by Opus International Consultants Ltd. to investigate the potential effects from the proposed marina on flood levels in the Whakatane River. This report outlines that investigation and describes the various assumptions associated with the analysis. Section 2 gives a brief background including relevant associated investigations. Section 3 describes the modelling approach taken and the assumptions made. Section 4 outlines the calibration procedure undertaken before the predictive simulations could be run. Sections 5 and 6 give the results of the investigation and offer a brief discussion.

1.1 Marina Proposal

Preliminary investigations are being carried out into the potential for a recreational boat marina in the Whakatane River by Bellingham Marine, Boffa Miskell, and Opus International Consultants. These investigations are centred around proposal alternatives Scheme C and Scheme D which involve a land reclamation and dredging development on 700m of the right bank immediately upstream of the Yacht Club Building. Details used in this investigation were taken from the preliminary Project Description (Boffa Miskell March 2004), and 1:1000 scale plans labelled: Scheme C (Bellingham Marine, August 2003), and Scheme D (Bellingham Marine, May 2004).

1.2 Whakatane River

The Whakatane River catchment covers some 1780 square kilometres. Its headwaters are in the steep, bush covered Urewera Ranges, and it has the capacity for large and dangerous floods. In its lower reaches the channel has the tendency to meander, and its mouth is restricted by an active sand spit. The township of Whakatane is protected from flood hazards by engineered stopbanks.

Chapter 2: Background

2.1 Stopbank Design Standards

The design level for the right bank stopbank upstream of the yacht club building is for the 1% AEP Storm with 0.80 m freeboard (Asset Management Plan). This is taken as either combination of a 1% flood and 5% tide, or 5% flood and 1% tide. During preliminary modelling it was found that the 1% flood, 5% tide combination was the more critical case. The simulation results shown in this report (sectionChapter 5:) do not include allowance for design freeboard.

2.2 Other Models of the Whakatane River

The lower reaches of the Whakatane River have been modelled several times previously in order to set stopbank design levels for the town flood protection scheme. The two analyses most relevant to this investigation are Balley's 2000 HecRas model and Wallace's 2004 Mike11 model. The downstream boundary water surface for this model was taken mid-reach near the Whakatane wharf directly from Balley's analysis. This is for a 1% flood of 2820 m^3/s coinciding with a tide level of 2.0 m. Balley assumed that the Piripai Spit would erode during the event to form a hydraulic fuse with a width at the time of peak discharge, of 50 m and crest level of 0.5 m.

In the case of the proposal going ahead, the relative backwater effects estimated in this investigation will be combined with design levels from the Wallace report to give absolute design stopbank levels for the reaches upstream of the proposed marina.

In this document, unless otherwise stated, the term 'model' refers to this investigation into the potential effects from the proposed marina.

2.3 Purpose of this investigation

The purpose of this investigation is to estimate the potential effects on flood water levels from the proposed marina development. Specifically to determine the relative upstream backwater effect from the proposed channel constriction at around Environment Bay of Plenty Survey Cross Section W3 and W2A. It is not intended that the results from this investigation be used to set absolute stopbank design levels but rather be used to set an additive quantity to more rigorous analyses.

Chapter 3: Modelling Approach

3.1 General Approach

- A simulation, Scheme Zero, was run on the calibrated geometry built from the most recently surveyed cross section data (July 2003). This gave estimates for the 1% AEP flood levels as they are, without any channel alterations.

- Baseline simulations were run for each of the two proposed schemes by including the corresponding synthetic cross section. (An outline of synthetic cross sections is included in Section 3.2; figure 1 shows the layout of the model). These baseline simulations were used to ensure, in each case, that the additional cross section had no effect on water levels.

- Predictive simulations were run for each proposed scheme by modifying the corresponding synthetic cross section, and the surveyed cross section W2A, to represent the proposed changes to the channel. Only one of the synthetic cross sections was used in any one simulation, i.e. the predictive Scheme D simulation would include section W3/2AschD but not W3/2AschC.

3.2 Synthetic Cross Sections

Two cross sections were synthetically produced in order to estimate the effect of the proposed river channel changes. They are labelled W3/2AschC and W3/2AschD and are identical in shape to section W2A. The placement of these cross sections is shown in Figure 1: Layout of the Marina Model. For each scheme, the corresponding cross section was inserted at the most constricted point. It is considered that this best describes the hydraulic effects in each case.

The method of producing the synthetic cross section data was straightforward. From inspection of aerial photographs it was considered that they would be better represented by a direct translation from section W2A rather than an interpolation between W2A and W3. Cross section data from the 2003 survey was corrected for 27 degrees angle error, elevated slightly, and inserted in the model geometry at the desired chainage.

The additional elevation applied to the synthetic cross sections was determined through an iterative process. Each baseline simulation was repeated with the inserted cross section at various elevations until a level was found that gave water level results identical to the Scheme Zero calibrated simulation. Cross section W3/2AschC was given 0.1 m additional elevation i.e. its data points are 0.1 m higher than the corresponding values for cross section W2A. W3/2AschD was given 0.05 m additional elevation.

Angle errors were also corrected in sections W3 and W3A.

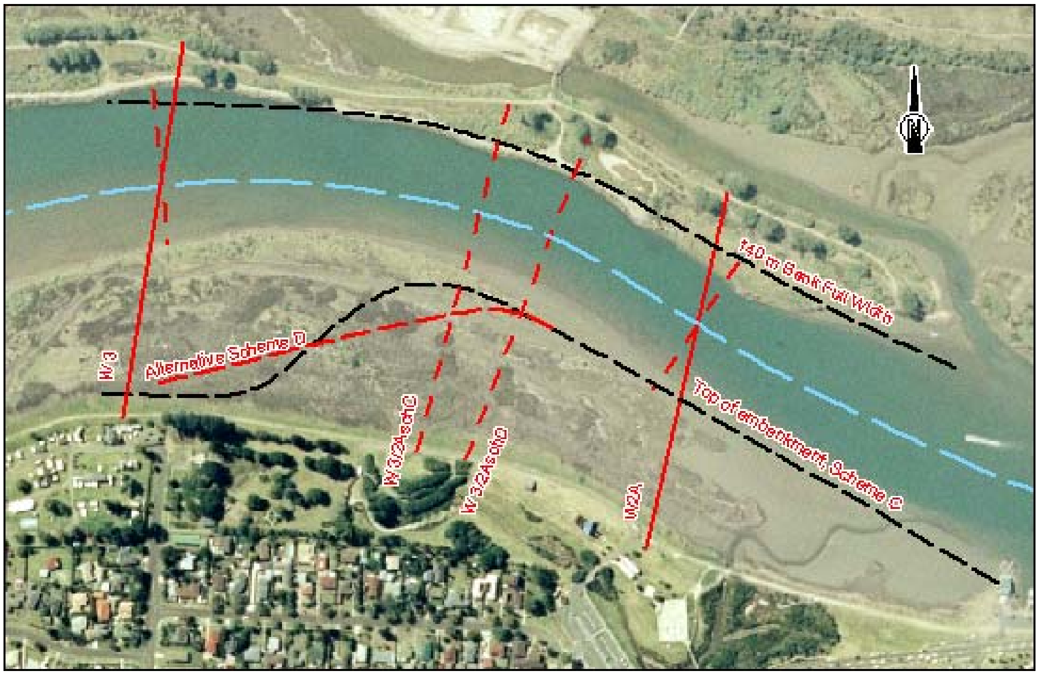


Figure 1: Layout of the Marina Model

3.3 Proposed Changes to the River Channel

3.3.1 Marina Reclamation and Basin

Each scheme plan gives details of a reclamation on the right bank. The top level of this reclamation was taken to be 3.44 m (0.5 m below the level of the adjacent stopbank). The extent of the reclamation was measured directly from the scheme plan with a scale rule, along the line of cross section W2A. It was considered that, due to the enclosing embankment, the basin of the marina would not convey significant floodwater.

3.3.2 Compensatory Dredging of the Left Bank

It is considered that the width of the channel is critical in determining the nature of the river meander pattern. In his Whakatane River Scheme Middle Reaches Investigation, (1992), Titchmarsh determined the critical bank-full width on this reach of the channel to be 140 m.

The main simulations were run with this 140 m width and 1V:2H revetment slopes. This would require considerable dredging of the left bank.

An additional simulation was run to test the scenario of the Scheme D marina in place without any compensatory dredging of the left bank. For this scenario the surveyed cross section data on the left bank were substantially unchanged except to represent a smoothing of the intertidal zone at elevation 0.5 m.

3.3.3 Effect of Scour

No allowance was given for the natural enlargement of the river channel following placement of the marina constriction. Although some scour may occur during flood flows, the timing of this in relation to a flood peak is not predictable.

3.4 Incorporation of Proposed Changes

The proposed changes to the river channel were incorporated by manipulating cross section W2A and the corresponding synthetic cross section. An energy loss due to a flow contraction was also included immediately downstream of W3.

Cross section W2A and the incorporated changes are shown in Figure 2. Data points were removed from, and added to the section to represent a reclamation on the right bank with an elevation of 3.44 m and a revetment batter of 1 vertical to 2 horizontal. The extremity of the reclamation was located in accordance with the scheme plan. A 140 m bank-full width channel was modelled with left bank batter slopes of 1:2.

The reclamation level was set at 0.5 m below the design level for the right bank stopbank. During the simulations the water level at W2A remained significantly below 3.44 m so minor changes to the reclamation level do not effect simulation results.

Once the changes had been incorporated into section W2A, the data was translated to the synthetic cross sections and elevated by the prescribed amount.

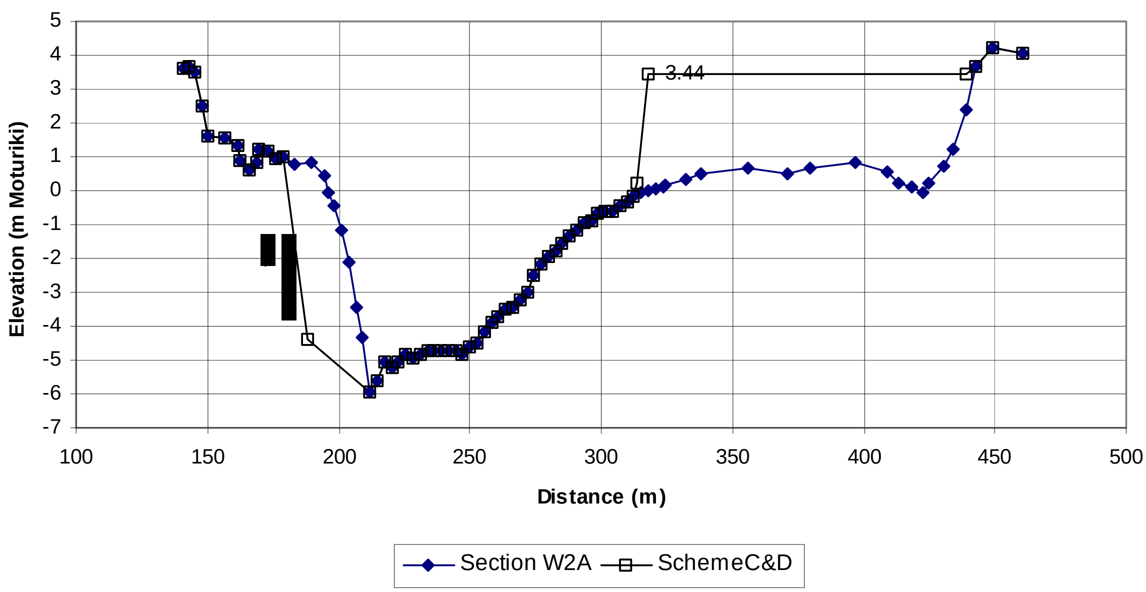


Figure 2: Modelled Cross Section W2A

3.5 Expansion and Contraction Losses

Energy losses from flow expansions and contractions were included in the calibrated model at various points on the reach. The HecRas modelling program uses the absolute difference between velocity heads approach as in equation 1.

$$h_{ce} = C\left|\frac{v_1^{\ 2}}{2g} - \frac{v_2^2}{2g}\right|$$

Where C is the loss coefficient, and v1 refers to the lumped flow velocity at the upstream cross section and v2 at the downstream cross section; hce is the head loss due to the contraction or expansion, and g is the gravitational acceleration constant. The head loss is applied at the upstream cross section.

Typical values for the coefficient C as given in the HecRas reference manual are shown in Table 1.

Table 1: Typical values for transition head loss coefficients for use in equation 1

	Contraction	**Expansion**
No transition loss computed	0.0	0.0
Gradual transition	0.1	0.3
Typical bridge sections	0.3	0.5
Abrupt transitions	0.6	0.8

In the calibrated geometry, an expansion loss coefficient of 0.3 was applied at W2A and immediately downstream of the Landing Road Bridge. A contraction loss coefficient of 0.1 was applied at W3A and immediately upstream of the Landing Road Bridge.

A contraction loss coefficient of 0.3 was added to the geometry at cross section W3 for the Scheme C predictive model simulation. For the Scheme D simulation this was reduced to 0.15 due the less abrupt nature of the contraction in this scheme.

3.6 Boundary Conditions

The downstream boundary for this model was taken to be the design water surface level at cross section W2 given by Balley in his 2000 report, which deals with the difficult hydraulics of the Whakatane Bar and spit fuse. Although the Balley model is based on 1998 survey data and is currently in the process of being replaced, it was considered the best information on which to base this investigation. The downstream boundary levels were therefore 2.56 m for the calibration model (2150 m^3/s) and 2.96 for the predictive model (2820 m^3/s).

The upstream boundary was set at normal depth at cross section W18, some 10.9 km upstream of the proposed marina. The energy slope used in the normal depth calculation was 0.001.

3.7 Flow

The flow for the 1% AEP flood was assumed to be 2820 m^3/s. This value is given in Environment Bay of Plenty's Data Summaries, and was obtained by analysis of 44 years of flow data on the Whakatane River, from 1957 to 2000, fitted using the Jenkinson method to an EV1 probability distribution.

The flood frequency analysis for the Whakatane River will require revisiting once the size of the 18 July 2004 flood is confirmed. This may result in small increases to design estimates.

Chapter 4: Calibration and Verification

4.1 Calibration Process

The model was calibrated against the 2 July 1998 flood using the relevant (1996) survey data. The downstream boundary condition was taken as the simulated water level at section W2 for the 1998 flood, given by Balley in his 2000 report. Expansion and contraction coefficients were selected and applied. Manning's roughness values were then applied and altered to give a good match to the recorded flood level data while retaining a smooth transition of values in the expected range.

The 1998 flood peak discharge was assumed to be the currently accepted value of 2150 m^3/s. This differs somewhat from the initially calculated value of 2243 m^3/s due to a re-analysis following the flood event.

The model was then verified by retaining the roughness and transition loss parameters and running the simulation for the November 1999 flood with 1998/99 survey data. This was compared against the recorded peak water level values for that flood.

4.2 Calibration Results

The results of the calibration simulations are given in Table 2. The simulated water levels from this investigation correspond closely with those given in Balley's 2000 report.

The absolute average water level error in the calibrated simulation on the reach from W2 to W7 is 0.162 m.

Table 2: Calibration Results Summary

X Section Name	Chainage	Recorded Levels	Balley's Report	Calib98pmw	Water level error for Calib98pmw
	(km)	(m Moturiki)	(m RL)	(m RL)	(m)
2	0.820	2.45	2.56	2.56	0.110
2b	1.195	2.47	2.61	2.587	0.117
3a	2.735	2.79	3.00	3.022	0.232
Bridge u/s	3.980	3.29	3.29	3.307	0.017
4	4.090	3.06	3.37	3.281	0.221
7	5.950	3.99		4.267	0.277

4.3 Discussion of Calibration

The slight difference in simulation characteristic between this simulation and Balley's is put down to slightly different modelling approaches.

Balley incorporated the expansion loss at W2A, and contraction loss at W3A, in his Manning's roughness values, as is the accepted practice when estimating design levels on a relatively static channel. However, in this analysis, because the effect under investigation was governed mainly by transition losses, it was concluded that these be best modelled explicitly.

Although the model includes geometry as far upstream as river cross section W18, it has only been recalibrated to W7, some 3.8 km upstream of the proposed marina.

4.4 Verification

The model was verified against the November 1999 flood of 1260 m^3/s. Balley verified his model against the July 1970 flood, so for this model the downstream boundary was taken as the recorded level at river cross section W1A. This gave good verification, with an absolute average water level error of 0.169 m on the reach between W1A and W7 inclusive.

Chapter 5: Model Results

The predicted water surface levels for the modelled scenarios are shown in Table 3. These values are the predicted peak water surface levels for a 2820 m^3/s flood in the lower Whakatane River. They refer to the water level at midstream and may vary from that experienced at the bank due to velocity head effects and super-elevation. The same results are shown graphically in Figure 3.

Table 3: Simulation Results; Predicted Peak Water Levels for the 1% Flood

Chainage (km)	X Section name	Balley 2000 Report	Scheme Zero (No Marina)	Baseline: Scheme C	Marina: Scheme C	Baseline: Scheme D	Marina: Scheme D	Marina: Scheme D; No dredging of left bank
0.000	1	2.49	2.490	2.490	2.490	2.490	2.490	2.490
0.265	1a	2.74	2.740	2.740	2.740	2.740	2.740	2.740
0.820	2	2.96	2.960	2.960	2.960	2.960	2.960	2.960
1.195	2b	3.01	3.003	3.003	3.003	3.003	3.003	3.003
1.670	2a	3.01	2.944	2.944	2.837	2.944	2.837	2.715
1.805	W3/2AschD					2.991	2.884	2.801
1.880	W3/2AschC			3.012	2.904			
2.195	3	3.17	3.138	3.139	3.261	3.139	3.225	3.424
2.735	3a	3.46	3.456	3.457	3.549	3.457	3.522	3.678
3.200	3b	3.52	3.497	3.498	3.585	3.498	3.559	3.709
3.960	Bridge d/s	3.77	3.663	3.664	3.736	3.664	3.714	3.841
3.980	Bridge u/s		3.791	3.792	3.821	3.792	3.801	3.920
4.090	4	3.87	3.787	3.788	3.817	3.788	3.797	3.916
4.550	5		4.046	4.046	4.068	4.046	4.053	4.146
4.960	6		4.403	4.403	4.421	4.403	4.409	4.484
5.415	6a		4.785	4.785	4.799	4.786	4.790	4.848
5.950	7		4.935	4.935	4.948	4.935	4.939	4.991

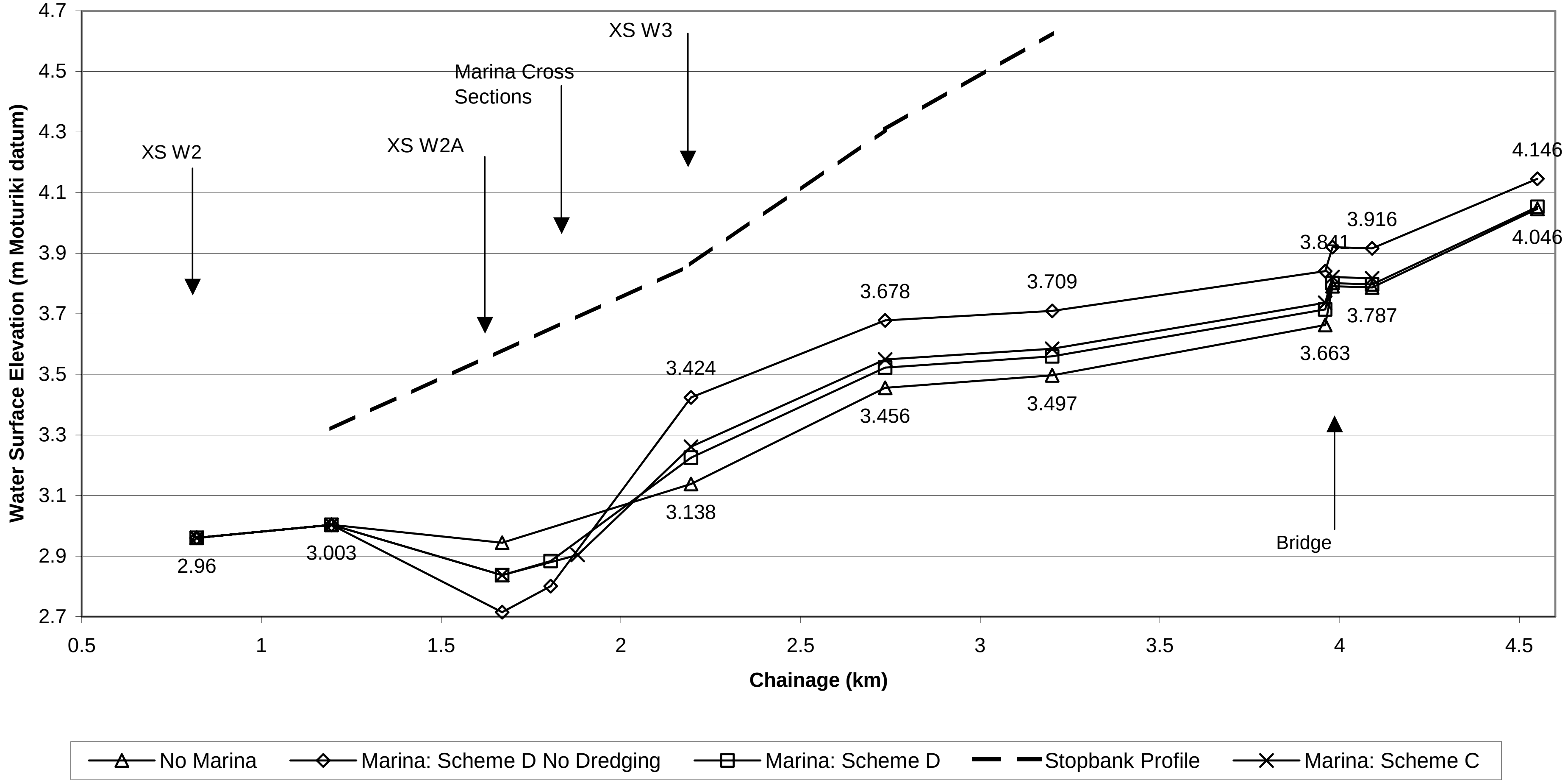
The greatest backwater effect due to the channel constriction was found as expected, to be at river cross section W3, immediately upstream of the proposed marina. This effect decreases in the upstream direction.

The results show the magnitude of the effect of the Scheme C proposal to be a rise in 1% AEP flood levels of 0.12 m at river chainage 2.195 km and 0.072 m at the Landing Road Bridge. The effect due to Scheme C gradually diminishes in the upstream direction to 0.013 mm at chainage 5.950 km.

The effect of the Scheme D proposal is a predicted rise in 1% AEP flood levels of 0.086 m at river chainage 2.195 km and 0.050 m at the Landing Road Bridge. Upstream of the bridge the effect due to Scheme D is insignificant at less than 0.010 m.

The simulation of Scheme D without any compensatory dredging on the left bank, showed a relative backwater effect of 0.285 m at cross section W3.

Figure 3: Predicted Water Levels for Whakatane River 1% AEP
Data Labels show Scheme D Levels without Left Bank Deep Dredging



Chapter 6: Discussion

6.1 Velocity Head Effect and Negative Water Surface Slope

Figure 3, water surface profiles of the various modelled scenarios shows a marked depression of water levels at cross section W2A and in some cases a negative water surface slope between W2A and W2B. This is the result of accounting for velocity head energy in the flow simulation. It is a characteristic of a horizontal constriction, expansion sequence in the channel. This negative water surface slope phenomenon does commonly exist in rivers but is seldom observed for a variety of reasons:

- It is often dampened and obscured by transitional energy-head losses from turbulence in the expansion.
- The slopes involved are so minor that at full scale it is difficult to see without surveying equipment.
- The actual negative slope across the expansion is generally less than that modelled due to complex flow patterns, flow separation, and recirculation of water in the reach immediately below the constriction (back eddies).
- Surveyed flood levels are usually taken from debris levels at the river bank, where the flow velocity is much less, giving a higher water level than in midstream.

6.2 Design freeboard and the recent July 2004 floods

The stopbanks on the right bank of the Whakatane River, upstream of the yacht club, are designed to protect the town from 1% AEP floods with an allowance for 0.8 m freeboard. This freeboard is to account for a variety of additional effects that are not explicitly included in a hydraulic assessment of flood levels, such as:

- Surface waves caused by wind or structures.
- Super-elevation on bends.
- Wear or settlement of the stopbank crest between maintenance works.

This freeboard also includes an allowance for imprecision in the estimates of flood level. It must not be thought of as surplus stopbank height above the level of the design flood.

The peak levels in the Whakatane River during the recent flooding (17/18 July 2004) were within 0.2 m of the stopbank crest in many places along the reach in question.

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