WAIRAU RIVER FLOODWAYS MANAGEMENT PLAN

APPENDICES I AND II:

HYDROLOGICAL AND HYDRAULIC ANALYSES

Wairau River Floodways Management Plan
Appendix I

Wairau River Flood Frequency Analysis

A Review to Determine the size of Flood for River Control Works in the Wairau River and its Floodplain Tributaries

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Executive Summary

Major river control works have been carried out on the Wairau river and its southern tributary, the Opawa-Taylor system for over 130 years since soon after the start of Pakeha settlement of the main Wairau Floodplain.

Serious attempts at measuring the size of floods were started by the Marlborough Catchment Board which was set up in 1957. The Marlborough Catchment Board established a continuous water level recorder on the Wairau at Tuamarina in 1960, and on the Taylor at Borough Weir in 1961. The design of the 1960 Wairau Valley Scheme river control works was without the benefit of this recorder information.

Flood flow gaugings are required to establish rating relationships to convert these water level records into flood size measurements. On the Wairau a gauging programme was carried out in the 1960's and early 1970's. The gaugings were limited as to how big a flood was gauged. Flood gaugings were virtually discontinued between 1974 and 1989. Ironically, this was a time of major changes just downstream of the Tuamarina recorder site on the Wairau with Wairau Diversion development, substantial gravel deposition in the lower Wairau, and river works. These channel changes would have changed the water level - flow rating relationships.

Doubt therefore existed as to the actual size and expected frequency of floods measured at Tuamarina and of the two very large 1983 floods in particular. The July 1983 flood was considered so large and so rare that it would not be worthwhile to construct river control works to control.

Information other than gauging is also available to estimate the 1983 and other floods. This includes surveyed river water levels along the stopbanks, river cross sectional survey, some temporary water level records at different sites, and rainfall records.

All the gauging data and all this other relevant information was analysed together in a major joint study with Works Consultancy Services so as to make the most accurate assessment possible of likely Wairau flood size and frequency, and the size of the 1983 floods in particular.

The main findings with regard to the Wairau were :

- The 1 in 100 year return period flood for the Wairau (ie a flood having a 10% chance of occurrence in the next 10 years) is assessed as being 5500m³/sec. This frequency of flood is commonly used for design of major river control works in New Zealand and is a suitable frequency for Wairau river works.
- The July 1983 flood is assessed as being 5800m³/sec, being a 1 in 150 year return period event and the October 1983 flood as 4400m³/sec as being a 1 in 30 year return period event. These estimates are respectively 1200m³/sec and 600m³/sec less than previously assessed, and therefore more frequent events than considered before.

- The analysis was supplemented by and in good agreement with a review of the less accurately recorded historical flood data from 1920 to 1960. The review showed that historical floods had previously been underestimated in size.

- The implications of these findings is that the current Wairau River Control works should be upgraded to cope with a flood at or nearly the size of the July 1983 flood. Currently the works are barely capable of passing an October 1983 sized event, which has a 30% chance of occurring in the next 10 years.

Under the 1960 Wairau Valley scheme major works have also been carried out on the Taylor - Opawa - Doctors Creek system (or in the case of Doctors Creek proposed diversion, proposed but not carried out).

This review of the Taylor - Opawa - Doctors Creek flood record is less comprehensive than for the Wairau. This is partly because the results are less likely to be controversial, partly because of a lack of data on which to cross check the gauging data, and partly because the large 1989 Taylor flood was easily coped with by the Taylor dam which was altered in 1980 by throttling the outlet culvert.

The main findings with regard to the Taylor - Lower Opawa :

- The Taylor dam is capable of detaining floods in excess of a 200 year return period event without using the spillway. The design 100 year return flood of $285m^3/sec$ peak will utilise an estimated 70% of dam storage, with an outlet flow of $105m^3/sec$. The September 1989 flood of $230m^3/sec$ into the Taylor dam is assessed as having a frequency of 1 in 30 year return period and filled the dam to 35% of storage. Doctors Creek tributary had little flow in that event.

- The lower Opawa - Taylor through Blenheim 100 year return period flood event is assessed at $170m^3/sec$. This is somewhat greater than the 1980 flood of $130m^3/sec$ which had significant Doctors Creek inflow. That flood overtopped and caused damage in the lower Opawa and Taylor. A $130m^3/sec$ flood is assessed as being of a 1 in 20 year flood event under current conditions.

- Further data collection is required to verify these flood estimates.

- This Taylor data analysis is a yardstick in helping assess flood sizes for other Wairau floodplain tributaries. This analysis confirms the previous design floods adopted for the Omaka, Fairhall, Doctors Creek, Wither Hills Streams, Upper Opawa and Riverlands floodway river works, but shows the Pukaka has been substantially under estimated.

- The implications of these findings are that further works are required on the Lower Opawa - Taylor - Doctors Creek system through Blenheim where the current standard of flood protection is for a 20 year return period flood (40% chance of occurring in the next 10 years).

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SECTION A : WAIRAU RIVER

1. Background

The report reviews the flood frequency of floods in the Wairau River so as to make recommendations for a design flood size for Wairau river control works.

The expected flood size for the Wairau River works is a fundamental parameter on which to design the size of river works. It is particularly important to assess this design flood for the Wairau as this river channel hydraulics have and are being changed by channel aggradation, diversions, stopbank relocation and other river control works. Flood levels for todays floods will be different from historical flood levels for the same size of flood.

The design flood size that is typically used elsewhere in New Zealand to protect valuable populated land is a flood that has a 10% chance of occurring within the next 10 years. Such a flood will on average occur or be exceeded once every 100 years, and the term "100 year flood" is a more succinct but less clear description of its frequency of occurrence.

Previous estimates of recommended design flood sizes, and/or 1 in 100 year flood size estimates are :

	Recommended Design Flood	1 in 100 year flood
(a) Vickerman and Lancaster 1924 (Report to Wairau River Board	4700 m^3/sec	not given
(b) Vickerman and Lancaster 1927 (Report to Wairau River Board)	4200 m^3/sec	not given
(c) C C Davidson 1959 (Wairau Valley Scheme Report)	5100 m^3/sec	4600 m^3/sec
(d) L N Pascoe & P A Thomson (1985) (Unpublished report to Marlborough Catchment Board)	5100 m^3/sec	5100 m^3/sec
(e) S N Rae (1987) (Water Resources of the Wairau, Volume 1)	Not given	5220 m^3/sec

To be noted is that each successive estimate has more information on which to base the estimate, and there is a general trend of increase in calculated flood size.

In carrying out this current review a very detailed analysis has been adopted. As far as possible, the base data has been re-examined, and information from a wide variety sources has been examined.

For example, all hydrological gauging and recorder site water levels were examined, and hydraulic backwater/slope area analysis of recorded flood levels down long channel reaches. The answers given by these different analyses were in reasonable agreement, but as could be expected, not perfect agreement. The overall final recommended flood frequency required value judgements on the relative accuracy of the various approaches.

The study has been a joint study between Marlborough District Council staff (E B Williman, W J Noell and V Wadsworth) and Works Consultancy Services (H J Freestone, D C Maslin, M G Webby). The joint study was complementary in that both organisations were involved in examining and analysing different base hydrological data. The joint study was an independent check in that both organisations each applied their own value judgements in balancing the various information sources to come up with their own final overall recommended flood sizes and flood frequencies.

The Works Consultancy Services prepared a report for the Council entitled Wairau River Flood Management Review. The recommended flood sizes in that February 1993 report are very similar to those recommended in this report herein.

2. Hydrological Data Source Reviewed

(Group responsible for analysing data in brackets -
MDC - Marlborough District Council
WCS - Works Consultancy Services)

2.1. Peculiarities of July 1983 flood.

2.1.1. Shape of hydrographic peaks (MDC)

2.1.2. Anaylsis of flood breakout sizes (MDC)

2.2. Water Level Recorder Sites

2.2.1. Lower Wairau at Ferry Bridge 1936-60 (MDC)

2.2.2. Wairau at Tuamarina Bridge 1960-92 (MDC & WCS)

2.2.3. Diversion at Pukaka Outlet 1968-80 (WCS)

2.2.4. Lower Wairau at Dicks Road 1964-76 (WCS)

2.3. High Stage Flow Gaugings

2.3.1. Lower Wairau at Ferry Bridge 1936 (MDC)

2.3.2. Wairau at Tuamarina 1958-1974 (MDC & WCS)

2.3.3. Diversion at Rarangi Bridge 1963-1990 (MDC & WCS)

2.3.4. Lower Wairau at Ferry Bridge 1963 - 1990 (MDC & WCS)

2.4. Monitored Flood Water Level

2.4.1. Diversion, 17 floods or freshes between 1963 and 1990 (MDC)

2.4.2. Lower Wairau, 17 floods or freshes between 1957 and 1990 (MDC)

2.5. River Cross Sections and Backwater Analysis, Model of monitored flood levels (MDC)

2.5.1. Diversion, 15 sets of reaches from 1963-1990 (MDC)

2.5.2. Lower Wairau, 4 sets of reaches from 1957-1989 (MDC)

2.6. Flow Split at Bothams Bend-Ferry Bridge area

2.6.1. Base cross sectional and monitored flood level data 1957 - 1992 (MDC)

2.6.2. Network hydraulic analysis using MIKE 11 programme (WCS)

2.7. Historical Floods (MDC)

2.8. Rainfall Records (MDC & WCS)

2.9. Other adjacent (regional) rivers (WCS & MDC).

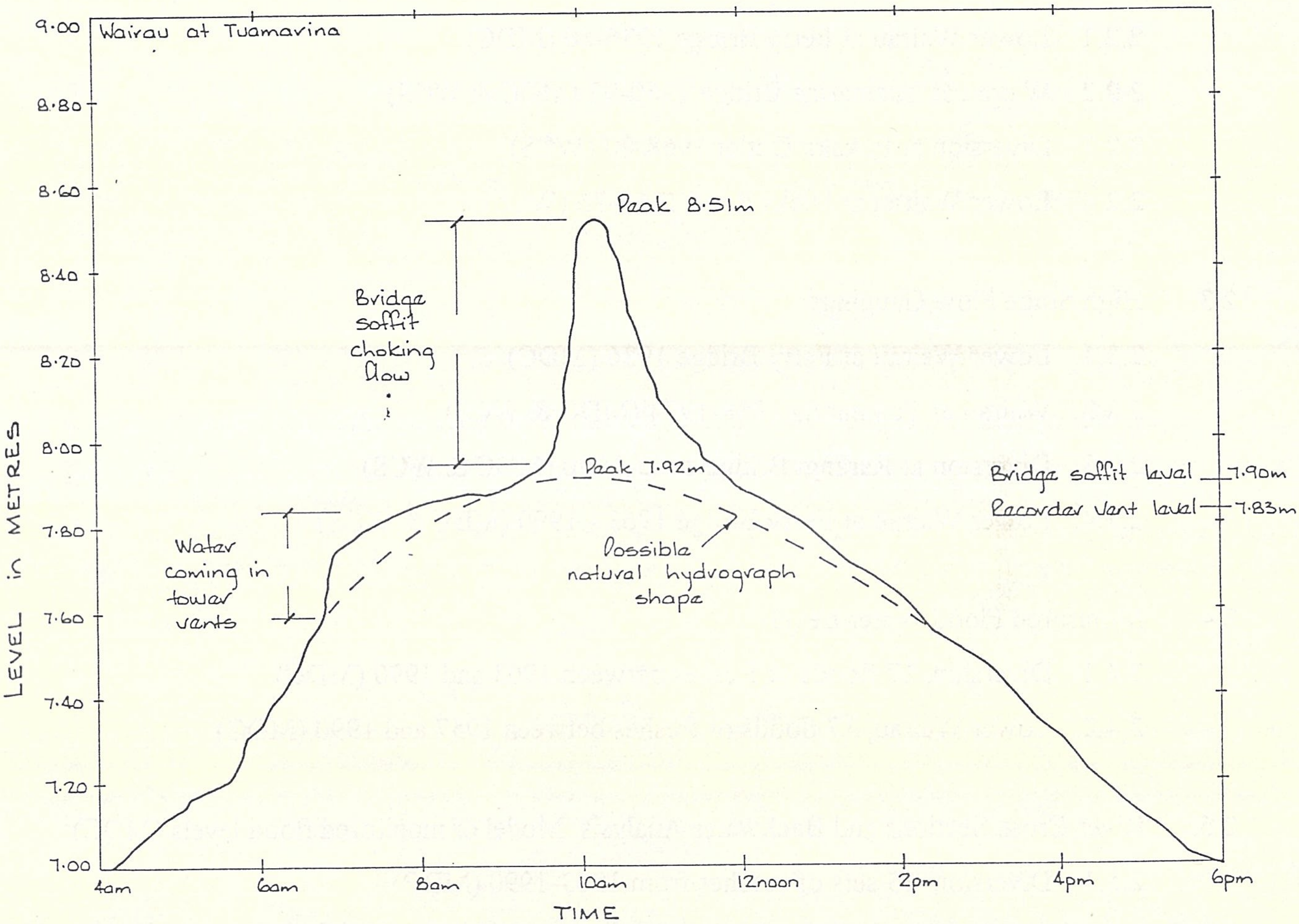


FIGURE 1 JULY 1983 FLOOD HYDROGRAPH RECORDED WATER LEVELS.

3. Peculiarities of July 1983 Flood

3.1 Background

The July 1983 was an extremely large flood. In European history only the February 1868 flood is documented as being of similar magnitude.

No estimates can be made of the Feb 1868 flood, an event that also caused exceptional flooding in the Waimakariri and other rivers.

Following the July 1983 flood an estimate of 7765m³/sec was proposed by Thomson (1983). In a review to the Marlborough Catchment Board a year later Pascoe and Thomson (1985) revised this estimate to 7000m³/sec.

A flood of 7000m³/sec is amongst the largest documented flood sizes of any flood in any river in New Zealand. Within the context of Wairau floods it was much larger than any other documented flood. The Marlborough Catchment Board therefore decided not to re-design the Wairau river control system to cope with such a size of flood. Instead, the flood water levels experienced during the October 1983 flood were adopted as the design standard. The October 1983 flood was recommended as being of 5100 m³/sec size.

This approach was echoed by Rae (1987) when drawing flood frequency plots for the Wairau at Tuamarina. The July 1983 flood was excluded as an outlier from the plot and the October 1983 flood was adopted as the maximum recorded flood, documented as 5000m³/sec and assessed as having a frequency of once in 75 years. On a contradictory note, 4550m³/sec is the documented October 1983 flood size using the MDC currently adopted rating curve.

Particular attention is therefore given in this review to the July 1983 flood event. Though of course the October 1983 flood and other major floods have been reviewed in some detail.

3.2 July 1983 Flood Hydrograph

The July 1983 flood water level record had a most peculiar shape at the peak of the flood hydrograph which is not credible for a normal situation, and has not been completely satisfactorily explained. This is shown on figure 1. (Figure 1 is in fact an edited version in which wave lap fluctuations have been removed).

The peculiarities to note are the sudden rises :

(i) Between 7.6 to 7.8 m in less than 10 min.
(ii) Between 8.0 to 8.5 m in 30 mins,

and sandwiched in between a fairly constant level of 7.9 m for over two hours. These sharp changes represent inconceivable changes in flood flows in short time periods. It has not been recorded on any other flood peak.

An explanation of the shape of the hydrograph lies in two factors :

(i) Vents of the water level recorder tower at a level of 7.83 m
(ii) Soffit level of bridge just downstream of 7.90 m.

The vents in the water level recorder tower provided access for water directly into the tower and thus recorded by the float mechanisms. The water level recorded through the static tubes set in the main flow is different and would be lower than at the tower where afflux has an effect. A difference of 0.23m is appropriate and thus it is quite believable that water will start coming through the vents (at 7.83 m) when the recorded water level is at 7.60. This would then explain the sudden rise in water level from 7.6 to 7.8 m, and also the sudden large increase in fluctuations of water level record starting from 7.6 m.

When the bridge soffit just downstream becomes submerged, a considerable increase in flow resistance will occur causing the water level to immediately rise substantially. The lowest beam soffitt level of this slightly humped shaped bridge is 7.90. The bridge soffit was observed to be submerged at the outer spans at the peak of the flood. The sudden rise in water level from 8.0 to 8.5 m is thus well explained by the submerging of the bridge beam soffitt and associated increase in bridge flow impedance.

The peculiar shape of the hydrograph can therefore be explained by "natural" causes. The July 1983 flood is the only flood to have exceeded a stage of 7.6 metres for this to have occurred. The question remains as to what water level would have been recorded had the vents not let in water and the bridge soffitt not become submerged. It would be this level that would be relevant to determining the flood size from the rating curve extrapolation.

A smooth hydrograph curve, as was recorded in October 1983, has therefore been drawn on figure 1. The peak level of this curve is 8.12 m. However, from this figure must be subtracted the increased level of approx 0.2 m, due to afflux recorded by the vent accessed recorder.

The effective rating curve water level is thus assessed at 7.92 m.

3.3 July 1983 Flood Breakouts

Substantial breakouts of the Wairau River occurred upstream of the Tuamarina water level recorder and this bypassed that recorder flow.

These were re-examined in this review.

(i) A detailed backwater/slope area analysis was carried out over a 3km reach of the Upper Opawa/Rose's overflow into which the breakouts from Upper Conders and lower Conders flowed. The calculated flow was 400m³/sec. This flow also included a component of 50-100m³/sec Omaka tributary flow estimated from aerial photographs of the event and discussion with farmers. Thus total breakout flow from Conders getting into the Opawa system is likely to be approx. 350m³/sec.

(ii) Surveyed flood levels of where this Conders breakout flow overflow crossed State Highway 6 at right angles, were plotted up, together with road levels and level of the ground downstream. This enabled (with the aid of aerial photographs of the event) waterway areas and water velocity to be assessed of the flow crossing the road. The resultant calculated flow was 350m³/sec, approximately equally split with water from upper Conders overflows water to lower Conders breach.

(iii) Substantial overflows occurred over the non stopbanked area at upper Conders. Examination of aerial photographs of the event showed that much of this water flowed back into the river shortly downstream. An estimate of 50 m³/sec was also made of seepage into groundwater based on the area of land inundated between upper Conders and S.H.6 and the probable infiltration rates of the soil. It is thus assessed that only 200m³/sec was effectively lost from the river at upper Conders, with 150m³/sec of this crossing S.H.6 and so into the Upper Opawa River. This is down from the estimate of 400 m³/sec of Pascoe (1983). (To be noted in the October 1983 floods practically all water overtopping at upper Conders flowed back into the river shortly downstream, as shown by aerial photographs).

(iv) At lower Conders a 50m length of stopbank overtopped and breached, scouring down to below ground level. The water level flowing through this breach would have dropped to well below stopbank level as this breach become a flow control. This is confirmed by aerial photographs. An estimate of 200m³/sec for this breach is considered appropriate and in agreement with further downstream calculations (i) and (ii). This is down from the 400m³/sec assessed by Pascoe (1983).

(v) Significant flood overflows occurred on the left bank immediately upstream of Tuamarina State Highway 1 bridge from stopbank overtopping (after the peak of the flood resulting in stopbank breaches), and on the right bank from stopbank breaching. From surveyed flood levels of depth of flow over these stopbanks Pascoe (1983) determined these flows as 280 and 120m³/sec respectively. This is the best estimate of these flow breakouts.

(vi) Summary : The individually identified breakouts and overflows total 800m³/sec. This estimate is less than the 1200m³/sec estimated previously by Pascoe (1983). It will be presumed that they occurred simultaneously with the peak of the flood.

3.4 Rainfall

Quayle et al (1983) examined the rainfall intensity in the Nelson Marlborough area. For the important mid catchment areas of the Branch, Waihopai, Hillersden and Northbank tributaries the 48 hour rainfall of typically 300mm were assigned a return interval in excess of 50 years. This central area is about half the total catchment. The probably more relevant 24 hour rainfall has a slightly smaller area shown to be in excess of the 50 year return interval. The upper and lower areas of the catchment had less extreme rainfall.

WCS (1993) and Rae (1987) have also independently examined rainfall probabilities for this storm event. For the important mid catchment rain gauge sites of Wairau Valley, the Leatham and Waihopai Power Station the return interval for the 24 hour and 48 hours rainfall events is shown as between 1 in 100 and 1 in 200 years. Storm rainfall frequency for the less important peripheral rain gauges was significantly less.

There is no evidence from the rainfall data of the July 1983 flood being extraordinarily rare.

3.5 Snowmelt

Rae (1988) suggests that snowmelt through warm rain melting snow could have been a factor that contributed to a popular viewpoint that the July 1983 flood was of an extraordinary rare nature.

While snowmelt occurred it was probably only a minor influence on the size of the flood.

The melting of snow requires considerable heat. Linsley et al (1958) indicates that from simple consideration of the latent heat and ice melt that rain falling at 10°C will approximately melt 12% of its own volume. Quayle et al (1983) state that the measured air temperature at 1500m over Christchurch rose from 2°C to 6°C during this storm of 9th July. A 10°C storm rainfall temperature is a reasonable estimate for the Wairau catchment snow areas.

Fitzharris et al (1980) from studies of snowmelt in the Clutha 1978 flood recommend that turbulent transfer of heat energy could account for a further similar amount of snowmelt as direct melting by rainfall. Moore and Prowse (1988) studied the July 1983 storm event in the Craigieburn catchment and calculated from meterological conditions that snowmelt would have contributed 21% of the storm run-off.

Applying these considerations to the Wairau July 1983 event indicates that the overall resulting snowmelt contribution may have been 25% of storm run-off. It will be restricted to those areas having significant snow (ie over 300mm depth), which could be approximately a quarter of the Wairau catchment.

To be noted is that the key period in which the storm runoff generated by flood peak was the intense 12 hour period of rainfall between 5.00pm 9 July and 5.00am 10 July. Snowmelt (and rainfall) occurring before or after this intense period had a much lesser effect on the flood peak, its effect will be more on the duration on the flood.

The resulting snow melt supply to the peak of the July 1983 flood may have been a few hundred m^3/sec, less than 10% of the flow, and more likely to be closer to 5%.

It is to be noted that snow in early winter (ie, early July) tends to be of low density - typically 0.2 - so that it appears far more extensive than in actuality. Visual assessment of snowmelt is likely to overestimate its disappearance, especially as this visual assessment will be over a longer period than the snowmelt generating the flood peak.

The greatest periods of snow melt in the Wairau are likely to be in September or October, as reflected by higher mean flows at that time of year. Such snowmelt will also affect the frequency of small floods, but again would not significantly affect the size of major floods.

These recommendations are quite consistent with observations that snow of approx half a metre depth of the Rainbow Skifield, and stretching down to the bushline was washed away during the whole period of the event. The Wairau at Dip Flat recorder includes the Rainbow Skifield and a considerable area of winter snow covered catchment.

The July 1983 flood at Dip Flat was recorded as having a peak of 550m^3/sec, this being assessed by Rae (1988) as being a 1 in 13 year return period event. This is in good agreement with recorded rainfall 24 hour event frequency, without making any extra allowance for snowmelt.

This recommendation is supported by examining the timing of the major Wairau floods. These have occurred in February 1868, March 1904, November 1916, May 1923, November 1926, December 1939, January 1945, June 1954, February 1955, June 1962, September 1970, April 1975, July 1983, October 1983.

In this list of the 14 largest known floods over 3000m^3/sec, 11 different months are represented. The exception is ironically August, a period of maximum snow accumulation. Similarly ironic is that February, a time of little snow, has had 2 major flood events.

Snowmelt certainly contributes to flood run-off by providing saturated soil pre storm conditions. In major storms however antecedant rainfall can equally provide saturated soil conditions. The July 1983 rainfall pattern was double peaked, typical of many major flood producing storms, in which the first peak provides the very saturated soil moisture conditions and the second intensive rainfall generated the flood peak.

Summary : There has been a popular viewpoint that snowmelt added to the already extreme rainfall to make the July 1983 flood of an extraordinary rare nature. The available evidence is contrary to this and indicates that snowmelt had a minor influence on the resulting flood peak.

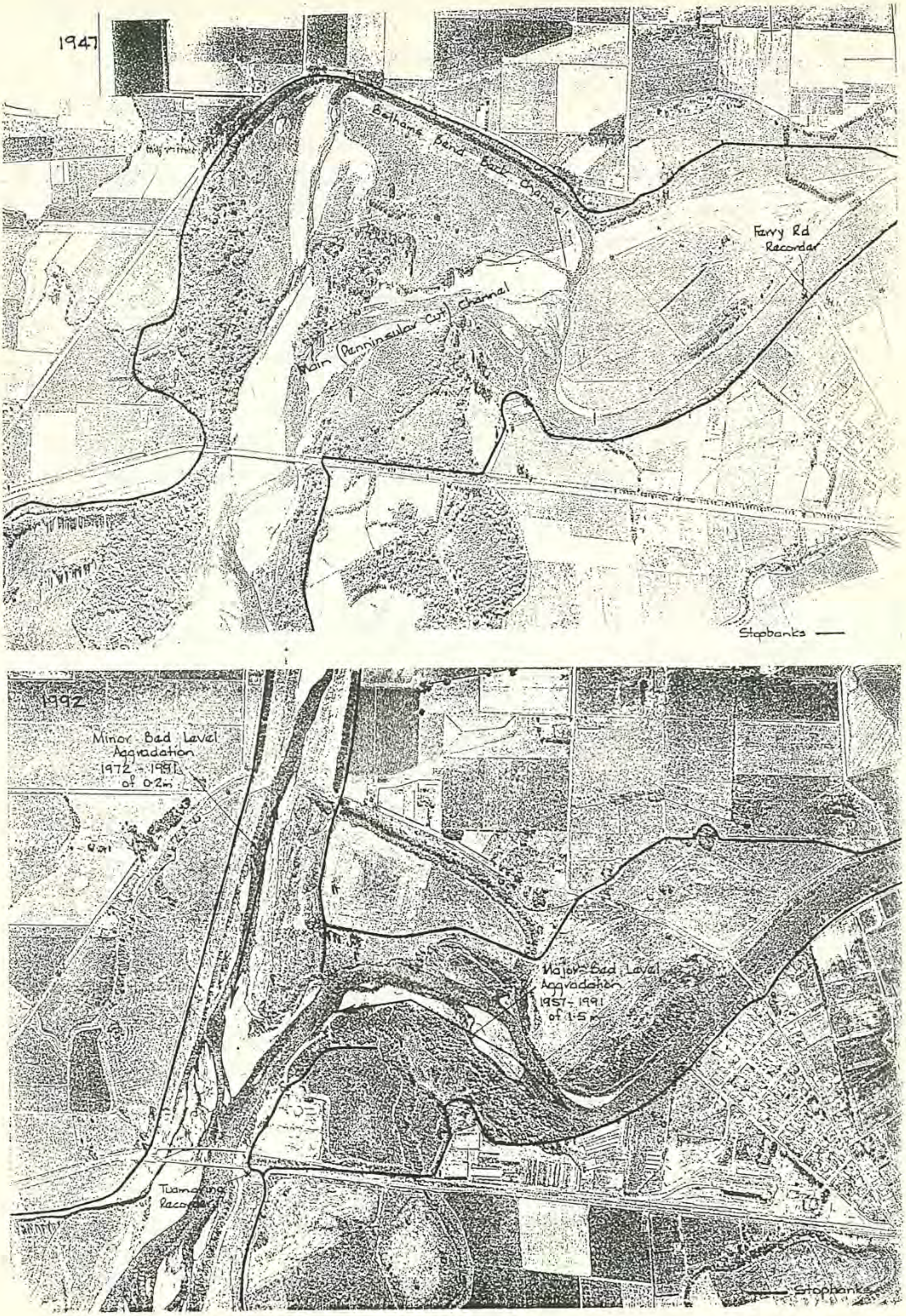


FIGURE 2: Wairau River Tuamarina to Ferry Road bridge; 1947 and 1992

4. Gauging Site Analysis

4.1 Lower Wairau at Ferry Bridge 1936 - 1960

4.1.1 General

The Wairau River Board maintained a water level recorder at the Ferry Bridge from 1936 to 1960. As the site is tidal the record was of no value for low or average river flows and was only used for flood purposes.

This data was the main basis for Davidson's (1959) flood frequency assessment for the Wairau Valley Scheme. Although the site has significant limitations it provided the best available information in 1959.

4.1.2 Site Description

The site is not a good one in that there is a 400m wide grassed flood berm flanking the wide 120m main river channel. Only the main river channel can be gauged. In major floods the grassed berm may carry over a third of the flow. Flood water levels on the berm are different from that in the main channel, being typically 0.5 m higher, but this is undoubtedly variable.

Floods start spilling across this berm at flows of about 800m³/sec, ie at only low flood stage.

4.1.3 Flood Gaugings and Rating Curves

Only one flood gauging was carried out in the 1936-1960period, that in 1938, at a flood flow of 1050m³/sec, and a visually estimated 150m³/sec across the berm. Actual channel water level was less than berm bank level and so certainly less than berm flow level. This gauging was a main basis for Davidson(1959) rating curve with flood slope area measurements in 1958 of a downstream reach to help establish the top end of the rating curve. This single rating curve was applied to the whole 1936-1960 water level record for this recorder.

4.1.4 Proportion of Flow on Berms

The flow split between what flow stayed in the main channel and went under the bridge, as opposed to the flow that spilt onto the left bank grassed berm is determined by channel conditions upstream of the recorder site.

Examination of historical aerial photographs, plans and other historical reports has shown that these channel conditions were variable, and thus the proportion of flow spilling on to the berm was variable. The amount of spilling to the berm would have been particularly influenced by the relative proportion of the flow in the "Botham Bend back channel" and the "main channel", for water will spill out of the Bothams bend channel more readily. The "main channel" is not actually the historical main channel but an artificial cut put through a "peninsular" as recently as 1927. This "main (peninsula cut) channel" would have been enlarging by scour from 1927 onwards and progressively taking a higher proportion of the flow.

Our earliest aerial photograph of the area is in 1943. Then, at average flows, the Bothams Bend back channel is taking all the flow, though by 1947 the aerial photographs shows the "main (peninsula) channel" was taking the higher proportion.

The 1947 photograph also shows the grassed berm being ploughed so as to encourage scour of the berm, and in a Wairau River Board report it is stated as having eroded by 0.6 m. More recently however, our cross section resurvey shows the berm building up with silt. Figure 2 shows the 1947 photograph of the area together with the latest 1992 photo.

In short therefore, the water level recorded at Ferry Bridge did not measure the variable proportion of flood flowing at different levels on the grassed berm opposite. The greater the flood the less reliable this Ferry Bridge recorder record is likely to be.

4.1.5 Flood breakouts

The record also was not able to measure flood breakouts upstream which happened in major floods especially between Tuamarina and Ferry Bridge. For example in the 1939 flood there was a 60 metre long stopbank breach at Bothams Bend, and in 1954 flood a 70 metres long breach at Morrins Hollow as well as other breaches and overtopping. Such breakouts are difficult to estimate but were clearly quite substantial. Significant flood overflows at Conders (both upper and lower) were documented for the 1939 and 1954 floods - perhaps of the order of 100m³/sec. The 1945 and 1958 floods also had substantial overtopping but mainly downstream of Ferry Bridge recorder.

4.1.6 Mean Annual Flood Estimates

Flood peak estimates from this 24 years of record were examined. The mean annual flood peaks for this period of record was assessed at 1890m³/sec using Davidson's rating curve applied to annual flood peak stages. This is some 10% less than the mean annual flood of 2100m³/sec for the 1960-1991 Tuamarina record (discussed later). Similarly the number of floods above 1420m³/sec averages 1.1 per year for Ferry Bridge and 1.5 per year for the Tuamarina recorder site. This implies that the Ferry Bridge record is low, and on average by 10%. This error will not be consistent and the larger floods which overlfow the berm may be higher than this.

4.1.7 Summary

The 1936-1960 record at Ferry Bridge, was the major basis of Davidson's 1959 estimate. While it was the best data available it had the drawbacks :

(i) Channel conditions made it an poor recorder site and not as reliable as the later 1960-1991 Tuamarina record.

(ii) Only one flood gauging taken udring the period of record.

(iii) It appears to have underestimated the 1936-1960 flood flows on average by 10%, but not consistently so, and may have underestimated major floods by more than this.

4.2 Wairau at Tuamarina

4.2.1 Site Description and Downstream Channel Changes

The site was established by the Marlborough Catchment Board in 1960 upstream of tidal influence at the S H 1 bridge where the channel is reasonably constricted and stable.

The site has the disadvantage that there have been changes to the river bed and channel both at the site and downstream. It is the channel, both at the site and downstream that "controls" the stage - discharge rating relationship.

At low flows it is the bed level in the immediate vicinity of the recorder that is the main control determining water levels. As the flow increases, it is the channel conditions further downstream that become the main control on the stage - discharge relationship. In major floods the channel quite some distance downstream will be an influencing control.

At the Tuamarina recorder changes to the bed and downstream channel have and are regularly occurring both at the bridge and for a considerable distance downstream. This is by a variety of mechanisms. Figure 2 shows a plan of the area.

(i) In the vicinity of the bridge gravel waves, or scour during floods, or the regular gravel extraction of this site have resulted in measured changes of mean bed level of up to 0.8 m.

(ii) From the channel bifurcation for the next 1km downstream down the 'Diversion' river channel the bed level has aggraded slightly by 0.2 m since 1972.

(iii) Over this same 1km area of channels there has been significant stopbanking work blocking off some of the berms, notably in 1972 and 1985.

(iv) At Bothams Bend 1.3 km downstream of Tuamarina the Diversion was constructed in 1963 as a 20m wide pilot cut within an overall 300m wide floodway. The entry to the 300m wide floodway was created by knocking down the existing river stopbank. This would have had an immediate effect on major flood levels and this influence would have transmitted right back up to the Tuamarina recorder.

(v) Development of the Diversion occurred following its initial 1963 pilot cut. Regular river cross sectional survey of the Diversion has enabled a good evaluation of its development change to be assessed Neilson (1993). In the upper 2km of the diversion this development for 1963-1975 consisted of erosion of the main channel that was approx. matched in volume by siltation of the berm. In this period the effect of diversion development on influencing the stage discharges relationship at Tuamarina would have been small.

The increase Diversion development from 1975 would have had a small influence on the stage discharge relationship at Tuamarina but this would have been counter balanced by the blocking of the Bothams Bend back channel in 1972.

Since 1983 erosion enlargement of the Diversion channel has considerably outweighed deposition of silt on the berms, with a likely significant influence on the rating relationship at Tuamarina.

(vi) Substantial changes also occurred in the lower Wairau River channel below the confluence, but these changes are much less well documented. The recent 1992 river cross sectional survey and the initial 1957 cross sectional surveys are the only complete surveys of this vital reach of the river down to Ferry Bridge. Cross sectional survey of the lower Wairau below Ferry Bridge is better with cross sections in 1957, 1963, 1967 and 1989.

From the channel bifurcation for the next kilometre downstream the lower Wairau has aggraded by typically 2 metres between 1957 and 1992. Further downstream of this aggradation lessens to typically 1 metre almost all the way to the estuary 10 kilometres downstream. This siltation has occurred since 1967, viz coinciding with increase in flows down the diversion and decreased flows in the lower Wairau.

These changes to the lower Wairau will have decreased the stage discharge relationship of floods at Tuamarina and thus tend to counterbalance the diversion developments effect on the Tuamarina stage discharge rating.

(vii) To summarise; the many changes to the bed and channel at the Tuamarina recorder, in the near downstream reaches and the more distant downstream reaches of the diversion and lower Wairau will have regularly changes the stage discharge rating relationship at the Tuamarina recorder. Changes close to the recorder will primarily influence at small floods, changes further downstream effect the large floods. Deposition in the lower Wairau will have couterbalanced enlargement of the Diversion, expecially in the pre 1983 period.

4.2.2 Flood Gauging

Some 50 gaugings have been taken at the recorder site of flows greater than 100m³/sec up to a maximum of 2300m³/sec. Full details of these gaugings were entered in and recalculated in the RICODA programme and transferred to TIDEDA. The gaugings were taken between 1961 and 1974. Those taken since 1972 were by jet boat usually 200 metres downstream of the recorder. The measured cross section of these gaugings was not that of the recorder site. Those jet boat gaugings are also considered less accurate as jet boat positioning was usually a visual assessment with regard to bridge piers. The other 38 gaugings were taken from the bridge. One gauging was also taken in 1983.

Flood gauging information was also directly provided from simultaneous (or close to simultaneous) gauging of the Diversion at Rarangi Bridge and the lower Wairau at Ferry Bridge. Five gaugings were carried out between 1967 and 1969 and another four gaugings between 1985 and 1990. Those 9 flood gaugings were between 700 and 2100m³/sec.

4.2.3 Flow into the Para Swamp

It has been noted that backflow up the Tuamarina tributary into the Para Swamp can occur. Thomson and Pascoe (1985) for their analysis of the July 1983 flood used a figure of 200m³/sec as a backflow estimate. This was based on a slope area measurement based on silt marks observed after the event. The very flat grade of the water slope, with a measured height difference of only 150mm makes this 'estimate' very imprecise.

In the October 1983 flood F J Bonnington was in the area for much of the time around the flood peak and after. He observed a slight but barely significant backflow up the Tuamarina River.

Backflow has been noted to happen by F J Bonnington in another flood and apparently a flow estimate of approx. 60m³/sec has been made for such a backflow during another flood in the 1970s. This does not appear to have been officially documented.

During a 2500m³/sec flood in October 1972 a flow of 74m³/sec <u>from</u> the Tuamarina into the Wairau is documented as being gauged at very close to the Wairau peak. Four hours later this flow was observed to be zero, as the Tuamarina had dropped faster than the Wairau. The Tuamarina has an estimated 100 year period flood of 300m³/sec and mean annual flood of 80m³/sec.

The Para swamp has a storage volume of roughly 7 million m³ for a 5m rise in Wairau water levels from normal levels to peak flood levels. In a flood hydrograph rise reaching the peak of 30 hours as for the July 1983 event - the average rate of flow absorbed by the swamp would be 80m³/sec. This could be as easily supplied by the Tuamarina as Wairau.

At most times therefore, the flow in the Wairau includes contribution from the Tuamarina River, but at some times there may be a backflow up the Tuamarina into the Para Swamp. This depends on the joint nature of the flood in the Wairau, the flood in the Tuamarina and the hydraulic conditions of the Tuamarina channel. The degree to which these were 'backflows' or 'forward flows' in the Tuamarina River, at the time of the peak of the flood in the Wairau river is unknown for there are no records. Backflow is unlikely to exceed 50m³/sec.

In a statistical analysis of flood flows in the Wairau at the Tuamarina bridge recorder site it is irrelevant as to what tributary inflows are upstream - or even if they are negative inflows. It is certainly not valid to include Tuamarina stream flow estimates for a few selected floods but not include any estimate for other floods.

For design purposes the flood estimates at the Tuamarina Bridge is of direct use for determining the required capacity of the Diversion and lower Wairau immediately downstream. Upstream of Tuamarina bridge design flood estimates will always need adjustment to take into account tributary inflows or outflows.

In summary then, estimates of significant backflows up the Tuamarina in some selected major floods are not relevant to flood frequency analysis. It is also of little value for channel capacity design purposes. Furthermore, such flows are likely to be quite small and not consistent.

Allowances for backflow from the Wairau into the Tuamarina and Para Swamp are **<u>not</u>** included in this review of the Wairau at Tuamarina bridge flood frequency.

4.2.4 Flood Breakouts

For the period of Tuamarina record from 1960 to 1992 there has been some flood breakouts that have bypassed the gauging site.

In July 1983 major breakout occurred, estimated at 800m³/sec and this has been discussed previously in Section 3.3.

The stopbanking overtopping or outflanking in 1962, 1975 and October 1983 was minor, probably totalling less than 50m³/sec and is not specifically included in this analysis.

4.2.5 Analysis

Works Consultancy Services (1993) analysed the gauging data to determine stage discharge rating relationships for the Tuamarina site. This was carried out as a separate exercise in itself as the main basis for determining the rating relationship.

Later WCS (Works Consultancy Services) considered the analysis of the Pukaka and Dicks Road recorder information and the hydraulic backwater anaylsis of other floods to fine tune that initial analysis.

In assessing the stage rating relationships WCS drew up separate stage area and stage velocity curves for the various gaugings. The extrapolation of these curves to high stages can be more precisely judged than for a lumped flow extrapolation.

There has been considerable variation in the waterway area, for as discussed in 3.2.1 the river bed level has varied by up to 0.8m. These variations show little trend with time. The variation is typical of gravel bed rivers as waves of gravel move through. WCS considered that this area variation should be regarded as affecting the stage discharge rating for moderate flows only. At large floods and high stages the influencing control on the stage rating would be further downstream. This downstream bed and channel control will not mirror the area changes measured at the narrow bridge recorded cross section itself.

WCS went as far to say :

"If (stage) rating curve extrapolation is to be based on this (gauging) data alone then the logical approach is to extend into a single curve (at high stages)."

ie, from the gauging data there is no evidence of any differences in high flood ratings (ie above 6 metres), for the record period 1960 - 1993 in spite of Diversion development and many other downstream changes.

It is noted that :

- The highest gauged floods are 2300m³/sec at a 6.2m stage.
- High stage gaugings were only regularly carried out in 1968 - 1974 period.

- There are potential inaccuracies in most gaugings due to lack of data on vertical angle or horizontal angle correction, and in the jet boat gaugings on accuracy of gauging location.

This analysis lead WCS to recommend 12 different stage rating relationship between 1960- and 1990, but these rating relationships had only three changes for the high stage above 6 metre range.

4.3 Diversion and Pukaka

4.3.1 Transcription of Record

WCS transcribed on to computer the 12 years of record from 1968 to 1980 that MDC held on circular Foxborough charts of water levels of the Diversion at the outlet of the Pukaka Stream. This record was then used in conjunction with the lower Wairau record at Dicks Road to give a check on the Tuamarina record.

4.3.2 Site Description

The Pukaka outlet is almost exactly at cross section 8 and some 300 metres upstream of the Rarangi bridge.

The Diversion has steadily and progressively enlarged at this cross section. Regular resurvey has taken place at approximately 2 yearly intervals. Since 1963 the waterway area has enlarged by some 230m² eroding a main channel over 2 metres deep. Over the period 1968 - 1980 period of water level record 140m² of channel enlargement occurred.

The location of the recorder right at the Pukaka outlet has occasionally resulted in instances of high water levels due to Pukaka inflow when the diversion was at a low flow. These occasions are usually quite recognisable.

4.3.3 Flood Gauging and Rating Relationship

Some 37 gaugings above 200m³/sec were taken at Rarangi bridge between 1963 and 1990 which were recalculated in RICODA and transferred to TIDEDA. WCS established a correlation of water levels at Pukaka recorder as being simply Rarangi bridge level + 190mm. For the 1968-1990 period WCS recommended 5 rating curves based on the gauging data. These rating curves were then used with the Pukaka water levels to construct a record of flood stage hydrographs for the site.

To be noted that the five rating curves derived are used to depict what is a regularly changing bed, and limited gauging information was available to determine the shape of the rating curve and the time when the rating curves should apply.

The separately carried out cross sectional surveys show significant bed changes between each of the 7 surveys between 1968 and 1980. Examination of these cross sectional surveys indicates a slightly different shape than the WCS rating curves, that more than five rating curves would be desirable, and to apply over different time periods.

This cross sectional information could therefore be used to refine the WCS proposed ratings. This has not yet been done and the effect may not be great.

4.4 Lower Wairau at Dicks Road

4.4.1 Transcription of Record

WCS transcribed on to computer the 12 years of water level record from 1964 to 1976 that MDC held on circular Foxborough charts of the lower Wairau at Dicks Road.

4.4.2 Site Description

Dicks Road recorder is close to MDC river survey cross section 18 and some 700 metres downstream of Ferry Bridge. The river here is confined to a single 120m wide main channel with no significant berms as the total stopbanked width is 200m. The three cross sectional surveys between 1957 and 1967 show only small bed changes, but the latest cross sectional survey in 1989 shows a rise in mean bed level of 1 metre and a reduction in waterway capacity of over 100 sq. metres.

4.4.3 Ferry Bridge Gaugings and Rating Relationships

The gaugings of flows in excess of 500m³/sec have been taken at Ferry Bridge, but only half these were in the 1964-1976 Dicks Road period of record. A water level correlation between Dicks Road and Ferry Bridge was determined, as 1.103 Dicks Road W.L + 0.036m. Thus the difference is dependent on the actual flow.

WCS derived a single rating curve at Dicks Road to cover the 12 year period. This rating curve was a good agreement with the 1969 backwater model carried out by MDC.

It is recognised that there should be more than one rating curve for the period, but data is too sparse on which to construct further rating relationships.

This rating curve, together with the stage record, allowed hydrographs of all flood levels in the 1964-1976 period to be assessed.

4.5 Combined Analysis

The construction of hydrographs for the lower Wairau and Diversion enabled the combined total to be compared with the Tuamarina record for the 8 year overlay period from 1968 to 1976. This provides a check on the proposed stage rating curve for Tuamarina. Ten floods of between 1900m³/sec and 4000m³/sec were so compared by WCS. The comparison of these showed differences in flood estimates for Tuamarina of -13.0% to +8%.

Two exceptions were at 30/10/68 when there is some doubt about the gauging data, and at 3/10/71 when the Pukaka Stream was in high flow when the diversion itself actually was not.

The -13.0% to +8% difference between the combined Dicks Road and Pukaka record compared to the proposed Tuamarina record was considered a reasonable check and acceptable. The difference is at least as likely to be due to poor definition of the Pukaka and Dicks Road rating curve as the Tuamarina definition. It was noted that the 5 floods recorded in the 1969-70 period compared 10% low while the 3 floods in the 1974-75 period plotted 6% high. This time trend could suggest fast silting up of the lower Wairau channel, and that the mismatch of estimates may be primarily due to reduction in the lower Wairau capacity.

Further examination of the results in the light of the hydraulic backwater analysis results could be carried out. Such further refinement is unlikely to significantly effect recommendations in the flood rating relationships at Tuamarina.

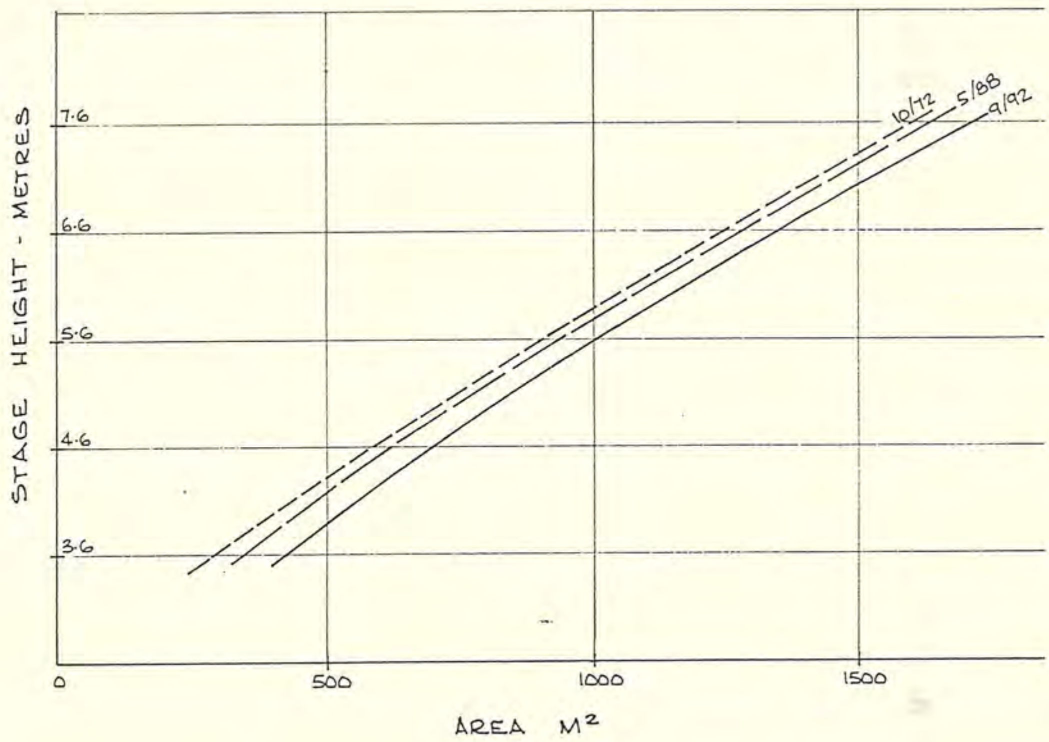


FIGURE 3. STAGE AREA RELATIONSHIPS AT CROSS-SECTION 23·5. 300mtr DOWNSTREAM - TUAMARINA RECORDER.
(For Tuamarina Recorder Add 0·4m)

5. Backwater Analysis of Recorded Flood Levels

5.1 Background

The lower Wairau and Diversion are both generally long reaches of fairly stright uniform channels of moderate velocity on which reasonable backwater flood flow estimates can be made and calibrated against gauged flows.

For the four largest major floods and for a selection of other floods since 1957, the flood peak levels have been measured by pegging and leveling deposited silt marks after the flood. Some other moderately large floods have been simultaneously pegged and gauged during the event. The levels were pegged for some or all of the Diversion, and/or some or all of the lower Wairau. On the occasion that floods were gauged and pegged at the same time has enabled calibration of mathematical computer hydraulic backwater analysis.

W J Noell input all recorded flood levels in the Diversion and lower Wairau to RICODA programme and used a consistent set of metric long section distances for definition.

Hydraulic backwater models were set up for the Diversion and lower Wairau and calibrated by the occasions of gauged flow. Where measurements and estimates have been made for both the lower Wairau and Diversion for the same event, their total gives the Tuamarina recorder flow.

The results are described in an internal memorandum "Estimation of Recorded Flood Sizes in the lower Wairau and the Wairau Diversion by Hydraulic (Backwater) Analysis" by W J Noell October 1992.

5.3 River Cross Section Survey Downstream of Recorder Site

As discussed earlier changes to the cross sections at the recorder site may have little relevance to changes to the stage rating relationships in large floods. Cross section 23.5 some 300 metres downstream of the Tuamarina recorder is the best single cross section indicative of likely changes to stage rating relationships. The three surveys of this cross section from 1972 show that the mean bed level has reduced here by 0.3 metres with a steady increase in waterway area as shown in figure 3. A small increase in flood flows at all stages would be expected since 1972.

5.3 Diversion

Noell set up 15 different hydraulic backwater models, a model for each set of cross section surveys between 1963 and 1990.

All floods were examined and four gauged floods, (two in Aug 1990, one in Oct 1972 and one in September 1970) were the main bases for calibration of Mannings "n" roughness. Three of the floods were over 780m³/sec, being the largest gauged on the Diversion. These three floods all calibrated well using a central channel roughness "n" = 0.021. The calibration of berm roughness was less definitive, and a roughness volume of 0.040 was adopted and used for modeling the important 1983 and 1975 floods.

In the modeling the problem of combining two distinct different zones of the "main channel" and the "berm" was recognised. Ackers (1993) and Smart (1993) have recently shown significant effective increase in hydraulic roughness with such double channels. Some allowances was made for this, but it is an area of possible error.

Seventeen different floods were modelled and assessed, and in most cases each flood was modelled using both the model of the cross sections surveyed prior to the flood, and the model based on sections surveyed after the flood. The recommended assessment was taken as an average of these two.

The models were generally restricted to the 1600 m reach between cross section 8 and 12. Downstream of section 8 the steep slope and resulting high velocities lead to mathematical instability in predicted water levels. Upstream of cross section 12, flood levels have only been measured for a few floods, as it has only been a defined stopbanked channel since 1985.

Until recently the flood slope in this reach from cross sections 8 - 12 was very flat, being in the order of 1:3000. There may also be a small effect of impedence of the Rarangi bridge piers 250m downstream of this reach, especially for the July 1983 flood when the bridge soffitt was just submerged. These backwater flood assessments may therefore have significant error especially for the 1983 floods which are over twice the size of flood flows on which the model was calibrated.

5.4 Lower Wairau

Noell set up four different hydraulic backwater models, a model for each set of survey cross section data.

This deep silt bedded channel calibrated well at a Mannings "n" of 0.021. These calibrations were based on four gauged floods, one in 1957, one in 1963, one in 1967 and one in 1980. The flows of between 500 and 1000m³/sec are much lower than major floods.

A particular problem was assessing the effect of thick willow trees overhanging the main river that have been steadily growing. The thick overhanging branches increasingly impede the flood waters as the water level rises. In 1948 aerial photographs showed such trees as lining 30% of the total bank length. This has increased to 55% in 1972 and 85% in 1991. Little impedence occurred at the moderate flood flows at which the model was calibrated. The resulting increases in hydraulic friction was catered for by assuming an increase in Mannings "n" of .001 for every metre rise in water level for each bank that was heavily tree lined. Thus at major flood levels, where trees lined both banks, the channel Mannings "n" was increased from 0.021 to 0.025. This was a subjective assessment - but strongly based on the reference "Roughness Characteristics of New Zealand Rivers" by Mason and Hicks (1991) in which other NZ willow lined rivers significantly increase Mannings "n" hydraulic roughness with flood level. The only way of confirming this estimate is to peg flood levels in a major flood that is simultaneously gauged. The size of the major floods due to the overhanging willow trees may well be over estimated.

5.5 Overall Results

Good agreement between the MDC backwater modelling and WCS gauging analysis was found for floods of a moderate size. For the highest 3 floods in 1975 and 1983 the backwater modelling estimated higher flood sizes. This could be due to underestimation of Manning's "n" at high flood levels or inaccuracies in flood level measurement, and is discussed in Section 7 in more detail.

(Some recent calculations in June 1993 using the MIKE II hydraulic analysis computer programme were able to mathematically model the steeper Wairau Diversion reach downstream of cross section 8 (Rarangi Bridge), unable to be modelled by the RICODA programme. These calculations indicated the October 1983 flood size as 1900m^3/sec, some 200m^3/sec lower than the previous RICODA backwater analysis. This is in closer agreement to the WCS gauging analysis.)

6. Bothams Bend Area Hydraulic Analysis

6.1 Background

The high stage rating relationship at Tuamarina is influenced by channel control changes significantly downstream, especally the opening of the Diversion as a pilot cut from Bothams Bend in 1963, the blocking off of Bothams Bend back channel and other berm stopbanking in 1972 and the construction of further stopbanks on the berms in 1985.

With the paucity of flow gauging at high stages, and the gauging being concentrated in the 1968-1972 period, the influence of the channel changes is poorly recorded.

Hydraulic analysis was therefore carried out as a guide to the influence these changes would have on the rating relationship at Tuamarina recorder. The area is very complex hydraulically with a bifurcation into two main parallel channels and also four distinct berm areas that carry high flows. The MIKE 11 computer programme capable of network analysis was used to model the situation. The work was carried out by Works Consultancy Services.

The data base was limited to a set of cross sections in 1991 and a set of 1957 cross sections. Two moderate floods in 1990 were documented for model calibration.

The 1957 cross sectional data was used to construct a model, and then used in a comparitive sense by modifications to reflect the changes of :

(i) Diversion opening in 1963.

(ii) Botham Bend channel block and berm banking in 1972.

Similarly the 1991 model was used in a comparitive sense to assess the effect of :

(i) The Bothams island berm stopbanking in 1985.

(ii) Development of the "outlet" into the Diversion from 1983-1991.

The available data was insufficient to quantitatively compare 1957 conditions with 1991 conditions, or in between periods.

6.2 Diversion Opening Change in April 1963

The Mike 11 model (on 1957 data) indicated that the opening of the divesion in 1963 would increase the flow at Tuamarina by 400m^3/sec at the 6 metre stage, and increase the flow by 1000m^3/sec at the 8 metre stage.

In actuality the gauging data indicated little difference in flow at a 6 metre flows stage. It would therefore appear that at the 6 metre stage the downstream diversion opening was at least counterbalanced by gravel deposition closer to recorder.

It is likely that while the flow rating remained much the same, or even decreased slightly over the period at the 6 metre stage, it increased by the order of 500m³/sec at the 8 metre stage.

6.3 Bothams Bend Blocking February 1972

The Mike 11 model (on 1957 data) indicated that the closing of the Botham Bend back channel and other associated banking work would reduce the flow by 200m³/sec at the 6 metre stage and 300m³/sec at the 8 metre stage.

Flow gauging data indicated little if any decrease at the 6 metre stage. The decrease at the 8 metre stage is also only likely to be approx. 100m³/sec.

6.4 Diversion Development 1983-1991

The MIKE 11 hydraulic model had an "outlet" into the Diversion. Stage rating relationships for this outlet were determine by Noell (1992) from his backwater analysis of the Diversion. The MIKE 11 model based mainly on 1991 cross section data but with different "outlet" ratings was used to assess the effect of the Diversion development from 1983 to 1991. This showed that both at the 6m and 8m level an increase in flow of 200m³/sec would be expected. The limited flow gauging data tended to confirm this.

6.5 Bothams Bend Berm Works 1985

The Mike 11 model (on 1991 data) indicated that the berm stopbanking work carried out in 1985 had no effect on flow amount at the 6 metre level, and only a small reduction of 100m³/sec at the 8 metre level. This small reduction will tend to counterbalance the increase due to diversion development from 1983-91 discussed in 6.4.

7. Summary of Recommended Tuamarina Stage Rating Relationships

7.1 Moderate Stage Flood Ratings

Flood flows up to 6 metre stage at Tuamarina (approx 2100m³/sec) can be considered as the moderate flood range. There is a reasonable amount of flood gauging data for this flood range, and Works Consultancy Services analysis of this gauging data is in reasonable agreement with the backwater/slope area analysis carried out by Marlborough District Council. The stage discharge relationships proposed by WCS (1993) are recommended for use.

7.2 High Stage Flood Ratings

At higher flood levels above 6 metres stage channel changes further downstream and on the berm become more apparent. Such changes are above the level at which gaugings have been taken. The MDC backwater analysis and the WCS MIKE 11 comparative hydraulic analysis are useful in assessing high stage rating.

The below table lists the major floods on which backwater analysis information is available, together with the WCS gauging analysis, and the WCS comparative MIKE 11 analysis.

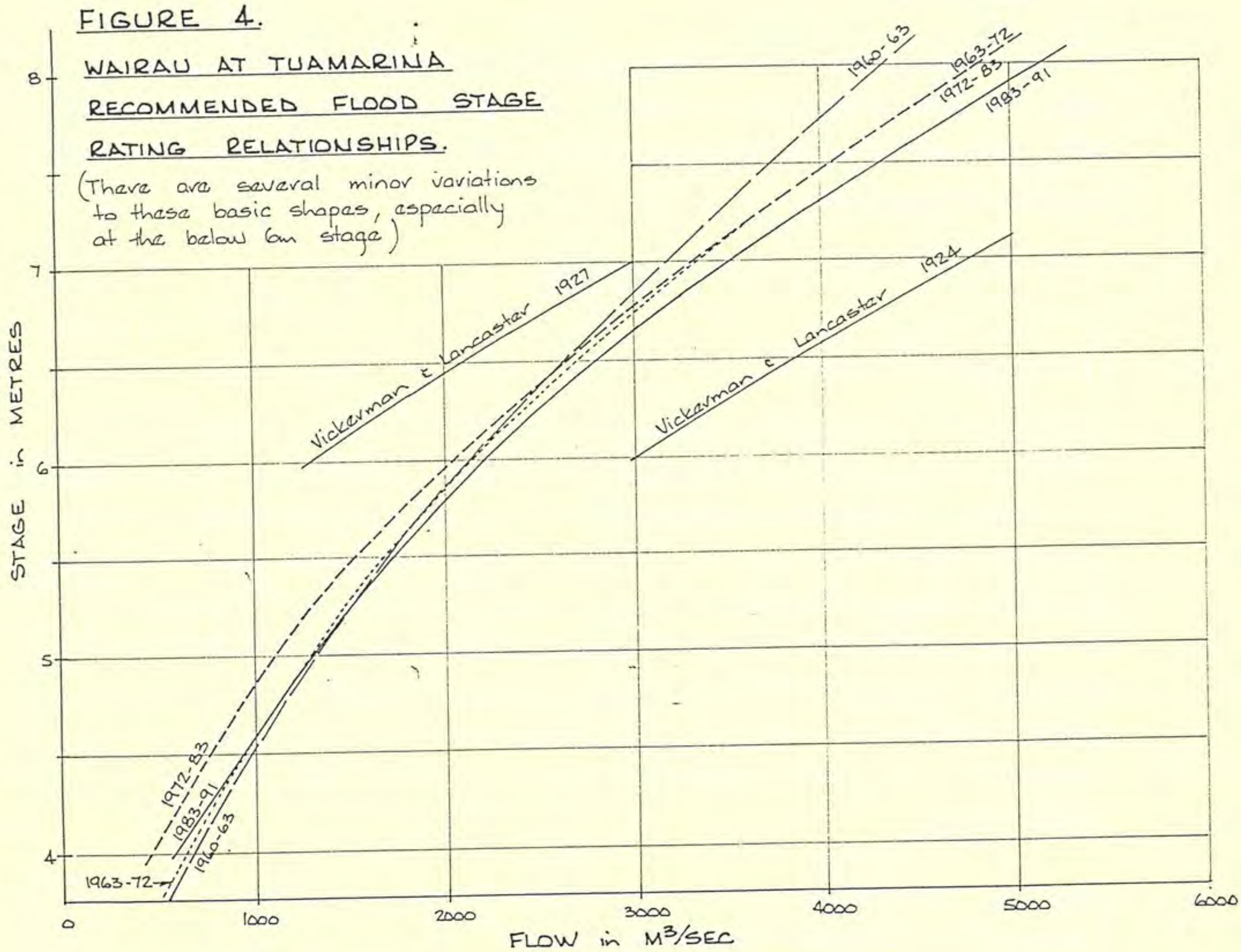
	Stage	WCS Rating Analysis	WCS MIKE 11 Comparative Study	MDC Backwater Analysis	WCS Final	RECOM-MENDED
1/6/62	7.36	3860	-250 = 3600	3600	3650	3620
8/8/63	6.52	2750	As is = 2750	2600	2750	2770
11/8/67	6.57	2790	As is = 2790	2800	2790	2800
17/9/70	6.82	3170	As is = 3170	3150	3130	3160
2/4/75	7.42	3840	-100 = 3750	4300	4000	4000
10/7/83	7.92	4620	+200 = 4800	5400	4700	5000
21/10/83	7.56	4145	+200 = 4350	4800	4330	4400

The final two columns show the overall final recommended values by (i) Works Consultancy Services; and (ii) recommended in this report. The difference reflects the degree of confidence that each group has given to the different information sources. In this report approximately equal weighting has been given to the MDC hydraulic backwater analysis as for the WCS gauging data information. (The limited later hydraulic analysis - see Section 5.5 - is in good agreement with the "recommended" flood sizes).

Four different stage discharge relationships are recommended for higher stages above 6 metres. These are shown as Figure 4 and the "recommended" flood flows above derive from them. The four rating relationships are for :

(i)	1960 - 63	Pre diversion
(ii)	1963 - 72	To Bothams Bend blocking
(iii)	1972 - 83	To diversion development due to 1983 floods
(iv)	1983 - 91	Current situation

There are variations to these, especially below the 6 metre stage level, which have not been shown for clarity. Also plotted (see Section 8.2) are Vickerman and Lancaster assessed 1924 and 1927 flood stage rating relationships.



8. Historical Floods

8.1 Background

Blenheim and the Wairau Floodplain have experienced at least one damaging flood every decade since European Settlement. Prior to 1920 so regularly did floods occur and so ineffective were the flood control works that Blenheim had the name the Beaver or Beavertown. The size of floods was virtually impossible to estimate because of their uncontained and widespread nature. It was however, clearly recognised that the February 1868 was an extremely large flood with water from hill to hill across the plain. The floods of 1904, 1911 and 1916 were also documented as being large and damaging, despite the intensive river works from 1877 to 1902.

The blocking of the entry to the Opawa Channel, the Waihopai Overflow into Gibsons Creek and other river control works in the Conders area constrained the Wairau virtually exclusively to the mainstem Wairau. These works were carried out in the 1914-1919 period. 1920 therefore marks a significant improvement of flood record, with further improvements in 1936 with the establishment of a water level recorder at Ferry Bridge, and 1960 with the Tuamarina recorder.

The effect of virtually all flow down the mainstem Wairau lead to newspaper reporting of the 1926, 1939, 1954, 1975 and July 1983 floods in turn as being the largest Wairau flood since 1868. (This report is later to rank them as (in order) 1983, 1926, 1954, 1939, 1975, and not as being progressively larger).

The floods since 1920 are examined in more detail.

8.2 Vickerman and Lancaster

Vickerman and Lancaster (1924) (consulting engineers to the Wairau River Board) made flood flow estimates based to a large degree on water levels measured at the railway bridges at Tuamarina (mainstem Wairau) and (for prior to 1920) over the Opawa in Blenheim. It is interesting to note that the stage rating relationship that they developed at Tuamarina was substantially different for their later 1927 report than for their 1924 report (See Figure 4). The November 1926 flood had a 7 metre level at Tuamarina railway bridge. The 1927 rating curve showed the flood as 3050 m^3/sec + 1150 m^3/sec overflow totalling 4200 m^3/sec; whereas the 1924 rating would have indicated 4800m^3/sec + 1150m^3/sec overflows, totally 6000m^3/sec. It is the smaller flood estimate that has been quoted by Rae (1987), and Davidson (1959).

The effect of these different flood flow assessments was to make quite different recommendations as to river control works required.

The 1923 flood levels recorded by Vickerman and Lancaster in the upper Conders area were very similar to those observed in July 1983. The 1923 Wairau Flood is now assessed as 3500m³/sec (see later). (The May 1923 Wairau flood unfortunately coincided with extremely large flood in the Taylor/Omaka/Opawa system causing considerable damage. This 1923 event, and the 1868 event are the only 2 known occasions when the weather pattern changed from the ESE to NNW (or vice versa) and brought major floods to both the Wairau floodplain river systems.)

Vickerman and Lancaster's flood estimates for floods prior to 1920 are even less accurate due to the divided nature of the flow in the mainstem Wairau and Opawa.

8.3 1929 Flood

The 1929 flood was not a major flood but it is significant in that flood levels were measured at various recognisable sites on the Wairau river including at Ferry Bridge and Tuamarina railway bridge. It was noted that the flood did not breakout anywhere, but was just at stopbank level for long stretches from Tuamarina downstream. (Overtopping of stopbanks did occur near the river mouth due to the partially blocked nature of the bar.).

This 1929 flood had a level of 6.4 m at Tuamarina bridge and was quoted as being 2000m³/sec - clearly based on the 1927 Vickerman and Lancaster rating curve. The <u>1924</u> V & L rating curve would have indicated 3700m³/sec.

At Ferry Bridge the recorded flood level of 4.63m (15.2 ft) would indicate 2800m³/sec by Davidson (1959) 1936-60 stage rating.

At Tuamarina on todays stage rating a flow of 2700 m³/sec would be indicated. The analysis carried out by WCS in Section 2.2. has shown that the rating curve at Tuamarina Bridge is reasonably stable and the modern rating curves can be used as a guide of these 1920 floods.

This 1929 flood is thus a "benchmark" on which to calibrate floods. The size of this flood is likely to be 2800m³/sec rather than the documented 2000m³/sec.

The estimate of the 1923 and 1926 Wairau floods by Vickerman and Lancaster in 1927 would have thus been underestimated. Using the modern rating curve as a guide the 1926 flood would be 3400m³/sec + 1150m³/sec breakout = 4500m³/sec (up from 4200m³/sec) and the 1923 flood as 3500m³/sec (up from 3000m³/sec). This is also a good agreement with the recognition that at the time the 1929 flood was very much smaller than the 1926 flood.

8.4 1939 and 1954 Floods

A photograph at the Tuamarina rail bridge of the 1939 flood at close to the height of its long peak shows the flood level to be 7.4m. Using the rating curve from Figure 4 as a guide, allowing that the bed level was 0.2 m lower (Noell 1991), and approx 100m³/sec outflowing at Conders leads to a recommended 4000m³/sec for this flood. This is up from the 3500m³/sec by Davidson (1959). However, it is in good agreement with the suggestion in Section 2.4 that Davidsons major flood estimates may be 10% or more low. The extent of stopbank breaching and overtopping including a major 60m breach of the high Bothams Bend bank upstream of the recorder site, plus Conders overflow, would indicate that the 150m³/sec of breakout allowed for by Davidson would also be low.

The 1954 flood estimate of 3800m³/sec by Davidson is similarly likely to be 10% or more underestimated, with a stopbank breach at Tuamarina, a 70m stopbank breach at Morrins Hollow, significant overflows at Conders, and in several places between Renwick bridge and Tuamarina. A flood figure of 4200m³/sec is now recommended. Davidson (1983 personal communication) has stated contemporary estimates of the 1954 flood ranged up to 4360m³/sec. His later observations of the 1962 flood, assessed at 3600m³/sec, lead him to subsequently consider that the 1954 flood had exceeded 4000m³/sec.

The other major floods of the period in 1945 and 1955 may also have been underestimated but there is no firm evidence to suggest other flood sizes than those recommended by Davidson, and his estimates of 3250 and 3400 respectively are kept.

8.5 Floods in New Zealand 1920 - 1953

The Soil Conservation and River Control Council published a book in 1957 that summarised all known floods between 1920 and 1953 in New Zealand. The data source was government files and newspaper reports.

The four largest Marlborough floods of the period had flow estimates as follows :

(i) May 1923. The Wairau was assessed as 4700m³/sec (including overflow estimates) with the river being at 6.8 metres at the rail bridge. (The flow estimate appears based on Vickerman and Lancaster 1924 rating.)

(ii) November 1926. The Wairau was assessed as 3000m³/sec + 1150m³/sec overflows. (based on Vickerman and Lancaster 1927 rating ?).

(iii) December 1939. Not mentioned.

(iv) Jan 1945. Stated as probably larger than the 1939 flood.

Summary : This publication generally confirms previous findings but also shows the inconsistent nature of newspaper flood reports.

9. Regional Flood Characteristics

Regional characteristics of flood frequency are a useful check, particularly if the catchment record appears to have an outlier. Regional flood formulae have been proposed by Beable and McKerchar (1982) who have shown that within hydrologically similar regions a flood of recurrence period Q_i can be directly related to the mean annual flood $\overline{Q}$. The ration of $Q_i/\overline{Q}$ varies from hydrological region to region.

McKerchar and Pearson (1989) have subsequently refined the definition of region by drawing contour lines of the $Q_i/\overline{Q}$ ratio. They demonstrate that the ration of $Q_{100}/\overline{Q}$ for the highest rainfall areas of the West Coast is less than 1.8, while that for the driest East Coast area exceeds 5.

Their plot of contour ratios for the Wairau spans the contour lines of 2.5 and 3.0 and is to quite a coarse scale. The available records can be examined in more detail.

Sub Catchment of Wairau Zone	% of Wairau Flood Flow	Recorder Site	Length of record years	$\overline{Q}$ m^3/sec	Assessed $Q_{100}/\overline{Q}$ for recorder	Average $Q_{100}/\overline{Q}$ ratio for zone	Contribution to $Q_{100}/\overline{Q}$ for Wairau at Tuamarina
Richmond Range Northbank	24%	Motueka Wairoa Pelorus Collins	18 29 9 26	292 707 377 31	2.72 2.35 2.3 2.84	2.55	0.61
Upper Wairau	16%	Dip Flat	32	342	2.32	2.32	0.37
Branch	22%	Branch	20	464	2.45	2.45	0.54
Waihopai	20%	Craiglochart	30	430	2.43	2.43	0.49
Other southern bank tributaries	18%	Waihopai at Craiglochart Taylor at Weir	30 32	430 68	2.43 4.00	3.0	0.54
				Total $Q_{100}/\overline{Q}$ For Wairau at Tuamarina			2.55

The expected ratio of $Q_{100}/\overline{Q}$ for the total Wairau from this use of adjacent regional records is thus 2.55. Some or all of these records may be no better or even less reliable than for the Wairau at Tuamarina. These records could be considered as cross checks of each other. The Waihopai and

Branch water level records are both known for significant bed movement at high stages and uncertainty of their major flood stage rating relationships.

It would be expected that the $Q_{100}/\overline{Q}$ ratio would increase from the wetter northern Richmond Range area to the drier Southern Waihopai subcatchment. The actual records do not show this, with all the assessed $Q_{100}/\overline{Q}$ ratios except the hydrologically different Taylor, plotting randomly between 2.3 and 2.8. This may be due to errors in those other regional river records.

This regional records analysis is certainly adequate to indicate that the expected ratio to be is the range of 2.45 to 2.7.

As the $\overline{Q}$ for the Wairau at Tuamarina is shown from Section 5 to be 2100m³/sec, this regional rivers analysis check would indicate a Q_{100} flood as being 2.55 x 2100 = 5350m³/sec.

10 Flood Frequency Analysis

10.1 Recorded Annual Peaks

The highest annual flood peaks at Tuamarina recorder as determined by the recommended stage rating relationships in Section 7 are :

DATE	STAGE	FLOW	RANKING (1960 - 1991)	RANKING (1920 - 1991)
05.08.60	4.63	1070	27	
06.03.61	4.55	1020	28	
01.06.62	7.36	3620	4	7
08.07.63	6.52	2770	7	
22.11.64	5.44	1640	19	
19.11.65	4.58	1080	26	
26.04.66	5.00	1300	23	
08.08.67	6.57	2800	6	
10.04.68	6.50	2700	8	
11.09.69	5.91	2130	13	
17.09.70	6.82	3160	5	11
04.10.71	5.70	1700	18	
09.10.72	6.32	2420	11	
12.08.73	4.50	740	31	
06.04.74	6.42	2550	9	
02.04.75	7.42	4000	3	5=
07.12.76	6.14	2230	13	
29.06.77	5.35	1240	24	
14.05.78	5.52	1430	20	
07.05.79	5.12	1010	29	
17.08.80	5.06	970	30	
06.10.81	5.31	1200	23	
04.06.82	4.58	640	32	
10.07.83♦	7.92	5000	1	1
22.10.83*	7.56	4400	2	3
16.12.84	5.80	2010 -	16	
20.04.85	5.88	2090	14	
25.01.86	5.80	2010	15	
20.01.87	5.05	1360	21	
24.07.88	6.27	2530	10	
11.06.89	5.60	1840	17	
12.08.90	6.01	2210	12	
02.09.91	4.71	1080	25	

- Breakout and overflows of 800m³/sec need to be added to the July 1983 flood, thus giving it a total of 5800m³/sec.

* The October 83 flood is also included because of its significance.

Estimates of other major floods in the 72 year 1920 to 1991 period are :

Date	Flow	Ranking 1920-1991
May 1923	3500	8
Nov 1926	4500	2nd
Dec 1939	4000	5th =
Jun 1954	4200	4th
Feb 1955	3400	9
Jan 1945	3250	10

10.2 Changing Calendar Year

The method of plotting annual flood peaks and fitting a Gumbel distribution to them is valid for any "year" period chosen. Traditionally, for simplicity, the Jan - Dec calendar year has been accepted.

WCS carried out the analysis survey also using a Sept - Aug year. In this way the major October 1983 flood was included in the analysis as it then lies in a separate analysis year from the July 1983 flood. The sets of moderately large floods in the 1970 and 1967 periods were also then included in two separate years.

The overall result was that the Sept - Aug year recorded higher annual floods, with the mean annual floods averaging 2200m³/sec compared to 2010 using the Jan - Dec 'year'.

As both "years" are equally valid, a mean annual flood of 2100m³/sec is recommended.

10.3 Distribution Plot

The flood peak data of Section 10.1 was plotted up as a Gumbel (EVI) distribution, using the Gringorten plotting position method to which a line was fitted by eye. This is shown in Figure 5.

Data from two sets was included. For flows above 3000m³/sec the 72 year data base from 1920 - 1991 was used including both 1983 floods. (Justified on the assumption of a Sept-Aug year). For flows below 3000m³/sec only the 32 year detailed Tuamarina record is included.

Also plotted is the WCS recommendations for a "Jan - Dec year" and a "Sept - Aug year". The data base is only for Tuamarina 1960 - 1991 and slightly different flow estimates were used for the 2 major 1983 and 1975 peaks (as discussed in Section 7).

Key features of the plot are :

- Good fitting of the data to a Gumbel plot.
- The July 1983 flood not plotting as a outlier.
- The 1 in 100 year return period flood survey shown as 5500m³/sec.
- The October 1983 flood plotting as a 1 in 30 year return period.
- The 1 in 2.33 return flood plotting as 2100m³/sec, ie, the mean annual flood.
- Good agreement with WCS analysis based only on Tuamarina 1960-1991 record.

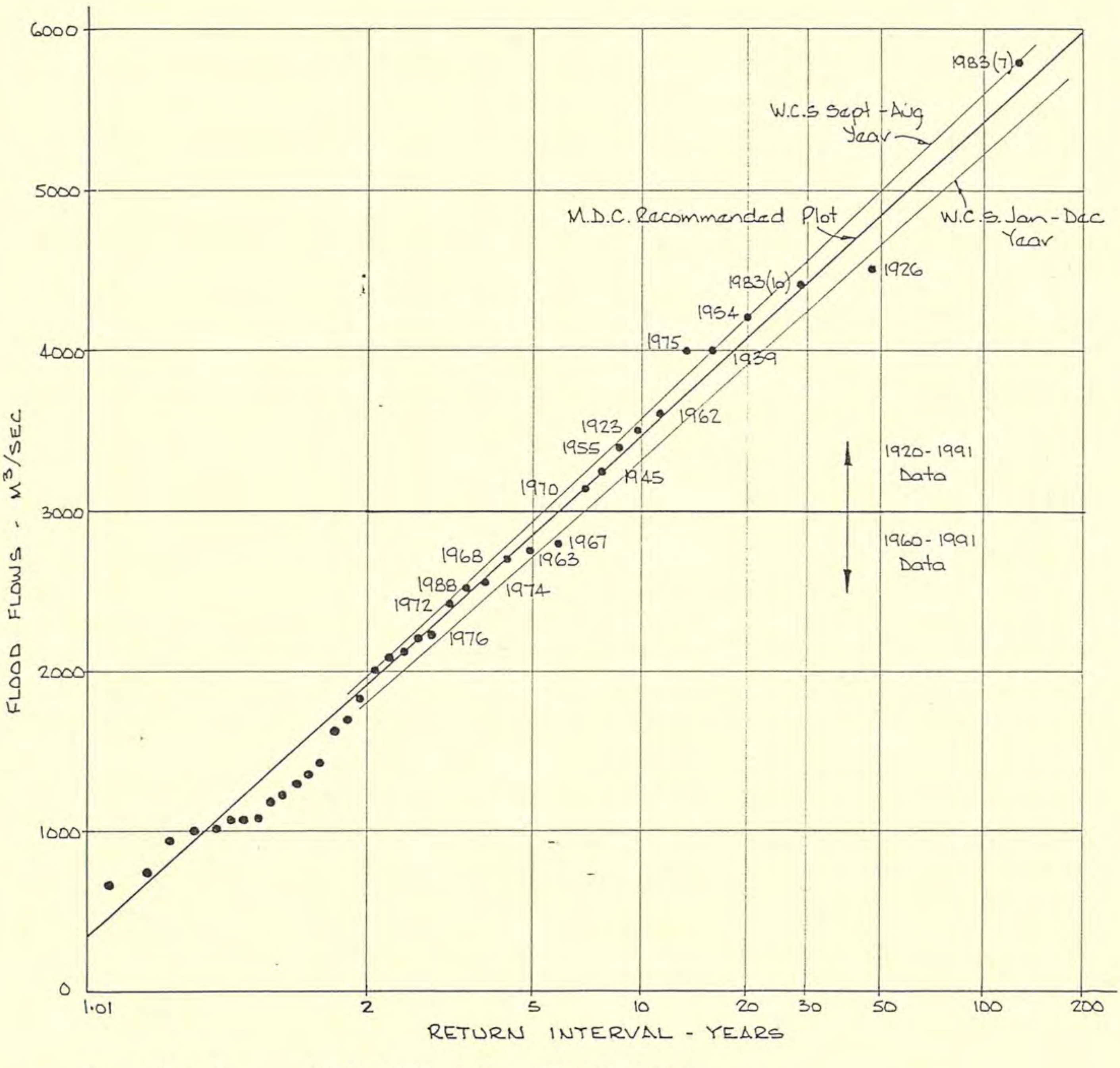


FIGURE 5. WAIRAU AT TUAMARINA.
FLOOD FREQUENCY ANALYSIS 1920-1991

11. Conclusions

The main findings of this report are :

- The 1 in 100 year return period flood (ie a flood having a 10% chance of occurring in the next 10 years) is assessed as being 5500m³/sec.

- The July 1983 flood is assessed at 5800m³/sec. as a 1 in 150 year return period event, and the October 1983 flood as 4400m³/sec. being a 1 in 30 year event. (NB: The October 1983 flood has been used as a design basis for Wairau River Control Works).

- The 1960 - 1991 record from the Tuamarina water level recorder was the main basis for analysis, to which stage rating relationships were derived from gauging data and backwater hydraulic calculations of monitored flood levels in the Diversion and lower Wairau.

- This analysis was supplemented and in good agreement with a review of the less accurately recorded historical flood data from 1920 to 1960. Prior to 1920 flood data was too inaccurate for use.

- An examination of regional flood frequency parameters of 8 subcatchments or adjacent catchment recorders was a confirmatory check.

- Works Consultancy Services, independantly analysing the information, came up with similar conclusions with a recommendation that the July 1983 flood was 5500m³/sec. being a 115 year return period flood, and 4300m³/sec for October 1983 flood.

- The 1 in 100 year return period flood recommended is larger than previous recommendations of earlier reports, which varied between 4200 and 5200m³/sec.

- Rainfall and snowmelt analysis confirm that there is no evidence to indicate that the July 1983 flood was of an extraordinary nature of extreme rarity.

- Stage rating relationships at the Tuamarina recorder regularly change. This is primarily due to waves of gravel moving through the bridge recorder site, with gravel deposition in the reaches shortly downstream of the channel bifurcation and gravel extraction.

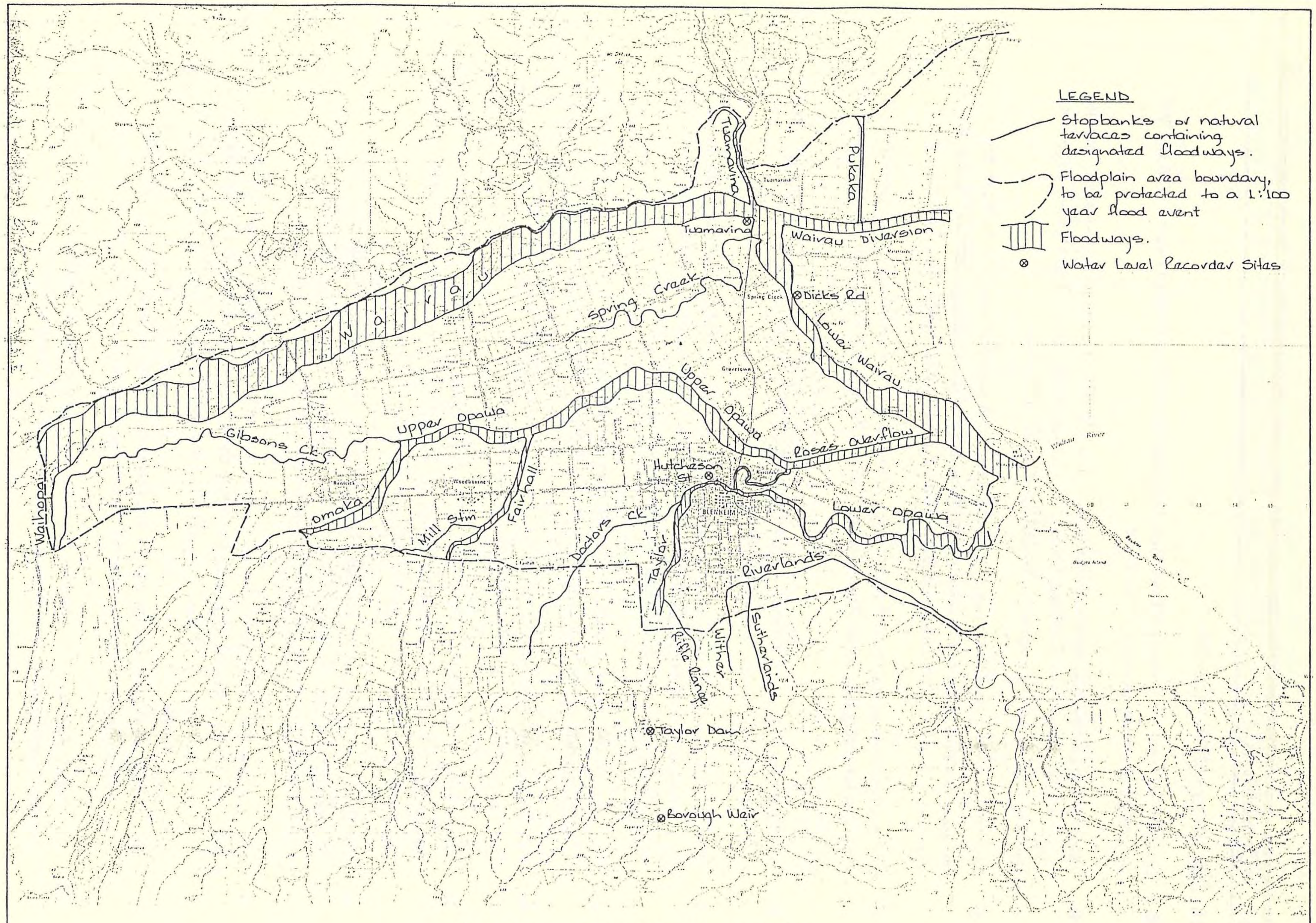
- The Diversion development has had a lesser effect on the stage rating relationships at Tuamarina. It has also been counterbalanced by gravel and silt deposition in the lower Wairau, and is limited to very high stages of major floods.

- The contemporary overestimation of floods in the 1960-1988 period was probably due to :
  - (i) A presumption of significant change to the stage rating relationship of the Tuamarina recorder due to Diversion construction and development - which has not been realised in actuality.
  - (ii) A presumption that the capacity of the lower Wairau channel remained the same - which has not been the case due to substantial siltation and also willow tree growth.

- The analysis has been very useful in providing an insight of the hydraulics of the bifurcation into the Diversion and lower Wairau, and of the State Highway bridge.

- That above a stage height of 7.7 m at Tuamarina, (approx 4800m³/sec), the State Highway bridge soffitt impedes the flow and progressively causes even higher flood levels.

FIGURE 7: Wairau Floodplain Rivers



WAIRAU FLOODPLAIN RIVERS.

1: 100,000.

SECTION B : TAYLOR AND OTHER WAIRAU FLOODPLAIN TRIBUTARIES

1. Taylor

1.1 Background

The Taylor is a major tributary of the Opawa, which joins the Wairau at its mouth. Flood analysis of this river is required directly for review of the adequacy of the Taylor detention dam design. It is the main basis for the stopbanked lower Opawa - Taylor - Doctors Creek system and downstream and is furthermore a good indicator of frequency analysis for other adjacent tributaries flowing on to the Wairau floodplain, and on which river control works are constructed. These other tributaries of Doctors Creek, upper Opawa, Fairhall, Wither Hills streams, Omaka and Pukaka are affected by the same south easterly storms as the Taylor. These other rivers are also stopbanked, again desirably to contain a flood with a 10% chance of occuring in the next ten years (100 year return period). Figure 7 shows the layout of the Wairau Floodplain tributaries.

In 1961 a water level recorder was established on the Taylor at the Borough weir 3km above the dam site. The catchment area above the recorder is 64km², at the Taylor Dam 76.5km² and just upstream of Blenheim 83km². This recorder has been operated continuously since 1961 and is the best data source for flood information on the Wairau floodplain tributaries.

1.2 Taylor Dam Design Intital Flood Estimation

In the initial scheme report Davidson (1959) had to rely on records of slope area assessments of historical floods, particularly those recorded by Vickerman and Lancaster (1927). The May 1923 flood was assessed as 620m³/sec, and other major floods of 1911 as 420m³/sec, 1929 as 280m³/sec and 1948 as 280m³/sec. (These estimates being upstream of town). The May 1923 flood size of 620m³/sec was recommended as the detention dam design flood peak.

By the time the Taylor Dam was being constructed in 1963, some information was available from the Borough weir recorder for 1961 and 1962 floods which together with unit hydrograph and other empirical methods lead Davidson (1963) to recommend a reduced design flood peak of 500m³/sec, with a flood hydrograph having a volume of 19 million m³ over the 25 hours in which it exceeded 40m³/sec.

During construction a significant flood occured in July 1963. Flood levels were also monitored through Blenheim. The Blenheim flood levels were substantially higher than expected for the size of flood measured at the Borough recorder. This indicated that the slope area estimates of the 1948 and other earlier floods would have been overestimated and that the July 1948 flood was more likely to have been 200m³/sec. The May 1923 flood was also considered to have been overestimated previously.

A revised design peak of 425m^3/sec was therefore adopted for the dam construction design with a hydrograph volume of 16 mllion m^3 over 24 the hours in which it exceeded 40m^3/sec. (40m^3/sec marks the dam culvert capacity above which storage in the dam begins to occur).

1.3 Borough Weir Record

The water level recorder established in 1961 at the Borough Weir has a low weir to stabilise the stage discharge relationship.

Essentially two rating curves apply to the site; 1961 to 1966 and 1966 to 1991. These dates correspond to changes to the weir construction. There have been some flood gaugings taken from the gauging bridge constructed immediately downstream, and correlated with several slope area flood assessments over the same reach. The majority of these gaugings and slope area assessments were carried out in the 1962-1966 period. One high stage flood gauging of 98m^3/sec was carried out in 1977. The recorded velocities in excess of 4m^3/sec limit the accuracy of this important gauging. No high stage gaugings have been carried out since.

The amount of gaugings to confirm the high stage ratings could clearly be improved. The available documented flood record however, appears in reasonable agreement with other measurements of water levels in the dam during flood events. The stage rating relationship will be presumed to be correct in this report. Further gaugings of flood events in the future for better definition of the flood stage rating is required.

1.4 Analysis of Borough Weir Record

This was analysed by Thomson (1980) as the basis for reducing the size of the dam culvert outlet. On the basis of the 18 years of data he assessed the 100 year return interval flood as 232m^3/sec at the recorder, and allowing for increased area down to the dam, 270m^3/sec was recommended as the dam inflow.

Thomson discounted the value of the May 1923 slope area estimate due to the hydraulic complexity of the channels through Blenheim with Omaka, Fairhall and Doctors Creek floodwater contributing as well as the Taylor; overflows in several places and very flat flood slopes.

Rae (1987) with a further 7 years of data (a-period of no significant flood) assessed the mean annual flood $\bar{Q}$ as 62m^3/sec and the 100 year return period flood as 223m^3/sec.

Most recently Neilson (1993) with a further six years of data has assessed the 100 year return period flood as 245m^3/sec and mean annual flood $\bar{Q}$ as 58m^3/sec. This extra six years of flood records included the largest ever recorded flood of 196m^3/sec in 1989 in an otherwise fairly flood free record. This 1989 flood peak has an assessed return period of 35 years. At the Taylor dam the flood flow would be expected to be 17% higher.

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Neilson also examined the volume of the hydrograph for a 100 year return period event and assessed this as 10 million m^3. The time period was assessed conservatively as 24 hours for a flow above $40m^3/sec$. The lesser the time period the more effect it has on the dam performance. This is shown by comparing the dam performance for the 1963 and 1989 floods. These floods respectively had flood volumes of 8.6 million m^3 over 31 hours and 8.1 million m^3 over 20 hours (above $40m^3/sec$). The storage level for the 1989 flood reached 64 metres R.L. behind the dam, representing 35% of dam storage. The larger 1963 flood would have recorded 62.5 metres R.L. representing 20% of dam storage. In this context of stored flood volume the floods are considered to be 25 year and 15 year return period frequencies.

1.5 Probable Maximum Flood

Davidson (1963) routed a short peaked "catastrophic" flood of $1700m^3/sec$ peak but only of short duration through the dam, using the original design intention outlet culvert capacity of $220m^3/sec$.

Terminology for "catastrophic" flood which a dam should safely cope with has now changed to "probable maximum flood" or PMF for short. Developments in the assessment of PMF's have shown that it is not a precise figure and that a range of values can be validly calculated.

NIWA (1993) has calculated several probable maximum flood scenarios for the Taylor as between 485 and $1545m^3/sec$, depending on length of rainstorm and assessed antecedant soil moisture conditions. An extreme case scenario for the dam is for a 12 hour storm and a peak of $1019m^3/sec$ and a flood volume of 21 million m^3, over a 15 hour period above $40m^3/sec$. An alternative, perhaps more realistic PMF scenario that NIWA examined has a flood peak of $658m^3/sec$ over a 21 hour period and a flood volume of 17 million m^3. The hydraulics of the spillway to pass this PMF total are examined separately in Appendix II.

2. Lower Opawa - Taylor - Doctors Creek

2.1 Background

The Taylor is a major tributary to the lower Opawa. Prior to 1967 the Opawa Loop joined the upper Opawa - Rose's Overflow system to the lower Opawa - Taylor System. It was a very complex hydraulic system with water able to flow either way in the Opawa Loop. Both the lower Opawa and Rose's Overflow channel took flood flows from both the Taylor and upper Opawa. The proportion that each channel took would depend on the timing and size of the flood hydrographs from the Taylor and upper Opawa inflows.

Channel blocks were constructed across the Opawa Loop through Blenheim in 1967 which has since prevented this from occurring. The Lower Opawa - Taylor is now a separate river system (except for low flows) from the upper Opawa - Rose's Overflow system.

Recorded flood levels prior to 1967 in the lower Opawa are now of little relevance to flood levels since or that will occur in the future. This also applies to the Taylor River through Blenheim which is strongly affected by the downstream "backwater" levels of the lower Opawa.

Doctors Creek is a major tributary of the Taylor, joining it on the western edge of Blenheim. Rifle Range Creek is a small diversion from the Wither Hills which with other small tributaries join the Taylor between the dam and Blenheim.

Since 1965 the Taylor flows are those from the outlet culvert of the dam, having been damped by the storage behind the dam.

2.2 Taylor Dam Outlet Culvert

The Taylor detention dam dampens the flood peak by storing flood waters and spreading the flood hydrograph over a longer period. The size of the outlet culvert is a major factor in determining the outflow from the dam. The initial size of the outlet culvert was such that a flow of 170m^3/sec was expected when the dam was full and the water level reached the spillway crest level.

Following the 1980 flood (when stopbanks were overtopped downstream even though the dam was only a quarter full) a review of the desirable outlet culvert size was carried out by Thomson (1980). This review lead to an orifice plate being installed to throttle the culvert outlet. Woolley (1980) had a design intention that the culvert overflow to be 113m^3/sec at dam full (spillway) level.

Recently the hydraulics of the dam have been reviewed for a 100 year return period flood event. Neilson (1993) calculates the water level in the dam to be 2.5 metres below dam full (spillway) level with an outflow of 105m^3/sec. (Presuming Woolleys culvert outlet relationship is correct). Figure 8 shows expected inflow and outflow hydrographs in routing a 100 year return period flood event through the dam and also the 1989 flood record.

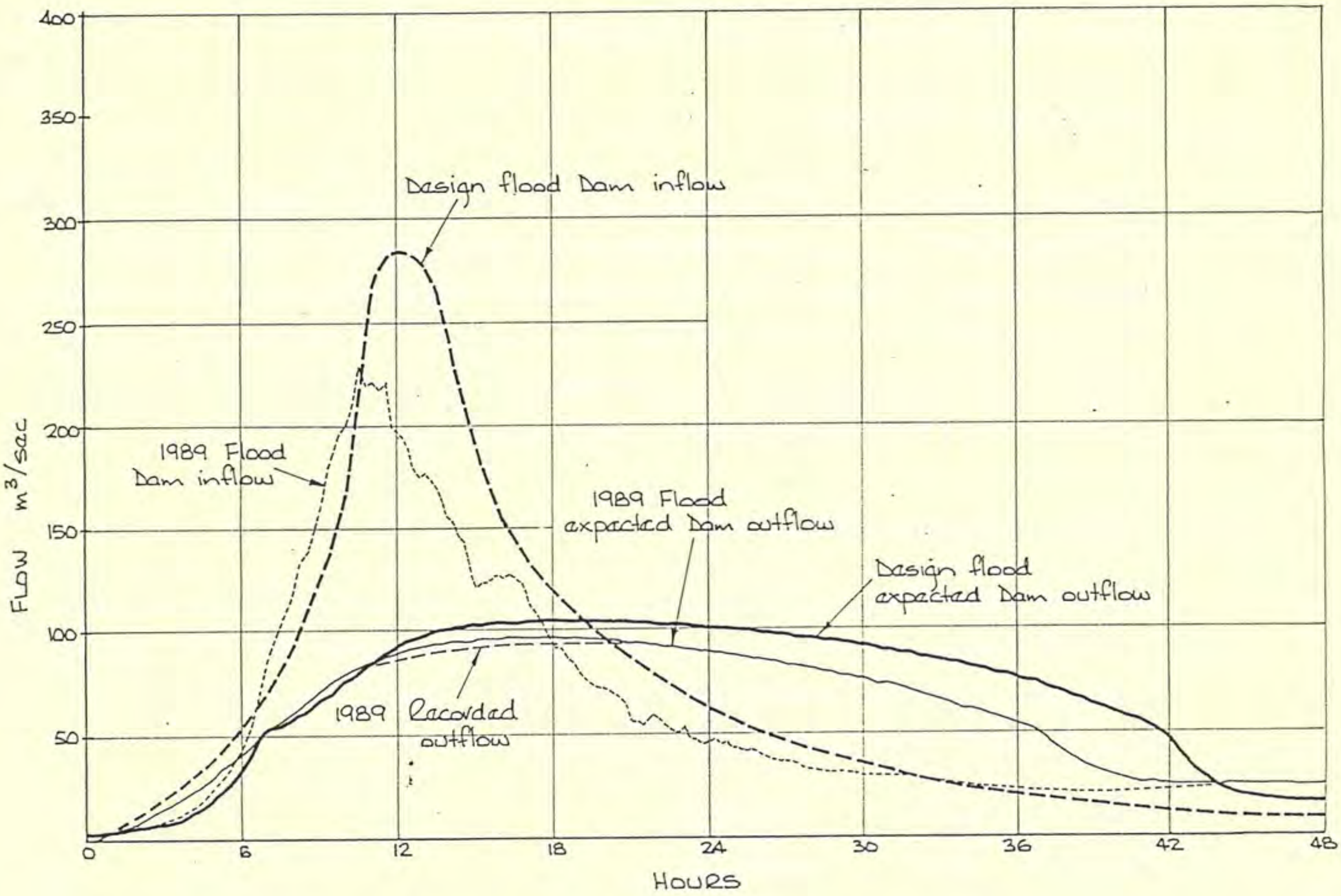

FIGURE 8. TAYLOR DAM FLOOD INFLOWS & OUTFLOWS.

(Note: Dam inflow shown, is measured Borough Weir Recorder flow × 1.17)

2.3 Doctors Creek

This is a major tributary of the Taylor. It has a hill catchment size of 24km², some 38% of the Taylor at Borough weir. The Taylor record can be used as the best estimates of flood frequency for Doctors Creek.

Storm rainfall is assessed as being 70% of Taylor rainfall from available records.

Beable and McKerchar (1982) showed that hydrologically similar catchments in the Marlborough area have flood flows proportional to $(Area)^{0.9}$.

0.25 x 1.17

On this basis Doctors Creek flood would be 29% of Taylor at Borough weir. A 100 year return period event is thus estimated as 70m³/sec. This estimate is identical with the earlier scheme estimate. It is also in good agreement with slope area assessment of 24, 34 and 41m³/sec of Doctors Creek at Purkiss Street in 1971, 1967 and 1966 respectively. The 1966 flood event had a return period of 20 years for the adjacent Taylor and approx. 50 years for the Omaka.

Before joining the Taylor, Doctors Creek spreads out over a large low lying area in the Batty's Road - Bells Road area. The natural storage ponding has been calculated to reduce the peak of the flood by approx $15m^3/sec$. The resulting inflow of a Doctors Creek one in a 100 year flood event into the Taylor is estimated as $55m^3/sec$.

2.4 Other Taylor Tributaries

Between the Taylor dam and Blenheim various small hill tributaries totalling $7km^2$ in area join the Taylor River. The most significant one, with $13.5km^2$ area is Rifle Range Creek. Davidson Ayson (1991) using the rational formula and rainfall intensity records estimated a 100 year event for this stream as $12m^3/sec$. This is in very good agreement with using the Taylor record, adjusting by $(Area)^{0.9}$ and allowing for a reduced 70% rainfall. The other hill tributaries total a similar area and these could also contribute approx $12m^3/sec$.

2.5 Taylor at Hutcheson Street Bridge

Although a water level recorder has not been installed on the Taylor at Hutcheson Street bridge in Blenheim, major floods have been gauged at this convenient site. Gaugings have been taken at or close to the flood peak of large floods. The peak flood levels were also pegged down the lower Opawa River. These flood peak sizes were :

May	1966	$150m^3/sec$
October	1971	$120m^3/sec$
July	1977	$110m^3/sec$
April	1980	$130m^3/sec$
Sept	1989	$100m^3/sec$

The record has proved very useful for calibration of the hydraulic backwater analysis of the lower Opawa - Taylor system.

It is less useful for flood frequency analysis. The record is limited to the largest floods only and is moreover of three frequency distributions - post and prior to 1981 when the Taylor Dam culvert was throttled, and also post and prior 1967 when channel blocks were put across the Opawa Loop.

It does have the advantage of directly measuring the Taylor - lower Opawa at the very relevant site of central Blenheim downstream of the Doctors Creek inflow.

2.6 Whole Lower Opawa - Taylor - Doctors Creek System

The Taylor River through town has flow contribution from tributaries whose individually assessed 100 year flood events are Taylor dam outflow 105, Doctors Creek 55, Rifle Range Creek12, other streams $12m^3/sec$.

Direct addition of these individual contributing tributaries would indicate a 100 year return flood event of 185m³/sec for the Taylor River through Blenheim and down the lower Opawa. However, the timing of the flood peak is likely to be different and so such a calculation is conservative. The timing of Doctors Creek flood peak can be earlier or later, and has been recorded as coincident, depending on the rainstorm pattern.

More importantly the severity of the Doctors Creek flood can be quite different from the Taylor.

This can be shown by examining the 1967 and 1989 floods.

In 1989 the Taylor and Borough weir had a flood of 196m³/sec, the highest on record, while Doctors Creek had only a mild flood - from aerial photographs examination approx. 10m³/sec - only 5% of the Taylor flow.

In 1967 the Taylor had an 85m³/sec flood, while Doctors Creek was assessed at 34m³/sec, some 40% of the Taylor flow.

The 1980 flood of 130m³/sec overtopped stopbanks of the lower Opawa and Taylor and as a result the dam culvert outlet was throttled down later that year. The flood peak has been reduced by some 50m³/sec. The flood peak is over a longer time period so that the chance of coincidence with Doctors Creek peak is more likely.

Overall 170m³/sec is recommended as a 100 year return period flood for the lower Opawa - Taylor. This estimate is based on the available data and it is suggested that the design flood will be between 150m³/sec and 200m³/sec. The most relevant data is only that since the dam culvert throttling after the 1980 flood. The 130m³/sec flood of 1980 is assessed as having a 1 in 20 year return period event under present conditions (a 40% chance of occurring in the next 10 years).

Considerably more data collection is desirable in future years to check and validate this flood peak. In addition to the Borough weir recorder useful data will be obtained from the water level recorder recently established in the Taylor dam. A new recorder is also recommended to be installed on the Taylor at Hutcheson Street bridge. The determination of Doctors Creek and other tributary inflows could then be assessed from examination of these 3 Taylor based recorders. There is no suitable water level recorder site on Doctors Creek.

3. Upper Opawa - Rose's Overflow

The Upper Opawa was formerly a distributary branch of the Wairau with estimated flood flows of up to 2000m^3/sec. Rose's Overflow diversion was constructed in 1902 to lead much of these flood waters directly to the Wairau estuary and bypass the Opawa Loop (Blenheim) and the lower Opawa.

Since the blocking of the Opawa breach in the 1910's much smaller overflows from the Wairau have come down the upper Opawa in large floods. With the exception of July 1983, these overflows were considered minor at the time. Given the intensive viticultural development in this Upper Opawa overflow channel area over the last 10 years such overflows may not be considered minor by today's landowners!

More recent and expected future river control works in the Conders area will further reduce the chance of flood waters from the Wairau except in extremely rare events. The upper Opawa - Rose's Overflow system need only be designed for inflows from its tributaries, the Omaka and Fairhall. Interestingly enough these are not natural tributaries but diversions in 1878 and 1930 respectively. (Formerly the Omaka and Fairhall joined together and combined with the Taylor near Doctors Creek confluence on the western edge of Blenheim). The Gibson's Creek tributary will have no significant contribution in flood time.

The Taylor at Borough weir flood frequency analysis is a major basis for estimation of the flood size for the 106km^2 Omaka catchment. Assuming similar storm rainfall intensities, and that flood runoff is proportional to (Area)$^{0.9}$, the 100 year flood event would be 58% larger than for the Taylor ie, 380m^3/sec.

Slope area estimates of the Omaka in major floods have been taken at Tyntesfield gorge of 320m^3/sec in 1966 and 200m^3/sec in 1963; and of 250m^3/sec below State Highway 6 bridge in 1989. These estimates are in good agreement with the above Taylor recorder based estimate of 380m^3/sec for a 100 year event.

For the 73km^2 Fairhall catchment, with rainfall assessed at 80% of the Taylor catchment, leads to a recommendation of 210m^3/sec for that river. This is only half the 420m^3/sec flood size used for design of the Fairhall diversion in 1930. It is not surprising that there is no record of Fairhall stopbanks being inadequate.

The upper Opawa below Fairhall diversion would therefore have a 100 year return period flood peak of nearly 600m^3/sec.

Backwater calculations of pegged flood levels were analysed by Williman (1989) for floods in 1966 of 400m^3/sec, and 1989 of 330m^3/sec. The flood levels were recorded over a 3 km length of the uniform Rose's Overflow and upper Opawa below State Highway 1 bridge. These backwater estimates of the two largest floods known on the upper Opawa since 1960 are in good agreement with the Taylor recorder based flood estimates.

The possible diversion of the majority of Doctors Creek floods into the Fairhall would increase that design flood estimate to 250m³/sec, which it should handle quite easily. The resultant design flood for the upper Opawa to slightly greater than 600m³/sec.

The comprehensive stopbanking system for the Omaka and Upper Opawa - Rose's Overflow has only been constructed since 1980 and the design flood flows have been 400m³/sec and 600m³/sec respectively, in line with recommended flood sizes as above.

4. Riverlands Co-op Drain and Wither Hills Streams

The Riverlands Co-operative Drain was originally constructed as a drain for the flat land to the east of Blenheim. If takes runoff from the streams draining the Wither Hills directly to the Vernon lagoons. These include the Wither Stream and Sutherlands Stream running through the southern part of Blenheim, and Mapps waterway draining hill catchments further east. The final tributary, Fifteen Valley, only joins just before the Vernon Lagoons.

In the scheme report of Davidson (1959) flood flows were estimated as $9m^3/sec$. for the combined Sutherland and Wither Streams runoff, increasing to $20m^3/sec$. for the bottom reaches. The Wither Stream itself was enlarged to carry a design $3.8m^3/sec$..

The 1980 flood showed the lack of capacity of the waterway in the upper reaches, and a decision was later made to upgrade the system, and was called Riverlands floodway. Fitzgerald and Carr (1987) used $12.5m^3/sec$ as the 50 year return period design flood.

Continuing increased urbanisation also lead to improved standards of works for Sutherlands Streams and Wither Stream. Fitzgerald and Thomson (1979) recommended a $10m^3/sec$ for Sutherlands Stream, a 100 year return period design and Davidson Ayson (1991) $5m^3/sec$ for Wither Hills Stream at its confluence with the Riverlands floodway. This latter was for a 50 year return period flood and was agreed on by Williman (1993) in recommending approval of a resource consent for the proposed works.

It should be noted that there is no water level recorder for this floodway or its tributary streams. The flood estimates by Fitzgerald and Davidson were based on rainfall measurements and runoff coefficients (the rational formula). The checking by Williman was based on the Taylor River recorder record, and allowing that flood flow was proportional to $(Area)^{0.9}$ but reduced to 70% of this due to lower storm rainfall. The two methods showed good agreement.

For a 100 year return period event the combined Sutherlands Stream and Wither Hills Stream runoff would now be estimated as $14m^3/sec$, in good agreement with the $12.5m^3/sec$ 50 year design for the Riverlands floodway. The floodway, still under construction, is likely to be capable of carrying this 100 year return period flood. This hydraulic problem is discussed in Appendix II.

5. Pukaka

The Pukaka has a 23km² hill catchment and is a significant stopbanked floodplain tributary that is affected by south easterly rainstorms as well as northerly ones.

The original scheme design for a 1 in 100 year return period event was 48m³/sec. Regular overtopping of the stopbanks lead to re-evaluation of its flood sizes. Pascoe (1977) made a slope area assessment of the March 1975 (Cylone Allison) flood as 82m³/sec, and a flood ayear later only slightly smaller size.

Wadsworth and Thomson (1986) estimated a 100 year flood return event of as 85m³/sec, based on regional flood frequency methods.

Based on the Taylor at Borough weir, a 100 year return period event would be 110m³/sec. Slope area assessment of the adjacent Waitohi Stream Picton have lead Davidson (1983 personal comms) to recommend 5m³/sec per hectare as being a 50 year return period runoff for the catchment. This would give a flood flow of 120m³/sec for a 100 year return period event.

A 100 year return period flood is thus recommended as 100m³/sec, double the design stopbank capacity.

6. Conclusions

The main conclusions of this report are :

- The 1 in 100 year return period flood (ie a flood having a 10% chance of occurring in the next 10 years) is assessed as being 170m³/sec for the Taylor (through Blenheim) - lower Opawa.
- The 1980 Taylor - Lower Opawa flood of 130m³/sec is assessed as having a 1 in 20 year return period. This flood overtopped stopbanks.
- The Taylor at Borough weir water level recorder is assessed as having a 1 in 100 year return period flood of 245m³/sec.
- The 1 in 100 year design flood hydrograph into the Taylor dam is recommended as having a flood peak of 285m³/sec with a flood volume of 10 million m³ over a 24 hour period in which it exceeded 40m³/sec.
- The recommended dam design flood flows are approximately half the original scheme design flows. An orifice plate that throttles the Taylor dam outlet culvert was installed in 1980, and this latest check on flood hydrology confirms it is appropriately sized, for it can pass a 1 in 100 year return period flood while using 70% dam storage, and water level 2.5 m below spillway crest level.
- The September 1989 flood of 230m³/sec into the dam and flood volume of 8 million m³ has an assessed return period frequency of approximately 1 in 30 years. As Doctors Creek was not coincidentally in flood the resulting 100m³/sec flood for the Taylor through Blenheim was only of a 5 year return period event.
- The 1 in 100 year return period flood for Doctors Creek is assessed as 70m³/sec.
- The coincidence of a major Taylor flood event and a major Doctors Creek flood event from the same storm can be quite different, and the combined timing of the flood peaks also varies.
- The flood estimates are not as accurate as desirable due to a lack of data. Further flood gauging and water level recorders on the Taylor are desirable to refine and verify design flood sizes, and the performance of the dam outlet culvert.
- The upper Opawa - Rose's Overflow 1 in 100 year flood event is assessed as being 600m³/sec, reducing to 400m³/sec above the Fairhall confluence. No allowance has been made or is recommended for any Wairau breakout flows in this estimate.
- The Omaka is recommended as having 380m³/sec and the Fairhall 210m³/sec for their 1 in 100 year flood events.
- The Pukaka Stream is recommended as having a 1 in 100 year flood event of 100m³/sec, double the scheme design flood.
- The Riverlands floodway, with its tributaries Sutherlands Stream, and Wither Stream is recommended as having a 1 in a 100 year flood flow of 14m³/sec, 10m³/sec and 5m³/sec respectively.

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