Wairau River Floodways Management Plan
Appendix II

Overview of Hydraulic Analyses of the Wairau River and its Tributaries on the Wairau Floodplain

Prepared by: E Brin Williman
Rivers Planning Engineer
June 1993

CONTENTS

1. Background

1.1 Introduction

Flood hydrology is the measurement and estimation of the size of floods. The flood hydrology for the Wairau river and tributaries is outlined in Appendix I.

Flood hydraulic analysis is the calculation of the size of a waterway required to convey flood flows - in terms of river width, stopbank heights and the impedance of trees and banks within the floodway. This report herein overviews the hydraulic analyses that have been carried out on the Wairau River and its tributaries. These analyses thus determine how wide the river floodways need to be and how high the stopbanks need to be to contain the design flood. Key features of such hydraulic analyses are:

(a) River bed cross-sectional survey is essential to the calculations.

(b) As the river bed changes through sediment movement the waterway capacity will also change. River cross-sectional survey and hydraulic analyses therefore need to be reviewed from time to time.

(c) Such hydraulic analysis needs to be compared and calibrated against recorded flood levels measured during floods. These recorded flood levels are measured down the full length of the stopbanks.

(d) Development of computer technology over the last 10 years has greatly increased the ability to carry out such hydraulic analysis.

(e) Flood levels that have been recorded over the last 30 years throughout New Zealand have shown that river hydraulic analysis is much more complex than had been previously appreciated. This applies particularly to wide rivers and flood levels on one side of the river are often quite different from those on the other.

1.2 Freeboard

The term freeboard is used by river engineers to denote an extra allowance over and above the calculated design flood level. The freeboard allows for the imprecision in determining flood levels by hydraulic calculations. Freeboard is not simply an extra factor of safety, and it is known that floods of the same size can have different flood levels depending on hydraulic conditions. The inaccuracies of hydraulic calculations that freeboard has to allow for are:

(a) Surface waves

(b) Two-dimensional flow effects with water level varying from one side of the river to the other side of the river and being influenced by the meander pattern at that time.

(c) Short or long-term build-up or movement of sediment.

(d) Short or long-term impedance of the floodway by trees, weed etc.

(e) Efflux, which is the effect of the velocity head of the water if it is brought to a sudden stop such as against groynes, rock work, trees etc.

(f) Flood levels may not be linear between surveyed river bed cross-section spacings. Widely spaced cross-sections will be less accurate.

(g) Also to allow for under-height stopbanks due to initial construction errors or subsequent deterioration.

Freeboard allowance should vary from river to river and is typically between 0.2 m and 1.0 metre. Fast flowing wide rivers such as the Wairau, will need greater freeboard than narrow slow flowing rivers such as the Lower Opawa. It should also be noted that if accurate records of large floods are held and these show a consistent pattern, then less freeboard can be provided.

1.3 1960 Wairau Valley Scheme Analysis

In the 1960 Wairau Valley Scheme hydraulic calculations were carried out by manually and recorded in computation books. The river cross-sectional information used was obtained from a comprehensive survey of all Wairau rivers carried out in 1957-58 with 400 metre spacings between each cross-section. The calculations were limited in scope because:

(a) The Wairau Diversion and its development was expected to preclude the need to raise the stopbanks for flood flows down the Lower Wairau river, and (it was expected) also to lower the flood levels for some distance upstream of Tuamarina.

(b) It was expected "Lacey" regime channels on the Wairau and Wairau Diversion would considerably improve channel efficiency and thereby lower flood levels.

(c) The Lower Opawa/Taylor river system would be affected by the Taylor Dam and new stopbanking would not be required.

(d) The Upper Opawa river was expected to be in excess of required capacity and need no new stopbanking.

(e) Details of flood levels of known flood sizes on which to calibrate any calculations were not available. Text book values of Mannings "n" were presumed.

In actuality the Lower Wairau, Wairau above Tuamarina, Lower Opawa, Upper Opawa and Roses Overflow, Omaka, Pukaka and Riverlands have all subsequently required extensive new stopbanking or upgrading of existing stopbanking. Hydraulic analyses of these waterways have been much more important than was apparent at the start of the Wairau Valley Scheme.

1.4 Analyses Since the 1960 Wairau Valley Scheme Report

Further manual calculations documented in computation books were the basis for the extensive stopbank raising of the Lower Wairau and the Wairau in the 1960's. It is only recently that more detailed computer-based hydraulic analyses have been carried out and again only these later analyses have been calibrated with flood levels measured down the stopbanks.

The following section of this report briefly reviews the hydraulic analyses done to date for individual rivers and river reaches.

2. Lower Wairau (mouth to Ferry bridge)

2.1 Wairau Valley Scheme Report Calculations

Moderately detailed hydraulic calculations were carried out for assessing alternatives for dealing with the Lower Wairau river. This included calibration against one measured flood of 930 cumecs. As a result of this hydraulic analysis the Wairau Diversion was designed and expected to carry, when developed, a flow of 3,200 cumecs leaving 1,900 cumecs to be carried by the Lower Wairau. (Pascoe (1958A), Pascoe (1958B), Davidson (1958)).

The capacity of the Lower Wairau at that time was assessed at 2,950 cumecs. (Davidson (1959))

2.2 Stopbanking Following 1962 Flood

The June 1962 flood occurred before the Wairau Diversion was constructed and overtopping of stopbanks over long reaches of the Lower Wairau caused considerable damage. This flood was assessed as 3,650 cumecs. It was decided to upgrade the Lower Wairau stopbanks to cope with this flood size. As the Diversion was assessed at having an initial capacity of 700 cumecs this would give a total combined capacity of approximately 4,300 cumecs at the initial opening of the Diversion. This would cope with a flow the size of the June 1954 event, assessed as being a 1 in 20 year return period event. This standard would also subsequently increase with development of the Wairau Diversion.

2.3 1989 River Cross-Sectional Survey

In 1989 river cross-sectional survey showed that the Lower Wairau River had silted up by typically 0.8 metres. This in itself indicated that the waterway capacity was likely to have been reduced by some 700 cumecs between the last survey in 1967 and 1989 (Williman (1991A)). As a consequence of this the 1983 floods were likely to have been over-estimated in size.

Preliminary hydraulic analysis by Williman (1992) using the RICODA computer software package confirmed these findings.

2.4 Recent Analyses

Further more detailed hydraulic calculations were carried out by Noell (1992A), again using the RICODA computer software package. In carrying out this analysis Noell examined every flood gauging that had been carried out from the Ferry Road bridge and all floods in which the levels had been pegged down the stopbanks, especially the August and September 1970 floods, the April 1975 flood, the July and October 1983 floods, the July 1989 and August 1990 floods.

Noell's analyses also considered the effect of the increased tree growth that had occurred on the banks over the last 35 years. Calibration of Mannings "n" roughness for the channel was carried out and based on the 1990 recorded flood levels. The assessed channel Mannings "n" hydraulic roughness of 0.021 was in good agreement with the work carried out by Pascoe in 1957. However work done by Hicks and Mason (1991) indicated where willow trees overhung rivers that Mannings "n" hydraulic roughness increased with flood water rise. Noell therefore increased Mannings "n" proportionately in the model with a figure of 0.025 at the highest stopbank level.

(The flood hydrology analysis outlined in Appendix I was carried out simultaneously to this hydraulic analysis. The findings of the hydrology study indicated the recorded flood sizes Noell used for calibration to be slightly high. An even better fit between the hydrological analysis and the hydraulic analysis could be achieved by a slight increase of Mannings "n" hydraulic roughness to 0.026 in peak flood conditions).

The main finding of Noell's work was that the Lower Wairau had a capacity of generally 2,500 m^3/sec. This allows for 500 mm freeboard. An exception is 500 metres on the right bank upstream of Watsons Road to the Spring Creek oxidation ponds where the bank is typically 300 mm low. A secondary finding was that downstream of Jones' Road there is adequate capacity without using much of the Beatson's Overflow berm area and that there is opportunity to relocate stopbanks closer in to the main river channel in this area.

Significant improvement of approximately 10% increased capacity can be achieved by removal of overhanging willow trees. Conversely further siltation would reduce capacity.

2.5 Wairau Mouth

The start point for hydraulic analysis is the predicted most downstream water level, i.e. at the Wairau mouth. Since 1961 this has been a fairly direct mouth opening out to the sea, a condition that achieves minimum flood levels. Historical floods, particularly as recorded in 1929 and 1939, show that when the mouth is partially blocked by a spit, higher flood levels result upstream, at least until the spit is overtopped and eroded through. These higher flood levels in the Lower Wairau occur as far upstream as Jones' Road.

Noell's backwater model did not actually start at the river mouth but at the head of the estuary section at cross-section 4, some 2.5 km upstream. This is due to a lack of information both of river cross-sections in the estuary reach and of any recorded flood levels.

2.6 Future Flood Levels.

Predicted flood levels in future may change due to -

(a) Continuing siltation of the river bed.
(b) Removal of overhanging willow trees.
(c) If the bar is not maintained in its present direct outlet position and a partial spit is allowed to form.

Further monitoring of bed levels, of flood sizes and flood levels and further hydraulic analyses will be required from time to time.

3. Wairau Diversion

3.1 Background

Considerable interest was shown by the Marlborough Catchment Board staff in assessing the hydraulic capacity of the Wairau Diversion as it developed through scour since its 1963 initial pilot cut construction. Cross-sectional surveys were carried out typically every three years, and flood levels regularly recorded by pegging and surveying.

The hydraulic manual calculations were facilitated by the fact that both the developing central channel and the grassed berm were even and rectangular in shape. Conversely the hydraulic calculations were made more difficult by the fact of the complex slope of the Diversion floodway. Heavy beach gravels form a submerged weir downstream of Rarangi bridge. For the 2 km downstream of this the developing channel was steep; for the 2 km upstream the flood slope was very flat at typically 1:5000. The interaction of berm flow and channel flow is also clearly quite complex.

The manual calculation used at the time to calculate the size of the 1975 and 1983 floods (Pascoe (1983)) presumed a Mannings "n" of 0.025. It is not known if any calibration of recorded flood levels and calculated hydraulics was carried out.

3.2 1990 Flood Calibration

In August 1990 two significant floods of 430 m^3/sec and 830 m^3/sec were simultaneously gauged and flood levels the full length of the Diversion were pegged and surveyed. Both these gauged floods were effectively contained within the main channel.

New cross-sections were surveyed and hydraulic models established using RICODA computer software. This calibration exercise enabled Mannings "n" hydraulic roughness for the channel to be assessed at 0.021.

3.3 Recent Analysis

Noell (1992A) set up 15 different hydraulic models, one for each of the cross-section surveys of the Diversion between 1963 and 1990. The 1990 calibrated channel Mannings "n" roughness of 0.021 together with other calibration studies was used for all the 15 other models representing different times and stages of the Diversion development.

The results from this analysis were used in cross-checking hydrological measurements of the flow and so is also discussed in Appendix I.

Good results were obtained for examined flood flows entirely within the channel, but less good analyses were obtained for the largest floods which used both channel and berm. Thus the backwater analysis of the major 1983 floods and the 1975 flood appeared significantly high by 600 m^3/sec compared with the hydrologically based assessments.

This discrepancy is considered to be a combination of under-estimation of the hydrologic based estimates and over-estimates of the hydraulic model. The errors in this hydraulic model are thought to be primarily because calibration of the model was restricted to the short 1600 m of flat graded reach of the Diversion upstream of Rarangi bridge. Better calibration would be achieved if the recorded flood levels in the steeper 2000 m reach downstream of the bridge was also used. Unfortunately this reach had higher velocities - close to critical velocity - and with flow suddenly narrowing down from full floodway width to the main channel, led to mathematical instability of the RICODA hydraulic model and meaningless results.

Some very recent calculations using the MIKE 11 hydraulic computer programme were able to mathematically model the steeper reach of the Diversion downstream of Rarangi bridge that had been unable to be modelled by the RICODA programme. These calculations indicated the October 1983 flood size as 1900 m^3/sec, some 200 m^3/sec lower than the previous RICODA backwater analysis. This is in closer agreement to the WCS gauging analysis. Only the October 1983 flood was modelled for as this work was carried out by Barnett (1993) consultants as a demonstration exercise on the capabilities of the MIKE 11 programme.

Reducing Noell's estimation of flood capacity appropriately, and allowing for 600 mm of freeboard, indicates that the waterway capacity at the top end of the Diversion is currently 2800 m^3/sec. The waterway capacity increases with distance down the Diversion.

3.3 Further Monitoring and Analysis

The Wairau Diversion is not up to desired capacity and further enlargement by erosion is anticipated - though this will also be counter-balanced by gravel deposition from upstream. Regular monitoring of floods and flood levels and development of a more accurate hydraulic computer model will continue to be required.

4. Flow Division Area of Diversion and Lower Wairau

The hydraulics of this area are critical to the control of the division of flow into the Wairau Diversion and Lower Wairau. It is also of relevance in controlling the level of flood water backing up Spring Creek.

The area is hydraulically very complex with a convoluted Lower Wairau channel which is heavily aggrading; substantial berm flow areas; recent stopbanking; an enlarging Diversion channel; and regular gravel extraction. A detailed set of river bed cross-sections was surveyed, in 1992, used specifically to assess the hydraulics of this reach.

Works Consultancy Services (1992) used the MIKE 11 computer programme to carry out this analysis. Even this analysis was limited for there was a restricted amount of flood level data on which to calibrate the model.

The model showed that the waterway capacity was generally to 2500 m^3/sec, but that Spring Creek would back up and overtop its stopbanks at a lower flood than this. This could be improved by routing the Spring Creek outlet further downstream.

The feasibility of building a simple flow control structure of earth banking to control flow into the Diversion is critically dependent on the hydraulics of this area. It is important to develop an accurate hydraulic network model such as MIKE 11 of this area, and considerably more work is required on this. This includes regular monitoring of flood levels and gauging of floods to enable calibration of the model.

5. Wairau (Tuamarina to Waihopai)

5.1 Marlborough Catchment Board Design Flood Level for Stopbanking

The Marlborough Catchment Board carried out extensive stopbanking on the right bank of this reach of the river, including relocating stopbanks by narrowing the river, in the late 1960's. The basis for the design stopbank heights shown on Plans 208/99, 208/100, 208/101 was manual calculations recorded in computation books, though there are no reports on these calculations. It is unlikely that the calculations were calibrated against any flood levels for there was little flood level data available before 1970. The freeboard allowance adopted was a relatively low 600 mm.

5.2 1983 Flood Response

Some stopbanking, notably for the 3 km upstream of Tuamarina was reconstructed to a higher level following the damaging 1983 floods.

The basis for design height for the upgraded stopbanks was the October 1983 flood levels plus a freeboard of 900 mm. At the time this flood was considered to be 5000 m^3/sec and of design flood size, and therefore of direct relevance in determining required stopbank levels. One drawback to these recorded flood levels was that in some places internal training banks were swept away. This resulted in lower recorded flood levels than would have been the case if the training banks had remained intact.

The recorded July 1983 flood levels and some computer analyses were used as a confirmatory check for design stopbank height. Unfortunately no report was written up of the computer analysis regarding its assumptions, methodology and findings, and so its value is uncertain.

Since 1983 the following changes have occurred:

(i) The internal training banks swept away in 1983 have been replaced in a generally stronger form.

(ii) The channel has aggraded significantly in some places.

(iii) A more accurate analysis of historic flood sizes has been carried out - Appendix I - assessing the October 1983 flood as 4400m^3/sec and being much less than the design 5500 m^3/sec.

(iv) The improvements and availability of computers has facilitated much more detailed hydraulic analysis.

Further review and analysis was therefore required to assess design stopbank levels and other channel changes for 1993 conditions.

5.3 Physical Changes to Floodway

Since 1957 the following changes have occurred to the Wairau floodway:

(a) Stopbanked floodway width has been reduced so that it is now typically 800 metres, down 20% from the pre-scheme situation.

(b) Training banks within the floodway reduce the effective width further still to typically 700 metres.

(c) The active non-vegetated gravel channel has reduced from 600 metres to 400 metres.

(d) Aggradation of the floodway has progressively occurred by deposition of gravel and silt in the channel and on the berms. This has raised the bed level by an average 0.4 m between Tuamarina bridge and Conders groyne.

(e) Stopbanks have been raised by typically 0.7 m, enabling approximately a 20% increase in flood water depth.

The overall effect of bed aggradation, reduction in active channel, and reduction of stopbanked floodway width has exceeded the increase in waterway capacity due to stopbank raising. In straight waterway capacity terms the waterway capacity is now approximately 20% smaller than the pre-scheme situation. This is counter-balanced by the in that cross-channel flows resulting in locally high flood levels was worse in the pre-scheme situation than today where there is a smoother more controlled meander pattern.

5.4 Recent Hydraulic Analysis

Noell (1992B) carried out a hydraulic analysis of the Wairau from Tuamarina to Waihopai using the RICODA computer software. As well as the 1991 cross-sectional survey based model, a 1969 and a 1984 were set up so as to calibrate the Mannings "n" hydraulic roughness with recorded flood levels of the 1967, 1970, 1973 and 1983 floods.

An initial major finding was that flood levels on each bank were strongly influenced by the meander pattern and training banks. Flood levels on the outside of the meander were typically 1.0 metre higher than flood levels on the other side of the river.

This presented difficulties in carrying out the modelling procedure, especially trying to calibrate the model with recorded flood levels.

Noell therefore developed a technique for modelling the right bank only where there is continuous stopbanking. Mannings "n" hydraulic roughness was varied between being artificially low on the inside of meanders to artificially high on the outside in order to calibrate it with measured flood levels.

The major findings of the report were:

(a) Flood levels would now be typically 0.4 m higher than for the same size flood in 1983.

(b) Removal of impeding training banks within the floodway will significantly improve its efficiency.

(c) For the 8 km reach from Hillocks Road to Giffords Road the stopbank is typically 0.8 m low.

(d) A 6 km reach in the Conders area is similarly low.

It should be noted that the flows used by Noell in his report were based on preliminary flood flow calibration estimates. These are now considered to be 5% high. Thus his "6000 m^3/sec" predicted flood levels is really for a 5700 m^3/sec, i.e. effectively design flood.

5.5 Further Hydraulic Modelling

A more sophisticated computer model capable of dealing with flood berms separately from the main channel is desirable i.e. a "network" model. This would more accurately define the effects of improving berm efficiency by tree removal etc.

6. Lower Opawa/Taylor

6.1 Background

The Lower Opawa/Taylor system is hydraulically very difficult to analyse, because of its complexities which include -

- A deep narrow main channel flanked by irregular wide berms. Flow discontinuity occurs as the flows overtops the main channel on to the berms.
- The river is tidal, with very flat flood slopes
- There is significant channel storage
- The berms at times have much shorter flow paths than the main channel
- Heavy tree growth flanking the river banks, and during summer heavy weed growth

It is not surprising that early manual calculations over-estimated its flood capacity.

The Wairau Valley Scheme report of Davidson (1959) estimated the Lower Opawa capacity as 310 m^3/sec and thus requiring no new stopbanking.

The May 1966 flood was in excess of the capacity of the system and considerable flooding occurred in the Opawa Loop area. The Lower Opawa was reported as at capacity and carried flood flows from both the Taylor and Upper Opawa at that time. The flow division with Upper Opawa/Roses Overflow cannot be known with accuracy and the contemporary estimate of 230 m^3/sec in the Lower Opawa is now considered excessive. The Lower Opawa/Taylor system has been made separate from the Upper Opawa with channel blocks constructed on the Opawa Loop in 1967.

Several large floods occurred in the 1970's and some of these floods were gauged. From this gauging data Rae (1979) estimated the hydraulic capacity of the Taylor through Blenheim at 158 m^3/sec. This calculation was based on the assumption of the hydraulic performance of the Lower Opawa downstream staying constant. This has not been the case for there has been increasing impeding tree growth. Rae also apparently made no allowance for freeboard. Manual calculations in computation books apparently indicated the Lower Opawa could have a capacity of 200 m^3/sec, though no report was written up of these calculations or the methodology. These estimates are now considered to be high.

6.2 The April 1980 Flood

The April 1980 flood overtopped stopbanks in the Lower Opawa and was at design level through Blenheim. This flood was therefore close to the river design capacity. It was gauged at 130 m^3/sec. Following this Thomson (1980) recommended that the Lower Opawa/Taylor design flood should be 150 m^3/sec with freeboard of at least 0.2 metres and preferably 0.4 metres (and also recommended various river improvement works). No specific hydraulic analysis was documented.

6.3 Recent Analysis

Williman (1991) carried out some analysis of the hydraulics of the Lower Opawa/Taylor channel. This analysis had the benefit of a recent resurvey of river cross-section in 1988, previously carried out in 1965, and monitored flood levels for the 1989 flood that was gauged at 100 m^3/sec. Monitored flood levels were also available for the 1971, 1977 and 1980 gauged floods, which enabled calibration of the hydraulic model, and compare the effect of willow tree growth on removal in various river reaches.

Aerial photographs during the 1989 flood were also very valuable for they showed the degree to which natural banking on the flood berms was seriously inhibiting flood flow on those berms.

The findings of this analysis were -

- There had been minimal siltation of the channel since 1957 and so little effect on changing flood hydraulics.
- In areas of thick willow trees a high Mannings "n" hydraulic roughness of 0.07 was modelled, but in the areas of recent willow tree removal and berm shaping works this reduced to 0.040.
- Natural banking on the berms prevented flood flow utilising the berm areas resulting in high flood levels in the Lower Opawa that backed water up in the Taylor through central Blenheim.
- The current safe capacity of the lower Opawa/Taylor between Malthouse Road and Hutcheson Street is approximately 130 m^3/sec, significantly less than the 170 m^3/sec recommended design flood.
- On the Taylor from Hutcheson Street upstream the river was of good capacity.
- The hydraulics of the Lower Opawa from Malthouse Road downstream are principally determined by flood levels in the Wairau estuary through Wairau floods and/or tide levels. Stopbanks are to an adequate height here provided a good Wairau mouth outlet is maintained.
- Removal of riparian willow trees from the Lower Opawa channel and berm shaping works would considerably improve Lower Opawa channel efficiency and thus achieve the design waterway capacity of 170 m^3/sec for both the Lower Opawa and Taylor.
- The model was limited in its ability to determine actual flood flows on the berms and thus quantify the extent of berm shaping works required to achieve improvements to design standards. A more sophisticated hydraulic model is desirable for this purpose so as to design the berm shaping works required.

7. Upper Opawa/Roses Overflow

Historically this system carried Wairau flows of 1000 m^3/sec and more - though with significant overflow at times. With increased land values through grape development there was pressure since the 1970's for narrowing the floodway by moving stopbanks in. A hydraulic review was carried out, reported on by Pascoe and Wadsworth (1980) and this was the basis for stopbanking relocation work, generally for the reach downstream of Thompsons Ford Road. In this manual calculated analysis, fairly wide-spaced (800 m) cross-sections surveyed in 1957 were used.

A more comprehensive analysis was carried out by Williman and Gardiner (1988) using cross-sections surveyed in 1988 at 400 m spacing, a more conservative Mannings "n" roughness of 0.050 and the calculations carried out using the CHANEL computer programme. This analysis confirmed the adequacy of the already upgraded stopbanking works in the lower part of the river system and was the basis for designing further stopbanking work upstream of Thompsons Ford Road. This hydraulic analysis also pointed to the need to keep the floodway relatively clear of tree and shrub vegetation - whether deliberately planted or self-generated weed species.

While no major concerns are held, it is desirable to establish a new computer model on the Council's new computer software which would incorporate the recent stopbanking and channel changes carried out and on which to monitor and calibrate flood flows.

8. Omaka River

The September 1989 flood which broke out of banks and through Renwick, demonstrated the need for the Omaka river to be a fully stopbanked system. The stopbanking work subsequently carried out (and not yet complete) was based on cross-sections surveyed in 1990 and a preliminary hydraulic analysis using the RICODA computer programme, but not written up as a report.

As for the Upper Opawa, it is desirable to establish a computer model to incorporate the recent stopbanking and channel changes and to monitor future flood events.

9. Riverlands/Wither Stream System

9.1 Riverlands

The upgrading of the Riverlands floodway system to a good standard (nearly completed) is based on manual hydraulic manual calculations carried out by V Wadsworth in 1982, though this was not written up in a formal report. Manual calculations are quite adequate for this regular trapezoidal man-made channel and conservative values were used for Mannings "n" roughness of 0.035 and initial expected water levels at Riverlands outlet into Vernon lagoons. Future monitoring and modelling its performance at flood times is desirable.

9.2 Wither Stream

Wither Stream is being upgraded to design capacity by making it into a rectangular concrete lined channel of quite predictable hydraulic performance as determined by Davidson (1991).

9.3 Rifle Range Creek

Again an artificial channel constructed to manual calculations by V Wadsworth in 1981, but not formally written up. Subsequent calculations by Davidson in 1992 (personal communication) suggest that the system may only just be to adequate capacity, depending on the performance of culverts under Wither and Taylor Pass roads. Future monitoring of flood events is desirable.

9.4 Sutherlands Stream

This has been reconstructed with a wide flood berm and secondary overflow berm. Any potential concerns of channel capacity would be due to sudden aggradation of the channel and not a nominal hydraulic capacity.

10. Other Floodplain Rivers

10.1 Pukaka

As discussed in Appendix I the design flood is now estimated at 100 m^3/sec, approximately double the initial design calculation. The culvert outlet into the Wairau Diversion is the dominating hydraulic feature of this artificial channel and the hydraulics of this culvert were examined by Sutherland (1985). Wadsworth (1986) examined the effect of increasing the culvert capacity and thus reducing the area flooded by the Pukaka overflowing via a deliberate spill weir.

Further hydraulic analysis is required to quantify the area of land affected by this deliberate spilling, and to examine options for reducing this spill area.

10.2 Doctors Creek

No hydraulic calculations have been carried out for Doctors Creek which sheet floods and ponds over approximately 300 hectares and is also affected by Taylor river levels. Such analysis is required to quantify the area flooded and examine options to reduce this flooded area.

10.3 School and Terrace Creeks, Renwick

Some limited hydraulic calculations were carried out for improvements to the Terrace creek system in 1964. There have been considerable changes since then and a detailed hydraulic analysis is required, including assessment of expected flood levels in Gibsons Creek into which they discharge to bring the system to an adequate standard and allow adequate disposal of Renwick stormwater.

11. REFERENCES

Barnett Consultants (1993): "Wairau Modelling Trials" Report to Marlborough District Council on use of MIKE II computer programme for hydraulic modelling.

Davidson C C (1958): "Preliminary Report on Alternative Proposals for the Wairau River, Tuamarina to the Mouth" Appendix VI, Wairau Valley Scheme report

Davidson C C (1959): "Wairau Valley Scheme" Marlborough Catchment Board

Davidson C C (1991): "Wither Stream Report" Report to Marlborough District Council.

Hicks D M & Mason P B (1991) "Roughness Characteristics of New Zealand Rivers" DSIR Marine and Freshwater

Noell W J (1992A): "Estimation of Recorded Flood Sizes in Lower Wairau and Wairau Diversion by Hydraulic Backwater Analysis"
Internal report, Marlborough District Council.

Noell W J (1992B): "Estimation of the Capacity of the Wairau River between Tuamarina and the Waihopai River" Internal report of the Marlborough District Council

Pascoe L N (1958A): "Mannings "n" for Tidal Reaches of the (lower) Wairau River" Appendix V, Wairau Valley Scheme Report

Pascoe L N (1958B): "Regime Investigations for the New Channel" Appendix V, Wairau Valley Scheme Report.

Pascoe L N (1983): "Observation and Comment on Flood of 10 - 11 July 1983" Internal memorandum of Marlborough Catchment Board.

Pascoe L N & Wadsworth V (1980): "Opawa River Floodway Guidelines" Staff report to Marlborough Catchment Board

Pascoe L N, Wadsworth V and Ingles A M (1985): "Wairau River Flood Protection - Review of Design Discharges and Stopbank Capacities" Staff report to Marlborough Catchment Board.

Rae S N (1979): "Taylor River Channel Capacity, Hutcheson Street to Blenheim Youth Club" Internal staff memorandum of Marlborough Catchment Board file 214.

Sutherland A J (1983)" "An Investigation of Pukaka Stream Outlet" Report to Marlborough Catchment Board

Thomson P A (1980): "Preliminary Report Reviewing the Operation of the Taylor/Lower Opawa Flood Control System" Staff report to Marlborough Catchment Board

Wadsworth V (1986): "Pukaka Stream" Report to Marlborough Catchment Board

Williman E B & Gardiner M R (1988): "Reassessment of Upper Opawa River" Staff report to Marlborough Catchment Board

Williman E B (1991A): "Upper Conders Stopbanking and Related Matters" Nelson-Marlborough Regional Council internal memorandum file 208.

Williman E B (1991B): "Lower Opawa River Hydraulics - Preliminary Note" Internal memorandum, Nelson-Marlborough Regional Council file 213.

Williman E B (1992): "Lower Wairau River Preliminary Backwater Analysis" Nelson-Marlborough Regional Council internal memorandum file 207.

Works Consultancy Services (1992): "Wairau River Flood Flow Hydrology Review" Report to Marlborough District Council

@P\0\RMR\dplan\Wairiver\HYDRAULI.DOC

adw

14.09.93