WATERWAY DESIGN PARAMETERS

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G H MACKY
A I MCKERCHAR

National Institute of Water and Soil, Christchurch.

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PO Box 2331, Lambton Quay, Wellington, New Zealand
Telephone (04) 473 0220; Facsimile (04) 499 0733

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AN IMPORTANT NOTE FOR THE READER

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EXECUTIVE SUMMARY

Transit New Zealand has adopted AUSTROADS (1994) for hydraulic design of bridges and culverts. This report addresses two aspects of hydraulic design which are considered only briefly in AUSTROADS (1994):

(i) The determination of a design peak flood flow rate; and

(ii) The estimation of the flow resistance of channels.

Existing knowledge and practice on these two topics has been reviewed in some depth. First, a questionnaire was sent to practising engineers to find out what methods they currently used. Second, university lecturers in engineering and geography departments were questioned about the methods they were teaching. Third, existing scientific knowledge on both these topics was reviewed, with an emphasis on any recent research developments.

Some site data have been obtained to test which of the methods used in New Zealand on ungauged sites appeared to be most valid. The sample of sites was too small to make absolute pronouncements about which methods were best, but they gave an indication of which methods had the most promise, and therefore might justify continued investigation.

Research worldwide has produced a considerable array of methods for determining design flow rates at ungauged sites. Many of these, ranging from the unit hydrograph to complex computer models, model the runoff which results from an assumed rainfall event. These models can be very useful in examining events more extreme that those measured or flows which might occur after changes in land use, for instance. These methods are taught and used in New Zealand, but they are not as useful as in other countries, because in general New Zealand is rich in flow data but has only limited rainfall data. In this study just one site was analysed for which a rainfall/runoff model had been developed. In that case the model gave an adequate prediction of the flow with an annual exceedance probability (AEP) of .01, but other simpler methods did just as well.

Students in degree courses were also introduced to regionalisation techniques, which transferred measured runoff data between catchments, as well as learning older empirical methods, such as the Rational Method and the TM61 method. Recent research efforts had concentrated on the regionalisation techniques, but there was a clear lag in the adoption of new procedures, and practising engineers often favoured the empirical methods.

The sites analysed in this study showed that the TM61 method gave erratic, usually high, estimates and which made it unsuitable for design. The New Zealand regional flood estimation (RFE) method of McKerchar and Pearson (1989) was reasonably successful in predicting design flood events, as was the Rational Method, despite minimal information on how to apply it to New

Zealand conditions. However, an uncertainty of up to 40% in estimates of design floods appeared unavoidable without a long flow record at the site.

There has been only limited recent research worldwide on channel resistance. The engineering profession tends to rely on site measurements (if available), photographs of comparable measured sites and tables of recommended roughness values matched to descriptions of the channel. Students were introduced to these and also to a selection of formulas derived for gravel beds.

The few sites analysed in this study do not give a clear indication that one method is markedly better than another. Two formulas by Griffiths (1981), derived from New Zealand gravel-bed river data, performed a little better than the other methods. The descriptive table given in AUSTROADS (1994) (their Table 4.1) was difficult to apply to New Zealand rivers and gave erratic results. The best present strategy appears to be to use two or more of these methods together, tempered with experience and judgement. At present, values of Manning's roughness coefficient n (the usual measure of channel resistance) can be expected to be in error by up to 40%.

Estimating flood flows and channel resistance are both difficult tasks justifying careful study by an experienced practitioner. But uncertainty will still remain, and designers should have regard for the consequent uncertainties in estimated water depths and velocities.

ABSTRACT

Two aspects of hydraulic design are addressed: the determination of a design peak flood flow rate; and the estimation of channel flow resistance. Existing practice on these two topics is reviewed by surveys of practising engineers and university lecturers in engineering and geography departments. A review of existing scientific knowledge on both topics is also presented, with an emphasis on any recent research developments.

For both flood flow rates and channel resistance, limited site data have been obtained to indicate which of the methods used for ungauged New Zealand sites appear to be the most valid, and therefore may justify continued development.

1. INTRODUCTION

1.1 Background

Transit New Zealand has recently adopted AUSTROADS (1994) for hydraulic design of bridges and culverts. AUSTROADS defers to Pilgrim (1987) for hydrological design, which states on page 4: *"There has been a growing recognition of the importance and implications of the difference in hydrological characteristics of the regions* [of Australia], *and the consequent inability of a single set of design data to be satisfactory for the whole of Australia".* It was therefore important to examine the methods available to New Zealand designers, and determine which was applicable in this country. This also applied to estimating the flow resistance of channels, which AUSTROADS treats only briefly by providing a table of recommended values of Manning's n.

It is usual in the design of hydraulic structures to design against a peak flow which has a particular annual exceedance probability (AEP), such as 0.02. It is not universally agreed that this approach is entirely valid. The choice is a matter of concern for many design engineers (see Section 3.1.2 and Appendix 2), and researchers (e.g. Davies 1993) have questioned whether choosing any particular AEP satisfactorily identifies the risk to property and life. However, the flood frequency part of this study has not addressed this issue, but considers the next step, of estimating the size of (for instance) the 0.02 AEP peak flow.

1.2 Research Objectives

The research described in this report considered two problems, estimating flood frequency in ungauged catchments and estimating channel resistance. There were two principle objectives:

- To undertake surveys of engineers and academic staff to determine which methods were being used by engineers, and which methods were being taught to future practitioners.

- To indicate which method or methods gave reliable estimates of channel resistance and flood frequency, by a literature search and by analysing a small representative sample of sites.

1.3 The Report

This report presents the research findings in the chronological order the work was done. Stage One, incorporating sections 2 and 3, describes the surveys of engineers and academic staff and the literature search. The results of Stage One were submitted to Transit New Zealand for review and also used to finalise the choice of methods to be tested in Stage Two.

Stage Two, incorporating Sections 4, 5 and 6, describes the analysis of two sets of representative sites, one set for flood frequency and one for channel resistance. Overall conclusions from the study are discussed in Section 7; and recommendations for design practice, technology transfer to the engineering profession and future research are itemised in Section 8.

STAGE ONE: EXISTING KNOWLEDGE AND PRACTICE

2. RESEARCH METHOD

A questionnaire was circulated to determine existing design practice in New Zealand. The flood frequency component of this research drew on a questionnaire circulated by the DSIR Hydrology Centre (now part of NIWA) in 1991. The results from this survey were set out in an internal report - two graphs from which are attached here as Appendix 1. To update these data and to obtain information about design practice for estimating channel roughness, another questionnaire was circulated in January 1996 (Appendix 2).

In order to determine the methods now being taught and recommended in engineering degree courses, individual discussions were held with the following lecturers:

Dr David Painter Dr Tim Davies	Natural Resources Engineering	Lincoln University
Dr Alex Sutherland Mr Roddy Henderson[1]	Civil Engineering	Canterbury University
Dr Bruce Melville Dr Steven Coleman	Civil Engineering	Auckland University

In addition, information was sought on what was being taught on flood frequency analysis in other degree courses. Written responses were obtained from the following lecturers:

Dr David Murray	Geography	Otago University
Dr Earl Bardsley	Earth Sciences	Waikato University
Dr Jack McConchie	Geography	Victoria University

Assessing and collating existing research into topics being taught drew on the combined knowledge of NIWA and university staff (particularly related to New Zealand research), and involved literature searches. Two "state-of-the-art" papers on frequency analysis (Potter 1987, Stedinger et al. 1993) assisted this process, but work on channel roughness appeared to lack such unifying papers.

[1] Mr Henderson is a NIWA scientist and has been lecturing on contract, teaching Surface Water Hydrology to 3rd Professional year undergraduates.

3. RESULTS

3.1 Existing design practice - Questionnaire returns

3.1.1 1991 Questionnaire

There were 66 responses from regional and district council staff and consulting engineers. Their responses (see Appendix 2) showed a strong reliance on the older methods of flood frequency estimation i.e. the Rational and TM61 methods, and (except in regional councils) Beable & McKerchar (1982) in preference to McKerchar & Pearson (1989). In their use of the Rational Method, values of the runoff coefficient C were widely spread (mostly 0.1-0.6) for residential and rural land. Many authorities required the use of the Rational Method and often promulgated recommended values for their territory.

3.1.2 1996 Questionnaire

There were 49 responses to this questionnaire, and these are tabulated in Appendix 2. The salient points were:

1. Flood frequency estimation in ungauged catchments was still determined using the TM61 method or the Rational Method. Some designers used the Rational Method for small catchments only, but others used it exclusively. TransRail use both these methods and also their own formula adapted from the TM61 method;

2. Engineers still used Beable & McKerchar (1982) rather than McKerchar & Pearson (1989) (although the researchers intended one to supersede the other) but neither was widely used;

3. Rainfall-runoff models (including the Unit Hydrograph) had a relatively modest following;

4. The Manning formula was almost universally preferred over other equations for frictional head loss;

5. Channel roughness was estimated using a variety of techniques, especially the tabulated values in Henderson (1966) and calculation from site measurements, but prescriptive equations were not popular;

6. "Waterway Design", the AUSTROADS manual adopted by Transit New Zealand as a standard, was not widely used for estimating channel roughness; and

7. Prompting engineers to identify other problem subjects in waterway design revealed a wide range of concerns, including scour problems, and choosing the return period of the design flood.

3.2 University Teaching

3.2.1 Flood Frequency Analysis

In the undergraduate engineering courses, a reasonable amount of time was spent on the statistical analysis of runoff data from a single site. At Lincoln the emphasis was on the Generalised Extreme Value (GEV) distributions, whereas Auckland University also introduced its students to other statistical distributions.

All three engineering courses present the RFE method of McKerchar & Pearson (1989) as the most up-to-date method of estimating extreme floods in ungauged catchments. The unit hydrograph was presented as the standard method when rainfall data were available rather than long-term runoff data, with the synthetic unit hydrograph used when there was no flow data for calibrating a unit hydrograph. More recent and sophisticated rainfall/runoff models were alluded to, as were older empirical methods including the Rational Method. Auckland University taught their students the TM61 method, but the other two engineering departments regarded it as obsolete.

Dr Davies had concerns about the choice of any particular annual exceedance probability (AEP) for design (Davies 1993), and it was also highlighted by the survey of practising engineers. This matter is outside the scope of this report.

Dr Murray at Otago University taught his geography students the unit hydrograph method and described regional estimation procedures in general and the McKerchar & Pearson method in particular. However, flood estimation was only a small part of the course and its treatment was brief.

At Victoria University, geography students were exposed to a range of flood estimation procedures, from the TM61 method to regional methods. In contrast, "subjective estimation" and a "non-parametric philosophy" (E. Bardsley, pers. comm.) were taught to earth science students at Waikato University (Bardsley 1994).

3.2.2 Channel Roughness

The Manning equation and its use in hydraulic calculations were covered in reasonable detail in the engineering courses, and other theoretical formulas such as that of Keulegan were also mentioned. However, the advice given students on the actual choice of a value for channel roughness reflected the difficulties in applying such formulas to typical field situations. At Lincoln and Auckland, students learned the Strickler formula (e.g. Henderson 1966), and were referred to the photographs in Chow (1959) and Hicks and Mason (1991) and the tables in Henderson (1966). At Canterbury University, undergraduates were referred to Hicks and Mason (1991) and to a modified Strickler formula. However, they also examined the three formulas proposed by Bray and Davar (1987), testing them against measured data for a North Island river. This exercise was in part intended to illustrate the uncertainty in applying any formula.

Geography students at Victoria University also learned the Manning equation, and were taught about assessing Manning's n by calibration against measured data; use of Hicks and Mason (1991); and (graduate students only) formulas such as those of Griffiths (1981) and Strickler.

3.3 Review of Existing Knowledge and Research

3.3.1 Flood Frequency Analysis

The science of flood frequency analysis can be conveniently divided into several categories, depending on what data are available. At-site statistical techniques are employed to evaluate an existing flow record. Where this record is short or non-existent, regional flood estimation techniques have been devised to take account of records from nearby catchments.

Rainfall/runoff models, ranging from the Rational Method and the unit hydrograph to much more complex models, address situations in which rainfall data has been gathered but runoff data is relatively limited or completely lacking. This includes assessing situations where the runoff hydrograph is expected to change due to land-use changes. These methods must be linked to a statistical treatment of the rainfall.

A common assumption in all these techniques is that the data are stationary, that is, there is no long-term trend. This assumption has been questioned and tested more in recent years with increased awareness of the possibility of global warming. Salinger et al. (1990) and Whetton et al. (1996) have considered possible future trends in the New Zealand climate. Their conclusions depend on assumptions made in their analyses, but in many parts of New Zealand 24-hour rainfall events that now have an AEP of 1% are expected to become twice as common in 50 years' time. Continuing research on this topic will look at what trends have occurred in the observational record. Withers (1992) looked for trends in some existing rainfall and runoff records, and found none.

3.3.1.1 Extrapolation of at-site data When flows have been measured at the site for many years, the only tasks in principle are to identify the statistical form of the annual flood series and to extrapolate that distribution to the design AEP. Popular choices for the distribution have been the generalised extreme value distribution (GEV) (e.g. Hosking et al. 1985, and in New Zealand: Beable & McKerchar 1982; McKerchar & Pearson 1989) and the log Pearson type 3 distribution. McKerchar and Pearson (1989) demonstrated that many New Zealand flood series could be modelled with the EV1 (Gumbel) distribution (which is a particular case of the GEV).

It has been recognised that extreme events at some sites have two or more alternative causes, such as snowmelt and rainfall events, or distinct weather patterns giving separate series of rainfall events. Rossi et al. (1984) developed the two-component extreme value distribution to model Italian floods. Various workers (e.g. Hirshboeck 1986; Dalin 1986) have identified different meteorological processes contributing to "mixed" flood series (including outlying events). Painter & Larsen (1995) explained apparent anomalies in flood data from the Hakataramea River by showing that they were caused by two types of storm.

Potter and Walker (1985) considered the problem of measurement error inherent in stage measurement of floods which exceed the largest flood actually gauged by flowmeter. They concluded that this error could be significant. It follows that a design AEP extrapolated from these floods could be significantly in error, and therefore the flood gaugings as well as the flow record should be examined before extrapolating the data.

3.3.1.2 Regional flood estimation (RFE) techniques Considerable research effort, both overseas and in New Zealand, has been directed at regionalisation techniques which can be used on ungauged sites or to improve estimates of extreme events at sites where the gauged record is too short to give statistically reliable estimates on its own.

Most of these methods seek to identify reasonably homogeneous groups of catchments for which the statistics of the flood series are similar. In general, these groupings are chosen by the researcher (based either on a subjective sense of geographical similarity or on a method of grouping such as Wiltshire's (1985,1986)) and then tested for homogeneity using the statistical properties of the flood series. L-moments (Chowdhury et al. 1991; Pearson 1991a) have proved more robust for this purpose than conventional statistical moments such as the coefficient of variation.

Among the simplest of these methods are index flood methods. The catchment's mean annual flood (or some other index flood) is derived from at-site data, but regional information is used to determine the distribution of annual floods and hence the size of extreme events. One such method has been popularised by Wallis (1980) and Hosking et al. (1985).

Waugh (1978) adopted a similar approach for catchments in Northland and Auckland. The shortness of the flow records available forced him to turn to Californian catchments to estimate the 0.02 AEP event.

Regional regression methods attempt to express statistical properties of the flood distribution as functions of catchment characteristics such as area and slope. (Stedinger et al. 1993, Pilgrim 1987). This approach was adopted for New Zealand by Beable & McKerchar (1982), who fitted GEV distributions to 152 sites and identified eight regions. For each of these regions regression analysis was used to express flood discharges as a function of catchment area and one or two rainfall parameters (as defined by Tomlinson 1980). However, the uncertainties inherent in estimates of rainfall statistics can be very large, especially in sparsely gauged mountainous regions. McKerchar & Pearson (1989), in updating this work, therefore did not identify discrete regions. Instead, they noted an empirical relationship between discharge and catchment area, and assumed that the remaining variability could be accounted for by physical location. For about 200 gauged sites they calculated two parameters which identify the mean annual and 0.01 annual exceedance probability flood, and hand-drew contour maps of these two parameters. The method thus relies on recorded flood data and does not require estimates of rainfall intensity.

Determining flood distributions in small New Zealand catchments (less than about 100 square kilometres in area) has proved to be especially difficult. However, Lawgun and Toong (1985) were able to estimate flood discharges for small catchments in Northland and Auckland with comparable accuracy to Beable and McKerchar (1982), by an analysis using data from small catchments only. Pearson (1991b) successfully applied Wakeby distributions to groups defined using Wiltshire's (1985) method. Thus the size of extreme events relative to the mean annual flood can be reasonably well defined. However, McKerchar (1991) attempted to determine the mean annual flood using the regional regression method, and was not particularly successful. This suggests that obtaining a short flow record for a small catchment (so that mean annual flood can be estimated) will greatly improve the accuracy of estimates of extreme events using regional flood estimation procedures.

A complication which can develop in applying regional methods is the modification of floods as they pass down a river channel. If a river has physical features not present in its neighbours, the distribution of its extreme floods may vary from those of its neighbours. Wolff and Burges (1994) have investigated the modification of floods as they pass down a river reach, depending on: the geometry, an EV1 distribution of extreme floods may transform to an EV3 (if the highest floods are the most modified) or an EV2 (if the highest floods are least modified). Pearson and Macky (1992) analysed the effect of ponding behind the Manawatu Gorge, and found that the EV1 flood distribution typical of the region was modified to an EV3 distribution in the Manawatu River at Palmerston North. Similar effects might be expected for the outflows of lakes.

3.3.1.3 Use of rainfall data When design flood events are estimated from rainfall records, there are two steps to the process:

1. Determining the rainfall that causes the design event; and

2. Determining the runoff that results from this design rainfall.

The first step is a frequency analysis very similar to those described above for flood flows. The same techniques are in fact applicable to rainfall data (Stedinger et al. 1993). However, an area reduction factor is often applied to point rainfall estimates.

The second step is discussed below under two headings, depending on whether runoff data are available to calibrate the rainfall/runoff models

3.3.1.3.1 Rainfall data only TM61 (the popular name for Ministry of Works and Development 1964, 1975) is an empirical method for estimating the design discharge in New Zealand catchments; the two versions are essentially the same except for the system of units. Parameters describing catchment shape and ground conditions must be estimated. A time of concentration must then be calculated (using one of several empirical formulas) so that a corresponding rainfall depth can be estimated from rainfall data such as that presented by Tomlinson (1980).

The method is described as empirical, a term which implies calibration against measured data. However, few examples of paired rainfall and runoff data were available in 1964, so that the TM61 method must be regarded as largely unverified. Waugh (1973) tested the use of TM61 on small rural catchments in Northland and Auckland. He concluded that TM61 underestimated floods in these catchments, but that a revised formula putting discharge proportional to catchment area would perform better. This change would effectively convert the TM61 method to a version of the Rational Method (see below).

The Rational Method has a history over 100 years, and remains popular.

Its basis is the formula $Q = CiA/3.6$

where	Q	=	peak discharge (m3/s)
	i	=	the design rainfall intensity (mm/hour)
	A	=	catchment area (km2)
and	C	=	a coefficient between 0 and 1

C may be regarded as reflecting the proportion of rain which contributes to the flood peak rather than infiltrating to groundwater. In doing so, it is assumed that rainfall and runoff reach equilibrium, an assumption that is reasonable for small catchments only. In practice, C may also account for storage of water and other effects.

The method is widely used in small urban catchments, where C is taken to be in the range 0.85-1. In contrast, values of C chosen for rural catchments vary widely (see Appendix 2). The method may be less reliable for rural catchments both for this reason and because (in some cases) the design rainfall will be less accurately known.

Beca Carter Hollings & Ferner (1992) detail an empirical rainfall/runoff model, based on the Rational Method, for catchments in the Auckland region. Calibration of the model consisted of determining suitable values of C to apply with the Rational Method for different locations, degrees of urbanisation and rainfall durations. This calibration used rainfall-runoff data from 19 catchments. The method is listed in this section because it does not require more runoff information from the user.

Rainfall-runoff models can be used without calibration from measured data, but their accuracy is then unknown and may depend on the modeller's experience with similar catchments. The use of synthetic unit hydrographs is an example which has been the subject of some research.

3.3.1.3.2 Rainfall data with limited flow data There are two typical situations in which there are adequate rainfall records but limited flow data. One is when changes in land use are expected within the catchment, so that existing flow data will not apply directly to the design situation. The other situation occurs when flow records are absent but the lead-in time for design allows a few floods to be gauged. In much of New Zealand this situation does not occur, because recording rain gauges (i.e., those that log a virtually continuous rainfall record) have not been as widely deployed as flow measurement sites. However obtaining a reliable continuous long-term discharge record is difficult in some rivers, such as the Kowhai River at Kaikoura, which has excessive bed changes and the risk of debris flows.

The procedure in both these situations is to calibrate a rainfall-runoff model against the gauged storms. Any expected changes to land use and the drainage network are then made to the model, before modelling the design rainfall.

The unit hydrograph (e.g. Sutcliffe 1978, Cordery 1987) is one of the earliest and simplest catchment models, and is still widely used. Its disadvantages are the assumptions of spatially uniform rainfall and linear catchment behaviour (Cordery 1987), and the method has not received much research attention in the last twenty years.

Many different more complex rainfall-runoff models have been developed, and a complete review of them all is beyond the scope of this study (see Ibbitt and McKerchar 1992). A few which have been used in New Zealand are briefly discussed below. It is generally accepted (e.g. Pilgrim 1987, p. 187) that all these models should be regarded as approximate only until calibration data have been obtained.

MIKE-II is a hydraulic model of channel flow which can be coupled to a rainfall-runoff hydrologic model (either HYCEMOS or NAM). NAM treats a catchment as a set of cascading reservoirs; the theory behind this type of approach is described by Dooge (1973).

RORB is a non-linear model which computes hydrographs for sub-catchments and then routes contributions to the catchment outlet. Rainfalls may vary in space and time (Laurenson and Mein 1985). The model is calibrated from features of the catchment outflow hydrograph.

RORB has been used successfully on catchments in Canterbury and north of Auckland (Griffiths et al. 1989, Auckland Regional Water Board, 1989).

HYCEMOS (Ibbitt and McKerchar 1992) is a physically-based catchment model which nevertheless relies heavily on calibration against measured data. It was developed by NIWA and has been tested against New Zealand data. A catchment is modelled as a set of plane surfaces (i.e. hillsides) and gutters (i.e. stream channels).

3.3.2 Channel Resistance

3.3.2.1 The "standard" river In practical river design problems, engineers have largely used the Manning formula and relied on photographs of measured rivers (Chow 1959, Barnes 1967) or on tables of values (Henderson 1959, Chow 1959) for finding an appropriate value for Manning's n. A similar table is presented by AUSTROADS (1994), and Hicks and Mason (1991) have presented photographs and channel data for New Zealand rivers in a similar manner to Barnes (1967). A recent hydraulics text (French 1985) endorses these practices. However, there has been only a limited research effort to test how accurate they are, and the results have been equivocal.

Bray (1979) applied several formulas to gravel-bed streams in Alberta. He found that the Lacey formula performed well, despite being derived mainly from sand- and silt-bed rivers. The Manning equation performed well when n was determined by either the "modified Cowan" method (which allows for channel sinuosity) or an empirical formula derived from California data by Limerinos (1970). Keulegan's equation (i.e. a logarithmic velocity profile) and the Manning-Strickler equation performed relatively poorly.

Griffiths (1981) used theoretical considerations to determine the general form for a channel resistance equation expressing the friction factor for gravel-bed rivers. He then used 136 datasets from 46 New Zealand rivers to calibrate two such equations, one for use with fixed beds, and one for use when the bed was mobile and bedforms may develop. He noted that these equations accounted for about half the variance in friction factor; the remaining scatter was still high.

Bray and Davar (1987) summarised previous research, and tested several formulas for flow resistance against measured data. Based on this comparison, they recommended three formulas which were all compromises between previously proposed formulas. One of these formulas was loosely based on the Lacey regime theory, one on the formulas of Limerinos (1970) and Griffiths (1981), and the third was part based on Bray (1979) and the United Kingdom data reported by Charlton et al. (1978).

Jarrett (1984) analysed 75 discharge measurements from 21 steep streams in the United States. His derived values of Manning's n decreased with increasing depth, except at some sites at peak flow when bank vegetation increased the resistance. He also found a correlation between Manning's n and channel slope, and therefore proposed an empirical formula for n as a function of slope and mean depth. Jarrett (1990) describes further progress with this research.

Channel resistance in wide sand-bed rivers is generally dominated by the form resistance (i.e., resistance due to bedform shape rather than bed material) of bedforms including ripples, dunes and antidunes. The resistance is then strongly dependent on flow velocity, as there is a transitional plane-bed regime between the lower-velocity regime of ripples and dunes and the higher-velocity regime where antidunes are present (Engelund and Hansen 1967; van Rijn 1984). Einstein (1950) suggested separating the form drag of bedforms from the "skin friction" due to the sand grains. A great amount of research effort since then (e.g. van Rijn 1984; Shen et al. 1990) has confirmed this approach and has provided methods for estimating the total resistance.

3.3.2.2 Some complications and special cases Related to Jarrett's research is that of Bathurst et al. (1982) who found that in streams with very high bed roughness (boulders emerging above the water line, etc.) neither the Manning equation nor a logarithmic equation remained valid. An equation for flow resistance in steep rough streams has also been presented by Aguire-Pe and Fuentes (1990).

Arcement and Schneider (1989) provide a method for estimating Manning's n for vegetated flood plains. The method was calibrated against United States data for which photographs were presented as well as a written description. Some New Zealand floodplains, but not all, may be similar enough to make this method very useful.

Research has also been carried out (e.g. Kouwen 1992) on flow resistance of grassed channels, a situation applicable to many flood diversion spillways.

A further factor in determining flow resistance is combined channel and floodplain flow. The conventional approach, due to Henderson (1966), has been to subdivide the cross-section with imaginary lines which do not contribute to the wetted perimeter. More recently it has been recognised that these boundaries between parts of a compound channel contribute substantially to the flow resistance of the main channel (F.M. Henderson, pers. comm.; Smart 1992). Flume experiments (e.g. Knight and Shiono 1990) and turbulence modelling (e.g. Kawahara and Tamai 1989) are advancing our understanding of this process. Rajaratnam and Ahmadi (1981) state a simple prescription for the extent of the mixing layer; this result offers a pragmatic solution to the problem pending a more definitive result from the experiments and modelling.

STAGE TWO: TESTING TECHNIQUES AGAINST TYPICAL N.Z. SITES

4. FLOOD FREQUENCY ANALYSIS AT UNGAUGED SITES

4.1 Research Method

In many New Zealand design situations there is no measured flow record at the site, and techniques of using flow records from adjacent basins (see 3.3.1.2) or rainfall records (see 3.3.1.3) must be employed. In this part of the study, the performance of three of these methods was tested against "at-site" estimates of flood events. One example of a rainfall/runoff model was also tested. In the absence of flow records approaching 100 years in duration, the "at-site" estimates must be taken as the "true" values.

Methods of estimating extreme flood flows were tested at 7 gauged sites to obtain direct statistical estimates of extreme flood events. Six of the sites were not used in developing the regional flood estimation (RFE) maps of McKerchar & Pearson (1989), and therefore offer an independent check of that method. These "at-site" estimates were then compared with "no-data" estimates obtained using the Rational Method and TM61 (see 3.3.1.3) and the regional flood estimation method (see 3.3.1.2).

The seventh site, Waikanae, was used for the regional flood estimation maps, but was chosen for this study because a RORB catchment model has been applied there. Four estimates in all could therefore be compared with the "at-site" estimate.

Standard Tideda processes (Rodgers and Thompson, 1992) were used to check the quality of the stage/discharge rating curves, and list annual maxima for those years for which complete records existed. The EV1 frequency distribution (a particular case of the GEV) was fitted to the annual maxima using the method of Probability Weighted Moments (PWM) (McKerchar and Pearson, 1989), and the applicability of the EV1 distribution examined.

Application of the Rational Method and TM61 requires estimation of "time of concentration" T_c using empirical formulae. These formulae use estimates of the main channel length and slope which are determined from maps. With the time of concentration T_c, rainfall intensities for that duration and specified frequencies are obtained from contour maps in Tomlinson (1980). The TM61 method also requires estimation of catchment slope, cover and shape coefficients, and a runoff coefficient must be chosen for the Rational Method. The TM61 coefficients are determined from estimates of channel length and slope as measured from maps, and information about catchment cover and soil types, also from maps. The Rational Method coefficient is normally taken from a table, but tables derived specifically for New Zealand conditions appear to not exist except for greater Auckland.

4.1.1 Selection of Basin and Checks of Data

Several criteria governed the choice of the seven basins:

- Records longer than 10 years;
- Reasonable geographical coverage;
- A range of catchment sizes, including catchments smaller than 100 km^2; and
- Reliable stage/discharge ratings.

Details of the basins selected for use in the study are listed in Table 1, and the locations of these basins are shown in Fig.1. Four North Island and three South Island basins were chosen.

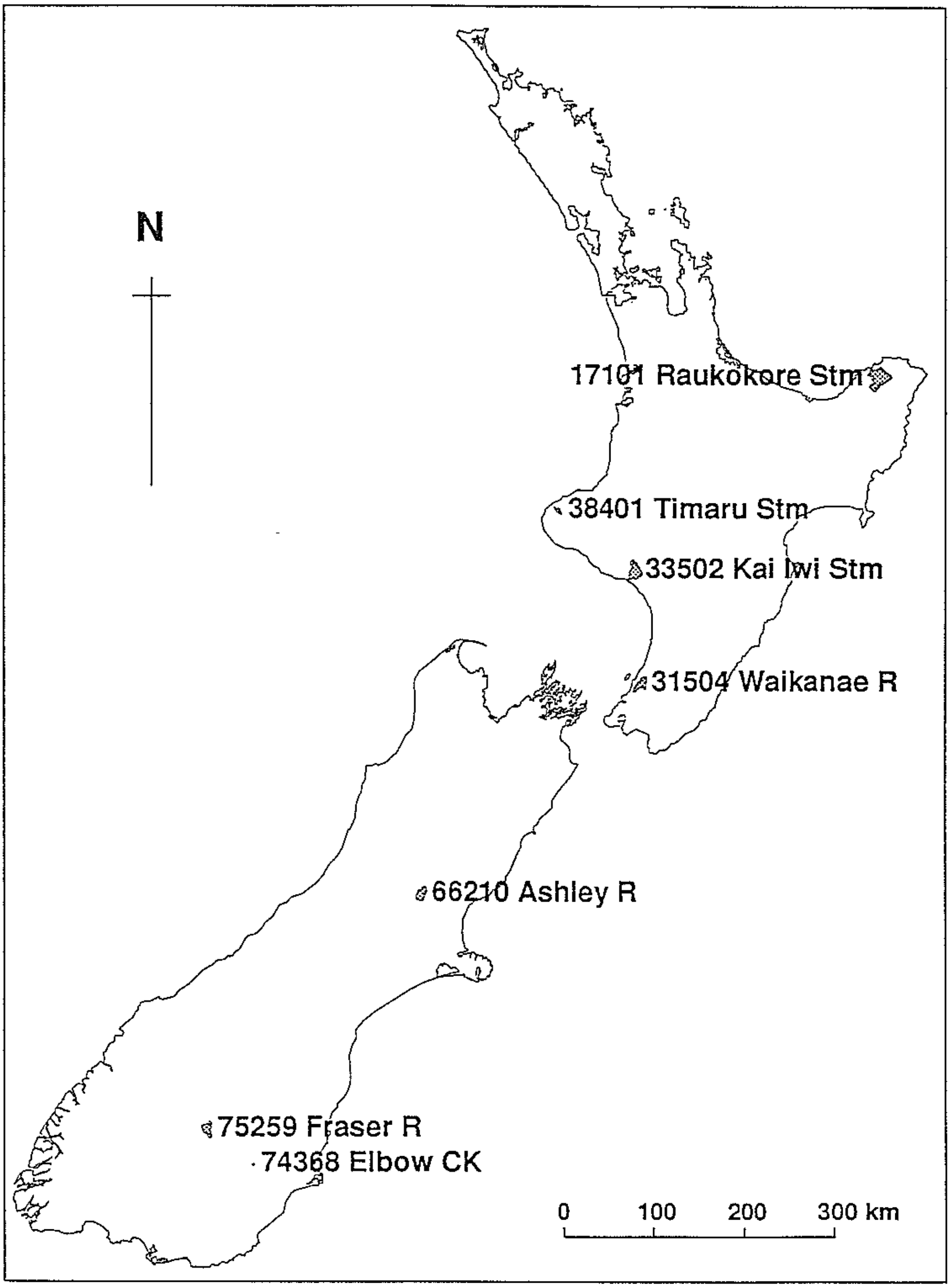


Fig. 1: Map showing locations of seven basins with streamgauge recorders used for flood frequency analysis in this study.

Table 1. Details of six basins selected for analysis.

Site No.	17101	33502	38401	66210	74368	75259	31504
River/stream	Raukokore	Kai Iwi	Timaru	Ashley	Elbow Ck	Fraser	Waikanae
Location	SH 35 Br.	Handley Rd	SH 45	Lees Valley	Muster Huts	Old Man Range	Water Treatment
Basin area A km^2	351	192	27.0	121	1.24	122	124
Mean discharge m^3/s	31.1	1.48	1.84	4.01	0.0349	2.25	
Runoff m/yr	2.80	0.243	2.14	1.05	0.888	0.583	

Rating curves for the stations were plotted to check the consistency of the methods used in their extrapolation. As a result of this check, ratings for the Fraser at Old Man Range (station 75259) were completely revised from those held in the archive (McKerchar and Pearson, 1989).

The range of basin areas is from 1.24 km^2 to 351 km^2, and mean area is 134 km^2. The basin area and mean discharge (m^3/s) were used to calculate the mean annual runoff for each basin which ranges from 0.24 m/yr to 2.8 m/yr. It is remarkable that both extremes are for North Island basins, whereas both drier and wetter basins in the country are in the South Island. It was hoped to include at least one basin for the much wetter western Southern Alps, but the one possible candidate (Bealey at Arthurs Pass, station 66419) was not used because its rating curves did not appear to be reliable.

4.2 Results

4.2.1 Frequency Analysis of the Gauged Site Flow Data

The annual maxima are listed in Table 2, and the key results from the application of the EV1 analysis are in Table 3. The extreme value analysis included an estimate of a parameter "k" and a test statistic to indicate whether it differed significantly from zero. This parameter is zero for an EV1 distribution, and non-zero for more general EV2 and EV3 distributions. We used a significance test to show that in five of the six cases, the k values estimated from the data were consistent with an EV1 distribution. This was considered sufficient to accept that the annual maxima followed an EV1 distribution.

In the sixth case (Elbow Creek) where k was negative, the test statistic was less than the 5% level, but not the 1% level. In other words, there was some evidence that the EV2 distribution, which curves upwards on Gumbel probability paper, might be more appropriate for Elbow Creek data. An alternative approach was adopted for Elbow Creek, by repeating the EV1 analysis with just eight annual maxima selected as the maxima for each two year partition of the data. These biennial maxima were represented well by an EV1 distribution, and the values in Table 3 pertain to this biennial analysis.

The results of the at-site EV1 analyses as given in Table 3 are the mean annual flood Q_b (which has an AEP of 1 in 2.33) and the flood of 1 in 100 AEP, Q_{100}. These values are sufficient to draw an EV1 frequency line and define a flood percentile for any required AEP.

Table 2. Annual maxima for the selected stations (peak flows in m^3/s).

Site No.	17101	33502	38401	66210	74368	75259	31504
River/stream	Raukokore	Kai Iwi	Timaru	Ashley	Elbow Ck	Fraser	Waikanae
Location	SH 35 Br.	Handley Rd	SH 45	Lees Valley	Muster Huts	Old Man Range	Water Treatment
Year							
1970						20.5	
1971						28.2	
1972						22.5	
1973						25.5	
1974						14.7	
1975						17.0	169
1976						53.0	177
1977						61.4	142
1978		44.5		69.3			99
1979		23.5		87.4		34.8	127
1980	742	21.0	57.5	117.9	**1.818**	84.6	232
1981	726	25.1	31.7	68.3	0.763	32.0	87
1982	438	26.4	49.1	80.1	**3.092**	29.4	181
1983	720	21.3	57.6	67.1	1.743	51.7	143
1984	579	13.3	46.3	98.0	**0.847**	32.2	118
1985		28.1	55.0	55.7	0.670	11.6	304
1986	1500	21.5	98.2	125.9	1.002	23.8	199
1987	737	14.0	49.9	121.0	**1.104**	36.0	72
1988	740	19.5	44.8	74.7	**1.758**	31.7	60
1989	1257	24.5	65.5	55.2	0.981	26.2	108
1990	795	24.5	138.7	34.8	0.687	15.3	273
1991	624	33.3	59.4	56.2	**1.241**	38.5	183
1992	938	30.5	50.5	39.0	1.361	24.3	
1993	555	14.0	74.2	127.4	**1.689**	46.4	
1994	575	34.4		79.9	**6.470**		
1995	880	15.2		42.1	2.006		

Note: The biennial maxima for Elbow Creek are shown in bold type.

Table 3. EV1 analysis results.

Site No.	17101	33502	38401	66210	74368	75259	31504
River/stream	Raukokore	Kai Iwi	Timaru	Ashley	Elbow Ck	Fraser	Waikanae
Location	SH 35 Br.	Handley Rd	SH 45	Lees Valley	Muster Huts	Old Man Range	Water Treatment
No. years	15	18	14	18	16	23	17
Qb	787	24.1	62.7	77.8	1.7	33.1	119
Qb/A0.8	7.24	0.36	4.49	1.68	1.43	0.71	2.51
Q100	1630	50.5	140	178	5.37	86	386
Q100/Qb	2.07	2.09	2.23	2.29	3.16	2.60	3.24

4.2.2 Testing of "No-data" Methods and RORB Model

Estimates of Q_b and Q_{100}, that assume no data for the six stations, are given in Table 4. These estimates were obtained using the Rational Method, TM61 and the regional flood estimation (RFE) method of McKerchar & Pearson (1989). Factors and coefficients used to obtain these estimates are also given in Table 4.

Table 4. Details of "no-data" estimates.

Site No.		17101	33502	38401	66210	74368	75259	31504
River/stream		Raukokore	Kai Iwi	Timaru	Ashley	Elbow Ck	Fraser	Waikanae
Location		SH 35 Br.	Handley Rd	SH 45	Lees Valley	Muster Huts	Old Man Range	Water Treatment
	Units							
Basin area A	km2	351	192	27.0	121	1.24	122	124
Elevation range	m	1395	435	1330	805	86	1040	730
Length main channel	km	49.3	50.1	14.9	18.5	1.44	22.7	15.7
Channel slope	m/m	0.0067	0.0048	0.0424	0.030	0.055	0.027	.0185
Estimated Tc	hrs	12	15	3	4	0.5	5	3.5
Rainfall intensity for Tc, 1/2.33 AEP	mm/h	10.1	5.5	26.95	13.2	20.1	4.3	9.9
Rainfall intensity for Tc, 1/100 AEP	mm/h	21.3	11.6	56.5	27.9	46.0	9.0	20.9
Rational Method								
C		0.5	0.2	0.2	0.3	0.3	0.3	0.3
Q_b	m3/s	493	59	40	134	2.1	43	103
Q_{100}	m3/s	1038	124	85	281	4.8	91	217
TM61								
C		730	1020	1600	1700	480	3000	1050
$R_{2.33}$, R_{100}		0.418, 0.881	0.251, 0.528	0.605, 1.274	0.340, 0.716	0.177, 0.404	0.118, 0.249	0.239 0.504
S		1.018	0.83	0.56	0.97	1.12	0.83	1.30
Q_b	m3/s	372	152	89	284	1.6	150	169
Q_{100}	m3/s	783	321	188	598	3.5	317	357
Regional method								
$Q_b/A^{0.8}$		6	0.8	6	1.4	1.0	0.55	2.7
Q_{100}/Q_b		2.6	2.0	2.3	2.9	3.6	3.5	2.2
Q_b	m3/s	652	54	84	65	1.2	26	128
Q_{100}	m3/s	1670	107	193	188	4.3	90	282

The estimate of the runoff coefficient C for the Rational Method is not objective because there is not a consistent set of nationally applicable coefficients. We therefore used a table in Pilgrim and Cordery (1993) which specifies values of C for particular descriptions of catchment conditions for urban situations. From this table, a value of 0.3 was adopted for the Waikanae and the three South Island basins and a value of 0.2 for Kai Iwi Stream and Timaru Stream in the North Island. The value of 0.3 is a mid-range value for "lawns, heavy soil, steep 7 percent" which seemed to be the most appropriate value for the South Island basins. The value of 0.2 for two North Island basins is an upper range value for "lawns, sandy soil, steep 7 percent", and was assigned because Kai Iwi is adjacent to the coast and the soils likely to be sandy, and Timaru, which drains radially from Mt Taranaki, is likely to have light volcanic soils. For Raukokore Stream, a value of 0.5 was adopted after consideration of rainfall/runoff studies for the adjacent Motu basin (Riddell, 1980). These coefficients, the estimated time of concentration, T_C, and two estimated rainfall intensities, those with AEPs of 1/2.33 and 1/100, are given in Table 4 together with the flood estimates.

Calibration of the RORB model requires assessment of rainfall and direct runoff volumes for several storms. Rainfall excess is determined as the volume of rainfall that is measured as direct, or storm response runoff, and the remaining rainfall volume is apportioned as initial and continuing losses. For each storm, a model storage parameter is determined that provides the best fit between the modeled and observed direct runoff, and the best parameters for all the calibration events selected. The model is normally verified with other recorded events before being used to determine the direct runoff hydrograph resulting from application of a design rainstorm over the catchment.

The RORB model offers a significant advance over the traditional unit hydrograph method because it allows non-uniform spatial rainfall distributions and non-uniform losses over a catchment to be modelled explicitly. It has been widely adopted in engineering practice, especially in Australia.

4.2.3 Comparison of "No-data" and RORB Estimates with "At-site" Estimates

In Table 5 the estimates of Q_b and Q_{100} from Tables 3 and 4 are summarised.

Table 5. Estimates of Q_b and Q_{100} from Tables 3 and 4.

	17101	33502	38401	66210	74368	75259	31504
River/stream	Raukokore	Kai Iwi	Timaru	Ashley	Elbow Ck	Fraser	Waikanae
Site No.	SH 35 Br.	Handley Rd	SH 45	Lees Valley	Muster Huts	Old Man Range	Water Treatment
Q_b							
At-site	787	24.1	62.7	77.8	1.7	33.1	119
Regional	652	54	84	65	1.2	26	128
TM61	372	152	89	284	1.6	150	169
Rational	493	59	40	134	2.1	43	103
Q_{100}							
At-site	1630	50.5	140	178	5.37	86	386
Regional	1670	107	193	188	4.3	90	282
TM61	783	321	188	598	3.5	317	357
Rational	1038	124	85	281	4.8	91	217
RORB model							495

The results in Table 5 are also displayed as a series of histograms in Fig.2. This set of plots shows immediately that for five of the six basins, the TM61 estimates of Q_b and Q_{100} are inferior to the other two "no-data" methods. To present a numerical assessment, the percent error E for each "no-data" estimate has been compared with the "at-site" estimate" by calculating

$$E = 100\ (Q_{no\text{-}data} - Q_{at\text{-}site})\ /\ Q_{at\text{-}site}$$

and the results for the Q_{100} estimates are given in Table 6. Note that negative values indicate that the "no-data" method under-estimates the flood.

Figure 2: Comparisons of "at-site" and "no-data" Qb and Q100 estimates

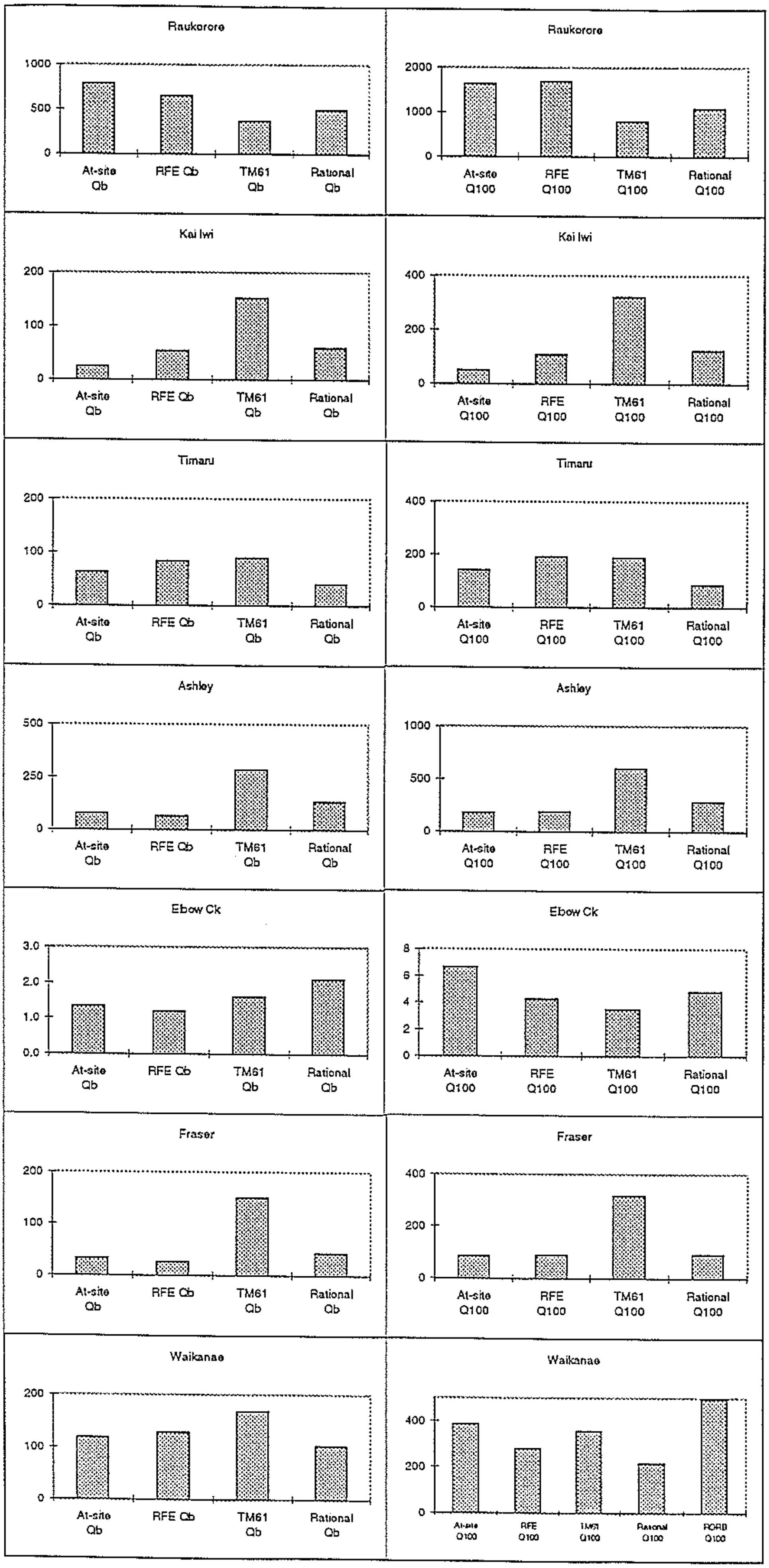


Table 6. Percentage errors of estimate for "no-data" 1/100 AEP estimates.

	Raukokore SH 35 Br.	Kai Iwi Handley Rd	Timaru SH 45	Ashley Lees Valley	Elbow Ck Muster Huts	Fraser Old Man Range	Waikanae Water Treatment	Mean	RMS Error
RFE	4	113	38	6	-36	4	-27	15	49
TM61	-52	536	34	236	-47	269	-8	138	219
Rational	-34	146	-39	58	-28	6	-44	9	70
RORB							28		

4.3 Discussion

The analysis presented above presumes that the "at-site" estimates in Table 3 are correct, whereas in reality they are only estimates of true, but unknown values. The estimates are affected by sampling error and model error. The sampling error occurs because we have flood maxima for only a limited number of years, and model error occurs because we in fact do not know the correct distribution for the annual maxima. However, results in McKerchar and Pearson (1989), which indicate that the EV1 distribution is applicable to many flood records in NZ, would suggest that the model error should be relatively low.

The magnitude of the sampling error for a particular record is a function of the mean and variance of the annual maxima and the number of years of record. For the five records where the annual maxima were used, it lies in the range of 13% to 15% for the 1/100 AEP estimates, and for Elbow Ck, where the biennial maxima were used, it is 26% for the 1/100 AEP estimate.

The six catchments used in the analysis show a range of hydrological conditions, even though none are from the wetter rainfall zones of the western Southern Alps. The data for Kai Iwi was particularly interesting: the annual runoff (Table 2) was unusually low for a North Island catchment and reflects the fact that it drains an area which lies to the southeast of Mt Taranaki, and is in a partial rain shadow, which is evident from close inspection of rainfall maps. None of the "no-data" methods was able to predict the unusually low flood statistics for this river (Table 6) but the RFE came closest.

In estimating the design rainfalls for the Rational Method and TM61, area reduction factors, which reduce point rainfall estimates to mean rainfalls over an area, were estimated to be not less than 0.89 for the areas and time of concentration considered in this study, using the British graph given in Tomlinson (1980). However these factors were not applied, because there is doubt about the applicability of such factors in New Zealand (Tomlinson, 1978).

The results presented show that characteristically quite wide variations in the design flood estimates occur. Of the three "no-data" methods, Table 5 shows that the RFE provides estimates with least error. The data in Table 6 show that all the methods are biased, such that the "no-data" estimates exceed the "at-site" estimates. For the RFE and the Rational Method, the biases might be considered tolerable, but for TM61, they are excessive. The biases and RMS errors are all affected by the particularly poor results for Kai Iwi, and hopefully for a larger sample of data (more basins) these results would improve.

It was surprising that the Rational Method performs as well as it does, despite a choice of coefficients from an overseas published handbook and some local knowledge, and uncertainties inherent in estimates of design rainfalls. It must be acknowledged that the choice of coefficients is not totally objective, and other engineers might choose different coefficients that would yield quite different no-data estimates. However it should not be discarded; it is a useful complement for dealing with small basins (e.g. less than 10 km^2) where the RFE method does not perform particularly well, and in urban situations for which the RFE method is not designed.

A second surprise is that most of the TM61 estimates are so much in error. Given the results in Fig. 2 and Tables 5 and 6, its continued use for design flood estimation cannot be recommended.

The results for Waikanae are unexpected; in this example the TM61 method predicted Q_{100} remarkably well. The RFE method was not very successful, despite the Waikanae site being one of those used in preparing the RFE maps. This illustrates difficulties with the RFE method in some locations where the spatial variation of the parameters derived in the method is marked. Further work, as yet unpublished, has been carried out to improve the RFE maps in the Kapiti region.

From the analysis presented here, the RFE method is the preferred "no-data" method. Where only short at-site records exist, these can be combined with the regional estimates using procedures described and illustrated in McKerchar and Pearson (1989). Errors in estimates are reduced dramatically when at-site data are incorporated into the analysis.

Application of the RORB model to Waikanae overpredicted Q_{100} by 28%. The model estimate differs from the other three estimates in that some at-site runoff data obtained from four storms used to calibrate the model.

This single result indicates that models, even when calibrated against several measured events, may give comparable estimates of flood flow to "no-data" methods, so that the cost of setting up a model may not always be justified. Nevertheless, models may be very useful in parts of New Zealand where the RFE method and other "no-data" methods perform poorly. In addition, catchment models are the only way of predicting very extreme floods (those for which the extrapolation required of measured flow records cannot be relied on), or the effect on flood peaks of changes in land use or other physical factors.

5. DETERMINATION OF CHANNEL RESISTANCE (I.E. MANNING'S ROUGHNESS COEFFICIENT n)

5.1 Research Method

This part of the study was directed at indicating whether flow resistance could be reliably estimated for design flood flows in New Zealand rivers. The approach was therefore different from other studies which have made use of a large number of observations at lower flows.

Estimates were made of the flow resistance at 7 gauged sites chosen to provide a variety of channel forms. These sites were also chosen to avoid the complicating factors (overbank flow,

very low depth relative to sediment size) alluded to in Section 3.3.2.2, so that the flow resistance could be described by a single parameter. The results have been presented in terms of Manning's n, for consistency both with Hicks & Mason (1991) and with common New Zealand practice.
Measurements of cross-sections, gauged flows and water levels, and bed surface material at these sites were used to obtain actual values of channel resistance for comparison with all the estimated values. Details of the sites chosen are given in Table 7:

Table 7. Sites used for channel flow resistance study.

River	Site	Site No.	Region	Catchment Area (km2)	Bed Material
Waimana	Ogilvie's Bridge	15536	Bay of Plenty	207	gravel
Hangaroa	Doneraille Park	21437	Hawkes Bay	596	bedrock
Waitara	Tarata	39501	Taranaki	725	silt/sand
Mangakahia	gorge	46618	Northland	246	gravel
Riwaka South Branch	Moss Bush	56901	Nelson	46	gravel
Maerewhenua	Kelly's Gorge	71106	North Otago	187	gravel
Haast	Roaring Billy	86802	Westland	1020	gravel

5.1.1 Estimates of Manning's n

The methods of estimating resistance were:

- Comparison with the tabulated descriptions given by Henderson (1966), and with those given by AUSTROADS (1994);
- For gravel-bed rivers, calculation using the formulas given by Griffiths (1981), and also using formulas recommended by Bray & Davar (1987); and
- Visual assessment, from site visits or from inspecting photographs, taking the particle size distribution of bed material into account.

The tabulated values in AUSTROADS (1994) are given solely for verbal descriptions of the channel. In contrast, Henderson (1966) provides some descriptions but for straight gravel reaches prescribes a form of the Strickler formula:

$$n = .031\ d_{75}^{1/6}$$

where d_{75} is the size for which 75% of the bed material is smaller, expressed in feet.

The two formulas recommended by Bray & Davar (1987) and used in this study are their equations (27) and (28) respectively:

$$1/\sqrt{f} = 1.9\ (R/d_{84})^{0.25}$$

$$1/\sqrt{f} = 2\ \log_{10}(R/d_{84}) + 1.1$$

where R is the hydraulic radius, d_{84} is the size for which 84% of the bed surface material is smaller, and f is the Darcy-Weisbach friction factor, related to Manning's n by:

$$n = 0.113 \sqrt{f} R^{1/6}$$

Two "rigid-bed" equations recommended by Griffiths (1981) are also used in this study:

$$1/\sqrt{f} = 1.33 \, (R/d_{50})^{0.287}$$

$$1/\sqrt{f} = 1.98 \log_{10}(R/d_{50})+0.76$$

where d_{50} is the size for which 50% of the bed material is smaller.

Griffiths also derived a "mobile-bed" equation, but it has not been used in this study. It was intended to account for the extra flow resistance of bedforms such as dunes. However, it could be calibrated on only a small dataset, and the values of channel resistance obtained are comparable to estimates using his rigid-bed equations.

The visual assessment was carried out by two observers independently of one another, using Hicks & Mason (1991). Both observers had a wide range of experience in selecting values of Manning's n and a knowledge of the physical factors which contribute to its value. The observers' notes are included as Appendix 3. Their technique was to choose two or more sites documented by Hicks and Mason that were comparable to the site being assessed, and choose an average of the measured values of Manning's n for high flows at these sites.

5.1.2 Derivation of Manning's n from Site Measurements

To measure directly the flow resistance of a reach, it is necessary to survey at least two cross-sections and measure water levels at these sections and the coincident flow rate, and from these calculate the mean velocity at each section and the energy slope. As the flow resistance of most interest is usually that which applies at high flows, these measurements require observers being on site during floods. Hicks & Mason (1991) obtained their data in this way, as did Griffiths (1981), but they were not always able to obtain data from significant floods (i.e. events comparable to the mean annual flood). Given the short time frame of the present study and the aim of examining design floods, it was considered unwise to rely on the chance that suitable floods would occur.

Instead, seven sites were chosen from a River Classification Survey undertaken in 1985 by the Ministry of Works & Development. In this survey, information was collated about river reaches each at or near a gauged site and chosen to be representative of rivers in the region. This information included:

- A standard "questionnaire" with mainly descriptive information about the reach and its surrounds;
- Four or five surveyed cross-sections defining the reach;
- A plan locating the sections with the locations of other features such as gravel bars and rock outcrops sketched;

- A longitudinal profile along the thalweg;
- A size analysis of the bed surface material by the Wolman method;
- Photographs taken from the river banks; and
- An aerial photograph with the sections located.

For a site to be suitable for the present study required a reasonably uniform reach. In addition, the water level recorder had to be at or near one of the surveyed cross-sections, the cross-section survey levels needed to be tied to the datum of the water level recorder, and there needed to be a reasonably stable stage/discharge rating. These three requirements eliminated many of the sites, but it was still possible to find a selection of sites (described in Table 7) that were representative of a range of New Zealand rivers.

To choose the flows for which channel resistance would be determined, the rated flow record for each site was examined. Usually three flows were modelled, with the lowest being the lowest of the annual flood peaks. Where possible, the highest rated flow was used as the highest of the three modelled values. However, at some sites the surveyed channel could not carry this flow, and the bankfull flow was used instead. The modelled flows are therefore a range of the largest flows for which the channel could be modelled. The middle flow modelled was approximately an average of the other two.

One shortcoming of most of these sites compared to those used by Hicks & Mason (1991) was that the water level was available at just one cross-section - at the water level recorder. This meant that the water surface slope had to be inferred from the channel geometry, assuming uniform flow, as must be done in the design situation. The simplest way to do this (but not the most accurate) was to take the mean thalweg slope to be the water surface slope. This could give undue emphasis to minor changes in cross-section shape, and it was instead assumed that the product $AR^{2/3}$, (where A is the cross-section area) was close to constant along the reach. This product features in the Manning equation expressed for flow rate Q:

$$Q = A\,R^{2/3}\,S^{1/2}/n$$

where S is the energy slope so that constant values of $AR^{2/3}$ and n are equivalent to a constant energy slope.

Steady state computations were carried out to compute Manning's n values between adjacent cross-sections and adjust these values to best match this criterion. An average of the computed Manning's n values, weighted by the distance between cross-sections, was taken as the measured value for the reach. (The existence of pools and riffles meant that a perfect match was not always possible). These computations took account of minor variations in velocity head along the reach (due to variations in cross-sectional area) which mean that the energy slope did not coincide with the water surface slope.

This approach was considered a reasonable one with steep channel slopes and reaches several hundred metres long, because the geometry required the energy slope to be reasonably close to the channel slope, otherwise depths upstream and downstream would vary markedly.

To check on the accuracy of this approach, further computations were done varying the channel resistance so that $AR^{2/3}$ was no longer constant along the reach. From these computations, and considering the range of plausible deviations from the assumed water surface slope (i.e. the range of plausible variations between upstream and downstream depths) it was possible to produce a somewhat subjective assessment of the accuracy of the original determinations of channel resistance. It was concluded that variations from the assumed water surface slope were unlikely to cause errors in excess of 15% in estimates of Manning's n.

The above approach does not work well in lowland rivers with low channel slope. In these channels a wide range of energy slope (hence channel resistance) might apply. The only such river of the 7 chosen was the Waitara, where the water surface slope was recorded during a flood in 1967. This slope has been used to determine the channel resistance, but there were difficulties (discussed below) in applying this slope to a wide range of flows.

5.2 Results

The various estimates of channel resistance, expressed as Manning's roughness coefficient n, are set out in Table 8. The actual values determined, as described in 5.1.2, are also included in Table 8; for all except the Waitara River these are the means of three values obtained for different flows. Table 9 shows the percentage deviation from the measured values and also includes the computed mean and standard deviation of these deviations. The mean indicates the bias of the method, i.e. whether it is consistently overpredicting or underpredicting; and the standard deviation indicates the precision of the method, with a high value corresponding to well-scattered estimates.

The variation of actual channel resistance with flow is shown in Table 10 and graphed in Figure 3 for 6 rivers (excluding the Waitara). For all the 6 rivers, the middle of the three flow values corresponds approximately with the mean annual flood.

The two formulas proposed by Bray & Davar (1981) for the resistance of gravel-bed rivers both overestimated Manning's n, by 9% and 21% respectively on average. Griffith's (1981) two formulas performed better, with no apparent bias and relatively good precision. All these formulas gave higher estimates of Manning's n than the estimate using Henderson (1966). This might be expected, given that these formulas have been obtained by regression analyses of field data, in contrast to Henderson's use of a theoretical Strickler formula for gravel-bed rivers. The idealised assumption of Henderson (1966) has produced underestimates of channel resistance, except for the Haast River, the largest in this study, but the good precision (20% standard deviation) suggests that a simple empirical correction could be made to the formula.

The table of Manning's n provided by AUSTROADS (1994) appears from this study to be of limited use for New Zealand rivers. The results show little bias but wide scatter, and do not fully illustrate the difficulties in using these tables. For each written channel description an extremely wide range of Manning's n is given, so that the actual choice of a value is largely an intuitive one. This criticism might also be made of Henderson's (1966) table when used for non-gravel beds.

Use of the Hicks & Mason handbook is also not as useful as might be hoped. Values obtained by the two independent observers generally varied both from one another and from the actual values. Overall, both observers overestimated Manning's n, by 6% and 18% respectively, and the precision was comparable to that of the Bray & Darvar formulas and poorer than that of the Griffiths formulas. Part of the cause for this might be traced to the examples in Hicks & Mason, some of which appear similar to one another but reveal quite divergent channel resistance. The final choice of a value of Manning's n then becomes partly intuitive.

One difficulty in applying the data of Hicks & Mason to design floods is that flows of the order of the mean annual flood are not common. Many sites show Manning's n decreasing with increasing flow, so that the designer must consider whether this trend will continue as the design flows are approached. Figure 3 indicates that, for flows around the mean annual flood, the Mangakahia and to some extent the Haast follow this trend, but four other sites do not. This might be expected at these higher flows, as the effects of bank vegetation will be expected to add to resistance at high flows.

Table 9 also shows the mean variation of all the estimated values of Manning's n from the measured value. These values range up to $\pm$ 40%, and have two possible causes:

(i) The measured values are in error; and

(ii) Individual sites have features that lead to different channel resistance from that expected from a site inspection and bed surface sample.

It is likely that both causes are valid. The likely error in the measured values due to the assumption of a particular water surface slope has been assessed, above, as up to 15%. Another source of error is in the assumed flow rate; both individual flow gaugings and the ratings derived from them may be in error by up to 10%.

The sites used by Hicks and Mason (1991) often indicate a wide range of channel resistance for apparently similar channels, and it must be accepted that some variation in resistance remains unexplained, both between sites and at a site for different flows. The bed surface material size distribution may be a source of error in the estimated values; only the AUSTROADS estimates do not depend on this distribution in any way.

Regardless of the reason for the discrepancies between measured and estimated values of Manning's n, the uncertainty for design engineers is real. The use in this study of assumed water surface slope, and of bed material sizes based on a Wolman-type sample, mirror the likely design assumptions. Further, a design flow accurate to within 10% is unobtainable at most sites, as indicated in the flood frequency part of the present study.

The Waitara River has been treated differently in this study, for two reasons: an actual water surface slope was measured during a flood in 1967, and the river is meandering with a very low channel slope (estimated as .0006 when the River Classification survey was undertaken). The water surface slope observed in 1967 was .00154, and has been applied in this study to a flow of 600 m^3/s, the highest for which reliable cross-section information existed. Water levels during these 1967 observations were 1 metre higher than those for the 600 m^3/s flow. It was considered that the water surface slope for events much smaller than 600 m^3/s might differ significantly from .00154, especially as the channel slope has been assessed at .0006.

The bank vegetation along the Waitara River within 1-2 km of the surveyed reach varied from pasture to dense willows, and the reach itself had willows and scrub. It is therefore likely that the local water surface slope is considerably more than the channel slope during floods high enough to reach the vegetation. This illustrates a major difficulty in determining water levels in lowland rivers. Because the water surface slope can vary markedly from the channel slope, applying the Manning equation using estimates of the local channel slope and the local channel resistance is not sufficient to determine water levels, and flow conditions upstream and downstream must also be considered. This means in practice that a steady-state hydraulic numerical model should be made of a considerable length of channel upstream and downstream of the site. In some circumstances a dynamic model will be needed.

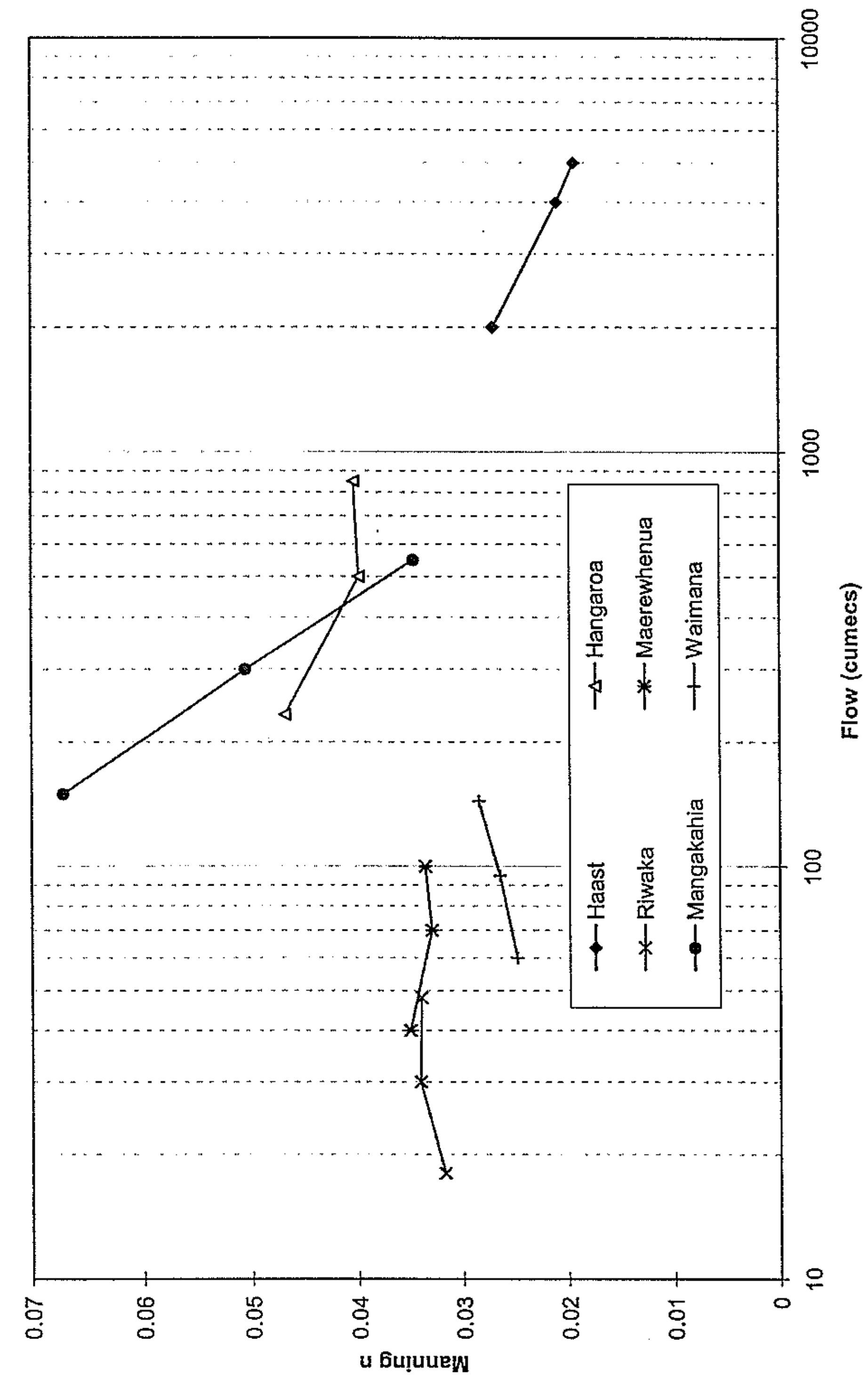


Figure 3: Variation of measured Manning's n with flow rate

River	Site No.	Bray & Davar eq (27)	Bray & Davar eq (28)	Griffiths (1981) power eq.	Griffiths (1981) logarithmic eq.	Hicks & Mason (1992) - observer #1	Hicks & Mason (1992) - observer #2	Henderson (1966)	AUSTROADS (1994)	**Measured**
Waimana	15536	0.035	0.038	0.033	0.032	0.034	0.045	0.025	0.045	0.027
Mangakahia	46618	0.035	0.038	0.038	0.037	0.035	0.034	0.027	0.045	0.051
Riwaka	56901	0.040	0.046	0.038	0.036	0.035	0.048	0.026	0.033	0.033
Maerewhenua	71106	0.032	0.035	0.031	0.030	0.031	0.026	0.024	0.040	0.035
Haast	86802	0.030	0.033	0.023	0.027	0.032	0.031	0.023	0.035	0.023
Hangaroa	21437					0.040	0.030	0.036	0.040	0.042
Waitara	39501					0.040	0.040	0.030	0.035	0.057

Table 8: Measured and estimated channel resistance, expressed as Manning n

Note: Measured values (except for Waitara) are the mean of three values

River	Site No.	Bray & Davar eq (27)	Bray & Davar eq (28)	Griffiths (1981) power eq.	Griffiths (1981) logarithmic eq.	Hicks & Mason (1992) - observer #1	Hicks & Mason (1992) - observer #2	Henderson (1966)	AUSTROADS (1994)	mean of 8 methods
Waimana	15536	30	42	23	18	28	69	-7	69	34
Mangakahia	46618	-32	-26	-24	-27	-31	-33	-47	-11	-29
Riwaka	56901	20	39	15	8	5	44	-21	-1	14
Maerewhenua	71106	-7	0	-13	-14	-11	-26	-32	14	-11
Haast	86802	32	48	3	21	42	38	4	56	30
Hangaroa	21437					-6	-29	-15	-6	
Waitara	39501					-30	-30	-47	-39	
mean variation - gravel bed		9	21	1	1	6	18	-21	25	
standard deviation - gravel bed		28	32	19	21	29	45	20	35	
mean variation - all 7 sites						0	5	-24	12	
standard deviation - all 7 sites						28	44	19	38	

Table 9: Percentage variation of estimated Manning's n from measured value

		n adopted for Table 8	Flow rates			Values of Manning's n		
River	Site No.		Q_1	Q_2	Q_3	n for Q_1	n for Q_2	n for Q_3
Waimana	15536	0.027	60	95	144	0.025	0.027	0.029
Mangakahia	46618	0.051	150	300	550	0.067	0.051	0.035
Riwaka	56901	0.033	18	30	48	0.032	0.034	0.034
Maerewhenua	71106	0.035	40	70	100	0.035	0.033	0.034
Haast	86802	0.023	2000	4000	5000	0.027	0.021	0.019
Hangaroa	21437	0.042	233	500	850	0.047	0.040	0.040
Waitara	39501	0.057			600			0.057

Table 10: Measured channel resistance for different flows, expressed as Manning n
Note: Values of n adopted for Table 8 (except for Waitara) are the mean of the three values

6. THE EFFECT OF ERRORS ON ESTIMATES OF WATER LEVEL AND VELOCITY

The information in this report on flood flows and on channel flow resistance (characterised by Manning's n) needs to be put into perspective by quantifying the effect of an error in assessing either the flow rate or Manning's n. The usual design problems are to determine velocity (usually for scour calculations) and water level. The Manning equation is:

$$V = R^{2/3} S^{1/2} /n$$

where V is the mean velocity in metres per second
R is the hydraulic radius in metres
S is the energy slope

n is Manning's roughness coefficient

The flow rate Q can also be expressed using the Manning equation, as:

$$Q = A R^{2/3} S^{1/2} /n$$

where Q is in cubic metres per second

A is the cross-sectional area in square metres

In wide shallow rivers with reasonably steep banks, the cross-sectional area A is given approximately by:

$$A=By$$

where B is the channel width

y is the depth

In addition, R and y are approximately equal. It follows that the depth y and velocity V can be expressed:

$$y = (Q/B)^{3/5} S^{-3./10} n^{3/5}$$

$$V = (Q/B)^{2/5} S^{3/10} n^{-3/5}$$

In wide triangular cross-sections A is given approximately by:

$$A = By^2/2$$

and y is approximately twice R. The depth y and velocity v can then be expressed:

$$y = 2^{5/8} (Q/B)^{3/8} S^{-3/16} n^{3/8}$$

$$V = 2^{-1/4}(Q/B)^{1/4}S^{3/8}n^{-3/4}$$

Most natural river channels fall between these two idealisations, so that (for instance):

- A 10% underestimate of n results in an underestimate of flow depth of 4-6%;
- A 10% underestimate of n results in an overestimate of velocity of 6-7.5%.
- A 10% underestimate of Q results in an underestimate of flow depth of 4-6%;
- A 10% underestimate of Q results in an underestimate of velocity of 2.5-4%.

7. CONCLUSIONS

This study has found a considerable variety of techniques available and used in New Zealand for determining flood flow rates and channel resistance. Present New Zealand practice lags behind current research knowledge, but most of the techniques used nevertheless have some value. A notable exception is the TM61 method of estimating flood flow, which appears quite unreliable.

Both flood flows and channel resistance are subject to some uncertainty, even when the "best" methods are employed by experienced practitioners. The corresponding uncertainties in water level and velocity should be allowed for by designers of bridges and culverts.

7.1 Flood Frequency Analysis

There is a strong body of research on the analysis of "at-site" flood records. There has also been widespread research on techniques for estimating flood frequency at ungauged sites, including regionalisation techniques, and rainfall/runoff models ranging from the unit hydrograph to complex computer modelling. Students in engineering and geography degree courses are introduced to these and to older empirical procedures such as the Rational Method. If recent research effort were the only indicator, it could be concluded that the Rational Method, TM61 and other empirical methods had been superseded. However, the two surveys of engineering design practice told a quite different story: a clear lag in the adoption of new procedures, with local authority engineers in particular preferring the older methods.

Of the methods available for design flood estimation at sites without flood records, the Rational Method (even using only crude estimates of its runoff coefficient) and the RFE method performed significantly better in our trials than TM61. The TM61 method is clearly inferior, and its use should cease unless it can be calibrated with NZ data.

Our sample of seven records suggests that 1/100 AEP design floods can be estimated with the RFE method with root mean square error of about 50% and a bias of about 15%. However these figures are unduly inflated by a particularly poor result for one basin. The RFE results are slightly closer to the "at-site" results than the Rational Method, and seems to be the best available method for design flood estimates where no "at-site" data are available.

The particularly poor result for the Kai Iwi basin may be attributable to a localised rain shadow effect although such a rain shadow has not yet been sought or identified in rainfall data. This underscores the importance of local knowledge and professional judgement in estimating flood flow. Estimates will be best when undertaken by a practitioner with hydrological expertise and experience.

Previous research has shown that where a limited series of at-site data are available, results from the RFE method are further improved by being able to estimate the mean annual flood directly. The practical implication of this is that it may be worth establishing a flow measurement site once a potential crossing site has been identified.

The single example of use of a rainfall/runoff model gave a reasonable estimate of the 1/100 AEP flood, but this estimate was no better than those obtained using other methods. It appears therefore that there will be some uncertainty in any estimate other than one obtained "at-site" from a flow record of many years' duration.

7.2 Determination of Channel Resistance (i.e. Manning's Roughness Coefficient n)

There is only a limited amount of useful published information on channel resistance, Section 3.3.2.1 of this report identifies several sources of information on the flow resistance of the "standard" river cross-section. The approach of the universities and the engineering profession has been a pragmatic one, relying on most of these sources of information. Site measurements, photographs of measured sites and tables of recommended roughness values are favoured over formulas. The literature search found only two attempts (Bray 1979, Bray and Davar 1987) at comparing available methods. Bray's (1979) results indicated that the "theoretical" formulas (Keulegan's and Strickler's) based on 2-dimensional idealisation were not as successful as empirical formulas.

The measured sites presented by Hicks & Mason (1991) displayed a wide range of values of Manning's n which were not always readily explained by either visual differences in the channel form or by differences in bed material. The present study adds weight to that impression, and one conclusion is that estimates of channel resistance will always be subject to some uncertainty.

Use of the Hicks & Mason (1991) handbook allows only moderately consistent and accurate estimates of Manning's n, but is probably an improvement on the tables of values provided by Henderson (1966) and AUSTROADS (1994). These are difficult to use, primarily because they generally quote a quite wide range of values for a particular channel description. The formula proposed by Henderson for straight gravel rivers appears to underestimate n in New Zealand gravel-bed rivers by an average of about 20%.

The channel resistance of gravel-bed rivers is moderately well described by the four formulas tested. The two equations of Bray & Davar (1987) gave results for the 5 sites that were comparable with those obtained using the Hicks and Mason (1991) handbook. On the limited evidence of this study, the formulas proposed by Griffiths (1981) were slightly more successful. This is in spite of Griffiths' formulas being applicable to rigid beds, and indicates that bedforms which form when the bed is mobilised might not contribute much extra resistance. One way of improving our ability to predict channel roughness in gravel-bed rivers would be to use the Hicks and Mason (1991) data to further calibrate or modify one of the formulas. The present situation with gravel-bed rivers is that the formulas are objective and easy to use, whereas the Hicks and Mason handbook greatly assists in illustrating the variations from "typical" channel resistance which occur in different types of reach. The best approach at present may be to use both approaches in tandem.

All these conclusions are based on quite limited data - only 7 sites - and the measured values of Manning's n are of necessity approximate. Evidence from further sites would be extremely useful, particularly from floods exceeding the mean annual flood.

The results of this study indicate that accurately estimating channel resistance is difficult. It follows that the engineer making the final choice of a resistance value should ideally have suitable experience and an understanding of the underlying causes of channel resistance. It appears that estimates of Manning's n can often be in error by up to 40%, and estimates of water depth and velocity can be expected as a consequence to be in error by up to 30%. This should be considered by designers of bridges and other engineering works.

8. RECOMMENDATIONS

There are three types of recommendation which emerge from this study. Section 8.1 sets out recommended design practice based on the results of this study. These recommendations are intended to be the best practice given present knowledge, and it is hoped and expected that they will change as further research results become available. Section 8.2 identifies topics where research has been done but design engineers are unfamiliar with the results, and Section 8.3 lists gaps in our knowledge where further research might benefit roading design.

8.1 Recommended Design Practice

8.1.1 Flood Flow Rates

- When the capital cost of a project is high and the lead time for design is more than, say, two years, a flow recording site should be established as soon as possible, to provide data to augment the "no-data" methods of estimating flood flows.

- The engineer making the final choice of a design flood flow should have suitable experience and an understanding of the assumptions and limitations present in the various methods.

- Design flood flows should be obtained from at least two methods chosen from the Rational Method, the RFE method (which in New Zealand should replace the methods outlined in Section 3.6 of AUSTROADS (1994)) and catchment models. The TM61 method gives erratic results and should not be used.

- The design of important structures should include considering the possible effects of climate change, and make use of up-to-date research on this topic.

8.1.2 Channel Flow Resistance (Manning's n)

- Any opportunity should be taken to directly measure water levels and channel resistance directly during flows close to the design value.

- The engineer making the final choice of a resistance value should have suitable experience and an understanding of the underlying causes of channel resistance.

- Rather than using Table 4.1 in AUSTROADS (1994) estimates of Manning's n for gravel-bed rivers should be made using at least one formula, e.g. one of the Griffiths "rigid bed" formulas as well as using Hicks & Mason (1991). If Henderson's (1966) formula is used, a factor of 1.2 should be applied to the calculated values of Manning's n.

- In low-gradient silt- and sand-bed rivers, determinations of Manning's n from sets of photographs (e.g. Hicks & Mason (1991) or from tables of values such as Table 4.1 of AUSTROADS (1994) should be taken as approximate only. In addition, any possible backwater effect from downstream features should be investigated. In these rivers direct measurements should be obtained whenever possible.

8.1.3 Final Choice of Design Parameters

- When structural integrity depends on the water velocity or depth used for design, values of flow rate and Manning's n should both be increased by about 30% to allow for errors in their estimation. These increases are not necessary if water depths exceeding the design value will cause inconvenience but not damage.

8.2 Suggested Topics Requiring Technology Transfer

- Use of the Regional Flood Estimation method of McKerchar & Pearson (1989).

The questionnaire showed that many engineers endorse RFE methods, but have continued to use the now superseded Beable & McKerchar (1982) method rather than its replacement, McKerchar & Pearson (1989).

- Head loss at culverts.

Questionnaire respondents indicated that this was a problem, yet there has been considerable research on the subject, and AUSTROADS (1994) provides good coverage.

- The analysis of flow records at a site to estimate extreme flood flows.

The statistical theory behind extrapolating a flow record is fairly well established (notwithstanding the research suggested in 8.3 below on special cases). However, some engineers have noted this topic as a concern.

8.3 Further Research

8.3.1 Suggested Research Objectives for Transit New Zealand

Some topics which may be of specific interest to Transit New Zealand and worthy of further research are itemised below, with explanatory notes.

- The effect of global warming and other trends in New Zealand climate on the frequency of extreme flow events.

Considerable research is already being carried out on the likely effects of global warming, including the possible effects on rainfall patterns. However, research specific to extreme rainfall events may need to be funded by Transit New Zealand, although the Electricity Corporation, for example, is another potential users of the results.

- The implications (economic and social) of choosing to design for a flood flow of a particular AEP.

The flood flow of 1% AEP has often been chosen for bridge design, although the flow of 10% AEP might be used for culverts and other structures where the consequences of more extreme flows are not severe. These choices have usually been made subjectively, and a more rigorous approach to risk requires research into the economic and social costs of a particular choice of AEP. This would include investigating (i) the expected damage and modes of failure due to a flood flow exceeding the design value; (ii) the costs of reconstruction and repair; and (iii) the costs of disruption to the roading network.

- Identifying situations where the frequency distribution of extreme flood flows may be modified by storage effects.

There is a large body of research into establishing the type of flood frequency distribution for individual sites and for hydrologically homogeneous regions. It is not recommended that Transit New Zealand fund further research on the general subject, but it is important to be able to identify the particular situations in which the distribution of extreme floods is modified by channel storage effects (see the final paragraph of 3.3.1.2 above). For instance, the example described by Pearson & Macky (1992) would have led to overdesign if it had not been identified. In other situations it is possible that channel storage effects would result in underdesign.

- Testing the accuracy of estimates of Manning's n using the photographs presented by Chow(1959) and Barnes (1967).

Given that the use of Hicks & Mason (1991) was only moderately successful, it would be worthwhile to extend the testing described in this report to other sets of photographs.

- Measuring the flow resistance of types of riverside and floodplain vegetation found in New Zealand.

The guide by Arcement & Schneider (1989) has no New Zealand equivalent, and the data presented by Hicks & Mason (1991) and the present study, indicate that at some sites vegetation causes high flow resistance. Field work augmented by numerical modelling could result in a useful guide for unmeasured sites, especially those where there is significant overbank flow.

- Determination of the coefficients C for the Rational Method of flood flow estimation, for classes of typical New Zealand landforms and vegetation.

The Rational Method performed rather better than expected in our trials, but there was a difficulty in deciding which value C to choose from a table of suggested values whose descriptors bore little resemblance to rural New Zealand. Such a study might also determine whether C varies with catchment size, and the limiting size for which the methods can be applied. The output might be an empirical method like that produced by Beca Carter Hollings & Ferner (1992) for Auckland, or along the lines suggested in Section 3.5 of AUSTROADS (1994).

8.3.2 Research Topics of General Interest

As well as the research objectives identified in 8.3.1 above, it is possible from this study to identify research topics whose outputs would have several applications besides roading design. It is possible that these topics are too general for Transfund New Zealand to fund the research on its own, but it is recommended that Transfund retain an interest in developments in these topics. Some examples are:

- Refinement of the RFE method of McKerchar & Pearson (1989) in regions of New Zealand where further data has become available.
- General research into aggradation and degradation of river beds.
- Global warming and other long-term climatic changes.
- The magnitude of very extreme floods and the Probable Maximum Flood.

9. REFERENCES

Aguirre-Pe, J., Fuentes, R. 1990. Resistance to flow in steep rough streams. *J. Hydraulic Eng.*, 116(11): 1374-1387.

Arcement, G.J., Schneider, V.R. 1989. Guide for selecting Manning's roughness coefficients for natural channels and flood plains. *Water Supply Paper 2339*, U.S. Geological Survey, Washington, D.C.

Auckland Regional Water Board. 1989. Application of the RORB catchment model to the Kaipara River catchment. Tech. Publ. No. 55.

AUSTROADS. 1994. *Waterway Design - A guide to the hydraulic design of bridges, culverts and floodways*. Sydney. 137 p.

Bardsley, W.E. 1994. Against objective statistical analysis of hydrological extremes. *J. Hydrology*, 162: 429-431.

Barnes, H.H. 1967. Roughness characteristics of natural channels. *Water Supply Paper 1849*, U.S. Geological Survey, Washington, D.C.

Bathurst, J.C. 1982. Flow resistance in boulder-bed streams. In: *Gravel-bed rivers*, Hey, R.D., Bathurst, J.C., Thorne, C.R. (Eds), John Wiley and Sons.

Beable, M.E., McKerchar, A.I. 1982. Regional flood estimation in New Zealand. *Water & Soil Tech. Pub. No. 20*, Ministry of Works & Development.

Beca, Carter, Hollings & Ferner 1992. Guidelines for the estimation of flood flows in the Auckland region. Prepared for Auckland Regional Council.

Beca, Carter, Hollings and Ferner, 1992: "Guidelines for the estimation of flood flows in the Auckland region" (Prepared for the Auckland Regional Council) Environmental and Planning Division Pub. 19.

Bray, D.I. 1979. Estimating average velocity in gravel-bed rivers. *Proc. ASCE* 105(HY9): 1103-1123.

Bray, D.I., Davar, K.S. 1987. Resistance to flow in gravel-bed rivers, *Canadian J. Civil Eng.* 14: 77-86.

Campbell AP, 1959: "Revision of Technical Memorandum No. 61: provisional standard for determination of peak flows in small catchment areas" from page 2-1 to 2-24 in "Hydrology", Proceedings of a meeting of design engineers employed on hydrological works, Soil Conservation and Rivers Control Council,

Charlton, F.G., Brown, P.M., Benson, R.W. 1978. The hydraulic geometry of some gravel rivers in Britain, *Report No. IT 180*, Hydraulics Research Station, Wallingford, Oxon, United Kingdom.

Chow, V.T. 1959. *Open channel hydraulics*. McGraw-Hill, New York.

Chowdhury, J.U., Stedinger, J.R., Lu, L.H. 1991. Goodness-of-fit tests for regional generalised extreme value flood distributions. *Water Resources Research* 27(7): 1765-1776.

Cordery, I. 1987. The unit hydrograph method of flood estimation in *Australian rainfall and runoff. A guide to flood estimation.* Pilgrim, D. (Ed.) Revised edition. Institute of Engineers, Australia. 374 p.

Dalin, J.J. 1986. Statistical analysis of mixed populated hydrological data (S.A.M.P.), paper presented at the *International Symposium on Flood Frequency and Risk Analysis*, Baton Rouge, Louisiana, U.S.A.

Davies, T.R.H. 1993. Fallacies in flood hydrologic design. Guest editorial, *J. Hydrology (N.Z.)* 31(2): 73-77.

Dooge, J.C.I. 1973. Linear theory of hydrologic systems, *Technical Bulletin 1468*, United States Department of Agriculture, 327 p.

Einstein, H.A. 1950. The bed-load function for sediment transportation in open channel flows, *Technical Bulletin No. 1026*, U.S. Dept. of Agr., Washington, D.C.

Engelund, F., Hansen, E. 1967. *A Monograph on sediment transport*, Technisk Forlag, Copenhagen, Denmark.

French, R.H. 1985. *Open-channel hydraulics.* McGraw-Hill.

Griffiths, G.A. 1981. Flow resistance in coarse gravel bed rivers. *Proc. ASCE* 107(HY7): 899-918.

Griffiths, G.A., Pearson, C.P., Horrell, G.A., 1989. Rainfall-runoff routing in the Waimakariri Basin, New Zealand. *J. Hydrology (N.Z.)* 28(2): 111-129.

Henderson, F.M. 1966. *Open channel flow.* MacMillan, New York.

Hicks, D.M., Mason, P.D. 1991. *Roughness characteristics of New Zealand rivers.* DSIR Marine & Freshwater, Wellington.

Hirschboeck, K.K. 1986. Hydroclimatically defined mixed distributions in partial duration flood series, paper presented at *the International Symposium on Flood Frequency and Risk Analysis*, Baton Rouge, Louisiana, U.S.A.

Horrell, G.A. 1992. Ashley River flood estimation. *Report R92(44)*, Canterbury Regional Council.

Hosking, J.R.M., Wallis, J.R, Wood, E.F. 1985. An appraisal of the regional flood frequency procedure in the UK Flood Studies Report. *J. Hydrological Sciences* 30(1): 85-109.

Ibbitt, R.P., McKerchar, A.I. 1992. Streamflow and the use of models. In: *Waters of New Zealand*, M.P. Mosley (Ed.), New Zealand Hydrological Society, Ch. 7.

Jarrett, R.D. 1984. Hydraulics of high-gradient streams. *J. Hydraulic Engineering,* 110(11): 1519-1539.

Jarrett, R.D. 1990. Hydrologic and hydraulic research in mountain rivers. *Water Resources Bulletin,* 26(3): 419-429.

Kawahara, Y., Tamai, N. 1989. Mechanism of lateral momentum transfer in compound channel flow. *International Association for Hydraulic Research, 23rd Congress,* Ottawa: B-463-470.

Knight, D.W., Shiono, K. 1990. Turbulence measurements in a shear layer region of a compound channel. *J. Hydraulic Research.* 28(2): 175-196.

Kouwen, N. 1992. Modern approach to design of grassed channels. *J. Irrigation & Drainage Engineering,* 118(5): 733-743.

Laurenson, E.M., Mein, R.G., 1985. RORB - Version 3 Runoff Routing Program - User Manual. Dept. of Civil Engineering, Monash University.

Lawgun, N., Toong, A.T. 1985. Regional flood frequency analysis of a small catchment in Northland, Auckland and Coromandel. *J. Hydrology (N.Z.)* 24(2): 64-76.

Limerinos, J.T. 1970. Determination of the Manning coefficient from measured bed roughness in natural channels. *Water Supply Paper 1989-B,* U.S. Geological Survey, Washington, D.C.

McKerchar, A.I. 1991. Regional Flood Frequency Analysis for Small New Zealand Basins. 1. Mean annual flood estimation. *J. Hydrology (N.Z.)* 30(2): 65-76

McKerchar, A.I., Pearson, C.P. 1989. Flood frequency in New Zealand. *Hydrology Centre Publication 20.* Hydrology Centre, DSIR Division of Water Sciences, Christchurch.

Ministry of Works and Development (1975). A method for estimating design discharge. *Metric version of Technical Memorandum No.61.* Planning and Technical Services Group, Water and Soil Division.

Ministry of Works and Development (1964). Provisional standard for empirical estimation of flood discharges. *Technical Memorandum No.61, 3rd Revision.* Water and Soil Division.

NWASCO, 1980: "Metric version of Technical Memorandum No. 61" Water and Soil Division, Ministry of Works and Development, Wellington.

Painter, D.J., Larsen, S.H. 1995. Effects of season and weather patterns on flood frequency: Hakataramea River, New Zealand. *J. Hydrology (N.Z.)* 34(2): 111-128.

Pearson, C.P. 1991. New Zealand Regional Flood Frequency Analysis using L-moments. *J. Hydrology (N.Z.)* 30(2): 53-64.

Pearson, C.P. 1991. Regional Flood Frequency Analysis for Small New Zealand Basins. 2. Flood frequency groups. *J. Hydrology (N.Z.)* 30(2): 77-92

Pearson C.P., McKerchar AI, 1989. "Flood estimation - a revised design procedure" *Transactions*, IPENZ, V16(2/CE), 59-65.

Pearson, C.P., Macky, G.H. 1992. Manawatu River flood estimation. Hydrology Centre Report CR92.12, DSIR Hydrology Centre, Christchurch.

Pilgrim, D. (Ed.) 1987. *Australian rainfall and runoff. A guide to flood estimation.* Revised edition. Institute of Engineers, Australia. 374 p.

Pilgrim D.H., Cordery I., 1993: "Flood runoff", Ch. 9 in Maidment DR (Ed) "Handbook of hydrology" McGraw-Hill.

Potter, K.W. 1987. Research of Flood Frequency Analysis: 1983-1986. *Reviews of geophysics*, 25(2): 113-118.

Rajaratnam, N., Ahmadi, R.M., 1981 Hydraulics of channels with flood plains. *J. Hydraulic Research,* 19(1): 43-60.

Riddell Associates, 1994. FLOEST evaluation and test. NIWA Christchurch Misc. Report 205.

Riddell D.C., 1980: "Flood hydrology of the Motu River". *Journal of Hydrology (NZ)* 19(1), 35-48.

Rodgers M.W., Thompson S.M., 1992: "Tideda reference manual" Publication No. 24, 2nd ed. Hydrology Centre, NIWA, Christchurch, 222p.

Rossi, F., Fiorentino, M., Versace, P. 1984. Two-component extreme value distribution for flood frequency analysis. *Water Resources Research* 20(7): 847-856.

Salinger, M.J., Mullan, A.B., Porteous, A.S., Reid, S.J., Thompson, C.S., Withers, C.S., Coutts, L.A., Fouhy, E. 1990. *New Zealand climate extremes: scenarios for 2050 AD.* Report prepared by NZ Meteorological Service and DSIR Applied Mathematics Group for the Ministry for the Environment.

Shen, H.W., Fehlman, H.E; Mendoza, C. 1990. Bed Form Resistances in Open Channel Flows, *J. Hydraulic Engineering* 116(6) 799-815.

Smart, G.M. 1992. Stage-discharge discontinuity in composite flood channels. *J. Hydraulic Research.* 30(6): 817-833.

Stedinger, J.R., Vogel, R.M., Foufoula-Georgiou, E. 1993 Frequency analysis of extreme events, in *Handbook of Applied Hydrology*, D.R. Maidment (Ed.), McGraw-Hill, Ch. 18.

Sutcliffe, J.V. 1978. Methods of flood estimation: a guide to the flood studies report. *Report No. 49*, Institute of Hydrology, Wallingford, Oxon, England.

Tomlinson A.I., 1978. "Storm rainfalls" *Journal of Hydrology (NZ)* 17(2): 57-77.

Tomlinson A.I., 1980. "The frequency of high intensity rainfalls in New Zealand Part 1", *Water & Soil Technical Pub. No. 19*. Water & Soil Division, MWD, Wellington. 36p & 4 maps.

van-Rijn, L.C. 1984. Sediment Transport, Part III: Bed Forms and Alluvial Roughness, *J. Hydraulic Engineering*, 110(12), 1733-1754.

Wallis, J.R., 1980. Risk and uncertainities in the evaluation of flood events for the design of hydraulic structures, in: *Piene e Siccita*, E. Guggino, G. Rossi and E. Todini, (Eds), Fondazione Politecnica del Mediterraneo, Catania, Italy.

Waugh, J.R. (1973). Report on the use of Technical Memorandum No. 61 (3rd Rev.) for estimating flood peak discharge for small rural catchments in the Northland-Auckland area. *Report WS146*, Ministry of Works and Development, Christchurch.

Waugh, J.R. (1978). Magnitude and frequency of floods in the Northland-Auckland region and their application to urban flood design. *Water & Soil Tech. Publ. No. 8*, Ministry of Works and Development, Christchurch.

Whetton, P., Mullan, A.B., Pittock, B. 1996. Climate change scenarios for Australia and New Zealand, in: *Greenhouse:Coping with climate change*, Bouma, W. J., Pearman, G. I., Manning, M.R. (Eds) CSIRO Press. 145 - 168.

Withers, C.S. 1992. An examination of the extremes of selected NZ rainfall and runoff records for evidence of trend. *Report No. 17*, Industrial Research Ltd, Lower Hutt.

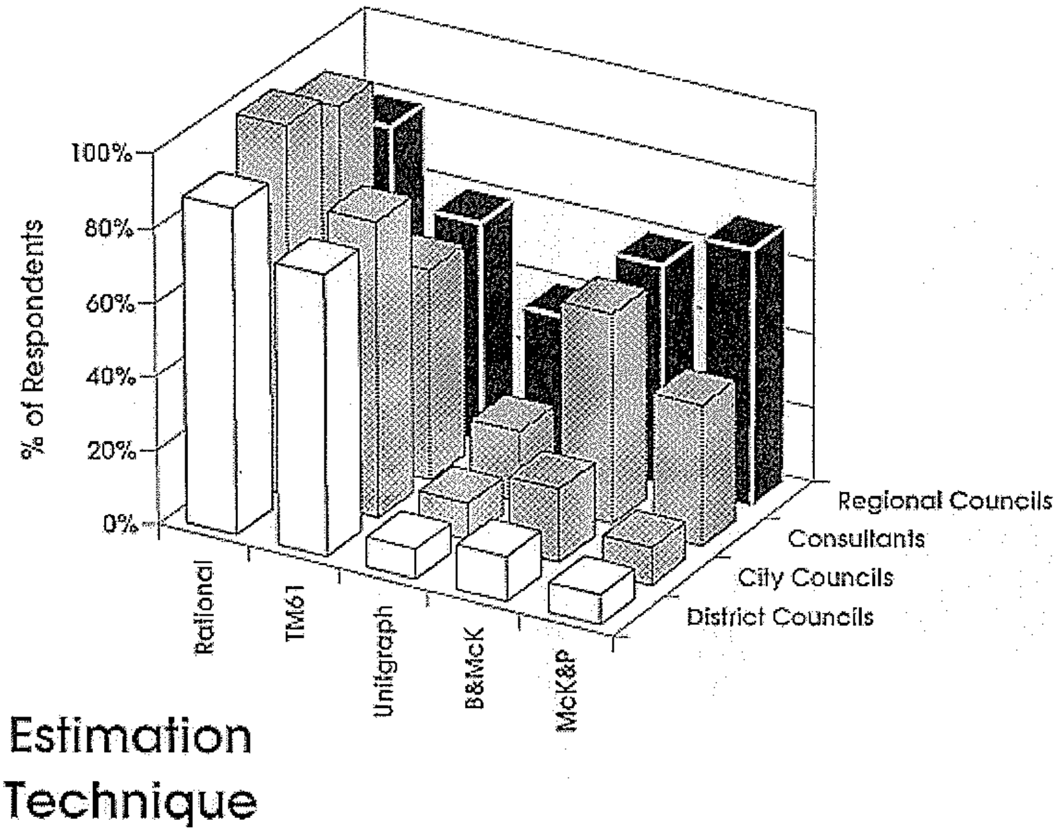
Wolff, C.G., Burges , S.J. 1994. An analysis of the influence of river channel properties on flood frequency. *J. Hydrology*, 153(1-4): 317-337.

10. GLOSSARY

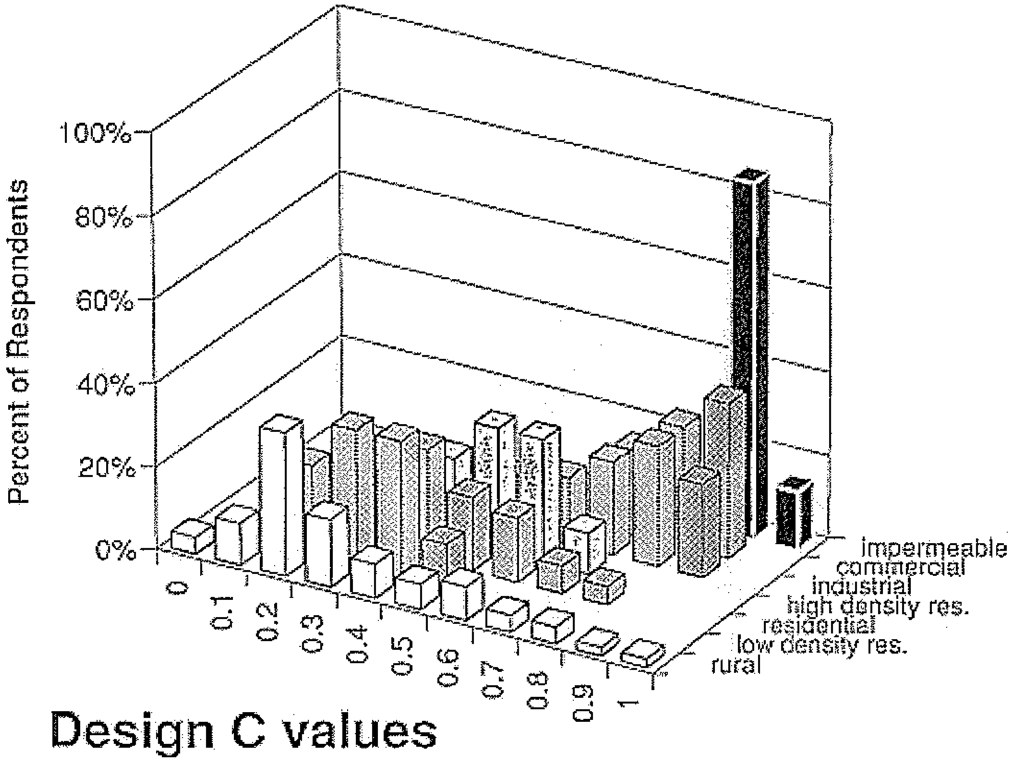
T_c	The time of concentration, i.e. the time interval between the start of rainfall and peak outflow
C	A runoff coefficient used in the Rational method
RMS	Root-mean-square
EV1 Distribution	A distribution for extreme values, also known as the Gumbel distribution. It is a particular case of the generalised extreme value distribution (GEV) and is often considered to describe flood flow maxima.
Annual Exceedance Probability (AEP)	For a given flow rate, the probability that it will be exceeded at least once in a year
Mean Annual Flood, Q_b	The mean of the annual flow rate maxima, presumed to have an AEP of 1 in 2.33
Q_{100}	The flow rate with an AEP of 1 in 100 (colloquially referred to as the 100-year flood)
Energy slope	For flow in a channel, the gradient (with distance downstream) of the total energy line given by $y + V^2/2g$, where y is the water surface elevation, V is the mean velocity and g is acceleration due to gravity
Channel slope	A representative gradient of the bed of the channel (see text for details of how this was determined).

APPENDIX 1. GRAPHS SHOWING FINDINGS FROM 1991 QUESTIONNAIRE

A1.1 Flood estimation methods used by design engineers



A1.2. Values of C used in the Rational Method



APPENDIX 2. 1996 QUESTIONNAIRE

A2.1 The Questionnaire

QUESTIONNAIRE for Transit NZ: Waterway Design Parameters

Part A: Respondent Details

Your name: ______________________________

Your position and responsibilities within your organisation ______________________________

____

The name of your organisation ______________________________

____

Part B: Flood Frequency

1 Which techniques do you use to estimate design floods in rural basins (and in rural parts of basins)? *(Please tick all those used.)*

- McKerchar & Pearson (1989)
- Beable & McKerchar (1982)
- Unit Hydrograph *(please give reference if known)*

  ____
- Rational Method
- TM61 (MWD 1975)
- Other (*please give references*)

  ____

  ____

  ____

....Questions 2-5 overleaf

2 Which other aspects **(if any)** of flood frequency analysis have given you concern. *(Please tick as appropriate*

- Analysis of discharge records to quantify extreme events

- Choice of a design extreme event (i.e. choice of return period

- other *(please specify)* ______________________________

Part C: Channel Roughness

3 What equation do you use to calculate the frictional head loss? *(Please tick)*

- Manning equation
- Other (e.g. Chezy equation) *(please specify)*

____

....Questions 4-5 overleaf

4 How do you estimate channel roughness (i.e. Manning n or equivalent)? *(Please tick all those used)*

- "Hand-calculation" from on-site measurements of cross-section, discharge and slope
- Calibration of on-site measurements of cross-section, discharge and slope against a numerical model
  *(please specify the model if known)*________________________

Comparison with tables of typical values in:

- Henderson (1966)
- AUSTROADS (1994) "Waterway Design"
- other *(please specify)*______________________________

Comparison with photographs of measured sites in:

- Chow (1959)
- Hicks & Mason (1991)
- other *(please specify)* ____________________________

Applying an equation:

- Strickler ($n=.041\ d^{1/6}$)
- Griffiths (1981)
- other *(please specify)* ____________________________

5 What other aspects **(if any)** of waterway design have given you concern? *(Please tick as appropriate)*

- Bed level changes during extreme events
- Unsteady flow in tidal or lowland rivers
- Head losses at culverts
- Pier scour
- Abutment scour
- other *(please specify)* ________________________________

A2.2 Summary statistics of returns

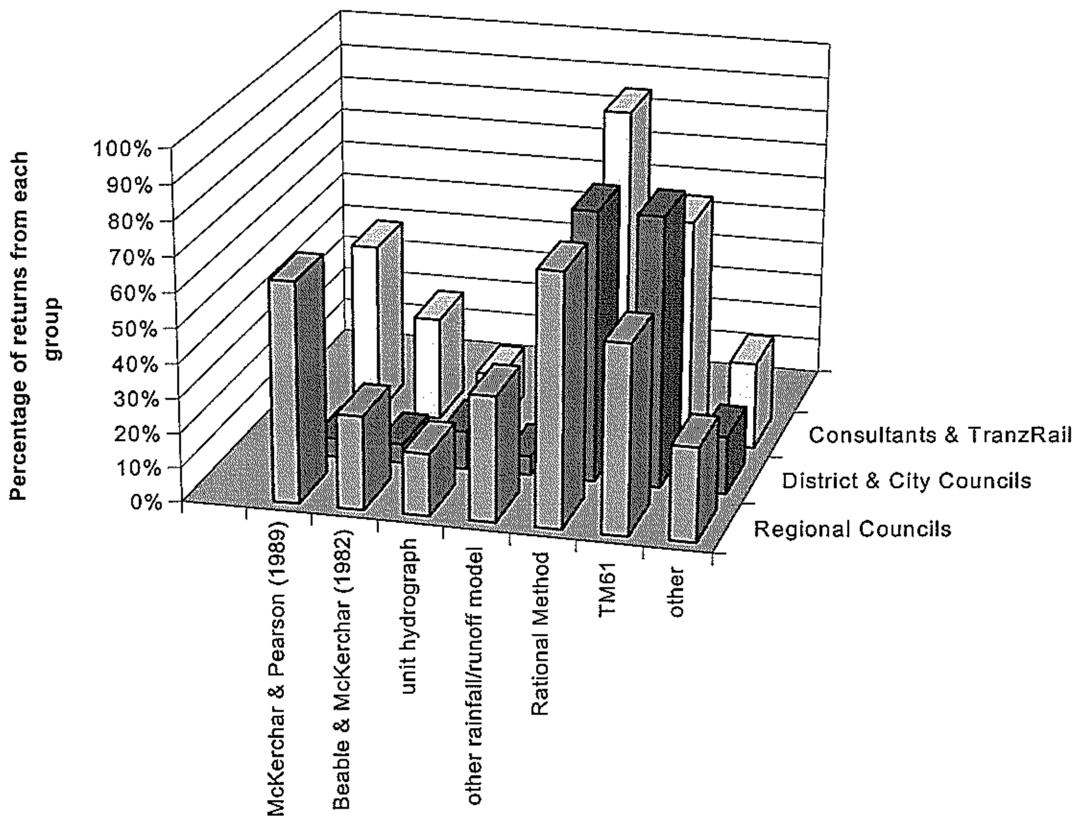
	Question No.	Response	Regional Councils		District & City Councils		Consultants		TranzRail	Total
		NUMBER OF RETURNS =>	**11**		**18**		**19**		**1**	**49**
Flood Frequency	1	McKerchar & Pearson (1989)	7	64%	1	6%	9	47%	1	**18**
		Beable & McKerchar (1982)	3	27%	1	6%	5	26%	1	**10**
		unit hydrograph	2	18%	2	11%	3	16%	0	**7**
		other rainfall/runoff model	4	36%	1	6%	4	21%	0	**9**
		Rational Method	8	73%	14	78%	18	95%	1	**41**
		TM61	6	55%	14	78%	12	63%	1	**33**
		other	3	27%	3	17%	4	21%	1	**11**
	2	analysis of discharge records	6	55%	4	22%	2	11%	1	**13**
		choice of design event	5	45%	10	56%	6	32%	1	**22**
		other	5	45%	2	11%	7	37%	0	**14**
Channel Roughness	3	Manning	11	100%	16	89%	19	100%	1	**47**
		other	2	18%	2	11%	3	16%	0	**7**
	4	hand-calculation from site measurements	7	64%	7	39%	12	63%	0	**26**
		model calibration from site measurements	9	82%	2	11%	4	21%	0	**15**
		tables: Henderson	6	55%	5	28%	13	68%	1	**25**
		tables: AUSTROADS	1	9%	2	11%	3	16%	0	**6**
		tables: other	2	18%	3	17%	5	26%	0	**10**
		photographs: Chow	6	55%	4	22%	6	32%	1	**17**
		photographs: Hicks & Mason	8	73%	2	11%	5	26%	0	**15**
		photographs: other	2	18%	0	0%	1	5%	0	**3**
		equation: Strickler	3	27%	2	11%	3	16%	0	**8**
		equation: Griffiths	1	9%	0	0%	1	5%	0	**2**
		equation: other	1	9%	0	0%	1	5%	0	**2**
	5	bed level changes	10	91%	6	33%	9	47%	1	**26**
		unsteady flow	3	27%	1	6%	5	26%	1	**10**
		head loss at culverts	7	64%	10	56%	7	37%	1	**25**
		pier scour	4	36%	5	28%	9	47%	1	**19**
		abutment scour	5	45%	6	33%	9	47%	1	**21**
		OTHER	4	36%	3	17%	6	32%	1	**14**

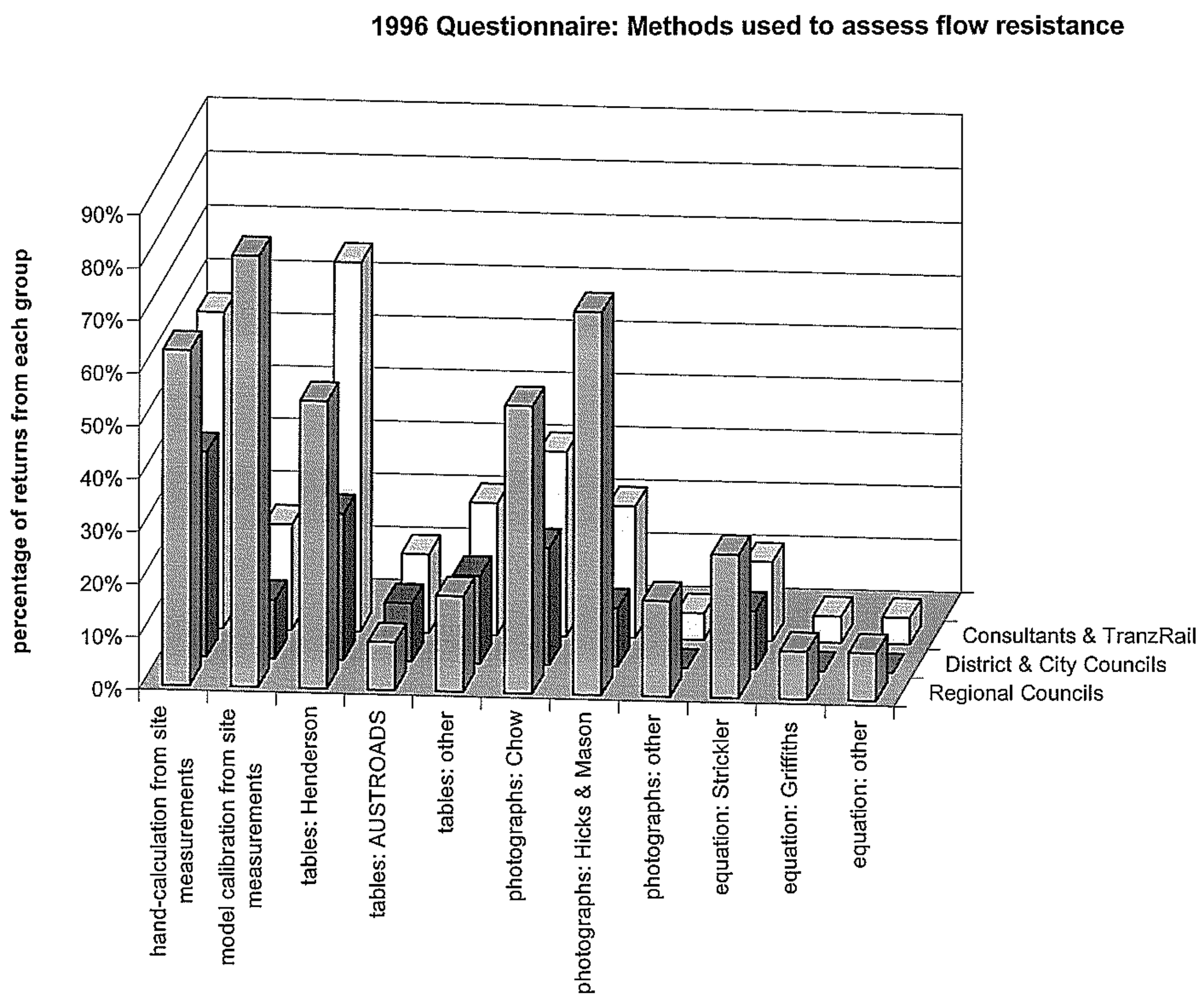
SUMMARY STATISTICS FROM 1996 QUESTIONNAIRE

A.23 Information graphed from summary statistics:

- **Flood Estimation methods used by design engineers (cf. Appendix 2)**
- **Methods used to assess flow resistance**

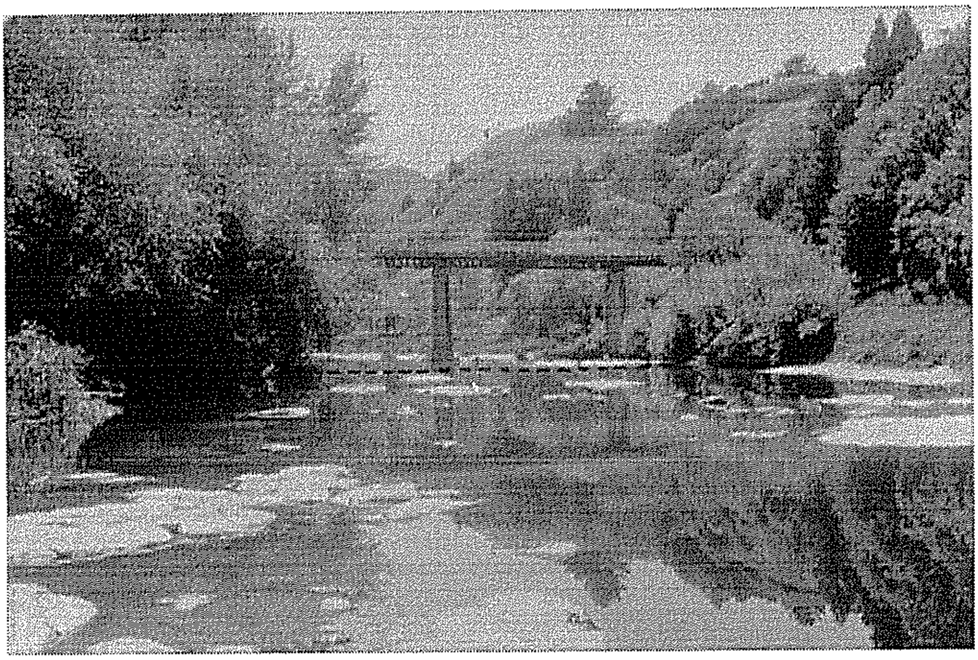
1996 Questionnaire: Flood frequency methods used by design engineers

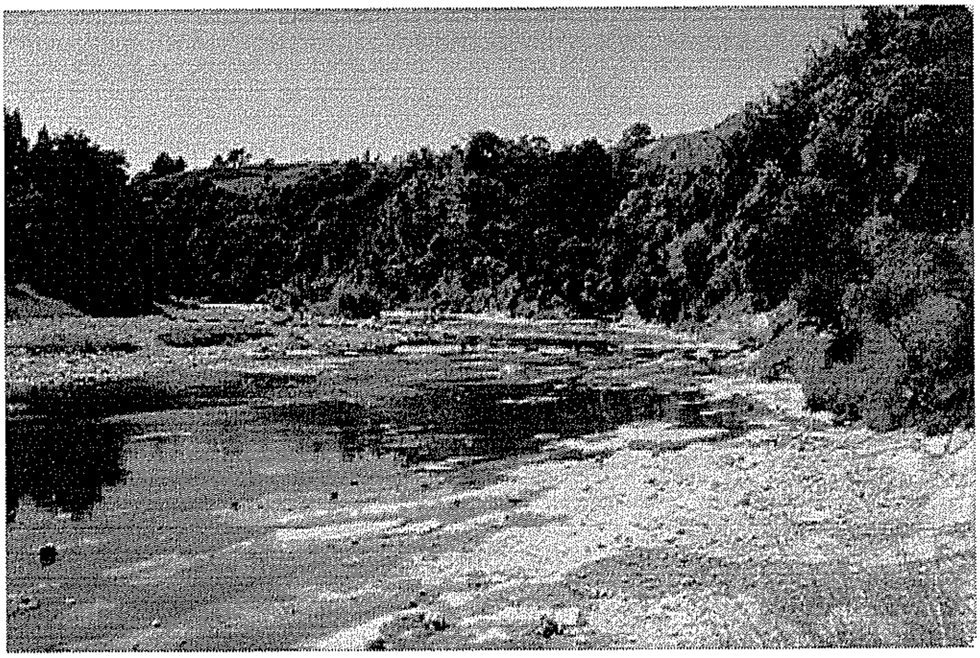



1996 Questionnaire: Methods used to assess flow resistance
percentage of returns from each group
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
hand-calculation from site measurements
model calibration from site measurements
tables: Henderson
tables: AUSTROADS
tables: other
photographs: Chow
photographs: Hicks & Mason
photographs: other
equation: Strickler
equation: Griffiths
equation: other
Consultants & TranzRail
District & City Councils
Regional Councils

APPENDIX 3. OBSERVERS' NOTE AND SITE PHOTOGRAPHS FOR CHANNEL ROUGHNESS SITES

A3.1 Hangaroa @ Doneraille Park





Hangaroa @ Doneraille Park **Site 21437**
Observer #1

Bed mostly rock; banks vegetated in parts strongly curved reach.
Compare with:

1043419	Pokaiwhenau	- bedrock and some cobbles - grass & blackberry on gentle banks n = .026 for higher flows
43435	Waipapa	- more heavily vegetated - straight reach n = .028 for higher flows
40708	Mokau	- bedrock and some silt, sand and gravel - dense willows on banks - gently curved reach n = .062 for higher flows
74315	Taieri	- bedrock and boulders - only a few willows, mostly tussock - straight reach n = .027
45311	Kaipara	- clay bed, grass and weed banks - slightly curved reach - n ~ .05 for high flows

The Mokau appears somewhat anomolous. However, the others have less flow resistance than the Hangaroa, particularly because they are nearly straight reaches.

Choose n = .04, but this is little more than a guess.

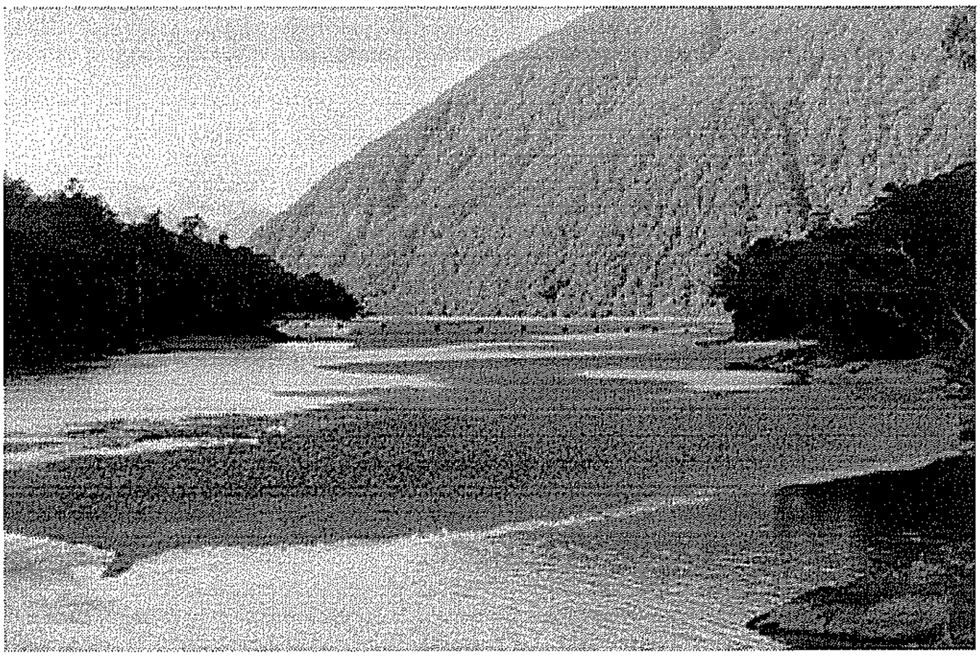
Hangaroa @ Doneraille Park **Site 21437**
Observer #2

Channel all bedrock; d_{84} and d_{50} problematic

Hicks and Mason (1991) p86 Taieri channel here is bedrock and some boulders and photos are somewhat similar, although Hangaroa flow is less.

Recommend n = 0.030

A3.2 Haast at Roaring Billy



Haast at Roaring Billy **Site No 86802**
Observer #1

Large wide gravel-bed river; measured reach is a relatively narrow gorge with single-thread channel and "alternate" bars.

d_{50} = 22 mm; d_{84} = 109mm; d_{90} = 135 mm

Heavy bush on true right bank; Bush and scrub on true left bank

Similar rivers:

Buller @ Woolfs 93208	- straight reach - Bed is coarser (d_{50} = 56 mm; d_{84} = 182 mm) - n = .04, n = .03 for high flows
Rangitikei @ Mangaweka 32702	- Bed material similar - Papa banks, partly vegetated - strongly curved reach n = .041 for high flows
Clarence @ Jollies 62105	- straight reach, grassed banks - coarser bed material - n = .027 for high flows
Waioehu @ Gorge	- similar bed material - smaller river, bends u/s and d/s of reach - bushed banks n = .035
Pomahaka @ Burkes Ford	- coarser bed; one bank grassed, one willowed - straight reach - n = .029
Grey @ Dobson	- finer sediment (d_{50} = 33 mm; d_{84} = 67 mm) - slightly curved reach; vegetated banks - n = .026

Wanganui @ Paetawa	- finer bed (d_{50} = 25 mm but d_{50} = 56 mm)
	- one bank grassed, one willowed
	- n = .033

Waioehu and Pomahaka appear the closest

Choose n = .032	- mean of these two
	- rough mean of all the above

<u>Haast R. at Roaring Billy</u> **<u>Site No 86802</u>**

Observer #2

Sediment

Sand - gravel

d_{84} = 109 mm

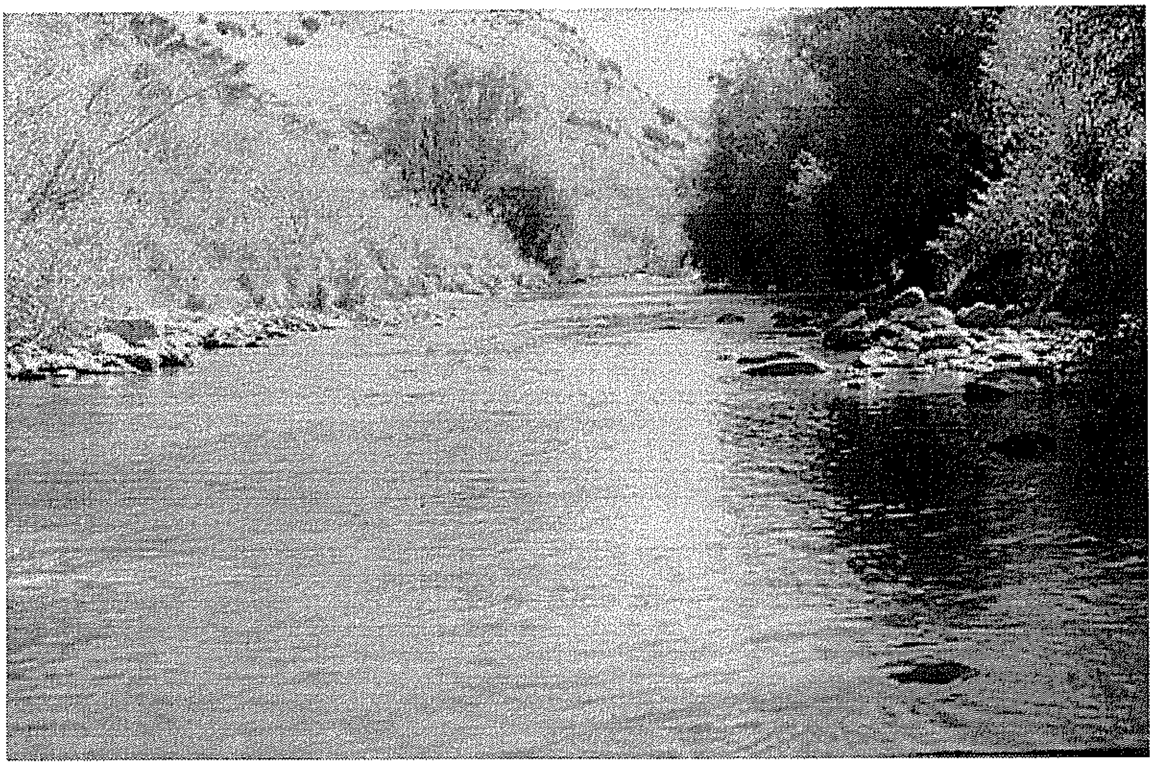
d_{50} = 22 mm

From Hicks & Mason (1991) it looks like:

Page	River	d_{50}	d_{84}	n
114	Waioehu	44	88	0.035
58	Grey	33	67	0.028

Recommend n = 0.031

A3.3 Maerewhenua



Maerewhenua **Site 71106**
Observer #1

"Similar" rivers

Arnold	- much bigger baseflow - more vegetation	.038
Cardrona	- willow trunks (versus, overhanging branches)	.035
Whangaehu	- coarser bed material - less vegetation	.04 - .03
Waireha	- much bigger	.034
Mangaheia	- slightly coarser - less vegetation	.031
Waikehu	- slightly coarser - less vegetation at measured fluxs	.032
Monowai	- coarser	.027
Orere	- similar material - little vegetation	.026 (variable)
Hakataramea	- vegetation on one side	.026 (variable)
Maruia	- finer materal	.028

Choose n = .031

Maerewhenua **Site 71106**
Observer #2

Mean flow
1970 - 1995, Q = 3.00 m^3/s

Bedslope, from longitudinal profile

$$S = \frac{0.32}{150} = 0.0021 \text{ m/m}$$

Bed material

$d_{50} = 33$ mm $d_{84} = 100$ mm

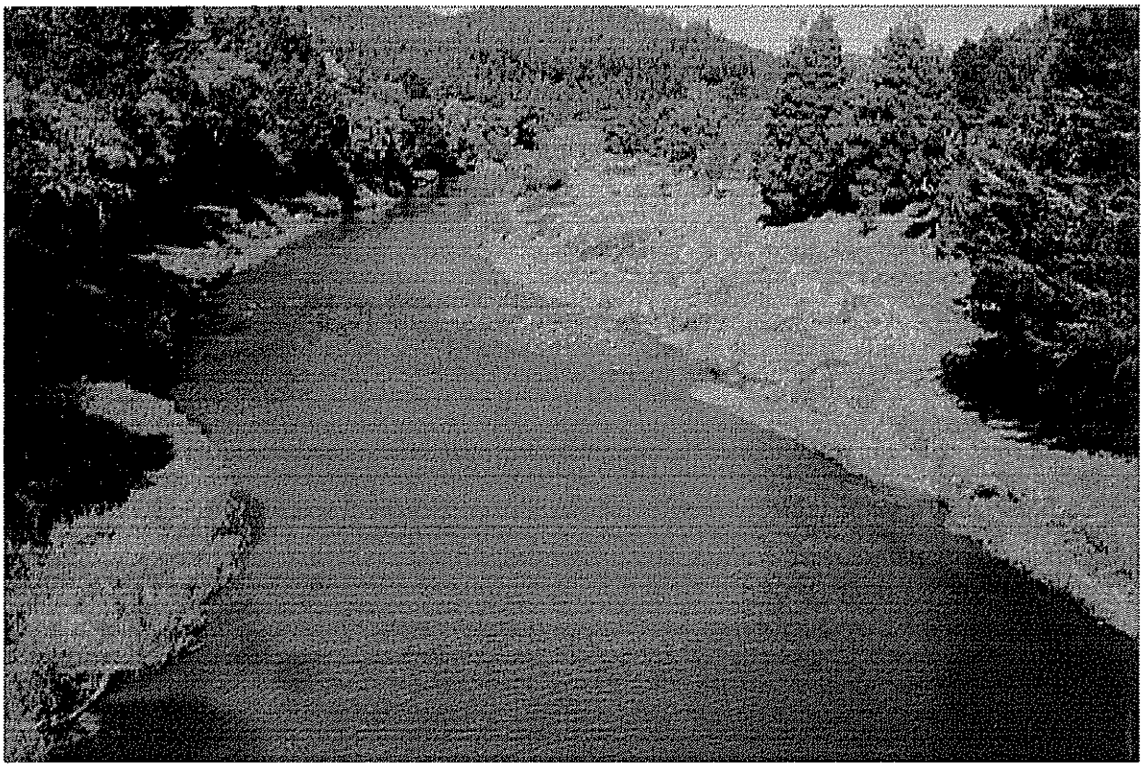
From Fig 2

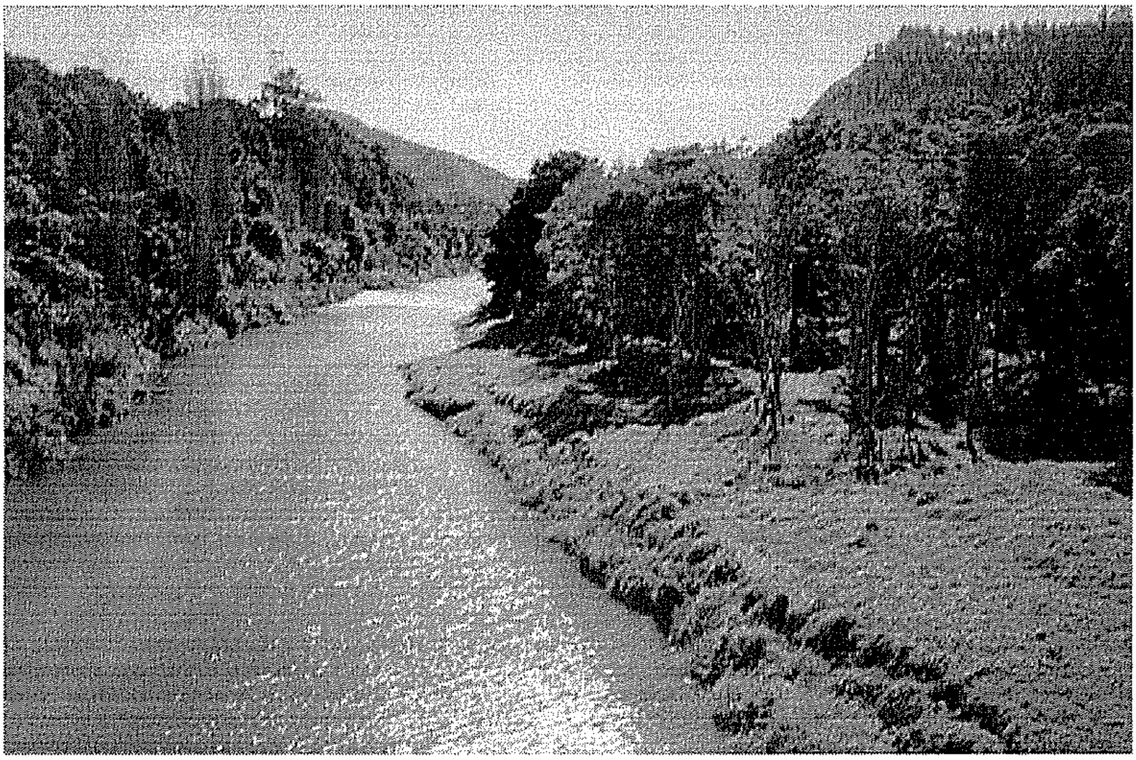
Similar streams could be:

	d_{84}	n	
Piako	1.4	0.025-0.031	-Sand bed Unlike Maerewhenua
Kaipara	?	0.051-0.061	Unlike Maerewhenua
Avon	57	0.026-0.040	-Somewhat similar but weed growth

Also
(P42) Hakataramea 57 0.022-0.031
Mean of 3 values is 0.226, and I would suggest this value for Maerewhenua.

A3.4 Mangakahia @ gorge





Mangakahia @ gorge **46618**
Observer #1

Banks in long grass (mostly), scrub and trees
Straight reach
Gravel bed: d_{50} = 101.5 mm; d_{84} = 168 mm

Compare with:

Fraser at Old Man Range 75259
- similar bed material,
- less vegetation
- straight reach

n = .05 (no high flow values available)

Cobb @ Trilobite 52916
- similar bed material,
- less vegetation,
- reach nearly straight
- n = .039

Collins @ drop structure 58301
- smaller stream,
- straight reach
- scrub and grass banks
- Some large boulders, otherwise similar material
- n = .046

Butchers Creek 90605
- similar bed material,
- straight reach banks thickly bushed
- n = .033

Kapoaiaia @ lighthouse 37503
- similar bed material,
- straight reach
- grassed banks
- n = .06 (no high flow values available)

Pelorus @ Bryants 58902
- similar bed material;
- slightly curved reach similar mix of vegetation;
- wider river.
- n = .024

Ruakokapatuna @ Iraia

- smaller stream; finer material
- (? large boulders visible)
- grassed banks
- n = .040

Waipapa 47804

- finer bed material;
- more bank vegetation
- nearby straight reach
- n = .033 (no high flow values available)

Ngaruroro @ Kuripopango 23104

- coarser bed;
- bush & scrub banks,
- similar size;
- straight reach
- n = .045

Tongariro @ Turangi 1043459

- curved reach,
- vegetated banks
- finer d50, d84 similar
- n ~ .03

Hutt @ Taita Gorge 29809

- straight reach heavily vegetated banks
- similar bed material
- n ~ .03 (?)

Buller @ Woolfs 93208

- large river! straight reach
- vegetated banks (willows)
- finer d50, d84 similar
- n = .03

Very variable; seasons not apparent
Choose n = .035

Mangakahia **Site 46618**
Observer #2

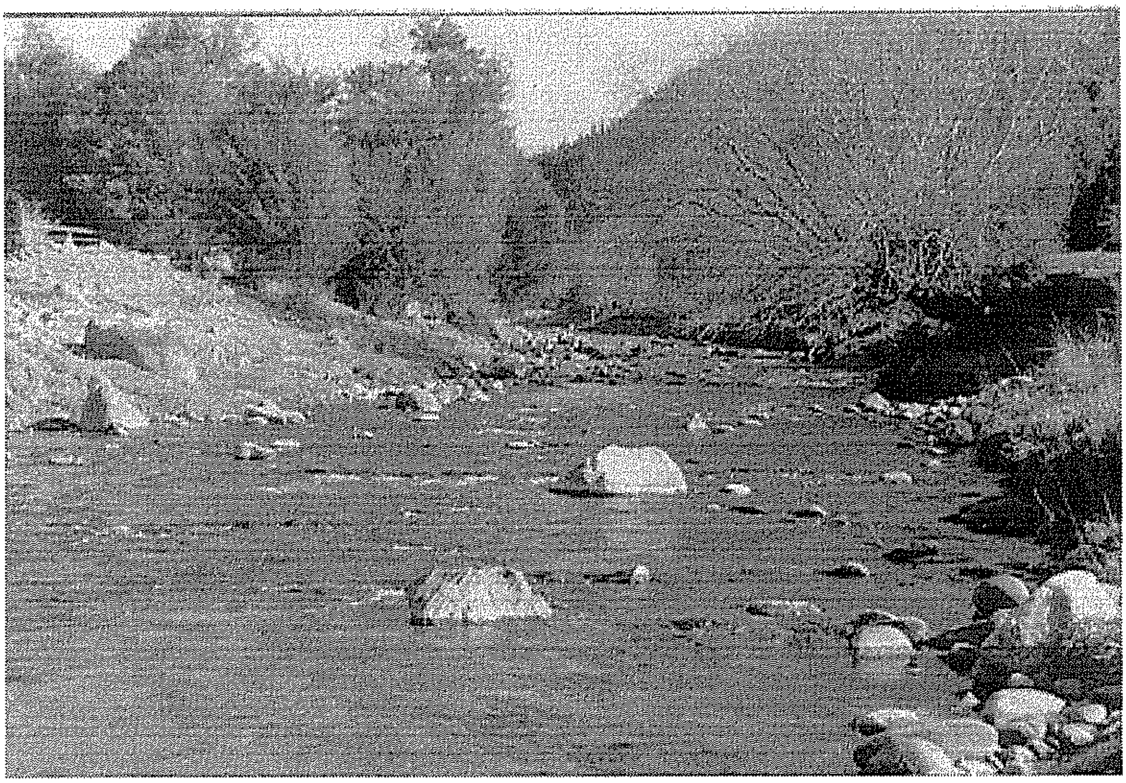
Sediment: cobbles basalt/variolite bedrock

d_{84} = 168 mm
d_{50} = 102 mm

Page	River	d_{50}	d_{84}	n
94	Pomahaka	112	178	0.032
102	Mangaheia	52	116	0.033
132	Clarence	104	200	0.037

Recommend n = 0.034

A3.5 Riwaka South Branch @ Moss Bush



Riwaka South Branch @ Moss Bush **56901**
Observer #1

Small stream, straight reach, grassed banks
Gravel bed: $d_{50} = 55$; $d_{84} = 181$

Ruakokapatuna @ Iraia

- Similar material;
- similar river;
- grassed banks
- n = .040

Clarence @ Jollies

- larger river;
- grassed banks;
- coarser material
- n ~ .027

Pomahaka

- one bank grassed,
- one in willows
- large river;
- straight reach
- d50 coarser;
- d84 similar
- n = .029 (no high flow values)

Collins @ drop structure

- scrub and grass banks;
- some large boulders,
- otherwise similar material
- n = .046

Fraser at Old Man Range 75259

- slightly coarser bed material
- straight reach;
- tussock bombs
- n = .05 (no high flow values)

These sites are variable; reasons not apparent

Choose n = .035

<u>Riwaka South Branch @ Moss Bush</u> **<u>56901</u>**

Observer #2

Boundary river bed

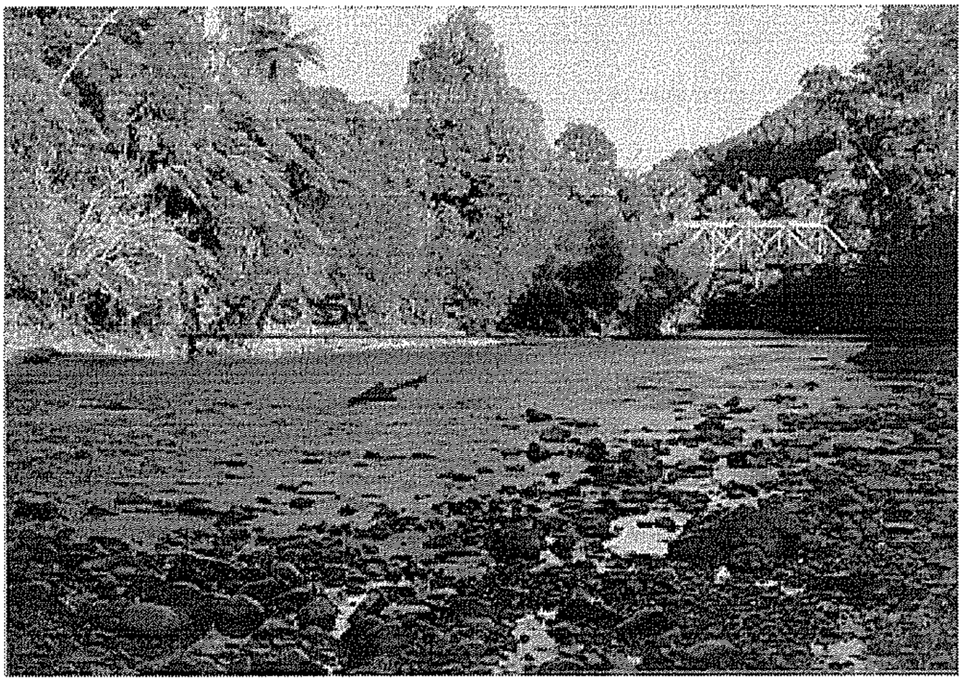
d_{84} = 181 mm

d_{50} = 55 mm

	River	d_{50}	d_{84}	n
126	Hutt @ Kaitoke	86	212	0.037
136	Loganburn	173	-	0.037
170	Hutt @ Taita Gorge	90	170	0.044
242	Fraser	89	208	0.061
250	Ruakokopatuna	45	119	0.065

Recommend n = 0.048

A3.6 Waimana @ Ogilvies Bridge



<u>Waimana @ Ogilvies Bridge</u> **<u>Site 15536</u>**
Observer #1

d_{50} = 40 mm; d_{90} = 150mm

Compare with:

Ngunguru @ Dugimores Rock	-	more bed vegetation
	-	n = .12 but n ~ .05 for higher flows (50% m.a.f.).
Waipapa @ Forest Ranger	-	looks very similar
	-	n = .061 but n ~ .035 for higher flows (1.4 of m.a.f.)
Whirinaki @ Galatea	-	looks similar
	-	n = .056 ~ .04 at higher flows (.038 at 60% of m.a.f.)
Stanley Brook @ Barkers	-	looks similar
	-	n = .054 ~ .03 for higher flows
	-	(.026 for 1/2 m.a.f.)
Jollie @ Mt Cook Station	-	straight, more
	-	n = .049, n = .040 for higher flows (< 40% m.a.f.) uniform
	-	looks coarser, but d50 = 33 mm
Pokaiwhenau	-	n = .063; n = .03 for m.a.f. - grassed banks
	-	rougher bed? bedrock etc
	-	similar curves in reach

A consistent feature of these and other similar rivers in the handbook is that n decreases with increasing Q. (This is not a consistent feature of straighter reaches with lower n).

Choose n = .034 (less than Ngunguru, mean of the others at high flow).

<u>Waimana @ Ogilvies Bridge</u> **<u>Site 15536</u>**
Observer #2

Bed Slope 0.005

d_{84} = 128 mm
d_{50} = 45 mm

Q = 9 m^3/s

$Q_2 = 175\ m^3/s$
max recorded = $385\ m^3/s$

Width 30 m   Some woody vegetation

Choosing from Fig 2 of Hicks & Mason   try pg 238
222
270
198

Comparison site 1

pg 198   Jollie @ Mt Cook Station
$Q = 8\ m^3/s$
$Q_2 = 70\ m^3/s$

$d_{84} = 90$ mm } NB coarser than Ogilvies
$d_{50} = 33$ mm }

Width = 25 - 35 m

Some woody vegetation encroaching into channel looks very similar
NB: difficult to gauge sinuosity of this from plan

n = estimate n = 0.03

Comparison site 2   Stanley Brook @ Barkers

$Q = 1.29$ similar riparian vegetation and bed material
$Q_2 = 64.2$

W = 15 m $d_{84} = 106$
$d_{50} = 32$

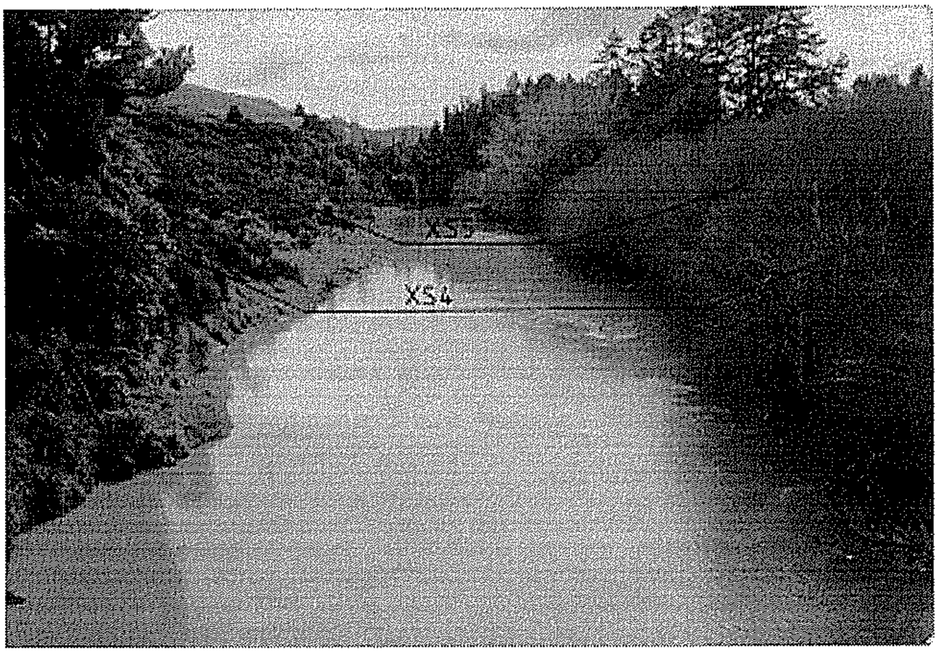
Estimate n at $Q_2 = 0.028$

Comparison site 3   Waipapa @ Forest Ranger pg 238
$d_{84} = 91$ $Q = 4.6\ m^3/s$
$d_{50} = 46.3$ $Q_2 = .250\ m^3/s$
Similar riparian vegetation to Waimana

Estimate n 0.033

Estimate n for Waimana 0.03 range 0.028 - 0.033

A3.7 Waitara @ Tarata



Waitara @ Tarata — **Site 39501**

Observer #1

Silt/sand bed (some gravel); straight reach vegetated banks (willow on one side; steep send and bush bank on the other)

Piako	9140	-	grassed banks;
		-	slightly curved reach
		-	n = .03 (very variable)
Tahuratara	1043428	-	grassed banks;
		-	straight reach
		-	n = .03
Whareama	25902	-	Grassed banks,
		-	straight reach
		-	Plant debris on silt bed;
		-	slope failures
		-	n = .06
Mokau	40708	-	generally rock bed;
		-	gently curved reach willowed banks
		-	n = .062
Kaipara	45311	-	Clay bed;
		-	grass and weeds on banks
		-	Curved reach;
		-	smaller river
		-	n = .05
Rangitaiki @ TeTeko	15412	-	willowed banks;
		-	straight reach
		-	bed some gravel,
		-	mostly silt and sand
		-	n = .043
Waipa @ Whatawhata	43433	-	willows and blackberry in banks slightly
		-	curved reach.
		-	n = .037

Adopt mean of these two sites

estimated n is .040

Waitara @ Tarata | **Site 39501**

Observer #2

Mean width - 60 m

Sinuousity - 1.5

Bed slope 0.006 m/m

Single channel

90% silt-sand particle size with 10% cobbles

Q = 33 m^3/s. Q2 576 m^3/s

Choosing from Figure 2.

try p166 Rangitaiki @ TeTeko
178 Waipa @ Whatawhata

Rangitaiki @ Te Teka

Q = 21.4
Slope 0.0005
width = 40 m

Cross-section shape similar
Riparian vegetation is similar

n = 0.043

Waipa @ Whatawhata

Q = 85 m^3/s
Slope = 0.00012

n @ Mean annual flood = 0.038

width = 45 m

material = sand and gravel

Conclusion: estimate n for Waitara = .04
range .038 - .043