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REDUCTION OF WATER AND BED LEVELS IN THE LOWER WAIKATO RIVER

A thesis
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by

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Abstract

The major aims of this thesis are to determine the historical qualitative and quantitative reduction in water and bed levels of the lower Waikato River, to identify probable factors influencing the reduction, and to predict the future water and bed levels at Huntly in relation to the Huntly Power Station management.

Sediments in the Waikato River are composed of gravelly pumiceous sands. In terms of the mean and median grain sizes, generally sands dominate the lower river bed from the river mouth to the cross section at about 100 km upstream of the mouth, while gravels dominate the bed upstream onward. A hiatus in the longitudinal mean and median grain size distributions from sand to gravel is apparent around Horotiu. In the Waipa River, the texture of bed materials was found to be similar but probably finer to those in the lower parts of the Waikato River.

On the basis that $\partial n / \partial Q = \partial W / \partial Q = \partial S / \partial Q = 0$, where n, W, S and Q are, respectively, Manning's coefficient, width, energy slope and discharge, water level of each gauging can be adjusted to one for an index flow of 350 m^3/s ($WL_{Q=350}$). Further assuming that $\partial n / \partial T = \partial W / \partial T = \partial S / \partial T = 0$ where T is time, variations in the time series of $WL_{Q=350}$ represent the mean bed level changes over time. These assumptions seem to be acceptable for those gauging data within a certain range of discharges in the lower Waikato River.

A quantitative analysis of the available river survey data, water level profile measurements, and gauging records has indicated that, in general, there was a continuous trend of reduction in water and bed levels on the lower Waikato River since the 1960s. The reasons for these reductions likely include sand mining operation along the river, consequent disturbance of the river bed surface, long term sand extraction around Mercer and further downstream, upstream effects of the significant bed level lowering at Mercer, and downstream effects of the 1947 Karapiro Dam closure.

Analysis of river bulk volume changes suggested that the amount of bedload transported into the river downstream of Ngaruawahia was about 160 000±24 000 m^3/yr between 1964–1989. Nearly two-thirds of this was contributed by the bed materials stored in the upstream Waikato River course by an analysis of data from 1974–1989, and the remainder by the Waipa River and the catchment yield from the Hamilton basin.

Two disparate time scales for water and bedload movements in practice result in singular-perturbation characteristics of the system. With a quasi-steady flow approach, linearization of the water and bedload movement system produces a hyperbolic equation or even a parabolic equation (uniform flow). The parabolic equation is a good approximation of the hyperbolic equation under the condition of large values of time or a large distance from the original disturbance. Variations along the Waikato River of an average mean bed level within a certain length of channel are expected to be small. Therefore the linear models can be applied. In the domain between 48.25–94.45 km upstream of the Waikato River mouth, the parabolic model has been used numerically to predict the future bed levels at Huntly in December 2040 for different scenarios. Potential effects on operation of the present cooling water system have been assessed for given discharges at that time.

The ratio of river width to water depth for the formation of alternate bars at the Huntly Railway Bridge has been found to be exceeding 100 by an analysis of the gauging data at this site and at the Ngaruawahia Cableway. This critical width-depth ratio, 100, is much bigger than those suggested in the literature. The corresponding conditions of discharge and

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CHAPTER ONE

INTRODUCTION

Chapter One

Introduction

1.1 Background: nature of the problems

The Huntly thermal power station, which at present is the biggest power station in New Zealand with a generating capacity of 1 000 megawatts (four 250-megawatt units), is located on the bank of the Waikato River at Huntly, North Island, New Zealand, and uses the river water for condensing the spent steam (Figures 1.1–2).

The Huntly power station has potentially serious problems affecting its



Figure 1.1 Photo of the Huntly Power Station. It is the biggest power station in New Zealand, built in 1973–1982 at Huntly by the Waikato River.

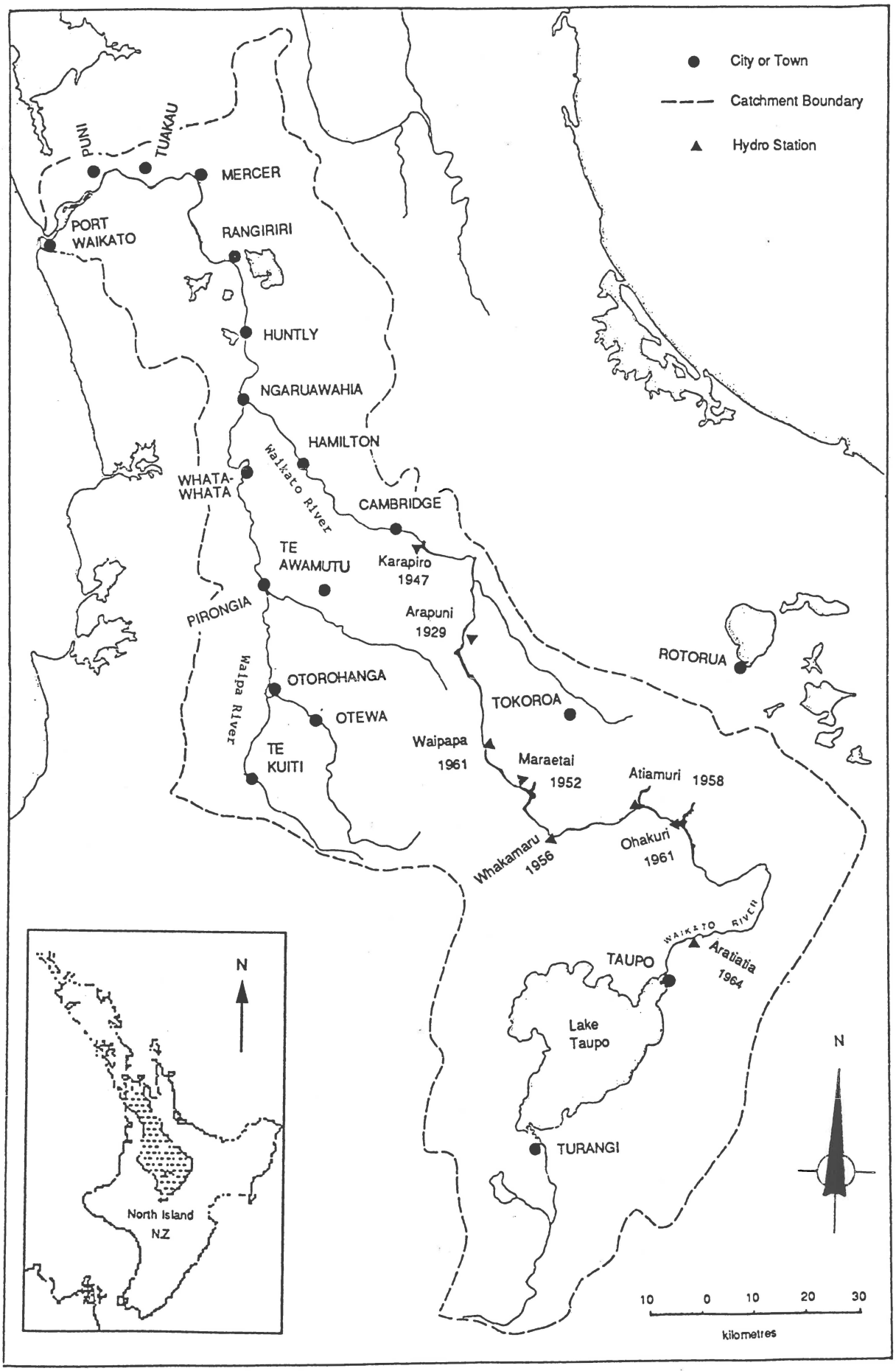


Figure 1.2 Location of the Waikato River catchment and the hydro power stations (modified after

power generation capacity arising from gradual reduction over time of the river bed elevations and water levels at Huntly, as well as from sedimentation in the cooling water intake fore bay (Dahm, 1987; Dahm and Hume, 1989; N. Pritchard, Group Environmental Manager, Electricorp NZ, 1990, pers. comm.). Reduction in water level at a given river flow discharge may affect operation of the cooling water system. Field studies and analyses of historical data available suggest that reduction in bed level may partly be due to the commercial sand extraction along the river, especially the long term operations downstream around Mercer (*cf.* Howard, 1982; Mulholland, 1983; Fenton, 1989). Also the hydro dams at Karapiro and further upstream have stopped the natural bedload sediment supplies from the upper catchment reaching the lower Waikato River, and perhaps induced downstream scouring and bed surface armouring (*cf.* Livesey, 1963; Schumm, 1977; Williams and Wolman, 1984; Sutherland, 1987; Fenton, 1989).

To date a number of river field surveys, broad scale studies of the river morphology and bedforms, and an analysis of effects of a possible Karapiro Dam failure have been undertaken (Ministry of Works, 1954; Ridall, 1967; Mulholland, 1983; 1987; Sledger, 1987; 1989; Fenton, 1989; Webby, 1989). However an enhanced understanding of variations in the future river bed and water levels is of major concern to the power station management. Therefore it is appropriate to undertake this investigation, even though the complexity of fluvial dynamics of the sandy river in conjunction with effects of sand extraction and dam construction, and lack of historical information, make it difficult.

Sponsored by Electricorp, New Zealand, this study has addressed quantitatively the historical changes in river water and bed levels and the corresponding influencing factors, and made a prediction of their future levels at Huntly.

1.2 Objectives of this study

As indicated above, the major concern is to predict the possible future changes in the river water and bed levels at Huntly. Accordingly, the following objectives are addressed in this research program:

- review of the catchment background setting;
- investigation of bed material characteristics, specifically pertaining to the extent of bed armouring and mobile sediments on the lower

Waikato River;

- investigation of historical river water and bed level changes from an analysis of the available river survey data and gauging data;
- identification of possible factors influencing the river changes;
- analysis of bedload sediment balance and investigation of bedload yields from catchments;
- derivation of a one dimensional water and bedload movement model and its application to the lower Waikato River;
- investigation of potential effects of river bed degradation on operation of the present cooling water system of the Huntly Power Station; and
- investigation of big bedform (bar) formation of the lower Waikato River at Huntly;

1.3 Structures of this thesis

In order to achieve the objectives listed above, this investigation involves an integration of information from: (i) field investigations and surveys, and (ii) information held by the Waikato Regional Council (formerly Waikato Catchment Board, and Waikato Valley Authority) and Works (formerly Ministry of Works).

This thesis is structured in the following way:

A brief description of the Waikato River catchment and its management as well as some characteristics of the lower river hydrology and morphology are contained in Chapter Two.

The detailed field studies of bedload sediment characteristics along the river are included in Chapter Three.

An analysis of the available historical river cross sectional survey and gauging data, and water level profile measurements is presented in Chapter Four.

Based on the results in Chapter Four, an analysis of the bedload sediment balance has been undertaken and the results are given in Chapter Five.

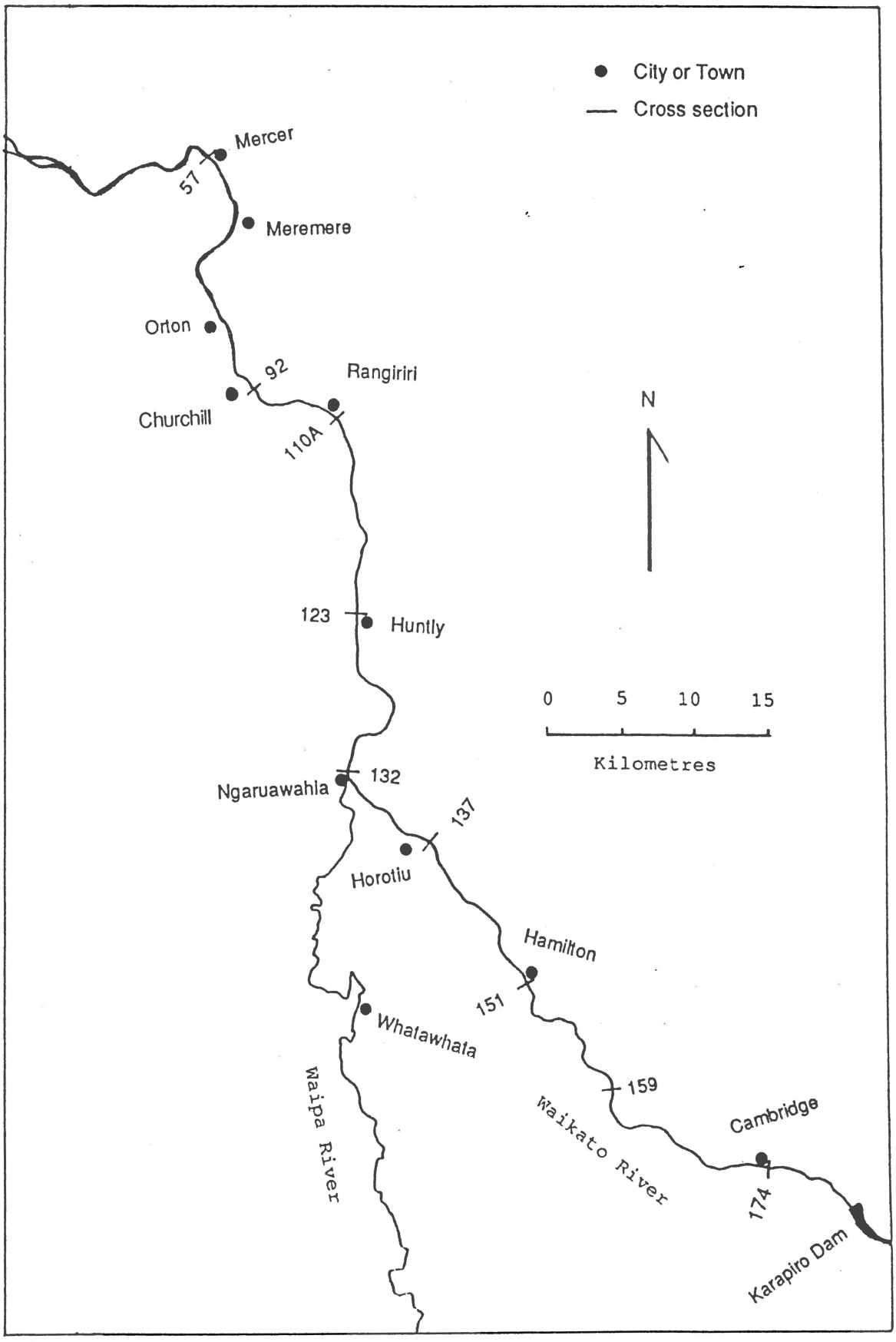


Figure 1.3 Location of the lower Waikato River and some key cross sections.

Derivation of a one dimensional water and bedload movement model, its characteristics and numerical schemes are presented in Chapter Six.

Chapter Seven contains the application of one dimension models to the lower Waikato River in the reach between Mercer and Ngaruawahia.

An analysis of alternate bar formations on the lower river by using the gauging data is given in Chapter Eight.

In the last chapter, a summary of the conclusions for the whole thesis, their implications for river management and recommendations for further research are included.

1.4 Study area

Although the main interest of this research is focused on the bed level degradation of the Waikato River affecting the Huntly Power Station, such degradation is believed due to the combined effects of sand extraction along the river, especially the long term operations at Mercer and further downstream, and of the Karapiro Dam. Accordingly, the study area extends from Karapiro to Mercer (Figure 1.3). and numerical models are applied to the reach between Mercer and Ngaruawahia. The study of the detailed formation of the big bedforms (bars) focuses on the cross section at the Huntly Railway Bridge using the historical gauging data from this site and the gauging data at Ngaruawahia Cableway.

There are various methods to subdivide the Waikato River into reaches (Ridall, 1967; Strachan, 1979). Since this study is more concerned with the reach from Karapiro to Mercer, the lower Waikato River is defined here as downstream of the Karapiro Dam to Mercer or sometimes to its mouth, and the major tributary of the lower river system, the Waipa River, is also included.

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CHAPTER TWO

WAIKATO RIVER DESCRIPTION

Chapter Two

Waikato River Description

2.1 Introduction

As with other rivers of the central volcanic plateau of New Zealand, the bed and banks of the Waikato River are composed mainly of gravelly pumiceous sands of acid volcanic origin. In addition, natural flow regimes of the river are regulated by the hydro dams upstream and flood control constructions on its lower system. These include control gates, spillways and pumps. (Mulholland, 1983). This chapter briefly introduces the catchment setting, its' management and development, and characterises the hydrology and morphology of the lower Waikato River.

2.2 River catchment setting, management and development

The Waikato River catchment is the most highly developed for hydro electricity generation in the country. It is located in the centre of North Island, New Zealand (Figure 1.2) with a catchment area of 14 258 km^2. The altitude in the catchment varies from sea level at the river mouth to 2 797 m (Moturiki Datum is used throughout this thesis), at the summit of Mt Ruapehu inland. About half of the catchment area is over 300 m in elevation.

The source of the river system is the central North Island volcanoes of Ruapehu, Ngauruhoe and Tongariro, from which it flows 425 km northward to the Tasman Sea (Strachan, 1979). From Lake Taupo, the river goes through the plateau-lands in a deeply incised channel across which are eight hydro dams, and then flows through plains. A number of small lakes are located in the upper reaches, while more extensive shallow lakes and peat swamps are features of the lower stretches of the river. The main uncontrolled tributary, the Waipa River, joins the Waikato at Ngaruawahia (Figure 1.2).

The river catchment can be subdivided into the following subcatchments.

Lake Taupo controls the upper catchment area of 3 289 km^2. The central sub-catchment drains an area of 4 419 km^2 in the reach between Lake Taupo and the Karapiro Dam. Below the Karapiro Dam towards Ngaruawahia is the Hamilton basin with a subcatchment area of 629 km^2. Joining at Ngaruawahia, the Waipa River has a catchment area of 3 059 km^2. Downstream of Ngaruawahia the lower sub-catchment area is 2 862 km^2 featured by shallow lakes and peat swamps. As a percentages of the whole catchment the subcatchments contribute respectively, 23%, 31%, 4.4%, 21.5% and 20.1% (Figure 2.1).

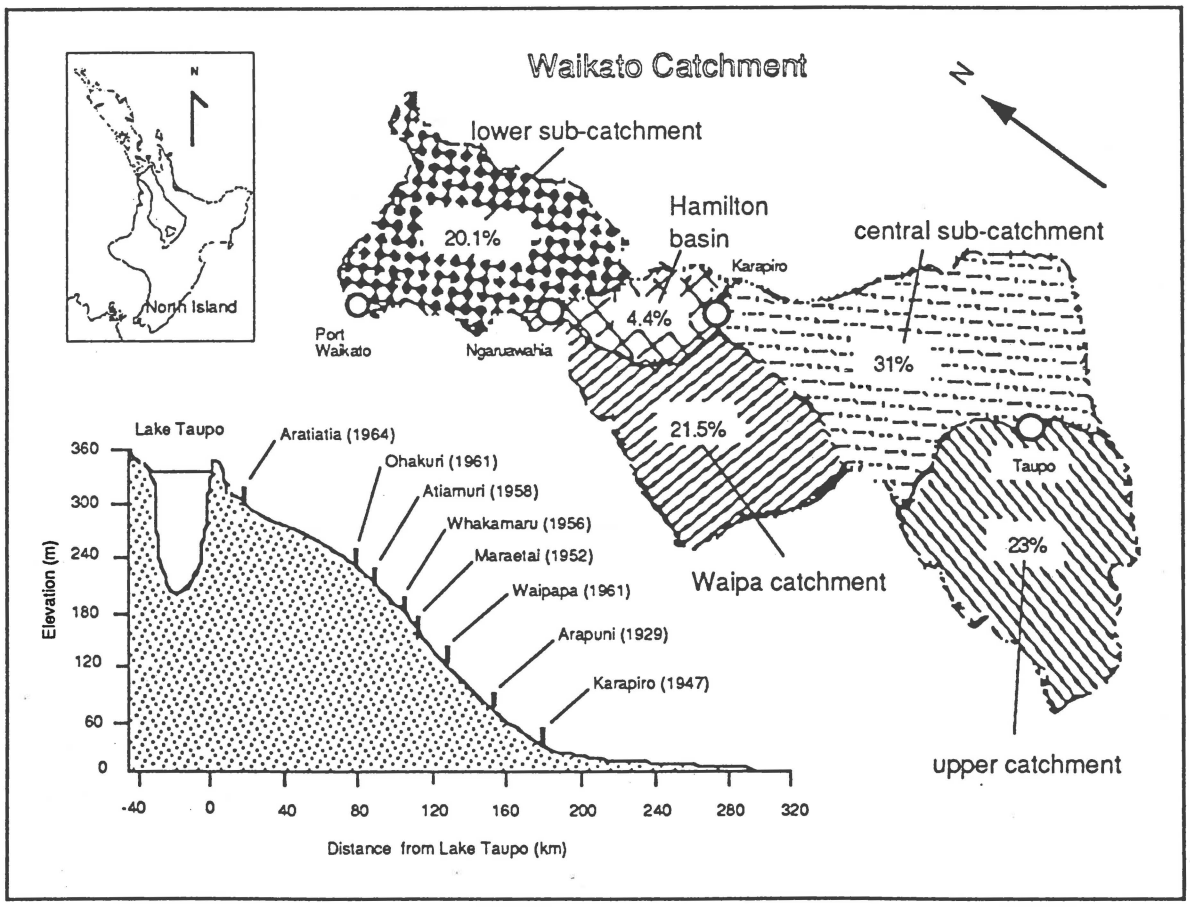


Figure 2.1 Subcatchments of the Waikato River, their area contributions, and longitudinal river profile (data from Strachan, 1979; and information held at the Waikato Regional Council).

As in many other areas of New Zealand, the basement rocks of the Waikato catchment are grey sandstone (greywacke) and dark mudstone (argillite), which were laid down in the sea some 200 million years ago (Mesozoic). These rocks have undergone various phases of tectonic movements which have resulted in their present distribution in the fault blocks around the perimeter of the catchment. The sediments deposited during the early and middle Tertiary during marine transgressions are termed the Te Kuiti

Group and the Waitemata Group respectively. Now these Tertiary sediments can be found only in restricted areas in the lower Waikato basin and the Waipa catchment (e.g., south-west of Otorohanga and Huntly north) as a result of long term erosion.

Volcanism has had a substantial influence on the catchment (Suggate, 1978; Strachan, 1979). Geologically recent explosive volcanic activity includes the so-called Taupo Eruptions. These began about 10 000 years ago with ash flows ejected from vents in the east of Lake Taupo resulting in great fans of pumice running across the lake and down the river. There were further eruptions every few hundred years, climaxing about 1 800 years ago. The impounded water in the lake flooded down the Waikato River carrying a sludgy mass of pumice debris, and formed thick deposits of pumice alluvium plastered high up on the river banks and on the low terraces, and spread pumice far and wide over the lower valley (Schofield, 1967; McCraw, 1971). Downstream of Lake Taupo the surface of the catchment is predominantly composed of acid volcanic rocks, mainly ignimbrite and rhyolite. There are large deposits of both water and air borne volcanic debris, mainly loose pumice gravel and quartz-pumice sand mixtures in the region. This geological background makes the catchment and the river bed and banks, which are primarily composed of easily erodible volcanic debris such as sand, pumice and gravelly alluvium.

Some river surveys of channel limits, shoals and depths were conducted as early as in August 1862 by Captain Greaves for navigation purposes (Sledger, 1987). Historical cross sectional surveys were undertaken in 1913 in the river reach downstream of Rangiriri. Prior to 1958, river surveys were conducted by the Public Works Department and also the Power Design Survey. Subsequently they have been carried out by the Waikato Regional Council (formerly the Waikato Catchment Board, formerly the Waikato Valley Authority). The 1963/1964 survey was the first comprehensive cross sectional survey in the lower Waikato River and since then regular surveys have been taken about every 5–6 years (Sledger, 1987; 1989).

Continuous hydrologic data collection has been carried out since 1958 in the Waikato River (Hannah, 1981). Before that time some hydrologic data was obtained by the Power Design Survey as a part of hydro investigations.

The first hydro station in the Waikato River catchment was the Horahora Station which operated from 1913 until 1947 when it was submerged by formation of Lake Karapiro. At present (1993), in addition to providing

cooling water for two geothermal stations at Wairakei and Ohaaki, and two thermal stations at Huntly and Meremere, the Waikato River system also supplies 10 hydro power stations (Figure 1.2, the other two are parts of the Tongariro Power Development). In total these 14 power stations can produce over 65% of the North Island's electricity requirements, and over 40% of New Zealand's total electricity generation.

During construction of the Arapuni Dam in 1928, about 5 400 000 m^3 of debris were deposited into its downstream channel because of emergency use of a partly natural spillway (Furkert, 1935) and caused a large amount of aggradation of the downstream river bed.

Since the late 1950s, commercial sand extraction has occurred on the lower river and a total of about 13 000 000 m^3 of sands have been extracted. Before that time although there was sand extraction (M. Tetslaff of W Stevenson & Co., pers. comm., 1991), its operation was small and no records are available. The 1990 annual extraction rate was restricted at 400 000–500 000 m^3, with about half of the volume taken from Meremere and Mercer, and the remainder from Tuakau and Puni further downstream. Maintenance dredging was also undertaken periodically at the fore bay of the Huntly power station. Since 1991 sand in the fore bay has been regularly pumped out and put back into the river downstream.

After floods in the 1950s, the *Lower Waikato-Waipa Control Scheme* was instigated by the Waikato Valley Authority in 1961 and all works of river training and controlling were substantially completed by 1982 (Mulholland, 1983). In the early 1970s, physical model studies of the lower Waikato River were undertaken by Hydraulic Research Station, Wallingford (1971) to investigate the ways of improving cross sections to minimise inundation. Some river training works were subsequently carried out on the recommendation of its reports. All the remedial works have radically changed the complex interplay of flows between the river and its adjacent ponding basins (Ridall, 1967; Mulholland, 1983). Flows of the Waikato River at present are regulated by the Taupo control gates (installed in 1941), hydro power stations upstream of Karapiro and the flood control features on the lower river system.

2.3 Sediment yield, hydrology and morphology

2.3.1 Sediment yield

The following discussion on sediment yield is based on measurement of sediment load, although other approaches, such as estimates of catchment erosion and sediment transport equations based on field and/or laboratory data, can also be applied.

According to the modes of sediment movement, total sediment load is divided into *bedload* and *suspended load*. The former is defined as sediment moving along and supported by the bottom of rivers, and the later as moving suspended in the water column. In terms of sediment composition, total sediment load can also be divided into *bed sediment load* and *wash load*, defined as, respectively, materials found in appreciable quantities and in very small quantities in the surface bed layer. Both bed sediment load and wash load can move as bedload and suspended load from the above definition although all wash load moves in suspension in practice (Vanoni, 1975).

Lake Taupo cuts off the bedload supply from the upper catchment, and the eight hydro stations between Karapiro and Lake Taupo are expected to stop sediment supply from the central subcatchment to the lower river and to cause downstream degradation and bed armouring. There is no historical information available about contribution of bedload sediments from the main tributary—the Waipa River. Other small tributaries in the lower river are expected to supply suspended sediments and volumes of water during storms but provide little, if any, bedload (Fenton, 1989). Rates of bedload sediment transport at Churchill East (cross sections 92–93, Figure 1.3) in the river have been measured at around 170 000 m^3 per year (Dahm, 1987; Fenton, 1989). Schofield (1975) gave an annual bedload movement of 42 000 m^3, which is the same order as annual suspended load yield and seems to be too small.

Suspended load in the lower Waikato River is small in comparison with the other rivers in North Island (Adams, 1979). But the suspended load concentration in the Waipa River seems to be a little high. This can be demonstrated in Figure 2.2 which shows the relationship between discharges and concentrations of suspended sediments at Hamilton in the Waikato River and at Otewa (catchment area 317 km^2) and Whatawhata (catchment area 2 826 km^2) in the Waipa River. The mean annual discharges at Otewa and Whatawhata are 12.5 m^3/s (1985–1992) and

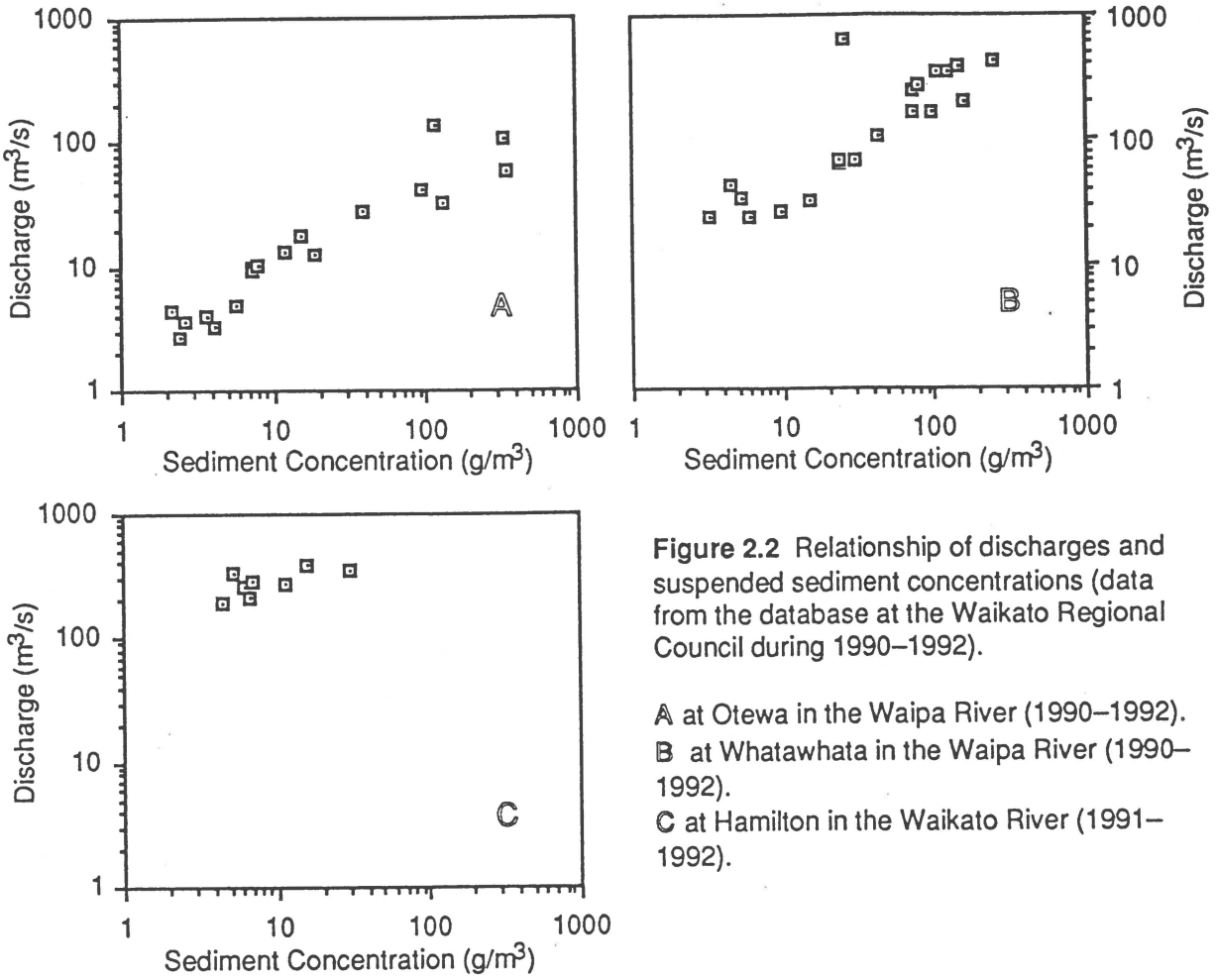


Figure 2.2 Relationship of discharges and suspended sediment concentrations (data from the database at the Waikato Regional Council during 1990–1992).

A at Otewa in the Waipa River (1990–1992).
B at Whatawhata in the Waipa River (1990–1992).
C at Hamilton in the Waikato River (1991–1992).

85.0 m^3/s (1969–1992) respectively. If suspended sediment concentrations of 13 g/m^3 and 22 g/m^3 are taken respectively for Otewa and Whatawhata (Figure 2.2), the annual suspended sediment yields are 5.12 kt and 58.97 kt, or 16.2 $t/km^2/yr$ and 20.9 $t/km^2/yr$. For the Waikato River at Hamilton, the annual suspended sediment yield is 63.02 kt for an annual discharge of 249.8 m^3/s (1976–1992) and suspended sediment concentrations of 8 g/m^3 (Figure 2.2). The annual suspended load at Ngaruawahia (including the Waipa River contributions) is about 120 kt, or 49 000 m^3 for a density of 2.46 t/m^3. This is much smaller than the annual bedload of 170 000 m^3, which was measured at Churchill East.

2.3.2 Lower river hydrology

The average annual rainfall over most of the Waikato River catchment is 1 200–1 600 mm, peaking around 3 000 mm in the mountains (Strachan, 1979). The mean annual rainfall at Ruakura, Hamilton, was 1 201 mm for the period 1905–1980 (NZ Meteorological Service, 1983). Rainfall is relatively evenly distributed throughout the year with a slight increase in winter

months (Figure 2.3) (de Lisle, 1967; Tomlinson, 1976; Hurnard and Coulter, 1979; Heine, 1985).

The total annual loss from Lake Taupo in terms of evaporation averages over 700 mm (Ministry of Works, 1972). Other workers have suggested an annual loss of 650 mm for the lower Waikato through evapotranspiration (Finkelstein, 1973; Strachan, 1979; McColl and Ward 1987).

River flows downstream of Ngaruawahia are not only controlled by operations of the hydro power generation system from Lake Taupo to Karapiro (draining 54% of the total catchment area), and the flood control constructions on its lower system, but also contributions from the tributaries

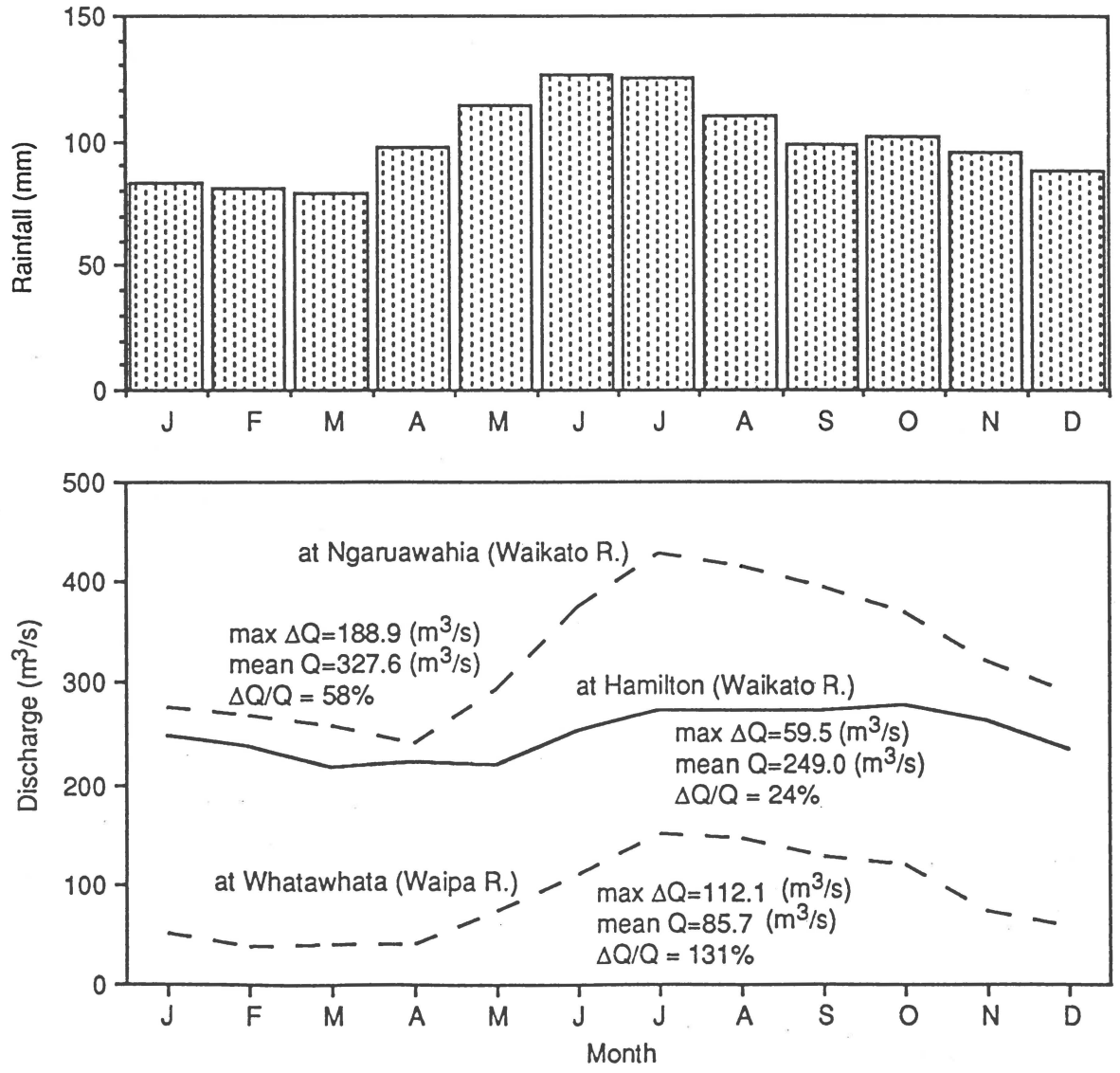


Figure 2.3 Distributions of long term annual rainfall at Ruakura, Hamilton in 1905–1980, and river flows at Hamilton in 1981–1989, at Ngaruawahia in 1958–1990 and at Whatawhata in 1970–1990 (data from NZ Meteorological Service, 1983, and the TIDEDA database at the Waikato Regional Council).

downstream, especially from the Waipa River (representing 21.5% of the total catchment area). Contributions from the tributaries to an extreme flow at Huntly may be important. Operation schedules of the hydro system are governed by some constraints and national electricity requirements. Flooding is of less concern in the upper catchment, and storm waters are largely accommodated in the hydro lakes or channels.

The Lake Taupo Compensation Claims Act 1947 sets the flood level of Lake Taupo at 357.39 m, and the Tongariro Offset Works Agreement requires discharges to support a minimum load of 40 megawatts (equating to a discharge of 148 m^3/s) at the Karapiro power station and smoothing out of flow variations as much as possible. However there are still some cyclical trends of flows in the lower river system due to power generation operations as well as tributary contributions (Figures 2.3–4).

Since the implementation of the Tongariro Power Development in the 1970s, an amount of water from adjacent catchments has been diverted to Lake

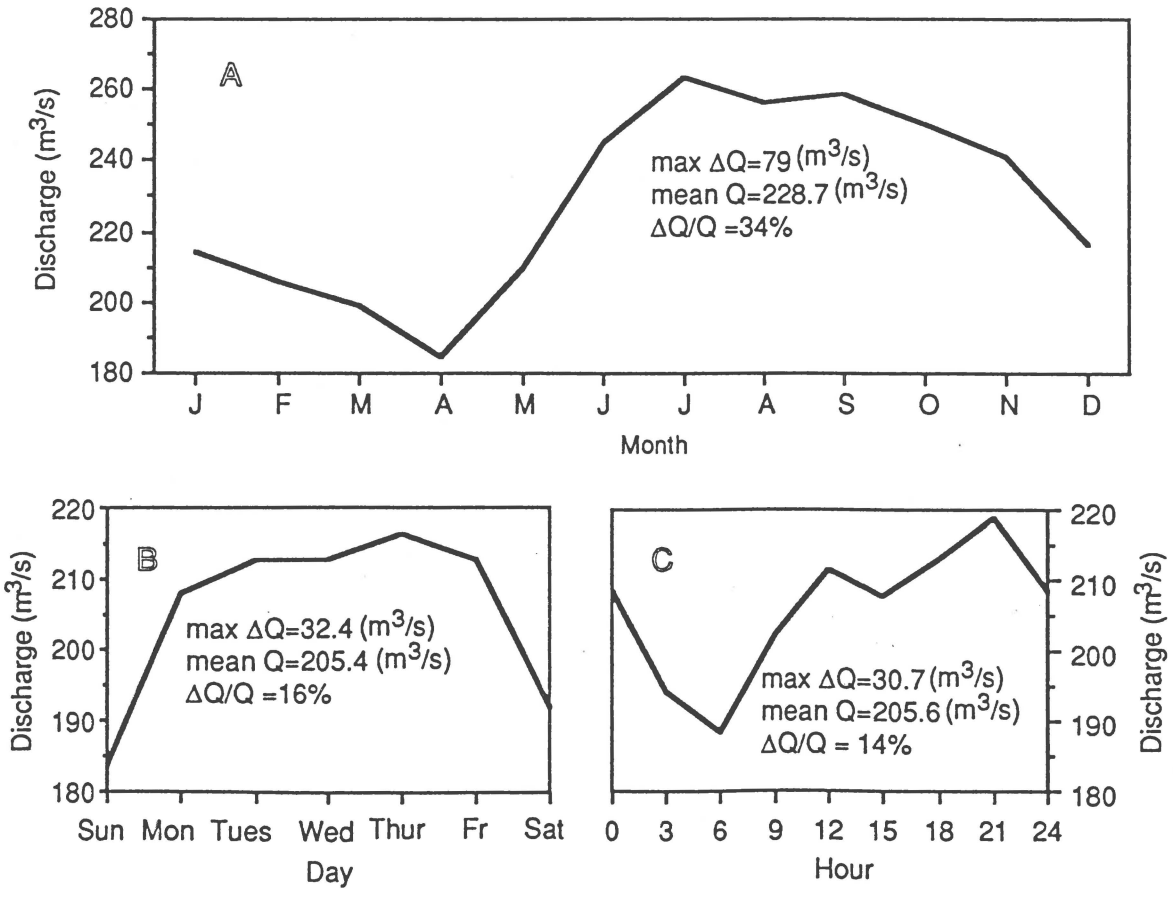


Figure 2.4 Variations of discharges over time at the Karapiro Dam. Figure A shows the mean monthly flows (1960–1985), and Figures B and C show the weekly and daily cycle of mean discharges respectively in 1985 (data from Mulholland, 1987).

Taupo and the Waikato River. The Western Diversion (diverting waters from the headwaters of the Wanganui River into Lake Rotoaira) began to operate in a small way in February 1971 and completely from November 1972. The East Diversion (diverting waters from the Rangitikei River to the Tongariro River) has been operating since October 1979. This project has contributed an extra long term mean flow of 34 m^3/s to the Waikato River, representing a 26% increase in mean Lake Taupo outflows.

An extra constant discharge at 34 m^3/s was suggested for the purpose of engineering design in the lower river system under normal conditions because of the Tongariro Power Development (Mulholland, 1983). Trends in flow increases due to the Tongariro Diversions can be found from Table 2.1. But the magnitude of flow changes decreases evidently as catchment area increases. The reasons may be that there was a decrease in runoff from Taupo to Karapiro during these times (Mulholland, 1987) and the average annual catchment rainfall for these three periods (1960–1970, 1971–1979 and 1980–1985) was 1 564 mm, 1 501 mm and 1 331 mm respectively, and these could partially account for the drop in runoff.

Table 2.1 Variations in mean discharge (m^3/s) due to the Tongariro Power Development for periods 1960–1970, 1971–1979 and 1980–1985 respectively (Data from Mulholland, 1987; and the Waikato Regional Council records).

Site	*Discharge (m^3/s)*		
	1960–1970	1971–1979	1980–1985
Taupo control gates	123.4	140.0	151.6
Whakamaru	172.9	180.3	185.5
Arapuni	212.5	212.9	215.6
Karapiro	227.8	230.4	228.1
Ngaruawahia	333.2	326.1	303.7

The data in Table 2.1 show that no great effects on the long term trend of discharge increases in the lower river can be expected due to implementation of the Tongariro Power Development because the random variation of rainfall-runoff is bigger than the effects of the project in these particular periods.

A seasonal pattern of discharges at Karapiro, typical of an uncontrolled catchment with a Winter maximum and an Autumn minimum, is shown in Figure 2.4A. In comparison with a natural river, the range of discharge variations is smaller. The maximum difference of the monthly discharge is 79 m^3/s, representing 34% of the mean discharge of 228.7 m^3/s. For the uncontrolled Waipa River the relative monthly variation is 131% (Figure 2.3).

Because of the effects of discharges from the Waipa River (draining 21.5% of the total catchment area) and the other small streams, there is a relationship between flow pattern and rainfall distribution in the river downstream of Ngaruawahia. For the 1958 flood of 1 427 m^3/s at Ngaruawahia, the individual contribution of the Waipa River could be 1 220 m^3/s (peak flow at Pirongia) in comparison with the discharge of approximately 900 m^3/s in the Waikato River at Hamilton (Waikato Valley Authority records).

The short term (weekly or daily) cyclical trends in flows at Karapiro are illustrated in Figures 2.4B–C. The relative variation is very small and the flows are reasonably uniform because of the constraints on maximum daily variation of flows and the requirements of smoothed out discharges.

Mercer is the upper point of tidal effects in the Waikato River (H. McMullan, staff of the Waikato Regional Council, 1992, pers. comm.) and short term variations in water level at Mercer have been observed with a range of about 10 cm under the conditions of high tides and low river flows.

The reach between Karapiro and Ngaruawahia has a catchment area of 629 km^2, representing 4.1% of the total catchment and a length of 54 km of water-way with some small tributaries. This section is deeply entrenched within the Waikato plains and except for storms, the flow patterns are totally controlled by the Karapiro Dam operation. At Ngaruawahia the Waikato River is joined by the Waipa River. Although water flow discharged from the Waipa River into the Waikato River is only 10 m^3/s under the condition of low flows, compared with about 200 m^3/s at Ngaruawahia in the Waikato River under the similar conditions, flow variations in the Waipa River as a result of storms are great and its contribution is significant (refer to the 1958 flood mentioned above and Figure 2.3).

Between Ngaruawahia and Mercer, the subcatchment area is about 2 145 km^2 (15% of the total catchment) and the length of this reach is about 55 km. There are large areas of peat swamps and shallow lakes in this subcatchment. The natural complex interplay between the river waters and pondings has been partly changed by the flood protection control schemes. The flow patterns in this reach are subject to the effects of power generation from the dams upstream, the levels of the river, swamps and lakes, the operation of the Whangamarino flood control gates, and the effects of tides, especially under low flow conditions.

Because of the reductions in bed elevation, the tendencies of reductions in

water level for a given discharge are evident at the lower parts of the river. From the rating curves used at the Mercer Bridge (gauging No. 1043446) water levels at discharge 350 m^3/s dropped by 1.28 m over the period 1960–1990. This reduction directly results from the bed level lowering assuming constant flow resistance, slope and river width. This study was mainly interested in the effects of bed elevation reductions on the water level changes.

2.3.3 River morphology

The Waikato River bank and bed are mainly composed of erodible and movable volcanic sediments, such as pumice, glass shards, quartz and feldspar, as well as the heavy mineral titanomagnetite. The river morphology on the sandy bed in the lower Waikato River (Ridall, 1967; Fenton, 1989) is typified by Pettijohn et al. (1972) description: "*Morphologically, the alluvial transport of sand creates forms ranging from the size of small ripples only a few centimetres high to meanders belts many kilometre wide and 40 to 50 metres or more thick. A wide range of scales of dunes and point and lunate bars occur in between.*" Various channel patterns—straight, sinuous, braided—as defined by Leopold and Wolman (1957), may also exist in the lower Waikato River.

Friedkin (1945) demonstrated alluvial river meandering processes in a laboratory. Natural channels are rarely stable over geological time periods (one million years), and even in short time scales (less than one hundred years) systematic erosion and deposition can occur (Hey, 1982). The lower Waikato River courses have greatly changed over the periods of geologic time (McCraw, 1967). Previously the river flowed out to the Bay of Plenty, and during another phase to the Firth of Thames, but following capture just above Karapiro the channel changed and it flowed into the Hamilton basin. McCraw (1967) identified the old river courses in Hamilton basin by the levees of coarse sediments and produced a map of generalised topography and old courses of the Waikato River. Recently, since human activities such as dams, flood control gates, river training works, bank protection, as well as the amount of heavy vegetation on the river banks resisting bank erosion, and rock outcrops, the lower river courses have been relatively stable. There have been no identifiable substantial channel changes such as cutoffs since 1942 based upon a casual aerial photograph comparison, and its banks are also relatively fixed with the exception of some places altered by river training works in the period of the 1964–1988 survey (Figure 2.5). The relative change of a river width in Figure 2.5 is defined as the ratio of a

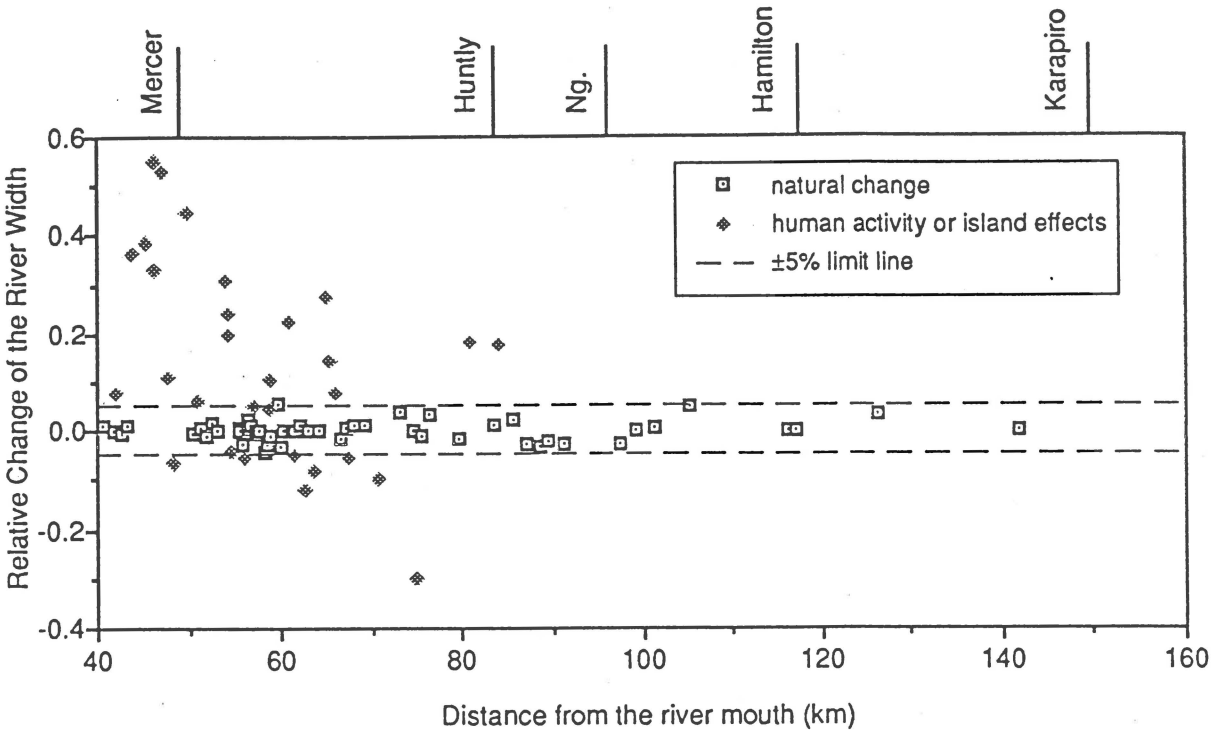


Figure 2.5 The relative changes of the river widths at the normal water level between the 1964 and 1988 surveys (data from the Waikato Regional Council river surveys). A positive value means reduction of the river width over time. Except for human activity (river training work, etc.) or island effects, the river boundaries have been very stable during the survey period 1964–1988.

difference between the two widths measured at the different times to their mean.

River surveys by the Waikato Regional Council and the echo sounding data from Hamilton to Puni by Fenton (1989) show that the large fluctuations of bed level may partly be attributed to channel restrictions (rock controls, gorges, bridges, etc.), sand extractions and big bedforms (bars, dunes). There was a noticeable aggradation of the lower river channel from 1928 caused by the debris deposition during construction of the Arapuni Dam during 1928–1929 (Furket, 1935). For the lower river, a phase of 3-year aggradation was reported using the survey data in 1928–1932, and the main deposition area was in the channels below Huntly and silting increased progressively downstream (Ministry of Works, 1954; J. van Kampen, 1992, pers. comm.). Subsequent changes of the bed levels were not significant when compared with the available survey data in 1954 (Dahm, 1987). Comparison of the mean bed levels and the cross section areas (see Appendix One) shows that in most parts of the lower river average bed elevations have reduced since 1964 (Mulholland, 1983; Sledger, 1987; 1989; Fenton, 1989). However the cross

sectional areas at some places have increased due to river training works and dumping materials (eg. at the Horotiu Bridge), as well as effects of big bedform (bar) movements (Figure 2.6). If we extend the time to 1913, the date of the first survey on the lower river which can be compared with the present (1987), bed levels at the river delta are lower by about 1.3 m (Sledger, 1987). The detailed analyses and discussions of river surveys and survey data are given by Mulholland (1983) and Sledger (1987; 1989).

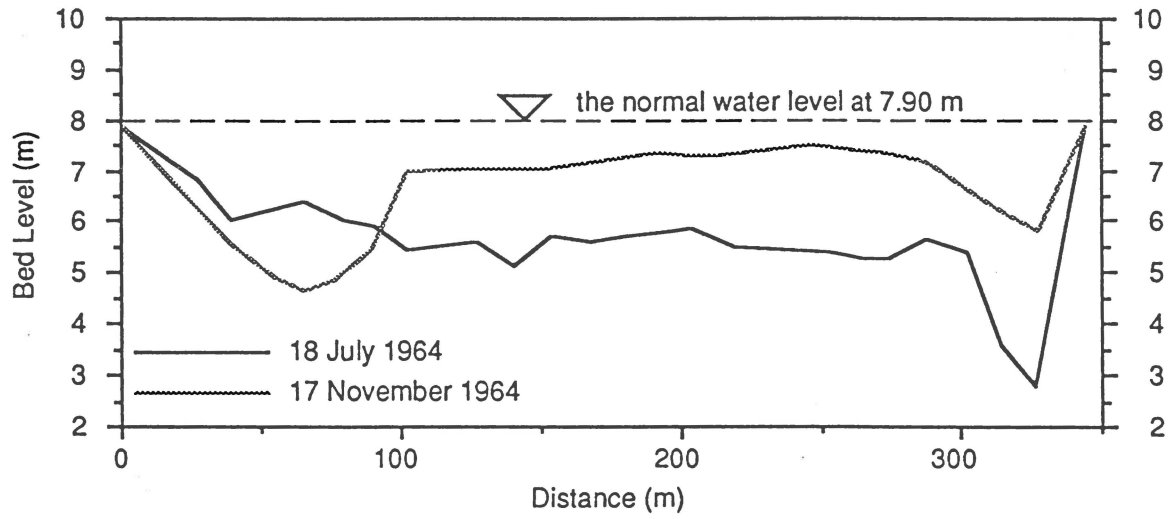


Figure 2.6 Effects of large bedforms on cross sectional configuration near cross section 121 at Huntly (data from Waikato Valley Authority, Plan No. 1646/434004). The cross sectional areas below the normal water level are 768 m^2 and 410 m^2 for the surveys of 18-07-1964 and 17-11-1964 respectively and their difference is 358 m^2. The mean bed level changed from 5.67 m to 6.71 m and the mean depth decreased by 1.04 m.

The longitudinal water profile of the lower Waikato River between the Karapiro Dam and Mercer is shown in Figure 2.7. This is derived from data of water profile measurement by the Waikato Regional Council between Ngaruawahia and Mercer on 8 October 1973, and by the Ministry of Works between the Karapiro Dam and Hamilton on 15 October 1973. The value of slopes shown in Figure 2.7 is an average in the corresponding channel reaches. Generally the slope decreases downstream except for those areas near Mercer, where long term sand extraction occurs.

Sediment grain size along a river in the downstream direction may increase, decrease or remain constant (Hack, 1957). However a reduction in size of bed materials in the downstream direction is common for many alluvial rivers (Knighton, 1987). Fenton (1989) produced a longitudinal mean grain size distribution of the lower Waikato River bedloads with 19 samples

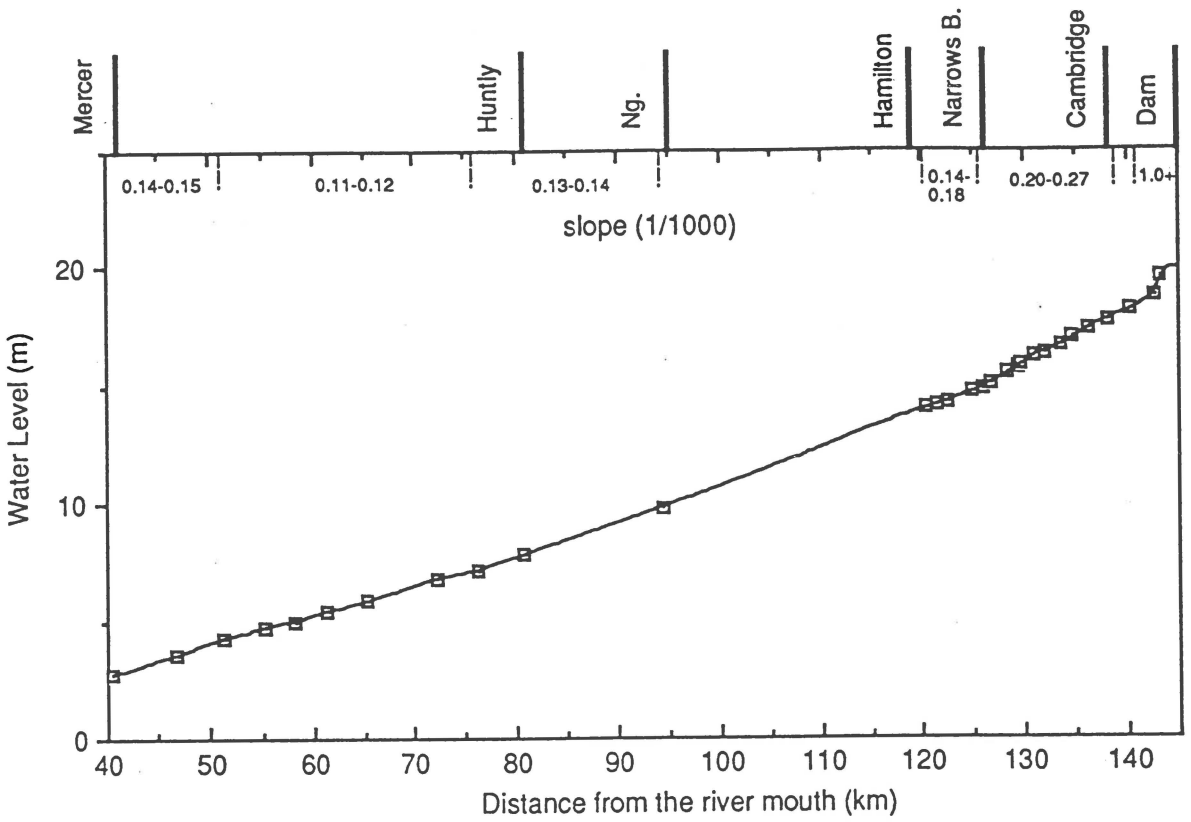


Figure 2.7 Longitudinal profile of the lower Waikato River between Mercer and Karapiro in 1973 (data from water profile measurements held at the Waikato Regional Council). The water levels between Mercer and Ngaruawahia were measured on 08-10-1973, between Hamilton and the dam on 15-10-1973.

concluded a slight decrease in trend in the lower parts of the river, which would be supported by the Passega C statistic, defined as the coarsest 1% of the sample (Figure 2.8).

The two different regimes of grain size distribution shown in Figure 2.8 can be respectively attributed to the downstream effects of the Karapiro Dam and the normal decrease in grain size of a river in the downstream direction. A similar phenomenon has been found in the Colorado River downstream of Hoover Dam (Williams and Wolman, 1984, page 33). So far there is no information about the bed material distribution prior to the dam construction to compare with the contemporary data to show effects of the Karapiro Dam. A natural (un-dammed) river can also have a similar sediment distribution as that shown in Figure 2.8 (cf. Yatsu, 1955; Monsalve and Silva, 1983). A more detailed discussion is presented in Chapter Three.

Sediment movement is accompanied by the organisation of grains into morphological elements known as bedforms. Ideally, a dynamic classification of bedforms based on bedform mechanics is required, but so far

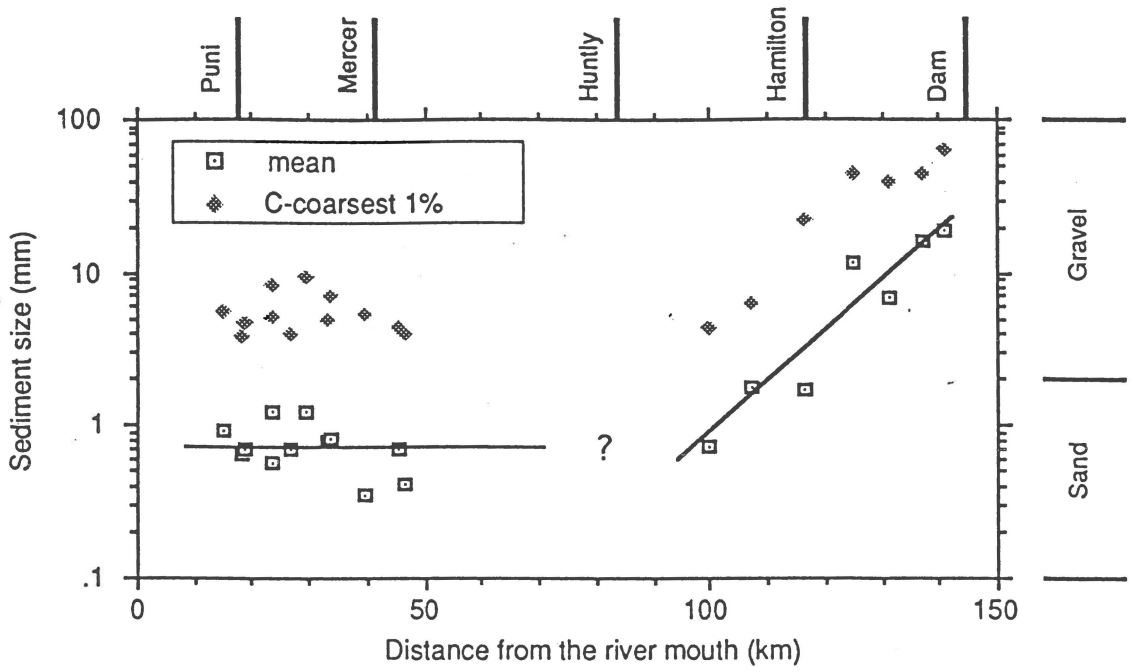


Figure 2.8 Variation of sediment size along the Waikato River (data from Fenton, 1989).

this goal has not been attained. The definition of common bedforms based on geometry is given by the ASCE (1966) Task Committee on bedforms. A comprehensive set of photographs and descriptions was produced by Pettijohn and Potter (1964), as well as Fairbridge and Bourgeois (1978), Leeder (1982), Allen (1985), and Collinson and Thompson (1989).

Field observations and echo sounding traces by Fenton (1989) show that a full variety of bedforms in sandy rivers from bars down to ripples exist in the Waikato River from Hamilton to Puni. The bedform hierarchies (Allen, 1968a) were clearly observed in the river bed configurations: ripples superimposed on the backs of dunes, and ripples and dunes on the stoss sides of larger dunes or sand bars. There were only isolated areas of small dunes in the reach between Hamilton and the Horotiu bridge. The reason is evidently related to the bed materials in this reach, which consist of gravels or a mixture of sand and gravel (see Chapter Three). Flume experiments indicate that there are few bedforms associated with sediment movement in gravel materials (Parker, et al., 1982).

Ripples with heights of 0.005–0.02 m and lengths of 0.1–0.5 m have been observed in the lower Waikato. Dunes at Churchill East in the lower Waikato River are usually up to 1.5 m high and can be of extremely regular shape with length of 76–92 m, corresponding to a R.I. (Ripple Index, which is defined as the ratio of wave length to wave height) of 51–61, but most of them

are rather irregular (Ridall, 1967). Dunes with heights of 0.22 m, lengths of 4.5 m and heights of 0.04–0.16 m, lengths of 0.5–2 m have also been observed by Fenton (1989) in most of the lower Waikato River.

The distribution of R.I. along the river from Hamilton to Puni determined from echo sounding traces (Fenton, 1989) shows that R.I. is generally below 100, and high R.I. occur only between Huntly and Mercer (Figure 2.9). Studies by Allen (1968b, 1970) and Costello (1974) indicated that for Ripple Index on average $R.I._{ripple} < R.I._{dune} \leq R.I._{bar}$. Dunes with higher R.I. have been termed "intermediate flattened dunes" (Pratt, 1973).

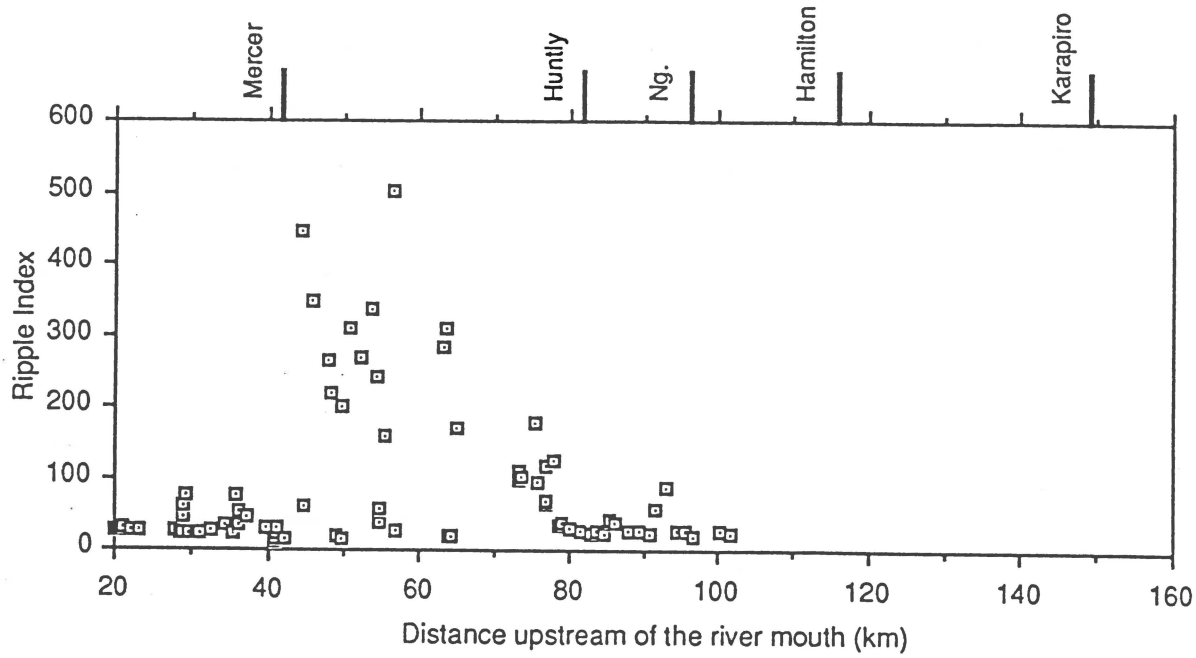


Figure 2.9 Distribution of Ripple Index along the lower Waikato River (data from Fenton, 1989). The figure shows that high Ripple Index representing big bedforms is dominated in sandy bed reaches in comparison with sediment distribution in Figure 2.8.

Detailed field surveys of the 1 000 m reach between cross sections 73–74 in the Waikato River below Orton (Figure 1.3) indicate that two forms of movable alternate bars exist. There was a layer of mud over the bed for some distance downstream of the sand bar crest, and only subtle variations in mean grain size and sorting occur along the face of the bar (Fenton, 1989). These phenomena are different from point bars. There is a trend of decreasing grain size in the downstream direction along the point bar surface (Levey, 1978; Struiksma, et al., 1985). With a flow increase, alternate bars appear to become rounded and flatten and eventually become undefinable. The reason may be that high flows apply greater transport power, and small and

undefinable bedforms migrate at a faster pace than bigger bedforms in the same system (Boothroyd and Hubbard, 1974; Jain and Kennedy, 1974).

Similar dynamics of big bedforms in the sandy Red Deer River of Alberta have been described by Neill (1969) using echo sounding techniques.

2.4 Concluding remarks

Sediments in the Waikato River are mainly gravelly pumiceous sands resulting from volcanic activity in the catchment area. The river channel pattern is more or less stable and no cutoffs have occurred since 1942 based on a comparison of aerial photos. Human activities, such as dam construction, river training works and commercial sand extraction have been undertaken on the river and have significantly changed the river regime. River bed and water levels have dropped over time. River flows downstream of Ngaruawahia are not only controlled by operations of the Karapiro Dam and the flood control constructions on its lower system, but also contributions from the tributaries downstream, especially the Waipa River. Both the Hamilton basin and the Waipa River catchment drain about 26% of the entire catchment area and their contribution to an extreme flow at Huntly may be important. Bedload sediment transport at Churchill East was measured at about 170 000 m^3/yr. Suspended load transport at Ngaruawahia is estimated at about 49 000 m^3/yr by using the data in 1990–1992. Bedforms due to sediment movement, such as current ripples, dunes, bars, in a sandy river as defined by ASCE (1966) have been observed in the lower reach of the Waikato River.

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Chapter Three

Sediment Characteristics in the Lower Waikato River

3.1 Introduction

Bed materials of the lower Waikato River were sampled in 1991, and in 1991 and 1993 for the Waipa River, in order to obtain their sediment characteristics. Sampling techniques, sample analysis methods and results are presented in this chapter. The purpose is to characterise the bed sediment distribution in these rivers as an aid to understanding sedimentation in the lower Waikato River. Results can be used as coefficients in an analysis of bedload sediment balance and bedload transport simulation.

3.2 Positions of bed material samples

During May and June, 1991, bed materials of the Waikato River between Mercer and Karapiro were sampled.

One to three bed samples (equally spread across the section on left, middle and right) were collected from each cross section sampled. No sample was able to be collected using the dredger near cross section 139 (105.2 km from the river mouth, above the Horotiu Bridge) because the bed materials consisted of consolidated silt and clay. Likewise at the left side near cross section 152 (117.3 km from the river mouth, around Hamilton) and the right side near cross section 173 (139.71 km from the river mouth). In the river reach between Cambridge and the Karapiro Dam, because the river bed had evidently been armoured and the bed materials had deeply consolidated, no sample was able to be obtained using the dredger. But some samples were taken by hand near the river bank just below the Karapiro Dam (approximately 500 m). The positions of the samples collected are shown in Figure 3.1.

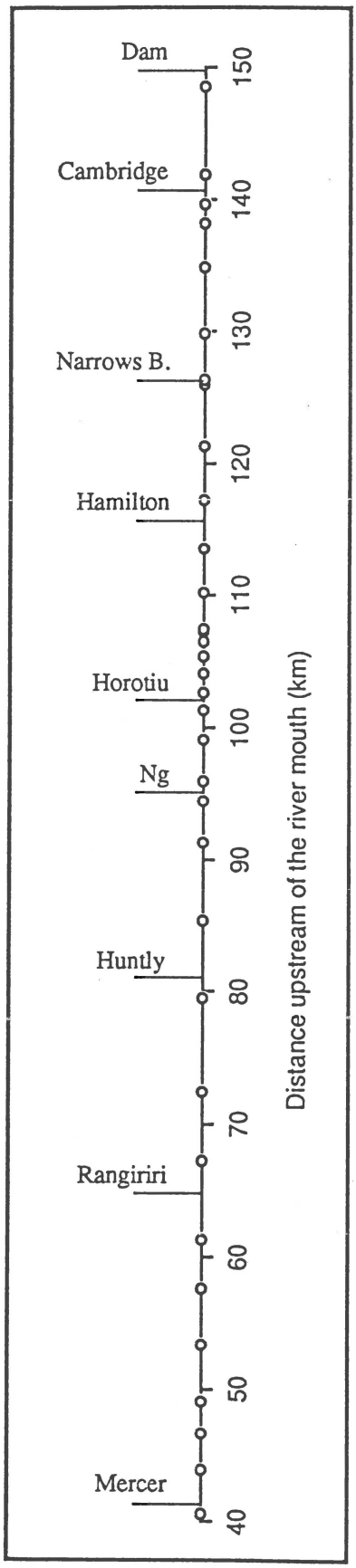


Figure 3.1 Positions of bed material samples in the lower Waikato River in 1991.

Table 3.1 Summary of bed material samples in 1991 and 1993.

XS	Dis. (km)	Left sample	Middle sample	Right sample	Note
57	40.56	√	√	√	
61A	43.98	√	√	√	
64	46.61	√	√	√	
68	48.99	√	√	√	
73	53.25	√	√	√	
90	57.63	√	√	√	
105	61.33	√	√	√	
111C	67.18	√	√	√	
115	72.34	√	√	√	
122	79.49	√	√	√	
126	85.46	√	√	√	
130	91.29	√	√	√	
132	94.45	√	√	√	
133	95.88	√	√	√	
135	99.15	√	√	√	
136	101.36	—	√	—	One sample
137	102.67	—	√	—	
138	104.03	√	√	√	
—	105.34	—	√	—	One sample
140	106.51	—	√	—	One sample
141	107.28	√	√	√	
—	107.38	—	√	—	One sample
144	110.11	√	√	√	
148	113.52	√	√	√	
152	117.26	×	√	√	No l. sample
156	121.38	√		√	M. sample lost
159A	125.95	√	√	√	
—	126.37	—	√	—	One sample
164	129.76	√	√	√	
168	134.80	√	√	√	
171	138.06	√	√	√	
173	139.71	√	√	×	No r. sample
174	141.77	√	√	√	
—	148.60	√	√	—	
—	148.60	√	√	—	Sub-surface.
—	WP1	—	√	—	Waipa River
—	WP2	√	—	—	Near bank
—	WP3	√	—	—	Near bank
—	WP4	—	—	×	No sample

One sample (WP1) was collected from the Waipa River bed at position of the middle channel about 100 m upstream of the Waipa River mouth in 1991. Two other samples (WP2 and WP3) were taken in July 1993 using a dredger from a river bank. They

were obtained near the bridges located at Whatawhata and Te Rore respectively, close to the left river banks. No sample was able to be collected from the bed at the right bank at Pirongia because of the fixed silt and clay on the bed from a diving observation (Figure 3.2).

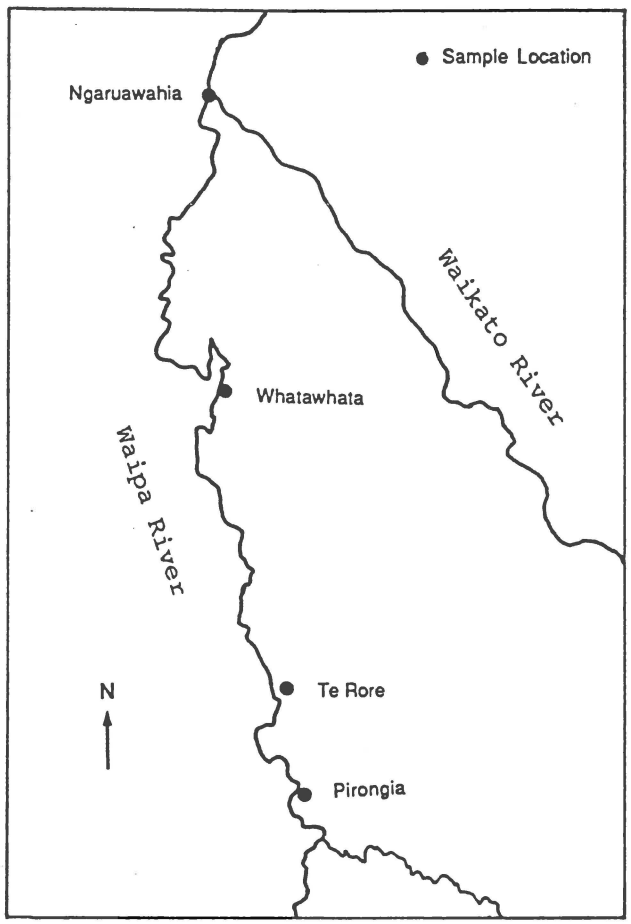


Figure 3.2 Location of bed material samples taken from the Waipa River. No sample was able to be collected by the bank at Pirongia because there was a fixed silt and clay bed.

Table 3.1 lists the samples collected. The original data are attached in Appendix Two. The total number of samples analysed was 91 including three samples taken from the Waipa River.

3.3 Sampling techniques and sample analysis methods

Except for two bed material samples taken by a dredger from river banks at Whatawhata and Te Rore in the Waipa River, all samples were collected with a dredger from a boat.

The dredger consisted of a cylinder (ϕ = 25 cm and H = 40 cm) with a cutting edge and chains (Figure 3.3). When sampling, the dredger was lowered to the river bed and dragged slowly in the downstream direction by means of a



Figure 3.3 Photo of the dredger used for bed materials sampling in the lower Waikato River in 1991. The dredger is consisted of a cylinder with a diameter of 25 cm and a height of 40 cm.

rope connected to the boat. For each sample, the cylinder was filled over two-thirds to full.

Since the RSA (Rapid Sediment Analysis) system at the University of Waikato cannot be used for gravel sediments, for consistency, all the samples were analysed by the dried sieving method (Vanoni, 1975; Jansen, et al., 1979) at intervals of 0.5–1.0 phi in the sedimentation laboratory of the Department of Earth Sciences.

Table 3.2 Sample size for different materials.

Materials	*Size (g)*
Sand	40–100
Sand and gravel	75–200
Gravel	>200

First, the sample was washed and dried for more than 24 hours, and split using a separator. Because there is a relationship between the sample size and the error in the grain size distribution (Jansen, et. al., 1979, page 220), the sample sizes

in Table 3.2 were followed in this study.

The hand sieving method was used for portions of samples with sieve diameters smaller than –1.5 phi (or bigger than 2.8 mm), and the remainder was vibrated in a Fritsch sieve shaker machine for about 6 minutes due to the presence of pumice, which is soft and easy to break down.

Several methods can be used to present the sediment analyses. The textural parameters of the sediment samples taken from the lower Waikato River are here presented using the Folk graphical method (Folk, 1968) so that they can be compared with historical sediment data.

Specific gravity of 15 samples was determined using the water displacement method (Appendix Three). The samples were collected from the middle channel except for one from the subsurface left bank just below the Karapiro Dam. Given the bulk density of sand, the results can be used to obtain a porosity.

3.4 Sediment characteristics

3.4.1 Grain size distribution

Normally, bed materials reduce in size down a river as a result of both particle abrasion and selective transportation of finer particles. Sternberg (1875) first addressed the problem of abrasion in a mathematical form, implying that the abrasion of a pebble is proportional to its weight P in water and the length of the path x covered by it. Sternberg's law in a modern version reads:

$$dP = aP\,dx \tag{3.1}$$

or

$$P = P_0 e^{-ax} \tag{3.2}$$

where a is a constant and P_0 is the weight at $x = 0$. P and P_0 can be proportional to the third power of the corresponding sediment diameters d and d_0 if the relative shape of the sediments remains similar during abrasion, i.e.

$$\frac{P}{P_0} = \left(\frac{d}{d_0}\right)^3 \tag{3.3}$$

A transformation of Eq. (3.2) using Eq. (3.3) is

$$d = d_0 e^{-ax/3} \tag{3.4}$$

This equation produces a linear relationship with semi-log coordinate. As Sternberg pointed out, when using these equations, the smaller grains have to be disregarded.

Sternberg's law has been criticised because reduction in sediment size downstream is not only subject to the abrasion but also to the flow sorting. Gölz and Tippner (1985) found that the observed distribution of the size reduction of pebbles along the downstream direction in the Upper Rhine was ascribed to a lesser degree to abrasion, but more to the sorting during sediment transport.

The following discussion relates to samples obtained in 1991 for this study. The average mean and median grain sizes of bed materials from 3 samples collected across the section are plotted in Figures 3.4-A–B. One sample collected at cross section 171 is omitted because of the abnormal cross section. It is clear that there is a general decrease of the average mean and median grain sizes in the downstream direction, but a hiatus occurs in the trend of grain size reduction. The same pattern of sediment distribution exists for the samples collected from the middle channel (Figures 3.5-A–B). There is no statistically significant relationship between sediment size and distance for the sediments with their mean or median sizes smaller than 2 mm in the reach downstream of Horotiu, especially for the sediments from the middle channel (Figures 3.4-A–B, 3.5-A–B).

The hiatuses of sediment distributions around Horotiu imply that Sternberg's law cannot readily be applied for the whole reach of the river downstream Karapiro. For the upper part of the reach between Horotiu and Karapiro, the reduction in sediment size downstream can be attributed to the dam effects—sorting or armouring—and the effects of abrasion. Because the Karapiro Dam have stopped bed load supplies from the upper stream and bed load supplies from the subcatchment between Horotiu and Karapiro are expected to be small, sorting during sediment transport or selective erosion (Livesey, 1963; Schumm, 1977; Sutherland, 1987) could be a major factor for the distributions of bed material sediment size in that reach. Even in a natural upper river, sorting can be the major factor in the distributions of bed material textural compositions (Gölz and Tippner, 1985).

It should be pointed out that the same kind of hiatus for the grain size reduction in the downstream direction has also been reported by Yatsu (1955)

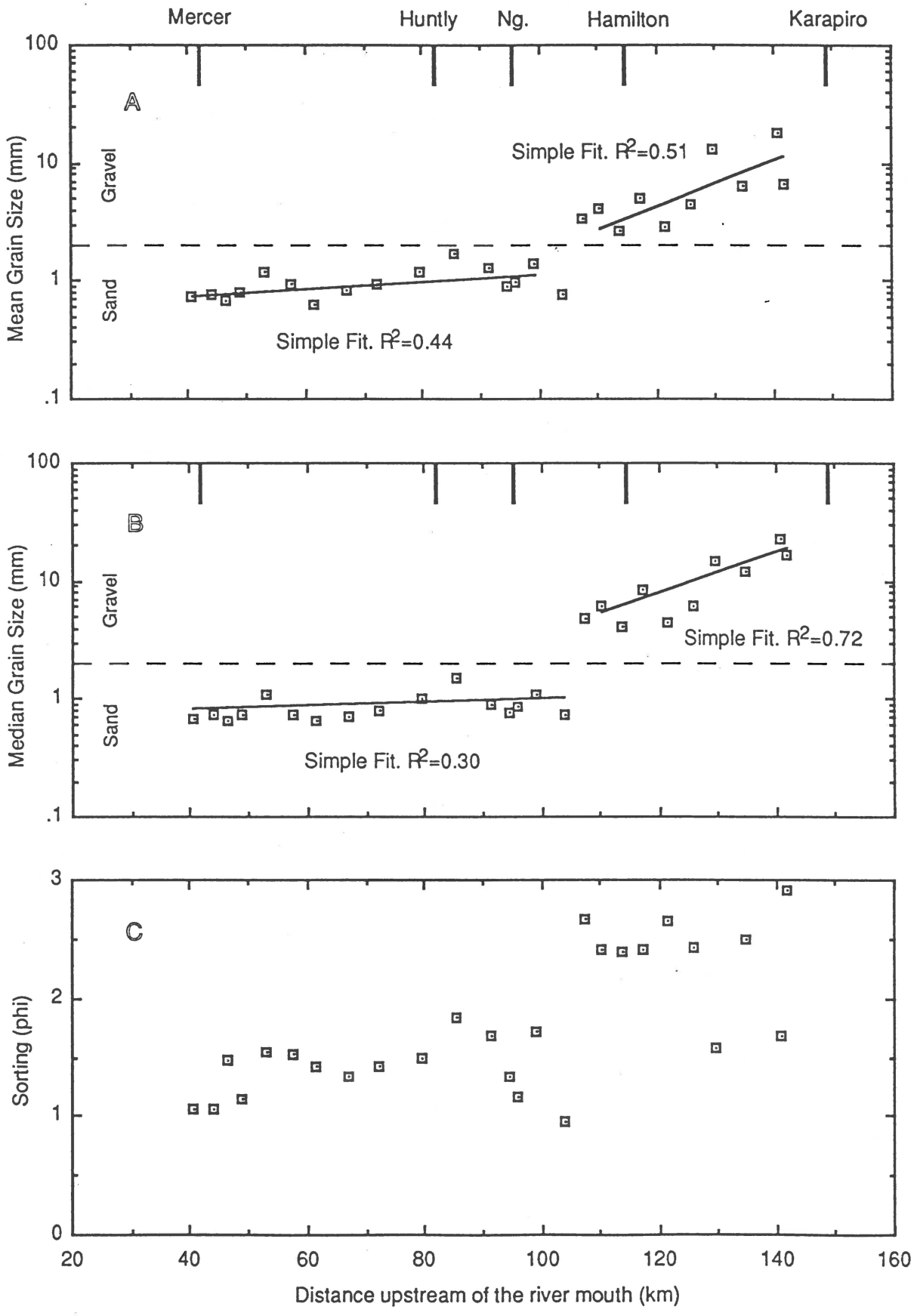


Figure 3.4 Parameters of bed materials along the Waikato River from an average of 3 samples at each cross section with a total of 26 cross sections sampled in 1991. Fit lines are determined for the distances of 40-100 km and 110-142 km respectively.

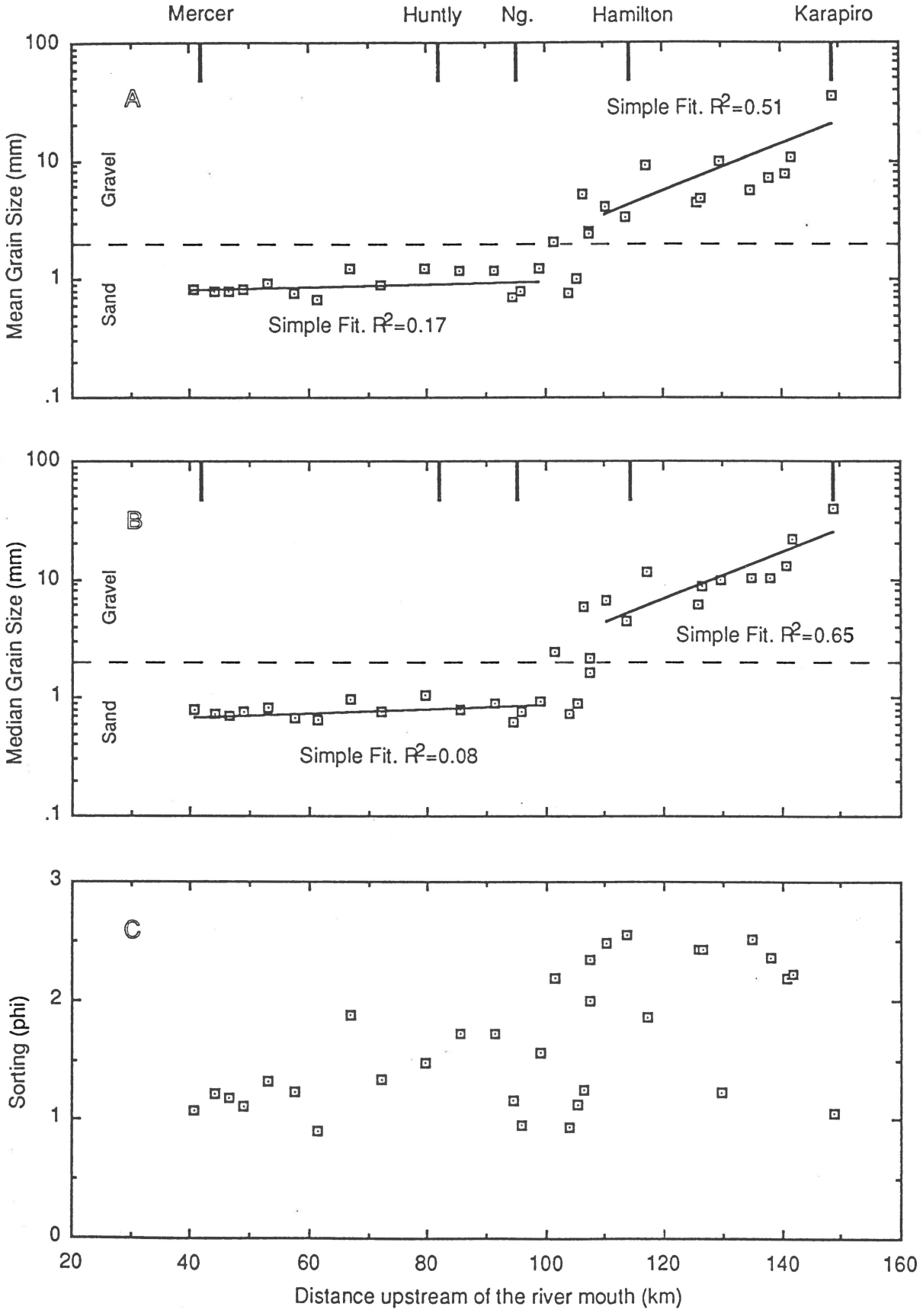


Figure 3.5 Parameters of bed materials along the Waikato River from one middle channel sample at each cross section with a total of 32 cross sections sampled in 1991. Fit lines are determined for the distances of 40-100 km and 110-149 km respectively.

and Monsalve and Silva (1983) in natural (un-dammed) rivers. Because there are no sediment size distribution data available prior to the Karapiro and Arapuni dam closures for comparison, the reduction in sediment sizes downstream between Karapiro and Horotiu shown in Figures 3.4–5 may not be caused solely by dam effects.

Sorting of the sediment samples along the river is shown in Figures 3.4-C and 3.5-C, in which the fluctuation can be explained as random effects, including the errors in representation of the samples, techniques and analysis methods used. The sorting here is defined by the following equation, which is very close to the standard deviation:

$$\sigma = \frac{\phi_{84} - \phi_{16}}{2} \tag{3.5}$$

where σ is the sorting, and ϕ_{84} and ϕ_{16} are, respectively, the sediment sizes with their cumulative percentages of 84% and 16% (Folk, 1968). In the case of phi unit used, the smaller the value of σ, the better the sorting.

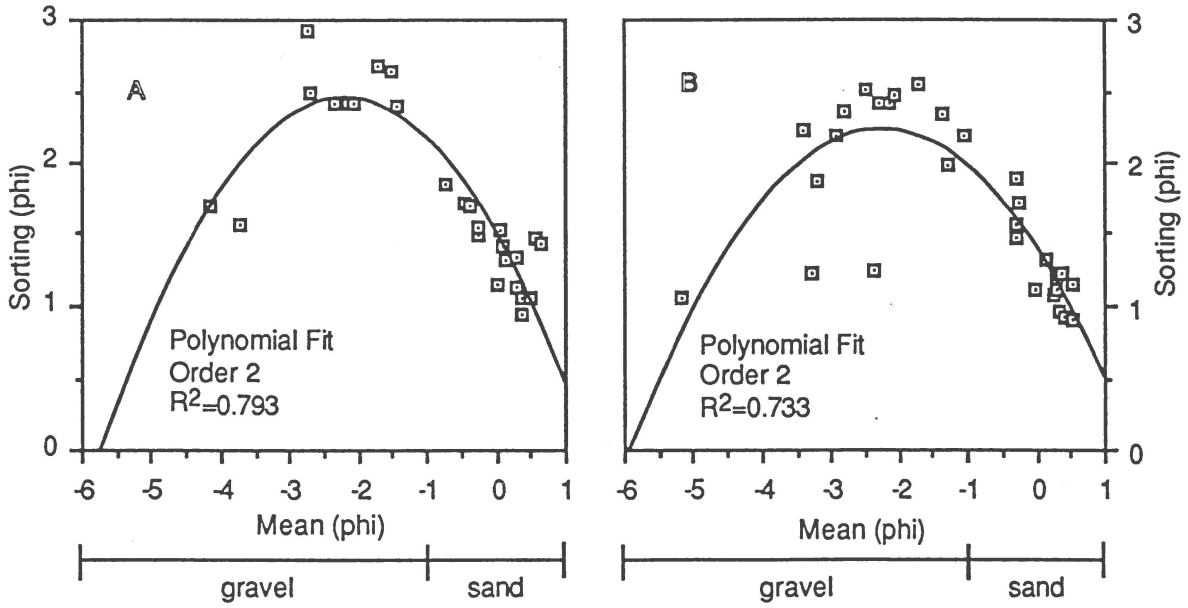


Figure 3.6 Relationship between mean grain size and sorting from the bed materials in the lower Waikato River in 1991. A is the relationship between mean grain size and sorting from the samples with 3-samples-averaged at each section sampled and a total of 26 samples. B is the relationship between mean grain size and sorting from the samples with one sample from the middle channel at each section sampled and a total of 32 samples.

Figures 3.6-A–B demonstrate a relationship between mean grain size and sorting existing in natural environments (Folk, 1968).

The textures of bed sediment samples taken between 40–100 km are mainly

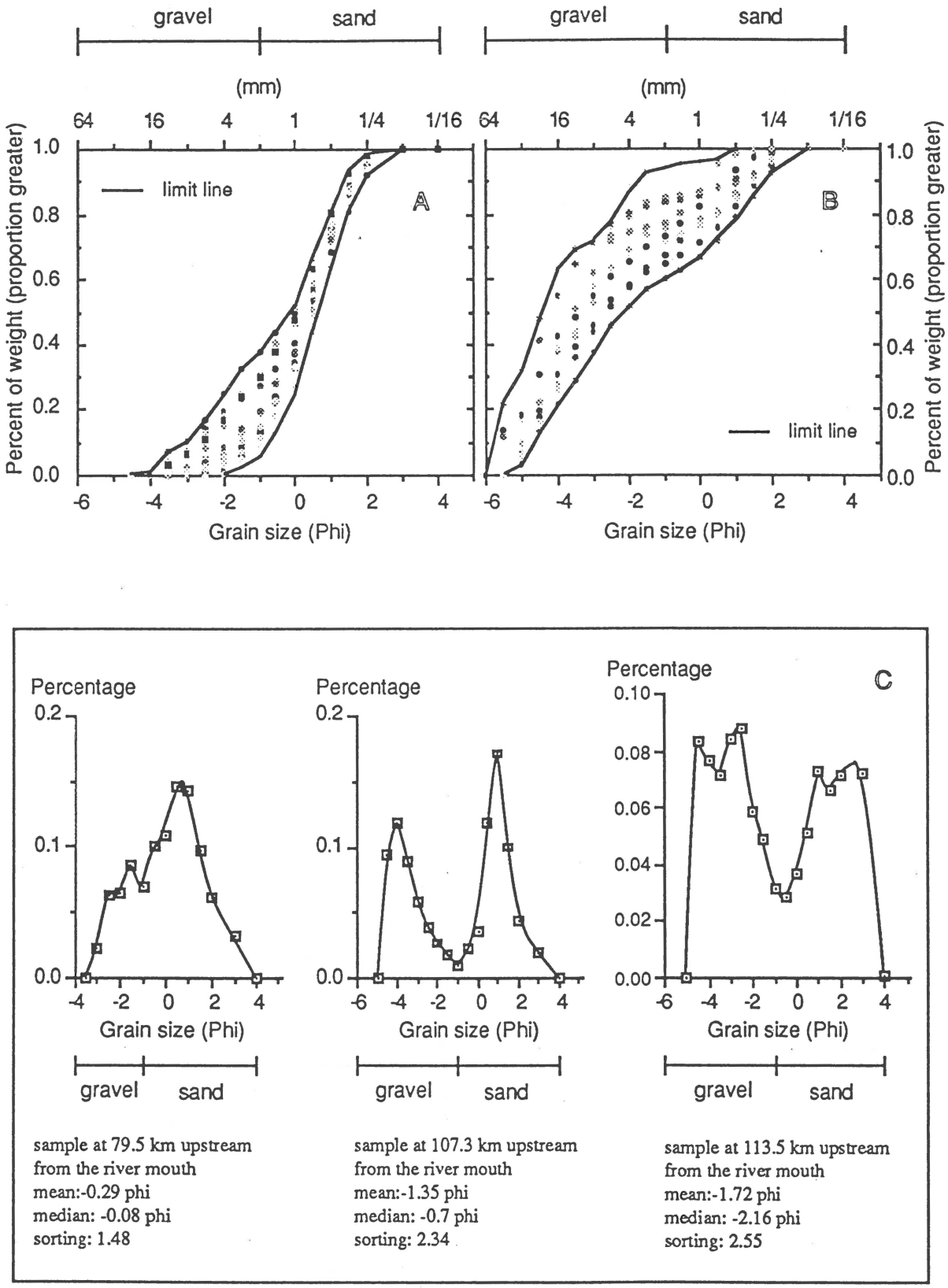


Figure 3.7. Grain size distributions of bed materials in the lower Waikato River in 1991. **A** is the grain size distribution of all the samples collected at the middle channel between 40–100 km upstream of the river mouth and shows that sand is a major part of bed materials (15 samples). **B** is the grain size distribution of all the samples collected at the middle channel between 110–142 km upstream of the river mouth and shows that gravel is a major part of bed materials (10 samples). **C** shows modes of grain size distributions from three samples.

composed of sand (Figure 3.7-A) and have a unimodal distribution or a close to unimodal distribution (Figure 3.7-C), while upstream of this they are mainly composed of gravel (Figure 3.7-B) and have some bimodal distributions (Figure 3.7-C), which normally occur in a gravel river (Church et al., 1987). Figures 3.4-A and 3.5-A also show this kind of sand and gravel distribution along the river in terms of the mean and median grain sizes.

An average of the mean grain size and the median grain size of the bed material samples taken between 40–100 km in the middle channel is 0.94 ±0.21 mm (mean ± standard deviation) and 0.80 ±0.12 mm (median ± standard deviation) respectively.

For the average of the three bed material samples taken at each section sampled, an average of the mean and the median grain sizes within the reach of 40–100 km is, respectively, 1.00 ± 0.30 mm (mean ± standard deviation) and 0.86 ±0.23 mm (median ± standard deviation).

The bed sediments collected from the river bed downstream of Ngaruawahia in the 1960s were sand sized, and their mean diameter was reported at 0.7–1.0 mm (Ridall, 1967). The average mean and median grain sizes of the 70 bed material samples taken from 24 cross sections between Huntly and Mercer in 1966 were estimated at 1.08 mm and 0.78 mm (Waikato Valley Authority, 1971b). In 1971, the average mean and median grain sizes of the 8 bed material samples taken from the river in the reach between cross sections 87 and 93 (56.8 km and 58.5 km upstream of the river mouth respectively) were 0.88 mm and 0.70 mm respectively (Waikato Valley Authority, 1971b). Fenton (1989) produced a value of the median grain size of 0.78 mm for the surface sediments near Mercer.

Historical sediment data upstream of Ngaruawahia can only be accessed from an interim report of Ministry of Works (1962) which was compiled from the river sediment (bedload and bed material) data taken around the Narrows Bridge and Cambridge in 1961. The median grain size of bed materials at Cambridge was about 25 mm—the same order of the data collected in 1991 (Figure 3.8). The surface of the river bed was partially armoured from a diving observation at that time (Ministry of Works, 1962).

The median grain sizes of all the historical data are plotted with those collected from the middle channel in 1991 (Figure 3.8). No obvious changes to the median grain sizes of the bed materials in the lower Waikato River during the period 1961–1991 can be detected.

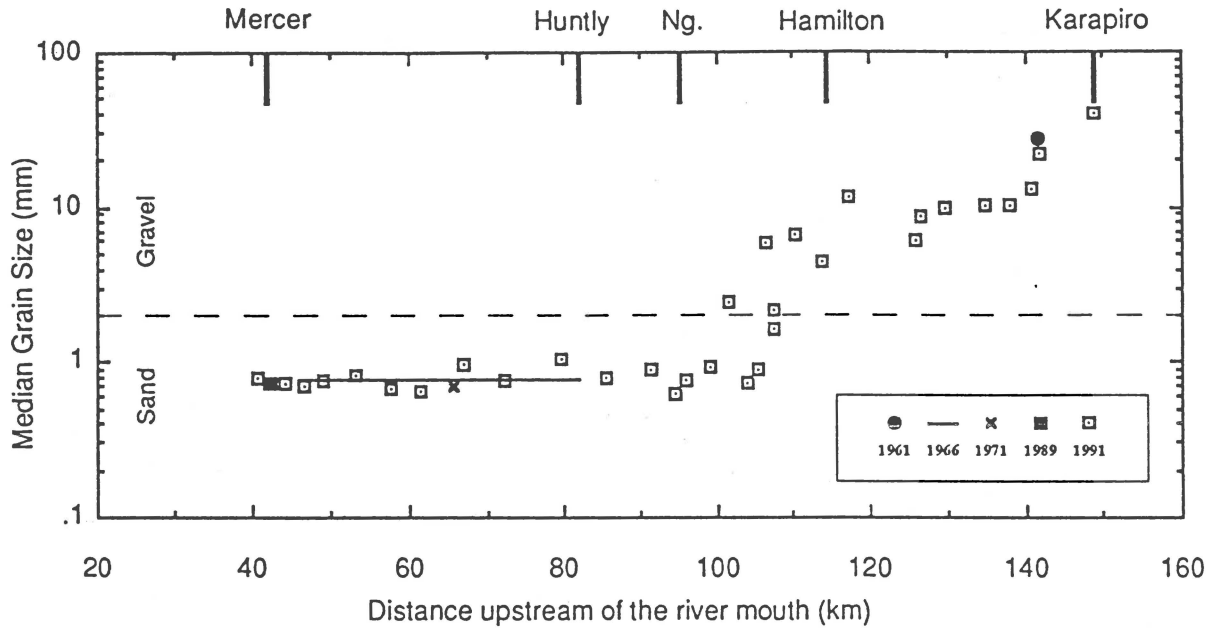


Figure 3.8 A comparison of the median grain sizes of bed materials in 1961–1991 (data from Ministry of Works, 1962; Waikato Valley Authority, 1971b; Fenton, 1989).

In the reach around Hamilton, although there was an amount of sand deposited near banks and point bars, the bed materials in the main channel (thalweg) were gravel, and gravel-sand mixture with diameters up to 50–60 mm in the 1960s (M. Tetslaff of W Stevenson & Co., pers. comm., 1991; J. Dahm, Waikato Regional Council, pers. comm., 1991). It can be expected that the river in the Hamilton reach and above was armoured or partially armoured by the 1960s due to operations of the Arapuni Dam since 1929, and the Karapiro Dam since 1947.

Broadly, in terms of the mean and median grain sizes, it is evident that sand dominates the river downstream of Ngaruawahia while gravel or sand-gravel mixture characterises the upper reach to Karapiro. That is one of the reasons why big bedforms (bars, dunes) are mainly observed in the lower parts of the river—downstream of Ngaruawahia (Figure 2.9) (Ridall, 1967; Fenton, 1989). Because of the difference between the gravel- and sand-bed rivers (Simons and Simons, 1987), few bedforms are identifiable with sediment movement in gravel bed rivers (Parker, et al., 1982).

Distribution of the bed material sample taken from the Waipa River is plotted in Figure 3.9. For the sample WP1 taken from the middle channel of the Waipa River at Ngaruawahia, there are no big differences from the sediment distribution of the lower parts of the Waikato River except for a proportion of clay components, which can be explained by the soft tertiary rocks

distributed in the Waipa River catchment as well as the backwater effects near the river mouth resulting in silt and clay deposition. For the other two samples WP2 and WP3, their distributions show that on average the grain sizes are smaller than those of the lower Waikato River bed materials although they are still sand. The probable reason for this may be that they were collected too close to the river banks. "*...the Waipa bed material has in all respects the characteristics of Lower Waikato sediment and could be a significant contributor of bedload to the Lower Waikato River.*" (Waikato Valley Authority, 1971a)

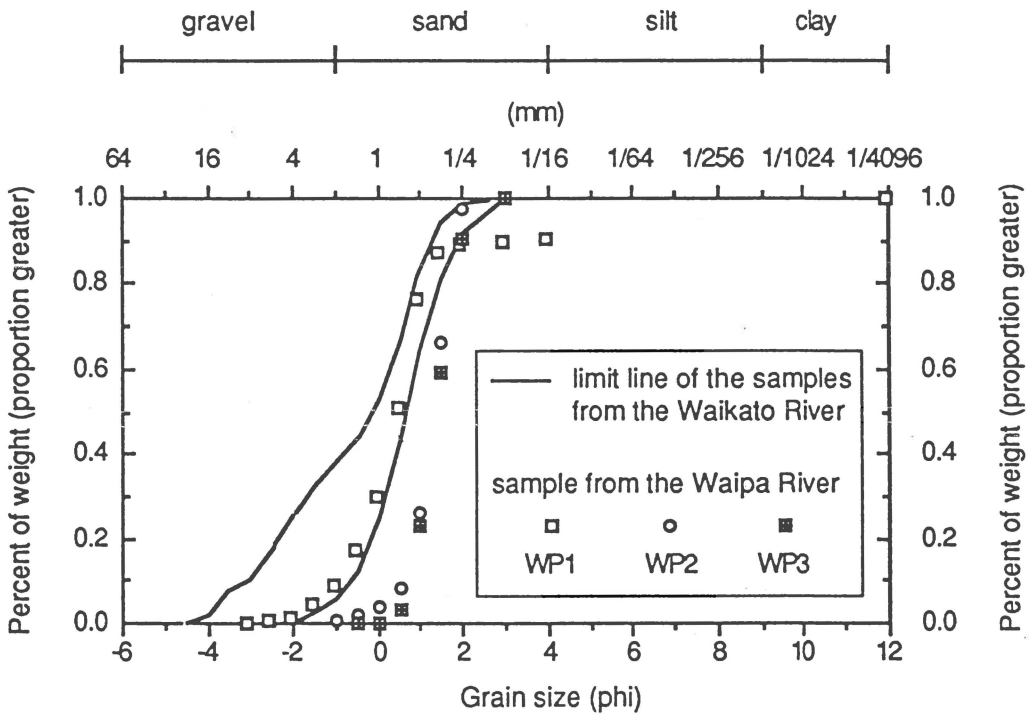


Figure 3.9 Comparison of particle size distributions for the samples from the Waipa River and the range of curves for the samples from the Waikato River downstream of Ngaruawahia. WP1 was collected from the middle channel about 100 m upstream of the river mouth and the pattern of its sediment distribution is very similar to those from the Waikato River of 40–100 km except that it contains more silt and clay. WP2 and WP3 were taken close to river banks at Whatawhata and Te Rore respectively and their distributions show that the grain sizes on average are smaller.

The evidence of a similar bed material distribution in the Waipa River and the lower parts of the Waikato River may indicate that the uncontrolled Waipa River could be one significant source of bed load supplies to the lower Waikato River.

The samples collected at 500 m downstream of the Karapiro Dam, close to the middle channel and near the bank respectively, show that sediments of the surface bed are coarser than those of the subsurface (Figure 3.10). For a river

with gravel or sand-gravel mixture bed materials, because of sorting during sediment transport, surface sediments become coarser, which hinders further river degradation, and as the surface layer becomes armoured, the bed tends to become stable if no disturbance or destruction of the armoured surface occurs. Figure 3.10 shows the armouring effects in the channel downstream of the Karapiro Dam.

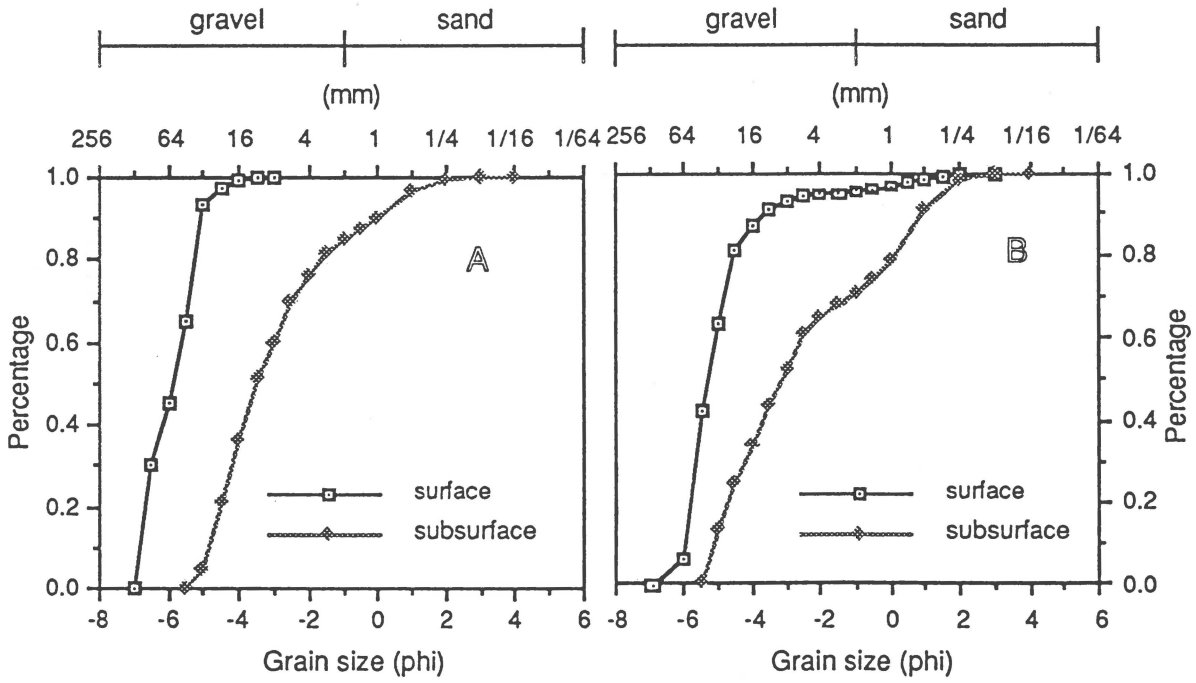


Figure 3.10 River bed armouring below the Karapiro Dam (data collected in 1991). A illustrates the samples taken close to the main channel. B shows the samples taken close to the river bank about 500 m below the Karapiro Dam. Both figures show that sediments of the surface bed are coarser than those of the subsurface.

3.4.2 Specific gravity and bulk density

As indicated in Chapter Two, the river bed and banks are primarily composed of volcanic debris such as sand, pumice and gravelly alluvium. Tilly (1987) investigated the physical properties of the Taupo pumice alluvium and a mean density of solid pumice 2 531.7 kg/m^3 was obtained. For the lower Waikato River, 24 bed material samples taken between Huntly and Mercer in 1966 gave a mean density of 2 250 kg/m^3 while 8 bed material samples taken between cross sections 87 and 93 (56.8 km and 58.5 km upstream of the river mouth respectively) gave a mean density 2 380 kg/m^3 (Waikato Valley Authority, 1971b). Fenton (1989) gave a mean density of 2 454 kg/m^3 from a number of bedload samples taken at cross section 73 near Orton. 3 bed material samples taken at Huntly in 1992 gave a mean specific gravity of 2.41 (Smith, et al., 1992).

Specific gravity of the 15 samples collected from the middle channel of the river bed was determined using the water displacement method. The specific gravity of these samples along the river is shown on Figure 3.11 and the original data are attached in Appendix Three.

The downstream trend of reduction of specific gravity can be seen from the figure. This may be due to the composition of the surface bed materials in the lower parts of the river being composed of more pumice, which is light and movable, than that in the upper parts of the river. The specific gravity of the sample collected at 40.6 km, which is mainly composed of pumice, is much smaller than that of the remaining samples (Figure 3.11).

The mean specific gravity of the total samples is 2.420 and if excluding the sample at 40.6 km, the mean of the remaining samples is 2.460±0.130 (mean±standard deviation) or a density of about 2 460±130 kg/m^3. This is very close to Fenton's result of 2 454 kg/m^3 and can be taken as a constant for the lower Waikato River sedimentation analysis.

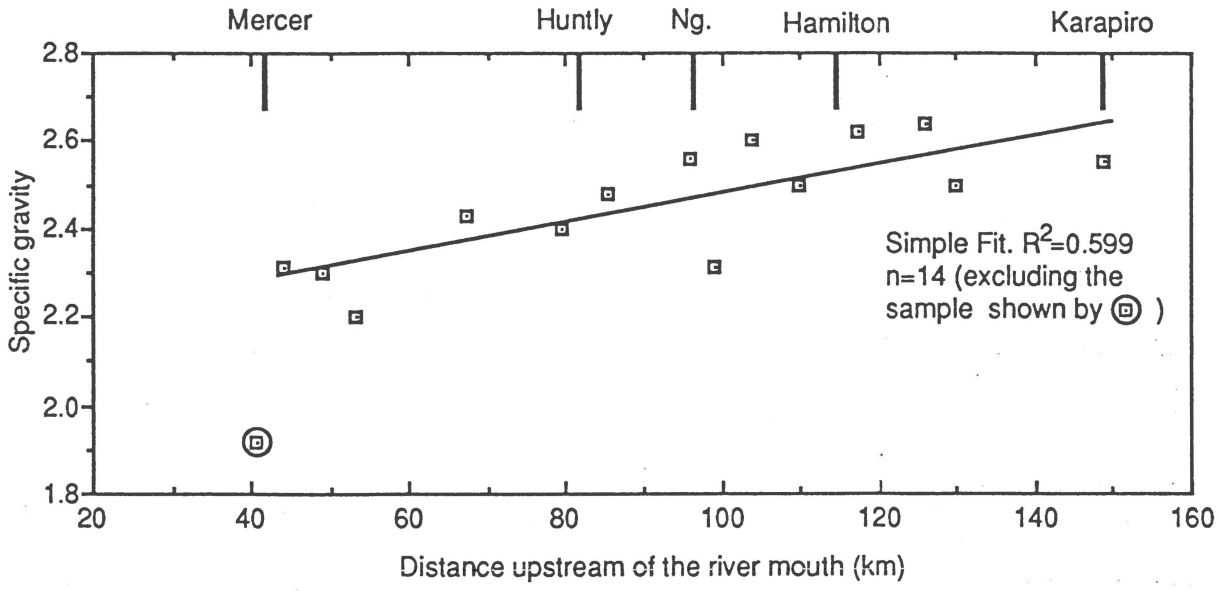


Figure 3.11 Distribution of the specific gravity of 15 samples along the Waikato River. All the samples were taken from the middle channel except for one from subsurface of the left river bank at 148.8 km upstream of the river mouth.

The bulk density of the moving bedload and the bulk density of the on-land dry sand were estimated at 962 kg/m^3 (60 bl/ft^3) and 1 122 kg/m^3 (70 bl/ft^3) respectively (Waikato Valley Authority, 1971c).

If a solid sand density of 2 460 kg/m^3 is taken, the porosity of the wet sand in the river and the loose dry sand on land is, respectively, 0.61 and 0.54. Both

porosity figures seem to be a little greater than 0.47 and 0.41, which generally are, respectively, those for wet and dry sand near lake banks (Graton and Fraser, 1935). Porosity of 0.36 was estimated by Smith et al. (1992) from 3 samples taken from the river bed at Huntly.

The difference between the moving bedload bulk density and the on-land dry sand bulk density means that the volumes of sand reported by companies as on-land dry sand are a little smaller than the same amount of sand volumes in the river. When the on-land dry sand volumes are used to analyse the Waikato River volume changes, a factor of 1.166 should be applied.

3.5 Conclusions

Bed materials in the lower Waikato River have been sampled and their textures and specific gravities have been analysed. The major findings are:

1. Generally, in terms of the mean and median grain sizes, sand dominates the Waikato River bed downstream of the section at about 100 km from the river mouth (above Ngaruawahia), while gravel dominates the bed upstream as far as the Karapiro Dam.

2. The bed materials decrease in size in the downstream direction. Although a power function—Eq. (3.4)—may be suitable for the gravel reach, the hiatus in the trends (Figures 3.4 and 3.5) indicates that it cannot be applied to the entire lower Waikato River. There is no significant relationship between sediment size and distance in the lower sandy river reach—downstream of Ngaruawahia.

3. Because some natural (un-dammed) rivers have a similar sediment distribution pattern (*cf.* Yatsu, 1955; Monsalve and Silva, 1983) to that in the lower Waikato River and no bed material data before construction of the Karapiro Dam are available, sediment distribution in the lower Waikato River shown in Figures 3.4 and 3.5 cannot be concluded as the dam effects only.

4. The texture of bed materials in the Waipa River is generally similar, but probably finer, to that from the lower parts of the Waikato River. This evidence may indicate that bed load supplies from the Waipa River into the lower Waikato River can be significant.

5. It can be expected that the river bed in the Hamilton reach and above was partially armoured by the 1960s.

6. The specific gravity of 2.460±0.130 (mean ± standard deviation) can be used for an analysis of the lower Waikato River sedimentation. Porosity of 0.61 and 0.54 for wet and dry sand respectively was determined based on the reported bulk densities (Waikato Valley Authority, 1971c).

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CHAPTER FOUR

ANALYSIS OF CROSS SECTIONAL CHANGES AND WATER AND BED LEVEL VARIATIONS IN THE LOWER WAIKATO RIVER

Chapter Four

Analysis of Cross Sectional Changes and Water and Bed Level Variations in the lower Waikato River

4.1 Introduction

River cross sectional surveys, gauging records, rating curves and water level profile measurements are analysed to ascertain historical changes in: cross sectional area, mean bed level, minimum bed level, water level at the gauging stations, and the water level profile. All records analysed were provided by the Waikato Regional Council. All water level data were adjusted to a standard discharge of 350 m^3/s. Possible factors influencing the historical changes have been evaluated.

4.2 Analysis of river cross sectional survey data

4.2.1 River cross sectional survey data

All of the Waikato River cross sectional survey data are held at the Waikato Regional Council. Definitions used in the surveys such as the normal water level, the mean bed level, etc. are attached in Appendix One.

Some surveys in the Waikato River downstream of Rangiriri were undertaken as early as 1913. Prior to 1958, the Waikato River surveys were conducted by the Public Works Department, and some of them were carried out for Power Design Survey. Subsequently they have been undertaken by the relevant catchment authority—presently the Waikato Regional Council (formerly the Waikato Catchment Board, formerly the Waikato Valley Authority). In the river reach between Ngaruawahia and Karapiro, no surveys were undertaken before construction of the Arapuni Dam (1929), and only one cross sectional survey record was available before the Karapiro Dam closure (1947)—cross section 148 at the Fairfield Bridge, Hamilton in 1933.

Fenton (1989) listed the cross sectional survey data from 1913 to 1987. It

should be mentioned that each survey might last 2–5 years and the sequencing of cross sections measured in each survey were different. The survey of 1963/1964 was the first set of complete cross sectional measurements in the lower Waikato River, with a few cross sections surveyed upstream of Ngaruawahia. A series of regular surveys were conducted at an interval of 5–6 years afterwards (Sledger, 1987, 1989). Over 174 cross sections have been measured since 1913, but each survey did not cover all of them.

For the reach between Puni and Ngaruawahia (cross sections 16–132), the 1963/1964, 1979/1983 and 1985/1989 survey data will be used in this study to analyse mean bed level variations and cross sectional changes. For upstream of Ngaruawahia, surveys in 1974/1976 and 1985/1989 will be used for the same analysis. The original data can be found from the survey files at the Waikato Regional Council or in Fenton (1989).

4.2.2 Errors in river cross sectional survey and analysis

Hand survey techniques (sounding rod or sounding line) were utilised for all the cross sectional measurements before 1963, and after that time both echo sounding survey and hand survey techniques were used.

At commencement of this study there was no information available about the systematic and random errors of the Waikato River survey data. Echo sounding charts were read with a ruler, or using a digitiser, to determine river bed levels and water surface elevations. For an echo sounding technique, Jansen *et al.* (1979) determined an overall error of 132 mm at a 95% confidence level.

For the Waikato River echo sounding survey data, the scale of depth used in echo sounding charts varied in different cross sections and different surveys, typically from 1 mm:79.68 mm (5-1-1982 at cross section 136) to 1 mm:160 mm (28-11-1963 at cross section 127). The standard deviation of the random error in determining positions of the water and bed levels from echo sounding charts using a ruler or a digitiser is, respectively, 0.25 mm and 0.50 mm (R. Lamb, surveyor of the Waikato Regional Council, 1992, pers. comm.). Therefore the total standard deviation of these two terms is 0.56 mm using the root-sum-square method (ISO, 1976). For a 95% confidence level, twice the standard deviation or 1.12 mm has to be taken. For the systematic error, we assume an error of ±10 mm (R. Lamb, surveyor of the Waikato Regional Council, 1992, pers. comm.); for determination of water levels in the field ±20 mm was suggested by Jansen, *et al.* (1979), so that the overall error at a

95% confidence level of the bed level measurements from echo sounding charts is ±179 mm for scale 1 mm:160 mm and ±90 mm for scale 1 mm:80 mm.

Therefore based on the report by Jansen, *et al.* (1979) and the analysis above, a total error of about ±150 mm, at a 95% confidence level, can be assumed for the Waikato River bed level measurements using echo sounding techniques.

The total error of bed level measurements from hand survey techniques mainly depends on the equipment used and river flow conditions. For sounding line techniques, corrections have to be made to the bed level measurements to take account of deviations of the line from the vertical (ISO, 1968). It is not known whether such corrections were made to the Waikato River survey data. Hence it is very difficult to determine quantitatively the overall error of the survey data. Therefore the same total error of ±150 mm, at a 95% confidence level, is also assumed for the hand survey techniques.

For cross sectional measurements with hand survey methods, the number of points taken also affects their accuracy.

The cross sectional area below a normal water level (A) is obtained from survey data by

$$A = \sum_{i=1}^{n} A_i = \sum_{i=1}^{n} W_i H_i \tag{4.1}$$

where A_i represents one piece of the whole cross section ($i = 1,\cdots,n$), W_i is its width and H_i is its depth from the normal water level to the bed level. From Eq. (4.1), the differential area (dA) is

$$dA = \sum_{i=1}^{n} dA_i \tag{4.2}$$

If the error for each small section is $\delta A_i = \varepsilon_i$ and all of them are independent of each other, the total error ε for the whole cross section A (δA) can be obtained by the following equation using the root-sum-square method (ISO, 1976)

$$\varepsilon = \delta A = \sqrt{\sum_{i=1}^{n} \delta A_i^2} = \sqrt{\sum_{i=1}^{n} \varepsilon_i^2} \tag{4.3}$$

The total error ε includes two parts, $\varepsilon = f(\varepsilon', \varepsilon'')$. ε' is the representative error from the original equation Eq. (4.1) using a sum of the total small piece sections to determine the whole cross section. ε' is a function of n, when

$n \to \infty$, $\varepsilon' \to 0$. ε'' is the error from all the W_i and H_i measurements which are used to calculate each segment of sectional area. Generally ε'' is proportional to $1/\varepsilon'$. ε'' can be estimated based on Eq. (4.1) as long as we know each independent error for W_i and H_i (δW_i, δH_i) by using:

$$\varepsilon'' = \sqrt{\sum_{i=1}^{n} (H_i \delta W_i)^2 + \sum_{i=1}^{n} (W_i \delta H_i)^2} \tag{4.4}$$

It is very difficult to determine the total error ε for each cross section surveyed in the lower Waikato River because we do not exactly know both ε' and ε''.

Accordingly we assume 5% of the cross sectional area as the total error at a 95% confidence level.

The average cross sectional area below the normal water level was estimated at about 550 m^2 using data from the gauging stations located at Ngaruawahia, Huntly, Rangiriri and Mercer. For the river reach from Ngaruawahia to Puni, if this figure is considered as a representative, then the total error at a 95% confidence level for cross sectional measurements in that reach is about ±28 m^2.

For the river upstream of Ngaruawahia, the cross section at the Hamilton Traffic Bridge is selected as representative. The area below the normal water level at this site is about 270 m^2, so that a total error at a 95% confidence level for cross sectional measurements in the reach is about ±14 m^2.

When comparing cross sectional areas over time Δt, net change (ΔA) is determined by

$$\Delta A = A_{t+\Delta t} - A_t \tag{4.5}$$

The error for the net area change ($\delta \Delta A$) is a function of measurement errors of $A_{t+\Delta t}$ and A_t ($\delta A_{t+\Delta t}$ and δA_t), and a random error ε''' from movements of large bedforms—dunes or bars (ref. Figure 2.6; Ridall, 1967; Fenton, 1989), $\delta \Delta A = f(\delta A_{t+\Delta t}, \delta A_t, \varepsilon'''(t))$. In a gravel river, because of absence of big bedforms ε''' can be omitted.

The random error ε''' will be diminished if we take an average of several measurements over time. For homogeneity, an average over space can be used instead of over time.

If both errors for cross sectional areas—$A_{t+\Delta t}$ and A_t—are δA and they are independent of each other, then an error for the net area increase ($\delta \Delta A$)

without ε''' is

$$\delta\Delta A = \sqrt{\delta A_{t+\Delta t}{}^2 + \delta A_t{}^2} = \sqrt{2}\delta A \tag{4.6}$$

The net area change for an average of n cross sectional areas is calculated by

$$\Delta A = \frac{1}{n}\left(\sum_{j=1}^{n} A_{j,t+\Delta t} - \sum_{j=1}^{n} A_{j,t}\right) \tag{4.7}$$

with the same error equation—Eq. (4.6).

4.2.3 Analysis methods of river cross sectional survey data

In order to investigate the river cross sectional changes and minimise effects of random bed level variations, Fenton (1989) studied river cross sectional changes using river volume changes between two different surveys of five immediate cross sectional mean data, and Mulholland (1982) and Sledger (1987) divided the river channel into several short reaches and compared the mean cross sectional areas below the normal water level over that reach between two different surveys (Figure 4.1).

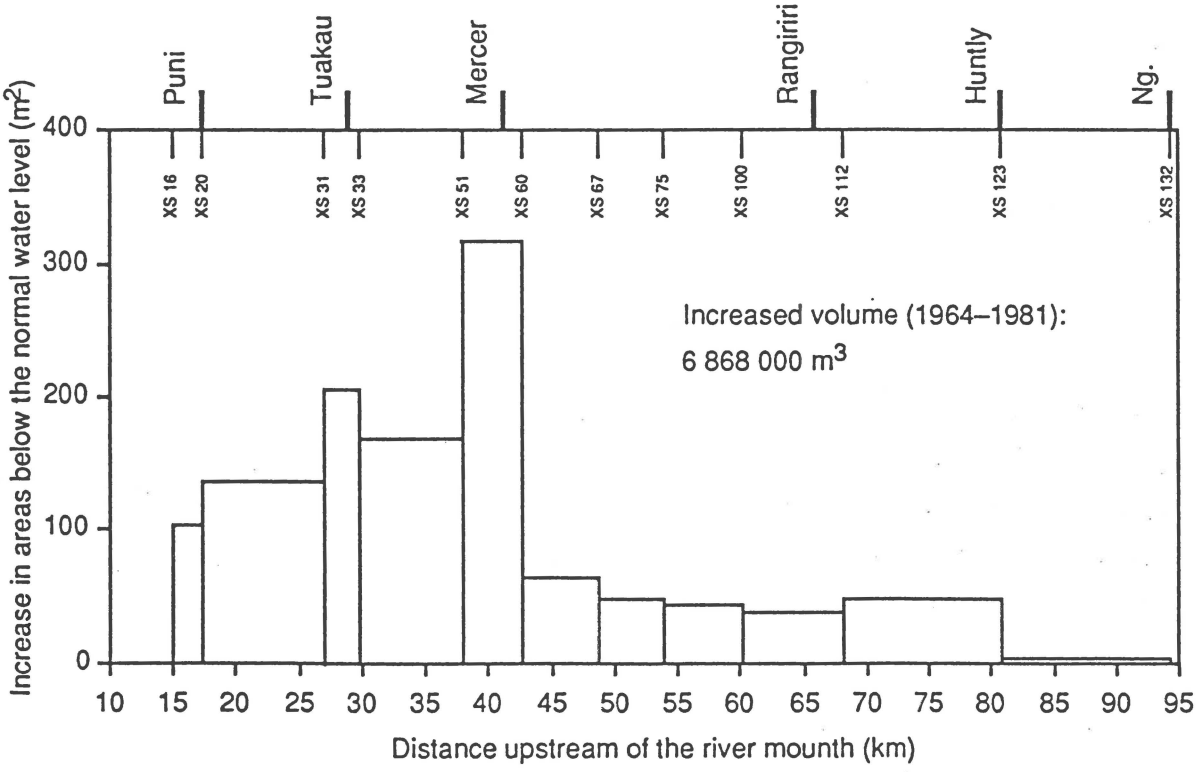


Figure 4.1 Increases in cross sectional area below the normal water level from 1964 to 1981 in the Waikato River between cross sections 16 and 132 (after Mulholland, 1983 or WVA No. 4474/434000/13).

There are several other methods that can be used to examine river cross sectional changes over time. For example, comparison between different smoothed mean bed level profiles from different surveys; comparison between cross sectional areas obtained from different surveys; comparison between different surveys by plotting them together at a particular cross section. These methods are used in this study.

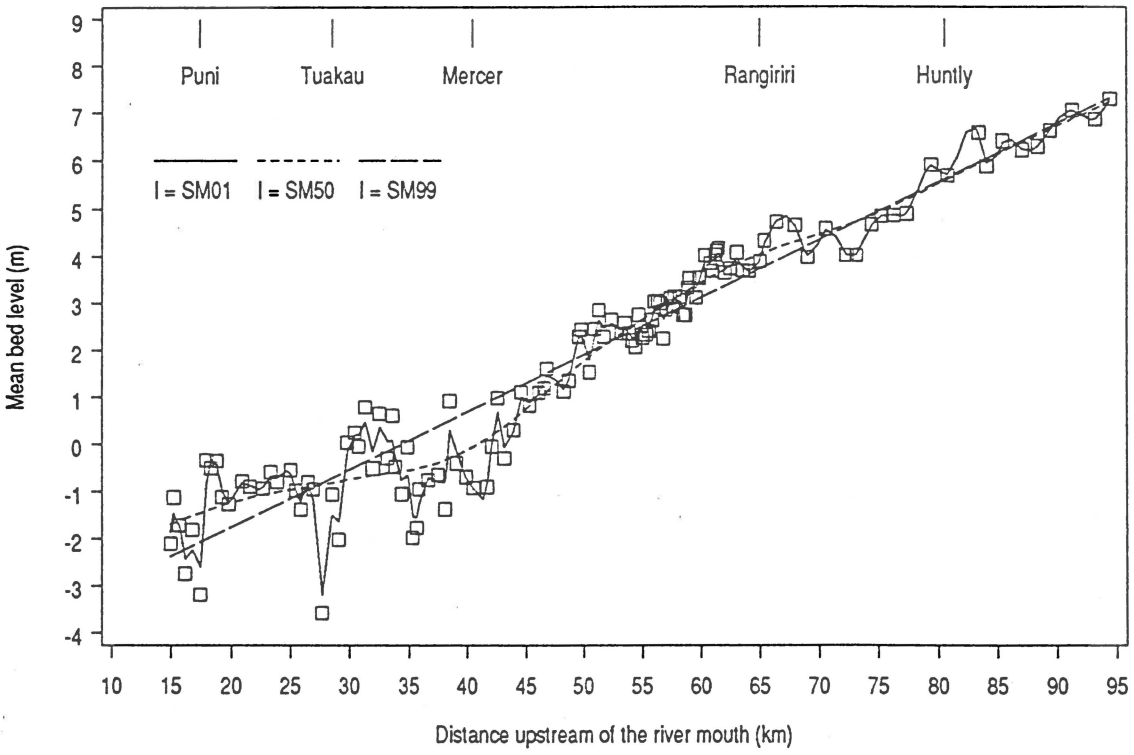


Figure 4.2 Effects of option I in the software SAS on smoothed curves of mean bed levels from the 1985/1989 survey (data from the river surveys held at the Waikato Regional Council).

Mean bed levels along the river between Puni and Ngaruawahia (cross sections 16–132) from the surveys in 1963/1964, 1979/1983 and 1985/1989 were treated as random data and interpreted by smoothed fit lines—the differences between these interpreting lines generally indicating the mean bed level changes. A cubic spline method, with minimisation of a linear combination of the sum of squares of the residuals of fit and the integral of the squares of the second derivative (Reinsch, 1967) was used to interpret the data and the software SAS (1985) was applied.

The smoothness of the fitted curve determined by this procedure can be altered by the value of ## in option I=SM## required by SAS (1985) from 01 to 99. The larger the value, the smoother the fitted curve. Figure 4.2 shows the effects of different values of ## on the fitted curves using mean bed level data

from the 1985/1989 survey. Option I=SM65 has been selected for curve fitting because it appears to be able to produce a fit line without any sudden changes, while also showing the difference in river gradation (Figure 4.3).

Changes in cross sectional area downstream of Ngaruawahia were determined for the survey periods between 1963/1964 and 1979/1983, 1979/1983 and 1985/1989, and 1963/1964 and 1985/1989. A moving average of three-sequent-cross-sectional areas was used to minimise random errors. The cross sectional changes were further used to estimate the total volume changes in the reach.

For the river reach between Karapiro and Ngaruawahia, data from the surveys of 1974/1976 and 1985/1989 upstream of Ngaruawahia were compared to estimate cross sectional, and volume changes in the reach were determined.

The term *stable* has been used here for those regions of the river where in terms of measurement errors no significant trends of bed level changes occurred within the survey period.

4.2.4 Changes in mean bed level and cross sectional area over time

Figure 4.3 shows mean bed levels of the cross sections surveyed between Puni and Ngaruawahia in the Waikato River obtained from three different surveys and their interpreting smoothed lines. The mean bed level variations along the river may partly be caused by variations of the river widths which were used to determine mean bed levels. Differences between the smoothed fit lines interpreting the mean bed levels along the river indicate that they have degraded over time (Figure 4.3). And the mean bed levels around Mercer and further downstream have clearly degraded more rapidly than those upstream. A similar pattern is indicated by cross sectional area changes (Figures 4.4-A–B).

For an analysis of changes in cross sectional area below the normal water level using the moving average of the areas of three adjacent cross sections in the reach between cross sections 16–132 (14.95 and 94.45 km upstream of

the river mouth), we assume that cross sectional areas at cross sections 16 and 132 are equal to, respectively, an average of the first three-sections areas and the last three-sections areas. There are no detailed surveys downstream of cross section 16 as far as to the river mouth except for a few surveys at cross sections 14 and 15 (14.16 and 14.47 km upstream of the river mouth). Cross sectional area at cross section 15 in 1964, 1970, 1975 and 1981 were,

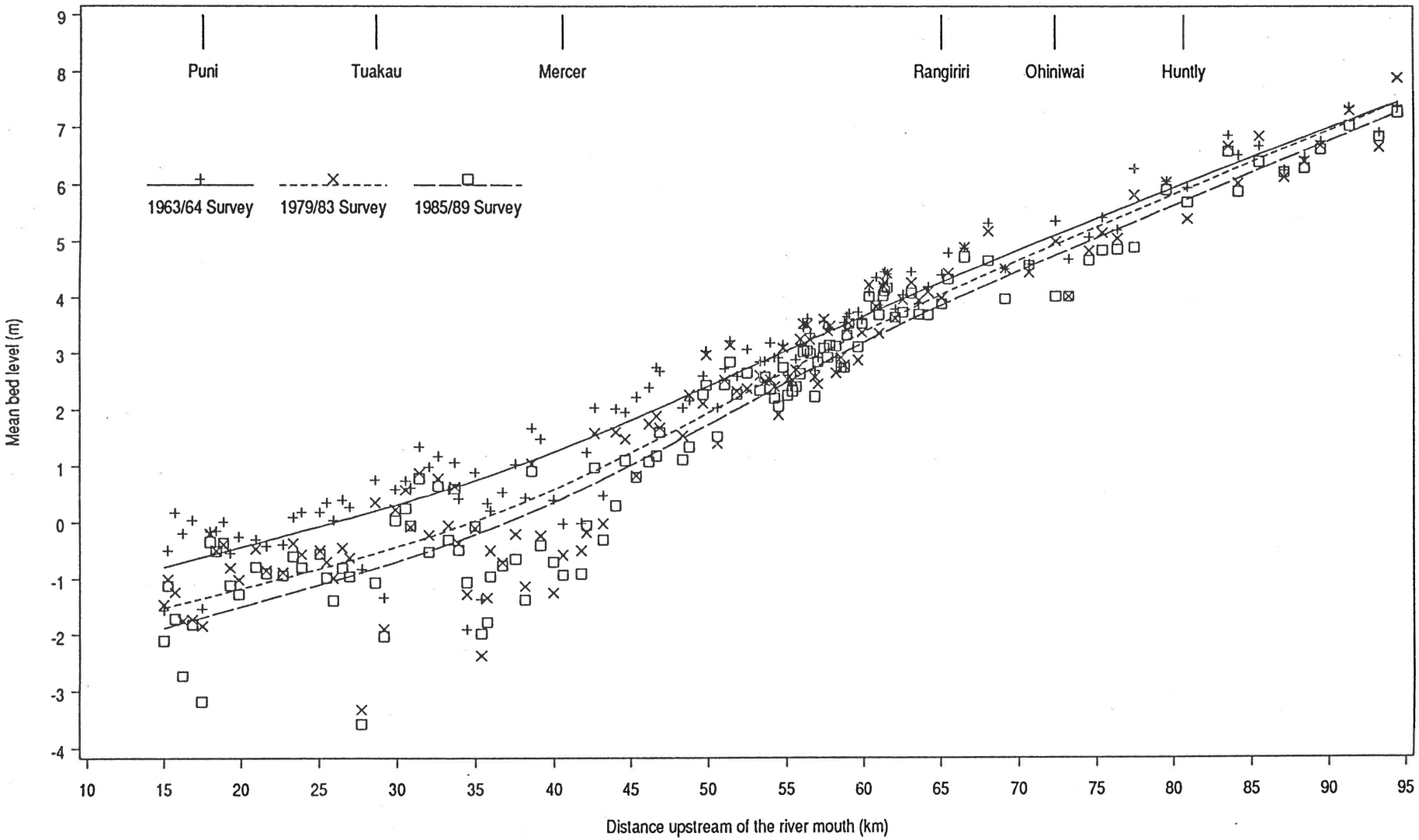


Figure 4.3 Mean bed levels in the lower Waikato River between cross sections 16–132 from three surveys of 1963/1964, 1979/1983 and 1985/1989. Smoothed curves interpreting the mean bed levels are obtained using SAS with option I=SM65 (data from the river surveys held at the Waikato Regional Council).

respectively, 633, 661, 607 and 712 m^2 with a mean 653 m^2. The area differences between the mean and those from each survey are 20, -8, 46 and -59 m^2 and except for the figure in 1981 their values are nearly the same order as the error of 46 m^2 (ref. Eq. (4.6), $\sqrt{2} \times 653 \times 5\% = 46$). Therefore we further assume that there are no cross sectional area changes downstream of the cross section at 14 km from the river mouth and a linear change from this cross section to cross section 16.

Figure 4.4 demonstrates the result of changes in cross sectional area in three periods—1963/1964 and 1979/1983, 1963/1964 and 1985/1989, and 1979/1983 and 1985/1989 with a total of 121 sections. The total river volume increases in the reach downstream of Ngaruawahia (cross section 132) between 1963/1964 and 1979/1983, and 1963/1964 and 1985/1989 are, respectively, estimated to be 6 527 000 m^3 and 10 980 000 m^3. Their difference of 4 453 000 m^3 is the volume increase between 1979/1983–1985/1989. The error limits in Figure 4.4 are determined by Eq. (4.6) and the cross sectional area measurement error is assumed to be ±28 m^2.

Based on the original survey data (cross sections 16–132 without moving average), the corresponding volume increases are estimated to be about 6 541 000 m^3, 10 794 000 m^3 and 4 253 000 m^3 respectively.

The three biggest increases in cross sectional area shown in Figure 4.4 are located near Puni, Tuakau and Mercer resulting evidently from the local operations of commercial sand extraction. Locations of the great cross sectional area increases upstream of Mercer, around 55 km and in a range of 70–75 km, might be caused by river training works, sand extraction and the fact that some islands upstream of Rangiriri were removed in 1987–1988 (W.M. Mulholland of the Waikato Regional Council, pers. comm., 1991).

The results in Figure 4.4 can also be demonstrated by using cumulative-with-distance river bed volume change. This cumulative change is shown in Figure 4.5.

For changes in cross sectional area of the river upstream of Ngaruawahia, Figure 4.6-A shows that the cross sectional areas of these cross sections surveyed generally increased in the survey period of 1974/1976–1985/1989 except for those of cross sections 137, 154 and 174. The error limits in Figure 4.6-A are determined by Eq. (4.6) using the cross sectional area measurement error ±14 m^2.

Cross section 174, located at the Leamington Bridge, Cambridge, is the first

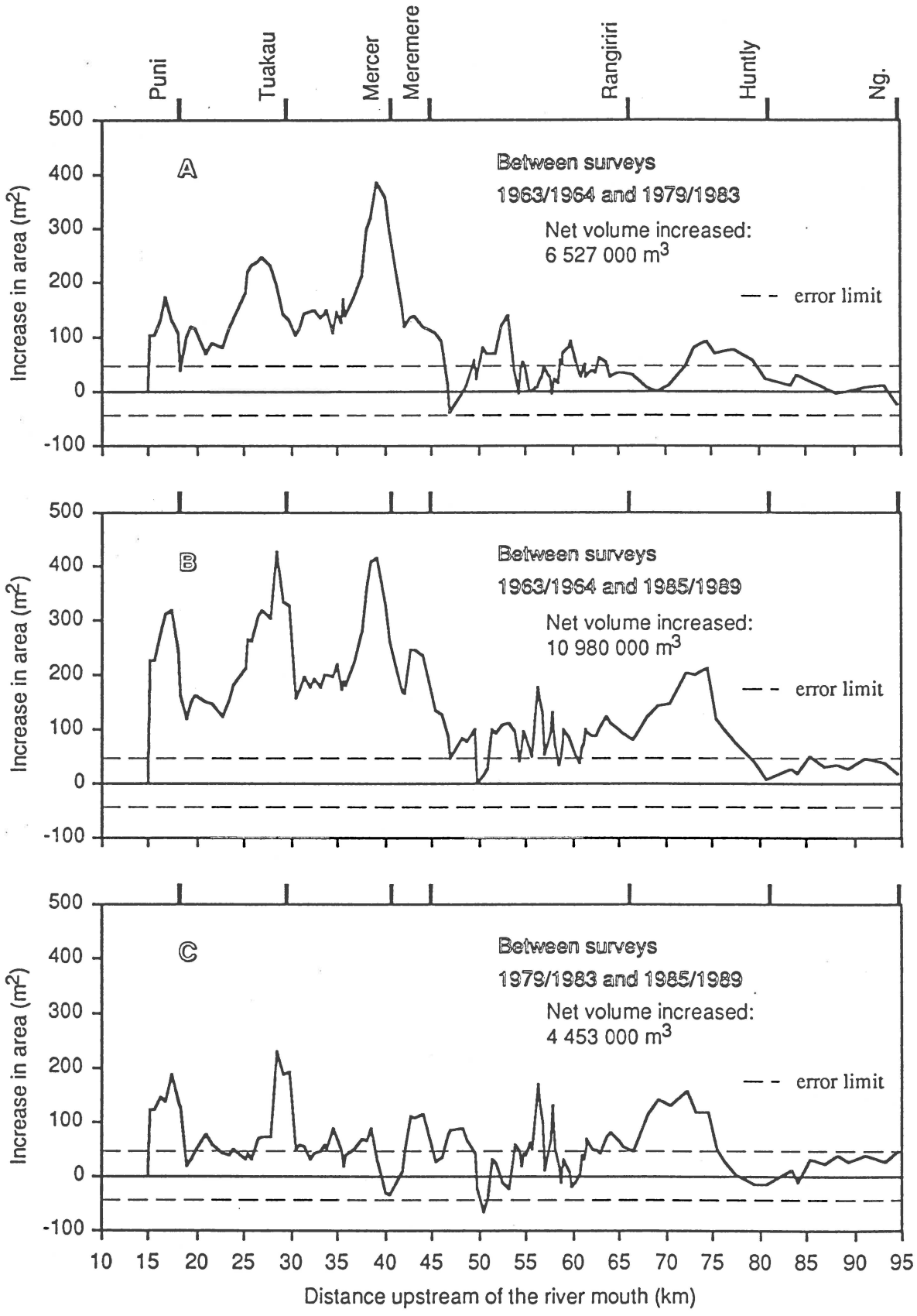


Figure 4.4 Changes in cross sectional area below the normal water level between two different surveys in the Waikato River downstream of Ngaruawahia. A moving average of the areas of three adjacent cross sections is used with a total of 121-sections data (data from the river surveys held at the Waikato Regional Council).

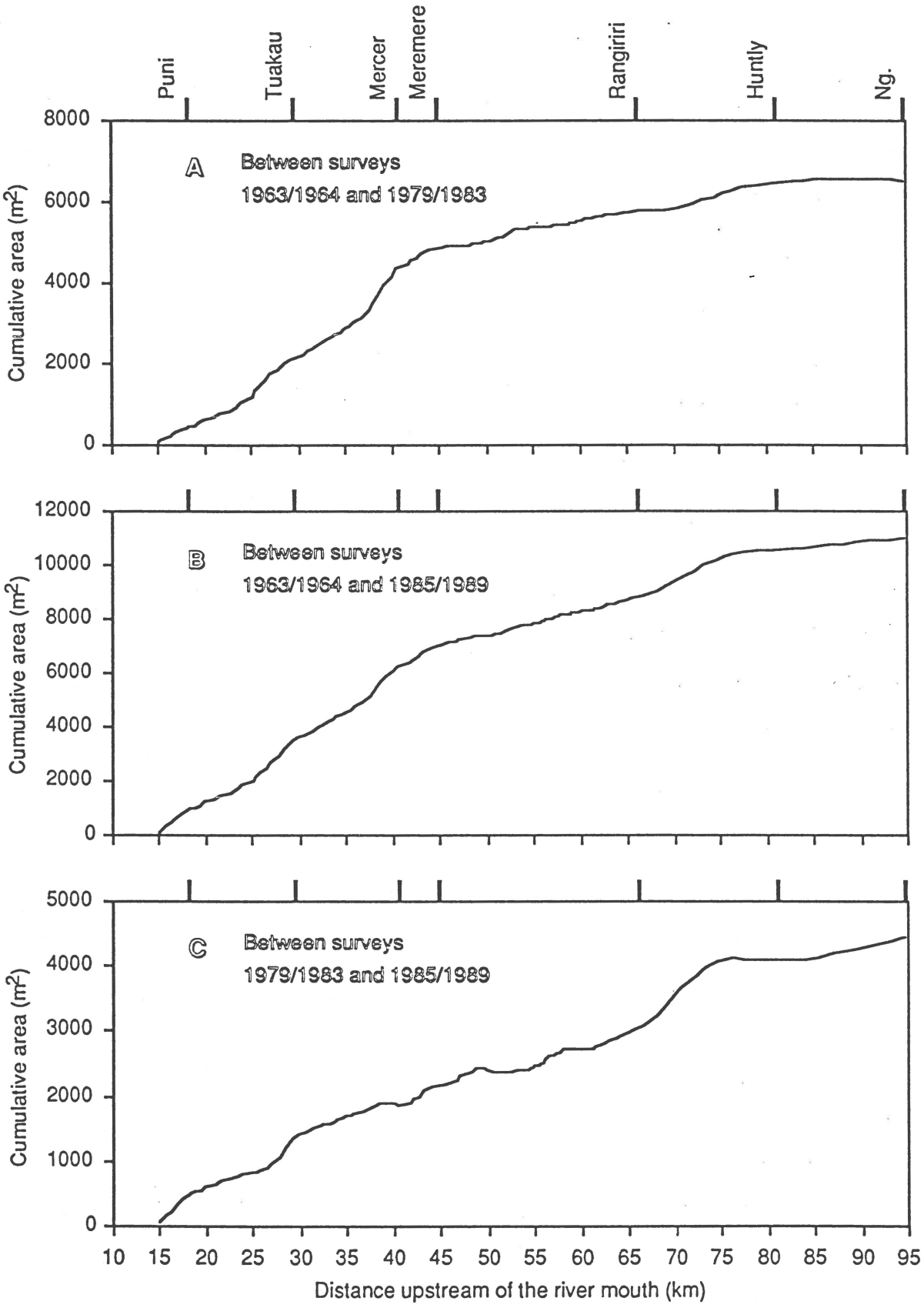


Figure 4.5 Cumulative-with-distance river volume changes between two different surveys in the Waikato River downstream of Ngaruawahia. Each figure is corresponding to those shown in Figure 4.4 (data from the river surveys held at the Waikato Regional Council).

one regularly surveyed below the Karapiro Dam. The bed materials around Cambridge were gravel and the bed surfaces were partially armoured based on a diving observation in 1961 (Ministry of Works, 1962). Except for the 1988 survey using echo sounding, the other surveys were obtained by hand survey techniques. In a comparison with the survey data from 1951 to 1988, although the differences in bed level changes at some positions are about one metre, no significant long term degradation or aggradation can be detected and the bed seems to be more or less stable. The amount of decrease in cross sectional areas in 1974/1976–1985/1989 is the same magnitude as the errors (Figure 4.6-A).

For cross section 154, in a comparison with plots of the surveys in 1974 and 1988 (plots of river surveys held at the Waikato Regional Council) it is evident that bed levels aggraded in its middle channel. There was dredging at this cross section at the end of 1973 (*Water Treatment Station-Dredging, 3108/434005* held at the Waikato Regional Council). It is likely that the 1974 survey was completed after the dredging and the aggradations resulted from a subsequent infill of sediments into the dredged hole.

The river width at cross section 137, located at the Horotiu Bridge, is very narrow (about 30 m) in a comparison with the others up and downstream. Its bed levels reduced by about 3 m in the survey period 1950–1963 but afterwards was relatively stable. A decrease in its cross sectional area between 1976 and 1988 from the surveys was caused by reductions of river width that may have resulted from the dumping of materials on its bank.

Assuming no changes in areas at cross section 174 between surveys 1974/1976 and 1985/1989, a linear variation of cross sectional areas between two cross sections surveyed, and omitting the survey data at cross sections 137 and 154, Figure 4.6-B is obtained. This shows an increase in cross sectional areas below the normal water level along the Waikato River upstream of Ngaruawahia. The total volume increased in this reach is estimated at 1 503 000 m^3.

In summary, the river cross sectional surveys suggest that the whole lower Waikato River has degraded over time since 1963 in terms of variations in mean bed level and cross sectional area below the normal water level. Net volume loss of about 10 980 000 m^3 occurred downstream of Ngaruawahia between the surveys of 1963/1964 and 1985/1989. The largest increases in cross sectional area were found to be around Puni, Tuakau and Mercer. The increases further upstream between Meremere and Ngaruawahia were

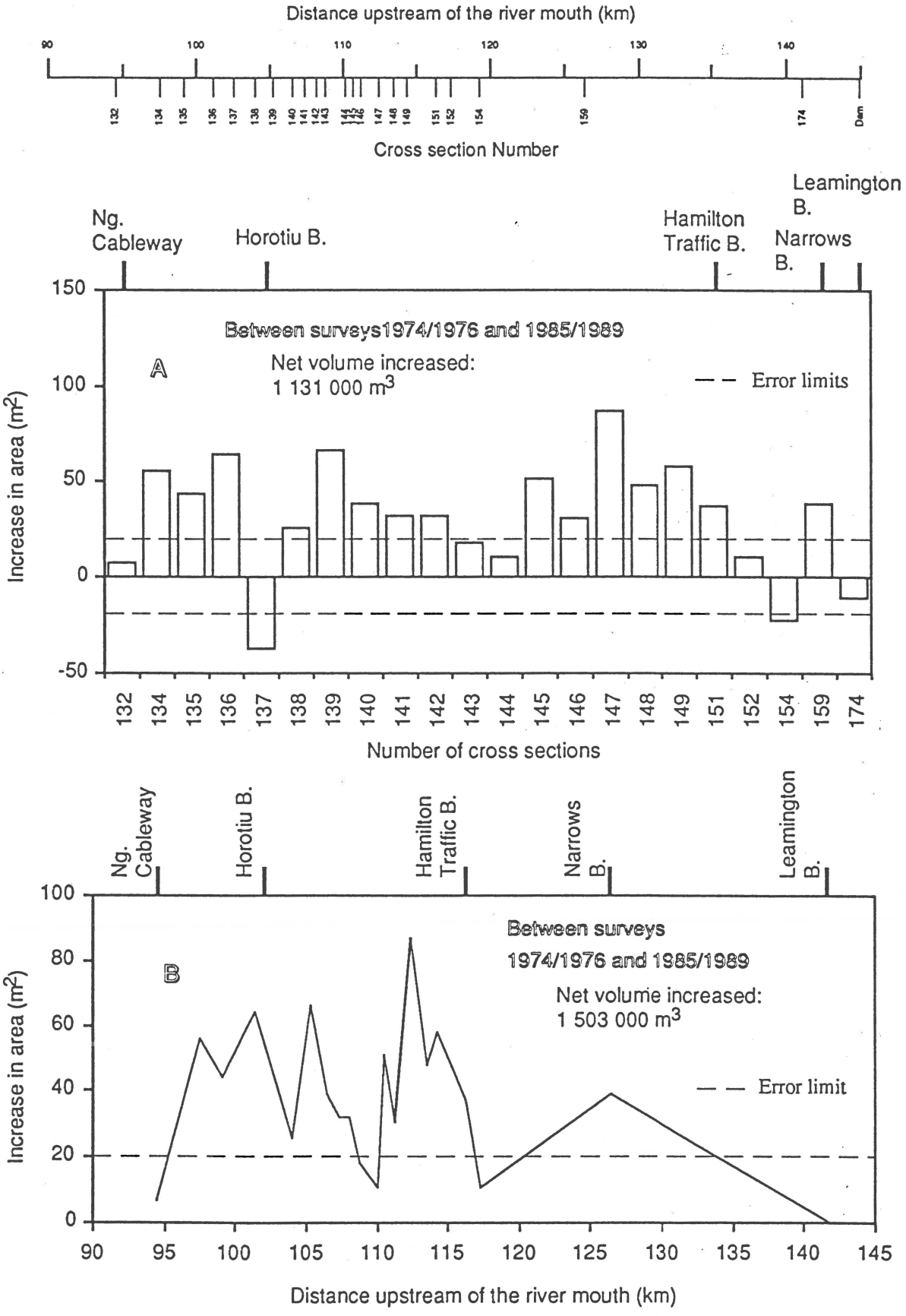


Figure 4.6 Changes in cross sectional area below the normal water level between surveys 1974/1976 and 1985/1989 in the Waikato River upstream of Ngaruawahia. In Figure B, linear variations and no changes at cross section 174 are assumed and the survey data at cross sections 137 and 154 are omitted (data from the river surveys held at the Waikato Regional Council).

relatively small in comparison with those downstream and probably they were caused by river training works as well as sand extraction and effects of the big fall in bed level around Mercer. The river reach between Cambridge and Ngaruawahia showed a general degradation too, by a comparison of changes in cross sectional area from surveys 1974/1976 and 1958/1989. The total volume increase of that river reach was estimated at 1 503 000 m^3.

4.3 Analysis of gauging data and rating curves

4.3.1 Gauging data and rating curves

There are five gauging stations, located at Hamilton, Ngaruawahia, Huntly, Rangiriri and Mercer, in the lower Waikato River. For each set of gauging data, the variables—time (T), water level (WL), cross sectional area below the water level (A), maximum depth (D_{max}), width at the water level (W) and discharge (Q)—are available.

Gauging data are used to obtain rating curves ($Q = f(WL)$). Each rating curve is determined from a number of gauging taken in a certain period ΔT ($Q = f(WL, \Delta T)$) with an acceptable error of ±8% discharge (McMillan, 1990). The same error of ±8% discharge has been used in determination of the lower Waikato River rating curves (H. McMullan of the Waikato Regional Council, pers. comm., 1992).

Table 4.1 Number of rating curves available at each gauging station in the lower Waikato River (data from the Waikato Regional Council).

Station	Hamilton Traffic Bridge	Ngaruawahia Cableway	Huntly Rail Bridge (ext.)	Rangiriri Bridge	Mercer Bridge (ext.)
Period	1960–1992	1957–1992	1968–1992	1958–1992	1960–1992
No. rating curves	9	8	12	9	10

Table 4.1 lists the total number of rating curves available at the five gauging stations in the lower Waikato River. The number of rating curves at the Huntly Rail Bridge includes those at the Tainui Bridge, and the number at the Mercer Bridge includes those at the Mercer Wharf.

Most gauging records have been checked for possible mistakes. Some of them have been omitted because of errors and others corrected. The detailed comments on each omitted record have been attached in Appendix Four.

For the gauging station at Huntly, discharges were measured at two sites—

the Huntly Rail Bridge (site No. 43419) in 1958–1986 and the Tainui Bridge (site No. 1143448, about 634 m upstream of the Huntly Rail Bridge) from 1984 up to now (1993). The corresponding water levels were also measured at two sites—the Huntly Rail Bridge in 1958–1986 (with some at Boatie, located close to the opposite side of the Huntly Power Station, some at the Huntly Power Station) and the Huntly Power Station (site No. 1543495) from 1984 up to now (1993).

For the gauging station at Rangiriri, its discharges were measured at the old Rangiriri Bridge in 1958–1968 and at the new Rangiriri Bridge (site No. 43420) from 1969 up to now (1993). Its water levels before 1965 might be measured at the staff gauge by the old bridge and after that time at the staff gauge beside the Rangiriri water level recorder upstream about 100 m (H. McMullan of the Waikato Regional Council, pers. comm., 1992).

For the gauging station at Mercer, some discharges were taken at the Mercer Wharf (site No. 43463) in 1960–1982, and others at the Mercer Bridge (site No. 1043446, about 100 m away from the Mercer Wharf) from 1980 up to now (1993). Except for the period 1960–1962, all the water levels were measured at the Mercer staff gauge.

Table 4.2 Number of gauging records used and omitted for an analysis in the lower Waikato River (data from the Waikato Regional Council).

Station	Hamilton	Ng.	Huntly Rail B.	Tainui B.	Old Rang. B.	New Rang. B.	Mercer Wharf	Mercer B.
Period	1960–1992	1958–1992	1958–1986	1984–1992	1958–1968	1969–1992	1960–1982	1980–1992
No. used	172	169	119	103	15	202	140	108
No. omitted	1	2	2	1	0	2	1	0

Table 4.2 lists the number of gauging records available for analysis at each gauging site in the lower Waikato River and a total of 9 records have been omitted.

4.3.2 Methods of analysis of rating curves and gauging data

Rating curves applied in each different period ($WL = f(Q,\Delta T)$) have been used to obtain variations in water level for a given discharge of 350 m^3/s at each gauging station ($WL = f(350,T)$). These may be able to indicate river bed changes.

Minimum bed levels, mean bed levels, water levels adjusted to an index flow of 350 m^3/s have been derived from all the gauging records as a time series.

All of them are believed to be an index of a cross sectional configuration and their variations over time can indicate the cross sectional changes.

The minimum bed level (BL_{min}), which is defined as the difference between the water level (WL) and the maximum depth (D_{max}):

$$BL_{min} = WL - D_{max} \tag{4.8}$$

can be obtained from each individual gauging. A time series of minimum bed levels ($BL_{min}(T)$) then can be established from all the gauging records at each gauging station.

In the case of gauging data taken from two different sites at the Huntly and the Mercer stations, only the data from one site are used for the analysis. Table 4.3 provides a summary of the data used for analysis of the minimum bed level variations at the gauging stations.

Table 4.3 Summary of the data used for an analysis of minimum bed level variations in the lower Waikato River (data from the Waikato Regional Council).

Station	Hamilton Traffic Bridge	Ngaruawahia Cableway	Huntly Rail Bridge	New Rangiriri Bridge	Mercer Wharf
Period	1960–1992	1958–1992	1958–1986	1969–1992	1960–1982
No. of Records	172	169	119	202	140

The mean bed level (BL), which is defined as the difference between the water level (WL) and the mean depth ($H = A / W$):

$$BL = WL - H \tag{4.9}$$

can be determined from each individual gauging record and a time series of mean bed levels ($BL(T)$) can be established from all the gauging data. Similar to the minimum bed level, for the gauging taken from different sites only the data from one site are used.

Table 4.4 Criteria and characteristics of the data used for an analysis of the mean bed level variations in the lower Waikato River (data from the Waikato Regional Council).

Station	Hamilton Traffic Bridge	Ngaruawahia Cableway	Huntly Rail Bridge	New Rangiriri Bridge	Mercer Wharf
Period	1960–1992	1958–1992	1958–1986	1969–1992	1960–1982
Q (m^3/s)	200–500	250–550	250–550	300–600	350–650
Water Level (m)	12.28–15.35	9.28–11.17	7.47–9.16	5.82–7.17	2.63–4.79
Width (m)	71.0–98.1	136.0–164.0	210.0–242.3	190.2–247.0	116.0–128.0
No. of Records	125	89	63	99	60

In order to minimise the non-rectangular (width-variations) effects on mean bed levels, gauging records within a certain range of discharges (equivalent to elevations) are used for an analysis of mean bed level variations. Table 4.4 lists the criteria and characteristics of the data used for this analysis at each station.

For the lower Waikato River, Table 4.5 lists the ratios of width to depth at each gauging station from a range of the gauging records. All of the ratios are greater than 10 which was suggested as a criteria for *a wide channel* (Chow, 1959, pages 26–27).

For a wide river channel, Manning's equation can be used. Therefor we have the following equations (Chow, 1959):

$$V = C\sqrt{HS} \tag{4.10}$$

$$C = \frac{1}{n} H^{1/6} \tag{4.11}$$

$$Q = WHV \tag{4.12}$$

where V is the mean flow velocity, C and n are, respectively, the Chézy's and the Manning's coefficients, S is the energy slope, W is the river width and H is the mean river depth. From Eqs. (4.10–12), we have

$$Q = W\frac{1}{n} H^{5/3} S^{1/2} \tag{4.13}$$

$$\frac{350}{Q} = \frac{W_{Q=350}\dfrac{1}{n_{Q=350}}(H_{Q=350})^{5/3} S_{Q=350}{}^{1/2}}{W\dfrac{1}{n} H^{5/3} S^{1/2}} \tag{4.14}$$

where $H_{Q=350} = A_{Q=350} / W_{Q=350}$, $A_{Q=350}$, $W_{Q=350}$, $S_{Q=350}$ and $n_{Q=350}$ are, respectively,

Table 4.5 Ratios of width to depth at the gauging stations in the lower Waikato River (data from the Waikato Regional Council).

Station	Hamilton Traffic Bridge	Ngaruawahia Cableway	Huntly Rail Bridge	New Rangiriri Bridge	Mercer Wharf
Period	1960–1992	1958–1992	1958–1986	1969–1992	1960–1982
Q (m^3/s)	200–500	250–550	250–550	300–600	350–650
Mean Width (m)	84.65	150.79	229.86	223.45	121.71
Sd. of Width (m)	5.50	5.42	8.63	11.42	2.46
Mean Depth (m)	3.19	3.01	2.37	2.54	5.38
Sd. of Depth (m)	0.57	0.43	0.38	0.32	0.51
Width/Depth	27	50	97	88	27

the cross sectional area, river width, energy slope and flow resistance at discharge 350 m^3/s, and 350 is an index flow.

The river width W, flow resistance n and energy slope S can be a function of discharge Q as well as time T. Given time, if the following conditions of

$$\frac{W_{Q=350}}{W} = \frac{n_{Q=350}}{n} = \frac{S_{Q=350}}{S} = 1 \tag{4.15a}$$

or

$$\frac{\partial W}{\partial Q} = \frac{\partial n}{\partial Q} = \frac{\partial S}{\partial Q} = 0 \tag{4.15b}$$

are held for each gauging, then Eq. (4.14) becomes

$$\frac{350}{Q} = \left(\frac{H_{Q=350}}{H}\right)^{5/3} \tag{4.16}$$

Because the mean bed level (BL) is defined as

$$BL = WL - H = WL_{Q=350} - H_{Q=350} \tag{4.17}$$

where $WL_{Q=350}$ is the water level at discharge 350 m^3/s, the following equation can be derived from Eqs. (4.16–17):

$$WL_{Q=350} = WL - \frac{A}{W} + \frac{A}{W}\left(\frac{350}{Q}\right)^{3/5} \tag{4.18}$$

All terms on the right hand side of the equation are available from each gauging record. Then a time series of water levels adjusted to discharge 350 m^3/s ($WL_{Q=350}(T)$) can be established. A time step of one year is taken in this study, i.e. gauging undertaken within a year is used to established $WL_{Q=350}(T)$ for that year.

The above method to derive adjusted water levels from gauging data has been applied to the lower Waikato River. The index discharge of 350 m^3/s has been used as a standard flow in the lower Waikato River (Mulholland, 1983).

In order to extend the time series of water levels adjusted to 350 m^3/s from gauging records, both sets of the gauging data available at the Mercer Wharf and the Mercer Bridge are used together for an analysis without any correction.

For the gauging station at Rangiriri, only those data taken from the Rangiriri Bridge starting from 1969 are used for an analysis of variations in

adjusted water level.

For the gauging data at Huntly, in order to follow consistency and to extend the time series, water levels measured at the Huntly Power Station of the new gauging data have been adjusted to those at the Huntly Rail Bridge by the following equation with n= 17, r^2= 0.998 obtained in 1983–1990 (Appendix Five)

$$WL_{\text{-Rail B.}} = 0.25929 + 1.0016 WL_{\text{-Power S.}} + 0.038 \qquad (4.19)$$

where $WL_{\text{-Rail B.}}$ and $WL_{\text{-Power S.}}$ are, respectively, water levels at the Huntly Rail Bridge and at the Huntly Power Station. The constant 0.038 is a coefficient for correction because zero of the staff gauge at the Huntly Rail Bridge has been found to be about 0.038 m higher than the Moturiki datum since 11-8-1982 when the staff gauge was installed, and discharges measured at Tainui Bridge have been adjusted to those at the Huntly Rail Bridge by multiplying 0.988 from a comparison of historical data (Appendix Six)

Table 4.6 lists the number of gauging records used for the analysis of adjusted water level variations and the criteria for the data selection. Determination of the criteria—the discharge limits—seem to be arbitrary as long as the assumptions in Eq. (4.15a or b) are held. But we try to include a discharge of 350 m^3/s in the limits and ensure there are sufficient data available within the limits.

There are many ways to interpret a time series of data and some of them can be used for forecasting (Box and Jenkins, 1976). Choice of a method seems to be more or less arbitrary. The same cubic spline method used in 4.2.1 is here applied to interpret the time series of minimum bed levels, mean bed levels and adjusted water levels by using the software SAS (1985). Option I required in the software can be determined by trial and error to produce a minimum deviation between the fit line and data. In this study, option I=SM65, which in general produces the best fit line for each time series of minimum bed levels, mean bed levels and adjusted water levels, has been taken.

Table 4.6 Data used for an analysis of adjusted water level variations in the lower Waikato River (data from the Waikato Regional Council).

Station	Hamilton Traffic Bridge	Ngaruawahia Cableway	Huntly Rail Bridge (ext.)	New Rangiriri Bridge	Mercer Bridge (ext.)
Period	1960–1992	1958–1992	1958–1992	1969–1992	1960–1992
Q (m^3/s)	200–500	250–550	250–550	300–600	350–650
No. of Records	125	89	129	99	101
Sd. (m)	0.28	0.09	0.09	0.07	0.17

4.3.3 Comments on assumptions, and the relationship between the adjusted water level and mean bed level

Eq. (4.18) also shows a relation between the adjusted water level ($WL_{Q=350}$) and the mean bed level ($BL = WL - A/W$). In summary, the assumptions made to get the water level adjusted to the index flow of 350 m^3/s ($WL_{Q=350}$) from each individual gauging record are that the results of the gauging can be interpolated or extrapolated to the index flow by Manning's equation and conditions indicated by Eq. (4.15a or b) are satisfied.

The assumptions indicated by Eq. (4.15a or b) are very difficult to test from field data. At this stage, we use only the gauging records within a range of discharges for analysis of their adjusted water level variations to try to satisfy these assumptions.

Because only data within a limited range of discharge are used, as long as no overflows occur, the condition of $W_{Q=350}/W = 1$ is statistically acceptable for a wide channel. For the other conditions $n_{Q=350}/n = S_{Q=350}/S = 1$, field data are not available to test them. But, for the same system, if some flows of gauging records used are greater than the index flow 350 m^3/s and others smaller, statistically we should have

$$\sum_i \frac{n_{Q=350}}{n_i} = \sum_i \frac{S_{Q=350}}{S_i} = 1 \tag{4.20}$$

Therefore the conditions above are probably at least acceptable in terms of a statistical average.

If the assumptions made for determination of adjusted water levels are held, their time series $WL_{Q=350}(T)$ can be obtained from all the gauging records. Based on Eqs. (4.13 and 4.18), we have

$$WL_{Q=350}(T) = BL(T) + \left(\frac{350n(T)}{W(T)\sqrt{S(T)}}\right)^{3/5} \tag{4.21}$$

If the following assumptions are held:

$$W(T) = \text{const.},\ n(T) = \text{const.},\ S(T) = \text{const.} \tag{4.22a}$$

or

$$\frac{\partial W}{\partial T} = \frac{\partial n}{\partial T} = \frac{\partial S}{\partial T} = 0 \tag{4.22b}$$

then variations of the adjusted water levels ($WL_{Q=350}$) can represent changes

of the mean bed levels (BL). Otherwise from Eq. (4.21) we have

$$\frac{\partial WL_{Q=350}}{\partial T} = \frac{\partial BL}{\partial T} + 0.6\left(\frac{1}{n}\frac{\partial n}{\partial T} - \frac{1}{W}\frac{\partial W}{\partial T} - \frac{0.5}{S}\frac{\partial S}{\partial T}\right)\left(\frac{350n}{W\sqrt{S}}\right)^{3/5} \tag{4.23}$$

which shows effects of non-constant W, n and S as well as BL over time on variations in $WL_{Q=350}$.

It is difficult to test the assumptions indicated by Eq. (4.22a or b), especially for $n(T) = \text{const.}$ and $S(T) = \text{const.}$ or $\partial n / \partial T = \partial S / \partial T = 0$. As the condition $W(T) = \text{const.}$, it can be examined from the gauging data.

Figure 4.7 shows variations in river width over time from the data used for an analysis of the mean bed levels at the gauging stations in the lower Waikato River (Table 4.4). The width at the Hamilton Traffic Bridge reduces over time in the period 1960–1992. The amount of its mean reduction (about 10 m) is less than the order of width variations (about 20 m) caused by the data within a certain range of discharges at a given time (Figure 4.7-A). The reduction may result from its non-rectangular cross section while the bed level is lowering (Figure 4.8). A constant width seems to be acceptable at the Ngaruawahia Cableway from the data in 1958–1992 (Figure 4.7-B). For the cross section at the Huntly Rail Bridge in 1958–1984 there was a sudden reduction of its width after 1980 that may have been caused by road construction on the river bank. Before that time no significant width reduction trends could be detected (Figure 4.7-C). Generally river width at Rangiriri and Mercer seems to be constant from the gauging data available in 1970–1992 and 1960–1980 respectively (Figures 4.7-D and E). In summary, condition $W(T) = \text{const.}$ in terms of a statistical mean seems to be acceptable at the gauging stations in the lower Waikato River.

For condition $n(T) = \text{const.}$, if bed materials at the cross section have no trends to become coarser or finer, it might be acceptable. For $S(T) = \text{const.}$, if the cross section is far away from fixed control points, such as the sea or a dam, it might also be acceptable.

If conditions in Eq. (22a or b) are held, and adjusted water levels can be obtained from gauging records as well, then a unit slope exists (ref. Eq. (4.21)) between the adjusted water level ($WL_{Q=350}(T)$) and the mean bed level ($BL(T)$). It is possible to test the unit slope by using the gauging data. This may be an indirect way to examine both conditions indicated by Eqs. (4.15a or b) and (4.22a or b).

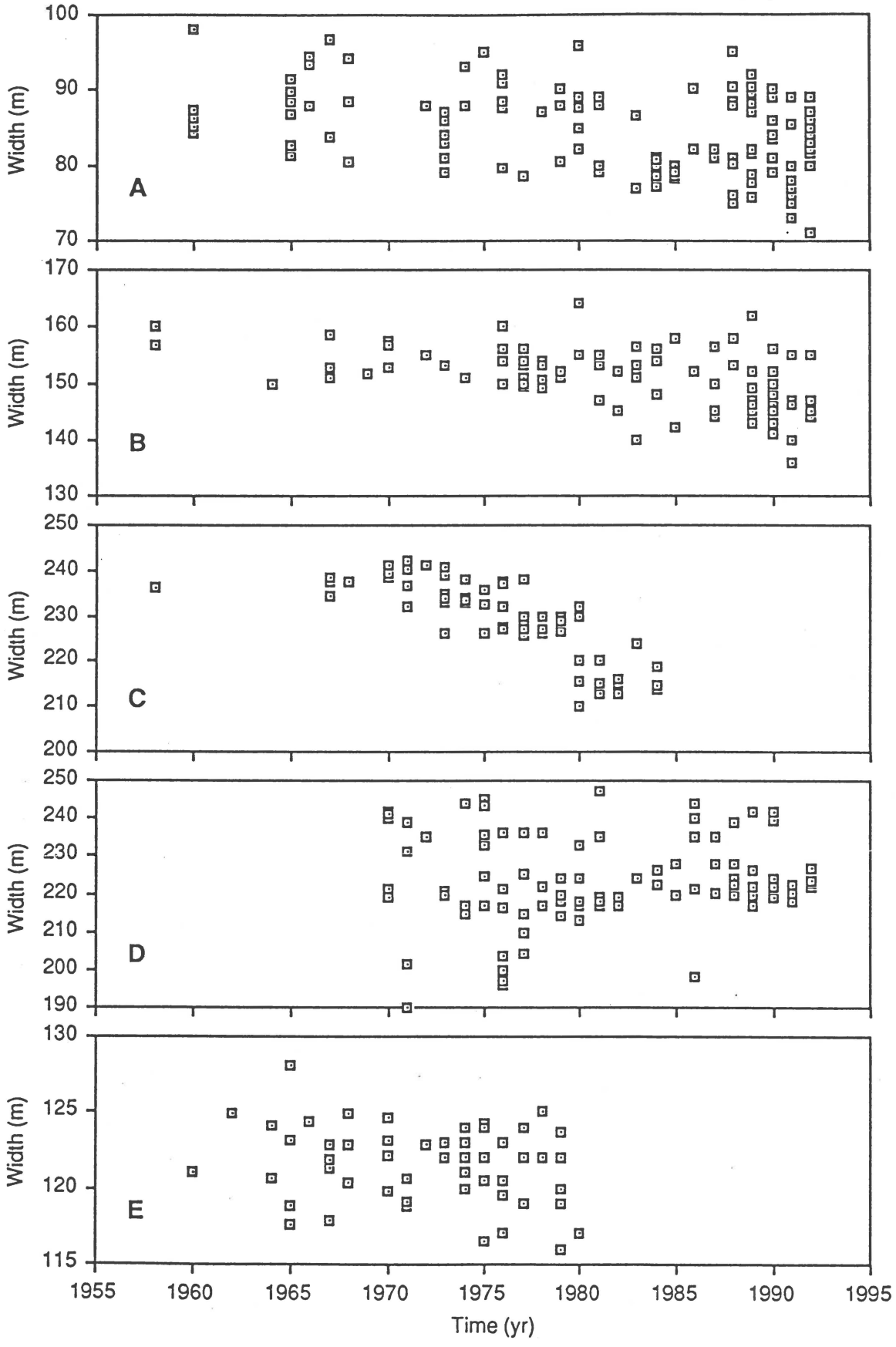


Figure 4.7 Variations in river widths over time at the Hamilton Traffic Bridge (A), at the Ngaruawahia Cableway (B), at the Huntly Rail Bridge (C), at the Rangiriri Bridge (D), at the Mercer Bridge (E) (data from the gauging database at the Waikato Regional Council in 1958–1992).

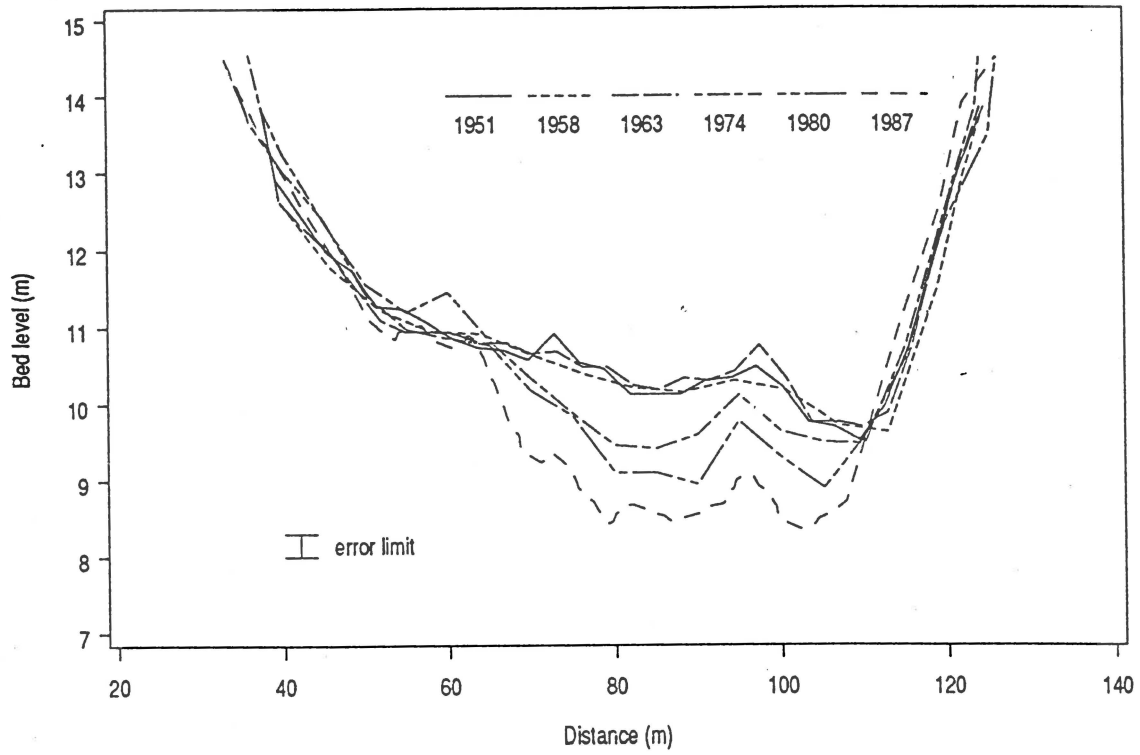


Figure 4.8 Cross section at the Hamilton Traffic Bridge from different river surveys. The normal water level is 13.84 m at this cross section (data from the river surveys in 1951–1987 held at the Waikato Regional Council).

Figure 4.9 shows the relationship between $WL_{Q=350}(T)$ and $BL(T)$ at the gauging stations in the lower Waikato River. The data used are the same as those indicated in Table 4.4, except for the data at the Huntly Rail Bridge, which are from 1958 to 1979 because of the different river widths after 1980 as discussed above. Plotted points of the relation at the Hamilton Traffic Bridge (Figure 4.9-A), at the Ngaruawahia Cableway (Figures 9–B), at the Huntly Rail Bridge (Figure 4.9-C) and at the Rangiriri Bridge (Figure 4.9-D) group around the unit slope line, especially for the data at the Hamilton Traffic Bridge and at the Ngaruawahia Cableway. These results demonstrate that the conditions in Eqs (4.15a or b) and (4.22a or b) may be acceptable. But the points at the Mercer Wharf (Figure 4.29-E) vary from the unit slope line and this likely relates to effects of a non-constant S (ref. Eq. (4.23)) because its location is near the sea and S becomes a function of time as bed levels have been lowering, and/or the mean bed levels BL at the Mercer Wharf may not be representative because of the local operation of sand extraction and its narrow width in comparison with those of the upstream and downstream cross sections.

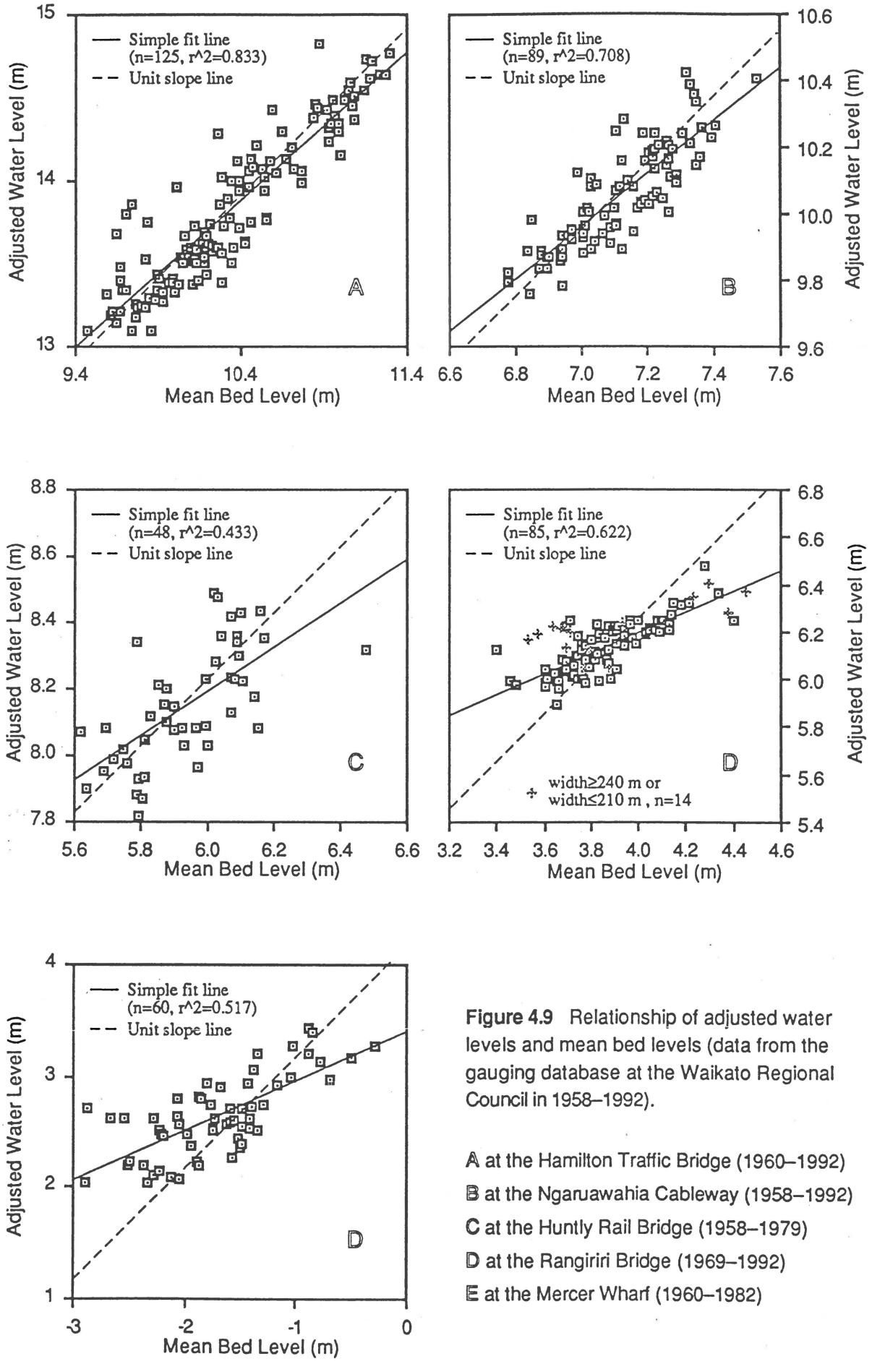


Figure 4.9 Relationship of adjusted water levels and mean bed levels (data from the gauging database at the Waikato Regional Council in 1958–1992).

A at the Hamilton Traffic Bridge (1960–1992)
B at the Ngaruawahia Cableway (1958–1992)
C at the Huntly Rail Bridge (1958–1979)
D at the Rangiriri Bridge (1969–1992)
E at the Mercer Wharf (1960–1982)

Table 4.7 Results of a linear regression analysis between adjusted water levels and mean bed levels of the gauging data in the lower Waikato River (data from the Waikato Regional Council).

Station	Hamilton Traffic Bridge	Ngaruawahia Cableway	Huntly Rail Bridge	New Rangiriri Bridge	Mercer Wharf
Period	1960–1992	1958–1992	1958–1979	1970–1992	1960–1982
Q (m^3s^{-1})	200–500	250–550	250–550	300–600	350–650
No. of Records	125	89	48	85	60
r^2	0.833	0.708	0.433	0.662	0.517
Relation	y=4.584+0.894x	y=4.419+0.792x	y=4.167+0.670x	y=4.428+0.442x	y=3.404+0.448x

A linear regression has also been applied to the same data in Figure 4.9 and their fit lines have been plotted together (Figure 4.9). Table 4.7 lists results of the regression analysis. There is a good relationship between adjusted water levels and mean bed levels at the Hamilton Traffic Bridge and at the Ngaruawahia Cableway with $r^2>0.7$. For Rangiriri, only the data of $240>W>210$ m are used (ref. Figure 4.7).

4.3.4 Results of an analysis of minimum bed levels, mean bed levels and water levels adjusted to 350 m^3/s from gauging records and from rating curves

Variations of minimum bed levels, mean bed levels, water levels adjusted to the index discharge of 350 m^3/s obtained from gauging records, and water levels obtained from rating curves at the discharge of 350 m^3/s of the five gauging stations, located at Hamilton, Ngaruawahia, Huntly, Rangiriri and Mercer in the lower Waikato River, are plotted in Figures 4.10–29. Standard deviations of the fit lines for adjusted water levels are given in Table 4.6. From these figures, some conclusions at each station can be drawn as follows.

- Hamilton Traffic Bridge from 1960–1992 (Figures 4.10–13):

1. All the figures show a consistent trend of bed and water levels dropping over time.
2. Mean bed levels started degrading after about 1967, minimum bed levels dropped steadily from 1960–1992 but somewhat more rapidly after about 1970.
3. Mean bed levels and adjusted water levels have dropped by about 1.0 m and minimum bed levels by 1.8 m.
4. Trends of the degradation are continuing.
5. Water levels from rating curves generally show a similar trend.

- Ngaruawahia Cableway from 1958–1992 (Figures 4.14–17):

  1. Mean bed levels and adjusted water levels have tended to drop over time, particularly in recent years, with an amount of about 45 cm.

  2. Adjusted water levels have been steadily dropping since about 1970.

  3. Mean bed levels started to drop rapidly after about 1982.

  4. Degradation of mean bed levels also occurred in 1958–1970, but the trend is less confident due to the limited data in this period.

  5. Variations in water levels from rating curves show a similar trend to adjusted water levels.

  6. Minimum bed levels fluctuated by about 1 m, but no marked constant trends of their reduction can be detected from the outline.

- Huntly Rail Bridge from 1958–1992 (Figures 4.18–21):

  1. Both mean bed levels and adjusted water levels dropped significantly in the period from 1970–1980.

  2. Trends determined prior to 1966 are not held in great confidence because only one record is available for the analysis.

  3. Adjusted water levels appeared to be stable (and perhaps increase slightly) after about 1985.

  4. Water levels from rating curves show a similar trend in adjusted water levels.

  5. Minimum bed levels varied by about 2 m, but there was no reduction trend in the outline. The outline even shows that they were fairly constant from 1970–1986.

- Rangiriri Bridge from 1970–1992 (Figures 4.22–25):

  1. Between 1970–1992, mean bed levels dropped by about 28 cm and adjusted water levels by 40 cm.

  2. A similar trend for falling water levels from the rating curves is evident.

  3. Minimum bed levels varied by about 2 m over the period 1969–1992 with no significant trends in degradation.

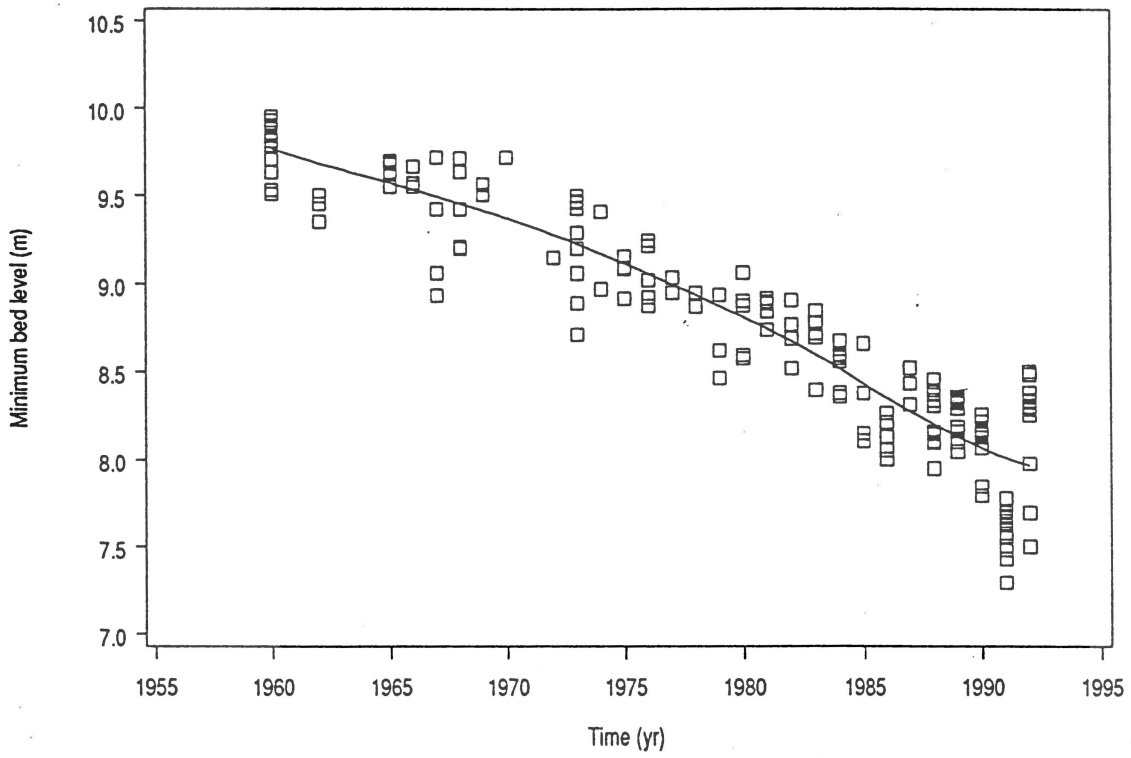


Figure 4.10 Variations of minimum bed levels at the Hamilton Traffic Bridge. Smoothed fit curve is obtained from 172 records using SAS with option I=SM65 (data from the gauging database at the Waikato Regional Council in 1960–1992).

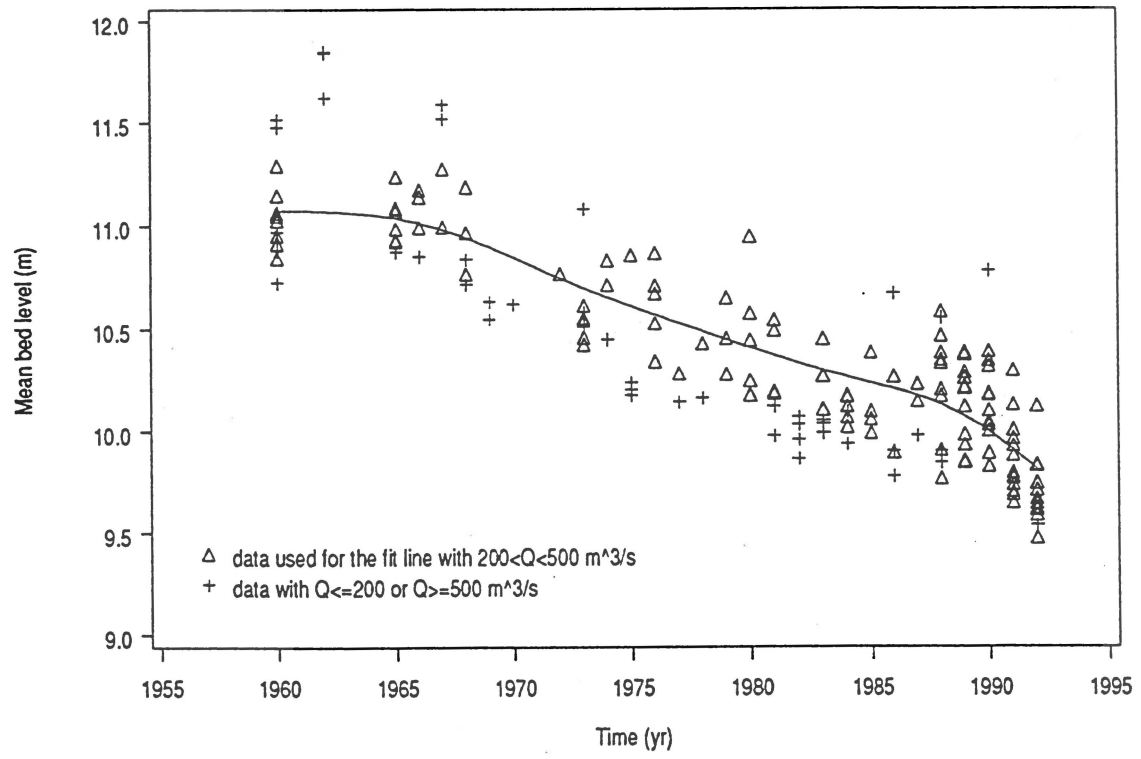


Figure 4.11 Variations of mean bed levels at the Hamilton Traffic Bridge. Smoothed fit curve is obtained from 125 records using SAS with option I=SM65 (data from the gauging database at the Waikato Regional Council in 1960–1992).

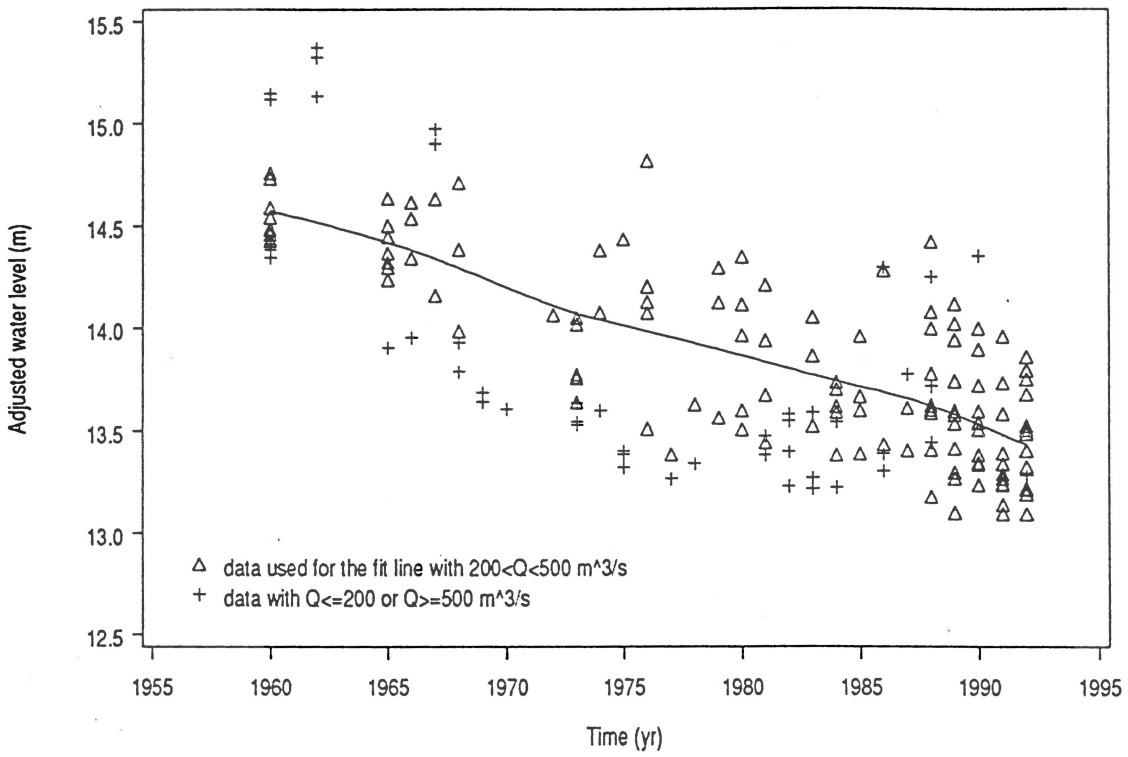


Figure 4.12 Variations of water levels adjusted to discharge 350 m^3/s at the Hamilton Traffic Bridge. Smoothed fit curve is obtained from 125 records using SAS with option I=SM65 (data from the gauging database at the Waikato Regional Council in 1960–1992).

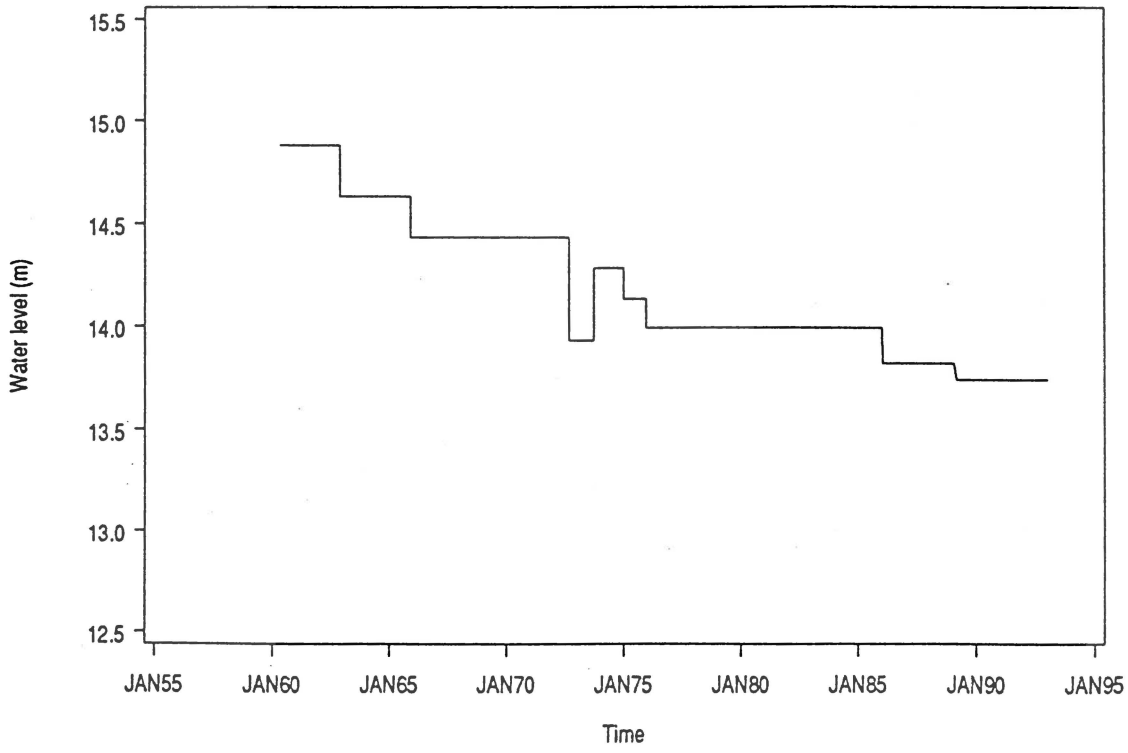


Figure 4.13 Variations of water level obtained from a total of 9 rating curves for discharge 350 m^3/s at the Hamilton Traffic Bridge (data from TIDEDA database at the Waikato Regional Council in 1960–1992).

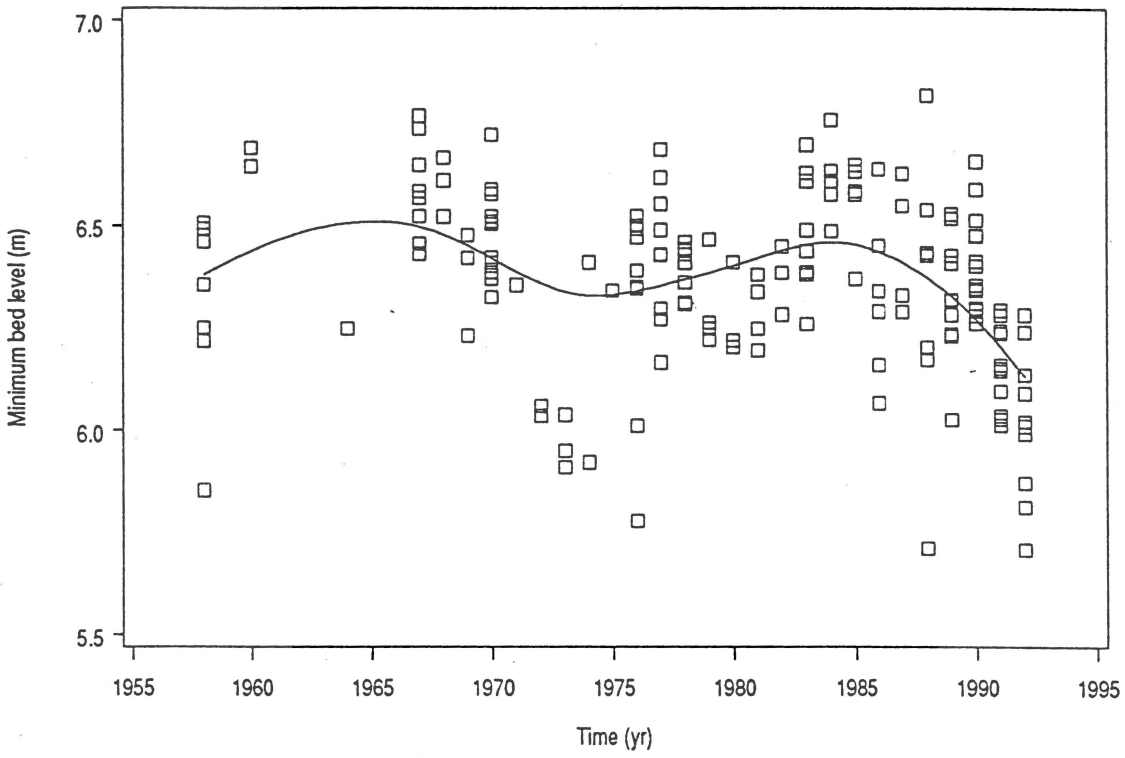


Figure 4.14 Variations of minimum bed levels at the Ngaruawahia Cableway. Smoothed fit curve is obtained from 169 records using SAS with option I=SM65 (data from the gauging database at the Waikato Regional Council in 1958–1992).

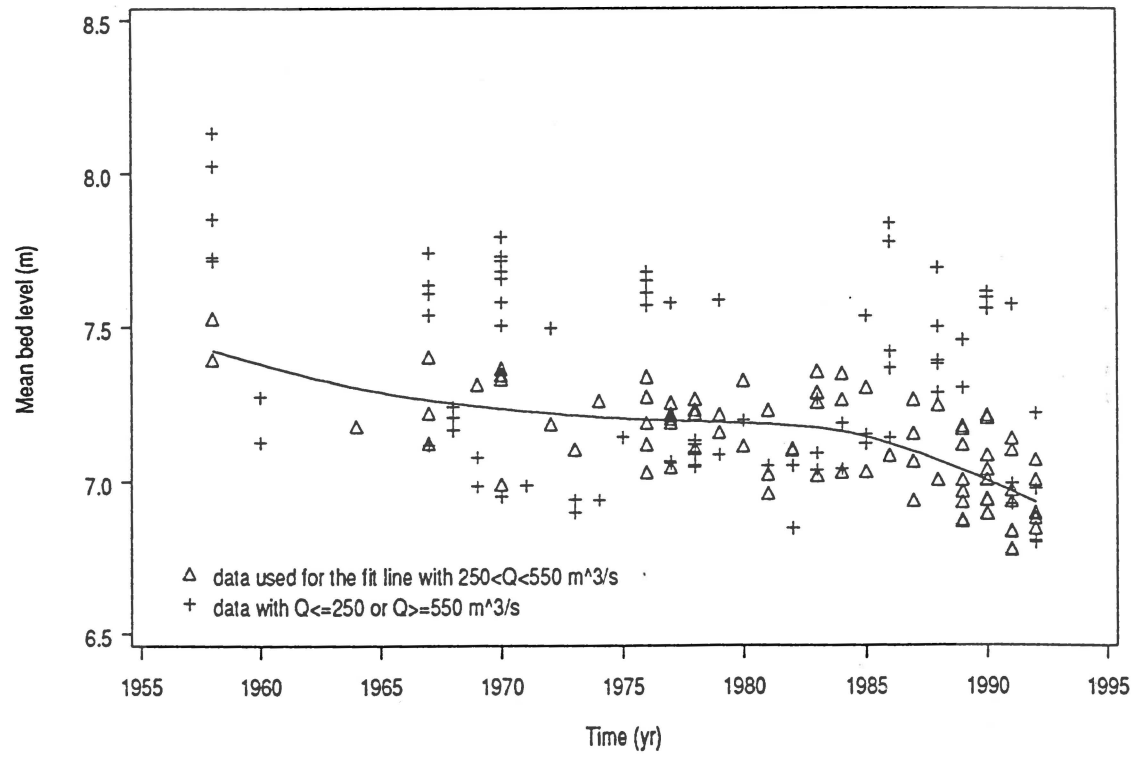


Figure 4.15 Variations of mean bed levels at the Ngaruawahia Cableway. Smoothed fit curve is obtained from 89 records using SAS with option I=SM65 (data from the gauging database at the Waikato Regional Council in 1958–1992).

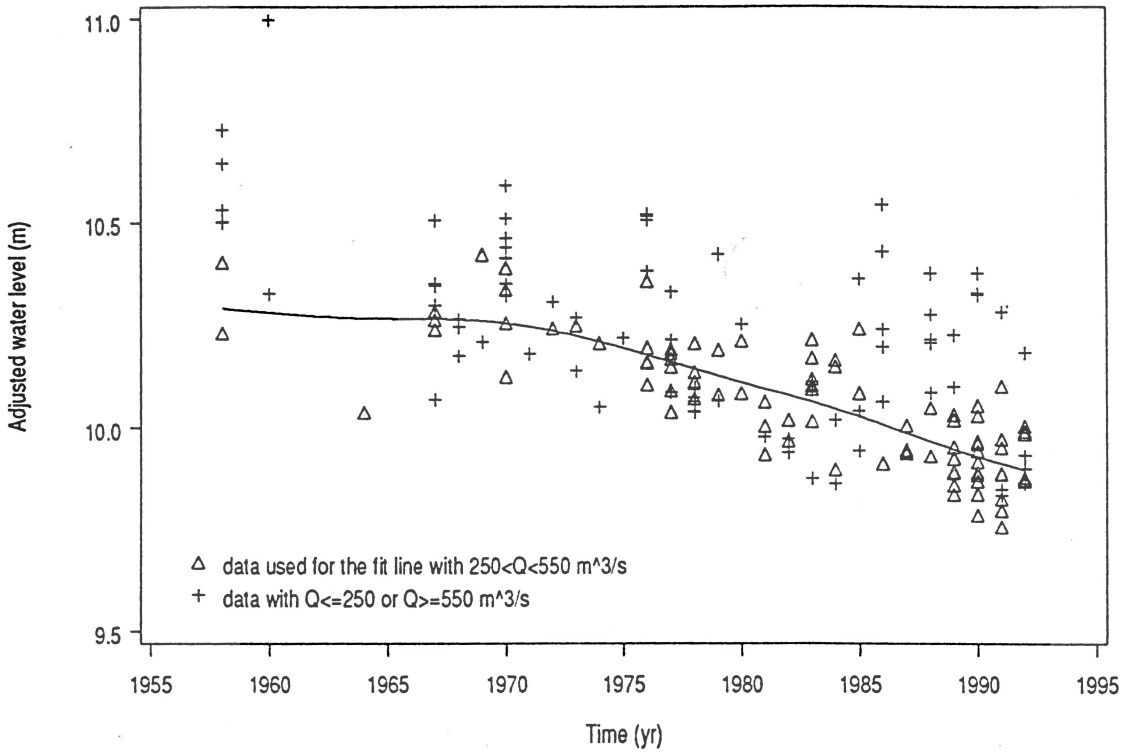


Figure 4.16 Variations of water levels adjusted to discharge 350 m^3/s at the Ngaruawahia Cableway. Smoothed fit curve is obtained from 89 records using SAS with option I=SM65 (data from the gauging database at the Waikato Regional Council in 1958–1992).

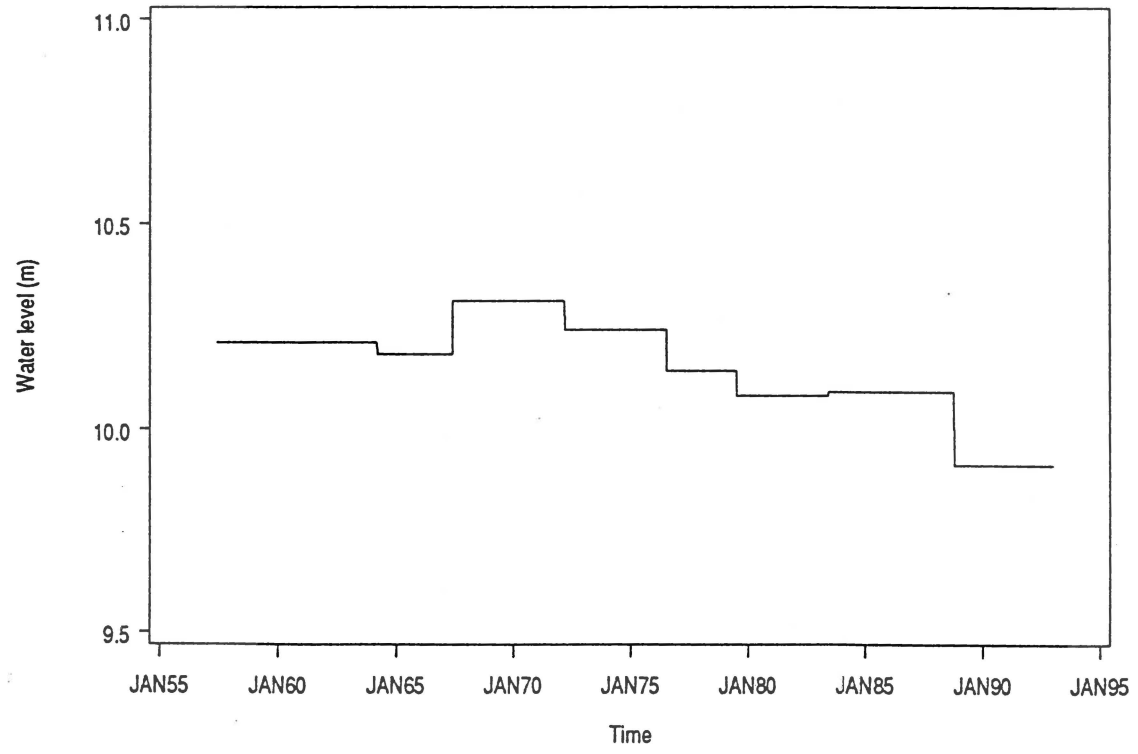


Figure 4.17 Variations of water level obtained from a total of 8 rating curves for discharge 350 m^3/s at the Ngaruawahia Cableway (data from TIDEDA database at the Waikato Regional Council in 1957–1992).

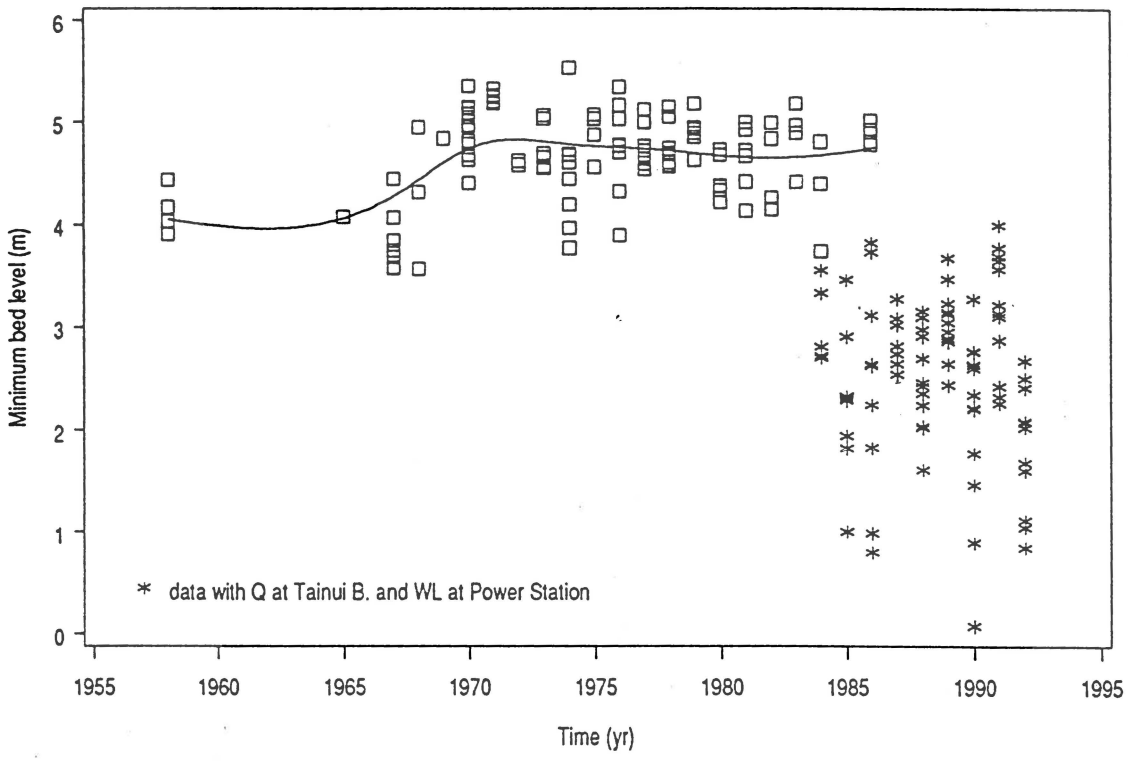


Figure 4.18 Variations of minimum bed levels at the Huntly Rail Bridge. Smoothed fit curve is obtained from 119 records in 1958–1986 using SAS with option I=SM65 (data from the gauging database at the Waikato Regional Council in 1958–1992).

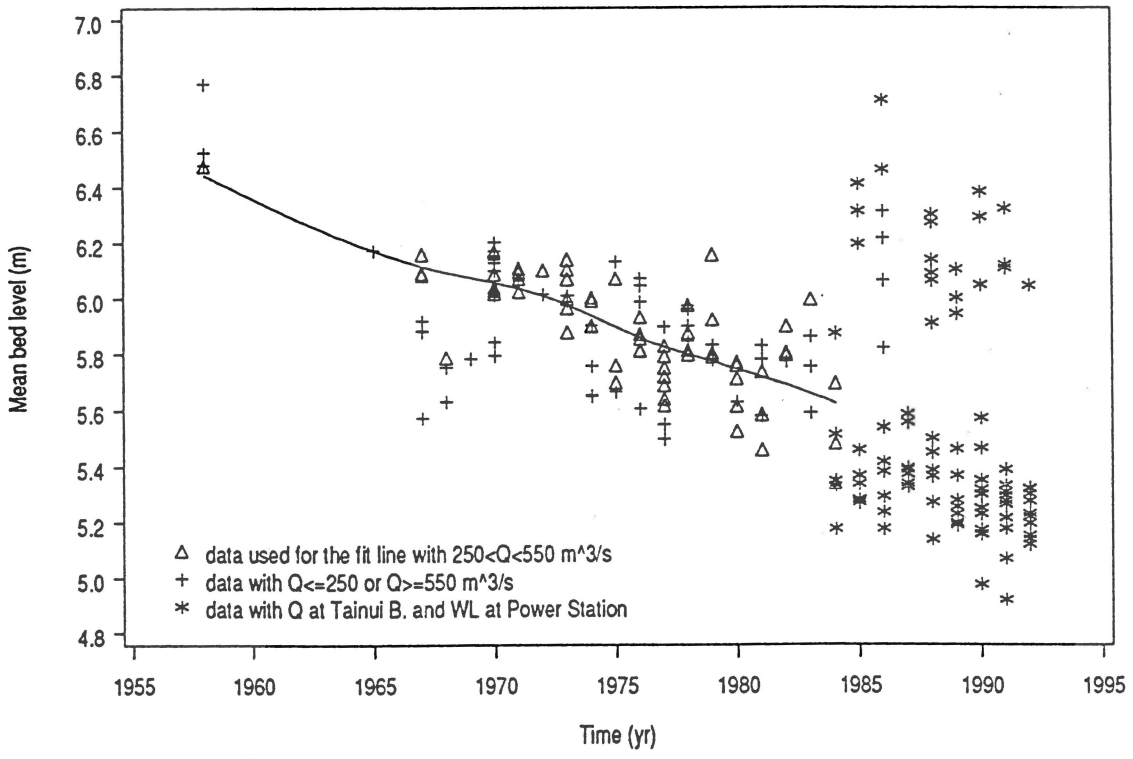


Figure 4.19 Variations of mean bed levels at the Huntly Rail Bridge. Smoothed fit curve is obtained from 63 records in 1958–1986 using SAS with option I=SM65 (data from the gauging database at the Waikato Regional Council in 1958–1992).

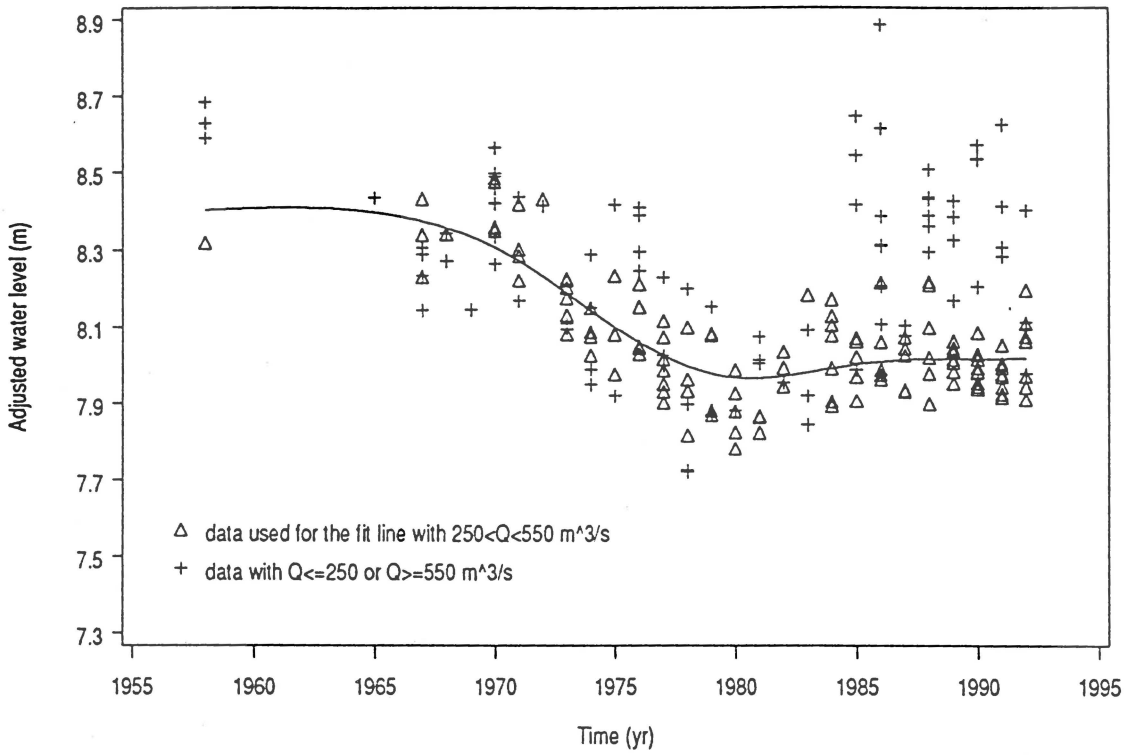


Figure 4.20 Variations of water levels adjusted to discharge 350 m^3/s at the Huntly Rail Bridge. Smoothed fit curve is obtained from 129 records using SAS with option I=SM65 (data from the gauging database at the Waikato Regional Council in 1958–1992).

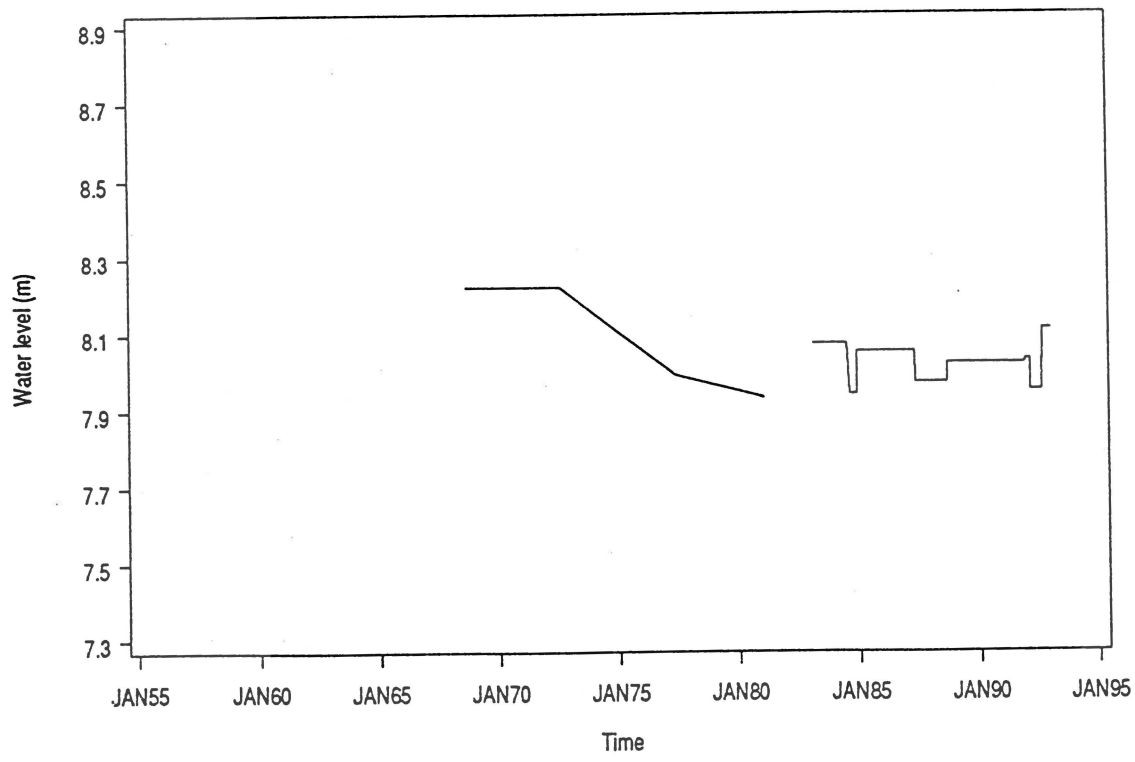


Figure 4.21 Variations of water level obtained from a total of 12 rating curves for discharge 350 m^3/s at the Huntly Rail Bridge (data from TIDEDA database at the Waikato Regional Council in 1968–1992).

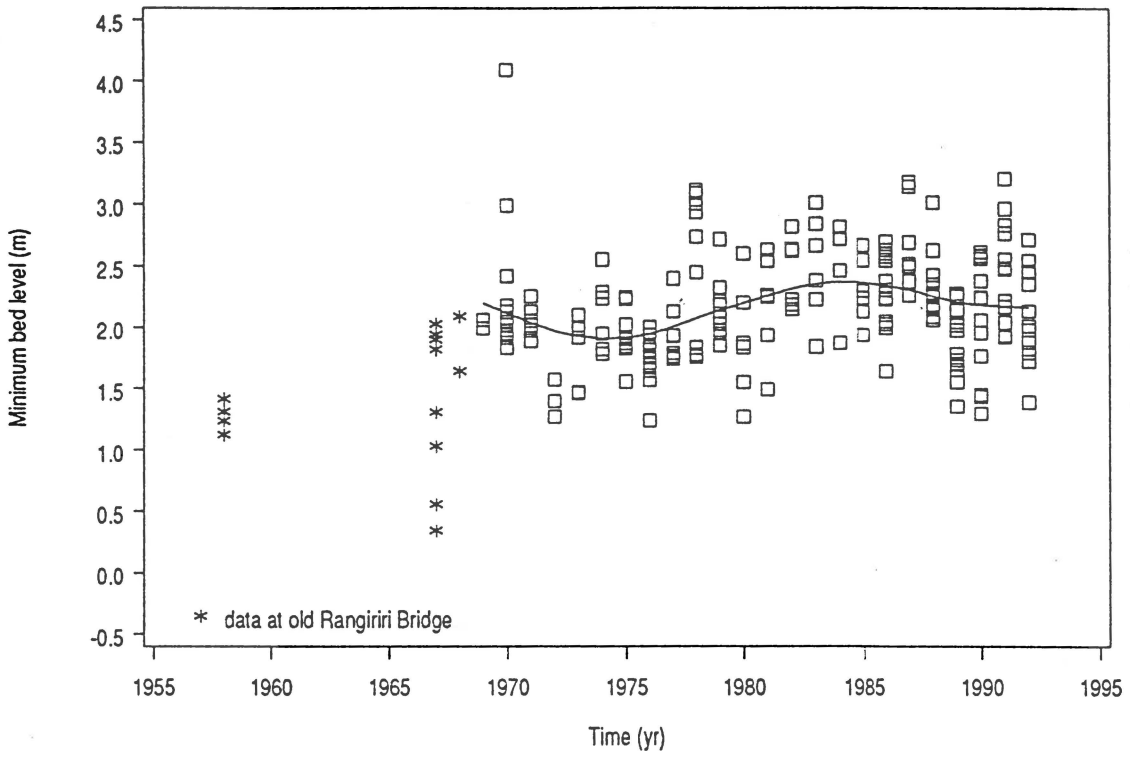


Figure 4.22 Variations of minimum bed levels at the Rangiriri Bridge. Smoothed fit curve is obtained from 202 records in 1969–1992 using SAS with option I=SM65 (data from the gauging database at the Waikato Regional Council in 1958–1992).

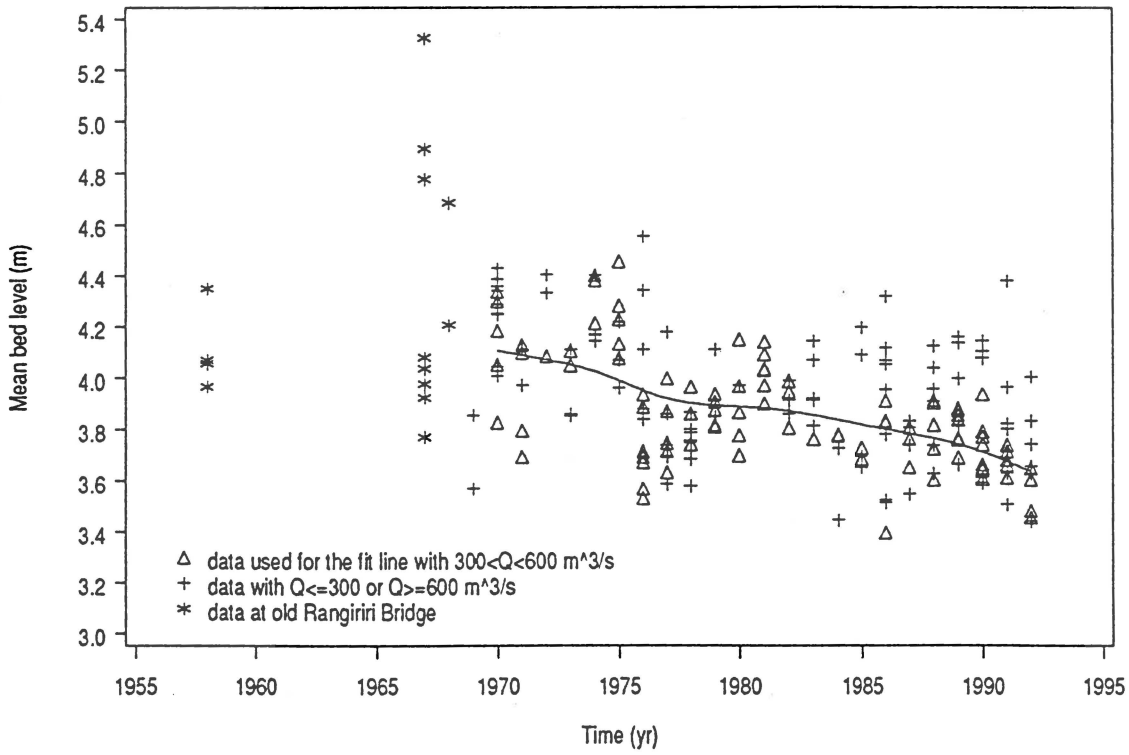


Figure 4.23 Variations of mean bed levels at the Rangiriri Bridge. Smoothed fit curve is obtained from 99 records in 1969–1992 using SAS with option I=SM65 (data from the gauging database at the Waikato Regional Council in 1958–1992).

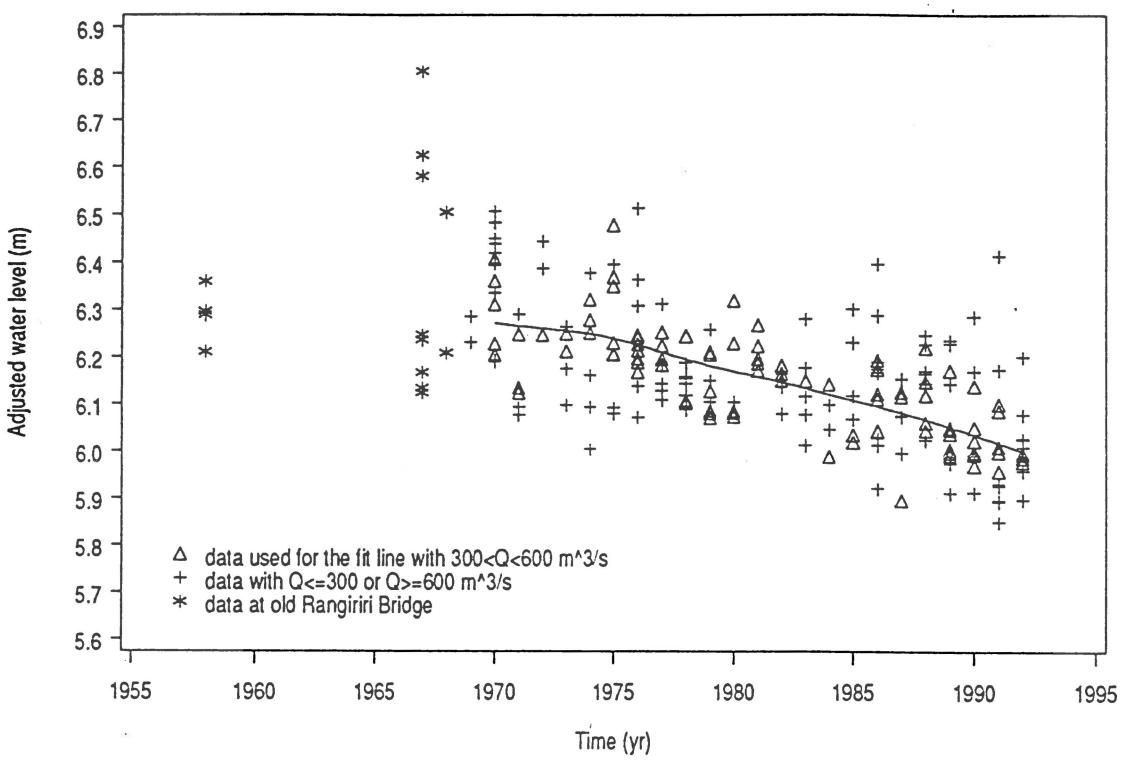


Figure 4.24 Variations of water levels adjusted to discharge 350 m^3/s at the Rangiriri Bridge. Smoothed fit curve is obtained from 99 records in 1969–1992 using SAS with option I=SM65 (data from the gauging database at the Waikato Regional Council in 1958–1992).

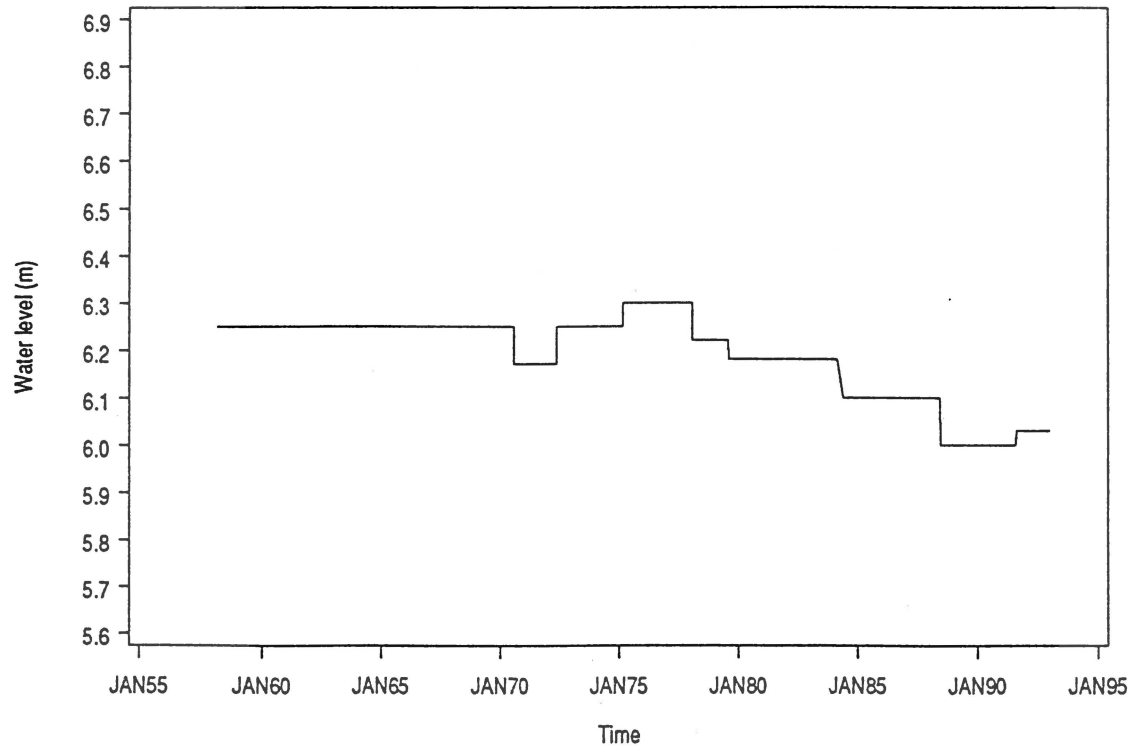


Figure 4.25 Variations of water level obtained from a total of 9 rating curves for discharge 350 m^3/s at the Huntly Rail Bridge (data from TIDEDA database at the Waikato Regional Council in 1958–1992).

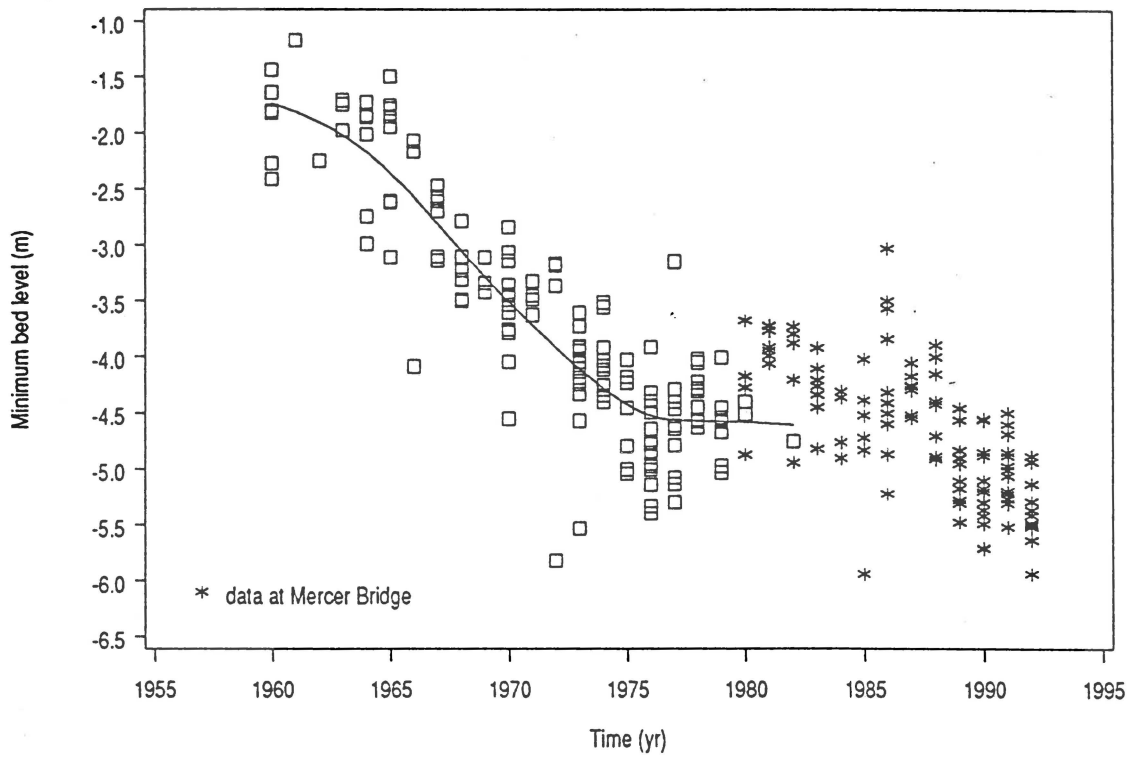


Figure 4.26 Variations of minimum bed levels at the Mercer Wharf. Smoothed fit curve is obtained from 140 records in 1960–1982 using SAS with option I=SM65 (data from the gauging database at the Waikato Regional Council in 1960–1992).

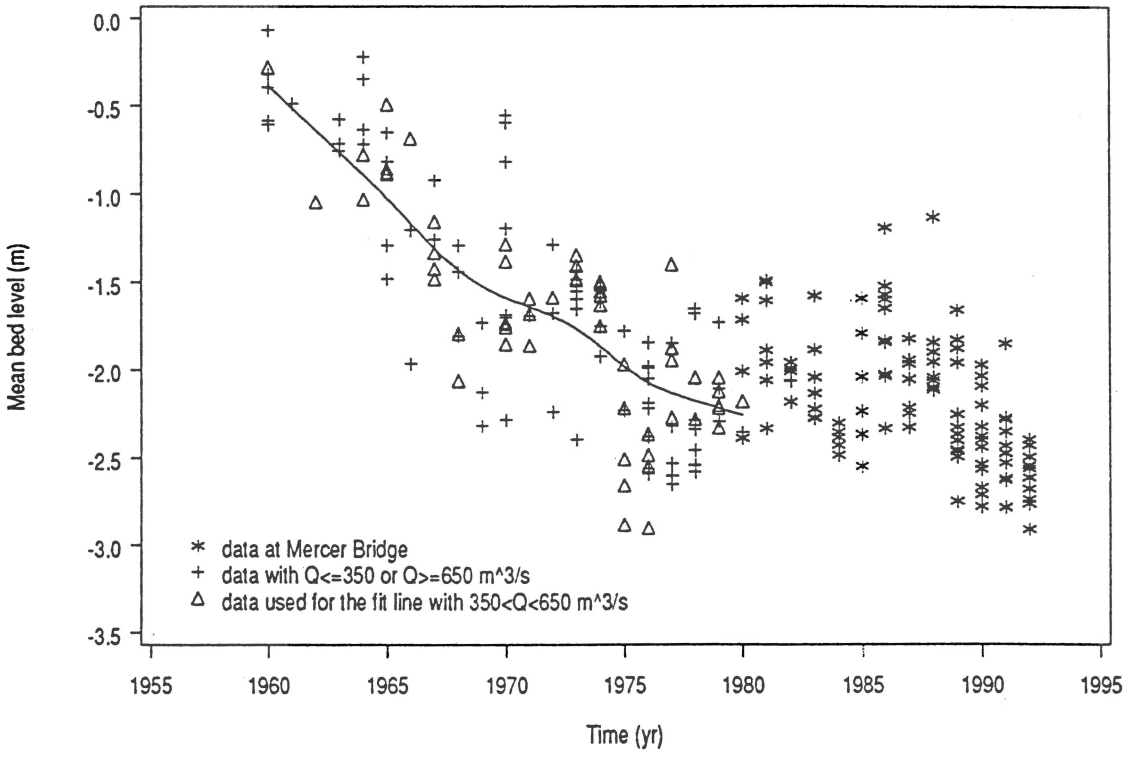


Figure 4.27 Variations of mean bed levels at the Mercer Wharf. Smoothed fit curve is obtained from 60 records in 1960–1982 using SAS with option I=SM65 (data from the gauging database at the Waikato Regional Council in 1960–1992).

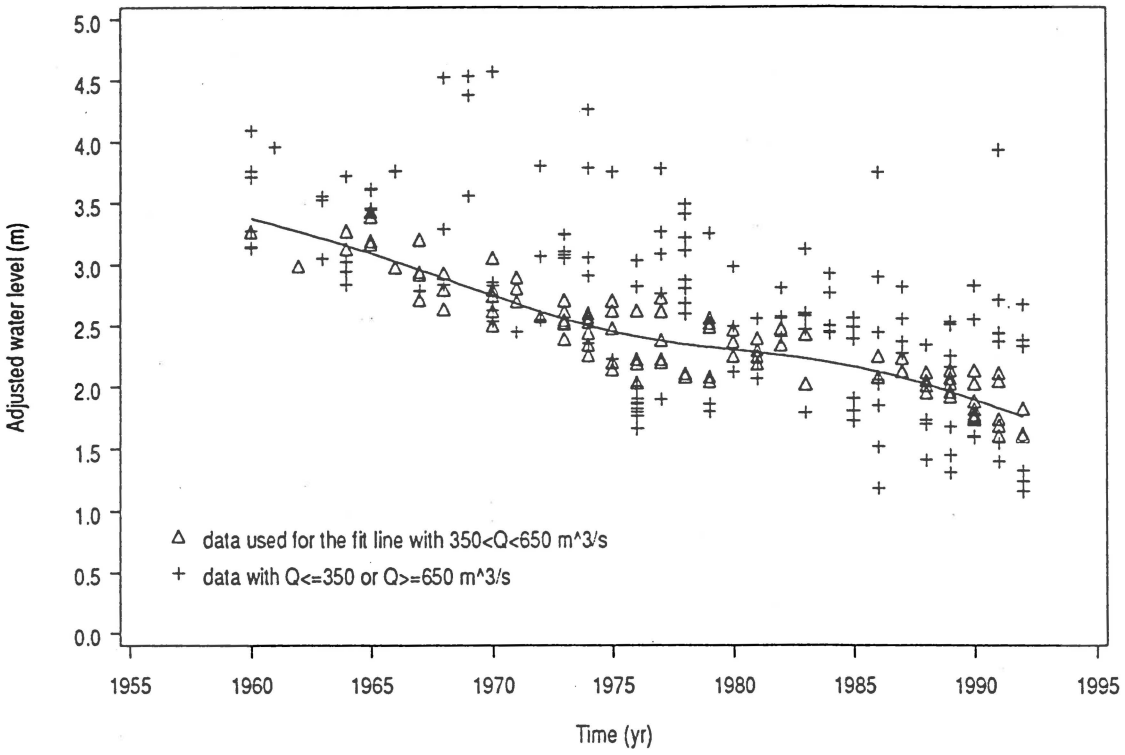


Figure 4.28 Variations of water levels adjusted to discharge 350 m^3/s at the Mercer Bridge. Smoothed fit curve is obtained from 101 records using SAS with option I=SM65 (data from the gauging database at the Waikato Regional Council in 1960–1992).

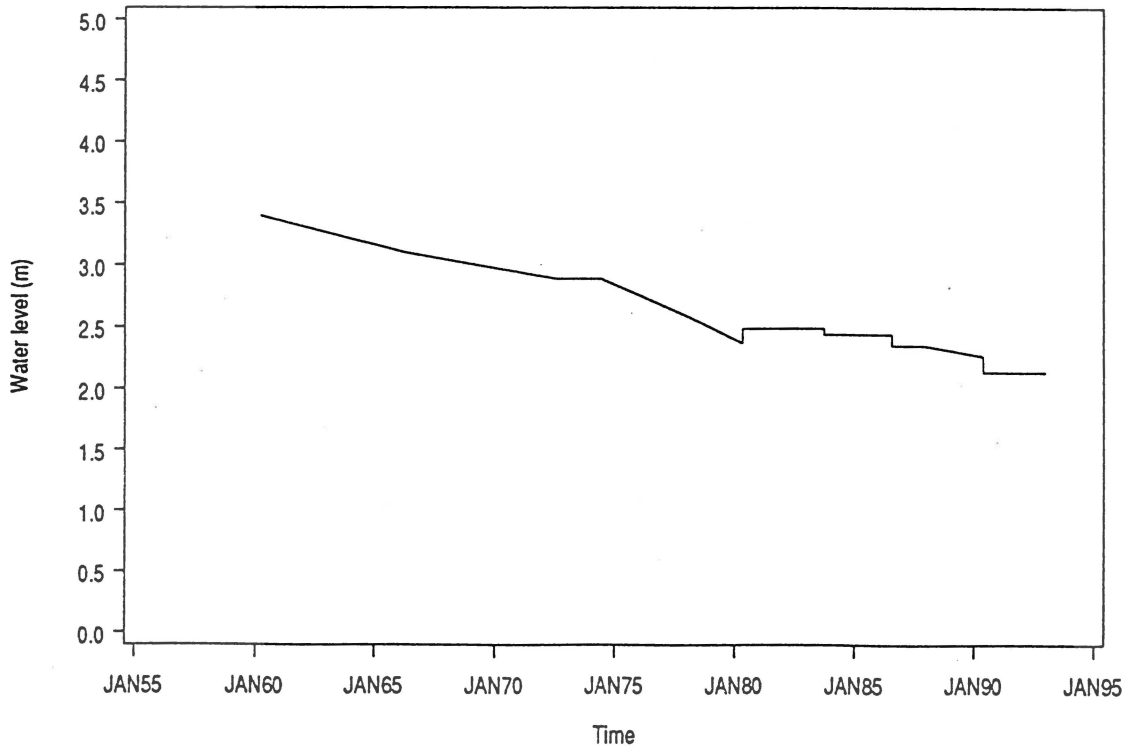


Figure 4.29 Variations of water level obtained from a total of 10 rating curves for discharge 350 m^3/s at the Mercer Bridge (data from TIDEDA database at the Waikato Regional Council in 1960–1992).

- Mercer from 1960–1992 (Figures 4.26–29):

  1. All variables show a marked dropping trend over time.
  2. Minimum bed levels of 1960–1982 and mean bed levels of 1960–1980 at the Mercer Wharf dropped by about 2.8 m and 2.2 m respectively.
  3. Adjusted water levels steadily dropped by about 2.4 m between 1960–1992 with accelerated dropping rate between 1960–1975, reducing between 1975–1985.
  4. There are similar eductions in the water level from rating curves and the outline of adjusted water levels.

4.4 Analysis of water level profiles at a discharge of 350 m^3/s

4.4.1 Water level profile measurements

Water level profiles in the reach between Ngaruawahia and Mercer were measured under a condition of river flows close to 350 m^3/s at Ngaruawahia and all the water levels along the river were measured within about 12 hours. Dates of the measurements and their daily mean water levels at Ngaruawahia Cableway are listed in Table 4.8.

Table 4.8 Dates of water level profiles measured and their daily mean water levels at the Ngaruawahia Cableway (data from the Waikato Regional Council).

D-M-Yr	WL at Ng.	D-M-Yr	WL at Ng.	D-M-Yr	WL at Ng.
25-06-1958	9.90	29-05-1974	10.17	24-05-1983	9.58
01-07-1963	9.97	11-10-1974	10.16	27-04-1987	9.80
01-12-1964	10.10	21-11-1974	9.96	24-05-1988	9.84
31-05-1967	9.97	07-11-1975	9.94	19-07-1988	10.25
03-09-1973	10.20	19-11-1976	10.07	17-08-1989	9.69
08-10-1973	9.84	12-10-1978	9.76	17-12-1990	9.88
09-11-1973	9.43	21-05-1982	9.73		

The original data for water level profile measurements at a discharge of 350m^3/s are held at the Waikato Regional Council and they have been attached in Appendix Seven. The original water levels at cross section 132 (the Ngaruawahia Cableway) are obtained from the TIDEDA database at the Waikato Regional Council using an average of the water levels at about 6:00 to 18:00 on that day when the profile was measured.

4.4.2 Determination of water level profiles at discharge 350 m^3/s

At a given time, the water level corresponding to a discharge of 350 m^3/s can be obtained from results of the adjusted water level analysis (4.3.4) at each gauging station. Differences between the water levels measured and obtained from the adjusted water level analysis results at each gauging station are linearly interpolated along the river between two immediate gauging stations. Water level profiles at the index flow of 350 m^3/s are then determined by adjusting their original water levels using the distributed differences. Because the flows were close to 350 m^3/s when the water profiles were measured, the linear interpolation of the differences is acceptable.

4.4.3 Water level profiles at discharge 350 m^3/s

The water level profile data at discharge 350 m^3/s, determined by adjusting the original measurements, are attached in Appendix Seven. Figure 4.30 shows the water level profiles of 25-Jun-1958, 3-Sep-1973, 21-May-1982 and 17-Dec-1990, in which water levels at 61.33 km (cross section 105) and at 75.34 km (cross section 109) in 1990 are omitted because they may have been subject to error. The water levels along the river significantly reduced with a big drop around Mercer in the period of 1958–1990 (Figure 4.30).

4.5 Discussion on possible factors influencing changes in water and bed levels in the lower Waikato River

It is very difficult to estimate the detailed downstream effects of the Arapuni Dam (closure in 1929) and the Karapiro Dam (closure in 1947) on changes in sediment characteristics and cross sections of the lower Waikato River there are the few data available relating to the river sediment characteristics and cross sectional measurements before and after the dam closures.

Experience in the US shows that dams normally trap the sediment supplied from their upper catchments. They also cause an imbalance between the sediment transporting capacity of flows and the sediment supply, leading to bed level degradation downstream, and development of meanders (Graf, 1971; Williams and Wolman, 1984). This imbalance usually also exists on upper gravel rivers in natural environments. As a result of selective erosion (Sutherland, 1987), bed armouring occurs (Livesey, 1963; Schumm, 1977) and the armoured bed hinders further river bed degradation. Although degrees of river sedimentological and morphological changes caused by the downstream effects of dams vary along the river in a given period (Williams

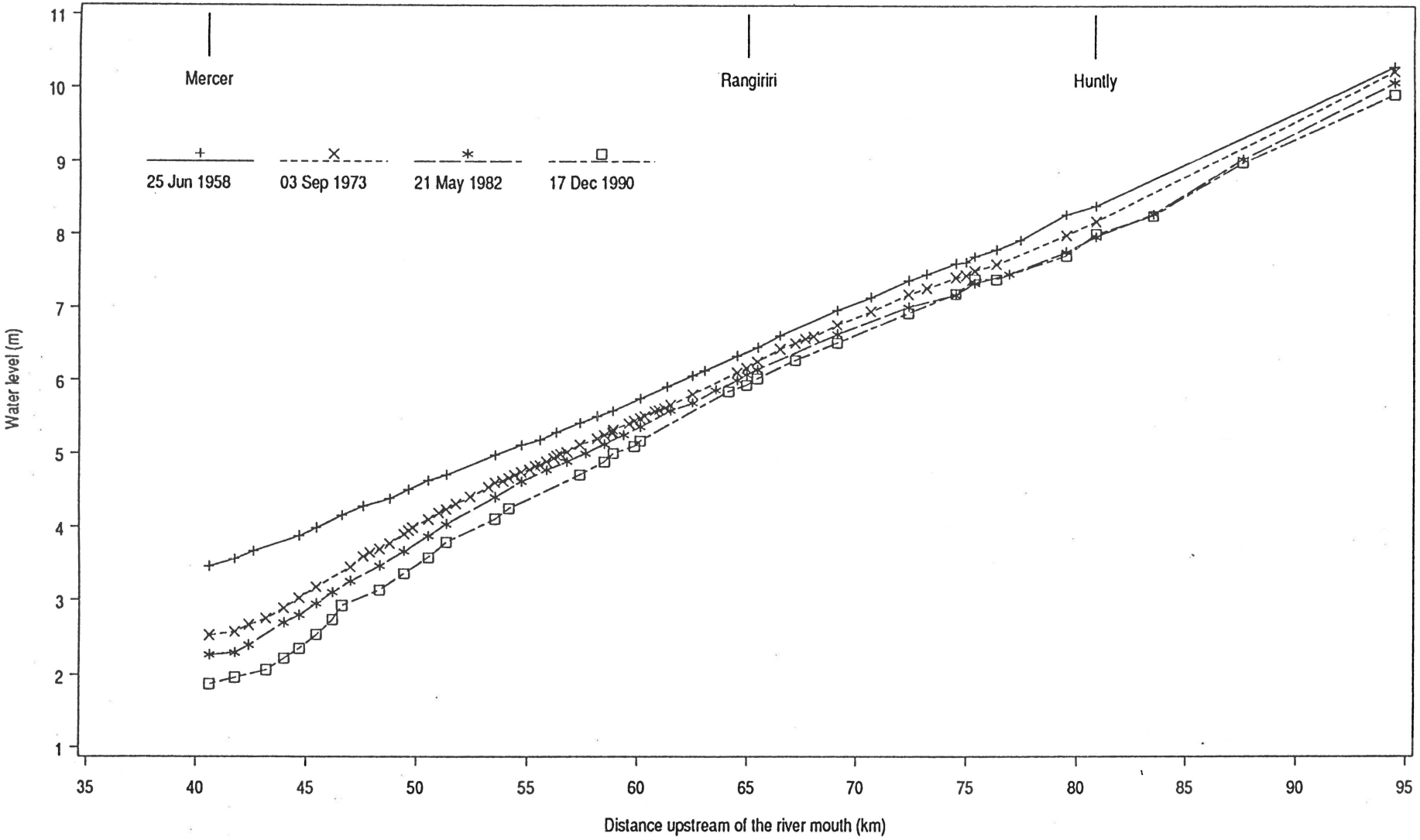


Figure 4.30 Water level profiles at discharge 350 m^3/s between Ngaruawahia and Mercer. Each profile is obtained by adjusting their original measurements using the results of adjusted water level analyses (data from the Waikato Regional Council in 1958–1990). Adjustments presumably small, because levels were measured at discharge around 350 m^3/s.

and Wolman, 1984, page 21, Figure 9), non-dimensional analysis demonstrates that the greatest amount of bed degradation occurs within the first 10 to 15 percent of the river adjustment period. Further, one-half of the predicted eventual maximum depth eroded occurs within the first few years after the start of degradation. A total of about 7 years is estimated by a model (Williams and Wolman, 1984, page 23, Figure 10-C; page 24, Figure 11).

For the lower Waikato River, a comparison of plots of cross sectional measurements held at the Waikato Regional Council shows that bed levels at cross section 159—the Narrows Bridge, 22.8 km downstream of the Karapiro Dam (Figure 4.31)—dropped by more than four metres between 1951–1963, but changes in bed levels from 1963–1988 were relatively small—less than half metre. A similar pattern of bed level reductions occurred at cross section 137—the Horotiu Bridge, 46.5 km downstream of the Karapiro Dam (Figure 4.31). Here bed levels were reduced by about three metres in the period of 1950–1963, and no significant bed level reductions occurred from 1963–1988, although there were some changes in its banks that might have partially resulted from dumping materials at the river bank. Both cross sectional widths are narrow compared with the other cross sections surveyed up and downstream, and the largest bed level reductions occurred within 16 years (1947–1963) after the Karapiro Dam closure (34 years after the Arapuni Dam closure between 1929–1963).

About the same period (1950–1963), there were no significant trends of bed level reductions at the other cross sections surveyed—cross sections 174, 151, 148, 133 (Figures 4.28, 4.31–32)—although there were some variations of their bed levels. For cross section 148, this conclusion can even be extended back to 1933, when the first survey was done. All the evidence demonstrates that river bed levels at these cross sections needed to make few adjustments for the upstream dam constructions during the period between 1950–1963 (Figure 4.31).

But bed levels of those cross sections surveyed downstream of cross section 159 (eg. cross sections 151, 148) show a significant degradation trend since 1963 from a comparison of the survey data (Figures 4.28 and 4.32). Most cross sections upstream of Ngaruawahia showed increases in their cross sectional areas in the survey period of 1974/1976–1985/1989 (Figure 4.6). For cross section 151, Table 4.9 shows variations in its sectional areas between distance 51–107 m and below the normal water level 13.84 m in 1963–1978 and the sectional area was increased by 66 m^2. An analysis of gauging data at

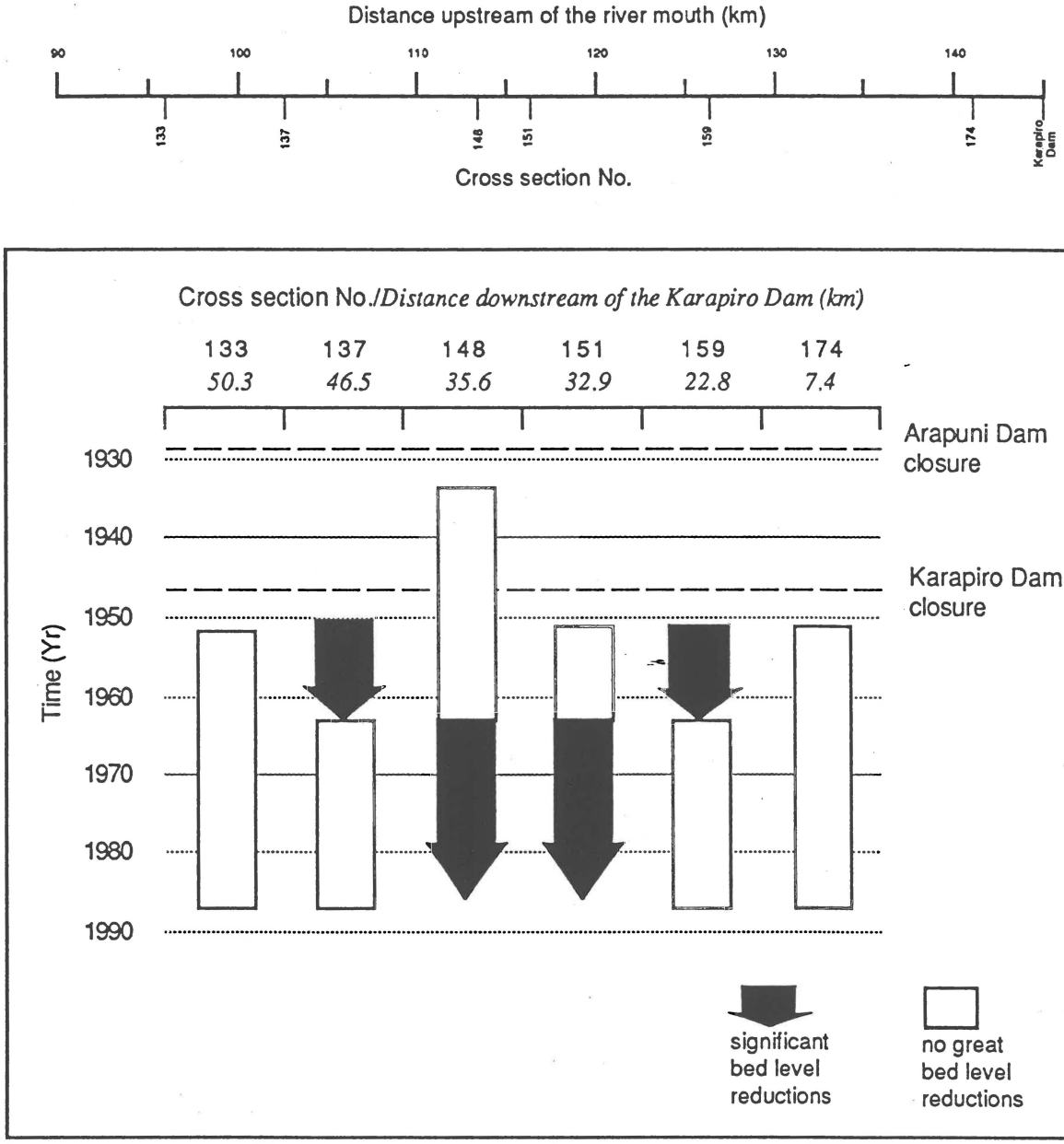


Figure 4.31 Sketch of changes in river bed levels at some cross sections upstream of Ngaruawahia from a comparison of survey data (data from the river surveys in 1933–1999 held at the Waikato Regional Council). The Arapuni Dam is located at 27.26 km upstream of the Karapiro Dam.

that cross section also shows a degradation trend of its bed levels (Figures 4.10–11).

Although the time when significant bed level degradation started at these cross sections was much later (16 years after the Karapiro Dam closure) than the beginning of degradation at cross sections 137 and 159 (Figure 4.31), it can still be explained by the downstream effects of dams (Williams and Wolman, 1984) because commencement of a degradation for each cross section downstream of dams may occur at different times.

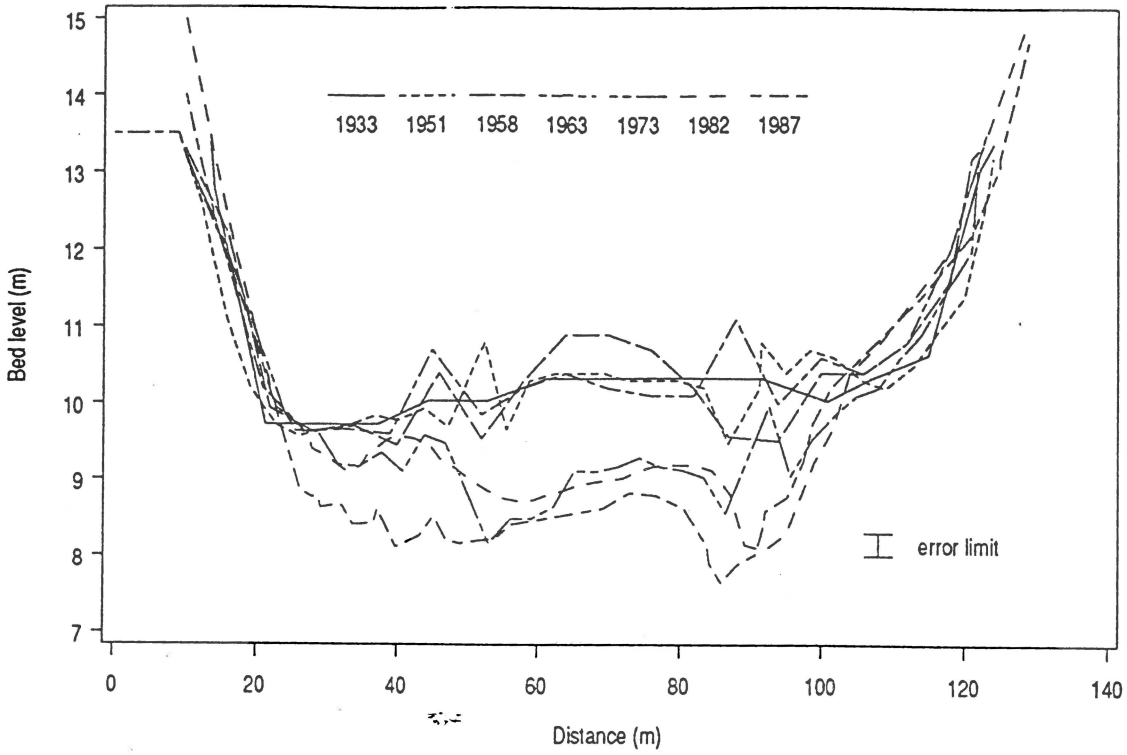


Figure 4.32 Cross sectional configuration at the Fairfield Bridge (cross section 148) from different river surveys (data from the river surveys held at the Waikato Regional Council). Zero of the distances of the 1933, 1982 and 1978 surveys has been adjusted.

Table 4.9 Variations in sectional areas between 51–107 m and below the normal water level 13.84 m at cross section 151, the Hamilton Traffic Bridge (data from the Waikato Regional Council).

Year	1963	1974	1980	1987
Area (m^2)	184	202	219	250

However commercial sand extraction undertaken from the river reach around Hamilton from the 1960s or before until 1976 (*New Zealand Herald*, 6-Apr-1976) could also be a major factor influencing the degradation. Extraction occurred downstream of cross section 159, which was shown on the licence of sand extraction held at the Waikato Regional Council (M. Tetslaff of W Stevenson & Co., pers. comm., 1991; R. Sang of the Hamilton City Council, pers. comm., 1991) Figure 4.33 shows the position of sand extraction licensed on the river in 1965, 1973, 1979–1994.

Bed materials in the main channel around Hamilton in the 1960s were a gravel or sand-gravel mixture, with a grain size diameter up to 50–60 mm (ref. Chapter Three). The river bed can be expected to have become partially

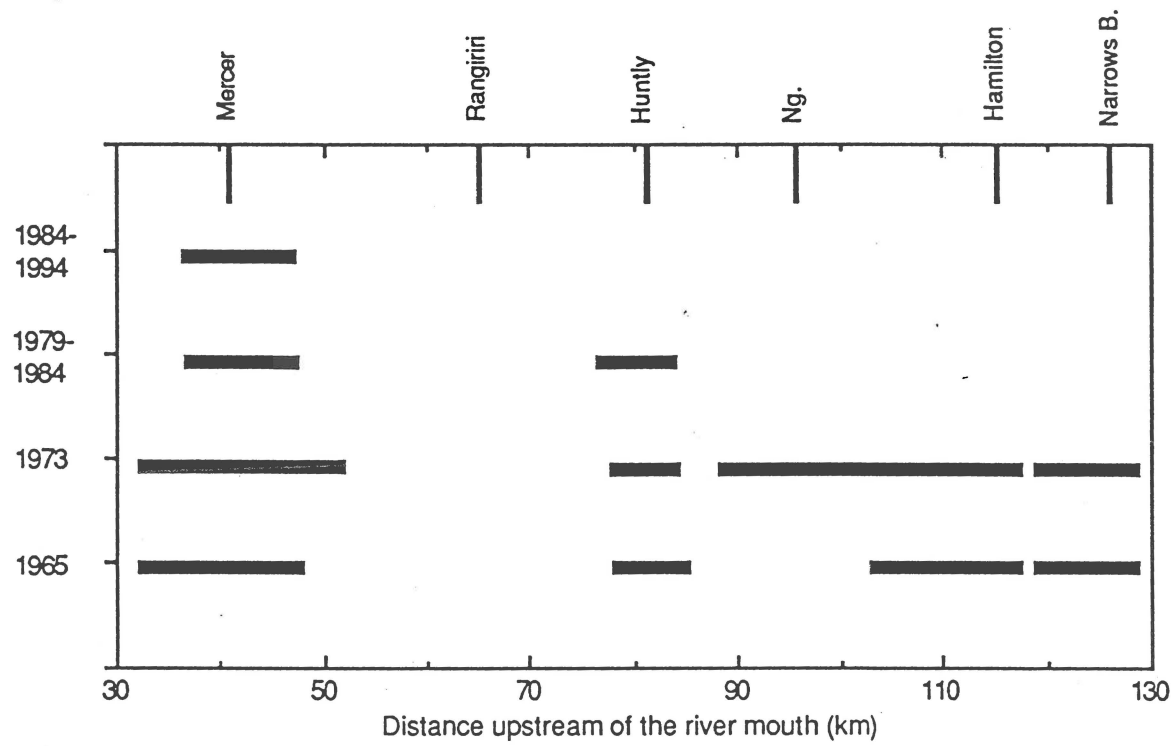


Figure 4.33 Position of commercial sand extraction licensed on the lower Waikato River in 1965, 1973 and 1979–1994 (data from files held at the Waikato Regional Council)

armoured at that time (16 years after the Karapiro Dam closure and 31 years after the Arapuni Dam closure) and more or less stable. The subsequent significant reductions of bed levels at cross sections 148 and 151 since about 1963 evident in the survey data, might result from the sand extraction in the Hamilton reach as well as bedload starvation following construction of the Karapiro Dam. The operation of sand extraction from the river disturbed the armoured or partially armoured bed surface layer, permitting sediments below the surface to move more easily, even after cessation of sand operations. But the degradation caused by disturbance of the surface layer can be expected to decline in the future because of river self-regulation and re-establishment of the armoured bed surface after cessation of the disturbance (Chorley and Kennedy, 1971; Parker and Sutherland, 1990).

Cross section 132 (the first cross section below the conjunction of the Waipa River, 94.4 km upstream from the river mouth) seems to be stable from a comparison of cross sectional plots between 1963–1988 (river survey data plots held at the Waikato Regional Council). But the detailed analysis of the gauging records in 1958–1992 at the Ngaruawahia Cableway—cross section 132—shows that there was a degradation in mean bed level and adjusted water level between 1958–1992, although the amount of degradation was small—the same order of the measurement errors (Figures 4.15–16). The

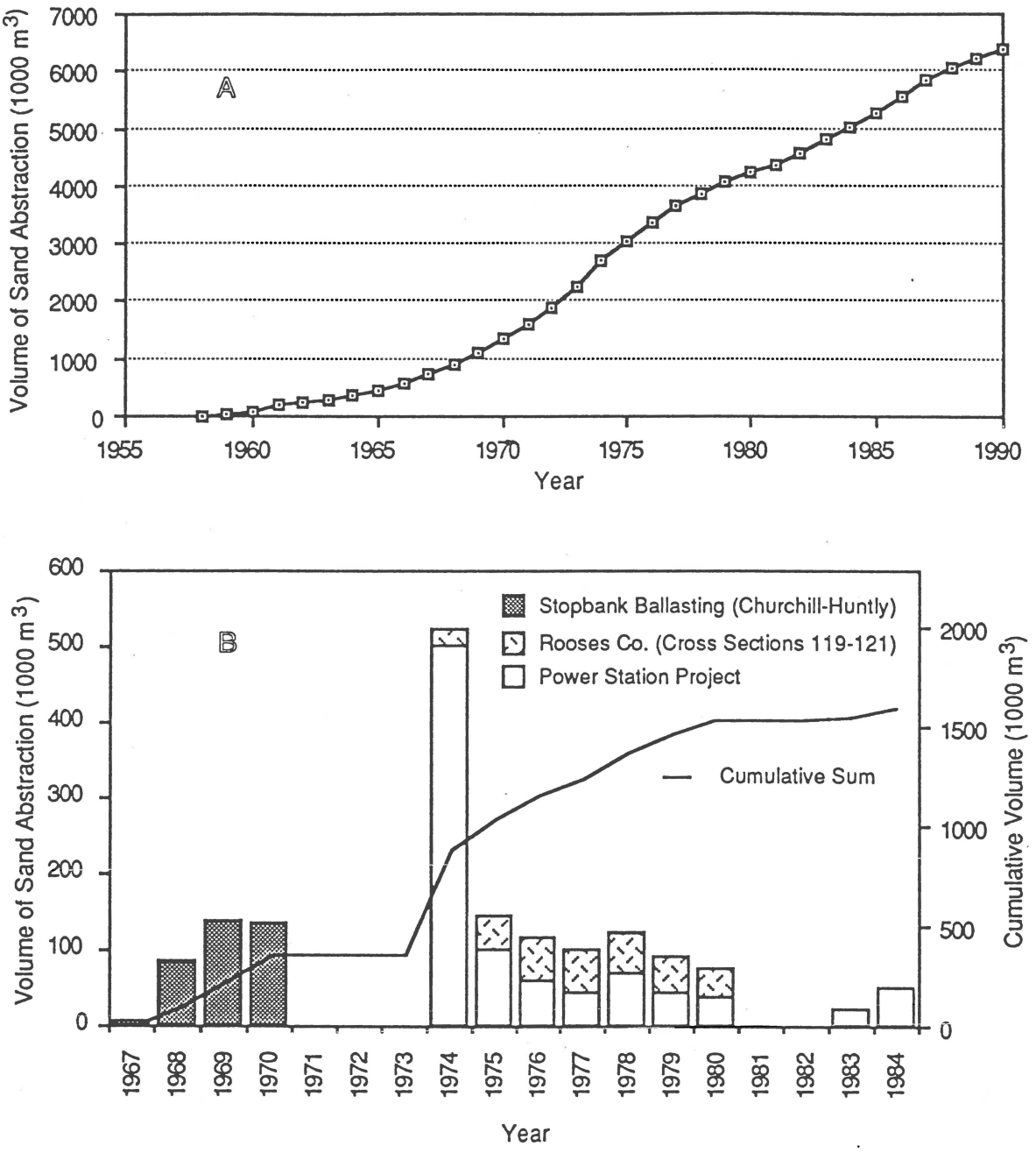


Figure 4.34 Sand extraction from lower parts of the Waikato River. A is the cumulative volume of sand taken around Mercer (data from Fenton, 1989; files at the Waikato Regional Council). B is the distribution of sand abstraction around Huntly and its cumulative volume (data from the Waikato Valley Authority, 1982; Fenton, 1989).

reductions probably result from the upstream effects (*cf.* Begin, *et al.*, 1981) caused by sand extraction around Huntly and Mercer (Figure 4.34), as well as reduction in sediment supply from its upper catchment because of the dam closures.

Drops in mean bed levels and water levels at 350 m^3/s at the Huntly Rail Bridge in 1970–1980 (Figures 4.19–20) were likely caused by the local sand extraction (Figure 4.34-B). The volume of sand extraction in 1974 for the

Huntly Power Station Project has been revised from 335 000 m^3 (Fenton, 1989) to 500 000 m^3, based on the figures of total sand extraction volumes in an internal report (Waikato Valley Authority, 1982) and a volume of 70 000 m^3 sand removed in 1978 (fill of Waahi Marae).

Generally, bed levels in the reach downstream of Ngaruawahia have been continuously dropping since about 1963/1964 with a more rapid reduction around Mercer and further downstream (Figure 4.3). These reductions may have directly influenced drops of water level profiles between Ngaruawahia and Mercer between 1958–1990 (Figure 4.30).

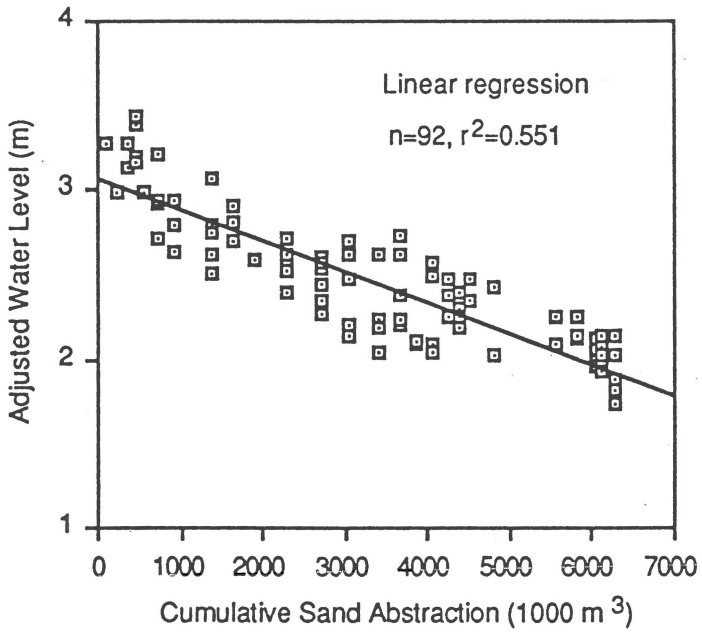


Figure 4.35-A
Relationship between cumulative sand extraction taken around Mercer and adjusted water levels at 350 m^3/s at the Mercer gauging station (data from the Waikato Regional Council of 1960–1990).

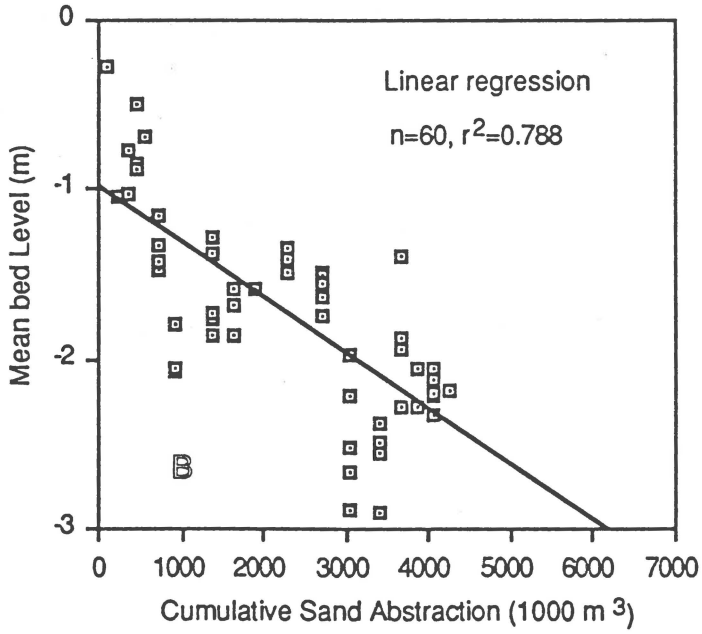


Figure 4.35-B
Relationship between cumulative sand extraction taken around Mercer and mean bed levels at Mercer Wharf (data from the Waikato Regional Council of 1960–1982).

The river training works (Mulholland, 1983), sand extraction from the river (Fenton, 1989), upstream effects from the major bed level drops around Mercer (*cf.* Begin, *et al.*, 1981; Howard, 1982), and decline of sediment supply from the upper catchment caused by construction of the dams (*cf.* Williams and Wolman, 1984) could be the reasons for the bed level reductions in the Waikato River downstream of Ngaruawahia.

The significant drops in bed levels around Mercer and further downstream may result from the long term operations of sand extraction at Mercer, Tuakau and Puni (Fenton, 1989). Figures 4.35-A–B show, respectively, the relationship of the cumulative sand extraction taken around Mercer and water levels adjusted to a 350 m^3/s discharge at the Mercer gauging station from 1960–1990 and mean bed levels at the Mercer Wharf from 1960–1982 using the gauging data with discharges between 350 and 650 m^3/s and a time step of one year.

4.6 Conclusions

Historical survey data and gauging records have been used to analyse changes in cross sectional area, bed level and water level of the Waikato River. Results have been used to determine the water level profiles at the discharge of 350 m^3/s in the reach between Mercer and Ngaruawahia. Possible factors influencing the changes have been evaluated. Because of the paucity of data available before the Arapuni Dam (1929) and the Karapiro Dam (1947) closures, it is difficult to analyse precisely the dam effects on the downstream river changes in bed levels and water levels, as well as sediment characteristics. Some conclusions have been drawn, viz.,

1. The total error of the bed level measurements using echo sounding in the Waikato River varied for different surveys and different cross sections. No information about hand survey errors was available. A total error at 95% confidence level of ±150 mm has been used for the entire echo sounding survey and hand survey data of the lower Waikato River in this study.

2. For the cross sectional measurements in the lower Waikato River, a total error at a 95% confidence level of ±28 m^2 has been assumed for the reach downstream of Ngaruawahia, and ±14 m^2 for the reach upstream as far as Cambridge.

3. Generally the mean bed levels of the river downstream of Ngaruawahia dropped between the surveys of 1963/1964–1985/1989, and that is shown in

their smoothed fit lines with a big drop around Mercer and further downstream.

4. Comparisons of cross sectional areas within the same reach show big increases around Puni, Tuakau and Mercer. The total net volume increase in the reach was estimated at about 6 527 000 m^3 between 1963/1964–1979/1983 and about 10 980 000 m^3 between 1963/1964–1985/1989.

5. Water level profiles at a discharge of 350 m^3/s between Mercer and Ngaruawahia have dropped over time, following the same pattern as the mean bed level changes.

6. Long term sand extractions around Puni, Tuakau and Mercer are the major factor causing the local bed and water level drops. The drops upstream of Mercer may result from river training works, sand extraction, upstream effects of the big drop around Mercer, decline and/or cessation of sediment supply from the upper catchment.

7. For the cross section at the Huntly Rail Bridge, there was a sudden drop in its mean bed levels and water levels for the 350 m^3/s index flow in the 1970s that resulted from local sand extraction for the power station construction. Variations of its water levels seem to become stable recently (1985–1992).

8. Minimum bed levels, mean bed levels and water levels adjusted to an index flow were obtained from gauging records. Their variations represent changes of cross sections. Assumptions for the adjustment of water level of each individual gauging record are that the river has a wide channel, and the same Manning's coefficient, same width and same energy slope at both the gauging discharges and the index flow.

9. Assuming a constant Manning's coefficient, constant width and constant energy slope, variations of adjusted water levels over time represent the mean bed level changes.

10. Plotted points of mean bed levels and adjusted water levels from the gauging data at Hamilton, Ngaruawahia, Huntly and Rangiriri, especially at Hamilton and Ngaruawahia, are grouped around a unit slope line. This suggests that variations of adjusted water levels represent the mean bed level changes in the lower Waikato River. The points from the data at Mercer varied from a unit slope line, possibly due to non-representative mean bed levels at the Mercer Wharf and/or a non-constant energy slope.

11. Bed levels at cross sections 159 (the Narrows Bridge, 22.8 km downstream

of the Karapiro Dam) dropped by about 4 m in 1951–1963, and became more or less stable between 1963–1988 from the surveys. At cross section 137 (the Horotiu Bridge, 46.5 km downstream of the Karapiro Dam), there were about 3 m reductions in bed levels from 1950–1963, and no significant changes afterwards. Both bed level reductions occurred within 16 years of the Karapiro Dam closure. These reductions are likely caused by downstream effects of dams (*cf.* Williams and Wolman, 1984).

12. Cross sections 148—the Fairfield Bridge, 151—the Hamilton Traffic Bridge—and 174—the Leamington Bridge (35.4 km, 32.9 km and 7.4 km downstream of the Karapiro Dam respectively) were more or less stable in the 1950–1963 surveys. For cross section 148, the stable period could be from 1933 when the first survey was undertaken.

13. Significant bed level reductions occurred at cross sections 148 and 151 (about 1.5–2 m) between 1963–1987 shown in the river survey data. These may have resulted from the upstream dam effects but they were also likely caused by operations of sand extraction from the river around Hamilton since the 1960s or until early 1976.

14. Cross sectional areas of most sections surveyed upstream of Ngaruawahia increased in the period of 1974/1976–1985/1989 and its net volume increase was estimated at 1 131 000 m^3 from the surveys.

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CHAPTER FIVE

BEDLOAD SEDIMENT BALANCE IN THE LOWER WAIKATO RIVER

Chapter Five

Bedload Sediment Balance in the Lower Waikato River

5.1 Introduction

A balance of bedload sediments in the lower Waikato River is determined by using results of the historical river survey analyses in section 4.2, and records of sand extraction taken from the river. The purpose is to estimate an average annual bedload sediment supply into the river downstream of Ngaruawahia from the historical records available. This information is necessary for river management, and for mathematical modelling. A total of bedload supply from both the Hamilton basin and the Waipa River is also determined. Assuming that bedload yield is proportional to catchment areas, and no long term volume changes in the Waipa River course, their individual bedload supply is approximated.

5.2 Data, methods and results

Volume changes in the lower Waikato River course during periods of two river surveys discussed in Chapter Four, and sand extraction volumes taken from the river listed by Fenton (1989) are used for an analysis of the bedload sediment balance in the Waikato River downstream of the Karapiro Dam.

Based on conservation of mass, for a particular river reach, Δx, in a period of $\Delta t = T_1 - T_0$, we have:

$$I \times \Delta t - O \times \Delta t = -\Delta V = V_{T_0} - V_{T_1} \qquad (5.1)$$

where I is an input rate of bedload materials into the reach Δx; O is an output rate of bedload materials from the reach Δx; ΔV is a total volume change in the river reach Δx during the period of Δt; and V_{T_0} and V_{T_1} are, respectively, the river course volumes at T_0 and T_1. If the river degrades, we have $\Delta V > 0$, otherwise $\Delta V < 0$.

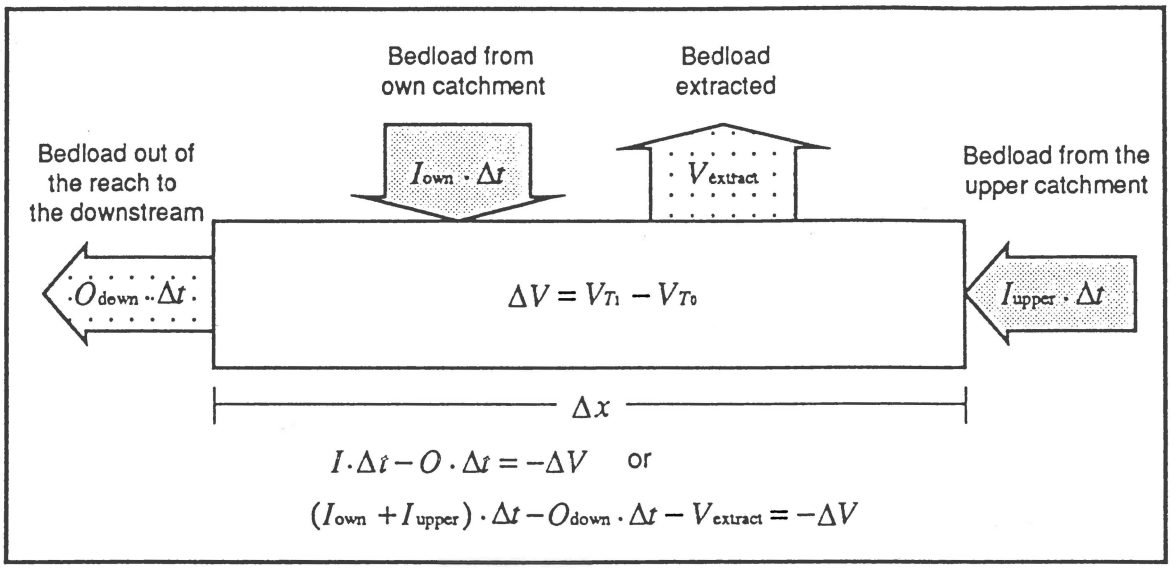


Figure 5.1 Sketch of the bedload sediment balance in a river reach Δx.

Given values for any two of the three variables I, O and ΔV in a certain period of Δt, the third one can be derived based on Eq. (5.1).

Variable I includes the bedload supply from its own subcatchment I_{own} of the reach concerned, and from the upstream river I_{upper}. Variable O includes rates of the bedload materials transported out of the reach to the river downstream or the sea O_{down}, and sand extraction taken from the river bed of the reach $O_{extract}$. Therefore Eq. (5.1) can be rewritten as

$$(I_{own} + I_{upper}) \times \Delta t - (O_{extract} + O_{down}) \times \Delta t = -\Delta V = V_{T0} - V_{T1} \tag{5.2}$$

Usually a certain amount of sand extracted during Δt is known, thus

$$V_{extract} = O_{extract} \times \Delta t \tag{5.3}$$

Then Eq. (5.2) becomes

$$(I_{own} + I_{upper}) \times \Delta t - O_{down} \times \Delta t - V_{extract} = -\Delta V = V_{T0} - V_{T1} \tag{5.4}$$

This equation is demonstrated in Figure 5.1.

In order to obtain the bedload supply from the upper catchment of cross section 132 at Ngaruawahia into the lower Waikato River, a balance of bedload sediments in the reach from the river mouth to Ngaruawahia has been studied.

The net volume increases in the river reach (ΔV) between the river surveys of 1963/1964 and 1979/83, and between the river surveys of 1963/64 and 1985/89

were, respectively, estimated at about 6 527 000 m^3 and 10 980 000 m^3 (see 4.2.2). Their difference—4 453 000 m^3—was the net volume increase in the period between the river surveys of 1979/83 and 1985/89.

The sand extraction volumes taken from the river downstream of Ngaruawahia can be obtained from Fenton (1989) and files held at the Waikato Regional Council. The figure of 335 000 m^3 listed by Fenton (1989) for the power project in 1974 has been revised to 500 000 m^3. A further total sand extraction volume of 70 000 m^3 was added in 1978 (see Chapter Four).

Only total sand extraction volumes taken by Roose Co. at its two operation sites of Mercer and Hamilton between 1964–1966 are available. Two-thirds of the total volumes have been assumed to be taken from Mercer, based on a comparison of the sand volume records in 1967–1975.

Commercial sand extraction volumes were measured as loose dry sand on land. Because of the difference between the bulk density for dry sand (1 122 kg/m^3) and wet sand (962 kg/m^3) (see Chapter Three), the commercial sand volumes have been multiplied by 1.166 before they are used for the analysis of the bedload sediment balance.

Because each river survey in the lower Waikato River lasted several years, e.g. the 1979/1983 survey lasted 5 years, it is difficult to determine the number of years between two surveys for the river volume changes. Therefore the minimum, mean, and maximum years of the period are considered in this study. For the 1963/1964 survey, year 1964 is taken because most cross sections were measured in that year.

Table 5.1 lists time periods (Δt), river volume changes (ΔV), and sand extraction volumes in the river reach from the river mouth to cross section 132 ($V_{extract}$). According to the bedload sediment balance—Eq. (5.4), given ΔV and $V_{extract}$, we have

$$I - O_{down} = (-\Delta V + V_{extract}) / \Delta t \qquad (5.5)$$

where $I = I_{own} + I_{upper}$. Values of $I - O_{down}$ are listed in the last column of Table 5.1.

If both the bedload materials transported out of the reach into the sea (O_{down}) and the bedload yield from its own subcatchment (I_{own}) are very small, we have

$$I - O_{down} = I_{upper} + I_{own} - O_{down} \approx I_{upper} \qquad (5.6)$$

Table 5.1 Bedload balance in the river reach downstream of cross section 132 at Ngaruawahia to the river mouth. ΔV is the river volume changes and $V_{extract}$, is the sand extraction volumes in a wet sand form (data from Fenton, 1989; and river surveys held at the Waikato Regional Council).

Period	Δt (yr)	ΔV (1 000 m^3)	$V_{extract}$ (m^3)	$I - O_{down}$ (m^3/yr)
1964–1979	16	6 527	8 902 841	148 000
1964–1981	18	6 527	9 679 231	175 000
1964–1983	20	6 527	10 661 533	207 000
1964–1985	22	10 980	11 768 086	36 000
1964–1987	24	10 980	13 035 189	87 000
1964–1989	26	10 980	13 869 436	111 000
1979–1985	7	4 453	3 403 370	–150 000
1979–1987	9	4 453	4 670 473	24 000
1979–1989	11	4 453	5 504 720	96 000
1981–1985	5	4 453	2 396 927	–411 000
1981–1987	7	4 453	3 664 030	–113 000
1981–1989	9	4 453	4 498 277	5 000
1983–1985	3	4 453	1 623 649	–943 000
1983–1987	5	4 453	2 890 751	–312 000
1983–1989	7	4 453	3 724 999	–104 000

Thus figures in the last column of Table 5.1 are the rate of bedload sediment supply from the upstream of cross section 132 at Ngaruawahia (I_{upper}). Generally they can also be considered as an average rate of bedload movement in the river reach concerned.

Possible factors influencing the differences can be the 1985/1989 survey, records of sand extraction, time periods selected for the river volume changes, and the coefficient for conversion between the dry and wet sand volumes. Some islands located upstream of Rangiriri were removed in 1987–1988 (W. M. Mulholland of the Waikato Regional Council, pers. comm., 1992) as river training works, but unfortunately no detailed information is available about the total sand volumes removed. Therefore the sand extraction volumes in the period of 1987–1988 used for the analysis in the reach concerned may be too small.

In order to circumvent the lack of information about sand volume removed for some river training works, the reach from the river mouth to cross section 101 near Rangiriri has been selected for an analysis of the bedload sediment balance. The results of $I - O_{down}$ are listed in Table 5.2 using Eq. (5.5).

Table 5.2 Bedload balance in the river reach downstream of cross section 101 near Rangiriri to the river mouth. ΔV represents the river volume changes, and $V_{extract}$ is the sand extraction volumes in a wet sand form (data from Fenton, 1989; and river surveys held at the Waikato Regional Council).

Period	Δt (yr)	ΔV (m^3)	$V_{extract}$ (m^3)	$I - O_{down}$ (m^3/yr)
1964–1979	16	5 560 000	7 762 106	138 000
1964–1981	18	5 560 000	8 457 548	161 000
1964–1983	20	5 560 000	9 418 850	193 000
1964–1985	22	8 272 000	10 475 403	100 000
1964–1987	24	8 272 000	11 742 506	145 000
1964–1989	26	8 272 000	12 576 753	166 000
1979–1985	7	2 712 000	3 153 551	63 000
1979–1987	9	2 712 000	4 420 653	190 000
1979–1989	11	2 712 000	5 254 901	231 000
1981–1985	5	2 712 000	2 325 778	–77 000
1981–1987	7	2 712 000	3 592 881	126 000
1981–1989	9	2 712 000	4 427 128	191 000
1983–1985	3	2 712 000	1 552 649	–386 000
1983–1987	5	2 712 000	2 819 752	22 000
1983–1989	7	2 712 000	3 653 999	135 000

If these results can also be applied in the reach from cross section 101 to Ngaruawahia, figures of $I - O_{down}$ from the two reaches above can be used together for a statistical analysis. The time period has a significant influence on determination of the values of $I - O_{down}$ (Tables 5.1–2). Table 5.3 summaries the results of these two reaches for the minimum, mean, and maximum time periods. In the reach from the river mouth to Ngaruawahia, only the results for the surveys of 1963/1964–1979/1983 are selected. The result for the minimum period of 1983–1985, –386 000 m^3/yr, is omitted in the statistical calculation. The total average is estimated at about 160 000±24 000 m^3/yr (mean±standard deviation). It is very close to the rate of bedload movement of 170 000 m^3/yr measured in the lower Waikato River (Dahm, 1987; and Fenton, 1989). Their relative difference is –6% ((160 000–170 000)/170 000).

Table 5.3 A summary of $I - O_{down}$ (m^3/yr) for the minimum, mean, and maximum time periods. "S.D." represents a standard deviation.

Minimum Period		Mean Period		Maximum Period		Note on
Period	$I - O_{down}$	Period	$I - O_{down}$	Period	$I - O_{down}$	River Reach
1964–1979	148 000	1964–1981	175 000	1964–1983	207 000	Mouth to Ng
1964–1979	138 000	1964–1981	161 000	1964–1983	193 000	Mouth to Rangiriri
1964–1985	100 000	1964–1987	145 000	1964–1989	166 000	Mouth to Rangiriri
1983–1985	(–386 000)	1981–1987	126 000	1979–1989	231 000	Mouth to Rangiriri
Mean	129 000	Mean	152 000	Mean	199 000	160 000 (average)
S.D.	25 000	S.D.	21 000	S.D.	27 000	24 000 (average)

For the river reach between cross section 132 at Ngaruawahia and the Karapiro Dam, the same method for the bedload balance analysis can be used to determine a total of bedload supply from both the Hamilton basin and the Waipa River.

Between the river surveys of 1974/1976 and 1985/1989, the river volume increase in this reach was 1 503 000 m^3 (see 4.2.2). Table 5.4 lists the number of years in the possible minimum, mean, and maximum time periods (Δt), the river volume changes (ΔV), sand extraction volumes taken from the reach in the corresponding period ($V_{extract}$), and the amount of bedload transported out of the reach into the river downstream by using the results above—$O_{down} \times \Delta t = 160\,000 \times \Delta t$. The last column of I in Table 5.4 is obtained by the following equation based on Eq. (5.4)

$$I = (-\Delta V + O_{down} \times \Delta t + V_{abstract}) / \Delta t \qquad (5.7)$$

where I includes three parts (Figure 5.2)—bedload supplies from the Hamilton basin ($I_{Hamilton}$), from the Waipa River (I_{Waipa}), and from upstream of the Karapiro Dam (I_{upper}) which is believed to be equal to nil because of the dam closure.

$$I = I_{Hamilton} + I_{Waipa} + I_{upper} = I_{Hamilton} + I_{Waipa} \qquad (5.8)$$

Since the long time span for each river survey (1974–1976 or 1985–1989), values of I vary between 10 400 and 72 200 m^3/yr (Table 5.4). Using 1975 and 1987 to represent the survey periods of 1974/1976 and 1985/1989 respectively, we have I = 47 600 m^3/yr (Table 5.4, Figure 5.2).

Table 5.4 Bedload sediment balance in the reach from cross section 132 at Ngaruawahia to the Karapiro Dam (data from Fenton, 1989; and the Waikato Regional Council). "S.D." represents a standard deviation.

Period	Δt (yr)	ΔV (m^3)	$V_{extract}$ (m^3)	$O_{down} \times \Delta t$ (m^3)	I (m^3/yr)
1974–1985	12	1 503 000	97 988	1 920 000	42 900
1974–1987	14	1 503 000	97 988	2 240 000	59 600
1974–1989	16	1 503 000	97 988	2 560 000	72 200
1975–1985	11	1 503 000	42 213	1 760 000	27 200
1975–1987	13	1 503 000	42 213	2 080 000	47 600
1975–1989	15	1 503 000	42 213	2 400 000	62 600
1976–1985	10	1 503 000	6 823	1 600 000	10 400
1976–1987	12	1 503 000	6 823	1 920 000	35 300
1976–1989	14	1 503 000	6 823	2 240 000	53 100
					Mean 45 700 S.D. 19 200

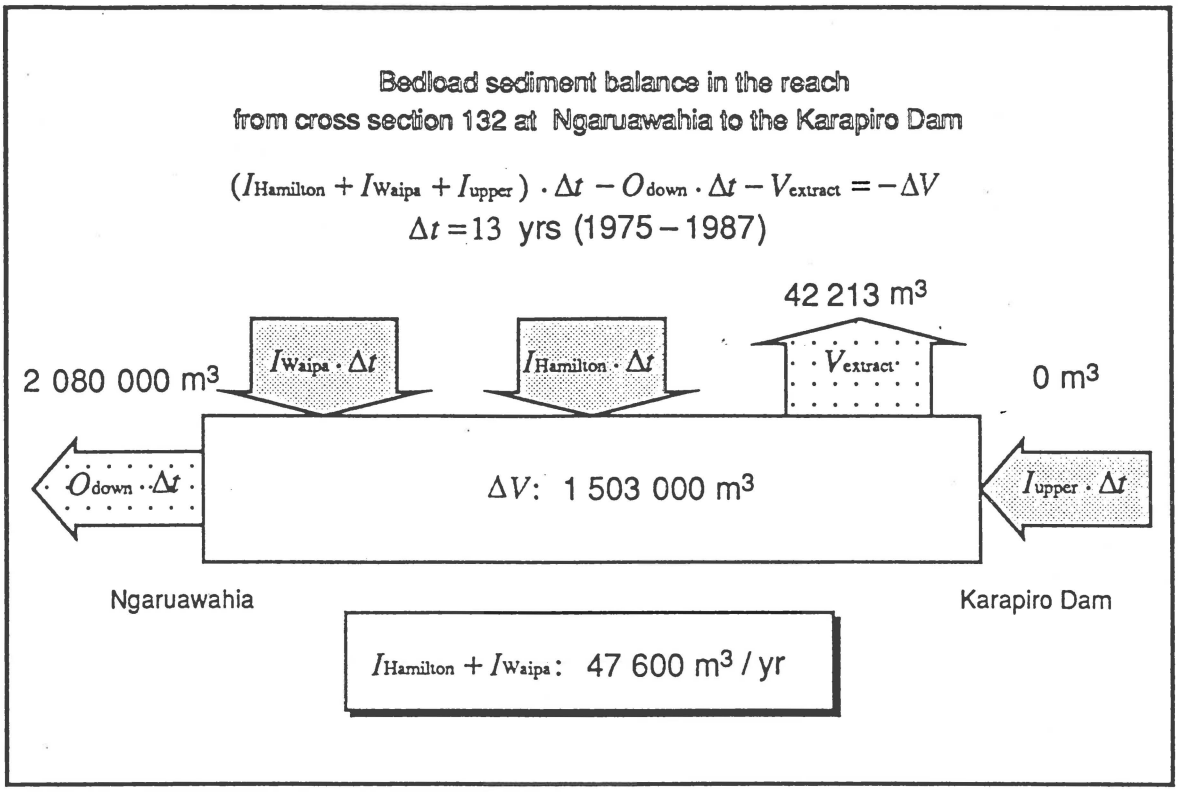


Figure 5.2 Bedload sediment balance in the river reach from cross section 132 at Ngaruawahia to the Karapiro Dam for the period of 1975–1987 by using O_{down}= 160 000 m^3/yr, and results of the 1974/1976–1985/1989 surveys.

The average of I for all the possible minimum, mean, and maximum time periods between river surveys of 1974/1976 and 1985/1989 is estimated at about 45 700±19 200 m^3/yr (mean±standard deviation; Table 5.4). Because of the uncertainty in O_{down} (160 000±24 000 m^3/yr, mean±standard deviation), which affects the results of I (Eq. (5.7)) directly, the standard deviation of I is expected to be bigger than 19 200 m^3/yr. Using the root-sum-square method, we have the standard deviation (S.D.) as

$$\text{S.D.} = \sqrt{19200^2 + 24000^2} \approx 30\,700 \ (\text{m}^3/\text{yr}) \qquad (5.9)$$

Thus I=45 700±30 700 m^3/yr (mean±standard deviation).

There are many factors influencing the amount of bedload yield, such as discharges, intensity and frequency of storms, catchment areas and slope, land use, forest coverage and others (Vanoni, 1975; Goldman, *et al.*, 1986). On the assumption that a bedload yield is linearly proportional to catchment areas and there have been no long term volume changes in the Waipa River course during the period concerned, the mean bedload supply from the Waipa River catchment (3 050 km^2) and from the Hamilton basin (629 km^2) are, respectively, 7 800 m^3/yr and 37 900 m^3/yr.

5.3 Discussions and conclusions

Based on volume changes in the river course obtained from the river surveys and records of sand extraction taken from the river, a balance of bedload materials in the reach of the Waikato River from the river mouth to cross section 132 at Ngaruawahia has been investigated.

No information is available about the amount of bedload transported into the sea since 1964 in the Waikato River. Locations of the sand mining operation at Puni, Tuakau and Mercer are close to the river mouth. A total of their annual sand extraction was over 200 000 m^3 since 1964 from the historical records. Because bedload movement in the lower Waikato River is believed to be about 170 000 m^3/yr (Dahm, 1987, Fenton, 1989), intuitively an assumption that no or very small amount of bedload was transported into the sea since 1964 is acceptable.

Accordingly, the difference between the bedload input and output into the sea in the reach concerned is the bedload sediment supply from both of its own subcatchment and the river upstream.

The subcatchment downstream of Ngaruawahia is much flatter, featured by lakes, ponds and swamps, than the catchment upstream. No information is available about the bedload yield from this subcatchment. If they are small and can be omitted, then the bedload input is the bedload supply from the catchment upstream of cross section 132. Generally this can also be considered as an average of bedload movement in the river reach concerned.

Because each river survey lasted several years in the lower Waikato River, it is difficult to determine the number of years between two surveys from which river volume changes were obtained. Since the time period selected has a significant influence on the analysis of the bedload balance, the minimum, mean, and maximum years of the period were considered in this study.

On average, the difference between bedload sediment input and output into the sea in the reach between the river mouth to cross section 132 at Ngaruawahia was estimated at about 160 000±24 000 m^3/yr (mean±standard deviation; Table 5.3) by using the results for the minimum, mean, and maximum time period. The average is very close to the annual rate of bedload movement in the lower Waikato River—170 000 m^3/yr (Dahm, 1987, Fenton, 1989).

By using results of the river surveys of 1974/1976 and 1985/1989, and the bedload output into the river downstream of 160 000 m^3/yr, the same method for the bedload balance analysis has been applied to the river reach from cross section 132 to the Karapiro Dam to obtain the bedload supply from both the Hamilton basin and the Waipa River. The bedload supply was estimated at about 45 700±30 700 m^3/yr (mean±standard deviation). The standard deviation here includes effects of the uncertainty in the bedload output into the river downstream.

If bedload sediment yield is proportional to catchment areas as well as assuming no long term changes in the Waipa River course during the period concerned, the individual bedload sediment supply from the Hamilton basin and from the Waipa River catchment was approximated at about 7 800 m^3/yr and 37 900 m^3/yr respectively.

It should be pointed out that for the average annual bedload supply to the river downstream of Ngaruawahia during the period between 1964–1989, 160 000 m^3/yr, nearly two-thirds was contributed by the bed materials stored in the upstream Waikato River course from the data in 1974–1989, probably resulting from bed surface disturbance by sand mining operation and downstream effects of the Karapiro Dam.

In the future, even if the bed surfaces upstream of Ngaruawahia in the Waikato River are totally armoured, an annual bedload sediment supply of about 45 700 m^3, yielded from catchment areas of the Hamilton basin and the Waipa River catchment, will still pass through Ngaruawahia to the lower river reaches under the condition of no great changes in land use. The reduction in bed level at the conjunction of the Waipa and Waikato rivers at Ngaruawahia may result in a certain amount of bed materials stored in the Waipa River course moving into the lower Waikato River. In order to know the quantitative effects, further research is required.

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CHAPTER SIX

ONE-DIMENSIONAL WATER AND BEDLOAD MOVEMENT SYSTEM

Chapter Six

One-dimensional Water and Bedload Movement System

6.1 Introduction

A one-dimensional river water and bedload movement system is discussed in this chapter with respect to its characteristics for application to the lower Waikato River. The governing equations consist of hydraulic equations (the St. Venant equations) and conservation of bedload, as well as a friction equation and a bedload transport equation. Under an assumption of a quasi-steady flow, linearization of the governing equations produces a hyperbolic equation, or a parabolic equation (uniform flow). Assumptions and limitations of these equations are discussed and the finite difference methods for their numerical solutions are presented.

6.2 Review of one-dimensional water and bedload movement models

Forecasting of river sediment loads is one of the most difficult tasks in river hydraulics. For a river catchment system, modelling of sediment yields should approximate mathematically the behaviour of upland catchment phase and lowland channel phase (Bennett, 1974). Studies on sediment yields from catchments have been discussed by Goldman, *et al.* (1986). The following discussion focuses on the lowland channel phase.

6.2.1 Definition of model and mathematical modelling

A model is a simplified representation of a complex system, involving similarity without identity (Singh, 1988). It is an abstraction and some system characteristics or natural processes are reproduced; others not.

There are many definitions for a system (Ackoff, 1962; de Neufville and Stafford, 1971). A system defined mathematically in a system-theoretics sense is a set of ordered pairs of inputs and outputs. Intuitively, a system can be considered as a collection of interacting components subject to inputs and producing outputs.

Models can be used not only to further understand natural processes, but also to make deterministic or probabilistic predictions given certain boundary and initial conditions (Woolhiser and Brakensiek, 1982). Model applications in river hydraulics were reviewed by de Vries (1973a) and a glossary of terms for hydraulic models was presented by ASCE (1982a).

Selection of an appropriate model depends largely on determination of the processes dominating the system. Therefore model development is an art, requiring judgement in abstracting dominant components from the real world, and judgement in expressing these components and relationships among them.

Models used in any discipline can be categorised as a material model and a formal model. A material model, including iconic and analog models, is a representation of the real original system by another system, which is assumed to be simpler and have similar properties. A formal model is a symbolic representation of an idealised simple situation that has structural properties of the original system. Symbolic models can be expressed variously (Lee, *et al.*, 1968). However symbolic models with a mathematical nature are used more widely following the development of mathematics and computers.

Mathematical models are an attempt to represent the real system by a set of equations expressing relationships of system variables and parameters based on physical laws or theories. They can sometimes be further subdivided into theoretical models, conceptual models and empirical models (Dooge, 1974); static models and dynamic models; deterministic models and stochastic models. Nevertheless these divisions are rather arbitrary. In practice nearly all the models used are hybrids (Odd, 1981).

An essential question raised is whether the theories can reasonably approximate reality and how we make a judgement. If they cannot, a new theory is required. The judgement is based on how well the model simulates the observed reality. Because of a proliferation of models, more and more studies on evaluations of model performances have been undertaken (Abraham, *et al.*, 1981; Ditmars, *et al.*, 1987).

Figure 6.1 indicates the principal components in mathematical modelling. Generally, the following four basic steps are included:

- determination of a well-posed problem based on theories which are thought to be able to describe the phenomena of interest (domain,

governing equations, boundary and initial conditions).

- selection of an algorithm to satisfy stability, convergence and consistency if no analytical solutions for the problem are available.
- calibration and validation of the model by using at least two different sets of observed data.
- application of the model.

The challenge is to keep inaccuracies within an acceptable limit. In practice, the most important step is to select an appropriate model (Fischer, *et al.*, 1979), and the most useful models for planning are often the simplest because they are easy to understand, to explain, and to use (Loucks, *et al.*, 1981).

6.2.2 Governing equations for a one-dimensional water and bedload movement system

One-dimensional mobile bed evolution models were first developed in the 1960s (de Vries, 1969; Thomas, 1979). Basic equations are based on conservation of sediment mass, water mass and water momentum (Jansen, *et al.*, 1979). The governing equations are given below by

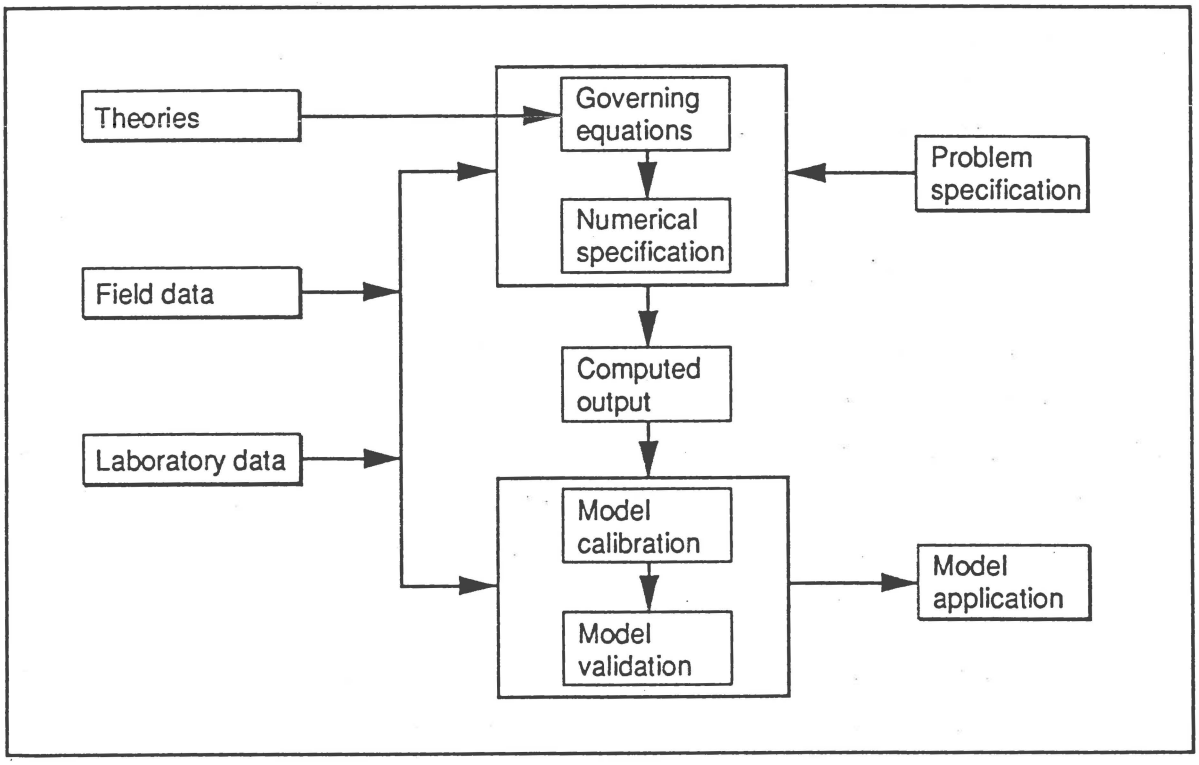


Figure 6.1 Principal components of mathematical modelling (after Thomann, 1982).

$$\frac{\partial A}{\partial t}+\frac{\partial Q}{\partial x}=q_l, \tag{6.1}$$

$$\frac{\partial Q}{\partial t}+\frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right)+gA\frac{\partial y}{\partial x}+gAS_f=u_l q_l, \tag{6.2}$$

$$W_B\frac{\partial z}{\partial t}+\frac{1}{1-p}\frac{\partial Q_B}{\partial x}=q_{Bs}. \tag{6.3}$$

In these equations (ref. Figure 6.2),

A: cross-sectional area;
t: time;
x: stream wise coordinate;
Q: water discharge;
q_l: lateral inflow per unit length of channel;
y: water surface elevation;
g: acceleration due to gravity;
S_f: slope of energy or friction;
u_l: velocity component of lateral inflow in the main flow direction;
W_B: width for an alluvial bed cross section;
z: bed level elevation;
p: bed sediment porosity;
Q_B: volumetric bedload transport rate;
q_{Bs}: source term of bedload inflows or outflows per unit length of channel;
A_B: change in an alluvial bed cross-sectional area;
W: cross-sectional width.

Eqs. (6.1–2) are the conventional St. Venant equations for one-dimensional unsteady channel flows (Liggett, 1975; Cunge, *et al.*, 1980). The significant departure from the normal fixed-bed context is that A and S_f depend on the variable thalweg elevation (bed level). Rahuel, *et al.* (1989) included a term, $+p(\partial A_B/\partial t)$, on the left hand side of Eq. (6.1) to take into account effects of bedload movement on cross-sectional changes, and Correia, *et al.* (1992) indicated that this term might be important under certain conditions, such as a long term evolution of the river.

Eq. (6.3) holds only if the quantity of suspended load can be neglected. Simons, *et al.* (1965) used a similar equation to study bedload transport.

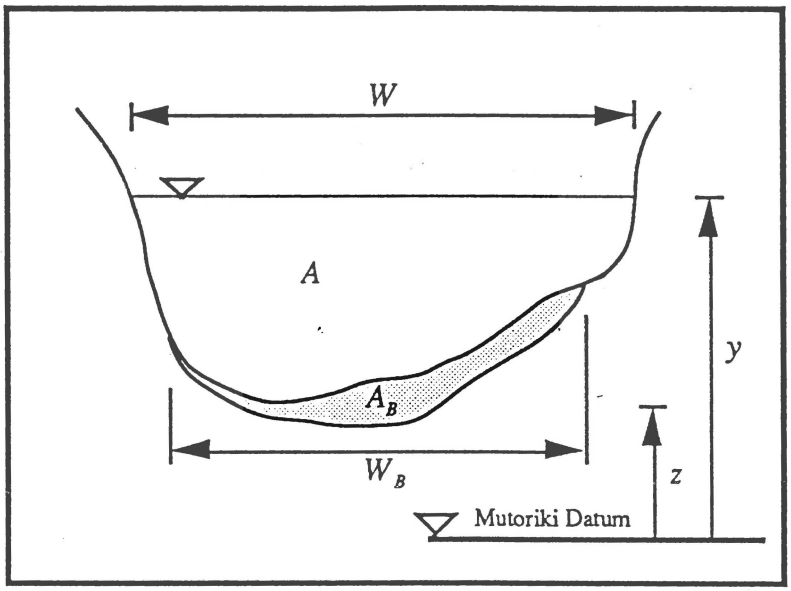


Figure 6.2 Sketch of definitions for a movable bed cross section.

Certainly the following fundamental assumptions underlying development of the St. Venant equations are invoked (Yevjevich, 1975):

> *"(1) The wave surface gradually varies, which is equivalent to stating that the pressure distribution along a vertical is hydrostatic, or that the vertical acceleration is small;*
>
> *(2) Friction losses in unsteady flow are not significantly different from those in the steady flow;*
>
> *(3) Velocity distribution across the wetted area does not substantially affect the wave propagation;*
>
> *(4) The wave movement can be considered as two-dimensional, with the effects of eventual difference of levels in cross sections negligible; and*
>
> *(5) The average slope of the channel bottom is so small that* $\sin\phi$ *may be replaced by* $\tan\phi$*, and* $\cos\phi$ *by unity, where* ϕ *is the angle made by the channel bottom with the horizontal."*

For the governing equations of water and bedload movements in a river channel, Eqs. (6.1–3), normally three boundary conditions are required for a particular physical problem. Two boundary conditions concerning water flows are the same ones used in a one-dimensional fixed bed flow model—the St. Venant equations (Cunge, *et al.*, 1980). For a subcritical flow, one of them is given at the downstream boundary and the other at the upstream boundary. For a supercritical flow, both boundary conditions are specified at the upstream boundary. The third boundary condition concerning bed level changes is imposed at the upstream boundary because the bed perturbations

travel in the downstream direction (Cunge and Perdreau, 1973; Chang and Hill, 1977).

For a wide rectangular channel, which implies that a unit channel width can be considered, Eqs. (6.1–3) can be rewritten as follows if effects of the lateral flows are omitted (ref. Figure 6.3):

$$\frac{\partial h}{\partial t}+u\frac{\partial h}{\partial x}+h\frac{\partial u}{\partial x}=0, \tag{6.4}$$

$$\frac{\partial u}{\partial t}+u\frac{\partial u}{\partial x}+g\frac{\partial h}{\partial x}+g\frac{\partial z}{\partial x}+gS_f=0, \tag{6.5}$$

$$\frac{\partial z}{\partial t}+\frac{1}{1-p}\frac{\partial q_B}{\partial x}=q_{Bs}, \tag{6.6}$$

in which,

h: depth of flow;
u: mean velocity;
z: bed elevation above a horizontal datum;
q_B: volumetric bedload transport rate per unit width;
q_{Bs}: source of bedloads per unit width and length.

It should be pointed out that the whole width considered for water and movable bed may be different (ref. Figure 6.2). Eqs. (6.4–6) will be discussed in general. Mathematically they comprise a quasi-linear, third-order hyperbolic system (Whitham, 1974). Given proper boundary and initial conditions, no analytical solutions for this system are obtained yet.

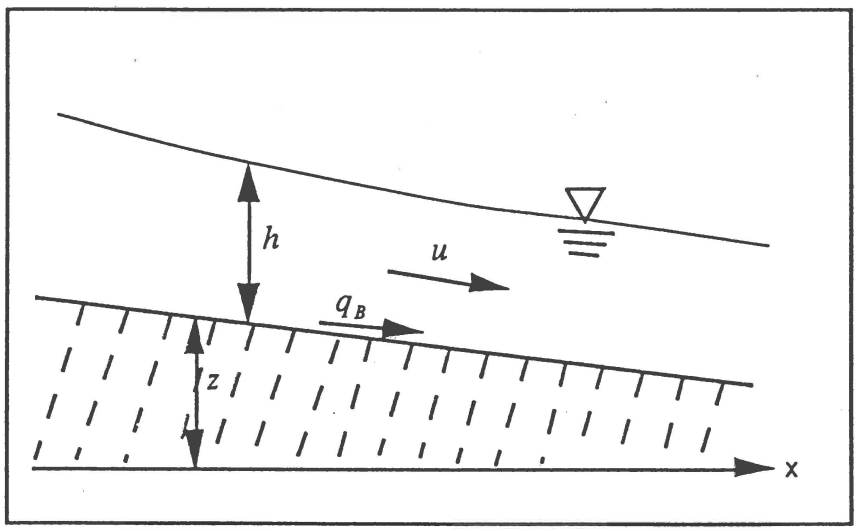


Figure 6.3 Sketch of definitions of the variables used in a mobile bed model with a unit width.

The unknowns are h, u, z, S_f and q_B. So at least two additional equations expressing S_f and q_B in terms of the other variables are required in order to solve the set of equations. At this stage they are usually taken to be an algebraic form—an empirical formula, like a state equation—based on experimental results rather than differential equations.

To pose a particular problem, initial and boundary conditions have to be specified. In practice the following conditions are generally given:

(1) Initially the dependent variables h, u and z are specified in the domain.

(2) At the upstream boundary, the water discharge, $q=uh$, and bedload supply, $q_{B,up}$, are given through time.

(3) At the downstream boundary, the stage-discharge relationship (rating curve) is given over time.

It should be pointed out that for the upper boundary condition concerning the bed level, the bedload supply is specified instead of the primary unknown z. Actually a straightforward upper boundary condition in terms of z is required (ref. Yang and Wang, 1990; Correia, *et al.* 1992). So in this case a relation between bed level and bedload supply at the upper boundary has to be known or assumed in order to give the boundary condition in terms of the bed level z or its derivative with respect to distance.

It is very difficult to determine whether the governing equations with specific conditions form a well-posed problem by mathematical analysis although a lot of work has been done in this area (Oliger and Sundström, 1978; Verboom, *et al.*, 1982). The following discussions are based mainly on physical meaning.

6.2.3 Characteristics of one-dimensional water and bedload movement model

Characteristics of the one-dimensional water and bedload movement model are discussed below by using characteristic velocities.

The Chézy-Manning equation is taken for calculation of S_f (Chow, 1959), and q_B is equal to bedload transport capacity of the flow $\tilde{q}_B$ (an equilibrium flow being normally assumed), which is proportional to a power of u:

$$S_f = \frac{u^2}{C^2 h} = \frac{n^2 u^2}{h^{4/3}}, \tag{6.7}$$

$$q_B = \tilde{q}_B = f(u) = au^b, \tag{6.8}$$

where C and n are Chézy's and Manning's coefficients respectively, and a and b are parameters containing information on sediment size, density, etc. and generally b is in the range of 2–8 (Jansen, *et al.*, 1979; Bell and Sutherland, 1983). Then the characteristic velocity c of Eqs. (6.4–6) should satisfy the following equation (Whitham, 1974):

$$\begin{vmatrix} u-c & g & g \\ h & u-c & 0 \\ f_u/(1-p) & 0 & -c \end{vmatrix} = 0, \tag{6.9a}$$

or:

$$-c^3 + 2uc^2 + \left(gh - u^2 + gf_u/(1-p)\right)c - ugf_u/(1-p) = 0 \tag{6.9b}$$

with $f_u = df/du$. Its dimensionless form is

$$-\phi^3 + 2\phi^2 + \left(Fr^{-2} - 1 + \psi Fr^{-2}\right)\phi - \psi Fr^{-2} = 0, \tag{6.9c}$$

where $\phi = c/u$, $Fr = u/\sqrt{gh}$ and $\psi = f_u/\left((1-p)h\right)$.

There are three roots for the above cubic equation of Eq. (6.9b or c) and they can be obtained by using the Cardano formula (Fogiel, 1980; Hazewinkel, 1988) in a very complicated expression.

Generally, ψ—a transport parameter—is very small and it can be estimated by using

$$\psi = \frac{df/du}{(1-p)h} = \frac{b}{(1-p)}\frac{q_B}{uh}. \tag{6.10}$$

Eq. (6.10) indicates that ψ is proportional to the ratio of bedload flux to water flux. For branches of the Rhine in the Netherlands, ψ is of the order 10^{-5}–10^{-6} (Jansen, *et al.*, 1979) and the same order is found in the lower Waikato River (ref. Chapter Seven).

Perturbation methods can be used to obtain an approximate solution of Eq. (6.9c). For a small parameter $\varepsilon \sim \psi$, assuming the solution to be

$$\phi = \phi_0 + \varepsilon\phi_1, \tag{6.11}$$

and a moderate Froude number, e.g., $Fr \le 0.6$ (Jansen, *et al.*, 1979), we obtain perturbation equations of Eq. (6.9c) as

$$-\phi_0^3 + 2\phi_0^2 + \left(Fr^{-2} - 1\right)\phi_0 = 0, \tag{6.12}$$

$$\phi_1 = \frac{Fr^{-2}(\phi_0 - 1)}{3\phi_0^2 - 4\phi_0 + 1 - Fr^{-2}}. \tag{6.13}$$

The first order equation—Eq. (6.12)—has three roots:

$$\phi_0' = 1 - Fr^{-1}, \quad \phi_0'' = 1 + Fr^{-1}, \quad \phi_0''' = 0. \tag{6.14}$$

corresponding to a fixed bed problem. Based on Eq. (6.14), then the higher order equation—Eq. (6.13)—has the following roots:

$$\phi_1' = \frac{Fr^{-2}}{2(1 - Fr^{-1})}, \quad \phi_1'' = \frac{Fr^{-2}}{2(1 + Fr^{-1})}, \quad \phi_1''' = \frac{1}{1 - Fr^2}. \tag{6.15}$$

Using Eqs. (6.11, 6.14–15), solutions of Eq. (6.9c) to order of ε are found to be

$$\begin{aligned} \phi' &= (1 - Fr^{-1})\left(1 + \frac{\varepsilon Fr^{-2}}{2(1 - Fr^{-1})^2}\right), \\ \phi'' &= (1 + Fr^{-1})\left(1 + \frac{\varepsilon Fr^{-2}}{2(1 + Fr^{-1})^2}\right), \\ \phi''' &= \frac{\varepsilon}{1 - Fr^2}, \end{aligned} \tag{6.16a}$$

or the velocities:

$$\begin{aligned} c_1 &= (u - \sqrt{gh})\left(1 + \frac{\varepsilon Fr^{-2}}{2(1 - Fr^{-1})^2}\right), \\ c_2 &= (u + \sqrt{gh})\left(1 + \frac{\varepsilon Fr^{-2}}{2(1 + Fr^{-1})^2}\right), \\ c_3 &= \frac{u\varepsilon}{1 - Fr^2}. \end{aligned} \tag{6.16b}$$

Both c_1 and c_2 represent the propagation speed of a disturbance at the water surface, and the third one, c_3, expresses the propagation speed of a disturbance at the bed surface.

Obviously there are two disparate time scales for water movement and bedload movement under a moderate Froude number ($c_2 >> c_3$ and $c_2 >> c_3$). Figure 6.4 shows the characteristic curves in the x and t plane, and the steepness of c_3 indicates its relatively small value in comparison to c_1 and c_2.

If $Fr \to 1$, Eq. (6.12) becomes

$$-\phi_0^3 + 2\phi_0^2 = 0. \tag{6.17}$$

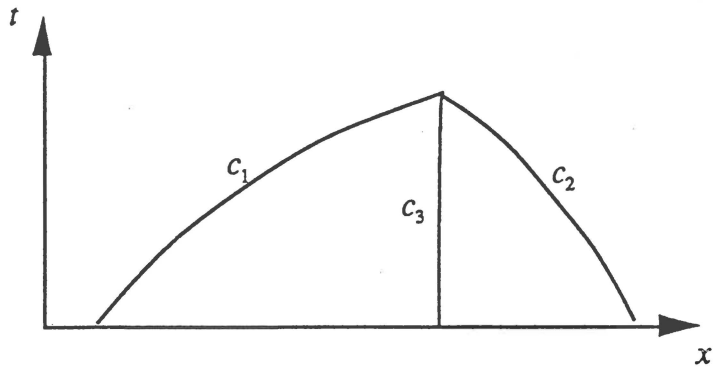


Figure 6.4 Sketch of the characteristic curves of water and bedload movements at $Fr \leq 0.6$. Both c_1 and c_2 are the characteristic velocities for water movement and c_3 for bedload movement.

One of the three roots is $\phi_0' = 2$ and the remaining two are $\phi_0'' = \phi_0''' = 0$. The higher order equation is also different from Eq. (6.13) in this case and the detailed discussions for the solutions can be found from Lyn (1987).

Solutions for c_1, c_2 and c_3 were also discussed by Jansen, *et al.* (1979).

6.2.4 Review and discussions

Many one-dimensional water and bedload movement models have been reported since the 1960s. They can be found from journals, such as *Journal of Hydraulic Engineering*, *Water Resources Research,* and *Journal of Hydraulic Research,* and various reports. Recently two-dimensional and three-dimensional mobile bed models have also been developed (Smith and McLean, 1984; Struiksma, *et al.*, 1985; McAnally, *et al.*, 1986; Shimizu and Itakura, 1989; Shimizu, *et al.*, 1990). Chang (1982) used minimum stream power as a width predictor to take account of the channel width changes in a one-dimensional model. Yang, *et al.* (1988) used the concept of a stream tube in a one-dimensional model to investigate two-dimensional and three-dimensional river changes.

A review of alluvial channel modelling in America was given by Dawdy and Vanoni (1986) and they stated:

> *"In conclusion, the choice of a model at this time is arbitrary, and the choice of a modeller is probably more important than the choice of a model."*

Some degradation models were analysed and compared by Lu and Shen (1986). A model study by the National Research Council (1983) indicated the following deficiencies in modelling:

> *"(1) unreliable sediment transport function,*
>
> *(2) inadequate formulation of the friction factor of erodible*

channels,

(3) inadequate understanding and formulation of bed armouring and its effects on sediment discharge and friction factor,

(4) inadequate understanding and formulation of bank erosion."

The difficulties result from not only the mixed sediment transport with hydraulic structure, but also the scale and detail, existing in all fields of hydraulic engineering (Liggett, 1990), as well as the crude numerical algorithms (Cunge and Holly, 1986; Laursen, 1986).

If Eq. (6.8) holds, the one-dimensional model of water and bedload movements is said to be an equilibrium flow, otherwise a non-equilibrium flow. Because sediment transport simulations are so important in determining changes in river channel, recently considerable efforts have gone into that area (Parker, *et al.*, 1982; Bell and Sutherland, 1983; Proffitt and Sutherland, 1983; van Rijn, 1984a; 1984b; 1984c; Parker, 1990; Parker and Sutherland, 1990; Hardwick and Willetts, 1991). In this study, an equilibrium flow is assumed.

Based on whether the decoupled techniques—hydraulic equations (the St Venant equations) and bedload balance equation being solved separately—are used, the model can be a coupled or decoupled model.

There are two types of decoupled model.

The first type involves decoupling the hydraulic equations (the St. Venant equations) from the bedload balance equation on the basis that bed level changes can be negligible within a time step (e.g., Chen, *et al.*, 1975; Bennett and Nordin, 1977). All numerical methods for the unsteady flow simulation (Mahmood and Yevjevich, 1975; Abbott, 1979; Cunge, *et al.*, 1980; Lai, 1986) as well as simplified methods (Henderson, 1966; Ponce, *et al.*, 1978; Weinmann and Laurenson, 1979; Ponce and Theurer, 1982) can be used to solve the hydraulic equations. The bedload balance equation is usually solved numerically by using a backward method since changes in alluvial bed level depend on upstream boundary conditions (Chang and Hill, 1977).

Unfortunately, this technique makes the problem ill-posed because as soon as given $u(x,t+\Delta t)$ from the fixed-bed step, bed levels are completely determined by (ref. Eq. (6.6))

$$z(x,t+\Delta t) = z(x,t) - \int_t^{t+\Delta t} \left(\frac{1}{1-p} \frac{\partial q_B(u(x,t))}{\partial x} + q_{Bs} \right) dt$$

with no freedom to pose a third boundary in terms of bed level z at the upper boundary.

The second type of decoupled model uses a steady or quasi-steady flow approximation. Because bed transients propagate at a much slower speed than water transients ($c_3 << c_1$ or c_2) under a moderate Froude number, say $Fr \leq 0.6$, when changes in bed levels are significant in a time step—$c_3 \sim O(1)$, the water surface transients will have already diminished—both c_1 and $c_2 \to \infty$—and a constant discharge can be assumed.

In the other words, in practice if variations in river discharge are significant in the time scale used, bed level changes in this time scale can be neglected. Conversely river flows can be treated as a constant discharge over time in the scale in which bed level changes become significant (Figure 6.5).

For the second type of decoupled model, the flows are assumed to be constant and the governing equations become

$$u\frac{\partial h}{\partial x} + h\frac{\partial u}{\partial x} = 0, \tag{6.18}$$

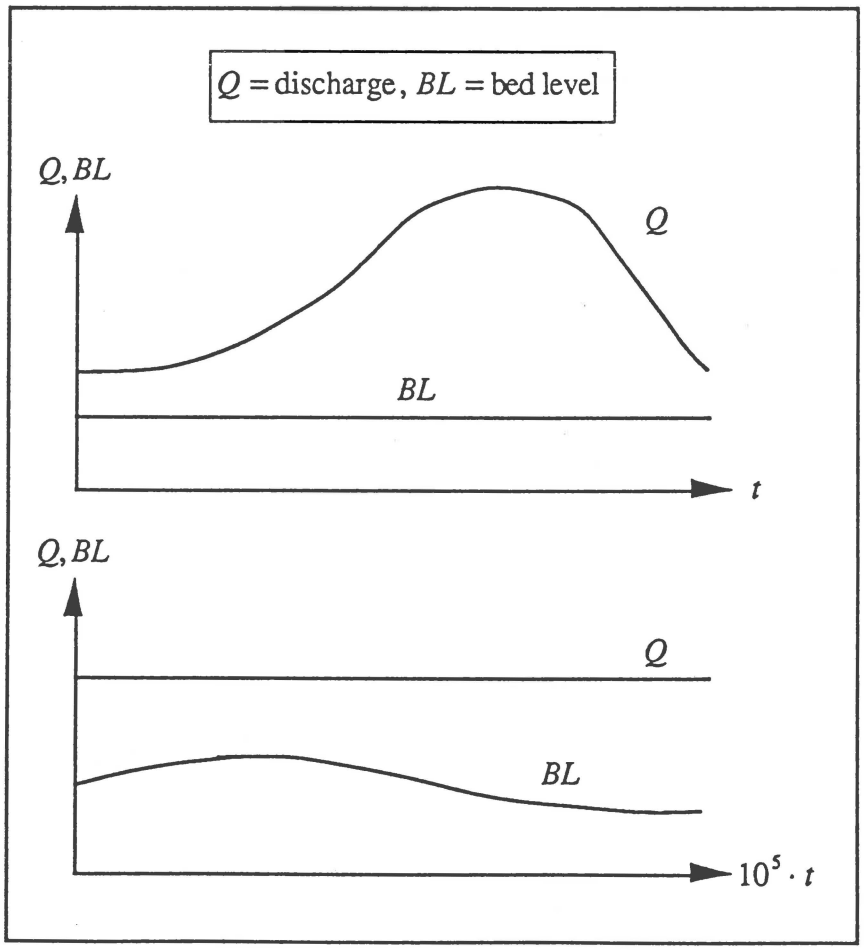


Figure 6.5 Sketch of time scale effects on variations in discharge and bed level.

$$u\frac{\partial u}{\partial x}+g\frac{\partial h}{\partial x}+g\frac{\partial z}{\partial x}+gS_f=0, \tag{6.19}$$

$$\frac{\partial z}{\partial t}+\frac{1}{1-p}\frac{\partial q_B}{\partial x}=q_{Bs}. \tag{6.20}$$

Given proper conditions, Eqs. (6.18–20) may form a well-posed problem and be solved numerically. However for the primary unknowns, u, h and z, their initial values, u_0, h_0 and z_0, have to satisfy the following equation (Eqs. (6.18–19) at $t=0$):

$$\frac{dh_0}{dx}=-\frac{dz_0/dx+S_f\big|_{t=0}}{1-u_0^2/gh_0}.$$

De Vries (1973b) presented a two-alternating-step method to solve the second type of decoupled model—Eqs. (6.18–6.20). In the first step, the bed level is fixed and the water level calculated by using a backwater curve calculation. In the second step, the bed level is changed by performing bedload balance calculations given bedload supply at the upper boundary. This procedure was used in the well-known HEC-6 model (Thomas, 1979). The backwater curve calculation can be undertaken by using standard methods (Henderson, 1966), and the bedload balance equation—Eq. (6.20)—can be approximated by using the backward-difference scheme. If a central-difference approximation is used in the bedload balance calculation, a non-zero artificial diffusion term should be added in order to get a stable numerical result.

Actually the two-alternating-step method assumes that the hydraulic equations and sediment balance equation can be solved separately. Similar to the first type of decoupled model, this makes the problem ill-posed.

The results of Lyn's studies (1987) indicated that because of the singular perturbation system of the unsteady water and bedload transport in a channel, approximations based on the assumption of a regular perturbation were unable to satisfy exactly either arbitrary boundary conditions or initial conditions. As demonstrated above, for the first type of decoupled model, the assumption of no changes in bed level within a time step makes the system ill-posed. The constant discharge assumption of the second type of decoupled model requires a particular initial condition in order to form a well-posed problem.

From Eqs. (6.8) and (6.18–19), Eq. (6.20) becomes

$$\frac{\partial z}{\partial t}+\frac{gf_u}{(1-p)(gq/u^2-u)}\frac{\partial z}{\partial x}=q_{Bs}-\frac{gS_f f_u}{(1-p)(gq/u^2-u)}, \tag{6.21}$$

in which q is the discharge per unit width, $q=uh$. Eq. (6.21) is a wave equation with a velocity of c

$$c=\frac{gf_u}{(1-p)(gq/u^2-u)}, \tag{6.22a}$$

or

$$c=\frac{uf_u}{h(1-p)(1-Fr^2)}. \tag{6.22b}$$

Here c is the same as c_3 in Eq. (6.16b) if $\psi=f_u/(h(1-p))$ is used instead of ε.

According to Eq. (6.21), obviously the upstream boundary condition concerning bed levels, the bed level variations, $z_{up}(t)$, or its derivative with respect to x, not bedload supply, should be specified. In the case of specification of bedload supply at the upper boundary, $q_{B,up}$, a relation between $q_{B,up}$ and the bed level needs to be established. Normally it is assumed that in a special reach of Δx_λ at the upstream boundary, the following equation is held (ref. Eq. (6.20)):

$$z_{up}(\Delta t)-z_{up}(0)=\Delta t(q_{B,up}-\tilde{q}_{B,up})/((1-p)\Delta x_\lambda), \tag{6.23}$$

where $\tilde{q}_{B,up}$ is the flow capacity of bedload transport determined by using one of bedload transport formulae.

Recently, by using the Preissmann scheme (Cunge, *et al.*, 1980; Lyn and Goodwin, 1987), some coupled or semi-coupled models have been reported (Yang and Wang, 1990; Bhallamudi and Chaudhry, 1991; Correia, *et al.*, 1992). Their applications were mainly limited to a short period of storms or for numerical tests with a small time step (e.g., 0.5 hr or less) in comparison with the time scale of bed level changes, although a time step of 100 hr was reported by Cunge, *et al.* (1980, page 290). The usefulness of this method is limited not only by computer CPU time, but also by the difficulties in collection of the detailed field data required within a short time step for calibration and validation and by determination of necessary boundary and initial conditions.

The second type of decoupled model with an equilibrium flow is here selected for investigation of long term changes in bed level of the lower parts of the Waikato River.

6.3 Computation methods

During discussions in 6.2 about the water and bedload movement models, it was assumed that formulas for q_B and S_f were known. Determination of these two variables are discussed below.

6.3.1 Sediment transport

There are two kinds of sediment yield measurement methods. One is a hydrological approach: field measurement of the sediment load or an estimate of the erosive activity in the catchment, or both. The other is a hydraulic approach: determination of the river flow transport capacity with some coefficients determined by field or laboratory data and assuming that the flow is equilibrium. The latter is used more widely in river engineering especially for mathematical modelling.

Characteristics of sediment transportation have been described by various authors (e.g., Allen, 1970; Graf, 1971; Yalin, 1972; Raudkivi, 1976; Middleton and Southard, 1984), but a complete analysis of general problems of sediment transportation has not been achieved yet. Shen (1979) and Vanoni (1984) presented an outline of problems in the sediment transportation.

Two different modes of sediment transportation exist (Scheidegger, 1991): suspended sediment transportation and bottom sediment transportation, and the transitional stage between these two modes is called saltation (Bagnold, 1966). Sediments, which move along the bottom of rivers and are supported by the bed, are also called bedload sediments. The following discussions focus on bedload transportation. A review of research on suspended load transportation can be found from Nordin and McQuivey (1971) and Bechteler (1986), and total sediment transportation from White, *et al.* (1975).

The distinction between bedload and suspended load is important in practice. Kresser introduced the following equation of a limiting grain size d_* (quoted by Mangelsdorf, *et al.*, 1990):

$$d_* = \frac{u^2}{360g}, \tag{6.24}$$

where u is the mean velocity. Sediments with their grain sizes bigger than d_* are bedload, and smaller suspended load.

Scheidegger (1991) summarised three types of theories on bedload transportation: drag theory, stochastic theory and physical semi-empirical

theory.

The drag theory has been known for a long time and the bedload transportation is described by an equation of the form

$$q_B = a'\tau(\tau - \tau_0), \tag{6.25a}$$

or more generally

$$q_B = a(\tau - \tau_0)^b, \tag{6.25b}$$

where q_B is a unit width bedload transport rate, a', a and b are parameters, τ and τ_0 are, respectively, the drag force and the critical drag force. The well known Du Boys formula was derived on this theory (Du Boys, 1879).

The stochastic theory is represented by Einstein formula (Einstein, 1950) and its modification (e.g., Cheong and Shen, 1976).

The third theory is based mainly on fundamental physical principles as well as some empirical equations. The following well-known sediment transport equations are based on it: Meyer-Peter and Müller (1948), Yalin (1963), Bagnold (1956, 1966), Engelund and Hansen (1967), Ackers and White (1973), Yang (1973; 1979; 1984) and others (Vanoni, 1975).

Representative grain-size, usually the median size d_{50}, is used in sediment transport formulae as a constant parameter. For graded sediment transportation, a separation of the total sediments by certain diameters is adopted in order to take into account non-uniform sediment distributions. Sediment transport rates are calculated separately for the different grain-sizes and then they are summed to produce the total sediment movements. However whether this technique improves the calculation of sediment transportation is in doubt and understanding of the structure of graded-sediment movement is still limited (Proffitt and Sutherland, 1983; Armanini and di Silvio, 1988). Collection of accurate field data to test the theories is very difficult.

A comparison of the accuracy of some sediment transport formulae was made with the field and laboratory data by White, *et al.* (1975), ASCE (1982b), Yang and Molinas (1982), Nakato (1990), Yang and Wan (1991). Nearly all formulae involved only the particles larger than 0.062 mm and mainly for sand size range.

Selection of a sediment transport formula in practice may be based on some criteria, such as simplicity, accuracy, available data (Shen, 1978). The best

way to obtain a reliable sediment transport rate for a given river is to collect suitably accurate field data to calibrate the formula selected.

Basically, most bedload transport formulae can be expressed as a relationship between the transport function, X, and the flow parameter, Y (Jansen, *et al.*, 1979):

$$X = f(Y), \tag{6.26}$$

$$X = \frac{q_S}{D^{1.5}\sqrt{g\Delta}}, \tag{6.27}$$

$$Y = \frac{D\Delta}{\mu h S}, \tag{6.28}$$

where q_S is volumetric bedload sediment transportation in unit width, D is a characteristic grain-size (normally the median size d_{50}), $\Delta = (\rho_s - \rho)/\rho$ where ρ_s and ρ are, respectively, densities of sediment and water, μ is a correction factor, h is water depth and S is an energy slope.

Laursen (1956) compared several well-known bedload equations in a reduced form. Karim and Kennedy (1990) used a non-linear multiple-regression analysis to derive relations between flow velocity and bedload rate in fluvial rivers based on field and laboratory data. The power law,

$$X = \alpha Y^{\beta}, \tag{6.29}$$

might approximate Eq. (6.26) with parameters of α and β, which may not be exactly constant. Based on the Meyer-Peter-Müller formula (Meyer-Peter and Müller, 1948), the Engelund-Hansen formula (Engelund and Hansen, 1967) and the Einstein-Brown formula (Brown, 1950), their β are, respectively, $-\infty < \beta < -1.5$, $\beta = -2.5$ and $\beta = -3$.

Eq. (6.29) can be rewritten as (Jansen, *et al.*, 1979)

$$q_B = au^b, \tag{6.8}$$

where a and b are parameters. From the analysis above, we have

$$a = \alpha D^{1.5}\sqrt{g\Delta}\left(\frac{D\Delta C^2}{\mu}\right)^{\beta}, \tag{6.30a}$$

$$b = -2\beta, \tag{6.30b}$$

where C is Chézy's coefficient.

It should be mentioned again that when the sediment transport formulae are applied directly to produce a volume of bedload, an equilibrium transport is assumed.

6.3.2 Friction calculation

In practice, frictional loss of energy is caused by the surface roughness from skin and bedforms, as suggested by Einstein and Barbarossa (1952), as well as covers like vegetation, etc. in natural environments (Chow, 1959; Simons and Richardson, 1961; Holtorff, 1982; Shen, *et al.*, 1990).

Chézy developed a formula in 1769 to calculate velocity for a uniform flow:

$$V = C\sqrt{RS_f}, \tag{6.31a}$$

or discharge:

$$Q = AV = AC\sqrt{RS_f}, \tag{6.31b}$$

where V is the mean velocity, C is a factor of flow resistance or Chézy's coefficient, R is the hydraulic radius, S_f is the energy slope and A is the cross-sectional area. Because this formula is used in uniform flow, the energy slope is equivalent to the slopes of both the water surface and bottom.

There are many equations proposed to determine the parameter C (Chow, 1959). Manning presented his formula in 1889 as

$$C = \frac{1}{n}R^{1/6}, \tag{6.32}$$

where n is Manning's coefficient and its value can be found in handbooks (e.g., Chow, 1959).

It is evident that Manning's coefficient is not a constant (Higginson and Johnston, 1988). But for a given river, a constant Manning's coefficient determined by trial and error using field data is still adopted in hydraulic engineering and it is also assumed that the slope of energy for a uniform flow can be used for an unsteady flow. Based on Eqs. (6.31a–b, 6.32), we have

$$S_f = \frac{n^2V^2}{R^{4/3}} = \frac{n^2Q^2}{A^2R^{4/3}}. \tag{6.33}$$

Strickler (1923) developed the following formula of k_r instead of $1/n$ in Eq. (6.32) from experiments for the grain roughness on the bottom with a rigid wall and fully developed turbulence:

$$k_r = \frac{26}{d_r^{1/6}}, \tag{6.34}$$

where k_r is Strickler's number, d_r is a critical grain diameter about d_{90} of the surface layer sediments. Considering the additional effects of shape or bedform roughness on friction, Eq. (6.34) was modified to

$$k_s = \frac{K}{d_r^{1/6}}, \tag{6.35}$$

where k_s is Strickler's number including shape roughness, and K is a constant between 16 and 26. A table of K was given by Meyer-Peter and Lichtenhahn (1963).

In practice, Chézy's or Manning's coefficients are determined from historical data, and they are normally assumed to be constant over time when applied. If the conditions, e.g. discharges, in application are much different from those of the historical data, corrections may be required.

6.4 One-dimensional water and bedload movement models with a quasi-steady and equilibrium flow

Linearization has been used in order to obtain linear equations to approximate one-dimensional water and bedload movement system of Eqs. (6.18–20). For homogeneous forms with simple boundary conditions, analytical solutions can be obtained. The assumptions and non-linear effects are discussed following.

The system has a mixed hyperbolic-parabolic character, which can be demonstrated by its linearized forms given below. For large time, diffusion or damping becomes a dominant process in river bed level changes.

6.4.1 Linearization of the water and bedload movement model with a constant flow discharge

Eqs. (6.7–8, 6.18–20) can be linearized by considering a small perturbation on an initial steady condition in which bed levels follow a straight line along a river. We refer to this condition as "bed continuity" (Vreugdenhil, 1982; Ribberink and van der Sande, 1985; Lenau and Hjelmfelt Jr., 1992).

Using $u = u_0 + u'$ with $u_0 >> u'$, $h = h_0 + \eta - z_b$, and $z = z_0 + z_b$ with $h_0 >> \eta$ and $h_0 >> z_b$, where the subscript "0" represents the steady conditions, and u', η and z_b are, respectively, small changes from the steady conditions of velocity,

water surface elevation and river bed elevation, then Eq. (6.19) becomes

$$(u_0+u')\frac{\partial(u_0+u')}{\partial x}+g\frac{\partial(h_0+\eta-z_b)}{\partial x}+g\frac{\partial(z_0+z_b)}{\partial x}=-\frac{g(u_0+u')^2}{C^2(h_0+\eta-z_b)}$$

$$=-\frac{gu_0^2(1+u'/u_0)^2}{C^2h_0(1+(\eta-z_b)/h_0)}, \tag{6.36}$$

which can be linearized to

$$-gi_0+u_0\frac{\partial u'}{\partial x}+g\frac{\partial\eta}{\partial x}\approx-\frac{gu_0^2}{C^2h_0}\left(1+\frac{2u'}{u_0}-\frac{\eta-z_b}{h_0}\right), \tag{6.37}$$

where $-i_0=\partial z_0/\partial x$, which is the original equilibrium bed level slope.

The zero-order term from Eq. (6.37) gives

$$-gi_0=-\frac{gu_0^2}{C^2h_0}, \tag{6.38}$$

which represents the uniform flow. The first-order term is

$$u_0\frac{\partial u'}{\partial x}+g\frac{\partial\eta}{\partial x}+gi_0\left(\frac{2u'}{u_0}+\frac{z_b-\eta}{h_0}\right)=0. \tag{6.39}$$

Eq. (6.18) can be linearized to be

$$u_0\frac{\partial(\eta-z_b)}{\partial x}+h_0\frac{\partial u'}{\partial x}=0. \tag{6.40a}$$

This equation can be integrated with respect to x assuming $\eta=0$, $z_b=0$ and $u'=0$ at the upstream or downstream boundaries (the river is in an initial steady state) to give

$$u_0(\eta-z_b)+h_0u'=0. \tag{6.40b}$$

Linearization of Eqs. (6.20) and (6.8) gives

$$\frac{\partial z_b}{\partial t}+\frac{f_u}{1-p}\frac{\partial u'}{\partial x}=q_{Bs}, \tag{6.41}$$

where $f_u=df/du$. Combination of Eqs. (6.39) and (6.40b) produces

$$Fr_0^2\frac{\partial z_b}{\partial x}+(1-Fr_0^2)\frac{\partial\eta}{\partial x}+\frac{3i_0}{h_0}(z_b-\eta)=0 \tag{6.42}$$

with $Fr_0=u_0/\sqrt{gh_0}$ and combination of Eqs. (6.40a) and (6.41) gives

$$\frac{\partial z_b}{\partial t}+\frac{u_0 f_u}{h_0(1-p)}\frac{\partial(z_b-\eta)}{\partial x}=q_{Bs}. \tag{6.43}$$

• Simple wave model

For a non-friction flow, $S_f=0$ or $i_0=0$, combination of Eqs. (6.42–43) by eliminating η gives a simple wave model for variable z_b:

$$\frac{\partial z_b}{\partial t}+c\frac{\partial z_b}{\partial x}=q_{Bs} \tag{6.44}$$

with

$$c=\frac{u_0 f_u}{h_0(1-p)(1-Fr_0^2)}. \tag{6.45}$$

Eq. (6.44) is called a propagating wave equation (Baldock and Bridgeman, 1981) and variable z_b is said to be a wave which propagates in the direction of x with velocity c.

• Parabolic model

For a uniform flow, the equation of motion—Eq. (6.19)—is

$$g\frac{\partial z}{\partial x}=-\frac{gu^2}{C^2h},$$

or

$$g\frac{\partial z}{\partial x}=-\frac{gu^3}{C^2q}.$$

Thus

$$g\frac{\partial^2 z}{\partial x^2}=-\frac{3gu^2}{C^2q}\frac{\partial u}{\partial x}$$

which can be linearized as

$$g\frac{\partial^2 z_b}{\partial x^2}=-\frac{3gu_0^2}{C^2q}\frac{\partial u'}{\partial x}=-\frac{3gi_0}{u_0}\frac{\partial u'}{\partial x}. \tag{6.46}$$

Combination of Eqs. (6.41) and (6.46) gives a parabolic model:

$$\frac{\partial z_b}{\partial t}-D\frac{\partial^2 z_b}{\partial x^2}=q_{Bs} \tag{6.47}$$

with

$$D=\frac{1}{3}\frac{1}{1-p}\frac{u_0}{i_0}\frac{df}{du}. \tag{6.48}$$

Eq. (6.47) is called a diffusion equation. Culling (1960) obtained the same equation from qualitative arguments without specifying the coefficient D.

• **Hyperbolic model**

Combination of Eqs. (6.42) and (6.43) by eliminating η gives

$$\frac{\partial z_b}{\partial t} - \frac{u_0 f_u}{3i_0(1-p)}\frac{\partial^2 z_b}{\partial x^2} - \frac{h_0(1-Fr_0^2)}{3i_0}\frac{\partial^2 z_b}{\partial x \partial t} = q_{Bs} - \frac{h_0(1-Fr_0^2)}{3i_0}\frac{\partial q_{Bs}}{\partial x}. \qquad (6.49a)$$

Assuming the term on the right hand side of Eq. (6.49a) to be equal to q'_{Bs}—a source term being function of q_{Bs}, we have

$$q'_{Bs} = f(q_{Bs}) = q_{Bs} - \frac{h_0(1-Fr_0^2)}{3i_0}\frac{\partial q_{Bs}}{\partial x}.$$

In the case of the lower Waikato River, q_{Bs} stands for sand extraction volumes. Because the exact sand extraction volumes and their distribution along the river are unknown and have to be assumed and determined by calibration, we use q_{Bs} to approximate to q'_{Bs}, where q_{Bs} is adjustable. Thus we have

$$\frac{\partial z_b}{\partial t} - \frac{D}{c}\frac{\partial^2 z_b}{\partial x \partial t} - D\frac{\partial^2 z_b}{\partial x^2} = q_{Bs}, \qquad (6.49b)$$

where c and D are the same as those in Eqs. (6.45) and (6.48).

Eq. (6.49b) is a hyperbolic equation of second order. It is important to notice that the third term on the left hand side of Eq. (6.49b) is a damping term. By comparing with Eq. (6.47), effects of the non-uniform flows are indicated by the second term on the left hand side of the equation.

Vreugdenhil and de Vries (1973) derived a non-linear equation for bed level z with a similar form to Eq. (6.49b), and parameters c and D were, respectively, given by

$$c = \frac{uf_u}{h(1-p)(1-Fr^2)},$$

$$D = \frac{uf_u}{3i_0(1-p)\phi}$$

in which $\beta = \partial z/\partial x$ and

$$\phi = \frac{1}{3}\left\{\frac{\beta}{i_0} + \left(\frac{u}{u_0}\right)^3\right\}\left(\frac{Fr^2+2}{1-Fr^2} + \frac{uf_{uu}}{f_u}\right) + \left(\frac{u}{u_0}\right)^3,$$

$$f_{uu} = \frac{d^2 f}{du^2}.$$

In summary, the assumption made for linearization of the water and bedload movement model is a small perturbation with an initial bed continuity. The system can be linearized to a simple wave equation for a steady and non-friction flow, a parabolic equation for a steady and uniform flow, and a hyperbolic equation with a damping term for a steady and non-uniform flow. The parabolic equation and the hyperbolic equation will be applied to the lower parts of the Waikato River.

6.4.2 Analytical solutions of the linear equations

Analytical solutions of the above three linear equations in their homogeneous forms are discussed below given simple initial and boundary conditions.

The x axis below is from the upstream to the downstream. The dimensionless variables used are

$$\tilde{x} = \frac{c}{D}x, \tag{6.50a}$$

$$\tilde{t} = 2\frac{c^2}{D}t, \tag{6.50b}$$

$$\tilde{z}_b = \frac{(1-p)c}{q_{B\infty} - q_{B0}}z_b, \tag{6.50c}$$

$$\tilde{q}_B = \frac{q_B}{q_{B\infty} - q_{B0}}, \tag{6.50d}$$

where $q_{B\infty}$ denotes the final steady rate of bedload supply at $x=0$ and q_{B0} denotes the initial steady rate of bedload supply. Then the dimensionless homogeneous governing equations of simple wave, parabolic and hyperbolic equations are, respectively, as follows:

$$\frac{\partial \tilde{z}_b}{\partial \tilde{t}} + \frac{1}{2}\frac{\partial \tilde{z}_b}{\partial \tilde{x}} = 0 \text{ (simple wave):} \tag{6.51}$$

$$\frac{\partial \tilde{z}_b}{\partial \tilde{t}} - \frac{1}{2}\frac{\partial^2 \tilde{z}_b}{\partial \tilde{x}^2} = 0 \text{ (parabolic):} \tag{6.52}$$

$$\frac{\partial \tilde{z}_b}{\partial \tilde{t}} - \frac{1}{2}\frac{\partial^2 \tilde{z}_b}{\partial \tilde{x}^2} - \frac{\partial^2 \tilde{z}_b}{\partial \tilde{x}\partial \tilde{t}} = 0 \text{ (hyperbolic).} \tag{6.53}$$

The equation of bedload balance becomes

$$\frac{\partial \tilde{z}_b}{\partial \tilde{t}} + \frac{1}{2}\frac{\partial \tilde{q}_B}{\partial \tilde{x}} = 0. \tag{6.54}$$

Initial conditions are taken to be $z_b(x,0)=0$ and a steady bedload supply q_{B0} in the domain. Boundary conditions are given as at $x=0$, bedload supply instantaneously changes to another of constant supply $q_{B\infty}$ and at $x=\infty$, solutions should remain bounded. Mathematically, these conditions are represented by the following dimensionless forms:

$$\tilde{z}_b(\tilde{x},0)=0, \quad \tilde{x}\geq 0, \tag{6.55}$$

$$\tilde{q}_B(\tilde{x},0)=\frac{q_{B0}}{q_{B\infty}-q_{B0}}, \quad \tilde{x}\geq 0, \tag{6.56}$$

$$\tilde{q}_B(0,\tilde{t})=\tilde{q}_{B\infty}(0,\tilde{t})=\frac{q_{B\infty}}{q_{B\infty}-q_{B0}}, \quad \tilde{t}>0. \tag{6.57}$$

Bed level $z_b(0,t)$ or its derivative with respect to x should be specified instead of bedload supply—Eq. (6.57). Jain (1981) gave the boundary condition at $x=0$ as:

$$-D\frac{\partial z_b}{\partial x}(0,t)=\frac{q_{B\infty}-q_{B0}}{1-p}, \quad \tilde{t}>0, \tag{6.58}$$

or in dimensionless form:

$$-\frac{\partial \tilde{z}_b}{\partial \tilde{x}}(0,\tilde{t})=1, \quad \tilde{t}>0.$$

For the simple wave model of Eq. (6.51), there is a general analytical solution of $\tilde{z}_b$ as

$$\tilde{z}_b=\mathrm{H}(\tilde{t}-2\tilde{x}), \tag{6.59}$$

where $\mathrm{H}(\tilde{t}-2\tilde{x})$ is a unit step function in which $\mathrm{H}=0$ for $\tilde{t}>2\tilde{x}$ or $\mathrm{H}=1$ otherwise. It should be pointed out that the unit step function does not satisfy the boundary condition of Eq. (6.58) given by Jain (1981).

For the parabolic model of Eq. (6.52), we have the following solution of $\tilde{z}_b$:

$$\tilde{z}_b=\sqrt{\frac{2\tilde{t}}{\pi}}\exp\left(-\frac{1}{2}\xi^2\right)-\tilde{x}\,\mathrm{erfc}\left(\frac{1}{2}\sqrt{2}\xi\right), \tag{6.60}$$

in which $\xi=\tilde{x}/\sqrt{\tilde{t}}$ and $\mathrm{erfc}(x)$ is the complementary error function defined by

$$\mathrm{erfc}(x)=\frac{2}{\sqrt{\pi}}\int_{x}^{\infty}e^{-u^2}du. \tag{6.61}$$

Gill (1983a; 1983b) gave analytical solutions for boundary conditions indicating aggradation and degradation by using both methods of Fourier series and an error function.

For the hyperbolic model of Eq. (6.53), its analytical solution is given below (Gill, 1988)

$$\begin{aligned}\tilde{z}_b=&\,\mathrm{H}(\tilde{x}-2\tilde{t})\Big\{(1+\theta-\tilde{x})\exp(-\theta)\mathrm{I}_0\Big(\sqrt{\theta^2-\tilde{x}^2}\Big)\\&+\sqrt{\theta^2-\tilde{x}^2}\exp(-\theta)\mathrm{I}_1\Big(\sqrt{\theta^2-\tilde{x}^2}\Big)\\&+(1-2\tilde{x})\big[1-\mathrm{J}((\theta-\tilde{x})/2,(\theta+\tilde{x})/2)\big]\Big\},\end{aligned} \tag{6.62}$$

in which $\theta=\tilde{t}-\tilde{x}$, I_0 and I_1 are modified zero and first order Bessel functions respectively, and function $\mathrm{J}(\phi_1,\phi_2)$ is defined as

$$\mathrm{J}(\phi_1,\phi_2)=1-\exp\big[-(\phi_1+\phi_2)\big]\sum_{k=1}^{\infty}\psi^{-k}\mathrm{I}_k(\zeta), \tag{6.63}$$

where $\psi=\sqrt{\phi_1/\phi_2}$ and $\zeta=2\sqrt{\phi_1\phi_2}$. The numerical values of Eq. (6.62) were also presented in the paper by Gill (1988) to compare with solutions of the parabolic equation.

Ribberink and Van der Sande (1985) gave approximate solutions for the hyperbolic model of Eq. (6.53) for small and large values of $\tilde{t}$ ($\tilde{t}\to 0$ and $\tilde{t}\to\infty$). Their solutions were, respectively,

$$z_b(\tilde{x},\tilde{t})=2\exp(-\tilde{x})-\exp(-\frac{1}{2}\tilde{t})\qquad \tilde{x}<\frac{1}{2}\tilde{t}, \tag{6.64a}$$

$$z_b(\tilde{x},\tilde{t})=\frac{\phi}{\sqrt{\pi}}\exp\left\{-\frac{\tilde{x}^2}{\phi^2}\right\}+\left(\frac{1}{2}-\tilde{x}\right)\mathrm{erfc}\left\{\frac{\tilde{x}}{\phi}\right\}\qquad \tilde{x}<\tilde{t}. \tag{6.64b}$$

Lenau and Hjelmfelt Jr. (1992) used a Laplace transform to solve the parabolic and the hyperbolic models analytically to the following conditions:

$$z(x,0)=-(\Delta z)e^{kx}, \tag{6.65-1}$$

$$z(0,t)=-\Delta z, \tag{6.65-2}$$

$$z(x,t)\to 0\qquad \text{as } x\to-\infty. \tag{6.65-3}$$

These conditions express the physical problem as how the bed levels

upstream degrade due to an abrupt drop in bed level at $x = 0$.

The linear diffusion model was obtained as early as in 1960 (Culling, 1960). It has been used for an analysis of both aggradation and degradation in an alluvial river (Ashida and Michuie, 1971; Soni, *et al.*, 1980; Jain, 1981; Gill, 1983a; 1983b; Phillips and Sutherland, 1986; Begin, 1988). Its non-linear form has also been studied (Jaramillo and Jain, 1984; Gill, 1987; Zhang and Kahawita, 1987), and Jaramillo and Jain (1984) solved the non-linear diffusion equation by using a method of residuals.

The linear hyperbolic model was developed and used in rivers by Vreugdenhil and de Vries (1973), Ribberink and van der Sande (1985), Zhang and Kahawita (1988; 1990), Lenau and Hjelmfelt Jr. (1992).

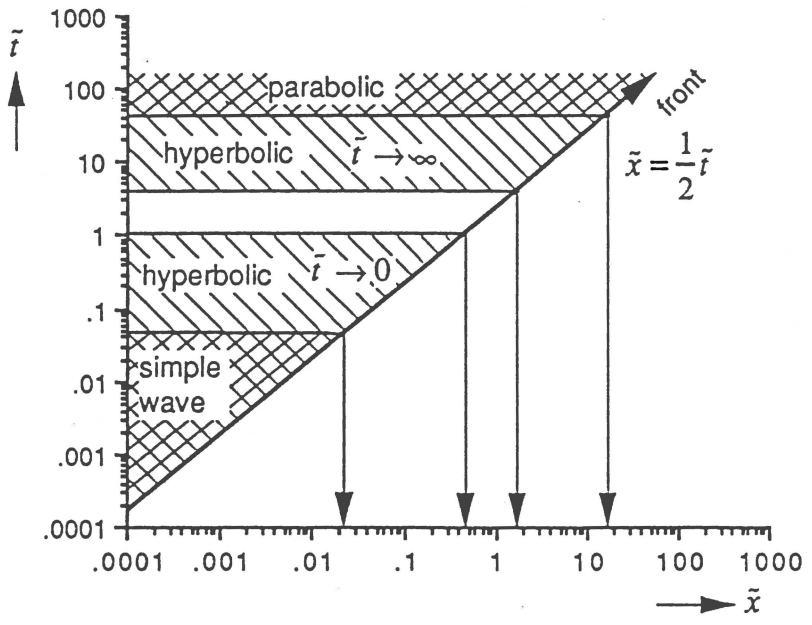


Figure 6.6 Validity diagram for simple wave, hyperbolic and parabolic models (after Ribberink and van der Sande, 1985).

For the three different models above, a validity diagram (Figure 6.6) was presented by Ribberink and van der Sande (1985). Figure 6.6 shows that for a long period (large values of time $\tilde{t}$), diffusion (the damping term in the hyperbolic equation) becomes a dominant process and the hyperbolic model (steady non-uniform flow) gives the same results as those from the parabolic model (steady uniform flow).

Figure 6.7 also shows the difference and likeness between hyperbolic and parabolic models. *"For small values of θ (large values of time) the results of both models approach each other. This effect is stronger for larger values of τ_0, i.e. for larger distances from the origin of the erosion....When non-linear*

effects are negligible, the linearized hyperbolic model presents a good approximation to the phenomenon." (Vreugdenhil and de Vries, 1973).

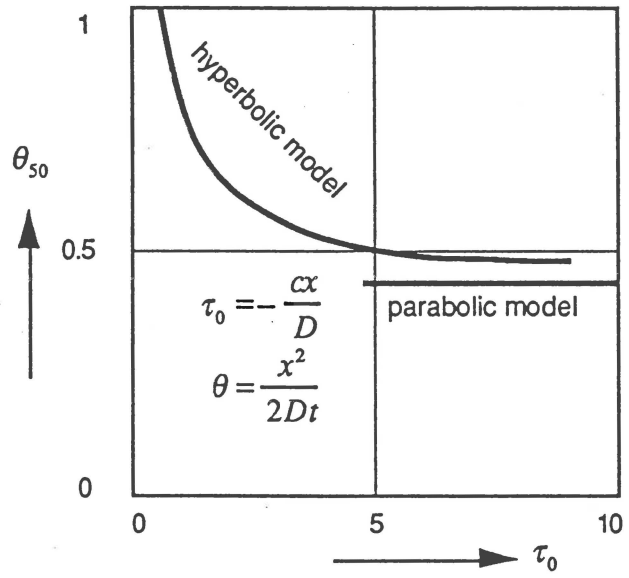


Figure 6.7 Difference between hyperbolic model and parabolic model (after Vreugdenhil and de Vries, 1973).

6.4.3 Non-linear effects

The linear equations in section 6.4.1 were derived based on an assumption of a small perturbation with an initial steady condition (bed continuity). In real rivers, generally the bed level shown in river surveys of distributed cross sections along the river is discontinuity, so that the non-linear effects need to be addressed before the linear equations are applied to the rivers.

In the report of Vreugdenhil and de Vries (1973), a problem of a degrading river with initial bottom discontinuity was solved numerically by using the original governing equations (ref. Eqs. (6.18–20)) in order to compare with the results from the linear hyperbolic model. The conclusion was that if the bottom discontinuity is small (i.e., smaller than 5% of the normal depth of flow), numerical results of the non-linear model are very similar to those of the linear model, but for larger values of bottom discontinuity, numerical results of the original equations show great discrepancies from those of the linear model, but *the non-linear effects are mainly at the places near the bottom discontinuity.*

Therefore if results at the place near the discontinuity are of no interest, the linear models can be still used for larger values of bottom discontinuity. Moreover a steady channel system was defined by Bagnold (1966) as:

> *"The system considered is defined as statistically steady and as representative not of conditions at a single cross-section but of average conditions along a length of channel sufficient to include all repetitive irregularities of slope, cross-section, and boundary."*

Accordingly by using an average of bed level over a certain river reach, the river bottom discontinuity caused by some random variables along the river, such as riffles and pools, width variations, and big bedforms, is expected to be smoothed out. Probably the linear models can be applied to a real river.

6.5 Finite difference approximation of the parabolic and hyperbolic equations

Descriptions of finite difference, finite element and spectral methods can be found in various places (e.g., Richtmyer and Morton, 1967; Zienkiewicz, 1971; Roache, 1972; Potter, 1973; Chung, 1978; Peyret and Taylor, 1983). The finite difference method is applied to the linear parabolic and hyperbolic equations below. The detailed mathematical analyses and discussions on the method are omitted.

6.5.1 Finite difference approximation of a one-dimensional linear parabolic equation

The linear parabolic or diffusion equation, Eq. (6.47), can be written in a general form:

$$\frac{\partial y}{\partial t} - D\frac{\partial^2 y}{\partial x^2} = f(x,t) \qquad a < x < b,\ t \geq 0, \tag{6.66}$$

with the initial condition:

$$y(x,0) = g_0(x), \qquad a < x < b, \tag{6.67}$$

and the boundary conditions:

$$y(a,t) = g_1(t), \qquad t > 0, \tag{6.68-1}$$

$$y(b,t) = g_2(t), \qquad t > 0, \tag{6.68-2}$$

or (ref. Jain, 1981)

$$y(a,t) = g_1(t), \qquad t > 0, \tag{6.69-1}$$

$$-D\frac{\partial y(b,t)}{\partial x} = g_3(t), \qquad t > 0, \tag{6.69-2}$$

where y denotes bed level, D is a coefficient and may be a function of time,

$f(x,t)$ represents a source term, $g_0(x)$, $g_1(t)$ and $g_2(t)$ denote, respectively, the initial bed levels, and variations in bed level at lower and upper boundaries, and $g_3(t)$ represents volumetric bedload supplies in a unit width at the upper boundary.

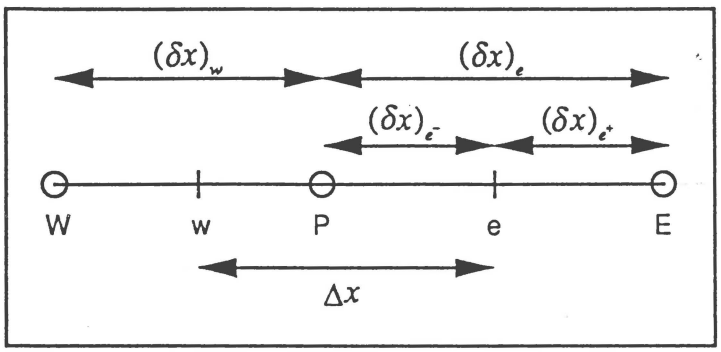


Figure 6.8 Scheme of finite differences.

Eq. (6.66) can be generally approximated by the following weighted average difference equation (ref. Figure 6.8):

$$\frac{y_P^{j+1}-y_P^{j}}{\Delta t}-\theta D^{j+1}\left(\frac{y_E^{j+1}-y_P^{j+1}}{(\delta x)_e}-\frac{y_P^{j+1}-y_W^{j+1}}{(\delta x)_w}\right)/\Delta x$$

$$-(1-\theta)D^{j}\left(\frac{y_E^{j}-y_P^{j}}{(\delta x)_e}-\frac{y_P^{j}-y_W^{j}}{(\delta x)_w}\right)/\Delta x=\bar{f}_P. \qquad (6.70)$$

If $(\delta x)_e=(\delta x)_w=\Delta x$, for the whole domain we have (ref. Figure 6.9)

$$\frac{y_i^{j+1}-y_i^{j}}{\Delta t}-\theta D^{j+1}\left(\frac{y_{i+1}^{j+1}-y_i^{j+1}}{\Delta x}-\frac{y_i^{j+1}-y_{i-1}^{j+1}}{\Delta x}\right)/\Delta x$$

$$-(1-\theta)D^{j}\left(\frac{y_{i+1}^{j}-y_i^{j}}{\Delta x}-\frac{y_i^{j}-y_{i-1}^{j}}{\Delta x}\right)/\Delta x=\bar{f}_i \qquad (6.71)$$

with $i=1,\cdots,n-1$, where, in practice, $0\le\theta\le1$, and y_i^0 is the initial condition, and $i=0$ and $i=n$ represent the lower and upper boundaries respectively (Figure 6.9), $\bar{f}_i$ is a linearization of the source term. For a point source, $\bar{f}_i$ can be represented by

$$\bar{f}_i=\theta f_i^{j+1}+(1-\theta)f_i^{j}. \qquad (6.72)$$

The variables of y_0^j and y_n^j can be obtained from Eqs. (6.68-1–2) for the first kind of boundary conditions:

$$y_0^{j}=g_1^{j}, \qquad (6.73\text{-}1)$$

$$y_n^j = g_2^j, \tag{6.73-2}$$

or from Eqs. (6.69-1–2) by using a central difference scheme to approximate the second kind of boundary conditions at the upper boundary (Smith, 1985):

$$y_0^j = g_1^j, \tag{6.74-1}$$

$$-D^j \frac{y_{n+1}^j - y_{n-1}^j}{2\Delta x} = g_3^j. \tag{6.74-2}$$

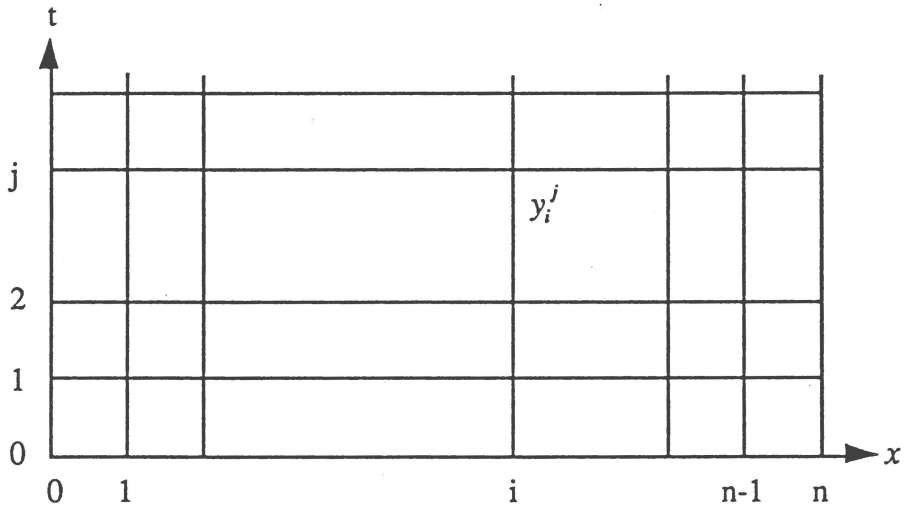


Figure 6.9 Scheme of a finite difference domain.

The variable of y_{n+1}^j is unknown and requires another equation. This can be obtained by assuming that the parabolic equation is satisfied at the end of the boundary ($i = n$ in Eq. (6.71)). Then the unknown y_{n+1}^j can be eliminated between Eqs. (6.71) and (6.74-2).

For a constant D, Eq. (6.71) has the following truncation error $e_T(x,t)$:

$$e_T(x,t) = \frac{1-2\theta}{\theta}\Delta t \frac{\partial^2 y}{\partial t^2} + O(\Delta t^2 + \Delta x^2).$$

Obviously when $\theta = 0.5$, we have a truncation error of order $O(\Delta t^2 + \Delta x^2)$, otherwise $O(\Delta t + \Delta x^2)$. Small Δt and Δx can reduce the truncation error but this usually causes an increase in a round error. So Δt and Δx should be selected to produce the minimum value for both truncation and round errors.

Eq. (6.71) gives an explicit scheme when $\theta = 0$, Crank-Nicolson scheme when $\theta = 0.5$, and a fully implicit scheme when $\theta = 1$. It is unconditionally stable and convergent for $0.5 \le \theta \le 1$, but for $0 \le \theta < 0.5$, the following equation needs

to be satisfied in order to obtain stable numerical solutions:

$$\frac{D\Delta t}{\Delta x^2} \le \frac{1}{2}\frac{1}{1-2\theta}. \tag{6.75}$$

The relationship between y_P and t for these three schemes can be demonstrated in Figure 6.10. Variable $y_P{}^t$ controls the whole time step for the explicit scheme, and variable $y_P{}^{t+\Delta t}$ for the fully implicit scheme. A linear variation of y_P with t in the period Δt is held for the C-N scheme.

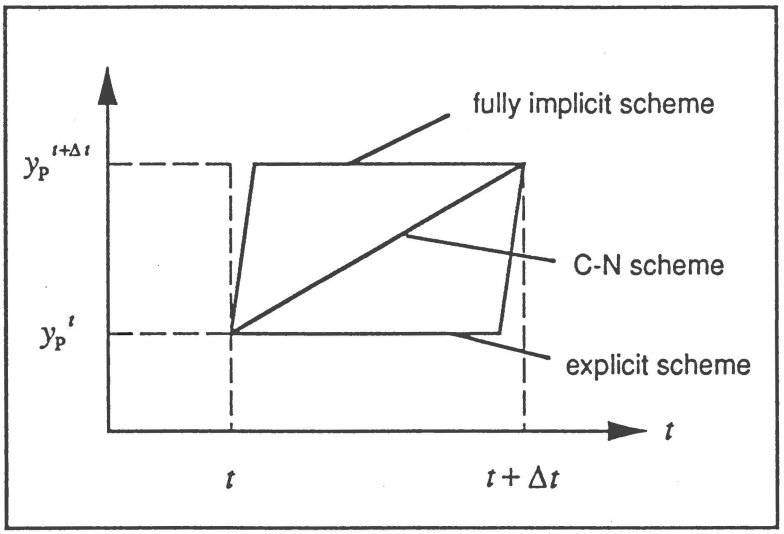


Figure 6.10 Relationship between y_P and t of the three finite difference schemes.

The finite difference equation Eq. (6.71) can be rewritten as

$$\begin{aligned} &-r^{j+1}\theta y_{i-1}{}^{j+1} + \left(1+2r^{j+1}\theta\right)y_i{}^{j+1} - r^{j+1}\theta y_{i+1}{}^{j+1} = \\ &r^j(1-\theta)y_{i-1}{}^j + \left(1-2r^j(1-\theta)\right)y_i{}^j + r^j(1-\theta)y_{i+1}{}^j + \Delta t\bar{f}_i, \end{aligned} \tag{6.76}$$

where $r^j = D^j\Delta t / \Delta x^2$, $i = 1, \cdots, n-1$. If boundary conditions $y_0{}^j$ and $y_n{}^j$ are given by Eqs. (6.73-1–2), the matrix form of Eq. (6.76) is

$$\begin{bmatrix} 1+2r^{j+1}\theta & -r^{j+1}\theta & & & \\ -r^{j+1}\theta & 1+2r^{j+1}\theta & -r^{j+1}\theta & & \\ & \ddots & & & \\ & -r^{j+1}\theta & 1+2r^{j+1}\theta & -r^{j+1}\theta & \\ & & -r^{j+1}\theta & 1+2r^{j+1}\theta & \end{bmatrix} \begin{bmatrix} y_1{}^{j+1} \\ y_2{}^{j+1} \\ \vdots \\ y_{n-2}{}^{j+1} \\ y_{n-1}{}^{j+1} \end{bmatrix} =$$

$$\begin{bmatrix} 1-2r^j(1-\theta) & r^j(1-\theta) & & & & \\ r^j(1-\theta) & 1-2r^j(1-\theta) & r^j(1-\theta) & & & \\ & \ddots & & & & \\ & & r^j(1-\theta) & 1-2r^j(1-\theta) & r^j(1-\theta) & \\ & & & r^j(1-\theta) & 1-2r^j(1-\theta) & \end{bmatrix} \begin{bmatrix} y_1^j \\ y_2^j \\ \\ y_{n-2}^j \\ y_{n-1}^j \end{bmatrix} +$$

$$\begin{bmatrix} r^{j+1}\theta g_1^{j+1} + r^j(1-\theta) g_1^j \\ 0 \\ \vdots \\ 0 \\ r^{j+1}\theta g_2^{j+1} + (1-\theta) g_2^j \end{bmatrix} + \Delta t \begin{bmatrix} \bar{f}_1 \\ \bar{f}_2 \\ \vdots \\ \bar{f}_{n-2} \\ \bar{f}_{n-1} \end{bmatrix}. \tag{6.77}$$

The total of $N-1$ unknown variables, $\mathbf{Y}^{j+1} = (y_1^{j+1}, \cdots, y_{n-1}^{j+1})^T$, $j = 0, 1, 2, \cdots$, in Eq. (6.77) can be solved by using the double sweep method given the initial condition of $\mathbf{Y}^0 = (y_1^0, \cdots, y_{n-1}^0)^T$ and boundary conditions of g_1 and g_2. The subroutine F04EAF of NAG (NAG, 1988) can be applied by using a FORTRAN program.

In the case of given a derivative upper boundary condition, Eq. (6.74-2), we assume that Eq. (6.76) is satisfied at $i = n$. Then combination of these two equations gives

$$\begin{aligned} &-2r^{j+1}\theta y_{n-1}^{j+1} + \left(1 + 2r^{j+1}\theta\right) y_n^{j+1} = 2r^j(1-\theta) y_{n-1}^j + \\ &\left(1 - 2r^j(1-\theta)\right) y_n^j - 2\Delta t \left(\theta g_3^{j+1} + (1-\theta) g_3^j\right) / \Delta x + \Delta t \bar{f}_n. \end{aligned} \tag{6.78}$$

Eq. (6.76) in a matrix form becomes

$$\begin{bmatrix} 1+2r^{j+1}\theta & -r^{j+1}\theta & & & & \\ -r^{j+1}\theta & 1+2r^{j+1}\theta & -r^{j+1}\theta & & & \\ & \ddots & & & & \\ & & -r^{j+1}\theta & 1+2r^{j+1}\theta & -r^{j+1}\theta & \\ & & & -2r^{j+1}\theta & 1+2r^{j+1}\theta & \end{bmatrix} \begin{bmatrix} y_1^{j+1} \\ y_2^{j+1} \\ \vdots \\ y_{n-1}^{j+1} \\ y_n^{j+1} \end{bmatrix} =$$

$$\begin{bmatrix} 1-2r^j(1-\theta) & r^j(1-\theta) & & & & \\ r^j(1-\theta) & 1-2r^j(1-\theta) & r^j(1-\theta) & & & \\ & \ddots & & & & \\ & & r^j(1-\theta) & 1-2r^j(1-\theta) & r^j(1-\theta) & \\ & & & 2r^j(1-\theta) & 1-2r^j(1-\theta) & \end{bmatrix} \begin{bmatrix} y_1^j \\ y_2^j \\ \vdots \\ y_{n-1}^j \\ y_n^j \end{bmatrix} +$$

$$\begin{bmatrix} r^{j+1}\theta g_1^{j+1}+r^j(1-\theta)g_1^j \\ 0 \\ \vdots \\ 0 \\ -2\Delta t\left(\theta g_3^{j+1}+(1-\theta)g_3^j\right)/\Delta x \end{bmatrix}+\Delta t\begin{bmatrix} \bar{f}_1 \\ \bar{f}_2 \\ \vdots \\ \bar{f}_{n-1} \\ \bar{f}_n \end{bmatrix}. \tag{6.79}$$

The total of N unknown variables, $\mathbf{Y}^{j+1}=(y_1^{j+1},\cdots,y_n^{j+1})^T$, $j=0,\ 1,\ 2,\ \cdots$, in Eq. (6.79) can be solved by using the same double sweep method given $\mathbf{Y}^0=(y_1^0,\cdots,y_n^0)^T$ and the boundary conditions of g_1 and g_3.

6.5.2 Finite difference approximation of a one-dimensional linear hyperbolic equation

The hyperbolic equation, Eq. (6.49b), can be written in a general form:

$$\frac{\partial y}{\partial t}-D\frac{\partial^2 y}{\partial x^2}-F\frac{\partial^2 y}{\partial x\partial t}=f(x,t) \qquad a<x<b,\ t\geq 0 \tag{6.80}$$

with the same initial and boundary conditions used by the parabolic equation—Eqs. (6.67, 68-1–2, 69-1–2). Both parameters D and F can be a function of time.

If the weighted average approximation used above is applied to Eq. (6.80) and different weight factors are used for the diffusion and mixed terms, then by using the central difference scheme we have

$$\begin{aligned} &\frac{y_i^{j+1}-y_i^j}{\Delta t}-\theta D^{j+1}\frac{y_{i+1}^{j+1}-2y_i^{j+1}-y_{i-1}^{j+1}}{\Delta x^2}-(1-\theta)D^j\frac{y_{i+1}^{j}-2y_i^{j}-y_{i-1}^{j}}{\Delta x^2} \\ &-\beta\frac{F^{j+2}(y_{i+1}^{j+2}-y_{i-1}^{j+2})-F^{j+1}(y_{i+1}^{j+1}-y_{i-1}^{j+1})}{2\Delta t\Delta x} \\ &-(1-\beta)\frac{F^{j+1}(y_{i+1}^{j+1}-y_{i-1}^{j+1})-F^{j}(y_{i+1}^{j}-y_{i-1}^{j})}{2\Delta t\Delta x}=\bar{f}_i \end{aligned} \tag{6.81}$$

with $i=1,\ \cdots,\ n-1$, $0\leq\theta\leq 1$ and $0\leq\beta\leq 1$. If $\beta\neq 0$, the initial conditions in two time levels of $\mathbf{Y}^j=(y_1^j,\cdots,y_{n-1}^j)^T$, $j=0$ and 1, and boundary conditions have to be given in order to obtain $\mathbf{Y}^j$, $j=2,\ 3,\ \cdots$, from Eq. (6.81). In practice it is very difficult to give initial conditions in two time levels. Therefore let $\beta=0$, Eq. (6.81) becomes

$$\begin{aligned} &\left(-r^{j+1}\theta+\frac{F^{j+1}}{2\Delta x}\right)y_{i-1}^{j+1}+\left(1+2r^{j+1}\theta\right)y_i^{j+1}-\left(r^{j+1}\theta+\frac{F^{j+1}}{2\Delta x}\right)y_{i+1}^{j+1} \\ &=\left(r^j(1-\theta)+\frac{F^j}{2\Delta x}\right)y_{i-1}^{j}+\left(1-2r^j(1-\theta)\right)y_i^j \end{aligned}$$

$$+\left(r^j(1-\theta)-\frac{F^j}{2\Delta x}\right)y_{i+1}{}^j+\Delta t\bar{f}_i, \tag{6.82}$$

where $r^j=D^j\Delta t/\Delta x^2$. If initial and boundary conditions are given by $y_i{}^0$, $y_0{}^j$ and $y_n{}^j$ (ref. Eqs. (6.73-1–2)), $\mathbf{Y}^{j+1}=(y_1{}^{j+1},\cdots,y_{n-1}{}^{j+1})^T$ with $j=0,\ 1,\ 2,\ \cdots$ can be solved by using the same method for the parabolic equation.

For the upper and lower boundary conditions given respectively by Eqs. (6.74-2) and (6.74-1), it is assumed that Eq. (6.82) is satisfied at $i=n$, then we have

$$\begin{aligned}&-2r^{j+1}\theta y_{n-1}{}^{j+1}+\left(1+2r^{j+1}\theta\right)y_n{}^{j+1}=2r^j(1-\theta)y_{n-1}{}^j+\left(1-2r^j(1-\theta)\right)y_n{}^j\\&-2\Delta t\left(\theta g_3{}^{j+1}+(1-\theta)g_3{}^j\right)/\Delta x-2\left(F^{j+1}g_3{}^{j+1}/D^{j+1}-F^j g_3{}^j/D^j\right)+\Delta t\bar{f}_n.\end{aligned} \tag{6.83}$$

The unknown $\mathbf{Y}^{j+1}=(y_1{}^{j+1},\cdots,y_n{}^{j+1})^T$ with $j=0,\ 1,\ 2,\ \cdots$ can be solved.

If the mixed term is approximated by using backward-difference formula, which is normally applied to a convective term, then Eq. (6.80) becomes

$$\begin{aligned}&\frac{y_i{}^{j+1}-y_i{}^j}{\Delta t}-\theta D^{j+1}\frac{y_{i+1}{}^{j+1}-2y_i{}^{j+1}-y_{i-1}{}^{j+1}}{\Delta x^2}-(1-\theta)D^j\frac{y_{i+1}{}^j-2y_i{}^j-y_{i-1}{}^j}{\Delta x^2}\\&-\beta\frac{F^{j+2}(y_i{}^{j+2}-y_{i-1}{}^{j+2})-F^{j+1}(y_i{}^{j+1}-y_{i-1}{}^{j+1})}{\Delta t\Delta x}\\&-(1-\beta)\frac{F^{j+1}(y_i{}^{j+1}-y_{i-1}{}^{j+1})-F^j(y_i{}^j-y_{i-1}{}^j)}{\Delta t\Delta x}=\bar{f}_i.\end{aligned} \tag{6.84}$$

Let $\beta=0$, we have

$$\begin{aligned}&\left(-r^{j+1}\theta+\frac{F^{j+1}}{\Delta x}\right)y_{i-1}{}^{j+1}+\left(1+2r^{j+1}\theta-\frac{F^{j+1}}{\Delta x}\right)y_i{}^{j+1}-r^{j+1}\theta y_{i+1}{}^{j+1}=\\&\left(r^j(1-\theta)+\frac{F^j}{\Delta x}\right)y_{i-1}{}^j+\left(1-2r^j(1-\theta)-\frac{F^j}{\Delta x}\right)y_i{}^j+r^j(1-\theta)y_{i+1}{}^j+\Delta t\bar{f}_i\end{aligned} \tag{6.85}$$

and given conditions of $y_i{}^0$, $y_0{}^j$ and $y_n{}^j$ (ref. Eqs. (6.73-1–2)), the known $\mathbf{Y}^{j+1}=(y_1{}^{j+1},\cdots,y_{n-1}{}^{j+1})^T$ with $j=0,\ 1,\ 2,\ \cdots$ can be solved.

We can also treat the mixed term in Eq. (6.80) as a source term:

$$S=F\frac{\partial^2 y}{\partial x\partial t}, \tag{6.86}$$

and solve Eq. (6.80) alternatively. Each step applies the same method used in the parabolic equation:

1. At the beginning, $S_i{}^j=0$ is assumed and then the first estimate of $\mathbf{Y}^{j+1}=\hat{\mathbf{Y}}_{(1)}{}^{j+1}=(\hat{y}_1{}^{j+1},\cdots,\hat{y}_n{}^{j+1})^T$ is obtained given $\mathbf{Y}^j=(y_1{}^j,\cdots,y_n{}^j)^T$, $j=0,\ 1,\ 2,\ \cdots$.

2. New S_i^j is determined by applying a backward difference equation to Eq. (6.86) given $\hat{\mathbf{Y}}_{(1)}^{j+1}$ in step 1 and $\mathbf{Y}^j$.

3. Solving the equation again by using the new S_i^j from step 2, we have the result $\mathbf{Y}^{j+1} = \hat{\mathbf{Y}}_{(2)}^{j+1}$.

4. If $\left|\hat{y}_{(2),i}^{j+1} - \hat{y}_{(1),i}^{j+1}\right| \leq \varepsilon$, $i = 1, \cdots, n$, $\mathbf{Y}^{j+1} = \hat{\mathbf{Y}}_{(2)}^{j+1}$ is the final result, where ε is a given small figure. Otherwise using $\hat{\mathbf{Y}}_{(2)}^{j+1}$ instead of $\hat{\mathbf{Y}}_{(1)}^{j+1}$, we have to go to step 2 again.

The stability of this method is an open question. Probably the condition $\left|\hat{y}_{(2),i}^{j+1} - \hat{y}_{(1),i}^{j+1}\right| \leq \varepsilon$ will be never met because of numerical divergence.

6.6 Discussions and conclusions

The one-dimensional water and bedload movement model is based on conservation of sediment mass, water mass and water momentum (Jansen, *et al.*, 1979). Under an assumption of equilibrium flows, the bedload transportation is determined by flow parameters.

Three governing equations comprise a quasi-linear hyperbolic system with no analytical solutions available given proper boundary and initial conditions.

There are two disparate time scales in the system for bedload movement and water movement, resulting singular-perturbation characteristics, and approximations based on a regular perturbation cannot satisfy either arbitrary boundary or initial conditions.

Decoupled techniques have been used in water and bedload movement modelling. There are two types of decoupled model. One of them decouples the hydraulic equations and the bedload balance equation assuming no changes in bed level within a time step. This procedure could make the problem ill-posed with no freedom to pose the bed level condition at the upper boundary. The other uses a constant water flow assumption, which results in requiring a particular initial condition.

For the upper boundary condition concerning bed levels, bed level variations or its derivative with respect to x have to be specified. If bedload supplies are given, as in most cases, its relation to bed level or its derivative with respect to x should be established.

Linearization of the water and bedload movement system with a quasi steady flow produces a hyperbolic equation with a damping term or even a parabolic equation (uniform flow). Both equations are derived under an assumption of small perturbation on an initial steady condition or bed continuity. For large values of time or large distance from the original disturbance, the parabolic equation is a good approximation of the hyperbolic equation.

Non-linear effects of bottom discontinuity are proportional to its value and mainly at the places near the discontinuity.

If an average of bed level over a certain length of river channel is used and results around big bed level discontinuity are of no interest, probably the linear hyperbolic equation and its long term approximation—the parabolic equation—can be applied to a real river. In this case, the bottom discontinuity caused by some random variables along the river course, such as riffles and pools, width variations, and big bedforms, is expected to be smoothed out.

Numerical schemes for solution of the linear hyperbolic and parabolic equations have been presented. Their application to the lower Waikato River will be presented in Chapter Seven.

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CHAPTER SEVEN

INVESTIGATION OF THE FUTURE BED LEVELS AT HUNTLY IN THE LOWER WAIKATO RIVER

Chapter Seven

Investigation of the Future River Bed Levels at Huntly in the Lower Waikato River

7.1 Introduction

The linear parabolic and hyperbolic equations discussed in Chapter Six are applied in this chapter to the reach from Mercer to Ngaruawahia in the lower Waikato River. Data used for calibration and validation of the models are generally based on the results of the water level profiles for the discharge of 350 m^3/s in Chapter Four. The main purpose of this study is to predict the future river bed levels at Huntly in the year 2040 for different scenarios. Potential effects of bed level reduction on operation of the present cooling water system at the Huntly Power Station are been assessed for given river discharges.

7.2 Assumptions, field data and parameters

7.2.1 Assumptions

For bedload movement with an equilibrium and quasi-steady flow, the linear parabolic (uniform flow) and hyperbolic (non-uniform flow) equations were derived under an assumption of small perturbation with an initial bed continuity (steady) condition and a constant width (Chapter Six).

Before the equations are applied to a river, the above assumption of bed continuity in unit width should be justified.

A normal cross section drawn on an undistorted scale is very useful when schematising a river course. Figure 7.1 shows a river cross section with a 200 m width at a bank-full flow of 4 m depth (Figure 7.1-A) and with a 300 m width at 3 m depth (Figure 7.1-B). These two cross sections can exist in a natural river within a distance of several meters or a few hundred meters. In this case, if their average width of 250 m is adopted, the relative difference is ±20% which is acceptable. "*In general, probable errors in predictions of*

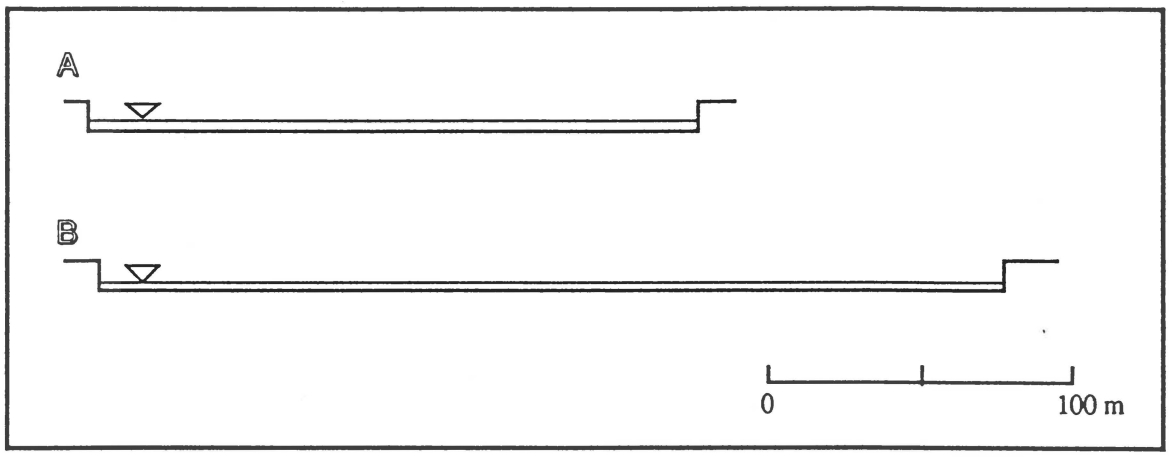


Figure 7.1 River cross sections in an undistorted scale.

±25%, or even ±50%, should not be considered unusual in this business." (Fischer, *et al.*, 1979, page 11)

Figure 7.2 shows variations in cross-sectional width in the lower Waikato River from the 1985/1989 survey. Generally the number of wide cross sections in the reach between 40–70 km is much more than that between 70–100 km. However the total number of cross sections surveyed in the latter reach is smaller.

Visual observation from a boat in the lower Waikato River in 1991 indicated that there were considerable amounts of aquatic vegetation on the river bed, with corresponding impeded sediment transport, especially at the places near river banks with a wide cross-sectional width. Because in practice an average of river conditions over a certain length of channel is usually of interest, it is probably acceptable to assume a constant representative river width in the reach between Mercer and Ngaruawahia for an analysis of long term changes in average bed level. Thus a unit width approach can be applied.

Bed levels in a natural river vary not only along the river course but also across the river. Some variations in bed level are caused by bedforms, others by riffles and pools, bedrock controls or knick points, effects of river width variations and others. Therefore the river bed levels are always changing and never smooth in a small scale (order of the river width or less). For example, the bed levels along the lower Waikato River changed by over 3 m within just about 20 m at Huntly (W.V.A., plan 1646/434004) because of existence of a big bedform.

Figure 7.3 shows big variations in depth, measured along the lower Waikato

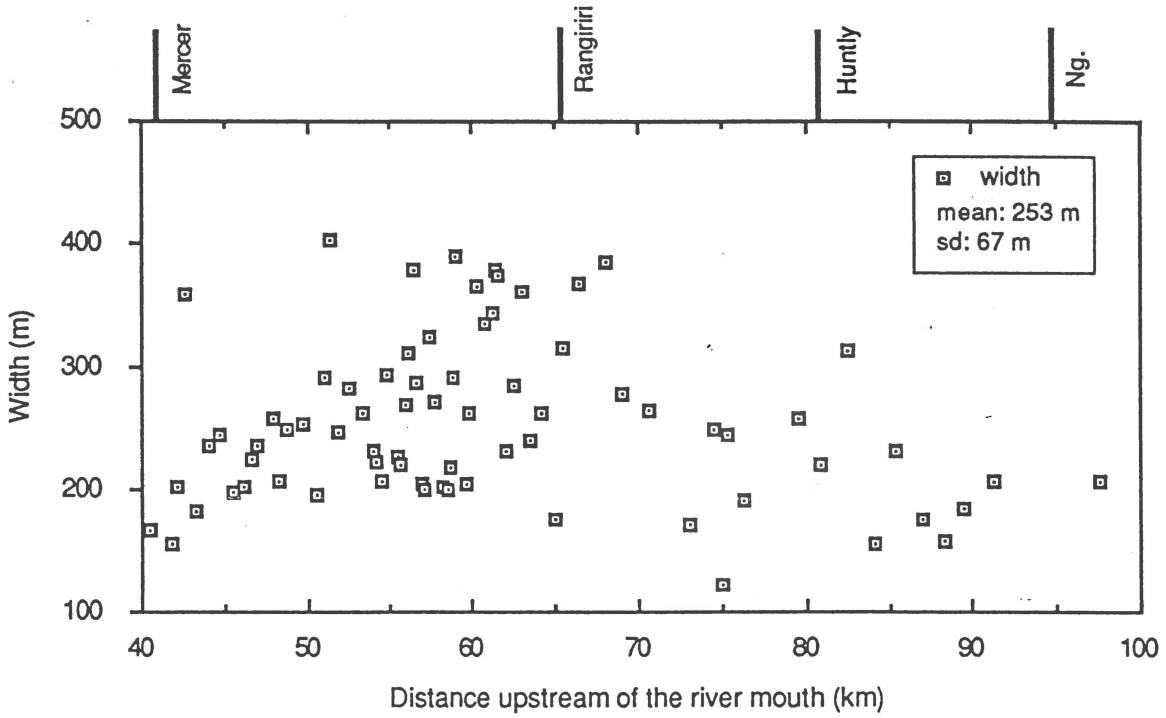


Figure 7.2 Variations in channel width in the lower Waikato River (data from the Waikato Regional Council 1985/1989 survey).

River nearly in the middle channel by using echo soundings, in terms of water level at 4 m in the reach between 40–100 km (Fenton, 1989, Figure 3.11). Generally bed levels varied randomly by up to 3 m along the river.

Mean bed levels of cross sections surveyed in the Waikato River also tend to show noisy variations along the river course (ref. Figure 4.3), that may partly result from the river width variations along the river (Figure 7.2), and big bedform occurrences (Figure 2.6).

Because a steady river system should represent an average of conditions along a length of channel sufficient to include all repetitive irregularities of slope, cross section and boundary, rather than conditions at a single cross section (Bagnold, 1966), it can be expected that a steady bed level in the lower Waikato River, determined by using an average of bed levels over a certain length of the river, say 1 000 m or more, should vary smoothly along the river course as the longitudinal discontinuity caused by some random variables is smoothed out. Unfortunately no data are available to undertake a quantitative analysis in the lower Waikato River.

Yang (1971) used the concept of entropy to demonstrate that the longitudinal bed profile for a maturity river, on average, should follow an exponential equation, which is a smooth line.

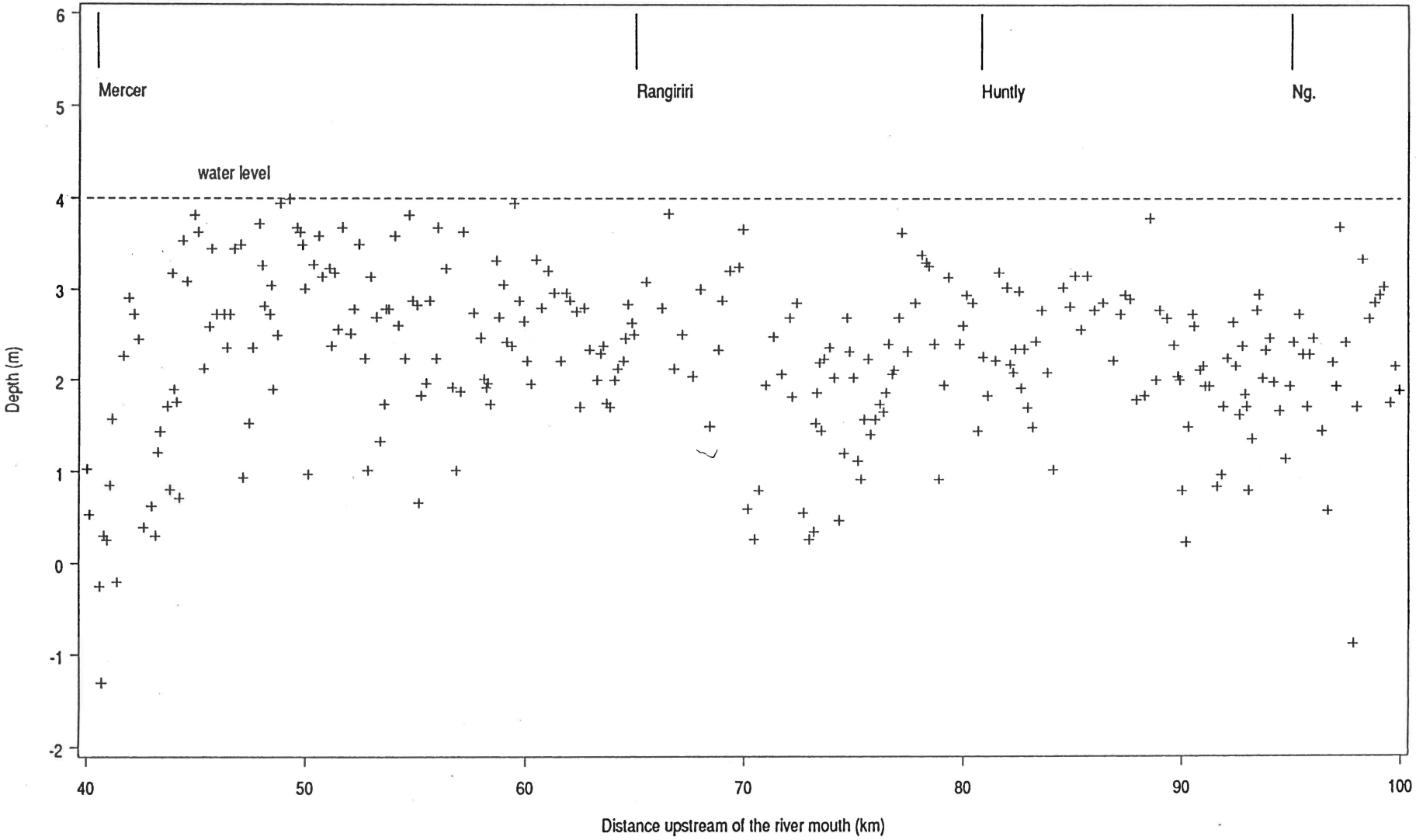


Figure 7.3 Variations in depth in the middle channel of the lower Waikato River in the reach between 40-100 km in terms of water level at 4 m (data from Fenton, 1989). The figure shows random variations in bed level along the river.

The discussions above demonstrated that generally longitudinal bed levels in the lower Waikato River exhibit large random variations. However an average of bed levels over a certain length of channel is expected to vary smoothly along the river. Thus the linear equations can be applied.

It is worth mentioning again that the non-linear effects caused by a bottom (bed level) discontinuity, such as bedrock controls, are mainly at the places just near the discontinuity (Vreugdenhil and de Vries, 1973). The linear models can still be used for the entire reach if one is not interested in such local effects.

7.2.2 Field data

An initial steady bed level should be determined by using a river contour map measured under steady conditions and taking an average of bed levels over a certain distance along the river. It is too expensive to obtain an entire river contour map. In addition, under natural environments steady conditions are never attained at any time in the whole river reach studied. Another method is to smooth or to take an average of mean bed levels, measured under a near steady condition, along the river if considerable representative cross-sectional data are available. We can also use theories and field data together to obtain the initial steady bed level with some assumptions.

In Chapter Four, a relationship between the water level at the discharge of 350 m^3/s ($WL_{Q=350}$) and the mean bed level (BL) was established. According to Eq. (4.21), we have

$$WL_{Q=350}(x,t) = BL(x,t) + \left(\frac{350n(x,t)}{W(x,t)\sqrt{S(x,t)}} \right)^{3/5}, \tag{7.1}$$

where n, W and S are, respectively, Manning's coefficient, the river width and the energy slope.

If an average of the second term on the right hand side of Eq. (7.1) over a certain distance, ΔX (1 000 m or more as indicated above), along the river between Mercer and Ngaruawahia can be obtained, the average bed levels then can be approximately determined by using the water level profile at the discharge of 350 m^3/s. The water level profile is generally smooth and continuous in the lower Waikato River (Figure 4.30).

The second term on the right hand side of Eq. (7.1) is a function of time and distance. It can be assumed that in the reach between Mercer and

Ngaruawahia, this term is a constant through time (section 4.3). However its average over a certain distance may vary along the river.

Figure 7.4 shows the water level profile in July 1963 at the discharge of 350 m^3/s between Ngaruawahia and Mercer (Appendix Seven), the mean bed levels and their smoothed profile from the 1963/1964 survey in the reach between cross sections 32–132 (28.56–94.45 km upstream of the river mouth). The smoothed profile was determined by using the cubic spline method (section 4.2). Although sand mining operations on the river occurred around Mercer and elsewhere at that time, the amount of sand extracted was relatively small in comparison to the total volume of sand taken from the river up to now, and the river can be expected to be close to a steady state at that time. The smoothed profile of the mean bed levels along the river in Figure 7.4 seem to indicate that it follows approximately a straight line. Generally the water depth tends to be greater in the upstream direction, which may show the effects of non-constancy over distance of the second term on the right hand side of Eq. (7.1).

It is very difficult to determine the second term on the right hand side of Eq. (7.1) from each individual width, Manning's coefficient and energy slope.

Between Mercer and Ngaruawahia, variations in width at the cross sections surveyed in 1958–1989 are shown in Figure 7.2 and generally the width seems to decrease somewhat in the upstream direction. For Manning's coefficient n, two different figures, 0.030 and 0.038, were suggested in the reaches of about 40–75 km and 75–95 km respectively (Mulholland, 1983). Because these figures were obtained by using the 1958 flood data only, they may not be representative for lower and moderate flow conditions. Few data are available about variations in energy slope along the river. Therefore it is impossible to get the initial steady bed levels based entirely on Eq. (7.1).

According to Figure 7.4, we assume that the river in the reach between Mercer and Ngaruawahia was in a steady state in June 1958 because during that time minimal human activities (sand extraction) occurred on the river, and the bed levels followed a straight line. The line can be represented by the following equation:

$$BL(x,t_0) = 1.60 + (1.60 - 7.67) \times \frac{x - 40.6}{40.6 - 94.45} \qquad x: 40 - 95 \text{ km}, \qquad (7.2)$$

where t_0 is the time of the initial steady state, June 1958, and x is the distance upstream of the river mouth (km).

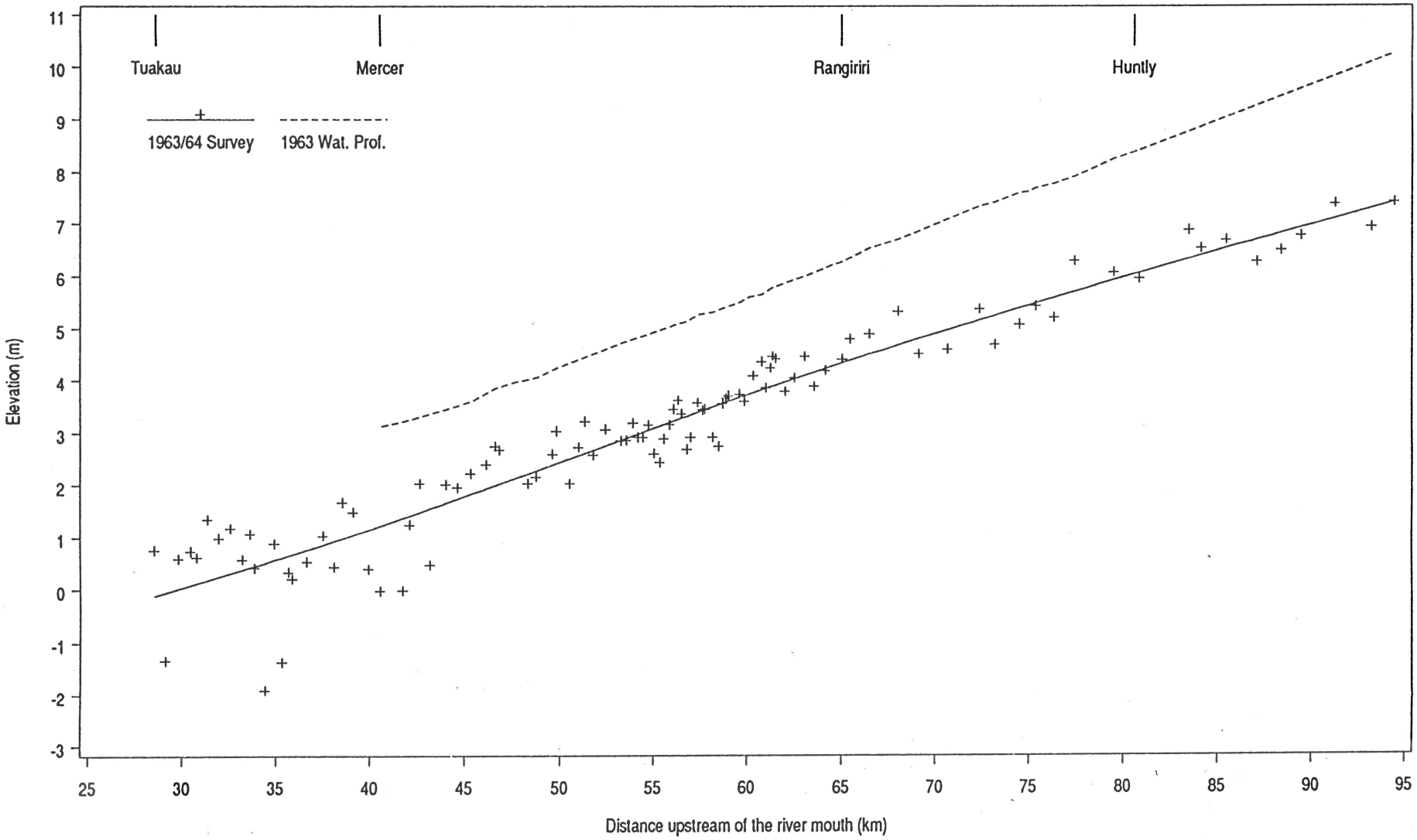


Figure 7.4 Comparison of the profiles of water levels and smoothed mean bed levels in 1963-1964. The smoothed line is determined by using a cubic spline method and software SAS with option I=SM65. The figure shows a trend of increase in water depth upstream.

As for bed levels after June 1958, they can be obtained from the known water level variations by using

$$BL(x,\Delta t+t_0)=BL(x,t_0)+WL_{Q=350}(x,\Delta t+t_0)-WL_{Q=350}(x,t_0). \tag{7.3}$$

Eq. (7.3) is derived from Eq. (7.1) under the assumption that the second term on its right hand side is a constant over time.

Figures 7.5–6 show, respectively, the bed level profiles in the reach between Mercer and Ngaruawahia in July 1963 and May 1982. The profiles were determined by using Eqs. (7.2–3) and the corresponding water level profiles at the discharge of 350 m^3/s. The mean bed levels from the 1963/1964 and 1979/1983 surveys are also plotted together for comparison. Generally the bed level profiles derived from the above method can represent the trend of changes in mean bed level of the cross sections surveyed along the river.

The bed level profiles in June 1958, May 1982 and December 1990, which were obtained by the above method, are illustrated in Figure 7.7. They have been used for an initial condition, model calibration and validation respectively. The original data have been included in Appendix Eight.

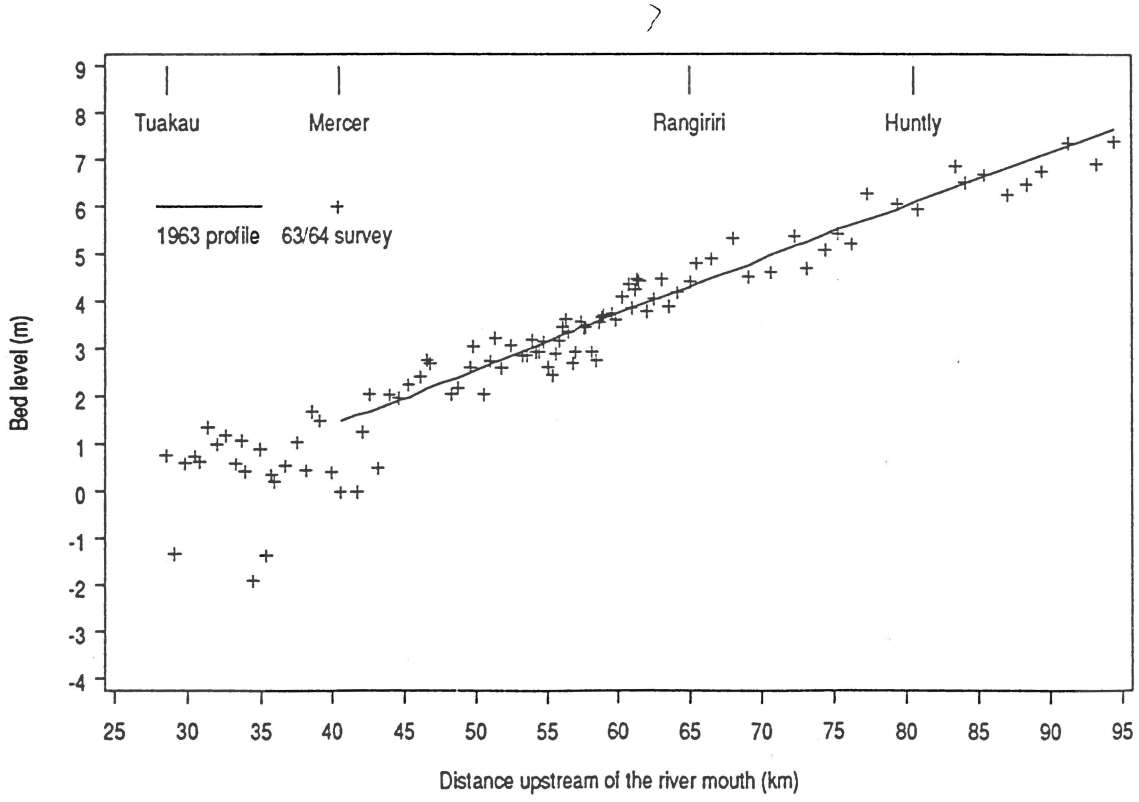


Figure 7.5 Comparison of the bed level profile in July 1963 and the mean bed levels from the 1963/1964 survey (data from the river surveys held at the Waikato Regional Council). The profile was obtained by using Eqs. (7.2–3) and the water level profile at the discharge of 350 m^3/s.

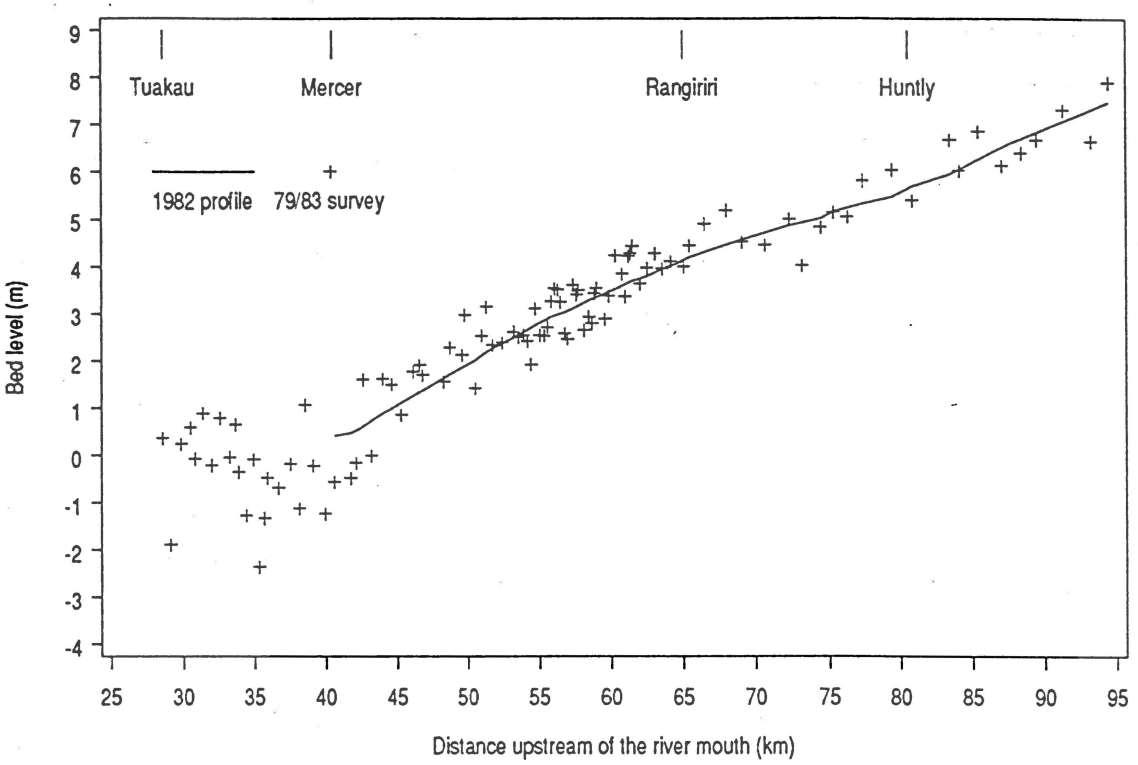


Figure 7.6 Comparison of the bed level profile in May 1982 and the mean bed levels from the 1979/1983 survey (data from the river surveys held at the Waikato Regional Council). The profile was obtained by using Eqs. (7.2–3) and the water level profile at the discharge of 350 m^3/s.

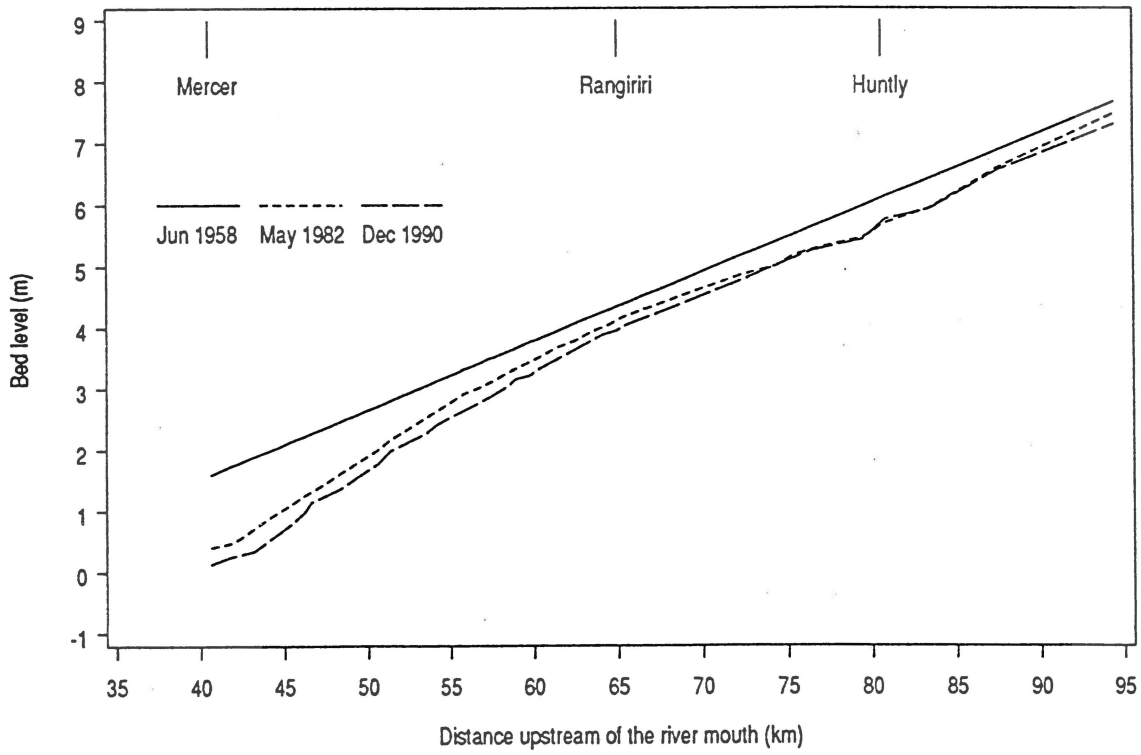


Figure 7.7 Bed level profiles in June 1958, May 1982 and December 1990. They were derived by using Eqs. (7.2–3) and the corresponding water level profiles at the discharge of 350 m^3/s.

7.2.3 Parameters

There are two parameters used in the parabolic and hyperbolic equations, D and c, which need to be determined by calibration. For the reach between Mercer and Ngaruawahia, their first estimates are discussed below.

From Eqs. (6.8) and (6.48), the parameter D can be initially approximated by using the following D_0:

$$D_0 = \frac{1}{3}\frac{b\bar{q}_s}{i_0}, \tag{7.4}$$

or

$$D_0 = \frac{1}{3}\frac{b\bar{Q}_s}{Wi_0}, \tag{7.5}$$

where $\bar{q}_s = \bar{Q}_s / W$, $\bar{Q}_s$ is an average of volumetric bedload discharge, b is a parameter used in the bedload transport equation (Eq. (6.8)), W is the river width and i_0 is the average bed slope.

If variations of D through time are considered, the following linearly-adjusted equation through discharges can be applied:

$$D = \frac{Q_T}{\bar{Q}_T} D_0, \tag{7.6}$$

where Q_T is the discharge at time T and $\bar{Q}_T$ is its long term average.

Given the parameter D as well as proper boundary and initial conditions, the parabolic equation can be solved numerically (Chapter Six).

Parameter c can be approximated initially by using an average of the propagation of a small bed disturbance denoted by c_0 from the following equation (Jansen, *et al.*, 1979):

$$c_0 = u_0 \frac{\psi_0}{1 - Fr_0^2} \tag{7.7}$$

with (ref. Eq. (6.10))

$$\psi = \frac{df / du}{h} = \frac{bau^{b-1}}{h} = b\frac{q_s}{q} = b\frac{Q_s / W}{Q / W} = b\frac{Q_s}{Q}, \tag{7.8}$$

$$Fr = \frac{u}{\sqrt{gh}}, \tag{7.9}$$

where u and h are the flow velocity and the water depth respectively, and a

subscript "0" represents an annual average, Q and Q_s are, respectively, the water and bedload discharges.

Considering variations of the variable c over time, the following equation similar to Eq. (7.6) can be used:

$$c = \frac{Q_T}{\overline{Q_T}} c_0. \tag{7.10}$$

Given the parameters D and c as well as proper boundary and initial conditions, the hyperbolic equation can be solved numerically (Chapter Six).

7.3 Calibration and validation of the parabolic and hyperbolic models

The field data used for initial conditions, calibration and validation in the parabolic and hyperbolic models have been discussed in section 7.2.2.

The domain was in the reach between 48.25–94.45 km upstream of the river mouth. The downstream boundary was selected at 48.25 km because in most cases this location was close to the upstream limit for licensed sand extraction around Mercer and Meremere (ref. Figure 4.33). The x axis is from the downstream to the upstream.

A constant of 0.15 km was used as a spatial step (Δx_i) and the number of total grid points in the domain was 309 ($i = \overline{0,308}$), including both upstream and downstream boundaries. For the temporal step (Δt_j), 1 month was applied. We assumed $t_0 = 0$ or $j = 0$ for the initial time of June 1958. In the following, j was used instead of t_j for the temporal grid. Given the month (1–12) and year, temporal grids can be obtained by using the following equation:

$$j = (\text{year} - 1958) \times 12 + (\text{month} - 6). \tag{7.11}$$

We have $j = 287$ and $j = 390$ for May 1982 and December 1990 respectively.

Bed levels at the spatial grids were determined by linear interpolation of the field data at two immediate cross sections (Appendix Eight).

At the downstream boundary of 48.25 km ($i = 0$), bed levels were specified. Table 7.1 lists the bed levels in June 1958, May 1982 and December 1990. Total bed level reductions between June 1958 and May 1982, and May 1982 and December 1990 are, respectively, 0.855 m (2.462–1.607 m) and 0.257 m (1.607–1.350 m).

Table 7.1 Bed levels at both upstream and downstream boundaries in June 1958, May 1982 and December 1990 from the field data, and average annual volumetric bedload supplies (G_0) at the upstream boundary in June 1958, December 1980, and December 1990 ($0< \kappa <1$).

Place	$i=0$ (48.25 km)			$i=308$ (94.45 km)			
Time	Jun 1958	May 1982	Dec 1990	Jun 1958	Dec 1980	May 1982	Dec 1990
j (time grid)	0	287	390	0	270	287	390
Bed Level m.	2.462	1.607	1.350	7.670		7.480	7.300
G_0 1000 m^3/yr				160±24	160±24		(160±24) κ

Because of the relationship between the cumulative volume of sand extraction around Mercer and reduction in water and bed levels at Mercer Wharf (ref. Figure 4.35), the monthly bed levels BL_j at the downstream boundary between June 1958 and December 1990 ($j=\overline{0,\ 390}$) can be approximated by using the following equations given a monthly cumulative sand extraction volume $CSEV_j$ (m^3) ($CSEV_0=0$ in June 1958):

$$BL_j = BL_0 - \frac{0.855}{5676398} CSEV_j, \quad j=\overline{0,\ 287}, \tag{7.12}$$

$$BL_j = BL_{287} - \frac{0.257}{1904407}(CSEV_j - 5676398), \quad j=\overline{287,\ 390}, \tag{7.13}$$

where BL_0=2.462, which is the initial bed level (m) in June 1958, 0.855 is the total reduction in bed level (m) between June 1958 and May 1982 ($j=\overline{0,\ 287}$), and 5 676 398 is the total sand volume (m^3) taken from the river around Mercer in the same period ($j=\overline{0,\ 287}$); BL_{287}=1.607 (m), which is the bed level in May 1982, 0.257 is the total reduction in bed level (m) between May 1982 and December 1990 ($j=\overline{287,\ 390}$), and 1 904 407 is the total sand volume (m^3) taken from the river around Mercer in the same period ($j=\overline{287,\ 390}$).

Dividing the quarterly sand extraction volumes (Fenton, 1989; Waikato Regional Council files) by 3 to produce a monthly sand extraction volume, the monthly cumulative sand extraction volumes around Mercer from 1958 to 1990 have been obtained (Appendix Nine). Thus $CSEV_j$ is known and Eqs. (7.12–13) can be applied.

At the upstream boundary of 94.45 km ($i=308$ near Ngaruawahia), the bed levels in June 1958, May 1982 and December 1990 were known. Linear variations in bed level between June 1958 and May 1982 ($j=\overline{0,\ 287}$), and May 1982 and December 1990 ($j=\overline{287,\ 390}$) were assumed. Results of the analysis of trends in water and bed levels seem to support this assumption (Chapter Four). By using the data in Table 7.1, bed levels BL_j (m; $j=\overline{0,\ 390}$) at the upstream boundary can be determined by the following equations:

$$BL_j = BL_0 + \frac{BL_0 - BL_{287}}{0-287}(j-0), \quad j=\overline{0,\ 287}, \tag{7.14}$$

$$BL_j = BL_{287} + \frac{BL_{287} - BL_{390}}{287-390}(j-287), \quad j=\overline{287,\ 390}, \tag{7.15}$$

where BL_0=7.670 m, BL_{287}=7.480 m and BL_{390}=7.300 m (Table 7.1).

Bedload supplies were also specified at the upstream boundary (Table 7.1) in order to use this kind of condition in the model application. The annual average volumetric bedload supply between June 1958 and December 1980 ($j=\overline{0,\ 270}$) was assumed to be G_0, which should be given within 160 000±24 000 m^3/yr (Chapter Five). From December 1980 to December 1990 ($j=\overline{270,\ 390}$) it was assumed to reduce linearly to κG_0 ($0<\kappa<1$) because of cessation of the sand mining operation on the river upstream of Ngaruawahia in 1976, which disturbed the sand-gravel bed surface allowing more sands below the surface to move to the river downstream. Considering monthly bedload supplies (ref. Eq. (7.6)), they can be indicated by the following equations (ref Eq. (7.6)):

$$G_j = -\frac{Q_T}{\overline{Q_T}}\frac{G_0}{12} \ (\mathrm{m^3/month}), \quad j=\overline{0,\ 270}, \tag{7.16}$$

$$G_j = -\frac{Q_T}{\overline{Q_T}}\left[1+\frac{1-\kappa}{270-390}(j-270)\right]\frac{G_0}{12} \ (\mathrm{m^3/month}), \quad j=\overline{270,\ 390}, \tag{7.17}$$

where the negative means that bedloads were supplied into the domain at the upstream boundary. When they are used in the model, a width has to be specified in order to give bedload supplies per unit width.

The sand extraction volumes taken in the reach between Mercer and Ngaruawahia (Figure 4.34B) were treated as a sink term in the models. Extraction for the power station was assumed to occur over a distance of 1 050 m from 79.675 to 80.725 km (grid points 210–216). The volumes of sands taken for stopbank ballasting and by Rooses Co. were, respectively, assumed to be linearly distributed in the reach from 69.925–85.075 km (grid points 145–245) and from 78.025–79.675 km (grid points 199–209).

Because of a difference between the dry bulk sand density and the wet bulk sand density, sand extraction volumes measured commercially for stopbank ballasting and by Rooses Co. were multiplied by 1.166 when they were used in the model as a sink term (Chapter Three). The monthly sand volumes used as a sink term in the models have been included in Appendix Ten.

The sink term has to be in a unit width and length required by the models, and we specify

$$q_{s,i} = \frac{Q_s}{W \times L}, \tag{7.18}$$

where $q_{s,i}$ is the sink term used in the model (m/month), Q_s is the monthly sand extraction volume (m^3/month), W is the width (m), and L is the length (m) where Q_s is distributed.

The best fit method was applied for calibration. The criterion used was the minimum average absolute differences between bed levels from the calculation and from the field data at all the spatial grids.

Monthly discharges at the Ngaruawahia Cableway were used when applying Eqs. (7.6), (7.10) and (7.16–17), and $\overline{Q}_T$=329 m^3/s. They have been included in Appendix Eleven.

7.3.1 Calibration and validation of the parabolic model

Eq. (7.6) was used for determination of D:

$$D = \frac{Q_T}{\overline{Q}_T} D_0.$$

By using Eq. (7.5) and assuming $b = \xi_1 \times 3$, $\overline{Q}_s = \xi_2 \times 160\,000$ m^3/yr, $i_0 = \xi_3 \times 0.011\%$ and $W = \xi_4 \times 220$ m, we have

$$D_0 = \frac{1}{3}\frac{b\overline{Q}_s}{Wi_0} = \frac{\xi_1\xi_2}{\xi_3\xi_4} \times 5.51 \times 10^5 = \xi \times 5.51 \times 10^5 \ \text{m}^2/\text{month}, \tag{7.19}$$

where ξ_i (i=1–4) is parameter for correction and

$$\xi = \frac{\xi_1\xi_2}{\xi_3\xi_4}, \tag{7.20}$$

which is a combined parameter.

For the sink term, Eq. (7.18), using $W = \xi_4 \times 220$ m, $\eta_1 \times Q_s$ instead of Q_s and $\eta_2 \times L$ instead of L, we have

$$q_s = \frac{\eta_1 \times Q_s}{\xi_4 \times 220 \times \eta_2 \times L} = \frac{Q_s}{\eta \times 220 \times L}, \tag{7.21}$$

where η_1 and η_2 are parameters for correction, and

$$\eta = \frac{\xi_4\eta_2}{\eta_1} = \frac{\xi_4}{\eta_s} \tag{7.22}$$

which is a parameter for correction of both the width and the sand extraction volumes, and η_s is a corrector for volumes of the sand extraction in a unit length.

For bedload supplies given at the upstream boundary ($x=94.45$ (km)), we have (Chapter Six; Jain, 1981):

$$-D\frac{\partial BL}{\partial x}=g_3(t), \qquad t>0, \tag{7.23}$$

where g_3 is the volumetric bedload supply, including pores, per unit width. In the case that the x axis is in the upstream direction, g_3 is negative for the bedload supplies to the river downstream. Based on Eqs. (7.16–17), g_3 can be presented by

$$g_3(t)=\frac{G_j}{\xi_4\times 220}, \tag{7.24}$$

where ξ_4 is the parameter for width correction.

Two variables, ξ and η, are calibrated in the case of bed levels specified at both upstream and downstream boundaries. In the case of bed levels and bedload supplies specified, respectively, at the downstream and upstream boundaries, ξ_4 needs to be determined because at the upstream boundary only a total of bedload supplies were given. Therefore three variables, ξ, η_s and ξ_4, are calibrated in this case. Theoretically values of ξ in these two cases should be the same if the boundary conditions are given correctly. However because of errors in the boundary conditions, they can be different.

All the parameters for correction above, including the combined parameters, are expected to be equal to one. Assuming $|\Delta\xi_1|=0$ ($b=3$), $|\Delta\xi_2|=15\%$ ($\overline{Q}_s=160\ 000\pm 24\ 000$ m^3/yr), $|\Delta\xi_3|=5\%$ ($i_0=0.011\%\pm 0.00055\%$), $|\Delta\xi_4|=10\%$ ($W=220\pm 22$ m), and $|\Delta\eta_s|=20\%$ (including effects of sand volumes extracted for river training work which were not recorded), and each of them is independent, deviations of ξ and η can be estimated by using the root-sum-square method (ISO, 1976):

$$\begin{aligned}|\Delta\xi|&=\sqrt{(\Delta\xi_1)^2+(\Delta\xi_2)^2+(\Delta\xi_3)^2+(\Delta\xi_4)^2}\\&=\sqrt{0^2+(15\%)^2+(5\%)^2+(10\%)^2}\approx 19\%,\end{aligned} \tag{7.25}$$

$$|\Delta\eta|=\sqrt{(\Delta\xi_4)^2+(\Delta\eta_s)^2}=\sqrt{(10\%)^2+(20\%)^2}\approx 22\%. \tag{7.26}$$

Then we have $\xi=1\pm 0.19$ and $\eta=1\pm 0.22$.

Table 7.2 A range for variations of ξ, η, ξ_4 and η_s in the models.

Parameter	ξ	η	ξ_4	η_s
Ranges	0.81–1.19	0.78–1.22	0.90–1.10	0.80–1.20

Table 7.2 lists the ranges acceptable physically for variations of ξ, η, ξ_4 and η_s in the models.

- **The parabolic model with bed levels given at both upstream and downstream boundaries**

Given bed levels at both upstream and downstream boundaries, Eq. (7.12–13) and (7.14–15), the parabolic equation with two adjustable parameters, ξ and η, were solved numerically with the fully implicit scheme (Chapter Six).

The average absolute differences ($|\overline{\Delta BL}|$) between bed levels from the parabolic model and from the field data in 1982 at the grids ($i=\overline{1,307}$) for different ξ and η are listed in Table 7.3. Values of ξ and η were selected to be equal to 0.87 and 0.82 respectively because of the corresponding minimum of $|\overline{\Delta BL}|$, 1.82 cm. Both ξ and η are acceptable based on the discussions above (Table 7.2). ξ=0.86 and η=0.82 or 0.83 can also be selected based on the criterion.

Table 7.3 Lists of the average absolute differences ($|\overline{\Delta BL}|$, cm) between bed levels from the parabolic model and the field data on the grids (n=307) in May 1982 for different ξ and η. Bed levels were specified at both upstream and downstream boundaries.

$\xi \backslash \eta$	1.00	0.98	0.96	0.94	0.92	0.90	0.88	0.86	0.84	0.82	0.80
1.10	4.03	3.87	3.72	3.58	3.48	3.41	3.37	3.35	3.36	3.40	3.46
1.00	3.36	3.15	2.95	2.77	2.62	2.51	2.45	2.43	2.43	2.46	2.51
0.90	3.40	3.14	2.87	2.62	2.38	2.18	2.01	1.91	1.86	1.85	1.90
0.89	3.44	3.17	2.90	2.64	2.40	2.20	2.02	1.90	1.85	1.83	1.88
0.88	3.48	3.20	2.93	2.67	2.43	2.22	2.03	1.89	1.84	1.83	1.87
0.87	3.52	3.25	2.97	2.71	2.46	2.25	2.05	1.89	1.83	1.82	1.87
0.86	3.57	3.30	3.01	2.75	2.50	2.28	2.08	1.90	1.82	1.82	1.88
0.85	3.63	3.35	3.06	2.80	2.55	2.33	2.12	1.94	1.83	1.83	1.89

Figure 7.8 shows results of the calibration—a comparison of the bed level profile from the parabolic model with ξ=0.87 and η=0.82 and the field data in May 1982.

The calibrated model (ξ=0.87 and η=0.82) was validated by using the field data in December 1990 and the results are illustrated in Figure 7.9. The average absolute bed level difference, $|\overline{\Delta BL}|$, was 3.34 cm, between those from

the model and from the field data on the grids. This figure (3.34 cm) was bigger than that of the calibration (1.82 cm in Table 7.2) by 1.52 cm.

At the Rangiriri Bridge and the Huntly Rail Bridge, depths between the water level at the discharge of 350 m^3/s (Appendix Seven) in June 1958 and the initial bed level from Eq. (7.2) (steady state) are 1.992 m (6.456–4.401 m) and 2.263 m (8.390–6.137 m) respectively. From Eq.(7.3), we have

$$BL(x,\Delta t+t_0)=WL_{Q=350}(x,\Delta t+t_0)-[WL_{Q=350}(x,t_0)-BL(x,t_0)].$$

Thus given variations in water level at 350 m^3/s, $WL_{Q=350}$, at these two sites (Figures 4.20 and 4.24), variations in bed level (BL) over time can be obtained by deducting the corresponding depth above from the variations of $WL_{Q=350}$.

The circles in Figures 7.10–11 are the bed levels determined by using the method above at the Rangiriri Bridge for 1970–1992 and at the Huntly Rail Bridge for 1958–1992 respectively. These data can be used to make a check of the parabolic model with ξ=0.87 and η=0.82.

The solid lines in Figures 7.10–11 are the annual variations in bed level from

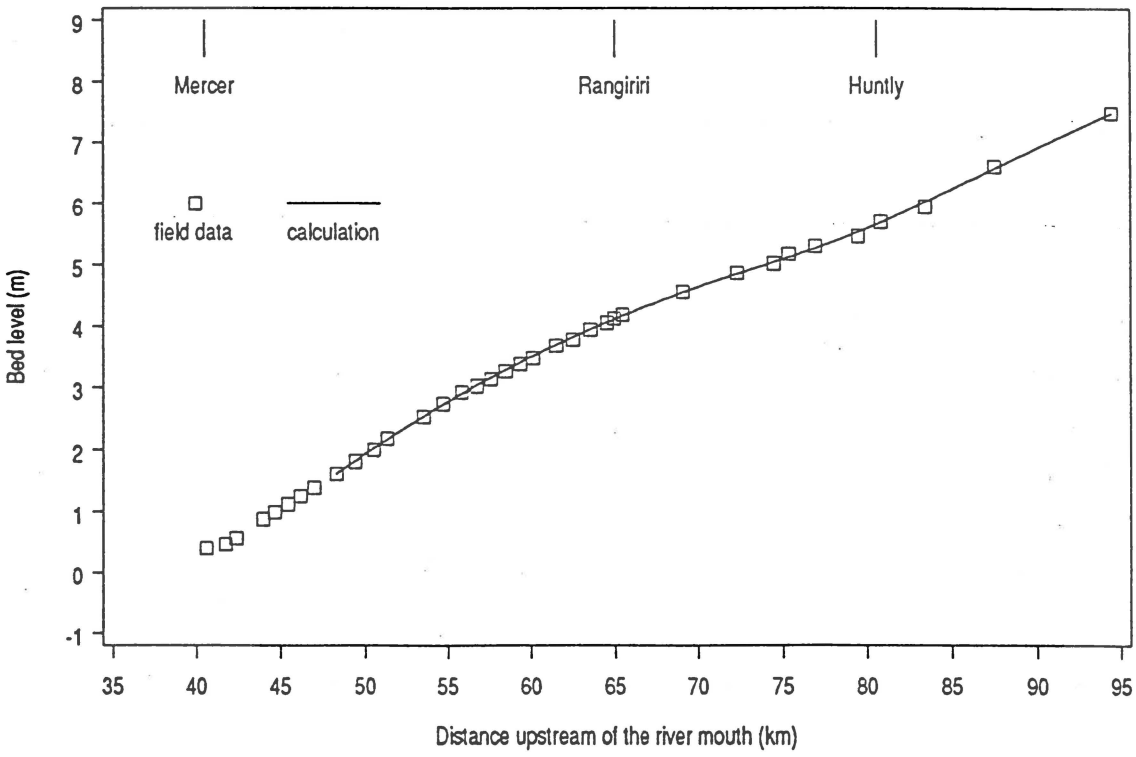


Figure 7.8 Calibration of the parabolic model by using the bed levels in May 1982. The profile was obtained by the model with ξ=0.87 and η=0.82. Bed levels were specified at both upstream and downstream boundaries.

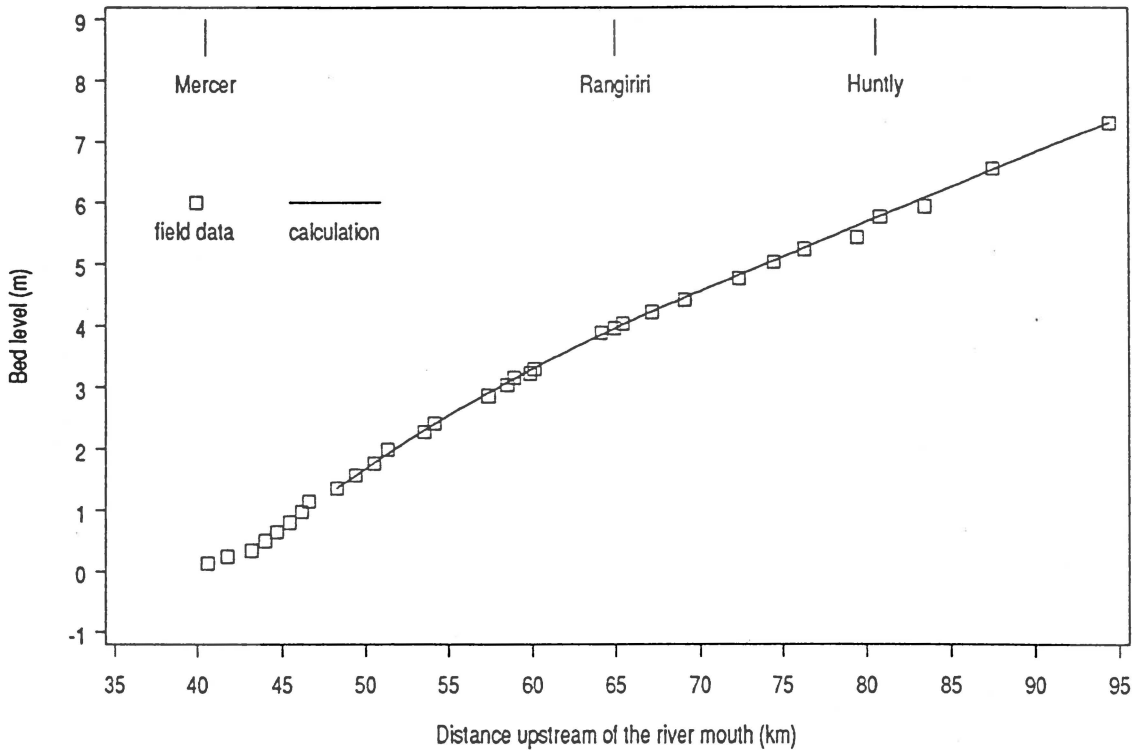


Figure 7.9 Validation of the parabolic model by using the bed levels in December 1990. The profile was obtained by the model ξ=0.87 and η=0.82. Bed levels were specified at both upstream and downstream boundaries.

the calibrated model between 1958–1990. For the incomplete year of 1958, an average of the monthly data between June–December was used.

In order to compare the results from the model to those from smoothed curve fitting, the sum-square-average-differences between the bed levels from the model and from the field data at the Rangiriri Bridge and at the Huntly Rail Bridge were determined. They are, respectively, equal to 0.09 m (n=90) and 0.12 m (n=106). Smoothed curves fitting for all the field data by using SAS with I=SM65 gave the standard deviations at these two stations 0.07 m (n=99) and 0.09 m (n=129) (Table 4.6) respectively. Differences between these two methods are, respectively, 0.02 m and 0.03 m at the Rangiriri Bridge and the Huntly Rail Bridge (Table 7.4).

A summary of the model calibration and validation, and a comparison of results from the model with ξ=0.87 and η=0.82 and from the curve fitting at the Rangiriri Bridge and the Huntly Rail Bridge are presented in Table 7.4.

All the evidence shows that the model with ξ=0.87 and η=0.82 can produce satisfactory results for long term changes in river bed level.

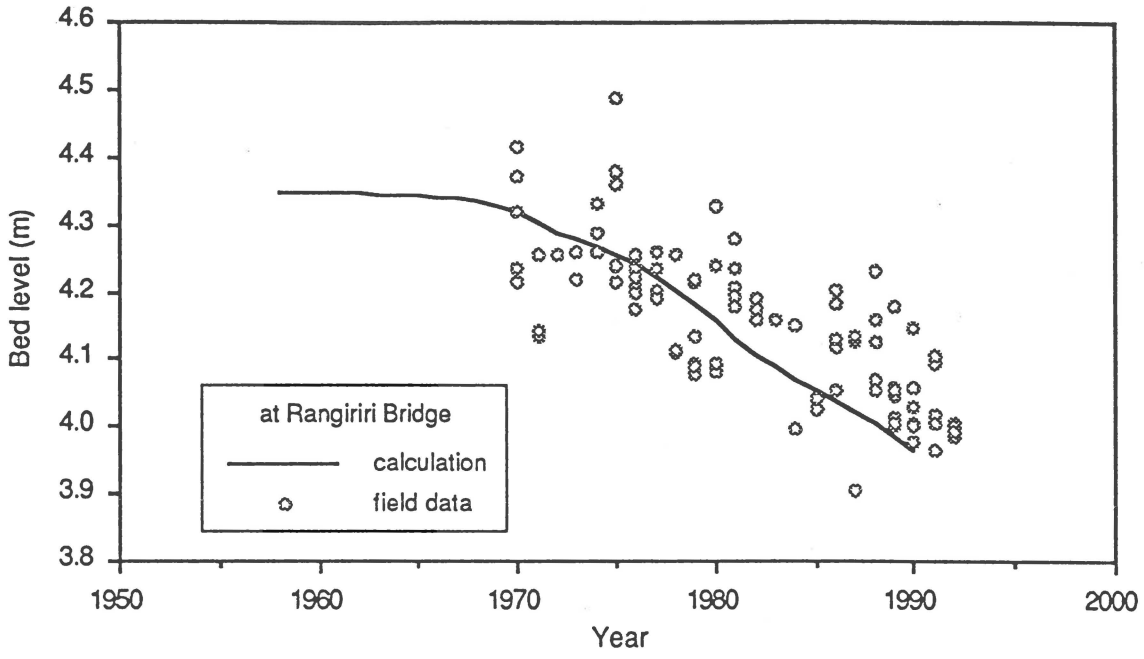


Figure 7.10 Comparison of annual bed level variations from calculation of the parabolic model with ξ=0.87 and η=0.82 and from the field data at the Rangiriri Bridge (1970–1992). Bed levels were specified at both upstream and downstream boundaries for 1958–1990. There are a total of 90 field records between 1970–1990 and the sum-square-average-difference between the bed levels from the model and from these field records is 0.09 m.

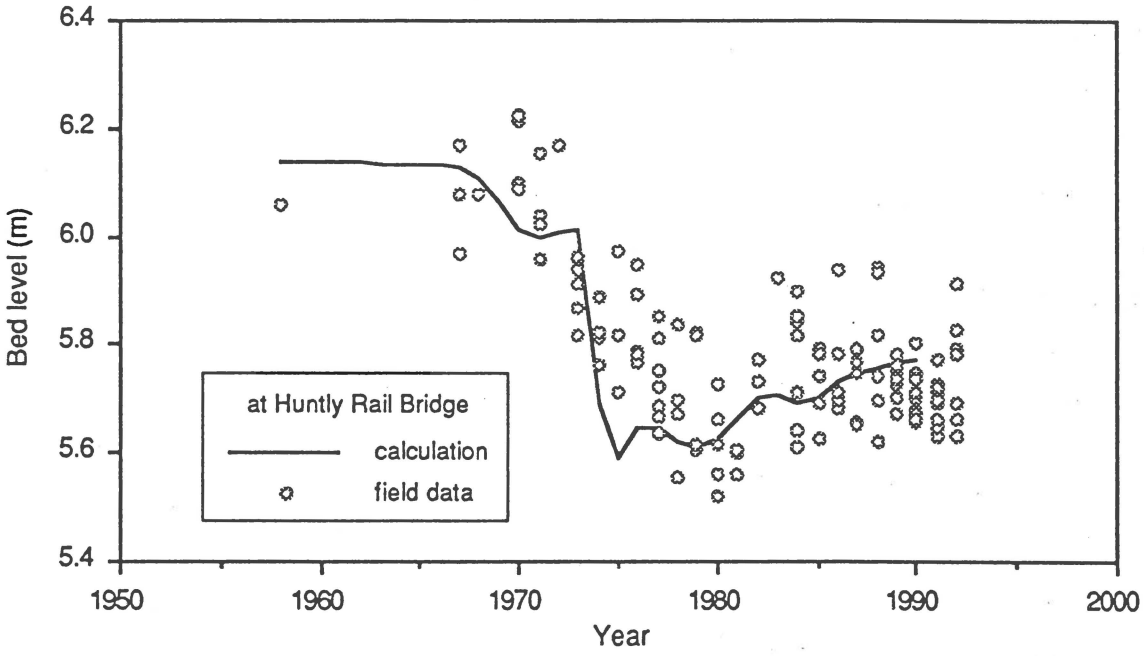


Figure 7.11 Comparison of annual bed level variations from calculation of the parabolic model with ξ=0.87 and η=0.82 and from the field data at the Huntly Rail Bridge (1958–1992). Bed levels were specified at both upstream and downstream boundaries for 1958–1990. There are a total of 106 field records between 1958–1990 and the sum-square-average-difference between the bed levels from the model and from these field records is 0.12 m.

Table 7.4 A summary of the parabolic model with ξ=0.87 and η=0.82, and bed levels given at both upstream and downstream boundaries. $|\overline{\Delta BL}|$ (cm) is the average absolute bed level difference on the grids (n=307), "S.S.A.D." (m) stands for the sum-square-average-differences between the bed levels from the model calculation and from the field data, and "S.D." (m) represents a standard deviation of the curve fitting of the entire field data by using SAS with I=SM65.

Content	$\|\overline{\Delta BL}\|$ (cm)	Station	Rangiriri B.	Huntly Rail B.
Calibration	1.82	S.S.A.D. (m)	0.09 (n=90)	0.12 (n=106)
Validation	3.34	S.D. (m)	0.07 (n=99)	0.09 (n=127)

- **The parabolic model with bed levels specified at the downstream boundary and bedload supplies at the upstream boundary**

Before using the bedload supply conditions, Eqs. (7.16–17), G_0 and κ have to be specified. Based on the result of ξ=0.87 above and Eq. (7.25), we assume that G_0=150 000 m^3/yr (within the range of 160 000±24 000 m^3/yr) and κ=0.9. This assumption is an open question and further research is required.

Given bed levels at the downstream boundary and bedload supplies at the upstream boundary, Eqs. (7.12–13) and Eqs. (7.16–17) with G_0=150 000 m^3/yr and κ=0.9, the parabolic equation were solved numerically with the fully implicit scheme (Chapter Six). As indicated before, ξ could be different from 0.87 determined above because of probable errors in the boundary conditions. Therefore a total of three variables, ξ, η_s, and ξ_4, were adjusted during calibration.

Table 7.5 lists the average absolute differences ($|\overline{\Delta BL}|$) between bed levels from the model and from the field data in May 1982 at the grids ($i=\overline{1,307}$) for different values of ξ, η_s and ξ_4, which are within their limits (Table 7.2). From this table, the minimum difference, $|\overline{\Delta BL}|$, is 2.41 cm corresponding ξ=0.96, η_s=1.20 and ξ_4=1.00. Further fine tuning still gave the same results.

Figure 7.12 shows the bed level profile from the model with ξ=0.96, η_s=1.20 and ξ_4=1.00 and the bed levels from the field data in May 1982.

Validation of the calibrated model was undertaken by using the field data for December 1990. The average absolute difference between bed levels from the model and from the field data, $|\overline{\Delta BL}|$, is 3.85 cm, and the results are shown in Figure 7.13.

Table 7.5 Lists of the average absolute differences ($\overline{|\Delta BL|}$, cm) between bed levels from the parabolic model and the field data on the grids (n=307) in May 1982 for different values of ξ, η_s and ξ_4. Bed levels and bedload supplies were specified at the downstream and upstream boundaries respectively.

$\eta_s \backslash \xi_4$	1.10	1.05	1.00	0.95	0.90	1.10	1.05	1.00	0.95	0.90
			ξ=0.87					ξ=0.89		
1.20	7.19	8.16	9.23	10.41	11.73	5.46	6.39	7.43	8.58	9.85
1.15	7.91	8.91	10.01	11.23	12.59	6.16	7.12	8.19	9.37	10.68
			ξ=0.91					ξ=0.93		
1.20	3.36	4.72	5.73	6.84	8.10	2.91	3.16	4.16	5.27	6.53
1.15	4.49	5.43	6.46	7.60	8.87	3.32	3.81	4.81	5.92	7.16
			ξ=0.95					ξ=0.96		
1.20	3.19	2.67	2.78	3.91	5.17	3.44	2.90	2.41	3.29	4.54
1.15	3.41	2.90	3.29	4.39	5.63	3.57	3.04	2.72	3.70	4.94
			ξ=0.97					ξ=0.98		
1.20	3.76	3.20	2.67	2.72	3.96	4.13	3.34	3.01	2.48	3.41
1.15	3.80	3.24	2.71	3.06	4.31	4.07	3.51	2.96	2.58	3.72
1.10	3.97	3.43	2.92	3.57	4.80	4.17	3.61	3.07	2.94	4.12
			ξ=0.99					ξ=1.01		
1.20	4.56	3.99	3.40	2.83	2.90	5.57	4.97	4.34	3.70	3.08
1.15	4.43	3.86	3.29	2.75	3.17	5.29	4.70	4.10	3.48	2.89
1.10	4.43	3.86	3.29	2.76	3.52	5.13	4.56	3.96	3.37	2.82
0.90	5.41	4.96	4.57	4.92	6.20	5.62	5.12	4.63	3.36	2.95

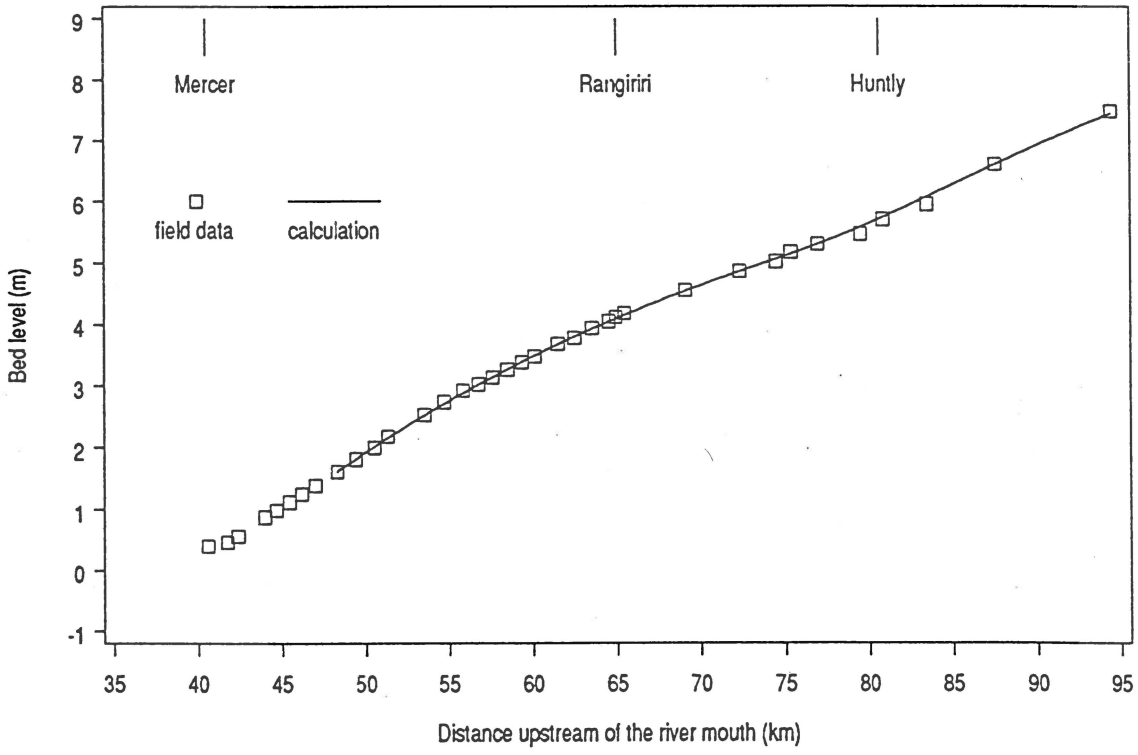


Figure 7.12 Calibration of the parabolic model by using the bed levels in May 1982. The profile was obtained by the model with ξ=0.96, η_s=1.20 and ξ_4=1.00. Bed levels and bedload supplies were specified at the downstream and upstream boundaries respectively.

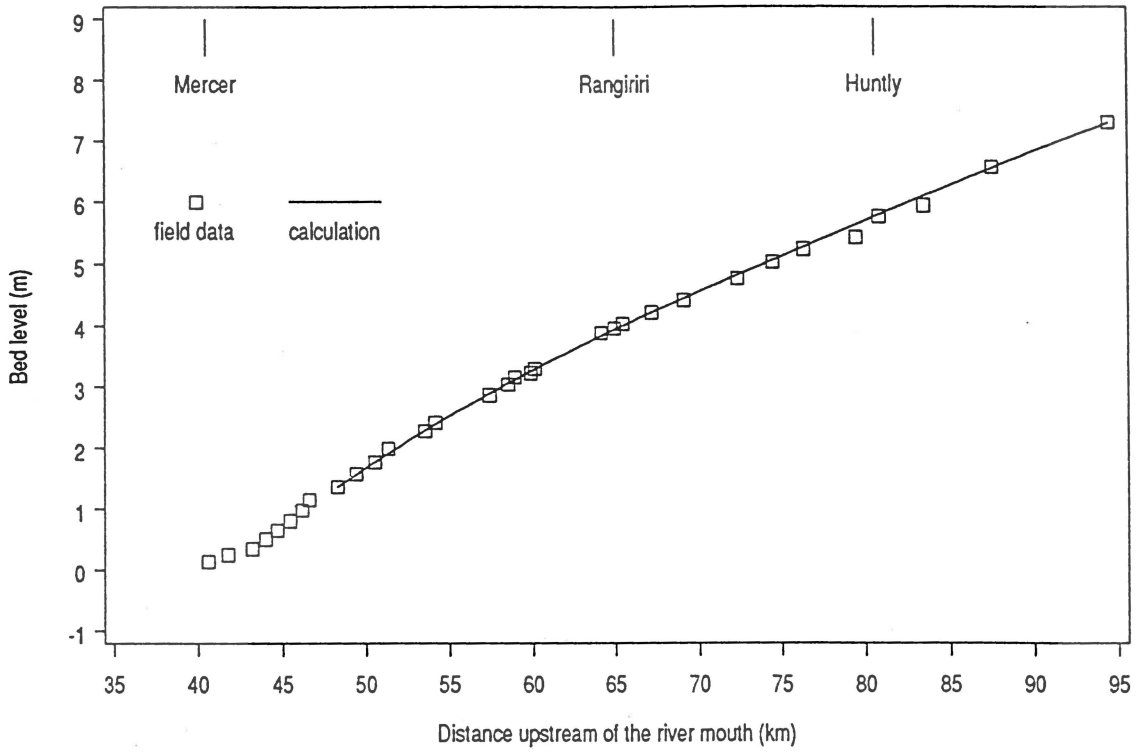


Figure 7.13 Validation of the parabolic model by using the bed levels in December 1990. The profile was obtained by the model with ξ=0.96, η_s=1.20 and ξ_4=1.00. Bed levels and bedload supplies were specified at the downstream and upstream boundaries respectively.

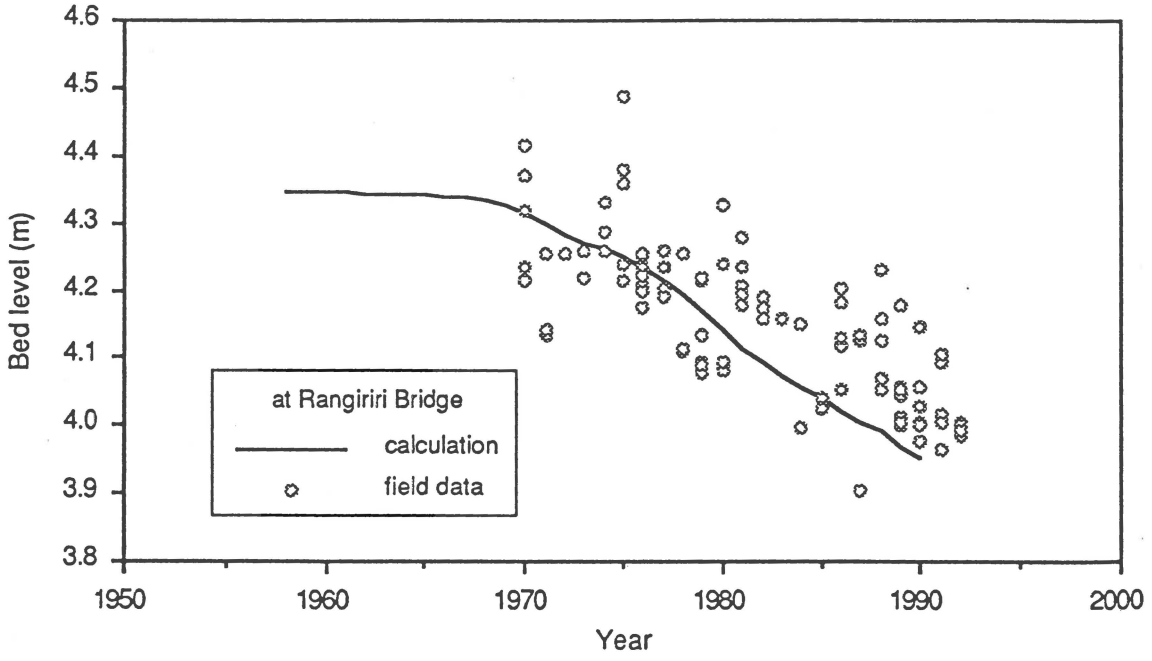


Figure 7.14 Comparison of annual bed level variations from the calculation from the parabolic model with ξ=0.96, η_s=1.20 and ξ_4=1.00 at the Rangiriri Bridge (1970–1992). Bed levels and bedload supplies were specified at the downstream and upstream boundaries in 1958–1990 respectively. There are a total of 90 field records for 1970–1990 and the sum-square-average-difference between the bed levels from the model and from these field records is 0.09 m.

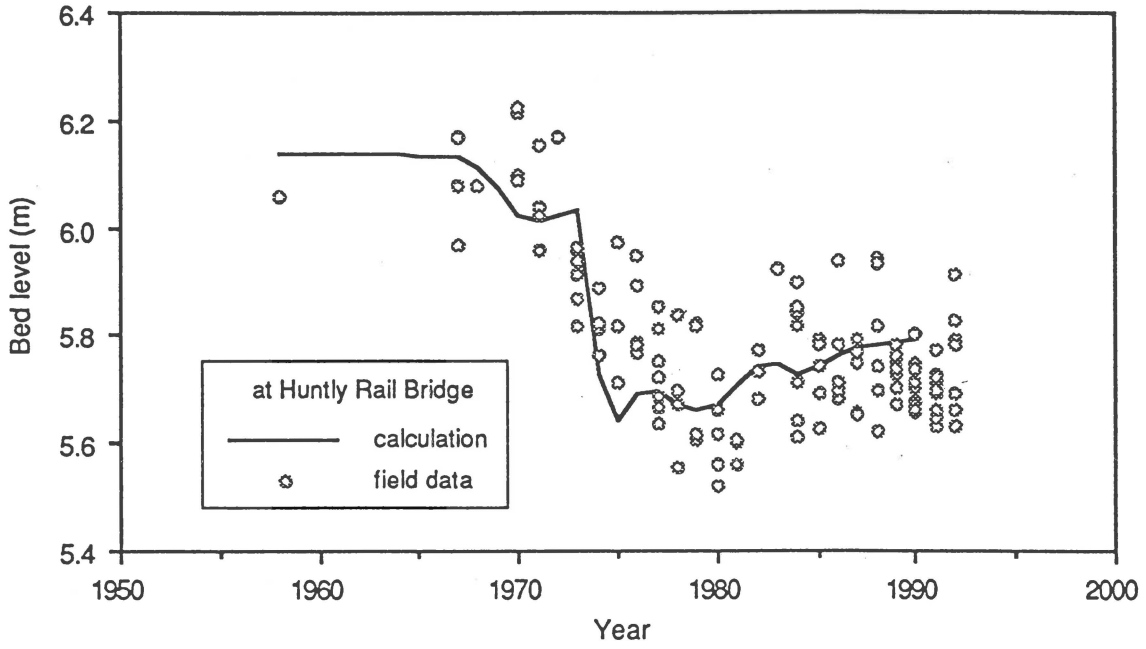


Figure 7.15 Comparison of annual bed level variations from the parabolic model with ξ=0.96, η_s=1.20 and ξ_4=1.00 and from the field data at the Huntly Rail Bridge (1958–1992). Bed levels and bedload supplies were specified at the downstream and upstream boundaries in 1958–1990 respectively. There are a total of 106 field records between 1958–1990 and the sum-square-average-difference between the bed levels from the model and from these field records is 0.11 m.

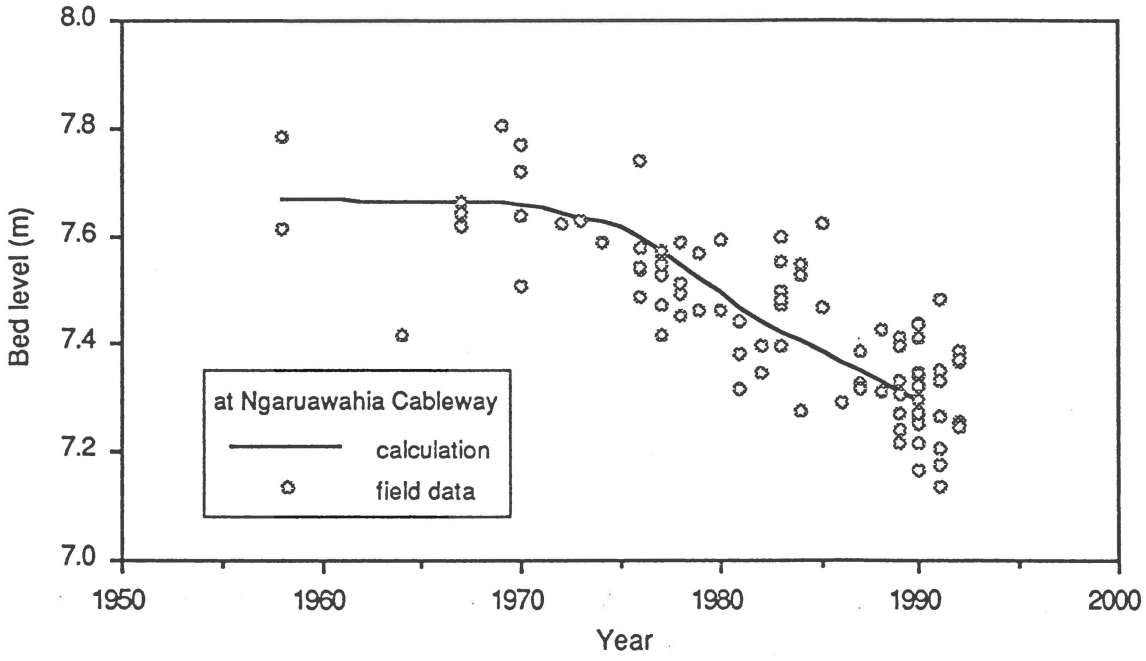


Figure 7.16 Comparison of annual bed level variations from the parabolic model with ξ=0.96, η_s=1.20 and ξ_4=1.00 and from the field data at the Ngaruawahia Cableway (1958–1992). Bed levels and bedload supplies were specified at the downstream and upstream boundaries in 1958–1990 respectively. There are a total of 75 field records for 1958–1990 and the sum-square-average-difference between the bed levels from the model and from these field records is 0.09 m.

Annual variations in bed level at the Rangiriri Bridge, the Huntly Rail Bridge and the Ngaruawahia Cableway were used to compare to the results from the parabolic model with ξ=0.96, η_s=1.20 and ξ_4=1.00. For the incomplete year of 1958, an average of the monthly data between June–December was taken. The comparisons at these three stations are shown in Figures 7.14–16.

The sum-square-average-differences between the bed levels from the model and from the field data in the period of 1958–1990 are, respectively, 0.09 m (n=90), 0.11 m (n=106) and 0.09 m (n=75) at the Rangiriri Bridge, the Huntly Rail Bridge and the Ngaruawahia Cableway, while the smoothed curves fitting all the field data by the cubic spline method using SAS with I=SM65 produced standard deviations at these stations of 0.07 m, 0.09 m and 0.09 m (Table 4.6).

A summary for the model calibration and validation, and a comparison of results from the model with ξ=0.96, η_s=1.20 and ξ_4=1.00 and the curve fitting at the Rangiriri Bridge, the Huntly Rail Bridge and the Ngaruawahia Cableway are given in Table 7.6.

Generally the model with ξ=0.96, η_s=1.20 and ξ_4=1.00 produced reasonable results.

Table 7.6 A summary of the parabolic model with ξ=0.96, η_s=1.20 and ξ_4=1.00, and bed levels and bedload supplies given at the downstream and upstream boundaries respectively. $|\overline{\Delta BL}|$ (cm) is the average absolute bed level difference on the grids (n=307), "S.S.A.D." (m) stands for the sum-square-average-differences between the bed levels from the model calculation and from the field data, and "S.D." (m) represents a standard deviation of the curve fitting of the entire field data by using SAS with I=SM65.

Procedures	$\|\overline{\Delta BL}\|$ (cm)	Station	Rangiriri B.	Huntly Rail B.	Ng. Cableway
Calibration	2.41	S.S.A.D. (m)	0.09 (n=90)	0.11 (n=106)	0.09 (n=75)
Validation	3.85	S.D. (m)	0.07 (n=99)	0.09 (n=127)	0.09 (n=89)

The parameters determined using two different boundary conditions (Cases A and B) and the same field data were compared to each other, and the results are presented in Table 7.7. For Case A, bed levels were specified at both upstream and downstream boundaries, and for Case B, bed levels and bedload supplies were specified at the downstream and upstream boundaries respectively. The parameter η in Case B was determined by using Eq. (7.22). Relative differences of ξ and η are, respectively, about 10% and 1%.

Table 7.7 A comparison of the parameters, ξ and η, determined by two different boundary conditions (Cases A and B) by using the same field data. "Dif." stands for difference.

Parameter	Range	Case A	Case B	Difference	Relative Dif.
ξ	0.81–1.19	0.87	0.96	0.09	10%
η	0.78–1.22	0.82	0.83	0.01	1%

It should be mentioned here again that bedload supplies at the upstream boundary were assumed because they could not be determined by field data. At this stage, in case B we adjusted the parameters, not the boundary conditions (bedload supplies), based on the assumption that the boundary conditions given were appropriate.

7.3.2 Calibration and validation of the hyperbolic model

Bed levels at both boundaries were specified in this model study. Two parameters are required to run the model, D and c. Determination of D has been discussed above. For the parameter c, Eq. (7.10) was applied:

$$c = \frac{Q_T}{\overline{Q}_T} c_0.$$

As a first estimate of c_0, taking $b=3$ (ref. 6.3.1), $\overline{Q}_s$=160 000 m^3/yr (Chapter Five) and $\overline{Q}$=350 m^3/s (a standard discharge), from Eq. (7.8) we have

$$\psi_0 = b\frac{\overline{Q}_s}{\overline{Q}} = 3\frac{160\ 000}{350\times(365\times24\times60\times60)} = 4.35\times10^{-5}.$$

Assuming width W=200 m, depth h_0=2.0 m, then $u_0 = 350/(220\times2.0) \approx 0.79$ m/s, $Fr_0 = 0.79/\sqrt{9.8\times2.0} \approx 0.18$ (Eq. (7.9)), and from Eq. (7.7) we have

$$c_0 = 0.79\times\frac{4.35\times10^{-5}}{1-0.18^2} \approx 3.55\times10^{-5}\ \text{m/s},$$

or $c_0 \approx$93.3 m/month, or $c_0 \approx 1.12\times10^3$ m/yr. Therefore we can take

$$c_0 = -\lambda\times95\ \text{m/month}, \tag{7.27}$$

where λ is a parameter for correction and the negative sing means that the direction of c_0 is in the downstream of the river.

Three parameters, ξ, η and λ, could be adjusted in calibration. Based on the results in section 7.3.1, η was taken to be 0.82. ξ is expected to be close to 0.87 (section 7.3.1).

The numerical methods discussed in section 6.5.2 have been used to solve the hyperbolic equation. Table 7.8 lists the average absolute differences ($|\overline{\Delta BL}|$)

between bed levels from the hyperbolic model using Eqs. (6.82) and (6.85) with $\theta=1$ and from the field data in 1982. Both methods gave the same differences for $\lambda \geq 20$ shown in Table 7.8.

The model with $\xi=0.87$ and $\lambda=200$ or bigger gives the smallest $|\overline{\Delta BL}|$, 1.82 cm. However whether the parameter, $\lambda=200$ or bigger, is physically acceptable is in doubt.

By using the model with $\xi=0.87$, $\lambda=200$ and the field data in December 1990 for validation, both methods produced the same $|\overline{\Delta BL}|$, 3.33 cm.

Table 7.8 Lists of the average absolute differences ($|\overline{\Delta BL}|$, cm) between bed levels from the hyperbolic model and the field data on the grids (n=307) in May 1982 for different values of ξ and λ. Bed levels were specified at both upstream and downstream boundaries.

	Method: Eq. (6.82)							Method: Eq. (6.85)						
$\xi \backslash \lambda$	1	20	40	80	100	200	400	1	20	40	80	100	200	400
0.85	2.88	1.84	1.83	1.83	1.83	1.83	1.83	2.89	1.84	1.83	1.83	1.83	1.83	1.83
0.87	2.97	1.84	1.83	1.83	1.83	1.82	1.82	3.08	1.84	1.83	1.83	1.83	1.82	1.82
0.89	3.08	1.86	1.85	1.84	1.84	1.84	1.84	3.18	1.86	1.85	1.84	1.84	1.84	1.84
0.91	3.19	1.92	1.90	1.89	1.89	1.88	1.88	3.29	1.92	1.90	1.89	1.89	1.88	1.88

The alternative, fully implicit method with the mixed term treated as a source term has also been applied to obtain the numerical results and the small value of ε required by the method was taken to be equal to 0.005 (section 6.5.2). However the program could not be run to completion because of computer overflows. This probably results from a numerical instability. Therefore the method should not be used in our case.

7.3.3 Comparison of results from the parabolic and hyperbolic equations

In the case of bed levels specified at both upstream and downstream boundaries, validations of the parabolic model with $\xi=0.87$ and $\eta=0.82$ and the hyperbolic model with $\xi=0.87$, $\lambda=200$ and $\eta=0.82$ indicated that the average absolute differences ($|\overline{\Delta BL}|$) of bed levels between these two models and the field data in December 1990 were, respectively, 3.34 cm (section 7.3.1) and 3.33 cm (section 7.3.2). The improvement in accuracy by using the hyperbolic model was 0.01 cm or 0.3% in relative percentage, which is not significant. In addition, physical acceptance of $\lambda=200$ in the hyperbolic model needs further confirmation.

The lack of improvement in accuracy from the hyperbolic model is theoretically expected because, in this case, the time periods used for calibration and validation were over 20 years, and for large values of time the

parabolic equation is an asymptotic model of the hyperbolic equation (Chapter Six).

Accordingly only the parabolic equation will be applied to the lower Waikato River, especially at Huntly, in terms of long term changes (50 years after 1990) in bed level.

7.4 Application of the parabolic model to predict future bed levels at the Huntly Power Station

The parabolic model has been applied to the lower Waikato River to investigate the future bed levels at the Huntly Power Station (79 km upstream of the river mouth) in December 2040 (50 years after 1990). In the application, the same time step Δt_j of one month was used, and j=0 was taken to represent the time of December 1990. Given year and month (1–12), the temporal grids can be determined by

$$j = (\text{Year} - 1990) \times 12 + (\text{month} - 12). \tag{7.28}$$

We have j=600 for the time of December 2040.

For the downstream boundary conditions at 48.25 km between December 1990 and December 2040, the following four cases, DB1–4, in which bed levels were specified, have been considered:

- **case DB1 (0):** There are no changes in bed level:

$$BL_j = BL_0, \; j = \overline{0,\; 600}, \tag{7.29}$$

where BL_0 is the initial bed level in December 1990, being at 1.35 m.

- **case DB2 (-0.5):** Bed levels linearly degrade by 0.5 m from 1.35 to 0.85 m:

$$BL_j = BL_0 + \frac{BL_0 - BL_{600}}{0 - 600}(j - 0), \; j = \overline{0,\; 600}, \tag{7.30}$$

where BL_0=1.35 m and BL_{600}=0.85 m.

- **case DB3 (-1.0):** Bed levels linearly degrade by 1.0 m from 1.35 to 0.35 m:

$$BL_j = BL_0 + \frac{BL_0 - BL_{600}}{0 - 600}(j - 0), \; j = \overline{0,\; 600}, \tag{7.31}$$

where BL_0=1.35 m and BL_{600}=0.35 m.

- **case DB4 (+0.5):** Bed levels linearly aggrade by 0.5 m from 1.35 to 1.85 m:

$$BL_j = BL_0 + \frac{BL_0 - BL_{600}}{0-600}(j-0),\ j = \overline{0,\ 600}, \tag{7.32}$$

where BL_0=1.35 m and BL_{600}=1.85 m.

These four cases for the downstream boundary conditions are illustrated in Figure 7.17. Variations in bed level between June 1958–December 1990 from Eqs. (7.12–13) were plotted together.

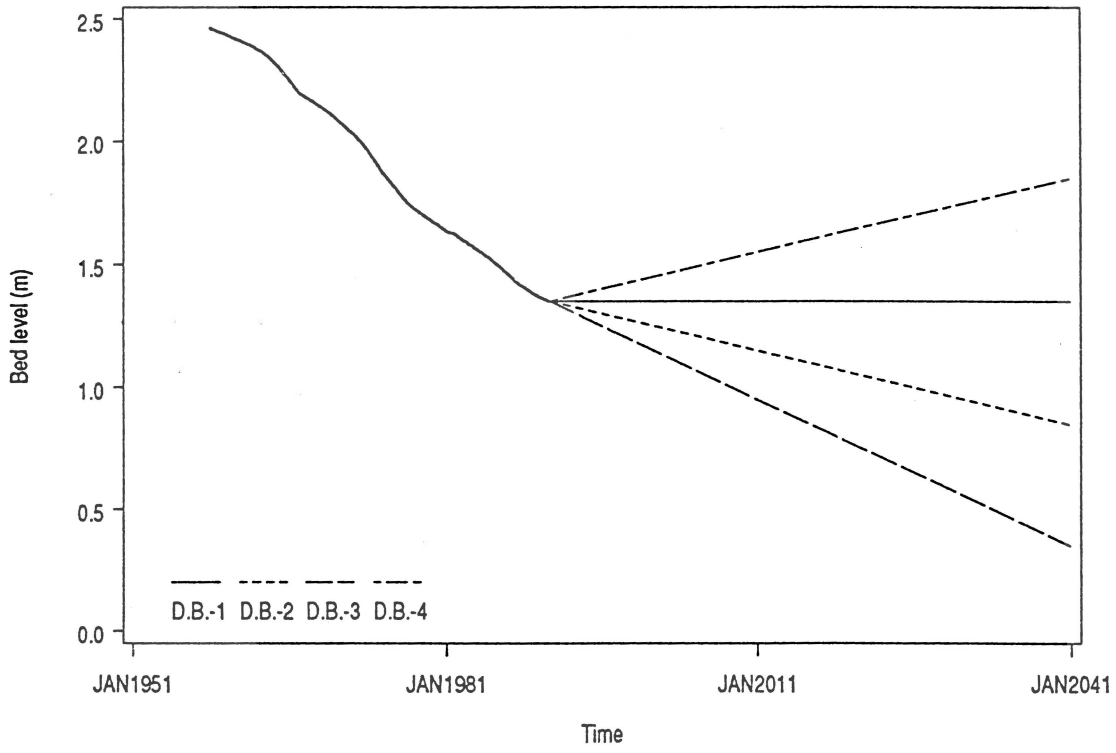


Figure 7.17 Bed level variations at the downstream boundary of 48.25 km upstream of the river mouth between December 1990 and December 2040 for four different cases. The solid line is from Eqs. (7.12–13), which have been used for model calibration and validation.

In practice, because of sand extraction, cases DB1 and DB4 may not occur. Even if sand mining were to cease right now (1993), the bed level there will still tend to drop, resulting from the bed level degradation further downstream (ref. Figure 4.4).

At the upstream boundary, two kinds of boundary conditions—bed levels and total bedload supplies—have been applied. For each of them, three different cases were considered.

For bed levels specified at the upstream boundary between December 1990 and December 2040, the following three cases, 1-UB1–3, have been studied:

• **case 1-UB1 (0):** There are no changes in bed level:

$$BL_j = BL_0,\ j = \overline{0,\ 600}, \tag{7.33}$$

where BL_0 is the initial bed level in December 1990, being at 7.30m.

• **case 1-UB2 (-0.2):** Bed levels degrade by 0.2 m from 7.30 to 7.10 m following an exponential function:

$$BL_j = BL_0,\ j = 0, \tag{7.34-1}$$

$$BL_j = BL_0 e^{-\frac{k}{599}(j-1)},\ j = \overline{1,\ 600}, \tag{7.34-2}$$

where BL_0=7.30 m and $k = 0.0278$.

• **case 1-UB3 (-0.3):** Bed levels degrade by 0.3 m from 7.30 to 7.00 m following an exponential function:

$$BL_j = BL_0,\ j = 0, \tag{7.35-1}$$

$$BL_j = BL_0 e^{-\frac{k}{599}(j-1)},\ j = \overline{1,\ 600}, \tag{7.35-2}$$

where BL_0=7.30 m and $k = 0.0420$.

For bedload supplies specified at the upstream boundary between December 1990 and December 2040, the following three cases, 2-UB1–3, have been investigated:

• **case 2-UB1 (0%):** There is a constant bulk bedload supply at 135 000 m^3/yr:

$$G_j = -\frac{G_0 \times 0.9}{12}\ (\mathrm{m}^3 / \mathrm{month}),\ j = \overline{0,\ 600} \tag{7.36}$$

where G_0=150 000 (section 7.3).

• **case 2-UB2 (-20%):** Bulk bedload supplies reduce by 20% from 135 000 m^3/yr to 108 000 m^3/yr following an exponential function:

$$G_j = -\frac{G_0 \times 0.9}{12}\ (\mathrm{m}^3 / \mathrm{month}),\ j = 0 \tag{7.37-1}$$

$$G_j = -\frac{G_0 \times 0.9}{12} e^{-\frac{k}{599}(j-1)}\ (\mathrm{m}^3 / \mathrm{month}),\ j = \overline{1,\ 600} \tag{7.37-2}$$

where G_0=150 000 and $k = 0.2231$.

• **case 2-UB3 (-40%):** Bulk bedload supplies reduce by 40% from 135 000 m^3/yr to 81 000 m^3/yr following an exponential function:

$$G_j = -\frac{G_0 \times 0.9}{12} \text{ (m}^3\text{/ month)}, \; j = 0 \tag{7.38-1}$$

$$G_j = -\frac{G_0 \times 0.9}{12} e^{-\frac{k}{599}(j-1)} \text{ (m}^3\text{/ month)}, \; j = \overline{1,\ 600} \tag{7.38-2}$$

where G_0=150 000 and k = 0.5108.

It should be pointed out that a part of historical bedload supplies into the river downstream of Ngaruawahia was from the sediment storage in the upstream river bed (ref. Chapter Five) and their decline in the future is expected because of armouring in the sand-gravel river (Chorley and Kennedy, 1971). However measures could be taken to let the bedload materials stored in the Karapiro Dam pass through to balance the decline.

During the period of December 1990–December 2040 a constant monthly discharge equal to the average was used in the model application.

Table 7.9 Bed levels at the Huntly Power Station (79 km upstream of the river mouth) for different scenarios in December 2040.

Case	1-UB1 (0)				1-UB2 (-0.2)				1-UB3 (-0.3)			
	DB1 (0)	DB2 (-0.5)	DB3 (-1.0)	DB4 (+0.5)	DB1 (0)	DB2 (-0.5)	DB3 (-1.0)	DB4 (+0.5)	DB1 (0)	DB2 (-0.5)	DB3 (-1.0)	DB4 (+0.5)
BL m	5.406	5.367	5.329	5.444	5.342	5.304	5.265	5.380	5.310	5.272	5.233	5.348
Case	2-UB1 (0%)				2-UB2 (-20%)				2-UB3 (-40%)			
	DB1 (0)	DB2 (-0.5)	DB3 (-1.0)	DB4 (+0.5)	DB1 (0)	DB2 (-0.5)	DB3 (-1.0)	DB4 (+0.5)	DB1 (0)	DB2 (-0.5)	DB3 (-1.0)	DB4 (+0.5)
BL m	5.273	5.226	5.178	5.320	5.193	5.146	5.098	5.240	5.104	5.057	5.010	5.151

In the case of bed levels given at both upstream and downstream boundaries, the model with ξ=0.87 was used. In the case of bed levels and bedload supplies specified at the downstream and upstream boundaries respectively, the model with ξ=0.96 and ξ_4=1.00 was applied. The results of bed levels at the Huntly Power Station in December 2040 are listed in Table 7.9.

As examples, annual variations in bed level over time at the Huntly Power Station are illustrated in Figures 7.18–19 for the different downstream boundary conditions (DB1–4) and two different cases of the upstream boundary conditions—1-UB2 and 2-UB2. The bed levels calculated in 1958–1990 were also plotted in the figures.

The bed level at the Huntly Power Station in the future is entirely controlled by both upstream and downstream boundary conditions. Both Figures 7.18–

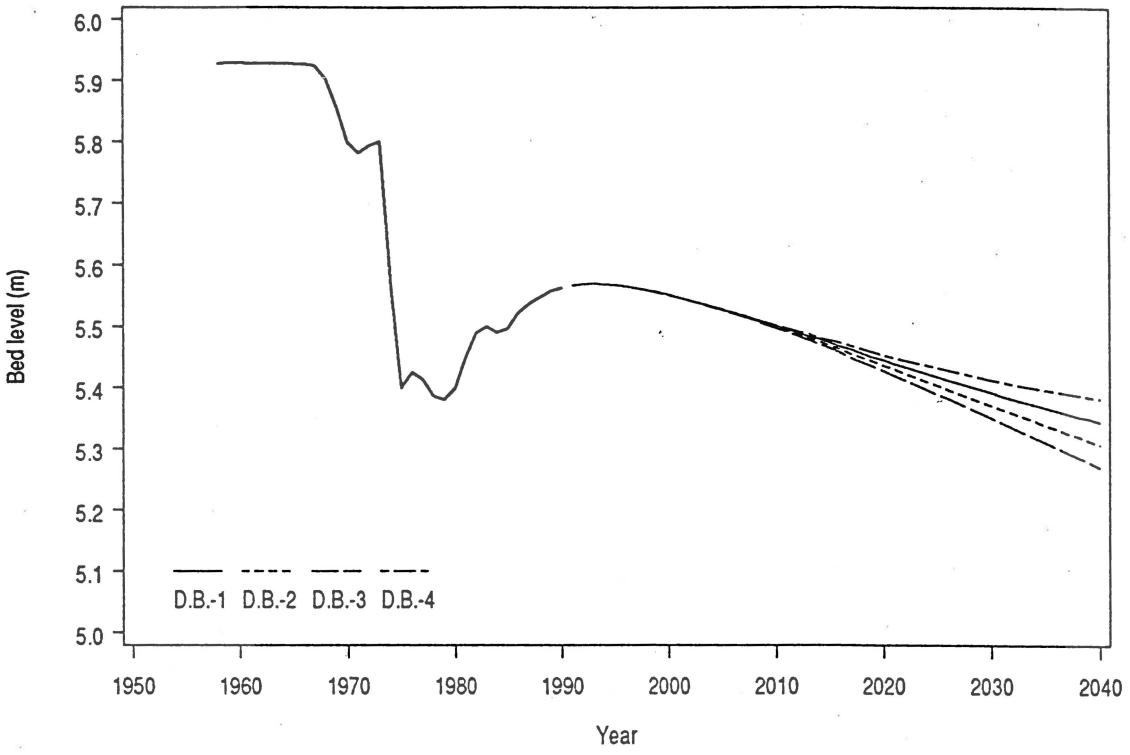


Figure 7.18 Annual variations in bed level at the Huntly power station (79 km upstream of the river mouth) for the different downstream boundary conditions with the upstream boundary condition of 1-UB2 after 1990.

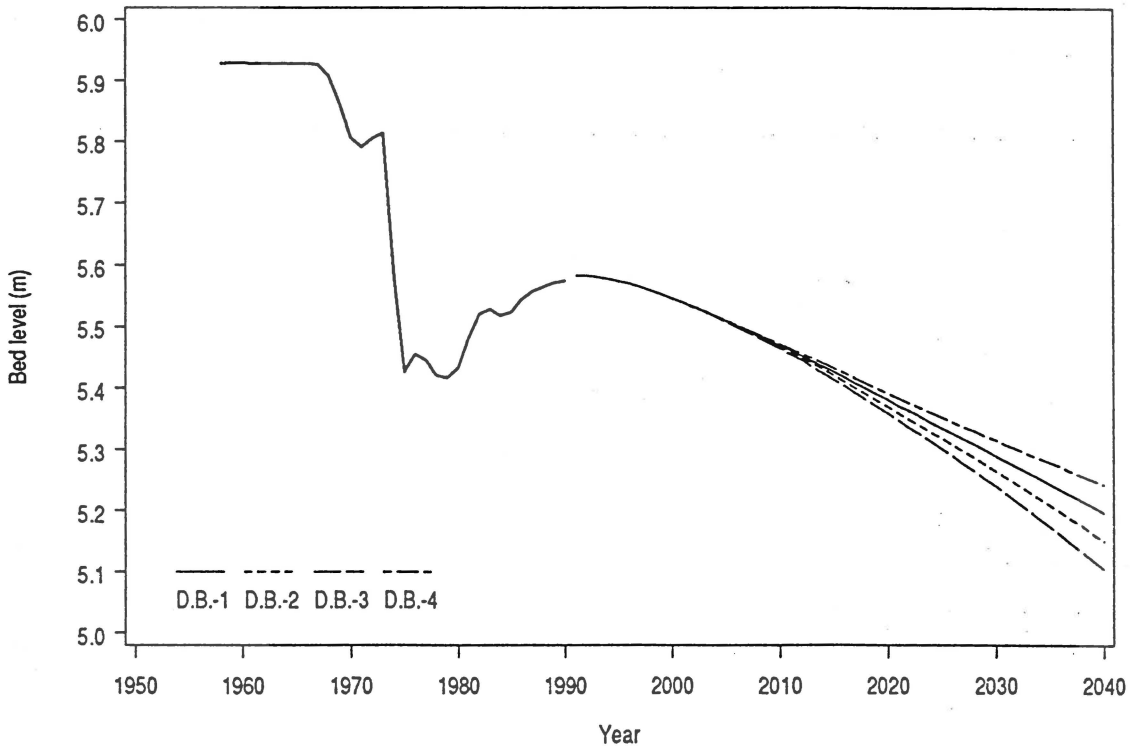


Figure 7.19 Annual variations in bed level at the Huntly Power Station (79 km upstream of the river mouth) for the different downstream boundary conditions with the upstream boundary condition of 2-UB2 after 1990.

19 show effects of the different downstream boundary conditions (DB1–4) on the annual variations in bed level at the Huntly Power Station after 1990 under the same upstream boundary condition. The trend of bed level degradation after 1990 is substantiated with the model results shown in the figures for these two cases although in the near future (10 years) bed level lowering seems to be not too serious.

7.5 Potential effects of bed level reductions on operation of the present cooling water system at the Huntly Power Station

Based on the bed levels predicted at the Huntly Power Station in December 2040 in Table 7.9, their corresponding water levels at the discharge of 350 m^3/s ($WL_{Q=350}$) can be obtained by using Eq. (7.3):

$$WL_{Q=350}(x,t) = BL(x,t) + [WL_{Q=350}(x,t_0) - BL(x,t_0)], \tag{7.39}$$

where x=79 km, t and t_0 are, respectively, the time of December 2040 and June 1958. The last two terms on the right hand side of Eq. (7.39) can be obtained from Figures 4.30 and 7.7 and their difference is 2.306 m (8.234 – 5.928 m).

Results of the water levels at discharge of 350 m^3/s in December 2040 for the different scenarios considered are listed in Table 7.10. They were determined by using Eq. (7.39) given the bed levels (BL).

Although Unit 3 of the MCW pumps, which are used in the Huntly Power Station, requires the river water level at the outside of the cooling water intake over 6.60 m from experiments, the trip limit of the pumps is at 6.50 m (Hornstra, 1983). Therefore the water level of 6.50 m is used as the critical water level required by operation of the present cooling water system.

River water levels are a function of not only river geometry but also river flows. At Huntly, the discharge of 150 m^3/s was used as a minimum allowable flow rate for investigation of effects of river degradation on the cooling water system operation (Hornstra, 1983). The discharges of 160 and 141 m^3/s were selected for modelling of hot water discharges at Huntly (Murray North-CMPS, 1991). From the historical flow records at the Huntly Rail Bridge (1968–1980), the Huntly North (1974–1983) and the Tainui Bridge or the Huntly Power Station (1983–1993), the minimum discharge was measured at 165 m^3/s on 25 February 1973 (TIDEDA database at the Waikato Regional Council). In this study, both discharges of 150 and 165 m^3/s were

used as the critical flow rates for investigation of the cooling water system operation requirement.

Table 7.10 Water levels (m) at the discharges of 350 m^3/s ($WL_{Q=350}$), 165 m^3/s ($WL_{Q=165}$) and 150 m^3/s ($WL_{Q=150}$) at the Huntly Power Station (79 km upstream of the river mouth) for different scenarios in December 2040 based on their corresponding bed levels (BL, m).

Case	1-UB1 (0)				1-UB2 (-0.2)				1-UB3 (-0.3)			
	DB1 (0)	DB2 (-0.5)	DB3 (-1.0)	DB4 (+0.5)	DB1 (0)	DB2 (-0.5)	DB3 (-1.0)	DB4 (+0.5)	DB1 (0)	DB2 (-0.5)	DB3 (-1.0)	DB4 (+0.5)
BL	5.406	5.367	5.329	5.444	5.342	5.304	5.265	5.380	5.310	5.272	5.233	5.348
$WL_{Q=350}$	7.712	7.673	7.635	7.750	7.648	7.610	7.571	7.686	7.616	7.578	7.539	7.654
$WL_{Q=165}$	6.672	6.633	6.595	6.710	6.608	6.570	6.531	6.646	6.576	6.538	6.499	6.614
Less 6.50 m	No	No	No	No	No	No	No	No	No	No	Yes	No
$WL_{Q=150}$	6.562	6.523	6.485	6.600	6.498	6.460	6.421	6.536	6.466	6.428	6.389	6.504
Less 6.50 m	No	No	Yes	No	Yes	Yes	Yes	No	Yes	Yes	Yes	No
Case	2-UB1 (0%)				2-UB2 (-20%)				2-UB3 (-40%)			
	DB1 (0)	DB2 (-0.5)	DB3 (-1.0)	DB4 (+0.5)	DB1 (0)	DB2 (-0.5)	DB3 (-1.0)	DB4 (+0.5)	DB1 (0)	DB2 (-0.5)	DB3 (-1.0)	DB4 (+0.5)
BL	5.273	5.226	5.178	5.320	5.193	5.146	5.098	5.240	5.104	5.057	5.010	5.151
$WL_{Q=350}$	7.579	7.532	7.484	7.626	7.499	7.452	7.404	7.546	7.410	7.363	7.316	7.457
$WL_{Q=165}$	6.539	6.492	6.444	6.586	6.459	6.412	6.364	6.506	6.370	6.323	6.276	6.417
Less 6.50 m	No	Yes	Yes	No	Yes	Yes	Yes	No	Yes	Yes	Yes	Yes
$WL_{Q=150}$	6.429	6.382	6.334	6.476	6.349	6.302	6.254	6.396	6.260	6.213	6.166	6.307
Less 6.50 m	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

It should be pointed out that because discharges are random variables, from an engineer's point of view the critical flow rate at Huntly should be selected from a frequency analysis of historical flow records given design criteria or return periods.

The water level at the discharge of 350 m^3/s ($WL_{Q=350}$) in December 2040 is known (Table 7.10). If the differences between water levels at discharges 350 and 165 as well as 150 m^3/s are given, the water levels $WL_{Q=165}$ and $WL_{Q=150}$ in December 2040 can be derived by deducting the corresponding water level differences from $WL_{Q=350}$. From a rating curve, the water level difference between two different discharges can be obtained.

Figure 7.20 is the latest rating curve used (September, 1993) at the Huntly Power Station and the original data have been attached in Appendix Twelve. The differences between water levels at discharges 350 and 165 m^3/s, and 350 and 150 m^3/s from the rating curve are, respectively, 1.04 m (7.67–6.63) and 1.15 m (7.67–6.52). Under an assumption that the same differences are held in 2040, the water levels at the discharges of 150 ($WL_{Q=150}$) and 165 m^3/s ($WL_{Q=165}$) in December 2040 were obtained by using

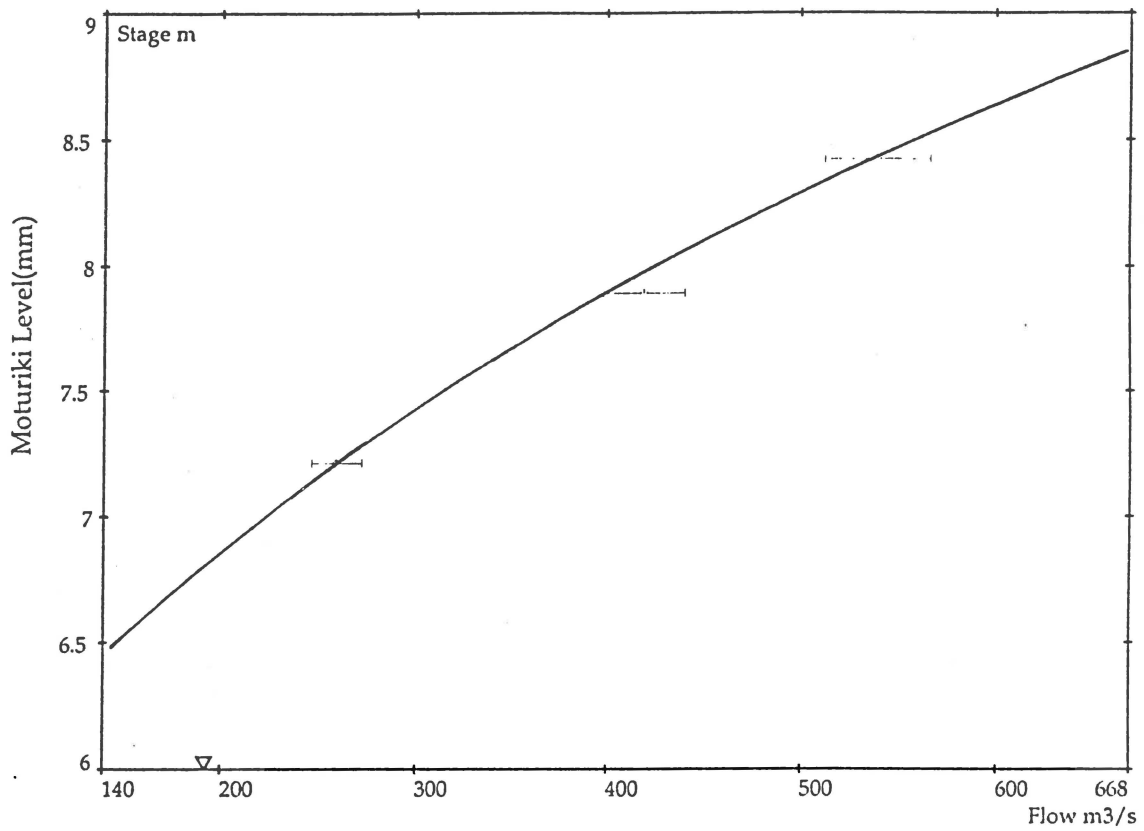


Figure 7.20 The latest (April 1993) rating curve used at the Huntly Power Station (data from TIDEDA database at the Waikato Regional Council).

$$WL_{Q=150} = WL_{Q=350} - 1.15 \text{ (m)}, \tag{7.40}$$

and

$$WL_{Q=165} = WL_{Q=350} - 1.04 \text{ (m)}. \tag{7.41}$$

Results of the water level for the different scenarios are listed in Table 7.10.

For the critical water level 6.50 m, under the condition of the river flow of 165 m^3/s, the present cooling water system at the Huntly Power Station can not be operated in 2040 in the following cases:

1-UB3&DB3,
2-UB1&DB2–3, 2-UB2&DB1–3, 2-UB3&DB1–4,

and under the river flow 150 m^3/s, in the case of

1-UB1&DB3, 1-UB2&DB1–3, 1-UB3&DB1–3,
2-UB& all the downstream boundary conditions considered.

It should be pointed out that in the case of the river flow of 150 m^3/s, even today (April 1993) the present cooling water system cannot be operated properly because the corresponding water level, 6.52 m (Appendix Twelve), is

very close to the critical level, 6.50 m. However the water level at the river flow of 165 m^3/s, 6.63 m (Appendix Twelve), exceeds the critical level by 0.13 m.

7.6 Conclusions and discussions

The linear parabolic and hyperbolic equations have been applied to the lower Waikato River in the reach between 48.25–94.45 km upstream of the river mouth. Spatial and temporal steps of 0.15 km and 1 month and a fully implicit scheme were used to obtain numerical solutions. An adjustable representative width was assumed in the domain for the movable bed simulation.

The initial steady bed levels in the domain were assumed to follow a straight line in June 1958 based on trends of the smoothed curve of the mean bed levels in the 1963/1964 survey as well as the fact of few sand operations on the river during that time. Bed levels in May 1982 and December 1990 were determined by deducting the differences between the corresponding water profiles at the discharge of 350 m^3/s from the initial bed levels under an assumption of constant Manning's coefficient, energy slope and width. The bed levels in July 1963 and May 1982 obtained by this method were compared to the mean bed levels from the 1963/1964 and 1979/1983 surveys respectively. Generally they seem to be a good approximation to the mean bed levels.

The bed levels in June 1958, May 1982 and December 1990 determined by the above method were accordingly used in the models for an initial condition, calibration and validation.

In two cases, Case A and Case B, with the same field data, different boundary conditions have been studied. For Case A, bed levels were specified at both upstream and downstream boundaries, and for case B, bed levels and bedload supplies were given at the downstream and upstream boundaries respectively. Because no field measurement data were available for bedload supplies, the annual bedload supplies given at the upstream boundary were assumed.

No significant improvements in accuracy by using the hyperbolic equation instead of the parabolic equation have been found in Case A. This is because the long term period (over 20 years) was used in our case for calibration and validation. Therefore only the parabolic model was applied to predict the

future (50 years after 1990) bed level at the Huntly Power Station.

The following discussions are related to the parabolic model. In Case A, the model with ξ=0.87 and η=0.82 gave a minimum average absolute difference $|\overline{\Delta BL}|$, 1.82 cm, between bed levels from the model and from the field data at the grids. Both parameters were physically acceptable and the validation by using the 1990 data produced the difference of 3.34 cm, which was bigger than that from the calibration by 1.52 cm. Trends of bed levels from the calibrated model in 1958–1990 at both the Rangiriri Bridge and the Huntly Rail Bridge were similar to the field data available.

In Case B, the model with ξ=0.96, η_s=1.20 and ξ_4=1.00 gave a minimum average absolute bed level difference $|\overline{\Delta BL}|$, 2.49 cm in calibration. The difference was 3.77 cm in the validation. All the parameters were physically acceptable with η_s close to its limit.

The relative difference of ξ from the calibration between these two cases is 10%. This may result from errors of the upstream boundary conditions specified.

In the model application, variations in bed level at the downstream boundary and both bed level and bedload supply at the upstream boundary were specified in the future. Several different scenarios have been considered, DB1–4, 1-UB1–3, and 2-UB1–3. The parameters used in the model depend on what kind of upstream boundary conditions is specified.

The bed level at the Huntly Power Station in the future is totally controlled by the upstream and downstream boundary conditions. Under the condition of declining bedload supplies at the upstream boundary and continued sand extraction in the lower parts of the river, the future bed level at Huntly is definitely likely to drop. For the downstream boundary, it should be noted that even if sand mining ceases immediately, bed levels there will probably still degrade due to the big holes further downstream. However the tidal effects in the lower river reaches may make the problem much more complicated and further research of the tidal effects on bedload movements upstream is required. The real bed level at Huntly in December 2040 depends on what is going to happen at both boundaries.

Effects of grain sorting on sediment movements were negligible in the model application. This was also assumed in the model calibration and validation. The fact of sand dominating the reach studied may support the no sorting assumption. But for long term river bed degradation, sorting and armouring

may become important and further research on these effects on the bed levels predicted in the future at Huntly is required.

Potential effects of the future bed level degradation on operation of the present cooling water system at the Huntly Power Station have been assessed. Water levels at the discharge of 350 m^3/s in December 2040 were determined given the corresponding bed levels with an assumption of constant Manning's coefficient, energy slope and width over time. They were deduced from the water level depth between discharges of 350 and 165 m^3/s to produce the water levels for the historical minimum flow of 165 m^3/s in December 2040. The water level depth between these discharges was obtained from the latest rating curve used at Huntly (April 1993). For the critical water level of 6.50 m required by the present cooling water system, the models indicated that it will not be operated properly in the case of 1-UB3&DB3, 2-UB1&DB2–3, 2-UB2&DB1–3 and 2-UB3&DB1–4 in December 2040.

A flow frequency analysis should be undertaken to give a design flow with a certain return period instead of the historical minimum flow used in the assessment.

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CHAPTER EIGHT

FORMATION OF ALTERNATE BARS IN THE LOWER WAIKATO RIVER

Chapter Eight

Formation of Alternate Bars in the Lower Waikato River

8.1 Introduction

An analysis of the formation of alternate bars at the Huntly Rail Bridge is undertaken by using the gauging data at that site, as well as at the Ngaruawahia Cableway, although a physical law cannot be established from the analysis. The purpose is to find out a critical width-depth ratio and its corresponding discharge influencing the alternate bar formation at Huntly .

8.2 River bedforms and origin of alternate bars

8.2.1 River bedforms

Various bedforms, such as ripples, dunes, bars, transition, flat bed, antidunes, and chutes and pools, can be formed on the bed surface of an alluvial river due to interactions between flowing water and bed materials. Current ripples, dunes, bars, sand walls (bars?), washed-out dunes, and bedform hierarchies (Allen, 1968a) were found at various places on the lower Waikato River by visual observations and echo sounding traces (Ministry of Works, 1962; Ridall, 1967; Finley, 1972; Dahm, 1987; Fenton, 1989).

Based on geometry of bedforms, ASCE (1966) provided definitions of bedform types with photographs. The geometry of bedforms depends on various hydraulic parameters and in turn, the bedforms also influence these parameters. Descriptions of bedforms and their characteristics associated with hydraulic parameters can be found at various places (Yen, 1970; Simons and Richardson, 1971; Yalin, 1972; Vanoni, 1974; Raudkivi, 1976; Allen, 1982; Leeder, 1982; Jaeggi, 1987; Nelson and Smith, 1989a; 1989b). Because there are many hydraulic variables affecting the formation of bedforms, and interactions among them are complicated, a complete relationship between bedforms and hydraulic parameters is not yet achieved. Therefore bedform phase diagrams, based mainly on experimental data, have been commonly used to show their relationships. For example,

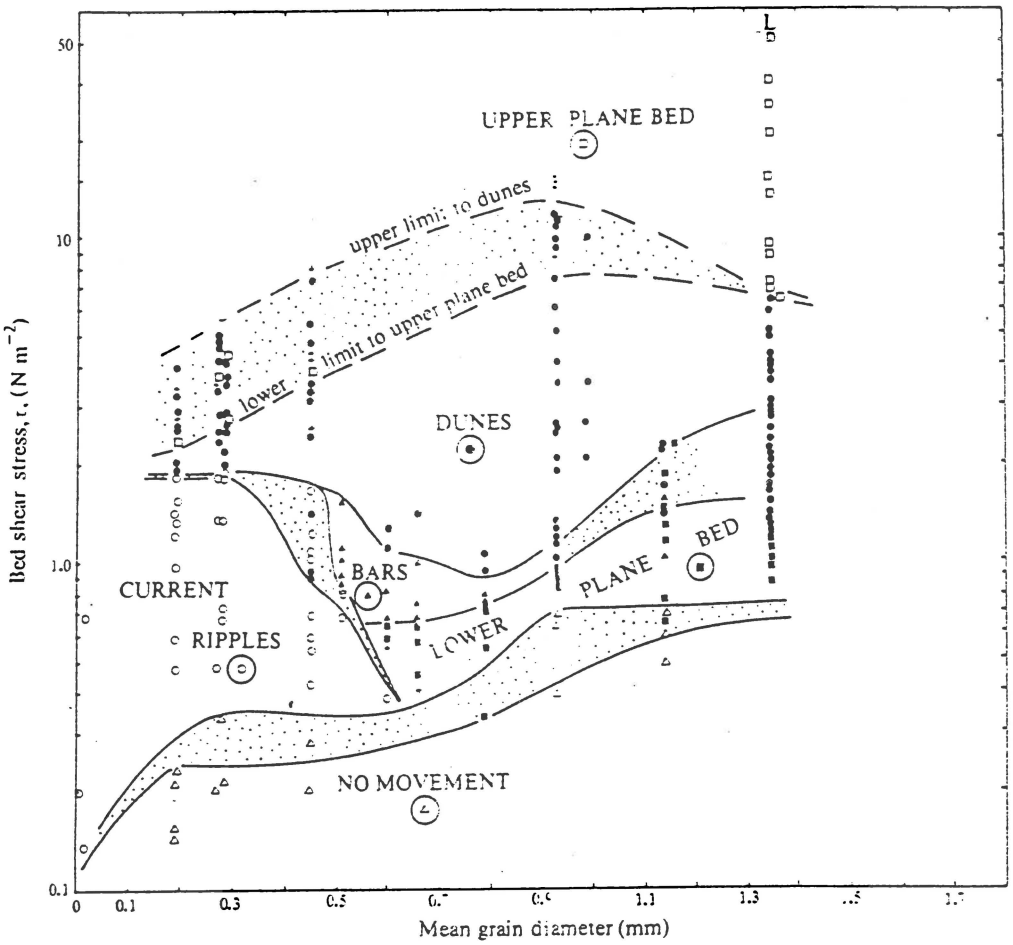


Figure 8.1 Relationship between bedforms and hydraulic parameters (after Leeder, 1982, page 85).

Figure 8.1 shows a relationship between various bedforms and parameters of mean grain size and bed shear stress.

Usually bedform diagrams are presented as non-dimensional boundary shear stress against grain size (e.g. Allen, 1985, page 72), and bed shear stress against mean grain size (Figure 8.1). Simons, *et al.* (1965a) used stream power and van Rijn (1984c) used transport stage parameter instead of stress in bedform diagrams. The other approach is a depth-velocity-size diagram (Southard, 1971). Reynolds and Froude numbers can also be included in diagrams (Fairbridge and Bourgeois, 1978, page 621).

A huge volume of data on the relationship between bedforms and hydraulic elements has been collected from several decades of laboratory experimentation on quartz-density sediments transported through straight and sinuous channels by uniform steady aqueous currents (ref. Allen, 1982). Because of the difference between experimental conditions and natural environments, there is uncertainty in application of the results from flume

data to natural rivers. The difference includes unsteady flows on a variety of scales in a natural river (Allen, 1973), and generally deeper water depth with wider channel for a natural river. Therefore caution is required when using the experimental results in practice.

The following discussion focuses on bars, especially alternate bars. Because of their significance in river science and engineering, recently more and more related experimental and theoretical studies have been undertaken by researchers all over the world (e.g. Ikeda and Parker, 1989; Lisle, *et al.*, 1993; Whiting and Dietrich, 1993a; 1993b; 1993c).

8.2.2 Bars and origin of alternate bars

Above the threshold of sand movement, an equilibrium plane bed exists on artificially planed bed with coarse sands (grain diameter over 0.7 mm) instead of ripples. This is termed the *lower-stage plane bed.* Bars can develop over the lower-stage plane bed on a coarse sand bedstock (Figure 8.1). They are three dimensional bed configurations, and their hydraulic origins remain poorly understood. Bars were defined by ASCE (1966) as *"bed forms having lengths of the same order as the channel width or greater, and heights comparable to the mean flow depth of the generating flow"* based upon the geometry or morphologic criteria. Ripples and dunes may occur on gentle sloping upstream or *stoss* sides of the bars (Leeder, 1982).

Four types of bars were originally distinguished by ASCE (1966): 1. *point bars*; 2. *alternating bars*; 3. *transverse bars*; and 4. *tributary bars*. However there are so many different configurations of bars observed in natural fluvial rivers, and no less than 32 specific terms preceding the word "bar" were found in a non-exhaustive survey by Smith (1978). In Japan, even the alternate bars have been further classified into three types: single-row alternate bars, double-row alternate bars, and alternate bars with higher mode (Ikeda, 1984). The concept of movable bars and stable bars was discussed by Jackson (1975). Recently *free bar* and *forced bar* have been used by Seminara and Tubino (1989).

Costello (1974) indicated that bars have a high R.I. (Ripple Index, which is defined as the ratio of wave length to wave height) value and a broad R.I. distribution in comparison to dunes and ripples. They show straight crests, leeside avalanche-induced migration and lack of leeside scour hollows, which are associated with dunes (Coleman, 1969; Leeder, 1982). Not only the local hydraulic factors but also the non-local effects due to the re-distribution of flow and sediment movement can be the factors influencing bed

deformation from experiments (Struiksma, *et al.*, 1985).

Although no strong correlations were suggested by Leeder (1982) between bar wavelength or height and water depth, field observations indicated that alternate bars flatten under the condition of high discharges and/or high water depth (Neill, 1969; Chang, 1985; Fenton, 1989). For example, alternate bars were formed on the Alpine Rhine after construction of river training works. However they vanished when further narrowing (high water depth?) of the central channel occurred (Jaeggi, 1984). The reason may be that small bedforms (ripples and dunes) migrate at a faster pace than bars in the same system (Boothroyd and Hubbard, 1974; Jain and Kennedy, 1974). Therefore more sediment can be transported at high flows associated with small bedforms.

Water depth seems to be one of the factors influencing the formation of alternate bars. The ratio of river width to water depth has been widely used as a criterion for their formation (e.g., Chang, et. al., 1971; Sukegawa, 1973; Jaeggi, 1984; Ikeda, 1984; Chang, 1985; Whiting and Dietrich, 1993a).

"Alternating bars tend to be distributed periodically along a channel, with alternate bars near opposite channel banks. Their lateral extent is significantly less than the channel width. Alternating bars move slowly downstream." (ASCE, 1966). Evidently alternating bars are related to stream forms and meandering thalweg path within the straight or even sinuous channel (Maddock, 1969; Seminara and Tubino, 1989; Whiting and Dietrich, 1993a), as well as flow discharges, slope and grain sizes (Figure 8.1). Simons and Richardson (1971) indicated that the position, shape and magnitude of alternate bars were a function of channel alignment, bed material and width-depth ratio. Dietrich, *et al.* (1979), Dietrich and Smith (1984), Hasegawa and Yamaoka (1984), Dietrich (1987), and Whiting and Dietrich (1993) investigated effects of meander on bar formation and migration. Processes of river meandering with bars in an artificially straightened reach with coarse-grained sediments under a condition of natural discharges were reported by Lewin (1976). Effects of varying flows on meander length were studied in a laboratory by Ackers and Charlton (1970).

Under an assumption of the same resistance in both river bank and bed, free meanders may occur. In contrast to free meanders, alternate bars are confined meanders within the width constraint of a straight or slightly sinuous channel. However the same resistance assumption above is hard to be accepted in natural environments. The time scale for formation of the

channel pattern of meanders and the river bedform of alternate bars is entirely different (Chang, 1988). Thus independent and dependent variables for the formation of alternate bars and river meanders may not be the same.

There are many theories on river meandering. A review of river meandering studies was given by Callander (1978). Scheidegger (1991) classified the theories of river meandering as *hydraulic geometry theory*, *mechanistic theory,* and *stochastic theory.* The hydraulic geometry theory is based on regime theory or river at grade. The mechanistic theory includes secondary flow, flow instability, external control. The stochastic theory is based on the most probable random walk and expectable paths. Shen (1971b) summarised the causes of meanders as:

1. *development of secondary currents.*
2. *maladjustment between the hydraulic properties of the flow and its available topographical conditions.*
3. *the dynamic stability of the flow.*
4. *principles of minimum variance and random walk.*

Three theories were critically reviewed by Chang (1988). They were *the most probable path* (Langbein and Leopold, 1964), *dynamic instability* (Callander, 1978; Engelund and Skovgaard, 1973; Parker, 1976; Ikeda, *et al.*, 1981; Blondeaux and Seminara, 1985), and *secondary currents* (Tanner, 1960; Einstein and Shen, 1964; Shen and Komura, 1968; Kitanidis and Kennedy, 1984). All of them treated meandering as a phenomenon (the effect) without giving the cause (Chang, 1988). Theories of *the Coriolis force* (Neu, 1967), and *bank erosion* (Friedkin, 1945) were reviewed by Yang (1971). He explained river meandering by using minimisation of stream power.

A review of the feasibility of modelling meanders was given by Cunge (1983) that summarised the existing approaches, which included correlative relationships; a synthetic approach based on overall river characteristics; an analytical approach based on detailed flow description; global simulation based on flow equations and calibrated coefficients. He concluded that: ɔ*There is no satisfactory operational model simulating meandering phenomena and it seems unlikely that such a model will be developed in the immediate future* (Cunge, 1983).

Practical application of achievements in meander modelling to natural environments is still limited. Further understanding of the mechanism of bars and meanders, and developments of applied technology and related

disciplines are required. Therefore nowadays empirical relationships are still a useful approach, and they are usually determined from experimental data by using a statistical analysis. Physical laws, however, cannot be established by this analysis (Snyder and Stall, 1965) and their applications are often limited.

8.2.3 Empirical relationships for alternate bars

In practice, information about the position, shape and magnitude of alternate bars in a river is useful for river engineering and management. Quantitative representation of this information includes parameters for alternate bar formation, bar length or bar wavelength, and bar height or scour depth (Figure 8.2). The bar wavelength is approximately equal to twice the bar length. The bar height and the scour depth are measured at a section including the lowest depression in a bar unit. The former is measured from the lowest point of bed level to the top of bed surface, and the latter from the lowest point of bed level to the reach averaged bed surface (Ikeda, 1984).

Channel alignment, bed material, slope, shear stress, width and depth are believed to be the parameters influencing the alternate bars (Simons and Richardson, 1971). In practice, for a given river reach, bed materials, river bank alignment and slope (?) are pre-determined in the time scale of the alternate bar formation (days or weeks).

In terms of alignment effects, alternate bars can be formed in a straight channel. This conclusion has been widely reported (e.g. Friedkin, 1945). The recent flume experiments even showed that in channels whose wavelength was about eight channel widths, alternate bars could exist in sine-generated channels with angles up to 10 (Whiting and Dietrich, 1993a). Otherwise point or forced bars might be formed.

Leopold, *et al.* (1964) indicated that natural rivers rarely flow in a straight channel for more than ten channel widths and that the term *straight* also covers all somewhat irregular, slightly winding, nonmeandering river channels.

Straight channels are assumed in the following discussions on criteria for the alternate bar formation.

Kinoshita (1957) first proposed a condition for the formation of alternate bars as a range of the width-depth ratio:

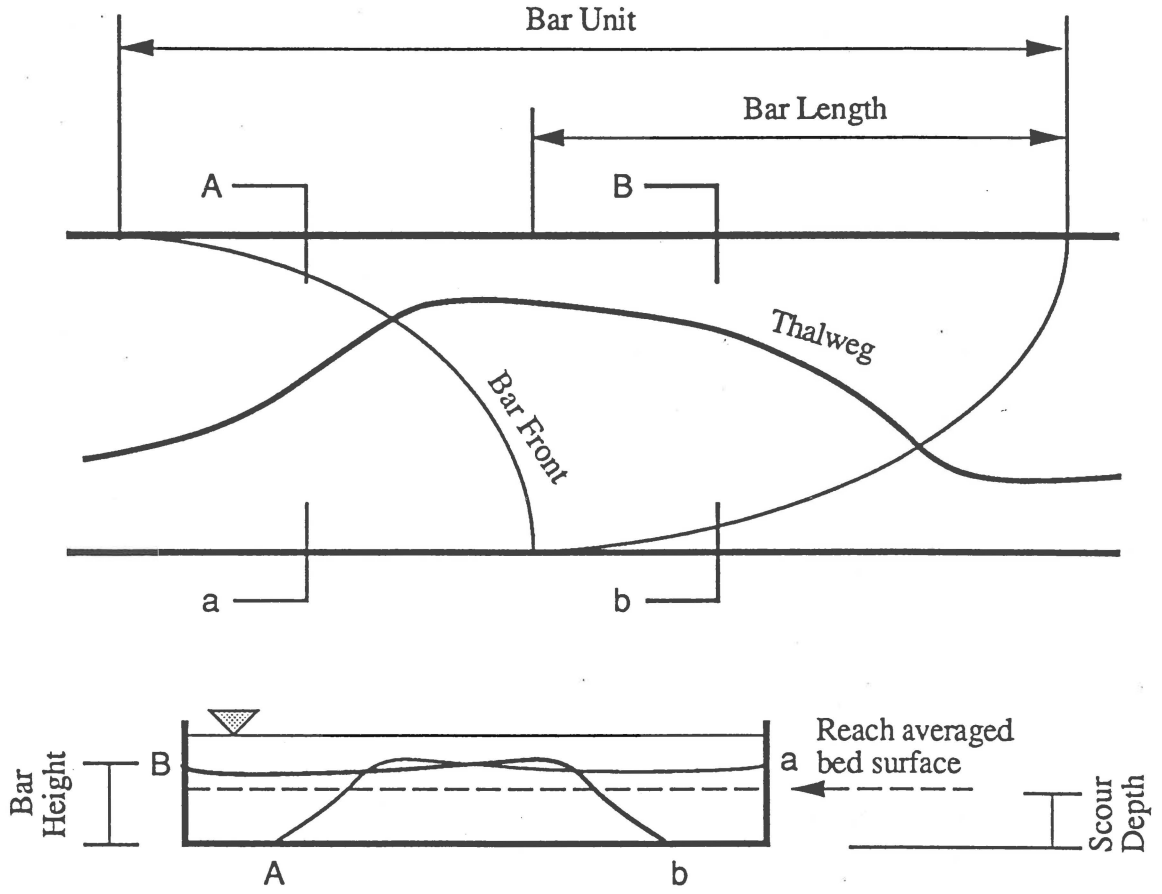


Figure 8.2 Definition of sketch of alternate bars used in this study.

$$20 > \frac{W}{H} > 5, \tag{8.1}$$

where W is the width, and H is the water depth.

Chang, *et al.* (1971) gave the condition as

$$\frac{W}{H} > 12. \tag{8.2}$$

Sukegawa (1973) used dimensional analysis and developed the following condition for alternate bar formation:

$$\frac{W}{R} \geq \frac{1}{125}\left(\frac{u_*}{u_{*c}}\right)^2 \frac{1}{S}, \tag{8.3}$$

where u_* and u_{*c} are the shear velocity and the critical shear velocity respectively, S is the slope, and R is the hydraulic radius.

Ikeda (1984) mainly studied bar wavelength and bar height. He presented the condition for alternate bar formation as

$$70 > \frac{W}{H} > 4. \tag{8.4}$$

Jaeggi (1984) used the parameter η the ratio of actual to critical Shields factor, $\eta = \theta / \theta_{cr}$ to indicate whether for a given channel, alternate bars can be developed for *any* discharge. The upper and lower limits for the alternate bar formation were given as:

$$\eta = 2.93 \ln \eta_B - 3.13 Z_B^{0.15} \quad \text{(upper)}, \tag{8.5a}$$

$$\eta = 1 \quad \text{(lower)}, \tag{8.5b}$$

where η_B is the ratio of actual to critical Shields factor related with channel width; Z_B is the ratio of channel width to mean grain size.

Chang (1985) indicated that alternate bars were formed because of meandering development within the confined channel if the streamflow s stable width was less than the confinement width. The stable width is a function of discharges. At high flows, if the stable width is greater than the confinement width, alternate bars are absent.

All the conditions above indicated that for a given river reach there is a critical ratio of river width to water depth, say ξ_c, and under the condition of $\xi > \xi_c$, alternate bars occur. However a universal value of ξ_c or its equation has not been obtained yet. Intuitively, another condition is also required that the river flow must be strong enough to initiate bedload transport.

Because of the relationship between water depth and discharge (e.g., by using the Manning equation), accordingly the critical ratio condition for the alternate bar formation can also be presented by a discharge for a given rectangular channel.

The alternate bar wavelength (λ) (about twice the bar length) and the meander wavelength were found to be proportional to the channel width (W). This was presented by Leopold and Wolman (1957) as

$$\lambda = \alpha W^{\beta} \tag{8.6}$$

with $\alpha = 6.5$ and $\beta = 1.1$. However Fujita (1980) indicated that α varied between 3 and 13, and β was approximately one.

After studying the configuration of alternate bars based on flume experiments and field data, Ikeda (1984) presented the following equations for estimate of the alternate bar wavelength:

$$\lambda = 5\left(\frac{WH}{C_f}\right)^{0.5}, \qquad \text{for } Fr < 0.8, \tag{8.7a}$$

$$\frac{\lambda}{W} = 181 C_f \left(\frac{W}{H}\right)^{0.55}, \qquad \text{for } Fr \geq 0.8, \tag{8.7b}$$

where C_f is a resistance coefficient, and Fr is a Froude number.

For the alternate bar height H_B, Kishi (1978) proposed that it was 5% of the channel width (W):

$$H_B = 0.05W. \tag{8.8}$$

Ikeda (1984) presented a more complicated equation for determination of bar height as

$$\frac{H_b}{H} = \left(\frac{W}{d}\right)^{-0.45} \Psi, \qquad \text{for } Fr \geq 0.74 \text{ and } 70 > \frac{W}{H} > 4, \tag{8.9}$$

where d is the median particle size, Ψ is a function of the width-depth ratio. A diagram for determination of Ψ was attached in the paper.

Jaeggi (1984) provided an equation to estimate the scour depth H_S. The scour depth, being 0.76 times the bar height, H_B, was a function of width (W) and ratio of width to mean grain size (Z_B):

$$H_S = 0.76 H_B = \frac{W}{6 Z_B^{\,0.15}}. \tag{8.10}$$

There are considerable prediction errors in the equations above for determination of bar length or bar wavelength and bar height. For example, the error of Eqs. (8.7a), (8.7b) and (8.9) as a percentage is, respectively, in a range of $-40 \sim +80\%$, $-40 \sim +75\%$, and $-45 \sim +85\%$. Therefore as a first estimate of the bar length and the bar depth, direct application of the bar definition (ASCE, 1966) seems a reasonable approach. Approximately, we can take the bar length to be 3 times the channel width (cf. Leopold and Wolman, 1957), and the bar height to be the mean flow depth of the generating flow (ASCE, 1966).

8.3 Alternate bars in the lower Waikato River

8.3.1 Field observations

The distribution of the Ripple Index (R.I.) along the lower Waikato River from Hamilton to Puni, which was determined by using echo sounding techniques, shows that generally the R.I.s were below 100. Some higher R.I.s (>150) occurred in the reach between Huntly and Mercer (Figure 2.9). This fact indicates that big bedforms (bars and dunes) probably occur in that reach because bars possess an R.I. which is generally equivalent to or bigger than that for dunes, which are greater than that for ripples (Allen, 1968b, 1970a; Costello, 1970). In the lower Waikato River, the R.I. for bars is over 150; about 20–50 for dunes; generally less than 40 for ripples (Fenton, 1989). The data from Ridall (1967) indicate the R.I. of 51–61 for dunes.

Detailed field three dimensional surveys from February 1988 to January 1989 by Fenton (1989) in a reach with length of 1 000 m near Orton (Figure 1.3) confirmed the existence of movable alternate bars there (Figure 8.3). However they disappeared or could not be defined under the condition of high flows or high water depth.

Sand bars were sometimes observed in the river just around the intake and outfall of the Huntly Power Station (N. Pritchard, Group Environmental Manager, Electricorp NZ, 1990, pers. comm.). Ridall (1967) also reported that a sand wall (bar?) varying in height from 7 to 9 feet (2.1 to 2.7 m) was observed at Huntly between June 1966 and January 1967. It moved downstream at an average speed of about 3 feet a day (0.9 m/day). However dunes with a length of 250 to 300 feet (76–92 m) and a height of 5 feet (1.5 m) moved at a speed between 8 to 20 feet a day (2.4–6.1 m/day) at Churchill East (Ridall, 1967).

Based on the cross-sectional configurations near cross section 121 at Huntly in Figure 2.6 as well as Figures 8.2 and 8.3-C, alternate bars most likely occurred there.

Gauging experiences from staff at the Waikato Regional Council also indicated that bed levels at the Huntly Rail Bridge had a more considerable variation associated with big bedforms than those at the Ngaruawahia Cableway, where the river bed seems to be stable (H. McMullan, the Waikato Regional Council, pers. comm., 1993). Results of the river surveys at these two sites further confirmed this conclusion (Figures 8.4–5).

The wavelength of the thalweg meanders or twice the alternate bar length was found to be 1100–1400 m by Fenton (1989) from aerial photographs in the

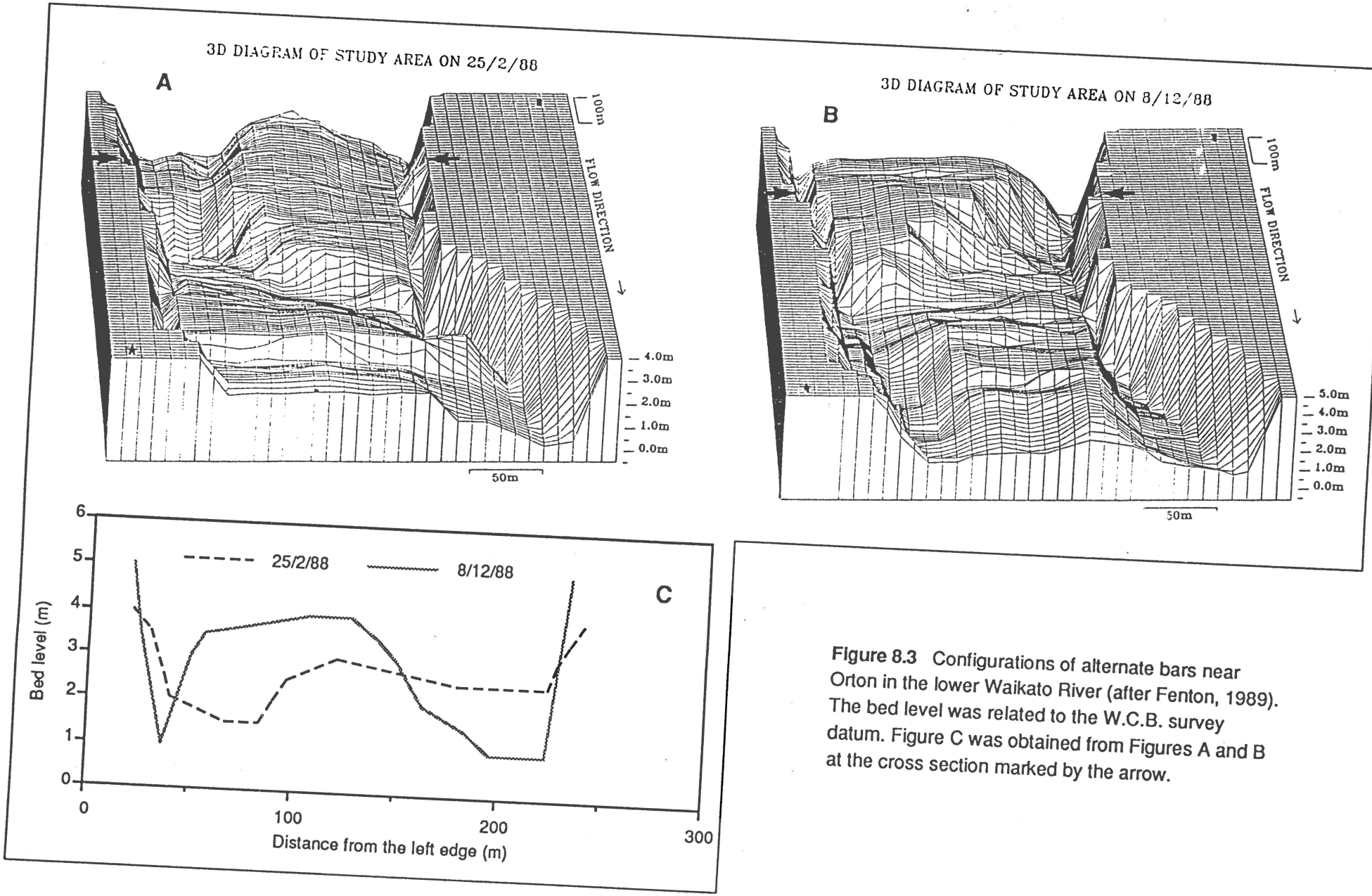


Figure 8.3 Configurations of alternate bars near Orton in the lower Waikato River (after Fenton, 1989). The bed level was related to the W.C.B. survey datum. Figure C was obtained from Figures A and B at the cross section marked by the arrow.

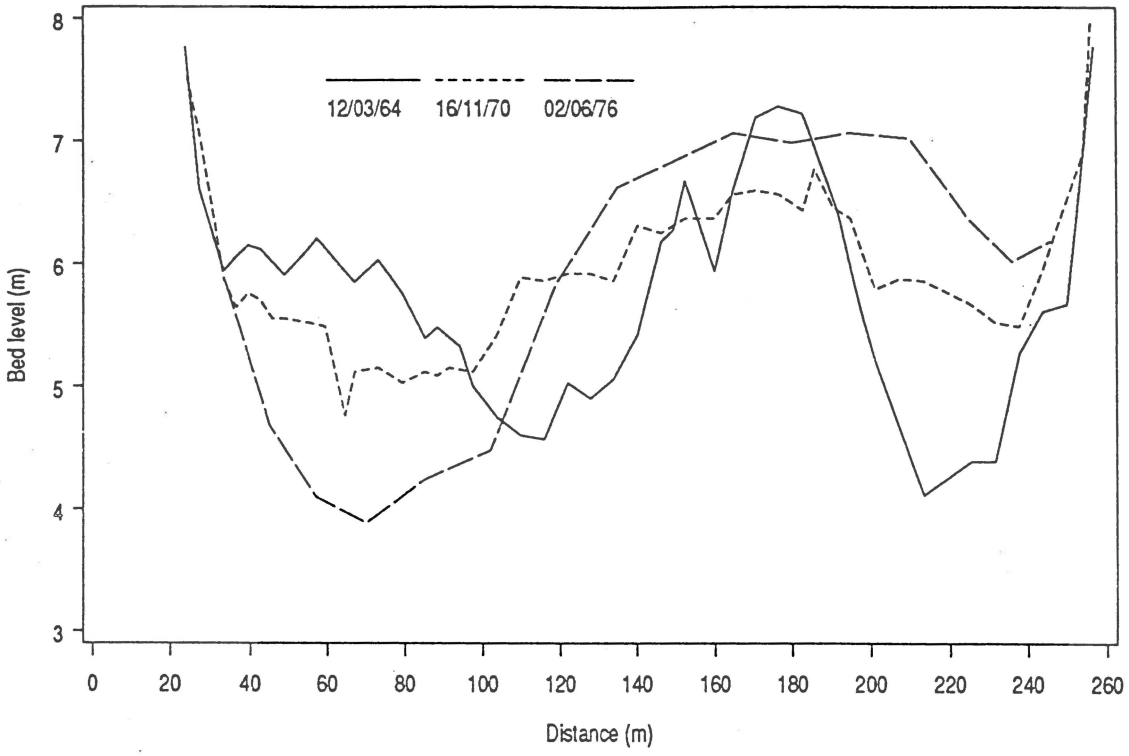


Figure 8.4 Cross-sectional configurations at the Huntly Rail Bridge on 12-03-1964, 16-11-1970 and 02-06-1976 (data from the river surveys held at the Waikato Regional Council). The bed level variations seems to indicate the occurrence of alternate bars.

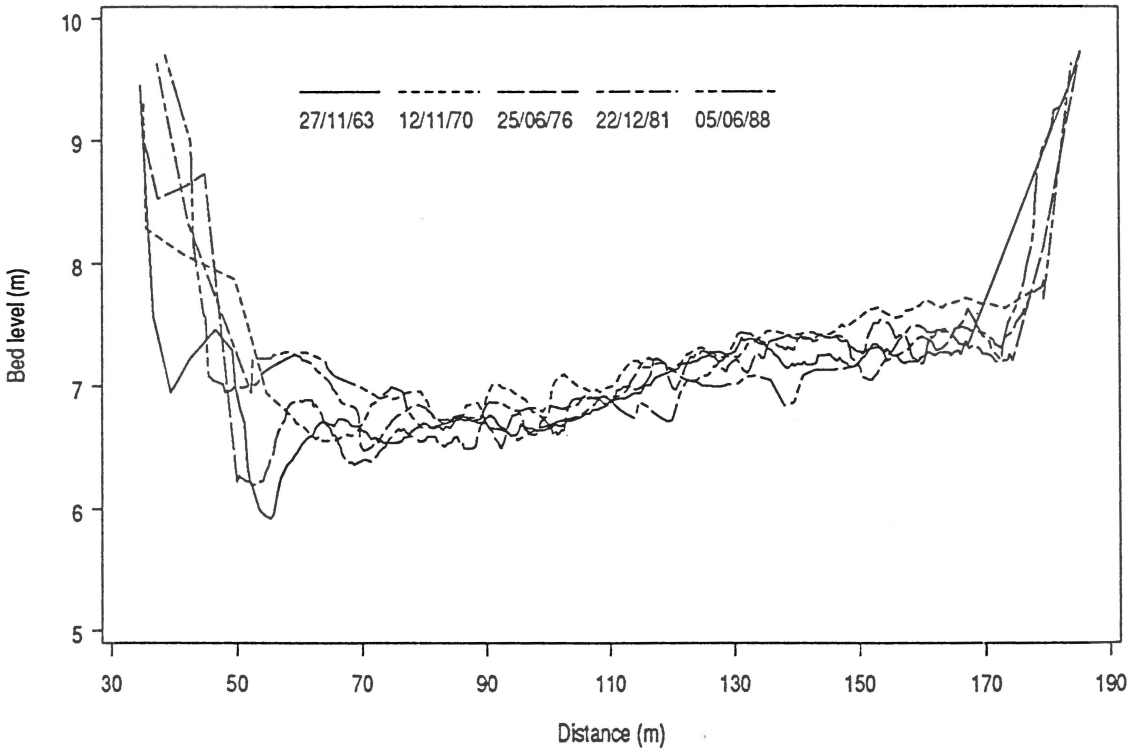


Figure 8.5 Cross-sectional configurations at the Ngaruawahia Cableway on 27-11-1963, 12-11-1970, 25-06-1976, 22-12-1981, and 05-06-1988 (data from the river surveys held at the Waikato Regional Council). Variations in bed level are relatively small in comparison with those at Huntly.

river reach around Orton. However in the Huntly reach a wavelength of 2800–3000 m, which is nearly twice that found by Fenton further downstream, was reported by Dahm (1987).

8.3.2 Gauging data at the Huntly Rail Bridge and the Ngaruawahia Cableway

Two gauging stations, one at the Huntly Rail Bridge and the other at the Ngaruawahia Cableway, are selected for analysis of the alternate bar formation. In terms of bed materials, there are no big differences in grain size between these two stations. Some piers exist in the river channel at the Huntly Rail Bridge. The river reaches around these two stations are shown

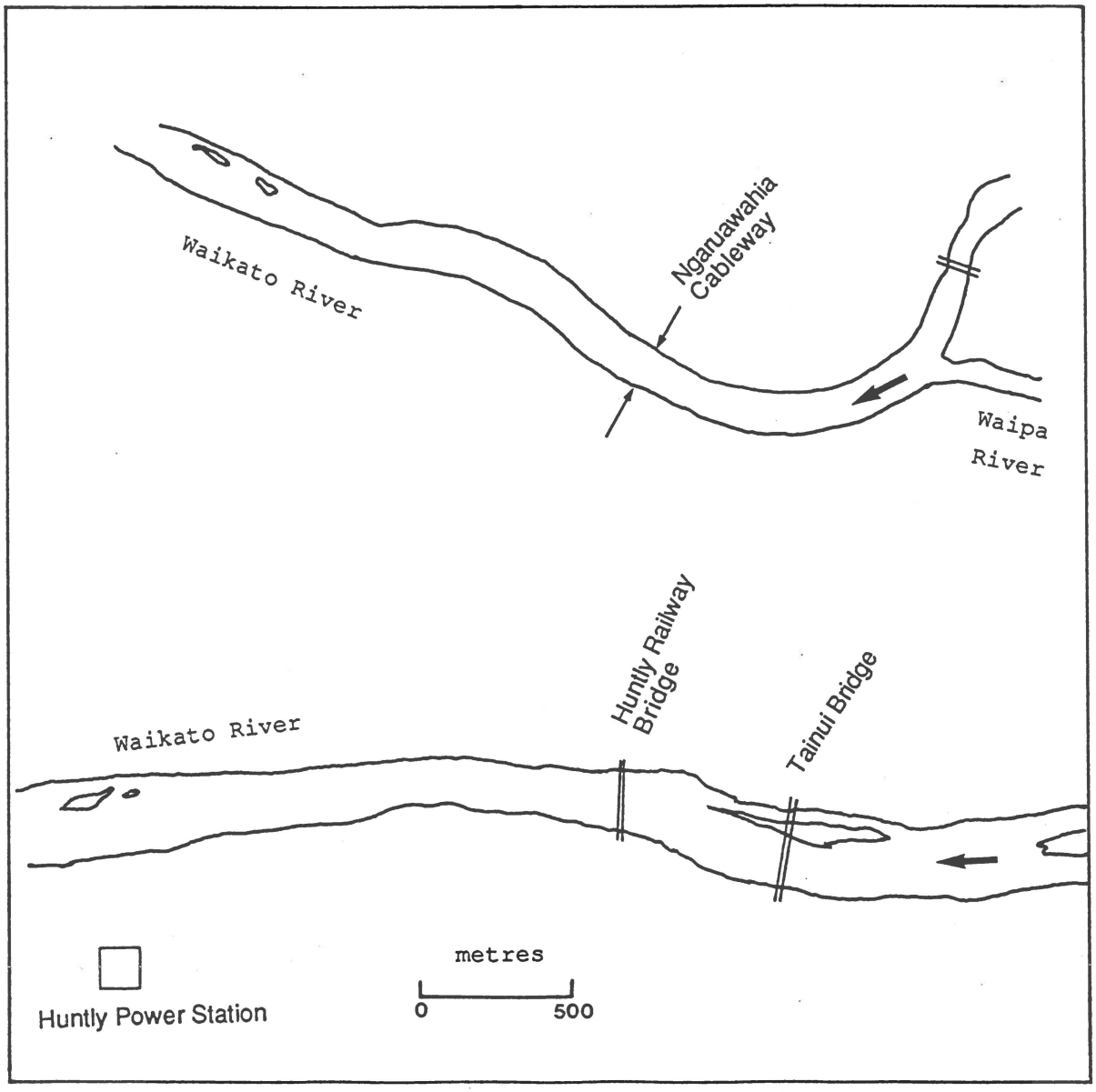


Figure 8.6 Sketch of locations of the Ngaruawahia Cableway and the Huntly Rail Bridge.

in Figure 8.6, and the channels seem to be essentially straight there. However, a big bend is located upstream of the Ngaruawahia Cableway.

Data used for analysis of the alternate bar formation consist of the cross-sectional measurements from gauging at the Huntly Rail Bridge and the Ngaruawahia Cableway.

The gauging data at the Huntly Rail Bridge are listed in Table Appendix 4.1. Among them, five records measured between 1958–1966 have been omitted in order to reduce the time period for the whole set of data, and the data measured after March 1980 have been deleted because of a river width change caused by road construction on the river bank since then (ref. Chapter Four). One point from gauging card No. 663 in 1968 has been deleted because it has been prone to errors.

For the gauging data at the Ngaruawahia Cableway, nine records taken before 1965 (seven in 1958, one in 1960 and one in 1964) have been omitted in this analysis. Records No. 927 (18-Aug-1970) and 958 (2-Oct-1970) have been omitted because the original gauging cards are missing and prone to errors. The latest record, No. 21124 (7-Oct-1993), has been deleted since the gauging was taken at other cross sections. Zero of the cross-sectional distance in six sets of gauging (No. 972, 8895, 8911, 9157, 11181 and 12753) has been adjusted by about 10–20 m. The gauging data used in this analysis at the Ngaruawahia Cableway is attached in Appendix 13.

A summary of the hydraulic parameters of discharge, water level, mean bed level, river width, water depth, and ratio of river width to water depth of the data used for analysis at these two stations is listed in Table 8.1. Figure 8.7 shows a frequency distribution of width-depth ratios at both stations. In terms of variations in ratio, there is an overlap between these two stations.

Table 8.1 A summary of hydraulic parameters of discharge, water level, mean bed level, width, water depth, and ratio of width to depth at the Huntly Rail Bridge and the Ngaruawahia Cableway (data from the Waikato Regional Council). "S.D." refers to a standard deviation.

	Huntly Rail Bridge (1967–1980; n=89)			Ngaruawahia Cableway (1967–1993; n=165)		
	Min.–Max.	Mean	S.D.	Range	Mean	S.D.
Discharge (Q, m^3/s)	167.6–1035.2	454.5	208.7	145.8–960.4	406.3	204.8
Water Level (WL, m)	7.16–10.58	8.51	0.87	8.83–12.64	10.26	1.03
Mean Bed Level (BL, m)	5.50–6.20	5.91	0.17	6.75–7.84	7.18	0.24
River Width (W, m)	225.0–250.5	235.0	7.17	134.0–179.0	152.8	9.59
Water Depth (H, m)	1.37–4.68	2.61	0.82	1.79–4.93	3.08	0.83
Ratio of W to H	50–166	98	28	35–83	52	11

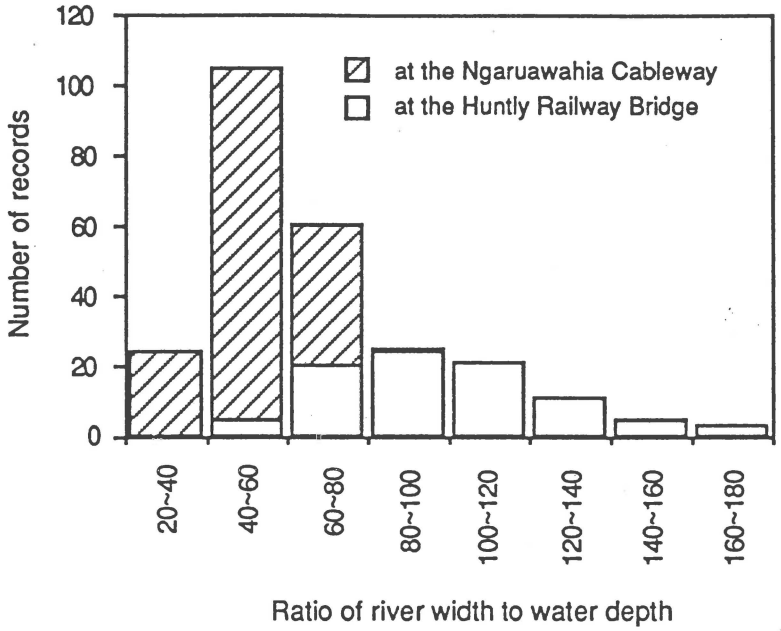


Figure 8.7 Frequency distribution of width-depth ratio at the Huntly Rail Bridge between 1958–1980 and the Ngaruawahia Cableway between 1958–1993 (data from the Waikato Regional Council).

8.3.3 Methods for determination of the alternate bar formation

Field three dimensional surveys associated with hydraulic measurements are one approach to investigate alternate bars: their formation, geometry characteristics, such as bar length and bar height, and effects on river current fields and bedload movements.

It is also possible to study alternate bars by analysing a time series of cross-sectional configurations measured at a given cross section with some known hydraulic parameters because alternate bars move along the river. Based on Figures 8.2 and 8.3, if alternate bars occur and a number of cross-sectional surveys, including measurements of the lowest levels of the alternate bars, are taken, variations in bed level at the cross section, especially at the places close to both river banks (Figure 8.3), must reach a critical value. This critical value for occurrence of alternate bars is expected to be larger than that without occurrence of alternate bars. According to the definition of bar height, the critical value should be close to the bar height. Unfortunately, no universal equations are available for determination of the critical value. For different channels, the critical value may be different.

For the following analysis of the alternate bar formation at the Huntly Rail Bridge, the critical value of about 2.5 m was selected based on Ridall (1967). This figure seems reasonable in comparison to the height between the lowest bed level to the top surface shown in Figures 8.3–4.

Direct comparisons of cross-sectional configurations and extreme variations in bed level at certain places for the data of two different width-depth ratio groups at Huntly are undertaken. One group with ratios less than 80 is supposed to include the data with occurrence of alternate bars, and the other with ratios bigger than 100 is not. Two places close to the left river bank between 50–65 m, and between 65–80 m at the Huntly Rail Bridge are selected. Statistical tests on the variance of bed level between the two groups at these two places were also undertaken.

Based on the discussions above, variations in bed level for occurrence of alternate bars and without alternate bars at the places close to a river bank are not in the same order. Therefore it is expected that the null hypothesis of the same variance in the two groups will be rejected.

The F test was used for the hypothesis test (Green and Margerison, 1978). The null hypothesis is given as

$$H_0: \sigma_1 = \sigma_2, \tag{8.11}$$

where σ_1 and σ_2 are, respectively, the standard deviations for group 1 and group 2. The alternative hypothesis corresponds to

(a) $H_1: \sigma_1 > \sigma_2$, or
(b) $H_1: \sigma_1 < \sigma_2$, or
(c) $H_1: \sigma_1 \neq \sigma_2$.

The test criterion, f, is determined by the following equations:

$f = s_1^2 / s_2^2$ for case (a),

$f = s_2^2 / s_1^2$ for case (b),

$f = \max(s_1^2, s_2^2) / \min(s_1^2, s_2^2)$ for case (c),

where s_1 and s_2 are, respectively, estimates of σ_1 and σ_2. f is believed to be distributed as $F_{\upsilon_1,\upsilon_2}$ under H_0 with the numbers of degrees of freedom $\upsilon_1 = n_1 - 1$ and $\upsilon_2 = n_2 - 1$, in which n_1 and n_2 are the sample sizes for group 1 and group 2 respectively.

We reject H_0 at a significant level of α if $f \geq F_{\upsilon_1,\upsilon_2}(1-\alpha)$ in case (a), or $f \geq F_{\upsilon_2,\upsilon_1}(1-\alpha)$ in case (b), or $f \geq F_{\upsilon_{num},\upsilon_{den}}(1-\alpha/2)$ in case (c) for a two-sided test, where υ_{num} and υ_{den} are the numbers of degrees of freedom in the numerator and denominator respectively. However it should be noted that if the null hypothesis cannot be rejected, the conclusion is that either the null

hypothesis is true or there are not enough data available to detect the small differences to be tested.

In our case, group 1 and 2 included those data with ratio less than 80 and bigger than 100 respectively, corresponding to the case (b) above.

8.3.4 Results and discussions

Figures 8.8–9 are, respectively, cross-sectional configurations from all the gauging records at the Huntly Rail Bridge and the Ngaruawahia Cableway.

Generally variations in bed level from a total of 165 gauging records at the Ngaruawahia Cableway are less than 1.5 m (Figure 8.9), while at the Huntly Rail Bridge exceed about 2.5 m at the places close to both river banks (Figure 8.8) from a total of 89 gauging records. Table 8.2 lists the minimum and maximum bed levels measured close to the left river bank in the distances of 50–65 and 65–80 m for all the data at the Huntly Rail Bridge. The number of points can be different from the total number of gauging records. Ranges for bed level variation are 2.66 and 2.51 m respectively (Table 8.2). The bed level variations close to the right river bank seem to be partially affected by the upstream island (Figure 8.6).

Table 8.2 Variations in bed level at the Huntly Rail Bridge close to the left bank within distances of 50–65 and 65–80 m. "BL" stands for bed level, and "S.D." represents a standard deviation.

Records	All		Ratio≤80		80<Ratio≤100		100<Ratio	
Place	50–65 m	65–80 m	50–65 m	65–80 m	50–65 m	65–80 m	50–65 m	65–80 m
Number	94	113	27	34	24	30	43	49
Min BL m.	3.780	3.901	4.366	4.326	4.770	4.390	3.780	3.901
Max BL m.	6.436	6.409	6.436	6.009	6.392	6.271	6.287	6.409
Range m.	2.656	2.508	2.070	1.683	1.622	1.881	2.507	2.508
S.D. m.	0.543	0.467	0.539	0.457	0.449	0.449	0.574	0.491

Based on the critical value for bed level variation with the alternate bar formation, 2.5 m (section 8.3.3), no alternate bars existed at the Ngaruawahia Cableway, and alternate bars occurred at the Huntly Rail Bridge.

Width-depth ratios of the gauging records at the Ngaruawahia Cableway are between 35–83 (Table 8.1). Most of them are less than 80 (Figure 8.7). Thus for the Huntly Rail Bridge, it is expected that alternate bars should not occur at least in these records with their width-depth ratios less than 80.

Figure 8.10 is the cross-sectional configurations at the Huntly Rail Bridge from the gauging records with their width-depth ratios less than 80 and a

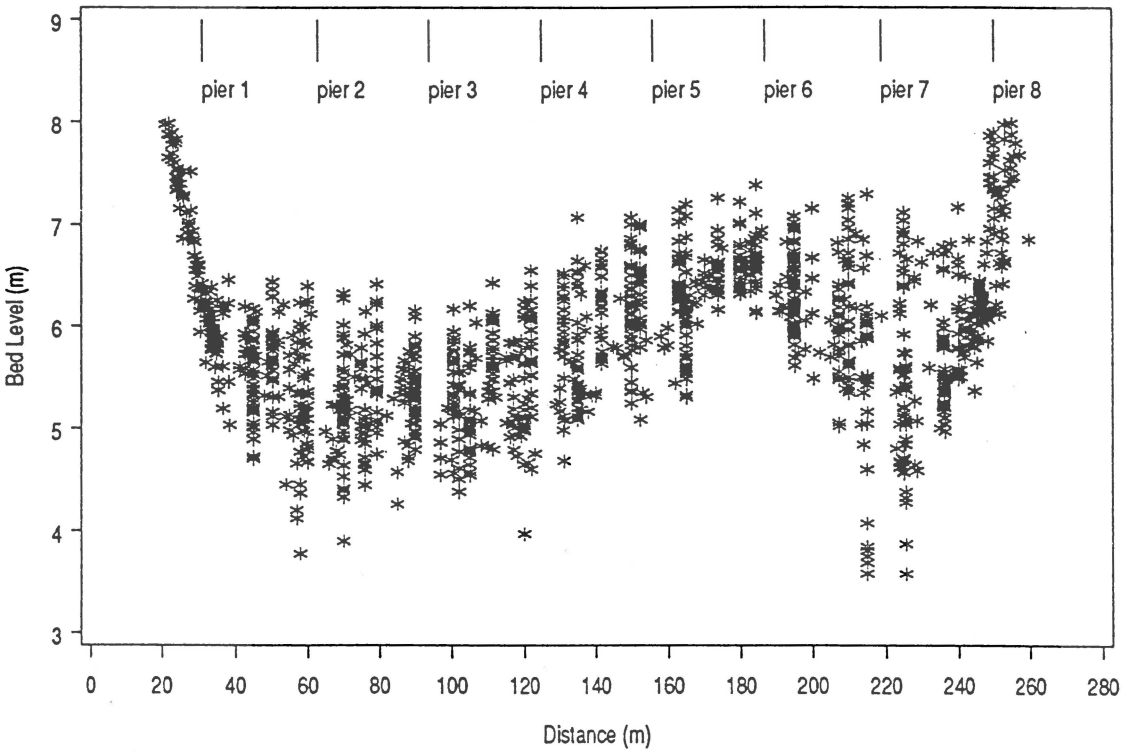


Figure 8.8 Cross-sectional configurations at the Huntly Rail Bridge with a total of 89 records between 1967–1980 (data from gauging records held at the Waikato Regional Council). Variations in bed level close to both banks exceed 2.5 m.

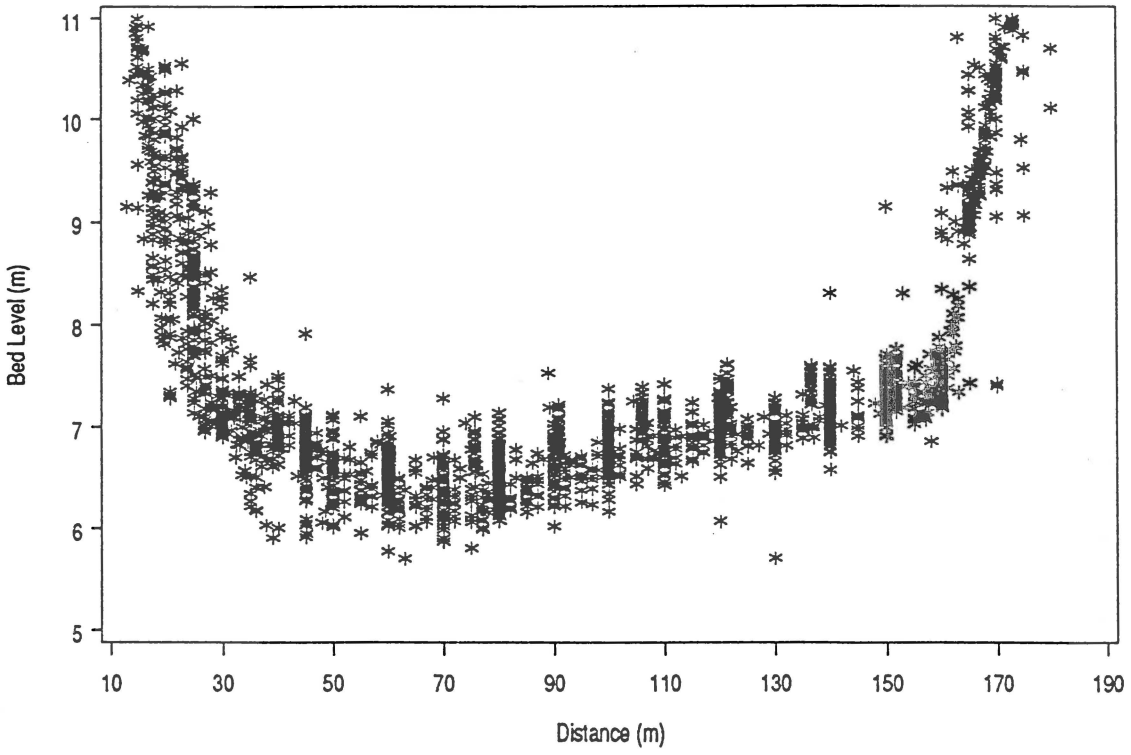


Figure 8.9 Cross-sectional configurations at the Ngaruawahia Cableway with a total of 165 records between 1967–1993 (data from gauging records held at the Waikato Regional Council). Generally variations in bed level are less than about 1.5 m.

total of 25 records. In comparison with the configurations from all the gauging records at that site (Figure 8.8), generally the range of bed level variations is small, especially at the places near both river banks. The little deep channel at the right river bank exists consistently. Only at the right bank, variations in bed level are bigger than 2.5 m, that may be caused by an upstream island control (Figure 8.6). However bed level variations close to the left bank are generally less than about 2 m. From Table 8.2, within the distances of 50–65 and 65–80 m, ranges of bed level variation are 2.07 and 1.68 m respectively. Therefore no alternate bars occurred there in these cases (ratios less than 80). Corresponding hydraulic parameters to the gauging records used in Figure 8.10 are summarised in Table 8.3.

Figure 8.11 is the cross-sectional configurations at the Huntly Rail Bridge from the gauging records with their width-depth ratios exceeding 100. A total of 40 records were used in this figure. Different labels were taken in the figure for the records with their ratios between 100–120 and bigger than 120. A summary of discharge, water depth and water level relating to these records is given in Table 8.3. Generally variations in bed level at the places near both river banks, especially at the left bank, reach about 2.5 m, and no deep sub-channel at the right bank can be clearly identified (Figure 8.11). Ranges of bed level variation are both 2.51 m in the distances of 50–65 and 65–80 m. Therefore alternate bars mostly likely occurred in these cases (ratios bigger than 100).

A comparison of bed level envelopes for the two groups of gauging records with ratios less than 80 and bigger than 100 is shown in Figure 8.12. The envelopes were drawn by eye. Obviously variations in bed level for the records with their ratios exceeding 100 are bigger than those with the ratios less than 80.

The gauging records with their width-depth ratios between 80–100 at the Huntly Rail Bridge were also plotted (Figure 8.13). Generally, bed level variations are similar to those in Figure 8.10. Except for those close to the right river bank, bed level variations are less than about 2 m. Ranges of bed level variation in the distances of 50–65 and 65–80 m are 1.61 and 1.88 m respectively (Table 8.2).

The analysis above indicates that alternate bars occur at the Huntly Rail Bridge under the condition that the ratios of river width to water depth are greater than 100. This figure is much bigger than those suggested in the literatures (ref. section 8.2.3). The corresponding conditions for discharge

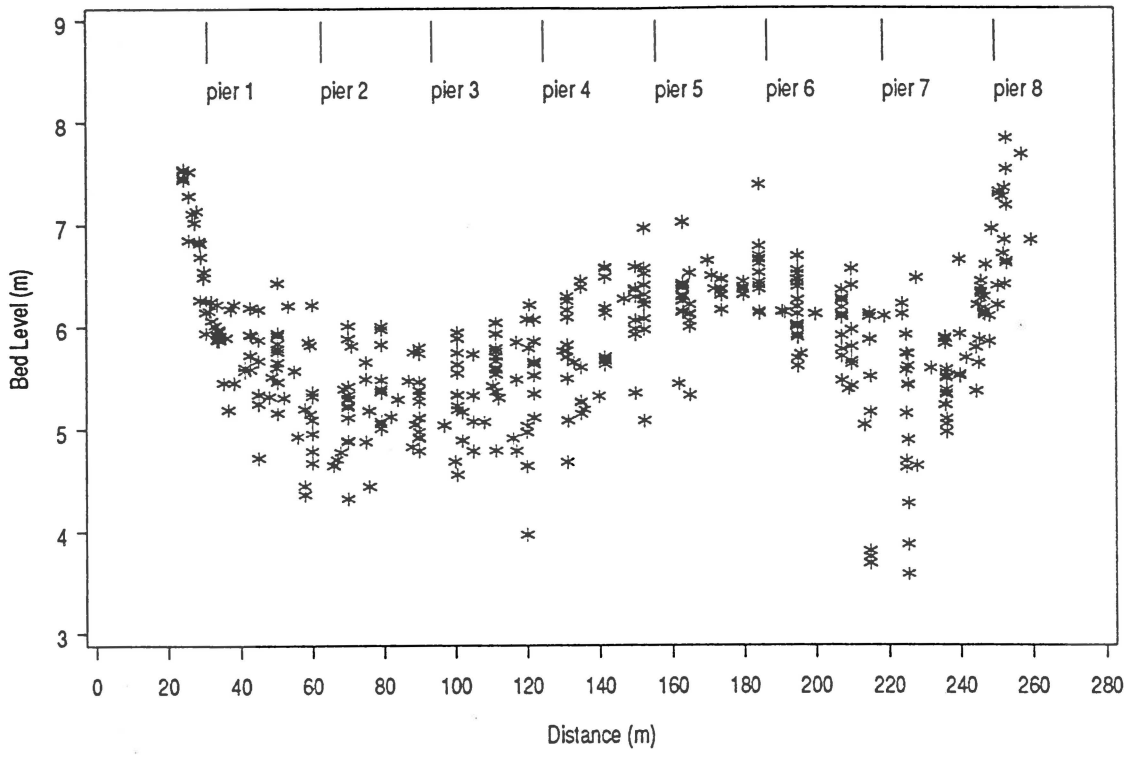


Figure 8.10 Cross-sectional configurations at the Huntly Rail Bridge using the records with width-depth ratios less than 80 (n=25) between 1967–1979 (data from the Waikato Regional Council). Generally variations in bed level are less than 2 m, except for those close to the right bank, which may be caused by an upstream control.

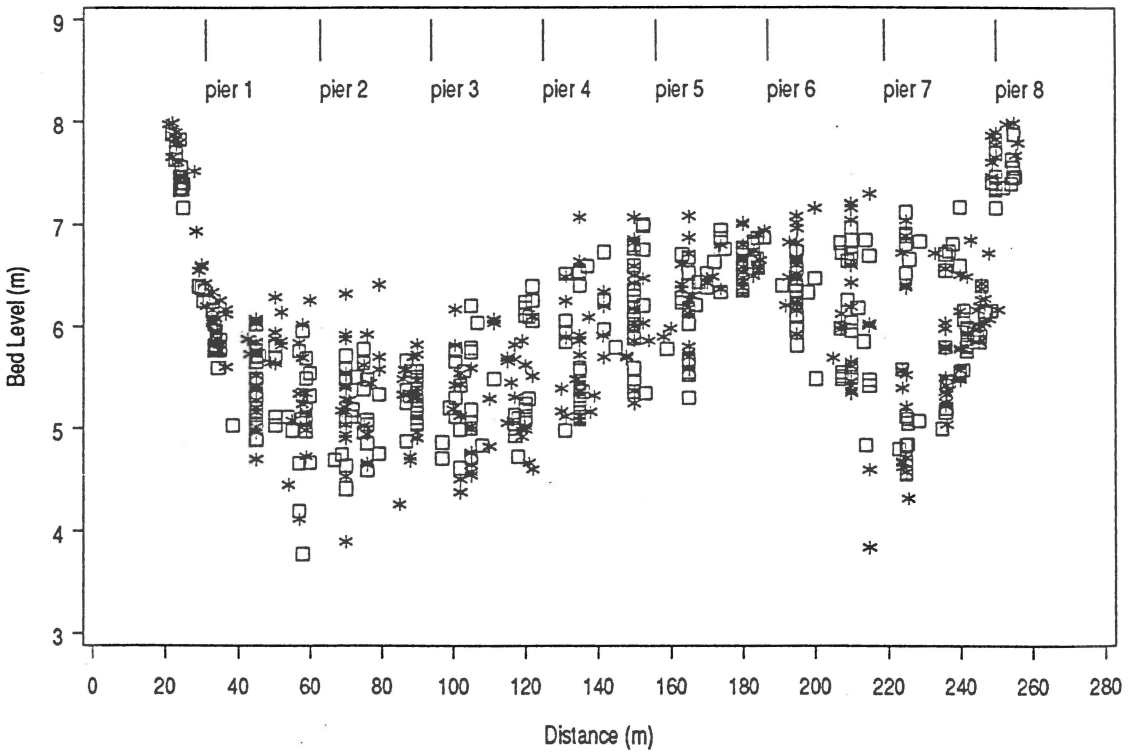


Figure 8.11 Cross-sectional configurations at the Huntly Rail Bridge using the records with width-depth ratios greater than 100 (stars for 100–120, n=21; and squares for 120+, n=19) between 1967–1980 (data from the Waikato Regional Council). Generally variations in bed level close to both river banks exceed 2.5 m.

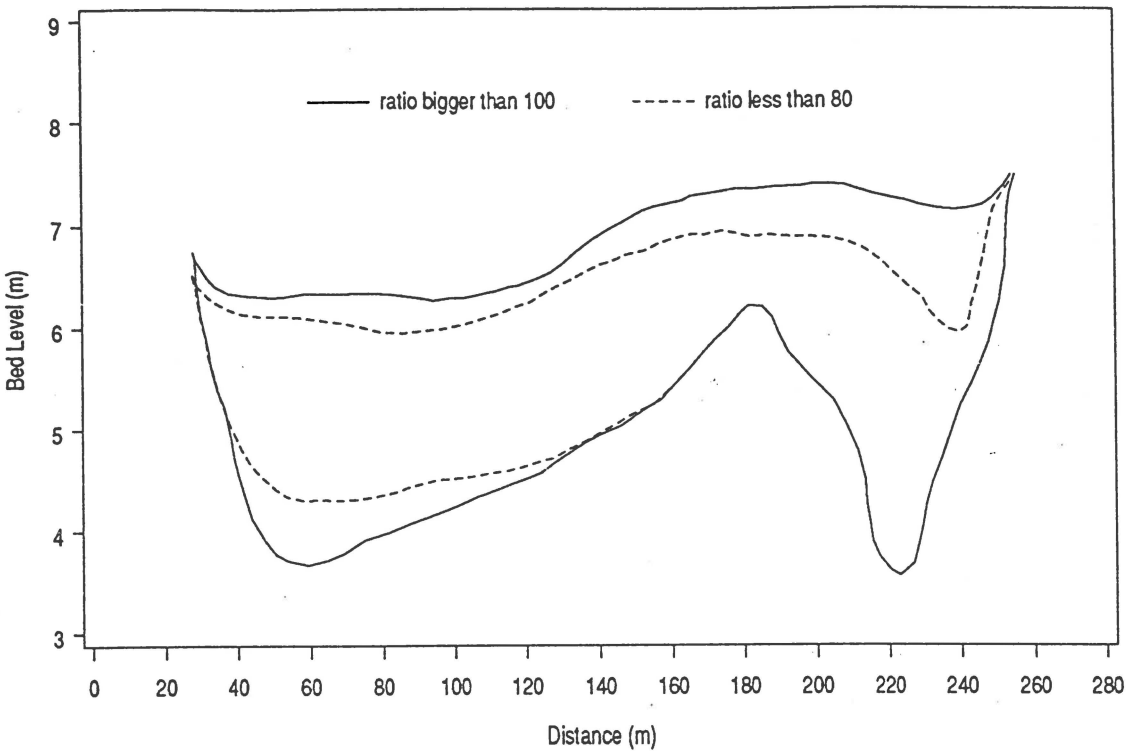


Figure 8.12 Envelopes of bed level variations for width-depth ratios less than 80 and greater than 100 at the Huntly Rail Bridge. They are drawn by eye based on Figures 8.10–11.

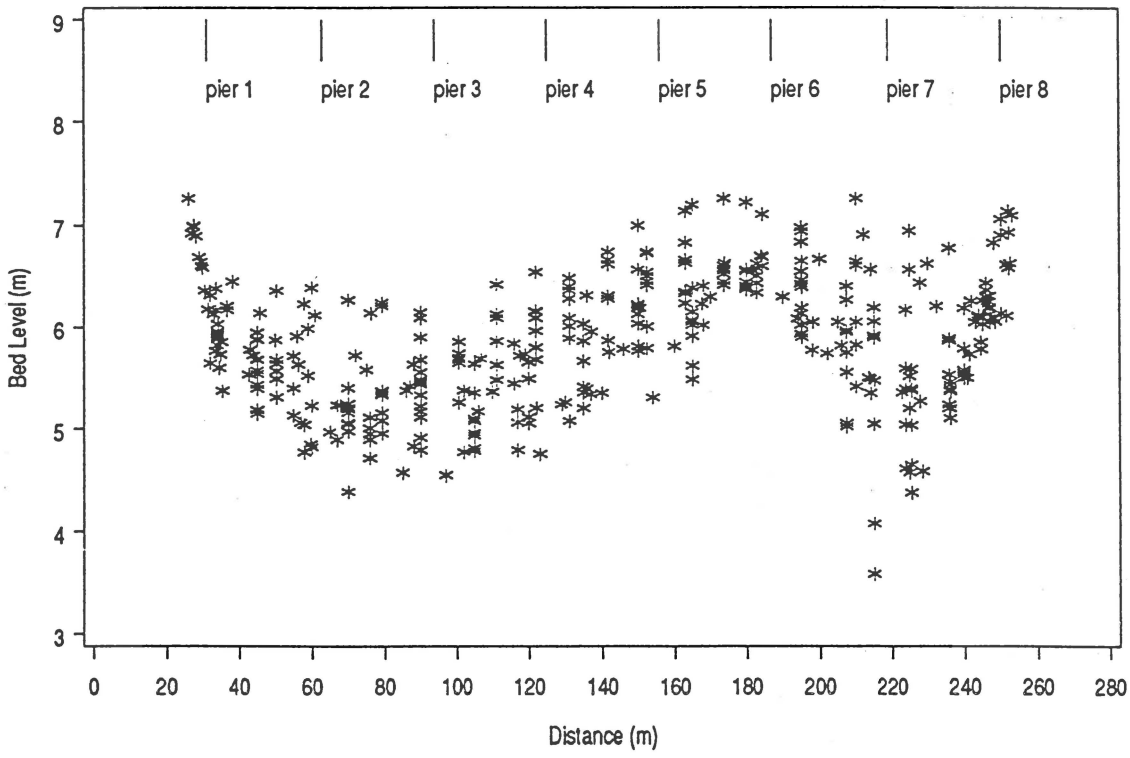


Figure 8.13 Cross-sectional configurations at the Huntly Rail Bridge using the records with width-depth ratios from 80–100 (n=24) between 1967–1980 (data from the Waikato Regional Council). Generally variations in bed level are less than 2 m except for those close to the right bank, which may be caused by an upstream control.

Table 8.3 A summary of the gauging data at the Huntly Rail Bridge with their width-depth ratios less than 80, between 80–100, and bigger than 100 respectively (data from the Waikato Regional Council).

Ratio (R)	No.	Period	Discharge (m^3/s) Min–Max	Water Depth (m) Min–Max	Water Level (m) Min–Max
R<80	25	1967–1979	523–1035	3.02–4.68	8.88–10.58
80≤R<100	24	1967–1980	334–556	2.31–2.94	8.02–8.99
100≤R	40	1967–1980	167–396	0.65–2.30	7.16–8.47

and water depth are, respectively, less than about 350 m^3/s and 2.30 m (Table 8.3).

The null hypothesis of the same variance of bed level within the distances of 50–65 and 65–80 m at the Huntly Rail Bridge for two groups with different width-depth ratios was undertaken (ref. section 8.3.3; Table 8.4). Group 1 includes the records with ratios less than 80 and group 2 bigger than 100. Unfortunately the null hypothesis cannot be rejected at the 5% significant level. As indicated early, this test conclusion can be due to that there are not enough representative data available to detect the small differences or H_0 is true.

Table 8.4 F test on variance of bed level within the distances of 50–65 and 65–80 m at the Huntly Rail Bridge.

$H_0: \sigma_1 = \sigma_2$; $H_1: \sigma_1 < \sigma_2$ $\alpha = 0.05$	Group 1: data with width-depth ratios less than 80. Group 2: data with width-depth ratios bigger than 100.	
Place	50–65 m	65–80 m
s_1	0.539	0.457
$\upsilon_1 = n_1 - 1$	26	33
s_2	0.574	0.491
$\upsilon_2 = n_2 - 1$	42	48
$f = s_2^2 / s_1^2$	1.134	1.154
$F_{\upsilon_1, \upsilon_2}(1-\alpha)$	1.862	1.752
Decision	do not reject H_0	do not reject H_0

It should be pointed out that hydraulic parameters, such as discharge and water level, vary over time in natural environments. By using simultaneous hydraulic parameters to investigate the bedforms in this study, their representativeness needs to be confirmed. The representativeness of the cross section at the Huntly Rail Bridge for the around river reach should also be studied. In the period of the data used, mean bed levels at both gauging stations degraded by about 0.4 m (Chapter Four) and these have not

been considered in the analysis. Whether reductions in mean bed level affect the final results is unknown. Therefore further research on alternate bars by using three dimensional surveys associated with hydraulic measurements is required to confirm the conclusions.

8.4 Conclusions

A complete explanation of the formation of alternate bars is not available yet. Further developments of theories and experiments are required. The existence of alternate bars in the lower Waikato River under certain conditions has been reported. For this study, a critical value, 2.5 m, of bed level variations for alternate bar occurrence was specified, especially at the places close to both river banks. However further studies on the critical value are required.

Because the ratio of river width to water depth is one of factors influencing the formation of alternate bars, an analysis has been applied to determine the ratio for the alternate bar formation at the Huntly Rail Bridge. The gauging data at this site and at the Ngaruawahia Cableway were used for the analysis.

The analysis indicated that alternate bars occurred at the Huntly Rail Bridge when the ratio of river width to water depth bigger than 100. Under this condition, variations in bed level within the distances of 50–65 and 65–80 m close to the left bank at the Huntly Rail Bridge reached 2.5 m. In the case of a ratio less than 80, the bed level variations were less than about 2 m, and probably no alternate bars occurred.

The null hypothesis test of the same variance of bed level within the distances of 50–65 and 65–80 m at the Huntly Rail Bridge for one group with ratio less than 80 and the other group bigger than 100 has been undertaken. The former group is supposed to include data with occurrence of alternate bars, and the latter is not. Unfortunately at the significant level of 5%, the hypothesis cannot be rejected yet.

The critical width-depth ratio, 100, for the formation of alternate bars at Huntly is much bigger than those suggested in the literature. Corresponding to the critical ratio, discharges and water depth are, respectively, less than about 350 m^3/s and 2.30 m.

Further field three dimensional surveys are necessary to confirm the conclusions.

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CHAPTER NINE

CONCLUSIONS AND RECOMMENDATIONS

Chapter Nine

Conclusions and Recommendations

9.1 Introduction

A brief summary of conclusions relating to the objectives outlined in Chapter One is presented in this chapter. In addition implications for river environmental management based on the conclusions are discussed, and recommendations made for further research on the lower Waikato River.

9.2 Conclusions and implications

9.2.1 Catchment background and bed material characteristics of the lower Waikato River

Sediments in the Waikato River are mainly gravelly pumiceous sands reflecting the volcanic history of the catchment area.

In terms of the mean and median grain sizes, generally sand dominates the lower Waikato River bed downstream of the section at about 100 km from the river mouth (above Ngaruawahia), while gravel dominates the bed upstream as far as the Karapiro Dam from the available data between 1961–1991. There is a hiatus evident in the longitudinal mean and median grain size distributions around Horotiu. Because no bed material data taken before construction of the Karapiro Dam are available for comparison, and a similar grain size distribution in natural rivers without dams has also been reported, these bed material distributions in the lower Waikato River cannot be used to indicate the downstream effects of the dams. The texture of bed materials in the Waipa River was generally similar, but probably finer, to those in the lower parts of the Waikato River. This implies that the bed materials stored in the Waipa River courses could be a potential source of bedload in the lower Waikato River.

9.2.2 Bedload sediment yield and bedload movement in the lower Waikato River

Sources of the erodible volcanic debris transported into the lower river, which have been commercially extracted from the river since the 1950s, are not only the sediment yields from catchments but also bed materials stored in the river courses years ago.

The bedload transport rate at Churchill East was estimated at about 170 000 m^3/yr in the 1970s, and the rate of suspended load at Ngaruawahia in the Waikato River was estimated at about 49 000 m^3/yr by using the sediment concentrations measured in 1990–1992 and the mean discharges. An analysis of bedload sediment balance indicated that 160 000±24 000 m^3/yr (mean±standard deviation) was the rate of annual bedload supply from the river above cross section 132 at Ngaruawahia. The available historical river survey data and sand extraction records for the period between 1964–1989, and an assumption of no bedload yields from the subcatchment between the river mouth and cross section 132, and no bedload transported into the sea during that corresponding period were used in this analysis. It is worth mentioning that for the average annual amount of bedload supply of 160 000 m^3 to the river downstream of Ngaruawahia, nearly two-thirds was contributed by the bed materials stored in the upstream Waikato River course from an analysis of upstream bedload balance in the period between 1974–1989, probably resulting from bed surface disturbance by sand extraction as well as downstream effects of the Karapiro Dam.

The bedload supply to the lower river from both the Hamilton basin and the Waipa River was estimated at about 45 700±30 700 m^3/yr (mean±standard deviation) by using results of the river surveys of 1974/1976 and 1985/1989, and an output rate of 160 000 m^3/yr.

9.2.3. Gauging data and rating curve analysis in the lower Waikato River: assumptions, methods, and results

On the assumption that $\partial n / \partial Q = \partial W / \partial Q = \partial S / \partial Q = 0$ where n, W, S and Q are, respectively, Manning's coefficient, width, energy slope and discharge, water levels adjusted to an index flow of 350 m^3/s ($WL_{Q=350}$) can be derived from each gauging record of the lower Waikato River. From this a time series of water levels adjusted to 350 m^3/s can be established by using the entire set of gauging records at each station. Therefore gauging data can be treated as a time series. A cubic spline method was applied in this study to interpret the time series.

Further assuming $\partial n / \partial T = \partial W / \partial T = \partial S / \partial T = 0$ where T is time, variations of the time series of $WL_{Q=350}$ represent the mean bed level changes over time. These assumptions seem to be acceptable for the gauging data within a certain range of discharges in the lower Waikato River. Plotted points of mean bed levels and adjusted water levels from the gauging data at Hamilton, Ngaruawahia, Huntly and Rangiriri, especially at Hamilton and Ngaruawahia, are grouped around a unit slope line. These facts support the assumptions above. That the points from the data at Mercer varied from a unit slope line may be caused by the non-representative mean bed levels used at Mercer and/or non-constant energy slope because of its location close to the limit of tidal influence.

Based on rating curves at each gauging station, water levels for an index flow of 350 m^3/s within certain periods can be obtained. Therefore their changes over time can be determined.

Results of the analysis of the gauging data and rating curves at Hamilton, Ngaruawahia, Huntly, Rangiriri and Mercer in the lower Waikato River in terms of variations in minimum bed level, mean bed level, water level adjusted to the flow of 350 m^3/s indicated a considerable reduction in river bed. The adjusted water levels dropped by about 1.0 m between 1960–1992 at the Hamilton Traffic Bridge; by about 0.45 m between 1958–1992 at the Ngaruawahia Cableway; by about 0.42 m between 1958–1992 at the Huntly Railway Bridge with a sudden drop in the 1970s, then a small recovery in the beginning of the 1980s, and nearly stable after 1985; by about 0.40 m between 1970–1992 at the Rangiriri Bridge; and by about 2.4 m between 1960–1992 at the Mercer Bridge.

9.2.4 River surveys and changes in channel pattern, river volume, discharge, and water and bed levels in the lower Waikato River

The channel pattern in the lower Waikato River is more or less stable. No cutoffs have been found since 1942.

River flows downstream of Ngaruawahia are not only controlled by operations of the Karapiro Dam, draining 54% of the total catchment area, and the flood control constructions on its lower system, but also contributions from the tributaries downstream, especially the Waipa River controlling 21.5% of the total catchment area, during storms. Both the Hamilton basin and the Waipa River catchment drain about 26% of the entire catchment area and their contribution to an extreme flow at Huntly may be important.

Regular cross section surveys in the lower Waikato River have been undertaken by the Waikato Regional Council since 1963 at an interval of 5–6 years. Each survey might last 2–5 years with different cross sections and sequences. This made it difficult to use the survey data as a time series for the entire lower river. It is also difficult to determine the number of years between two surveys. It is worth considering extending surveys for the same cross sections and sequences and completing them within a short period, e.g. six months or a year, in the future river surveys. A total error at 95% confidence level of ±15 cm was suggested for the bed level measurements of the lower Waikato River surveys in this study. For cross-sectional area measurements, the total error was assumed to be 5% of its entire area.

The total net river volume increase in the reach between the river mouth and Ngaruawahia was estimated at about 6 527 000 m^3 between surveys of 1963/1964–1979/1983 and about 10 980 000 m^3 of 1963/1964–1985/1989. For the reach from Ngaruawahia to the Karapiro Dam, the river volume was increased by 1 131 000 m^3 in the period between surveys of 1974/1976–1985/1989.

Generally mean bed levels of cross sections surveyed in the river downstream of Ngaruawahia dropped between the surveys of 1963/1964–1985/1989. Their smoothed fit lines also indicated the bed level reduction with a big drop around Mercer and further downstream. A comparison of cross-sectional areas in the same reach from the survey data demonstrated big net increases around Mercer and further downstream of Puni and Tuakau.

Water level profiles at a discharge of 350 m^3/s between Mercer and Ngaruawahia between 1958–1990 also show a drop over time with the same pattern as the mean bed level reductions. The profiles were obtained from results of the gauging data analysis and water level profile measurements.

Reductions in bed level of the Waikato River upstream of Ngaruawahia were clearly indicated by changes in cross-sectional area from the survey data and the gauging data at the Hamilton Traffic Bridge in the period of 1950–1992. These reductions contributed a great amount of bedload sediments to the lower river sedimentation.

9.2.5 Possible factors influencing the water and bed level reduction in the lower Waikato River

Human activities that utilise natural resources, such as dam construction, river training works and commercial sand extraction, have had a

significant influence on the river regime.

The long term commercial sand extraction located around Puni, Tuakau and Mercer is the major factor causing the local bed level drops. The bed level reduction upstream of Mercer as far as Ngaruawahia may result from river training works, sand extraction, upstream effects of the big bed level drop around Mercer, and decline and/or cessation of sediment supply from the upper catchment because of dam closures.

The sudden drop in its mean bed level and adjusted water level at the Huntly Rail Bridge in the 1970s, shown in the results of the gauging data analysis, evidently resulted from local sand extraction associated with the power station construction.

Reductions in bed level of the Waikato River upstream of Ngaruawahia could be the results of downstream effects of dam closures. However the sand mining operations from the 1950s to 1976 in the Hamilton reach must also be one of the influencing factors. The sand mining operations not only contributed to the local bed level reduction, but their disturbance of the bed surface layer may also have caused sands below the surface to move more easily even after cessation of the sand mining. Re-establishment of a stable bed is expected in that reach because of river self-adjustment and armouring of the gravel or sand-gravel mixture bed. This implies a decline of bedload supplies to the lower river in the future.

9.2.6 Formation of alternate bars in the lower Waikato River

A complete explanation of the formation of alternate bars has not been obtained yet. Existence of alternate bars in the lower Waikato River under a certain condition has been reported.

Because the ratio of river width to water depth is one of the factors influencing the formation of alternate bars, the gauging data at the Huntly Railway Bridge and at the Ngaruawahia Cableway were analysed to determine the critical width-depth ratio for the alternate bar formation. A criterion of bed level variation for alternate bar occurrence has to be given for this analysis. In this study the criterion of 2.5 m was used.

Alternate bars seem likely to occur at the Huntly Railway Bridge under the condition of the ratio of river width to water depth greater than 100. This critical width-depth ratio is much bigger than those suggested in literatures. The corresponding conditions of discharge and mean water depth in this case are, respectively, less than about 350 m^3/s and 2.30 m. However further

three dimensional surveys in the field associated with hydraulic measurements are required to confirm the conclusions.

9.2.7 Characteristics of a one-dimensional water and bedload movement system

A one-dimensional water and bedload movement model, based on conservation of sediment mass, water mass and water momentum, is presented as a quasi-linear hyperbolic system with no analytical solutions available given proper boundary and initial conditions. Water flows and bedload transportation are usually connected under an assumption of equilibrium flows, in which bedload transport is equal to the flow transport capacity determined by hydraulic parameters.

In terms of the upstream boundary condition concerning bed levels, a straightforward bed level or its derivative with respect to x have to be specified. If bedload supplies are given, as in most cases in practice, its relationship to bed level or bed level derivative with respect to x should be established. In this case non-equilibrium flows at the boundary are assumed.

The two disparate time scales of the system for bedload movement and water movement in practice result in its singular-perturbation characteristics, and approximations based on a regular perturbation cannot satisfy either arbitrary boundary conditions or initial conditions.

By using one of the decoupled techniques—a quasi steady flow approach, linearization of the water and bedload movement system produces a hyperbolic equation with a damping term or even a parabolic equation (uniform flow). Both equations are derived under an assumption of small perturbation on an initial steady condition or bed continuity. For large values of time or a large distance from the original disturbance, the parabolic equation is a good approximation of the hyperbolic equation because the damping term in the hyperbolic equation becomes dominant in this case.

Non-linear effects of bottom discontinuity are proportional to its magnitude and mainly occur at the places adjacent the discontinuity. For a small bottom discontinuity (e.g. smaller than 5% of the normal depth of flow), numerical results of the non-linear model are very similar to those of the linear model.

9.2.8 Application of a parabolic equation to the lower Waikato River

In the case of the Waikato River, variations along the river in mean bed level of those cross sections surveyed are generally over 1 m, much greater than 5% of the normal depth of flow. However a steady condition is defined as the average conditions along a length of channel. Therefore longitudinal variations of the average mean bed levels within a certain length in the Waikato River are expected to be small because some variations caused by random variables along the river, such as riffles and pools, width variations, and big bedforms, should be filtered. Therefore the linear hyperbolic equation and its long term approximation, parabolic equation, can be used.

The domain between 48.25–94.45 km upstream of the river mouth in the lower Waikato River was selected for the model applications, and a fully implicit scheme was used to produce the numerical solutions of the equations with spatial and temporal steps of 0.15 km and 1 month respectively.

Data used for calibration and validation were generally based on the results of water level profiles for the discharge of 350 m^3/s. The initial steady bed levels were assumed to follow a straight line in June 1958. Bed levels in May 1982 and December 1990 were determined by deducting the differences between the corresponding water profiles for the discharge of 350 m^3/s from the initial bed levels. Bed levels in May 1982 and December 1990 were, respectively, used for calibration and validation.

Criteria used for calibration are the minimum sum of the absolute differences between bed levels from the calculation and from the field data at the spatial grid points.

At the downstream boundary, bed levels were specified. For the upstream boundary conditions, two cases of Case A and Case B were considered: in Case A, bed levels were given; and in Case B bedload supplies were specified. Because no field measurement data were available for bedload supply at the upstream boundary, the total amounts of bedload were assumed.

Since the time period for calibration, validation and application was over 20 years, only the parabolic equation was applied. No improvements in terms of accuracy have been found in Case A by using the hyperbolic equation instead of the parabolic equation.

Parameters determined by calibration of the parabolic equation are physically acceptable for both cases concerned. Effects of grain sorting on

sediment movements were negligible in the model calibration, validation and application. However the fact of sand dominating the domain may support the assumption made.

Different scenarios at both boundaries from January 1991 to December 2040 were assumed. At the downstream boundary, they were DB1–4; at the upstream boundary, 1-UB1–3 and 2-UB1–3 (Table 9.1). The corresponding bed levels at the Huntly Power Station in December 2040 were predicted by the model. Water levels for the discharge of 350 m^3/s were also obtained given their bed levels on the assumption of constant Manning's coefficient, energy slope and width over time. By using the latest rating curve of the gauging station at Huntly (April 1993), water levels for any other discharges in December 2040 can be approximated.

Table 9.1 A summary of the different scenarios at the downstream boundary (DB) and the upstream boundary (UB) from January 1991 to December 2040.

Boundary	*Label*	*Boundary Condition*
Cases A and B Downstream Boundary	DB1	No changes in bed level, being at 1.35 m.
	DB2	Linear degradation by 0.5 m from 1.35 to 0.85 m.
	DB3	Linear degradation by 1.0 m from 1.35 to 0.35 m.
	DB4	Linear aggradation by 0.5 m from 1.35 to 1.85 m.
Case A Upstream Boundary	1-UB1	No changes in bed level, being at 7.30 m.
	1-UB2	Exponential degradation by 0.2 m from 7.30 to 7.10 m.
	1-UB3	Exponential degradation by 0.3 m from 7.30 to 7.00 m.
Case B Upstream Boundary	2-UB1	No changes in bedload supply, being at 135 000 m^3/yr.
	2-UB2	Exponential decline by 20% from 135 000 to 108 000 m^3/yr.
	2-UB3	Exponential decline by 40% from 135 000 to 81 000 m^3/yr.

9.2.9 Implication for operation of the present cooling water system at the Huntly Power Station

In the case of a critical low river discharge of 165 m^3/s, potential effects of the bed level degradation on operation of the present cooling water system at the Huntly Power Station in December 2040 were assessed. For the river water level of 6.50 m required by the present cooling water system at the outside of the intake, the model indicated that normal operation of the cooling water system would not be possible in the case of 1-UB3&DB3, 2-UB1&DB2–3, 2-UB2&DB1–3 and 2-UB3&DB1–4 for the river flow of 165 m^3/s at Huntly in December 2040.

The real bed level in the future at Huntly will be controlled mainly by the conditions at both upstream and downstream boundaries. Under the

conditions of decline of bedload supplies at the upstream boundary and sand mining operations in the lower parts of the river, the future bed level at Huntly is definitely likely to drop.

9.3 Recommendations for further research

9.3.1 Possibilities of removing bedload sediments trapped in the Karapiro Dam into the river downstream

As indicated above, most parts of bedload sediments passing through Ngaruawahia down to the lower Waikato River were from the bed materials stored in the river course of the Hamilton reach years ago. With the expected bed armouring in that reach, resulting directly in a decline of bedload supply to the lower river, investigations to balance or partially balance the decline should be considered. One of the possible procedures is to allow bedload materials trapped in the Karapiro Dam to pass through it to the river downstream. Also this will probably increase the dam operation capacity to produce more power.

9.3.2 Effects of bed level reduction at Ngaruawahia on a possible increase of bedload supply from the Waipa River

One of the potential sand sources is the bed materials stored in river courses. With the same texture of the bedload in the lower Waikato River, bed materials stored in the Waipa River courses may be important for the sand mining in the lower river, especially under the condition of bed level reduction at the junction of the Waipa and Waikato rivers at Ngaruawahia. The quantitative effects of the bed level reduction on movements of the bed materials stored in the Waipa River courses should be studied, although theoretically an increase of bedload supply is expected. Analysis of historical river changes and field bedload measurements may be a suitable start for this investigation.

9.3.3 Bedload yields from catchments

In order to make full use of the natural sand resources and at the same time to keep the river stable, determination of reliable bedload sediment yields from catchments to the lower Waikato River seems to be very important for the regional council to manage the river sand mining quantitatively. Although the bedload sediment balance has been attempted in this study to determine the catchment bedload yield, further research is required as there were many assumptions and uncertainties in this study.

9.3.4 Tidal effects on bedload movement

Sediment movements in a tidal river reach are complicated. If commercial sand mining operations in the lower Waikato River are restricted to a tidal reach, e.g. further downstream of Mercer as planned, to extract local bed materials as well as bedload sediments supplied from the river upstream, effects of the local bed level reductions on sedimentation of the upstream non-tidal river are unknown. Research and monitoring to produce a quantitative assessment of their influences are necessary for the river environmental management.

9.3.5 Possibility of armouring on the lower Waikato River

Although sand dominates the reach downstream of Ngaruawahia in terms of the mean and median grainsize, sorting and armouring may become important under the condition of continued long term bed degradation. Research on their possibilities and their effects on the bed level at Huntly in the future should be undertaken.

9.3.6 Variations in river flow on the lower Waikato River (a flow frequency analysis)

Because the river water level at Huntly is not only controlled by river configurations or bed levels, but also river flows, discharges with certain return periods are required to investigate the effects of bed level reductions on operations of the present cooling water system at the Huntly Power Station. The future mean bed level has already been addressed in this study, and a river flow frequency analysis should be undertaken in order to determine a design flow at Huntly. This analysis may include studies of flow contribution from the Karapiro Dam (deterministic?), possibilities to increase the minimum flow at the dam set by the Tongariro Works Agreement, and a random contribution from subcatchments, particularly the Waipa River.

In terms of operations of the present cooling water system, the critical river water level required for different generation capacities also needs to be given precisely.

9.3.7 Formation of alternate bars and corresponding river hydraulic characteristics around the intake and outfall of the Huntly Power Station

Occurrence of alternate bars in the river around the Huntly Power Station has been reported. Because a complete explanation for their formation is not available yet, investigation of the detailed effects of their existence on current

fields and bedload movements should be studied. This investigation is important for further research on the possibilities of making use of alternate bars for efficiently discharging cooling water into the river and reducing the amount of bedload transported into the intake forebay at the Huntly Power Station. It can be carried out through field 3-d configuration surveys associated with hydraulic measurements, laboratory experimentation, theoretical analysis and mathematical simulation.

APPENDIXES

Appendix One

River Survey Concepts

The following concepts have been used in the Waikato River cross sectional surveys as illustrated in Figure Appendix 1.1.

• *The normal water level (N.W.L.)*

N.W.L. is the elevation chosen for a particular cross section in terms of Moturiki Datum. The value of N.W.L. increases in the upstream direction and it does not relate to any return periods of water level or water discharge.

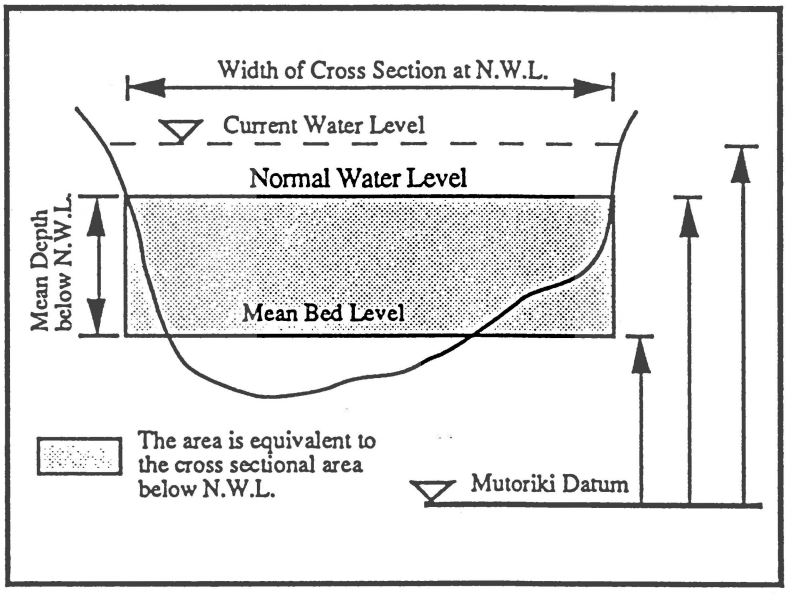


Figure Appendix 1.1 Illustration of river survey concepts.

• *Current water level*

This is the water elevation in terms of Moturiki Datum at a particular time when the cross section is being surveyed. It can be above or below the normal water level.

• *Width of cross section at N.W.L.*

This is the distance between the right and left river bank at the elevation of N.W.L.

• *Mean depth below N.W.L. or mean depth*

This is defined as the ratio of cross sectional area to width of cross section at N.W.L.

• *Mean bed level (M.B.L.)*

This is the elevation which is determined by subtraction of N.W.L. from mean depth.

Appendix Two

Texture Data of Bed Material Samples from the Lower Waikato River

The following table lists texture data of bed material taken from the lower Waikato River in 1991 and 1993 for the purpose of this study. The dried sieving method was used to determine the grain size distributions and the cumulative weight (gram) was listed in the table for different sieve sizes. "Distance" refers to the distance upstream from the river mouth (km), "L" refers to a sample from the left side of the river, "M" from the middle sector, and "R" from the right side of the river.

Table Appendix 2.1 Grain size distributions of the bed material samples from the lower Waikato River in 1991 and 1993.

		phi																				
Distance	Sample	-7	-6.5	-6	-5.5	-5	-4.5	-4	-3.5	-3	-2.5	-2	-1.5	-1	-0.5	0	0.5	1	1.5	2	3	4	12
40.56	L								0	0.8	1.2	1.8	3.1	5.3	10.7	15.5	31.8	50	65.9	77.3	85.7	86.8	
	M									0	0.35	0.92	4.43	9.36	17.48	27.32	41.2	55.05	64.53	70.12	72.71	72.8	
	R									0	0.43	0.7	2.06	4.66	9.42	17.43	28.73	41.06	51.13	56.56	56.22	56.29	
43.94	Left									0	1.4	3.6	7.4	13.46	24.75	39.5	63.56	88.13	107.13	117.73	119.88	119.9	
	Middle								0	1	6	10.3	16.5	23.34	37.68	55.95	83.59	114.57	140.97	157.21	163.49	163.6	
	Right										0	1.24	3.23	6.87	13.79	22.36	40.32	61.01	75.96	80.76	81.5	81.5	
46.61	L									0	0.46	0.91	4.43	9.54	19.99	33.26	51.76	68.76	79.41	84.42	85.14	85.17	
	M									0	4.81	4.94	8.72	13.54	20.57	30.31	47.43	67.22	82.22	90.46	93.17	93.22	
	R								0	4.95	11.64	21.14	27.98	32.2	36.96	41.01	45.28	51.71	62.5	96.55	142.58	144.34	
48.99	L									0	1.09	3.28	9.76	18.58	35.37	55.27	84.7	114.69	133	140.96	142.8	142.84	
	M								0	1.22	2.39	5.95	13.65	23.87	36.98	55.63	84.97	117.54	142.65	156.31	159.33	159.37	
	R									0	3.2	6.9	11.9	16.04	21.82	30.67	46.46	67.77	87.37	99.8	107.46	107.55	
53.25	L								0	29.62	50.86	77.22	116.84	167.62	237.07	306.24	376.62	434.43	480.16	509.17	521.51	521.94	
	M									0	6.01	8.94	14.66	21.99	31.75	44.38	63.53	82.68	97.14	107.14	110.18	110.23	
	R								0	6.11	19.1	34.04	53.7	67.33	81.94	93.58	104.37	120.81	132.63	146.82	160.38	161.34	
57.43	L										0	2.36	5.42	9.9	19.26	33.88	59.1	86.09	112.12	125.79	129.66	129.73	
	M										0	4.36	7.23	10.62	16.31	23.95	34.89	47.95	59.65	68.57	71.14	71.19	
	R								0	14.4	37.4	60	76.2	86.38	95.67	103.84	114.68	129.15	148.31	171.58	185.85	186.69	
61.33	L											0	8	9.7	13.8	20.5		53.5		88.1	92.1	92.1	
	M										0	0.64	3.13	6.6	14.09	26.66	49.36	75.56	94.28	105.25	108.12	108.22	
	R									0	4.5	10.89	25.18	34.29	52.09	72.37	100.11	125.35	141.43	149.51	152.3	152.32	
67.18	L										0	1.88	4.93	9.85	20.63	35.83	57.63	79.93	97.65	106.13	111.33	111.33	
	M								0	8.7	23	33.4	43.7	51.1	59.2	67.2	77.8	92.7	109.2	124	134.5	134.8	
	R										0	0.7	2.1	3.92	9.43	20.81	42.37	69.1	90.35	102.37	105.05	105.11	
72.34	L										0	1.19	4.16	8.49	15.05	23.49	37.56	54.66	68.99	76.43	78.36	78.36	
	M								0	3	11	22.9	39.3	53.72	76.32	103.28	147.71	191.66	241.83	263.01	273.39	273.8	
	R							0	7.9	11.8	16.5	26.5	37.9	45.99	57.02	69.32	88.41	105.88	115.64	124.51	134.55	139.79	
79.49	L										0	2.71	5.31	9.72	17.96	28.78	45.49	56.9	64.63	68.07	69.87	69.87	
	M								0	2.9	10.3	16.8	29.4	36.09	50.47	63.96	82.13	99.87	112.02	119.69	123.64	123.71	
	R								0	18.68	21.68	27.77	37.12	46.36	56.91	70.23	82.71	94.94	106.2	116.11	119.12	119.16	
85.46	L							0	2.8	8.3	16.8	24.7	40.8	56.69	75.98	90.45	104.4	115.99	126.75	135.64	143.58	143.64	
	M							0	8.9	13.4	18.6	25.1	30.7	35.42	42.3	52.09	72.15	99.77	116.96	123.41	129.85	129.86	
	R						0	11.3	34.6	70.7	112.5	151.6	175.8	187.33	200.84	213.02	223.36	250.08	276.34	292.39	295.5	295.55	
91.29	L							0	16.7	51.8	94.8	123.4	152.7	176.51	200.05	220.32	244.57	262.54	269.87	271.73	272.18	272.18	
	M							0	12.3	22.2	37.6	51.8	69.5	84.32	103.74	129.77	164.64	209.21	247.25	270.32	281.2	281.2	
	R												0	0.67	2.6	10.1	33.84	69.96	92.76	100.87	103.16	103.17	
94.45	L							0	19.6	42.5	78.6	98.4	113.2	128.08	150.13	176.46	220.48	273.34	310.27	322.84	325.4	325.4	
	M									0	2.7	5.6	10.1	16.39	25.12	36.1	60.14	88.36	113.73	129.96	137.75	137.8	
	R										0	0.17	0.29	0.97	3.56	12.81	35.87	57.71	66	68.12	68.44	68.45	
95.88	L								0	1.3	9.6	32.3	85.6	152.09	203.68	229.18	253.91	273.86	281.99	283.81	284.3	284.1	
	M						0	6.12	6.12	6.12	11.48	16.79	25.2	44.63	87.77	169.77	280.81	369.3	459.35	492.47	505.45	505.8	
	R											0	0.2	0.69	3.61	14.25	42.48	76.82	95.82	105.72	106.82	106.84	
99.35	L						0	39.79	111.73	194.31	277.37	337.93	389.2	423.74	461.97	506.69	576.36	674.24	762.2	830.63	883.96	884.9	
	M						0	6.43	26.39	47.85	81.38	127.81	176.44	223.35	281.49	351.82	467.1	594.14	685.55	724.64	734.84	734.34	
	R								0	2.3	6	14.7	25.4	37.46	50.61	75.86	101.86	127.88	143.62	148.47	149	149	
101.36	M						0	30.84	112.91	183.97	272.51	337.87	389.76	420.8	453.89	489.35	532.94	604.81	674.93	747.31	807.82	806.16	

Table Appendix 2.1 (Continued)

		phi																				
Distance	Sample	-7	-6.5	-6	-5.5	-5	-4.5	-4	-3.5	-3	-2.5	-2	-1.5	-1	-0.5	0	0.5	1	1.5	2	3	4	12
102.76	M.									0	7.25	13.28	22.35	31.5	46.23	61.33	85.24	109.27	123.14	127.41	128.37	128.59	
104.05	L.											0	0.31	1.82	3.96	12.6	37.5	67.83	84.07	90.65	92.5	92.55	
	M.									0	0.9	2.6	5.3	9.72	18.78	34.49	62.85	89.86	107.13	114.86	117.68	117.7	
	R.								0	1.65	5.77	10.04	19.34	30.17	45.83	64.85	93.72	126.69	144.57	149.56	150.17	150.25	
105.34	M.									0	3.6	9.4	17.8	28.15	44.44	66.37	94.25	125.05	142.85	147.56	150	150	
106.91	M.						0	5.3	43.4	113.5	222.3	297.2	340.7	354.97	366.69	372.05	381.21	394.66	403.95	421.46	415.35	415.45	
107.28	L.						0	37.96	138.12	254.85	450.46	572.48	670.93	729.55	779.88	815.58	846.92	874.04	888.37	907.45	936.97	938.63	
	M.					0	78.72	301.82	582.72	794.82	932.42	1027.52	1090.72	1132.22	1154.75	1207.33	1295.59	1377.25	1985.1	2223.76	2328.17	2373.04	2373.42	
	R.					0	185.4	383.6	700.4	830.2	906	946.6	974	992.6	1018.3	1058.1	1128.8	1251.7	1375	1509.1	1783.2	1791.8	
107.38	M.						0	13.79	76.32	99.77	121.1	131.77	140.7	161.54	181.41	204.44	235.13	272.28	297.9	308.86	314.59	314.9	
110.11	L.				0	191.45	259.7	350.7	548.85	751.65	931.35	1021.45	1094.65	1132.74	1177.38	1243.19	1374.54	1492.63	1590.57	1705.23	1794.91	1796.75	
	M.				0	36.68	220.75	383.12	487.52	569.24	660.87	722.72	769.47	794.84	835.5	877.38	936.5	1016.43	1101.53	1170.72	1235.5	1236.67	
	R.					0	292.4	495.7	651.9	785.1	958.8	1085.9	1194.1	1242.53	1357.78	1468.87	1574.41	1664.94	1751.85	1825.17	1877.46	1880.1	
113.52	L.					0	12.42	43.35	122.96	224.85	340.57	393.72	434.39	464.09	491.88	529.54	566.54	616.32	652.43	718.72	767.11	769.39	
	M.				0	60.72	149.26	230.9	307.28	396.72	490.2	553.05	605.12	634.89	669.08	706.25	762.76	840.33	910.76	987.17	1064.35	1065.12	
	R.					0	34.71	134.77	351.87	554.57	720.67	846.27	930.37	970.63	1006.78	1047.37	1127.11	1270.5	1410.46	1584.9	1790.51	1794.17	
117.26	M.				0	154.36	530.86	907.06	1207.86	1483.06	1716.86	1887.36	1991.46	2020.23	2038.86	2056.95	2092.51	2129.45	2195.58	2287.85	2375.71	2377.26	
	R.				0	124.58	450.58	581.53	685.31	760.25	816.36	871.46	953.71	1036.43	1137.66	1247.43	1384.74	1543.39	1730.85	1846.98	1905.76	1905.21	
121.38	L.				0	228.1	342.3	539.6	744.2	919.6	1110.3	1347.2	1455.8	1512	1567.94	1619.11	1690.6	1788.25	1889.87	2034	2185.56	2198.9	
	R.					0	35.12	48.91	111.44	134.89	154.22	166.89	175.82	182.35	192.41	204.09	221.45	248.49	285.36	354.14	392.53	399.32	
125.95	L.					0	63.83	137.41	294.14	421.67	522.34	604.19	655.44	690.58	730.54	787.02	889.89	1017.32	1136.34	1305.14	1447.49	1450.04	
	M.				0	180	621.8	848.9	1109.6	1348.5	1596.8	1779.3	1948.8	2042.63	2225.77	2358.63	2477.3	2639.29	2782.25	2958.41	3042.14	3043.3	
	R.				0	295.5	951.1	1576.9	2254.5	2545.7	2697.4	2756.7	2799.1	2822.53	2843.98	2869.76	2916.98	2948.76	2964.8	2983.91	2999.99	3000.7	
126.37	M.				0	295.5	680	813	1253.2	1553.3	1836.7	2036.7	2165.3	2231.87	2275.94	2309.05	2379.1	2535.19	2668.89	2819.3	2941.94	2943.5	
129.76	L.				0	789.3	1348.4	1715.5	1906.8		2163.5	2251.1	2319.1	2340.8	2364.63	2386.08		2471.69		2506.32	2517.84	2518.7	
	M.				0	307.09	977.85	1474.08	1804.08	1966.58	1390.38	1590.88	1693.48	1721.52	1738.96	1750.06	1764.44	1785.51	1794.67	1812.11	1823.43	1823.78	
	R.				0	162.81	504.68	935.91	1318.41	1544.81	1696.58	1771.31	1809.41	1829.65	1842.08	1853.41	1865.3	1880.96	1899.32	1949.58	2076.77	2091.41	
134.8	L.			0	450.9	950.1	1376.2	1656.5	1994	2211.2	2353.7	2419.7	2458	2477.84	2502.86	2540.14	2605.56	2682.39	2756.67	2829.83	2875.67	2877.7	
	M.				0	554.3	1006.5	1288.7	1589.4	1831.4	2068.6	2219.4	2329.7	2386.41	2448.91	2534.27	2653.56	2834.48	3008.93	3227.26	3321.82	3324.6	
	R.				0	34.16	391.56	610.66	760.46	861.96	981.96	1070.26	1148.36	1205.53	1278.09	1378.28	1556.05	1756.44	1865.85	1943.72	1989.59	1992.06	
138.06	L.					0	50.4	116.6	205	358.4	541.8	778.6	871.89	943.34	976.5			996.83		1011.82	1018.06	1018.45	
	M.			0	213.8	293.8	487.1	670	763.5	872.7	965.9	1039	1109.8	1140.72	1228.12	1309.44		1486.52	1544.06	1579.29	1587.31	1588.1	
	R.					0	46.41	85.41	112.6	145.44	171.46	184.79	193.12	197.42	201.83	210.97	262.14	452.22	693.68	954.21	1258.33	1273.92	
139.71	L.			0	103.93	443.05	998.43	1256.53	1438.83	1547.43	1597.13	1618.03	1627.53	1631.97	1635.57	1639.68	1644.26	1650.54	1663.41	1680.37	1705.49	1706.1	
	M.			0	376.3	1037.1	1350.4	1787.2	2198.8	2314.9	2512.4	2615	2678.5	2715.99	2750.2	2786.83		2942.15		3211.57	3254.83	3255.5	
141.77	L.			0	859	2207.5	2455.6	2645.2	2873.9	3042.3	3223.1	3308.4	3336.9	3341.47	3407.52	3463.45	3601.87	3648.61	3655.11	3659.27	3663.35	3663.5	
	M.			0	497.8	752.3	1117.9	1471.5	1617.9	1668.2	1713	1736.7	1766.8	1799.23	1880.06	2031.53	2229.71	2305.17	2320.72	2326.94	2329.89	2330.2	
	R.					0	153.2	260	389.4	474.7	541.5	572	583.9	586.5	588.17	591.14	598.36	643.49	774.54	1000.81	1395.82	1402.5	
148.6	L. Surf.		0	245	1540.2	2190.2	2942.1	3165.4	3284.5	3356.2	3405.1	3426.5	3445.1	3460.4	3483.3	3506.72	3548.86	3584.09	3605.89	3618.61	3621.87	3621.87	
	L. Subsurf.					0	258.9	465.5	641.1	822.6	980.6	1138.9	1225.6	1284.1	1340.24	1401.53	1481.23		1717.9		1868.14	1885.3	1886.7	
	R. Surf.	0	1140.2	1747.6	2512.6	3593.6	3756.1	3835	3855.1	3859.6	3859.6												
	R. Subsurf.					0	103.86	455.36	786.26	1118.36	1300.36	1512.66	1649.76	1764.46	1838.77	1894.51	1949.47	2094.22		2154.6	2166.39	2167.76	
	WP1									0	1.61	6.39	21.85	45.09	86.23	146.22	251.35	375.34	429.38	440.06	444.15	444.1	492.8
	WP2												0	0.7	1.84	3.87	8.63	27.63	69.9	103.22	105.86		
	WP3													0	0.07	0.26	2.89	20.5	53.22	81.18	89.63		

Appendix Three

Specific Gravity of 15 Samples from the Lower Waikato River

The specific gravity of 15 bed material samples was determined by using the water displacement method. These samples were taken from the middle channel of the Waikato River in 1991, except one from the subsurface bed at the left bank. The results are given in the following table.

Table Appendix 3.1 The specific gravity of 15 bed material samples from the middle channel of the Waikato River in 1991.

Sample Position (XS No.)	Dis. from R. Mouth (km)	Sediment Weight (g)	Water Weight (g)	Reading of Total Volume (1000 cm^3)	Total Volume adjusted (cm^3)	Water Volume (cm^3)	Specific Gravity
57	40.6	475.8	266.7	0.522	515	266.85	1.92
61A	44.0	546.5	261.5	0.505	498	261.64	2.31
68	49.0	457.3	277.8	0.484	477	277.95	2.30
73	53.2	420.7	268.9	0.467	460	269.05	2.20
111C	67.2	504.5	294.5	0.509	502	294.66	2.43
122	79.5	424.9	244.8	0.429	422	244.93	2.40
126	85.5	769.8	269.4	0.587	580	269.55	2.48
133	95.9	470.6	267.1	0.458	451	267.25	2.56
135	99.2	573.1	259.8	0.515	508	259.94	2.31
138	104.0	539.1	270.9	0.485	478	271.05	2.60
144	110.1	532.9	300.3	0.521	514	300.47	2.50
152	117.3	602.5	273.0	0.510	503	273.15	2.62
159A	126.0	657.7	272.3	0.529	522	272.45	2.64
164	129.8	545.1	267.8	0.493	486	267.95	2.50
near 181	148.8	470.5	288.4	0.480	473	288.56	2.55

Note:

1. Sample positions are given by a cross section (XS) number and the distance upstream of the river mouth. The sample at 148.8 km was taken at the subsurface bed from the left bank.

2. Water temperature was 12.5°C (water density = 0.99945 t/m^3) when it was used for determination of sediment specific gravity.

3. Water Volume = Water Weight/Water Density.

4. The scale on the glass, which was used for measurement of volume, was adjusted by subtracting 7 cm^3 based on the volume test (Table Appendix 3.2). The average volume difference between the readings and water volumes is 7 cm^3.

5. Specific Gravity is determined by the following equation:

$$\text{Specity Gravity} = \frac{\dfrac{\text{Sediment Weight}}{\text{Sediment Volume}}}{\text{Water Density}}$$

where Sediment Volume = Total Volume - Water Volume.

Table Appendix 3.2 Tests of the glass volume reading scales.

Reading of Volume from the Glass (cm^3)	Water Weight (g)	Water Volume (cm^3)	Volume Difference (cm^3)
100	92.3	92.35	6.87
600	592.8	593.13	7.64

Appendix Four

Comments on Gauging Records of the Lower Waikato River

A total of 173 gauging records in 1960–1992 are available at the Hamilton gauging station (site No. 43466), which is located at the Hamilton Traffic Bridge (cross section 151). 20 original gauging cards have been checked and corrected. One gauging record—card No. 954 of 1-Oct-1970—has been omitted because there is no information about the datum used.

The gauging station at Ngaruawahia is located at the Ngaruawahia Cableway (site No. 43402, cross section 132), just downstream of the Waipa River conjunction. The data available are from 1958 to 1992 with a total of 171 records available and few gauging undertaken in the period between 1958–1966. 13 original gauging cards have been checked and corrected, and two gauging records—No. 2172 of 8-Oct-1976 and No. 10171 of 17-Feb-1987—have been omitted because they are prone to error.

The site of the gauging station at Huntly changed. Before and including 1986 the gauging station was located at the Huntly Rail Bridge (site No. 43419) with the first gauging taken in 1958 and a total of 121 records available. Water levels of the gauging—No. 8884, 8893, 8897 and 8906—in 1986 were measured at the Huntly Power Station and they have been adjusted to those at the Huntly Rail Bridge using the relationship between water levels at the Huntly Rail Bridge and at the Huntly Power Station (Appendix Five).

All of the water levels between 1958 and 1968 have been checked and recalculated by using both the water level records at the Huntly Pumping Station (Waikato Valley Authority plan 441/HYD/2) which is located upstream of the Huntly Rail Bridge; and the following equation (Waikato Valley Authority plan 1334/HYD/1):

$$WL_{\text{-Rail B.}} = \frac{1.031 WL_{\text{-Pump. S.}} - 4.495}{3.28} - 28.68 \qquad \text{(App. 4.1)}$$

where $WL_{\text{-Pump. S.}}$ is the water level at the Huntly Pumping Station (feet, Waikato Valley Authority datum), and $WL_{\text{-Rail B.}}$ is the water level at the Huntly Rail Bridge (m, Moturiki datum).

Table Appendix 4.1 Gauging data for analysis at the Huntly Rail Bridge (data from the Waikato Regional council). "Dis" refers to discharge, "MaxD" refers to maximum water depth.

No	Date	Stage (m)	Dis (m^3/s)	Area (m^2)	Width (m)	MaxD (m)
8	26-Feb-58	11.56	1611.0	1208.0	252.4	7.65
13	28-Feb-58	11.24	1385.4	1190.8	252.4	7.07
15	1-Mar-58	10.90	1162.1	1104.1	249.9	6.46
28	24-Jun-58	8.17	304.7	400.3	236.2	4.14
368	27-Aug-65	9.81	770.6	908.7	249.9	5.73
483	7-Feb-67	10.00	844.8	1030.2	250.5	5.55
489	9-Feb-67	9.39	681.1	860.8	245.4	5.70
492	13-Feb-67	8.71	423.1	597.4	234.4	4.63
497	1-Jun-67	8.33	376.8	534.4	237.7	4.48
501	23-Jun-67	8.77	468.2	639.3	238.6	5.18
506	14-Aug-67	9.18	595.4	784.9	240.8	5.43
592	22-Nov-67	9.62	744.6	980.4	242.3	6.04
614	6-Mar-68	7.80	235.5	474.7	232.2	3.48
663	2-Jul-68	10.22	876.8	1118.3	243.9	6.64
675	1-Aug-68	8.52	391.2	649.4	237.7	3.57
774	10-Nov-69	7.56	216.8	408.6	230.4	2.71
886	26-Feb-70	7.40	167.6	356.6	228.9	2.99
901	6-May-70	7.47	175.0	387.3	231.0	2.84
906	5-Jun-70	8.97	474.8	708.3	241.1	3.99
910	9-Jun-70	9.16	522.9	757.5	241.1	4.48
915	21-Jul-70	8.63	423.0	607.1	238.6	3.81
919	15-Aug-70	10.40	935.9	1052.0	246.9	5.61
926	17-Aug-70	9.84	752.5	911.6	243.8	4.88
929	19-Aug-70	9.21	559.2	769.6	240.8	4.12
937	28-Sep-70	9.57	669.9	839.3	243.8	4.42
959	2-Oct-70	10.58	976.7	1093.2	249.9	5.49
964	7-Oct-70	9.99	802.9	943.6	246.9	4.94
971	12-Oct-70	9.25	589.2	751.3	242.3	4.12
978	16-Oct-70	8.95	510.8	697.4	239.6	3.87
985	27-Oct-70	8.47	382.4	554.9	241.1	3.11
1005	15-Apr-71	7.46	175.4	319.9	230.1	2.26
1011	30-Jun-71	8.78	486.5	651.8	242.3	3.54
1012	5-Jul-71	9.32	594.8	785.0	241.4	4.05
1014	8-Jul-71	8.99	502.6	701.8	240.5	3.78
1017	13-Jul-71	8.48	400.8	569.7	232.2	3.29
1020	19-Jul-71	8.20	343.7	494.5	236.5	2.87
1091	24-May-72	8.92	479.1	678.7	241.1	4.33
1101	19-Jul-72	9.90	780.4	947.1	243.8	5.27
1129	30-Jan-73	7.63	228.0	376.2	231.6	2.56
1162	21-Feb-73	7.36	170.8	311.1	227.0	2.70
1255	25-Jul-73	8.60	454.9	650.6	239.0	3.56
1272	3-Sep-73	8.32	377.4	520.9	235.0	3.75
1279	8-Oct-73	7.83	256.6	382.1	226.0	3.13
1284	9-Nov-73	7.88	296.1	446.4	233.0	3.32
1289	16-Nov-73	8.67	473.8	644.7	241.0	4.10
1292	7-Dec-73	8.04	324.6	461.3	234.0	3.38
1305	29-Jan-74	7.99	327.7	486.9	233.0	3.54
1448	2-May-74	7.40	222.1	394.4	225.0	3.20
1451	15-May-74	7.40	214.6	392.1	225.0	3.62
1460	17-Jun-74	9.27	621.1	812.0	241.0	5.30
1467	1-Aug-74	9.09	607.3	803.7	241.0	4.40
1482	22-Nov-74	8.06	342.6	484.0	234.0	3.38
1484	28-Nov-74	8.02	348.1	471.1	233.5	2.48
1491	12-Dec-74	8.47	436.8	611.9	238.0	3.86
1613	6-Mar-75	7.35	214.4	378.1	225.0	2.78
1655	27-May-75	8.56	442.3	587.6	236.0	3.52
1675	17-Jun-75	9.83	780.2	906.1	245.0	4.95
1743	6-Nov-75	8.04	365.5	529.8	232.7	3.00
1748	11-Nov-75	8.02	333.9	523.9	226.0	2.94
1883	16-Jan-76	9.21	587.4	873.0	242.0	4.88
2091	2-Jun-76	8.10	370.1	493.6	227.5	4.20
2109	8-Jul-76	10.47	1015.3	1060.6	241.3	6.14
2128	27-Jul-76	8.69	497.8	669.4	237.5	3.92
2139	12-Aug-76	9.90	790.6	952.0	247.0	5.12
2145	23-Aug-76	8.88	530.7	716.8	237.0	4.17
2159	16-Sep-76	9.78	798.6	935.7	247.0	4.74
2183	8-Nov-76	8.47	466.6	616.4	232.0	3.12
2191	6-Dec-76	7.83	295.8	458.8	227.0	2.66
2209	18-Jan-77	7.61	277.2	443.6	225.5	2.48
2377	11-Mar-77	7.36	211.7	408.7	225.5	2.36
2431	1-Apr-77	7.47	230.5	443.0	225.0	2.74
2504	10-Jun-77	8.39	427.1	631.6	228.0	3.84
2513	4-Jul-77	9.73	800.4	927.7	242.0	5.08
2537	8-Aug-77	8.75	525.9	695.2	238.0	3.98
2549	14-Sep-77	8.41	456.5	611.4	230.0	3.40
2559	31-Oct-77	7.78	307.2	474.9	227.0	3.10
2585	2-Dec-77	7.66	270.4	443.0	228.0	3.06
2599	20-Dec-77	7.90	342.4	479.0	227.0	3.26
2681	24-Jan-78	7.41	221.6	350.9	225.0	2.82
2854	10-Mar-78	7.47	254.6	377.1	226.0	2.78
2979	27-Apr-78	7.16	188.8	315.0	250.0	2.55
2983	12-Jun-78	7.34	231.9	343.5	250.0	2.63
2986	21-Jul-78	8.91	556.1	706.0	240.0	4.34
2994	25-Aug-78	7.83	311.0	426.1	230.0	2.77
2997	20-Sep-78	8.45	446.1	591.3	230.0	3.70
3017	6-Dec-78	7.70	289.4	429.3	227.0	2.55
3211	5-Mar-79	7.36	220.0	356.4	225.0	2.50
3227	18-Apr-79	7.90	358.0	475.4	227.0	2.96
3232	18-May-79	9.62	789.4	923.6	244.0	4.98
3239	25-Jun-79	7.87	346.1	470.7	226.5	2.97
3268	9-Nov-79	8.32	415.1	551.0	230.0	3.36
3276	17-Dec-79	8.21	389.8	470.0	229.0	3.02
3363	20-Feb-80	7.98	395.5	512.8	232.0	3.60
3365	3-Mar-80	8.11	401.4	541.2	230.0	3.72
3398	12-May-80	7.58	299.0	431.5	210.0	2.89
3413	6-Aug-80	8.89	648.5	733.9	225.0	4.55
3421	2-Oct-80	8.44	512.8	599.8	220.0	3.70
3424	6-Nov-80	8.15	389.8	546.2	215.5	3.92
3441	7-Jan-81	7.87	360.9	486.1	213.0	3.72
3491	3-Mar-81	7.41	217.4	386.4	211.0	2.98
3682	6-Aug-81	8.81	560.4	664.1	223.0	3.80
3701	3-Sep-81	8.73	560.6	661.4	224.0	4.00
3736	8-Oct-81	8.39	506.5	584.7	220.0	3.46
3774	15-Dec-81	8.00	382.3	546.1	215.0	3.32
4044	21-May-82	7.91	315.8	431.8	215.0	3.75
4053	11-Aug-82	7.98	356.5	472.0	216.0	3.71
4080	13-Oct-82	7.61	264.4	385.2	213.0	2.77
5209	15-Dec-82	7.45	232.5	342.4	204.0	2.45
5788	16-Mar-83	7.17	188.7	302.4	213.5	2.27
5866	21-Apr-83	7.31	215.8	362.1	211.0	2.88
5963	7-Oct-83	9.28	731.4	769.4	225.5	4.30
5992	15-Nov-83	8.68	507.5	602.4	224.0	3.50
6020	26-Jan-84	7.69	267.6	427.4	213.9	2.88
6026	5-Apr-84	7.82	294.8	500.8	214.5	3.41
7032	6-Jul-84	8.48	502.0	686.6	219.0	4.73
8884	27-Jul-86	10.39	1068.6	1045.6	242.0	5.60
8893	28-Jul-86	10.46	1171.5	1053.5	248.0	5.65
8897	30-Jul-86	10.09	978.7	947.8	251.0	5.16
8906	1-Aug-86	9.48	733.1	824.9	225.5	4.46

It should be pointed out that some water levels of the data in 1980–1982 were measured at Boatie, whose location is close to the opposite side of the Huntly Power Station, and they have been adjusted to those at the Huntly Rail Bridge.

Comments on the gauging record (card No. 28) of 24-Jun-1958 indicated that it was measured at the Huntly Pumping Station and the gauging has been recalculated using Eq. (App. 4.1). Two gauging records—cards 3002 of 12-Oct-1978 and 4036 of 12-Apr-1982—have been omitted because of, respectively, suspect notes written on the original card by staff of the Waikato Regional Council and no information of the gauging position. The water level of the gauging on 12-May-1980 (card 3398) has been changed from 7.272 m to 7.578 m which was the water level at 12:00, 12-May-1980 from the TIDEDA database at the Waikato Regional Council. The gauging data, taken at the Huntly Rail Bridge, used for an analysis of water and bed level variations have been listed in Table Appendix 4.1.

The new gauging station at Huntly was set up on the Tainui Bridge (site No. 1143448, about 634 m upstream from the Huntly Rail Bridge) in 1984. It should be noted that water levels of the new gauging were measured at the Huntly Power Station (site No. 1543495, downstream of the Huntly Rail Bridge). For the new gauging station, a total of 104 records from 1984 to 1992 are available and one gauging record—card 10529 of 27-Mar-1987—has been deleted from the analysis because the gauging was undertaken near the power station by a boat.

The site of the gauging station at the old Rangiriri Bridge was shifted to the new Rangiriri Bridge (site No. 43420) in 1969 when it was built by the old bridge downstream. The first new gauging was done on 10-Nov-1969. There are a total of 219 gauging records available in 1958–1992 with 15 records taken from the old bridge in 1958 and 1967–1968. 21 original gauging records have been checked and corrected, and records—card Nos. 7804 of 24-May-1985 and 8946 of 19-Aug-1986—have been omitted because of suspect comments written on the cards by staff of the Waikato Regional Council. Two staff gauges were installed at Rangiriri. One has been located by the bridges probably since 1958 or before and the other beside the Rangiriri water level recorder upstream about 100 m since 1965. All the new gauging data were related to the staff gauge at the water level recorder.

The site of the gauging station at Mercer changed in 1980. Discharges in 1960–1980 were taken at the Mercer Wharf (the old Mercer bridge, site No.

43463), and since 1980 they have been measured on the Mercer Bridge (site No. 1043446). Some discharge in 1980–1982 were also measured at the Mercer Wharf. The distance between the Mercer Wharf and the Mercer Bridge is about 100 m. Except for the period of 1960–1962, water levels of all the gauging data are taken at the Mercer staff gauge. There are a total of 141 gauging records at the Mercer Wharf in 1960–1982 and 97 records at the Mercer Bridge in 1980–1991. Eight original cards have been checked and corrected, and gauging card 216 of 29-Nov-1962 is omitted because of the suspect notes written on the card.

Appendix Five

Relationship of Water Levels between the Huntly Rail Bridge and the Huntly Power Station

The relationship between water levels at the Huntly Rail Bridge, $WL_{\text{-Rail B.}}$ (site No. 43419) and at the Huntly Power Station, $WL_{\text{-Power S.}}$ (site No. 1543495) has been studied in order to analyse the gauging data at both sites together. A total of 17 historical records available in the following table were selected from the original gauging cards, field books (water profile measurement) and TIDEDA database at Huntly Power Station during 1983 and 1990.

Table Appendix 5.1 Relationship between water levels at the Huntly Rail Bridge and at the Huntly Power Station in 1983–1990 (data from the Waikato Regional Council).

Date	$WL_{\text{-Rail B.}}$	$WL_{\text{-Power S.}}$	Difference	Note
16-Mar-1983	7.170	6.88	0.290	gauging card
21-Apr-1983	7.305	7.02	0.285	gauging card
7-Oct-1983	9.275	8.985	0.290	gauging card
15-Nov-1983	8.684	8.387	0.297	gauging card
26-Jan-1984	7.692	7.39	0.302	gauging card
5-Apr-1984	7.815	7.553	0.262	gauging card
6-Jul-1984	8.476	8.22	0.256	gauging card
14-Aug-1984	7.935	7.667	0.268	gauging card
23-Aug-1984	8.564	8.305	0.259	gauging card
3-Oct-1984	8.203	7.968	0.235	gauging card
10-Apr-1985	7.710	7.450	0.260	field book
11-Apr-1985	7.555	7.318	0.237	field book
27-Apr-1987	7.970	7.674	0.296	field book & database
24-May-1988	8.050	7.776	0.274	field book & database
19-Jul-1988	8.370	8.073	0.297	field book & database
17-Aug-1989	7.970	7.704	0.266	field book & database
17-Dec-1990	8.003	7.761	0.242	field book & database

In Table Appendix 5.1, $WL_{\text{-Rail B.}}$ and $WL_{\text{-Power S.}}$ are, respectively, the water levels at Huntly Rail Bridge (m) and at Huntly Power Station (m). Their linear relationship is demonstrated by

$$WL_{\text{-Rail B.}} = 0.25929 + 1.0016 WL_{\text{-Power S.}} \qquad \text{(App. 5.1)}$$

with r^2=0.998 (Figure Appendix 5.1).

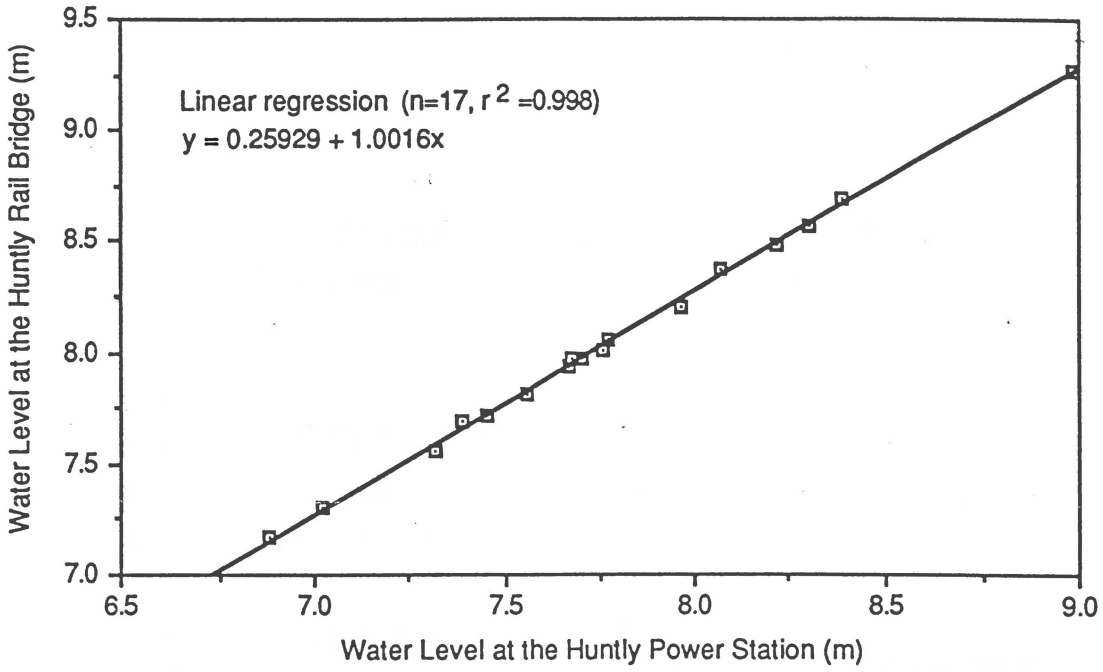


Figure Appendix 5.1 Relationship between the water levels at the Huntly Rail Bridge and at the Huntly Power Station (Moturiki datum, data from the Waikato Regional Council in the period 1983–1990).

Because the survey undertaken on 10 and 11 April 1985 indicated that zero of the staff gauge at the Huntly Rail Bridge was 0.037 m higher than the Moturiki datum, *"All S.G. readings at Huntly Rail since the instalment by Huntly Borough are suspected."* (the Waikato Regional Council field book, 18-4-1985) Another survey taken on 6 and 27 May 1991 also indicated that zero of that staff gauge was 33–42 mm higher with a mean 37.5 mm than the Moturiki datum (the Waikato Regional Council field book).

It has been found from the gauging record—card No. 4053—that the latest staff gauge at the Huntly Rail Bridge was reinstalled on 11-8-1982. So all of the staff gauge readings at least since that time should be adjusted by plus 0.038 m and Eq. (App. 5.1) based on the data taken in 1983–1990 should be corrected to:

$$WL_{\text{-Rail B.}} = 0.25929 + 1.0016 WL_{\text{-Power S.}} + 0.038 \qquad \text{(App. 5.2)}$$

which is the same as Eq. (4.19) in Chapter Four.

Prior to 11-8-1982 it is not known whether the readings from that staff gauging need to be corrected.

Appendix Six

Relationship of Discharges between the Huntly Rail Bridge and the Tainui Bridge

In order to extend the time series of adjusted water levels at the Huntly Rail Bridge using gauging data at the Tainui Bridge, the relationship between discharge at the Huntly Rail Bridge and at the Tainui Bridge has been studied.

The distance between these two bridges is about 634 m and no tributary flows enter into that reach and no waters are drawn from it either.

If monthly discharges at the Ngaruawahia Cableway $Q_{N.C.}$, at the Tainui Bridge $Q_{T.B.}$ and at the Huntly Rail Bridge $Q_{R.B.}$ are assumed to be stationary random variables, historical flow data for 1968–1980 at the Huntly Rail Bridge, for 1983–1992 at the Tainui Bridge and for 1968–1992 at the Ngaruawahia Cableway can be used to investigate the relationship between $Q_{R.B.}$ and $Q_{T.B.}$. In the case of the monthly discharges within 220–420 m^3/s at the Ngaruawahia Cableway, results of the mean ratios of $Q_{N.C.}$ to $Q_{R.B.}$ and $Q_{N.C.}$ to $Q_{T.B.}$ are, respectively, given in the following table.

Table Appendix 6.1 Relationship between monthly discharges at the Ngaruawahia Cableway $Q_{N.C.}$ and at the Tainui Bridge $Q_{T.B.}$ as well as at the Huntly Rail Bridge $Q_{R.B.}$ (data from TIDEDA at the Waikato Regional Council in 1968–1992).

Station	Ratio	No.	Period
Tainui Bridge	$Q_{N.C.} / Q_{T.B.} = 0.889$	83	1968–1980
Huntly Rail Bridge	$Q_{N.C.} / Q_{R.B.} = 0.900$	49	1983–1992

From Table Appendix 6.1, the relationship between discharges at the Tainui Bridge and at the Huntly Rail Bridge is determined by $Q_{R.B.} / Q_{T.B.} = 0.988$. Thus discharges at the Tainui Bridge are adjusted by multiplying 0.988 when they are used together with the data at the Huntly Rail Bridge for an analysis of adjusted water level variations in Chapter Four.

Appendix Seven

Data of the Water Profiles at the Discharge of 350 m³/s between Ngaruawahia and Mercer

The following table lists all original water level profile measurements held at Waikato Regional Council and the profiles adjusted for the discharge of 350 m³/s between 1958–1990. In the table, "Xs No" represents the cross section number, "dis" is the distance from the river mouth (km), "Mea" and "Adj" represent, respectively, measured and adjusted water levels in meters relative to Moturiki Datum.

Table Appendix 7.1 Water level profiles for the discharge of 350 m³/s in the reach between Mercer and Ngaruawahia in the Waikato River (data from the Waikato Regional Council in 1958–1990).

		Jun-58		Jul-63		Jan-64		May-67		Sep-73		Oct-73		Nov-73		May-74		Oct-74		Nov-74	
Xs No	dis (km)	Mea	Adj	Mea	Adj	Mea	Adj	Mea	Adj	Mea	Adj	Mea	Adj	Mea	Adj	Mea	Adj	Mea	Adj	Mea	Adj
rec	40.60	3.418	3.460	3.643	3.190	3.491	3.130	3.326	2.970	3.128	2.530	2.729	2.530	2.499	2.530	3.024	2.510	3.323	2.490	2.550	2.490
58	41.75	3.512	3.560	3.710	3.279	3.549	3.203			3.143	2.573	2.753	2.582	2.521	2.569						
58A	42.09											2.820	2.657								
59	42.38							3.399	3.065	3.221	2.666	2.808	2.652	2.598	2.655	3.180	2.711	3.454	2.665	2.644	2.608
60	42.61	3.619	3.671	3.774	3.359	3.598	3.262														
60A	43.17									3.287	2.750	2.893	2.756	2.659	2.727			3.470	2.700	2.705	2.680
61A	43.98									3.406	2.889	2.997	2.879	2.869	2.950	3.366	2.937	3.604	2.854	2.990	2.976
61	44.68	3.814	3.877	3.957	3.580	3.896	3.588	3.723	3.415	3.524	3.024	3.168	3.067	3.015	3.106	3.424	3.012	3.720	2.987		
62	45.45	3.921	3.987	4.040	3.677	3.954	3.656	3.811	3.513	3.659	3.177	3.317	3.235	3.174	3.276			3.832	3.119	3.314	3.320
62A	46.19													3.317	3.431	3.842	3.467	3.957	3.262	3.439	3.455
64	46.61	4.082	4.155	4.241	3.900	4.079	3.795					3.518	3.464	3.430	3.549	3.866	3.502	4.037	3.352	3.530	3.552
64A	47.00									3.896	3.452	3.595	3.549	3.515	3.641	4.034	3.680	4.089	3.413	3.586	3.614
65	47.56	4.201	4.278	4.342	4.018	4.268	3.997	4.210	3.937	4.024	3.593	3.723	3.691	3.637	3.771	4.049	3.709	4.210	3.549		
66	47.87									4.070	3.647	3.771	3.747	3.677	3.815			4.271	3.618		
66A	48.31									4.110	3.697	3.805	3.791			4.171	3.850	4.317	3.674	3.839	3.885
67	48.77	4.296	4.379	4.406	4.105	4.357	4.101			4.174	3.772	3.851	3.848	3.765	3.917			4.372	3.741	3.921	3.973
69	49.42									4.287	3.900	3.994	4.007	3.899	4.060	4.323	4.030	4.457	3.842	3.990	4.051
69A	49.63	4.418	4.505							4.332	3.951										
70	49.81			4.564	4.283	4.515	4.272	4.460	4.214	4.369	3.992	4.046	4.068	3.973	4.139	4.396	4.113	4.531	3.925		
71	50.53	4.534	4.625			4.579	4.346			4.460	4.101	4.127	4.166	4.031	4.208	4.468	4.203	4.607	4.019	4.116	4.192
71A	51.00									4.540	4.191	4.195	4.246	4.113	4.297						
72	51.34	4.610	4.706	4.744	4.491	4.649	4.426	4.643	4.415	4.576	4.236	4.253	4.312	4.174	4.363			4.698	4.130	4.221	4.308
72A	51.78									4.640	4.311	4.305	4.374	4.214	4.409	4.643	4.409	4.756	4.199	4.296	4.389
73A	52.43									4.720	4.405	4.390	4.475	4.308	4.513	4.735	4.517	4.832	4.291	4.350	4.452
73	53.25			4.976	4.758					4.832	4.538	4.491	4.595	4.406	4.623	4.863	4.666	4.970	4.449	4.496	4.609
OR(74)	53.56	4.869	4.976			4.960	4.766	4.957	4.756	4.887	4.600	4.537	4.649								
75	53.91									4.896	4.618			4.460	4.688	4.927	4.746	5.043	4.538	4.569	4.691
76	54.18									4.933	4.661	4.576	4.703								
77	54.44									4.967	4.701	4.610	4.743	4.537	4.772	5.000	4.833	5.113	4.621	4.627	4.756
78	54.73	5.000	5.113	5.116	4.926	5.079	4.900			5.003	4.744	4.640	4.780	4.564	4.803						
79	55.09									5.028	4.777	4.665	4.813	4.570	4.815	5.040	4.888	5.174	4.698	4.640	4.778
80	55.35									5.067	4.823	4.686	4.841	4.589	4.837						
81	55.57	5.064	5.181			5.143	4.974			5.079	4.840	4.707	4.868	4.604	4.855	5.092	4.952				
82	55.86			5.238	5.069					5.119	4.887	4.738	4.905	4.649	4.905	4.878	4.746	5.278	4.821	4.759	4.908
84	56.19									5.159	4.935										
85	56.31	5.162	5.283	5.290	5.129	5.287	5.127	5.293	5.124	5.189	4.968	4.820	4.998	4.753	5.016	5.235	5.114	5.345	4.899	4.848	5.003
86	56.50									5.207	4.991	4.845	5.027								
87	56.79			5.320	5.168					5.226	5.016	4.866	5.055	4.781	5.050	5.244	5.135	5.387	4.954	4.884	5.046
88	56.99																				
89	57.38	5.287	5.413	5.436	5.295	5.421	5.275	5.399	5.243	5.314	5.119	4.927	5.130	4.842	5.120						
90	57.63					5.433	5.290									5.366	5.278	5.464	5.051	4.974	5.147
92	58.17	5.375	5.505	5.473	5.347	5.512	5.377	5.442	5.295	5.375	5.199	5.000	5.223	4.906	5.195			5.546	5.146		
93	58.49									5.418	5.249	5.037	5.267	4.954	5.249	5.436	5.370	5.598	5.206	5.069	5.254

Table Appendix 7.1 (Continued)

		Jun-58		Jul-63		Jan-64		May-67		Sep-73		Oct-73		Nov-73		May-74		Oct-74		Nov-74	
Xs No	dis (km)	Mea	Adj	Mea	Adj	Mea	Adj	Mea	Adj	Mea	Adj	Mea	Adj	Mea	Adj	Mea	Adj	Mea	Adj	Mea	Adj
95	58.87	5.442	5.576	5.552	5.439	5.579	5.453	5.564	5.426	5.448	5.289										
96	58.90									5.482	5.323	5.095	5.335	5.018	5.319	5.546	5.490	5.671	5.289	5.152	5.342
97	59.37																				
98	59.60			5.622	5.523					5.549	5.407	5.153	5.409	5.073	5.384					5.195	5.395
99	59.85									5.579	5.443										
100	60.11	5.610	5.750	5.723	5.633	5.671	5.560	5.701	5.577	5.607	5.477	5.210	5.479	5.125	5.444	5.674	5.648	5.842	5.490	5.241	5.448
101	60.33									5.634	5.510	5.244	5.518								
102	60.78			5.756	5.679					5.692	5.578	5.290	5.575	5.220	5.548						
103	60.98									5.701	5.592										
104	61.23									5.723	5.620	5.323	5.619	5.278	5.612						
105	61.33	5.765	5.911	5.863	5.796	5.884	5.789	5.863	5.754												
106	61.50									5.759	5.663	5.393	5.696	5.354	5.693	5.872	5.881	5.970	5.652	5.496	5.722
107	62.50	5.909	6.061	5.997	5.952	6.049	5.969	5.991	5.896	5.884	5.812	5.521	5.848	5.473	5.826			6.070	5.778	5.594	5.834
108	63.05	5.979	6.133	6.055	6.020	6.101	6.028														
108A	63.54															6.110	6.170	6.207	5.940	5.742	5.996
109A	64.13																				
109	64.52	6.177	6.339	6.232	6.224	6.290	6.235	6.229	6.157	6.134	6.110	5.750	6.125	5.741	6.124	6.262	6.347	6.351	6.108	5.870	6.137
110A	64.95			6.274	6.275			6.314	6.248	6.183	6.169			5.778	6.167	6.299	6.394	6.409	6.176	5.905	6.178
R(110)	65.45									6.262	6.260	5.863	6.260	5.863	6.260	6.412	6.520	6.470	6.250	5.970	6.250
110	65.45	6.290	6.456	6.345	6.354																
110C	65.68																				
111	66.48	6.445	6.617	6.506	6.535	6.518	6.489	6.491	6.443	6.445	6.436	6.049	6.441	6.052	6.442	6.595	6.428	6.619	6.398	6.125	6.403
111C	67.18									6.531	6.516							6.695	6.473		
111D	67.48													6.183	6.565						
111E	67.64									6.595	6.577	6.250	6.636								
112	68.01			6.646	6.704	6.744	6.734	6.701	6.672	6.628	6.608	6.281	6.665	6.290	6.668	6.787	6.612	6.799	6.576	6.364	6.638
113	69.10	6.781	6.966	6.778	6.855	6.866	6.870	6.848	6.831	6.796	6.768	6.436	6.815	6.445	6.816	6.945	6.765	6.954	6.730	6.508	6.780
114	70.63	6.948	7.141	6.979	7.085	7.052	7.076			6.988	6.950	6.616	6.988	6.628	6.988	7.128	6.941	7.177	6.951	6.716	6.984
115	72.34	7.171	7.372	7.192	7.330	7.281	7.326	7.278	7.299	7.235	7.185	6.814	7.178	6.839	7.186	7.439	7.243	7.418	7.190	6.934	7.198
116	73.16	7.250	7.456	7.256	7.409	7.360	7.416			7.320	7.264	6.915	7.274	6.939	7.281	7.503	7.303	7.503	7.274	7.032	7.294
117	74.48	7.393	7.605	7.409	7.586	7.503	7.576	7.509	7.556	7.479	7.413	7.043	7.396	7.073	7.406	7.665	7.458	7.668	7.437	7.175	7.434
118	74.95	7.406	7.620	7.430	7.616	7.531	7.610			7.509	7.441							7.701	7.470	7.197	7.455
119	75.34	7.482	7.698	7.491	7.684	7.579	7.664	7.601	7.658	7.576	7.505	7.125	7.474	7.146	7.473	7.802	7.591	7.768	7.537	7.240	7.498
120	76.32	7.573	7.795	7.558	7.770	7.662	7.759			7.671	7.593	7.195	7.539	7.235	7.554	7.890	7.675	7.869	7.636		
HulyN	76.90																			7.395	7.649
121	77.41	7.698	7.925	7.662	7.894	7.768	7.879	7.790	7.872			7.329	7.668	7.366	7.678					7.448	7.701
122	79.49	8.037	8.274	7.957	8.228	8.104	8.241			8.092	7.991	7.662	7.990	7.683	7.980	8.241	8.010	8.204	7.968	7.696	7.944
123	80.85	8.146	8.390	8.104	8.400	8.235	8.390	8.247	8.370	8.290	8.180	7.848	8.170	7.883	8.170	8.378	8.140	8.378	8.140	7.895	8.140
124	83.48																				
126	85.46																				
MangC	87.55																				
132	94.45	9.896	10.290	9.948	10.270	10.067	10.270	9.942	10.270	10.207	10.230	9.835	10.230	9.436	10.230	10.186	10.210	10.189	10.210	9.960	10.210

Table Appendix 7.1 (Continued)

		Nov-75		Nov-76		Oct-78		May-82		May-85		Apr-87		May-88		Jul-88		Aug-89		Dec-90	
Xs No	dis (km)	Mea	Adj	Mea	Adj	Mea	Adj	Mea	Adj	Mea	Adj	Mea	Adj	Mea	Adj	Mea	Adj	Mea	Adj	Mea	Adj
mc	40.60	2.940	2.430	2.800	2.390	2.500	2.350	2.550	2.270	2.270	2.250	2.493	2.080	2.375	2.020	2.750	2.020	2.350	1.940	2.310	1.880
58	41.75	2.950	2.466			2.561	2.423	2.560	2.299	2.280	2.271	2.584	2.191	2.443	2.103	2.840	2.131	2.440	2.051	2.380	1.969
58A	42.09																				
59	42.38	3.040	2.571	2.890	2.505	2.584	2.453	2.650	2.399			2.600	2.219	2.490	2.158						
60	42.61																				
60A	43.17					2.747	2.625					2.670	2.303			2.920	2.238	2.480	2.118	2.460	2.071
61A	43.98	3.260	2.828	3.130	2.768	2.861	2.748	2.920	2.696	2.650	2.661	2.697	2.344	2.596	2.284	2.960	2.293	2.510	2.163	2.600	2.224
61	44.68	3.380	2.964	3.310	2.957	3.033	2.927	3.010	2.797			2.768	2.428	2.659	2.356	3.000	2.346	2.680	2.346	2.720	2.356
62	45.45					3.163	3.066	3.150	2.950					2.821	2.527	3.140	2.500	2.800	2.480	2.890	2.538
62A	46.19	3.660	3.278	3.620	3.289	3.337	3.248	3.290	3.102	3.090	3.122	3.084	2.770	3.014	2.730	3.270	2.643	3.010	2.703	3.080	2.740
64	46.61	3.740	3.368																	3.260	2.927
64A	47.00	3.840	3.477	3.820	3.500	3.498	3.418	3.430	3.256	3.230	3.269	3.215	2.916			3.400	2.788	3.220	2.928		
65	47.56	3.950	3.600			3.591	3.517														
66	47.87	3.990	3.647	3.980	3.672																
66A	48.31	4.020	3.687	4.010	3.709	3.709	3.643	3.620	3.467	3.450	3.501	3.441	3.165	3.375	3.118	3.610	3.023	3.410	3.143	3.440	3.134
67	48.77					3.779	3.718														
69	49.42	4.150	3.842	4.160	3.874	3.864	3.810	3.800	3.666	3.660	3.722	3.657	3.401	3.621	3.378	3.820	3.253	3.570	3.323	3.650	3.362
69A	49.63																				
70	49.81																				
71	50.53	4.300	4.018	4.310	4.040	4.043	4.001	3.990	3.874	3.860	3.932	3.888	3.652	3.862	3.633	4.050	3.504	3.780	3.554	3.850	3.580
71A	51.00																				
72	51.34	4.480	4.216	4.410	4.151	4.168	4.135	4.140	4.037			4.060	3.838	3.999	3.780	4.170	3.639	3.920	3.709	4.050	3.793
72A	51.78	4.530	4.276	4.490	4.237																
73A	52.43	4.630	4.391	4.580	4.337																

Table Appendix 7.1 (Continued)

		Nov-75		Nov-76		Oct-78		May-82		May-85		Apr-87		May-88		Jul-88		Aug-89		Dec-90	
Xs No	dis (km)	Mea	Adj	Mea	Adj	Mea	Adj	Mea	Adj	Mea	Adj	Mea	Adj	Mea	Adj	Mea	Adj	Mea	Adj	Mea	Adj
73	53.25	4.710	4.490			4.446	4.433														
OR(74)	53.56			4.730	4.503	4.490	4.481	4.470	4.404	4.310	4.410			4.340	4.149	4.550	4.060	4.290	4.120	4.330	4.109
75	53.91																				
76	54.18															4.660	4.181	4.370	4.211	4.460	4.249
77	54.44																				
78	54.73					4.630	4.634	4.660	4.613												
79	55.09	4.910	4.732	4.880	4.674																
80	55.35																				
81	55.57																				
82	55.86	4.990	4.830	4.970	4.775	4.728	4.744	4.800	4.772			4.713	4.571								
84	56.19																				
85	56.31	5.070	4.920	5.090	4.901	4.832	4.853														
86	56.50																				
87	56.79			5.120	4.938			4.900	4.887												
88	56.99	5.110	4.976																		
89	57.38											4.850	4.736	4.860	4.718			4.790	4.691	4.870	4.710
90	57.63	5.220	5.101			4.964	4.999	5.000	5.001												
92	58.17																				
93	58.49	5.320	5.220	5.330	5.172	5.087	5.131	5.110	5.125			5.055	4.960	5.117	4.989	5.320	4.921	5.020	4.941	5.030	4.888
94	58.85																				
95	58.87																				
96	58.90	5.400	5.310													5.410	5.019	5.100	5.029	5.140	5.005
97	59.37					5.228	5.282	5.220	5.250	5.090	5.244										
98	59.60			5.460	5.318																
99	59.85																	5.190	5.136	5.220	5.100
100	60.11	5.500	5.438	5.530	5.395	5.302	5.364	5.320	5.362											5.290	5.174
101	60.33																				
102	60.78																				
103	60.98																				
104	61.23	5.650	5.613																		
105	61.33													5.530	5.438			5.420	5.394	6.010	5.914
106	61.50	5.730	5.699	5.780	5.664	5.513	5.590	5.520	5.585			5.503	5.462	5.558	5.468	5.770	5.427				
107	62.50	5.810	5.802			5.620	5.708	5.610	5.691												
108	63.05											5.179	5.165	5.749	5.679	5.970	5.656	5.660	5.666		
108A	63.54	5.970	5.986	6.040	5.953	5.708	5.807	5.770	5.869												
109A	64.13																			5.900	5.849
109	64.52	6.090	6.129	6.160	6.087	5.906	6.016	5.890	6.005					5.952	5.900	6.160	5.873	5.850	5.883		
110A	64.95	6.120	6.169	6.185	6.118	5.948	6.063	5.960	6.062	5.850	6.055	5.960	5.980	6.010	5.964	6.250	5.971	5.900	5.941	5.980	5.942
R(110)	65.45	6.180	6.240	6.280	6.220	6.070	6.190	6.020	6.150	5.940	6.150	6.061	6.090	6.110	6.070	6.340	6.070	6.000	6.050	6.060	6.030
110	65.45																				
110C	65.68																				
111	66.48	6.340	6.401	6.430	6.367																
111C	67.18											6.302	6.328					6.220	6.265	6.310	6.280
111D	67.48																				
111E	67.64																				
112	68.01	6.550	6.613	6.670	6.603	6.424															
113	69.10	6.710	6.775					6.520	6.641			6.511	6.534	6.531	6.482	6.770	6.470	6.460	6.499	6.550	6.520
114	70.63	6.910	6.977																		
115	72.34					7.010	7.130	6.900	7.012			6.900	6.917	6.934	6.877	7.180	6.853	6.820	6.849	6.960	6.931
116	73.16	7.130	7.200																		
117	74.48	7.350	7.422			7.210	7.330	7.070	7.177			7.121	7.134	7.158	7.096	7.410	7.065	7.040	7.062	7.220	7.191
118	74.95	7.370	7.442																		
119	75.34	7.430	7.503					7.230	7.334			7.395	7.407	7.575	7.511	7.710	7.358	7.320	7.339	7.420	7.391
120	76.32													7.379	7.312	7.680	7.320	7.240	7.256	7.420	7.391
HulyN	76.90	7.550	7.625	7.660	7.570	7.360	7.480	7.360	7.460												
121	77.41	7.600	7.676									7.457	7.465								
122	79.49	7.780	7.858			7.660	7.780	7.670	7.764			7.682	7.686	7.753	7.678	8.070	7.683	7.630	7.636	7.740	7.712
123	80.85	8.010	8.090	8.150	8.050	7.870	7.990	7.880	7.970	7.838	8.050	8.008	8.010	8.088	8.010	8.408	8.010	8.008	8.010	8.038	8.010
124	83.48	8.180	8.293	8.440	8.377	8.190	8.349	8.140	8.280			8.190	8.233			8.580	8.201	8.180	8.230	8.270	8.259
126	85.46	8.700	8.838																		
MaugC	87.55	9.010	9.174	9.190	9.184	8.930	9.149	8.820	9.038			8.900	9.007	8.906	8.930	9.250	8.900	8.840	8.964	8.980	8.995
132	94.45	9.940	10.190	10.080	10.170	9.780	10.100	9.730	10.080	9.560	10.070	9.775	9.990	9.840	9.970	10.260	9.960	9.690	9.940	9.860	9.920

Appendix Eight

Data of Bed Level Profiles in Figure 7.7

The following table lists the averaged bed level profiles in meters relative to Moturiki Datum in 1958, 1982 and 1990. They are determined based on the corresponding water profiles at the discharge of 350 m^3/s. "Xs No" represents the cross section number, and "dis" is the distance from the river mouth (km).

Table Appendix 8.1 Bed level profiles in the reach between Mercer and Ngaruawahia in 1958, 1982, and 1990.

Xs No	*dis (km)*	*Bed Level (m)*					*Bed Level (m)*		
		Jun-58	May-82	Dec-90	Xs No	dis (km)	Jun-58	May-82	Dec-90
rec	40.60	1.600	0.399	0.129	97	59.37	3.716	3.388	
58	41.75	1.730	0.462	0.245	99	59.85	3.770		3.231
59	42.38	1.801	0.555		100	60.11	3.799	3.483	3.298
60	42.61	1.827			105	61.33	3.937		
60A	43.17	1.890		0.339	106	61.50	3.956	3.686	
61A	43.98	1.981	0.873	0.501	107	62.50	4.069	3.782	
61	44.68	2.060	0.987	0.641	108	63.05	4.131		
62	45.45	2.147	1.119	0.798	108A	63.54	4.186	3.939	
62A	46.19	2.230	1.251	0.975	109A	64.13	4.252		3.884
64	46.61	2.277		1.148	109	64.52	4.296	4.053	
64A	47.00	2.321	1.388		110A	64.95	4.345	4.126	3.959
65	47.56	2.385			R(110)	65.45	4.401	4.189	4.039
66A	48.31	2.469	1.618	1.358	110	65.45	4.401		
67	48.77	2.521			110C	65.68	4.427		
69	49.42	2.594	1.812	1.575	111	66.48	4.517		
69A	49.63	2.618			111C	67.18	4.596		4.225
71	50.53	2.719	1.999	1.765	113	69.10	4.813	4.557	4.419
71A	51.00	2.772			114	70.63	4.985		
72	51.34	2.811	2.177	1.988	115	72.34	5.178	4.866	4.777
OR(74)	53.56	3.061	2.533	2.279	116	73.16	5.270		
76	54.18	3.131		2.416	117	74.48	5.419	5.023	5.037
78	54.73	3.193	2.742		118	74.95	5.472		
81	55.57	3.287			119	75.34	5.516	5.179	
82	55.86	3.320	2.925		120	76.32	5.626		5.249
85	56.31	3.371			HulyN	76.90	5.692	5.305	
87	56.79	3.425	3.029		121	77.41	5.749		
89	57.38	3.491		2.868	122	79.49	5.984	5.473	5.436
90	57.63	3.520	3.140		123	80.85	6.137	5.707	5.767
92	58.17	3.580			124	83.48	6.433	5.952	5.943
93	58.49	3.617	3.269	3.044	MangC	87.55	6.892	6.609	6.566
95	58.87	3.659			132	94.45	7.670	7.480	7.300
96	58.90	3.663		3.164					

Appendix Nine

Monthly Cumulative Sand Extraction Volumes around Mercer in 1958–1990

Monthly cumulative sand extraction volumes taken from the Waikato River around Mercer in 1958–1990 are estimated based on the quarterly figures from Fenton (1989) and the files held at the Waikato Regional Council. They are listed in the following table.

Table Appendix 9.1 Monthly cumulative sand extraction volumes (m^3) around Mercer in 1958–1990 (data from Fenton, 1989; the Waikato Regional Council).

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1958	10475	20949	31424	41208	50992	60777	69510	78244	86977	96936	106895	116854
1959	125202	133549	141897	152078	162259	172440	181540	190640	199740	209733	219725	229718
1960	239575	249433	259291	271927	284563	297199	307723	318248	328773	339463	350154	360845
1961	369977	379109	388241	399102	409963	420824	430956	441088	451221	461257	471293	481328
1962	489817	498305	506793	519952	533110	546269	560425	574581	588737	602381	616025	629669
1963	641540	653411	665282	680727	696172	711618	730333	749049	767764	783277	798790	814303
1964	838949	863596	888242	912080	935917	959754	983103	1006452	1029801	1054537	1079273	1104009
1965	1132570	1161131	1189692	1220767	1251841	1282916	1310257	1337597	1364938	1391882	1418826	1445771
1966	1474576	1503381	1532186	1564332	1596477	1628623	1660115	1691606	1723097	1758851	1794606	1830360
1967	1841875	1853391	1864906	1880221	1895537	1910852	1927844	1944835	1961827	1975923	1990020	2004116
1968	2017122	2030129	2043135	2057898	2072661	2087424	2105439	2123454	2141469	2156086	2170704	2185321
1969	2199898	2214475	2229052	2245640	2262228	2278816	2298691	2318567	2338442	2358484	2378525	2398567
1970	2417081	2435596	2454110	2472620	2491129	2509639	2530247	2550854	2571462	2593860	2616259	2638657
1971	2657921	2677184	2696448	2718472	2740497	2762521	2785691	2808862	2832032	2853369	2874707	2896044
1972	2912105	2928166	2944227	2968513	2992798	3017084	3043980	3070875	3097771	3123936	3150101	3176266
1973	3203079	3229893	3256706	3284883	3313061	3341238	3376458	3411679	3446899	3481781	3516662	3551544
1974	3583687	3615829	3647972	3685573	3723173	3760774	3797803	3834831	3871860	3905965	3940069	3974174
1975	3999767	4025359	4050952	4080507	4110063	4139618	4169270	4198923	4228575	4258962	4289349	4319736
1976	4338462	4357189	4375915	4410555	4445195	4479835	4509601	4539366	4569132	4599342	4629553	4659763
1977	4685141	4710518	4735896	4760827	4785757	4810688	4831287	4851885	4872484	4892493	4912502	4932511
1978	4947950	4963388	4978827	4996531	5014234	5031938	5046767	5061597	5076426	5093451	5110477	5127502
1979	5145541	5163580	5181619	5197340	5213060	5228781	5244077	5259373	5274669	5292141	5309613	5327085
1980	5341256	5355428	5369599	5385060	5400521	5415982	5432731	5449480	5466229	5483164	5500100	5517035
1981	5534727	5552420	5570112	5576912	5583712	5590512	5590512	5590512	5590512	5609935	5629358	5648781
1982	5664995	5681209	5697423	5717299	5737175	5757051	5775469	5793888	5812306	5827939	5843573	5859206
1983	5877326	5895447	5913567	5928723	5943879	5959035	5980224	6001414	6022603	6040221	6057840	6075458
1984	6092810	6110163	6127515	6144926	6162338	6179749	6201538	6223327	6245116	6260456	6275797	6291137
1985	6311907	6332676	6353446	6375045	6396644	6418243	6441038	6463833	6486628	6505914	6525201	6544487
1986	6564867	6585248	6605628	6631212	6656797	6682381	6706167	6729954	6753740	6775828	6797915	6820003
1987	6844595	6869188	6893780	6918940	6944101	6969261	6993824	7018387	7042950	7063401	7083853	7104304
1988	7125750	7147195	7168641	7177201	7185760	7194320	7213667	7233014	7252361	7271433	7290505	7309577
1989	7326041	7342504	7358968	7376289	7393609	7410930	7423505	7436079	7448654	7463571	7478488	7493405
1990	7506169	7518933	7531697	7542957	7554216	7565476	7578934	7592391	7605849	7617760	7629671	7641582

Appendix Ten

Monthly Sand Extraction Volumes used as a Source Term in the Models between Mercer and Ngaruawahia

Monthly sand extraction volumes taken from the Waikato River between Mercer and Ngaruawahia during 1958–1990 are estimated based on the quarterly figures from Fenton (1989) and the files held at the Waikato Regional Council. They include three parts: sand taken for stopbank, for construction of the power station, and commercial sand taken by Roose Co. at Huntly. All of them are listed in the following table.

Table Appendix 10.1 Monthly sand extraction volumes (1000 m^3) from the river between Mercer and Ngaruawahia in 1958–1990 (data from Fenton, 1989; the Waikato Regional Council).

Stopbank (1000 m**3)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1967	0.561	0.561	0.561	0.561	0.561	0.561	0.561	0.561	0.561	0.561	0.561	0.561
1968	8.257	8.257	8.257	8.257	8.257	8.257	8.257	8.257	8.257	8.257	8.257	8.257
1969	13.343	13.343	13.343	13.343	13.343	13.343	13.343	13.343	13.343	13.343	13.343	13.343
1970	13.042	13.042	13.042	13.042	13.042	13.042	13.042	13.042	13.042	13.042	13.042	13.042

Power Station (1000 m**3)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1974	41.667	41.667	41.667	41.667	41.667	41.667	41.667	41.667	41.667	41.667	41.667	41.667
1975	8.333	8.333	8.333	8.333	8.333	8.333	8.333	8.333	8.333	8.333	8.333	8.333
1976	4.917	4.917	4.917	4.917	4.917	4.917	4.917	4.917	4.917	4.917	4.917	4.917
1977	3.667	3.667	3.667	3.667	3.667	3.667	3.667	3.667	3.667	3.667	3.667	3.667
1978	5.000	5.000	5.000	5.000	5.000	5.000	5.000	5.000	5.000	5.000	5.000	5.000
1979	3.750	3.750	3.750	3.750	3.750	3.750	3.750	3.750	3.750	3.750	3.750	3.750
1980	3.167	3.167	3.167	3.167	3.167	3.167	3.167	3.167	3.167	3.167	3.167	3.167
1983	1.750	1.750	1.750	1.750	1.750	1.750	1.750	1.750	1.750	1.750	1.750	1.750
1984	4.167	4.167	4.167	4.167	4.167	4.167	4.167	4.167	4.167	4.167	4.167	4.167

Roose Co. at Huntly (1000 m**3)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1974	0.000	0.000	0.000	0.720	0.720	0.720	3.582	3.582	3.582	4.228	4.228	4.228
1975	3.745	3.745	3.745	4.031	4.031	4.031	4.373	4.373	4.373	5.309	5.309	5.309
1976	2.833	2.833	2.833	6.348	6.348	6.348	7.133	7.133	7.133	6.026	6.026	6.026
1977	5.364	5.364	5.364	5.041	5.041	5.041	5.794	5.794	5.794	5.268	5.268	5.268
1978	4.324	4.324	4.324	5.220	5.220	5.220	4.673	4.673	4.673	5.942	5.942	5.942
1979	3.777	3.777	3.777	5.296	5.296	5.296	4.176	4.176	4.176	4.374	4.374	4.374
1980	3.883	3.883	3.883	4.194	4.194	4.194	3.481	3.481	3.481	2.709	2.709	2.709
1981	0.050	0.050	0.050	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Appendix Eleven

Monthly Discharges at the Ngaruawahia Cableway in 1958–1990

Table Appendix 11.1 Monthly discharges (m^3/s) at the Ngaruawahia Cableway in 1958–1990 (data from the Waikato Regional Council).

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Mean
1958	270	496	391	303	325	321	393	392	323	283	270	408	348
1959	315	296	288	354	389	408	336	340	273	338	265	222	319
1960	230	276	264	219	227	376	395	412	505	328	275	215	310
1961	201	194	197	192	215	285	486	325	315	265	240	207	260
1962	190	203	312	263	369	574	499	452	491	562	570	542	419
1963	297	245	216	228	249	325	456	354	435	265	242	195	292
1964	229	233	344	230	273	333	572	540	514	543	365	316	374
1965	286	432	335	297	324	422	421	478	312	248	304	244	342
1966	320	297	326	258	424	410	587	499	469	330	300	276	375
1967	302	441	295	239	242	304	312	408	386	280	385	386	332
1968	272	229	197	254	303	527	469	428	407	371	327	342	344
1969	264	333	235	225	330	348	380	369	307	268	192	208	288
1970	175	159	150	156	242	412	413	482	447	581	455	239	326
1971	301	231	199	175	222	366	397	407	575	599	402	357	353
1972	252	187	403	199	358	353	520	444	363	303	255	239	323
1973	218	181	207	184	256	329	331	368	411	260	299	240	274
1974	207	226	163	173	199	380	507	454	402	396	302	290	308
1975	323	184	217	241	376	557	538	457	494	417	349	272	369
1976	331	349	278	264	326	398	578	579	499	455	336	237	386
1977	222	187	217	213	342	445	543	468	436	368	252	250	329
1978	232	231	194	183	199	247	414	323	306	286	341	246	267
1979	208	257	289	310	439	369	438	486	423	498	433	355	375
1980	433	349	318	321	303	373	476	446	464	367	357	401	384
1981	311	267	262	298	282	451	482	502	425	391	349	299	360
1982	275	254	239	236	293	309	304	274	250	277	240	207	263
1983	220	198	165	194	263	297	292	250	296	389	414	274	271
1984	245	216	255	231	227	306	348	353	297	269	230	265	270
1985	248	225	225	227	234	355	293	293	324	235	273	335	272
1986	474	282	227	187	243	318	483	504	418	347	295	230	334
1987	231	183	212	298	292	343	304	266	336	259	230	278	269
1988	242	223	205	189	240	345	399	533	431	590	441	401	353
1989	474	401	288	288	300	468	473	339	384	555	345	277	383
1990	273	308	338	320	405	381	401	680	461	384	374	292	385
Mean	275	266	256	241	294	377	432	421	399	373	324	289	329

Appendix Twelve

Rating Curve at the Huntly Power Station in April 1993

The following table is the latest available rating curve (April 1993) used at the Huntly Power Station (site number 1543495). It is taken from the Waikato Regional Council database. The left column and the first row in the table are water levels (cm, Moturiki Datum), and the others are discharges (m^3/s). For example, the discharge of 92 m^3/s is associated with the water level of 609 cm from Table Appendix 12.1.

Table Appendix 12.1 Rating curve at the Huntly Power Station in the Waikato River in April 1993 (data from the Waikato Regional Council).

WL cm	*0*	*1*	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*
600	81	82	83	84	86	87	88	90	91	92
610	93	95	96	97	99	100	101	102	104	105
620	106	108	109	110	112	113	114	116	117	118
630	120	121	122	124	125	126	128	129	130	132
640	133	134	136	137	139	140	141	143	144	146
650	147	148	150	151	153	154	155	157	158	160
660	161	163	164	166	167	169	170	171	173	174
670	176	177	179	180	182	183	185	186	188	189
680	191	193	194	196	197	199	200	202	203	205
690	207	208	210	211	213	215	216	218	219	221
700	223	224	226	228	229	231	233	234	236	238
710	239	241	243	245	246	248	250	251	253	255
720	257	258	260	262	264	266	268	269	271	273
730	275	277	279	281	283	284	286	288	290	292
740	294	296	298	300	302	304	306	308	310	312
750	314	316	318	320	322	324	326	328	331	333
760	335	337	339	341	343	345	348	350	352	354
770	356	359	361	363	365	367	370	372	374	376
780	379	381	383	386	388	390	393	395	397	400
790	402	404	407	409	412	414	416	419	421	424
800	426	429	431	433	436	438	441	443	446	448
810	451	454	456	459	461	464	466	469	472	474
820	477	479	482	485	487	490	493	495	498	501
830	503	506	509	511	514	517	520	522	525	528
840	531	534	536	539	542	545	548	550	553	556
850	559	562	565	568	571	573	576	579	582	585
860	588	591	594	597	601	604	607	610	613	616
870	620	623	626	629	633	636	639	642	646	649
880	652	656	659	662	666	669	672	676	679	683
890	686	690	693	696	700	703	707	710	714	717

Appendix Thirteen

A Summary of Gauging Records at the Ngaruawahia Cableway

The following table is a summary of the gauging data used for an analysis of the formation of alternate bars at the Ngaruawahia Cableway between 1967–1993.

Table Appendix 13.1 A summary of gauging data for the analysis at the Ngaruawahia Cableway (data from the Waikato Regional council). "Dis" refers to discharge, "MaxD" refers to maximum water depth.

No	Date	Stage (m)	Dis (m^3/s)	Area (m^2)	Width (m)	MaxD (m)	No	Date	Stage (m)	Dis (m^3/s)	Area (m^2)	Width (m)	MaxD (m)
464	17-Jan-67	10.43	387.1	489.1	152.7	3.78	2118	12-Jul-76	11.98	700.3	740.3	171.0	5.48
477	4-Feb-67	12.53	873.4	816.8	170.7	5.76	2123	15-Jul-76	11.17	520.7	612.8	160.0	4.82
480	6-Feb-67	12.28	838.8	795.3	167.6	5.70	2127	27-Jul-76	10.71	459.2	536.0	156.0	4.24
481	7-Feb-67	12.13	809.9	760.2	169.2	5.70	2138	12-Aug-76	11.77	635.5	697.9	168.0	5.24
486	8-Feb-67	11.80	730.9	702.0	167.6	5.06	2154	15-Sep-76	11.79	688.4	711.3	168.5	5.40
491	10-Feb-67	11.14	546.4	592.2	158.5	4.57	2163	23-Sep-76	10.68	455.7	547.7	154.5	4.90
496	15-May-67	9.41	229.9	337.4	147.2	2.96	2182	8-Nov-76	10.46	418.8	527.4	154.0	4.44
505	3-Aug-67	10.12	319.9	451.4	150.9	3.60	2190	6-Dec-76	9.69	262.8	375.1	150.0	3.20
595	26-Feb-68	9.39	201.0	325.1	149.3	2.87	6552	11-Mar-77	9.25	189.9	321.4	147.0	3.08
613	6-Mar-68	9.38	207.1	317.7	148.4	2.77	2430	1-Apr-77	9.27	207.1	325.1	147.5	3.00
650	25-Mar-68	9.13	164.0	277.5	141.2	2.47	2506	14-Jun-77	10.22	357.1	461.0	153.0	3.79
754	30-Aug-69	10.20	308.3	436.2	151.5	3.72	2511	4-Jul-77	11.89	738.6	733.0	170.0	5.46
773	7-Nov-69	9.26	190.9	321.7	147.8	2.84	2540	9-Aug-77	10.72	472.1	539.9	156.0	4.10
805	20-Nov-69	9.10	172.6	310.6	147.2	2.87	2550	14-Sep-77	10.47	412.6	501.7	154.0	3.92
887	27-Feb-70	8.95	145.8	294.1	147.5	2.62	2558	31-Oct-77	9.74	284.6	405.9	151.0	3.44
908	9-Jun-70	11.04	482.8	584.7	157.6	4.63	2583	2-Dec-77	9.59	262.3	358.0	149.5	2.90
921	15-Aug-70	12.30	762.1	786.9	171.6	5.79	2600	21-Dec-77	9.87	289.1	397.1	150.0	3.38
962	6-Oct-70	12.22	801.3	774.8	172.5	5.85	2760	3-Feb-78	9.30	208.2	320.4	148.0	2.84
963	7-Oct-70	11.96	739.9	731.0	170.7	5.58	2855	10-Mar-78	9.25	208.2	317.5	147.0	2.82
966	8-Oct-70	11.72	667.5	681.0	167.6	5.00	2978	27-Apr-78	9.01	170.0	284.5	145.0	2.55
972	12-Oct-70	11.25	551.9	593.1	158.5	4.72	2982	6-Jun-78	8.94	160.7	267.4	145.0	2.53
979	16-Oct-70	10.90	489.9	557.2	157.6	4.48	2989	24-Jul-78	10.37	404.2	477.5	154.0	3.93
980	19-Oct-70	10.91	506.9	614.4	157.0	4.33	2993	16-Aug-78	9.90	303.3	402.4	150.5	3.45
984	27-Oct-70	10.34	351.3	457.8	152.7	3.84	2996	19-Sep-78	10.50	409.6	499.5	153.0	4.04
1004	15-Apr-71	9.07	171.2	305.0	146.6	2.71	3005	19-Oct-78	9.42	233.8	349.8	148.0	3.11
1097	13-Jul-72	11.70	685.2	709.3	168.5	5.67	3008	10-Nov-78	9.55	249.5	362.0	149.0	3.19
1120	30-Aug-72	10.66	433.5	537.6	154.8	4.60	3020	6-Dec-78	9.63	267.9	376.1	149.0	3.32
1127	22-Jan-73	9.25	205.1	349.7	148.7	3.30	3214	7-Mar-79	9.50	246.5	359.6	149.0	3.24
1160	21-Feb-73	8.93	148.5	288.2	144.8	3.02	3224	17-Apr-79	9.83	300.8	402.5	151.0	3.58
1258	27-Jul-73	10.26	351.5	482.2	153.0	4.22	3229	18-May-79	11.74	660.5	697.6	168.0	5.52
1450	15-May-74	9.02	179.2	304.1	146.0	3.10	3251	24-Jul-79	10.17	345.6	448.1	152.0	3.70
1483	27-Nov-74	9.71	257.2	369.6	151.0	3.30	3397	12-May-80	9.53	222.6	361.3	155.0	3.12
1611	6-Mar-75	9.10	164.7	285.5	146.0	2.76	3414	7-Aug-80	10.70	463.8	560.1	164.0	4.54
2115	9-Jul-76	12.61	875.6	857.1	174.0	6.26	3420	2-Oct-80	10.52	439.5	527.4	155.0	4.30

Table Appendix 13.1 (Continued)

No	Date	Stage (m)	Dis (m^3/s)	Area (m^2)	Width (m)	MaxD (m)	No	Date	Stage (m)	Dis (m^3/s)	Area (m^2)	Width (m)	MaxD (m)
3442	7-Jan-81	9.56	270.1	390.3	153.0	3.38	14243	19-Sep-89	10.14	389.7	468.5	149.0	3.74
3492	3-Mar-81	9.14	199.1	313.0	150.0	2.80	14303	16-Oct-89	11.73	752.6	739.1	167.0	5.50
3683	6-Aug-81	10.81	509.6	550.0	155.0	4.40	14306	18-Oct-89	12.29	883.8	820.7	170.0	6.26
3743	15-Oct-81	10.01	360.3	444.8	147.0	3.74	14405	22-Nov-89	9.62	299.0	397.6	145.0	3.30
4015	23-Apr-82	9.26	219.6	306.9	139.0	2.81	14764	14-Dec-89	9.63	301.7	390.7	146.0	3.08
4056	17-Aug-82	9.87	319.8	419.9	152.0	3.48	15198	14-Feb-90	9.88	348.6	419.1	146.0	3.40
4070	17-Sep-82	9.37	248.0	352.1	140.0	3.08	15317	7-Mar-90	9.49	278.5	372.9	146.0	3.20
5158	8-Dec-82	9.38	261.7	348.3	145.0	3.06	15342	13-Mar-90	10.92	545.9	577.9	156.0	4.58
5865	9-Feb-83	9.29	208.3	322.8	147.0	3.03	15416	3-Apr-90	9.43	281.0	358.7	144.0	3.02
5790	16-Mar-83	8.83	162.8	251.9	140.5	2.45	15524	17-May-90	9.57	288.3	378.2	144.0	3.22
5871	28-Apr-83	9.49	253.2	345.6	140.0	3.10	15587	13-Jun-90	10.12	391.2	455.1	148.0	3.60
5912	14-Jul-83	10.00	330.3	410.3	151.5	3.30	15720	11-Jul-90	10.29	419.5	467.8	143.0	4.00
5980	15-Sep-83	10.26	379.5	454.9	153.0	3.65	15803	8-Aug-90	11.74	697.0	689.9	165.0	5.15
5954	29-Sep-83	9.94	297.2	405.1	151.0	3.45	15814	9-Aug-90	11.88	739.1	719.4	168.0	5.40
5962	7-Oct-83	11.31	628.7	614.4	152.0	4.70	15931	24-Aug-90	12.18	807.4	780.4	171.0	5.52
5978	3-Nov-83	10.39	396.5	474.8	156.5	3.76	16118	10-Oct-90	10.00	357.7	422.3	145.0	3.70
6014	24-Nov-83	10.32	395.7	463.5	153.0	3.88	16158	6-Nov-90	10.26	408.7	486.9	150.0	3.90
7005	23-Feb-84	9.07	193.1	289.9	155.0	2.45	16219	26-Nov-90	10.20	386.5	455.2	152.0	3.80
6031	12-Apr-84	9.37	232.0	332.9	143.0	2.79	16432	19-Dec-90	9.32	253.6	341.8	141.0	3.06
7033	6-Jul-84	10.44	405.7	487.9	154.0	3.80	16556	16-Jan-91	9.29	246.0	331.1	144.0	3.05
7062	14-Aug-84	9.86	324.2	405.2	148.0	3.28	16826	19-Feb-91	10.55	475.3	526.2	147.0	4.40
7066	23-Aug-84	10.56	439.2	500.5	156.0	3.80	17034	28-Mar-91	9.32	265.8	363.4	147.0	3.30
7574	30-Jan-85	9.35	234.7	316.2	144.0	2.70	17157	8-May-91	9.34	261.3	347.6	136.0	3.30
7824	25-Jun-85	11.65	654.0	679.5	165.0	5.02	17778	12-Jun-91	9.28	255.5	350.2	140.0	3.25
7825	27-Jun-85	10.99	510.8	582.2	158.0	4.62	17915	16-Jul-91	9.48	272.1	386.0	147.0	3.30
7831	9-Jul-85	9.47	242.5	339.5	145.0	2.88	17997	29-Jul-91	11.00	543.1	597.2	155.0	4.90
7847	21-Aug-85	9.93	320.5	411.4	142.0	3.35	18024	12-Aug-91	12.38	909.8	817.2	170.0	6.10
8112	6-Jan-86	12.06	734.4	715.5	169.5	5.42	18164	25-Sep-91	10.69	508.3	556.4	155.0	4.40
8901	27-Jul-86	12.64	960.4	870.4	179.0	6.30	18300	30-Oct-91	10.54	487.6	525.6	146.0	4.30
8895	30-Jul-86	12.21	858.8	790.1	166.0	6.02	18553	9-Dec-91	9.15	221.2	310.6	140.0	3.00
8911	1-Aug-86	11.49	638.4	663.8	161.0	5.20	18852	14-Jan-92	9.34	244.9	330.4	140.0	3.20
9157	24-Oct-86	10.03	373.8	446.9	152.0	3.96	19194	20-Feb-92	9.37	256.9	361.2	145.0	3.35
9575	22-Dec-86	9.15	187.5	275.2	137.0	2.70	19338	23-Mar-92	9.51	282.0	376.0	144.0	3.50
10554	9-Apr-87	9.54	274.2	372.5	150.0	3.00	19488	28-Apr-92	9.19	222.8	336.2	141.0	3.10
11181	28-Aug-87	9.81	321.6	414.9	156.5	3.52	19586	3-Jun-92	9.24	240.0	349.4	143.0	3.25
11352	3-Nov-87	9.44	256.9	358.8	144.0	3.10	19748	8-Jul-92	10.51	453.3	538.7	147.0	4.70
11694	24-Dec-87	9.70	286.8	352.5	145.0	3.07	19826	23-Jul-92	11.71	699.3	726.4	162.0	6.00
12279	11-Feb-88	9.32	199.9	273.5	142.0	2.50	20040	17-Sep-92	10.77	511.6	583.8	155.0	4.90
12681	10-Jun-88	11.21	567.6	630.0	161.0	5.00	20107	29-Oct-92	10.48	453.4	528.5	155.0	4.20
10611	29-Jul-88	11.90	754.1	752.7	171.0	6.19	20148	24-Nov-92	9.64	306.3	397.9	145.0	3.40
10865	10-Oct-88	12.50	923.7	826.9	172.0	5.96	20472	12-Feb-93	8.87	186.4	276.4	134.0	2.80
10885	14-Oct-88	11.14	561.6	600.7	160.0	4.70	20580	8-Mar-93	8.91	193.4	294.8	138.0	2.80
12753	8-Nov-88	10.46	440.1	526.3	158.0	4.29	20686	15-Apr-93	8.91	193.7	289.2	135.0	2.90
12948	5-Dec-88	10.41	430.1	484.8	153.0	3.99	20782	19-May-93	11.44	730.9	695.2	170.0	5.40
13641	17-Mar-89	9.83	307.7	387.8	147.0	3.30	20889	24-Jun-93	10.61	510.5	548.7	156.0	4.55
13707	17-Apr-89	9.62	291.9	381.8	144.0	3.30	20891	30-Jun-93	11.61	740.8	709.4	165.0	5.40
13780	12-Jun-89	10.12	398.1	485.0	162.0	3.88	20966	27-Jul-93	9.32	257.4	361.0	146.0	3.30
13907	26-Jul-89	10.25	398.4	467.4	152.0	3.82	21040	1-Sep-93	9.18	240.1	348.0	143.0	3.10
14191	21-Aug-89	9.42	272.5	365.0	143.0	3.14							