The Waimakariri River and its Environment: Frequency Analysis of Extreme Events and the Effects of Flow Modifications

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Barbara Katharina Nagy

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The Waimakariri River is known for its unpredictability; flood and drought events can occur at any time of the year. Knowledge about the return period of extreme events is crucial for water management and planning. Frequency analysis uses the stochastic nature of these extreme events as a basic concept to account for uncertainty by describing the expected occurrence of floods and droughts. While frequency estimates for the Waimakariri River have been produced in the past, sampling from the historical stream record has almost exclusively relied on annual maximum series (AMS), in combination with a few selected probability distributions based on theoretical considerations. Low flow frequency analysis, albeit an informative tool for water planning, is seldom utilised and no estimates are published for the Waimakariri River. However, low flow frequency estimates can provide valuable information for the management of instream biota under pressures from flow modification.

This thesis produced flood and low flow frequency estimates for the Waimakariri River, using the partial duration series (PDS) approach to sampling for the first time. Previous estimates utilised in floodplain management have solely relied on the use of AMS. Graphical and statistical testing methods were employed to determine the best fitting probability distributions, guided by the empirical streamflow record. Low flow frequency estimates were produced with 'runs theory' to reflect the multiple dimensions that characterise low flows. Current developments in the Waimakariri catchment are reflected by developing a discharge series subtracting consented water abstractions, which was compared to the current discharge series. Together with a rapid systematic literature review, the results from the low flow frequency analysis were used for a qualitative narrative assessment summarising the likely effects of flow modifications on the Waimakariri River environment.

Statistical results indicate that the PDS produces a better fit to the empirical flood and low flow data than the often applied AMS approach. The procedures for threshold selection and best fit decision making in this study identified methods for choosing the optimum PDS. Contrary to the published literature, the Gumbel distribution is not a good fit for the flood series of the Waimakariri River. Results indicate a better fit of four alternative distributions, i.e. the Log-Pearson III, Pearson III, Generalised Extreme Value and Generalised Pareto distributions. All dimensions of low flow series are equally well described by these four distributions. Design estimates for floods produced by the four alternative distributions are markedly higher than current estimates used in floodplain management. Low flow frequency estimates were produced for duration and deficit series, reflecting the current water allocation regime of the Waimakariri River. The longer low flow durations and higher deficits are expected to significantly impact not only on the reliability of river flow dependent catchment developments, but also on the flow dependent biotic and abiotic environment. Results showed that both habitat quantity and quality ensuring flows are significantly affected by flow modifications.

Keywords: Waimakariri River, frequency analysis, extreme events, flow-ecological relationship, water abstractions, stochastic modelling, historical streamflow data, probability theory, critical thresholds.

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This is dedicated to you, '*a szívem csücske'*.

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List of Abbreviations

7dMALF	7 day mean annual low flow
ACF	autocorrelation function
AEP	annual exceedance probability
AMS	annual maximum/minimum series
ANOVA	Analysis of Variance
CCC	Christchurch City Council
cdf	cumulative distribution function
CPWES	Central Plains Water Enhancement Scheme
CPWL	Central Plains Water Ltd.
CWMS	Canterbury Water Management Strategy
DI	Dispersion Index
ECan	Environment Canterbury
EV1	Extreme Value Type 1 (distribution)
EV2	Extreme Value Type 2 (distribution)
EV3	Extreme Value Type 3 (distribution)
FCC	Filliben Correlation Coefficient
FDC	flow duration curve
FRE_3	frequency of flood events three times the median flow
GEV	Generalised Extreme Value (distribution)
GP	Generalised Pareto (distribution)
IFIM	Instream Flow Incremental Methodology
iid	independent and identically distributed
IQR	interquartile range
KS test	Kolmogorov Smirnov test
L-moments	Linear moments
LN	log-Normal (distribution)
LN2	two parameter log-Normal (distribution)
LN3	three parameter log-Normal (distribution)
LP3	log-Pearson Type 3 (distribution)
MfE	Ministry for the Environment (NZ)
ML	Maximum Likelihood
MOM	Method of Moments
N	Normal (distribution)
NB	negative-binomial (distribution)

NCCB	North Canterbury Catchment Board (NZ)
NERC	National Environment Research Council (UK)
NIWA	National Institute of Water and Atmospheric Research (NZ)
NZFS	New Zealand Forest Service
OHB	Old Highway Bridge discharge measurement site
P3	Pearson Type 3 (distribution)
pdf	probability density function
PDS	partial duration series
PHABSIM	Physical Habitat Simulation
PP plot	probability-probability plot (theoretical vs. empirical)
QQ plot	quantile-quantile plot (theoretical vs. empirical)
RHYHABSIM	River Hydraulic Habitat Simulation
RMA 1991	Resource Management Act 1991
SDC	Selwyn District Council
TCEV	Two-Component Extreme Value (distribution)
WUA	Weighted Usable Area

Nomenclature

Latin letters

A	catchment area (km^2)
d	test statistic corresponding to the Dispersion Index
D	Kolmogorov Smirnov test statistic
df	degrees of freedom
d_i	duration of low flow event *i*
$f(x)$	probability density function (pfd)
$F(x)$	cumulative density function (cdf)
F_X	non-exceedance probability of event *X*
m_i $(\frac{s_i}{d_i})$	magnitude of low flow event *I* according to Yevjevich (1967)
n	number of records
p or *P*	probability
$P(x)$	probability of exceedance
Q	discharge
q_i	probability plotting position
r	correlation coefficient (KS test)
s_i	severity of low flow event *i* (deficit volume)
T	return period
t'	event identifying break-point in time series
T_a	annual return period
T_p	partial duration return period
V	variance
X_i	discrete extreme event
x_o	threshold or truncation value
z	standardised Normal variate

Greek letters

α	level of significance
α	scale parameter
α, β,and ϒ	parameters of the Pearson Type 3 family of distributions
Θ	time interval
κ	shape parameter
λ	average number of event per year above the threshold

μ	mean
ξ	location parameter
ρ	Spearman's Rank Correlation Coefficient
σ	standard deviation
τ_2	LCV (L-coefficient of variation)
τ_3	L-skewness
τ_4	L-kurtosis
τ_a	Kendall's tau-a Correlation Coefficient
ϕ	standard Normal distribution
χ^2	chi-squared test statistic

Chapter 1
Introduction

1.1 Introduction

The threats posed by floods and droughts in New Zealand have been extensively recorded since the European settlement period, and even earlier (Cowie, 1957). Maori tales tell of rough, sweeping rivers and their unpredictability, and it is no surprise that the Waimakariri translates to 'river of cold rushing water' (Logan, 2008). Intensive human settlement, land-use change and channel modification are believed to have greatly increased the intensity and occurrence of such extreme events (Mosley & Pearson, 1997). In other instances, it was simply early migrants unknowingly settling on floodplains; a costly choice that now has to be managed. Payments for flood damages averaged NZD $12.85 million per year[1] between 1976 and 2015 (Insurance Council of New Zealand [ICNZ], 2015). The other spectrum of the extreme has equal effects. The drought of 1997-1998 resulted in an estimated NZD $500 million loss for farmers in Otago (Pearson & Henderson, 2004).

Floods and droughts are random events which cannot be predicted. Therefore, uncertainty is inherently present in water resource planning and management. Uncertainty in the data used for water management stems from a limited understanding of how the water resource functions as a system. Reciprocal feedback between the abiotic and biotic environment is only one of the many puzzling mechanisms that is still not fully understood and thus can only be predicted with limited certainty (Corenblit et al., 2007). Some of this uncertainty can be accounted for by adopting a deterministic approach in which the random variable is replaced by its average or a worst-case value. Concerning the stochasticity of an important variable within the system, an analysis of the performance of the system, and its risk and magnitude of failure is preferred. Furthermore, stochastic methods are often necessary, as some physical processes are too complex to be accounted for in deterministic models (Pearson & Davies, 1997). Therefore, probability theory, which uses random variables as a basic concept, has long been applied in water resource management and planning. Stochastic variables, such as observed flood flows and drought periods, are fitted with probability distribution functions. By definition, the exact value of a random variable cannot be calculated with certainty. However, stochastic methods reduce uncertainty by describing the expected value of the random variable, or the likelihood of occurrence and exceedance (Stedinger, Vogel, & Foufoula-Georgiou, 1993). For convenience, the output of such calculations is described in terms of return periods or quantiles rather than frequency. The return period is the average time between exceedance events of the random variable. The term 'average' is often

[1] inflation adjusted to the year 2011

misleading, as a 100-year event could occur twice within 10 years; however, the probability of such an occurrence is rather low (Revfeim, 1984).

Typical critical quantiles used are, for example, the 100-year flood event for flood hazard planning, or the 10-year return period of the seven day mean annual low flow (7dMALF) for water quality planning (Smakhtin, 2001). The outputs of such quantile estimates are frequently used for further analyses, for example, in combination with hydraulic modelling to assess inundation by flood flows. Flood frequency analysis is also a prerequisite for flood defence schemes as a tool for assessing the reliability and risk of failure of built structures. Water demand and water supply are equally stochastic in nature and can be described in terms of reliability. Therefore, low flow frequency analyses can provide information not only to water planners assessing future developments, but also to ecologists who need to quantify the relationship between ecological aspects and the physical environment.

Environmental or ecological threshold criteria are either defined in terms of quality or quantity conditions of water resources. They are operationalised to minimise or assess the negative effects of water resource developments on aquatic ecosystem health. Criteria have greatly evolved since their first application. Early criteria were based on hydrological rules, such as the Tennant Method (Tennant, 1976), to define the minimum flow necessary for fish habitat. More recent developments account not only for minimum and maximum values, but also consider hydrological variation in spatial and temporal scales (Arthington et al., 1992). Sensitive or highly valued indicator species are used for assessments, acknowledging that maximising the well-being of all affected organisms is impossible. In such assessments, the relationship between habitat suitability indices are quantified in terms of relevant hydrological characteristics (Caissie & El-Jabi, 2003). It is clear that ecosystem diversity thrives with larger variation in hydrological conditions. However, water resource development relies on a reduction in variation and an increased reliability of water resources, and thus conflicts with the functioning of water resources as ecological systems (Arthington, Bunn, Poff, & Naiman, 2006). Therefore, the principal question of water resource management is not “Can we take water?” but rather “How much?”, and frequency analyses in accordance with ecological threshold criteria can be utilised to aid the decision-making process.

1.2 Problem definition

The Waimakariri River drains one of the largest catchments in the South Island of New Zealand and is known as an exceptional habitat for many endangered species. The river is also highly valued for recreational use by, for example, anglers and jet boaters (Hughey et al., 2015). However, as a braided river in close proximity to the largest city of the South Island, it also constitutes a significant flooding hazard. The occurrence of extreme events on both ends of the spectrum is characterised by large unpredictability. Floods can occur during any time of the year. With the rapid development of dairy farming in the area, reliability of flow for irrigation is desirable (Environment Canterbury [ECan], 2009;

Gunningham, 2011). Unfortunately, the occurrence of low flows and droughts is equally unpredictable. Therefore, the analysis of frequency of extreme events in the Waimakariri River can provide valuable information to water resource planning and management in the area.

Due to the close proximity to Christchurch, '*making the Waimakariri go, not where it wanted, but where the engineers dictated*' (Logan, 2008, p. 137) has been a struggle since the 19th century. As a result of a major flood in 1957, the first flood frequency analysis on the Waimakariri River was published using a series of annual maximum (AMS) streamflow measurements (Stephen, 1958). Every subsequent assessment of floods and low flows in the Waimakariri River has been based on AMS, which form the foundation for many floodplain management decisions. The scientific literature, however, has often advocated the use of partial duration series (PDS), in which not only the maximum value of each year, but every value above a predetermined threshold is selected. To date, there has been no study specifically addressing the use of PDS for the Waimakariri River to estimate the probability of occurrence and magnitude of floods and low flows.

The Canterbury region has undergone significant land-use change in the form of a transition from predominantly dry-land farming to dairying over the last few decades. The higher irrigation demand has put increasing pressure on water resources; the Waimakariri is no exception (ECan, 2009). Recently granted resource consents allow for further abstractions from the river in the future, potentially impacting on river ecology by reducing the reliability of flow for meeting in-stream needs. While low flow events occur naturally, an increase in the duration, deficit volumes and number of events can significantly alter ecosystem processes. An analysis of low flow frequencies using ecologically informed threshold flows can thus provide useful information for water resource managers and planners.

1.3 Research aims and objectives

This research has two guiding aims. The first aim is to produce flood and low flow frequency estimates for the Waimakariri River. Previous estimates for flood frequencies have relied on the use of the AMS and a small number of selected probability distributions that have been endorsed on theoretical grounds. Low flow frequency estimates are seldom produced, as the analysis of drought events is often limited to the assessment of flow duration curves. Thus the specific objectives are to:

(1) investigate the use of PDS sampling for the analysis of flood frequencies and to compare frequency estimates with those produced by AMS.

(2) produce low flow frequency estimates using annual minimum series of the 7dMALF and to compare results to those produced by frequency analysis applying 'runs theory'.

(3) investigate the implications of novel results for floodplain and water resource management in the Waimakariri catchment.

The second aim of the study is to apply low flow frequency analysis to determine the extent to which flow regime modifications can alter the probability of meeting in-stream flow requirements. The specific objectives are to:

(4) identify attributes of the Waimakariri flow regime relevant to riverine biota.
(5) determine ecological thresholds in response to flow, which are relevant in braided rivers and specifically to the Waimakariri River.
(6) produce low flow frequency estimates from pre- and post-abstraction conditions.
(7) evaluate the response of biota to altered frequencies of low flow durations and deficits using ecologically informed threshold levels.

1.4 Statement of scope and delimitations

The study is geographically focussed on the Waimakariri River. It should be kept in mind that a river does not 'have' a flood or low flow frequency curve and that the estimates produced are a direct reflection of the sampled data and its quality. Data in this study are used as provided by the Canterbury Regional Council (ECan) and assumed to be correct within standard margins of error (± 8 %). The data used to produce frequency estimates is primarily data from 1967-2015. Annual maxima that exist for 1930-1966 are included into analyses where appropriate. In order to meet objectives 4-7, the literature is reviewed to extract quantifiable relationships between the flow regime and chosen indicator variables. This thesis is not concerned with proposing alternative ways for managing water resources. Instead, the comparison of low flow frequency components under pre- and post-abstraction conditions is aimed at assessing the ability of the resource system to provide the defined necessary flows for appropriate functioning.

1.5 Organisation of the thesis

The thesis is organised into two parts, comprising a total of ten chapters (Figure 1.1). The introduction (Chapter 1) and description of the study area (Chapter 2) are common to both parts of the study. The first part of the thesis (Chapters 3 to 5) deals with the frequency analysis of extreme events in the Waimakariri River. Chapter 3 begins with a literature review of background information useful in the domain of frequency analyses. This chapter also includes an account of previously produced design estimates in New Zealand and the Canterbury region, including approaches to sampling and distribution choice. In Chapter 4 the methods for frequency analyses produced in this thesis are described in detail. Chapter 5 provides results for (a) the flood frequency and (b) the low flow frequency analysis of the Waimakariri River.

The second part of the thesis (Chapters 6 to 8) focusses on an assessment of the reliability of water resources in the Waimakariri River based on frequency analyses of low flows vs. established ecological threshold criteria. Chapter 6 introduces the flow regime as a critical determinant for ecological functioning in the river system, describes past and current methods of establishing in-stream flows, and contextualises developments affecting the flow regime in the Waimakariri River. Chapter 7 describes the methods used to assess the reliability of water resource allocations based on low flow frequency analyses. In Chapter 8, the results from the second part of the study are presented.

Chapter 9 combines both parts of the study in a joint discussion of the results in reference to the wider literature. The final chapter, Chapter 10, concludes the thesis with a summary of research findings, implications, limitations and thoughts on future research directions.

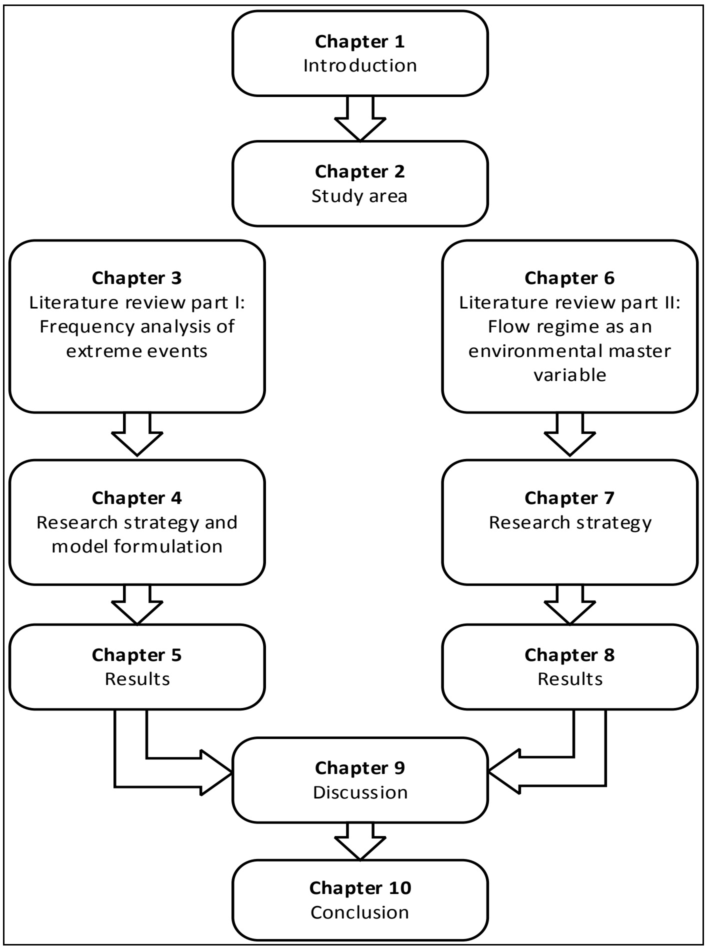


Figure 1.1 Organisation of the thesis.

Chapter 2

Description of the study area

An overview of the study area is presented in this chapter. This includes a catchment overview, the description of the climate and hydrology of the catchment, and a short account of historical flooding and hazard mitigation attempts. In addition, current river values associated with the study area are shortly introduced.

2.1 Catchment overview

The present study investigates the Waimakariri River in the South Island of New Zealand. This river is a braided gravel-bed river with a total length of 151 km and a maximum catchment elevation of 2402 m a.s.l. at its origin in the Southern Alps (ECan & Waimakariri District Council, 2003). The Waimakariri River catchment (Figure 2.1) comprises 3654 km^2 and is the largest catchment within North Canterbury (ECan et al., 2007; Logan, 2008). It emerges as a narrow river, flowing through trough valleys but develops a braided planform below Crow junction until the Esk confluence. Further downstream, the Waimakariri flows through the 300 m deep bedrock Upper Gorge. Once the river emerges from the mountainous terrain, it follows a shallow trench across the Canterbury Plains. 2460 km^2 of the catchment lies upstream of the Lower Gorge in the mountains, encompassing much of Arthur's Pass National Park and Craigieburn Forest Park (ECan & Waimakariri District Council, 2003), of which a total area of 580 km^2 consists of snow tussock, with the majority of less than 40 % permanent snow cover (Blakely & Mosley, 1987b; Logan, 2008). Through the Lower Gorge it once more converges to a narrower trench (ECan, 1991). From here onwards until Crossbank the floodplain can extend up to 2 km in width and the riverbed is characterized by extensive braiding; a number of unstable streams intermitted by gravel or partly vegetated islands. Downstream of Crossbank, the river is confined to a narrower path as a result of previous flood management measures (ECan, 1991). Greywacke gravel and interstitial smaller materials, such as sand and silt, make up most of the riverbed and banks, except at close proximity to the sea, where silt is predominantly found (ECan, 1991). The estimated long-term average annual bedload is 275,000 m^3, with a mean grain diameter of 24 mm (Carson & Griffiths, 1989; Hicks, Westaway, & Lane, 2003). Previous findings point out minimal armouring of the riverbed, indicative of an ample supply of sediment in relation to the transport capacity of the river (Hicks et al., 2007). The Waimakariri River enters the Pacific Ocean approximately 15 km north of Christchurch's city centre, at Pegasus Bay. However, some of the lower reaches extend as close as 5 km to the city's outer suburbs (Logan, 2008).

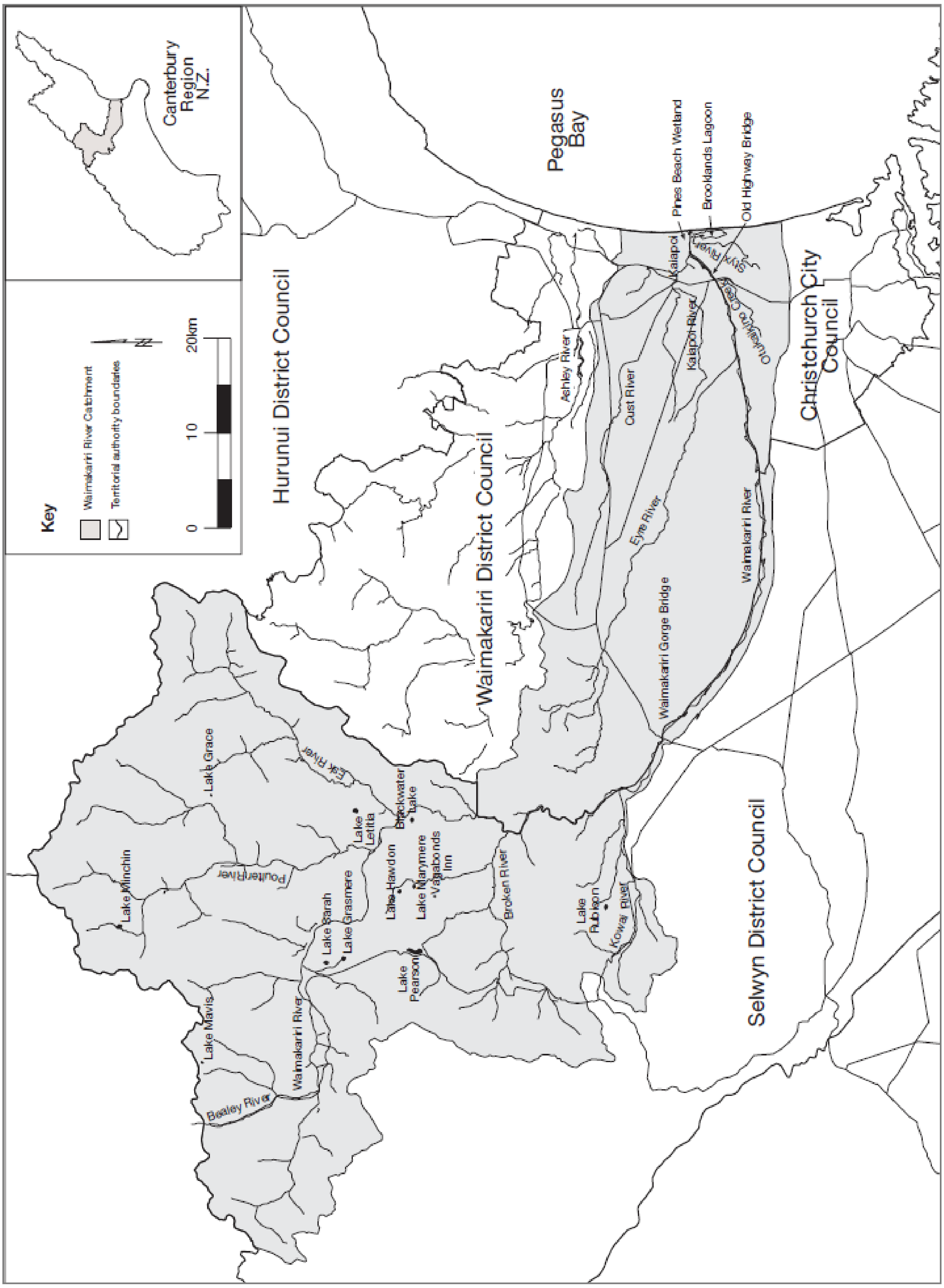


Figure 2.1 The Waimakariri River catchment (ECan, 2011b with permission).

2.2 Climate

Five principal influences determine the climate of the Waimakariri River catchment (ECan, 2008; North Canterbury Catchment Board [NCCB], 1986);

i. The upper basin is characterised by westerly wind conditions, which often bring abundant rainfall with high frequency but low variability. Mean annual temperatures have a small range. Northwest winds can lead to strong gales at times and snow may persist for long periods over winter.

ii. In the lower basin, easterly conditions are predominant, resulting in moderate amounts of rainfall with high variability and a large range of annual temperature. Rainfall is slightly higher in winter compared to other seasons. Summer northwest Föhn winds can raise temperatures to 30 degrees and above. Frosts are frequent in winter, but snow cover is infrequent.

iii. Orographic effects result in high annual rainfall and cold winters. The conditions vary with altitude and exposure. Rain from the altitudinal effect is highest during frontal disturbances with intense cyclones, bringing moist subtropical air from the northern Tasman Sea.

iv. Subtropical depressions can bring occasional heavy rainfall from the north east, causing significant flooding of tributaries, such as the Esk and Kowhai rivers.

v. The Southern Annular Mode, a climate pattern circling the Antarctic, also has strong influences on the South Island of New Zealand by influencing air pressure patterns and associated winds and rainfall (National Institute of Water and Atmospheric Research [NIWA], 2006).

2.3 Land use

High production pasture (27 %) dominates the land use of the Waimakariri catchment, followed by forestry (21 %), semi-natural valley, mid-altitude tussock (17 %) and high country snow tussock (16 %). Only 6 % of the catchment is used for low production pasture, and the gravel riverbed occupies 5 % of the total catchment area. Forty-six percent of the total catchment lies in steep mountainous areas of nearly 2000 m heights (Logan, 2008). The upper reaches of the river flow through an old glacial valley. The end-of-summer snowline lies at 1800 m, whereas midwinter snowlines are at approximately 600 m (NCCB, 1986). Urban centres within the catchment area include Rangiora, Kaiapoi, Woodend, Belfast and Oxford (ECan, 2011b).

2.4 River hydrology and flow statistics

The Waimakariri has several sources of water: (i) rainfall, (ii) snow and ice melt, (iii) aquifer resources below the Canterbury Plains, and (iv) numerous lakes and wetlands. The river hydrology is, however, mainly influenced by the climatic conditions of the upper basin (NCCB, 1986). Precipitation in 70 % of the high country catchment contributes to more than 90 % of the river flow, while snowmelt has previously been shown to contribute approximately 8 % to the mean annual flow in the Waimakariri River (Kerr, 2013). Annual rainfall varies significantly across the catchment area. While measurements record up to 5950 mm close to the Main Divide, only an average rainfall of 600 mm is measured on the Eastern Plains, near the river mouth (Logan, 2008). Large floods in the Waimakariri River are mostly a result of heavy rainfall in the upper catchment area, near the Main Divide (NCCB, 1986). Spring time generally produces higher flows due to snow and ice melt in the high country, contributing to 30 % of the flow (Gray, Scarsbrook, & Harding, 2006), and late summer produces the lowest flows of the year. This results in strong seasonality of the observed flows. Nevertheless, flood and low flows have been recorded at any time during the year, resulting in low predictability.

There are several discharge recording stations along the Waimakariri River; the Old Highway Bridge (OHB) gauging site has the longest and best quality record[2]. The mean annual flow of the Waimakariri River is 119 m^3/s (Table 2.1), with fluctuations ranging from low flows as low as 25 m^3/s and flood flows potentially exceeding 4,000 m^3/s (cf. historical accounts in Cowie, 1957). Figure 2.2 shows the annual maximum discharge recorded at the Waimakariri OHB site starting from the year 1930. The highest discharge at this site was recorded in 1957, with a flow of 3990 m^3/s; although accounts of early settlers indicate that even higher floods occurred within the last century (Cowie, 1957; Logan, 2008). The highest discharge measured since the installation of an automated recorder in 1967 at the OHB site has been 2835 m^3/s in 1979. Yearly flood flows average 1500 m^3/s (ECan, 2011a; Table 2.1).

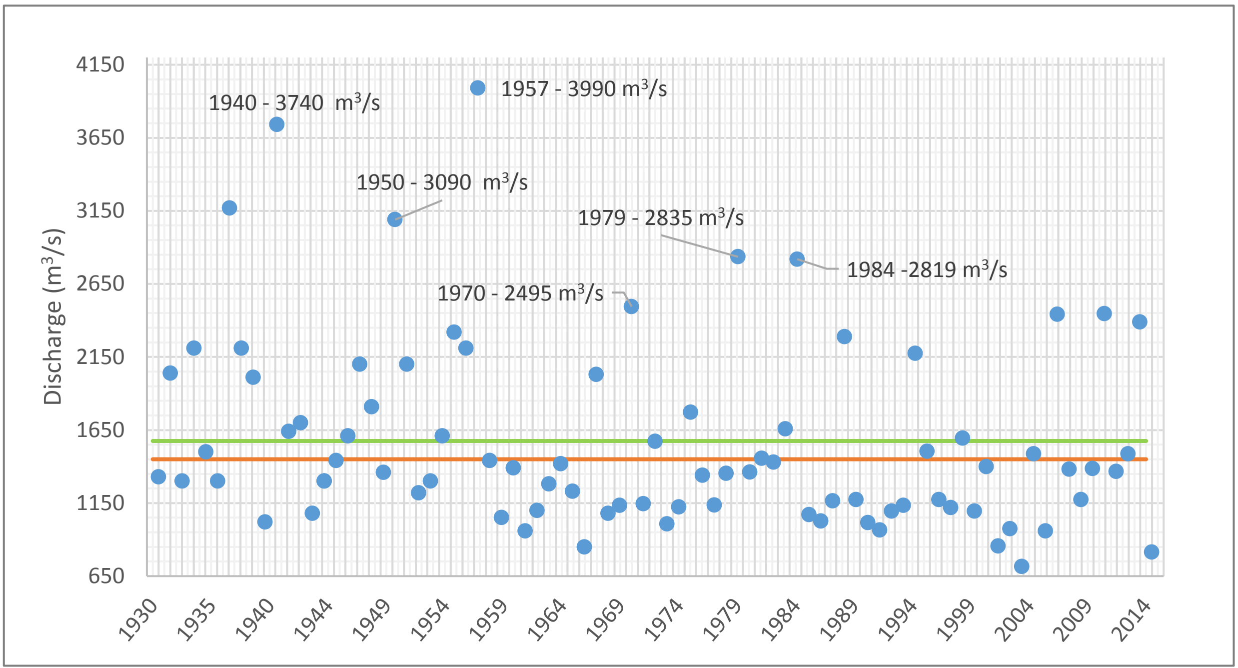


Figure 2.2 Annual maximum discharge at Waimakariri OHB (1930-2014). The orange line represents the mean annual flood level (1967-2015); the green line represents the mean annual flood level (1930-2015).

[2] For a detailed review of the history of hydrometric measurements in New Zealand and the quality of hydrometric data, refer to Appendix A.

Low flows occur most frequently during late summer through to April. The longest period of low flow was recorded in 1971 with 71 consecutive days of discharge below 40 m^3/s. The variation in mean annual flow as measured at the OHB site is visible in Figure 2.3. A summary of the most commonly used flow statistics is given in Table 2.1.

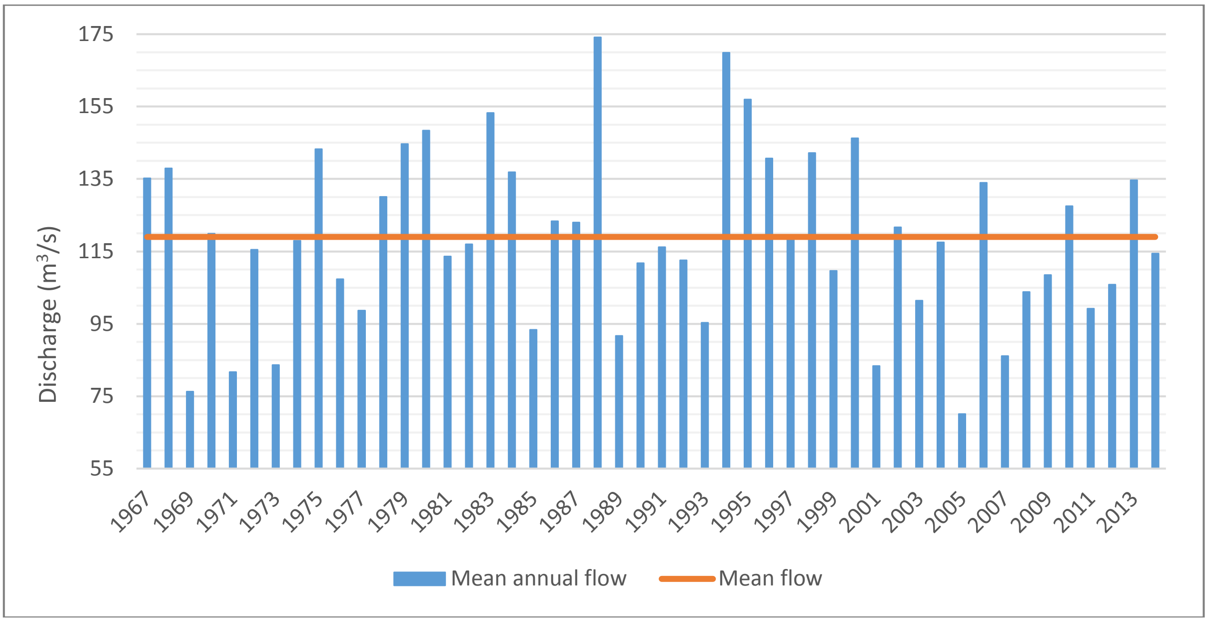


Figure 2.3 Yearly variations in the mean annual flow. The orange line represents the mean flow from 1967-2014.

Table 2.1 Summary of commonly used flow statistics. Waimakariri River at OHB. [a] data from 1930-2015, [b] data from 1967-2015.

Flow statistic	Discharge
Mean flow [b]	119 m^3/s
Mean annual flood	1575 m^3/s [a], 1450 m^3/s [b]
Median flow [b]	87 m^3/s
Mean annual minimum flow [b]	37.88 m^3/s
7d MALF [b]	39.18 m^3/s
FRE_3 [b]	18 per year[3]
Highest flood on record	3990 m^3/s in 1957 [a], 2835 m^3/s in 1979 [b]
Lowest instantaneous flow on record	22.03 m^3/s in 1971

2.5 History of flood protection

Early European settlers had not previously been confronted with a large, unpredictable braided river on an alluvial fan, such as the Waimakariri River. The seriousness of the threat posed by the river was soon recognised. Early accounts of the settlement history describe the frequent inundation of then Kaiapoi

[3] FRE_3 is the mean annual frequency of flow events > 3 time median flow

Island during flood flows. Indeed, the location chosen for the establishment of Christchurch and other surrounding towns was unfortunate. It is now known that the floodplain stretched out as far as to Te Waihora/Lake Ellesmere in the last few thousand years (NCCB, 1986).

By the late 19th century, flood control measures, such as banks, had been established to reduce the flooding risk. However, as the account of Samuel Butler (n.d.) describes, many of the early measures were unsuccessful:

> *"If it eats about a hundred yards more of its gravelly bank in one place the river will find an old bed several feet lower than its present. This bed will conduct it into Christchurch. Government had put up a wooden defence, at a cost of something like two thousand pounds, but there was no getting any firm standing ground, and a few freshes carried embankment, piles and all, away, and ate a large slice off the bank into the bargain; there is nothing for it but to let the river have its own way" (Logan, 2008, p. 9).*

As a result of the many flooding events observed between the 1860s and 1920s, the Waimakariri River Improvement Act 1922 gave responsibility to the Waimakariri River Trust to find a solution to the flooding problem on both the north and south banks of the river. Hays No. 2 Scheme, named after the principal engineer, was suggested as a comprehensive scheme to control the river to the fullest extent possible. This design included cuts in the river's course, and the construction of groynes and stopbanks that would allow for the passage of 4250 m^3/s, the assumed maximum discharge at the time (ECan, 1991; Logan, 2008). Flood events in 1940, 1950 and the largest recorded in 1957 (Figure 2.2), demonstrated that Hays No. 2 Scheme needed major upgrades. The previous events, albeit lower than the design discharge of the stopbanks, resulted in breaches and significant flooding of surrounding areas. This was partly attributed to the effects of gravel accumulation within the river channel (ECan, 1991). The subsequent 1960 scheme was aimed at providing protection for a 4730 m^3/s flood event. The scheme included improvements to the existing stopbanks and extensive gravel mining to reduce gravel aggradation in the channel. The largest floods (Figure 2.2) since the commencement of the upgrades (1970, 1979, 1984) were successfully contained (ECan, 1991).

Since then, discussions about the flood risk to the areas surrounding the Waimakariri River and Christchurch have been ongoing (ECan, 1991). Suggestions included the containment of a 1 in 10,000-year flood event (0.01 % annual exceedance probability [AEP], estimated 6500 m^3/s), justified by the high economic loss should such an event occur. For that purpose, the *Waimakariri River Flood Protection Project* is aimed at upgrading the existing flood protection measures along with the construction of a secondary stopbank at the southern bank of the river (ECan, 2013).

2.6 Waimakariri River values

The braided planform of the Waimakariri River makes it a highly dynamic and high-energy environment. The variability in discharge regime and sediment flux are ideal processes for the creation and maintenance of spatially complex and temporarily variable microhabitats (Gray et al., 2006). The extensive river floodplain is equally valuable in terms of biodiversity and ecosystem functioning. Therefore, the Waimakariri River and its floodplain (i) provide valuable habitat for many aquatic and terrestrial flora and fauna, some of which are classed as endangered or threatened with extinction, and (ii) significantly constitute economic, recreational and cultural values.

2.6.1 Flora

The 67,500 ha of indigenous forest in the Waimakariri catchment are dominated by mountain beech (*Nothofagus*) species. Above the timberline (ca. 1250 m) flowering shrubs, mosses, lichens and other plant species are found in the area's extensive screes. Twelve species of plants are either classed as rare or endangered in the upper catchment. The Castle Hill buttercup (*Ranunculus paucifolius*), for example, is an endemic species that has previously been threatened with extinction (Logan, 2008). Whipcord hebe (*Hebe armstrongii*) and climbing everlasting daisy (*Helichrysum dimorphum*) are two other endangered plant species only found in the Waimakariri basin (Biodiversity Strategy Advisory Group, 2008). *Olearia adenocarpa* is a recently discovered rare dryland shrub that is found on the margins of the Waimakariri River (Department of Conservation [DoC], 2007). On the other hand, there are several non-native, invasive species that have successfully established themselves within the catchment, or even on the floodplain. These species include wilding conifers (*Pinus* spp.), willows (*Salix* spp.) and lupins (*Lupinus* spp.). (Biodiversity Strategy Advisory Group, 2008).

2.6.2 Fauna

In terms of avifauna, the braided rivers of the South Island of New Zealand are exceptional and provide high value habitat. Many of the birds found here have adapted to the highly dynamic nature of the braided rivers and occupy unique habitat niches, which are only found in these environments (O'Donnell, 2004). Some of these adaptations include specialised morphological features (e.g. the laterally recurved beak of the wrybill), set migratory patterns and specialised foraging behaviour (O'Donnell, 2000). A number of endangered and rare species occupy the Waimakariri River as key breeding or foraging habitats. Four threatened species have been identified in a recent bird survey (Jolly, 2015): wrybill (*Anarhyncus frontalis*), banded dotterel (*Charadrius bicinctus*), black-billed gull (*Larus bulleri*) and black-fronted tern (*Sterna albostriata*). The black-fronted tern only breeds on eastern South Island braided rivers and due to habitat decline, the population consists of only < 10,000 individuals (O'Donnell, 2000). Wrybill are equally dependent on shingle islands that are found between braids.

Based on previous surveys, the Waimakariri River was classed as an outstanding habitat (cf. criteria used by O'Donnell, 2000 in Appendix 1) due to the presence of threatened species in high numbers.

Apart from supporting a highly valued sports fishery, the Waimakariri is also a significant habitat for many native fish species. NCCB (1986) extensively reviewed the species commonly found in the Waimakariri River and its associated wetlands. Some significant species are:

(i) lower river migrants: Inanga (*Galaxias malculatus*), and the common bully (*Gobiomorphus cotidianus*)

(ii) river system migrants: longfinned eel (*Anguilla dieffenbachia*), shortfinned eel (*Anguilla australis*), koaro (*G. brevipinnis*), and torrentfish (*Cheimarrichtys fosteri*)

(iii) river system inhabitants: common galaxias (*G. vulgaris*), alpine galaxias (*G. paucispondylus*), and the upland bully (*G. breviceps*)

Some of these species, e.g. koaro and torrentfish, are predicted to decline in the future (Goodman et al., 2014).

2.6.3 Economic values

The total economic value includes market and non-market values, which people derive from the river. While some values, such as irrigation use, can be quantified with ease, the valuation of other uses poses challenges, especially the valuation of non-use values (e.g. the value derived from the knowledge that the resource is managed well). Kerr, Sharp, and Leathers (2004) used postal surveys and interviews of Waimakariri River households to estimate that the protection of instream flows in the Waimakariri River was valued between NZD $ 11 million and NZD $ 30 million at the time of the study in 2004, indicating that residents place a significant value on the protection of the river and its amenities. An older survey by Sheppard et al. (1993) indicated that the benefits of improved water quality in the Lower Waimakariri River would yield a value of NZD $ 94.4 million, based on willingness to pay by Canterbury residents.

2.6.4 Recreational values

The Waimakariri River supports many recreational activities, partly due to its close proximity and easy access to recreationists from urbanised areas. Salmon, trout, mullet and whitebait support the significant use of the river by anglers. Swimming, sailing, jet boating and kayaking are also popular activities along the river. The Waimakariri Gorge is a key kayaking destination and is a destination in the annual Coast to Coast multi-sport race (ECan, 2011b). Other common activities include tramping, walking, sightseeing and bird watching (NCCB, 1986).

2.6.5 Cultural values

One of the primary hapū of Ngāi Tahu, Te Ngāi Tūāhuriri Rūnanga, is located within the Waimakariri catchment and has strong cultural associations with the river. Especially Kaiapoi is a culturally significant area of importance, as it once was a major trading centre for further travel into the South Island. Ngāi Tahu have a strong relationship with the river through wāhi tapu (sacred places) and wāhi taonga (treasured places and things), including Pā Tawhite (ancient pā sites), Urupā (burial sites) and mahinga kai. Over generations, Ngāi Tahu developed mahinga kai traditions based on local seasonal patterns and lifecycles of flora and fauna. Mahinga kai is a key value that is central to the culture and identity of mana whenua (customary authority). It described the practices that link mana whenua to the environment through interests in traditional foods and natural resources, and the places where these are obtained (Goodwin, 2011; Te Rūnanga o Ngāi Tahu, 2015).

2.7 Summary

This chapter served to introduce the study area for this thesis. It identified the Waimakariri River as a significant river for flora, fauna, economic activity, recreation and cultural heritage. The next chapter introduces the theory of frequency analysis with a comprehensive literature review, and thus marks the beginning of Part I of the thesis.

Chapter 3

Literature review part I: Frequency analysis of extreme events

This chapter introduces frequency analysis as a method for the estimation of hydrological extremes. The approaches to fitting frequency distributions to empirical data for individually gauged streams, including sampling methods, parameter estimation methods and goodness of fit methods are discussed. The annual maximum series (AMS) and partial duration series (PDS) methods for sampling from the streamflow record are compared and each approach is discussed in detail. Probability distributions associated with frequency analyses are also introduced. The literature review includes an overview of previous design estimates for New Zealand, and particularly the Waimakariri River. The concepts introduced in this literature review will be explained in greater detail in Chapter 4.

3.1 Introduction

The frequency analysis of extreme events is a commonly used method in hydrology to infer the probability of occurrence and magnitude of hydrological extremes from historical stream flow data (Keast & Ellison, 2013; Pearson, 1992). The use of stochastic methods allows the estimation of hydrological extremes and their frequencies beyond the observed record, and improves the reliability of estimates within the recorded period (Pearson & Davies, 1997). Such probability is commonly stated as the average length of time between events and referred to as the 'return period' or 'average recurrence interval' (Kidson & Richards, 2005). It is a statistical measure, describing extreme events as a sequence of independent, random trials. Therefore, the terms 'return period' and 'recurrence interval' do not imply a regular, reliable predictor in time but are rather related to the probability of occurrence. For the analysis of frequencies, three main steps are necessary: (i) sampling of extreme events, (ii) choice of an appropriate model, which requires knowledge about the underlying distribution of the events, and (iii) parameter estimation (Kidson & Richards, 2005). A probability distribution is fitted to the sample data from the observed historical record, from which a probability of a design event can then be extrapolated (Keast & Ellison, 2013). As straightforward as this approach may seem, there has been no scientific consensus about the ideal method for sampling original data for extreme events, or the choice of distribution fitted to the data.

3.1.1 Annual maximum/minimum series

The AMS is composed of the single greatest event for each year of observation as the basis for the time series (Langbein, 1949). Therefore, the number of years on record dictates the data series length. This approach is often used as it increases the chances that chosen events are independent of each other. Furthermore, annual series are easily extracted and are directly related to the return period of the design events. The choice of AMS also simplifies the choice of distribution fitted to the data, as AMS usually conform to theoretical distributions (Keast & Ellison, 2013). Concurrently, the literature is extensive on the shortcomings of the AMS. As only one hydrological event per year is included in the series, other significant stream flow records that are not the highest of the year are excluded. At the same time, the series may include small maxima for other years. The second highest event of the year might outrank other chosen annual events. By choosing this approach, not only is the series relatively short, but it might also lead to a systematic underrepresentation of more frequent, smaller events (Keast & Ellison, 2013; Mohssen, 2009). The estimation of small to intermediate extreme events can be equally important, especially for calculating expected cost variability during a year or calculating the vulnerability to damage during only part of the year.

3.1.2 Partial duration series

To overcome the disadvantages that are inherent to AMS, the use of PDS is often quoted. In the PDS, independent flows that exceed the *a priori* set threshold (x_0) are selected from the discharge record (Ashkar & Rouselle, 1987). Therefore, some years may produce more extreme events than others. Some years will produce no significant events at all and thus, the total number of events sampled from the record is random. Especially when the historical stream record is short, the PDS can produce a larger series by using multiple events per year as the basis for the frequency analysis. Such a method ensures a more representative sampling procedure of extreme values (Silva et al., 2014).

However, the PDS approach is not without shortcomings, due to the lack of guidelines surrounding its proper application (Lang, Ouarda, & Bobée, 1999). One of the reasons why PDS are seldom applied is the uncertainty of independence and the associated choice of threshold levels or truncation value (x_0). Not only does the truncation value directly influence the sample size, but the assumption of independence of the extreme events is in question. A high threshold level decreases the number of events chosen from the stream record, but a low threshold increases the chance that events are not independent. A previous event may set the stage for another, e.g., antecedent soil moisture conditions affect the generation of subsequent flooding events (Keast & Ellison, 2013; Langbein, 1949). Some of the various guidelines that exist in the literature for setting the threshold value for the analysis of flood flows are as follows:

- The National Environment Research Council [NERC] (1975) suggested a minimum time interval of three days, in which the flow between peaks must fall below at least two thirds of the earlier peak.
- Ashkar and Rouselle (1987) stated that the threshold should be fixed high enough as to validate a Poisson assumption. In theory, a threshold that allows no more than three exceedances per year should achieve stochastic independence, in which case the antecedent catchment conditions should theoretically not influence subsequent flood generation.

- Malamud and Turcotte (2006) proposed that time intervals between selected peaks should be 7-60 days.
- Svensson, Kundzewicz, and Maurer (2005) chose thresholds depending on catchment size. According to this criterion, events in the Waimakariri catchment (3,654 km^2) should be at least five days apart.

To overcome this disadvantage, it can be useful to supplement any threshold selection with visual confirmation of the peaks within the time series. While hydrological series are not entirely stochastic processes (e.g. influences of meteorological processes, such as the Southern Oscillation pattern, e.g. Mosley, 2000), there is a general consensus that a high enough truncation value can be selected to ensure the premise of independence.

3.2 Fitting a distribution

Distribution candidates for the use of frequency analyses are often either (a) two-parameter models or (b) three-parameter models (Kidson & Richards, 2005). Two-parameter models are characterised, as the name suggests, by two parameters; location and shape, employing the mean and variance of the sample population. The biggest advantage in using a two-parameter model is its simplicity in application. The Extreme Value Type I (EV1 or Gumbel) distribution and the log-normal (LN) distribution fall into this category. Three-parameter distributions on the other hand, are characterised by three parameters; are location, shape and scale, using the mean, variance and skewness of the distribution. The log Pearson III (LP3) and Generalised Extreme Value (GEV) models fall into this category (Kidson & Richards, 2005). Mathematically, the more parameters are used in a function, the easier will the fitting of empirical distributions be. However, statistically, the number of moments or other statistics needed for describing the parameters increases with the number of parameters. An increase in the order of moments for such estimation decreases the reliability of moments and thus the reliability of the parameter estimated from it. Consequently, the selection of the distribution function is a compromise between the ease of fitting a distribution and the reliability of the estimated parameter (Yevjevich, 1972a). The choice of the model is empirical, with the result that a large variety of models can be utilised to describe the parent distribution based on the sample at hand (Griffiths, 1989). While each model comes with its merits and drawbacks, selection has to be based on the specific sample data, as the choice of the underlying frequency distribution can have a marked effect on quantile estimation. This is especially important for the estimation of extreme events, as they lie in the tails of the distribution (Ware & Lad, 2003).

3.2.1 Annual maximum series

The AMS employs a cumulative distribution function for modelling the yearly extreme flows. The AMS approach achieved its popularity in application due to Gumbel's extreme value theory, stating that the largest peak in a given year has special statistical properties and must therefore be of one of three types

(Extreme Value distribution I, II and III) (NERC, 1975). The statistical groundwork for this theory was undertaken by Fisher & Tippett (1928) (as cited in Stedinger et al., 1993), however, it was only through Gumbel's extensive hydrological applications that EV distributions were adopted as the standard distribution for frequency analyses of floods (NERC, 1975). The EV1 distribution is most often employed on theoretical grounds. It is characterised by a positive skew, with most values in the lower ranges and few values in the right-hand tail, representative of the behaviour of many rivers (Pearson & Davies, 1997). The shape parameter (κ) of the distribution determines its type (κ = 0 → Type 1, κ < 0 → Type 2, κ > 0 → Type 3)(Pearson & Henderson, 2004). NERC (1975) contested Gumbel's arguments for using exclusively GEV distributions on theoretical grounds, and rather proposed using a distribution as dictated by the sample data. Another family of distributions commonly used, and also recommended is the Pearson Type III (P3) distribution (Foster, 1924 as cited in Kirby & Moss, 1987), utilising the sample mean, standard deviation and skewness of the sample data (Kirby & Moss, 1987). The Gamma distribution[4] has been recommended for general use in the United States (within the Water Resource Council Guidelines) due to its flexibility in usage (Kirby & Moss, 1987). On the other hand, in order to eliminate skew from the sample data, the log-Normal distribution (Hazen, 1930 as cited in Kirby & Moss, 1987) is also often employed.

3.2.2 Partial duration series

The peaks-over threshold approach in frequency analyses uses two distinct probabilistic models: one for arrivals above the threshold in a given year, and a cdf for modelling the magnitude of events (Bhunya et al., 2013; Madsen, Rasmussen, & Rosbjerg, 1997). The most frequently assumed, and generally applied model for over-threshold arrivals of events is a Poisson model. Shane and Lynn (1964) first described the arrival of peaks over threshold in a given year by a Poisson process. Here, the number of events occurring in a year, *x*, of the variable *X* (annual number of occurrence of a given event), follow a Poisson process, implying independence of occurrences (Beguería, 2005). At the same time, numerous studies have shown that this general assumption of Poisson arrivals first requires validation. The studies by NERC (1975) and Cunnane (1979) tested the negative-binomial (NB) model as an alternative to Poisson arrivals, as it allows for higher variances. Ben-Zvi (1991) also showed that the NB model provided a better fit to the sample data. Önöz and Bayazit (2001) tested the binomial model, comparing it to the performance of Poisson and NB arrivals, but the results did not clearly identify a superior model.

Often, for the probability distribution of the peaks over the threshold (or exceedance magnitudes), an Exponential[5] or Generalised Pareto (GP) model[6] is assumed (Ashkar & Rouselle, 1987; Claps & Laio, 2003; Pearson & Davies, 1997). The justification for the GP distribution comes from the

[4] a special case of the Pearson Type III distribution
[5] exponential decay compared with likelihood of larger events
[6] contains the Exponential distribution as a special case

second theorem of extreme value theory, stating that the asymptotic tail distribution of a random variable *X* is of this family of distributions (Silva et al., 2014). The Poisson - GP assumption for modelling PDS is seen as the most viable and useful alternative to using AMS. However, as is the case with the general assumption of Poisson arrivals, the literature has explored several other models to describe exceedance magnitudes (e.g. Bhunya et al., 2013; Bhunya et al., 2012; Guru & Jha, 2015), some of which yield better results for estimating design floods.

3.3 Parameter estimation

One of the first steps after choosing an appropriate probability distribution is the testing or estimation of parameters of the underlying distribution, *F(x)* (Claps & Laio, 2003; Pearson, 1991). Parameters of a distribution are estimated through the analysis of the sample data, under the assumption that the selected sample data is representative of the population. The random outcome of the sample data is assumed to have been generated by stochastic processes, producing the underlying distribution of the events. Therefore, the aim of parameter estimation is finding the parameter values characterising the unobservable distribution, assuming that the probability distribution is dependent on the descriptive measures or parameters (Ware & Lad, 2003). The underlying probability distribution influences the goodness of parameter estimation, and in turn, the fitted distribution (Singh & Strupczewski, 2002).

There are several methods for estimating parameters, such as the method of least squares, method of maximum likelihood (ML), method of moments (MOM) and linear moments (L-moments); all of which have been extensively applied in the literature. For a simple analysis of linear regression, the parameters are most often estimated using the method of least squares. The least squares method chooses parameters based on the minimum difference between observed and predicted values. However, for many procedures used in frequency analyses, such as logistic and Poisson distributions, the least squares method is not sufficient for parameter estimation.

The ML technique is often cited in the frequentist literature for parameter estimation. It is a consistent technique, especially when fitting parameters for sample data that include different types of data[7]. Furthermore, it is a robust technique, able to deal with data that does not adhere to the statistical assumption of normality. The ML estimate of a parameter is the value of the unknown parameter that maximises its likelihood of occurrence, or in other words, it is a value that makes the sample data the most probable. However, the ML technique is problematic when fitting parameters for events with low expected counts, such as flood flow data (Kidson & Richards, 2005).

The MOM is based on the statistical moments of the sample data, such as the mean (1st moment), variance (2nd moment), skew (3rd moment), and kurtosis (4th moment) (Kidson & Richards, 2005). In its simplest form, the MOM technique aims to equate sample moments with theoretical

[7] e.g. different flood producing phenomena, each with different underlying distributions

moments to find the parameters of the underlying distribution. While it is easy in its application, it is seldom used because of its inefficient estimates.

Alternatively, L-moments can be used for the estimation of sample data. Hosking (1990) first introduced L-moments and they have since been recommended for hydrological frequency analyses (e.g. Stedinger et al., 1993). This method is analogous to the method of moments, using dispersion, skewness, location, and kurtosis to provide a description of the distribution. However, L-moments are constructed using linear combinations of the ordered sample data values (Kidson & Richards, 2005; Pearson, 1991; Vivekanandan, 2015). One major advantage of L-moments is the avoidance of non-linear transformation of sample data[8], which can often lead to poorer parameter estimation due to distortions. The effect of data outliers is therefore reduced when using L-moments, compared to other conventional methods (Pearson, 1991; Pearson & Davies, 1997).

3.4 Flood frequency analysis

Unsurprisingly, the debate whether to use AM series or PDS for sampling flood events from the historical record extends as far back as the pioneering works of extreme value theory by Fisher & Tippett (1928)(as cited in Stedinger et al., 1993). One of the earlier comparative works was done by Cunnane (1973), whose results were also published in the British *Flood Studies Report* (NERC, 1975). Cunnane (1973) investigated which of the two methods was preferable for flood frequency analysis, assuming a Poisson-Exponential model for the PDS and a Gumbel distribution for the AMS, and fitting parameters with ML estimation. His results showed that for return periods greater than 20 years, assuming the PDS contains at least 1.65 exceedances per year, the PDS estimator has a significantly smaller variance than the AMS estimator. While these results were published in the NERC Report (1975), no mention was made about the obvious recommendation for the use of the PDS over the AM series within the report. The work of Cunnane (1973, 1979) was further investigated by Tavares and da Silva (1983) and Rosbjerg (1985). While both studies agreed with the general conclusions of Cunnane (1973, 1979), his estimations were found to underestimate (overestimate) the quantile variances of the PDS model, when the average number of arrivals over threshold is greater (less) than two.

However, much of the work done in New Zealand on flood frequency analysis has followed the approach of the *British Flood Studies Report* (NERC, 1975), e.g. Beable & McKerchar (1982), and subsequently McKerchar & Pearson (1989, 1990), and more recently ECan (2011a). Beable and McKerchar (1982) chose not to use PDS as "*in certain circumstances estimations from an annual series can be more efficient statistically than those from a partial duration series taken from the same record*" (p. 17). Instead, the authors chose to extract annual maxima from catchments in New Zealand, split into more than ten regions, in order to estimate the mean annual flood at ungauged sites, where no recorded

[8] raising data to powers of 2, 3 and 4

data was available. Distributions used in this study were either the EV1 or the EV2. McKerchar and Pearson (1989, 1990) later argued that the geographic delineation of the regions was rather arbitrary and improved this method by using contour maps, based on an additional ten years of measured discharge data. Together with mean annual floods and dimensionless 100-year return period floods, the contour maps were (and still are) used as part of regional flood estimation in New Zealand. The conclusions of these publications were that EV1 distributions describe at-site flood frequency distributions well on an annual basis. Some records (47 of the 274) appeared to be EV2 or EV3 distributed; however, by extracting biennial records, the EV1 distribution fit sufficiently. In the conclusion, the authors mentioned that one of the assets of this method is the simplicity of its application.

Pearson (1991) later followed up on these results by plotting L-moment ratios for the estimation of distribution parameters. He concluded that contrary to earlier works, some areas, notably catchments in the east, are better described by three-parameter EV2 distributions. A number of years later, Madsen, Rasmussen, et al. (1997) published the first comparative study of AM series and PDS using New Zealand data. The authors partitioned the South Island into two separate regions, east and west, using annual rainfall as the selection criterion, as previously suggested by Pearson (1991). The results indicated that an Exponential distribution (PDS) was the best fit for western catchments, and the GP distribution was the best fit for the east, similar to results obtained by Pearson (1991) for EV1 and EV2 respectively, using AMS. Overall, Madsen, Rasmussen, et al. (1997) also concluded that the PDS model is generally preferred for at-site quantile estimation. As a follow-up on the results obtained by Pearson (1991), Connell and Pearson (2001) further investigated the EV2 tendencies of Canterbury rivers. The authors proposes an alternative to the EV2 distribution, the Two-Component Extreme Value (TCEV) distribution, which described the maxima of two independent EV1 distributions integrated in one flow record. This model suggests that there are two separate flood-producing phenomena, resulting in the flood record of such rivers. Each process is a Poisson process with an Exponential distribution. One of the distributions represents the 'outlier' larger flood events; the other represents the more frequent, smaller floods. Connell and Pearson (2001) concluded that the TCEV distribution was a better descriptor of the underlying parent distribution for many Canterbury rivers, compared with EV2 distributions. At the same time, L-moment ratios were used to identify distinct regions within Canterbury; South Canterbury rivers, with two-parameter tendencies; North Canterbury rivers with lesser two-parameter tendencies; and Main Divide rivers[9], whose predominant flood producing process comes from westerly storms and are described by EV1 distributions.

The first modern statistical flood frequency analysis for the Waimakariri River was produced by Stephen (1958) as part of a report on the significant flood event of 27 December 1957, which led to a

[9] to which the Waimakariri River belongs

stop bank breach. Stephen (1958) estimated the 1 % AEP, or 100-year event using an EV1 distribution, with the least squares method for parameter estimation. He employed an AM series, using data from 1929 to 1957. His estimate was 4290 m^3/s, including a 10% safety factor for future improvements on the stop banks.

A later report produced by Pearson (1988) established similar results. The at-site analysis of the OHB data assumed an EV1 distribution of the AMS data. Parameter estimation was done by probability weighted moments. However, it was found that the EV1 distribution did not produce an optimum fit. Therefore, the biennial series was extracted, as was later done by McKerchar and Pearson (1989, 1990). The 1 % AEP was estimated to be 3830 m^3/s, comparable to results achieved by Stephen (1958) 30 years earlier[10]. Pearson (1988) also suggested that the 1 % AEP would not significantly change over the next 30 years to come. The same calculations were used for the Waimakariri River in the *Draft Waimakariri River Floodplain Management Plan* by Griffiths (ECan, 1991) which were subsequently used for the *Waimakariri Flood Protection Project, Hydraulic Modelling* report[11] (ECan et al., 2007). The hydraulic calculations in this report are solely based on AEP estimates produced by AM series of the OHB site. The most recent report by ECan (2011a) assessing flood frequency of the Waimakariri River is an update of the previous nation-wide study done by McKerchar and Pearson (1989), with the benefit of an additional 22 years of data. The 100-year flood estimate is 4155m^3/s; however, this is also an estimate based on AMS, using an EV1 distribution and probability weighted moments. The report mentions that fitting an EV1 distribution at a site displaying EV2 tendencies will underestimate larger return periods. A more recent report by ECan (Steel, 2016) presented an investigation of various methods for obtaining flood frequency estimates for the Waimakariri River, including an analysis of historical data and a range of distributions.

However, there has been no study specifically addressing the use of PDS for the Waimakariri River to assess the probability of occurrence and magnitude of floods. This is particularly surprising, as a number of studies (e.g. Madsen, Rasmussen, et al., 1997; Mohssen, 2009) have suggested that PDS may be the better suited approach, especially when assessing higher frequency, lower magnitude flood events.

3.5 Low flow frequency analysis

In relative comparison with the extensive literature on flood frequency analysis, the literature on low flow frequency analysis has little to offer. However, the field of low flow hydrology is equally conflicted in terms of satisfactory methods for frequency analyses (Stedinger et al., 1993; Tallaksen, Madsen, & Clausen, 1997; Zelenhasic, 2002). In addition, the literature is conflicted on a set definition for the terms

[10] The inclusion of a 10 % safety factor to Pearson's results (1988) would equate to 4213 m^3/s.
[11] This report was produced to assess the benefits of a potential secondary stop bank on the south bank of the lower Waimakariri River.

low flow and *drought* (Smakhtin, 2001). While low flow refers to one component of a natural flow regime of the river, a drought usually results from below average precipitation for an extended period of time. Therefore, low flows are inherently related to drought; however, a drought is characterised by more than just low flow conditions (Lee & Kim, 2008; Pearson & Davies, 1997). Some of the causes for droughts are natural phenomena, while anthropogenic factors, such as direct water abstractions or artificial afforestation, can also play a large role (Pearson & Henderson, 2004; Smakhtin, 2001). By its simplest definition, a drought is therefore characterised by a condition of shortage of water that negatively influences one or more water uses (Pearson & Henderson, 2004).

Rather than using frequency analyses of low flows, an often employed method for the visualisation of low flows is a predefined low flow section (e.g. below mean annual runoff) of a flow duration curve (FDC). Such FDCs display the relationship between a discharge value and the time this discharge is equalled or exceeded in a given year. While FDCs easily show the whole range of flows at one glance, they only illustrated proportional frequencies of flows, irrespective of sequence of occurrence (Smakhtin, 2001). A low flow frequency analysis, on the other hand, gives an indication of the proportion of years a flow of interest is equalled or exceeded, and attempts to place a probability on the likelihood of occurrence (Stedinger et al., 1993).

The focus of the majority of studies concerned with low flow analyses or hydrological drought conditions has been in terms of streamflow deficits (Tallaksen et al., 1997). However, the general discussion is not centred around the use of AMS or PDS for sampling from the flow record, but rather on the use of the 'traditional' approach (what Stedinger et al., 1993 refer to as 'annual-event-based low-flow statistics') vs. the 'theory of runs', as developed by Yevjevich (1967).

The traditional approach typically samples the stream record for the minimum annual *n*-day average discharge. Often this *n*-day discharge is represented by the 1-day minimum, 7-day minimum or 10-day minimum, reflecting countries' different preferences (Clausen & Pearson, 1995; Tallaksen et al., 1997; Yevjevich, 1967). This approach is satisfactory when only one critical parameter, the duration or magnitude, is of interest. At the same time, it has to be acknowledged that low-flow events or droughts are characterised by multivariate processes, such as duration, intensity and deficit volume (Yevjevich, 1967). The traditional approach does not take into consideration that one river may have few but severe streamflow deficits, and another may display many small, yet prolonged streamflow deficits (Smakhtin, 2001).

Yevjevich's (1967) theory of runs (sometimes referred to as crossing theory) offers an opportunity for including the time of occurrence, duration and severity of a specific drought event and therefore defines a drought as a period of time, in which streamflow falls below a certain threshold level. Despite the similarity of definitions, this should not be confused with the discussion on sampling from the stream record, as the theory of runs can be applied with both, data from an AM series, and data from a PDS series. The runs theory employs a truncation level x_0, for the selection of the critical drought.

The streamflow deficit condition starts when the flow falls below the threshold and ends when the flow returns above the threshold. Furthermore, the following parameters can be defined (Yevjevich, 1967):

- The duration, d_i, also called the low flow spell or run length
- The severity of the deficit, also called the run sum (cumulative water deficit), s_i
- The magnitude, m_i, calculated as $\frac{s_i}{d_i}$
- The minimum flow of each deficit event

Figure 3.1 provides a visual representation of the parameters used in runs theory.

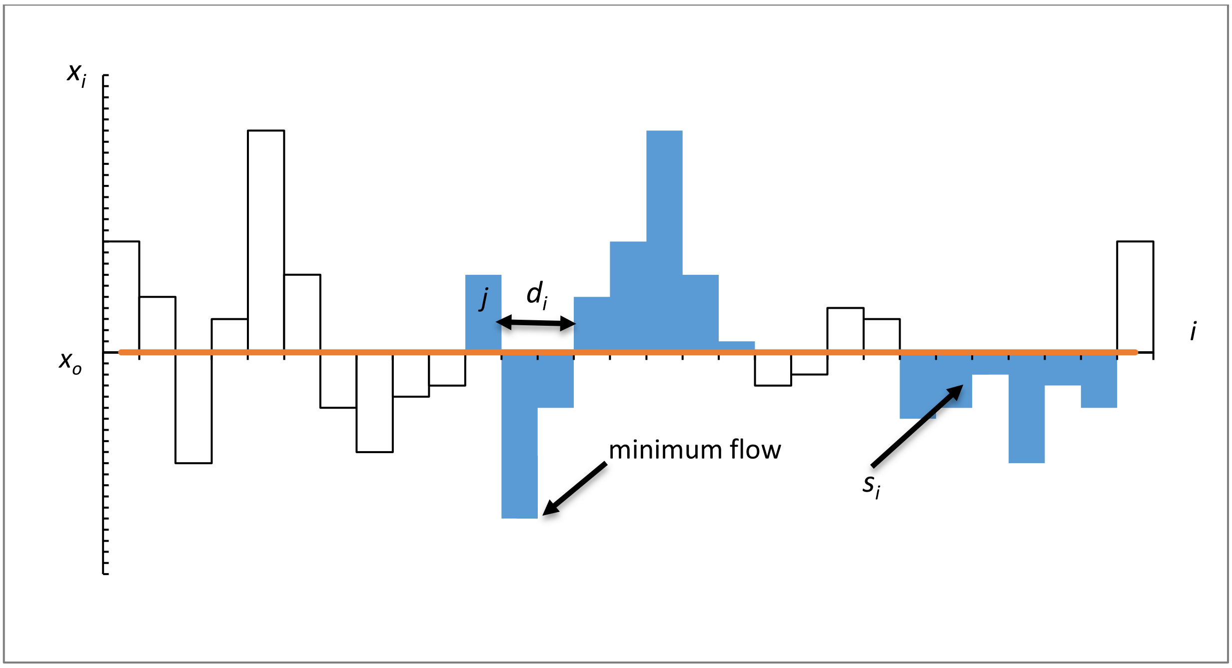


Figure 3.1 Definitions used in runs theory, where d_i is the duration or run length, s_i is the run sum for a discrete series, x_i. j is a selected event above the threshold. The minimum flow is indicated. The orange line is the chosen truncation value x_o.

Setting a threshold faces similar challenges, as the threshold selection for flood peaks. The purpose and region of the study should guide the selection of the threshold, for example, selection could be guided by minimum water requirements for in-stream ecology or by water abstractions for irrigation. While a flood peak might be easy to confirm visually, drought or low flow conditions have the added dimension of duration. In such a case, a drought condition can be interrupted by small but frequent above-threshold events (cf. event denoted by j in Figure 3.1); however, they do not indicate the return to normal flow conditions (Tallaksen et al., 1997). The flow deficits cannot be treated as independent events and require statistical pooling procedures.

Despite the introduction of the threshold idea into runs theory, many studies worldwide employ the AMS approach for the selection of the annual minimum deficit. For example, Clausen and Pearson (1995) used the runs theory in a regional drought study in New Zealand. The parameter of interest was the longest low flow duration of each year. The threshold for the truncation level was chosen as the mean flow and 75 % of the mean flow. The authors found that the three-parameter log-Normal distribution was the best fit to the data. In addition, Pearson (1995) used a variation of annual low flow minima (1-day, 7-day and 30-day mean flows) from catchments around New Zealand to analyse low flows and regional patterns.

Numerous other studies have explored the use of PDS in combination with runs-theory (Fleig et al.,2006; Kjeldsen, Lundorf, & Rosbjerg, 2000; Madsen & Rosbjerg, 1995; Zelenhasic, 2002; Zelenhasic & Salvai, 1987). Zelenhasic and Salvai (1987) used partial duration series assuming a Poisson distributed number of drought events and exponentially distributed drought duration for their Yugoslavian dataset. Madsen and Rosbjerg (1995) introduced a GP distribution for their Danish dataset. Kjeldsen et al. (2000) in turn, introduced a two component exponential distribution, and concluded that it was a valuable exceedance distribution in the PDS analysis of droughts.

Yet another approach is described by Önöz and Bayazit (2002), who used the traditional approach to low flow analysis (i.e. event-based low flow statistics, rather than runs theory), but sampled using PDS. The authors termed this approach 'troughs under threshold' as opposed to 'peaks over threshold' analysis.

3.6 Summary

This chapter concludes the literature review for Part I of the study, which examines approaches to estimating flood and low flow frequencies for the Waimakariri River. The literature review introduced frequency analysis as a method for obtaining flood and low flow quantile estimates. In addition, the two approaches for sampling events from the continuous stream flow record were contrasted. Afterwards, the general steps involved in the production of quantiles were introduced. The literature review also presented previous frequency analyses that have been undertaken in New Zealand, and specifically for the Waimakariri River. The next chapter will elaborate on the research design and will introduce further details concerning the model formulation for frequency analyses.

Chapter 4

Research strategy and model formulation

This study applies the partial duration series (PDS) method in comparison with the annual maximum/minimum series (AMS) method to generate flood and low flow quantile estimates for the streamflow record of the Waimakariri River at the Old Highway Bridge (OHB) site. An extensive literature review precedes this chapter. The literature review is part of the research and served as the guide for the formulation of the model processes to follow. This chapter proceeds by elaborating in detail on the discrete steps necessary to perform frequency estimates of floods and low flows, as depicted in Figure 4.1. The chapter following the *Research Strategy* contains the experimental results of flood and low flow frequency analyses performed for the study.

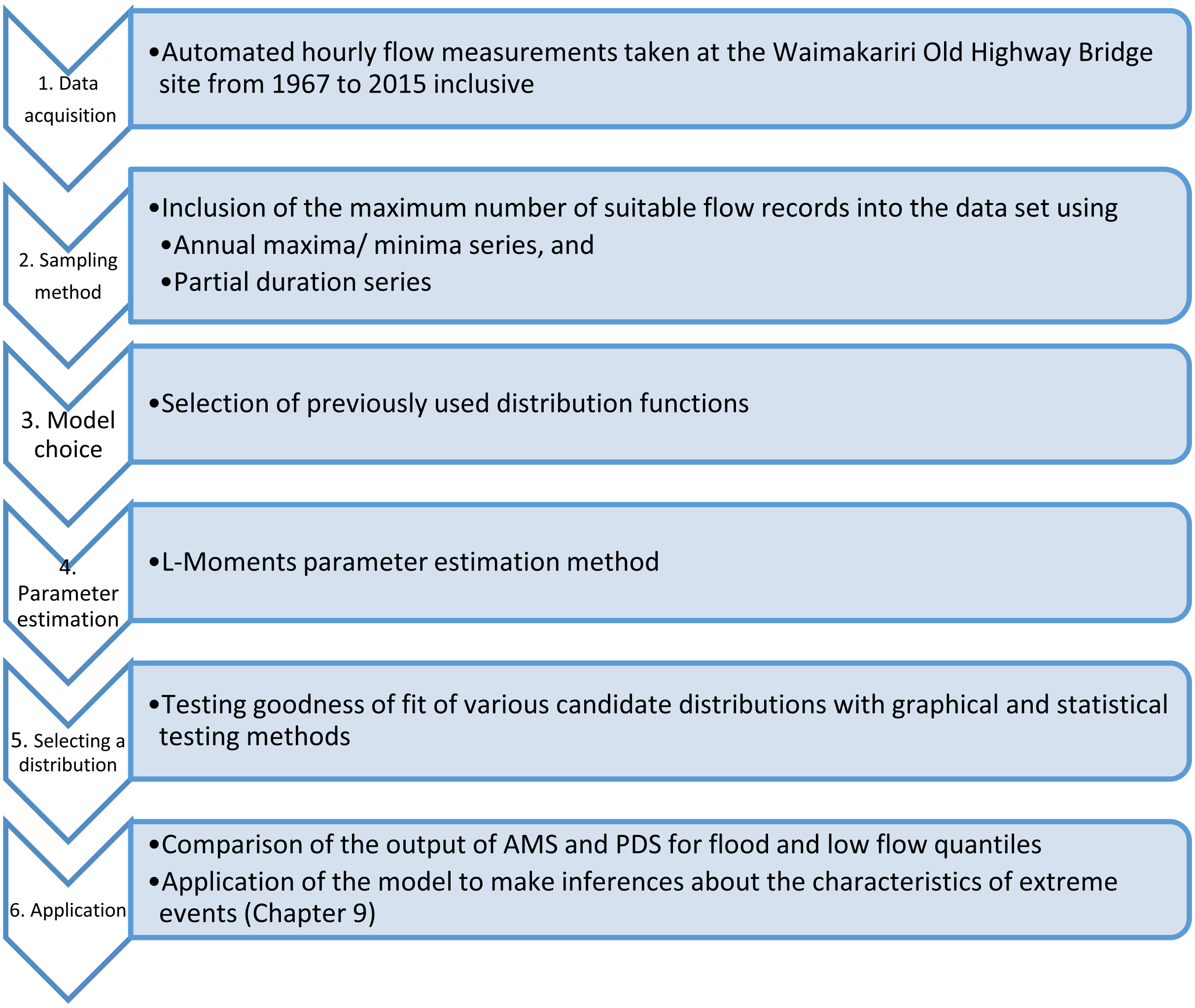


Figure 4.1 Schematic representation of the steps involved in the frequency analysis of extreme events.

4.1 Introduction

The goal of frequency analyses within this study is to obtain quantile estimates for extreme events. Suitable methods, as explored in Chapter 3, are applied to find the flood and low flow magnitude (*Q*) – return period (*T*) relationship, often referred to as the *Q - T* relationship. At the chosen OHB site, the aim is to estimate the F^{th} quantile of non-exceedance probability. The cumulative distribution function (cdf) $F_X(x) = P(X \leq x)$, also the non-exceedance probability, describes the probability that a value of a random variable *X* is equal to or less than *x*. It is the *F* % chance that in any year, discharge (Q_i) is less than x_i. Such probability is commonly expressed as a one in *n* year event, describing the average length of time between two events of a given size or larger.

4.1.1 Return period in the annual maximum series

As the annual series only considers the most significant event per year, the recurrence interval in the AMS is the average time interval in which an extreme event of selected magnitude x_i will recur as an annual maximum/minimum (Langbein, 1949). In the AMS, exceedances of extreme events are modelled using the cdf *F(x)* (Bhunya et al., 2013). The return period of such an event is calculated as the reciprocal of its probability of exceedance *P(X)* in one year, $T = 1/P(X)$, where $P(X) = 1- F(x)$, and is based on integers of years (Beguería, 2005). What is referred to as a 100-year return period (T_{100}) has a 1/100 chance or 1 % probability of exceedance per year. It is defined by $F(x) = 0.99$ (Pearson & Henderson, 2004; Stedinger et al., 1993). To calculate the probability that an event will occur or be exceeded at least once in the *m* year time interval, the following formula can be used: *p(*exactly *n* years until $Q_i \geq x_i) = 1-[1-1/T]^m$ (Beable & McKerchar, 1982; Pearson & Henderson, 2004). Therefore, the probability of a 100 year event being exceeded at least once in 100 years is in fact only 63 %, exceedance at least once in 10 years has a probability of 9.6 %.

4.1.2 Return period in the partial duration series

In the PDS, the recurrence interval is the average time between extreme events X_i, irrespective of the annual time interval that is used as reference in the AMS. The return period is rather based on time spans (Langbein, 1949), and has the form of:

$$1 - F\left(\frac{Q_T}{(Q_T \geq q_0)}\right) = \frac{1}{\lambda T}, \qquad \text{(Equation 4.1)}$$

q_0 being the chosen threshold and λ representing the number of peaks per year included in the series. While for larger events AMS and PDS approach the same value for calculated recurrence intervals, the difference in return periods is evident for smaller events. As multiple significant events above a set threshold can be chosen for the time series for the one year period in the PDS, the question of translating the exceedance probabilities and return periods into annual terms arises (Mohssen, 2009).

4.1.3 Translating the PDS into the annual domain

The assumption of a Poisson process for the arrival of events above the threshold in the PDS has been widely used in the literature due to its simple modelling application (Ashkar & Rouselle, 1987; Cunnane, 1979; Rosbjerg, 1985; Shane & Lynn, 1964). A Poissonian flood count was previously necessary for the translation of the return periods of the PDS into the annual domain. This requirement even resulted in the suggestion for the selection of threshold levels with the aim of obtaining an exceedance series described by a Poisson process (Ashkar & Rouselle, 1987). Stedinger et al. (1993) provide a formula which enables the estimation of annual exceedance probabilities from PDS, assuming a Poissonian arrival rate of flood peaks:

$$T_p = -\frac{1}{\ln(1-\frac{1}{T_a})}, \qquad \text{(Equation 4.2)}$$

where T_p is the return period obtained from the PDS and T_a is the annual return period given by the relationship:

$$1 - F_a(x) = 1/T_a. \qquad \text{(Equation 4.3)}$$

Later studies showed that the Poisson hypothesis is not essential and that in fact the binomial and NB distributions offer alternatives to the Poisson assumption (Ben-Zvi, 1991; Önöz & Bayazit, 2001). In this case, equations analogous to the one offered by Stedinger et al. (1993) have been suggested. A newly derived formula by Mohssen (2009) does not assume or require a Poissonian arrival rate for floods and can be applied with any value or distribution of λ, as long as $T_a > 1$ and therefore $T_p > \lambda$. The independence of flood events in the PDS is the only requirement. To obtain the annual return period of floods from a PDS, the following equation can thus be solved:

$$\frac{1}{T_a} = \lambda\left(\frac{1}{T_p}\right)\left(1-\frac{1}{T_p}\right)^{\lambda-1}. \qquad \text{(Equation 4.4)}$$

The application of this formula in Mohssen (2008, 2009) resulted in significantly higher annual return estimates in the lower ranges than with the application of the commonly used formula as given by Stedinger et al. (1993). This in turn means that the new equation produces lower design floods for smaller return periods when compared to the more commonly used equation. For higher return periods, both formulas approach similar design estimates. Overall, the new formula was shown to be more reliable for the translation of the PDS return period into the annual domain. Therefore, any subsequent quantile estimates using PDS are obtained in this thesis using the newly derived formula by Mohssen (2009).

4.2 Data acquisition

Since the structural changes of government funded authorities tasked with the oversight of New Zealand's water resources in 1989, records for water flows have been the responsibility of regional/district councils and regional branches of the National Institute of Water and Atmospheric Research (NIWA)(Mosley & McKerchar, 1989; Pearson, 1998). In the case of the Waimakariri River, the Canterbury Regional Council, hereafter Environment Canterbury (ECan), holds historical records of river flows dating back to 1930 (ECan, 2013). While the length of this record is short compared to European equivalents, the records of the Old State Highway Bridge site are one of the longest and best quality measurements in New Zealand (ECan et al., 2007).

Flow data

Prior to 1953 the OHB site was not operational. The data for annual maxima between 1930 and 1952 have been estimated from stage records at the Gorge Bridge site post-flood event using slope-area calculations, which take into account water level, cross-sectional area, slope of the channel bed and theoretical velocities using a standard water depth-flow velocity relationship table (Mosley & McKerchar, 1993; Ware & Lad, 2003). Additionally, the stage records from the Gorge Bridge between 1939 and 1945 are sparse at best (Young, 1990). An analysis of the Gorge-OHB relationship beyond 1956 is confounded by the fact that the Gorge Bridge stage measurement site was moved to a new site, 60 m downstream, in 1957 (NCCB, 1986). Therefore, an analysis of streamflow at OHB on the basis of the Gorge stage measurements, as was done by Young (1990), necessitates two non-linear models.

The first stage measurements at the OHB site between 1953 and 1966 were done with the aid of a Lea flow recorder by Lea Recorder Co Ltd[12] (K. Osten, personal communication, 13/08/2015). This chart recorder uses the mechanical input of the river level to record data on paper, in proportion to the signal. However, only maxima were gauged for discharge (Pearson & Henderson, 2004). This early period also contains six relatively low annual maxima about which little is known, which suggests that discharges were estimated from now lost rating curve information (Young, 1990). Continuous measurements began in 1966 and high quality automated gauging data is available from 1967 onwards (McKerchar, 1986). Currently, the cross-section of the OHB site is remapped to retain a high quality stage-discharge relationship, whenever flooding is suspected to have substantially altered the site (Ware & Lad, 2003). The data from 1967 onwards is regarded to be of high quality because of a relatively level

[12] http://www.gracesguide.co.uk/Lea_Recorder_Co, retrieved 14/08/2015

riverbed in the cross-section, uniform flow at the measurement site[13], 28 to 29 vertical measurement points for high flow gauging and a flat water surface after peak flow; all of which yield reliable quality data for the calibration and use of the rating curve (Young, 1990). For a more detailed description of the quality of the discharge measurements at the site, see Appendix A. Recent flows of the Waimakariri River at the gauging site can be publicly accessed on the Council's website[14]. The data provided by ECan is naturalised to remove the tidal influence on the measurement site.

For the purpose of frequency analysis, a minimum scale of hourly time intervals is needed. Any larger intervals may miss the record of significant peaks. The basis for the statistical frequency analysis in this study are records of hourly mean flow discharges from 1967 until 2015 inclusive at the ECan operated site as detailed in Table 4.1 (Walter, 2000):

Table 4.1 Site used for frequency analysis. NHD= National Hydrometric Reference Network; R= Regional database; CRC= Canterbury Regional Council (Source: Walter, 2000).

Site index	**River and site name**	**Map reference (NZMS260)**	**Catchment Area (km²)**	**Archive**	**Recording authority**	**Length of record**	**Comment**
66401	Waimakariri at Old Highway Bridge	M35:818547	3,210	R, NHD	CRC	48 years	Digital records available from 1967 onwards

While the recorded annual maximum stream flows of the OHB site extend back to 1930, earlier significant extreme events, especially floods, have been reported even pre-European settlement in Christchurch. These reports are only descriptive statements of the events that severely impacted on communities and no accurate estimates of the streamflow exist. Therefore, such historical events, albeit very informative, are not included in this study. Furthermore, as the full records dating back to before 1967 are not currently available in digital form, the records cannot be used in PDS sampling and are thus omitted from the frequency analysis.

4.3 Data sampling

The first step in the model development requires the extraction of flows from the historical gauging record. For the annual maxima or minima series, this is straightforward. The yearly maximum or minimum value is chosen to represent the single most extreme value of the year. This yields 49 values each for the annual maximum series and the annual minimum series. Additionally, for the analysis of low flows, several interpretations of the low flow have been included, such as the 7-day mean annual low flow, and the absolute minimum flow. One reason often cited in the literature advocating the use

[13] neither converging nor diverging
[14] http://www.ecan.govt.nz/services/online-services/monitoring/river-flows/Pages/river-flow-chart.aspx?SiteNo=66401

of the AMS is that events are very likely independent of each other. Indeed, many statistical methods inherently assume independence of sample data. Such independence cannot be assumed in cases when, for example, the maximum flood occurs in December and is followed by a maximum flood peak in January. The extreme events extracted from the flow record were thus visually inspected via flow hydrographs to rule out such occurrences.

4.3.1 Threshold selection for the PDS of flood flows

In the PDS approach, independence of events is the guiding principle behind the selection of the threshold level. When the average return period between extreme events is relatively long, it is plausible to assume that events are independent of each other (Ashkar & Rouselle, 1987). However, events often appear in clusters, showing strong autocorrelation (Beguería, 2005). Ensuring independence of data series is a complex task and no clear guidelines have been uniformly adopted in the literature (Chapter 3). Lang et al. (1999) outlined several methods commonly used for the selection of appropriate threshold values for sampling flood events. As is the case with the choice of distribution for the data set, each statistical decision should be based on the behaviour of the river in question itself. For the dataset of the Waimakariri River at OHB, various threshold levels were tested. Some of the guiding methods for determining a threshold level were:

(i) Mean number of events above threshold: Cunnane (1973) and subsequently Stedinger et al. (1993) suggested that PDS perform better than AMS in terms of variance when the average number of events is at least 1.65 per year.

(ii) Suggestions by NERC (1975): Flood events should be separated by at least 5 days plus the natural logarithm of the catchment area in square miles. Otherwise, the flows between two peaks must drop below 75% of the lower of the two peaks[15]. Therefore, the second flood peak must be rejected if:

$$\Theta < 5 \text{ days} + \ln(A/2.589) \text{ or } Q_{min} < (3/4) \min [Q_1, Q_2], \qquad \text{(Equation 4.5)}$$

where Θ is the time interval, A is the catchment area in square kilometers[16], and Q is the peak discharge.

(iii) Fixed percentile: A value may be chosen based on visual guidance by the flow hydrograph. Other arbitrary criteria, such as three times the median flow may be chosen.

[15] The recession limbs of flood events tend to be relatively short. Therefore, where the flow has dropped below 75% of the lower of the two peaks, there is reasonable security in the assumption of independence, i.e. the assumption that the effects of the previous flood event do not significant contribute to the development of the second pulse.

[16] The original publication uses square miles in the equation. This was transformed into km^2 by dividing the area by 2.589.

4.3.2 Threshold selection for the PDS of low flows

Setting the threshold for the analysis of low flows of the Waimakariri River has an equal lack of consensus. First, there is a lack of consensus on the definition of a low flow event and drought events. Often, the two terms are used interchangeably. In this study, a low flow is considered to be river discharge below a predefined truncation level. Therefore, the upper bound for low flows has to be set. Fleig et al. (2006) recommend finding a threshold level, which excludes both, multi-year droughts, but also excludes years without any significant low flows and droughts. The common options for the upper bound are (Smakhtin, 2001):

(i) the mean annual flow
(ii) median annual flow, which is often seen as a more conservative option, as hydrological data are positively skewed and hence the median flow is smaller than the mean flow
(iii) a percentile of the FDC, usually a flow that is exceeded between 5-30 % of the time.

The separate dimensions described in the runs theory (Yevjevich, 1972b) were further explored in the analysis of low flows (Chapter 3). Thus, various truncation values were chosen to analyse run length (duration) and the severity (water deficit) of low flows, in addition to troughs under threshold. A summary of the sampling methods is given in Table 4.2.

Table 4.2 Summary of sampling procedures for the analysis of extreme events.

Annual maximum/ minimum series	High flows	Q_{max} of n_i	
	Low flows	Q_{min} of n_i	Univariate
		7dMALF	Univariate
Partial duration series	High flows	Peaks over selected threshold	threshold selection based on criteria described in text
	Low flows	Troughs under selected threshold	
		Magnitude	Bivariate

4.4 Model choice

Once significant flow events have been selected, the statistical information of the selected random variables are summarised within an appropriate distribution. Typically, to obtain the best fit, all data are used for fitting a distribution function. In this case, however, the focus lies on the extreme tails of the distribution. The overall aim is to achieve an approximation of the true underlying distribution of events. Model candidates that have been commonly used worldwide and in New Zealand were previously addressed in Chapter 3. None of the proposed distributions in the literature are universally accepted and the relative merits of each distribution are still a topic of discussion. Therefore, several authors, e.g. Mohssen (2009) and Keast and Ellison (2013), suggest testing models for their goodness of fit and appropriateness for each case. This section aims to give an overview of the distributions and their parameters. Of the many distributions that can be applied for frequency analyses of extreme events, the following have been identified as the most fitting candidate distributions, based on findings of previous research (Chapter 3)[17]:

4.4.1 Generalised Extreme Value family: GEV, Gumbel and Weibull

The Generalised Extreme Value (GEV) distribution family is most widely used for analysing the frequency of extreme events. The justification for the use of the GEV distributions is given by the following: Let X_i represent an observed daily average flow at a site for year *n* with a base distribution of *F*. Furthermore, let $M_n = \max(X_1, X_2, \ldots X_n)$ be the maximum value of *n* yearly maximum flow values. In the study of extreme values, the focus lies on the distribution of M_n, rather than the distribution of X_i. Knowledge about the exact parameters of *F* is not required with the use of GEV distributions. The cdf for the GEV distribution is written as:

$$F(x) = exp\left\{-\left[1-\frac{\kappa(x-\xi)}{\alpha}\right]^{1/\kappa}\right\},\ \kappa \neq 0, \qquad \text{(Equation 4.6)}$$

with parameters κ (shape), ξ (location) and α (scale), where $1 + \kappa (x - \xi)/\alpha > 0$. The shape parameter κ of the distribution determines its type and therefore the special cases of the GEV distribution. A Type I distribution (EV1), corresponding to $\kappa = 0$, is also known as the Gumbel distribution in the literature. The EV1 distribution has a constant skew of 1.1396. The average recurrence interval for the mean value is 2.33 years, which is the reason why the mean annual flood is often defined with an average recurrence interval of 2.33 years (Gordon et al., 2004). The cdf of the EV1 is written as:

$$F\ (x) = exp\left\{-exp\ \left[\frac{-(x-\xi)}{\alpha}\right]\right\}. \qquad \text{(Equation 4.7)}$$

The Type II distribution (EV2), also known as the Fréchet distribution, is characterised by the shape parameter $\kappa < 0$. The type III distribution (EV3) is denoted by $\kappa > 0$. If *X* corresponds to an EV3 distribution, then –*X* has a Weibull distribution, which is often used in the analysis of low flows (Pearson & Henderson, 2004). This has the effect of creating a lower limit. The location parameter ξ is defined as the low flow value that will be exceeded with a recurrence interval of 1.58 years. This is often referred to as the characteristic drought. The Weibull distribution cdf is:

$$F(x) = 1 - \ exp\left[-\left(\frac{x}{\alpha}\right)^{\kappa}\right]. \qquad \text{(Equation 4.8)}$$

[17] Equations are taken directly from Stedinger et al. (1993), unless otherwise indicated.

Considering the domain, the EV1 is a distribution with no upper or lower limits, EV2 has limits on the lower end, and EV3 has a fixed upper bound.

4.4.2 Normal family: Normal, log-Normal and log-Normal III

Flood peak data rarely conforms to the Gaussian distribution. For one, flood data have a finite lower limit (often zero) or a theoretical limit that represents the smallest annual flood or the threshold value of a PDS. Moreover, flood data usually display a positive skew, creating a tail on the upper side of the distribution (Gordon et al., 2004). Therefore the N distribution is used for well-behaved phenomena, such as average annual streamflow (Stedinger et al., 1993).

The N distribution is a two-parameter distribution, utilising the mean and variance as defining parameters. The third parameter, shape, is zero because the distribution is symmetric. In a N distribution, the mean is represented by a 0.5 probability, which corresponds to a 2-year recurrence interval in the AMS. For a generic Normal distribution, the cdf is:

$$F\ (x) = \ \Phi\ \frac{x-\mu}{\sigma}\ , \qquad \text{(Equation 4.9)}$$

with mean μ and deviation σ. The Q - function, i.e. the probability that the value of a variable *X* will exceed *x*, is given by

$$Q\ (x) = 1 - \ \Phi\ (\mathrm{x}). \qquad \text{(Equation 4.10)}$$

The cdf is not available in its closed form, but tables are used to obtain the standardized Normal variate *z*. Normal distribution functions are used in certain cases in hydrology, as described by Yevjevich (1972a): for fitting symmetrical empirical frequency distributions of random variables, as a pdf for analysing random errors, as a bench mark distribution for comparisons, and for statistical inferences as many hydrological statistical parameters are exactly or approximately normally distributed.

In the case that the logarithms of the data follow a Normal distribution, the distribution of *X* is log-Normal. It is often recommended for the elimination of skewness from the data (Hazen, 1930 as cited in Kirby & Moss, 1987). The cdf for the three-parameter log-Normal distribution (LN3) is:

$$F\ (x) = \ \Phi\ \left(\frac{\log(x- \xi)- \mu}{\sigma}\right), \qquad \text{(Equation 4.11)}$$

where x > ξ and Φ are parameters of the standard Normal distribution. Changes in the location parameter ξ of the function have no effect on the other two parameters. In the case that ξ = 0, the equation reduces to a two-parameter log-Normal distribution (LN2), which has the probability density function (pdf):

$$f(x) = \frac{1}{x\sqrt{2\pi\sigma_y^2}} exp\left[-\frac{1}{2}\left(\frac{\ln(x)-\mu_x}{\sigma_x}\right)^2\right].$$ (Equation 4.12)

4.4.3 Pearson Type 3 family: Pearson III, log-Pearson III

This family of distributions is one of many proposed by the statistician Karl Pearson (Stedinger et al., 1993) in an attempt to model skewed observations. The Person Type III (P3) distribution is a special case of the Gamma distribution, but has an additional parameter. The third parameter introduces a lower bound on the left-hand tail of the distribution. The other two parameters are location and shape. The P3 distribution belongs to the so-called thin-tailed class of distributions, in which exceedance probabilities typically decrease exponentially in the extreme tails (Kirby & Moss, 1987). The cdf of the P3 distribution is expressed by (Singh, 1998):

$$F(x) = \frac{1}{\beta\Gamma(\alpha)}\int_0^\infty \left(\frac{x-\Upsilon}{\beta}\right)^{\alpha-1} exp\left(-\frac{x-\Upsilon}{\beta}\right) dx,$$ (Equation 4.13)

where β ,α, and Υ are the scale, shape and location parameters of the distribution respectively, and $\Gamma(\alpha)$ describes the Gamma distribution.

In the log-Pearson II (LP3) distribution, the random variables' logarithms are P3 distributed. This type of distribution is used as a standard procedure for the estimation of design floods in the United States (Kirby & Moss, 1987). The LP3 distribution belongs to the thick-tailed distributions, whose exceedance probabilities typically decrease as a reciprocal power function at extreme discharges (Kirby & Moss, 1987). Therefore, these distributions are more likely to estimate the occurrence of observations far beyond the majority body of observations, when compared to thin-tailed distributions. When log *X* is symmetric about its mean, the LP3 distribution reduces to a LN distribution.

4.4.4 Above-threshold arrivals: Poisson, binomial and negative-binomial

Cunnane (1979) proposed the use of the dispersion index (DI) to determine the best-fitting distribution for modelling exceedances above the threshold. The dispersion index is defined as the ratio between the variance (V) and mean of exceedances above a threshold (E). The test statistic corresponding to the DI is:

$$d = \sum_{i=l}^{N} \frac{(m_i-\bar{m})^2}{\bar{m}} = \frac{(N-1)V}{E},$$ (Equation 4.14)

where m_i is the annual number of exceedances in the given year *i*, and $\bar{m}$ is the mean of m_i. *d* follows a chi-squared distribution with *N*-1 degrees of freedom. A Poissonian distribution of exceedances has a dispersion index of DI = 1. Binomial distributions display 0 < DI < 1 tendencies, and negative-binomial has a dispersion index of DI > 1. The Poissonian hypothesis of DI = 1 is rejected if DI is outside the range of $(\chi^2_{\alpha/2}, \chi^2_{1-\alpha/2})$, where α is the chosen significance level, in this case, 0.05 (Önöz & Bayazit, 2001). The dispersion index is used as one method to assess the best fitting threshold level when modelling flood frequencies. However, as described earlier, the choice of a distribution for modelling above-threshold arrivals is irrelevant in this study, as the newly derived formula by Mohssen (2009) does not require this element.

4.4.5 Above-threshold magnitude: Generalised Pareto and Exponential

The Generalised Pareto distribution (GP) is a three-parameter continuous probability distribution, described by shape, scale and location. It is a suitable distribution for modelling the magnitude of events that exceed a specified threshold or lower bound, and at which the cdf has a maximum (κ < 1). The equation of the cdf is:

$$F(x) = 1 - \left[1 - \kappa \frac{(x-\epsilon)}{\alpha}\right]^{1/\kappa - 1}, \text{ for } \kappa \neq 0. \qquad \text{(Equation 4.15)}$$

The parameters are analogous to those of the GEV distribution. In the case that κ = 0, the GP distribution is reduced to the Exponential distribution, which is expressed by:

$$F(x) = 1 - exp[-\beta(x - \xi)]. \qquad \text{(Equation 4.16)}$$

When κ < 0 the distribution is long-tailed. When κ > 0 it becomes bounded at the upper end and should be applied with caution, unless physical evidence suggests upper bounding of the events (Beguería, 2005). The threshold level ϵ is determined beforehand.

4.4.6 Summary of distribution model choice

For the selection of the appropriate distribution, a fundamental question constitutes the guiding principle: Is the proposed distribution consistent with the available data for the Waimakariri OHB site? Each distribution described above was applied to obtain the distribution function with the best fit to the selected data. Goodness of fit statistics were used to answer this question.

4.5 Parameter estimation

The parameters of a pdf are estimated from the sample data. Such parameters are properties of location (central tendency), scale (dispersion or concentration), shape, asymmetry, peakedness, etc. Mathematically, the more parameters used in a function, the easier the fitting of an empirical distributions will be. However, statistically, the number of moments or other statistics needed for describing the parameters increases with the number of parameters. An increase in the order of moments for such estimation decreases the reliability of moments and thus the reliability of the parameter estimated from it (Yevjevich, 1972b).

As the literature often favours using L-moments for parameter estimation, as opposed to other methods summarised in Chapter 3, the focus in this research was on the L-moments technique. One major advantage of L-moments is the avoidance of non-linear transformation of sample data, which can often lead to poorer parameter estimation due to distortions. The effect of data outliers is therefore reduced when using L-moments, compared to other conventional methods (Hosking, 1990; Pearson, 1991; Pearson & Davies, 1997).

L-scale, L-skewness and L-kurtosis are analogous to the standard deviation, skewness and kurtosis of conventional moments. Standardised L-moments are termed L-moment ratios and are equally comparable to ordinary moments. Each distribution function has theoretical population L-moments, which can be estimated from the sample L-moments of the empirical distribution. The first L-moment is the sample mean, the second L-moment is a measure of dispersion/ scale. L-moment ratios, which are dimensionless quantities, are obtained by dividing higher order L-moments by their dispersion, such as the measure of scale (L-CV):

$$\tau_2 = \frac{L_2}{L_1}. \qquad \text{(Equation 4.17)}$$

τ_3 is the measure of skewness (L-skewness) and τ_4 is a measure of kurtosis (L-kurtosis), where:

$$\tau_3 = \frac{L_3}{L_2}; \qquad \text{(Equation 4.18)}$$

and $\tau_4 = \frac{L_4}{L_2}$. (Equation 4.19)

Hosking's (1990) paper provides a list of L-moments for some of the commonly used distributions. Equations for the calculation of L-moments are available in Appendix B.

4.6 Selecting a distribution

With a large number of distributions available, the best fitting distribution, i.e. the distribution that best agrees with the empirical observations, has to be chosen objectively. For this purpose, several techniques have been employed in frequency analyses. Graphical techniques are considered less objective, as the choice of the fitting distribution is a subjective judgement. However, goodness of fit statistics have also been criticised in the past, as they often fail to distinguish between candidate distributions for the same application. Therefore, the methods described in detail below are used to reject some distributions over others, and not to select the best fitting distribution.

4.6.1 Graphical testing

Probability plots and quantile plots

For an initial graphical display and analysis of the flood and low flow data, probability plots (PP plots) are used. They are especially useful for determining if the sample chosen for the analysis is consistent with the population distribution, and thus guide the choice of the most fitting distribution candidate. PP plots compare theoretical and empirical probabilities to reject non-fitting distributions. In quantile-quantile (QQ) plots, empirical data are displayed with their respective plotting position as a straight line in an *x-y* plot by using an inverse distribution scale of the theoretical distribution. The smaller the deviation of the points are from the linear function $y = a + bx$ (a = location, b = scale), the better the evidence that the chosen distribution produces the observed data (Gordon et al., 2004; Stedinger et al., 1993). The QQ plot is assembled by plotting ordered observations of Q_i, $i = 1, 2, \ldots n$ against the inverse of the chosen cumulative distribution function (cdf). This is defined as:

$$y_i = F^{-1}\left(\hat{F}(Q_i)\right), \qquad \text{(Equation 4.20)}$$

where $\hat{F}(Q_i)$, the plotting position, is estimated based on the i^{th} ordered observation. An unbiased plotting formula, the Gringorten formula, is often used in combination with an EV1 distribution. Other unbiased plotting positions that are frequently employed include the Weibull, Blom, Cunnane or Hazen formulae (Stedinger et al., 1993). It is to be noted, however, that both plotting techniques are deemed to be rather subjective and sensitive to random occurrences in the data set. Therefore, it is not advisable to rely solely on PP or QQ plots for the choice of the distribution.

L-moment ratio diagrams

L-moment ratio diagrams are used to determine the suitability and goodness of fit of various distributions to the sample data. L-moment diagrams display the dimensionless ratio of L-kurtosis vs. L-skewness for a variety of statistical distributions. Two parameter distributions plot as a single point and three parameter distributions are shown as curves. Hosking (1991) provides approximations for obtaining τ_4 from τ_3 for selected distributions. Typically, L-moment ratio diagrams have been used in regional flood frequency analyses, using average values of skewness and kurtosis from stations in the study area. The goodness of fit is judged by a comparison of values with the fitted regional data. L-moments within this context are also useful in describing regional data for making inferences about ungauged streams (Madsen, Pearson, & Rosbjerg, 1997; Pearson, 1991; Yue & Wang, 2004). As this study only uses data from a single station, no regional data are available to use for comparison. However, L-moment ratio diagrams are still useful in comparing the observed series with the candidate distributions.

4.6.2 Statistical testing

While probability plotting techniques are useful in deciding whether the observed series is generated by the underlying chosen distribution, deciding what constitutes deviations from the linear equation is rather subjective. Therefore, objective analytical goodness of fit techniques have a wide appeal in frequency analyses for the selection of population distributions. They reveal whether the observed lack of fit can be attributed to sample-to-sample variability or if the deviations of the data from the model is statistically significant (Stedinger et al., 1993). A calculated test statistic or index is used as the basis for rejecting some distributions over others.

One of frequently used goodness of fit tests in the frequency analysis domain is the non-parametric chi-squared (χ^2) test. In chi-squared tests, the frequencies of the observed events in class intervals are compared with expected values for a certain distribution for all values. The test statistic (χ^2) is given by:

$$\sum_{i=1}^{k} \frac{(O_i - E_i)^2}{E_i}, \qquad \text{(Equation 4.21)}$$

where O_i is the observed frequency and E_i is the expected frequency for each class interval. The choice of class intervals has no objective rules, but at the same time can have a significant effect on the calculated test statistic. In this study, the choice of intervals follows 1 + 3.3 $\log_{10}$ (n), where n is the number of observations in the series (Benjamin & Cornell, 1970, p. 8).

The Kolmogorov-Smirnov test (KS test), on the other hand, determines the largest discrepancy between the theoretical ($F_n(x_i)$) and empirical ($F_0(x_i)$) cumulative distribution functions of the data set (Zeng et al., 2015). The test statistic (D) is:

$$D = \max |F_n(x_i) - F_0(x_i)|. \qquad \text{(Equation 4.22)}$$

Although empirical distribution function tests are generally more powerful than chi-squared statistics, the chi-squared statistic is more flexible in its use. Zeng et al. (2015) advocate the use of the chi-squared statistic in instances where little is known about the empirical distribution's characteristics. Both test statistics have been used in this study to reject distributions over others. Rejection of a distribution occurs when the calculated index value lies in the extreme tail of the specified distribution. In this case, there is doubt that the sample came from the sampling distribution.

Additionally, the Filliben Correlation Coefficient test (FCC) (or sometimes Probability Plot Correlation Coefficient test) (Filliben, 1975) was used to determine the suitability of candidate distributions. In this test, the correlation coefficient r between ranked observations and the corresponding quantiles, as determined by plotting positions, are assessed such that r measures the linearity of a probability plot. The plotting position used to determine the empirical quantiles was $q_i =$

$\frac{i}{n+1}$, where *i* is the rank assigned to the event in descending order and *n* is the total number of events (see 4.6.1. Probability plots).

4.7 Summary

This chapter detailed the strategy and modelling procedures used in the study to obtain quantile estimates for floods and low flows in the Waimakariri River. It identified the method used to sample data from the continuous discharge record, introduced candidate distributions and ways of testing the goodness of fit of those distributions. The following chapter present the results obtained from applying the above described research strategy.

Chapter 5

Results part I: Frequency analysis

This chapter presents the results obtained from annual maximum/minimum (AMS) and partial duration series (PDS) modelling of flood frequency and low flow frequency based on the continuous streamflow record (1967-2015) of the Waimakariri River at the OHB discharge measurement site. The first part details the application of AMS and PDS to flood flow events. The subsequent section deals with the findings of the application of AMS and PDS to low flow events.

5.1 Flood frequency analysis

5.1.1 Data selection

The continuous streamflow record measured at the OHB site of the Waimakariri River (1967-2015) was used in this thesis as the basis for the extraction of events in the subsequent frequency analysis. The data was obtained from the regional council, ECan. The extraction of annual maxima yielded 49 events for 49 years on record, which are tabulated in Appendix B. The highest discharge measured since the installation of the automatic recorder in 1967 has been 2836 m^3/s (1979). This was exceeded twice in the historical record with 3990 m^3/s (1957) and 3740 m^3/s (1940). The lowest recorded annual maximum is 716 m^3/s (2003). A separate annual series, the historical annual maximum series (AMS_{hist}), which includes data from 1930 onwards, is also modelled for comparative purposes, as previous frequency analyses of the Waimakariri River utilised this data. However, no continuous measurements are available from 1930 - 1966. Therefore, the PDS only account for data from 1967 - 2015 inclusive. For a more detailed account of data quality and data uncertainty, the reader is referred to Appendix A.

The AMS was tested for serial dependence using a plot of autocorrelation vs. time lags. Figure 5.1 shows no significant autocorrelation and serial independence of the data. A slight negative trend was detected in the time series using Kendall's tau (τ_a) and Spearman's Rank Correlation (ρ) at α=0.05. However, this trend is statistically not significant, as the calculated value is smaller than the tabulated critical value (τ_a = -0.031, p = 0.756; ρ = -0.042, p = 0.773). The historical AMS, including manually gauged data from 1930-1966, equally shows no trend (τ_a = -0.002, p = 0.979; ρ = -0.001, p = 0.991), and no autocorrelation, i.e. serial independence within the 86 extracted annual maxima (refer to Appendix B for plot).

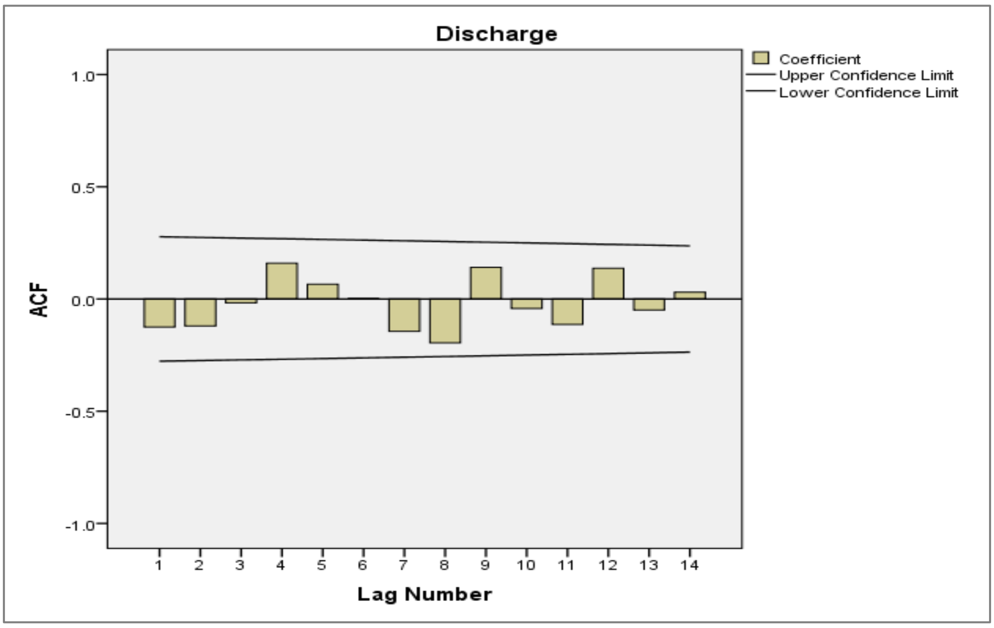


Figure 5.1 Autocorrelation plot of sample annual maxima (1967 - 2015) vs. time lags. The black lines represents the 95 % confidence limits.

For the frequency analysis of flood events using PDS, various thresholds were chosen. The thresholds applied for the extraction of series and resulting number of peaks are summarised in Table 5.1. Cunnane (1973) previously suggested that PDS perform better than AMS in terms of variance when the average number of events is at least 1.65 per year (λ = 1.65). Therefore, 1000 m^3/s was the highest threshold level used in the study, as this threshold produced λ = 1.837. A higher threshold, such as 1050 m^3/s, resulted in λ = 1.55, and thus contradicts Cunnane's (1973) findings and suggestions.

Independence of events was ensured by applying recommendations given by the UK National Environment Research Council (NERC, 1975), which state that a second flood peak must be rejected if:

$$\Theta < 5 \text{ days} + \ln(A/2.589) \text{ or } Q_{min} > (3/4) \min [Q_1, Q_2], \qquad \text{(Equation 5.1)}$$

where Θ is the time interval, A is the catchment area in square kilometers[18], and Q is the peak discharge. Therefore, a minimum of 12 days must lie between two flood peaks if the flow between two peaks does not drop below 75 % of the lower of the two peaks. Examples of accepted and rejected flood peaks are given in Figure 5.2 for illustrative purposes. The figure shows two peaks that occur above the selected threshold of 500 m^3/s. The time interval between the peak occurring on 13/10/2012 and 18/10/2012 is only 5 days. For the lower of the two peaks to be selected, the flow must drop below 75 % of the lower peak, which in this case is 170.9 m^3/s. However, the flow only reduces to 190.8 m^3/s and therefore, the peak occurring on 18/10/2012 is rejected and omitted from further analysis. Table 5.1

[18] The original publication uses square miles in the equation. This was transformed into km^2 study by dividing the area by 2.589.

details the number of events extracted from the continuous discharge data using different threshold levels and the resulting average rate of high flow peaks per year (λ) using the aforementioned guidelines.

Table 5.1 Applied thresholds and the resulting number of peaks extracted from the stream record (1967-2015). λ is the average exceedance rate above the threshold.

Threshold	Number of peaks selected	Number of peaks omitted	λ
350 m^3/s	486	196	9.918
400 m^3/s	428	156	8.735
500 m^3/s	323	87	6.592
600 m^3/s	252	46	5.143
650 m^3/s	219	40	4.469
700 m^3/s	195	25	3.980
750 m^3/s	178	22	3.633
800 m^3/s	162	20	3.306
900 m^3/s	122	7	2.490
1000 m^3/s	89	3	1.837
1050 m^3/s	76	1	1.551

It is acknowledged that the application of these guidelines for the selection of independent peaks may eliminate flood peaks that are in fact independent of their predecessors. Flood peaks that follow closely may in fact be a result of separate storm events. The Waimakariri catchment is no exception, as flood events are often initiated from storm events from various directions, e.g. northwest or south. However, in order to maintain a standard and repeatable methodology in the selection of peaks, the aforementioned guidelines are adopted without exceptions.

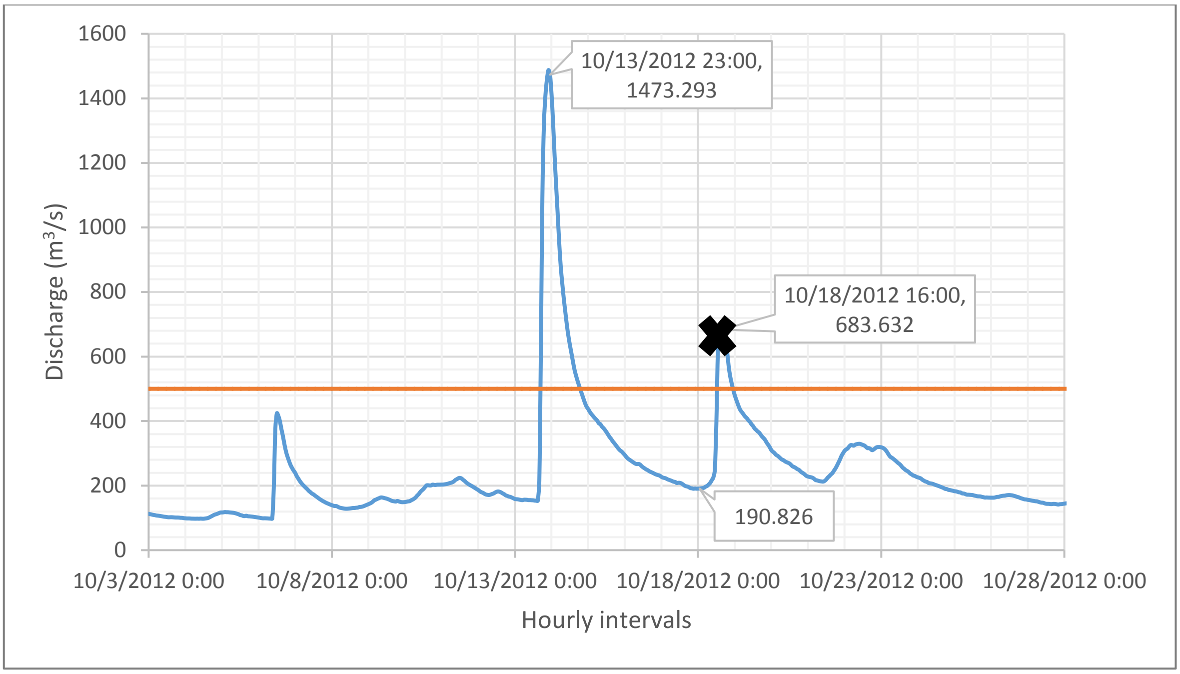


Figure 5.2 Flow hydrograph of the Waimakariri River at OHB, from 03/10/2012 until 28/10/2012. The orange line represents the chosen threshold level at 500 m3/s. Two peaks above the threshold are labelled.

Trend was tested by means of Spearman's Rank Correlation and Kendall's tau (Table 5.2); and visual confirmation of autocorrelation plots was used to detect serial dependence. The events resulting from applying threshold levels at 350 m^3/s and 400 m^3/s resulted in a significant positive trend at $\alpha = 0.05$. Therefore, these threshold levels and the resulting occurrences above the threshold are omitted from further analyses. No serial dependence was detected by visual inspection of autocorrelation plots as a result of any of the chosen threshold levels (Appendix B).

Table 5.2 Summarised results for trend tests of selected PDS, testing for correlation of the series.

Threshold	Spearman's ρ	Kendall's τ
350 m^3/s	0.123 > 0.007 *	0.086 > 0.006 *
400 m^3/s	0.117 > 0.014 *	0.080 > 0.12 *
500 m^3/s	-0.012 < 0.834	0.010 < 0.795
600 m^3/s	-0.030 < 0.633	-0.020 < 0.632
650 m^3/s	-0.053 < 0.431	-0.035 < 0.429
700 m^3/s	-0.015 < 0.831	-0.010 < 0.833
750 m^3/s	-0.058 < 0.437	-0.039 < 0.440
800 m^3/s	-0.033 < 0.678	-0.020 < 0.706
900 m^3/s	-0.063 < 0.492	-0.044 < 0.475
1000 m^3/s	0.018 < 0.863	0.018 < 0.796

* indicates detected significance at $\alpha = 0.05$

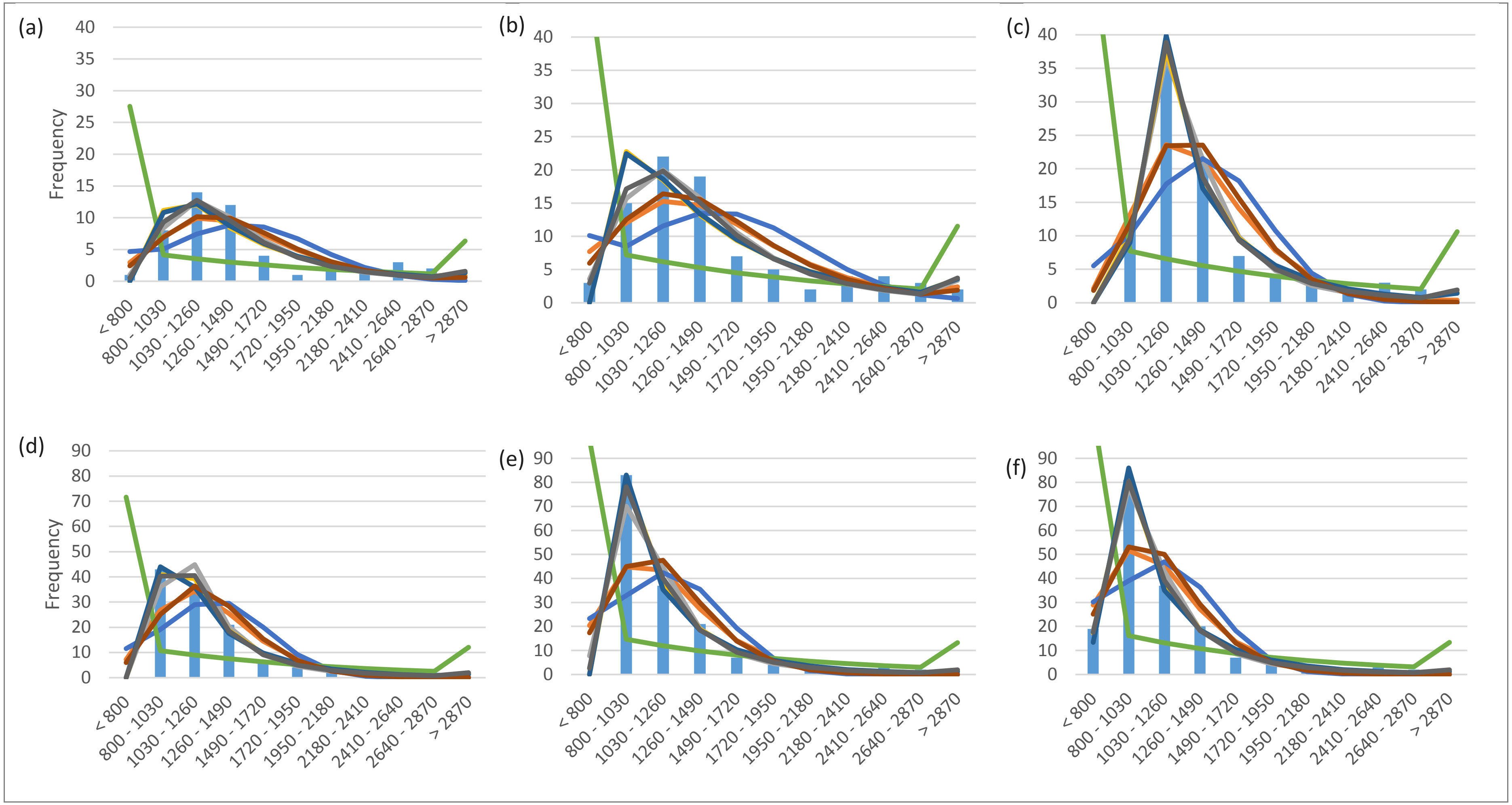
5.1.2 Parameter estimation

For the analysis of flood frequencies, eight distributions were considered. Of the eight distributions, five have three parameters, namely; GP, LN, GEV, P3 and the LP3. The remaining three distributions have two parameters, i.e. EV1, Exponential and N. The parameters of these distributions were estimated using the L-moments approach and respective equations as detailed in Appendix B, along with calculated L-moments and L-moment ratios for the AMS and each PDS.

5.1.3 Selection of best fitting distribution

Graphical selection

For an initial graphical display of the fit of various distribution functions, histograms for AMS and PDS were produced. As is the case with all judgements based on graphical analysis alone, subjectivity in decision-making is very high. However, based on the histograms showing a comparison between observed flows and expected flows in Figure 5.3, the Exponential and N distributions can be rejected as potential best-fit distributions with reasonable certainty. The GP, P3, LP3 and GEV distributions appear to match the observed data better than the EV1 and N distributions in all series.


(a)
(b)
(c)
(d)
(e)
(f)
Frequency
Frequency
< 800
800 - 1030
1030 - 1260
1260 - 1490
1490 - 1720
1720 - 1950
1950 - 2180
2180 - 2410
2410 - 2640
2640 - 2870
> 2870

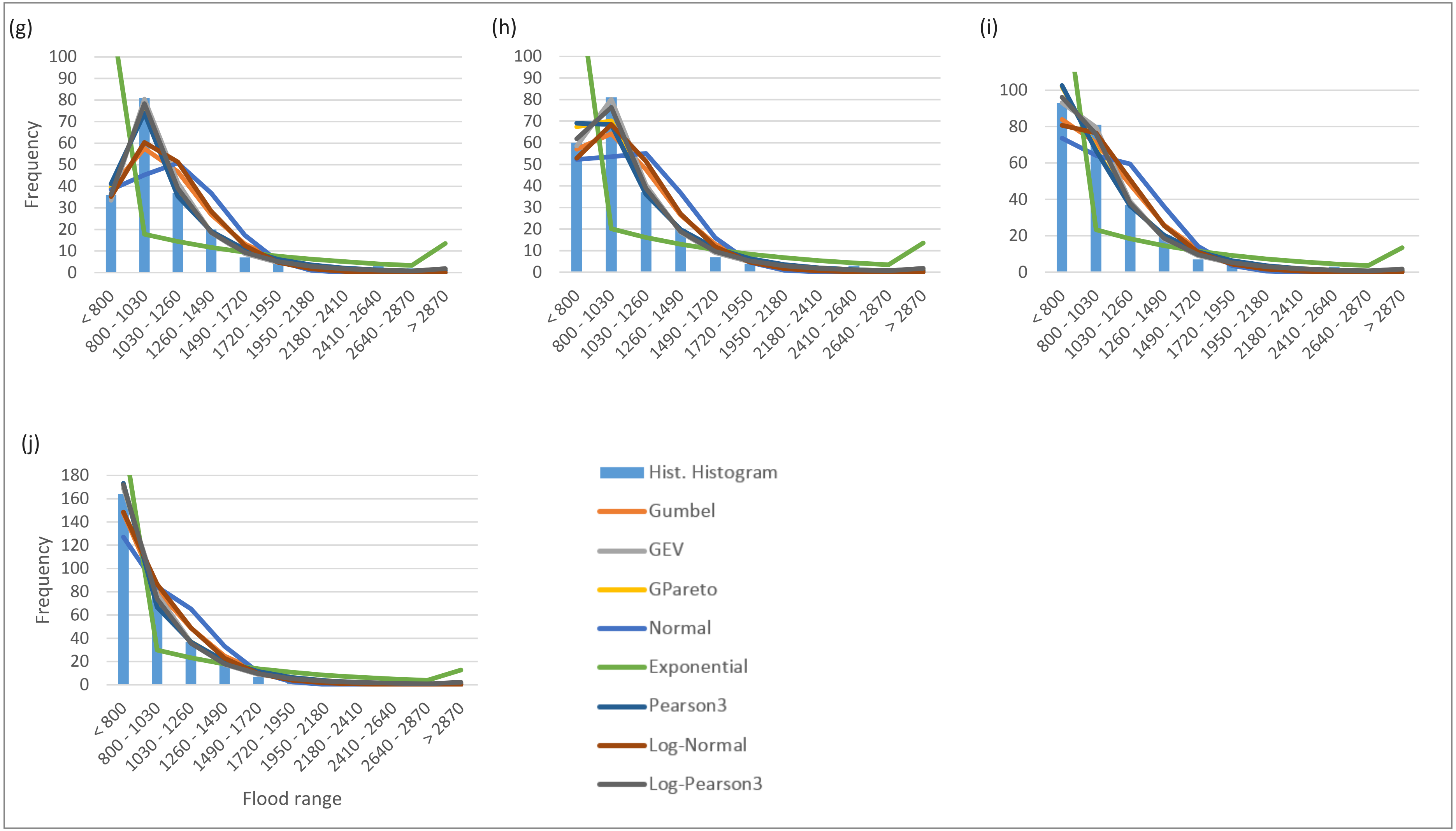


Figure 5.3 Histograms of observed vs. expected flows of AMS and PDS series. (a) AMS (1967-2015), (b) AMS_{hist} (1930-2015), (c) PDS threshold 1000 m^3/s, (d) PDS threshold 900 m^3/s, (e) PDS threshold 800 m^3/s, (f) PDS threshold 750 m^3/s, (g) PDS threshold 700 m^3/s, (h) PDS threshold 650 m^3/s, (i) PDS threshold 600 m^3/s, (j) PDS threshold 500 m^3/s.

L-ratio diagrams

As discussed earlier, L-moment diagrams display the dimensionless ratio of L-kurtosis vs. L-skewness for a variety of statistical distributions. Two parameter distributions plot as a single point and three parameter distributions are shown as curves in Figure 5.4. The goodness of fit is judged by a comparison of values with the fitted theoretical ratios.

Figure 5.4 clearly excludes the Normal and EV1 (Gumbel) distribution as candidates for any of the obtained AMS or PDS. The GEV curve to the right of the Gumbel point indicates an EV2 tendency, while the curve to the left of the Gumbel point indicates an EV3 tendency. It is evident that none of the sample data are described by EV3 tendencies. The historical AMS (including data from 1930-2015), and the PDS resulting from 600 m^3/s, 650 m^3/s and 700 m^3/s thresholds appear to be closest to the GEV curve. The AMS (data from 1967-2015), and PDS resulting from 500 m^3/s, 750 m^3/s, 800 m^3/s and 900 m^3/s are closest to the LN and GP curves. The PDS resulting from a 1000 m^3/s threshold lies between the P3 and LN curve; however, neither distribution is visually a better fit.

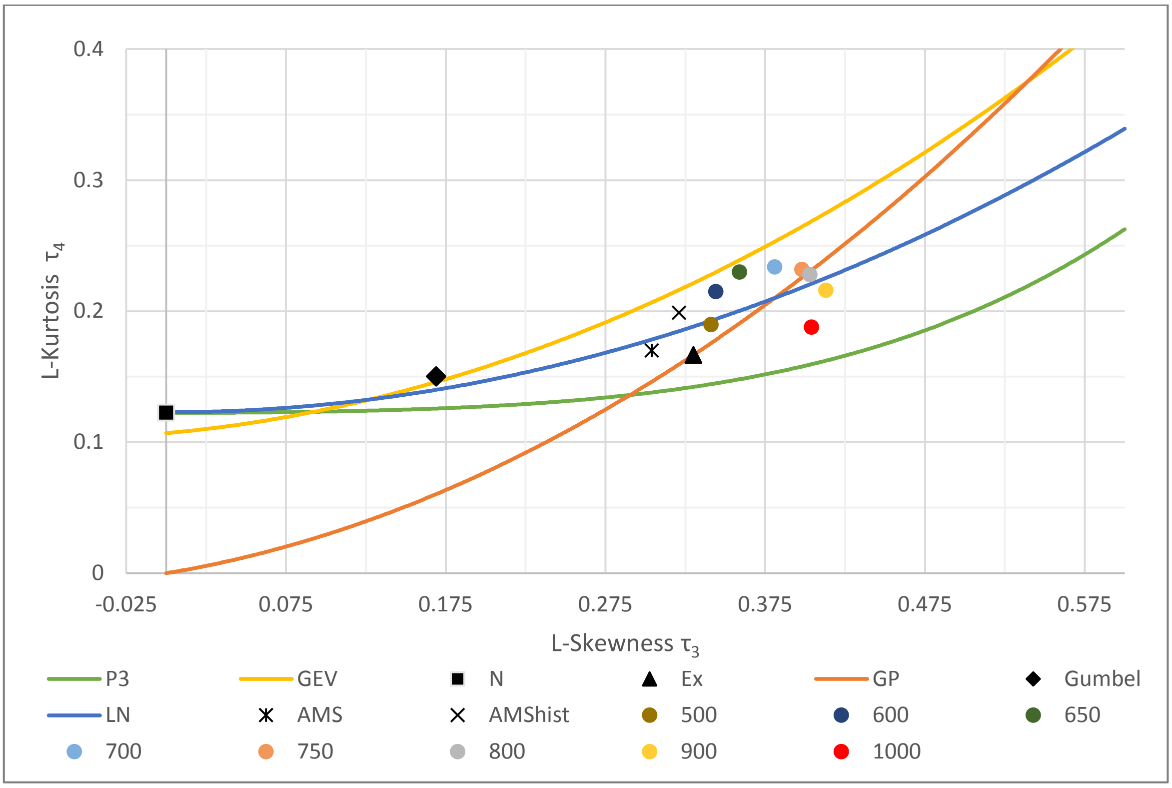


Figure 5.4 Theoretical L-moment ratio diagrams of various distributions and sample L-moment ratios.

Probability plots and quantile plots

Based on the L-moment ratio diagram (Figure 5.4), viable distribution candidates are GEV, LN, P3, Exponential and GP distributions. The PP plots comparing theoretical and empirical probabilities of various PDS and AMS in Figure 5.5 further validate this choice of distribution candidates. However, the Exponential, Normal and LN distributions can be additionally excluded based on a purely graphical analysis of the plot. The PP plot indicates a better fit of the observed data to the GP, P3, LP3 and GEV distributions. Note that Figure 5.5 only shows PP plots for the AMS (1967-2015) and PD series resulting from 1000 m^3/s and 650 m^3/s thresholds. Appendix B includes PP plots for all thresholds used in this study.

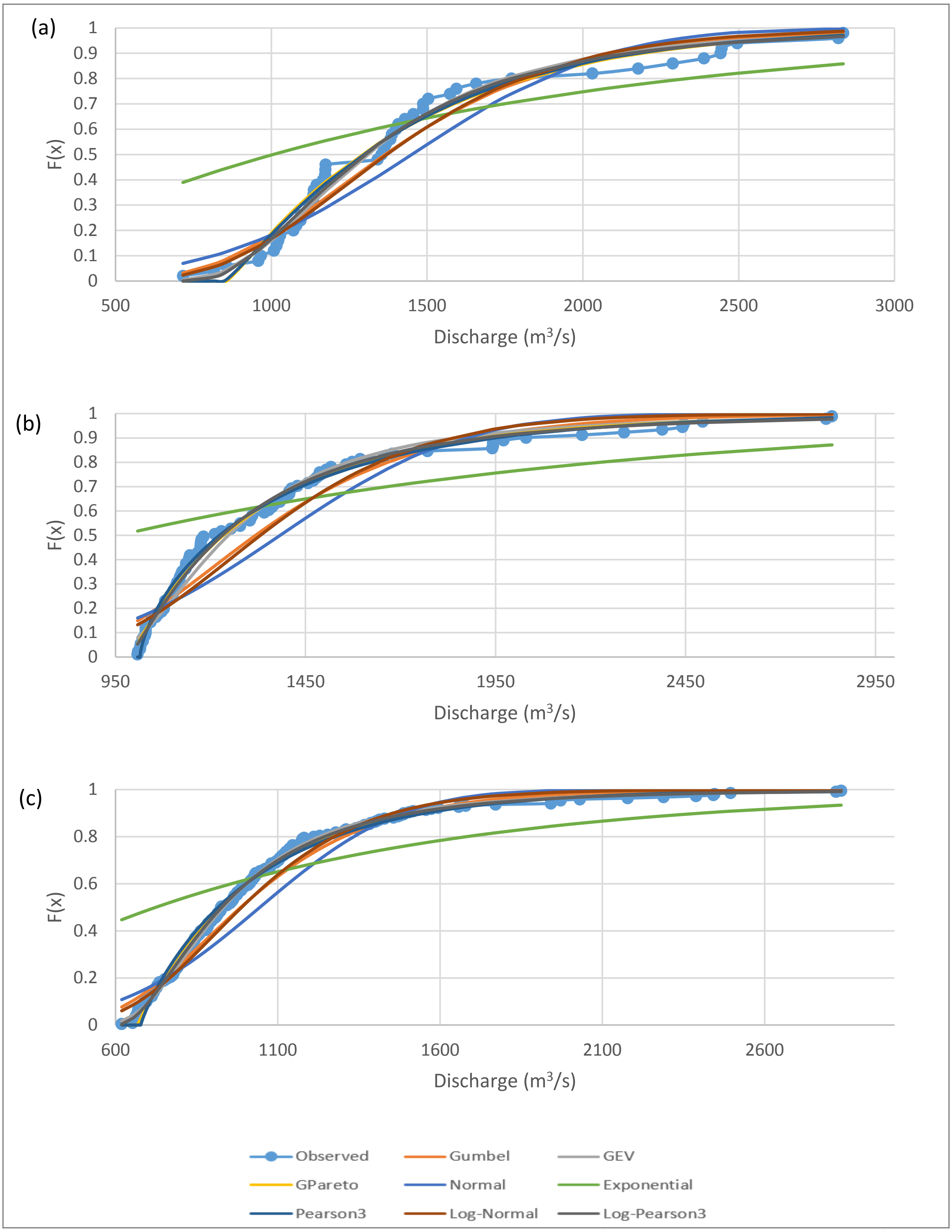


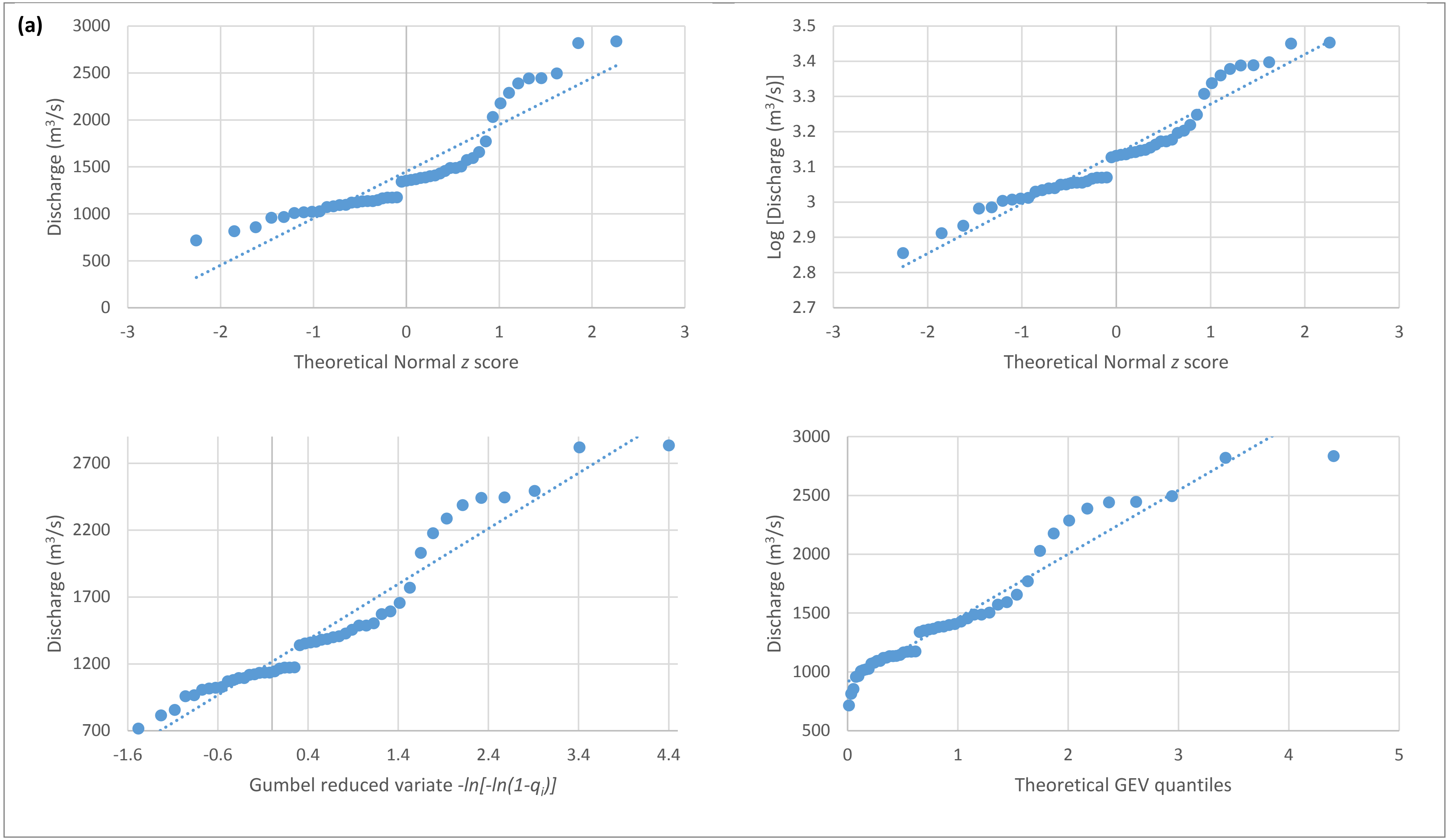
Figure 5.5 PP plots of (a) AMS (1967-2015), (b) PDS threshold 1000 m^3/s, and (c) PDS threshold 650 m^3/s.

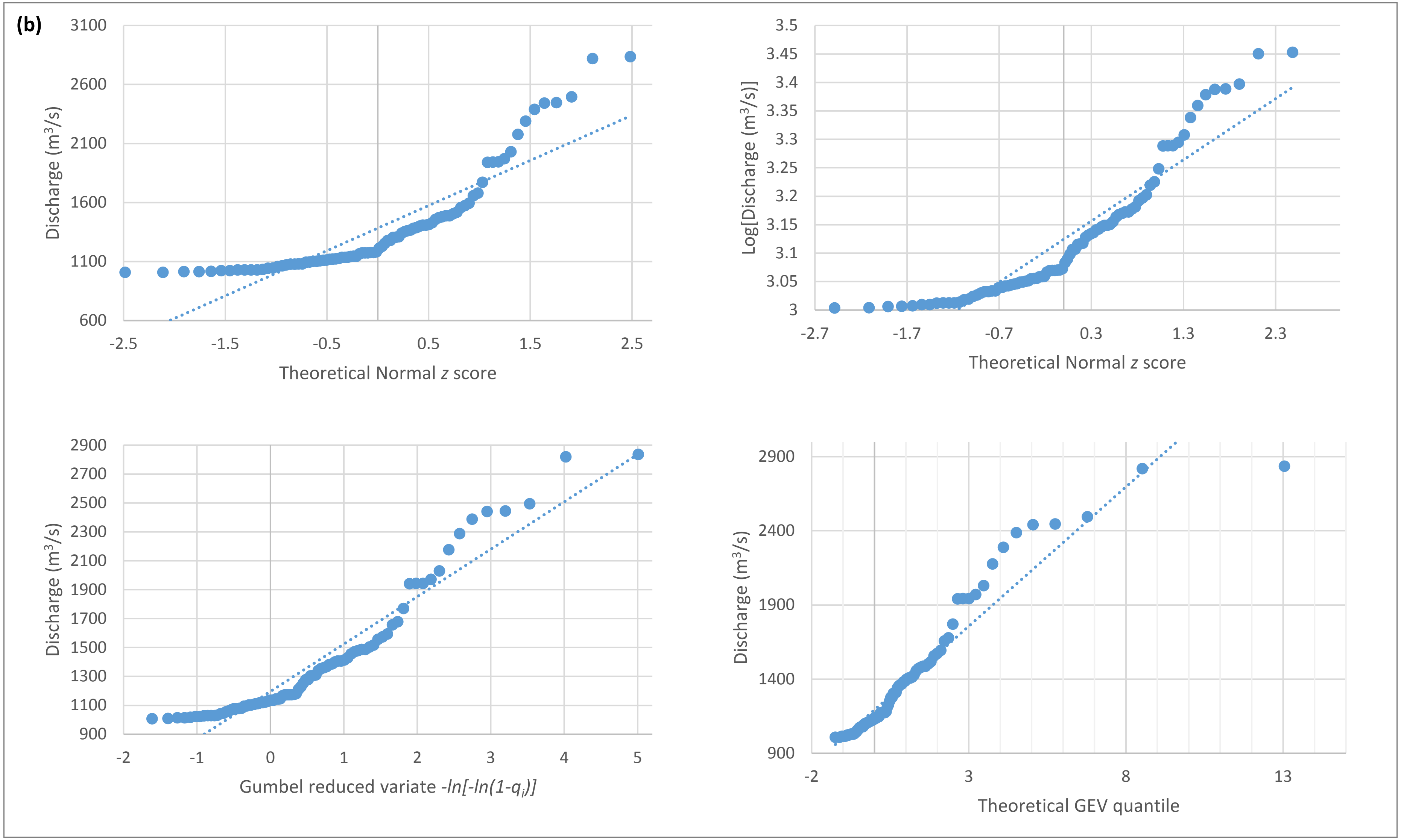
QQ plots were produced using the Cunnane plotting position to estimate the exceedance probability of the i^{th} largest events. The quantile plot shows the ranked observations x_i against their expected theoretical value depending on the chosen distribution (Stedinger et al., 1993). The Cunnane plotting position formula was chosen as it produces approximately unbiased quantiles for all distributions, as opposed to the Gringorten formula, which is optimised for the Gumbel distribution (Stedinger et al., 1993). The Cunnane formula has the following expression:

$$q_i = \frac{i-0.40}{n+0.20}, \quad \text{(Equation 5.2)}$$

where q_i, the probability plotting position, is an estimate of the exceedance probability.

For P3 or LP3 distributions, the frequency factor depends on the skewness when constructing a quantile plot. An often employed alternative is the use of N quantile plots, in which a P3 distribution should plot as a curved line (Stedinger et al., 1993). Similarly, a GP distribution will plot as a curved line on an Exponential plot due to the shape parameter. Quantile-quantile plots were thus only generated for the following distributions: N, LN, Exponential, GEV and Gumbel. Figure 5.6 shows a selection of QQ plots used for the selection of the best fit distributions. Appendix B contains all QQ plots generated for the purpose of this study. The quantile plots of the N distribution show heavy tails for both sides of the distribution. Both the left and right tails are not modelled well with the N distribution, meaning that the theoretical distribution is too thin to reproduce the extreme events present in the empirical sample. While the LN quantile plot is a better match visually for the AM series and the PDS obtained with a 650 m^3/s threshold, it is still an inadequate distribution for the heavy tails. The remaining concave shape of the data indicates that a LP3 distribution may be a better fit to the empirical data (Helsel & Hirsch, 1992). Furthermore, the GEV distribution appears to produce a more linear fit of the data than the Gumbel distribution with increasing number of events included in the series, i.e. lower PDS threshold. Examination of the Exponential plots (Appendix B) indicate that the GP distribution may be a better fit to the data, due to the curved line.


(a)
Discharge (m³/s)
Theoretical Normal z score
Log [Discharge (m³/s)]
Theoretical Normal z score
Discharge (m³/s)
Gumbel reduced variate -ln[-ln(1-qi)]
Discharge (m³/s)
Theoretical GEV quantiles



(b)
3100
2600
2100
1600
1100
600
Discharge (m³/s)
-2.5
-1.5
-0.5
0.5
1.5
2.5
Theoretical Normal z score
3.5
3.45
3.4
3.35
3.3
3.25
3.2
3.15
3.1
3.05
3
Log[Discharge (m³/s)]
-2.7
-1.7
-0.7
0.3
1.3
2.3
Theoretical Normal z score
2900
2700
2500
2300
2100
1900
1700
1500
1300
1100
900
Discharge (m³/s)
-2
-1
0
1
2
3
4
5
Gumbel reduced variate -ln[-ln(1-qi)]
2900
2400
1900
1400
900
Discharge (m³/s)
-2
3
8
13
Theoretical GEV quantile

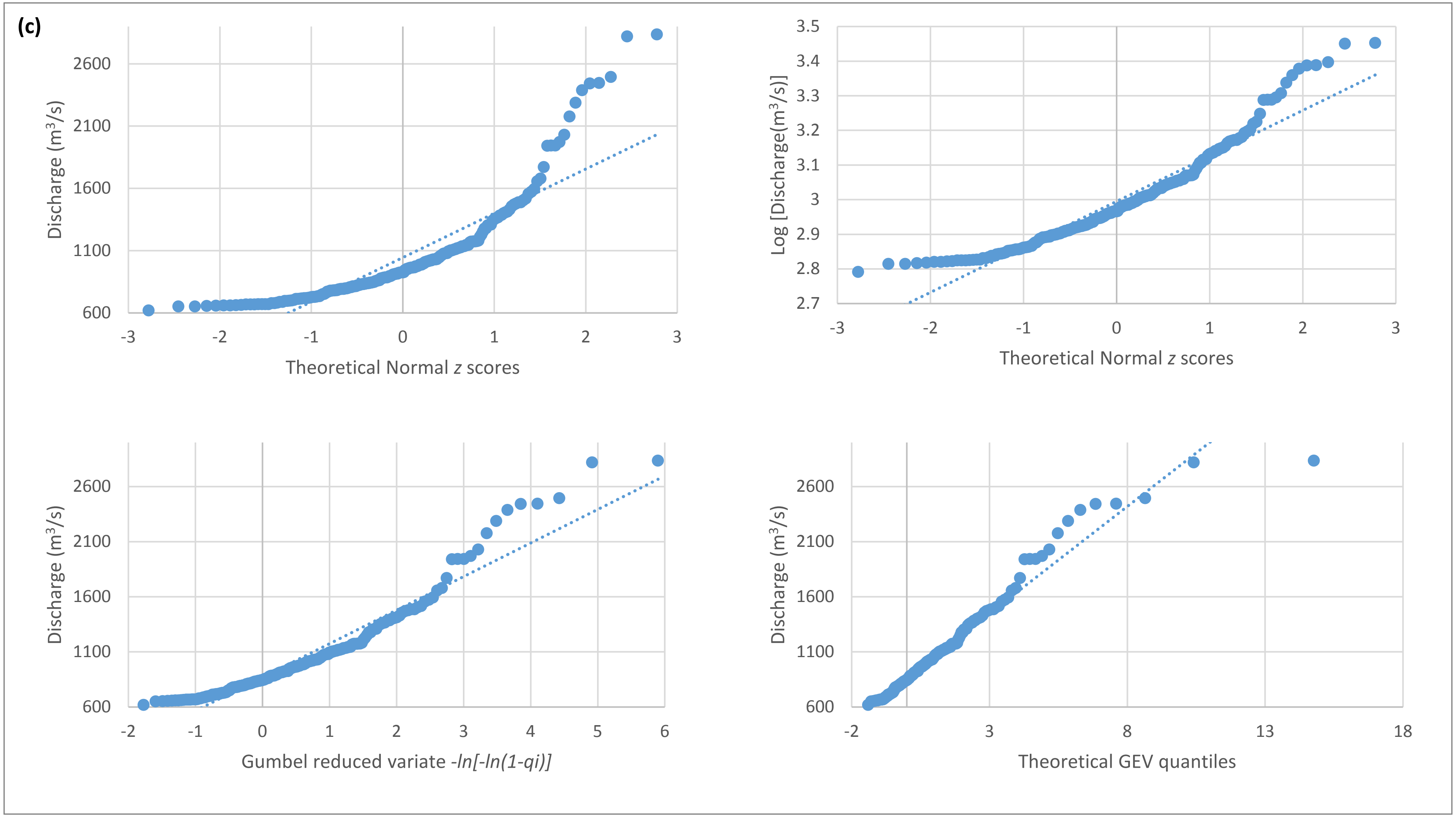


Figure 5.6 QQ plots of (a) AM series, (b) PDS with 1000 m^3/s threshold, (c) PDS with 650 m^3/s threshold. Included distributions are the N, LN, Gumbel and GEV distribution.

Goodness of fit statistics

Chi-squared (χ^2), Kolmogorov-Smirnov (*D*) and Filliben Correlation (FCC) test statistics were computed for each of the eight distributions to judge the goodness of fit for modelling the magnitude of exceedances above the threshold. The details concerning the calculation and rationale behind each test's statistics were previously discussed in Chapter 4. Results of χ^2, *D* and FCC statistics are shown in Table 5.3. The table indicates the rejected distributions at the 5 % significance level with an asterisk (*).

Table 5.3 Summary of goodness of fit statistics for modelling the magnitude of exceedances above the indicated threshold and the AMS (1967-2015) and AMS_{hist} (1930-2015) record.

Threshold (m^3/s)		**500**	**600**	**650**	**700**	**750**	**800**	**900**	**1000**	**AMS**	**AMS_{hist}**
GP	χ^2	12.64	12.72	11.21	8.95	8.393	8.481	8.711	8.441	11.84	11.69
	D	0.040	0.051	0.063	0.056	0.045	0.046	0.057	0.076	0.081	0.066
	FCC	0.994	0.993	0.993	0.993	0.992	0.992	0.989	0.987	0.982	0.992
LN	χ^2	*	*	*	*	*	*	*	*	*	14.62
	D	*	0.084	0.092	*	*	*	*	*	0.151	0.108
	FCC	0.967	0.964	0.959	0.949	0.939	0.936	0.929	0.929	0.971	0.9798
GEV	χ^2	8.672	8.994	9.548	10.32	10.92	*	13.55	11.09	13.58	9.942
	D	0.067	0.052	0.034	0.035	0.043	0.048	0.065	0.109	0.113	0.068
	FCC	0.992	0.991	0.989	0.986	0.983	0.981	0.977	0.975	0.975	0.993
P3	χ^2	13.20	13.42	12.45	9.622	10.52	7.880	7.555	8.101	12.08	11.49
	D	0.041	0.054	0.080	0.082	0.067	0.074	0.098	0.058	0.083	0.063
	FCC	0.993	0.992	0.992	0.992	0.993	0.993	0.993	0.991	0.981	0.992
LP3	χ^2	8.706	8.686	8.799	8.497	8.469	11.48	8.960	8.80	13.168	9.622
	D	0.056	0.046	0.037	0.039	0.039	0.041	0.054	0.076	0.099	0.058
	FCC	0.996	0.997	0.997	0.997	0.996	0.995	0.993	0.99	0.983	0.995
EV1	χ^2	*	*	*	*	*	*	*	*	*	*
	D	*	*	*	*	*	*	*	*	0.140	0.110
	FCC	0.967	0.965	0.962	0.958	0.954	0.954	0.953	0.957	0.971	0.974
Exp	χ^2	*	*	*	*	*	*	*	*	*	*
	D	*	*	*	*	*	*	*	*	*	*
	FCC	0.993	0.991	0.99	0.989	0.988	0.988	0.988	0.988	0.98	0.992
N	χ^2	*	*	*	*	*	*	*	*	*	*
	D	*	*	*	*	*	*	*	*	0.192	*
	FCC	0.889	0.887	0.883	0.876	0.87	0.87	0.872	0.882	0.926	0.913

* indicates a rejected distribution at the 5 % significance level.

The χ^2 and *D* goodness of fit statistics exclude four of the eight distribution candidates at the 500 m^3/s threshold level (Table 5.3). Based on the FCC, the LP3 and GP distributions are the better fit. For a 600 m^3/s threshold, the LN, EV1, Exponential and N distributions were rejected in the χ^2 test at the 5 % level. The remaining distributions are suitable candidates. However, the FCC is clearly lower for GEV distributions. At the 650 m^3/s level four distributions, namely GP, GEV, P3 and LP3 are accepted. The remaining distributions are rejected by either χ^2 or *D* goodness of fit statistics. Results are similar for the 700 m^3/s and 750 m^3/s thresholds. Only the GP, GEV, P3 and LP3 distributions are remaining candidates. At the 800 m^3/s threshold, results are equivalent with those at the 700 m^3/s and 750 m^3/s level, except that the GEV distribution is additionally excluded by the chi-squared test at the 5 % significance level. Only the GP, GEV, P3 and LP3 distributions are valid distribution candidates to describe the events obtained by the 900 m^3/s and 100 m^3/s thresholds. However, the FCC is lower for the GEV distribution than any of the other three candidate distributions (i.e. GP, P3 and LP3) and these threshold levels.

Based on these results of the test statistic, it is inferred that the GEV, GP, LP3 and P3 distributions are suitable candidates to model the magnitude of exceedances in the PDS series at the chosen threshold levels. However, it is noted that the FCC highlights the use of the P3 or LP3 distribution over the other two distributions.

For the AMS the chi-squared test leads to the rejection of the LN, EVI (Gumbel), Exponential and N distributions at the 5 % significance level. Based on the KS test, only the Exponential distribution can be rejected. For the historical annual series, the EV1, N and Exponential distributions are rejected by the chi-squared test and the Exponential and N distribution are further excluded by the KS test as suitable candidate distributions. It is interesting to note that while the LN distribution has been rejected for all other series, it is statistically accepted for the historical AM series. Among the remaining distributions, the FCC leads to the conclusion that the use of either the LP3 for modelling the AMS and AMS_{hist} are appropriate.

5.1.4 Selection of best fitting PDS

The subjectivity involved in selecting one PDS or threshold level is one of the reasons why PDS sampling is seldom applied in practice. The choice of a desired threshold level should not reflect practical advantages, but should consider, first and foremost, accuracy and performance in relation to quantile estimates. One test proposed by Lang et al. (1999) uses the Dispersion Index (DI), as shown in Table 5.4, to choose possible threshold levels. The DI, as proposed by Cunnane (1979) and described previously in Chapter 4, has been calculated for each of the PDS to determine the best distribution for modelling the exceedances above the selected threshold (Table 5.4). With application of this test, the series obtained by the 500 m^3/s and 600 m^3/s threshold level are rejected, as they do not conform to a Poisson process of peak arrivals. However, as discussed earlier, with the new formula of translating the PDS return period into the annual domain (Mohssen, 2009), such a selection criterion is redundant. A further test dictates that the mean number of exceedances per year (λ) should be larger than 2 or 3, based on previous studies investigating the sampling variance of quantiles (Lang et al., 1999). Such a criterion would lead to the conclusion that the optimal threshold levels are 900 m^3/s, 800 m^3/s and perhaps 750 m^3/s. The final criterion proposed by Lang et al. (1999) argues that the optimal threshold level is found where the mean value of exceedances above threshold $E(X_s)$ is a linear function of the chosen threshold level (S). This test suggests a threshold that maximises the stability of the PDS distribution parameter estimates. Figure 5.7 shows a plot of the mean value of exceedances above the threshold as a function of the threshold (green line) and λ as a function of the threshold (blue line). The ideal threshold belongs to the interval [650, 800].

Table 5.4 Calculation of the Dispersion Index (Cunnane, 1979) and the resulting recommended distribution for arrival above threshold modelling.

Threshold	N	Mean*	Variance*	DI	d_{calc}	$(\chi^2_{\alpha/2}, \chi^2_{1-\alpha/2})$	Distribution
500 m^3/s	323	6.571	9.458	1.439 > 1	463.47	274; 374	Negative Binomial
600 m^3/s	252	5.306	7.009	1.321 > 1	331.56	209; 297	Negative Binomial
650 m^3/s	219	4.470	4.713	1.054 > 1	229.85	179; 261	Poisson
700 m^3/s	195	3.980	4.062	1.021 > 1	197.99	157; 234	Poisson
750 m^3/s	178	3.714	4.25	1.144 > 1	202.54	142; 216	Poisson
800 m^3/s	162	3.306	3.800	1.150 > 1	185.06	128; 198	Poisson
900 m^3/s	122	2.490	2.630	1.056 > 1	127.80	92; 153	Poisson
1000 m^3/s	89	1.816	1.778	0.979 < 1	86.16	64; 116	Poisson

* mean and variance of measured exceedance values above threshold

The results indicate, that apart from the exceedances above the 500 m^3/s and 600 m^3/s threshold levels, the average time between flood peaks can be modelled using a Poissonian distribution. None of the series extracted are described by binomial flood count above the threshold.

Looking at chi-squared and FCC test results from Table 5.3 can give further indications of optimal PDS, as these results compare the empirical data series with the expected modelled series. These two test statistics further confirm previous conclusions. Threshold selection should be in the range of 650 m^3/s to 800 m^3/s as these thresholds produce series with the best fit to theoretical distributions. While the focus lies on PDS resulting from these thresholds, PDS with higher or lower peak events will also be included in analyses.

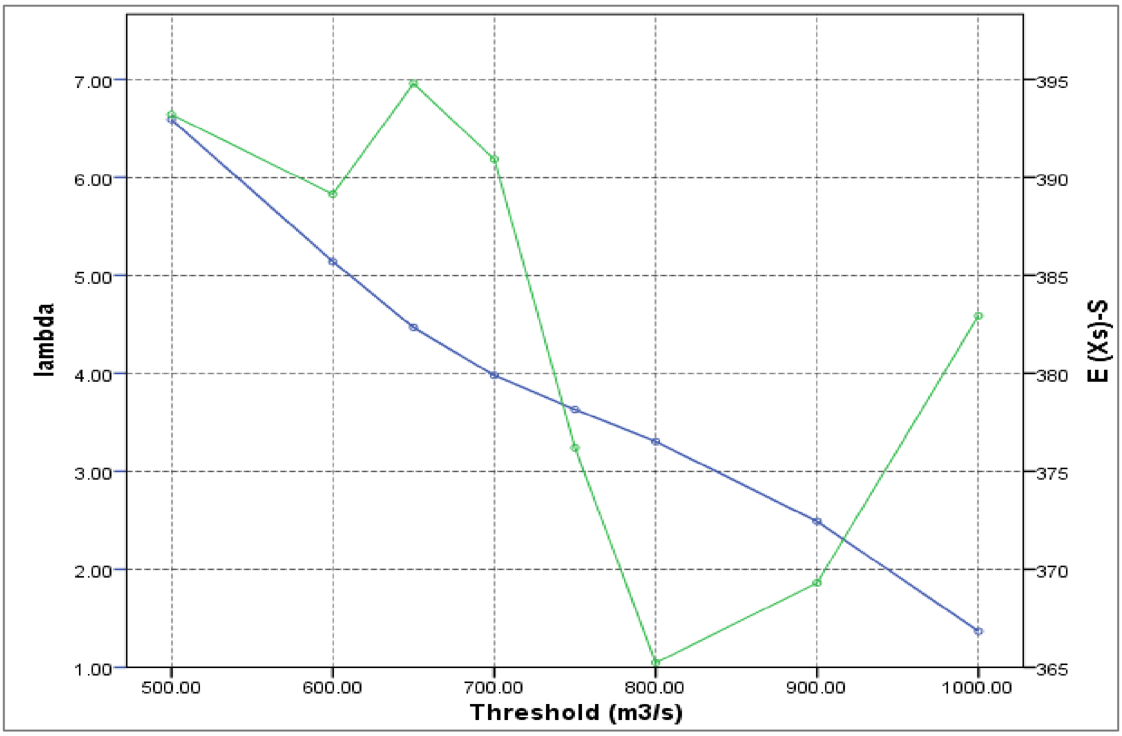


Figure 5.7 Threshold selection test. The green line is the mean exceedance above the threshold, the blue line shows the average number of exceedances per year.

5.1.5 Summary of goodness of fit

The graphical analysis based on histograms, probability plots and quantile plots indicate that the GP, LP3, P3 and GEV distributions are a better fit to the observed data than the EV1 (Gumbel), N, LN and Exponential distributions. The histograms in Figure 4.3 give the strongest indication out of all graphical methods that the Exponential distribution can be excluded from the list of potential candidates. The L-moment ratio diagrams (Figure 5.4) clearly exclude the EV1 (Gumbel) and Normal distributions as possible candidates, which was not evident from histograms or probability plots. The quantile plots also exclude the N, LN distribution. Concurrently, the choice of best fit distribution should not be based on graphical analyses alone. For example, the exclusion of the LN distribution is not apparent from the L-Moment ratio diagram alone. However, the LN distribution has been ruled out by the goodness of fit tests (Table 5.3) from all but one series (i.e. AMS_{hist}). Nevertheless, the LN and Gumbel distribution were added for comparative purposes with other studies that have generated flood quantile estimates for the Waimakariri River.

While a range of PDS have been extracted from the daily streamflow measurement record with differing threshold levels, not all threshold levels produce series with ideal fit to the theoretical distributions employed. Therefore, several tests were employed to conclude that PDS resulting from threshold ranges from 650 m^3/s to 800 m^3/s are ideal for the Waimakariri River discharge series.

5.1.6 Flood frequency quantile estimates

For the AMS, flood quantiles were modelled using the GEV, P3, LP3 and for comparative purposes the EV1 and LN distributions. While the EV1 and LN distributions have been rejected by goodness of fit test statistics, they are included in the analysis as especially the EV1 distribution is a routinely chosen distribution for flood frequency modelling internationally and in New Zealand. The magnitude of exceedances in PDS was modelled using the GEV, GP, P3, and LP3 distribution and for comparative purposes, LN and EV1 distributions. Quantile estimates were produced for 3, 4, 5, 10, 25, 50, 100, 250 and 500 year return periods.

The Gumbel and LN Q10 quantile estimates are lower than the remaining better fitting distributions' estimates (Figure 5.8). For the AM series, the Gumbel and LN Q10 estimates are 2105 m^3/s and 2062 m^3/s respectively, while a better fitting distribution, such as the GP distribution, gives an estimate of 2191 m^3/s. It is evident from Figure 5.8 that the greater the number of events included in the series, the greater the variance between the estimates of different distributions becomes. At the 500 m^3/s threshold, the difference between the Q10 estimate generated by the Gumbel distribution and the estimate generated by the LP3 distribution is 384 m^3/s, a 17 % difference.

The difference between estimated design floods is more pronounced at the 100-year return level, which is illustrated in Figure 5.9. The AMS_{hist} generated the highest discharge for each distribution. For the AM series, the 100-year design flood is 3078 m^3/s estimated by the Gumbel distribution, and 3763 m^3/s, estimated by the LP3 distribution. The largest estimate is generated by the LP3 distribution with the inclusion of 323 events, corresponding to a threshold level of 500 m^3/s. This estimate is 4156 m^3/s for the 100-year design flood. The difference between the Gumbel and LP3 estimate at this point is 1606 m^3/s, or 39 %. It is interesting to note, that the PDS estimates for the 100-year flood event using the LN, Gumbel and P3 distributions are smaller than for the AM series. The PDS estimates using the GP, GEV and LP3 distributions are slightly larger at the 900 m^3/s and 800 m^3/s threshold levels.

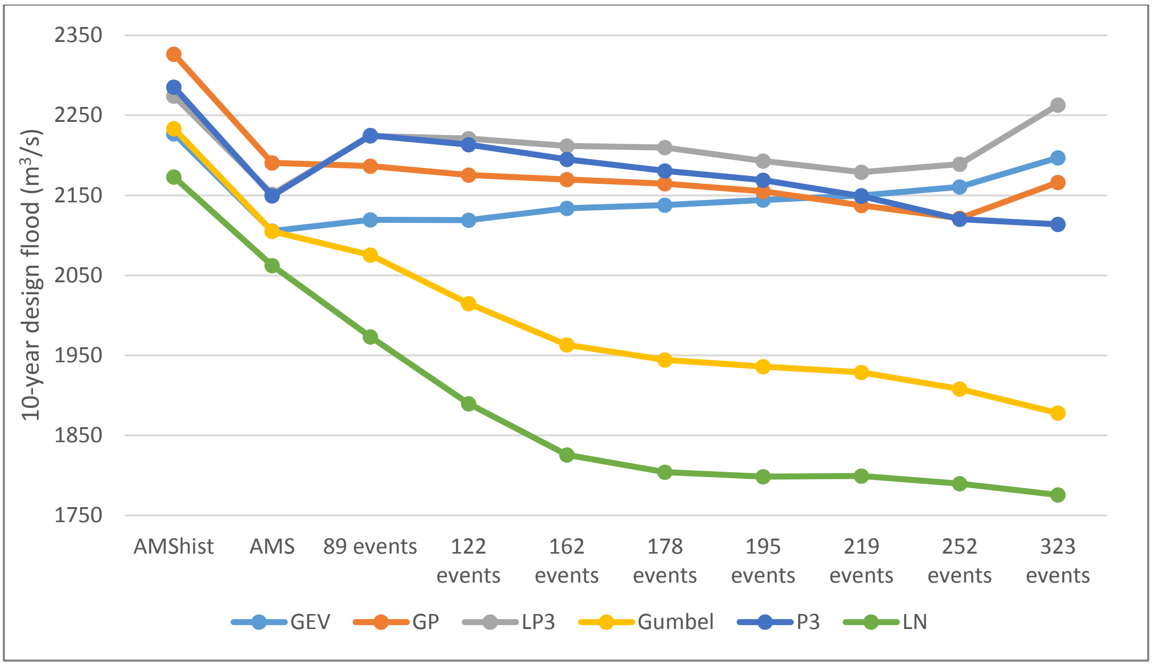


Figure 5.8 10-year design flood estimate for selected distributions and thresholds.

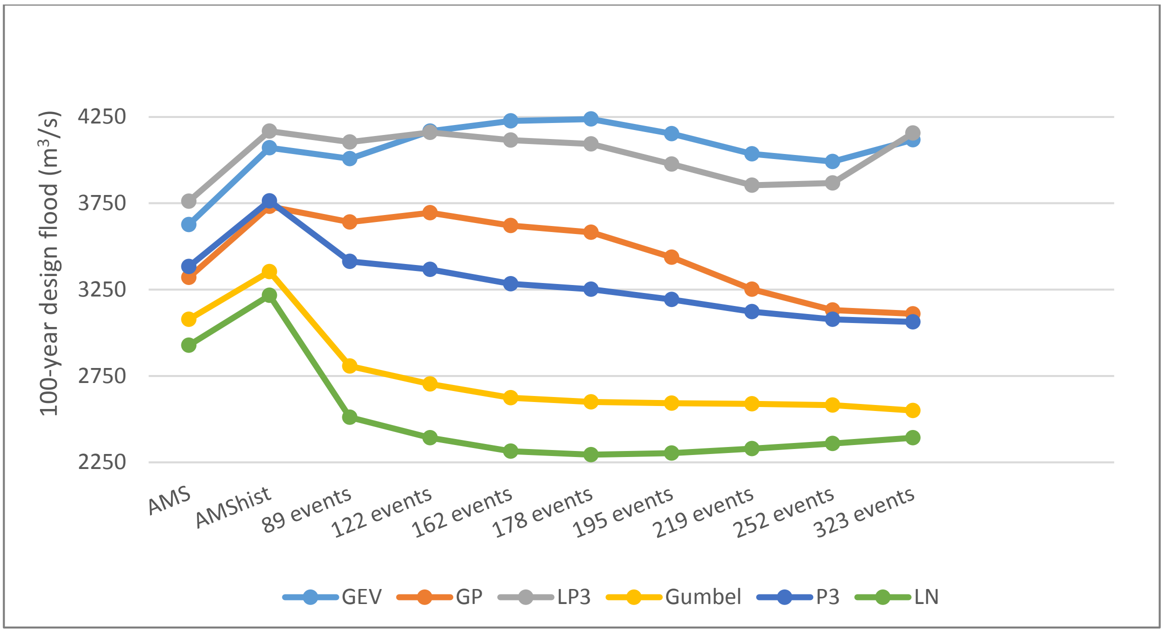


Figure 5.9 100-year design flood estimate for selected distributions and thresholds.

The variance in estimates increases once again for the 500-year design discharge (Figure 5.10). For the AM series, the Gumbel distribution gives an estimate of 3730 m^3/s, while the LP3 distribution generates 6086 m^3/s. The highest estimate in the chosen series is generated with the inclusion of 122 flood events, corresponding to the 900 m^3/s threshold, for the LP3 and GP distributions. The GEV distribution estimates the highest design flood with the inclusion of 162 events, i.e. the 800 m^3/s threshold level. For the remaining three distributions, the AMS_{hist} series returns the highest flood discharge for the 500-year return period.

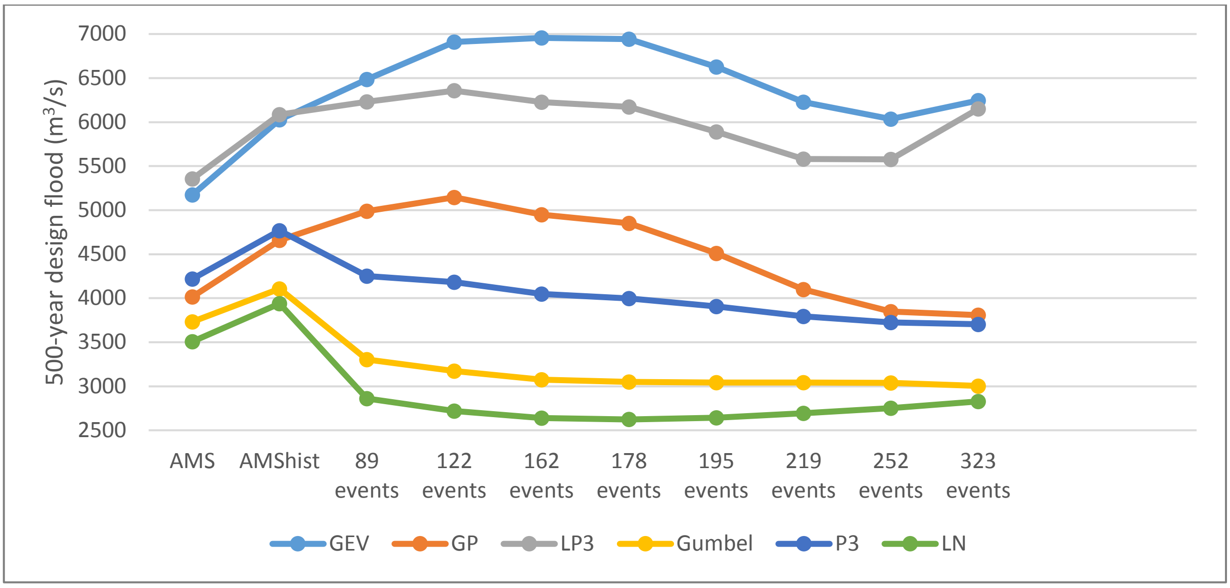


Figure 5.10 500-year design estimate for selected distributions and thresholds.

Figure 5.11 shows design discharge plotted as a function of return period. It is equally evident from this figure that an increase in counted flood events results in higher design flood estimates for each quantile. The AM series with 49 years produces the lowest estimates, while the series with 162 and 122 events produce the highest estimates. Overall, lowering the threshold level, i.e. increasing the number of events in the series, does not equate to higher estimated design discharge. The same conclusion can be reached for the estimates produced by the LP3 distribution (Figure 5.12).

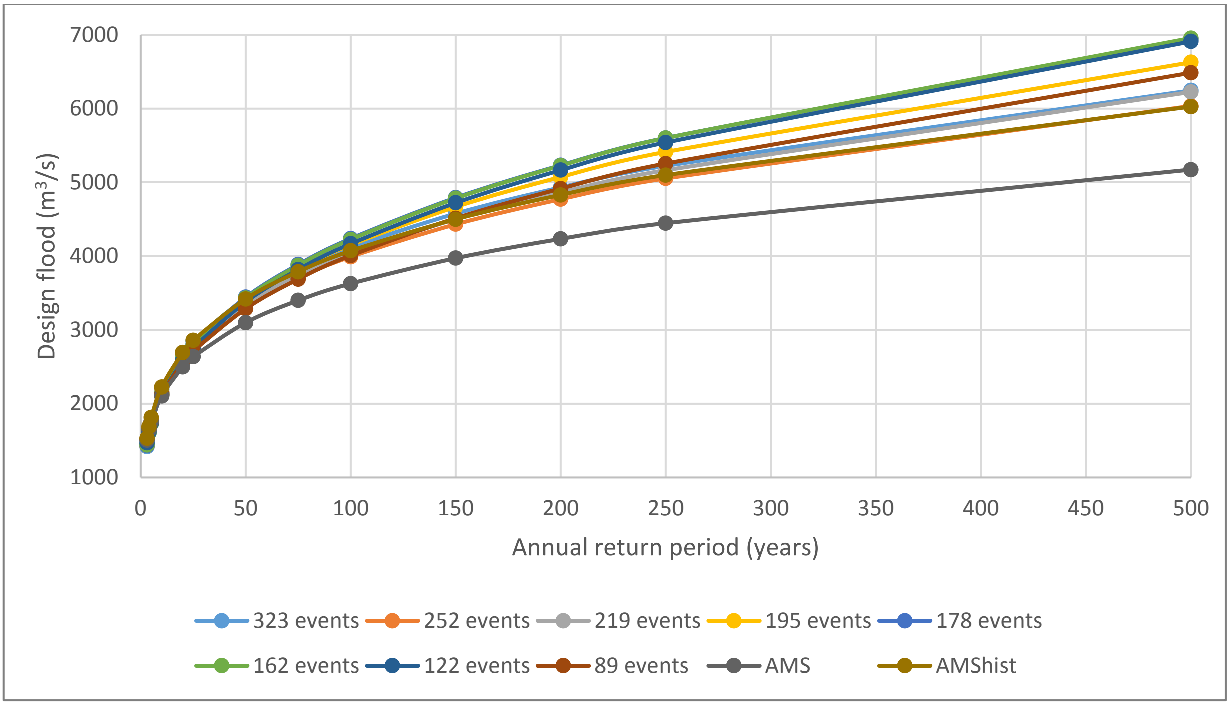


Figure 5.11 GEV flood discharge estimates for selected return periods.

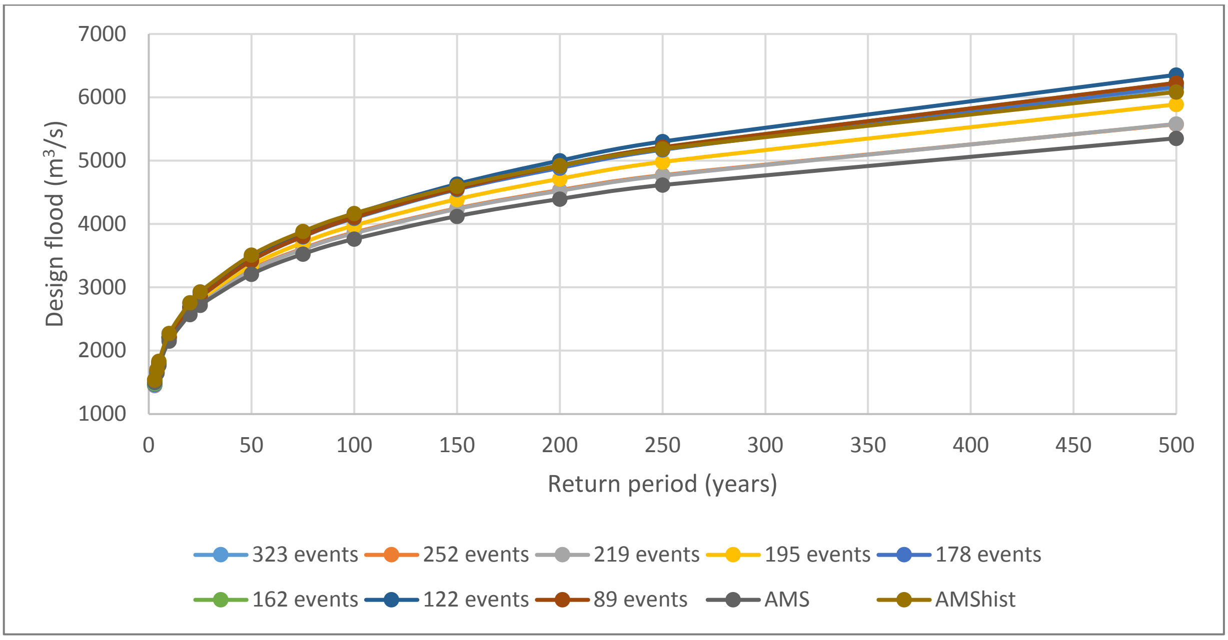


Figure 5.12 LP3 flood discharge estimates for selected return period.

The quantile estimates generated by each distribution/ threshold level combination were pooled together to produce the boxplots shown in Figure 5.13. For example, the GEV Q5 estimate generated by each threshold level were grouped together to produce the first boxplot in the figure, showing the four quartiles. The mean value is indicated by a red asterisk in each boxplot. At a first glance, the boxplots show that the majority of the data are negatively skewed, with the mean located in the first quartile, i.e. below the median or second quartile. Furthermore, the variance of the boxplots increases with increasing quantile estimates. This is a result of different distributions' ability to model events in the extreme tails, especially for high quantiles, such as the 500-year event. The Gumbel (EV1) distribution consistently provides the lowest estimated design flood discharge, while the GEV and LP3 distributions give the highest design flood discharge estimates at Q25 and higher. The Gumbel distribution has the smallest interquartile range (IQR). The IQR of the remaining four distributions becomes larger with higher quantiles. This indicates that while for estimates of smaller quantiles the selection of the threshold level may not be central, the selection of the adequate threshold level for larger estimates (Q25 and larger) has a significant influence on the calculated design flood level for the GP, LP3, P3 and GEV distributions.

In order to evaluate whether design flood estimates are statistically different across distributions, the central position of the model outputs was compared using ANOVA. To verify the underlying assumption of homogeneity of variances, the Levene's test was performed for each quantile comparison. The results of the Levene's test are tabulated in Table 5.5.

Table 5.5 Results of the Levene's test for H_0 = homogeneity of variance. *df1* = 4, *df2* = 35, α = 0.05.

Group	Levene's statistic	*p*	Conclusion
Q5	4.461	0.005	$p < \alpha$, reject H_0
Q10	2.248	0.084	$p > \alpha$, accept H_0
Q25	1.762	0.159	$p > \alpha$, accept H_0
Q50	3.931	0.010	$p < \alpha$, reject H_0
Q100	5.470	0.002	$p < \alpha$, reject H_0
Q500	7.877	<0.001	$p < \alpha$, reject H_0

In the case that the assumption of homogeneity of variance is violated, i.e. the H_0 is rejected, the Welch test for unequal variances, also referred to as the unequal variances t-test, was performed in lieu of ANOVA. For post-hoc exploratory testing the Tukey's HSD test for ANOVA and the Games-Howell test for the Welch test were used.

According to the Welch test, there is a significant difference between the Q5 estimates modelled by the five selected distributions [$F(4, 16.82) = 25.572$, $p < 0.001$]. Post-hoc test results are tabulated in Table 5.6. The Games-Howell test results indicate that the mean estimate produced by the GEV distribution was significantly different from the estimates produced by the LP3, P3 and GP distributions ($p < 0.001$ for all tests). However, the mean values produced by the GEV and EV1 distributions did not differ significantly ($p = 0.994$). No significant difference can be reported between LP3 and GP/EV1 estimates ($p = 1.0$ and $p = 0.117$ respectively). The P3 mean design flood estimates are significantly different than results produced by any of the other distributions.

Table 5.6 Games-Howell post-hoc testing for Q5 estimates.

(I) Distribution	(J) Distribution	Mean Difference (I-J)	Std. Error	Significance	95 % Confidence Interval	
					Lower Bound	Upper Bound
GEV	LP3	-53.10025*	7.34547	.000	-76.0042	-30.1963
	P3	-99.88663*	12.68737	.000	-142.0726	-57.7007
	GP	-53.53500*	6.24796	.000	-73.2169	-33.8531
	EV1	8.31963	21.56642	.994	-66.6692	83.3084
LP3	GEV	53.10025*	7.34547	.000	30.1963	76.0042
	P3	-46.78638*	12.83849	.029	-89.1644	-4.4083
	GP	-.43475	6.54940	1.000	-21.1808	20.3113
	EV1	61.41988	21.65567	.117	-13.5900	136.4298
P3	GEV	99.88663*	12.68737	.000	57.7007	142.0726
	LP3	46.78638*	12.83849	.029	4.4083	89.1644
	GP	46.35163*	12.24370	.030	4.5706	88.1326
	EV1	108.20625*	23.99962	.006	30.5261	185.8864

GP	GEV	53.53500*	6.24796	.000	33.8531	73.2169
	LP3	.43475	6.54940	1.000	-20.3113	21.1808
	P3	-46.35163*	12.24370	.030	-88.1326	-4.5706
	EV1	61.85463	21.30843	.112	-13.1179	136.8272
EV1	GEV	-8.31963	21.56642	.994	-83.3084	66.6692
	LP3	-61.41988	21.65567	.117	-136.4298	13.5900
	P3	-108.20625*	23.99962	.006	-185.8864	-30.5261
	GP	-61.85463	21.30843	.112	-136.8272	13.1179

*difference significant at the 0.05 level.

As the assumption of homogeneity of variances is met for the design flood estimates at the 10-year return period level, ANOVA and subsequently Tukey's HSD testing can be performed. There is a statistically significant difference between distributions as determined by one-way ANOVA [$F(4, 35) = 51.716$, $p < 0.001$] for the Q10 quantile estimates. Tukey's HSD test results are available in Appendix B. The test results indicate that the mean estimate produced by the EV1 distribution is significantly different from the other distributions' results ($p < 0.001$ for all tests). However, P3 results do not differ significantly from results obtained by the GEV, LP3 and GP distributions ($p = 0.688$, $p = 0.250$, $p = 0.899$ respectively).

ANOVA and Tukey's HSD test were used for the 25-year return period estimates, as homogeneity of variance was present (Table 5.5). One-way ANOVA determined a statistically significant difference in means between the distributions [$F(4, 35) = 142.741$, $p < 0.001$]. Tukey's HSD test results for Q25 results are available in Appendix B. Test results indicate that the mean of the LP3 and GEV distributions are not significantly different ($p = 0.350$) and that equally, the means of the P3 and GP distribution are not significantly different ($p = 0.897$). All other distributions are significantly different ($p < 0.001$ for all tests).

For the remaining quantile estimate comparisons, i.e. Q50, Q100 and Q500, variances are non-homogeneous (Table 5.5) and therefore, the Welch test and Games-Howell test were used. The Welch test determined that the chosen distributions are significantly different in all quantile estimates ($p < 0.001$ for all tests). The GEV Q50 and LP3 distribution means, and P3 and GP distribution means are not significantly different ($p = 0.999$ and $p = 0.428$, respectively). For the tabulated results, the reader is referred to Appendix B. Results are similar for the Q100 estimates ($p = 0.658$ and $p = 0.245$, respectively). Means for the Q500 estimates are significantly different, except for P3 and GP distribution means ($p = 0.113$).

Overall the test results confirm what is visible in Figure 5.13. The EV1 distribution is significantly different from all other selected distributions, except for the Q5 estimates. In all pooled groups apart from Q5, the difference between LP3 and GEV, and P3 and GP is not significantly different. The group difference among LP3/GEV and P3/GP, however, is significant.

5.1.7 Summary of flood frequency quantile estimates

Design flood quantiles have been produced for four of the best fitting distributions, i.e. LP3, P3, GEV and GP, and for two rejected distributions, i.e. Gumbel and LN. The Gumbel and LN distributions consistently produce the lowest design flood estimate for each of the example quantiles used in this study. On the other hand, the LP3 and GEV distributions provide the highest design quantile estimates as a rule. It is evident from the figures provided (Figure 5.8, Figure 5.9 and Figure 5.10), that the absolute difference between the lowest and highest theoretical estimate increases with increasing quantile. Furthermore, the AMS_{hist} series generates the largest estimate of all series due to the inclusion of three significant historical events that are not included in the modern record that has been used for the other series (1967-2015). The boxplots (Figure 5.13) confirm that the four best fitting distributions are significantly different from each other. The GEV and LP3 distributions generate statistically similar design estimates as a rule; as do the P3 and GP distributions. All four distributions that have been shown to provide a good fit differ significantly from the Gumbel distribution. At the same time, the Gumbel distribution shows the lowest variance between estimates for all quantiles, while the other four fitting distributions show varying degrees of variance. The highest variance between estimates occurs for the 500-year quantile. The increasing variance between estimates of the four fitting distributions with increasing quantiles shows that the choice of the ideal PDS is vital for accurate design estimates of flood discharges.

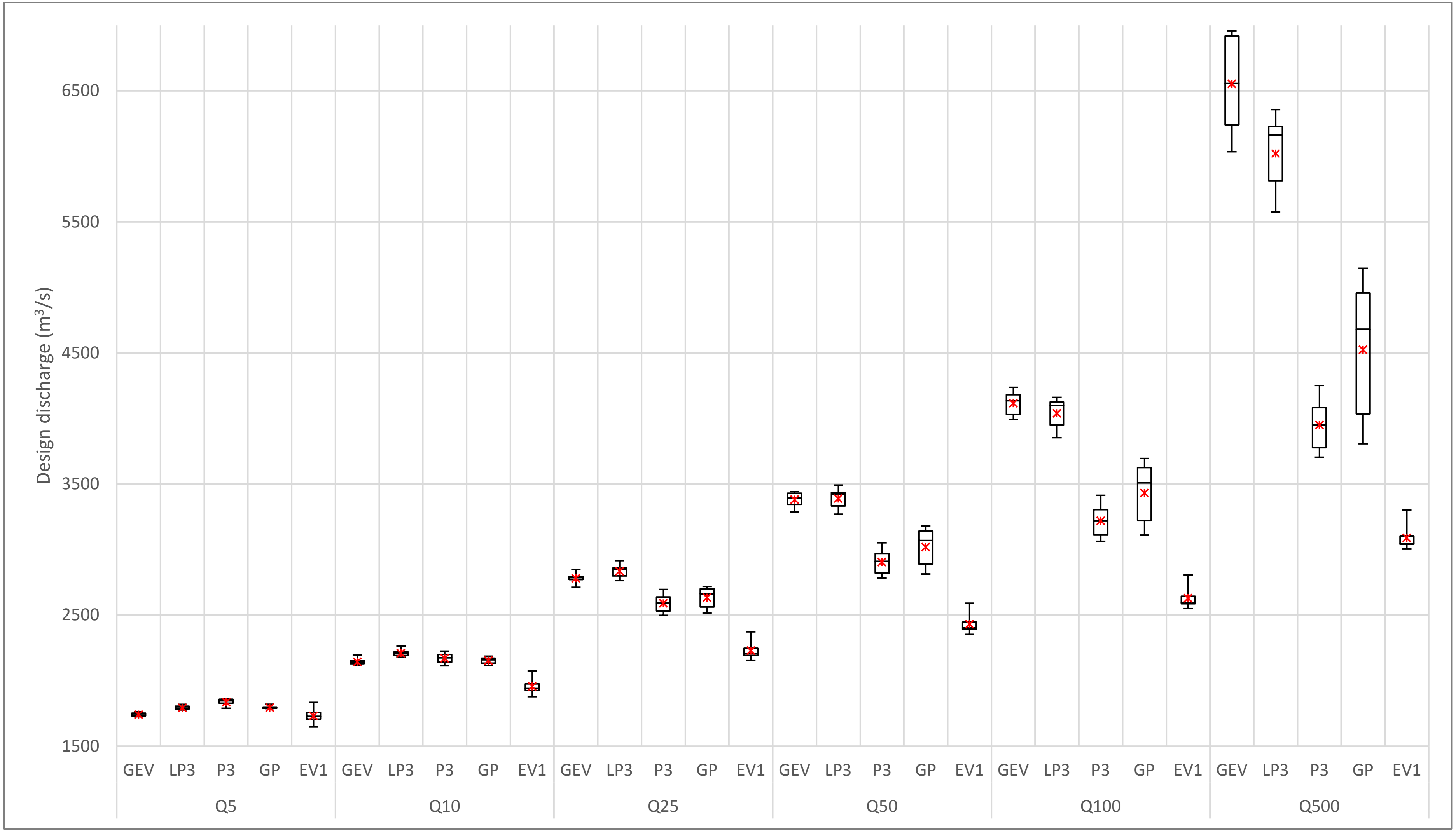


Figure 5.13 Boxplots of quantile estimates based on PDS. Note: PDS estimates are pooled together for each distribution and quantile. The red asterisk indicates the mean of estimates.

5.2 Low flow frequency analysis

5.2.1 Data selection

For the low flow frequency analysis, continuous hourly streamflow measurements from the Waimakariri River at the OHB site (1967-2015) were used. The extraction of annual minima yielded 49 events for 49 years on record. Annual low flow measurements are typically recorded as averaged measurements, such as the seven day mean annual low flow (7dMALF). Instantaneous minimum measurements and the 7dMALF measurements are tabulated in Appendix C. The lowest instantaneous measurement since 1967 has been 22.03 m^3/s, recorded in 1971. The highest instantaneous annual minimum measurement is 55.26 m^3/s, recorded in 1998. Concurrently, the lowest 7dMALF measurement is 22.45 m^3/s in 1971, and the highest 7dMALF was measured in 1996 with 59.28 m^3/s. Information about low flows in the Waimakariri River before 1967 is sparse at best. While a separate flood frequency analysis using data from 1930-2015 was possible due to the availability of historical annual maximum recordings form 1930-1966, analysis of low flows only uses data from 1967 onwards.

Both, the annual minimum series and the 7dMALF series were tested for serial dependence with the aid of autocorrelation plots. Figure 5.14 shows no significant autocorrelation and thus serial independence of the data.

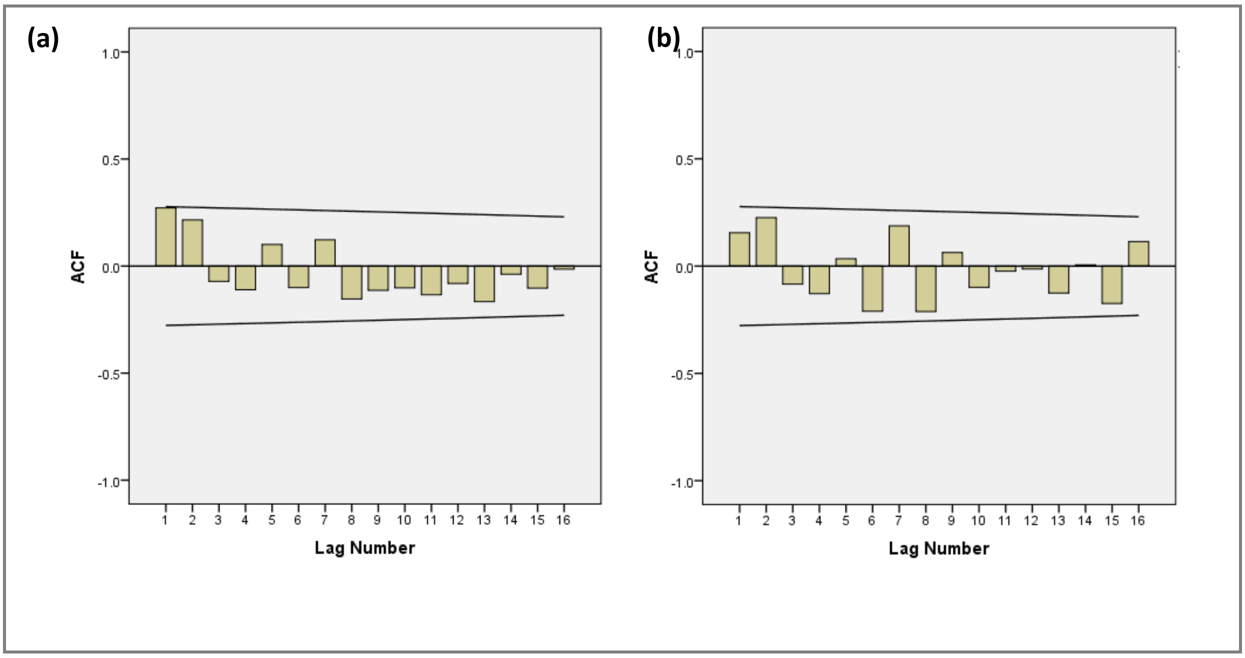


Figure 5.14 Autocorrelation plot of (a) sample annual minima (1967-2015) vs. time lags and (b) 7dMALF vs. time lags (1967-2015). The black lines represent the 95 % confidence limit.

No significant trend was detected in either of the series by Kendall's tau (τ_a) and Spearman's Rank Correlation (ρ) at α = 0.05, with a smaller calculated value (τ_a, ρ) than the tabulated critical value (p) [(a) $\tau_a = -0.110$, $p = 0.266$; $\rho = -0.159$, $p = 0.276$, (b) $\tau_a = -0.0063$, $p = 0.524$; $\rho = -0.073$, $p = 0.618$].

For the extraction of PDS of low flow events, six thresholds were applied, which are as follows; 60 m^3/s, 50 m^3/s, a threshold set at the 90^{th} percentile (45 m^3/s), a threshold set at the 7dMALF (40 m^3/s), 35 m^3/s, and a threshold set at the 95^{th} percentile (27 m^3/s). The thresholds applied and resulting number of troughs are summarised in Table 5.7. The analysis of low flow events has the added dimensions of run length and deficit. Therefore, the resulting series contain the lowest flow of selected events, the calculated deficit below the threshold and the run length.

For the selection of low flow events, independence criteria were applied as low flow events closely following one another may in fact be mutually dependent minor events. Low flow events with run lengths of fewer than 24 hours were removed from the series, as these are deemed very minor drought events with a non-significant influence on the larger *Q-T* estimates (Zelenhasic & Salvai, 1987). This also simplified the curve fitting application. Zelenhasic and Salvai (1987) suggested the removal of events that are 0.5 - 1 % of the maximum deficit on the series. However, Madsen and Rosbjerg (1995) argued that a percentage based on the maximum deficit is prone to outliers and therefore the removal of an unnecessary number of smaller events. The authors suggested the use of the mean deficit instead. Therefore, following the removal of events with fewer than 24 hours, events with deficit of 2.5 % of the average deficit of a series were also excluded.

Table 5.7 Applied thresholds and the resulting number of events extracted from the stream record. λ is the average exceedance rate below the threshold.

Threshold	Number of troughs identified	Number of low flow events	λ
60 m^3/s	661	425	8.67
50 m^3/s	421	249	5.08
45 m^3/s	309	176	3.59
40 m^3/s	217	122	2.49
35 m^3/s	169	70	1.43

Due to the low number of events below the 27 m^3/s threshold, the PDS was excluded from further analyses. Trend testing was done with Kendall's tau and Spearman Rank Correlation testing. This was done for PDS of minimum flows, run lengths and deficits for all threshold levels. Test results are tabulated in Appendix C.

Test results show statistically significant trend detected in the PDS of lowest values with thresholds 35 m^3/s, 45 m^3/s, and 50 m^3/s at $\alpha = 0.05$. Significant trend was also detected in the PDS of deficits with threshold 35 m^3/s. And lastly, statistically significant trend was detected in the PDS of duration with the threshold 35 m^3/s (Appendix C). This directly violates the assumption of stationarity for the further frequency analysis of the low flow data. Such non-stationarity can arise from a number of reasons, primarily from climatic factors such as atmospheric circulation patterns and temperature anomalies, but also from human activities, such as water abstractions and land use changes. The magnitude of each factor is hard to account for in time series analyses and subsequently this study does not attempt to conclusively attribute changes in the time series to human activities or climate factors. However, for the further analysis of low flow frequencies and in order to obtain unbiased estimates for design discharges, stationarity is required. Non-stationary frequency analyses have been previously considered in the literature (Liu, Guo, Lian, Xiong, & Chen, 2014). Censored series of time series displaying non-stationarity can be thus used for further investigation.

The Pettitt change point test (Pettitt, 1979) was used to define the break point in series that displayed non-stationarity. This was performed in *R i386 3.2.0* (R. Development Core Team, 2010) using the *trend* package *pettitt.test()* function (Pohlert, 2016). Test results are listed in Table 5.8. In each PDS of low flows, duration and deficits that failed the Mann-Kendall and Spearman's Rank Correlation test at the 5 % significance level, a break point was detected at *t'*. The subsequent trend test of the obtained subsets of the series, Q_1 and Q_2, showed no significant trend at the 5 % significance level (Table 5.8).

Table 5.8 Test results for Pettitt change point test and subsequent trend testing of subset series by Mann-Kendall test and Spearman Rank Correlation.

PDS	Detected break point, $\alpha = 0.05$	Stationarity of Q_1, Q_2; $\alpha = 0.05$
$PDS_{35\_lowest\ value}$	$t' = 11$, $p = 0.03502$	Invalid as $\lambda < 1$
		Q_2: [$\tau_a = 0.034$, $p = 0.704$; $\rho = 0.037$, $p = 0.780$]
$PDS_{35\_deficit}$	$t' = 35$, $p = 0.0218$	Invalid as $\lambda < 1$
		Q_2: [$\tau_a = 0.130$, $p = 0.274$; $\rho = 0.178$, $p = 0.306$]
$PDS_{35\_duration}$	$t' = 35$, $p = 0.02846$	Invalid as $\lambda < 1$
		Q_2: [$\tau_a = 0.013$, $p = 0.910$; $\rho = -0.009$, $p = 0.961$]
$PDS_{45\_lowest\ value}$	$t' = 22$, $p = 0.02979$	Invalid as $\lambda < 1$
		Q_2: [$\tau_a = -0.019$, $p = 0.726$; $\rho = -0.024$, $p = 0.771$]
$PDS_{50\_lowest\ value}$	$t' = 147$, $p = 0.01237$	Q_1: [$\tau_a = -0.024$, $p = 0.661$; $\rho = -0.042$, $p = 0.615$]
		Q_2: [$\tau_a = 0.048$, $p = 0.381$; $\rho = 0.075$, $p = 0.352$]

5.2.2 Selection of series

The results obtained from the change point analysis are equally visible in graphs of the selected series. Figure 5.15 shows the series of low flows, deficits and durations obtained by the 35 m^3/s threshold. The break points in the deficit and duration series are especially noticeable, with non-stationarity in variance and mean. Figure 5.16 contains a combination of graphs of the series obtained by the 40 m^3/s threshold, showing the relationship between the lowest value in the top section and durations in the bottom section. Low flow duration coincides with the lowest value obtained, and both are in turn related to the total recorded deficit. The two extreme low flow events in Figure 5.16 at $t = 5$ and $t = 15$ are both extreme events that occurred in 1971 and 1973 respectively, both with durations > 1500h, or > 62 days below 40 m^3/s. These two events are also identifiable in the series of Figure 5.16 at $t = 2$ and $t = 9$. However, the lower threshold enlarged these events in their magnitude in comparison with Figure 5.16, as events below 35 m^3/s are rare and generally short. The duration of events with increasing thresholds is increasing and as such, the magnitude of the extreme events in Figure 5.16 decreases in relation. This is of notable importance, as these extreme events below selected (low) thresholds are the reason for the non-stationarity observed in some series. With this in mind, the following thresholds and obtained series are used for further analyses:

- PDS of lowest discharge, deficit and duration below the 60 m^3/s threshold
- PDS of lowest discharge, deficit and duration below the 40 m^3/s threshold
- Censored PDS of lowest discharge, deficit and duration below the 35 m^3/s threshold

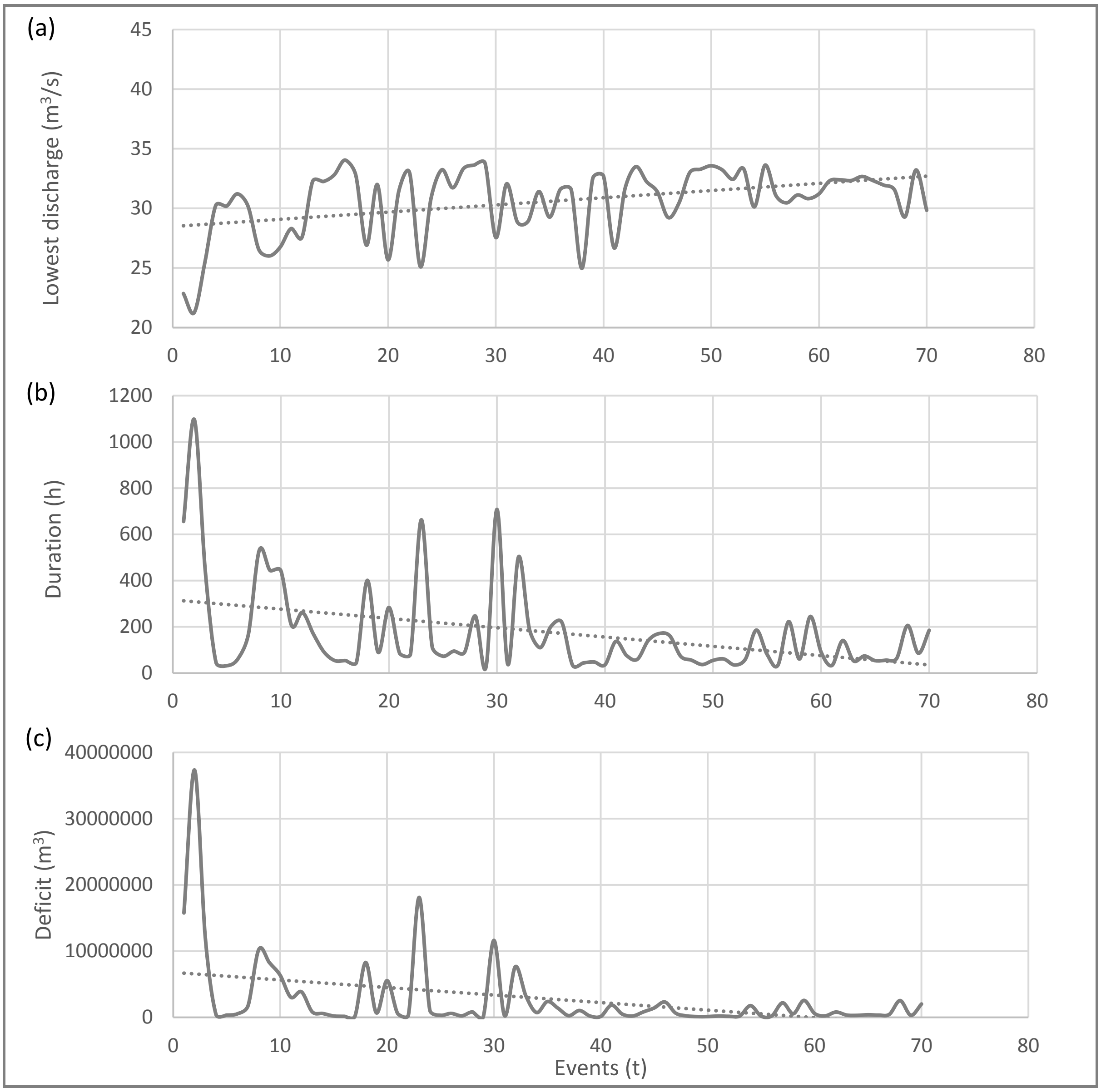


Figure 5.15 Partial duration series of events obtained below the 35 m^3/s threshold. Series of observed (a) lowest value, (b) duration, and (c) deficit.

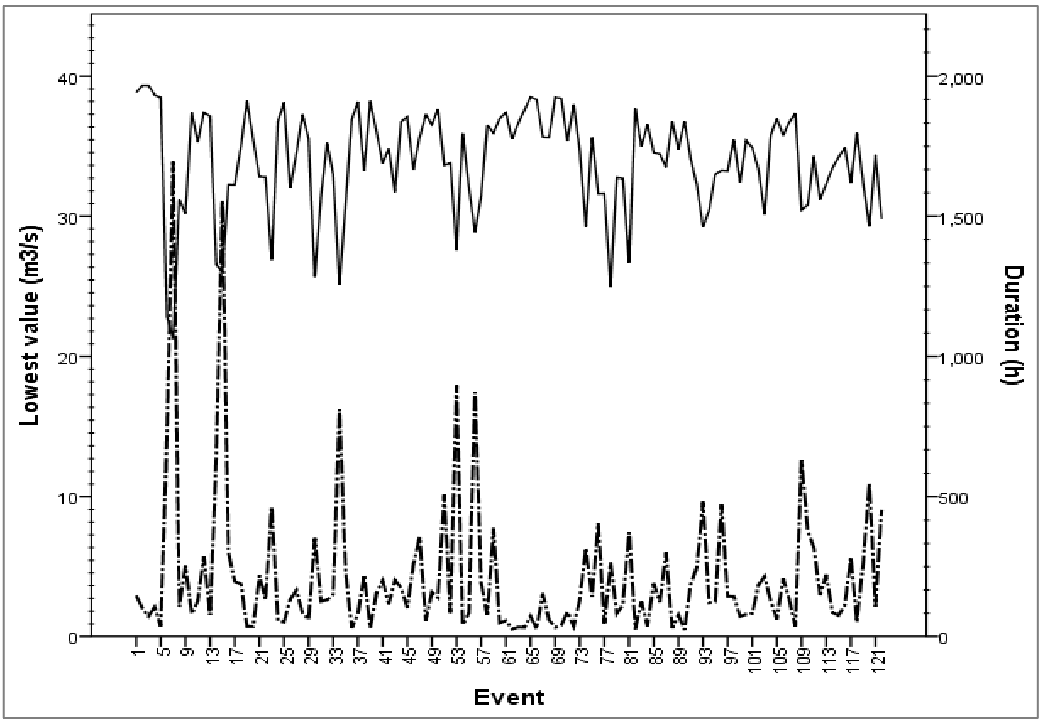


Figure 5.16 Series of duration and lowest observed discharge below the 40 m^3/s threshold. The dashed line represents the duration of events, the solid line represent the series of the lowest values.

5.2.3 Parameter estimation

For the analysis of low flow events, seven distributions are considered. The parameters of each distribution were estimated using the L-moments approach. Calculated L-moments and L-moment ratios for the annual minimum series, the 7dMALF series and each PDS (respective series of lowest value, duration and deficit) are listed in Appendix C.

5.2.4 Selection of best fitting distributions

Graphical selection

L-moment ratio diagrams

The use of L-moment ratio diagrams has previously been described in the presentation of the flood frequency results (5.1.3 L-ratio diagrams). As low flows are typically characterised by several dimensions (i.e. duration of low flows, total deficit, and lowest value of the event), several L-moment ratio diagrams are presented below. In addition, L-moment ratios of unmodified and censored series below 35 m^3/s are presented in comparison.

Lowest value

The L-moment ratio diagram in Figure 5.17 shows that the Exponential and N distributions can be excluded as the best fit for all series. The AMS and 7dMALF series fit most closely with a Weibull distribution (EV Type 3), or a P3 distribution. The PDS series obtained from the 60 m^3/s threshold is best described by a GP distribution, while the censored 35 m^3/s Q2 threshold series seems to fit equally well with a P3, GP or LN distribution. The non-stationary 35 m^3/s series is best described by the GP or P3 distribution. The 40 m^3/s threshold series lies between the LN and P3 distribution.

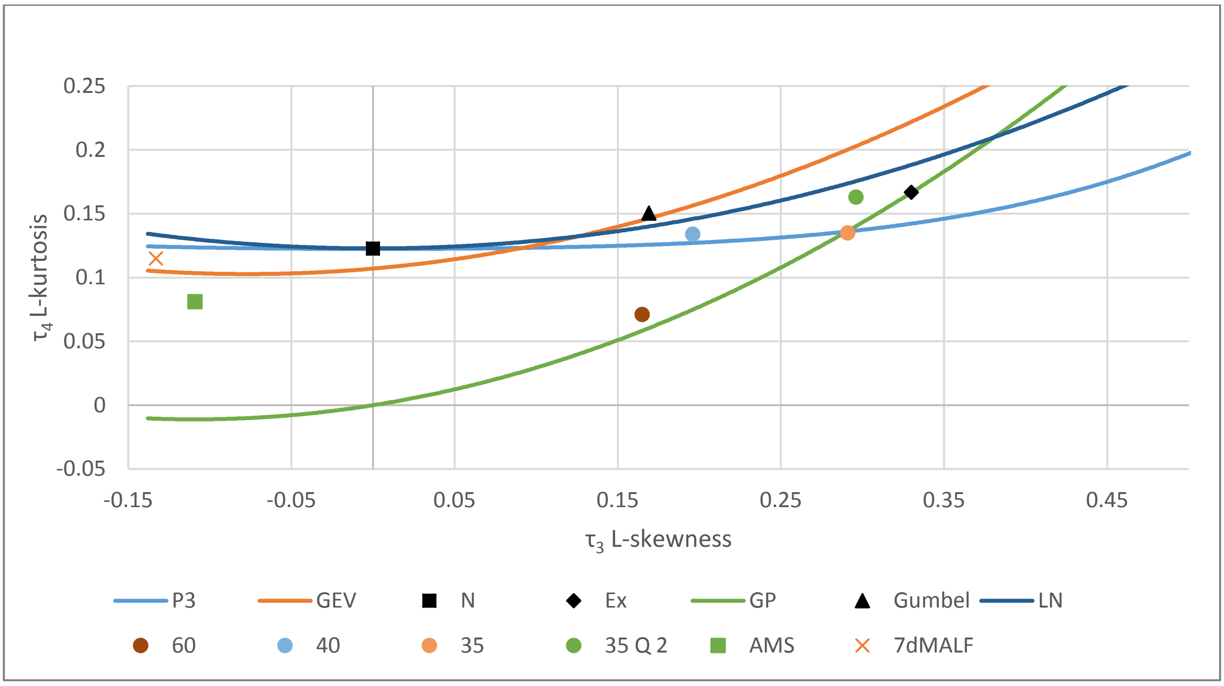


Figure 5.17 L-moment ratio diagram of low flow series obtained by selected thresholds (lowest observed flow).

Duration

The L-moment ratio diagram of low flow durations (Figure 5.18) leads to the exclusion of the Gumbel and Exponential distributions as good fits. All of the series plot between the P3/GEV (with Weibull tendencies) and GP distributions. There is a large difference between the position of the unmodified, non-stationary 35 m^3/s series and the censored 35 Q2 series. The 35 Q2 series plots furthest away from all distributions.

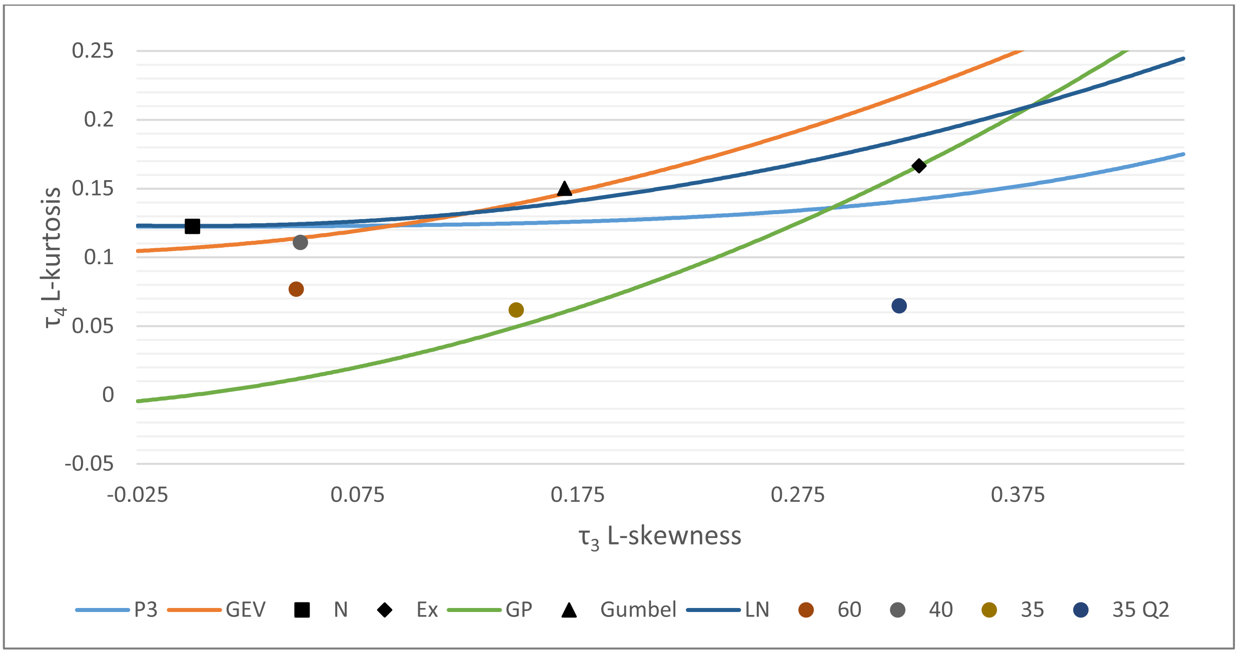


Figure 5.18 L-moment ratio diagram of low flow durations for selected thresholds.

Deficit

The Gumbel and Exponential distribution can be excluded as a good fit for any of the deficit series obtained by selected thresholds (Figure 5.19). The AMS and 7dMALF series appear to be closest to the LN distribution. Apart from the deficit series obtained by the 40 m^3/s threshold, which plots closest to the GEV and P3 distributions, all series are dispersed between the GEV and GP distributions. The censored 35 m^3/s threshold series is plotted in relation to the unmodified, non-stationary 35 m^3/s threshold series. The large distance between the two 35 m^3/s series indicates that the separated series is markedly different from the unmodified series, indicative of significant changes in the processes governing the low flow deficits over time. However, both series are most closely described by a GP distribution.

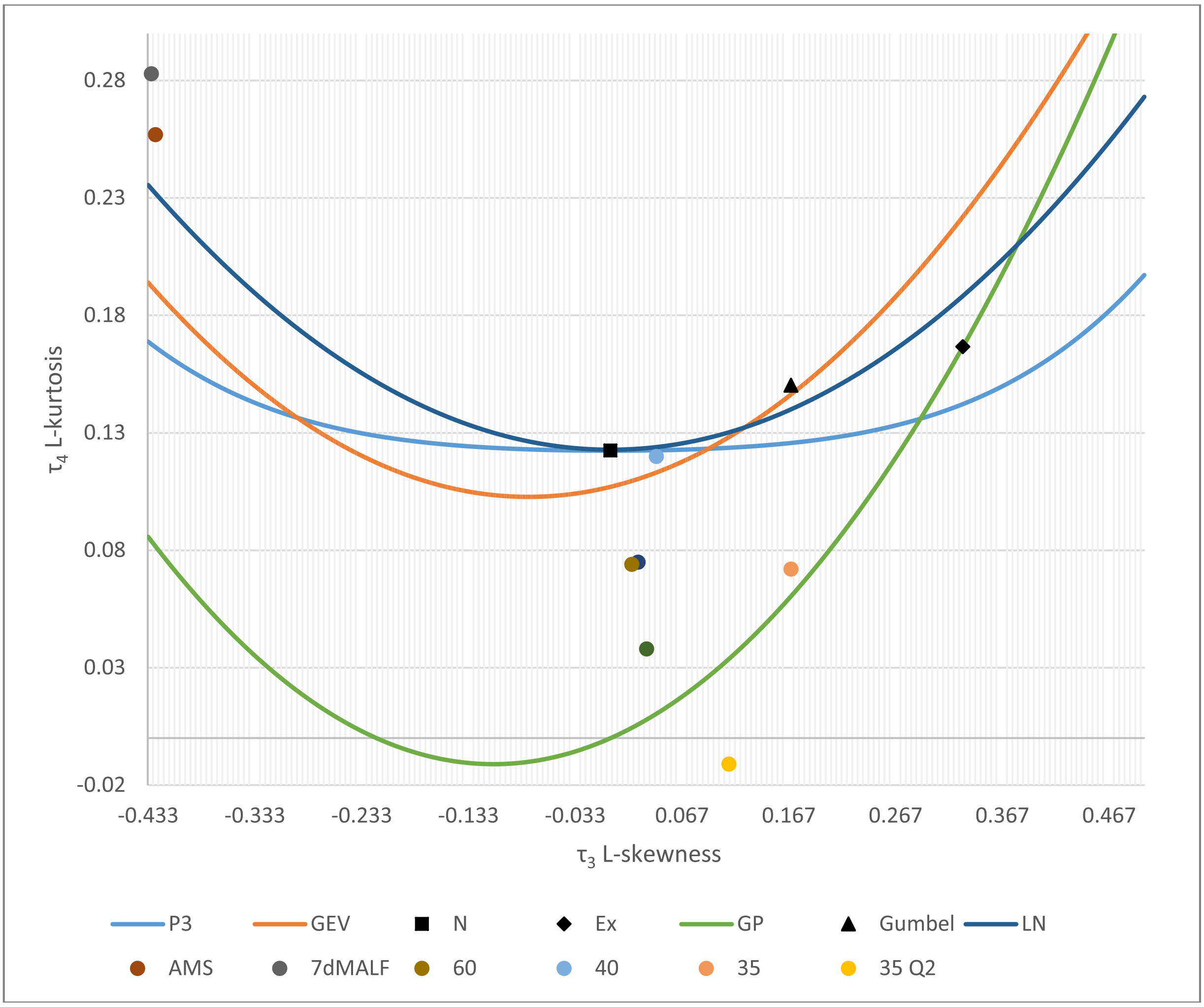


Figure 5.19 L-moment ratio diagram of low flow deficits for selected thresholds.

For a better overview, Table 5.9 summarises the best fitting distribution for each series, as based on a subjective judgement of the L-moment ratio diagrams. It is clear that the Exponential, EV1 and N distributions can be excluded as best fit candidates for any of the PDS extracted from the low flow record.

Table 5.9 Best fitting distributions as identified by L-moment ratio diagrams.

Series	Lowest flow	Duration	Deficit
60	GP	GP, GEV	GP, GEV
40	LN, P3	GEV	GEV, P3
35 Q2	GP, P3, LN	GP, P3	GP
AMS	GEV, P3	-	-
7dMALF	GEV, P3	-	-

Probability plots

PP plots comparing the theoretical and empirical probabilities of various PDS are available in Appendix C. Table 5.10 lists the distributions that can be excluded as a good fit based on subjective visual examination of PP plots. The Exponential and N distributions were not included in PP plots, as they have been ruled out by visual examination of the L-moment ratio diagrams. The PP plots agree with the majority of the findings of the L-moment ratio diagrams. It is evident from the table that the Gumbel distributions can be excluded as good fits for any of the obtained series. It can also be seen that the P3 distribution does not fit well with series of observed deficits.

Table 5.10 Excluded distributions as identified from PP plots.

Series	Lowest Value	Duration	Deficit
60	N	EV1	EV1, GEV, P3
40	EV1, N	EV1, LN	EV1, P3
35 Q2	EV1, N	EV1, P3	EV1, GEV
AMS	EV1, N, GP	-	-
7dMALF	EV1, N, GP	-	-

Goodness of fit statistics

Chi-squared (χ^2), Kolmogorov-Smirnov (*D*) and Filliben Correlation Coefficient (FCC) test statistics were calculated for each of the eight distributions in order to reject distribution from the model candidates. Results of the test statistics are tabulated in Tables 5.11 and 5.12. Rejected distributions at the 5 % significance level are designated with an asterisk (*).

The results confirm previous findings. The N, LN, EV1 and Exponential distributions perform more poorly than the remaining distributions or are rejected by statistical tests at the 5 % significance level. However, the EV1 distribution is an acceptable, albeit statistically less favourable, distribution for modelling the low flow series of the PDS. For the low flow series, both PDS and AMS, the GP and P3 distributions are the best fitting distributions. The GEV distribution performs better than the EV1 distribution. The LP3 distribution was not modelled, as it is solely used for flood data and represents the upper tail of the distribution better; however, it can be used for modelling durations and deficit values. For modelling the deficits of low flow events of PDS, the P3 and LP3 distributions perform best. Surprisingly, the deficit series below the 60 m^3/s threshold is best modelled with a LN distribution. AM series of deficits perform better with P3, GEV and GP distributions. Durations of low flow events are best modelled with either the LP3 or GP distribution.

5.2.5 Summary of goodness of fit

The graphical analysis based on L-moment ratio diagrams and PP plots indicate that the LP3, GP and perhaps P3 distributions are the best fit for observed low flow data. The L-moment ratio diagrams in Figure 5.17 clearly exclude the Exponential, N and EV1 distributions for modelling any of the low flow series, both for AMS and PDS. The test statistics are in agreement with these findings and clearly exclude the N, Exponential and LN distributions at the 5 % significance level. However, the EV1 (Gumbel) distribution is a statistically good candidate for modelling the low flow series of the PDS.

For modelling the durations of low flow events, all goodness of fit methods indicate that the N, Exponential and LN distributions are not suitable. Instead, the GP, LP3 and P3 distributions are good candidates for the observed data. The GEV distribution performs slightly weaker; however, it can also be used to model the durations of events.

Series of deficits are modelled consistently well by the LP3 and GP distributions, as indicated by graphical methods and statistical testing. The P3, GEV and LN distributions can also be used for modelling some of the extracted series. However, these distributions generally perform weaker in the test statistics. Overall, the GP, P3 and GEV distributions are used to produce design estimates for low flow, duration and deficits. The LP3 distribution is additionally used for durations and deficits. The EV1 and LN distributions were also used to highlight significant differences (if any) in design estimates.

5.2.6 Low flow frequency quantile estimates

Univariate quantile estimates

All series were modelled using the GP, P3 and GEV distributions. Additionally, estimates obtained from the EV1 were included for comparative purposes. Quantile estimates were generated for 5, 10, 25, 50, 100, 200 and 500 year return periods for the lowest flow, low flow deficits and low flow durations separately.

Lowest flow

The 10-year return estimate for the lowest flow is similar across distributions (Figure 5.20). The largest difference between estimates is visible at the 60 m^3/s threshold level, where the GP distribution models 27 m^3/s compared with the EV1 distribution value of 22 m^3/s. The highest estimates are produced by the 7dMALF series; all within the range of 26 m^3/s and 28 m^3/s. The 50-year and 100-year design estimates (Figure 5.21 and Figure 5.22) for the lowest flow also show that the choice of distribution makes little difference to the design value, when choosing the 40 m^3/s or 35 Q2 series. Results are all within 20 % of each other. However, distributions model markedly different results when choosing the 60 m^3/s threshold, or the AMS and 7dMALF series. Here, results can differ up to 70 %, due to the low estimate of the EV1 distribution and the relatively high estimate of the GP distribution. The combination of the EV1 distribution and the AMS or 7dMALF series especially estimate very small values in comparison with the other candidate distributions. This is especially evident in Figure 5.23, which shows generated quantile estimates for each selected series, modelled with the EV1 distribution. The EV1 distribution estimates negative flow values for the AMS, 7dMALF and PDS from the 40 m^3/s and 60 m^3/s thresholds for the 500 year return period. While this negative value is a theoretical possibility with the EV1 distribution, river flows can only go as low as zero. Therefore, the GP distribution, which is statistically a better fitting distribution, is preferred for further use in this study.

Table 5.11 Summary of goodness of fit statistics for modelling the magnitude of lowest instantaneous flows below the indicated threshold and the AMS (1967-2015).

		Lowest value					
Threshold (m^3/s)		**35 m^3/s**	**Q2 35 m^3/s**	**40 m^3/s**	**60 m^3/s**	**AMS**	**7dMALF**
GP	***χ^2***	1.40	1.25	2.33	3.02	8.37	10.03
	D	0.046	0.053	0.049	0.025	0.092	0.126
	Filliben	0.996	0.993	0.992	0.999	0.987	0.982
GEV	***χ^2***	3.20	1.81	2.82	*	10.69	1.314
	D	0.060	0.522	0.046	0.052	0.077	0.07
	Filliben	0.990	0.987	0.998	0.992	0.990	0.991
P3	***χ^2***	1.87	1.35	1.77	7.98	11.97	2.900
	D	0.048	0.050	0.033	0.044	0.081	0.059
	Filliben	0.995	0.992	0.998	0.995	0.988	0.991
EV1	***χ^2***	4.10	3.79	3.47	12.37	*	*
	D	0.101	0.098	0.047	0.053	0.145	0.164
	Filliben	0.985	0.982	0.997	0.991	0.940	0.933
N	***χ^2***	*	*	*	*	*	8.90
	D	*	0.168	0.095	*	0.875	0.096
	Filliben	0.936	0.933	0.964	0.973	0.986	0.98

* indicates a rejected distribution at the 5 % significance level

Table 5.12 Summary of goodness of fit statistics for modelling the magnitude of low flow deficits and durations below the indicated threshold.

		Deficit				**Duration**			
Threshold (m^3/s)		**35 m^3/s**	**Q2 35 m^3/s**	**40 m^3/s**	**60 m^3/s**	**35 m^3/s**	**Q2 35 m^3/s**	**40 m^3/s**	**60 m^3/s**
GP	***χ^2***	*	3.98	9.13	*	*	4.37	3.90	7.20
	D	0.128	0.113	0.042	*	0.092	0.105	0.050	0.019
	Filliben	0.990	0.962	0.879	0.990	0.992	0.974	0.988	0.994
LN	***χ^2***	15.16	3.35	1.25	*	10.03	5.87	6.93	*
	D	0.119	0.127	0.053	0.066	0.126	0.144	0.043	0.044
	Filliben	0.975	0.973	0.993	0.994	0.982	0.969	0.993	0.994
GEV	***χ^2***	*	6.02	*	*	*	6.96	8.92	*
	D	0.150	0.151	0.097	*	0.121	0.139	0.050	0.054
	Filliben	0.989	0.948	0.877	0.982	0.988	0.961	0.991	0.990
P3	***χ^2***	6.92	1.32	*	18.10	13.58	4.69	9.27	10.22
	D	*	0.095	*	*	0.129	0.108	*	*
	Filliben	0.980	0.969	0.798	0.983	0.992	0.973	0.974	0.985
LP3	***χ^2***	12.63	3.469	7.441		*	13.52	5.656	14.79
	D	0.057	0.099	0.053		0.073	0.103	0.050	0.039
	Filliben	0.993	0.978	0.992		0.992	0.975	0.997	0.997
EV1	***χ^2***	*	7.45	*	*	*	8.22	*	*
	D	*	0.183	*	*	*	0.173	*	*
	Filliben	0.807	0.946	0.476	0.838	0.921	0.962	0.892	0.927
Exp	***χ^2***	*	15.47	*	*	*	*	*	*
	D	*	0.133	*	*	0.139	*	0.104	*
	Filliben	0.881	0.970	0.543	0.909	0.970	0.973	0.945	0.971
N	***χ^2***	*	11.14	*	*	*	13.69	*	*
	D	*	*	*	*	*	0.231	*	*
	Filliben	0.687	0.892	0.365	0.704	0.826	0.920	0.788	0.823

* indicates a rejected distribution at the 5 % significance level

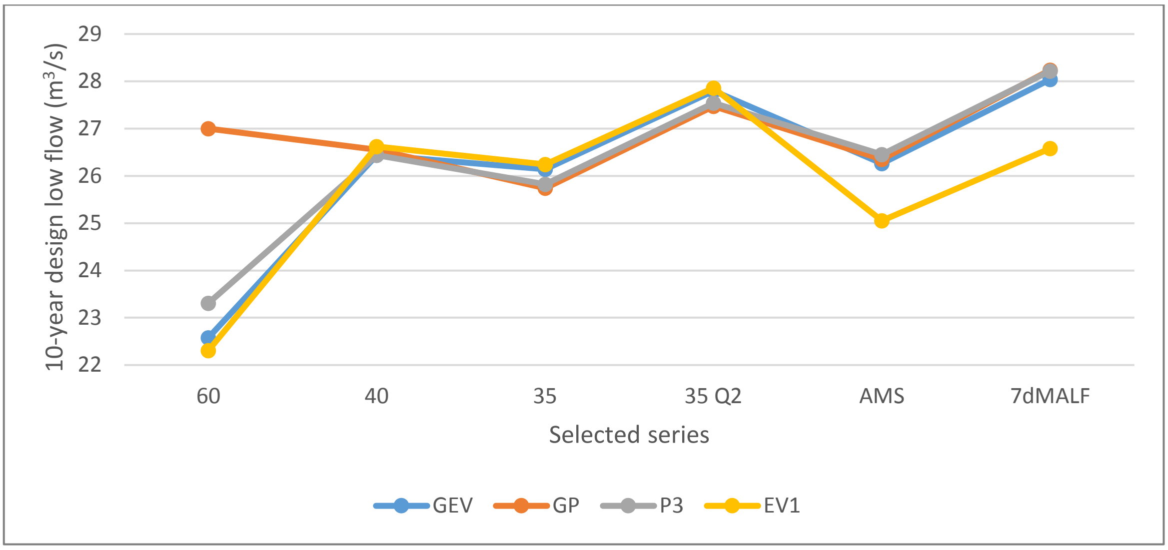


Figure 5.20 10-year design low flow estimate for selected distributions and thresholds.

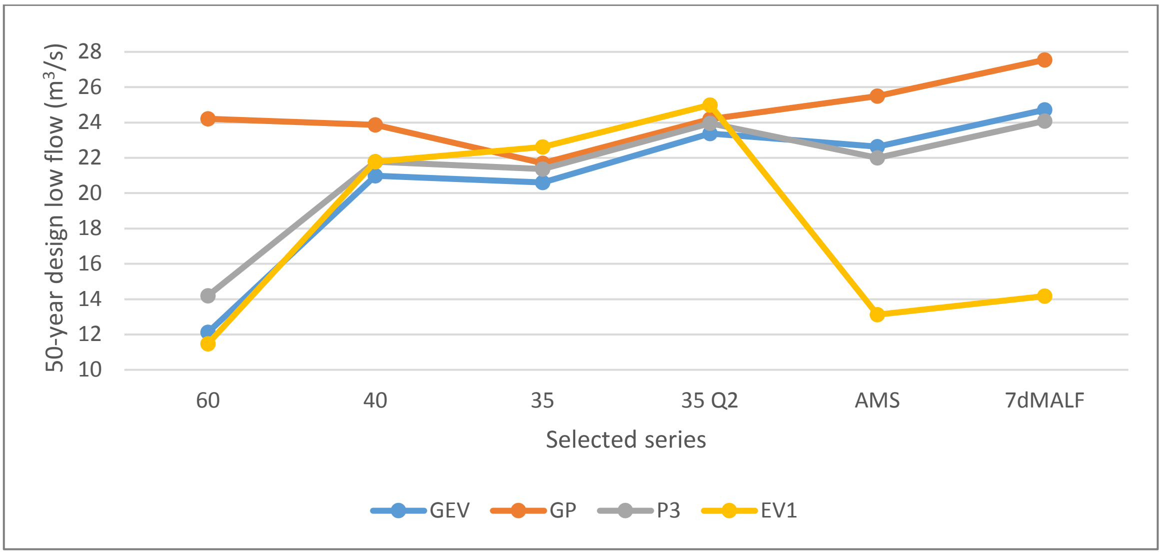


Figure 5.21 50-year design low flow estimate for selected distributions and thresholds.

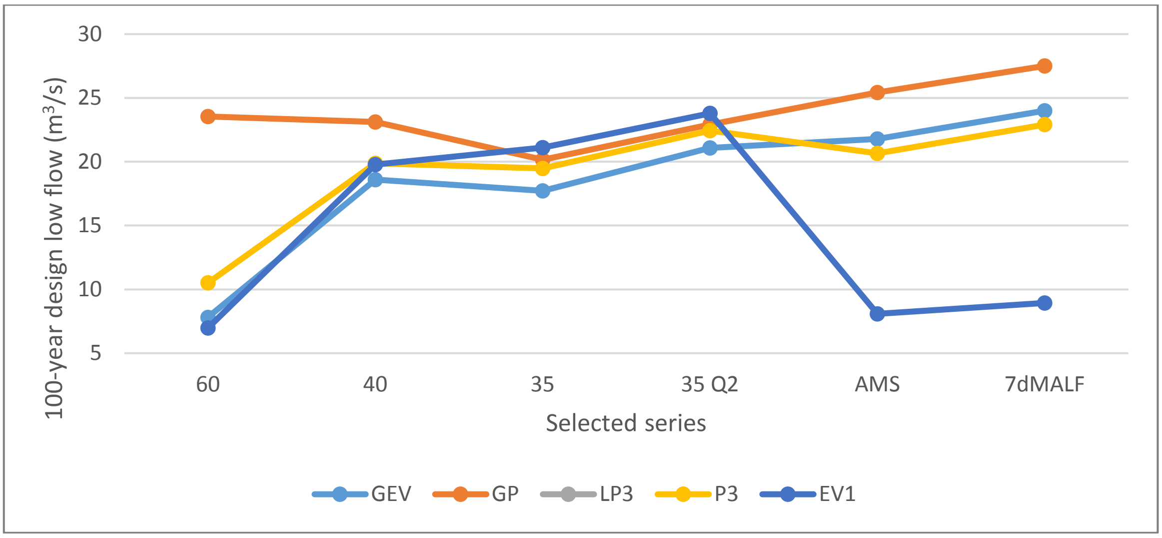


Figure 5.22 100-year design low flow estimate for selected distributions and thresholds.

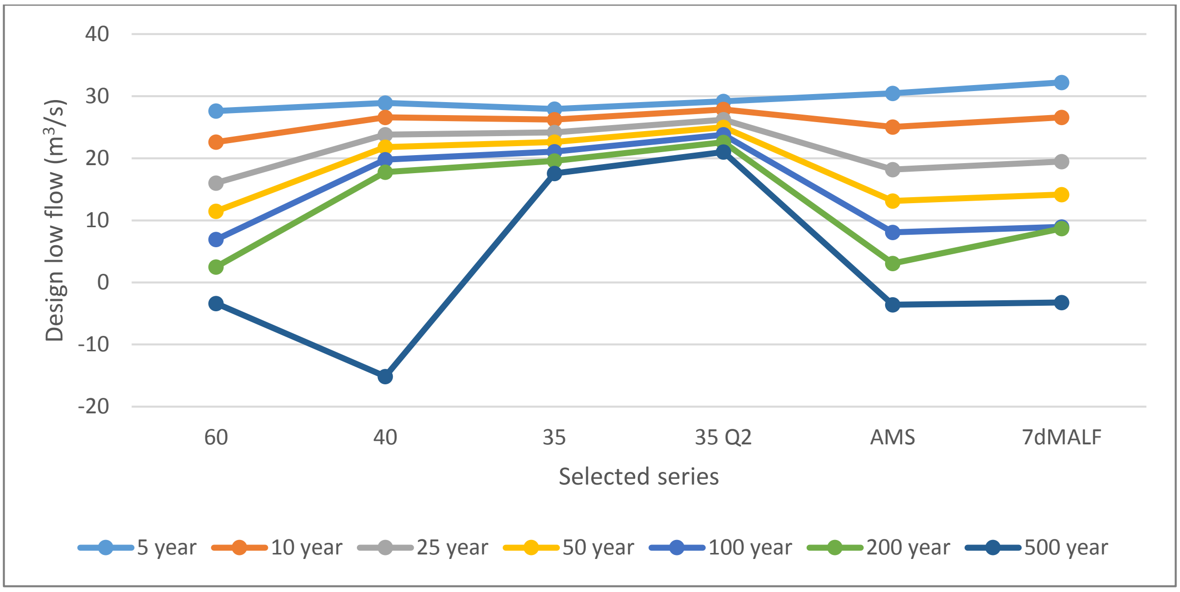


Figure 5.23 EV1 low flow quantile estimates for selected thresholds.

Duration

Figures 5.24 to 5.26 show that an increase in selected events leads to an increase in the design low flow duration estimates. The 35 Q2 series estimate of duration is lower than the unmodified 35 m^3/s threshold series, as it does not include the significant low flow events that occurred in the 1970s. With increasing threshold value, the time spent below the threshold (i.e. duration) also increases, which is reflected in the figures across all distributions. The EV1 distribution models the lowest estimates for each threshold level for the 10-, 50- and 100- year return period.

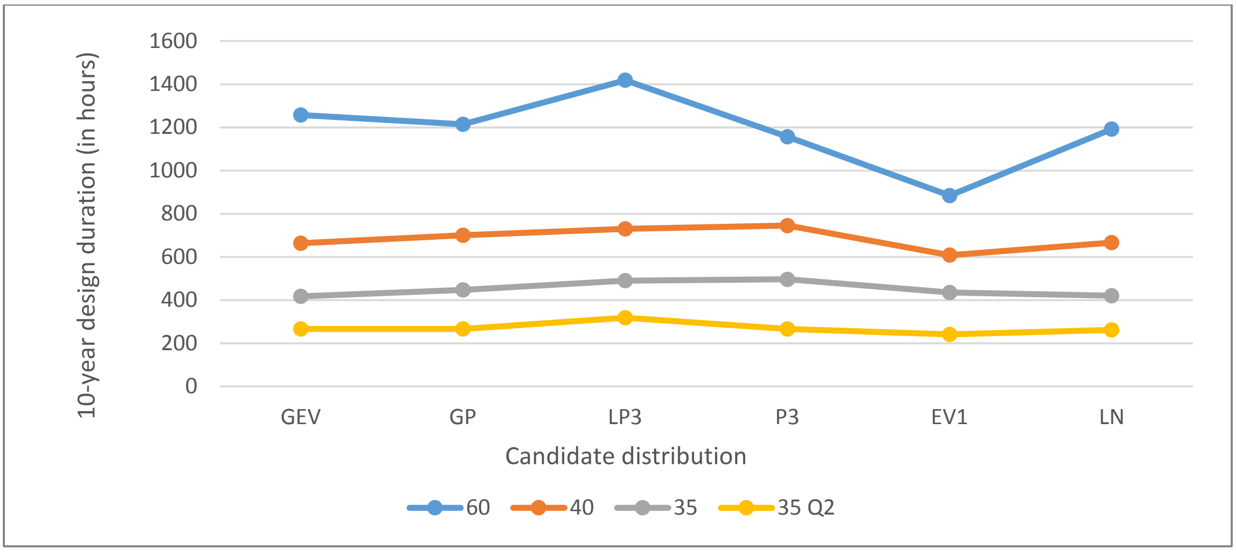


Figure 5.24 10-year design duration estimate for selected distributions and thresholds.

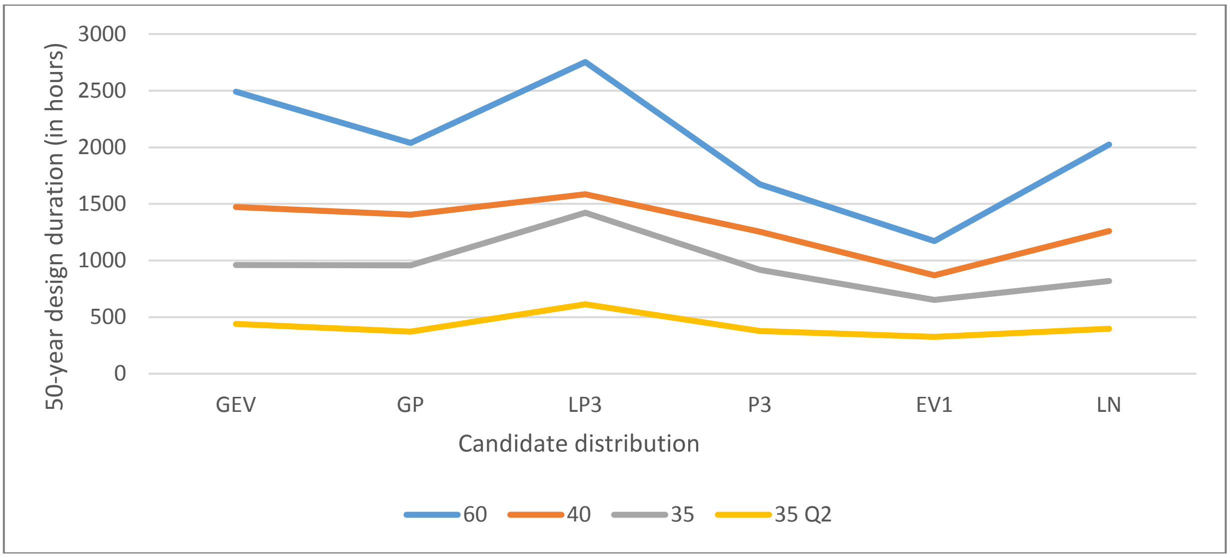


Figure 5.25 50-year design duration estimate for selected distributions and thresholds.

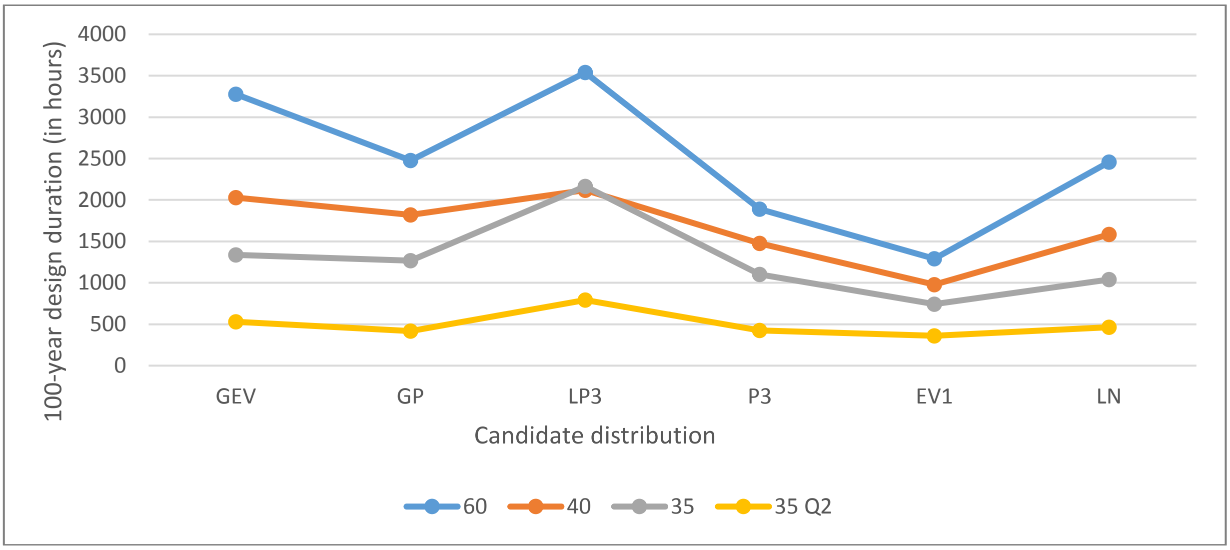


Figure 5.26 100-year design duration estimate for selected distributions and thresholds.

Deficit [19]

As a function of the threshold level, the total deficit increases with higher threshold levels. This is also reflected in the design estimates. Apart from the two series obtained by the unmodified 35 m^3/s threshold and the 35 Q2 series, candidate distributions give markedly different design deficit estimates. The similar estimates for the two 35 m^3/s series are expected, as the two series only differ in the number of events in the series, rather than the severity of the deficits. Values from other threshold levels range from 57,825 m^3 (EV1 distribution) to 97,908 m^3 (P3 distribution) for the 10-year return deficit (Figure 5.27). For the 100-year design deficit, values range between 8,625 m^3 (EV1) and 396,895 m^3, using the 60 m^3/s threshold series (Figure 5.28). As with previous design estimates, the EV1 distribution has a tendency to produce the lowest modelled estimates when compared with the other distribution candidates.

[19] Note: all deficit values are given in 10^3 m^3

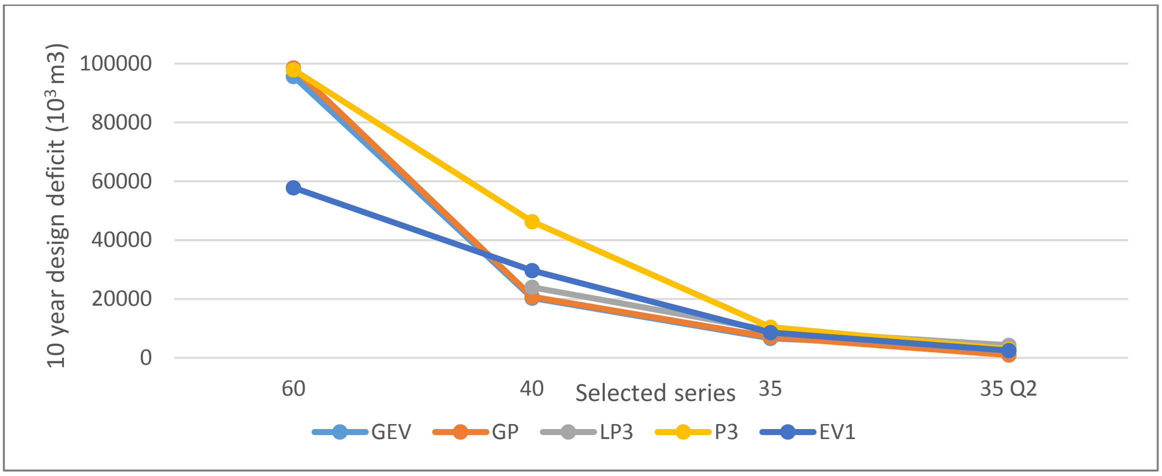


Figure 5.27 10-year design deficit estimate for selected distributions and thresholds.

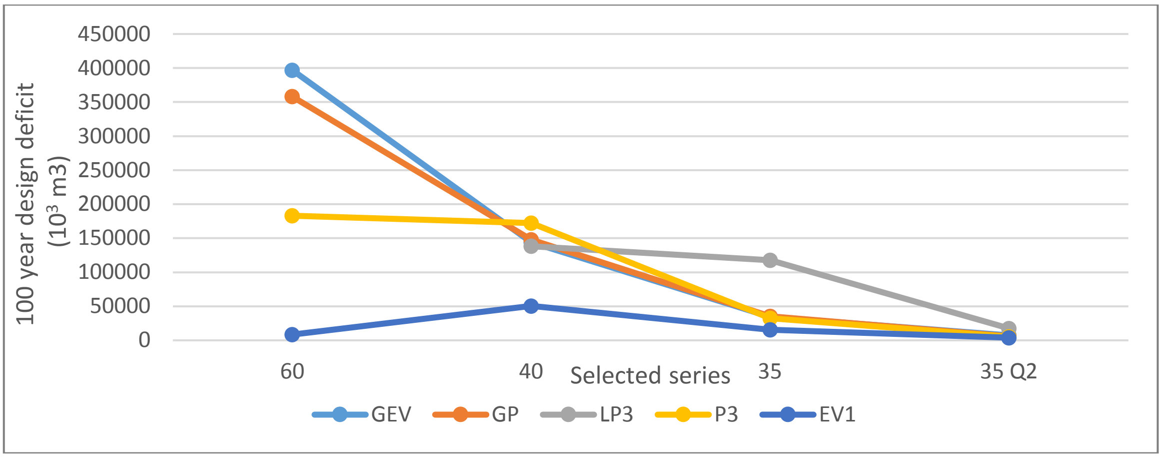


Figure 5.28 100-year design deficit estimate for selected distributions and thresholds.

Magnitude quantile estimates

Yevjevich (1967) introduced the idea of defining low flow events as a multivariate function of duration, d_i, and severity, s_i, in this case total deficit below the threshold. The magnitude of a low flow event is thus described by $m_i = \frac{s_i}{d_i}$. In this thesis, magnitude was derived by adding the dimensionless duration and dimensionless deficit together, using the equation:

$$\mathrm{m_i} = \frac{\mathrm{s_i}}{\bar{\mathrm{s}}} + \frac{\mathrm{d_i}}{\bar{\mathrm{d}}}, \qquad \text{(Equation 5.3)}$$

where $\bar{s}$ and $\bar{d}$ represent the mean of low flow deficit volumes and durations, respectively. This served to avoid obscuring events with large deficits and long durations, which could be undermined by events with long durations but small deficits if Yevjevich's equation was adopted. The frequency of the magnitude of low flow events was calculated using a threshold level of (a) 60 m^3/s and (b) 50 m^3/s. The obtained dimensionless magnitude series were tested for stationarity. Test results are available in

Appendix C. Stationarity and serial independence were confirmed by the chosen test statistics. The GP distribution was the only candidate distribution accepted both by the KS test and the chi-squared test for the 60 m^3/s level threshold series, and the best fitting distribution for the 50 m^3/s threshold series [(a)D = 0.0346, D_{crit} = 0.066; χ^2 = 9.792, χ^2_{crit} = 14.08; (b) D = 0.0397, D_{crit} = 0.08619; χ^2 = 26.27, χ^2_{crit} = 14.08]. While the GP distribution was rejected by the chi-squared test for the 50 m^3/s series, it provided the lowest calculated test value in comparison with the critical tabulated value. Furthermore, the FCC Test displayed a clear preference for the GP distribution over the other candidate distributions [(a) FCC = 0.9951, (b) FCC = 0.9945]. Therefore, quantile estimates of magnitude were only calculated for the GP distribution. The good fit of the observed dimensionless magnitude to the GP distribution is visible in Figure 5.29.

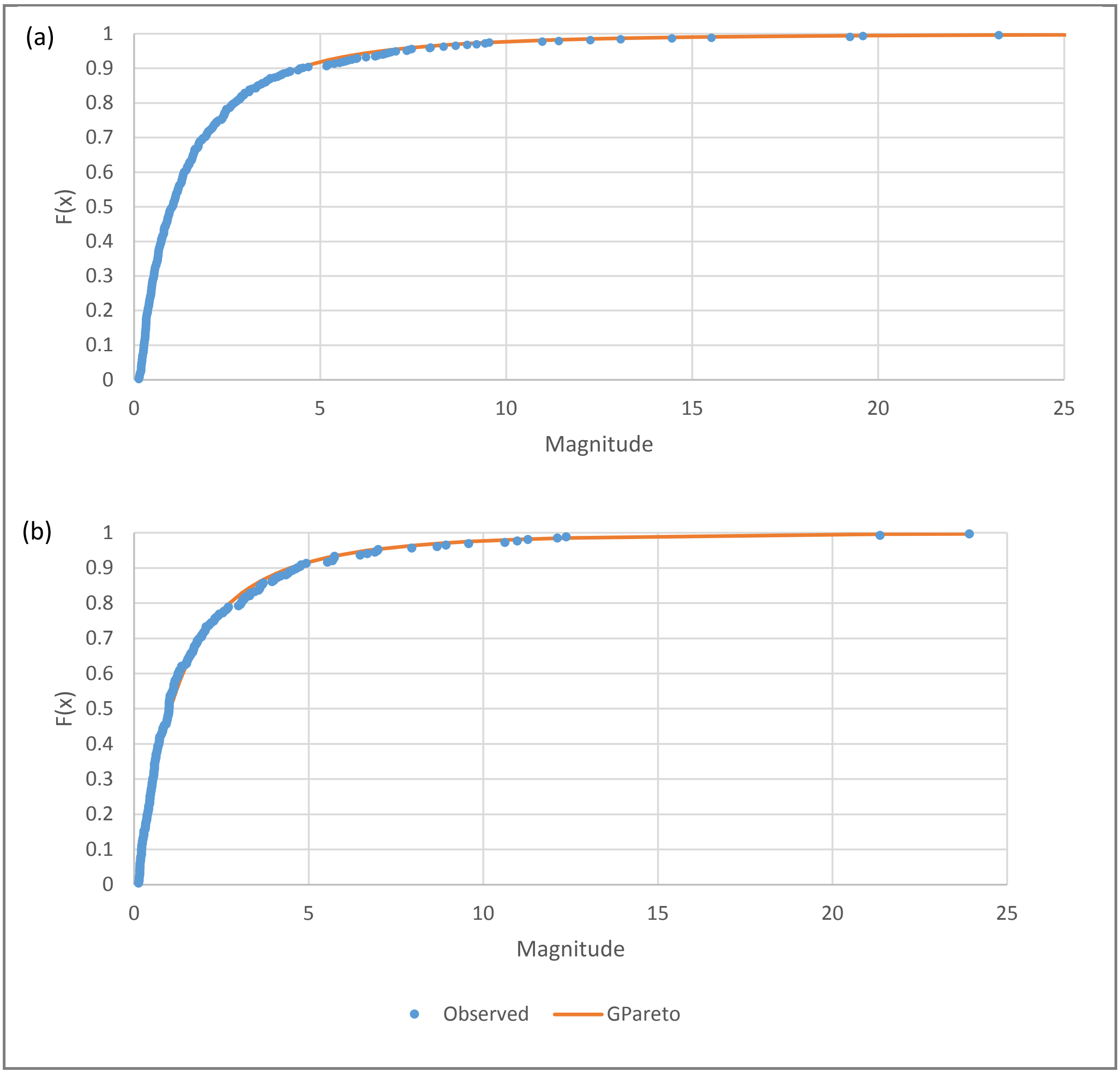


Figure 5.29 Comparison of theoretical GP and empirical cdf of dimensionless magnitude series obtained from (a) 60 m^3/s threshold and (b) 50 m^3/s threshold.

Figure 5.30 shows design estimates of magnitudes obtained by using the GP distribution. It is evident from the figure, as expected, that the design estimates using a higher threshold are also higher. As the duration and deficit of low flows below the threshold are a function of the threshold itself, this is not surprising. Using the GP distribution, a 10-year return magnitude of low flow events is estimated to be 10.33 using the 50 m^3/s series, and 13.30, using the 60 m^3/s series. Events of this magnitude correspond to low streamflow events as recorded in March 2001. The March 2001 event had a duration of 1072 h (45 days) below the 50 m^3/s threshold. The deficit for this particular event was 65,209,954 m^3, or 33.1 m^3/s discharge averaged over the duration of the event. The 50-year event magnitude is estimated to be 22 (50 m^3/s series) or 33.6 (60 m^3/s series), which corresponds to the magnitude of the February 1971 drought event. The February 1971 event, for example, had a duration of 985 hours (41 days) below 50 m^3/s, amounting to a 61,153,707 m^3 deficit, which equates to an averaged 32.75 m^3/s flow over the 41 days.

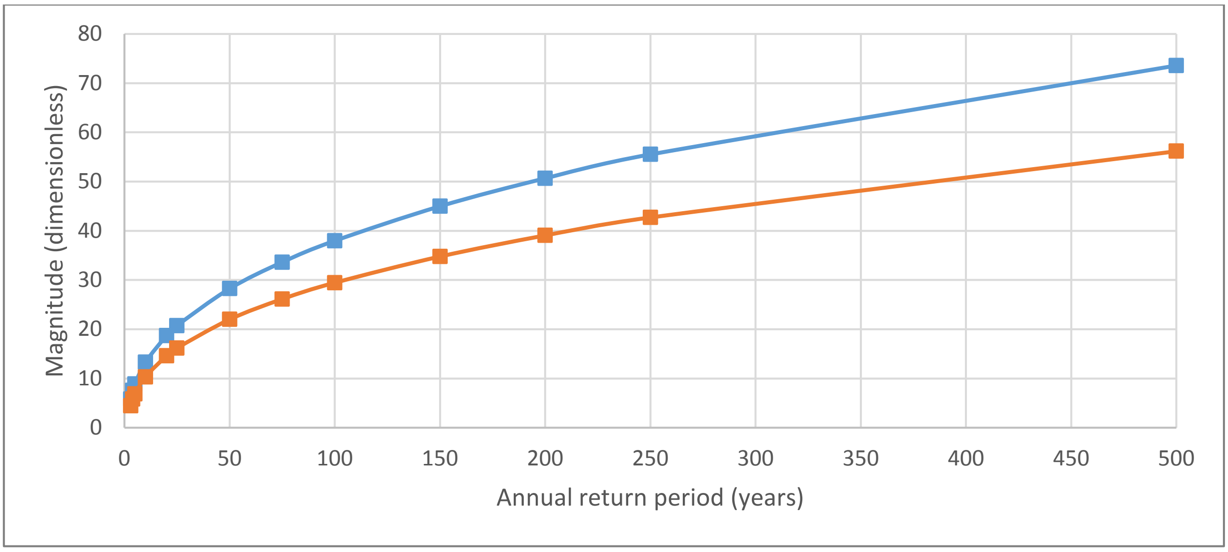


Figure 5.30 Design estimates of dimensionless magnitude using a 50 m^3/s threshold (orange) and a 60 m^3/s threshold (blue).

5.2.7 Summary of low flow frequency quantile estimates

Design estimates were produced for series of lowest instantaneous measurements, deficits below the threshold and durations of low flows. For each series, multiple threshold levels were explored and fitted with the four best fitting distributions, i.e. GP, LP3 (excluded in modelling of lowest flow), P3 and GEV. Additionally, series were modelled using the EV1 distribution. Estimates of durations are similar for all distributions, except the 60 m^3/s threshold series. The estimates produced using this series vary largely depending on the distribution used. The estimates calculated for low flow deficits are as expected. An increase in the threshold level also means an increase in design estimate. However, the EV1 distribution, which is statistically not a good fit for the deficit series, produces markedly lower estimates. Finally, the GP distribution proved to be a very good fit for the two series of magnitude used in this study. Estimates of magnitudes are as expected; the 60 m^3/s threshold series produces larger estimates than the 40 m^3/s threshold series.

5.3 Summary

This chapter presented the results from calculating the frequency of floods and low flows in the Waimakariri River, including a comparison of commonly used distributions and a comparison of the PDS/AMS approach to sampling. The results indicated that PDS gives markedly different results when compared to AMS. Furthermore, the commonly used distribution candidates that are used in the cited New Zealand literature significantly underestimate flood frequencies. The implications of these results are discussed in Chapter 9. The following chapter introduces the second part of this study with a comprehensive literature review, followed by the research design and the results. Results of both parts of the study are jointly discussed in Chapter 9.

Chapter 6

Literature review part II: Stream flow as an environmental master variable

This chapter serves to introduce the current state of knowledge about the flow regime as a 'master variable' in terms of water resources management. The chapter first elaborates on current practices in water management for setting minimum flows in rivers. It then identifies incompatibilities between water management practices and ecosystem needs. The literature review focuses on the three specific dimensions that are assessed in this study: avian ecology, periphyton communities and river geomorphology. While the sections about avian ecology and periphyton are New Zealand focussed, the review on the state of knowledge of the hydrogeomorphology component also draws on international studies and insights.

6.1 Introduction: Management of water resources and riverine ecosystems

All aspects of the natural environment and human society are rooted in the availability of freshwater. Not only does it shape landscapes, but it is also valued for the provision of ecological and cultural services (Costanza et al., 1997; Naiman et al., 2002). In an attempt to manage water resources for their various uses, unintended consequences, such as ecological degradation and morphological changes have largely compromised the ability of freshwater ecosystems to provide these services, with boundaries of resilience and sustainability largely surpassed (Richter et al., 2003; Rockstrom et al., 2009). In their natural state, functioning freshwater ecosystems are a product of hydrological variability, characterised by seasonal high and low flows, and infrequent floods and droughts (Richter et al., 2003). A lack of understanding of the large temporal and spatial scales of freshwater ecosystems (DeFries, Foley, & Asner, 2004) and the system's water flow requirements can be blamed for the wide-spread ecological degradation as witnessed. Many of the current management approaches and restoration efforts fail to attribute the integrity of the functioning ecosystem to natural hydrological variability (Poff et al., 1997). The leading management approach comes with the intention that any benefit from human appropriation of water should outweigh the unintended consequences for ecosystem functioning, often disregarding the essential needs of riverine ecosystems (DeFries et al., 2004).

The presence of low flow in rivers is part of the natural flow regime and it often occurs at similar times each year, attaining a certain degree of predictability. However, human uses of water, already amounting to over half of the accessible runoff worldwide (Dewson, James, & Death, 2007), artificially creates or extends low flow conditions, or changes the origin of water in streams during low flow events (Smakhtin, 2001). Therefore, the largest challenge in water management thus far has been balancing the needs of instream and out-of-stream uses. The notion of considering rivers themselves as legitimate 'users' of available water has been broadly accepted in society, carrying with it the understanding that the same level of advocacy and resource allocation as is characteristic for human uses is necessary (Arthington et al., 2006; Naiman et al., 2002). Poff et al. (1997) introduced the concept that the flow regime of a river should be considered as the 'master variable' for management decisions, governing the distribution and abundance of riverine species by influencing critical physicochemical characteristics, such as temperature, habitat diversity, energy input and geomorphology. Bunn and Arthington (2002) further developed this idea by introducing four basic principles that link hydrology, specifically altered flow, with riverine ecosystem development. The authors assert that (i) flow determines physical habitat, and in turn biotic composition of riverine ecosystems, (ii) instream biota have adapted life history strategies to the natural flow regime, (iii) longitudinal and lateral connectivity of the floodplains is crucial for the viability of riverine species, and (iv) an altered flow regime facilitates the invasion success of exotic species.

As any, or in the worst case, all of the aforementioned components are affected by water resource management decisions, the proper protection of freshwater biodiversity and ecosystem functioning necessitates at least the imitation of the naturally variable character of rivers, accounting for magnitude, frequency, timing, duration, rate of change and predictability of flow events (Arthington et al., 2006). This however, runs counter to the idea of water management, which seeks to provide stable and reliable water supplies for a range of uses (Richter et al., 2003). With the increasing pressure on water resources during the last decades came the recognition that previous attempts at protecting river ecosystems by means of focusing on indicators of water quality and arbitrary minimum flows had not been suitable (Poff et al., 1997). It was also recognised that ecological responses to altered flow regime largely depend on; the hydrological components affected, the rate of change relative to the natural character, and the response of the ecological processes within the system. Subsequently, the same activity can have markedly different degrees of change in different locations (Poff et al., 1997). Therefore, the knowledge about the magnitude and frequency of past flow events is crucial for balancing aspects of water management, such as maintenance of quality and quantity of available water for irrigation planning, recreation, drinking water supply, and the conservation of wildlife.

Richter et al. (2003) proposed a framework for the ecologically sustainable management of river ecosystems. The first step of the three step problem definition process necessitates the estimation of ecosystem flow requirements, based on the natural river regime and the holistic approach. The second step requires the identification of the human influences on the natural flow of the river, including an assessment of the influence of hydrological extremes, which can have significant impacts on the ecosystem. The last step of the problem definition process requires the combination of the results obtained from the first two steps: To identify incompatibilities between ecosystem flow requirements and the altered flow regime to meet human needs.

6.2 Step 1: Assessing in-stream environmental flows

The goal of water flow requirements is to provide quantifiable ranges within which certain flow components have to be maintained in order to safeguard the functioning of rivers for management purposes (Richter et al., 2003). 'Good' assessments of water allocation ensure that (i) the geomorphological structure and function of the natural river channel are maintained and (ii) individuals, populations, communities, and ecosystem processes are preserved (Dollar et al., 2007). It is inherently understood that the objective of such assessment is not recommending the unmodified regime as the ideal. It is rather a negotiation about which lesser amount of river flow will satisfy stream health and anthropogenic resource use concurrently. However, describing flow requirements provides many difficulties for managers. Relationships between flow and biota might not be well enough understood (Richter et al., 2003). Despite the challenges, it is crucial to consider the spatial and temporal context of the key links between geomorphology, hydrology, and ecology for environmental water allocation purposes (Dollar et al., 2007).

As a result of the challenges posed in the scientific field of 'environmental flows', more than 200 methods for assessing the water requirements of riverine ecosystems have been proposed to date[20], which can be broadly grouped into: hydrological rules, hydraulic rating methods, habitat simulation models, and more recent holistic methodologies, each category increasing in complexity (Davie, 2008).

6.2.1 Historical streamflow method

The historical streamflow method is the oldest and quickest assessment technique for analysing water needs for instream biota, based on hydrological rules. Some of the commonly used methods that fall into this category are the Tennant method (in New Zealand known as the Montana method), the Median Monthly Flow, or the 7Q2 method. The historical method considers the generic needs of a specific river under study, without focusing specifically on species (Caissie & El-Jabi, 2003) and uses historical flow ranges to set water abstraction limits. This assumes a linear relationship between flow parameters and biological response and ignores the concept of non-linear dynamics and thresholds in ecology and geomorphology (e.g. Groffman et al., 2006). The most widely used historical streamflow method is the Tennant Method (Tennant, 1976), based on a fixed percentage of the mean annual flow (MAF). While the method was developed from a study of streams in Montana, Wyoming and Nebraska, many prescriptive flow requirements worldwide have adopted this approach and often state a percentage of the MAF as a recommended allocation value. One of the largest drawbacks of this, albeit simple and quick method, is the lack of transferability to other streams outside of the study area. Direct transfer of

[20] For a list of methodologies worldwide see the database of the International Water Management Institute (2015).

the conclusions and recommendations necessitates that the streams are morphologically similar and Tennant (1976) does not provide criteria to assess morphological similarities.

6.2.2 Hydraulics rating method

While for hydrological rules no direct fieldwork data is required, the methods in the hydraulics rating category require higher level complexity and some field data (Caissie & El-Jabi, 2003). Knowledge about hydraulic data such as wetted perimeter, width, water velocity, and depth are often required in combination with rating curves or Manning's equations to derive flow (Gordon et al., 2004). A direct relationship between the selected hydraulic criterion and habitat for biota is assumed, in which case a reduction in the hydraulic criterion represents a reduction in available habitat. The most commonly used hydraulic criterion is the wetted perimeter, set at the level where habitat for biota starts to decline sharply (Caissie & El-Jabi, 2003). The advantage of the wetted perimeter method is the inclusion of stream geomorphology. However, the selection of optimal flow requirements can be subjective and has a significant effect on the determination of the minimum flow requirements (Gordon et al., 2004).

6.2.3 Habitat preference method

As an extension of the hydraulics method, the habitat preference or simulation method offers an opportunity for considering the biological factors that govern the dynamics of selected/studied species as a response to river hydraulic characteristics. The most commonly studied and applied model worldwide is the Instream Flow Incremental Methodology (IFIM), making use of computer models, such as PHABSIM (Physical Habitat Simulation) or RHYHABSIM (River Hydraulic Habitat Simulation)(Davie, 2008). The method combines knowledge about the habitat preferences and life history of a species and its response to flow. The predicted simulated habitat is often referred to as the Weighted Usable Area (WUA), and it generally increases to a maximum WUA with increasing flow, before declining at higher flows (Caissie & El-Jabi, 2003). However, it is to be noted that within the model, constant water quality parameters are assumed, as only the physical flow regime of the river is modelled.

6.2.4 Holistic method

Despite the many uses of the methods described above, Arthington et al. (1992) argue that a more holistic methodology for the assessment of environmental flows is necessary. To properly account for the spatial and temporal scales that govern river channels, both, bottom-up and top-down approaches are to be included in any water allocation decision (Dollar et al., 2007). One of the theoretical objections raised in Arthington et al. (1992), especially for Southern hemisphere rivers, is the presence of '*extremely heterogeneous geomorphologies with highly variable and unpredictable hydrology'* (Arthington et al., 1992, p. 69) and the inability of methods such as IFIM to adequately characterise hydraulic and habitat needs of species. A holistic approach is argued to consider the needs of the entire ecosystem by combining best available biological knowledge, expert opinions and water needs of the river to maintain a natural hydrological regime, including functioning geomorphological processes (Arthington et al., 1992; Poff et al., 1997; Richter et al., 2003). The three assumptions that are implicit in the holistic approach are described by Arthington et al. (1992, p. 70) and are: (i) the river is a legitimate user of water and water belongs to the environment. Other resource uses have to be accounted for from excess production of the resource. (ii) The ecosystem produces more of the resource than necessary for maintenance, i.e. excess. (iii) Under the adoption of a true natural flow regime, no ecological or functional degradation of the ecosystem should occur.

6.2.5 In-stream environmental flows in New Zealand

Within New Zealand, environmental flow decisions are made on several levels. The Resource Management Act [RMA] 1991 provides the framework for decisions on national and regional policy statements, and objectives of regional plans. Regional councils have varied methods of setting minimum environmental flow standards, depending on the type of river under consideration. The Ministry for the Environment (MfE, 2008) published the proposed *National Environmental Standards on Ecological Flows and Water Levels* as a 'guiding' document in 2008; however, final decisions on the standards are currently on hold. Nevertheless, the document provides a framework for selecting appropriate methodologies for determining minimum environmental flows and suggests increasing complexity of methods employed with increasing value of the in-stream habitat and increasing degree of hydrological alteration. The components used to assess alterations to the hydrological regime are: (i) the magnitude and duration of minimum flows, (ii) the magnitude, frequency and duration of high flows, and (iii) the magnitude, frequency and duration of flood flows sufficient to cause substantial movement of the armour layer and erosion of banks. An example would be the use of a historical flow method or expert opinions for the determination of minimum flows in cases where alteration of the regime is deemed minimal in a lesser significant stream (*sensu* ecosystem services). In contrast, significant modification of the regime, such as through the construction of dams in highly valued streams would require a range of assessment methods, such as 2D habitat models, flow variability analyses, sediment entrainment models, or groundwater models (MfE, 2008).

6.3 Step 2: Identification of the human influences on the natural flow and environment

6.3.1 Water management in Canterbury

The sustainable management of water resources is not only a contentious issue worldwide, but is also a primary topic of discussion within New Zealand (ECan, 2009). The increasing, and often conflicting demand on already constrained water resources has initiated the slow deterioration of water quality and quantity (Russell & Frame, 2011). Within Canterbury, the largest region in New Zealand, water is especially pivotal for the world renowned braided gravel-bed rivers that are characteristic of the landscape. Canterbury is further characterised by a high-quality aquifer system and a number of spring-fed, alpine and lowland streams, which together form the basis for the continuous expansion of highly water intensive forms of agriculture, despite low levels of rainfall in the agricultural part of the region (ECan, 2009; Russell & Frame, 2011). The conversion from traditional dryland farming and forestry to dairying has created conflict for the allocation of Canterbury's freshwater resources (Weber, Memon, & Painter, 2011) and water demand is much higher than availability within sustainable boundaries (Rockstrom et al., 2009). Currently, it is estimated that 500,000 ha of land are irrigated in Canterbury, out of 1.3 million ha of potentially irrigable land (ECan, 2009; Srinivasan & Duncan, 2011). In fact, Canterbury contains 70 % of the total irrigated land in New Zealand, and 60 % of the total consumptive use of freshwater in New Zealand occur in Canterbury. Irrigation makes up 90 % of Canterbury's total consumptive freshwater use (ECan, 2009; Gunningham, 2011).

Despite strong indications that water resources are declining in quality and quantity within the region, the reversal of the ever increasing pressures on the region's water resources is not likely. Water, and particularly access to irrigation, is the key driver for a large portion of New Zealand's primary production exports, and thus national GDP (Quinn et al., 2013). To address the region's challenges around freshwater, the Canterbury Water Management Strategy (CWMS) was finalised in 2009 with the primary goal of meeting quality and quantity requirements to sustain the ecological, social, cultural, and economic function of freshwater resources (ECan, 2009). Some of the key challenges addressed in this document are maintenance (and establishment) of environmental flows, over-allocation of groundwater resources, declining quality of water with cumulative effects on ecosystems, climate change effects, and water use efficiency. As a non-statutory, collaborative planning document, the CWMS aims to support existing regulation by providing clear targets for water management. Nevertheless, the increasing dichotomy between economically driven goals and the environment threatens the successful achievement of the mentioned targets (Barclay, 2015). The development of irrigation schemes within the region, for example, was cited as one reason for failing to achieve set targets as human activities, in this case abstractions, can artificially create or extend critical low flows that deviate from the natural flow regime of a river (Dewson et al., 2007).

6.3.2 The Central Plains Water Enhancement Scheme

The Central Plains Water Enhancement Scheme (CPWES) is a major irrigation project in the Canterbury region, planned to provide water to 60,000 hectares of land on the Canterbury Plains. The scheme, once completed, will abstract and redirect water from two braided rivers, the Waimakariri and Rakaia, with the aid of a 56 km long headrace canal (Central Plains Water Ltd. [CPWL], 2015). The first explicit plans for this project were assessed in a joint feasibility study by the Christchurch City Council and the Selwyn

District Council (SDC) in 1999 (Christchurch City Council [CCC], 1999) and the initial plans were introduced to the public with the creation of a joint steering committee (CPWL, 2015b). The plans for the scheme included the abstraction and diversion of water, a storage dam, tunnels, and canals to supply the Canterbury Plains. By 2010, the proposals were consented in a revised form, omitting the Waianiwaniwa Valley storage dam. After two years of negotiations and hearings before the Environmental Court (CPWL, 2011), costing more than NZD $ 2 million, all necessary resource consents were granted by ECan and the SDC and construction of stage one of three stages commenced. Stage one was completed in August 2015 (New Zealand Government, 2015) and currently provides irrigation water for the area around Hororata and Te Pirita with water from the Rakaia River. Stage two of the construction aims for completion by spring 2016, and stage three, which includes run of river water from the Waimakariri, by spring 2018 (CPWL, 2015c).

The RMA 1991 is the main legislative document governing the management of New Zealand's freshwater resources. Through the RMA 1991, the national government takes on a 'steering' function, by providing National Policy Statements and Environmental Standards (Lennox, Proctor, & Russell, 2011). Regional governments have the statutory responsibility of managing regional water resources either through (i) regional plans or (ii) water permits (resource consents). As the Canterbury regional plan was only introduced in 2004, the primary mechanism for the management of freshwater quantities has thus far been the use of resource consents, granted on a first-come-first-served basis (Lennox et al., 2011; Russell & Frame, 2011).

While the initial consent application by CPWL pursued the abstraction of up to 40 m^3/s from the Waimakariri River, the current consent allows water takes of up to 25 m^3/s at an intake at the Waimakariri Gorge Bridge. As per the Waimakariri River Regional Plan at the time of the consent approval (ECan, 2011b), the minimum unmodified flow of the river is set at 41 m^3/s (at the OHB site) for allocation 'A' permits. The existing consents for water take are as follows:

- Waimakariri Irrigation Ltd. – 10.5 m^3/s
- Other irrigation permits – 5.9 m^3/s
- Stockwater – 3.6 m^3/s

As the total water allocation for 'Band A' permits is restricted to 22 m^3/s, the existing permits leave 1 m^3/s for the CPWES for abstraction. However, in case more water is required, 'B' permits can be obtained, which are restricted by higher minimum flows of 63 m^3/s. The Waimakariri River Regional Plan (ECan, 2011b) therefore is a mechanism for the allocation of water between the minimum flows for 'A' and 'B' permits. For the purpose of the CPWS, a total of 25 m^3/s of take from the Waimakariri is permitted (1 m^3/s of allocated but unused 'Band A' water, and 24 m^3/s of 'Band B' water). Restrictions further state that any take of water in 'Band B' must be matched with the river, i.e. a take of 24 m^3/s must still leave 24 m^3/s of water above the minimum 'Band B' unmodified flow in the river (CPWL, 2011; Figure 6.1). Furthermore, restricted abstraction rates apply during the summer period, when the unmodified flow lies between 80 m³/s and 95 m³/s. During such conditions, CPWL can only take water for a maximum of 12 hours per day at a restricted rate (ECan, 2015; CPWL, 2011).

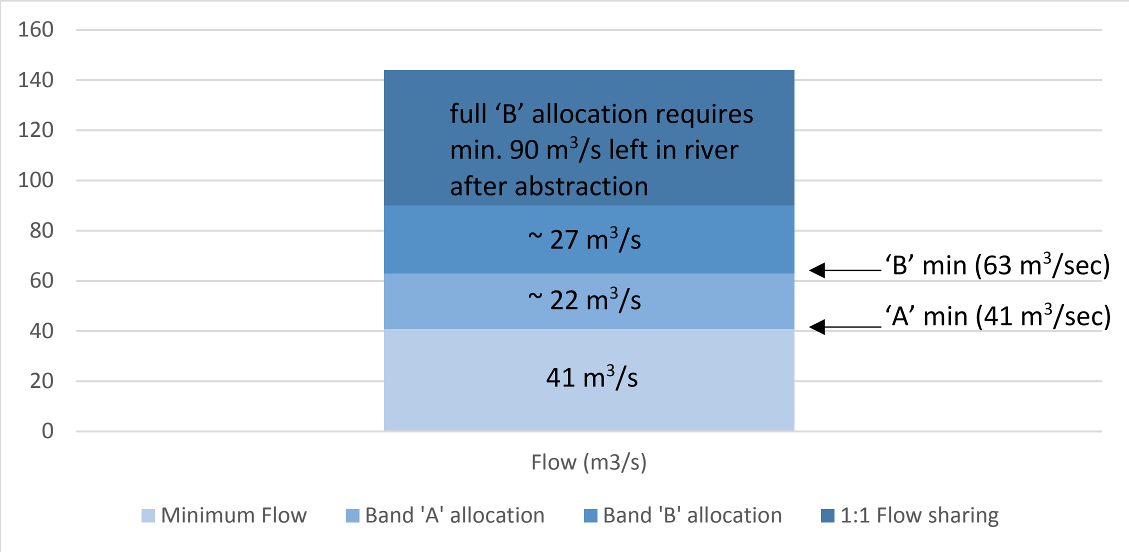


Figure 6.1 Existing user rights and water resource allocation regulation at the time of the CWPL consent approval, as per the Waimakariri River Regional Plan (ECan, 2011b)

6.4 Step 3: Incompatibilities between ecosystem flow requirements and altered flow regime

Rivers are complex, continuously changing systems, interacting with and reflecting conditions of the surrounding atmosphere (hydrological and climate processes), biosphere (ecosystem processes), and geology (geomorphological processes)(Brierley & Fryirs, 2005; Dollar et al., 2007). However, the common approach in river management often fails to implement a whole-of-system direction and rather combines inputs from the various sub-disciplines to solve pressing problems without an understanding of the large picture at hand (Dollar et al., 2007). Subsystems that form the integrated whole and that are of relevance to the identification of incompatibilities between ecosystem flow requirements and altered flow regimes are; the geomorphology, hydrology, and ecology of riverine systems. Geomorphology, or specifically fluvial geomorphology, is concerned with continuous landform evolution driven by fluvial dynamics, such as resistance (sediment cohesion and landform configuration) and periodic constraints (flow variability) (Corenblit et al., 2007; Dollar et al., 2007). Hydrology, on the other hand, considers the movement of water through the landscape and river systems. Finally, ecology investigates the response of the biota to changes in hydrology (e.g. water supply), geomorphology (e.g. sediment movement), and other biotic phenomena (Dollar et al., 2007).

Given the complexity of natural systems and the large number of variables that are inherent to them, there is a clear necessity for a conceptual model to organise the discrepancies between altered flow regimes and riverine system functioning. A conceptual model is able to help visualise patterns across various disciplinary boundaries. It is a graphical representation of a tentative theory, showing key factors and system variables and the assumed relationships between them (Robinson, 2010). In this brief review, it is also used as a framework to organise the vast amount of literature on the management of riverine environments in a systematic way within the scope of the research objective (Figure 6.2).

The proposed water abstraction activity by CPWL reduces the flow available to the ecosystem and thus has the potential to modify the hydrological regime, as summarised in the previous section *6.3*. Arroita et al. (2015) summarise some of the consequences that can result from this resource use. While there is a larger amount of literature on water quality deterioration due to decreased water availability (Dewson et al., 2007), stream ecosystem processes and their interacting components (hydrology, geomorphology and ecology) are less noticeably represented, despite the now well established notion of water flow being the 'master variable' for river management (Poff et al., 1997).

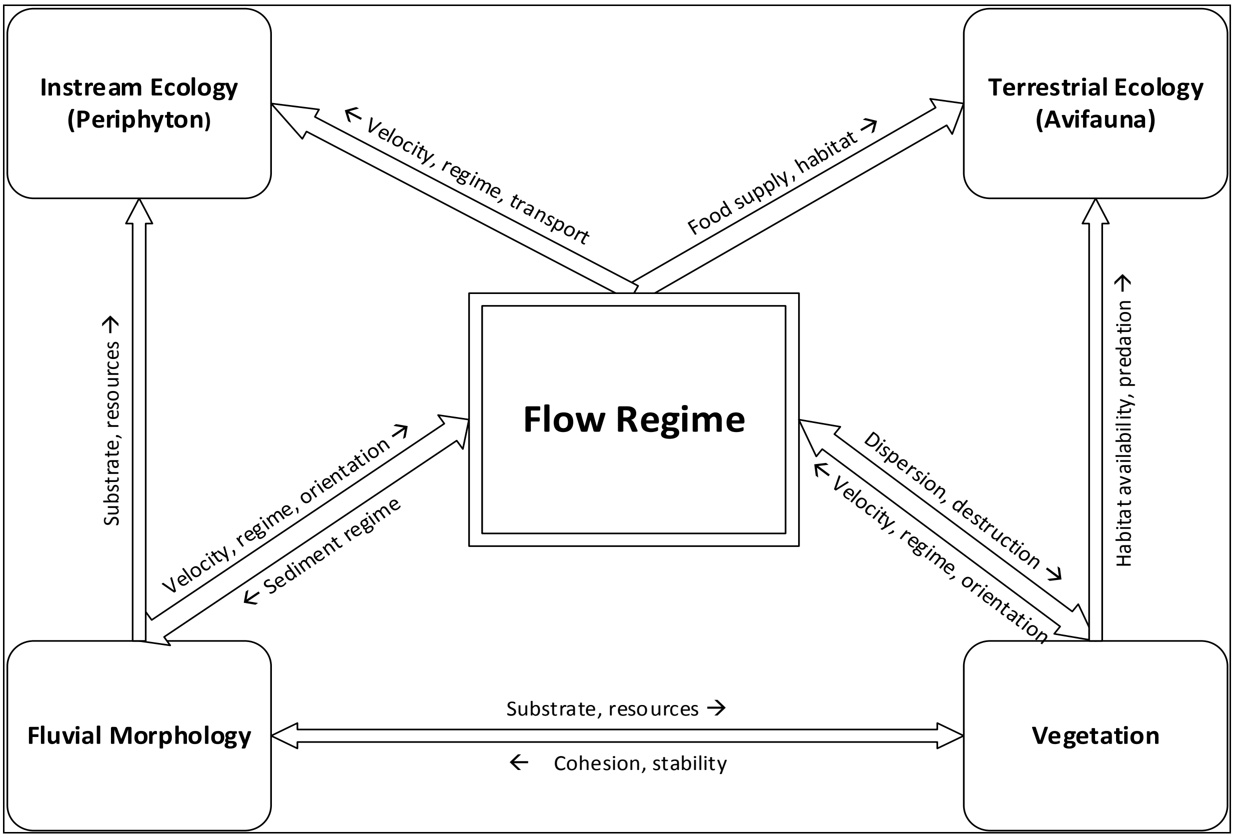


Figure 6.2 Conceptualisation of the fundamental interactions between water flow, ecology and hydrogeomorphic processes (Sources: Biggs & Close, 1989; Clausen & Biggs, 1997; Corenblit et al., 2007; Keedwell, 2002)

6.4.1 The master variable 'flow'

Worldwide, there are very few large streams and rivers of significance left untouched by human intervention and displaying a natural, unregulated flow regime (Bunn & Arthington, 2002). It is now widely understood that arbitrary minimum flow regulations for the management of rivers is unsuitable as the functioning of river systems largely depend on flow quantity, timing and the resulting longitudinal, lateral and vertical connections (Lloyd et al., 2004; Poff et al., 1997). Several concepts were developed with the above in mind, notably the flood pulse concept (Junk, Bayley, & Sparks, 1989) and its extension, the flow pulse concept (Tockner, Malard, & Ward, 2000). With the publication of the natural flow regime paradigm (Poff et al., 1997), scientists and water managers realised that by accounting for the specific characteristics of a flow regime, the consequences of human activities can be explicitly considered and quantified, especially in terms of ecological impacts. Furthermore, as a 'master variable', flow is correlated with physicochemical indicators and processes, such as water temperature and channel geomorphology, and thus ecological habitat (Poff et al., 1997).

However, the description of the natural flow regime of a river requires knowledge about a long record of observations, due to the large natural variability ranging from hours and days to years and even longer. The five critical facets that characterise the flow regime of a river are (Poff et al., 1997, pp. 770 - 771);

- flow rate: the total amount of water moving past a point per unit time
- frequency of occurrence: the number of occurrences of a flow above a given magnitude within a fixed time interval
- duration: the time period associated with a specific flow condition
- timing or predictability: the regularity of occurrence of a specified flow, and
- rate of change or flashiness: the speed at which flow changes from one magnitude to another.

While all of the five critical components interact in complex ways to shape the resulting flow regime, they are treated separately for illustration. Furthermore, the focus lies on flood flow and low flow events, as they often act as 'bottleneck events' for riverine ecology (Poff et al., 1997). When considering geomorphological processes, the flow magnitude, frequency of occurrence, and duration are most significant. Within the river channel, the wide variety of flows in unregulated natural rivers creates and maintains different physical features (Arthington, 2012). The magnitude and variability of flows have been shown to be most significant for instream ecology (Clausen & Biggs, 1997). The duration of flow is equally significant, when considering. (i) riparian plant tolerance to flooding (Caruso, Pithie, & Edmondson, 2013), or (ii) low flow and periphyton accrual period (Biggs, 1990). The timing of flow events is equally important, as many aquatic and riparian species have adapted life cycles and strategies to exploit or avoid present flows (Hughey, 1985a).

Within the conceptual model presented above (Figure 6.2), *flow regime* acts as a central node, representing its role as the 'master variable', regulating hydrogeomorphic and ecological processes. Each relationship as shown in Figure 6.2 is considered in detail in the sections below.

6.4.2 Geomorphology

The scientific field of geomorphology seeks to understand the processes that actively shape the topographic landscape. It draws on other disciplines, such as geology, physics and biology to gain an understanding of how weathering, erosion and deposition form the land (Bierman & Montgomery, 2014). The direct relation between a process and the resulting form is at the centre of discussion. The movement of water on the surface of the earth is a key influence on landscape evolution, by initiating and facilitating the movement of sediment. Concurrently, the shape of the landscape at the larger spatial scale directly controls the smaller scale processes of erosion, transport and deposition (Arthington, 2012). At smaller scales, geomorphological processes are often a reflection of the interaction with ecological and hydrological systems. An example, which will be discussed in more detail in a later section, is the stabilising effect of riparian vegetation roots on stream channel sediment (Corenblit et al., 2007).

Fluvial processes, which belong to the field of geomorphology, describe the complete process of the formation and evolution of river systems at temporal and spatial scales. Channel and floodplain morphology is an expression of the interaction of flowing water, sediment availability and large-scale topology. The balance between the cohesiveness of the landform (lithology, sediment texture, vegetation) and mobilisation by water flow results in the observed fluvial style (Francis, Corenblit, & Edwards, 2009). Stream and river channels carry sediment along the course of the river system, with fluvial processes controlling the mobilisation and deposition of sediment. Sediment availability is a factor of regional tectonic forces. Combined, these two factors, together with stream power, result in different rates of transport capacity and therefore river patterns. Previous categorisations have included those of Leopold and Wolman (1957), Schumm (1985) and Ferguson (1987). Every known river planform can be described as a combination of three main river configurations, which are straight, meandering or branched (Alabyan & Chalov, 1998).

There are multiple theories explored in the literature that explain the braiding planform of alluvial systems (Ashmore, 1991). The most commonly evoked theory, the functional explanation, relates the process of braiding in a river to the sum of its external environmental forces, such as stream discharge, sediment morphology and channel gradient (Ashmore, 1991). Leopold and Wolman (1957) defined the threshold between meandering and braiding rivers using riverbed slope and bankfull discharge as characterising variables. A sudden decline in slope and the deposition of the coarse bedload is an essential process in the formation of the braiding form. Other theories are based on 2D stability analyses of channel bars or focus on the physical sedimentary and hydraulic processes required for the initiation of the braiding process in alluvial systems (Ashmore, 1991).

Previous studies found that the effectiveness of discharge, measured by the sediment transport capacity and frequency of occurrence determine the channel pattern (Kleinhans, 2010). Braided rivers are the result of high flow strength, which is determined by a high energy gradient and channel-forming discharge. Furthermore, sediment supply exceeds the river's transport capacity (Kleinhans, 2010). The resulting multiple, active stream threads are located in an often wide, low sinuosity channels and braid bars (Bierman & Montgomery, 2014; Francis et al., 2009). Braiding is also characteristic of variability in discharge, a lack of stabilising vegetation and high loads of coarse sediment. Such rivers are rarely in steady-state equilibria and braiding is often an indicator of active valley evolution. Braided rivers can be categorised according to their stage of evolution (Piégay et al., 2009). During times of high sediment supply, typically aggradation and widening is the norm. This expansion phase is also indicative of major channel shifting. During contraction, in which sediment supply is limited and/or peak flows are reduced, braided rivers typically narrow or incise their riverbeds (Piégay et al., 2009).

The highly dynamic, non-equilibrium nature of braided rivers makes it particularly difficult to make decisions concerning floodplain management. Although these rivers are highly regarded for their ecological values, many of them have undergone significant morphological and ecological changes over the last century, such as channel narrowing and straightening, gravel mining and the installation of dams (Francis et al., 2009; Piégay et al., 2009). Often a shift in boundary conditions, such as the ones that govern rates of sediment supply or sediment mobilisation, is enough to initiate long-term changes to the morphodynamics of a river (Brierley & Fryirs, 2005). Therefore, the magnitude, intensity, duration and frequency of flow are significant for physical processes. For example, bankfull discharge is responsible for the maintenance of river bars and riffle-pool sequences, as it moves significant amounts of bed and bank sediment (Naiman et al., 2002; Poff et al., 1997). The onset of sediment entrainment is further related to local hydraulic conditions, a direct function of river flow. Therefore, the reduction of characterising flow conditions influences the transport capacity of a river, which can have significant impacts on morphodynamics, both at small and large temporal/spatial scales. Such changes have previously been shown to induce metamorphosis of braided rivers into transitional or meandering rivers (Kleinhans, 2010; Schumm, 1985).

6.4.3 Periphyton development

Periphyton is a term to describe the communities of algae, single-celled and colonial chlorophytes, bacteria and fungi covering the benthos of surface water bodies. These communities are also often referred to as benthic algae and phytobenthos (Biggs, 1990; Biggs & Kilroy, 2004). While periphyton is often invisible to the naked eye, it can be seen in periods of biomass accrual or proliferation, when it forms thick slimy layers or filaments. Within freshwater ecosystems, periphyton plays an important role as a primary producer and purifier by taking up dissolved nutrients (Biggs & Kilroy, 2004). However, extensive proliferation of periphyton can have widespread negative effects on freshwater ecosystems and aesthetics (Biggs & Kilroy, 2004). Proliferation in New Zealand streams can generally be observed during summer time, when flows are naturally low. Concerns are often raised over the reduced water quality in rivers during that time. Biggs and Close (1989) observed two blooms of *Ulothrix zonata,* forming dull-green mats, in the Waimakariri during summer and autumn low flows within a 13 month study period. In a study investigating filamentous algal growth in New Zealand rivers, Biggs and Price (1987, p. 177) found that 80 % of the surveyed sites in the South Island supported filamentous algal growth, even during winter. In addition, the majority of the sites were recorded in Canterbury. *U. zonata* proliferations found in Canterbury rivers during winter, however, are associated with higher quality waters.

Periphyton growth and community composition is controlled by a variety of factors, starting with 'ultimate controls' at the continental/catchment scale (e.g. climate, geology and land use) and encompassing the point scale (e.g. light, nutrients, hydraulic processes) (Biggs & Kilroy, 2004), with smaller scale states often conditional upon the larger scale (Biggs, 1990; Biggs & Gerbeaux, 1993). Stream systems further operate on a temporal scale. Significant large events have the potential to influence stream processes for decades, while small yearly freshes can induce 'immature' non-equilibrium states (Biggs & Gerbeaux, 1993). Moreover, controlling factors can be broadly grouped into hydrological, chemical and biological factors (Biggs & Close, 1989). For a review of literature within the New Zealand setting dealing with the chemical and biological factors, the reader is referred to Biggs and Kilroy (2004). For the purpose of this review, only the hydrological controlling factors, which are the disturbance regime, water velocity and bed sediment stability, and one of the chemical factors, nutrient balance (Biggs & Close, 1989), will be addressed.

The most commonly cited abiotic determinant of periphyton development within river systems are flow-related processes, which directly influence the balance between accrual and various loss processes (Biggs & Kilroy, 2004). Flow not only exerts direct physical pressure onto periphyton, but it also indirectly controls development via its influence on substrate movement and stability, water chemistry and the mass transport of limiting nutrients (Bunn & Arthington, 2002; Dewson et al., 2007; Hart et al., 2013). Therefore, the hydrological regime of a river can control periphyton development directly in two distinct ways: (i) water velocities during flood conditions increase shear stress near the river bed, causing removal of periphyton and (ii) increased flow initiates bed load movement, causing physical abrasion (Biggs & Close, 1989, p. 225).

While the hydrological regime has been recognised as a key component controlling the composition and development of periphyton communities in New Zealand streams, the exact significance of each factor and the relationships between periphyton dynamics and the frequency and duration, average flow conditions and overall variability of extreme events has not been fully established (Biggs & Kilroy, 2004; Clausen & Biggs, 1997; Lloyd et al., 2004; Poff et al., 1997; Snelder et al., 2014). In addition, although nutrients might be the largest controlling factor of periphyton growth in lentic water systems, nutrients might only contribute significantly during inter-flood periods as far as lotic systems are concerned (Biggs & Gerbeaux, 1993).

Dewson et al. (2007) mentions that one can generally find contrasting results. The effect of flow regime on periphyton development and removal depends on the periphyton growth form. Some growth forms might even increase with higher velocities, due to the increased nutrient delivery rate, while most filamentous communities, the majority of communities found in South Island rivers (Biggs, 1990; Biggs & Price, 1987), generally decrease, as they only loosely adhere to the substratum. There is also a significant difference in the removal rate of actively growing algae and the removal of the mature periphyton matrix. While the mature growth form was shown to be removed with only minor flow changes, active growth forms tend to survive even major changes in flow (Biggs & Close, 1989).

In an attempt to quantify the relationship between benthic community structure and the hydrological regime, Clausen and Biggs (1997) determined that the absolute size of the flow (mean or median) and measures of variability (flood frequency or low flow frequency) were good predictors of the variance in average benthic community structures. For this study, partial duration series of floods from 83 rivers in New Zealand were used, setting thresholds at different levels of the median flow. One of the key results was that in streams with fewer than 10 floods per year (> 3 x the median flow, or FRE_3), high mean monthly periphyton biomass can be reported. On the other hand, periphyton in streams with higher average flood peaks and more frequent floods had reduced periphyton species richness. Subsequently, the suggestion has been made to use FRE_3 as an indicator for not only periphyton but overall benthic community changes as a result of the flow regime (Clausen & Biggs, 1997). Indeed, the FRE_3 parameter has been widely applied within New Zealand (e.g. Booker, 2013).

At the same time, flood disturbance or increases in water discharge can have markedly different effects on instream ecology in different streams (Biggs & Close, 1989), or even different effects within a stream but varying with season. While some of the apparent differences can be attributed to the periphyton growth forms (Dewson et al., 2007), as described above, some of the variation could be explained by differences at smaller spatial scale (i.e. velocity and bed sediment movement). Many rivers in New Zealand are described as harsh environments due to frequently moving bed sediment. Higher water velocities usually initiate sediment movement. Streams with high sediment supply and unstable beds can mobilise sediment at lower discharges, even during mean flow conditions (Dietrich et al., 1989). On the other hand, armouring of gravel beds can reduce the adverse effects of flood disturbance on periphyton communities (Biggs & Smith, 2002). Biggs, Smith, and Duncan (1999) concluded that bed sediment stability could potentially be a more important factor in controlling periphyton biomass than flood disturbance. Stable bed sediment sites resulted in higher mean monthly biomass of periphyton compared with unstable sites. The distinction between hydrological 'harsh' and 'benign' environment in terms of bed sediment also produces distinctly different food-webs. Usually, benign and more stable environments, although healthy, are less diverse (Biggs, Ibbitt, & Jowett, 2008). It is thus evident that small changes in local velocity rates and turbulence are important micro-scale factors in determining periphyton biomass response (Hart et al., 2013).

At the larger catchment scale, geology and the geomorphological setting of the river has a large influence on periphyton communities. Especially in regions with significant heterogeneity in climate, geology and land use, large scale influences will dominate over micro-scale factors when regulating instream ecosystem processes (Biggs & Gerbeaux, 1993). Inorganic nutrients, which are linked to land-use intensification and weathering of sedimentary rock in the catchment provide the resources and substratum necessary for periphyton growth (Biggs & Kilroy, 2004).

6.4.4 Riparian vegetation

It is clearly established that landforms and physical processes exert abiotic control over biological communities. However, certain living organisms are equally capable of modifying or even controlling their environment, or in the case of rivers, the fluvial landscape. By influencing geomorphic processes, such as erosion, transport and deposition of sediment, these living organisms can control the fluvial environment at larger spatial scales (Corenblit et al., 2007). Research on the interaction between plants and fluvial geomorphology typically focus either on the bio-engineering capabilities of riparian vegetation in the fluvial context, or the reverse, the abiotic control of hydrogeomorphic processes on vegetation (Caruso, Edmondson, & Pithie, 2013), as recognised by, for example,. the flood and flow pulse concepts (Junk et al., 1989; Tockner et al., 2000). Between the temperate and arid climate zones, however, the distinction over which variable exerts dominance over the other may be a small one (Coulthard, 2005). Recently, the concept of 'fluvial biogeomorphic succession' combined the two approaches to describe the two-way interaction between fluvial landforms and riparian vegetation (Corenblit et al., 2007). The concept described by Corenblit et al. (2007) highlights the successional shift between the dominance of hydrogeomorphological and ecological processes, the positive feedbacks between them and the role they each play in the evolution of fluvial landforms.

Vegetation acts as ecosystem engineers by exerting influence on water velocity, shear stress, and increasing resistance to channel erosion and sediment movement. Roots additionally increase substrate cohesion, especially along river banks. Overall, vegetation is responsible for a large amount of energy loss within the fluvial system by acting as a mediating agent between fluvial forces and the landscape (Corenblit et al., 2007; Gurnell, Bertoldi, & Corenblit, 2012). Rivers on the other hand, have an effect on vegetation by inducing scour, shear stress or facilitating seed dispersal. The distribution and composition of riparian vegetation communities is to a large extent controlled by the specific hydrological regime and its components, such as flood frequency, duration and intensity. Timing of the regime plays an additional role during seed dispersal, as water acts as the principal agent for seed transport in the fluvial landscape (Corenblit et al., 2007).

Within braided rivers, vegetation is a determinant in the development of channel geometry, braiding and islands (Coulthard, 2005; Gurnell et al., 2012). In an experiment, Coulthard (2005) showed that the establishment of plants in a flume experiment caused significant bar and channel movement.

Plants caused, contrary to previous studies, an increase in the braiding index. However, the establishment of vegetation also caused the stabilisation of islands, reducing the migration and dynamic movement that is typical of braided rivers. Indeed, the vegetation-mediated transitions of rivers from the braided to single thread planform is not unheard of and has been previously explored by the literature (e.g. Francis et al., 2009).

The chances of establishment and subsequent growth performance of riparian vegetation largely depends on the tolerances of species to the extremes of the hydrological regime. Within New Zealand, introduced exotic species of trees and shrubs are of particular concern in braided river habitats. Some of the species capable of modifying braided river morphology are plants within the Salicaceae family (esp. Crack willow, *Salix fragilis*), *Lupinus spp*. (e.g. *Lupinus polyphyllus*), and Sweet briar (*Rosa rubiginosa*). The plants within the Salicaceae family (willow and poplar species) especially display life history traits that allow them to dominate active river corridors (Karrenberg, Edwards, & Kollmann, 2002).

The threats to braided river morphology posed by increasing riparian vegetation has been widely explored within the New Zealand setting. Caruso (2006) summarised the restoration efforts by Project River Recovery in the Upper Waitaki Basin, influenced by hydroelectric power development. Riparian vegetation management is one of the key foci of the project, due to the adverse effects of exotic weed establishment on river morphology and in turn aquatic/ terrestrial ecology. Caruso, Edmondson, et al. (2013), Caruso, Ross, Shuker, and Davies (2013) and Caruso, Pithie et al. (2013) further studied the dynamics of a South Island braided river under the ecosystem engineering influence of invasive vegetation. The authors found that the large floods within the river system are crucial for the removal of vegetation in the floodplain. The frequency and number of large floods are controlling variables, especially for *Lupinus* spp. The duration of floods is a further controlling factor, as prolonged inundation of the floodplain and the submergence of plant roots can cause significant mortality.

6.4.5 Avian ecology

There is a considerable amount of literature on the ecology of braided riverbed birds (e.g. Hughey, 1985a, 1997, 1998; Keedwell, 2005; Keedwell & Sanders, 2002; O'Donnell, 2000; O'Donnell & Hoare, 2011; Rebergen et al., 1998). O'Donnell (2000) grouped New Zealand's water birds into distinct guilds, based on foraging and nesting behaviour, general habitat requirements and microhabitat characteristics. Microhabitat requirements are the result of evolutionary adaptation responses to spatial and temporal dynamics of the flow regime which created species' subtle preferences to specific river substrates, vegetation cover and depth and velocity of water (Poff et al., 1997). Of the eight guilds described, seven guilds of indigenous water birds use braided rivers as a foraging and breeding habitat.

The impacts of changes in the physical environment are as diverse and different as each guild of birds, and largely depend on the flexibility and specialisation of species within guilds to specific microhabitats. The heterogeneity of microhabitats of braided rivers means that they can support larger number of bird species of different guilds in comparison with single channel rivers. This variability in channel size, shape, flow rate, vegetation cover, terraces and gravel islands can therefore support as many as 80 species of birds within one braided river habitat (O'Donnell, 2004). Braided rivers can be separated into terrestrial and aquatic habitat. They can support areas with varied substrate composition, levels of disturbance and characteristic flow regime (O'Donnell, 2000).

Within Canterbury, four bird species have evolved with the challenging and dynamic nature of braided river systems, namely the wrybill (*Anarhynchus frontalis*), black stilt (*Himantopus novaeseelandiae*), black-billed gull (*Larus bulleri*) and black-fronted tern (*Sterna albostrita*). Additionally, two further endemic species, the banded dotterel (*Charadruis bicinctus*) and South Island pied oystercatcher (*Haematopus ostralegus finschi*), use braided rivers as breeding habitat (O'Donnell & Moore, 1983). Some of these birds are of particular conservation interest to New Zealand due to their declining numbers. The black-fronted tern[21], wrybill[22], banded dotterel[23], black-billed gull[17] and black stilt[17], for example, are classed as threatened species. The South Island pied oystercatcher is classed at risk with a declining population number (Robertson et al., 2012).

Each braided river specialist has preferences for certain conditions and their distribution and population number is reflected in the availability of preferred microhabitat. For example, while wrybills prefer shingle bars and shallow channels for foraging and exclusively nest on shingle riverbeds (Hughey, 1985a, 1998), black-fronted terns can make use of a wider variety of habitats such as riparian areas, terraces, open water and deep channels and shallow channels for foraging, but equally prefer to use gravel beds for nesting (O'Donnell & Hoare, 2011). Many specialist species within Canterbury have varying requirements depending on season and activity. However, microhabitat preferences across both, spatial and temporal dimensions, are stable among rivers in Canterbury (O'Donnell, 2000).

O'Donnell and Moore (1983) list specific adaptations of braided river birds, including, among others, specific migratory patterns matched to the onset of higher flows of braided rivers during spring time. The arrival of these specialists coincides with an increase in aquatic invertebrates for foraging. Hughey et al. (1989) for example, showed that while during normal flow conditions smaller channels support larger invertebrate biomass, post flooding increased the productivity of the major channels. This rapid increase in food is necessary for replenishing energy levels after long migrations and for the onset of the breeding season (O'Donnell, 2004). The timing of arrival and breeding of bird species varies significantly among species, but it is evident that the breeding season of braided river specialists is relatively short compared to other bird species (O'Donnell, 2004).

[21] nationally endangered
[22] nationally vulnerable
[23] nationally critical

The food source of many riverine bird species is among one of the factors that is also significantly influenced by the flow regime of the river. Braided rivers, especially the Waimakariri River, support a large diversity of benthic communities (Gray et al., 2006). There is a broad literature base covering the response of invertebrates to changes in water regime (Suren & Jowett, 2006 and references within). Flood disturbance often removes significant numbers of invertebrate taxa, though, recovery is usually rapid. Prolonged low flow is typically associated with an increase in population densities. However, a shift in community composition (e.g. from insect dominated to snail dominated) can also be observed as a result of prolonged low flow conditions (Suren & Jowett, 2006). This change in benthic community composition can be particularly worrying, as species such as wrybill largely depend on *Deleatidium spp.* as their primary food source (Pierce, 1979).

At the same time, a reduction in flow and a change in flow regime can encourage the establishment of exotic vegetation (Caruso, Edmondson, et al., 2013; Surian et al., 2015), such as willows (*Salix sp.)*, lupins (*Lupinus sp.),* broom (*Cytisus scoparius*) and gorse (*Ulex europeus*) which in turn reduces the available nesting sites and foraging habitat for birds that require bare shingle sites (Balneaves & Hughey, 1990). Vegetation, as discussed in a previous section, has the ability to significantly change and influence hydrogeomorphic processes within riverbeds (Corenblit et al., 2007). Thus, specific microhabitats such as shallow channels and gravel islands could be reduced. Vegetation may also offer a better opportunity for mammalian predation of bird eggs, chicks and adults by providing cover, and findings suggest that the number of flowing channels, a function of flow, correlates with higher nesting success and survival (Boffa Miskell and Urtica Consulting, 2007; Duncan et al., 2008).

6.5 Summary

Managing the human demand of water is a contentious problem worldwide. With a growing acceptance that rivers themselves are 'users' of water came also the recognition that previous attempts at setting minimum water level thresholds within rivers were unsuccessful. New Zealand, and especially the Canterbury region, is faced with a growing demand for water, particularly for the purpose of water intensive agriculture. However, with the development of the natural flow paradigm (Poff et al., 1997) came also the awareness that the natural hydrological regime of a river provides the often highly valuable ecosystem services humans depend on.

With the development of the CPWES, the Waimakariri faces direct pressures on its water resources through the proposed abstraction of run-of-the-river flow. The potential adverse effects of the proposed activity on the 'master variable' flow and its components were summarised within a conceptual model (Figure 6.2) in order to highlight the interacting components: hydrology, geomorphology and ecology. While each component has been summarised separately within the review, the interrelations between them have been pointed out with examples from the literature.

Chapter 7

Research strategy

The following chapter describes the research approach adopted to answer the questions as described in Chapter 1. In order to fully explore the research questions, a combination of secondary (desk-based) data analysis and modelling was adopted. The secondary research is comprised of a rapid systematic literature review which seeks to explore the current base of knowledge about the effects of water abstractions on chosen environmental components. The modelling part utilises simulated time series data to analyse the frequency of low flows under consented abstraction rates. The remainder of this chapter elaborates on the details of the methods adopted in this part of the thesis.

7.1 Desk-based (secondary) data analysis

Desk-based research or sometimes secondary data analysis relies on secondary data, i.e. data that has been aggregated from existing research and analyses. The most common type of research based on secondary data is an informal literature review. When the literature review is conducted in a systematised order and is a piece of research on its own, it is often referred to as a systematic review (Goodwin, 2012). Heaton (2004, p. 15) defines secondary data analysis as a research strategy that "*…makes use of pre-existing quantitative or pre-existing qualitative research data for the purpose of investigating new questions or verifying previous studies"*. Not only is the use of secondary data more cost and time effective, it can also provide an opportunity for finding research gaps and trends that have been previously overlooked. Through reviewing, cross-analysing and interpreting secondary data, insights into new research questions can be gained. However, the individual researcher has to keep in mind that the data used was collected for a different purpose than the study and a judgement has to be made about the quality and potential bias of the data (Stewart & Kamins, 1993).

The secondary data analysis approach was used to determine the effect of flow regulations in the form of water abstractions on the environment. Specifically, the literature was searched to obtain quantitative variables predicting the effect of low flow on (i) birdlife native to braided rivers, (ii) periphyton communities, and (iii) river geomorphology.

7.1.1 Literature review

The desk-based research commenced with an exploratory stage, in which a general literature review was conducted assessing the importance of river flow for the health of a functioning river ecosystems. The literature review that emerged from the initial exploratory stage was introduced in Chapter 6. This stage of the desk-based approach served two specific purposes; (i) ensuring that the proposed research was based on the current knowledge base within the field of study, and (ii) narrowing research aims and questions (Flick, 2011). From the literature review, the effects of flow variation on three environmental components were identified for further use in the study, namely: avian fauna; periphyton communities; and river geomorphology.

7.1.2 Rapid systematic literature review

Due to time constraints for the completion of a Master's thesis (one year full-time), a rapid systematic literature review (Ganann, Ciliska, & Thomas, 2010) was chosen in favour of a traditional systematic review. Traditional reviews address a specific research question by identifying and critically evaluating all relevant studies pertinent to the research question. Such reviews, while originally developed for the use in clinical medical research (Moher et al., 2009), are now widely applied in other fields of study. They are considered pieces of research in their own right and often have the possibility of uncovering links between individual empirical studies. Systematic literature reviews are objective, transparent and above all, replicable. Therefore, the research methodology necessitates a description of the search process to locate studies, criteria used to include or exclude studies from the review, and criteria for assessing the quality of the research. However, traditional systematic reviews typically can take between six months to one year to complete due to the number of individual studies to be evaluated. Typically, more than one researcher is involved in the search and assessment process to ensure transparency of study choices. The research problem to be assessed is narrow, which has limited use to decision-makers who need rapid scientific evaluations of broad concepts (Khangura et al., 2012).

While a formal definition of rapid systematic literature reviews does not exist, Tricco et al. (2015, p. 2) offered the following definition: *"Rapid reviews are a form of knowledge synthesis in which components of the systematic review process are simplified or omitted to produce information in a timely manner"*. Some of the frequently omitted or simplified processes include a smaller number of databases searched, the conduction of the review by only one reviewer, stricter exclusion criteria for the selection of studies and a descriptive categorisation of the data rather than a statistical meta-analysis. However, both methods strive to determine the validity of the evaluated studies in order to make generalisations about applications in practice.

Search strategy

Initially, the search strategy was based on the identification of pertinent and influential summary reviews of the topics highlighted by the research questions. The inclusion of summary reviews produced by New Zealand experts in the field facilitated a rapid summary of the knowledge base up to the publication date and they represent the core articles for the processes to follow. With careful consideration, the trends highlighted in these chosen reviews were included for full consideration.

The search strategy for each of the three questions was based on the same sequential three-step process. Firstly, online databases and catalogues were searched, using keywords as listed in Table

7.1. Consulted databases were: CAB Abstracts, Scopus, ScienceDirect, NZ Science, JSTOR and Web of Science. Additionally, a web search on Google Scholar was performed. The first 50 hits from each web search were examined for appropriateness and usefulness. The second step involved searching governmental and non-governmental organisations for technical reports and publications. A list of consulted governmental and non-governmental organisations (i.e. evidence submitted to ECan hearings by consulting companies) can be found in Table 7.2. The third step involved making use of the literature review developed in Chapter 6. Bibliographies of relevant peer-reviewed literature were screened to identify further publications that have not previously been found through the first two steps, until this strategy was exhausted.

Table 7.1 Keywords used in the systematic review.

Research object	Search words
Avian fauna	bird* AND braided AND river*, native AND bird* AND South Island AND New Zealand, (stream)flow AND bird* AND braided AND river*, native AND bird* AND habitat AND New Zealand
Periphyton communities	periphyton AND water AND (stream)flow, periphyton AND abstraction*, algae AND water AND (stream)flow, periphyton AND New Zealand,
River geomorphology	braided AND river* AND New Zealand, braided AND river* AND Waimakariri, braided AND river* AND geomorphology, Waimakariri AND morphology, braided AND river* AND morphology AND Canterbury

Table 7.2 Consulted governmental and non-governmental organisations.

Organisation	Accessed website
Department of Conservation Science Publications	http://www.doc.govt.nz/about-us/science-publications/
Environment Canterbury Publications Library	http://ecan.govt.nz/publications/Pages/default.aspx
NIWA Library Catalogue	https://library.niwa.co.nz/
Environment Canterbury – hearing evidence by institution submitters	http://ecan.govt.nz/GET-INVOLVED/CONSENT-PROJECTS/PAST-NOTIFICATIONS/CENTRAL-PLAINS-WATER/Pages/institution-submitters-evidence.aspx
Environment Canterbury – hearing evidence by applicant	http://ecan.govt.nz/GET-INVOLVED/CONSENT-PROJECTS/PAST-NOTIFICATIONS/CENTRAL-PLAINS-WATER/Pages/applicant-further-evidence.aspx
Cawthron Institute	http://www.cawthron.org.nz/publications/

Relevant peer-reviewed literature, technical reports, literature review analyses and reference books were compiled in a database for further screening and selection. To reduce publication bias, unpublished reports were also included in the review. The first screening process removed any articles that were not published in English. Secondly, duplicates were removed by a manual scan to remove duplicates with differing syntax. In the next step, the database was screened as described in the *Selection strategy*.

Selection strategy

Once the search was completed, relevant articles were selected based on *a priori* defined inclusion and exclusion criteria (Table 7.3). Each publication was first assessed based on its title only. The second filtering process required the screening of the abstract for relevance. If there was doubt over the relevance of a publication, it was retained for the next screening process. The remaining articles were assessed in full to determine their relevance, based on the inclusion and exclusion criteria.

Table 7.3 Inclusion/exclusion criteria for each research aim.

Aim	Inclusion	Exclusion
Effect of flow modification on avian fauna	All studies addressing the direct effect of streamflow and the indirect effect of streamflow (vegetation dynamics, predation) on population numbers of endemic New Zealand birds of braided rivers in the South Island of New Zealand.	Studies addressing the effect of streamflow on non-endemic birds, bird of non-braided rivers, birds in other locations than the South Island of New Zealand. Studies focussing on population success resulting from variables other than streamflow, vegetation dynamics and predation.
Effect of flow modification on periphyton communities	All studies addressing the effect of streamflow (and related changes in abiotic and biotic conditions) on the development, accrual and removal of periphyton in braided rivers of the South Island of New Zealand.	Studies addressing the development, accrual and removal of periphyton due to reasons other than streamflow (and related changes in abiotic and biotic conditions). Studies primarily focused on locations other than the South Island of New Zealand.
Effect of flow modification on geomorphology	All studies addressing the effect of streamflow modification on braided river geomorphology.	Studies addressing channel form modification arising from damming, channel straightening and other large-scale engineering projects.

Only the 30 most significant texts (excl. the identified reviews) for each environmental component were retained at most. Quality and significance was based on the processes described in *Quality assessment*.

Quality assessment

Quality assessment is a crucial part of systematic literature reviews. While it is based on subjective judgements, the assessment needs to be transparent and most importantly reproducible. Previous studies have used a hierarchy of quality (Pullin & Knight, 2003). However, due to the vast amount of study systems present in the field of environmental sciences and also the small number of selected articles, such an assessment system is unsuitable. Furthermore, as two of the three environmental components under study are limited by their geographic scope, articles are scarce at best and each of the small number of studies available offers valid insight into occurring processes in the Waimakariri River. Therefore, the quality assessment is based on a distinction between the funding parties for the study (i.e. independent scholarly research vs. contracted work) and the relevance to the Waimakariri River.

Data extraction

Data extracted from articles was recorded on a spreadsheet, taking into account the structure of the synthesis following in the Results section (Chapter 8). For each article, the following information was recorded: (1) full reference of article, (2) location of the study, (3) the aim of the study or review, (4) the response of the component under study, and (5) relevance and quality based on quality assessment.

Data presentation

Due to the low number of relevant studies that have been performed in the selected study area, a quantitative analysis in the form of a meta-analysis is not possible. However, data are analysed in a narrative descriptive report in Chapter 9 to expose common patterns in the literature and identify gaps in research.

7.2 Modelling

The modelling part of the study utilises the same research strategy as described for Part I of this thesis in Chapter 4. In order to assess differences between flow conditions, data from before and after treatment, in this case water abstractions, must be readily available for analysis. However, the consented abstractions from the Waimakariri River are not yet being abstracted. Therefore, this part of the thesis is a formative assessment and is forward looking.

In order to gain an understanding of the likely effects of such abstractions, the hourly time series data from 1967-2015 inclusive has been subjected to the full amount of ECan consented abstractions. Ideally, such a modelling process would incorporate additional information about abstraction rates from existing users and likely water requirements, such as in an assessment presented by Srinivasan and Duncan (2011). Such a procedure is extremely time consuming and relies on additional modelling of irrigation demand based on soil moisture models, which is not within the scope of this research.

7.2.1 New time series

The full amount of consented abstraction rates were subtracted from the existing hourly streamflow record of the Waimakariri River at OHB, using data from 1967 to 2015 inclusive. No considerations were shown for seasonal differences in water demand, due to time constraints and a lack of data. Detailed modelling of the time series would also have to include actual water abstractions by other users, rather than consented abstractions. However, with growing water demand in the region and further land use intensification, this modelling approach can be regarded as a potential worst case scenario, in which the fully consented abstractions are taken from the river. The deducted abstractions took into account minimum required unmodified discharge in 'Band A' and 'Band B' and also flow sharing rules (Figure 6.1). The subtractions from the real hourly streamflow time series were performed in Microsoft Excel (2013) using a conditional nested *IF Function*.

7.2.2 Analysis of new time series

The modified time series containing the consented abstractions were analysed as described in Chapter 4. Furthermore a descriptive comparison of hydrological data between the unmodified and modified time series was performed, highlighting above average 'wet' years and below average 'dry' years.

7.3 Combination

The third step takes into account the results obtained from the previous two steps, which were described above. The combination of both (Figure 7.1), the output of the rapid systematic review and the frequency analysis, were used to summarise the state of knowledge on the research questions and to make judgements on likely effects resulting from proposed water abstractions. The external validity of previous research and assessments were interpreted in light of low flow frequency analysis results under abstractions. Inductive reasoning was thus used to reach a generalisation, based on the observations and patterns. While this process might not be able to explain why certain patterns exist in this case, it identified these patterns (Babbie, 2011).

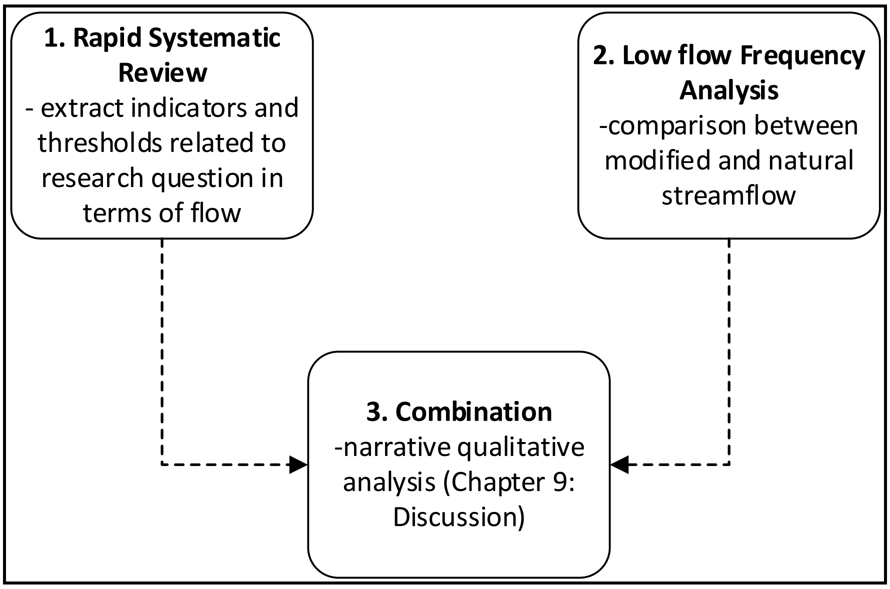


Figure 7.1 Summary of research approach.

7.4 Summary

This chapter introduced the research approach used in the second part of the thesis. The research approach included the analysis of secondary data in the form of an exploratory literature review and a rapid systematic literature review, alongside the modelling of the streamflow time series under consideration of proposed abstractions. The next chapter summarises the results from these two approaches, before combining results from Part 1 of the thesis with Part 2 in Chapter 9, the overall discussion.

Chapter 8

Results part II: The master variable flow

The chapter presents the results obtained from (i) the rapid systematic literature review and (ii) modelled abstractions and subsequent frequency analysis of low flows. The first part of the chapter details the number of articles assessed for the review and also presents a table with key variables extracted from the literature relating to biotic and abiotic effects of flow regime changes. The second part presents a detailed comparison of descriptive statistics pre- and post-abstractions, as well as low flow frequency modelling.

8.1 Rapid systematic literature review

8.1.1 Search strategy output

Figure 8.1 shows the amount of relevant published and unpublished material found at each step of the search strategy. After the first screening step, which included the removal of duplicates and foreign language publications, a total of 251 records were retained in the database. The second selection step involved screening the titles and abstracts for the appropriateness of the material. Subsequently, a total of 111 papers were retained in the database. After the third step, 70 records were identified which assessed the effects of flow modifications on the chosen variables and were retained for the qualitative synthesis of extracted data.

8.1.2 Extraction of variables

It was established in Chapter 6 that the flow regime of a river is the most important determinant of biological response in a hierarchy of physical influences. As such, the timing, magnitude and frequency of different levels of discharge, but especially magnitude of change relative to the preceding base level, are key determinants of aquatic and associated terrestrial ecology. The rapid systematic literature review was aimed at identifying the most influential characteristics of the flow regime to biotic and abiotic conditions in the Waimakariri River. Table 8.1 summarises the key findings from the literature search, organised according to ecologically and morphologically significant flow ranges. Findings are colour coded, so that effects of flow regime on periphyton are highlighted in green, effects on bird habitat are highlighted in blue, and effects on geomorphology are highlighted in yellow. Findings relating to vegetation encroachment, while influential for both fluvial geomorphology and bird habitat, are coded in yellow.

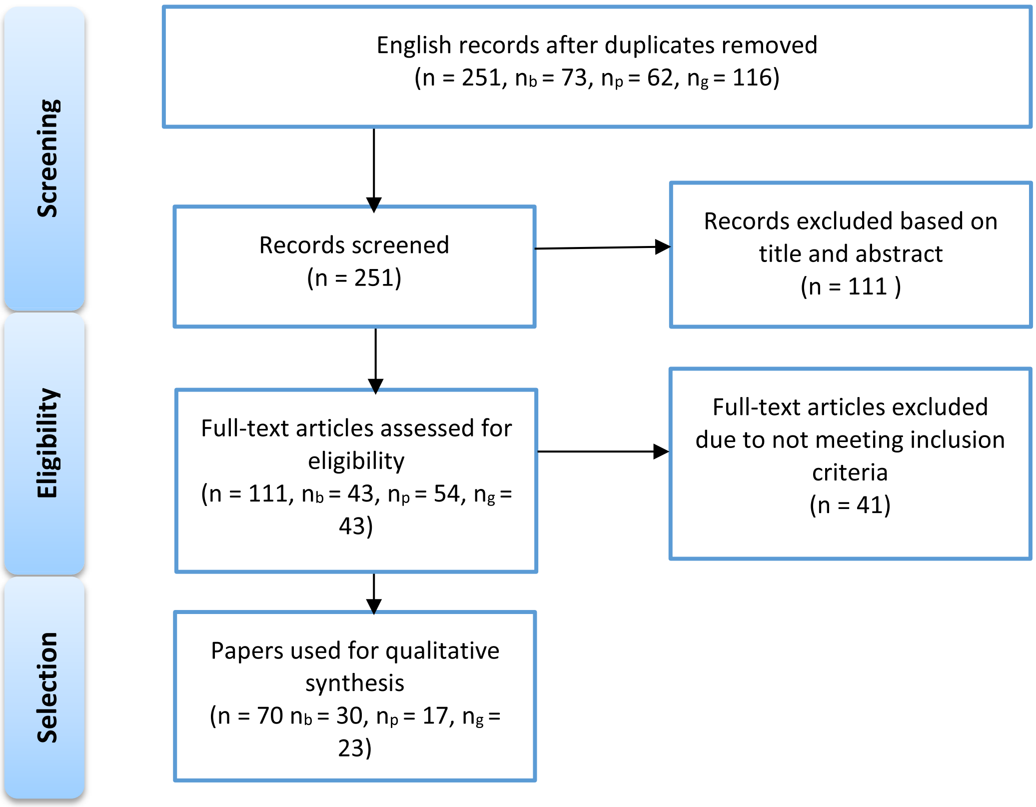


Figure 8.1 Flowchart outlining the search strategy and screening process for the rapid systematic literature review. Flowchart modified from the Preferred Reporting Items for Systematic Review Recommendations (PRISMA) (Moher et al., 2009). N = total number of records, n_b = number of records related to bird habitat, n_p = number of records related to periphyton development, n_g = number of records related to geomorphology.

Table 8.1 Aspects of the flow regime most influential for biotic and abiotic conditions in the Waimakariri River. Blue = effects on bird habitat; green = effects on periphyton; yellow = effects on geomorphology and vegetation.

Flow range	Respective Waimakariri River flows	Response/ effect	Literature
Extreme flood flows (> 10 year flood event) > 1600 m^3/s			
50-year return event	3000 m^3/s - 3500 m^3/s	May have little impact on well-established woody tree species (Ahuriri River): 4 % removal of woody trees; 70 % removal of lupins	Caruso, Edmondson, et al., 2013
10-15-year return event	2000 m^3/s - 2500 m^3/s	Floodplain and channel morphology altering, sediment regime altering, avulsion	Carson, 1984; Caruso, Pithie, et al., 2013; Boyle, 2009; MfE, 1998; Reinfelds & Nanson, 1993
20 x median flow	> 1740 m^3/s	Substantial reduction in invertebrate species richness and densities	Biggs et al., 1990; Biggs, Nikora, & Snelder, 2005; Hughey, 1985a
Extreme flood flows	> 1600 m^3/s	Catastrophic for periphyton communities of all growth forms	Biggs, 2000; Biggs et al., 2005

Flood pulses (~mean annual flood) 900-1600 m3/s			
Mean annual flood or 10 x mean flow	~1500 m^3/s	Strongest correlation between bed-moving flows for d_{50} and d_{84} with mean annual flood in New Zealand rivers	Clausen & Plew, 2004
Mean annual flood	~1500 m^3/s	Major suspended and bed load sediment movement, gravel transport, channel forming	Bertoldi et al. (2010); MfE, 1998
Flood pulse	1000 m^3/s	Movement time of particles in Waimakariri River: 25 % of time from erosion to deposition	Habersack, 2001
Flood pulse	600 m^3/s – 1100 m^3/s	Modelled critical discharge for bedload entrainment in Waimakariri River (single value used for shear stress)	Carson & Griffiths, 1989
Flood pulse	900 m^3/s – 1600 m^3/s	Removal of large woody debris in river channel and removal of vegetation on islands in Hurunui and Waiau rivers	Hughey, 2012
Bankfull discharge	900 m^3/s - 1000 m^3/s	1000 m wide inundation at Crossbank, Waimakariri River	Hicks et al., 2002
1-year return event	< 900 m^3/s	Even with 25-40 % invasive vegetation encroachment, such events cause significant channel migration in Ahuriri River	Caruso, Edmondson, et al., 2013
Bankfull discharge and smaller flood	< 900 m^3/s	High rates of nest failure (dependent on species and nesting site) in Rakaia, Ashley, Ohau, Pukaki and Ahuriri rivers	Hughey, 1985a, 1985b; Sanders and Maloney, 2002
0.5-1 year return event	< 900 m3/s	Catastrophic for periphyton communities of all growth forms	Biggs, 2000
Freshes > 130 m^3/s < mean annual flood			
Fresh flow	< 1000 m^3/s	Period of small and infrequent floods: annual yield of 154,000 m^3 of gravel movement (1986-1987), compared to average (1955-1983) of 275,000 m^3 in Waimakariri River	Carson & Griffiths, 1989
Fresh flow	~470 m^3/s	Development of new channel, shift of 29,000 m^3 bedload in study reach in Waimakariri River	Carson & Griffiths, 1989
Fresh flow	~470 m^3/s	Island maintenance, vegetation removal in Waimakariri, Hurunui and Waiau Rivers	Carson & Griffiths, 1989; Hughey, 2012
10x low flow	> 400 m^3/s < 500 m^3/s	Habitat creation, channel forming	Biggs et al., 2008; Carson & Griffiths, 1989; Duncan, 2008
Fresh flow	~288 m^3/s	Short filamentous algae and diatoms removed from 80 % of the bed surface in Waimakariri River	NIWA, 2008
Fresh flow	~288 m^3/s	80 % of bed has velocity greater than that suitable for short filamentous algae in Waimakariri River	NIWA, 2008
FRE3	~260 m^3/s	Habitat resetting (for benthos) in Waimakariri River	NIWA, 2008
< 30 days between FRE3	~260 m^3/s	Low periphyton biomass (< 200 mg/m2)	Biggs, 2000
3 x- 6 x median flow	260 m^3/s - 520 m^3/s	Main type periphyton flushing flows in New Zealand rivers	Hay & Kitson, 2013
3 x to 6 x median flow	260 m^3/s - 520 m^3/s	Sediment flushing in New Zealand rivers	Hay & Kitson, 2013
Fresher	~288 m^3/s	80 % of the bed is flushed of sediment in Waimakariri river	NIWA, 2008

5 x minimum flow (41 m^3/s)	~205 m^3/s	72 % of diatoms are surface washed; 49 % of diatoms are deep flushed (preceding condition = minimum flow) in Waiau river	Duncan & Bind, 2008
Fresh flow	~200 m^3/s	Movement time of particles only 2.7 % of the time from erosion to deposition in Waimakariri River	Habersack, 2001
Fresh flow	< 200 m^3/s	Zero modelled sediment transport (derived with fixed width method) vs. 50 % using non-uniform model in Waimakariri River	Nicholas, 2000
Fresh flow	~192 m^3/s	Critical flow for long filamentous algae, surface flushing (preceding condition= median flow) 66 % of bed is being surface flushed; 44 % is deep flushed in Waimakariri River	NIWA, 2008
1.35 x median flow	~140 m^3/s	80% of the bed has a velocity greater than that suitable for long filamentous algae (preceding flow = median flow) in Waimakariri River	NIWA, 2008
Fresh flow	> 130 m^3/s	Supply rate (1995-2001) estimated at 173,000 m^3/year and (2001-2007) 230,000 m^3/year – lower than long-term average in Waimakariri River	Boyle & Surman, 2009
Fresh flow	> 130 m^3/s	Facilitates hydrochory	Caruso, Edmondson, et al., 2013
Fresh flow	> 130 m^3/s	Timing of freshes important for vegetation removal, spring freshes more important than summer freshes	Caruso, Edmondson, et al., 2013
Fresh flow	> 130 m^3/s	Predator control in South Island braided rivers	Hughey, 2012; O'Donnell & Hoare, 2011
1.4 x preceding median flow	87 m^3/s - 130 m^3/s	Periphyton (long filamentous algae) scouring in main channel (80 %) in gravel bed rivers	Biggs & Close, 1989; Duncan, 2008; NIWA, 2008b
6 x preceding baseflow		Periphyton (long filamentous algae) scouring in side channel in gravel bed rivers	Biggs & Close, 1989
5 x baseflow		Periphyton biomass reduction	Biggs et al., 1990
> 13 FRE3 events per year		Low periphyton biomass	Biggs, 2000
Velocity increase	0.3 -1.5 m/s	< 50 % of low biomass diatom film and > 80 % of higher filamentous algae removed in laboratory studies	Biggs and Thomsen (1995) as cited in Biggs & Kilroy, 2004
		Mean discharge 120 m^3/s	
< mean discharge	< 120 m^3/s	Number of braids decline in Rakaia and Ashley rivers	Hughey, 1985a
< mean discharge	< 120 m^3/s	< WUA, reduced size and number of small channels	O'Donnell, 2000
		Q_{90} to Q_{50} ~> 45 m^3/s < ~ 87 m^3/s	
Decrease in flow	350 m^3/s to 50 m^3/s	2.5 fold decrease in mean wetted width in Waimakariri River	Golder Kingett Mitchell, 2007
	~86 m^3/s	8 flowing channels in Waimakariri River	Kingett Mitchell Ltd., 2006
2x minimum flow (41 m^3/s)	~82 m^3/s	80 % of the bed has a velocity greater than that suitable for long filamentous algae (preceding flow = minimum flow) in Waimakariri River	NIWA, 2008

	~70 m^3/s	11 gravel islands with area > 2 ha in Waimakariri River	Duncan, 2008
	60 m^3/s - 90 m^3/s	Optimal flow range for bird nesting habitat in Waimakariri river	Duncan et al., 2008
	< 60 m^3/s	Reduction of number of gravel islands; only 10 gravel islands larger than 2 ha in Waimakariri River	Duncan, 2008; Hughey, 2008; Kelly et al., 2015
Increase in flow	52 m^3/s to 63 m^3/s	Decline in filamentous algae in Waimakariri River	Golder Kingett Mitchell, 2007
	> 50 m^3/s	Number of flowing channels increases as discharge increases in Rakaia River, especially above 50 m^3/s	Mosley, 1983
Increase in flow	41 m^3/s to 82 m^3/s	Flushing of long filamentous algae (80 %), in Waimakariri River	Duncan & Bind, 2008; NIWA, 2008
Increase in flow	41 m^3/s to 52 m^3/s	WUA for long filamentous algae drops rapidly in Waimakariri River	Golder Kingett Mitchell, 2007
Increase in flow	41 m^3/s to 85 m^3/s	Depth increase from 0.31 m to 0.41 m, velocity increase from 0.51 to 0.62 m/s, mean wetted channel width increases from 230 m to 320 m in Waimakariri River	Golder Kingett Mitchell, 2007
Increase in flow		Ashley River: percentage of flow (40-100 %) in main channel decreases with increasing flow (in ranges of 1-100 m^3/s) (similar for Hurunui, no relationship for Rakaia)	Mosley, 1983
	31 kg/s at 10 m width	Actual width of the maximum transport capacity channel in Waimakariri River	Griffiths & Carson, 2000
	Velocity 0.41 m/s	Subsidy-stress response for didymo in New Zealand rivers	Bray et al., 2016
Minor channel flows (10 % of total discharge)		> 60 % contribution to WUA for wrybill and banded dotterel on Rakaia/Ashley Rivers	Hughey, 1985a
Number of flowing channels		Predator control (nest success) in Tekapo, Ohau, Ahuriri, Pukaki, Hurunui and Waiau Rivers	Hughey, 2012; O'Donnell & Hoare, 2011; Rebergen et al., 1998
Reduction in mean wetted area		Decline in invertebrate biomass in New Zealand braided rivers	Kelly et al., 2015
	0.48 m ± 0.18 m depth; 0.75 m/s velocity	Optimal depth and velocity for *Deleatidium* spp.	Jowett & Richardson, 1990
Low flows (statutory minimum flow) ca. < 41 m^3/s			
	< 52 m^3/s	Habitat availability for long filamentous algae increases in Waimakariri River	Kingett Mitchell Ltd., 2006
Prolonged minimum flow (2-3 months)	FRE3 > 34 days < 45 m^3/s	Periphyton accrual (20-100 % coverage of minor braids in Waimakariri)	Biggs, 1990; Biggs, Ibbit and Jowett, 2008; Duncan and Bind, 2009;
Prolonged minimum(2-3 months)	< 45 m^3/s	Aquatic snail and worm communities develop and replace the mayfly dominance in the Waimakariri River	Gray et al., 2006; Jowett, 1987; Kingett Mitchell Ltd., 2006
Prolonged minimum flow (2-3 months)	< 45 m^3/s	Sediment load accumulation and subsequent effect on periphyton development	Biggs, Ibbit and Jowett, 2008; Hay and

			Kitson, 2013; Olsen, 2006
Decrease in Q_{90}	< 45 m^3/s	Periphyton biomass increase with increasing Q_{90} in New Zealand rivers	Clausen & Biggs, 1997
Decrease in Q_{90}	< 45 m^3/s	Hatching success of banded dotterel decreased in Tasman River	Cruz et al., 2013
Minimum flow	< 40 m^3/s	Reduction in number of gravel islands; only 8 islands with areas larger than 2 ha in Waimakariri River	Hughey, 2008
Minimum flow	25 m^3/s to 47 m^3/s	Increase in number of flowing channels from 6 to 7 in Waimakariri River	Kingett Mitchell Ltd., 2006; MfE, 1998
Prolonged low flow	~20 m^3/s	Long filamentous algae accrual in edge habitats (minor braids) in Waiau River	Duncan & Bind, 2008
Prolonged low flow	> 14 days	Significant duration for periphyton accrual in Waimakariri river	NIWA, 2008
Prolonged low flow		Only finer gravel is moved in Waimakariri River	Carson and Griffiths, 1989
Prolonged low flow		Vegetation encroachment in Ohau, Tekapo, Ahurir, Ashley and Rakaia Rivers	Hughey, 1985a and b; MfE, 1998; Rebergen et al., 1998
Prolonged low flow		Bank and island stabilisation through vegetation	MfE, 1998
Prolonged low flow		Bed armouring in gravel bed rivers	Dietich et al., 1989; MfE, 1998
Prolonged low flow		Predator access to islands in Hurunui and Waiau Rivers	Hughey, 2012
Low river levels in winter		Selection of low lying nests by wrybill in Ashley and Rakaia Rivers	Hughey, 1985a

Based on the table presented above it is evident that a large variability of flow ranges is required for the maintenance of the river as a functioning process system. Flows between Q_{90} and Q_{70} primarily ensure adequate habitat quantity (in terms of WUA). Regular flushing flows, or freshes, ensure the required habitat quality is maintained by intermittently flushing sediment, nuisance periphyton accrual, established vegetation on islands and banks, and predators from the floodplain. These freshes are thus considered habitat resetting flows in the ecological sense. Flood pulses in the ranges of the mean annual flood or higher permit the transport of bed load downstream and thus maintain the river morphology by preventing an imbalance of gravel input vs. output along the river continuum. Such flows are considered channel forming and can significantly alter the dynamics of braided rivers. These flows are generally catastrophic for riverbed nesting birds and their food sources (invertebrates), and periphyton communities. Extreme flood events with magnitudes corresponding to a 10-year event or higher have a high potential of altering the floodplain and channel morphology of a braided river. After such events, the river may persist in a non-equilibrium state *sensu* bed load and sediment regime for a number of years. The low flow ranges lie on the other end of the spectrum of extremes. Prolonged periods of low flow (i.e. at or below the minimum flow, 7dMALF) can have extensive negative effects on habitat quality and quantity of riverbed nesting birds and their food sources. Prolonged low flow events also permit the accrual of nuisance periphyton biomass and vegetation encroachment on gravel islands, which in turn can alter river morphology, a process known as biogeomorphology.

8.2 Modelling of abstraction

8.2.1 Descriptive analysis

The available time series of discharges of the Waimakariri at the OHB site from 1967 to 2015 inclusive was modified to reflect a consented abstraction of max. 25 m^3/s. Assumptions and conditions of abstractions are summarised in Chapter 6 (Figure 6.1). Comparison of the time series before and after modelled abstractions reveals the following:

- The mean flow is reduced from 119 m^3/s to 106 m^3/s (-11 %)
- The median flow is reduced from 87 m^3/s to 75 m^3/s (-14 %)
- The mean summer flow (November to March)[24] is reduced from 112 m^3/s to 100 m^3/s (11 %)
- The median summer flow is reduced from 83 m^3/s to 72 m^3/s (-13 %)
- The mean flow during river bed bird breeding season[25] (September-January) is reduced from 146 m^3/s to 130 m^3/s (-11 %)
- The median flow during bird breeding season is reduced from 109 m^3/s to 90 m^3/s (-17 %)
- The 1 day mean annual low flow (1dMALF) is only slightly reduced from 37.40 m^3/s to 36.86 m^3/s
- The 7dMALF is also only slightly reduced from 39.74 m^3/s to 39.26 m^3/s.
- The average take of water across all years is 12.55 m^3/s (median take is 11 m^3/s).
- The average take of water during the summer months is 11.42 m^3/s (median take is 10.35 m^3/s)
- The average take (consented take in light of restrictions) of water during the bird breeding season is 16.12 m^3/s (median take is 18.92 m^3/s).
- FRE_3 is reduced from an average of 18 per year to an average of 10 per year[26]

Figures 8.2 and 8.3 show the effects of water abstractions on the mean and median flow of each year on record. Both figures reveal that in wet years (mean/ median flow above overall mean/median) abstractions on average are higher than in dry years (mean/median flow below overall mean/median). This is expected, as drier years (i.e. flows more frequently at or near the 7dMALF) also result in more stringent water take restrictions. It is evident that the distribution of discharge at the OHB site displays a positive skew (i.e. the mean is larger than the median). However, based on Figure 8.2 and 8.3, the effect of abstractions is more pronounced on the median, indicating that abstractions had the largest effect on mid-range magnitude flows that are < 120 m^3/s.

[24] note that only 48 summer seasons are included in this calculation
[25] as above
[26] FRE_3 calculations are based on mean daily measurements as opposed to hourly measurements

The median abstraction rate across all years is 11 m^3/s. However, the estimated median reduction in river flow due to abstraction is 13 m^3/s. This discrepancy is explained by the fact that the consented maximum rate of water take also increases with increasing discharge due to flow sharing rules.

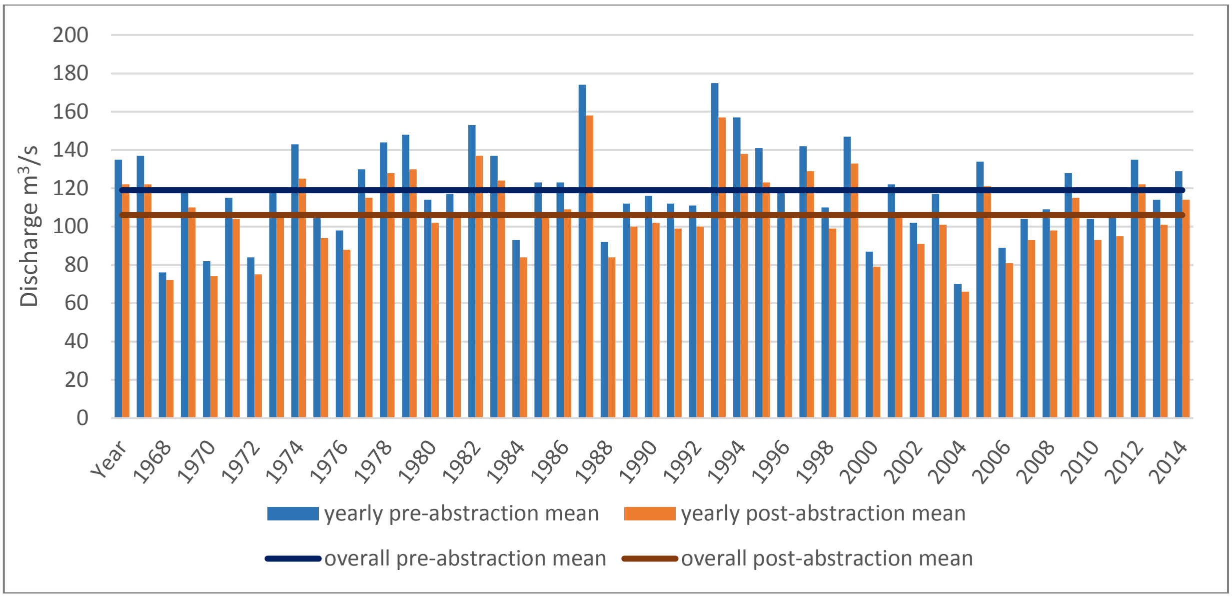


Figure 8.2 Mean annual flow at the Waimakariri OHB site. Blue colours represent values corresponding to pre-abstraction conditions. Orange colours represent post-abstraction (PA) conditions.

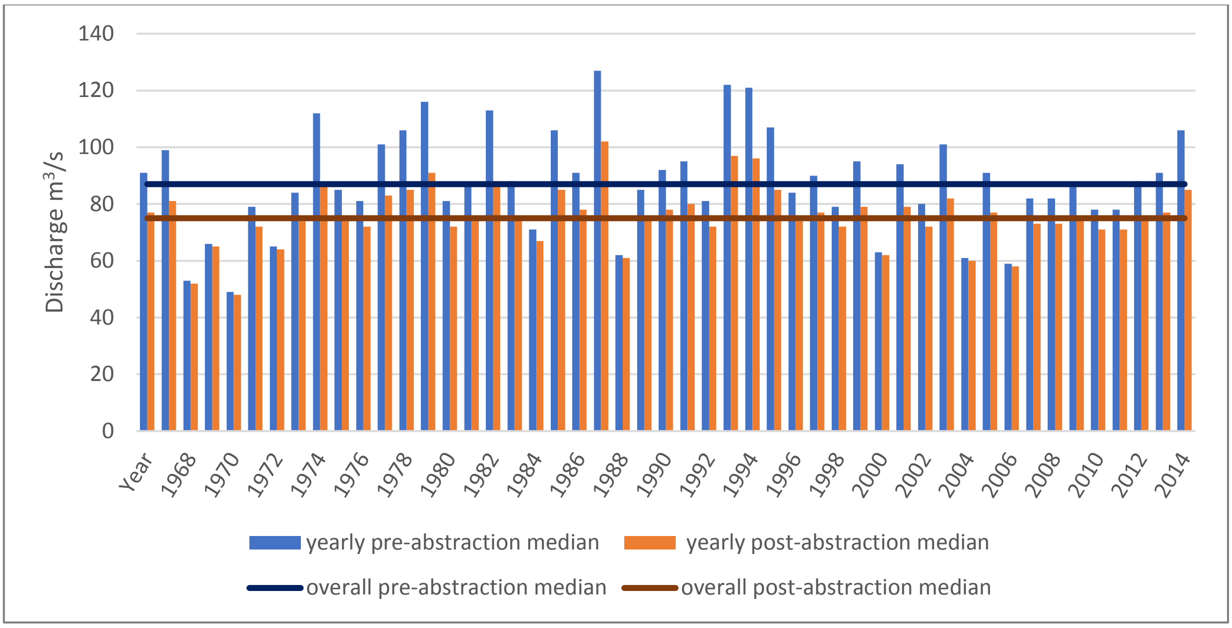


Figure 8.3 Median annual flow at the Waimakariri OHB site. Blue colours represent values corresponding to pre-abstraction conditions. Orange colours represent post-abstraction (PA) conditions.

The reductions in the 1dMALF and 7dMALF are only marginal, as consents do not allow abstractions when the unmodified flow of the river is at or below 41 m^3/s. 7dMALF values pre-abstraction are at or above 41 m^3/s in 18 of the 49 years on record, with the highest being 59.28m^3/s in 1996. 7dMALF values post-abstraction are at or above 41 m^3/s in 16 of the 49 year, with the highest being 58.20 m^3/s in 1996. Results for the 1dMALF reflect a similar trend.

The examination of the flow duration curves in Figure 8.4 also reveals that flows between the minimum 'Band B' flow of 63 m^3/s and 111 m^3/s will be most affected, with a proportional increase of the rate of abstraction with flow. Above 111 m^3/s the maximum allocation can be abstracted at any point. Between the two band minima, abstractions have a negligible effect, as only 1 m^3/s is allocated for abstraction between the two bands. Flat-lining is avoided due to flow-sharing rules above the 'Band B' minimum flow.

Based on Figure 8.2 and Figure 8.3, representative dry, wet and normal (average) years were chosen for further examination. Dry years are interpreted as years where flows are below the overall average and median flow. Concurrently, wet years are years in which average and median flows are above the overall average and median. Normal years are years in which the average and median flow correspond approximately to the overall mean/ median. A representative wet year thus occurred in 1988. The driest year on record occurred in 1971 and 1982 was a representative average year.

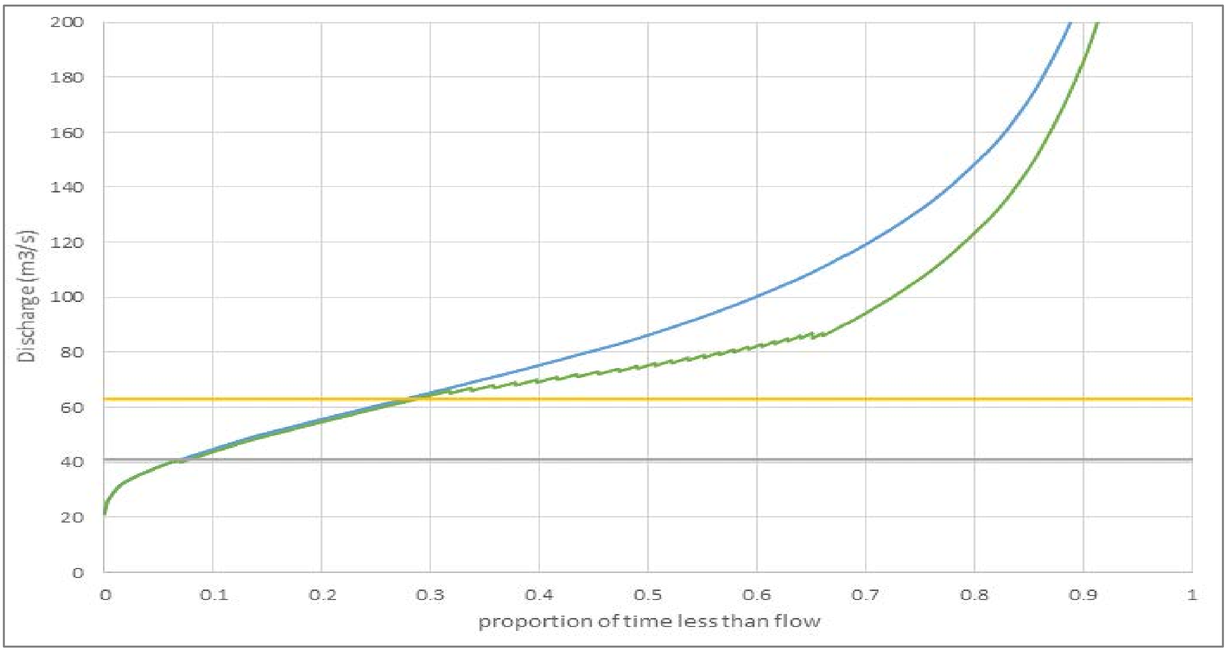
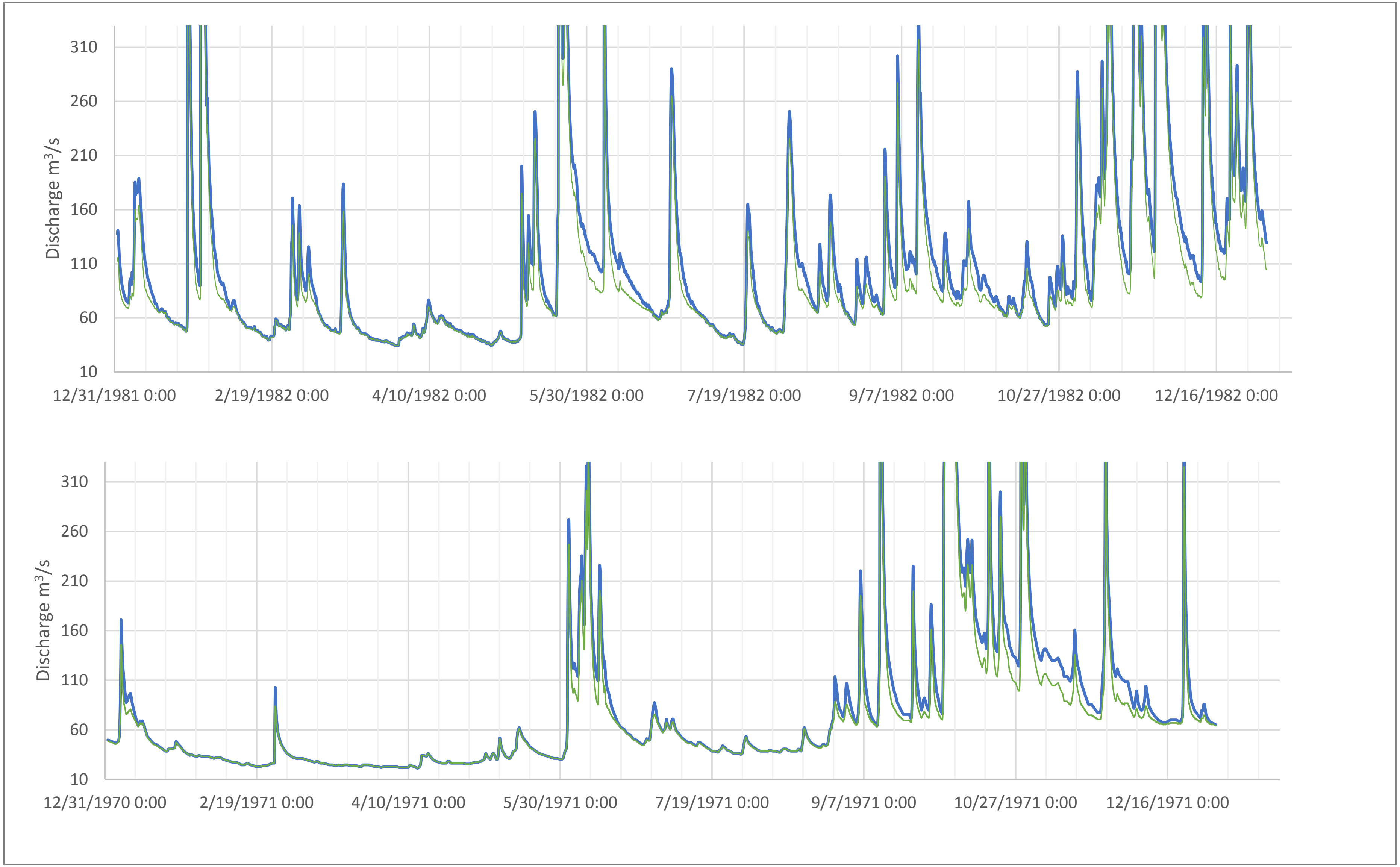


Figure 8.4 Flow duration curves for actual data (blue) and post-abstraction (PA) data (green) for the Waimakariri River. The two horizontal lines represent the 'Band A' and 'Band B' minima, 41 m^3/s and 63 m^3/s, respectively.

Figure 8.5 highlighting representative dry, wet and normal years shows that extreme low flows (< 40 m^3/s) will not be affected by abstractions. Flows below the 7dMALF value of 40 m^3/s will equally not occur more often, as stringent restrictions do not permit abstractions below the 'Band A' minimum (Figure 8.5b). Panels a, and c show however, that flows in the mid-ranges, i.e. above 63 m^3/s and below 120 m^3/s, are most affected. The frequency of larger flushing events (> 300 m^3/s) is not significantly affected by abstractions. However, the FRE_3 indicator is significantly affected. Especially unmodified flows between 261 m^3/s and 285 m^3/s, where full abstraction of 24 m^3/s is permitted reduce the amount of disturbances that are three times the median flow. Based on mean daily measurements from 1967-

2015, the average number of events greater than three times the pre-abstraction median flow reduces from 18 to 10. In 16 out of the 49 years, FRE_3 is reduced by 50 % or more. In 31 out of the 49 years, FRE_3 reduction is between 20 % and 50 %. Only one year out of the 49 years shows a reduction in FRE_3 less than 20 %, which is in 1976, where FRE3 numbers reduced from 11 to 9 occurrences. None of the years on record display an unchanged number of FRE3 occurrences post-abstraction. Figure 8.4 also shows that the total percentage of low flows markedly increases with abstractions. Equally, the duration of flood disturbances in the summer season is markedly affected (Panel b in Figure 8.5), which will be further discussed in section 8.2.2 below.

Figure 8.6 shows monthly minima and monthly median values for selected representative years in comparison. The median was chosen in this instance over the mean, as the median is not as prone to outliers (extreme flood events). Figure 8.6 highlights that the lowest flows occur during the late summer until autumn period. Spring experiences higher flows, which also coincides with the start of the riverbed bird breeding season. Once again it is visible that the abstractions affect flows above 63 m^3/s most significantly. The effect of abstractions on summer flow (period from November to March inclusive) and on the riverbed bird nesting season (period from September to January inclusive) is shown in Figure 8.7. Summer flows are least affected when flows fall below the 'Band B' minimum threshold. 'Wet' summers, i.e. mean and median flows above average flows, are most affected by abstraction as higher abstraction rates are permitted. While the lowest flow values during the summer period often coincide with the lowest flow value of the bird nesting season, this is not always the case. The bird nesting season in 1971/72 was not the driest on average, while at the same time it was one of the driest summers on record. However, the abstractions during the 1971/72 bird nesting season resulted in a reduction of 17 % in the median flow. Overall, it can be noted that the abstractions from the river result in larger reductions relative to the unmodified flow during low flow periods (e.g. summer) and smaller relative reductions during times at high flows.



310
260
210
160
110
60
10
Discharge m3/s
12/31/1981 0:00
2/19/1982 0:00
4/10/1982 0:00
5/30/1982 0:00
7/19/1982 0:00
9/7/1982 0:00
10/27/1982 0:00
12/16/1982 0:00
310
260
210
160
110
60
10
Discharge m3/s
12/31/1970 0:00
2/19/1971 0:00
4/10/1971 0:00
5/30/1971 0:00
7/19/1971 0:00
9/7/1971 0:00
10/27/1971 0:00
12/16/1971 0:00

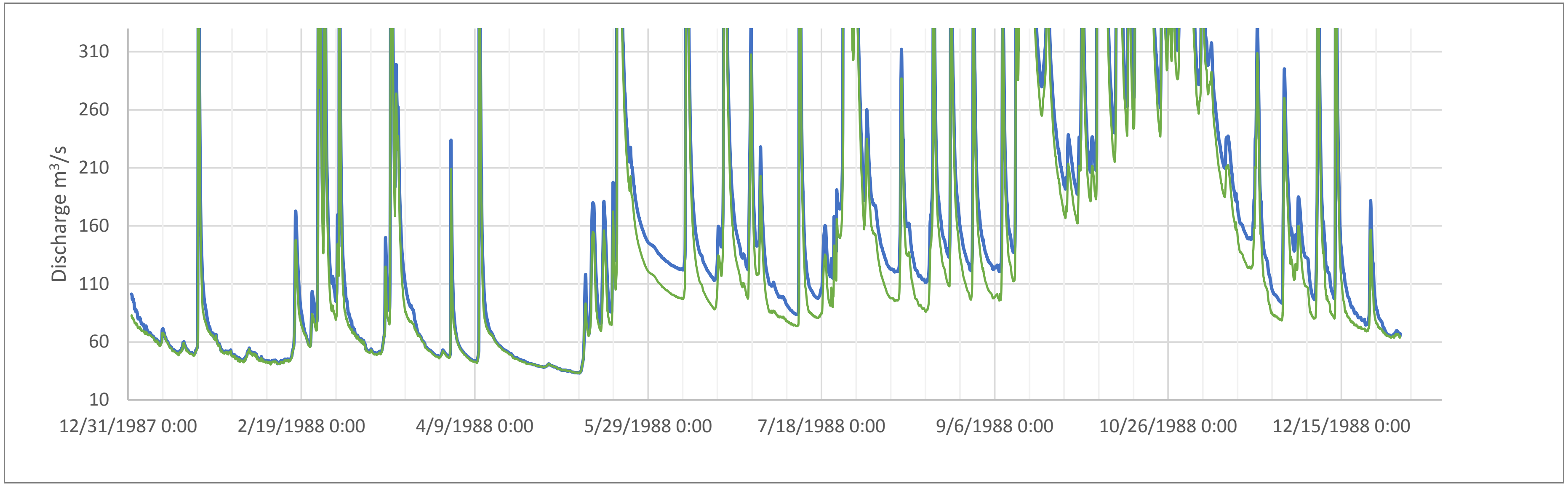


Figure 8.5 Hydrograph of (a) an average year 1982, (b) a dry year 1971 and (c) a wet year 1988. The blue line shows the actual hydrograph; the green line represents the post-abstraction time series.

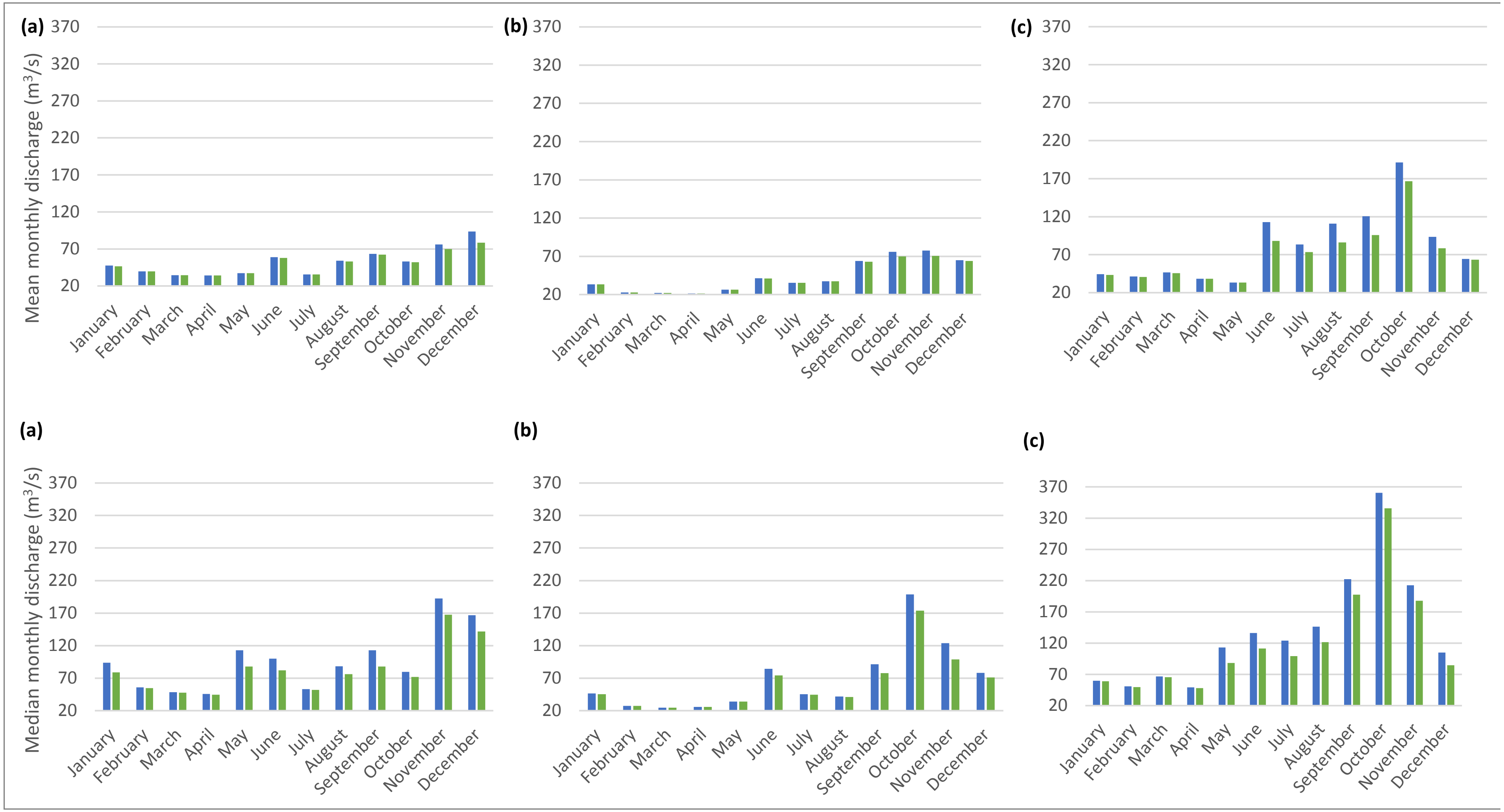


Figure 8.6 Monthly minima and median flows for a selected (a) normal, (b) dry, and (c) wet year. Pre-abstraction series are in blue colours, post-abstraction series are presented in green colours. Upper panels are monthly minimum flows, lower panels are monthly median flows.

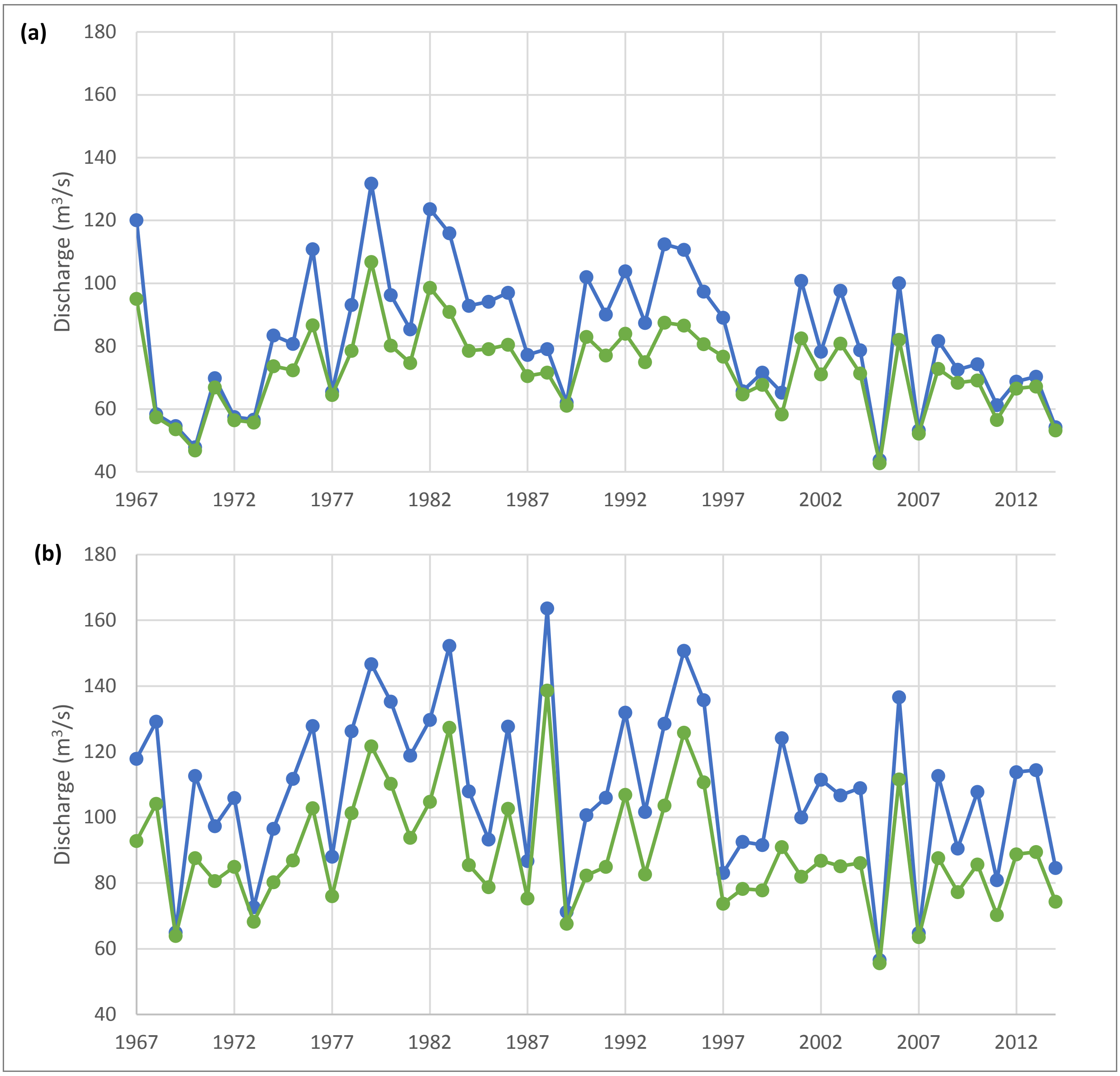


Figure 8.7 Yearly median flows for (a) the summer season (Nov-Mar), and (b) the riverbed bird nesting season (Sep-Jan). Pre-abstraction series are in blue colours, post-abstraction series are in green.

8.2.2 Frequency analysis with new time series

As most of the effects from water abstractions are visible above the 'Band B' minimum flow of 63 m^3/s, a threshold of 70 m^3/s was chosen for the extraction of low flow events and the subsequent frequency analysis of deficits, durations and magnitudes. The selected threshold includes a proportion of flows above the 'Band B' minimum flow. A higher threshold selection, such as for example, 80 m^3/s approaches the median flow of the Waimakariri River at OHB and thus does not fall into the domain of low flows. The extraction of events from the abstraction adjusted streamflow record was done in accordance with methods described in Chapter 4. Table 8.2 provides a summary of values in comparison with the pre-abstraction series of events extracted below the 70 m^3/s threshold.

Table 8.2 Summary of events below 70 m^3/s.

	Pre- abstraction	Post- abstraction
Total number	604 events	672 events
Mean duration	241 h (10.04 days)	256 h (10.67 days)
Mean deficit	16,105,029 m^3	15,625,665 m^3
Mean low	52 m^3/s	53 m^3/s
Median duration	147 h (6.13 days)	158 h (6.58 days)
Median deficit	5,189,749 m^3	4,455,484 m^3
Median low	54 m^3/s	55 m^3/s

The results indicate that the total number of events below 70 m^3/s increased from 604 to 672, with an associated increase of the mean and median duration, from 241 to 256 hours and from 147 to 158 hours, respectively. The mean and median low flow, and mean and median deficit volume decrease with abstractions. This is expected, as the abstractions primarily affect river flows above 63 m^3/s and therefore introduce a higher proportion of flows between 63 m^3/s and 70 m^3/s. Flows just slightly below the threshold level thus have smaller deficit volumes.

The series of extracted deficits, durations and low flows were modelled as described in detail in Chapter 4. The extracted time series were tested for independence. Results in chapter 5 indicated that the GEV, P3, GP and LP3 (for durations and deficits) distributions performed best and thus the selection of the best fitting distribution was based on those distributions. Goodness of fit statistics were primarily used for the selection of the best fitting distribution. The details of these results are provided in Appendix D.

8.2.2.1. Duration

The LP3 distribution was identified as the best fitting candidate for the series of extracted durations. Figure 8.8 shows design estimates for durations of flows below 70 m^3/s under current conditions (in blue) and post-abstraction (in green). An overall increase in low flow durations is noticeable. While a 3-year event would last 798 h pre-abstraction, durations are 888 h after abstractions have taken place. This constitutes a 10 % increase in durations for a 3-year event. The 10-year duration estimate is 9 % longer after abstractions have taken place (i.e. an increase from 1708 h to 1878 h), and the 100-year duration estimate is 8 % longer for the abstraction scenario (i.e. increase from 4344 h to 4698 h).

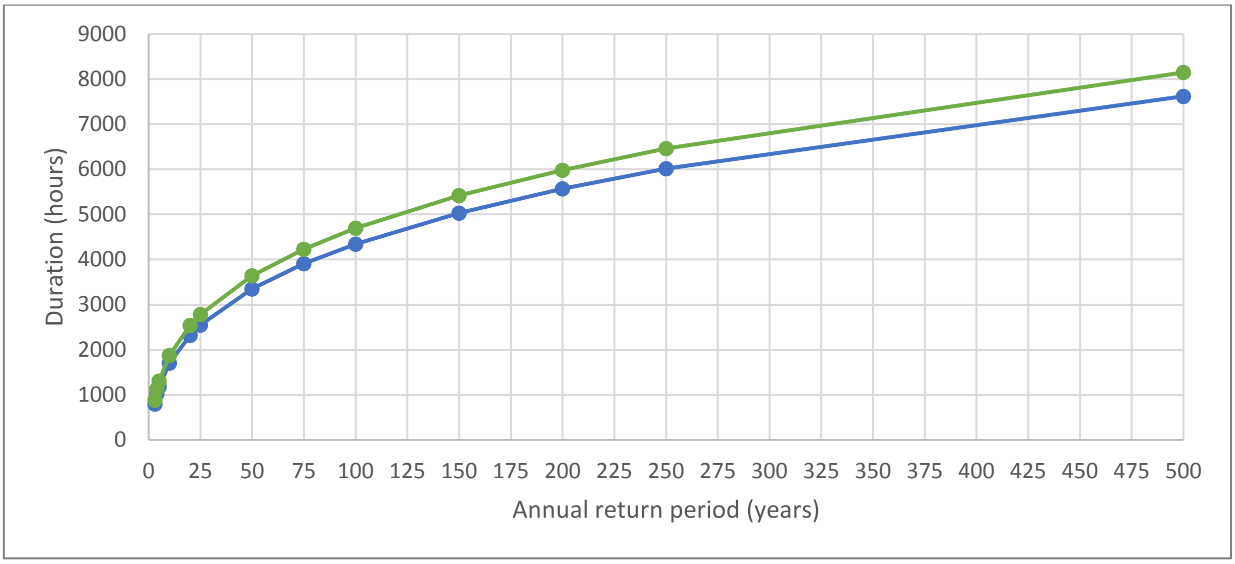


Figure 8.8 Design estimates for durations of flows below 70 m³/s. The green line shows results for the time series post-abstraction, the blue line is the pre-abstraction series.

8.2.2.2. Deficit[27]

For the series of deficits, the LP3 distribution was identified as the best fitting distribution for pre- and post-abstraction series. A large increase in deficit estimates is evident from Figure 8.9, especially with increasing magnitude of events. Currently, a 3-year return event has a total deficit volume of 77,140 m^3 (LP3). This number increases by 13 % to 88,800 m^3 after abstractions have taken place. The 10-year estimate increases from 244,750 m^3 to 329,690 m^3 (a 26 % increase), and the 100-year estimate deficit volume increases from 965,700 m^3 to 1,648,880 m^3 (a 41 % increase).

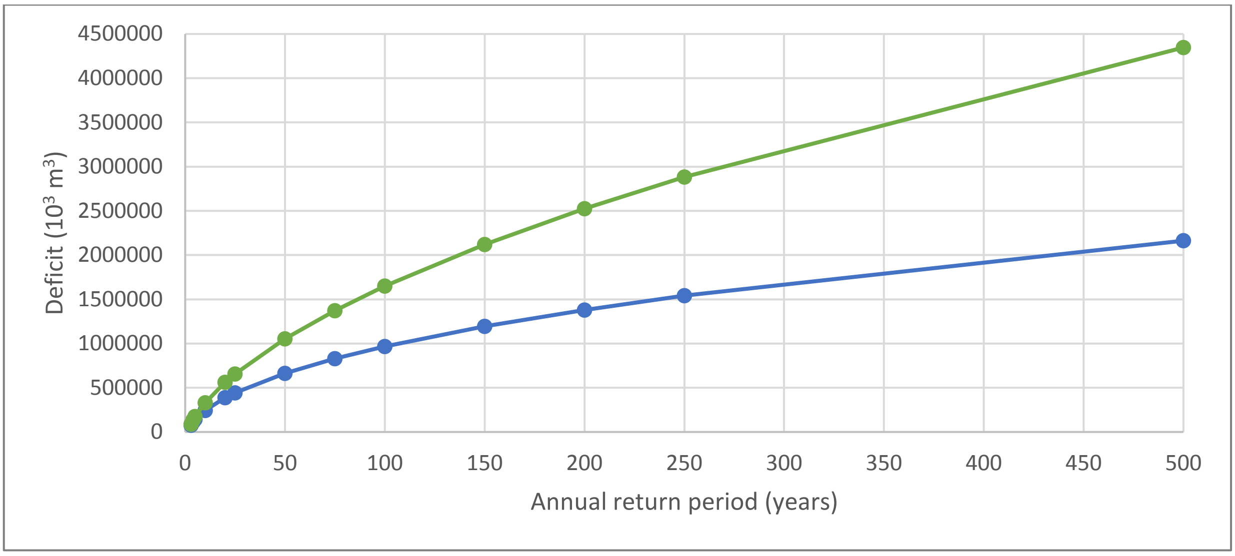


Figure 8.9 Design estimates for deficits for flows below 70 m³/s. The green line shows results for the time series post abstraction, the blue line is the pre-abstraction series.

[27] Note: all deficit values are given in 10^3 m^3

8.2.2.3. Lowest value

The GP distribution was chosen for the series of low flow events occurring below the 70 m^3/s threshold, as it was the statistically best fitting distribution. Figure 8.10 shows that with abstractions, the lowest instantaneous flow recorded for each low flow event only marginally decreases. This is expected, as stringent restrictions do not allow abstractions below 41 m^3/s. A large proportion of low flows in both time series fall below this level. For each design estimate produced, the lowest flow after abstractions is within 1 m^3/s of the values produced for the pre-abstraction series.

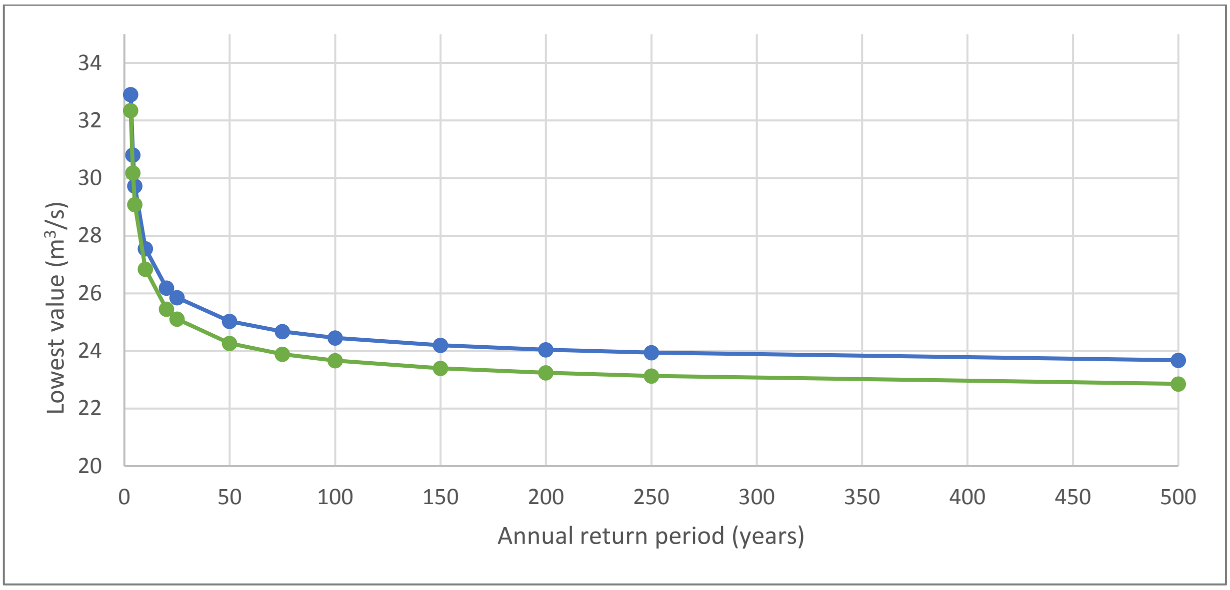


Figure 8.10 Design estimates for lowest flow recorded per event below 70 m^3/s. The green line shows results for the time series after abstractions, the blue line is the pre-abstraction series.

8.2.2.4. Magnitude

To produce design estimates of dimensionless magnitudes (i.e. deficit/duration), the GP distribution was identified as the best fitting distribution. Figure 8.11 shows that the decrease in river discharge leads to a higher overall magnitude for the design estimates produced. While a 3-year return period magnitude under current conditions is estimated to be 6.97, the magnitude after abstraction is 7.38. A 10-year event currently has a magnitude of 15.45, while it increases to 16.65 with abstractions. The 100-year event currently corresponds to a magnitude of 42.29, while the post-abstraction series produces an estimate of 47.06. The longest drought and lowest flows on record during the summer season of 1971 has an estimated magnitude with a 50-year return period under pre-abstraction conditions.

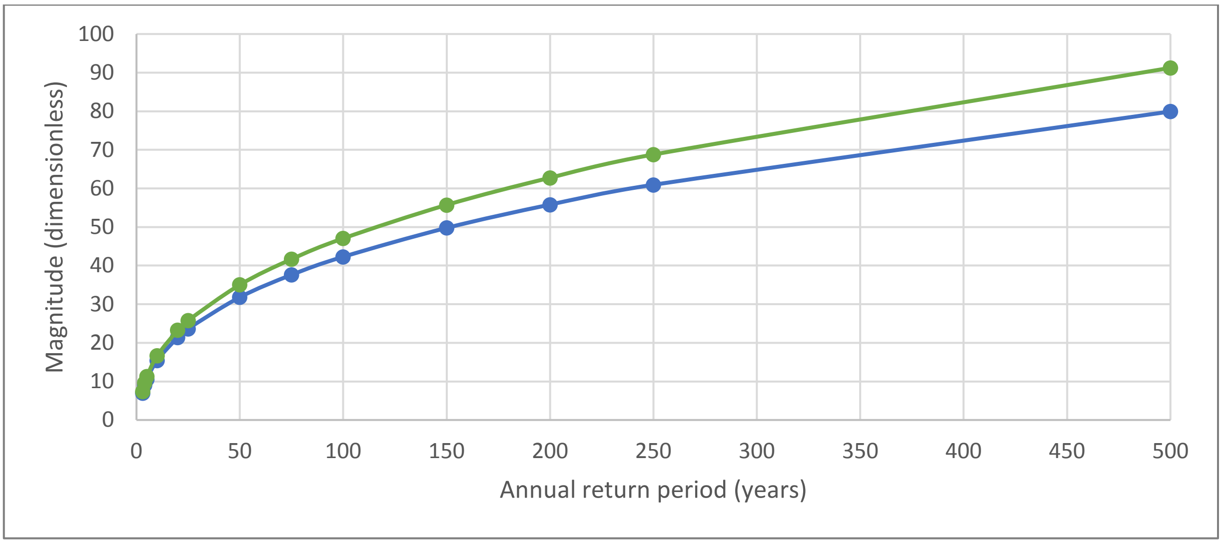


Figure 8.11 Design estimates for magnitudes below 70 m^3/s. The green line shows results for the time series post-abstraction, the blue line is the pre-abstraction series.

8.3 Summary

The results from the rapid systematic literature review highlight the importance of varied flow magnitudes for the maintenance of habitat quality and quantity. The second part presented a detailed account of changes to the flow regime as a result of abstractions from the river. Results indicated that flows between 63 m^3/s and 120 m^3/s, thus habitat quantity ensuring flows, are most affected by abstractions. The frequency analysis also showed that overall, an increase in the duration, deficits and magnitudes of low flows can be expected. In the next chapter, the results from both parts of the study, the frequency analysis of extreme flows, and the effects of abstractions on the river regime and environment are discussed.

Chapter 9

Discussion

The purpose of this study has been the modelling of extreme events in the Waimakariri River using and comparing the well-developed annual maximum series (AMS) approach and the less frequently used partial duration series (PDS) approach. Additionally, low flow frequency quantile estimates were calculated for the discharge series from 1967-2015, and for a modelled post-abstraction series. The second part of the study was aimed at identifying and quantifying the environmental consequences occurring from water abstractions in the Waimakariri River by interpreting changes to the discharge regime. The results obtained from both parts of the study (presented in Chapters 5 and 8) are discussed in light of the current state of knowledge reviewed in Chapters 3 and 6, in order to answer the questions guiding this research (Chapter 1).

9.1 Flood frequency analysis

The first part of this thesis dealt with the estimation of design floods for the Waimakariri River, using hourly discharge measurements from 1967 to 2015 inclusive. Flood estimates were also produced by adding historical gaugings from 1930 to 1966 to AMS extracted from 1967 onwards in order to compare the estimates produced in this thesis with other studies' results. This study used partial duration series of the discharge measurements at the OHB site to sample extreme events in the Waimakariri River for the first time. Previous frequency analyses of the Waimakariri River have exclusively depended on the use of AMS to make predictions about the return period of significant flood events. A valid explanation for the exclusive choice of AMS over PDS is ensuring the independence of subsequent flood peaks, a major premise in frequency analyses. It is advisable, however, to validate independence of events even when sampling annual maximum series, as subsequent peaks can occur, for example, at the end of one year and the beginning of the next. By chance, this was not observed in the AMS for the Waimakariri River. It is generally understood in the literature that a selection of a sufficiently high threshold validates the premise of independence for PDS.

9.1.1 Threshold selection

Statistical independence of selected PDS in this study was ensured by applying strict selection criteria to flood events occurring above the threshold. The selection of the highest threshold was guided by Cunnane's (1973) recommendations, in which the average number of exceedances is at least 1.65. This put an upper limit to the threshold selection at 1000 m^3/s. In light of a lack of guidelines regarding the selection of thresholds below this maximum, NERC (1975) recommendations were adopted and deemed sufficient to ensure statistical independence of PDS and to rule out serial correlation. However, it could be argued that the minimum number of interval days imposed between two peak events[28] is unwarranted. Indeed, the fact that flood events in the Waimakariri catchment can be the results of storm events with varying prevailing winds is an argument for the abandonment of strict inter-event guidelines altogether. Connell and Pearson (2001) suggested the use of a Two-Component Extreme Value distribution for this precise reason. The TCEV distribution is particularly attractive in cases where two separate flood-producing phenomena are present. Within the Waimakariri catchment, northwest winds are the common underlying factor causing large 'outlier' floods, such as the floods of 1940 and 1957. Smaller flood events are likely caused by southerly and south-easterly wind conditions, and sometimes north-easterly wind directions from subtropical depressions. Identifying and distinguishing separate flood-producing phenomena within the Waimakariri catchment, perhaps with the aid of rainfall data, could therefore reduce the imposed inter-event time between flood peaks, without compromising independence of flood events from antecedent conditions. However, an investigation of such scale was not within the scope of this research and therefore the recommendations of NERC (1975), one of many alternative choices, were adopted as a suitable choice.

The clear lack of uniformly applicable guidelines for PDS sampling explains the preferred choice of AMS. However, as visible in the results, the PDS, even with a relatively high threshold such as 1000 m^3/s adds 45 % more events to the time series of flood events, which is particularly useful in cases when the discharge record is short, relative to the return period to be estimated. A reasonable 100-year design estimate is based on at least 50 years of annual maxima. However, the 250 years of records required for a robust 500-year return period estimate are not available to date, as most systematic gauging only began at the start of the 20th century. Systematic records in New Zealand are even younger, with many catchment records only commencing in the 1960s (Appendix A). The PDS proves a valuable tool for extending the amount of data, i.e. flood events, to be extracted from the record. A standardised, reliable method for selecting the optimal threshold level was found when plotting the mean of exceedances above the threshold as a linear function of the threshold level $[E(X_s)-S]$. The PDS resulting from a threshold level where $E(X_s)$ is a linear function of the threshold level S, maximises the stability of the distribution parameter estimates (Lang et al. 1999). This method alone was found to be sufficient to determine that the best threshold level for the Waimakariri River flood series was between 650 m^3/s and 800 m^3/s.

Irrespective of the choice of distribution to produce the design estimates, a comparison of design estimates produced by AMS and PDS shows marked differences. The AMS including the historical annual flood data (AMS_{hist}) at the OHB site (1930-2015), consistently produces higher estimates than the AMS derived from hourly measurement data (1967-2015). This is due to the influence of three particularly large flood events, namely 1940 (3740 m^3/s), 1950 (3090 m^3/s) and 1957 (3990 m^3/s), which have not

[28] min. 5 days inter-event time, unless flow drops below 75 % of the lower of the two peaks

been surpassed since. Since the installation of the automated recorder, the highest peak has been documented in 1979 with 2835 m^3/s. Concurrently, the results show that the three PDS including 122 to 178 events (i.e. threshold levels of 900 m^3/s, 800 m^3/s and 750 m^3/s) approach the estimates produced using the AMS_{hist}. This is particularly obvious for the series including 162 and 178 events[29] (LP3 and GEV distributions Figure 5.9 and Figure 5.10), which also coincidentally fall into the ranges of the optimal threshold levels as determined by the *[E(X$_s$)-S]* plot. It could be thus argued, that in the case of the Waimakariri catchment, the result produced in this thesis support the use of PDS sampling to make up for 37 years of missing systematic data (1930-1966) to end up at similar estimates as those produced by the use of the AMS_{hist}. By extension, it is also conceivable that the addition of partial duration series data from 1930-1966, if it were available, would return significantly higher estimates than those of the AMS_{hist}, or at the least, higher estimates than currently produced by the PDS record form 1967-2015. This would also be conducive to the idea that lesser weight is to be attached to the historical data (1930-1966), given the smaller confidence in the accuracy of measurements of discharge data at the time, or perhaps that design estimates including such data are given varying degrees of certainty, depending on the data source. Bayesian flood frequency analysis has been previously explored for such reasons (Kuczera, 1987) and is increasingly being used.

9.1.2 Distribution choice

Among eight distribution candidates, the Log-Pearson Type 3, Generalised Pareto, General Extreme Value and Pearson Type 3 distributions performed the best, as determined by extensive graphical and statistical testing. While L-moment ratio diagrams are typically used in regional rather than in at-site frequency estimation, it unequivocally ruled out the Normal, Gumbel (EV1) and Exponential distributions as possible best fit candidates. Probability and quantile plots further supported this result. The LP3, P3, GP and GEV distributions were statistically confirmed as good fitting candidates by Kolmogorov-Smirnov and chi-squared tests. Statistical testing also concluded that the Exponential, Normal, Log-Normal and EV1 distributions were not well fitting distributions at the 5 % significance level. This was particularly surprising, as the bulk of the scientific literature supports the use of the Gumbel distribution for the purpose of flood frequency analysis.

As a two-parameter distribution, the EV1 distribution is preferred for the ease of deriving the describing parameters statistically. It thus also has a lower standard error associated with estimates produced from short records (e.g. 10 to 20 years). The application of the EV1 distribution in extreme value analysis is well documented on theoretical grounds, as it describes a distribution with most values in the lower ranges and few values in the right-hand tail, i.e. a positively skewed distribution (Pearson & Davies, 1997). Slade (1936) (cited in Dalrymple, 1960) noted that estimating skewness from a sample of

[29] thresholds at 800 m^3/s and 750 m^3/s

50 events or smaller is rather meaningless and that only in the ranges of 140 samples does it truly give a meaningful characterisation of the distribution. New Zealand studies of flood frequency similarly have a clear preference for the use of the EV1 distribution. Beable and McKerchar (1982) chose to use EV1 distributions for AMS of New Zealand catchments and concluded that the EV1 distribution performs well for at-site frequency analyses using AMS. Series that did not display EV1 tendencies but rather conformed to the EV2 distribution were modified by extracting the biennial or triennial interval series, so that the EV1 distribution fitted well. Connell and Pearson (2001) further confirmed that Main Divide rivers, whose predominant flood producing processes come from westerly storms, are described by EV1 distributions. The PDS counterpart was well modelled by a GP distribution. However, Pearson (1991) showed that eastern catchments of the South Island, to which part of the Waimakariri catchment belongs, are better described by the EV2 distribution using an AM series. A review of flood frequency in the Canterbury region also confirmed the EV2 fit to eastern South Canterbury rivers (ECan, 2011a). Ware and Lad (2003) first reported the use of other distribution candidates, such as the Generalised Logistic and the LN3 distribution, for flood quantile estimates of the Waimakariri River. The suitability of the P3/LP3 distributions for the purpose of PDS or AMS frequency analysis of the Waimakariri River has only recently been tested (Steel, 2016) and it was concluded that the LP3 distribution is a good fit for the historical and systemic record at the OHB site.

The LP3 distribution, which conformed well to the empirical partial duration and annual maximum series in this study, is the standard distribution adopted within the United States as part of the Bulletin 17B flood frequency analysis by federal agencies (Kirby & Moss, 1987), and the recommended distribution for the description of floods in Australia by the Institute of Engineers. NERC (1975) commented on the better fit of three-parameter distributions in general (to which the LP3 distribution belongs). However, the LP3 distribution has to be applied with care, as the use of the logarithm can overemphasise the importance of smaller events within the series and result in large variability; the latter has been confirmed by the results in this study. Numerous investigations in Australian catchments have since concluded that the GP and GEV distributions, which in this study rank 3rd and 4th respectively in regards to goodness of fit, are better fitting distributions (Haddad & Rahman, 2008; Rahman, Karim, & Rahman, 2015).

9.1.3 Comparison of AMS results

Although the EV1 distribution was not a well-fitting distribution for the PDS and the AMS in this study, design estimates were produced for comparative purposes. The results for the AMS, including historical data from 1930-1966, produced a design estimate of 3352 m^3/s with the Gumbel distribution for the 100-year flood. The AMS using data from 1967-2015 has a 1 % AEP estimate of 3075 m^3/s. The lower estimate reflects the three large missing flood events that are representative of the historical series. The 100-year estimates produced in this study using the Gumbel distribution and L-moments technique for parameter estimation are lower than previously produced estimates for the Waimakariri River. Stephen (1958) estimated the 100-year return period flood at 4290 m^3/s. This estimate used data from 1929-1957 and the EV1 distribution with the least squared method for parameter estimation. The 4290 m^3/s estimate also includes a 10 % safety factor. Therefore the estimate is 3900 m^3/s, which is 540 m^3/s higher than the results in this study (AMS_{hist}). Pearson (1988) came to a similar conclusion 30 years later, also employing the EV1 distribution as the best fitting distribution for the AMS. The 1 % AEP was estimated to be 3830 m^3/s (± 380 m^3/s). At the lower confidence range (3830 m^3/s - 380 m^3/s), Pearson's (1988) estimate is comparable to the one in this study. Pearson (1988) also asserted that the 1 % AEP would not significantly change in the next 30 years to come. ECan's review of the flood frequency in Canterbury (2011a), however, estimated the 100-year flood some 23 years later at 4155 m^3/s. This estimate is 800 m^3/s higher than the one produced in this study (AMS_{hist}), and 325 m^3/s higher than the estimate of Pearson (1988). The report by ECan assessing flood frequency in the Waimakariri River (2011a) states that fitting an EV1 distribution to a site displaying EV2 tendencies will underestimate larger return periods, which is perhaps an explanation for the low 100-year estimates in this study relative to previous estimates. Pearson (1988) for example extracted biennial series from the annual maximum series to ensure an adequate EV1 fit to the data. The results from ECan (2011a) are based on a similar procedure. The difference in design estimates is also a function of varying frequency analysis procedures (e.g. parameter estimation techniques) and the relatively larger number of low annual minima reflected in the study's dataset compared to previously employed data sets (cf. Figure 2.2). Better fitting distributions, such as the LP3 and GEV distributions, match previous design estimates more closely. The 100-year estimate (AMS_{hist}) for the LP3 and GEV distributions in this study are 4170 m^3/s and 4070 m^3/s, respectively. Ware and Lad (2003) estimated 4232 m^3/s as the 100-year return flood, using the GEV distribution. This is comparable to the results in this study producing 4070 m^3/s (GEV distribution, AMS_{hist}). The results in this study are also comparable to the most recently published report by ECan (Steel, 2016). The 1 % AEP estimates produced in this study using historical data (AMS_{hist}) and the GEV/ LP3 distributions are within 5 % of the estimates produced by Steel (2016).

9.1.4 Comparison of PDS results

The comment has been previously made, that the chosen PDS with 49 years of data (1967-2015) produces similar estimates to those of the AMS_{hist} (86 years of data). Concurrently, the AMS using only 49 years underestimates the magnitude of events, relative to the AMS_{hist}. For the 100-year estimate, the choice of 162 or 178 events for the PDS is optimal.

Based on statistical goodness of fit testing of these series in comparison with the AMS, it is evident that the choice of the PDS gives a smoother and more homogeneous fit to the empirical data overall. In the case of the LP3 distribution, the Filliben Correlation Coefficient performs more strongly for both PDS in comparison with the AMS. Chi-squared and KS tests also indicate that the PDS is the preferred choice, as the empirical data fits smoother to the theoretical distribution. Results are equivalent for the GP, GEV and P3 distributions. Either of the four fitting distributions fit more closely to the PDS with 178 or 162 events than to the AMS. The choice of the PDS over the AMS also resulted in a significantly higher design estimates for the LP3, GP and GEV distributions.

Apart from the influence of candidate distributions on design estimates, the choice of the threshold level and resulting PDS exerts significant influence. Both effects are confirmed graphically in Figure 5.13. The boxplot represents pooled quantile estimates from PDS for each distribution and increasing return period. Quantile estimates from all selected threshold levels were pooled together to display (i) the variances between distributions for each return period estimate, and (ii) the variance within each distribution candidate resulting from threshold selection. ANOVA testing showed that the variance within distribution estimates progressively increases with higher return periods. This shows that for higher return periods, the threshold level choice exerts a significant influence on the resulting design estimate. At lower return periods (i.e. 5 years, 10 years and 25 years) the GEV, LP3, P3, and GP display smaller within variances; and thus the threshold choice has a smaller influence. The boxplot also shows that the EV1 distribution consistently produces the smallest design estimate at each return period level. The differences between distributions are in fact statistically significant at return periods higher than 5 years, i.e. the EV1 distribution is statistically significantly different from the other (well-fitting) distributions. This agrees with the comments made in the report by ECan (2011a) which states that fitting of the EVI distribution, when not appropriate, will underestimate larger return periods.

9.1.5 Implications for floodplain management

The flood hazard posed by the Waimakariri River is translated into an ever increasing risk due to the fast expansion of nearby urban areas. The risk is not only limited to economic costs, but extreme floods can have an equal impact on society and the environment. It is now standard practice to employ estimates of probabilities of extreme flood events in the conceptualisation and design of flood protection systems, despite large discrepancies in methodological approaches, and thus design estimates (Klemes, 1988). A flood frequency curve is not a characteristic of the catchment under study, but rather a function of the data on record. This distinction should be kept in mind, as only a few data points can have a large influence on the produced frequency curve, which was demonstrated by the large difference in design estimates produced by the AMS and AMS_{hist}, and is further demonstrated below.

Perhaps the largest discrepancy between the design estimates produced within this study are estimates for return periods of 100 years and larger. Frequency estimation larger than the 100-year return period are fraught with large uncertainties; estimation for events with no comparable entry in the record (500 years or larger) are even more challenging. The 500-year return period estimate in this study ranged from 4048 m^3/s (P3 distribution) to 6956 m^3/s (GEV distribution), using the PDS series with an 800 m^3/s threshold. For the 1 % AEP, the lowest estimate was 3283 m^3/s (P3) and the largest estimate was 4227 m^3/s (GEV). The larger difference between the 500-year estimates is a function of the relatively small number of flood events used to predict such a large return period. The magnitude of differences also gives an indication of the influence of individual candidate distributions' shapes on the resulting design flood discharge estimate and could be interpreted as a measure of uncertainty arising from the choice of probability distributions.

The uncertainties presented above from the choice of distribution alone can have major implications for hazard planning and floodplain management in the region. It is alarming that an uncertainty of 42 % is present from a choice among statistically well-fitting distributions. To give an indication of this level of discrepancy, the reader should bear in mind that under this scenario, a flood defence designed to cope with a 100-year flood based on the P3 distribution and the PDS with 162 events (3280 m^3/s) would be well below the 1 in 50 year event estimate based on the GEV distribution with 162 events (3430 m^3/s). Therefore, the level of protection offered by flood protection works in the Waimakariri catchment has to be questioned, particularly as the results in this study clearly demonstrate that four alternative distributions fit the empirical flood data better than the one currently employed for hazard planning (EV1 distribution). Differences between estimates at such scale may also be an explanation why and how flood defences fail during flood events that have much smaller return periods than the designed level of protection.

The current level of protection for the Waimakariri River is designed to withstand a 1 in 450 - 500 year flood event plus 0.9 m freeboard, estimated at 4700 m^3/s (ECan & Waimakariri District Council, 2003). Continuous gravel extraction is required to maintain the river's capacity to convey a flood of such extent. The risk of stopbank breach is estimated to be 5 % during a 20-year return period flood event in the range of 2250 m^3/s - 2750 m^3/s. The results based on the PDS with 49 years of data agree with these estimates. However, it is expected that PDS sampling of the entire record, including data from 1930-1966, would achieve higher estimates for the 5 % AEP, especially with the inclusion of the three largest flood events on record. The chance of a breach is significantly higher with 15 % during a 100-year flood event (3300 m^3/s - 3750 m^3/s). The GEV distribution predicts flood flows of 4230 m^3/s during a 100-year event, which is markedly larger than the flood level estimated in the Waimakariri Floodplain Flood Warning and Emergency Evacuation Plan (ECan, 2013). Events of the size predicted in this study approach the flood ranges attributed to a 500-year event flood, according to ECan (2013). The 10,000-year flood is estimated to be 6500 m^3/s (ECan, 2013), during which multiple breaches are predicted to occur along the primary stopbank system. Apart from the fact that a 10,000-year flood event estimation based on an 87 year data record is speculation at best, the 500-year flood estimate, based on the PDS with 162 events and a GEV distribution, returns a markedly higher estimate at 6956 m^3/s. The LP3 500-year estimate is also significantly larger than the design capacity of the primary stopbank system, with a flood of 6230 m^3/s. The GP distribution estimate is equally higher than the design capacity, at 4950

m^3/s. Among the best fitting distributions, the P3 distribution is the only one that produces estimates agreeable with the 500-year return period capacity of the primary stopbank system (4050 m^3/s).

In light of the large risk of flooding from the Waimakariri River posed to Christchurch, the construction of a secondary stopbank is underway. This stopbank is designed to provide an added level of protection from large flood events with return periods of up to 10,000 years, estimated at 6500 m^3/s. Extensive hydraulic modelling has been undertaken to assess the extent of such flooding and to quantify the advantages of the secondary stopbank system (ECan et al., 2007). The assessment of the reliability of the primary stopbank system strongly emphasised the risk of failure due to erosional processes. However, no mention has been made about the uncertainty of the flood frequency estimation processes that underpin such assessments. Given the flood damage costs of the maximum probable flood are estimated to be NZ $ 5 billion (ECan et al., 2007), the adoption of a conservative approach in the design of flood defences is preferred. With the availability of statistical software it is now a relatively simple task to identify and compare the best fitting candidate distribution and their estimated design floods for selected catchments. With the effect of climate (Mosley, 2000) and climate change on flood producing phenomena (NIWA, 2010), the adoption of more conservative (higher) flood estimates for flood defence systems can accommodate for anticipated increases in flood magnitudes, while also increasing the life-time of flood defences.

9.2 Low flow frequency analysis

In addition to flood frequency analyses, the frequency of low flows in the Waimakariri River were assessed. Contrary to the wealth of information present in the literature regarding flood frequency analyses for Canterbury Rivers, little to no information is present about low flow frequency analyses. However, the estimation of low flow frequency curves is not only fundamental to river/water engineers for assessing management strategies, but it is also relevant for environmental scientists and ecologists who need such data to quantify relationships between aquatic ecosystem processes and river flows, a discussion of which will follow later.

Common practice for water management strategies, which includes water allocation to out-of-river uses, are primarily based on flow duration curves (FDC). Specific allocation limits are, for example, based on the lower part of the FDC, which may be determined as the median flow, or a portion of the MALF. Indices, such as the *n*-day MALF are also used for abstraction licencing (Franklin et al., 2012). While FDCs show a relationship between discharge and the percentage of time that discharge is exceeded at a first glance, the relative percentage is directly related to the record period. Furthermore, a FDC gives no indication about the probability of a discharge being exceeded, which is valuable information for judging the reliability of the river as a water source.

For this purpose, low flow frequency curves were constructed in this study, using and comparing two different approaches: (i) low flow analysis on the basis of a series of *n*-day mean annual minima

(AMS), and (ii) low flow analysis based on 'runs theory', i.e. analyses of the frequency of low flows, deficits and durations below a chosen threshold (PDS).

9.2.1 *n*-day mean annual low flow

Both the 1-day mean annual low flow series (1dMALF) and the 7-day mean annual low flow series (7dMALF) were extracted from the hourly discharge record (1967-2015) of the Waimakariri River at the OHB site. The 7dMALF is used worldwide for low flow analysis, as it eliminates the day-to-day variations present in the hydrograph (Smakhtin, 2001). It is extensively used in the USA, in Canada, the UK, in Australia and in New Zealand (Franklin et al., 2012; McMahon & Arenas, 1982; Pryce, 2004; Smakhtin, 2001). A comparison of the 1dMALF and the 7dMALF in this study show only slight differences, which has also been noted previously by Smakhtin (2001). No serial correlation or trend was detected in either one of the series, which is consistent with Pearson's (1995) previous findings from 500 catchments within New Zealand. No previously published studies have addressed low flow frequency analysis by means of the 7dMALF specifically in the Canterbury region. However, studies have explored low flow frequency analysis New Zealand wide or in selected regions. Caruso (2000), for example, explored low flow frequency in Otago rivers, using the 10-year return period of the 7dMALF for comparison. He employed the LN, EV3 and EV1 distributions and concluded that in most cases, the LN estimate was higher than the other two candidates' estimates. Estimations by the EV1 distribution were generally lower. In a subsequent study, the LN distribution was confirmed as a good fit (Caruso, 2002). Pearson (1995) derived regional patterns of low flows, using 500 catchments in New Zealand. He used 1d, 7d and 30d MALF series to produce an index low flow, analogous to the index mean annual flood flow, used in contour maps (Pearson, 1991). For Canterbury rivers, the Generalised Normal distributions was a good fit. For a conference paper, Gamage (2008) utilised the LP3 distribution for the Ahuriri River and Mary Burn River, also using the 10-year return low flow of the 7dMALF to assess the environmental effects of low flows.

From the review of the literature in Chapter 3, it is evident that, analogous to the flood frequency analysis, no clear guidelines exist for the preferred choice of a distribution. In this study, the GEV and P3 distributions were statistically the best fitting distributions for the series of 1dMALF and 7dMALF. This is consistent with the general recommendations in the literature, which suggest the EV3 (Weibull) distribution for low flow frequency analyses (Pearson & Davies, 1997; Smakhtin, 2001). The GEV distribution contains the EV3 distribution as a special case (Stedinger et al., 1993). The EV1 distribution was rejected by the chi-squared test, however, was accepted by the KS test. For the 7dMALF series, the GP and N distribution ranked 3rd and 4th, respectively. As the N distribution was rejected by the chi-squared test, it has been excluded as a distribution candidate. The use of the EV1 distribution for low flow frequency analyses has to be treated cautiously, as there are no bounds on the lower tail of the distribution. This means, that theoretically, negative values can be estimated, which have to be treated as zero flows. The frequency analysis of the 7dMALF series, using and EV1 distribution, did in fact return a negative value for the 500-year estimate (-3.2 m^3/s). Caruso (2000) notes that the presence of negative return values indicates that the EV1 distribution underestimates low flows in relation to other distributions. The results in this study showed that, in comparison with the three preferred distributions, the EV1 distribution only returned lower estimates at return periods of 10 years or higher. The estimates for the 7dMALF are consistent across the three preferred distributions, with values ranging from 37.46 m^3/s (GP) to 37.92 m^3/s (P3) for a 2-year return period, and from 28.04 m^3/s (GEV) to 28.24 m^3/s (GP) for a 10-year return period. The variance between the distributions' estimates increases with increasing return period, however, variances are not as large as for flood frequency estimates. For the 100-year return period, estimates range from 22.91 m^3/s (P3) to 27.51 m^3/s (GP). The frequency analysis of this study thus suggests that the severe drought period of 1971 (7dMALF was 22.45 m^3/s) was of the magnitude of a 100-year event according to the P3 distribution, and greater than a 500-year magnitude event, according to the GEV distribution. Overall, the P3, GP and GEV distributions return similar estimates for smaller (< 500 years) quantiles. Results produced in this thesis estimate larger 10-year low flows than previous estimates, which were presented at the CPWL consents hearings. De Joux (2008) estimated a 7dMALF flow of 26.8 m^3/s for the 10-year return period, using an EV3 distribution.

For the analysis of low flows based on annual minima, the fixed duration (here 1 day, or 7 days) becomes the critical parameter. This might constitute a problem in terms of the definition of the extreme value region of low flows, as in wet years, the extracted 7dMALF may not belong to true extreme low flows (e.g. the 1996 7dMALF was 59.28 m^3/s). In the truncation level approach, the chosen threshold level, based on the problem definition, becomes the critical parameter and events that do not fall below the threshold are excluded from analyses.

9.2.2 Truncation level approach/ runs theory

Unlike flood flows, low flows are characterised by multiple dimensions. The description of the severity (deficit) and duration of low flows can give additional information to water resource managers and engineers. The truncation level approach, in combination with PDS sampling was used in this study to add value to the frequency estimates produces by *n*-day mean annual minima. Since the introduction of the idea in 1967 (Yevjevich, 1967), the work of Zelenhasic and Salvai (1987) has been cited most often in relation to its application. Runs theory requires the choice of a threshold level, analogous to the idea of a threshold level in PDS modelling of flood frequencies. For the purpose of this part of the study, thresholds were set at 60 m^3/s, 40 m^3/s and 35 m^3/s. The second part of the study, which includes modelling of abstractions also employs a threshold at 70 m^3/s. The 40 m^3/s threshold was chosen, as it represents the average 7dMALF conditions over the 49 years of data. The remaining thresholds were chosen as a reflection of the water allocation regime for out-of-stream uses present for the Waimakariri River. The range between 41 m^3/s and 63 m^3/s is considered 'Band A' water, which currently is fully allocated, albeit not all used. The results presented in this study should give an indication of the reliability of the water resource, by providing probabilities of low flow events, including severities of deficits and durations. Thirty-five m^3/s was chosen as it represent the true extreme of the low value region. Seventy m^3/s was chosen in the second part of the study as it gave an indication of return periods of deficits and durations in the flow region describing smaller freshes in the Waimakariri River. Studies that have previously employed runs theory for the purpose of low flow frequency analysis set thresholds at similar levels (i.e. 90th and 95th flow percentile) (Madsen & Rosbjerg, 1995; Zelenhasic & Salvai, 1987) or higher at the mean annual flow and at 75 % of the mean annual flow (Clausen & Pearson, 1995). Other studies have explored several options, including 70th percentile of the FDC or a monthly varying threshold (Sung & Chung, 2014). In either case, it is advisable that the threshold is chosen to reflect the objective of the study and the flow regime of the study river.

The PDS of low flows, durations and deficits of threshold levels 40 m^3/s, 60 m^3/s and 70 m^3/s displayed no trend or serial correlation. However, all three series resulting from the 35 m^3/s threshold were non-stationary. Figure 5.15 provided clear graphical evidence of the non-stationarity. In addition, the Pettitt change point test identified the events which marked the changes in mean and/or variance of the series. For the PDS of low flows, this point was identified at t' = 11 (March 1973), for the series of deficits and durations it was t' = 35 (May 2001).

The presence of the change point for the series of low flows is likely a reflection of the severe drought of 1971, in which a low flow of 21.26 m^3/s was recorded, an equivalent of a 100- to 500-year return flow. For the duration and deficit series it is interesting to note that the majority of low flow events (42 events) occurred after 2001, i.e. within the last 15 years. The average number of events below the threshold (35 m^3/s) was 2.8 from 2001-2015, whereas the average number of events below the threshold for 1967-2000 was only 0.8. While the average volume of deficits and duration of low flows has thus decreased relative to the 35 m^3/s threshold, the average number of events below the threshold has markedly increased. It is also interesting to note that the period pre-break point (1967-2000) had 68 % of years without low flow events below the 35 m^3/s threshold, while the period post-break point only recorded 40 % of years without low flow events below the threshold. This trend is likely the reflection of changes in catchment conditions that affect the hydrological response of the river, such as land use change and intensified consumptive water use, especially during the summer and late summer seasons[30]. The influence of large scale climatic changes is not a likely contributor to this trend, as no break-point was identified in the low flow series below 40 m^3/s and below higher thresholds. However, Waimakariri Irrigation Ltd. became fully operational in 2003 (consented abstractions 10.5 m^3/s in 'Band A'). The analysis of such trends and break-points were not within the scope of this research, however, could prove fruitful for further investigation in the future.

[30] 86 % of low flow events below 35 m^3/s occur from February to May

The series of durations and deficits resulting from the 35 m^3/s threshold were censored, so that only events from 2001 onwards were used for low flow frequency modelling purposes. A similar procedure was previously considered in the literature (Liu et al., 2014) for non-stationary modelling of low flows.

Distribution selection

Low flow frequency analyses using the truncation approach have limited application in the literature, and are even rarer in the New Zealand context. Clausen and Pearson (1995) explored runs theory for the drought analysis of 44 catchments in New Zealand. However, only the most severe deficit below the threshold of each year (annual maximum) was used in the frequency analysis to estimate the 100-year annual maximum drought severity. Some of the distributions explored were the Generalised Logistic, GEV, LN, P3, GP and LN3 distributions. The LN3 distribution was the recommended distribution for Canterbury rivers. The use of only the most severe deficit of each year would thus constitute AMS sampling of deficits.

International examples of runs theory in combination with PDS sampling include Zelenhasic and Salvai (1987), Madsen and Rosbjerg (1995), and Tallaksen et al. (1997). Zelenhasic and Salvai (1987) assumed a GP distribution for modelling the magnitude of deficits and durations below the threshold, analogous to flood frequency modelling. Later studies agreed with the good fit of the GP distribution. The choice of the best fitting distribution for low flow values, deficits and durations was guided by the same principles as applied in the flood frequency analysis: the goodness of fit dictated the choice. For the series of low flow values, the GP distribution was the most consistently well-fitting distribution, agreeing with the literature. The GEV and P3 distributions could equally be used to model low flow estimates, as statistical testing confirmed the good fit. Equally, the EV1 distribution was permissible. However, as discussed earlier, the EV1 distribution tends to underestimate low flows and may return negative values, as the distribution is not restricted at the lower tail.

Deficit and duration series are best modelled with GP, LP3, P3 and GEV distributions. The EV1 distribution was rejected by both, chi-squared and KS testing for the series of deficits and durations. These results were expected, as the shape of the empirical distribution resembles that of the series of flood events; few occurrences at the upper tail of the distribution and the majority of occurrences at the lower tail. Three-parameter distributions, in general, offer a more consistent fit to the empirical data (NERC, 1975).

Low flow quantile estimates

As was the case with the estimates of the 7dMALF series, the three preferred distributions (GP, P3 and GEV) returned similar results. The 35 Q2 series only includes events post break-point (i.e. after March 1973) and returned estimates similar to those produced by the 7dMALF series for estimates of 10 years and higher. The 35 Q2 10-year estimates were between 27.47 m^3/s (GP) and 27.80 m^3/s (GEV) compared to 28.04 m^3/s (GEV) to 28.24 m^3/s (GP) for the 7dMALF series. This similarity was expected, as a large proportion of 7dMALF values equally fell below 35 m^3/s. The 40 m^3/s series estimated design flows in similar ranges. However, the larger the return period, the larger was the difference between estimates, between thresholds and also between distributions, analogous to results from the flood frequency analysis. Estimates from the 60 m^3/s threshold were significantly smaller when using the GEV and P3 distributions. The GP distribution, which was the most consistently well-fitting distribution statistically, returned estimates similar to those found by the 35 Q2 series and the 40 m^3/s series (10-year return estimate is 27 m^3/s and 100-year return estimate is 23.55 m^3/s).

As the magnitude of the durations and deficits below thresholds are a direct function of the threshold level itself, no direct comparison between thresholds can be made. As was the case with the flood frequency results, the EV1 distribution (statistically not a good fit) gave the lowest estimate for durations and deficits for all series. Equally, the GEV and LP3 distributions returned the highest estimates among the four well-fitting distributions. Ten-year design estimates for durations ranged between 663 h (40 m^3/s threshold, GEV) and 1418 h (60 m^3/s threshold, LP3). Ten-year deficits ranged between 20,315 x 10^3 m^3 (40 m^3/s threshold, GEV) and 138,560 x 10^3 m^3 (60 m^3/s threshold, LP3). To facilitate a meaningful comparison between the effect of deficit and duration as a function of the threshold, design estimates of magnitudes were also modelled. Yevjevich (1967) first introduced the idea of magnitudes as a ratio between the duration and deficits of low flow events in a PDS. In his work, magnitude was calculated with absolute numbers, i.e. relatively small values of duration in hours were divided by relatively large deficits in m^3. In this study, dimensionless deficits and durations were obtained by dividing the values by their respective means. However, instead of using the ratio of duration/deficit, the two values were added. This ensured that magnitudes of events were appropriately reflected. A significant event (i.e. large deficit and long duration) could otherwise be obscured by dividing a large number by a large number and thus arriving at a ratio close to 1. Other events with, for example, long durations but small relative deficits could thus be ranked higher. It is however, acknowledged that adding the two values gives equal weight to both dimensions. This formula could be improved upon by assigning lesser weight, for example, to the duration and more weight to the deficit, depending on the application and the desired outcome. As the magnitude is a function of the threshold level, it is not surprising that the 60 m^3/s series estimated higher magnitudes than the 50 m^3/s series. Both magnitude estimates can be used, however, to give an indication of the return period of significant events relative to past events that have occurred.

9.2.3 Implications for water management

Low flow frequency analysis can be a useful tool in addition to flow duration curves for water resource management and reliability assessments of water resources for out-of-river uses. It can effectively inform about the probability of events and their return periods, which in turn a FDC cannot deliver. Thus, one could deduce that a 50-day duration low flow below a threshold of 60 m^3/s has a return period of 10 years. Such a statement is not specific to the time of year, and it certainly doesn't predict the occurrence of such an event. However, knowing that such an event occurs with a 10 % chance each year is a useful guide for water users with consents to abstract water from the Waimakariri River within 'Band B' (above 63 m^3/s). Comparatively, a 38 day duration low flow event has a 20 % chance of occurrence each year. Equally, this knowledge proves useful for environmental scientists who need to set allocation limits and minimum water levels for in-stream uses. Other uses could include an assessment of low flow conditions for the formulation of in-stream environmental guidelines and thresholds. Snelder and Hughey (2005), for example, suggested that low flow events with durations of 20 days or longer (> 480 h) should occur no more than once every two years to restrict periphyton blooms.

As with flood frequency analyses, the application of a distribution model should be guided by its goodness of fit to the empirical data and the study area. While indicators, such as the 10-year return period of the 7dMALF, are often used, the analysis of events in regards to deficits and durations under thresholds can add additional dimensions to water resource planning, especially when the threshold is chosen to reflect desirable yield of the catchment. In the case of low flows, it is often not clear which dimension plays a larger role in ecosystem processes. Is a long low flow period with a small deficit more severe than a short duration low flow with a large deficit? A combination of both indicators in the form of the magnitude of flow events, as is presented in this study, gives an overall indication of the magnitude of a low flow event. Currently, flow duration curves are the basis for many decision making processes, such as the allocation of abstraction licensing and minimum flow in rivers for stream ecology. The subsequent section of the discussion will focus on the second part of the study, which aimed to show the application of low flow frequency analysis as a tool for environmental decision making.

9.3 The master variable 'flow'

Functioning freshwater systems are a product of natural hydrological variability, characterised by seasonal high and low flows, and infrequent floods and droughts (Richter et al., 2003). In order to evaluate the potentially detrimental effects of anthropogenic pressures on freshwater systems, direct pressures, in the form of abstractions from the river, were more closely evaluated in a three step process, as proposed by Richter et al. (2003). Thus the continuing discussion is organised in three parts: (9.3.1) an assessment of the instream environmental flow requirements based on the rapid systematic literature review, (9.3.2) an assessment of the anthropogenic pressures on the flow regime based on modelling of abstractions, and (9.3.3) a discussion of the incompatibilities between them.

9.3.1 Assessing In-stream environmental flow requirements

It was previously established (Chapter 6) that for enabling and safeguarding multiple river values, an understanding of the river system's water flow requirements is necessary, as the flow regime acts as the ‘master variable’ regulating hydrogeomorphic and ecological processes. The interacting nature of these components complicates the establishment of quantifiable guidelines governing the use and management of water resources within environmental and ecological limits. A good assessment of in-stream guidelines needs to include provisions for the maintenance of hydrogeomorphic and ecological processes, functions of the flow regime within the spatial and temporal context in which they operate.

The rapid systematic literature review was aimed at identifying aspects of the flow regime that are ecologically and geomorphologically relevant for the functioning of the river system. The majority of the published and unpublished reports, and studies used for the review were based on New Zealand data, and often reflected studies in braided rivers, or even the Waimakariri River itself. It is evident from the criteria extracted (Table 8.1) that there is a greater emphasis on average to high-flow events within the literature. Rolls, Leigh, and Sheldon (2012) found a similar trend and noted that effects from human-induced alteration of the low flow regime are in fact poorly understood. Studies addressing rates of change relative to antecedent conditions have emerged only within the last 20 years or less (e.g. NIWA, 2008). However, hydrological variability is crucial for the integrity of a functioning lotic system (Poff et al., 1997), which is visible from the organisation of Table 8.1. The three nodes surrounding the ‘master variable’ flow in Figure 6.2, each respond differently to flow ranges and variability of flow in the river and therefore, the linkages between hydrology, geomorphology and ecology to predict responses, are complex.

Flows belonging to the range of extreme flood flows (> 10 year flood event) are primarily habitat resetting, affecting periphyton communities, in-stream vegetation, invertebrates and geomorphology. Ecological responses to such flows are generally catastrophic, but recovery is relatively quick (Fowler, 2004). For braided river morphology, extreme flood flows have the potential to cause avulsions of the active river bed, as has been documented for the Waimakariri River in the past (Reinfelds, 1995). These flows are also necessary for the removal of large quantities of bedload and suspended sediment and established woody vegetation, which could otherwise significantly alter floodplain and channel morphology through what has been termed biogeomorphical processes (Corenblit et al., 2007).

Flows in the ranges of flood pulses (900 m^3/s - 1600 m^3/s) on the other hand, are habitat quality maintaining flows. Flood pulses in the range of the mean annual flood equally prevent vegetation encroachment, periphyton nuisance growth and sediment build-up. While nest survival of river-bed breeding birds is also negatively associated with such flows, these birds have established adaptations that cue adequate timing of nesting based on flow in the river. Birds therefore tend to nest after high flows, when the probability of occurrence of a subsequent flood of this size is comparatively low (Hughey, 1985a). Periphyton is significantly negatively affected by floods in the range of the mean annual flood; however, such flows are not the determining factor in periphyton removal as these flood events occur too infrequently.

Freshes (130 $m^3/s < Q <$ mean annual flood), on the other hand, are considered to be the primary control over periphyton accrual (NIWA, 2008). Freshes are the principal aquatic habitat quantity controlling factor. Larger freshes (> FRE_3) form channels in braided rivers and maintain gravel islands by removing substantial amounts of smaller vegetation. However, freshes larger than FRE_3 flows are also substantial for the removal of short filamentous algae and diatoms (NIWA, 2008) and thus remove the food source of invertebrate grazers, such as *Deleatidium* spp., in turn a main food source for riverbed nesting birds (Pierce, 1979). Freshes in the dimensions of the FRE_3 occur more often, up to 20+ times per year. These are the flows necessary to flush long filamentous algae and sediment from the bed, preventing substantial build-up. FRE_3 freshes are also considered important for mammalian predator control in braided rivers, as flows in this range prevent predators from accessing bird nests, despite some predators' swimming abilities (Duncan et al., 2008; Veale et al., 2012).

Average flow conditions, i.e. flows between Q_{90} and Q_{50}, are habitat quantity ensuring. Flows in the range of the median flow ensure the largest number of flowing channels in the Waimakariri River, while also maintaining the largest number of gravel islands (Duncan et al., 2008). Any reduction affecting the Q_{50} flow is correlated with a reduction in channel depth, width, velocity, channel number and gravel island number. However, the reduction in channel width is comparatively larger than the reduction in channel depth. In braided rivers, a reduction in the width to depth ratio can reduce braiding intensity and thus change river morphology substantially, according to classical stability analyses (Kleinhans, 2010). The optimal depth and width preference for *Deleatidium* spp., a main food source for nesting birds, also correlates with median flow conditions (Jowett & Richardson, 1990). A reduction from the Q_{50} to the Q_{90} flow, on the other hand, significantly increased the WUA for long filamentous algae, less palatable to mayfly larvae in general (incl. *Deleatidium* spp.).

While low flow conditions are also a natural characteristic of river processes, long periods of sustained low flow are generally associated with detrimental effects to river ecology and morphology of braided rivers. Such conditions favour the proliferation of nuisance periphyton and the accrual of fine sediment with a subsequent effect on periphyton accrual, and possibly bed armouring. Vegetation encroachment, predator access to islands containing bird nests and subsequent nest mortality are some of the studied effects (Hughey, 1985b, 2012; O'Donnell & Hoare, 2011; Rebergen et al., 1998). Bank stabilisation through vegetation encroachment and bed erosion in the main channel are associated with decreased braiding intensity and lateral migration of the river, which has previously enabled morphological change from braided to single thread rivers (Piégay et al., 2009).

It is evident from this short summary of research in New Zealand regarding in-stream flow requirements that the variability of flow conditions is crucial for a wide variety of ecological and physical processes within the river system. Despite this understanding, the proposed *National Environmental Standard for Ecological Flows and Water Levels* (MfE, 1998) and the *Draft Guidelines for the Selection of Methods to Determine Ecological Flows and Water Levels* (MfE, 2008) still list hydrological methods based on the MALF as the minimum determinant for flow values. In practice, rivers of importance, such as the Waimakariri, are often assessed based on habitat preference methods or to the least, hydraulic rating methods. Additional assessments ensured that the minimum flow of the Waimakariri was set higher than the proposed minimum, which is determined at 80 % of the MALF (min. 32 m^3/s). However, water allocations in 'Band A' alone currently exceeds the minimum standard, which is a maximum of 50 % of the MALF (max. 20 m^3/s). Arthington et al. (2006) and Poff and Zimmerman (2010), among others, suggest that rules for environmental flows need to be established based on process-based relationships between ecosystem structure and flow regime, adaptive management processes and best available hydro-ecological knowledge. Any other approach risks to undermine the temporal and spatial hierarchy of the flow regime and will introduce an unnecessary threat to ecological and hydrogeomorphic processes by altering the duration, frequency and timing of flow events.

9.3.2 Identification of the anthropogenic influences on flow

The broad aim when identifying the anthropogenic influences on river flow is to identify ecological and physical thresholds and to investigate if modelling of this relationship is possible in some way. For the generally applied 'rule of thumb' hydrological methods for setting minimum flow values, this includes an assessment of the influence on hydrological extremes. The consented abstractions for the CPWES from the Waimakariri River were subtracted from the entire hourly flow record, spanning from 1967 to 2015 inclusive, in order to model such post-abstraction conditions. For the purpose of this study, a 100 % abstraction rate was assumed whenever conditions allowed them. No provisions have been made for differences between summer and winter irrigation requirements. Previous assessments have taken account of irrigation demand, e.g. the assessment of Kingett Mitchell Ltd. (2006) used modelled irrigation demand data from 1967-2001, supplied by *Aqualinc,* to assess the effect of a 40 m^3/s water take from the Waimakariri River. A subsequent report (Golder Associates, 2007) assumed the granted consent with water takes of up to 25 m^3/s. The 100 % take all year around assumed in this study therefore represents a 'worst case' scenario. However, subsequent studies could include a more detailed assessment of irrigation demand based on precipitation and soil moisture modelling, as has been done for the Assessment of Environmental Effects supplied by Kingett Mitchell Ltd. on behalf of CPWL in combination with methods described in a study by Srinivasan and Duncan (2011) to assess the reliability of river-based irrigation schemes.

A comparison between pre-abstraction (PRA) and post-abstraction (PA) conditions thus slightly differs from what has previously been found by the reports mentioned above. The results indicate that the mean flow PA is 11 % smaller (reduction from 119 m^3/s to 106 m^3/s), whereas the median flow would undergo a reduction of 14 % from 87 m^3/s to 75 m^3/s. These results are consistent with the 1st report (Kingett Mitchell Ltd., 2006), which assumed a 40 m^3/s total abstraction (median reduced from 90 m^3/s to 77 m^3/s). The 2nd report (Golder Associates, 2007), assuming a 25 m^3/s maximum consented abstraction, reports a larger proportional reduction from a median of 75 m^3/s to 63 m^3/s (16 %). The results from this study indicate that the median summer flows (here from November to March inclusive) are reduced by a similar percentage as the overall flows (from 112 m^3/s to 100 m^3/s). Submitted reports (Golder Associates, 2007; Kingett Mitchell Ltd., 2006) indicate a much larger reduction in median summer flows (> 30 % reduction). This could be attributed to the inclusion of an additional month (summer season is interpreted as November to April inclusive), or a result of the shorter period included in the analysis (1967-2001). The reduction in mean and median flow is greatest during bird nesting seasons (September to January inclusive). Mean flow is reduced from 146 m^3/s to 130 m^3/s (11 % reduction), and median flow is reduced from 109 m^3/s to 90 m^3/s (17 % reduction). The higher mean and median flow of the bird nesting season compared to summer flows are likely a reflection of hydrological/ agricultural droughts which are more pronounced later in the summer season (Feb/March)(Srinivasan & Duncan, 2011). The bird nesting season, however, only extends until late January for the purpose of this analysis. Results are not comparable with previous reports, as no assessment of this kind has been published to date. It can also be seen that the median take of water is 11 m^3/s throughout the year, 10.35 m^3/s during the summer season and the largest, 18.92 m^3/s, during the bird breeding season. The small median take in comparison with the consented 25 m^3/s is a result of flows falling below the statutory minimum flow of 41 m^3/s or flows within 'Band A', and can be interpreted as the reliability of the scheme.

Contrary to evidence supplied by Golder Associates (2007) and Kingett Mitchell Ltd. (2006), the number of times the flow exceeds three times the median flow is significantly smaller post-abstraction. FRE_3 value were extracted from the average daily flow measurements from 1967 to 2015. FRE_3 values pre-abstraction were on average 15 per year, ranging from four (2005) to 34 (1995). This is expected for the Waimakariri River, displaying a non-glacial mountain type flow regime (Ministry for the Environment, 1998), and the calculated values agree with previous studies and reports (Clausen & Biggs, 1997; Duncan & Woods, 2013; Kingett Mitchell Ltd., 2006). Post-abstraction, FRE_3 values drop to an average of ten exceedances per year, ranging from two (1969, 2005) to 21 (1995). This is an overall reduction of 44 % in the number of events exceeding three times the median flow, compared to the reported < 14 % (Golder Associates, 2007). Overall, in 31 years of the 49 years on record, the reduction in the number of events is between 20 % and 50 %. In 16 years, the reduction was greater than 50 %, e.g. in 2001 FRE_3 events dropped to only three post-abstraction, compared to twelve pre-abstraction. The discrepancies between the results presented here and previous findings could have multiple explanations. The FRE_3 value in this report was calculated at 261 m^3/s, compared to 284 m^3/s (Golder Associates, 2007) and 270 m^3/s (Kingett Mitchell Ltd., 2006). It is likely that the significant reduction of events above the FRE_3 value in this study are due to pre-abstraction flows in the ranges of 261 m^3/s and 285 m^3/s, as these would permit full abstractions of 25 m^3/s and thus, in the case of the upper values, would reduce the PA river flow to just below the FRE_3 value. It is to be assumed that river flows of 260

m^3/s and slightly below would equally have the same ecological and physical effects as flows just above 261 m^3/s. Furthermore, it is expected that the extraction of FRE_3 events based on hourly discharge data would yield different results. Daily data likely 'hides' FRE_3 events by averaging hourly data, thus appearing less 'flashy'. The lack of definition for the application of the FRE_3 value as an ecological indicator has previously been discussed by Booker (2013). However, the daily time resolution was adopted in this study, as it has been extensively used in other published reports and studies. Booker (2013) also showed that using a 5-day filter period can have a significant influence on the calculated values. No filter was applied for the calculation of FRE_3 events in this study, however, it is likely that in some instances FRE_3 values follow each other closely (< five days inter-event time) and thus would not exert a cumulative effect on ecology as more than five days are required for benthos recovery.

The above analysis of descriptive river discharge statistics (including Figure 8.4) shows that flows between 63 m^3/s and 111 m^3/s will be most affected by the abstractions, with a proportional increase of abstraction rates with flow. These flows fall into the category of habitat quantity forming flows (Table 8.1). However, the magnitude of small freshes and even larger freshes, as shown with the reduction of FRE_3 events, are also significantly affected by abstractions. Such flows belong to the ranges of habitat quality maintaining flows (Table 8.1). Minimum flows, as measured by the instantaneous 1dMALF and the 7dMALF are not significantly affected, due to water take restrictions below the 'Band B' minimum unmodified flow.

In order to assess the probabilities of low flow conditions (low flow value, deficit, duration and magnitude), the frequency of pre- and post- abstraction low flow conditions was analysed, using a 70 m^3/s threshold for the selection of low flow events. The choice of the 70 m^3/s threshold was motivated by the 'Band B' minimum unmodified flow (63 m^3/s). Setting the threshold at 70 m^3/s thus allows the inclusion of events within 'Band B', without approaching the median flow, which by definition in this study is not considered to belong to the low flow domain. A first comparison of low flow events reveals that the total number of events below the threshold increased from 604 to 672 events (+10 %). Mean and median durations also increased by 15 hours and 11 hours, respectively. However, mean and median deficit volumes decrease with abstractions. This is due to the larger proportion of events falling just below the 70 m^3/s thresholds, skewing the mean and median. The largest deficit, however, increases from 328,409 x 10^3 m^3 to 329,224 x 10^3 m^3 post-abstraction. It is thus evident that while the presence of low flow events is a natural part of the flow regime of the river, abstractions artificially create, extend and exacerbate low flow conditions. This also has an effect on the frequency estimates of low flow events.

The LP3 and GP distributions were chosen for frequency analyses of low flow conditions. Durations and deficits were modelled using the LP3 distribution, magnitude and low flow were modelled using the GP distribution. The results produced by the frequency analysis of low flow events show no significant change post-abstraction, due to the stringent restrictions limiting water takes when flow falls below 41

m^3/s. Therefore, the post-abstraction series is within 1 m^3/s of the pre-abstraction series for design estimates, and also closely corresponds to the estimates produced by the 7dMALF series (Chapter 5). However, design estimates are notable for management purposes. A 3-year event, for example (PA), already returns an estimate that is smaller than the statutory minimum, with 32.35 m^3/s. This can also be compared to the 7dMALF series low flow results (32.67 m^3/s). Ten-year events are estimated to reach a low of 26.85 m^3/s (28.20 m^3/s using the 7dMALF series). The 100-year event approaches the low flow values recorded during the record drought during 1971, with an estimate of 26.65 m^3/s (27.50 m^3/s for 7dMALF series). Extrapolation beyond the 100-year event only slightly decreases values, due to the shape of the GP distribution, e.g. the 250-year event estimate is 23.10 m^3/s. While such values give an indication of low flow values and the likelihood of their occurrence, other criteria, such as durations, provide much better information for decision making. The 10-year low flow event with 26.85 m^3/s may be severe for river ecology by introducing a bottleneck event. However, this value does not provide information about the duration of the low flow event, which can exert much greater influence on ecology. A 5-day duration under 10-year low flow conditions is not likely to provide opportunities for vegetation encroachment or periphyton accrual.

Durations of low flow events markedly increase with abstractions, which also implies that the length of time between freshes and flood events increases. The 3-year event duration pre-abstraction is 798 h (33.25 days) long, while post-abstraction the same event would last for 888 h (37 days). Ten-year events are seven days longer post-abstraction, lasting 1878 hours (78.25 days). The longest drought on record lasted 2298 h (95.625 days) below 70 m^3/s, corresponding to a 100-year event (PA). The duration of low flows has significant implications for water management, as has been previously mentioned in 9.3.1 and will be further discussed later. However, for the estimation of durations, the threshold selection has to be guided by the ecological and geophysical processes under consideration. Seventy m^3/s is a relevant threshold for the Waimakariri River for this study, as it is within the optimal bird nesting flow ranges, is a flow significant for habitat quantity maintenance, and contains optimal flow ranges for periphyton accrual. It also coincides with flows that support near optimum habitat conditions of *Deleatidium* spp.

The selection of the deficit threshold should be guided by equal consideration. In this study, the 70 m^3/s was also adopted. However, previous studies suggested the use of monthly varying thresholds, reflecting desired yield. Due to time constraints, such modelling was not undertaken, but could reveal interesting results if further explored. Threshold selection for deficit analysis could reflect irrigation demand with varying seasons, or could equally use ecologically significant flow ranges as minimum conditions (e.g. minimum flow required during bird nesting season to support food production and reduce mammalian predation, or minimum flow required to flush vegetation in spring/summer growth periods). Such an analysis would therefore shed light on the reliability of minimum desired water yield and the frequency of 'system failure'. The deficit results in this study are based on a uniform threshold of 70 m^3/s. Such a threshold contains a small part of 'Band B' allocated water from 63 m^3/s to 70 m^3/s. While the analysis of deficits does not reveal the timing of deficits (i.e. drought conditions during the relevant irrigation season), it can give an indication of the overall deficit to be expected. Srinivasan and Duncan (2011) have previously indicated that the most significant drought conditions in the Waimakariri are generally observed later in the summer season (January-April), based on an analysis of data from 1972 to 2008, which is consistent with the data selected for this analysis. Deficits post-abstraction with an annual return period of three years are 88,800 x 10^3 m^3. The 10-year return period estimate is 329,700 x 10^3 m^3, and the 100-year return period estimate is 1,648,880 x 10^3 m^3. Pre-abstraction estimates are 77,139 m^3, 244,750 x 10^3 m^3, 965,700 x 10^3 m^3, respectively. The post-abstraction 10-year return period estimate corresponds well to the deficit resulting from the low flow during 1971 (328,410 x 10^3 m^3). Such a deficit would reflect average flow of 31 m^3/s for 95 days. The significance of the deficit volume is thus only relevant in combination with the duration of the event. Shorter durations indicate a much lower flow below the minimum flow on average. However, the disadvantage of analysing the frequency of duration and deficit separately is that one cannot combine, for example, the 100-year estimate for low flow durations and the 100-year estimate of deficits to make conclusions about such an event, meaning that a 100-year duration does not necessarily accumulate a deficit that is expected from a 100-year deficit event and vice versa. The magnitude, as has previously been presented in Chapter 5, combines both values by adding the dimensionless deficit and duration to then produce a frequency curve, and can be more readily utilised for decision making.

9.3.3 Incompatibilities between flow requirements and altered flow regime

The synthesis to follow outlines the likely responses of biota and hydrogeomorphic processes to the altered flow regime due to consented abstractions, based on the results in Chapter 8 and the previous discussion about flow regime changes in the Waimakariri River. The modification of the average flow conditions (Q_{90} to Q_{50}), and of freshes in the ranges from > 130 m^3/s and < mean annual flood are most pronounced. The effect on larger flood pulses (> 900 m^3/s) and extreme floods is not significant and thus system processes relying on such high energy events remain unchanged. However, clear discrepancies between the modification of the river regime due to anthropogenic changes and the flow regime necessary for the maintenance of the river system are evident.

Periphyton

Periphyton responds well to prolonged low flow conditions, and associated low flow velocity. Studies have stressed the importance of rate of change in discharge relative to preceding base flow conditions, and the importance of freshes in the removal of periphyton nuisance growth (Biggs et al., 1990; Biggs & Close, 1989; NIWA, 2008). Rivers with more than 13 FRE_3 events have previously been shown to support only very low levels of periphyton biomass (Biggs, 2000). Thus, a reduction in FRE_3 values from an average 15 to 10 per year, as modelled in this study, can benefit biomass accrual, especially in side channels. Modelled reductions in FRE_3 can be as large as 75 % in some years. Long filamentous algae blooms (*Ulothric zonata*), although very rare in the dynamic Waimakariri River under current conditions, have been reported in the past during summer and autumn low flows (Biggs & Close, 1989). The median flow during summer months pre-abstraction is 83 m^3/s. This is approximately the flow rate at which 80 % of the river bed has a velocity greater than optimal long filamentous algae growth conditions (NIWA, 2008). The median summer discharge supports a velocity of circa 0.62 m/s and a mean channel depth of ca. 0.41 m, as determined by two dimensional habitat modelling along a 3 km reach at Crossbank (Golder Kingett Mitchell, 2007). An increase from minimum flow conditions (7dMALF) to 83 m^3/s or higher also imposes shear stress and drag forces strong enough to scour 80 % of long filamentous algae present in the river (Duncan, 2008; NIWA, 2008). The velocity encountered at a flow of 85 m^3/s is also the subsidy-stress response level of *Didymosphenia geminata* (Bray et al., 2016). However, post-abstraction, the median summer discharge is reduced to 72 m^3/s, providing better accrual conditions. The reduction in the FRE_3 events also indicates a longer duration of accrual periods in the river. Thirty-four days between significant flushing events is already sufficient for substantial periphyton growth, covering between 20 % and 100 % of minor braids in the Waimakariri River (Biggs, 1990; Biggs et al., 2008; Duncan & Bind, 2009). Ideally, the 34 day duration low flow events is not surpassed on a yearly basis. Indeed, frequency analysis of the duration of flow events below 70 m^3/s suggests that a 33.2 day duration is a one in three year event pre-abstraction. Post-abstraction, the one in three year event surpasses the 34 day target (modelled 37 days), suggesting an increased probability of occurrence for nuisance algal blooms. Modelling of pre-abstraction events below 60 m^3/s show that the 34 day duration low flow is a one in four year event. The 60 m^3/s series was not modelled to reflect post-abstraction conditions; however, it is expected, based on a comparison with the other data, that an increase in duration is likely. Frequency analysis of events below 40 m^3/s suggests that a 34 day duration below the statutory minimum is a one in ten year event. This value is not expected to change post-abstractions, as restrictions limit the take of water below the 'Band B' minimum. It is evident that the probability of occurrence of such an extreme duration is lower with decreasing thresholds. However, even flows dropping below 60 m^3/s significantly increases habitat availability for long filamentous algae (Golder Associates, 2007). In combination with lower discharge, especially in the summer months, the reduction in the number of FRE_3 events could significantly impact on the amount of biomass flushed from the channel after prolonged low flow conditions, as only flows of > 140 m^3/s, and some results even report > 190 m^3/s, are substantial enough to cause 66 % surface flushing and 44 % deep flushing of the river bed (NIWA, 2008). In contrast, diatoms and short filamentous algae have much higher flow preferences. Significant flushing only occurs at flows in the range of 280 m^3/s and above. The change in flow conditions can have profound effects on periphyton community composition, and thus indirectly on invertebrate community composition. At flows below 45 m^3/s, aquatic snail and worm communities, dominated by chironomids, replace mayfly and caddis fly populations, which primarily feed on diatoms and short filamentous algae (Gray et al., 2006). Habitats with frequent bed resetting flushes (FRE_3), such as the Waimakariri River, tend to support 'clean water' invertebrate taxa, such as mayflies and caddis flies, which are adapted to recover quickly form such disturbances (Jowett, 1997). Invertebrate fauna abundance displays a curvilinear relationship with values of FRE_3, peaking at about 10 - 15 events per year. Coincidentally, the optimal habitat conditions for *Deleatidium* mayflies is provided at depths of 0.48 ± 0.18 m and at velocities of 0.75 m/s (Jowett & Richardson, 1990), which correlates to a flow of > 90 m^3/s in main channels, reflecting the median flow during bird breeding seasons pre-abstraction (109 m^3/s).

Avifauna

The provision of the right food supply in sufficient quantities is an important prerequisite for nesting and fledging success of river bed breeding birds during the bird breeding season (September to January). Macroinvertebrates, but especially larvae of the order Ephemeroptera (mayflies), are key prey for the endangered and threatened birds nesting and foraging in the Waimakariri River (Pierce, 1979). Changes to the community composition and densities of invertebrate food sources can have a detrimental effect on the survival of these birds. A potential decline in the availability of Ephemeroptera during the nesting season appears to be especially damaging populations with evolved high energy-cost feeding strategies (O'Donnell, 2004). The increase in invertebrate productivity measured by the increase of the relative chironomid larvae abundance thus does not offset the overall reduction in mayfly larvae, contrary to some suggestions (Kingett Mitchell Ltd., 2006). Additionally, Kelly et al. (2015) suggest that an assessment of the flow requirements of invertebrates based on hydraulic habitat modelling using WUA tends to underestimate the effect of flow reductions on invertebrates. Flood disturbance during early spring, before the onset of nesting, ensures an abundant food supply (of the right kind) for nesting and returning migratory species (Fowler, 2004; O'Donnell, 2004). Freshes and above median flow during this period also have an effect on the selection of nesting sites for gravel-nesting birds, such as the wrybill and banded dotterel (Hughey, 1985a). The selection of higher nesting grounds relative to moving water helps prevent or reduce nest mortality due to flooding later in the nesting season. Especially wrybill, banded dotterels and pied stilts suffer from high nest mortality due to flooding (Hughey, 1985a). A reduction in freshes (FRE_3), as shown in Chapter 8, but especially immediately at the onset of the nesting season, could have a detrimental impact on the survival of these birds, as higher magnitude floods remove lower than average lying nests.

The effect of decreased discharge largely depends on the hydraulic characteristics of the river. For braided rivers, which have a high width to depth ration, a decrease in discharge typically equates to a large reduction in wetted width (Gordon et al., 2004). A 100 % increase from the minimum flow in the Waimakariri River results in a mean wetted channel width increase from 230 m to 320 m (Kingett

Mitchell Ltd., 2006). Such changes also affect the number of smaller braids, which have been shown to contribute to > 60 % of weighted usable area for wrybill and banded dotterel (Hughey, 1985a). Reduced flow may not significantly impact the main channel in terms of depth and velocity (Kingett Mitchell Ltd., 2006). Minor channels, on the other hand, are the first microhabitat to significantly undergo changes (by drying up).

The preferred flow ranges for nesting birds lies between 60 m^3/s - 90 m^3/s (Duncan et al., 2008). This range provides the largest available number of flowing channels and islands for nesting, and also the best availability of food sources. At flows below 120 m^3/s, the total number of braids starts to decline (O'Donnell, 2000). Modelling of a 3 km reach at Crossbank has shown that at flows of 86 m^3/s, 8 flowing channels are present. At flows of 70 m^3/s, 11 gravel islands with areas greater than 2 ha are available for breeding and nesting. This number reduces to 10 islands at flows below 60 m^3/s and to only 8 islands at flows below 40 m^3/s; with a concurrent decrease in flowing channels (Duncan et al., 2008; Hughey, 2008; Kelly et al., 2015). For highly territorial birds, such as the wrybill, the available habitat is a critical determinant for population numbers. At the same time, the availability of the preferred habitat (minor channels) reduces the required home range size (Hughey, 1998). Thus a flow of 60 m^3/s should be the absolute minimum regulated flow during nesting season to provide adequate food supply and habitat availability.

The mean flow during the river bird nesting season is larger than the overall mean flow of the Waimakariri River (146 m^3/s compared to 112 m^3/s, respectively). This suggests that the majority of the higher flow ranges occur during the bird breeding season, which is confirmed with the inspection of Figure 8.7. As a result, the largest modelled water take (mean take 16.12 m^3/s, median take 18.92 m^3/s) occurs during this environmentally sensitive season, as water flows naturally fall below the 'Band B' minimum flow less frequently. The abstraction of water during the bird nesting season increased the number of events below the 70 m^3/s threshold from 176 to 238, a clear discrepancy in relation to the required optimum flow. Deficits are generally larger and low flow periods are longer. During a selected August events (02/08/1973) deficits (below 70 m^3/s) equate to an average flow of 40.67 m^3/s post-abstraction, rather than an averaged 41.13 m^3/s pre-abstraction over 38 days. For a selected December event (04/12/2005), flows averaged 48.70 m^3/s over 26.5 days vs. 47.6 m^3/s after abstractions.

The magnitude of events (i.e. duration + deficit) has a large influence on the establishment of vegetation, which is also linked to nesting site selection and survival. Wrybill in the Raikaia have been shown to only tolerate vegetation cover of round 5 % at nesting sites (Hughey, 1985a). Encroaching vegetation appears to be correlated with a choice of lower lying nesting sites within the riverbed, where even small flood flows can lead to nest failure (Hughey, 1985a). In the Ashburton River, endangered species population numbers decreased during the 1980s as a result of vegetation growth (O'Donnell, 2000). The presence of vegetation has also been linked to increased mammalian predation (Maloney et al., 1997; Rebergen et al., 1998). Studies on the braided Ohau and Tekapo rivers indicated that nest mortality was significantly higher if sites were within 50 m of encroaching vegetation and thus potential predator den sites (Rebergen et al., 1998). Flows between 60 m^3/s and 90 m^3/s produce the largest number of flowing braids, unfavourable conditions for vegetation encroachment and a barrier to mammalian predation. Minor channels only supporting flows of 1 m^3/s or less already inhibit access to nesting sites by predators (Boffa Miskell and Urtica Consulting, 2007), despite the swimming abilities of some predator species (Veale et al., 2012). Flows in the optimal ranges ensure that minor braids remain active channels during nesting season.

The removal of vegetation from the braided river bed is dependent on flushing flows (of the magnitude of FRE_3 and higher) and flood flows of the extreme ranges (larger than mean annual flood). Extreme flood flows have been shown to remove up to 70 % of lupins (Caruso, Edmondson, et al., 2013). However, well-established woody tree species, such as willows, can remain even after 50-year flood events, as has been reported in the Ahuriri River (Caruso, Edmondson, et al., 2013). Large woody debris and smaller established vegetation on islands are removed during events in the ranges of the mean annual flood (Hughey, 2012).

The probability of occurrence of floods in these ranges does not change as a result of abstractions from the river, as modelled in this thesis. However, the time between smaller freshes, and thus the time available for the establishment of problem species is significantly increased. A flow of 85 m^3/s markedly increases the wetted width of the channel when compared to the minimum flow (230 m vs. 320 m, respectively), (i) reducing the available habitat for invasive vegetation on side bars and on gravel islands, and (ii) removing already established vegetation. The modelled duration of low flows after abstraction is higher than pre-abstractions. The increased duration of flows and their return periods was previously mentioned in relation to periphyton accrual in the Waimakariri River. Concurrently, the timing of flushing flows is equally important. Flushing flows during spring and summer time are especially important for the removal of not yet fully established, immature plants. International examples have shown that high summer flows and floods inhibit vegetation establishment (Piégay et al., 2009). Unfortunately, summer flows in this study are further decreased due to abstractions, from median of 83 m^3/s to a median of 72 m^3/s. Once plants have rooted and are established, higher shear stress and drag forces, as provided by larger flood flows, are required to remove unwanted establishment and growth (Caruso, Pithie, et al., 2013).

The modification of flow in the Waimakariri River is argued to be minor or less than minor (Kingett Mitchell Ltd., 2006). In a braided river, floodplain inundation during flood events is determined by the stochastic probability of flooding at any given time, the location of main channels and the past history of the river. The scientific literature, however, has yet to establish the cumulative effect of low flows on vegetation encroachment in braided rivers over multiple growing seasons.

Fluvial geomorphology

Establishing relationships between morphodynamic processes in riverine systems is a difficult task, as each river system reacts differently to physical change. However, albeit difficult, this task is necessary for the maintenance of key hydrogeomorphic processes and braided rivers (Reid & Brierley, 2015). Human-induced changes to river morphology may take several years or decades to fully unfold, yet most management strategies and goals do not reflect this large time scale. River morphology is rather assumed to be in a steady-state equilibrium (Hicks et al., 2007). Developing a quantifiable relationship between human-induced pressures to morphodynamic processes and river response is therefore desirable in order to predict changes to the morphology of the river landscape and the physical instream/ terrestrial habitat. For braided rivers, such as the Waimakariri, knowledge of the causes of braiding are vital for the anticipation of changes due to human-induced pressures.

The direct influence of flow regime on riparian vegetation is evident from the earlier mentioned examples. However, Corenblit et al. (2007), describes the equally important influence of vegetation on fluvial geomorphology, in a concept termed 'fluvial biogeomorphic succession'. Laboratory studies of braided rivers indicate that morphological change into a single-thread river is possible if gravel turn-over from flooding is too infrequent to clear established vegetation (Hicks et al., 2007; Paola, 2011). The hydro-power influenced Waitaki River is a New Zealand example showing the effects of reduced discharge, altered sediment transport capacity and vegetation establishment on the river's planform. Narrowing channels, reduced braiding intensity and positional stability of the main braids are only few of the consequences (Hicks et al., 2007).

Locally, vegetation exerts an influence on hydraulic conditions, by influencing flow velocity, depth and thus erosional processes. Vegetation encourages suspended sediment deposition, which induces aggradation of material on vegetated areas. Larger vegetation, such as willow trees, encourages bed-material deposition and accumulation, stabilising and establishing banks of greater permanency by imposing stronger cohesion to the bed and banks (Coulthard, 2005; MfE, 1998). Resulting effects include channel narrowing at the reach scale, and the elevation of islands due to concentrated flows in channels. These processes over time, together with channel incision, decouple the channel from its floodplain, with severe ecosystem consequences (Piégay et al., 2009). As of now, sub-annual flood flows are sufficient to clear vegetation in the Waimakariri River (Hicks et al., 2007). Concurrently, changes in rivers may take decades to fully manifest after alterations to the river regime have taken place.

The modelled abstractions do not change the return period or magnitude of the mean annual flood or floods with even greater magnitudes. Therefore, the braided planform of the river is maintained. However, the continuum of meandering-braiding morphology is a reflection of the ratio of time required for vegetation to establish and the average time between scour events. The abstractions during spring/summer seasons especially favour vegetation establishment to a higher degree (Paola, 2011). Prolonged periods between large freshes enable the development of strongly rooted, stabilising vegetation cover, more resistant to larger flood events. In the Waitaki River, even the 200-year flood event only removes around 25 % of vegetation from river bed islands (Hicks et al., 2007). The magnitudes of low flow events (i.e. the duration and deficit combined) are more severe post-abstraction, and return periods of higher magnitude events are shorter. Thus the modelled abstractions can potentially induce morphological changes to the river, by altering the vegetation establishment time/flood frequency ratio.

At the smaller scale, the modification of the flow regime has an influence on the sediment transport capacity of the river, with consequences for river ecology, reflecting the multitude of feedback relationships as conceptualised in Figure 6.2. Streams with high sediment supply and unstable beds, such as the Waimakariri River, are able to mobilise sediment at smaller discharges (Dietrich et al., 1989). Freshes larger than FRE_3 events are thus the main sediment flushing flow, by which up to 80 % of the bed is flushed from fine sediment (Hay & Kitson, 2013; NIWA, 2008), often during a 1-day time span (Hicks et al., 2007). Flows below 200 m^3/s only initiate 0 - 50 % sediment transport, depending on the model employed in the calculations (Nicholas, 2000). Habersack (2001) equally shows that the movement time of radio-tracked particles is significantly decreased at flows around 200 m^3/s in the Waimakariri, with only 2.7 % movement time recorded, compared to 25 % movement time at flows around 1000 m^3/s. Sediment flushing from both the bed and the water column have been shown to increase habitat quality, even though in the short term the effects on the zoobenthos can be detrimental due to physical abrasion and increased drift by invertebrates (Biggs et al., 2005; Fowler, 2004). The accumulation of fine sediment between gravel reduces the habitat available to aquatic macroinvertebrates, thus limiting the food supply of river nesting birds (Biggs et al., 1990). Fine sediment has also been shown to reduce the aquatic grazer activity, and benefit the accumulation of nuisance algae (Biggs et al., 2008). Armouring of gravel beds by filling interstitial spaces, for example, reduces the adverse effects of flood disturbance on periphyton (Biggs & Smith, 2002) and as a result can play an equally large role in the control of nuisance long filamentous algae accumulation during low flow conditions. It is not expected that abstraction as modelled in this thesis will have an influence on the large flushing flood pulses (~1000 m^3/s) required to clear the riverbed of sediment. However, the duration of low flow events, especially during summer, coupled with the smaller number of FRE_3 events, is expected to have an effect on sediment accumulation, and indirectly on periphyton accrual and invertebrate abundance/ community composition.

9.4 Summary

The results obtained from this research were discussed in detail in this chapter, reflecting and incorporating previous research findings from the literature. The following chapter summarises the key findings of the research in light of the research limitations, and also highlights some potential future research avenues.

Chapter 10

Conclusions

The main findings of the thesis are summarised in this final chapter. Findings are organised according to the main objectives that have been set out in Chapter 1. The chapter also comments on the significance of the findings for stakeholders in a broader context. This chapter is concluded with a brief discussion on the limitations of the research and some ideas for future research possibilities.

10.1 Main findings

10.1.1 Objective 1: Flood frequency analysis of the Waimakariri River

Correct return period estimations for flood events underpin engineering projects in rivers and are of immense importance for rivers with high associated flooding risk due to the proximity of urban areas. The flood frequency estimates calculated in this thesis were the first estimates obtained using a partial duration series (PDS) approach to sampling the continuous discharge record of the Waimakariri River. Previous studies have solely relied on annual maximum series (AMS) for the estimation of design events, partly based on well-developed theoretical grounds, but also due to a lack of prescriptive guidelines governing the use of the partial duration series sampling approach.

This study used a standardised approach to the selection of events above multiple thresholds, as proposed by NERC (1975), which achieved the statistical independence of events necessary for frequency analysis. The application of a recently developed formula to convert partial duration series return periods into the annual domain (Mohssen, 2009) made previous premises about the Poissonian distribution of occurrences above the threshold redundant, simplifying the use of the PDS. Instead, the optimum threshold was determined to lie between 650 m^3/s and 800 m^3/s, with $3.306 < \lambda < 4.469$. Based on these results, it can be thus recommended that threshold selection should reflect a linear relationship between the mean of exceedances above the threshold and the chosen threshold level.

Contrary to previous recommendations for the choice of the distribution to model the magnitude of flood events above the threshold and to model AMS, the Gumbel (EV1) distribution was excluded as a fitting distribution with confidence, based on graphical and statistical testing. The findings in this study therefore directly contradict previous flood frequency analyses of the Waimakariri River, which used the Gumbel (EV1) distribution for annual maxima. The results in this study support the use of four other distributions (out of eight tested) over the EV1 distribution, namely the LP3, GP, GEV and P3 distributions, for both AMS and PDS analyses of the Waimakariri River. ANOVA testing of PDS design estimates indicated that the EV1 distribution significantly underestimates design events for larger return periods (> 25 years), as has been previously indicated in the reviewed literature (ECan, 2011a).

Previous AMS flood design estimates of the Waimakariri River have used AMS data from 1930 onwards. However, continuous data for the extraction of PDS are only available from 1967 onwards. A comparison of PDS with the historical AMS showed that the PDS using a 750 m^3/s or 800 m^3/s threshold level produced design estimates of comparable magnitude. This indicates that these PDS based on 49 years of data (rather than 86 years of data) are able to 'make up' for the missing continuous data record. Furthermore, the PDS series provided a smoother and thus better fit to the empirical data than the extracted AMS series. This is also confirmed by statistical testing using the chi-squared, Kolmogorov-Smirnov and Filliben Correlation Coefficient test and thus suggests that the application of PDS analysis should be preferred over AMS analysis for at-site frequency analysis of flood events in the Waimakariri River.

10.1.2 Objective 2: Low flow frequency analysis of the Waimakariri River

Low flow frequency analyses are seldom undertaken, as the assessment of low flows for the purpose of water resource management is primarily based on flow duration curves. The few examples of low flow frequency analyses that are present in the literature are based on series of annual *n*-day mean low flows, and thus are equivalent to AMS modelling. However, low flow events are described by multiple environmentally significant dimensions, such as duration and total deficit. The frequency analysis of the 7dMALF, on the other hand, only gives an indication of a single dimension characteristic of low flows. This study thus analysed low flow frequencies based on not only commonly used annual 7dMALF series, but also based on 'runs theory' as developed by Yevjevich (1967). Series of low flow values, low flow durations and deficit volumes, derived from various threshold levels, were analysed.

As is the case with the application of the PDS for flood frequency analysis, the use of runs theory for low flow analysis is plagued with uncertainty. The threshold choice was based on the water allocation regime of the Waimakariri River, reflecting a common agreement that the threshold choice for drought analysis should be guided by the study area and desired application. The series of events resulting from a 35 m^3/s threshold choice displayed serial dependence and trend, indicating non-stationarity. While the trend in series of low flows can be explained by the influence of a significant event during 1971, the trend shift in the series of durations and deficit volumes occurs much later in the record, in 2001. Post-2001, a larger proportion of low flow events below the 35 m^3/s threshold are recorded, perhaps indicating significant changes in catchment conditions. Series resulting from higher thresholds displayed no such trend.

In agreement with the distribution choice for flood frequency modelling, the EV1 distribution was statistically and graphically a bad fit for all series of all threshold levels. For the series of low flow events, one has to be specifically cautious using the EV1 distribution, as it is not bounded at the lower end and thus can produce negative values for flows. Low flow series were best modelled with the GP, P3 and GEV distributions. The GP distribution was the most consistently well-fitting distribution statistically, as previously found in similar international studies (e.g. Madsen & Rosbjerg, 1995). Design low flow estimates did not significantly differ as a result of threshold levels. The GEV and LP3 distributions were statistically a good fit for modelling the durations and deficit volumes of low flows. Direct comparison of the influence of threshold level choice is not possible, as both the duration and deficit volume are a function of the chosen threshold level. Instead, a formula derived from Yevjevich (1967) was introduced to describe the magnitude of events by combining the duration and deficit of low flows below the threshold.

10.1.3 Objective 3: Implications for water resource and river management

Flood frequency analysis

The estimation of flood return periods is a fundamental part of floodplain management, especially in close proximity to urban environments, such as Christchurch. The first Waimakariri flood frequency estimation was in fact motivated by a significant flooding event in 1957 and the resulting need to build better flood protection structures. With increasing years on record and higher computational capacities, the estimation of flood frequencies has become more accurate, less time consuming and overall more conceivable. However, it is not advisable to estimate flood return periods that extend more than twice the available record. While PDS can significantly extend the available record, as was demonstrated in this study, the estimation of a 10,000 year event produced for the design of the secondary stop bank for the Waimakariri River, is speculation at best. The assessment of the reliability of the existing stop bank system has to be questioned in light of the results presented in this thesis. Selected thresholds and resulting PDS fit the four chosen distributions better than the annual maximum series, as indicated by goodness of fit. Additionally, the EV1 distribution performed better with the PDS than with the AMS, signalling the superior application of the PDS for flood frequency analysis. The comparison of PDS with varying thresholds further confirmed that the EV1 distribution significantly underestimates design discharge for return periods > 5 years and showed that four alternative distributions describe the empirical data better, despite differences of up to 42 % between estimates. The results thus indicate that the preferred method of flood frequency analysis in the region, which is primarily based on annual maximum series and extreme value distributions, has to be revised.

Low flow frequency analysis

An analysis of low flow events using the runs theory can be a valuable addition to the assessment of flow duration curves and thus a useful tool for water resource management. Low flow frequency analysis can especially drive the assessment of the reliability of water resources by informing about the probability and return periods of significant events. Previous studies have largely relied on an analysis of the annual 7dMALF, which was historically primarily applied to water quality issues and also limits the insight to only one dimension of the low flow phenomenon. The benefit of low flow frequency analyses is especially highlighted when threshold levels are combined with relevant applications. Low flow frequency analyses in combination with runs theory can thus be used to quantify water deficits and durations relative to the desired water yield for agricultural and/or ecological applications.

10.1.4 Objectives 4-5: Ecological thresholds related to the flow regime

In light of the granted (not yet active) consents for water abstractions from the Waimakariri River as part of the CPWES development, the discrepancies between altered flow regime and in-stream environmental flow requirements were assessed in a three step process. The first step in the process required the assessment of the current state of knowledge on the effects of flow reductions on selected environmental variables. The variables under question in study were periphyton communities, riverbed nesting avifauna and braided river geomorphology. The rapid systematic literature review revealed that the variability in flow magnitudes is a key determinant for maintaining the river process system.

Larger flood events with return periods of 10 years or more are floodplain altering events, which have the potential of imposing a persistent non-equilibrium state in the sense of bed load and sediment regime for a number of years. Such events are generally catastrophic for river ecology but are equally important for resetting ecological systems. Mean annual flood events or higher maintain the morphological planform by permitting the transport of bed load downstream and are considered channel forming events. These flows are of significant importance to braided rivers as they clear encroaching vegetation, counteracting negative ecological effects and possible fluvial biogeomorphic feedback. Freshes, in the ranges of FRE_3 flows are significant for habitat quality maintenance by removing nuisance periphyton on a regular basis, ensuring the 'right' invertebrate community composition and removing introduced flora and fauna. Flows between Q_{90} and Q_{70} primarily ensure adequate habitat quantity (in terms of WUA). While low flows form part of the natural river regime, prolonged natural and artificially created low flows are generally associated with ecological and morphological braided river degradation.

It has thus been established that changes to the variability of flow magnitudes and their frequency of occurrence is strongly associated with changes to biotic and abiotic conditions along the river continuum, impacting on a multitude of spatial and temporal scales.

10.1.5 Objectives 6-7: Frequency estimates for pre- and post-abstraction series

The second step in the process was aimed at removing the consented abstractions from the available streamflow record (1967-2015) and modelling low flow frequency under post-abstraction conditions. An initial assessment of the descriptive statistics revealed significant changes in the habitat quantity and habitat quality maintaining flows. Flood flows of the ranges of the mean annual flood and higher are not significantly impacted by the modelled abstractions. Results at this stage were also compared with technical reports prepared for the consents hearing process of CPWL. While the reduction of mean and median flows is consistent with technical reports, the results in this study indicate a significant effect on FRE_3 flows, which were described as marginal by the assessed technical reports.

The reduction in FRE_3 values as modelled in this study, has a potentially significant impact on the accrual of nuisance periphyton by increasing the duration between flushing flows and by reducing the magnitude of flushing flows. The reduction in events is also likely to impact directly on sediment accumulation and increased vegetation encroachment, and indirectly on riverbed nesting birds. Low flow frequency analyses of the pre-abstraction and post-abstraction series revealed a higher number of events below the chosen threshold post-abstraction, a longer duration of events post-abstraction and higher deficit volume estimates. 7dMALF values are not changed, as restrictions limit water take below the 'Band B' minimum flow.

The altered flow regime, as reflected by changes to low flow duration frequencies, is likely to have a significant effect on periphyton accrual rates, especially through the reduction of FRE_3 events and the increased duration of low flow events. The altered community composition of periphyton can also significantly impact on the community composition of aquatic invertebrates, which are the main food source of braided riverbed nesting birds. Optimum conditions for mayfly larvae are consistent with velocities that also support diatoms and short filamentous algae, while scouring long filamentous nuisance algae.

Riverbed nesting avifauna face increased pressures from lowered habitat quantity and quality due to abstractions. As deficit volumes and their magnitude in relation to return period increase, available minor channels dry up, invertebrate food sources diminish, and vegetation encroachment increases. The timing of flushing flows vs. low flows is considered to be particularly important for nesting and fledging success, reflecting the availability in food sources, habitat quantity and influencing behavioural patterns (e.g. location and timing of nesting). Vegetation establishment on braided river islands and banks is also closely associated with mammalian predation and river planform changes.

The reciprocal feedback between vegetation establishment and changes to the river planform/ regime could be witnessed as a result of water abstractions. While larger flood event with higher return periods are not changed, prolonged low flows with larger deficit volumes, which were modelled in this study based on abstracted flows, enable the ongoing establishment of woody vegetation, resistant to smaller flushing flows, once established. The establishment of vegetation on banks and rivers has previously been associated with the decrease in braiding intensity, channel permanence and channel incision, driving the decoupling of the river from the floodplain. The reduction in flow in the mid-ranges, which was also concluded in this research, reduces the frequency of the primary sediment mobilising flows, resulting in increased sediment accumulation with negative consequences for periphyton invertebrates, and thus riverbed nesting birds.

10.2 Contributions to the sum of knowledge

10.2.1 Theoretical contributions

The research in this thesis revisited an ongoing discussion about the use of partial duration series or annual maximum series for the frequency analysis of extreme events. The choice of the AMS is mainly guided by the application of extreme value theory (Fisher & Tippett, 1928 as cited in Stedinger et al., 1993) and has been applied by many scholars since. AMS sampling from the stream record is also straightforward in its application as the premise of independence is more readily achieved, and therefore lends itself to a standardisation process of frequency analyses. However, PDS sampling is argued to offer a valid alternative and extends the sample used for frequency analysis by including events above/ below a predetermined threshold. This research showed that independence of events in the PDS can easily be resolved with the application of predefined guidelines (e.g. NERC, 1975), and that the PDS offers a considerable extension of the number of samples used in the analysis. This is particularly relevant for streams with short records to date. The selection of an appropriate threshold level was extensively discussed and methods for such selection, as suggested by Lang et al. (1999) were further endorsed. Despite the lack of 37 years of data available for the AMS but not the PDS, the PDS sampling method achieved comparable quantile estimates and additionally offered a smoother fit between the empirical data and theoretical distributions.

The theoretical distributions considered in this research were guided by previous research in New Zealand and world-wide. Despite assertions in the literature that the distributions of the extreme value family should provide a good fit for flood peaks, the results presented here suggest that alternative distributions are a better choice. Therefore, the results are in agreement with what has previously been suggested by, for example, NERC (1975) and it is concluded that the choice of distribution should be guided by information of available data and not theoretical considerations.

The low flow extreme of a river's discharge regime is traditionally reviewed by the use of flow duration curves, which offer little insight to the recurrence interval of extremes. Despite the pioneering works of Yevjevich (1967), runs theory is seldom applied for the analysis of low flows. The few regularly cited examples of runs theory primarily employ annual maximum sampling of the low flow dimensions under study, and very few studies (e.g. Zelenhasic & Salvai, 1987) approached the problem with PDS sampling. This research first compared PDS of minimum flows with annual minima and also produced estimates for durations and low flow deficits. As was the case with flood estimates, the PDS of minima provided a smoother fit between the empirical data and theoretical distributions.

Yevjevich (1967) also introduced the analysis of magnitudes of low flow events, given by the absolute ratio between the deficit and the duration. However, it was argued in this thesis, that the calculation of this ratio with absolute numbers obscures the real effect of a low flow magnitude. A new formula was suggested in this research to use the sums of dimensionless durations and deficits. The calculation of the sum eliminates the 'masking' of the effect of magnitudes, which frequently occurs when dividing deficits and magnitudes. With Yevjevich's (1967) ratio calculation, relatively smaller low flow events may in fact appear more significant by arriving at larger ratios. The calculation of magnitudes as a sum of dimensionless durations and deficits also enables selectively adding weight to one or the other dimension of low flow events, depending on the research question.

10.2.2 Applied contributions

It is expected that the approach and flood frequency results presented in this thesis are relevant to organisations tasked with flood risk management in the study area and elsewhere. The example from the study area clearly showed that a choice of methods on theoretical grounds rather than based on information of available data leads to the underestimation of the flooding hazard. In this particular case, PDS sampling offered a better description of the flood peaks' distribution due to the larger number of events chosen from the stream record. It is expected that a similar comparative study elsewhere would reach the same conclusion. Adequate floodplain management, facing increased pressures due to climate induced changes to the river regime and discharge extremes, should reflect a more conservative approach, by testing and producing a range of design estimates and selecting the most statistically fitting distribution to model exceedances. This argues for a move away from a standardised approach to flood frequency analysis as is practised in the region and advocates adopting the most suitable method for the catchment in question. This is priority task, as Christchurch faces a significant flooding risk from the Waimakariri River.

The discussion on setting minimum flows in rivers has been ongoing for a number of decades. The assessment of minimum flow requirements has progressed from hydrological rules, such as the Tennant Method, to a more process-based understanding, incorporating a range of flows and species' specific needs (IFIM). The challenge when setting minimum environmental flows in rivers is to determine the degree of change acceptable before ecosystem values degrade noticeably. This research explored a method for analysing the reliability of meeting in-stream values under changed flow conditions. Identifying the necessary components of the flow regime for optimal ecosystem functioning through the systematic literature review enables the formulation of nominal objectives regarding return periods of critical durations and/ or deficits. One could thus impose maximum annual low flow durations/deficits under a threshold of 60 m^3/s during the bird breeding season to ensure habitat quality and quantity for avifauna or alternatively maximum annual low flow durations/deficits under a threshold of 80 m^3/s during summer seasons to minimise periphyton accrual, etc. depending on the desired environmental objective. This analysis would allow an assessment of the capacity and reliability of the use of out water resource under current or altered conditions and thus could guide the allocation of water with methods complementing those already in place, reflecting the holistic approach to water management, as advocated by Arthington et al. (1992).

10.3 Research limitations and future research directions

10.3.1 Research limitations

The results presented in this thesis should be interpreted in light of their limitations. First and foremost, the design estimates of floods and low flows are specific to the study catchments and the data used in the analysis. All steps were taken to ensure data series used for frequency analyses were independent and they were assumed to be identically distribution, '*iid*'. Data supplied by the regional council was assumed to be correct. In reality, the margin of error, especially for high flows, can be large as flood peaks are rarely measured directly and instead estimated based on stage-discharge relationships. Other sources of error and uncertainty include the subjective selection of thresholds, the parameter estimation method and the choice of probability distribution, which all dictate the obtained quantile estimate results.

For the analysis of low flow frequencies of altered stream conditions, simplified assumptions were made about abstraction rates from the stream record. No account was taken of actual water demand for irrigation and details of consent conditions which impose stricter water take guidelines after long periods of low flows. Furthermore, a single threshold was chosen to compare pre- and post-abstraction series due to time constraints. In reality, a moving threshold reflecting varied in-stream water needs dependent on seasons and life stages would provide more insight. The time limit also meant that only a limited number of published and unpublished reports and studies were included for the assessment of in-stream environmental needs. Once again, the search for literature forming the basis of the rapid systematic review was mostly limited geographically to New Zealand and the Canterbury region.

10.3.2 Future research directions

Throughout the thesis, multiple references have been made to possible research projects which could improve the methods and results presented here. Recommendations for future research are thus summarised below.

- The frequency analysis using PDS sampling only included data from 1967 onwards, as continuous measurements are not available in digital form. However, records are available in the form of chart recorders, which could significantly extend the record used for analyses. As of now, only peak flows between 1930 and 1966 are gauged. Future analyses could potentially make use of available stage-discharge relationships to gauge significant peaks above or below selected thresholds, which date back to 1930.
- Independence of selected peaks above the threshold was ensured by applying criteria as proposed by NERC (1979). Future investigations could make use of rainfall data to ascertain whether closely following peaks are in fact independent, thus reducing the imposed inter-event time between

subsequent high flows. Rainfall data could also be used to distinguish flood events resulting from storms of different wind directions.

- Bayesian analysis could be further explored to make use of valuable paleo-flood data and historical accounts of flooding from the grey literature. The use of such historical accounts could thus reduce the uncertainty in flood quantile estimates.
- Trend testing of the low flow series below 35 m^3/s revealed significant trend and serial correlation. A detailed investigation into the sources of such trend within the Waimakariri River could be considered. The detected break point at which a change in trend occurs does not correlate with known changes in IPO trends.
- Threshold selection for low flow frequency analysis for the Waimakariri catchment could reflect seasonal changes in desired water yield, either based on irrigation demand or based on environmental requirements. This could be a useful tool for reliability assessment of rivers as process systems. An assessment of reliability in relation to irrigation demand has previously been explored by Srinivasan and Duncan (2011).
- A similar assessment could be done based on detailed knowledge about environmental flow requirements. This could also lead to the formulation of nominal guidelines in terms of return periods of low flow events, including durations and deficit volumes below desired thresholds.

10.4 Summary of research

This research was motivated by current issues in water management in the Waimakariri catchment reflected by the unpredictability of extreme events and uncertainty of resource capacity for out-of stream users and in-stream ecology. This final section of the thesis is aimed at summarising the key findings and recommendation that emerged from the research.

- Results produced in this thesis strongly indicate that PDS modelling of flood events provides a better statistical fit to the historical empirical data than AMS modelling. At the same time, the PDS design estimates using only 49 years of data approach estimates of AMS with 86 years of data.
- Four distributions (i.e. P3, LP3, GEV and GP) are statistically a good fit for modelling flood frequencies of the Waimakariri River. The currently employed EV1 distribution for flood hazard planning was statistically not a good fit and significantly underestimates design events for return periods > 5 years. Based on the fitting distributions, the existing stop bank system provides protection for flood events with a return period of ca. 100 years (GEV and LP3 distribution estimates). GP, LP3 and GEV estimates for the 500-year event markedly surpass the 4700 m^3/s capacity of the stop bank system, which is estimated to withstand a 500-year flood event.

- Despite the greater effort required for PDS modelling due to the standardisation of threshold and event selection, it is recommended that PDS modelling be explored further for floodplain management due to the smoother fit of statistical characteristics of empirical data to theoretical distributions, and thus overall produces better flood design estimation. The results in this study indicate that this is a pressing issue for the Waimakariri River floodplain, which extends to the outer bounds of Christchurch.
- The example of pre- and post-abstraction low flow frequencies showed that minimum flow requirements based solely on hydrological indicators can compromise ecological health and fluviomorphological processes in the river. Ecologically informed low flow frequency analysis based on runs theory is recommended as an additional tool for assessing the reliability and capacity of water resources to meet in-stream and out-of stream needs in the present and future.

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Appendix A

Quality of hydrometric data and data uncertainty

A.1 History of hydrometry in New Zealand

As with any other product or service, maintaining quality standards in hydrometry is a necessity. One common definition of quality is '*fitness for purpose'* (Hudson, McMillan, & Pearson, 1999, p. 826), meaning that the data collected are for their intended purpose, communicated effectively to users, and meet quality requirements for the predefined purpose. Otherwise, data quality can be understood as a metric for deviations of measurements created by a standards process that is assumed to be reliable. In New Zealand, the question of managing quality standards in hydrometric network operations was first approached in the scientific literature by McKerchar (1986) and subsequently Mosley and McKerchar (1989).

Systematic data collection of New Zealand's major rivers was first considered relevant after 1906, for assessing the viability of hydroelectric operations, irrigation and river control schemes (Pearson, 1998). Monitoring of major rivers, other than for their user-potential, began in the 1930s (Pearson, 1998), such as the case for the Waimakariri River. Low flow conditions were of particular interest, due to the need for irrigation in the growing season, but rating curves were poorly supported by irregular gaugings at the time. McKerchar (1986) states that this was due to personnel entering military service during World War II. Therefore, much of the data collected in the earlier days are of little use to current data analyses (K. Osten, personal communication, 11/09/2015; McKerchar, 1986). Later data collection, namely from 1941 onwards, mainly focussed on data for flood protection works. A meeting of engineers in 1959 was dictated by the need for a systematic approach to hydrometric data collection in New Zealand, setting the scene for the adoption of UNESCO's policies during the *International Hydrological Decade* from 1965-1974 (Mosley & McKerchar, 1989). However, before 1960, the quality of data obtained from hydrometric measurements solely depended on the know-how of field staff in charge of maintenance of measurement sites, as there was no nation-wide adopted quality assurance program in place (Ware & Lad, 2003). While the deployment of digital recorders in the 1960s greatly reduced uncertainty in discharge measurements, even with the rapid development of new technologies, data quality was still only informally assessed, often by end users of collected data (McKerchar, 1986). It wasn't until 1983 that the National Water and Soil Conservation Authority set up a quality assurance program for all hydrological surveying in the country, ensuring a chain of feedback from the end-user to the surveyor (Mosley & McKerchar, 1989). With the first steps taken towards a nation-wide quality assurance program as described by Mosley and McKerchar (1989, p. 190), the following objectives were set:

(1) To enable estimation of hydrological parameters at any point on any natural water course with the following accuracies:
    - Mean and median discharge to a standard error of ± 20 %
    - Mean annual flood to a standard error of ± 30 %
    - 50-year return period flood to a standard error of ± 30 %
    - Seven-day duration, two-year return period low flows to a standard error of ± 10%

(2) To identify, with 90 % confidence, time varying trends in mean discharge from all catchments greater than 800 km^2 in area

The second objective, especially, required the continuous measurement of large river, such as the Waimakariri. Furthermore, the following standards were introduced (Table A.1):

Table A.1 Standards adopted by the Water Resources Survey, Department of Scientific & Industrial Research. (Source: Mosley & McKerchar, 1993, p. 8.33)

	Water level of river flow sites
(a)	Installed equipment and operating procedures shall be such as to ensure that 95 % of instantaneous measurements are within ± 3 mm or ± 10 mm of the water level above the sensing device, depending on site instrumentation; specifications in clause 7 of ISO Standards ISO 4373-1979 ‘*Measurement of liquid flow in open channels –Water level measuring devices*’ should also be met.
(b)	Instantaneous values shall be available on the Water Resources Archive within a maximum of six months of their being recorded.
(c)	At any one measuring station, there shall be not more than 2 % (approx. 7 days) missing record in a given calendar year, and not more than one calendar year in ten shall have any missing record,
(d)	Field practice shall conform to standards specified in the Water Resources Survey Hydrologists’ Field Manual (Hydrology Centre, 1988).
	Measurement of discharge, and rating curve construction
(a)	Flow gauging shall conform to the appropriate ISO standards as outlines in ISO Handbook 16 (ISO, 1983), and documented in the Water Resources Survey Field Hydrologists’ Manual (Hydrology Centre, 1988); in any case, 95 % of discharges shall be measured to an accuracy of better than ± 8 % of the rated value, and a frequency specified by reference to flood event frequency, bed stability, and historical evidence.
(b)	Stage discharge rating curves shall conform to specification of ISO Standard ISO1100/2-1982 *‘Liquid flow measurement in open channels- Part 2: Determination of the stage-discharge relation’*, clause 7.1: *the curves shall invariably express the stage-discharge relation objectively and shall therefore be tested for absence from bias and goodness of fit in the periods between shifts of control, and for the shifts in control*
(c)	Flow gaugings and revised rating curves shall be available on the Water Resources Archive within a maximum of six months of the date of the gaugings.
(d)	At any one measuring station, 95 % of all flows estimated from a stage record with a rating applied shall be within ± 8 % of the actual values.

The mid-1980s were characterised by major political and economic changes, placing an emphasis on market mechanisms and the deregulation of the economy. The hydrometric sector was not spared and saw the introduction of a ‘user pays’ philosophy to water monitoring and data supply (Pearson, 1998). This development went hand-in-hand with the introduction of the quality assurance program, to ensure that end users received quality data, fit for use. With the establishment of the Crown Research Institutes in 1992, all scientific surface-water hydrological field capabilities were transferred to the National Institute of Water and Atmospheric Research (NIWA). This development also shifted the responsibility of water resource monitoring for all non-scientific purposes to regional councils (Pearson, 1998).

A.2 Streamflow measurement

Streamflow or discharge measurements are either described by instantaneous or continuous techniques (Davie, 2008). The often used velocity-area method is an instantaneous streamflow measurement technique, whereby discharge is expressed as the volume of water per unit of time (i.e. m^3/s). It is calculated as the product of stream velocity and cross-sectional area of a river section and is described by the equation $Q = v_i \, a_i$, where Q = discharge (m^3/s), v_i = velocity (m/s), and a_i = cross-sectional area (m^2). Depending on the roughness of the river channel, the stream is divided into smaller segments to account for the range in velocities across the channel. Velocities are measured using a flow or current meter and as a rule of thumb are measured at a stream depth of 60 % from the surface (Davie, 2008; Gordon et al., 2004; McKerchar, 1986). In the past, current metres have been used either from the side of the riverbank or by dropping a current meter from a boat (Mosley & McKerchar, 1993). Today, flow velocity is measured automatically by means of electronic recordings while driving a boat along the river. However, during significant flood events, such method is not advised and therefore other methods are often used. These include measurements from bridges or by indirectly calculating velocity through hydraulic depth-velocity relationships (Mosley & McKerchar, 1993). The instantaneous method only allows for a single measurement at a specific location. However, for a discharge hydrograph, continuous measurements are necessary.

These measurements can be done in a number of ways, the stage-discharge relationship being the most commonly used method. This involves three steps: (i) recording of water levels, or stage, above the datum level; (ii) producing a relationship between stage and discharge (stage-discharge rating curve); (iii) calculating a record of discharge from the record of stage. A measurement station is referred to as gauging station (Mosley & McKerchar, 1993). For the deduction of the stage-discharge relationship, multiple at-site instantaneous measurement of flow velocity and cross-sectional area of the river are needed to produce a relationship together with the water stage relative to a datum level (Davie, 2008; Reid, Poynter, & Brown Copeland and Co. Ltd, 1982). Analysts have to be careful in interpreting data derived from stage-discharge relationships, as it is the stream stage that is being measured and actual discharge is only inferred information (Davie, 2008).

Every major New Zealand river is underpinned by a stage-discharge curve for the estimation of discharge. In order to guarantee the quality of such curves, especially in highly dynamic braided rivers such as the Waimakariri, remapping of the cross-sectional area is occasionally required (Ware & Lad, 2003). The entire standardised measurement procedure represents a series of approximations. Especially flood flows are inherently less precise than average flow values (Mosley & McKerchar, 1993; Ware & Lad, 2003). It is also standard to assume that flow measurements of NIWA and regional councils are within ± 8 % of the true flow 95 % of the time, as per standards set by the National Hydrological Reference Network (Mosley & McKerchar, 1989). Therefore, the recorded measurement value represents the true flow value at the time, plus some unknown random and systematic error (Ware & Lad, 2003).

A.3 Uncertainty in frequency analysis

Uncertainty by definition is the '*interval within which the value of a measured quantity can be expected to lie with a stated probability*' (McKerchar, 1986, p. 6). Quantifying uncertainty is the underlying mathematical concept of probability (Hofer, 1996). For the frequentist school of thought, the interpretation of probability refers to the proportion of times the event occurs in a long series of independent, identically distributed (iid) trials (Winkler, 1996). The process of flood frequency analysis, involving only iid trials by definition, becomes problematic when samples or trials are insufficient to describe the phenomenon of interest or when used outside the domain of natural uncertainty (Pate-Cornell, 1996). As the true parameters cannot be adequately described, probabilistic (stochastic) models, such as used for frequency analyses, replace unknown driving parameters with statistics derived from the observations (Klemes, 1988). Therefore, frequency analysis includes a series of approximations, inherently containing a number of epistemic uncertainties. The consideration of uncertainty is an important factor when analysing results and deciding their reliability for decision making processes (Merz & Thieken, 2005). This statement holds true for many applications that include quantitative analyses of measured data, but are especially important when risk assessment, as the case with flood risk assessment, is involved. Merz and Thieken (2005) make a distinction between two fundamental classes of uncertainty: natural and epistemic uncertainty. Natural uncertainty stems from variability in the stochastic processes of natural phenomena (Merz & Thieken, 2005; Yen, 2002). It has also often been referred to as objective uncertainty, randomness, aleatory (Hofer, 1996) or type-A uncertainty in the literature. Epistemic uncertainty, on the other hand, is a result of the lack of knowledge about the stochastic processes under study, and is sometimes referred to as subjective uncertainty, specification error or type-B uncertainty in the literature. Yen (2002) and Merz and Thieken (2005) further distinguished between different sub-classes of epistemic uncertainty in their reviews, which are listed in Table A.2. While natural uncertainty is inherently present in all natural phenomena under study (i.e. is a property of the system), epistemic uncertainty arises mainly because of measurement errors or the lack of appropriate technology to quantify all variables driving the natural phenomena. Furthermore, each analyst will add to the uncertainty due to aspecific state of knowledge and skill in measurement and analysis (cf. Yen, 2002: operational uncertainty), and therefore epistemic uncertainty is considered to be reducible in principle (Merz & Thieken, 2005; Winkler, 1996; Yen, 2002).

Table A.2 Uncertainties in water resources analyses (Merz & Thieken, 2005; Yen, 2002)

Source		Explanation	Example
Natural uncertainties	associated with underlying stochastic processes of natural phenomena		assumption of stationarity, homogeneity, independence
Epistemic uncertainty	associated with incomplete knowledge about the system's processes		
	Model uncertainties	representing the inability to model the system's true behaviour	distribution functions: GEV, Gumbel, Lognormal, Weibull, Pearson Type 3
	Model parameter uncertainties	associated with the difficulties of estimating the model's characterising parameters	parameter estimation methods: L-moments, method of moment, method of maximum likelihood
	Data uncertainties	representing a lack of quality assurance for the sample data, including errors in measurement, transcription and recording, data inconsistencies and non-representativeness	sampling uncertainty, annual maxima or partial duration series, length of record, consideration of historical data, errors in stage-discharge rating curves
	Operational uncertainties	including those associated with construction, manufacture, deterioration, maintenance, and other human factors that are not accounted for in the modelling or design procedure	

Natural uncertainty

Sources of natural uncertainty stem from the assumptions on which flood frequency analyses are built. The assumption of stationary in geophysical time series particularly has been argued to have limited validity (Beven, 2015; Klemes, 1988). Hydrological time series are considered stationary if statistical parameters of events fluctuate around a constant value in a statistically constant pattern (Gordon et al., 2004). However, urbanisation, deforestation, afforestation and land-use changes significantly impact on the assumption of stationarity within a watershed (Davie, 2008; Merz & Thieken, 2005; Olsen, Kiang, & Waskom, 2010). If non-stationarity results from clearly observable deterministic functions of time, such as land use changes, the trend or shift can be removed or adjusted with deterministic models (Olsen et al., 2010). However, a question that is often raised deals with the uncertainty of extrapolating to the future based on historical data in case of non-stationary conditions and several methods have been suggested as a solution (see e.g. 'The Workshop on Nonstationarity, Hydrologic Frequency Analysis and Water Management' edited by Olsen et al., 2010).

Another required assumption is that the events under study are generated by the same physical conditions, belong to the same population, and thus are homogeneous (Klemes, 1988). Sources of non-homogeneity include on the larger scale, e.g. instantaneous changes in the physical processes generating hydrological time series due to catastrophic natural hazard events, such as earthquakes (Gordon et al., 2004). On the other hand, non-homogeneity can occur due to changes in the data collection method or the environment in which it is done. This can include changes in the recording location or method, changes in channel configurations, diversion of water, or even a change in data analyst/observer.

For the Waimakariri River dataset dating back to 1930, natural uncertainties can arise from the following factors; land use changes, diversion of water, and changes in channel configuration, all of which are results of settlement in the vicinity of the river and resulting land use. The Waimakariri catchment was dominated by native forest and high-country tussock pre-settlement (Logan, 2008). Even before the European arrival, a large proportion of native bush in the Plains has been removed during the Polynesian settlement period. However, post- European arrival, burning advanced the retreat of native forests in the upper catchment (NCCB, 1986) and by 1880 much of the previously forested area was occupied by grazing activities. These land use changes occurred well before the first attempts of hydrometric data collection in the area and are, apart from occasional historical accounts of high flows (Cowie, 1957), not detectable in the modern datasets. Additionally, forest clearance has an instantaneous effect on catchment water yield, as opposed to afforestation, in which case effects can still be detected decades after planting (Brown et al., 2005). Following the removal of the majority of native forests, the New Zealand Forest Service (NZFS) undertook three main planting operations in the Waimakariri catchment (NCCB, 1986). Craigieburn Forest Park was established as a Douglas Fir and Pine plantation between 1965 and 1971, Eyrewell State Forest was established in 1926, and the planting of Oxford State Forest was initiated in 1965 as a remedy to flooding problems in the area (NCCB, 1986). Paired catchment studies, such as the study by Brown et al. (2005) illustrate that afforestation in catchments induces a significant reduction in water yield until maturity of the forest stand is reached. Depending on climate, forest type and soil conditions, it can take years before the catchment reaches a new equilibrium in water yield. As such, the activities undertaken by the NZFS between 1926 and 1971 could have resulted in changed water yields, reflected in measured stage or discharge.

Since the mid-1800s, protection works have been undertaken to reduce the flooding hazard in the inhabited areas around the lower Waimakariri reaches. Earlier works were limited to the installation of groynes, and it wasn't until the 1920s that the Waimakariri River Improvement Act 1923 enabled the design of extensive stopbanks and diversions along 40 km of the river to facilitate a shorter, straighter course to the sea (Blakely & Mosley, 1987; Reid et al., 1982). The Waimakariri Improvement Scheme 1960 was approved to achieve flood protection up to discharges of 4730 m^3/s with a combination of high levels of channel modifications and channel containment via stopbanks (ECan & Waimakariri

District Council, 2003). The viability of the construction works necessitates the ongoing extraction of gravel and sediment to maintain the channel capacity. Other anthropogenic influences on the river bed occurred between 1960 and 1966 for the construction of the Northern Motorway. At this time, small, arguably non-significant amounts of water were diverted upstream of the OHB in combination with gravel extractions (Ware & Lad, 2003).

Epistemic uncertainty

As described before, epistemic uncertainty results from the incomplete knowledge about the stochastic processes governing the physical processes and potential analysts' errors. Yen (2002) grouped epistemic uncertainties, as listed in Table A.2.

The first set of uncertainties, model uncertainty and model parameter uncertainty, arise due to the process inherent in frequency analyses. The fundamental understanding of probability is that each observed record represents one of infinite equally likely realisations of a stochastic process at play. Thus, a number of different distribution functions can be fitted to the observed data, each giving different extrapolation values. Equally, parameters of the distribution can be estimated with a range of methods, each resulting in different values. As a remedy, tests of goodness of fit are employed to evaluate if the chosen distribution function and parameters represent the sample and give a 'good' probabilistic model, i.e. a model that can reveal more about a physical phenomenon than the observations themselves can (Klemes, 1988). Which distribution function results in a 'good' model, has been widely debated in the literature (cf. Chapter 3).

Data uncertainties and operational uncertainties are perhaps the most reducible in principle. The most significant data uncertainty arises due to measurement errors. As described earlier, discharge is commonly inferred from a rating curve, rather than directly measured. The stage-discharge rating curve relationship depends on a number of, often assumed to be constant, variables. Changing cross-sections as a result of deposition or erosion can introduce errors into the rating curve and thus inferred discharge (Merz & Thieken, 2005). Vegetation encroachment can also induce changed hydraulic conditions (McKerchar, 1986). Incorrect accuracy of instruments, installation and maintenance of the site by staff can all introduce further errors. Additionally, changing the recording method and location can result in inconsistencies (Gordon et al., 2004). Especially high flow data is plagued by inaccuracies, as discharge is seldom measured during peak flows. It is rather a result of the extrapolation of the rating curve, far beyond its measurement range (Clarke, 1999; Di Baldassarre, Laio, & Montanari, 2012; Haque, Rahman, & Haddad, 2014). However, as Gordon et al. (2004) state, assessing streamflow records for errors is a time consuming and often a difficult task and subsequently most published data is accepted as true and accurate for the purpose of further studies.

Appendix B

Complementary results for Chapter 5: Flood frequency analysis

Table B.1 L-moment of the most commonly used distributions (Asquith, 2011; Hosking, 1990).

Distribution	L-moments
Exponential	$L_1 = \xi + \alpha, \quad L_2 = \frac{1}{2}\alpha, \quad \tau_3 = \frac{1}{3}, \quad \tau_4 = \frac{1}{6}$
EV1	$L_1 = \xi + \gamma\alpha, \quad L_2 = \alpha \log 2, \quad \tau_3 = 0.1699, \quad \tau_4 = 0.1504$
Normal	$L_1 = \mu,\ L_2 = \pi^{-1}\sigma,\ \tau_3 = 0,\ \tau_4 = 30\pi^{-1}\ \tan^{-1}\ \sqrt{2-9} = 0.1226$
GP	$L_1 = \xi + \alpha/(1+k),\ L_2 = \alpha/(1+k)(2+k),\ \tau_3 = (1-k)/(3+k),\ \tau_4 = (1-k)(2-k)/(3+k)(4+k)$
GEV	$L_1 = \xi + \frac{\alpha}{k}\{1-\Gamma(1+k)\},\ L_2 = \frac{\alpha(1-2^{-k})\Gamma(1+k)}{k},\ \tau_3 = \frac{2(1-3^{-k})}{1-2^{-k}} - 3,\ \tau_4 = \frac{(1-6.2^{-k}+10.3^{-k}-5.4^{-k})(2-k)}{1-2^{-k}}$
LN	$L_1 = \xi + \exp\left(\mu + \frac{\sigma^2}{2}\right),\ L_2 = \exp\left(\mu + \frac{\sigma^2}{2}\right)\mathrm{erf}\left(\frac{\sigma}{2}\right),\ \tau_3 = 6\pi^{-\frac{1}{2}}\int_0^{\sigma} \mathrm{erf}(x/\sqrt{3)}\ \exp(-x^{2)}\,dx/\mathrm{erf}(\frac{\sigma}{2})$
P3	$L_1 = \xi + \alpha\beta,\ L_2 = \frac{\pi^{-1/2}\beta\Gamma\left(\alpha + \frac{1}{2}\right)}{\Gamma(\alpha)},\ \tau_3 = 6\, I_{\{\frac{1}{3}\}}(\alpha, 2\alpha) - 3$
LP3	P3 fit to the logarithms of the variable

Note: γ is Euler's constant, I_x is the incomplete beta function.

Table B.2 Annual maximum series of discharge measured at Waimakariri OHB. Values for each corresponding year are rounded to the nearest integer and are given in m^3/s.

1967	2030	**1984**	2820	**2001**	855
1968	1080	**1985**	1071	**2002**	975
1969	1134	**1986**	1028	**2003**	716
1970	2495	**1987**	1166	**2004**	1487
1971	1146	**1988**	2288	**2005**	958
1972	1573	**1989**	1175	**2006**	2442
1973	1008	**1990**	1017	**2007**	1382
1974	1123	**1991**	965	**2008**	1173
1975	1771	**1992**	1095	**2009**	1387
1976	1341	**1993**	1135	**2010**	2446
1977	1136	**1994**	2176	**2011**	1367
1978	1353	**1995**	1504	**2012**	1488
1979	2836	**1996**	1173	**2013**	2389
1980	1361	**1997**	1120	**2014**	806
1981	1456	**1998**	1594	**2015**	1408
1982	1429	**1999**	1094		
1983	1657	**2000**	1400		

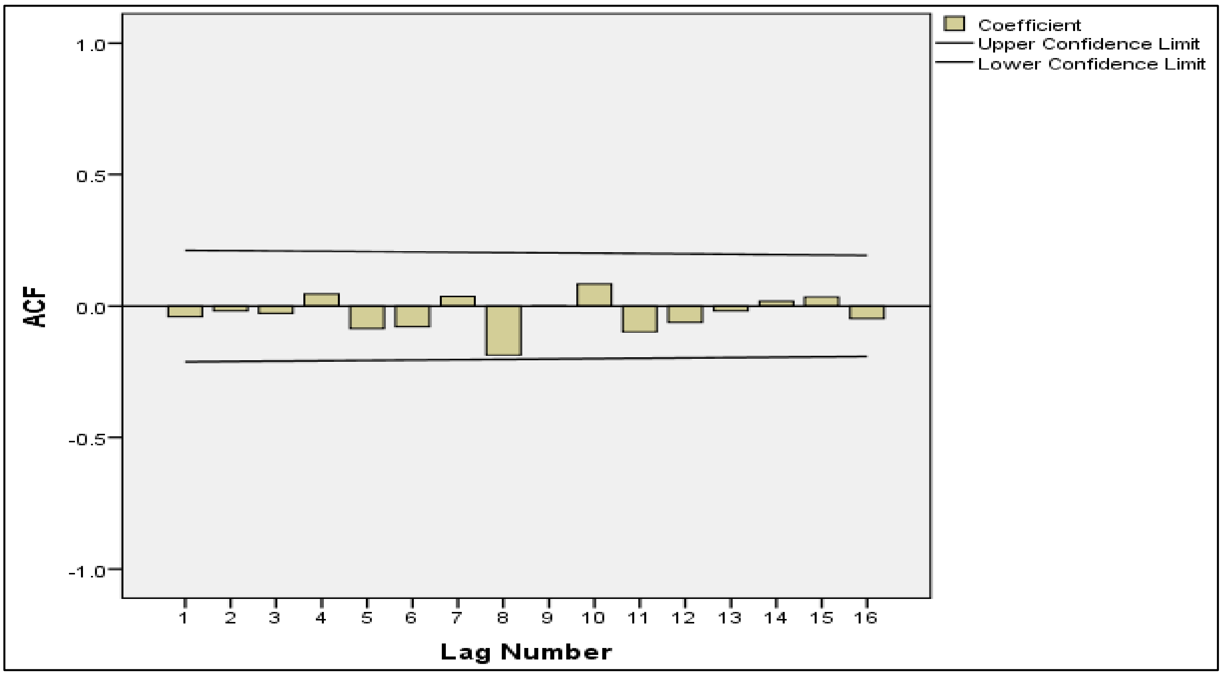
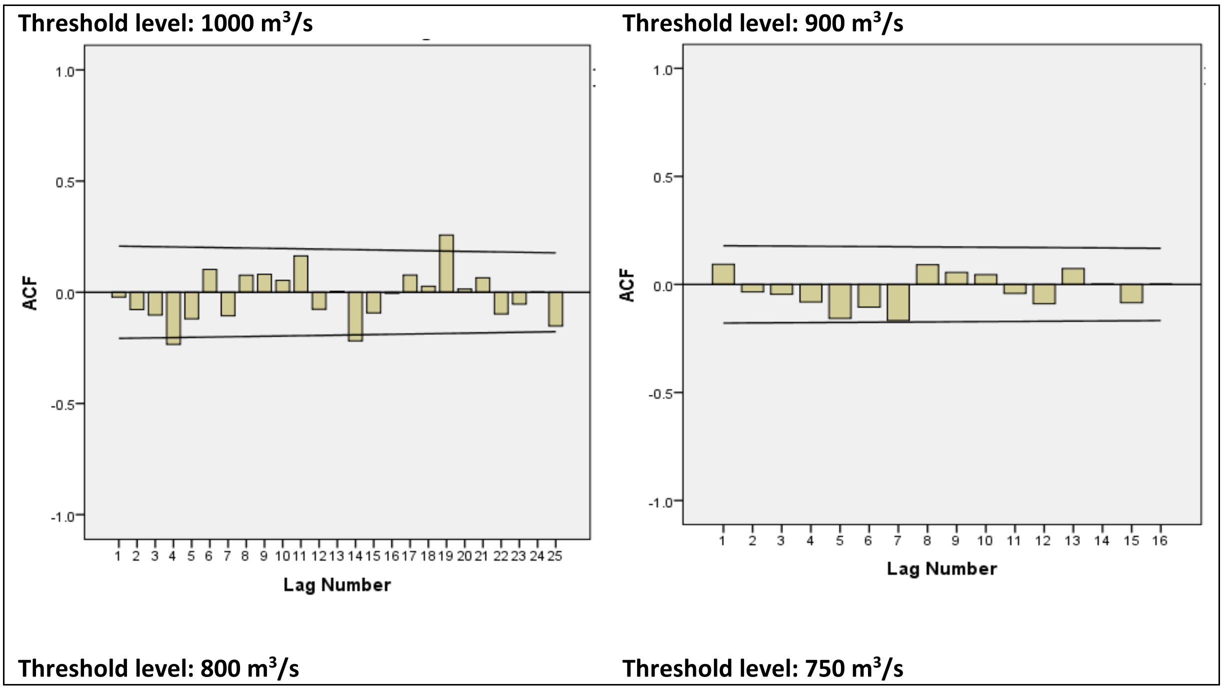


Figure B.1 Autocorrelation plot of AMShist (1930-2015). Black lines represent the upper and lower confidence limit.



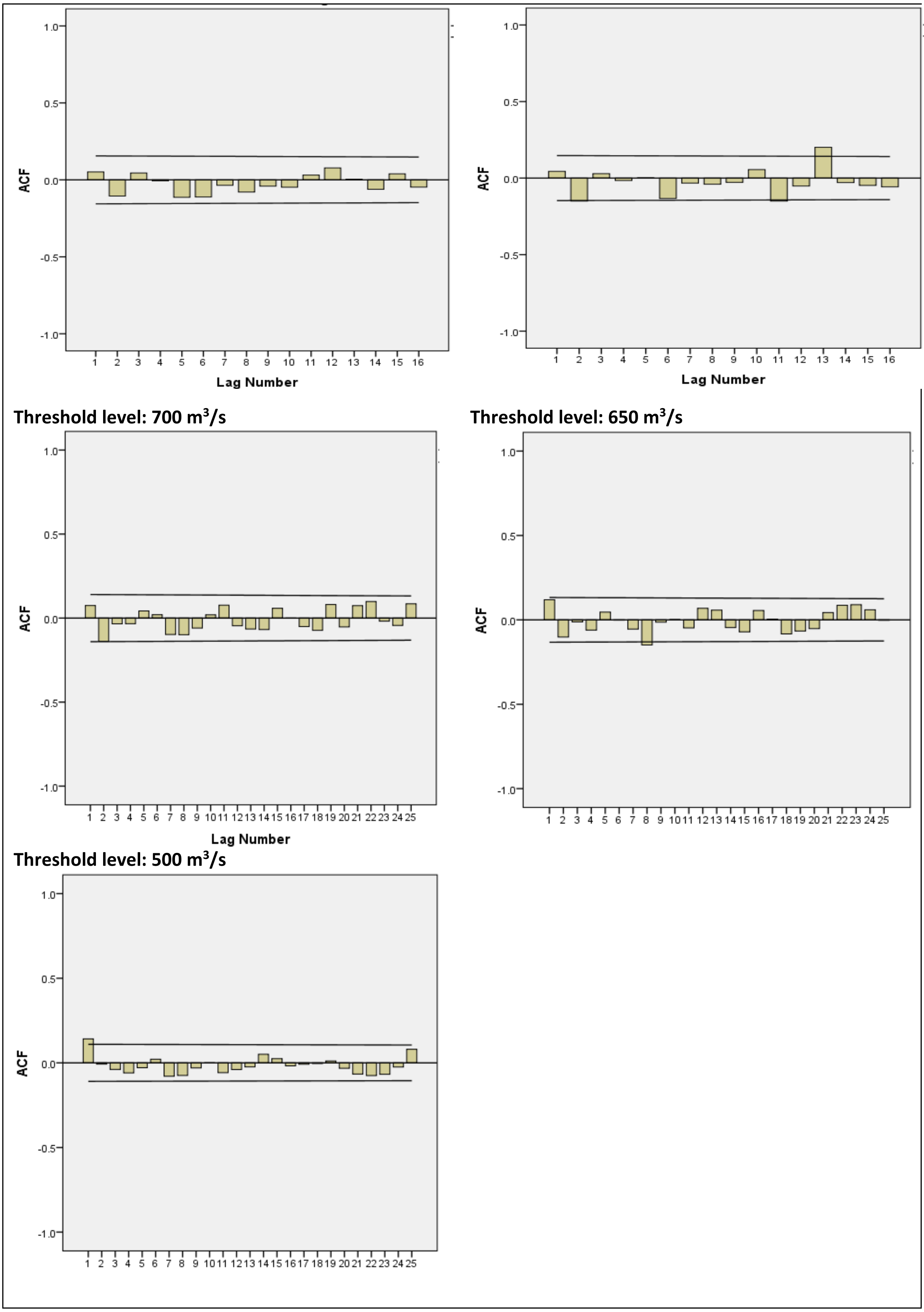
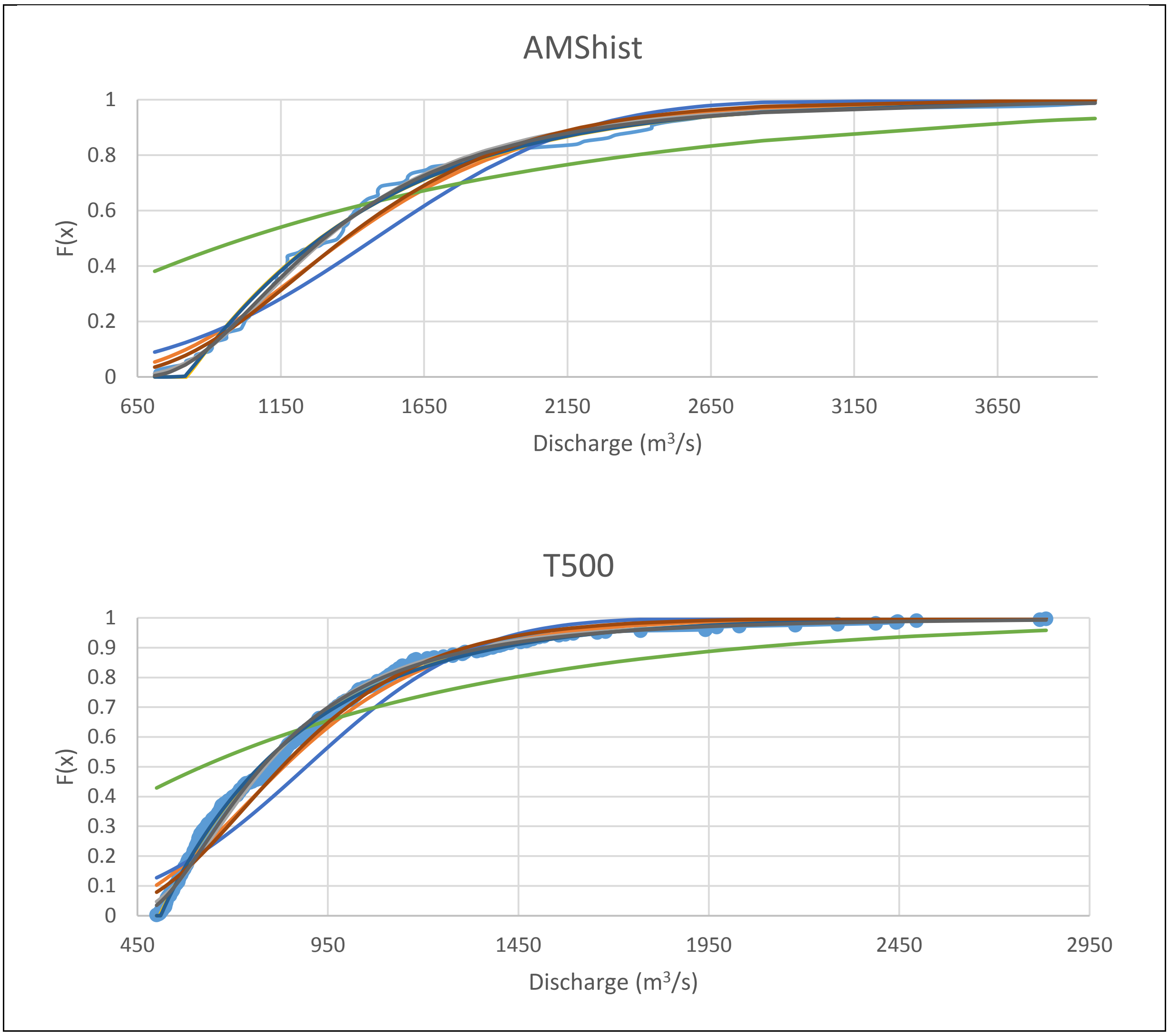
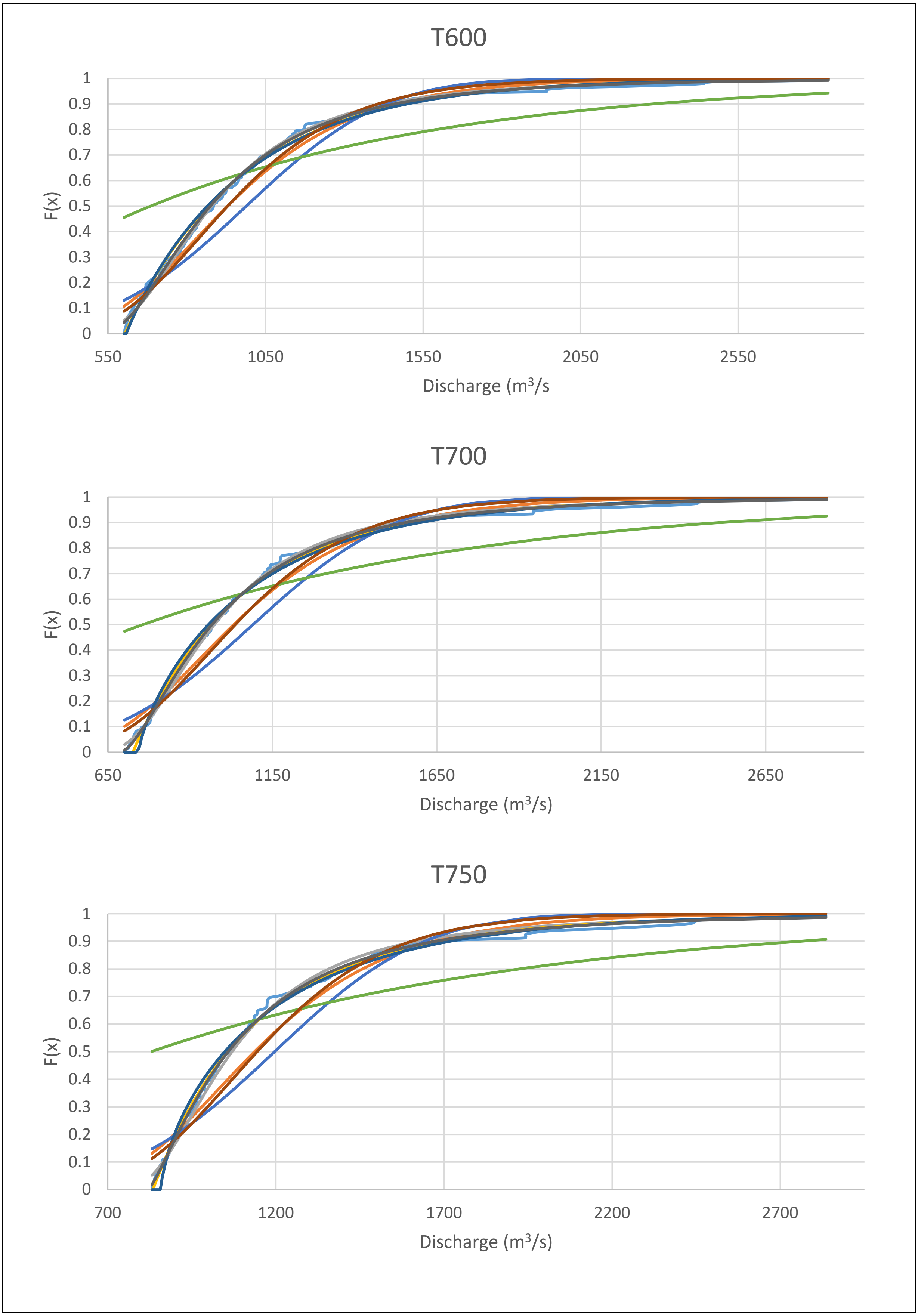


Figure B.2 Autocorrelation plots of partial duration series with chosen threshold levels. Black lines represent upper and lower confidence limits.

Table B.3 Calculated L-moments and L-moment ratios for the annual maximum series and each partial duration series.

Threshold	L_1 = mean	L_2 =scale	τ_3 = L-skewness	τ_4 = L-kurtosis
AMS	1450.83	280.327	0.304	0.17
AMS_{hist}	1479.194	323.148	0.321	0.199
500 m^3/s	893.215	194.516	0.341	0.19
600 m^3/s	989.148	195.143	0.344	0.215
650 m^3/s	1044.79	194.029	0.359	0.23
700 m^3/s	1090.937	192.634	0.381	0.234
750 m^3/s	1126.213	192.378	0.398	0.232
800 m^3/s	1165.237	193.916	0.403	0.228
900 m^3/s	1269.315	201.514	0.413	0.216
1000 m^3/s	1383.223	213.137	0.404	0.188




T600
F(x)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
550
1050
1550
2050
2550
Discharge (m3/s
T700
F(x)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
650
1150
1650
2150
2650
Discharge (m3/s)
T750
F(x)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
700
1200
1700
2200
2700
Discharge (m3/s)

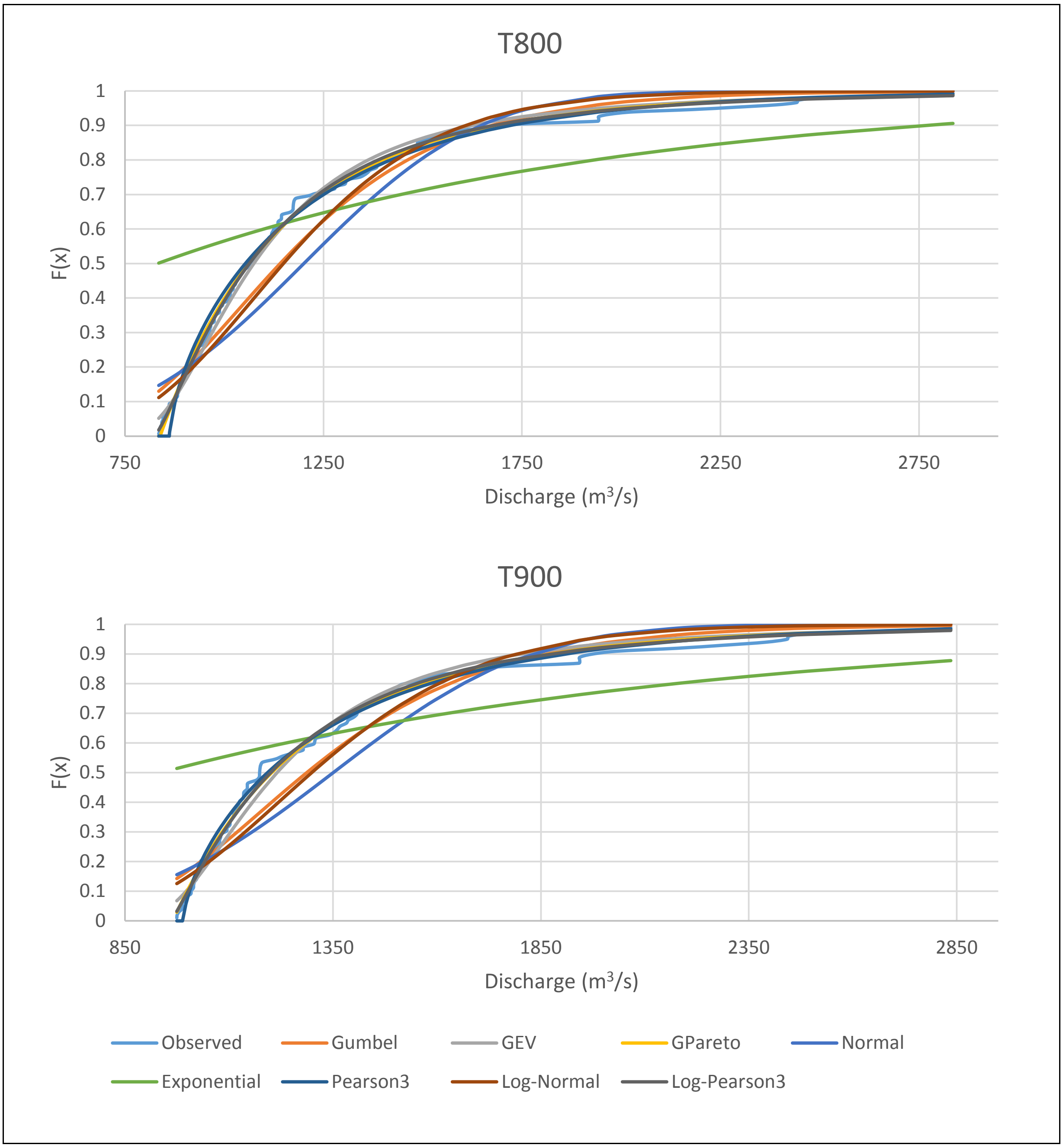
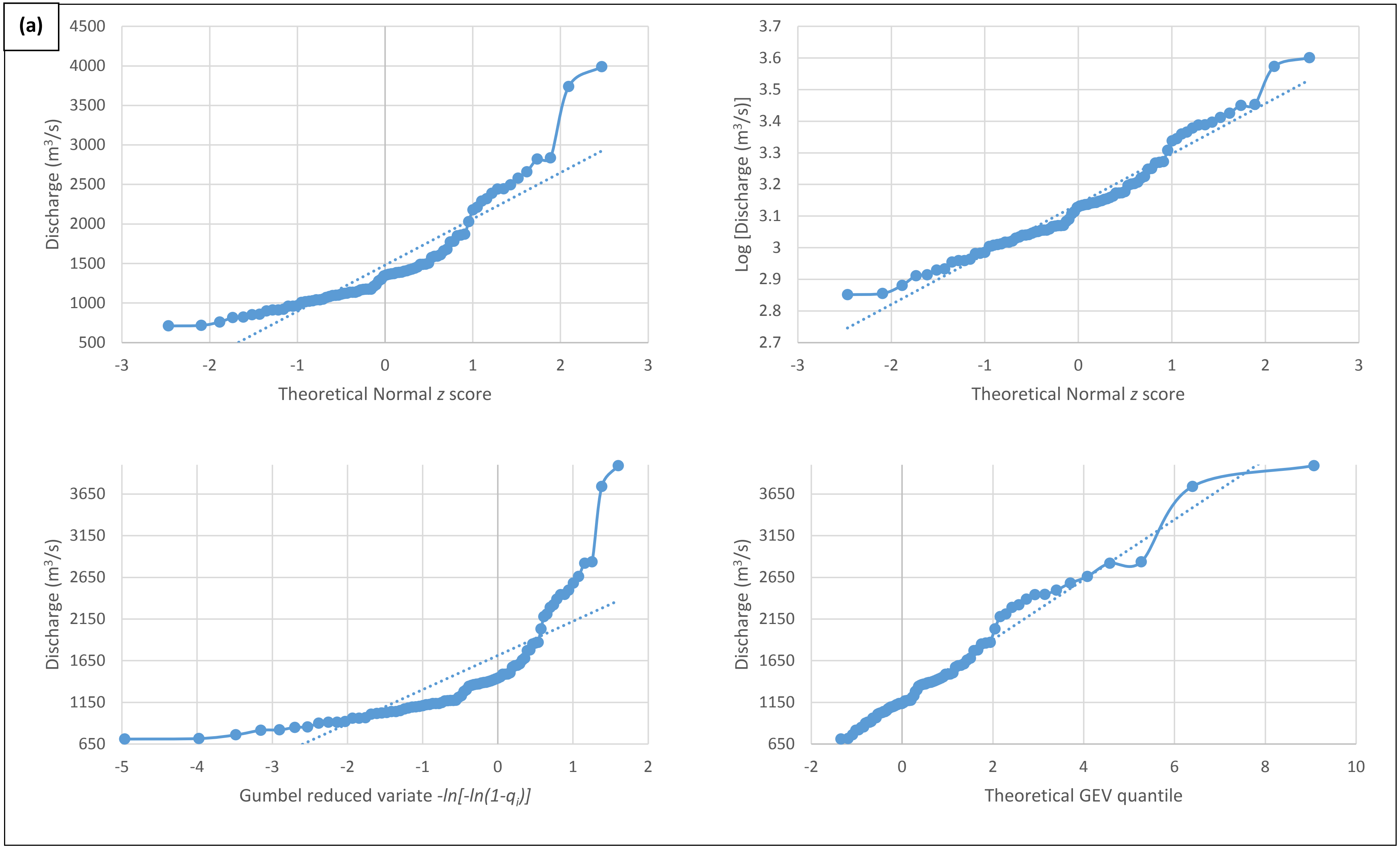
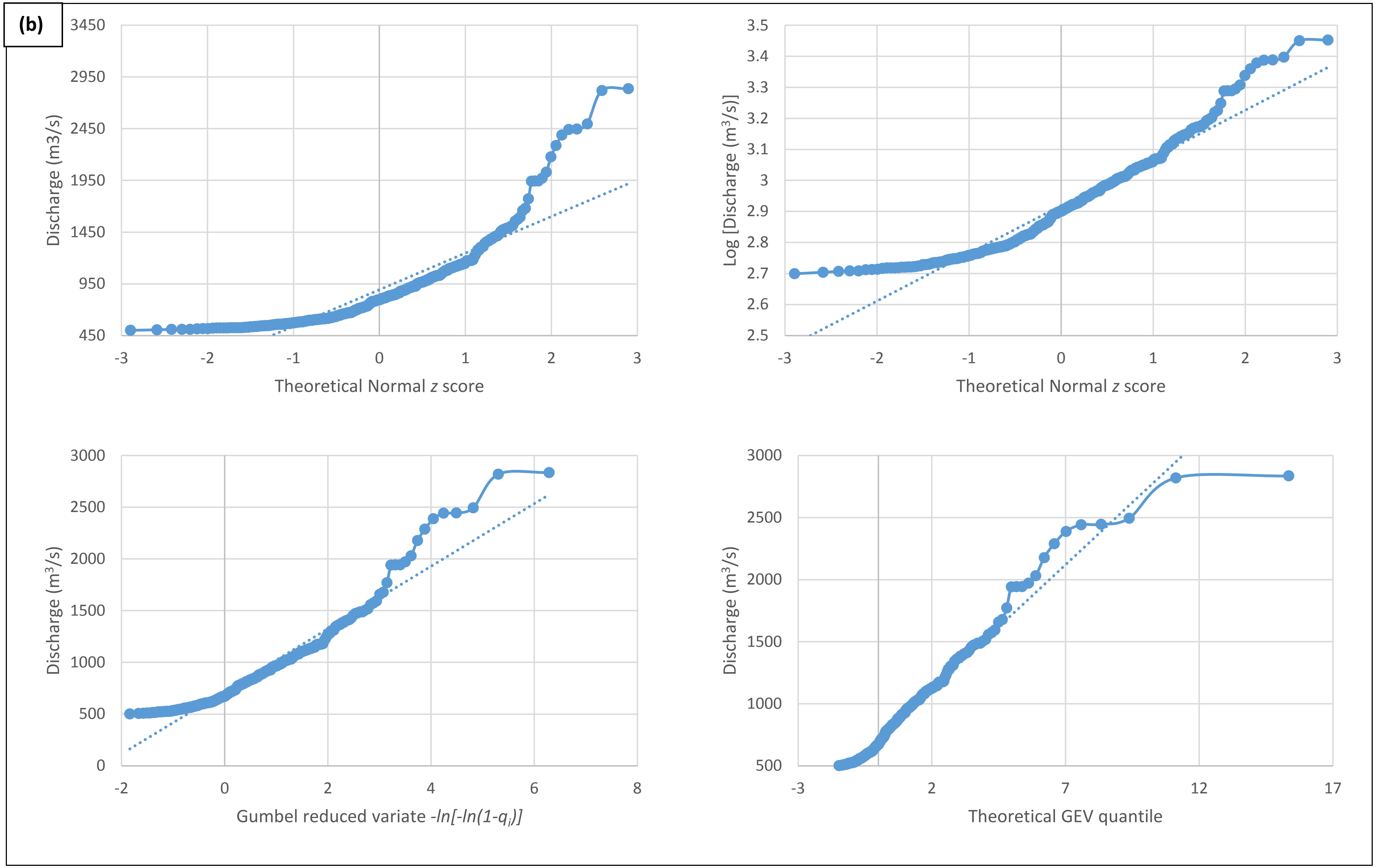
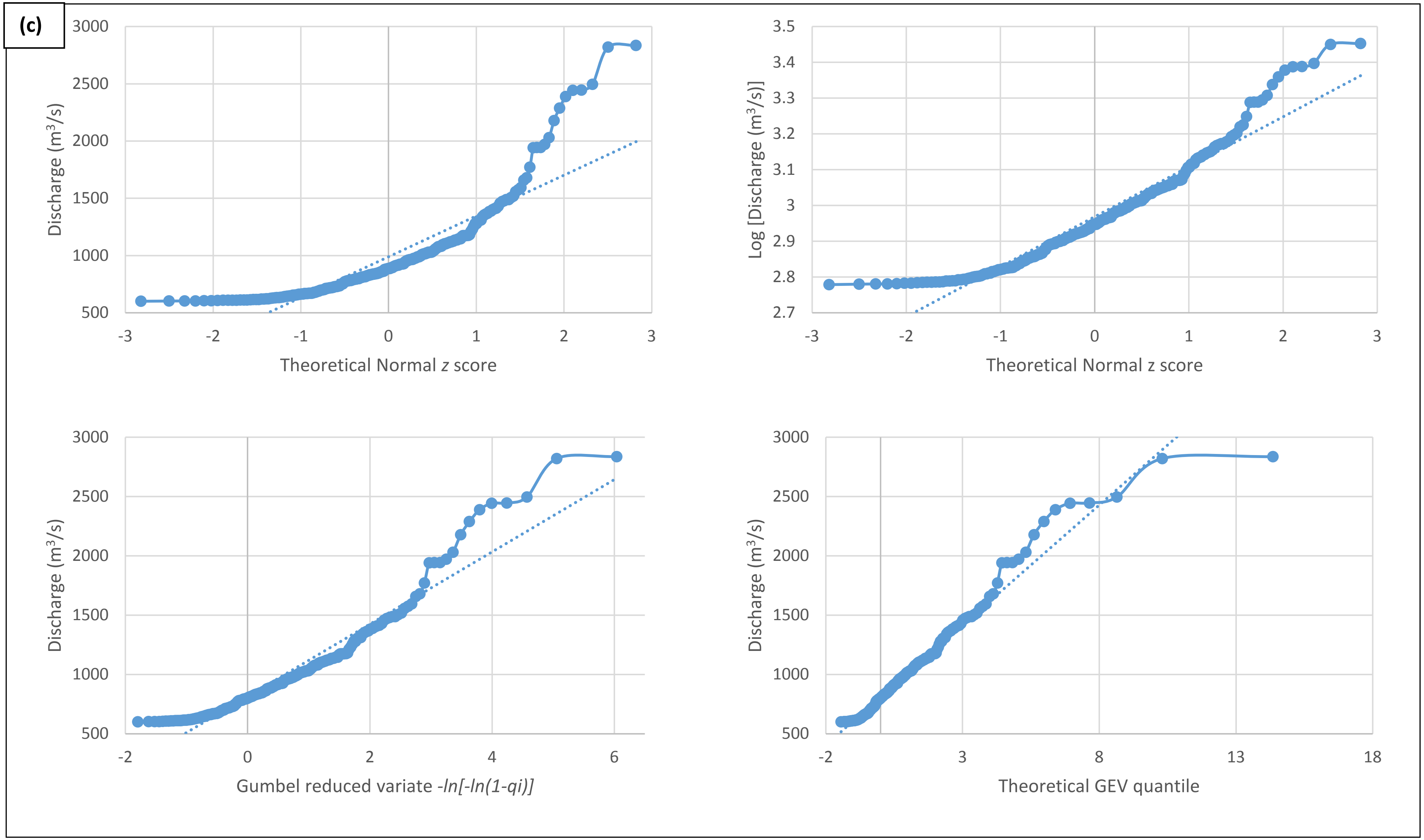


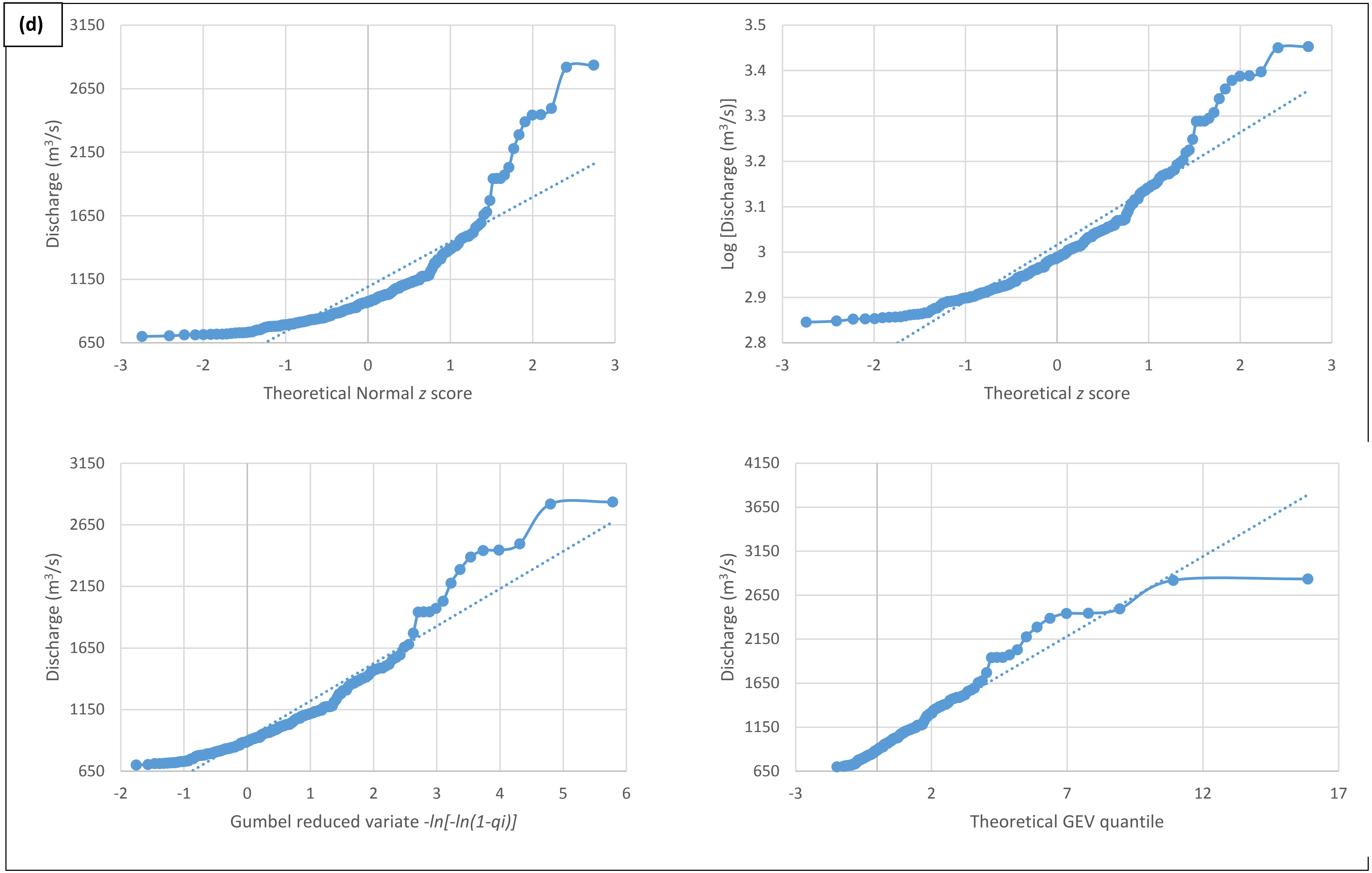
Figure B.3 Probability plots of PDS and AMS_{hist} series vs. theoretical distribution frequencies. T = threshold.


(a)
Discharge (m3/s)
Theoretical Normal z score
Log [Discharge (m3/s)]
Theoretical Normal z score
Discharge (m3/s)
Gumbel reduced variate -ln[-ln(1-qi)]
Discharge (m3/s)
Theoretical GEV quantile

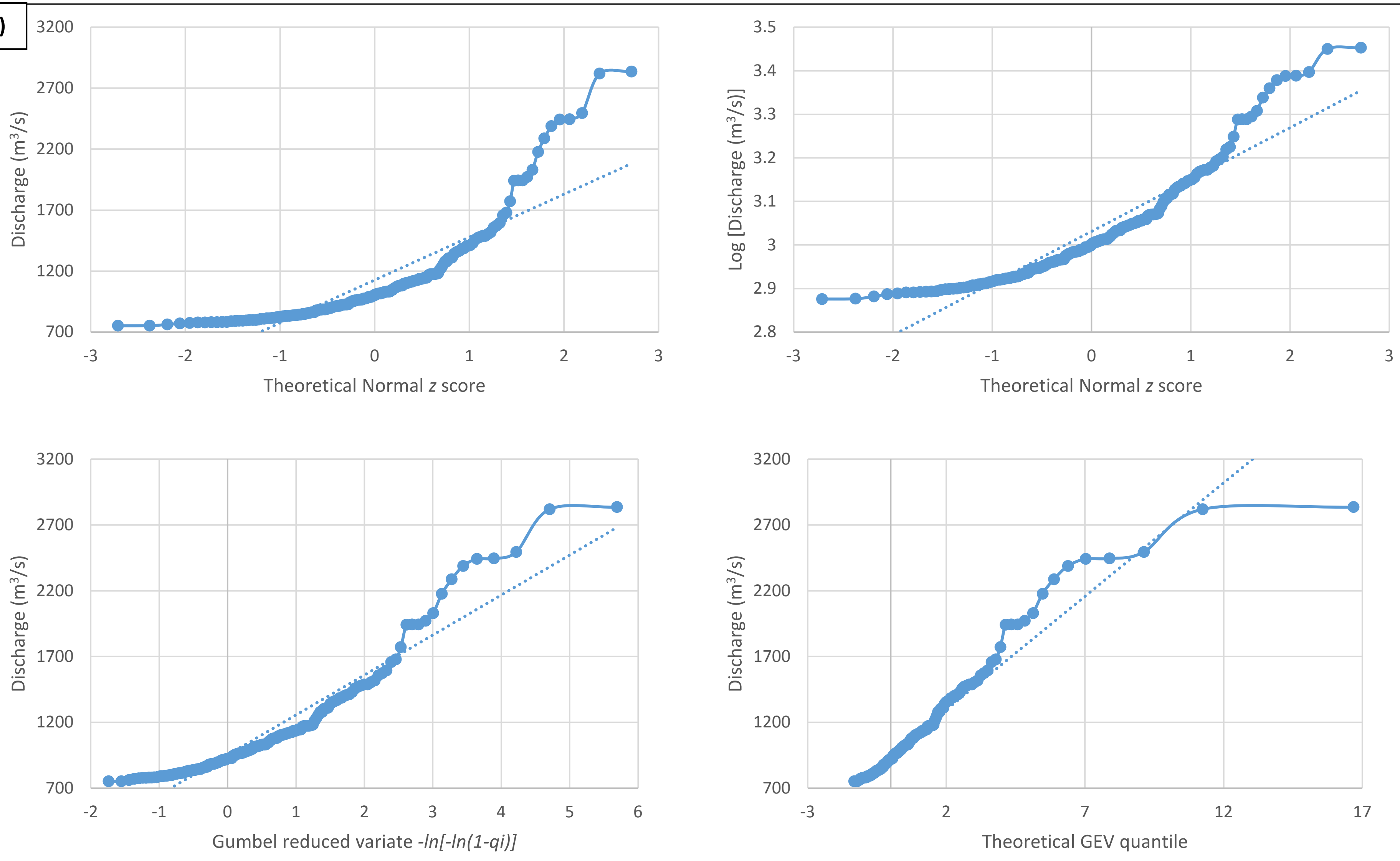


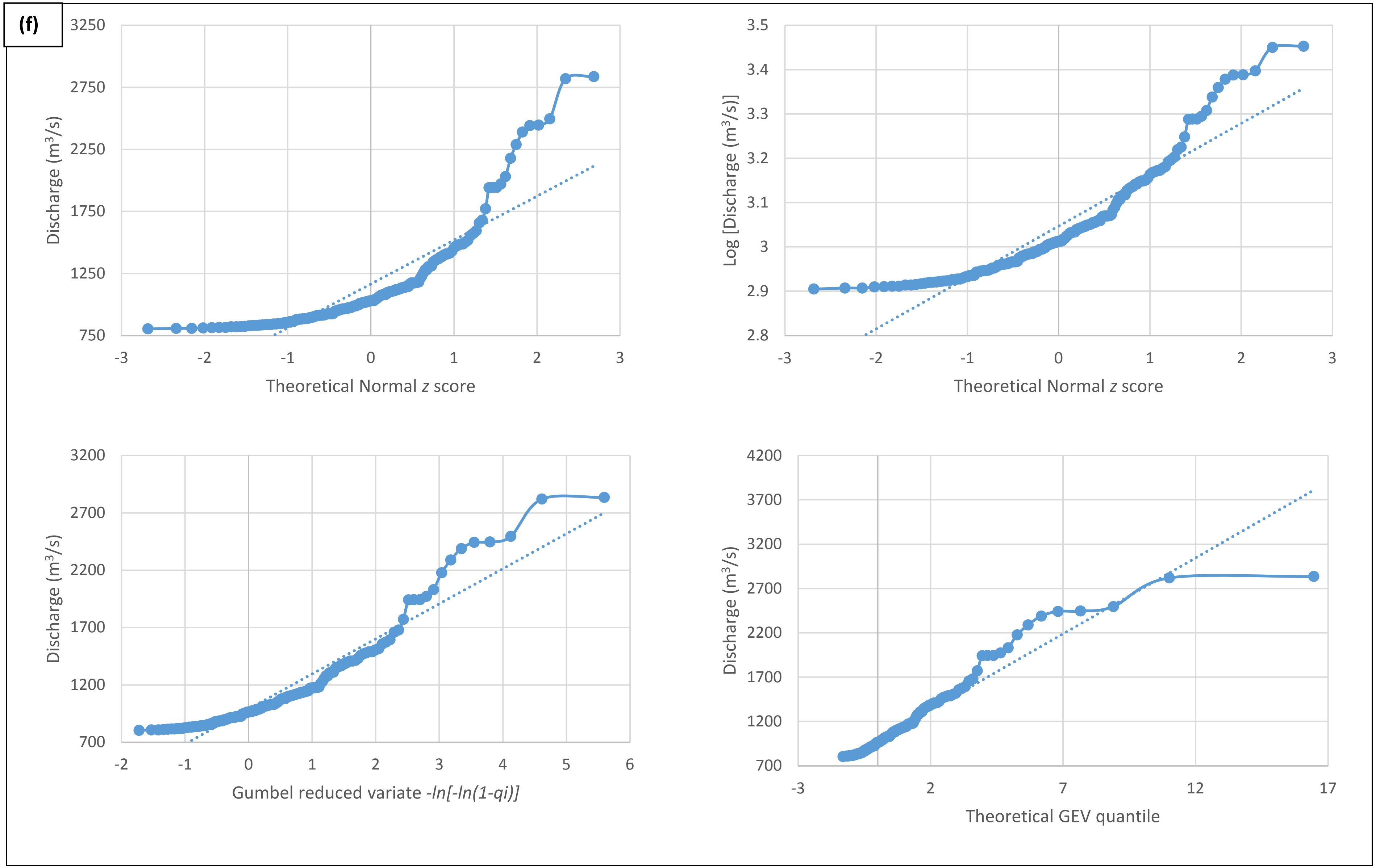
(b)
Discharge (m3/s)
3450
2950
2450
1950
1450
950
450
-3
-2
-1
0
1
2
3
Theoretical Normal z score
Log [Discharge (m3/s)]
3.5
3.4
3.3
3.2
3.1
3
2.9
2.8
2.7
2.6
2.5
Theoretical Normal z score
Discharge (m3/s)
3000
2500
2000
1500
1000
500
0
-2
0
2
4
6
8
Gumbel reduced variate -ln[-ln(1-qi)]
Discharge (m3/s)
3000
2500
2000
1500
1000
500
-3
2
7
12
17
Theoretical GEV quantile


(c)
Discharge (m³/s)
3000
2500
2000
1500
1000
500
-3
-2
-1
0
1
2
3
Theoretical Normal z score
Log [Discharge (m³/s)]
3.5
3.4
3.3
3.2
3.1
3
2.9
2.8
2.7
Theoretical Normal z score
Discharge (m³/s)
-2
0
2
4
6
Gumbel reduced variate -ln[-ln(1-qi)]
Discharge (m³/s)
-2
3
8
13
18
Theoretical GEV quantile


(d)
Discharge (m³/s)
Theoretical Normal z score
Log [Discharge (m³/s)]
Theoretical z score
Discharge (m³/s)
Gumbel reduced variate -ln[-ln(1-qi)]
Discharge (m³/s)
Theoretical GEV quantile

(e)


Discharge (m³/s)
Theoretical Normal z score
Log [Discharge (m³/s)]
Theoretical Normal z score
Discharge (m³/s)
Gumbel reduced variate -ln[-ln(1-qi)]
Discharge (m³/s)
Theoretical GEV quantile


(f)
Discharge (m³/s)
Theoretical Normal z score
Log [Discharge (m³/s)]
Theoretical Normal z score
Discharge (m³/s)
Gumbel reduced variate -ln[-ln(1-qi)]
Discharge (m³/s)
Theoretical GEV quantile

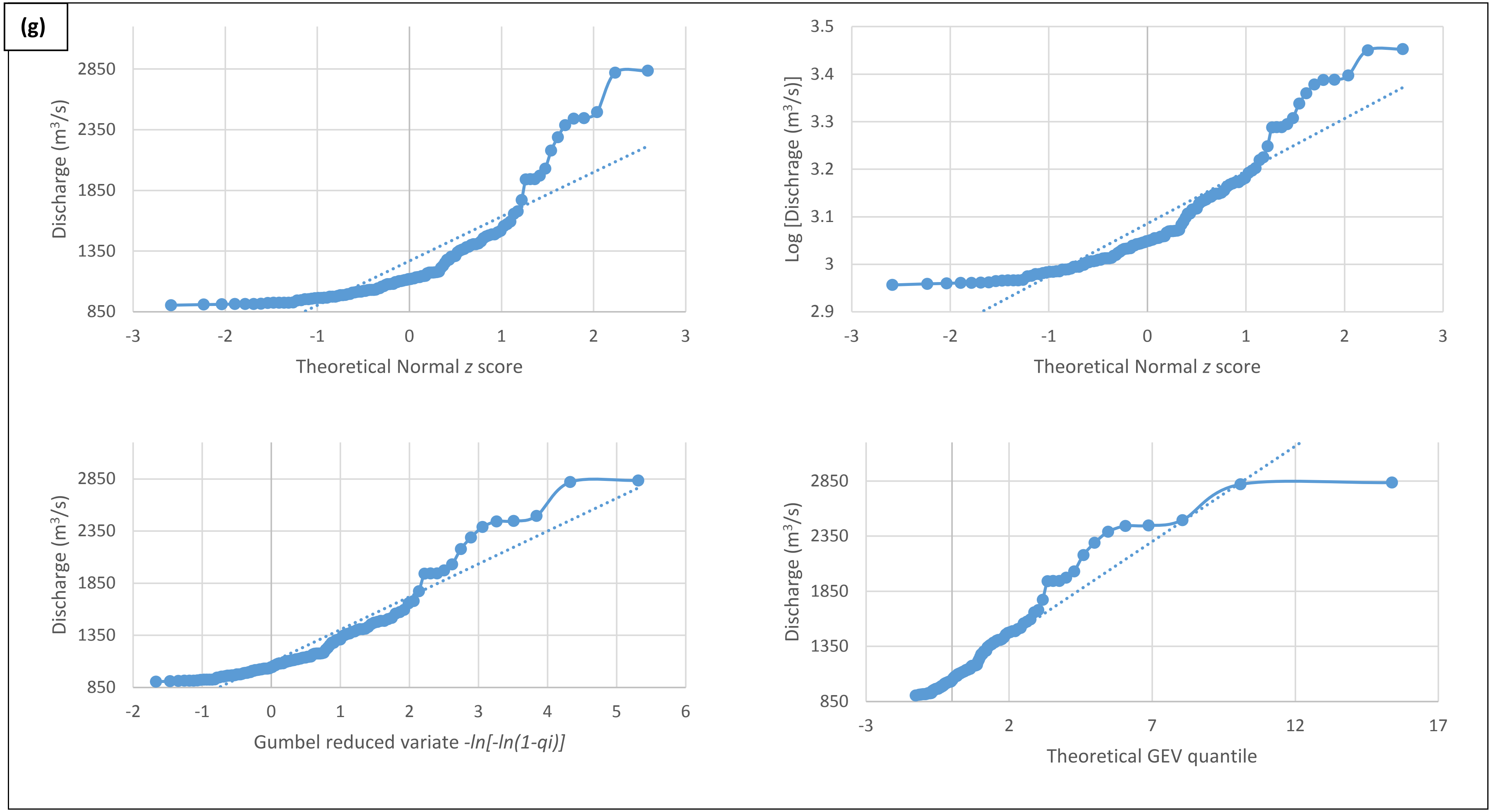
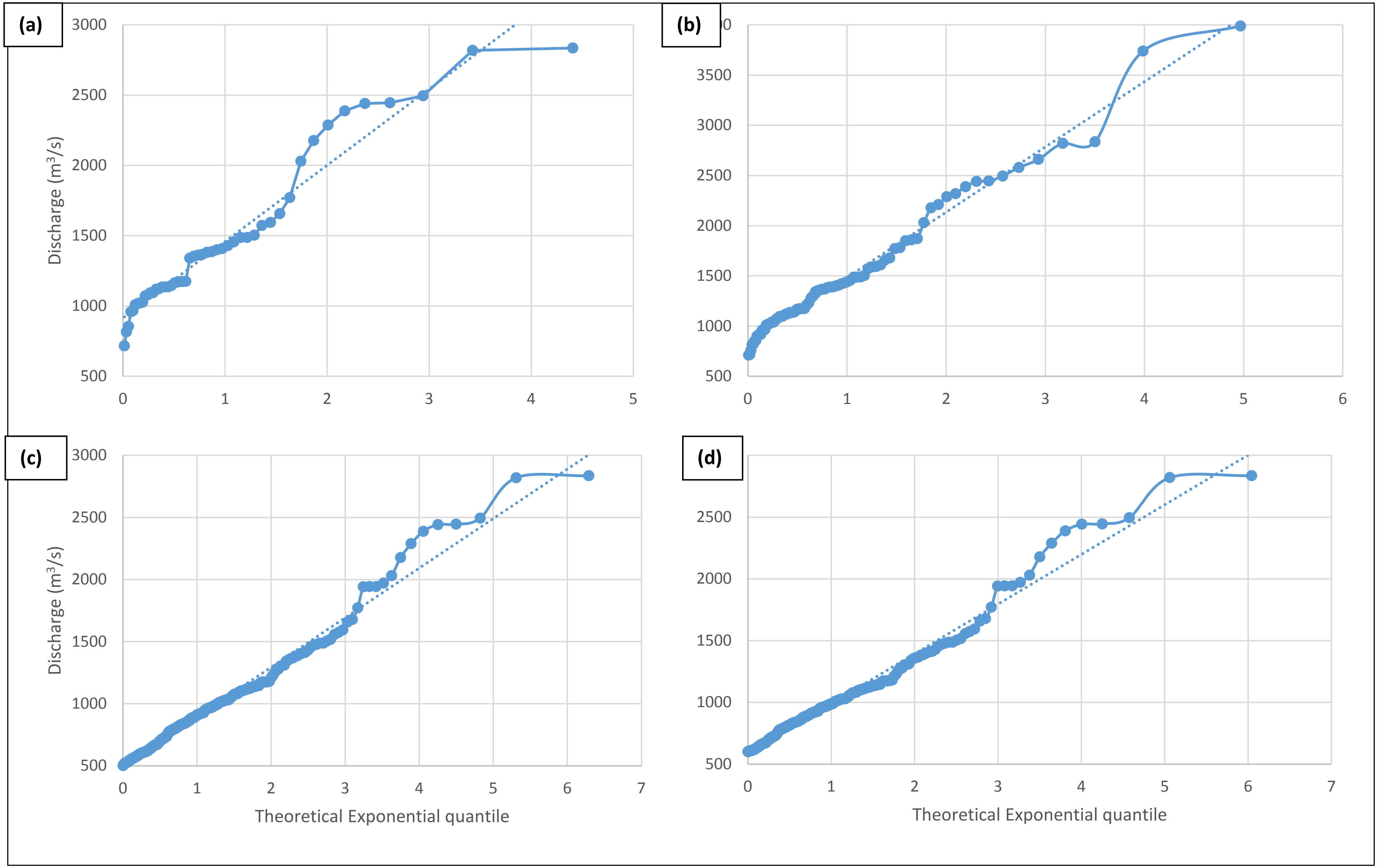
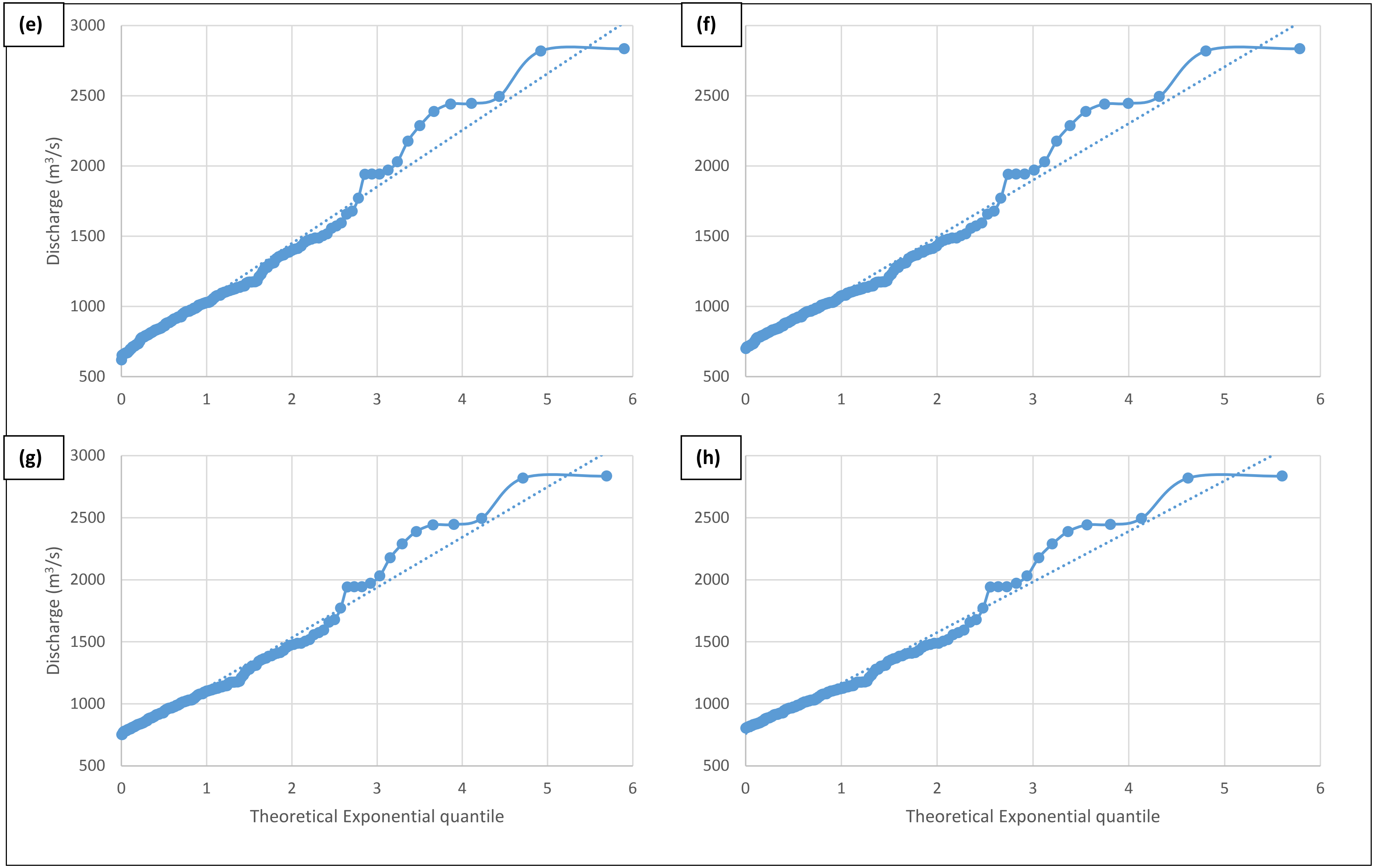


Figure B.4 Quantile-quantile plots of PDS and AMS series vs. theoretical distribution quantiles. (a) AMShist series, (b) PDS with 500 m^3/s threshold, (c) PDS with 600 m^3/s threshold, (d) PDS with 700 m^3/s threshold, (e) PDS with 750 m^3/s threshold, (f) PDS with 800 m^3/s threshold, and (g) PDS with 900 m^3/s threshold.


(a)
(b)
(c)
(d)
Discharge (m³/s)
Theoretical Exponential quantile
3000
2500
2000
1500
1000
500
3500
0
1
2
3
4
5
6
7


(e)
(f)
(g)
(h)
Discharge (m³/s)
Theoretical Exponential quantile
3000
2500
2000
1500
1000
500
0
1
2
3
4
5
6

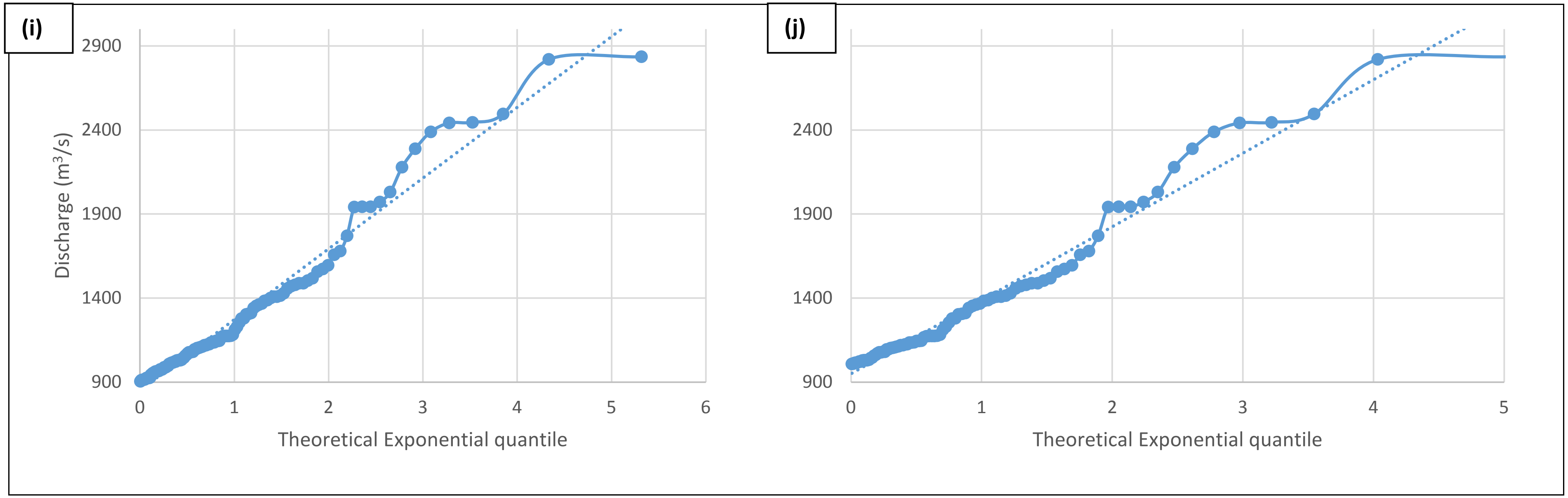


Figure B.5 QQ plots of PDS and AMS vs. theoretical Exponential quantiles. (a) AMS, (b) AMS_{hist}, (c) PDS with threshold 500 m^3/s, (d) PDS with threshold 600 m^3/s, (e) PDS with threshold 650 m^3/s, (f) PDS with threshold 700 m^3/s, (g) PDS with threshold 750 m^3/s, (h) PDS with threshold 800 m^3/s, (i) PDS with threshold 900 m^3/s, (j) PDS with threshold 100 m^3/s.

Table B.4 Tukey's HSD post-hoc testing for Q10 estimates.

(I) Distribution	(J) Distribution	Mean Difference (I-J)	Std. Error	Significance	95 % Confidence Interval Lower Bound	Upper Bound
GEV	LP3	-66.06662*	19.47653	.014	-122.0628	-10.0704
	P3	-25.48287	19.47653	.688	-81.4791	30.5133
	GP	-8.17312	19.47653	.993	-64.1693	47.8231
	EV1	189.07963*	19.47653	.000	133.0834	245.0758
LP3	GEV	66.06662*	19.47653	.014	10.0704	122.0628
	P3	40.58375	19.47653	.250	-15.4125	96.5800
	GP	57.89350*	19.47653	.040	1.8973	113.8897
	EV1	255.14625*	19.47653	.000	199.1500	311.1425
P3	GEV	25.48287	19.47653	.688	-30.5133	81.4791
	LP3	-40.58375	19.47653	.250	-96.5800	15.4125
	GP	17.30975	19.47653	.899	-38.6865	73.3060
	EV1	214.56250*	19.47653	.000	158.5663	270.5587
GP	GEV	8.17312	19.47653	.993	-47.8231	64.1693
	LP3	-57.89350*	19.47653	.040	-113.8897	-1.8973
	P3	-17.30975	19.47653	.899	-73.3060	38.6865
	EV1	197.25275*	19.47653	.000	141.2565	253.2490
EV1	GEV	-189.07963*	19.47653	.000	-245.0758	-133.0834
	LP3	-255.14625*	19.47653	.000	-311.1425	-199.1500
	P3	-214.56250*	19.47653	.000	-270.5587	-158.5663
	GP	-197.25275*	19.47653	.000	-253.2490	-141.2565

*. The mean difference is significant at the 0.05 level.

Table B.5 Tukey's HSD post-hoc testing for Q25 estimates.

(I) Distribution	(J) Distribution	Mean Difference (I-J)	Std. Error	Significance	95 % Confidence Interval Lower Bound	Upper Bound
GEV	LP3	-52.76163	28.16116	.350	-133.7266	28.2034
	P3	191.03087*	28.16116	.000	110.0659	271.9959
	GP	216.21762*	28.16116	.000	135.2526	297.1826
	EV1	552.02087*	28.16116	.000	471.0559	632.9859
LP3	GEV	52.76163	28.16116	.350	-28.2034	133.7266
	P3	243.79250*	28.16116	.000	162.8275	324.7575
	GP	268.97925*	28.16116	.000	188.0142	349.9443
	EV1	604.78250*	28.16116	.000	523.8175	685.7475
P3	GEV	-191.03087*	28.16116	.000	-271.9959	-110.0659
	LP3	-243.79250*	28.16116	.000	-324.7575	-162.8275
	GP	25.18675	28.16116	.897	-55.7783	106.1518
	EV1	360.99000*	28.16116	.000	280.0250	441.9550
GP	GEV	-216.21762*	28.16116	.000	-297.1826	-135.2526

	LP3	-268.97925*	28.16116	.000	-349.9443	-188.0142
	P3	-25.18675	28.16116	.897	-106.1518	55.7783
	EV1	335.80325*	28.16116	.000	254.8382	416.7683
EV1	GEV	-552.02087*	28.16116	.000	-632.9859	-471.0559
	LP3	-604.78250*	28.16116	.000	-685.7475	-523.8175
	P3	-360.99000*	28.16116	.000	-441.9550	-280.0250
	GP	-335.80325*	28.16116	.000	-416.7683	-254.8382

*. The mean difference is significant at the 0.05 level.

Table B.6 Games-Howell post-hoc testing for Q50 estimates.

(I) Distribution	(J) Distribution	Mean Difference (I-J)	Std. Error	Significance	95 % Confidence Interval	
					Lower Bound	Upper Bound
GEV	LP3	-8.17612	34.48602	.999	-117.6300	101.2777
	P3	475.77175*	40.50866	.000	344.3887	607.1548
	GP	362.25763*	56.50842	.001	171.4764	553.0388
	EV1	950.42925*	33.42213	.000	844.7620	1056.0965
LP3	GEV	8.17612	34.48602	.999	-101.2777	117.6300
	P3	483.94788*	45.59133	.000	340.9741	626.9217
	GP	370.43375*	60.25628	.001	174.7094	566.1581
	EV1	958.60538*	39.42913	.000	835.7104	1081.5003
P3	GEV	-475.77175*	40.50866	.000	-607.1548	-344.3887
	LP3	-483.94788*	45.59133	.000	-626.9217	-340.9741
	GP	-113.51413	63.89433	.428	-316.5597	89.5314
	EV1	474.65750*	44.79199	.000	333.7893	615.5257
GP	GEV	-362.25763*	56.50842	.001	-553.0388	-171.4764
	LP3	-370.43375*	60.25628	.001	-566.1581	-174.7094
	P3	113.51413	63.89433	.428	-89.5314	316.5597
	EV1	588.17163*	59.65377	.000	393.4500	782.8932
EV1	GEV	-950.42925*	33.42213	.000	-1056.0965	-844.7620
	LP3	-958.60538*	39.42913	.000	-1081.5003	-835.7104
	P3	-474.65750*	44.79199	.000	-615.5257	-333.7893
	GP	-588.17163*	59.65377	.000	-782.8932	-393.4500

*. The mean difference is significant at the 0.05 level.

Table B.7 Games-Howell post-hoc testing for Q100 estimates.

(I) Distribution	(J) Distribution	Mean Difference (I-J)	Std. Error	Significance	95 % Confidence Interval Lower Bound	Upper Bound
GEV	LP3	76.07237	55.73107	.658	-99.1892	251.3340
	P3	896.07387*	57.25619	.000	715.4823	1076.6655
	GP	683.63675*	90.76243	.000	379.9383	987.3352
	EV1	1486.23888*	45.01269	.000	1345.6407	1626.8370
LP3	GEV	-76.07237	55.73107	.658	-251.3340	99.1892
	P3	820.00150*	63.92961	.000	620.7524	1019.2506
	GP	607.56437*	95.11331	.000	297.8877	917.2411
	EV1	1410.16650*	53.24345	.000	1240.9557	1579.3773
P3	GEV	-896.07387*	57.25619	.000	-1076.6655	-715.4823
	LP3	-820.00150*	63.92961	.000	-1019.2506	-620.7524
	GP	-212.43712	96.01490	.245	-523.6630	98.7887
	EV1	590.16500*	54.83780	.000	415.2131	765.1169
GP	GEV	-683.63675*	90.76243	.000	-987.3352	-379.9383
	LP3	-607.56437*	95.11331	.000	-917.2411	-297.8877
	P3	212.43712	96.01490	.245	-98.7887	523.6630
	EV1	802.60213*	89.25655	.000	500.2815	1104.9227
EV1	GEV	-1486.23888*	45.01269	.000	-1626.8370	-1345.6407
	LP3	-1410.16650*	53.24345	.000	-1579.3773	-1240.9557
	P3	-590.16500*	54.83780	.000	-765.1169	-415.2131
	GP	-802.60213*	89.25655	.000	-1104.9227	-500.2815

*. The mean difference is significant at the 0.05 level.

Table B.8 Games-Howell post-hoc testing for Q500 estimates.

(I) Distribution	(J) Distribution	Mean Difference (I-J)	Std. Error	Significance	95 % Confidence Interval Lower Bound	Upper Bound
GEV	LP3	530.23700*	167.27194	.046	6.8848	1053.5892
	P3	2602.15238*	147.24779	.000	2126.4533	3077.8514
	GP	2029.09500*	229.89811	.000	1298.6406	2759.5494
	EV1	3462.97363*	132.88167	.000	3004.5629	3921.3844
LP3	GEV	-530.23700*	167.27194	.046	-1053.5892	-6.8848
	P3	2071.91537*	129.68506	.000	1660.0416	2483.7891
	GP	1498.85800*	219.06493	.000	790.7941	2206.9219
	EV1	2932.73662*	113.10986	.000	2547.3251	3318.1482
P3	GEV	-2602.15238*	147.24779	.000	-3077.8514	-2126.4533
	LP3	-2071.91537*	129.68506	.000	-2483.7891	-1660.0416
	GP	-573.05737	204.18486	.113	-1259.9730	113.8583
	EV1	860.82125*	80.59684	.000	596.1609	1125.4816

GP	GEV	-2029.09500*	229.89811	.000	-2759.5494	-1298.6406
	LP3	-1498.85800*	219.06493	.000	-2206.9219	-790.7941
	P3	573.05737	204.18486	.113	-113.8583	1259.9730
	EV1	1433.87862*	194.08009	.001	751.7355	2116.0218
EV1	GEV	-3462.97363*	132.88167	.000	-3921.3844	-3004.5629
	LP3	-2932.73662*	113.10986	.000	-3318.1482	-2547.3251
	P3	-860.82125*	80.59684	.000	-1125.4816	-596.1609
	GP	-1433.87862*	194.08009	.001	-2116.0218	-751.7355

*. The mean difference is significant at the 0.05 level.

Appendix C

Complementary results for Chapter 5: Low flow frequency analysis

Table C.1 Annual minimum series at Waimakariri OHB. Values are given in m^3/s.

1967	37.97	**1984**	39.32	**2001**	27.56
1968	41.79	**1985**	25.11	**2002**	39.99
1969	38.83	**1986**	36.92	**2003**	35.95
1970	38.66	**1987**	48.90	**2004**	36.97
1971	21.26	**1988**	33.24	**2005**	35.51
1972	26.55	**1989**	38.21	**2006**	29.26
1973	26.02	**1990**	31.73	**2007**	24.97
1974	48.24	**1991**	36.78	**2008**	29.23
1975	52.55	**1992**	33.33	**2009**	33.24
1976	35.00	**1993**	37.65	**2010**	30.12
1977	45.98	**1994**	50.05	**2011**	41.95
1978	26.91	**1995**	47.72	**2012**	35.78
1979	52.00	**1996**	52.74	**2013**	30.47
1980	53.54	**1997**	49.09	**2014**	31.22
1981	32.00	**1998**	54.59	**2015**	29.30
1982	34.17	**1999**	33.63		
1983	25.68	**2000**	42.00		

Table C.2 7dMALF series at Waimakariri OHB. Values are given in m^3/s.

1967	40.19	**1984**	44.12	**2001**	28.10
1968	40.61	**1985**	25.80	**2002**	40.50
1969	39.93	**1986**	41.56	**2003**	37.42
1970	39.82	**1987**	53.70	**2004**	40.65
1971	22.45	**1988**	34.96	**2005**	42.86
1972	27.51	**1989**	40.96	**2006**	31.26
1973	27.25	**1990**	34.72	**2007**	31.80
1974	33.71	**1991**	38.17	**2008**	32.27
1975	55.60	**1992**	34.60	**2009**	35.77
1976	36.77	**1993**	48.21	**2010**	32.39
1977	50.17	**1994**	39.55	**2011**	47.39
1978	27.85	**1995**	48.19	**2012**	37.17
1979	56.79	**1996**	59.28	**2013**	31.62
1980	57.38	**1997**	52.41	**2014**	33.66
1981	35.58	**1998**	55.98	**2015**	31.14
1982	37.34	**1999**	33.92		
1983	27.59	**2000**	43.18		

Table C.3 Summarised results for trend tests of selected unmodified PDS.

Series	Threshold	Spearman's ρ	Kendall's τ
Lowest Value	**60 m3/s**	-0.034 < 0.479	-0.018 < 0.589
	50 m3/s	-0.190 > 0.003 **	-0.127 > 0.003 **
	45 m3/s	-0.211 > 0.005 **	-0.143 > 0.005 **
	40 m3/s	0.098 < 0.150	0.086 < 0.105
	35 m3/s	0.337 >0.004 **	0.231 > 0.005 **
Deficit	**60 m3/s**	0.035 < 0.468	0.021 < 0.514
	50 m3/s	0.094 < 0.140	0.063 < 0.142
	45 m3/s	0.091 < 0.230	0.061 < 0.227
	40 m3/s	0.087 < 0.342	0.058 < 0.342
	35 m3/s	-0.335 > 0.005 **	-0.223 > 0.006 **
Duration	**60 m3/s**	0.064 < 0.186	0.044 < 0.179
	50 m3/s	0.103 < 0.106	0.069 < 0.108
	45 m3/s	0.029 < 0.701	0.020 < 0.697
	40 m3/s	-0.001 < 0.982	-0.003 < 0.971
	35 m3/s	-0.289 > 0.015 *	-0.200 > 0.015 *

* indicates detected significance at α = 0.05, ** indicates detected significance at α = 0.01

Table C.4 Summary of parameters obtained by the methods of L-moments for unmodified series.

	Lowest value				**Duration**				**Deficit**			
	L_1	L_2	τ_3	τ_4	L_1	L_2	τ_3	τ_4	L_1	L_2	τ_3	τ_4
AMS	37.136	5.006	-0.109	0.081	n/a				n/a			
7dMALF	39.16	5.209	-0.133	0.115	n/a				n/a			
60 m^3/s	46.493	4.432	0.165	0.071	5.086	0.504	0.047	0.077	8.327	0.858	0.02	0.074
40 m^3/s	33.956	1.993	0.196	0.134	4.897	0.527	0.049	0.111	7.254	0.885	0.043	0.12
35 m^3/s	30.62	1.507	0.291	0.135	4.715	0.512	0.147	0.062	6.749	0.82	0.169	0.072

Table C.5 Summary of parameters obtained by the methods of L-moments for censored Q2 series.

PDS_{Lowest_value}				**$PDS_{Duration}$**				**$PDS_{Deficit}$**			
L_1	L_2	τ_3	τ_4	L_1	L_2	τ_3	τ_4	L_1	L_2	τ_3	τ_4
31.26	1.193	0.296	0.163	96.543	34.71	0.32	0.065	6.20	0.566	0.11	-0.011

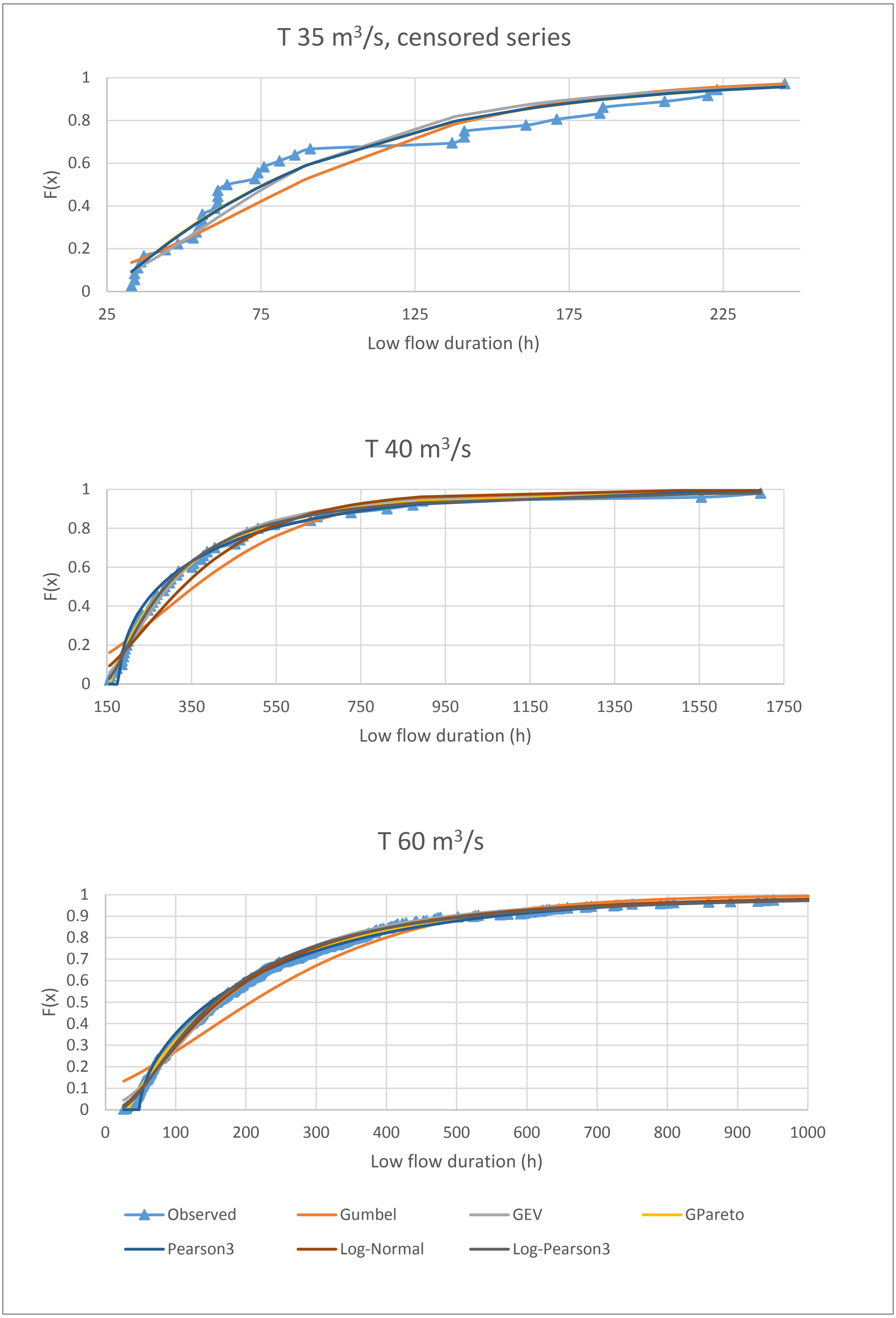


Figure C.1 Probability plots of low flow durations for various thresholds. T = threshold.

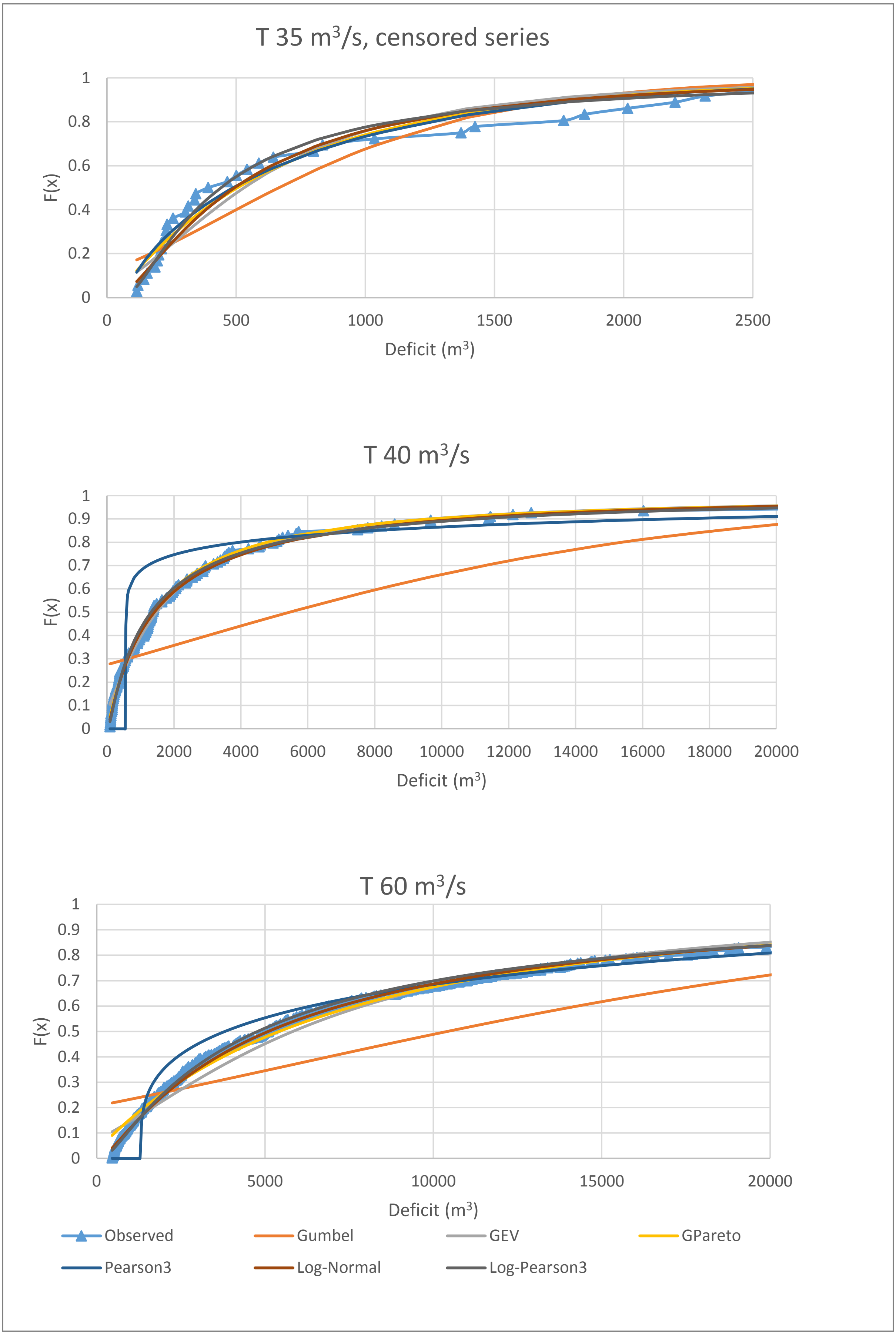
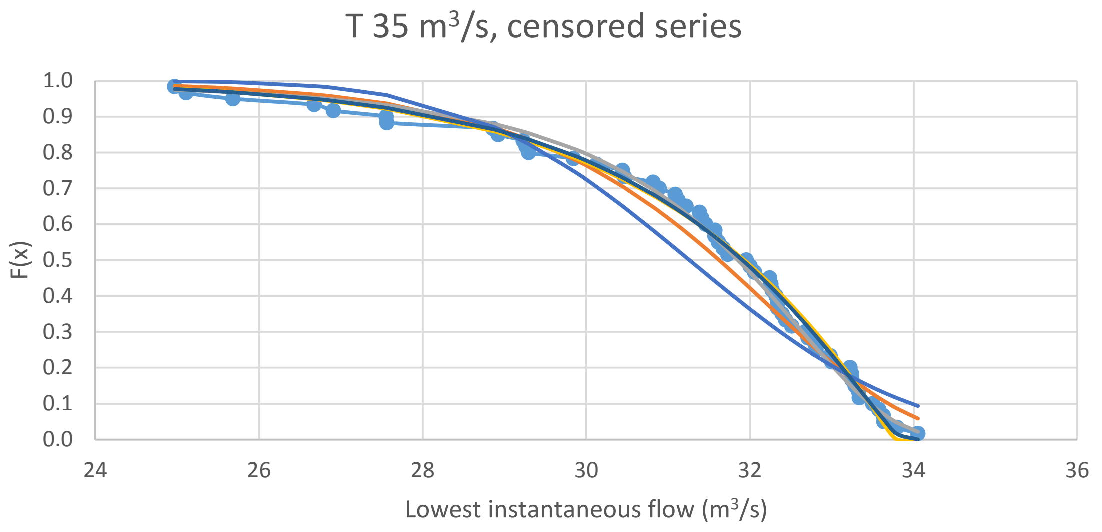
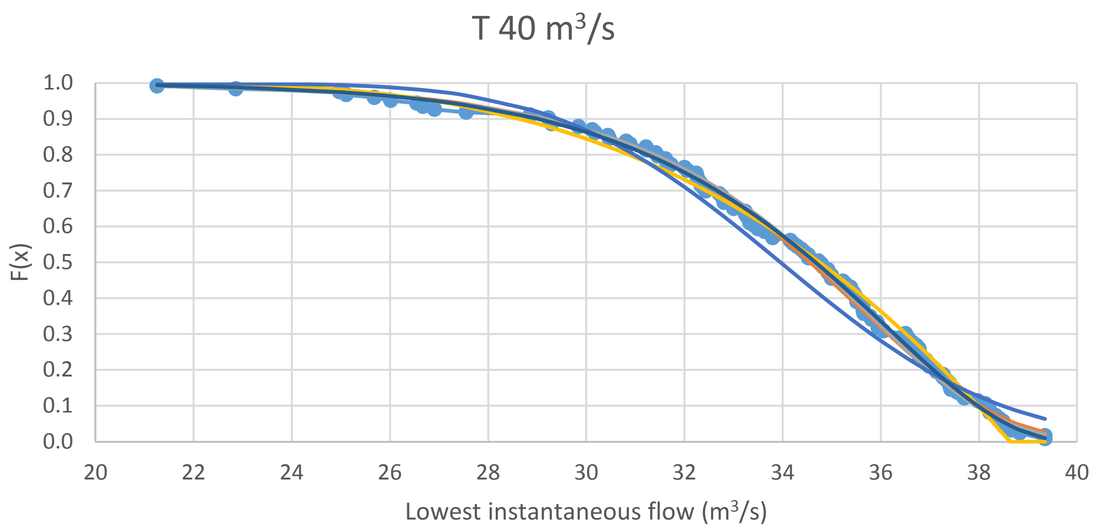
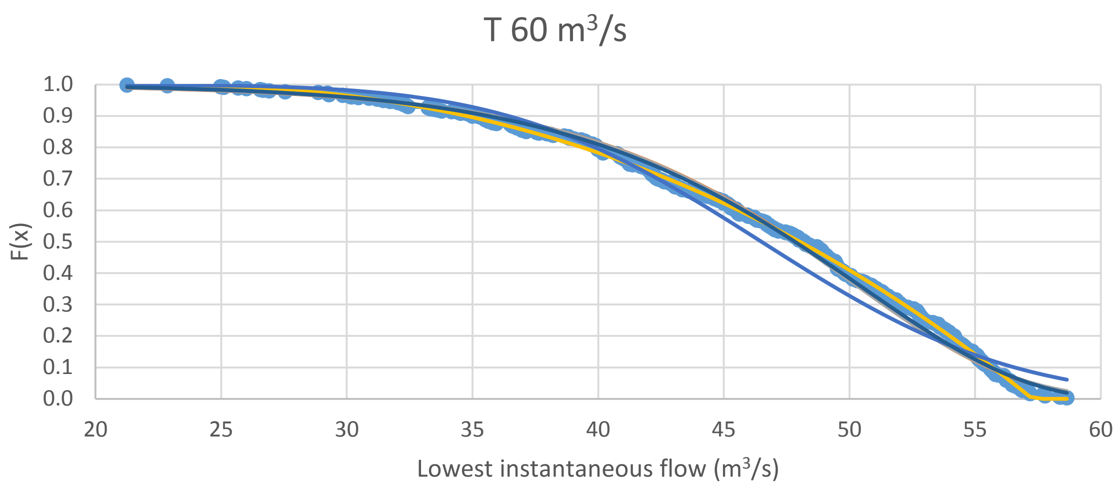


Figure C.2 Probability plots of low flow deficit volumes for various thresholds. T = threshold.


T 35 m³/s, censored series
F(x)
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
24
26
28
30
32
34
36
Lowest instantaneous flow (m³/s)


T 40 m³/s
F(x)
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
20
22
24
26
28
30
32
34
36
38
40
Lowest instantaneous flow (m³/s)


T 60 m³/s
F(x)
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
20
25
30
35
40
45
50
55
60
Lowest instantaneous flow (m³/s)

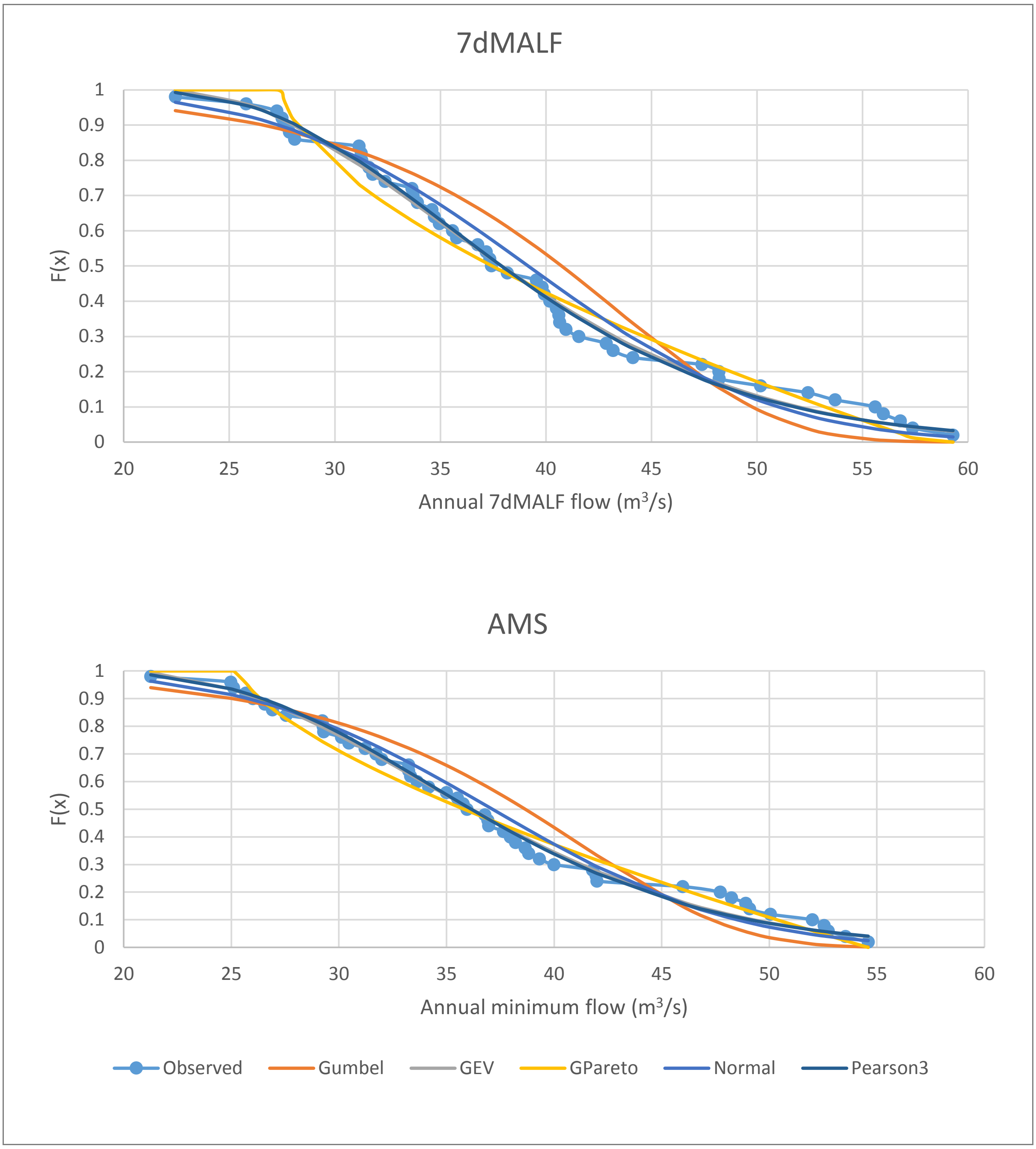


Figure C.3 Probability plots of low flow values for various thresholds. T = threshold.

Table C.6 Trend testing of magnitude series with thresholds 60 m^3/s and 50 m^3/s.

Threshold	Spearman's ρ	Kendall's τ
60 m3/s	0.053 < 0.273	0.036 < 0.266
50 m3/s	0.069 < 0.115	0.064 < 0.079

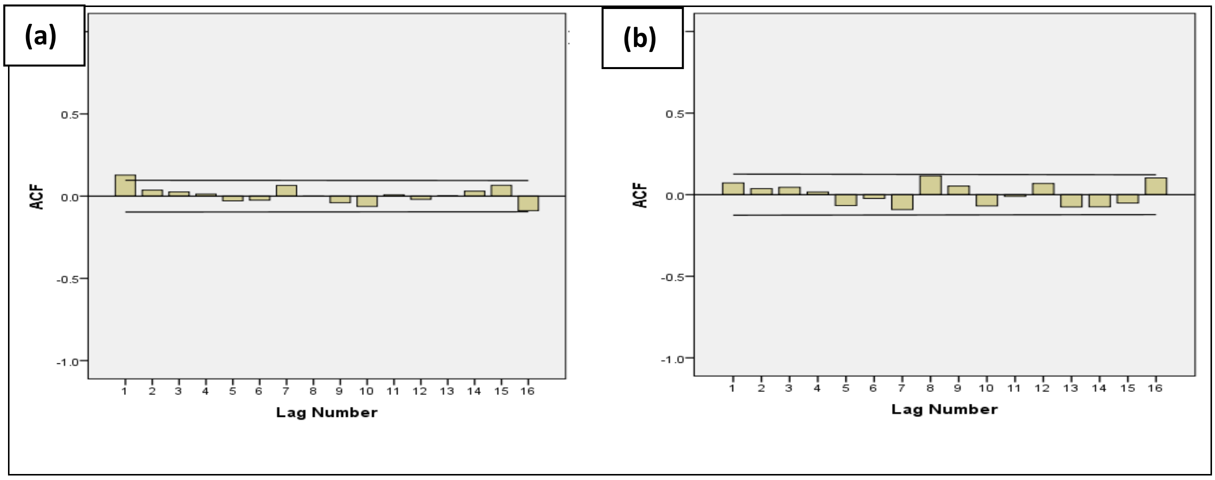


Figure C.4 Autocorrelation plots of series of magnitudes with threshold levels 60 m^3/s (a) and 50 m^3/s (b).

Appendix D

Complementary results for Chapter 8

Table D.1 Trend testing of pre- and post- abstraction series.

Series		Spearman's ρ	Kendall's τ
Pre-abstraction	Low flow	0.031 < 0.452	0.018 < 0.499
	Duration	0.065 < 0.110	0.043 < 0.113
	Deficit	0.044 < 0.283	0.029 < 0.281
Post-abstraction	Low flow	0.041 < 0.291	0.031 < 0.225
	Duration	0.055 < 0.074	0.050 < 0.051
	Deficit	-0.077 > 0.047*	-0.051 > 0.047*

*correlation significant at the 0.05 level.

Table D.2 Summary of parameters obtained by the method of L-moments for pre- and post- abstraction series.

	Lowest value				**Duration**				**Deficit**			
	L_1	L_2	τ_3	τ_4	L_1	L_2	τ_3	τ_4	L_1	L_2	τ_3	τ_4
Pre	51.99	5.572	0.151	0.065	5.063	0.512	0.053	0.078	16105	11058	0.588	0.365
Post	53.12	5.91	0.152	0.041	5.129	0.51	0.051	0.087	15626	11037	0.604	0.372

Table D.3 Summary of goodness of fit statistics for modelling the pre- and post-abstraction series under a 70 m^3/s threshold.

		Pre-abstraction series				**Post- abstraction series**			
		Duration	**Deficit**	**Low flow**	**Magnitude**	**Duration**	**Deficit**	**Low flow**	**Magnitude**
GP	χ^2	10.78	*	6.200	*	2.679	*	4.872	*
	D	0.0263	*	0.024	0.0409	0.031	*	0.003	0.0523
	Filliben	0.994	0.993	0.999	0.996	0.997	0.987	0.999	0.995
GEV	χ^2	*	*	*	*	*	*	*	*
	D	*	*	0.047	*	*	*	*	*
	Filliben	0.99	0.984	0.991	0.989	0.988	0.975	0.986	0.984
P3	χ^2	*	*	*	*	6.588	*	*	*
	D	*	*	0.041	*	*	*	*	*
	Filliben	0.983	0.976	0.993	0.976	0.991	0.983	0.989	0.983
EV1	χ^2	*	*	*	*	*	*	*	*
	D	*	*	0.048	*	*	*	*	*
	Filliben	0.923	0.836	0.988	0.871	0.934	0.838	0.983	0.876
LP3	χ^2	*	22.231	n/a		11.13	*	n/a	
	D	0.0332	0.038			0.0266	0.0503		
	Filliben	0.997	0.995			0.998	0.991		

* indicates a rejected distribution at the 5 % significance level.

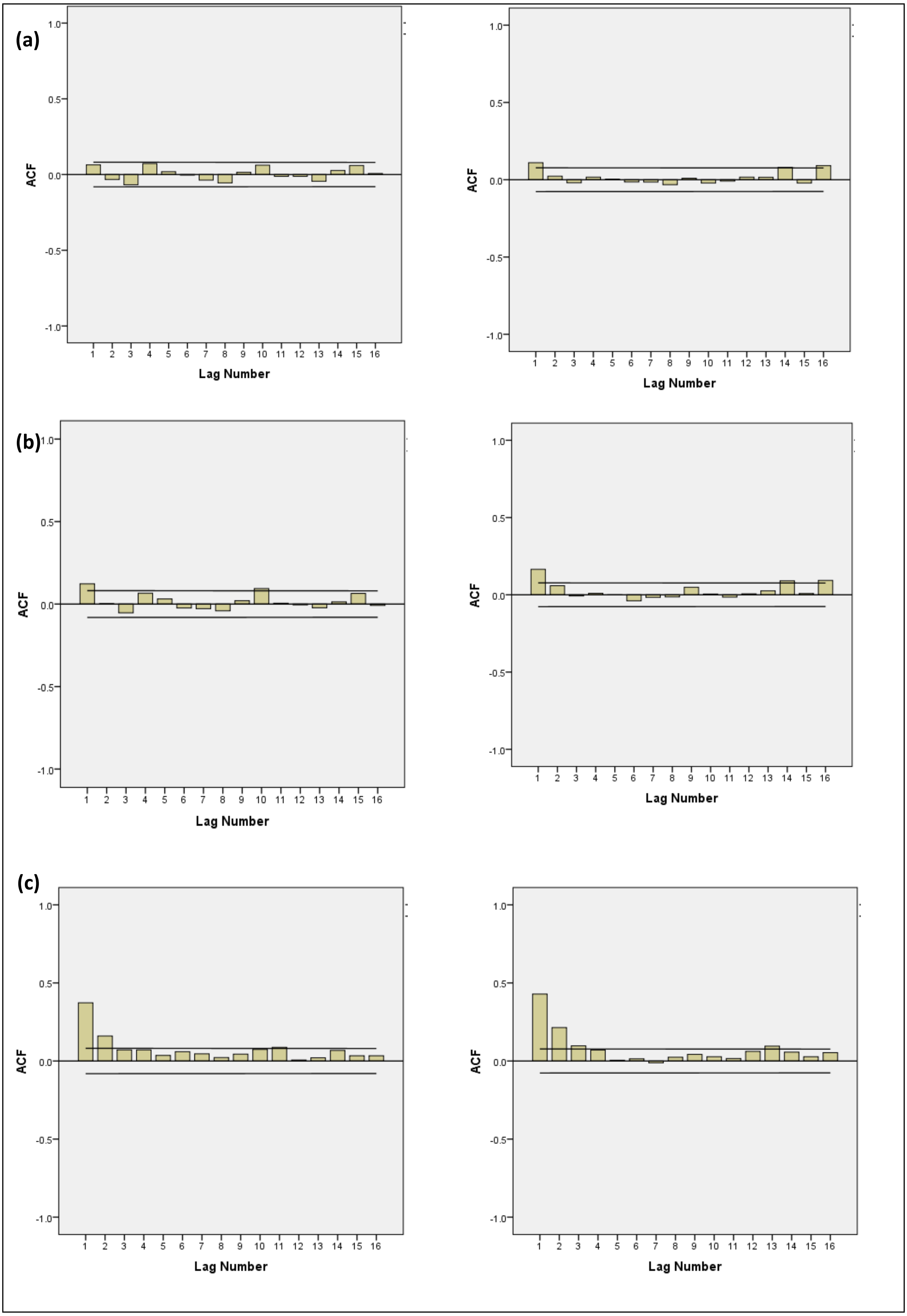


Figure D.1 Autocorrelation plots of pre- and post-abstraction series. (a) duration, (b) deficit and (c) low flow. Left panels are pre-abstraction series and right panels are post-abstraction series.