Journal of Hydrology (NZ) 48 (1): 29-38 2009

Climate variability and the design flood problem

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Abstract
The relationship between the risk of at least one exceedance, the design life, and the cumulative probability of non-exceedance of a given flood peak magnitude constitutes the design flood problem. Parametric and non-parametric risk formulae available for its solution are reviewed. These formulae may be applied to annual maximum flood series which exhibit no trend, periodicity, persistence or shifts. At least 15 series longer than 40 years from South Island, New Zealand do however display shifts in flood regime which are stationary, decades in length and correspond in time of occurrence and duration to shifts in the Interdecadal Pacific Oscillation. In hydraulic design, these shifts may be accommodated by considering a future design life of up to about 35 years in length to be a sequence of stationary flood regimes, each having a specific flood frequency relationship. For a longer design life, predicted changes in rainfall regime derived from climate change modelling will also need to be incorporated in the design. Examples are given illustrating the use of risk formulae, with both stationary annual series and series with shifts in flood regime.

Keywords
Flood estimation, flood frequency, design flood, climate variability, non-stationarity

Introduction
In the design of hydraulic structures and systems which must withstand the effects of rare large floods, it is essential to consider the risk associated with the choice of design capacity. For a particular design it is usually a straightforward exercise to estimate the maximum flow that can be passed without surcharging—for instance, the largest flood a stopbanked river reach can safely contain. The designer then seeks, using past flow records, to determine the risk of the critical flood being exceeded in a given period or design life. The relationship between risk, r, design life, n, and probability of non-exceedance, $F(Q_d)$, where Q_d is the critical flood peak discharge, constitutes the design flood problem. Expressed formally, r can be estimated using

$$r = f\left[n, F(Q_d)\right] \qquad (1)$$

in which f is an unknown function and r increases with n and decreases with Q_d. Specified magnitudes of risk and design life (in years) in Eq.(1) are nowadays determined using a combination of economic, environmental, social and political criteria. The assessment process is complicated and partly subjective, and almost invariably involves community input as a part of a resource consent or planning process conducted under the terms of the Resource Management Act (1991) (Griffiths, 1991).

The flood record normally employed to determine the probability of non-exceedance $F(Q_d)$ in Eq.(1) is a series of annual maximum flood peak discharges measured at a relevant hydrological recording station. At least 20 years of actual or equivalent record is commonly required for useful analysis. This series is compiled, assessed and possibly adjusted until it is stationary and the peak values behave as independent, identically distributed random variables. It is important in the construction of these series to consider whether paleoflood or historical flood information, regional analysis, rainfall-runoff modelling and so on, can be used to supplement, expand or modify the series (Merz and Blöschl, 2009).

In practice, a range of solutions to Eq.(1) is obtained to evaluate appropriate risk and design life and to assist in assessing the effects of errors in the variables. For specific values of r and n, and using the annual flood series described above, Eq.(1) may be solved by both parametric and non-parametric methods involving, for instance, the binomial probability distribution, the distribution of exceedances, and Bayesian statistics. A New Zealand application of some of these methods is given in Griffiths (1989a).

Many New Zealand annual flood series are approximately stationary and show little, if any, evidence of trend, periodicity, shift or persistence. However, about half of 15 South Island flow recording sites, examined by McKerchar and Henderson (2003), with more than 40 years of record show evidence of shifts in flood regime. These shifts last for decades and correlate in duration with phases of the Interdecadal Pacific Oscillation (IPO) (Salinger *et al.*, 2001; Parker *et al.*, 2007). An important question arises here about how to treat changes in flood regime which may occur during a future design life. This life may be short (less than 15 years), medium (15–35 years), or long (35–50 or more years), where the choice of 35 years corresponds to the duration of a resource consent under the Resource Management Act (1991) and 50 years to the life of structures under the new Building Act (2004) (Ministry for the Environment, 2008).

There are at least two approaches for dealing with the effects of the IPO. For short and medium terms, one can partition past and future years into periods, each one corresponding to an IPO phase, and assign a flood regime, $F(Q)$, to each period where Q is annual maximum flood peak discharge. For a long term, $F(Q)$ could be derived using a rainfall-runoff model where changes from the present relevant rainfall regime are forecast from projected changes in local temperature (Ministry for the Environment, 2008); the latter approach is not explored in detail herein.

The main purpose of this paper is to review in part the solution of the design flood problem, with an example, where the existing flood record is stationary; and from this basis present a solution and example where the flood regime in a future design life is affected by a change of phase of the Interdecadal Pacific Oscillation. The aim is to present a rational approach to the solution of the design flood problem under such climate variability.

Theory

Risk formulae

In considering what might occur during the life of an hydraulic structure or system, the worst eventuality is the occurrence of one or more supradesign floods. If the designer seeks to reasonably avoid this outcome, then the value of r in Eq.(1) is the risk of at least one exceedance in n years. Assuming that the relevant record of annual maximum flood peak discharges is stationary and that annual values are independent and identically distributed, then three main formulae for the solution of the design flood problem are available.

Parametric risk formula

The term “risk formula” is used here because it is customary to express Eq.(1) with r as the dependent variable. The probability, P, of exactly s floods or exceedances occurring in n years is given by the binomial distribution

$$P(s) = {}^{n}C_{s}(1-F)^{s}F^{n-s} \tag{2}$$

where, as before, F is the probability of non-exceedance. Since

$$\begin{aligned} r &= P\text{ (at least one exceedance)} \\ &= 1-P\text{ (no exceedance)} \end{aligned} \tag{3}$$

we have using Eq.(2)

$$r = 1 - {}^{n}C_{o}(1-F)^{o}F^{n} = 1 - F^{n} \tag{4}$$

which is the standard risk formula (Reed, 1999). The number of years to the first exceedance of some specified flood, Q_s, follows a geometric distribution with an expected or average value of

$$T(Q_s) = 1/\left[1 - F(Q_s)\right] \tag{5}$$

where T is the return period. Put another way, T is the average time between years containing one or more floods greater than Q_s. Elimination of F between Eqs.(4) and (5) yields

$$r = 1 - \left[1 - (1/T)\right]^{n} \tag{6}$$

an alternative and commonly used version of the standard risk formula. The model used for the frequency-magnitude relationship, $F(Q)$, in New Zealand is either the Extreme Value Type I (EV1) or the General Extreme Value (GEV) distribution fitted to the annual maximum flood peaks by probability-weighted moments (McKerchar and Pearson, 1989).

Non-parametric risk formula

Regardless of the particular form of $F(Q)$, the probability that in n future years the kth of m past floods will be exceeded exactly y times is (Gumbel, 1958)

$$P(y) = \frac{\binom{m}{k}\binom{n}{y}}{\binom{n+m}{k+y}\binom{k}{k+y}} \tag{7}$$

a formula originally derived by Thomas (1948). Note that the first bracketed expression in the numerator is equivalent to $m![k!(m-k)!]$. In Eq.(7), k=1 corresponds to the largest flood in the record. From Eqs.(3) and (7) with $y=0$ we may write

$$r = 1 - \frac{\binom{m}{k}}{\binom{n+m}{k}} \tag{8}$$

which is a nonparametric risk formula.

Bayesian risk formula

A number of distributions are available from which to derive a risk formula. For example, if the number of independent floods occurring randomly in some interval of time is a Poisson variate; and if the parameter of this distribution varies itself according to the conjugate prior Gamma distribution, then the probability of z future exceedances in n years, given w exceedances in the past m years, is, from Benjamin and Cornell (1970)

$$P(z) = \frac{n^{y}m^{w+1}(y+w)!}{(m+n)^{w+y+1}\,y!\,w!} \tag{9}$$

With $y=0$ in Eq.(9) and using Eq.(3) we may write

$$r = 1 - \left(\frac{m}{m+n}\right)^{w+1} \tag{10}$$

a Bayesian risk formula.

Climate variability

As noted above, some annual flood series in New Zealand show clear evidence of shifts in average flood magnitude over an interval but are largely free of trend, periodicity and persistence (serial correlation). Incidentally, examination of partial-duration series shows that these shifts are accompanied by a change in the frequency of large floods as well. This is not detectable, of course, in a series of annual maxima. Where shifts occur, the flood record can be viewed as a series of shifts from one stable flood regime to another.

Following Griffis and Stedinger (2007), one might expect to see the influence of both the IPO and the Southern Oscillation Index (SOI) in a flood record. The IPO exhibits low frequency variability, with shifts that last the order of decades; the SOI operates in a much shorter time frame of less than a couple of years. La Niña does have an effect on seasonal flows in New Zealand (McKerchar *et al.*, 1998) and may influence the magnitude of the annual maximum flood in a given year only (Olsen *et al.*, 1999), although in applications it is prudent to check this by correlating, for instance, annual flood peak magnitude with the seasonal value of the SOI for the season in which the peak occurs. In the longer term, temperature and other changes are predicted which are expected to result in changes in the intensity of rainfalls of a given duration and frequency at a given site (Ministry for the Environment, 2008, appendix 4).

Risk formulae cannot, by definition, be applied to an entire annual flood series containing shifts. Instead, $F(Q)$ or its equivalent can be calculated for each flood regime corresponding to an IPO phase. One or more of these $F(Q)$ distributions can then be used in the solution of Eq.(1), depending on what IPO behaviour is assumed in a future design life. The detail of this approach varies with n as follows.

Short-term design

For $n \leq 15$ yr we assume, in the absence of a reason not to do so, that the present IPO phase will continue along with the current flood regime. The calculation of r does, however, need to incorporate the probability that the current $F(Q)$ will continue for the next $n \leq 15$ yr. If E_1 is the event that Q_d is exceeded at least once in n_1 years and E_2 is the event that the present phase of the IPO continues for n_1 years or more, then from the definition of conditional probability the adjusted risk, r_1 is

$$r_1 = P(E_1 E_2) = P(E_1 / E_2) P(E_2) \qquad (11)$$

Here $P\ (E_1/E_2)$ is $r[n_1,\ F(Q_d)]$ determined from, say, Eq.(4) and $P(E_2)$ has to be estimated from the long-term behaviour of the IPO, assuming for instance that phase changes follow a pure Poisson process so that phase durations are exponentially distributed.

Medium-term design

For $15 < n \leq 35$ yr we consider an IPO Phase (1) of duration at least n_2 years and then a shift to an IPO Phase (2) of duration at least n_3 years. In the solution of Eq.(1) it is assumed here that, for the specified Q_d, both $F(Q_d)_1$ for IPO Phase (1) and $F(Q_d)_2$ for IPO Phase (2) are known. We require the probability of at least one exceedance of Q_d in n_2+n_3 years, that is $r(n_2+n_3, F(Q_d))$. Recognising that Q_d could be exceeded in Phase (1) or Phase (2), or both, let E_3 be the event that Q_d is exceeded at least once in n_2 years, E_4 the event that Phase (1) is of length n_2 years or more, E_5 the event that Q_d is exceeded at least once in n_5 years, and E_6 the event that Phase (2) is of length n_3 years or more. Then, by definition, for events which are not mutually exclusive

$$\begin{aligned} & r(n_2 + n_3, F(Q_d) \qquad (12) \\ & = P\left[E_3 E_4\right] + P\left[E_5 E_6\right] - P\left[E_3 E_{4.} E_5 E_6\right] \end{aligned}$$

Long-term design

For $35 < n \leq 50$ (or more) years, the contribution to climate variability from a predicted increase in atmospheric temperature needs to be incorporated. Most of this increase is attributed to anthropogenic contributions to greenhouse gas emissions (Ministry for the Environment, 2008). Under this scenario, annual flood series should exhibit a trend, at least because warmer air holds more moisture, and this change will alter the magnitude and intensity of rainfalls of a given duration at a given location. The degree and timing of these effects will vary from one catchment to another and may be superimposed on the behaviour of the IPO. One approach to solving Eq.(1) in this situation is to use a weather model such as the Universal Model (Drost *et al.*, 2007) to estimate rainfall intensity, frequency and duration relationships for the design life. This information can then provide the input to a calibrated rainfall-runoff model to forecast *F(Q)* for a specific catchment. Such a forecast is quite speculative, but it has a rational basis, more so perhaps than current allowances made in hydraulic design for the effects of development and land-use change. A New Zealand example of the above general approach is given by Gray *et al.* (2005).

Application

To illustrate the practical application of the theory by example, the design flood problem is solved for two sites: Waimakariri River at Old Highway Bridge and Lake Te Anau at Park HQ. Both the annual flood series on the Waimakariri River and the annual maximum inflow series (where inflows are averaged over three days) for Lake Te Anau (McKerchar and Henderson, 2003, p. 640) exhibit no trend, periodicity, persistence or effects of the SOI. The Lake Te Anau record does, however, show shifts in inflow regime. These two sites were chosen because both have very long, high-quality records by New Zealand standards.

Example 1

The problem is to estimate design flood size for Waimakariri River at Old Highway Bridge (Site 66401, Walter, 2000) when $r=0.3$ and $n=20$ *yr*. As noted previously, there is no evidence in the annual flood series of the effects of climatic variability. Accordingly, this record can be used directly to estimate Q_d by means of the various risk formulae—Eqs.(4), (8) and (10). Figure 1 is a flood frequency plot for the site. Note that the data are positively skewed, with a skewness coefficient of 1.7, and so will not be well modelled by an EV1 distribution: this distribution does however provide a good model of the biennial series

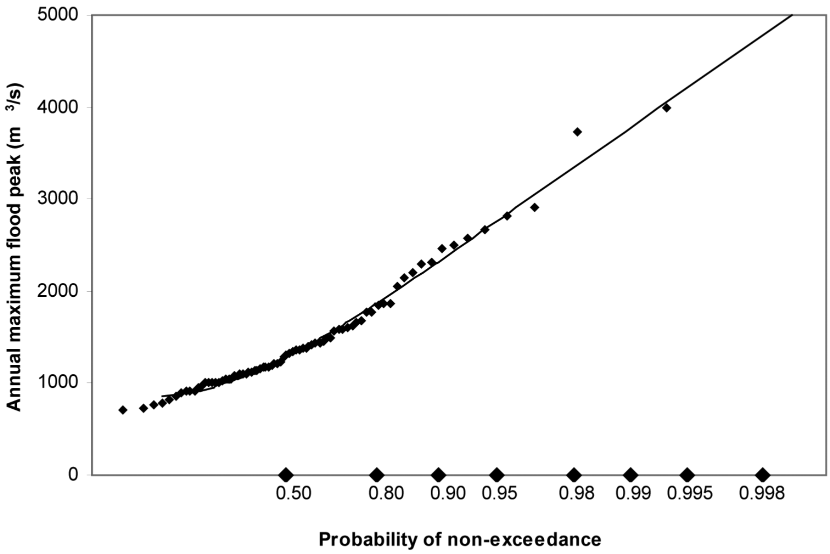

Figure 1 – Annual maximum flood peak discharge versus probability of non-exceedance for the Waimakariri River at Old Highway Bridge (1930–2007).

(McKerchar and Pearson, 1989). Instead, a two-parameter Wakeby distribution is fitted, giving

$$F = 1 - \left\{1 - \left[(Q-u)/x\right]\right\}^{m-1} \qquad (13)$$

in which $u = 819.8$ and $x = 51022$ as estimated by moments, with $m = 78$ yr. For details about this distribution and fitting procedures see Griffiths (1989b).

With $r = 0.3$ and $n = 20$, Eq.(1) yields $F = 0.982$ ($T = 56$ yr) and with this value Eq.(13) yields $Q_d = 3414 \pm 355$ m^3/s. Here the standard error is determined using Griffiths' (1989b) Eq.(50) with $r = 0.3$ and $m = 78$. Another answer is provided by the nonparametric risk formula, Eq.(8). This is solved by trial for nominated integer values of k, as m and n are known. For $k = 1$ in Eq.(8), $r = 0.204$ and from the data for Figure 2, $Q = 3990$ m^3/s, and for $k = 2$, $r = 0.368$ and $Q = 3740$, giving by linear interpolation (using MS Excel or similar software, for instance) for $r = 0.3$ a value of $Q_d = 3844 \pm 410$ m^3/s. Here the standard error is estimated by the method of Wilks (1962, p. 332) using large sample statistics. Finally, using the Bayesian model with $m = 78$, $n = 20$ in Eq.(10), then again by trial for $w = 0$, $r = 0.204$, and for $w = 1$, $r = 0.368$, which is the same as the result from Eq.(8) in this instance. The results for Q_d are not different within a standard error and the parametric estimate of $Q_d = 3414 \pm 355$ m^3/s should be adopted because of the goodness of fit in Figure 1 and because it is obtained using the standard approach. A further reason is that the highest ranked floods used in the nonparametric approach are likely to be the values least well supported by the site's stage versus discharge or rating curve.

Before leaving the Waimakariri annual series it is of interest to note, from Figure 2, that from 1930–1957 there were four large floods or exceedances greater than 2580 m^3/s in 28 years. How likely is this? By definition

$$P\,[\text{at least 4 floods}] = 1 - P\,[\text{0 flood}] - P[1] - P\,[2] - P[3] \qquad (14)$$

And using Eqs.(2) and (14) (with $F = 0.933$ from Eq.(13)) the answer is 0.12, showing that the occurrence is unusual but not highly unlikely. Also from 1995–2005 there were

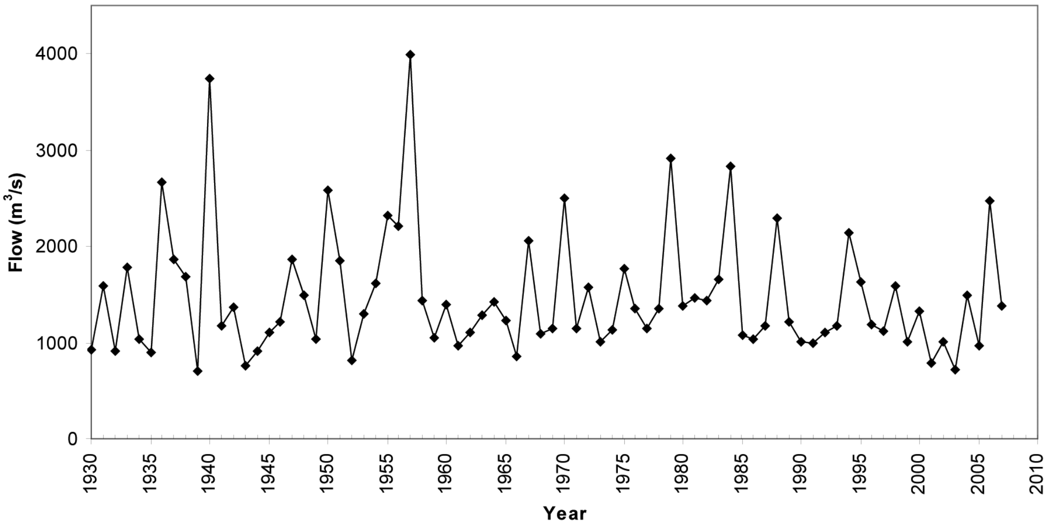

Figure 2 – Time series of annual maximum flood peak discharges for Waimakariri River at Old Highway Bridge (1930–2007).

no floods greater than 1627 m^3/s in 11 years. How likely is this? We can obtain an approximate answer without introducing the details of run theory by employing a result from Cramer (1946) that the expected value of the longest run, $E\left[\ell(\max)\right]$, may be estimated from

$$E\left[\ell(\max)\right] = \ln(m)/\ln(1/F) \qquad (15)$$

With $F = 0.707$ from Eq.(13), and $m = 78$ in Eq.(15), we obtain an expected value of 12.6 years, showing that the observed run length of 11 years is in fact quite likely.

Example 2

The problem is to estimate the risk for Lake Te Anau at Park HQ (Site 79704, Walter, 2000), when $Q_d = 2100$ m^3/s and $n = 35$ yr.

As mentioned above, there is evidence of shifts in flood regime in the Te Anau annual maximum inflow series (Fig. 3). The years in which changes from one regime to another occur are difficult to discern in Figure 3, but are more clearly shown in Figure 4, which exhibits cumulative departures from the mean. From Figure 4 new regimes began approximately in the years 1944, 1977 and 2000. These dates are consistent with shifts in annual rainfall totals for the West Coast of the South Island noted by Salinger *et al.* (2001), and correspond with the phase changes of the IPO (Table 1). If these changes are assumed to occur as a Poisson process, then by definition the duration, *d*, of a phase is an exponential variate. A reasonable model for the durations in Table 1 is given by

$$F_d = 1 - e^{-\lambda d} \qquad (16)$$

as evidenced by a plot, in which F_d is the probability of non-exceedance of a duration and $1/\lambda = 17.4$ yr is the mean. Figure 5 shows the flood frequency data for positive phases (1932–43, 1977–99) and negative phases (1944–76, 2000–07) (Table 1) of the IPO modelled by EV1 distributions fitted by probability-weighted moments.

Now, in a design life of $n = 35$ yr beginning, say, in 2009, it is reasonable to expect a shift in regime from the present which began in 2000 (Fig. 4) and which has an average duration of 17.4 years. For the current regime, *pn*, we have from Figure 5, for $Q_d = 2100$ m^3/s, that $F_{pn} = 0.907$, and for the other regime, *pp*, $F_{pp} = 0.793$. Assuming that the current regime lasts at least 17 years (2000–2017), then from Eq.(16), with $d = 8$ it follows that $F_d(pn) = 0.368$. This implies that after this predicted change in 2017, the next regime will last at least 27 years, as $n = 35$ yr. For the second regime we have from Eq.(16) with $d = 27$ that $F_d(pp) = 0.778$. From Eq.(4)

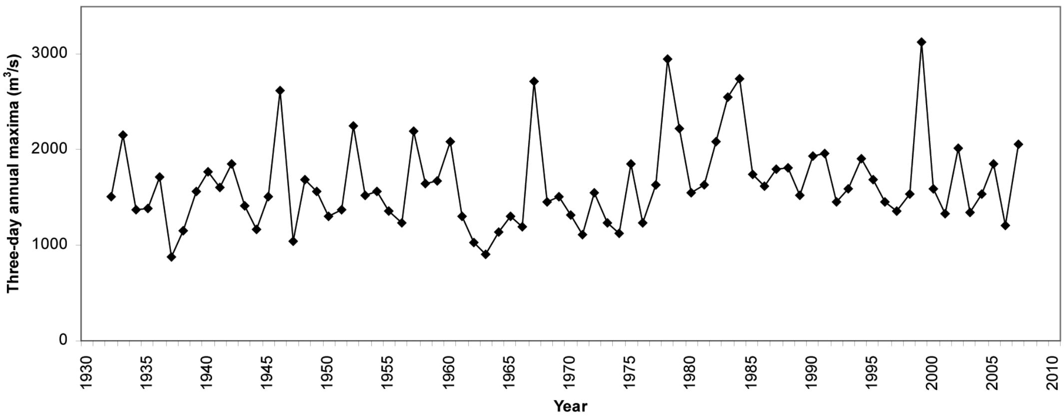

Figure 3 – Time series of 3-day annual maximum inflows for Lake Te Anau at Park HQ (1932–2007).

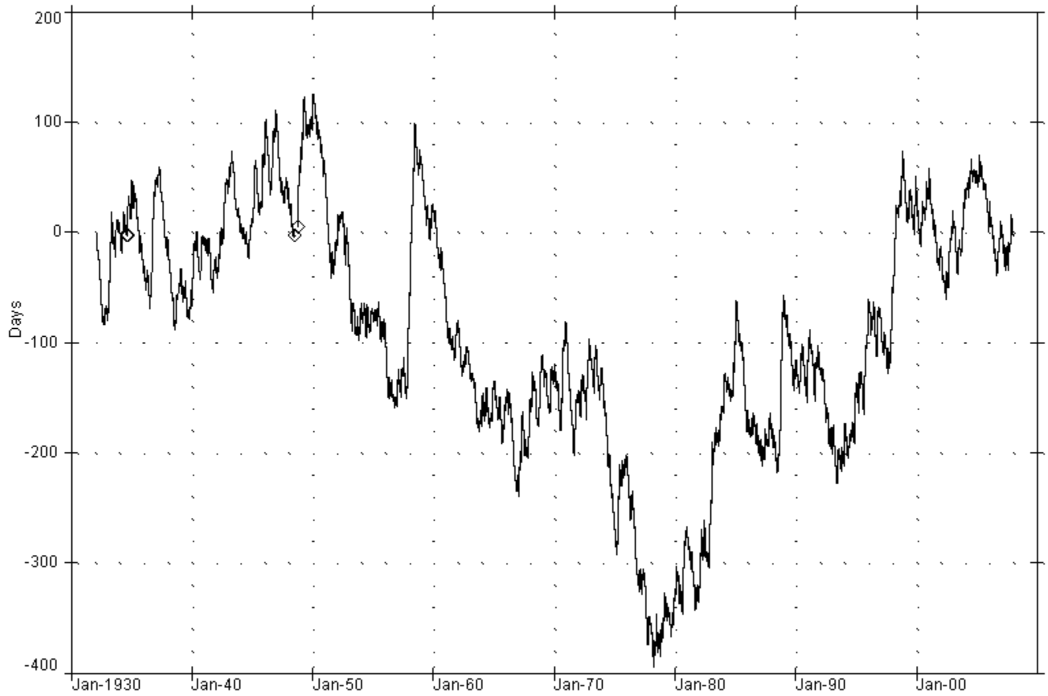

Figure 4 – Time series of cumulative departures (days of mean flow) from the mean of 3-day annual maximum inflows for Lake Te Anau at Park HQ (1932–2007).

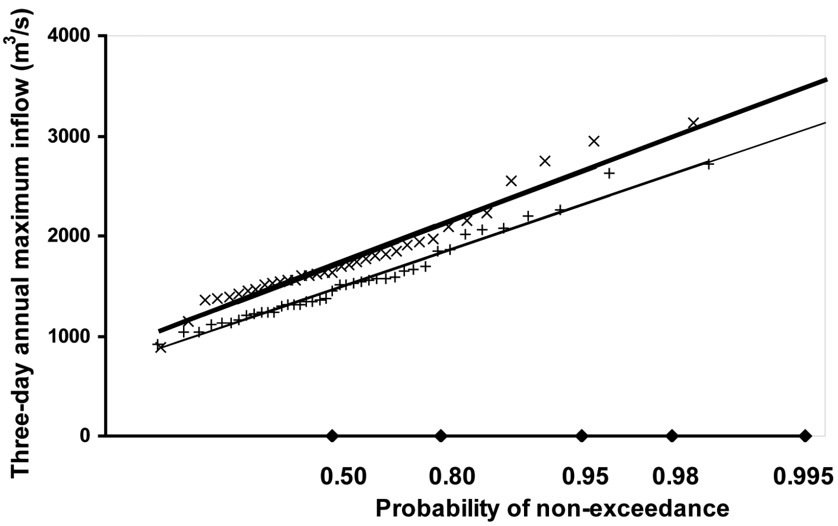

Figure 5 – 3-day annual maximum inflows versus probability of non-exceedance for Lake Te Anau at Park HQ. Upper line applies to negative IPO phases (Table 1) (data from 1944–1976 and 2000–2007). Lower line applies to positive IPO phases (data from 1932–1943 and 1977–1999).

Table 1 – Phase changes of the Interdecadal Pacific Osciliation (1850–2007 incl.) (A. Coleman, Hadley Centre, Metrological Office, United Kingdom, pers.com.)

Year of phase change	Phase sign	Duration of phase (yr)
?	-ve	At least 28
1877	+	10
1887	-	8
1895	+	17
1912	-	12
1924	+	20
1944	-	33
1977	+	22
2000	-	At least 8

with n = 8, 27 yr, we obtain r_{pn} = 0.542 and r_{pp} = 0.998. The final part of the calculation is to obtain a composite value for r, using Eqs.(11) and (12). In Eq.(11), $P[E_2] = 1\text{-}F_d(pn)$ and $P[E_1/E_2] = r_{pn}$. With an appropriate change of subscripts from E_1 to E_3, for example, we can write using Eq.(12).

$$F(n = 35,\ Q_d = 2100) = (0.542 \times 0.632) + (0.998 \times 0.212) - (0.343 \times 0.212) = 0.482$$

that is, the composite risk is 48%. As an independent check, Eq.(8) with m = 41, n = 8 and k = 5 yields r_{pn} = 0.607 and with m = 35, n= 27, k = 7 yields r_{pp} = 0.986. Similarly, Eq.10 with m = 41, n = 8 and w = 4 gives r_{pn} = 0.590, and with m = 35, n = 27, k = 6 gives r_{pp} = 0.982. All the check values confirm the parametric estimates of r_{pn} and r_{pp}.

In this way a composite risk versus Q_d relationship can be compiled for a nominated design life.

Conclusion

The design flood problem can be expressed both parametrically and nonparametrically as a risk formula. These independent expressions provide useful checks in applications.

Fifteen of some of the longer annual flood series (40 year plus) available from the South Island of New Zealand exhibit climatic variability in the form of shifts in regime; but show little or no evidence of trend, periodicity, persistence or effects of the SOI. Each regime is stationary and their time of occurrence and duration correspond with shifts in the IPO.

In the design of hydraulic structures and systems with a design life of up to 35 years or so, shifts in flood regime should, and can, be readily incorporated statistically in the solution of the design flood problem. The design life may be considered as a sequence of stationary flood regimes, each with a specific, variable duration and flood frequency relationship.

For longer design times, the effects of predicted increases in atmospheric temperature on catchment rainfall regime could be incorporated in a calibrated rainfall-runoff model to generate an annual flood series for use in solving the design flood problem.

Acknowledgements

This work was funded through the Foundation for Research Science and Technology Regional

Riskscape Model programme (Contract C05X0409). The Lake Te Anau data are used with permission from Meridian Energy Ltd.

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Manuscript received 10 September 2008; accepted for publication 5 May 2009

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