The rational formula from the runhydrograph

Mohamed Parak and Geoffrey GS Pegram*
Civil Engineering Programme, University of KwaZulu-Natal, Durban, 4041, South Africa

Abstract

The rational formula is possibly the simplest flood estimation technique available using rainfall-runoff relationships. In spite of the many criticisms regarding its over-simplification of the processes of rainfall conversion into runoff, it remains possibly the most widely used method for estimating peak flood flows for urban drainage systems and small (<100 km^2) rural catchments. However, as a result of the criticisms, the formula carries with it many cautions. One such caution regards the determination of the formula's runoff coefficient *c*, which is seen as the main difficulty in the design application of the formula. Mindful of this, it was decided to investigate the calibration of this coefficient, on past flood peak and flood volume pairs for a number of catchments in South Africa. To this end the "data set" of runhydrographs, which describe the characteristic peak and volume discharges of a catchment for a given recurrence interval, was used to calibrate the coefficients for selected catchments and to explore the assumptions underpinning this simple model. This article describes the methods employed in achieving this as well as a discussion of the results.

Keywords: design flood estimation, probabilistic rational formula, runhydrograph, calibration of runoff coefficients

Introduction

The rational formula is perhaps the best known and most widely used method for the determination of peak flood flows from rainfall events. It has survived numerous criticisms regarding its over-simplification of the complex hydrological processes of flood production but nonetheless is possibly the most favoured method used by practitioners for peak flood estimation. The rational formula owes its popularity to the fact that it is easy to understand and simple to use. The peak flood flow due to a rainfall event on a catchment, determined from the rational formula, is expressed (in SI units) as:

$$Q_{RF} = ciA/3.6 \qquad (1)$$

where:

Q_{RF} is the flood peak in m^3/s
c is the runoff coefficient, which is (in the traditional deterministic approach) defined as the proportion of precipitation that contributes to runoff
i is the storm rainfall intensity in mm/h
A is the catchment area in km^2.

The criticisms concerning the rational formula in the above form are not unfounded and the use of this method carries valid cautions that are based on the following assumptions built into the formula (which are not always explicit in its presentation):

- The maximum rate of runoff from a catchment is achieved when the duration of rainfall is equal to the time of concentration (T_c) of the catchment, which is defined as the time taken for the outflow from a catchment to reach near equilibrium due to a storm uniformly spread in space and time,
- The spatial and temporal characteristics of rainfall are consequently ignored and the storm rainfall, as input into the formula, is assumed to be a rectangular pulse of duration T_c, deposited in lumped form on the catchment (i.e. there is no routing component implicit in the formula).

As a consequence, the rational formula was previously limited in its application to small catchments (<15 km^2 in South Africa (HRU, 1972)) and was only to be used as a check method (it was not to be used in isolation). It was further noted that sound engineering experience and judgment was required for its use. However, work that has since been done, locally by Alexander (2002) and Pegram (2003), and abroad in Australia (Institute of Engineers Australia, 1987), has shown that these cautions were too timid and its use may well be extended beyond small catchments.

For the estimation of design floods, a probabilistic approach to the rational formula is needed, where the variables *c* and *i* (the runoff coefficient and rainfall intensity respectively) of the formula are associated with a probability of exceedance. A probabilistic approach is different to a deterministic approach (which is the form shown in Eq. (1)), as it does not involve the representation of a historic event. As opposed to the latter case, no unique combination of rainfall and catchment conditions (such as storm patterns, ground cover conditions, antecedent moisture conditions, etc) exist to reproduce the design flood. In a probabilistic approach, the rational formula is used to estimate, for a given probability of exceedance, the magnitude of the peak discharge from a site; this peak would be equivalent to a discharge estimated from a frequency analysis of flood records if a long and representative record were available at that site.

Pilgrim and Cordery (1993) stated that the design situation is exactly suited to the probabilistic approach of the rational formula and has little similarity with the deterministic rational formula, so that the criticisms associated with the deterministic approach are not necessarily valid for the probabilistic design case. Alexander (1990) stated that as the catchment size increases the value

* To whom all correspondence should be addressed.
☎ +2731 260-3057; fax: +2731 260-1411;
e-mail: pegram@ukzn.ac.za
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of c becomes more probabilistic than deterministic in its derivation. The probabilistic approach to the rational formula has the same form as Eq. (1) but is defined more specifically as:

$$Q_{(T)} = c_{(T)} i_{(T_c,T)} A / 3.6 \tag{2}$$

where:

$Q_{(T)}$ is the flood peak in m^3/s of recurrence interval (RI) T-years

$c_{(T)}$ is the runoff coefficient for a T-year event

$i_{(Tc,T)}$ is the T-year storm rainfall intensity in mm/h of duration equal to the time of concentration T_c (h) of the catchment

A is the catchment area in km^2.

In this approach, the value of $c_{(T)}$ purports to transform a T-year design storm $i_{(Tc,T)}$, of duration T_c, into a T-year flood peak $Q_{(T)}$ for a catchment of area A. The variable $i_{(Tc,T)}$ can be determined, for a particular site, from suitable intensity-duration-frequency (IDF) relationships of design storms. However, the estimation of the runoff coefficient $c_{(T)}$ remains the main source of uncertainty in the probabilistic application of the rational formula. It is the least precise variable of the rational formula, in spite of it being bounded in the interval (0; 1), and suggests that a fixed ratio of peak runoff rate to rainfall rate exists for the site, which in reality is not the case (Chow et al., 1988: 497). It is this characteristic (the estimation of the design runoff coefficient) of the rational formula that forms the main focus of this article. To this end, this article investigates the calibration of the runoff coefficient, on past flood peak and flood volume pairs for a number of catchments in South Africa, to assist with its determination. The calibration of runoff coefficients on past floods is also the practice that was adopted in Australia (Institute of Engineers Australia, 1987). It was shown in Australia that the use of calibrated coefficients in a probabilistic approach to the rational formula could consistently provide flood estimates for catchments up to $250km^2$. In this research, the "data set" of runhydrographs produced by Hiemstra and Francis (1979) was used to calibrate the coefficients in order to investigate this for the selected catchments.

In South African practice, the idea of calibrating the rational formula's runoff coefficient is not new. Alexander (2002) proposed a new standardised regional flood estimation technique called the standard design flood (SDF). This method is essentially a probabilistic approach to the rational formula, as advocated by Alexander (1990), utilising calibrated runoff coefficients. The SDF method is based on the calibration of the runoff coefficient against design floods determined from a frequency analysis, using the LOG-Pearson-III (LPIII) distribution, of recorded events from a number of catchments in South Africa. According to Alexander (2002), the SDF can be applied to all sizes of catchments in South Africa, ranging in size from 10 km^2 to 40 000 km^2. Alexander has also suggested a standard design hydrograph for the SDF with a fixed triangular shape that has a rising limb equal to the time of concentration of the catchment T_C and a falling limb equal to $2T_C$, i.e. an effective time base-length of $3T_C$. This idealised hydrograph is the same as that proposed by Rooseboom et al. (1981) where, in this instance, it is noted that the runoff volume is greater than the proportionate part of the storm rainfall that runs off during the time of concentration. In an independent test, the average ratio of Alexander's 50-year SDF flood peak to the 50-year LPIII flood peak was found to be approximately 210% (Görgens, 2002). Alexander's method was designed to be purposefully conservative and he states that the over-estimates fall within the range of uncertainties associated

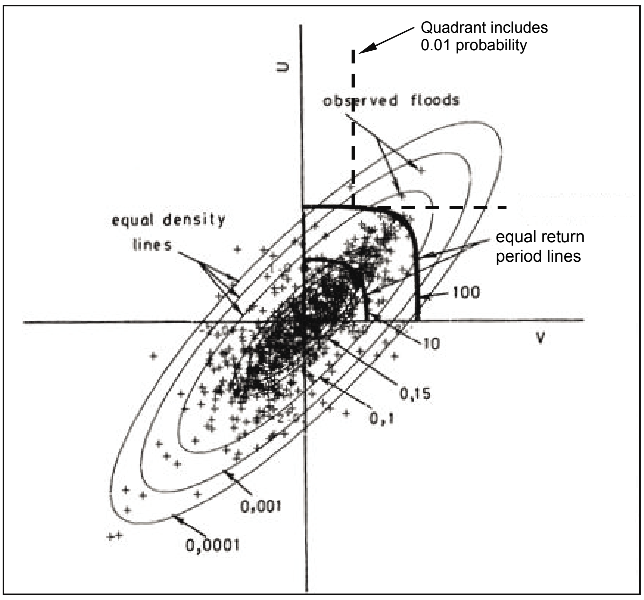

Figure 1

A standard bivariate normal probability density function, with a cross correlation coefficient of 0.85, plotted with log-transformed observed flood peak-volume pairs in probability space (from Hiemstra and Francis, 1979: 14). The bold lines in the positive quadrant are the 10- and 100-year return period joint-exceedance contours. The dashed lines include a quadrant to the upper right which, on average, will include 1% of the observations.

within all design flood procedures. However, Görgens (2002) states that although the cost and implications associated with a conscious over-design in terms of a bridge/culvert is relatively minor, by contrast it is not acceptable for dam spillway design, where the cost of the spillway is a significant component of the total dam cost. An average over-estimate of 200% might render some projects infeasible. As such, Görgens recommends that the SDF should be seen as a conservative approach similar to that of the regional maximum flood (RMF) method.

Conscious of this, it was thought that where this investigation would add value would be in the calibration of the runoff coefficient on past flood peak and volume pairs, as offered by the runhydrograph method. Thus, it was hoped that this would yield coefficients that could, in a design situation, describe a complete design flood hydrograph (peak, volume and time base-length). The following sections describe and explain the theory behind the runhydrograph method, the methods employed in this investigation and the results achieved.

The runhydrograph

The runhydrograph method (Hiemstra and Francis, 1979) summarises, for a given catchment, the family of characteristic peak and volume discharges for a given recurrence interval. These hydrographs were based on the frequency analyses of all rare hydrographs (which were carefully screened for reliability) in a continuous stream flow record and, as such, are independent of rainfall input and catchment characteristics. This set of statistics was thus a valid data set against which to calibrate the runoff coefficient towards a probabilistic approach of the rational formula.

The runhydrograph method was developed by Hiemstra and Francis (1979) (in the sequel referred to as H&F) and was

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based on earlier work by Hiemstra (1972; 1973; and 1974), Hiemstra et al. (1976) and Francis (1979). It is based on the joint probability analyses of same-event flood peak and flood volume pairs of recorded data from 43 catchments around South Africa (see Table A1 in the Appendix). H&F discovered that the natural logarithms of the flood peak and its corresponding volume were approximately normally distributed and well correlated, with a cross-correlation coefficient with mean 0.78 and standard deviation 0.12 (a relatively narrow range) whose mode is 0.85. Fig. 1 shows the natural logarithms of the recorded flood peak and volume pairs plotted together with the contours of equal probability density of a standardised bivariate normal probability density function (with a cross-correlation coefficient of 0.85). Also shown in Fig. 1 (in the positive quadrant) are 10- and 100-year return period exceedance probability contours (bold lines). The dashed lines intersecting on the 100-year exceedance contour define an area in the plane whose probability density integrates to 0.01. Thus, on average, 1% of the observations will lie within this area, and within other areas defined similarly on the 100-year contour.

The contours describe the joint probability of flood peak and flood volume exceedance and are able to produce "families" of hydrographs (peak-volume pairs) of equal probability of jointly being exceeded, but of varying shape. These families can range from the marginal peak (associated with any volume), to the "most likely" joint peak and volume pair through to the marginal volume, each with an equal probability of joint exceedance. However, it can be seen from Fig. 1 that the plotted peak-volume pairs cluster around the 45° line in an elliptical shape. If the cross-correlation coefficient approaches unity, the minor axis of the ellipse reduces to zero. Thus, although more than one combination of a peak-volume pair exists that has the same probability of jointly being exceeded, the most likely (modal) pair will be found at the intersection of the 45° line on the exceedance contour, the point where the probability density is highest.

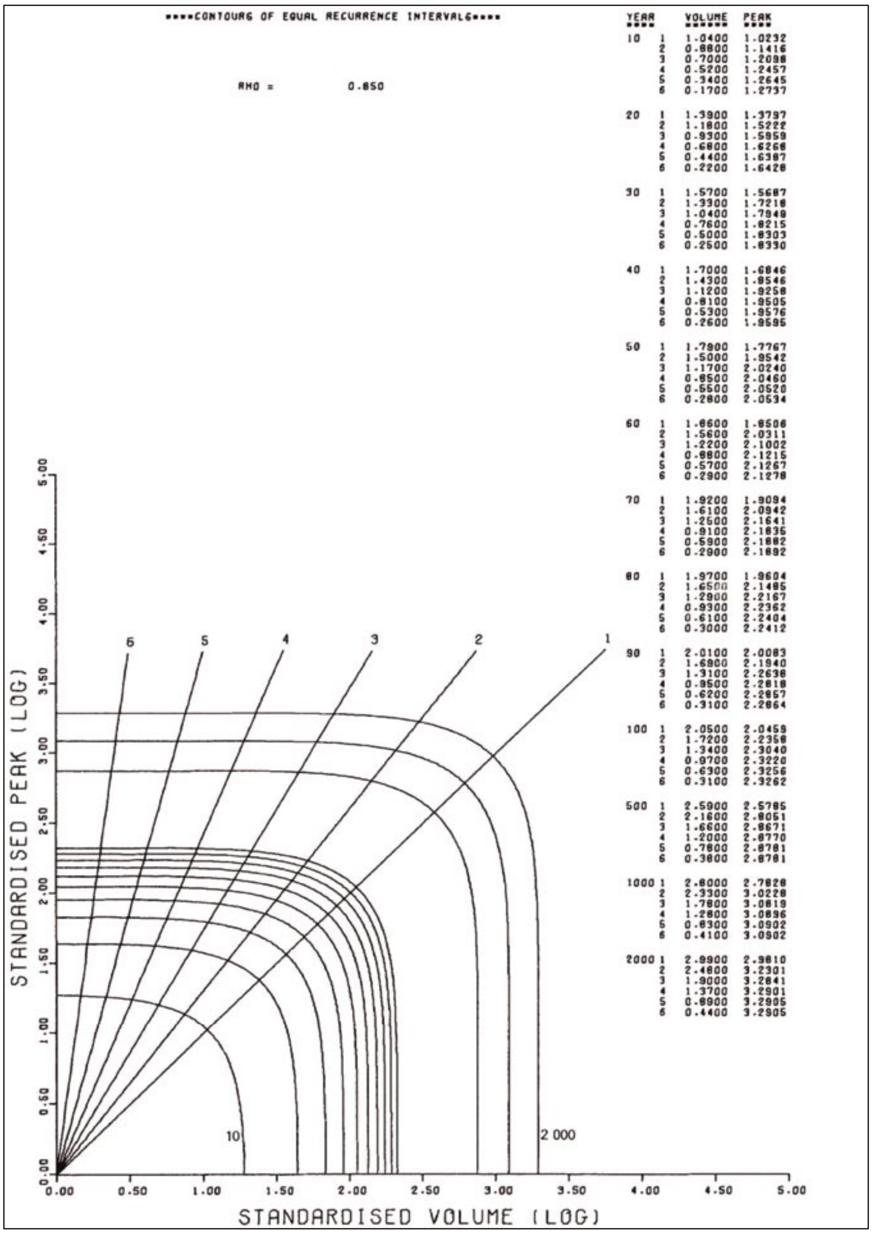

Figure 2
Joint flood peak and flood volume exceedance contours, in probability space for a peak-volume cross-correlation coefficient of 0.85 (from Hiemstra and Francis, 1979)

Figure 2 shows the application of the runhydrograph method for design flood peak and volume estimation for a cross-correlation coefficient of 0.85. The listed numbers on the top right of Fig. 3 are the standardised ordinates of the peak-volume exceedance contours for the selected recurrence intervals. They describe the joint exceedance of the most likely peak-volume pair (corresponding to line #1) through to the exceedance of the marginal peak (corresponding to the vertical axis to the left of line #6). It is unlikely that a peak-volume pair will occur on *lines 4*, *5* and *6* for this relatively high correlation, and thus for the purposes of this investigation the modal peak-volume pair was chosen in order to limit the number of variables.

In passing, it is interesting to note that this idea of describing hydrographs with a joint probability of exceedance of peak and volume, has surfaced again more recently to be exploited in the evaluation of dam safety (De Michele et al., 2005).

Method and results

The methods employed in this investigation were typical of those used in the derivation of a probabilistic rational formula utilising calibrated coefficients, of which an explanation follows which is adapted from Pilgrim and Cordery (1993) (quotes appearing in italics):

- *Where a set of long and reliable record of flood data from a particular catchment exists, a frequency analysis should be carried out on the observed data to determine design values of flood peaks for a range of recurrence intervals.* In this study, flood peak and volume pairs (Q_T in m^3/s and V_T in m^3 respectively) for the 'most likely' runhydrograph was computed for each of the selected catchments for RIs of 10-, 20-, 50-, 100- and 200-years. These appear in Tables A2 to A6 in Appendix A. As a result of this, values of B, the time base-length of the triangular approximated hydrographs, were also computed.

- *A design formula for the calculation of time of concentration T_c must be selected and used consistently throughout the derivation and use of this method.* In this study the Kirpich (1940) formula was used, following the lead of Petras and Du Plessis (1987):

$$T_C = 0.0633[L^2 / S]^{0.385} \tag{3}$$

  where:
  T_c is the catchment's time of concentration (in hours)
  L is the length (in km) of the longest water course
  S is the slope of the longest water course.

- *Design rainfall intensities, $i_{(Tc,T)}$, for the corresponding time of concentration of the catchment and recurrence interval should be determined from a suitable Intensity-Duration-Frequency (IDF) database.* These were determined from Smithers and Schulze's (2002) design rainfall data-base for South Africa. These data appear in the same tables in the **Appendix**. (A computer program with a graphical user interface has been developed to obtain design rainfall depths for any location in South Africa from this database. The software may be downloaded from the following website: http://www.beeh.unp.ac.za/hydrorisk/ and follow the "Design Rainfall" option).

- *From these data, values of $c_{(T)}$ can be back calculated by the following equation (where the variables are as defined for Eq. (2))*:

$$c_{(T)} = \frac{3.6 \cdot Q_T}{i_{T_c,T} \cdot A} \tag{4}$$

  This data also appears in the same tables in the **Appendix.**

- *These calibrated values of $c_{(T)}$ can then be regressed on any physical characteristic of the catchment.* In order to validate th e calibrated coefficients at untested sites, regional parameters with which to relate $c_{(T)}$ with RI were sought. However, it was noted by Pilgrim and Cordery (1993) that the probabilistic runoff coefficients determined for Australia did not show much sensitivity to physical characteristics of a catchment.

It is important to note that the values of $c_{(T)}$ obtained in this manner are conditioned on the use of a consistent formula for the calculation of T_c and a consistent database for the derivation of the IDF rainfall relationships. A detailed explanation of each of the steps listed above and the results of each exercise are given in the following subsections.

The streamflow database

The 43 catchments used by H&F in their study were based on the Department of Water Affairs and Forestry's drainage region delineations. These and their derived statistics are listed in Table A1 in the **Appendix**. As a point of departure, runhydrograph data from H&F were combined with catchment parameters from Petras and Du Plessis (1987), namely area (A) and time of concentration (T_c - based on Kirpich's formula). The number of catchments from the H&F database, for which T_c values were available from the Petras and Du Plessis catalogue, reduced the number of available catchments for calibration to 29. These are listed in Table A2 in the **Appendix** and formed the core data set on which the rational formula calculations were performed.

The rainfall database

For each of the 29 catchments, a number of locations (depending on the size of the catchment) along the main watercourse within the catchment were chosen for which design rainfall estimates were obtained from Smithers and Schulze (2002). The output from this rainfall database provides point rainfall depths (in mm) for durations ranging from 5 minutes to 7 days and for return periods ranging from 2 to 200 years at a spatial resolution of 1 arc minute in South Africa. The mean depth for each catchment was computed and thereafter the intensity, duration and frequency (IDF) relationships were computed by fitting a simple power-law function of storm duration to the mean rainfall depths. For the selected recurrence intervals, these took the form of: P (rainfall depth in mm) = ad^b and i (rainfall intensity in mm/hr) = ad^{-c}, where d is the storm duration in hours and a, b and c (which equals b-1) are the fitted power-law parameters. The mean intensity, corresponding to the time of concentration T_c, was calculated from the IDF relationships for the 10-, 20-, 50-, 100- and 200-year recurrence intervals for each catchment. The parameters fitted to the rainfall duration, for the selected recurrence intervals, are listed in the Appendix (Tables A2 to A8). It was found that rainfall depth scaled, on average, to the power of *0.238* of rainfall duration and thus rainfall intensity to the power of *-0.762* of rainfall duration with a standard deviation of 0.0419.

Area reduction factors (ARFs) were not used in this study to scale the point rainfall depths into average depths over the catchment. Instead simple averages of a few representative points along the longest watercourse within the catchment, determined from the Smithers and Schulze (2002) database, were used to account for the variation in precipitation with position and altitude for large catchments. ARFs were deemed not necessary based on the findings of Pegram (2003), of which a summary is presented here. He investigated the scaling properties of rainfall in South Africa and found that they could be expressed as a function of three factors: the median one-day rainfall (which is a function of location), a function of return period (the reduced variate of the general extreme value (GEV) distribution) and a function of duration. He used this finding to modify the intensity expression of the rational formula. The storm duration used by Pegram was the catchment's time of concentration T_c, as in this study, from the Kirpich (1940) formula. When this duration T_c was plotted against catchment area, it was found that the points clustered about a curve to which a power-law relationship could be fitted. This was superimposed on the area reduction factor (ARF) diagram, published in the *Flood Studies Report* (FSR, 1975). He found that the *Area* ~ T_c curve yielded an almost constant ARF value of 87% across the FSR curve. The implication of this is that, as long as the precipitation intensity used in the rational formula corresponds to the time of concentration of the catchment, the point rainfall is automatically scaled by a constant ARF. It is likely

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that the FSR's ARF curves over-estimate the relationship in South Africa, but the degree is likely to be a matter of climate (Pegram and Parak, 2004), so it is also likely that the scaling behaviour will be maintained. However, the reduction factor would automatically be absorbed into the fitted $c_{(T)}$-values. The first thing to note then is that because *c* is explicitly a function of T_c, it is therefore implicitly independent of the ARF.

Calibration of the runoff coefficients ($c_{(T)}$)

The next thing to explore was the dependency of *c* on the flood regime of catchments of various sizes and locations. The first task was to relate *c* to the peaks of each catchment for varying recurrence interval, *T*. *c*-values were fitted to the flood peak of the calculated modal runhydrograph at each site, using the parameters for that site as estimated by H&F.

The summary of results from the calibration of the runoff coefficients is shown below in Table 1. They are the 10-, 20-, 50-, 100- and 200-year runoff coefficients for each of the 29 catchments that formed the core data set. These data and results are drawn from the **Appendix** (Tables A2 to A6).

Coefficients from 6 of the 29 catchments (marked with an asterisk in Table 1) produced results that did not increase in magnitude with recurrence interval. As mentioned in Alexander (1990), an increase in *c* with recurrence interval is necessary to accommodate the known effects which also increase with rainfall intensity but are not accounted for in the formula's calculation process. The main effect, requiring this increase of *c* with recurrence interval, is that the catchment is likely to be more saturated at the start of a storm with a longer recurrence interval (Rooseboom et al., 1981). This initial saturation caused by pre-event rainfall is the main reason why one can expect to obtain a higher percentage runoff with an increase in the recurrence interval of an event. Alexander (2002) states that in many of the destructive events observed, severe rainfall events were often preceded by above-normal seasonal rainfall.

The calibrated values of *c* (values of $c_{(T)}$ for all 29 catchments) were coaxially plotted with *c*-values from Chow et al. (1988: 498) against recurrence interval. This relationship is shown in Fig. 3, where the coefficients from Chow et al. (1988) correspond to the "flat" slopes type (i.e. for ground slopes between 0 and 2%, since all the test catchments in this calibration exercise had slopes of less than 2%) and are for the three "undeveloped" (rural) coverage types (i.e. cultivated land, pasture/range and forest/woodland). These values are shown in Table 2 and were determined for small rural catchments (i.e. less than 100 km²) of Austin, Texas (USA).

TABLE 1
The results of the calibration of the *c*-coefficient of the rational formula on flood peak and flood volume pairs from Hiemstra and Francis (1979)

Num.	Station	River	Latitude (degrees decimal)	Longitude (degrees decimal)	Catchment area (km²)	Time of concentration T_c (hours)	Calibrated *c*-coefficients				
							10-year	20-year	50-year	100-year	200-year
1	A2M03	Hex	25.77	27.28	494	6.4	0.301	0.303	0.304	0.305	0.306
2	A2M12	Krokodil	25.82	27.92	2 586	18	0.089	0.093	0.095	0.097	0.098
3	A3M01	Klein Marico	25.53	26.10	1 002	8.7	0.084	0.092	0.104	0.113	0.123
4	B2M01	Bronkhorstspruit	25.80	28.77	1 585	18.1	0.210	0.228	0.244	0.254	0.262
5	B4M03	Steelpoort	25.02	29.53	2 271	19.6	0.091	0.102	0.112	0.125	0.135
6*	B7M04	Klaserie	24.55	31.03	130	3.7	0.234	0.233	0.214	0.227	0.224
7	C1M01	Vaal	26.95	29.27	8 254	74	0.396	0.419	0.444	0.460	0.476
8	C4M01	Groot Vet	28.48	26.67	5 504	34	0.368	0.386	0.409	0.425	0.442
9*	C4M02	Vet	27.85	25.90	17 550	111	0.179	0.175	0.170	0.167	0.164
10	C5M03	Modder	29.17	26.58	1 650	18.3	0.419	0.440	0.458	0.469	0.479
11	C5M04	Modder	28.85	26.18	5 012	38	0.528	0.592	0.660	0.706	0.749
12	C5M12	Riet	29.65	25.98	2 383	23	0.218	0.235	0.252	0.264	0.274
13	C5M15	Modder	28.80	26.10	6 545	43	0.280	0.302	0.325	0.341	0.355
14	C7M01	Renoster	27.27	27.18	5 255	57	0.236	0.300	0.379	0.438	0.498
15	D1M05	Oranje	30.03	28.50	10 891	60	0.261	0.266	0.270	0.272	0.274
16*	D5M01	Renoster	31.65	20.62	2 129	27	0.263	0.264	0.264	0.264	0.264
17*	D5M04	Sak	31.65	21.77	5 799	28	0.130	0.128	0.125	0.123	0.121
18	E2M02	Doring	32.50	19.53	5 778	30	0.389	0.420	0.459	0.487	0.516
19	H1M06	Bree	33.42	19.27	754	7.6	0.454	0.457	0.461	0.464	0.468
20*	H1M07	Wit	33.57	19.15	83	2.4	0.814	0.800	0.790	0.787	0.786
21	H7M04	Huis	33.92	20.72	26	2.3	0.278	0.307	0.336	0.353	0.368
22	J2M03	Gamka	33.53	21.65	17 941	42	0.076	0.082	0.090	0.095	0.099
23	J3M04	Olifants	33.48	23.03	4 330	23	0.163	0.180	0.194	0.200	0.205
24	Q1M01	Groot Vis	31.90	25.48	9 150	18	0.089	0.097	0.108	0.116	0.124
25	Q9M10	Groot Vis	33.22	26.87	29 376	108	0.176	0.227	0.282	0.318	0.349
26	Q9M12	Groot Vis	33.10	26.45	23 041	85	0.113	0.133	0.158	0.178	0.198
27*	T3M02	Kinira	30.48	28.62	2 100	26	0.186	0.172	0.156	0.145	0.135
28	W4A03	Pongola	27.42	31.52	5 843	31	0.267	0.278	0.284	0.285	0.284
29	W5M05	Hlelo	26.83	30.73	751	17.8	0.177	0.193	0.212	0.225	0.237

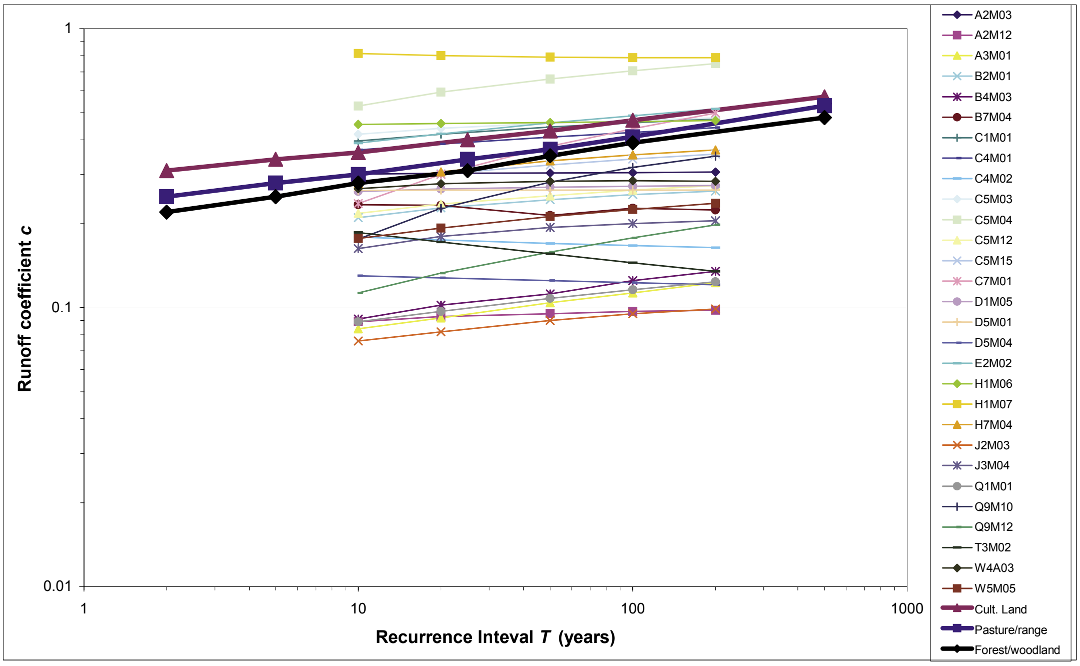


Figure 3
A comparison of the runoff coefficients c from Chow et al. (1988: 498) with those calibrated in this study $c_{(T)}$. The c-values plotted from Chow et al. are shown in thick bold lines and extend from the 2- to 500-year recurrence intervals.

It is evident from Fig. 3 that the $c_{(T)}$-values obtained from this exercise are spread around those of Chow et al. (1988) but are generally lower in magnitude. The $c_{(T)}$-values obtained from this exercise range from 0.084 to 0.786, while the values from Chow et al. are between 0.28 and 0.57 (for the recurrence interval range of 10- to 200-years). However, the scatter associated with the latter data set is not known and hence not shown, so it is conjectured that they are curves fitted to the high side of the original data.

TABLE 2
Runoff coefficients for use in the rational method for undeveloped (rural) regions in Austin, Texas in the USA (from Chow et al., 1988: 498)

	Runoff coefficients c							
Character of surface	**2-year**	**5-year**	**10-year**	**25-year**	**50-year**	**100-year**	**200-year (inter-polated)**	**500-year**
Undeveloped								
Cultivated land								
Flat, 0 - 2%	0.31	0.34	0.36	0.40	0.43	0.47	0.51	0.57
Average, 2 - 7%	0.35	0.38	0.41	0.44	0.48	0.51	0.55	0.60
Steep, >7%	0.39	0.42	0.44	0.48	0.51	0.54	0.57	0.61
Pasture/range								
Flat, 0 - 2%	0.25	0.28	0.3	0.34	0.37	0.41	0.46	0.53
Average, 2 - 7%	0.33	0.36	0.38	0.42	0.45	0.49	0.52	0.58
Steep, >7%	0.37	0.4	0.42	0.46	0.49	0.53	0.56	0.60
Forest/woodlands								
Flat, 0 - 2%	0.22	0.25	0.28	0.31	0.35	0.39	0.42	0.48
Average, 2 - 7%	0.31	0.34	0.36	0.4	0.43	0.47	0.50	0.56
Steep, >7%	0.35	0.39	0.41	0.45	0.48	0.52	0.54	0.58

Hydrograph time base-length *B*

The use of flood peak and volume pairs for calibration in this investigation (from the runhydrograph method of H&F (1979)) was thought to have the added advantage in that complete design flood hydrographs could be calculated from these calibrated coefficients. From the flood database computed for the calibration exercise, hydrograph time base-lengths B for each RI were determined for each catchment. Out of interest, they were then expressed as ratios to the catchment's time of concentration T_c for the respective recurrence intervals (which, in terms of the rational formula, is effectively a ratio to the hydrograph's time to peak). The average ratio of B/T_c, for each recurrence interval, was then determined and the results are presented in Table 3 together with their standard deviations. These results exclude three catchments whose area is 130 km^2 or less as they gave B/T_c ratios in excess of 7. It is noted here that there is an increase of base-length with recurrence interval, which means that the volumes of the floods relative to the peaks, as modelled by the runhydrograph, also increase with T. The figures in the third row of Table 3 show the proportion of floods whose base-length B exceeds $3T_c$, so that when T is 100, the proportion is approximately one third.

Validation of the runoff coefficients ($c_{(T)}$)

The purpose of validation is to test whether the model operates in the manner for which it was designed in "ways that were not explicitly built into the model" (Basson et al., 1994). Validation tests are necessary to convey confidence that the model works as expected. In order to validate the $c_{(T)}$-values achieved in calibration, it was necessary to find some physical regional descriptor(s) on which to regress the coefficients. This was required so that the calibrated coefficients may be extended to un-gauged catchments.

Several regional descriptors were tested in combination with the $c_{(T)}$-values to examine if a relationship existed on which to

TABLE 3
The mean and standard deviations of the ratio of the hydrograph time base-length B to the catchments' time of concentration T_c as a function of recurrence interval T. The proportion of B/T_c values above 3 in each group are given in the third row.

Recurrence interval T (years)	10	20	50	100	200
Mean of B/T_c ratios	1.92	2.06	2.25	2.40	2.56
Standard deviation	0.981	1.09	1.29	1.48	1.71
Proportion > 3	0.14	0.19	0.28	0.34	0.40

regress the coefficients. Descriptors such as catchment slope, mean annual precipitation (MAP), percentages of land coverage and Kovaćs' regional K-values (Kovaćs, 1988) were tested. From these analyses, no meaningful relationships between any of the descriptors tested and the $c_{(T)}$-coefficients were found. There were also no relationships found between parameters (multiplier and exponent) of a power-law function fitted to the $c_{(T)}$-values as a function of recurrence interval and regional descriptors. This result is in line with the comments of Pilgrim and Cordery (1993) for conditions in Australia, where the calibrated runoff coefficients did not show much sensitivity to catchment characteristics and indicate that the c-values are essentially functions of T and T_c as conjectured. Because there was no dependency observed between c-values and catchment properties, we were left with a problem: what values to use for validation?

It was decided to use the curves from Chow et al. (1988: 498), shown in Fig. 3, where it can be seen that the calibrated coefficients are generally lower than those of Chow et al. and the latter coefficients can be viewed as an approximate upper bounding set of curves. This choice, although conservative, was based on the premise that a practitioner will make a choice of the value of c based on catchment slope and land usage, knowing that it is bounded in the interval (0.1) and usually in the range 0.3 to 0.6.

Twenty-one catchments, which were not used in the calibration exercise and for which flood records were available, were selected for validation. These catchments ranged in size from 126 km^2 to 24 044 km^2. The flood records were modelled using a general extreme value (GEV) distribution in a previous study (Pegram and Parak, 2004) which was shown to be the most appropriate distribution generally speaking for flood peaks in the region. For these catchments, times of concentration (T_c) values were obtained from Petras and Du Plessis (1987) and representative design rainfall intensities from Smithers and Schulze (2002) in the same manner as for the calibration set. These data are summarised in Table A7 (Parts 1, 2 and 3) in the **Appendix**.

In order to obtain appropriate c-values from Chow et al. (1988: 498) for each catchment, it was necessary to relate the land coverage type and slope of each catchment with theirs (see Table 2 above). These catchment characteristics are given in Petras and Du Plessis (1987) where the percentages of land coverage for each catchment are catalogued as forest, dense bush wood, thin bush wood, cultivated land, grass and bare. At this stage it then became necessary to relate each catchment's coverage type (Petras and Du Plessis, 1987) to the generalised coverage types of Chow et al. (1988: 498). In order to easily accomplish this, several assumptions were made. They were:

- That the greatest percentage of land coverage (the modal type) was representative of the entire catchment
- That the following coverage types (from the descriptions of Petras and Du Plessis (1987) and Chow et al. (1988) respectively) were equivalent (shown in Table 4 below).

TABLE 4
Equivalent land coverage types from the descriptions of Petras and Du Plessis (1987) and Chow et al. (1988: 498)

Equivalent land coverage types	
Actual catchment land coverage (as described in Petras and Du Plessis (1987))	***c*-coefficient land coverages (as listed in Chow et al. (1988: 498))**
Forest	Forest/woodland
Dense bush wood	Forest/woodland
Thin bush wood	Forest/woodland
Cultivated land	Cultivated land
Grass	Pasture/range
Bare	Cultivated land

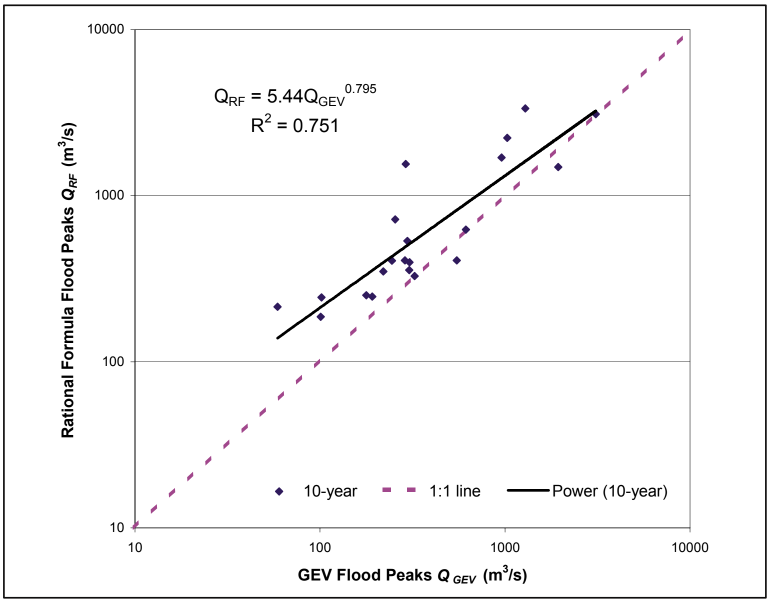
From the above procedure, design flood peaks were obtained using the rational formula method (Q_{RF}), i.e. a function of catchment area, design rainfall (of duration equal to the catchment time of concentration and the desired recurrence interval) and the runoff coefficients from Table 2. These design flood peaks were compared with the statistically modelled flood peaks (Q_{GEV}), from the same catchments, for the corresponding recurrence intervals. The results of this exercise, for the 10-, 50- and 200-year recurrence intervals are shown in Figs. 4, 5 and 6 respectively and are summarised for all recurrence intervals in Table 5.

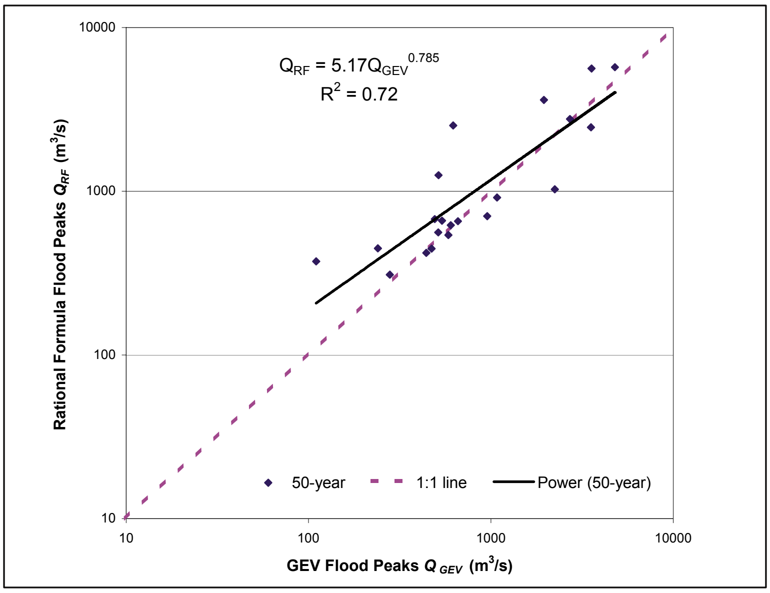
Although there is a fairly large scatter around the trend-line in log-space in Figs. 4, 5 and 6, some conclusions can be drawn from this validation exercise.

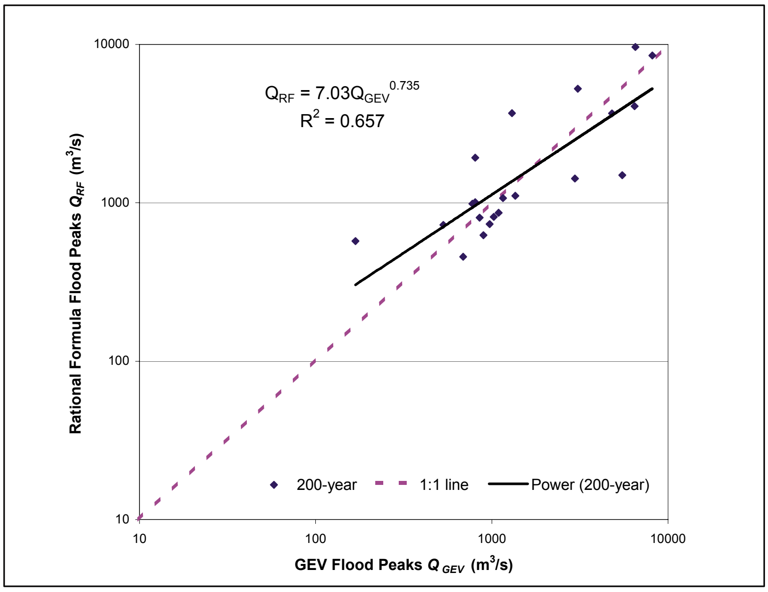
TABLE 5
A summary of the *power-law* curves, of the form $Q_{RF}=aQ_{GEV}^{\,b}$, fitted to the graphs of Q_{RF} vs. Q_{GEV} (where Q_{RF} are the flood peaks obtained from the rational formula and Q_{GEV} are the statistically modelled flood peaks). The average ratio of Q_{RF}/Q_{GEV} for each recurrence interval is also given.

Recurrence interval T (years)	10	20	50	100	200
Factor: *a*	5.44	5.10	5.17	5.75	7.03
Exponent: *b*	0.795	0.798	0.785	0.766	0.735
R^2	0.751	0.746	0.726	0.699	0.657
Mean Q_{RF}/Q_{GEV}	1.84	1.64	1.42	1.31	1.21

It is evident from the 10-, 50- and 200-year validation graphs (shown in Figs. 4, 5 and 6 respectively) that the estimated rational formula flood peaks Q_{RF} tend to be larger than the GEV modelled flood peaks Q_{GEV}, especially for the lower magnitude floods, however, their trend-lines cross the 1:1 line at the larger flows – peaks at about 7 000 m^3/s. This trend is also exhibited for the 20- and 100-year validation tests (the results of which are not shown here) and is confirmed in Table 5 where the average ratio of Q_{RF}/Q_{GEV} across all recurrence intervals is approximately 1.5, reducing from 1.84 for T = 10 to 1.21 for T = 200. This observation is to be expected since the c-values used to compute Q_{RF}, from Chow et al. (1988: 498), were generally larger than the calibrated runoff coefficients obtained in this study (see Table 2 and Fig. 3). Although the R^2-values are reasonable, the correlation is calculated in log-space and may disguise the fact that







some flow peak ratios are occasionally different by up to a factor of 5 (see Table A7, Part 3 in the Appendix for the full list of values). As a consequence, the *c*-values adopted for this validation exercise, from Chow et al., were treated as trial upper bound estimates, conceding that although consistent, the method is prone to error.

Discussion of results

Calibration

Calibration of the rational formula's runoff coefficients, using runhydrograph flood peak and volume pairs of given recurrence intervals, was performed with the intention of removing the subjectivity involved in this parameter's estimation in the design environment. Use was made of characteristic *T*-year flood peak and volume pairs together with *T*-year design rainfall intensities, as a function of the catchments time of concentration, in order to obtain the coefficients. The results of this exercise produced calibrated runoff coefficients, as a function of recurrence interval, which were scattered (see Fig. 3) around published values from Chow et al. (1988: 498). The calibrated values spread around the latter set of coefficients but were, in general, lower in magnitude (bar two catchments) and had gentler growths as a function of recurrence interval. Although this result did not produce a good match, the calibrated coefficients were sensible in magnitude. However, it was worrying to note that calibrated coefficients from six catchments (of the original 29) had a tendency to decrease in magnitude with increasing recurrence interval. This deviation from the norm is attributed to the fact that the flood runoff data (calculated using the runhydrograph method) had a gentler growth curve, as a function of recurrence interval, than the design rainfall data. It was found that the fitted *c*-values could not be regionalised in agree-

Figure 4 (top left)
A graph, for the purposes of validation, showing a plot in log space of the 10-year rational formula flood peaks Q_{RF} (using Chow et al.'s (1988) c-values as substitutes for calibrated runoff coefficients) vs. the 10-year GEV modelled flood peaks Q_{GEV}.

Figure 5 (middle left)
A graph, for the purposes of validation, showing a plot in log space of the 50-year rational formula flood peaks Q_{RF} (using Chow et al.'s (1988) c-values as substitutes for calibrated runoff coefficients) vs. the 50-year GEV modelled flood peaks Q_{GEV}.

Figure 6 (bottom left)
A graph, for the purposes of validation, showing a plot in log space of the 200-year rational formula flood peaks Q_{RF} (using Chow et al.'s (1988) c-values as substitutes for calibrated runoff coefficients) vs. the 200-year GEV modelled flood peaks Q_{GEV}

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ment with the conclusions of Pilgrim and Cordery (1993). Thus it is confirmed that *c* is a function of land-use, slope, T_c (through the design storm) and *T*. The fitted *c*-values (Fig. 3) were generally lower than those suggested by Chow et al. (1988: 498); it was therefore decided to accept the latter values for the purpose of validation, conscious of this discrepancy.

Hydrograph time base-length *B*

It was initially thought that this investigation would be able to produce entire design hydrographs (albeit in an idealised triangular form) from the rational formula since the flood data used (from the runhydrograph method) described characteristic peak and volume pairs for each catchment. It was hoped that the ratio of *B* to T_c (effectively a ratio of *B* to the time to peak of a rational formula hydrograph) would be consistent and that a particular outflow hydrograph could be prescribed with the use of this method. However, the results (see Table 3) indicate that, firstly, the average ratios are not constant across all recurrence intervals and, secondly, that the coefficients of variation are quite high (they range from 0.51 to 0.66). Also, the results shown in Table 3 exclude three catchments of area less than 130 km^2 as they gave ratios in excess of seven, however, several points are worth noting.

Firstly the base-lengths are, on average, 2.25 times the catchments' time of concentration across all recurrence intervals. This result is somewhat less than the length of the hydrograph suggested by Rooseboom et al. (1981) and Alexander (2002), which was $3T_c$, but Table 3 also indicates that a fair proportion of the calculated base lengths exceeded this number. As explained earlier, the hydrograph shape suggested by Rooseboom et al. (1981) was not meant to maintain continuity but was instead designed to be conservative. The hydrographs derived in this study are thus expected to have a smaller base-length as continuity is implicitly maintained, so the result is in line with expectation.

Secondly, the tendency of the base-length to increase with T is possibly due to the method employed by H&F in extracting their hydrographs and the non-linearity of the rainfall runoff process (abstractions reduce with *T*). As depicted in Fig. 7, H&F employed a truncation level for each catchment in order to extract independent hydrographs from their continuous records of streamflows. Flood volumes were obtained by extrapolating the rising limb and the recession limb of the discharge curves downwards towards zero flow from the first point below the truncation level which showed a reversal in slope. Depending on this level, a higher truncation level is likely to result in a reduction in the modelled volume when compared to the actual volume of the flood event. Thus it is likely that the base-lengths achieved in this study are smaller (as a function of T_c) for the smaller floods (more frequent events) than the base-lengths for the larger events, thus exhibiting the trend in Table 3.

Finally, it interesting to examine the relationship between *B* and T_c using a linear rainfall-runoff model as a comment on the values appearing in Table 3. If a constant (pulsed) input of rainfall of intensity *i* (in mm/h) on a catchment of area *A* (in km) lasts for the time of concentration T_c (hours), the total volume of rain that falls is $V = 1000 \cdot i \cdot T_c \cdot A$ (in m^3). The average rate of flow onto the catchment is $1\,000 \cdot i \cdot A$ (in m^3/h) and the peak outflow *Q* must be a fraction of this, say $\alpha \cdot 1000 \cdot i \cdot A$ (m^3/h), where $0 < \alpha < 1$ (α is a factor related to the closeness of the peak to its asymptotic value as defined by its nearness to equilibrium). The base-length of the equivalent triangular hydrograph is thus $B = {}^{V}/_{Q} = 2 \cdot T_c/\alpha$ (in hours). If there are no losses, the maximum peak that occurs at T_c can only be approaching equilibrium asymptotically,

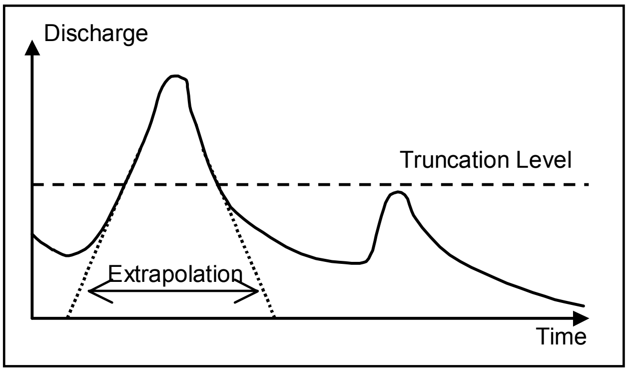

Figure 7
The method employed by Hiemstra and Francis (1979) to extract independent hydrographs from a continuous flow record, showing that a lower truncation level is likely to provide a bigger volume.

so α has to be chosen close to 1. If $\alpha = 0.9$, then it turns out that $B \approx 2.2T_c$, which is close to the *average ratio* (determined from Table 3 above).

Validation

The validation exercise was necessary to test whether the calibrated coefficients behaved in the probabilistic manner for which they were designed, i.e. to predict design floods of magnitudes equivalent to those derived from a statistical analysis of flood records from that site. However, since it was shown that *c* is not dependent on physical properties nor location of the un-gauged catchments, *c*-values from Chow et al. (1988: 498), which are a function of *T*, catchment slope and land-use characteristics, were substituted for the calibrated coefficients as approximate upper bound values. Based on this substitution, the validation exercise was ultimately reduced to a test of whether the *c*-values from Chow et al. (or possibly some other summary values) could provide reasonable design flood estimates such as those obtained from a statistical distribution (such as the GEV) fit to historical flood data.

The result of this exercise showed that the floods estimated using the substitute *c*-values from Chow et al. (1988) produced floods from the rational formula that were, on average, approximately 1.5 times larger than the floods estimated from the statistical distributions of the historical data (see Table 5 and Figs. 4, 5 and 6), with a tendency to overestimate for lower flood peaks and *T*. This result is in line with expectation as the substitute *c*-values from Chow et al. (1988) were adopted as upper bound estimates. Given that, in order to make use of the coefficients of Chow et al., a crude matching of land coverage types was performed (see Table 4), this result is relatively pleasing especially since the catchments used in validation ranged in size from small to large (170 to 24 000 km^2 – see Table A7). The precision of the method is of course still low, as indicated by the spread of results in Figs. 4, 5 and 6, and relies heavily on the judgement of the practitioner

Conclusion

The rational formula, which is possibly the simplest rainfall-runoff flood estimation technique available was reviewed by means of calibrating the most uncertain variable of the formula, i.e. the runoff coefficient *c*. The "data set" used to achieve this was the set of runhydrographs produced by Hiemstra and

Francis (1979). The results of the calibration were reasonably encouraging, producing *c*-coefficients that were scattered around, but generally lower than, those offered by Chow et al. (1988: 498), whose precision is not known. It was discovered that the fitted $c_{(T,Tc)}$-coefficients of this investigation did not show any variation with catchment characteristics, in line with Australian experience (Pilgrim and Cordery, 1993), and hence validation of these values at other sites was only possible using land-use and average slope of validation catchments together with recurrence interval as guides for the choice of *c*-values. It was thus decided to use the *c*-values from Chow et al. as approximate upper bound estimates of the fitted $c_{(T)}$-coefficients in validation. In order to use their values, a match of land coverage types was required. The results of the validation were as expected, producing floods from the rational formula that were on average 1.5 times larger than the floods estimated from a statistical analysis of the validation set (not used for calibration), but with a wide scatter. Of minor importance, it was discovered that the time base-lengths of the derived triangular hydrographs of this investigation were approximately between 1.9 and 2.6 times the catchment's time of concentration, depending on the recurrence interval of the flood, lower than suggested elsewhere. It can be concluded, from the results of this investigation, that the rational formula is a simple, consistent, approximate tool when used in its probabilistic frame-work and although not suitable as a stand-alone design tool for flood estimation, can be useful as a quick check method for calculating flood hydrographs for large catchments as it is for small.

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APPENDIX

TABLE A1
Selected information, extracted from Hiemstra and Francis (1979), on the streamflow stations used by them in the development of the runhydrograph method

No.	Station	River	Lat. (deg & min)	Long. (deg & min)	Area (km²)	Length of record (years)	Quality of data	No. of hydro-graphs	Trun-cation level (m³/s)	Deg. of trunc. %	*ln* (peaks)		*ln* (vol)		Cross correl-ation coeff.	Chi-square bi-variate	Accep-tance level Chi-square
											Mean	St. Dev.	Mean	St. dev.			
1	A2M01	Krokodil	25 44	27 52	2 907	18	A	20	323.4	43.51	5.964	1.133	7.812	1.727	0.843	13.36	70.1
2	A2M02	Magalies	25 44	27 51	1 206	18	A	17	116.6	64.36	4.282	1.297	5.896	1.461	0.907	18.24	70.1
3	A2M03	Hex	25 46	27 17	494	19	A	48	79.8	12.90	5.061	0.601	6.001	0.839	0.846	8.64	88.0
4	A2M12	Krokodil	25 49	27 55	2 586	36	D	37	109.6	49.87	4.699	0.698	6.432	1.065	0.750	12.97	88.0
5	A3M01	Klein Marico	25 32	26 06	1 002	33	A	33	52.4	73.44	3.338	0.992	5.307	0.776	0.704	20.31	88.0
6	B2M01	Bronkhorstspruit	25 48	28 46	1 585	47	A	42	145.9	36.14	5.242	0.731	7.664	1.062	0.923	21.70	88.0
7	B4M03	Steelpoort	25 01	29 32	2 271	10	A	9	76.4	45.57	4.423	0.787	7.871	0.555	0.748	8.59	54.4
8	B7M04	Klaserie	24 33	31 02	130	10	D	24	41.9	33.07	4.048	0.716	5.848	1.163	0.586	7.98	70.1
9	C1M01	Vaal	26 57	29 16	8 254	54	A	56	433.8	44.73	6.162	0.678	10.158	0.907	0.847	14.39	88.0
10	C3M04	Dry Hartz	27 34	24 43	8 039	24	A	18	47.1	25.81	4.241	0.600	6.561	1.187	0.676	21.39	70.1
11	C4M01	Groot Vet	28 29	26 40	5 504	24	B	40	325.3	40.22	5.962	0.718	9.359	0.663	0.827	11.69	88.0
12	C4M02	Vet	27 51	25 54	17 550	14	A	41	217.5	13.79	5.899	0.473	10.392	0.645	0.839	11.27	88.0
13	C5M03	Modder	29 10	26 35	1 650	36	A	35	247.4	9.04	6.189	0.504	8.806	0.778	0.889	6.86	88.0
14	C5M04	Modder	28 51	26 11	5 012	28	A	24	449.6	12.44	6.863	0.652	9.640	0.838	0.879	6.49	70.1
15	C5M10	Kromellenboog	29 50	25 38	1 994	17	A	28	43.4	11.61	4.686	0.765	7.353	1.055	0.893	9.96	70.1
16	C5M12	Riet	29 39	25 59	2 383	21	D	20	127.7	11.46	5.547	0.577	8.491	0.758	0.877	5.07	70.1
17	C5M15	Modder	28 48	26 06	6 545	18	A	18	298.9	18.30	6.252	0.611	9.278	0.894	0.892	6.07	70.1
18	C7M01	Renoster	27 16	27 11	5 255	25	A	14	218.8	38.02	5.671	0.931	9.566	0.924	0.892	145.16	54.4
19	C9M03	Vaal	28 31	24 42	10 8081	16	C	19	382.6	9.01	6.715	0.569	11.599	1.112	0.892	5.62	70.1
20	C9M06	Vaal	27 39	25 35	10 2384	22	A	23	519.6	35.57	6.554	0.815	11.565	1.151	0.754	9.18	70.1
21	D1M05	Oranje	30 02	28 30	10 891	29	A	26	574	59.87	6.195	0.631	10.593	0.599	0.683	8.04	70.1
22	D2M05	Caledon	28 53	27 54	3 815	14	A	37	171.5	26.04	5.538	0.613	9.724	0.731	0.692	7.56	88.0
23	D3M05	Oranje	29 48	24 26	91 994	19	A	22	2 302.6	64.51	7.509	0.625	12.75	0.476	0.439	12.85	70.1
24	D5M01	Renoster	31 39	20 37	2 129	26	E	31	82.19	6.86	5.174	0.510	7.918	0.769	0.839	9.81	70.1
25	D5M04	Sak	31 39	21 46	5 799	31	E	36	149.56	14.05	5.511	0.466	8.421	0.712	0.651	6.69	88.0
26	D6M02	Brak	30 07	23 34	6 360	16	A	36	73.9	16.47	4.925	0.638	7.688	0.995	0.760	9.77	88.0
27	E2M02	Doring	32 30	19 32	5 778	37	C	40	210.9	46.64	5.414	0.741	9.310	0.826	0.935	32.01	88.0
28	G1M02	Vier-en-twintig	33 08	19 04	186	10	A	9	261.4	9.12	6.010	0.331	7.388	0.683	0.577	4.92	54.4
29	H1M06	Bree	33 25	19 16	754	10	A	12	360.8	32.31	6.083	0.425	8.962	0.384	0.573	10.83	54.4
30	H1M07	Wit	33 34	19 09	83	10	A	27	171.5	66.75	4.918	0.524	6.884	0.847	0.608	18.19	70.1
31	H7M04	Huis	33 55	20 43	26	10	A	9	10.2	24.98	2.881	0.837	5.118	1.223	0.707	5.34	54.4
32	J2M03	Gamka	33 32	21 39	17 941	19	A	25	81.7	20.37	5.120	0.867	7.718	1.667	0.765	10.47	70.1
33	J3M04	Olifants	33 29	23 02	4 330	44	E	29	172.7	19.70	5.723	0.671	7.802	0.731	0.852	10.57	70.1
34	L7M02	Groot Vet	33 20	24 21	25 587	19	A	35	165.6	10.65	5.703	0.523	8.560	0.935	0.716	8.10	88.0
35	Q1M01	Groot Vis	31 54	25 29	9 150	43	A	30	242.1	60.84	5.273	0.787	7.399	1.314	0.882	89.34	70.1
36	Q7M01	Groot Vis	32 57	25 49	18 954	23	A	72	243.3	18.11	6.059	0.620	9.133	0.698	0.808	18.83	107.8
37	Q7M02	Groot Vis	32 43	25 51	18 436	26	A	19	275.6	16.35	6.230	0.623	9.221	0.880	0.858	6.58	70.1
38	Q9M10	Groot Vis	33 13	26 52	29 376	26	A	13	671.4	22.34	7.192	0.898	10.513	0.942	0.899	7.25	54.4
39	Q9M12	Groot Vis	33 06	26 27	23 041	24	B	35	86.6	22.88	5.242	1.052	8.615	1.315	0.916	11.21	88.0
40	T3M02	Kinira	30 29	28 37	2 100	12	A	15	207.3	8.01	5.671	0.238	8.953	0.929	0.494	7.01	54.4
42	W4A03	Pongola	27 25	31 31	5 843	16	A	13	655.7	36.17	6.710	0.636	9.328	0.700	0.877	40.60	54.4
42	W5M05	Hlelo	26 50	30 44	751	19	A	25	41.6	49.55	3.739	1.003	5.831	1.541	0.666	14.84	70.1
43	X2A02	Wit	25 19	31 03	176	20	B	20	48.3	10.51	4.511	0.503	6.194	1.231	0.736	6.85	70.1

TABLE A2

Data used for the calibration of the rational formula's *c*-coefficient for the 10-year case (i.e. all floods and rainfall estimates correspond to a 10-year recurrence interval)

No.	Station	Lat. (dec deg)	Long (dec deg)	Area (km^2)	Time of conc. (h)	Runhydrograph design flood (used for calibration)			Design rainfall (used in rational formula for calibration)				Cali-brated c-coeff-icient, $c_{(T)}$	Comment
						Peak (m^3/s)	Vol x 10^6 (m^3)	Hydro. base-length (h)	a	b	c	Inten-sity *i* mm/h		
1	A2M03	25.77	27.28	494	6.4	510.8	7.5	8.2	52.2	0.225	0.776	12.37	0.301	
2	A2M12	25.82	27.92	2 586	18	331.4	12.1	20.2	46.8	0.237	0.763	5.16	0.089	
3	A3M01	25.53	26.10	1 002	8.7	204.3	3.4	9.3	48.2	0.210	0.790	8.73	0.084	
4	B2M01	25.80	28.77	1 585	18.1	531.2	34.4	36.0	53.9	0.227	0.773	5.74	0.210	
5	B4M03	25.02	29.53	2 271	19.6	249.7	20.5	45.5	41.7	0.240	0.760	4.34	0.091	
6	B7M04	24.55	31.03	130	3.7	215.9	10.8	27.7	69.1	0.240	0.760	25.56	0.234	Anomalous
7	C1M01	26.95	29.27	8 254	74	1 404.4	396.7	156.9	42.8	0.229	0.772	1.55	0.396	
8	C4M01	28.48	26.67	5 504	34	1 524.0	147.6	53.8	45.5	0.200	0.800	2.71	0.368	
9	C4M02	27.85	25.90	17 550	111	968.5	444.5	255.0	44.0	0.218	0.782	1.11	0.179	Anomalous
10	C5M03	29.17	26.58	1 650	18.3	843.5	56.0	36.9	41.5	0.227	0.773	4.39	0.419	
11	C5M04	28.85	26.18	5 012	38	1 807.6	125.4	38.5	41.1	0.226	0.774	2.46	0.528	
12	C5M12	29.65	25.98	2 383	23	479.1	39.8	46.2	38.3	0.220	0.780	3.31	0.218	
13	C5M15	28.80	26.10	6 545	43	1 106.8	116.7	58.6	40.1	0.225	0.776	2.17	0.280	
14	C7M01	27.27	27.18	5 255	57	666.7	117.2	97.7	45.1	0.221	0.779	1.93	0.236	
15	D1M05	30.03	28.50	10 891	60	1 339.0	372.4	154.5	44.1	0.204	0.796	1.69	0.261	
16	D5M01	31.65	20.62	2 129	27	331.0	25.5	42.8	32.6	0.172	0.828	2.13	0.263	Anomalous
17	D5M04	31.65	21.77	5 799	28	418.4	36.5	48.4	31.0	0.177	0.823	2.00	0.130	Anomalous
18	E2M02	32.50	19.53	5 778	30	835.1	172.0	114.4	20.7	0.194	0.806	1.34	0.389	
19	H1M06	33.42	19.27	754	7.6	765.5	46.5	33.7	29.8	0.355	0.645	8.05	0.454	
20	H1M07	33.57	19.15	83	2.4	475.9	26.4	30.8	42.9	0.399	0.601	25.37	0.814	Anomalous
21	H7M04	33.92	20.72	26	2.3	43.0	2.2	28.1	38.3	0.301	0.699	21.42	0.278	
22	J2M03	33.53	21.65	17 941	42	567.6	84.7	82.9	31.6	0.186	0.814	1.51	0.076	
23	J3M04	33.48	23.03	4 330	23	500.1	15.0	16.7	30.3	0.211	0.789	2.55	0.163	
24	Q1M01	31.90	25.48	9 150	18	683.0	47.7	38.8	29.9	0.209	0.791	3.04	0.089	
25	Q9M10	33.22	26.87	29 376	108	1 962.9	199.4	56.4	38.1	0.289	0.711	1.37	0.176	
26	Q9M12	33.10	26.45	23 041	85	1 114.7	182.4	90.9	33.5	0.307	0.693	1.54	0.113	
27	T3M02	30.48	28.62	2 100	26	370.0	71.7	107.7	45.1	0.207	0.793	3.40	0.186	Anomalous
28	W4A03	27.42	31.52	5 843	31	1 833.4	98.1	29.7	54.0	0.259	0.741	4.23	0.267	
29	W5M05	26.83	30.73	751	17.8	230.8	16.8	40.4	57.6	0.229	0.771	6.25	0.177	

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TABLE A3
Data used for the calibration of the rational formula's *c*-coefficient for the 20-year case

No.	Station	Lat. (dec deg)	Long (dec deg)	Area (km^2)	Time of conc. (h)	Runhydrograph design flood (used for calibration)			Design rainfall (used in rational formula for calibration)				Calibrated *c*-coefficient, $c_{(T)}$	Comment
						Peak (m^3/s)	Vol x 10^6 (m^3)	Hydro. base-length (h)	a	b	c	Intensity *i* mm/h		
1	A2M03	25.77	27.28	494	6.4	597.9	9.3	8.7	60.7	0.225	0.776	14.39	0.303	
2	A2M12	25.82	27.92	2 586	18	404.2	16.3	22.4	55.2	0.237	0.763	6.08	0.093	
3	A3M01	25.53	26.10	1 002	8.7	258.6	4.1	8.8	55.6	0.210	0.790	10.06	0.092	
4	B2M01	25.80	28.77	1 585	18.1	680.3	49.3	40.2	63.7	0.227	0.773	6.79	0.228	
5	B4M03	25.02	29.53	2 271	19.6	317.7	24.2	42.4	47.3	0.240	0.760	4.93	0.102	
6	B7M04	24.55	31.03	130	3.7	255.2	14.1	30.7	82.2	0.240	0.760	30.39	0.233	Anomalous
7	C1M01	26.95	29.27	8 254	74	1 719.7	520.1	168.0	49.6	0.229	0.772	1.79	0.419	
8	C4M01	28.48	26.67	5 504	34	1 841.8	175.8	53.0	52.3	0.200	0.800	3.12	0.386	
9	C4M02	27.85	25.90	17 550	111	1 090.5	522.7	266.3	50.9	0.218	0.782	1.28	0.175	Anomalous
10	C5M03	29.17	26.58	1 650	18.3	1 019.7	75.1	40.9	47.8	0.227	0.773	5.06	0.440	
11	C5M04	28.85	26.18	5 012	38	2 334.5	174.2	41.5	47.3	0.226	0.774	2.83	0.592	
12	C5M12	29.65	25.98	2 383	23	594.7	52.9	49.5	44.1	0.220	0.780	3.82	0.235	
13	C5M15	28.80	26.10	6 545	43	1 374.5	160.1	64.7	46.2	0.224	0.776	2.50	0.302	
14	C7M01	27.27	27.18	5 255	57	976.4	171.2	97.4	52.0	0.221	0.779	2.23	0.300	
15	D1M05	30.03	28.50	10 891	60	1 590.5	438.5	153.2	51.4	0.204	0.796	1.98	0.266	
16	D5M01	31.65	20.62	2 129	27	394.6	33.2	46.8	38.7	0.172	0.828	2.52	0.264	Anomalous
17	D5M04	31.65	21.77	5 799	28	486.5	45.9	52.5	36.6	0.177	0.823	2.36	0.128	Anomalous
18	E2M02	32.50	19.53	5 778	30	1 041.5	220.0	117.4	24.0	0.194	0.806	1.55	0.420	
19	H1M06	33.42	19.27	754	7.6	864.7	51.9	33.3	33.4	0.355	0.645	9.03	0.457	
20	H1M07	33.57	19.15	83	2.4	528.0	31.2	32.8	48.4	0.399	0.601	28.62	0.800	Anomalous
21	H7M04	33.92	20.72	26	2.3	57.4	3.3	32.1	46.4	0.301	0.699	25.92	0.307	
22	J2M03	33.53	21.65	17 941	42	740.8	141.4	106.0	37.9	0.186	0.814	1.81	0.082	
23	J3M04	33.48	23.03	4 330	23	667.0	20.6	17.1	36.6	0.211	0.789	3.09	0.180	
24	Q1M01	31.90	25.48	9 150	18	869.3	71.4	45.6	34.6	0.209	0.791	3.51	0.097	
25	Q9M10	33.22	26.87	29 376	108	3 080.4	320.0	57.7	46.3	0.289	0.711	1.66	0.227	
26	Q9M12	33.10	26.45	23 041	85	1 533.4	271.7	98.4	39.2	0.307	0.693	1.81	0.133	
27	T3M02	30.48	28.62	2 100	26	398.5	95.8	133.5	52.6	0.207	0.793	3.97	0.172	Anomalous
28	W4A03	27.42	31.52	5 843	31	2 287.1	125.1	30.4	64.7	0.259	0.742	5.07	0.278	
29	W5M05	26.83	30.73	751	17.8	299.3	25.0	46.4	68.4	0.229	0.771	7.43	0.193	

TABLE A4

Data used for the calibration of the rational formula's *c*-coefficient for the 50-year case

No.	Station	Lat. (dec deg)	Long (dec deg)	Area (km²)	Time of conc. (h)	Runhydrograph design flood (used for calibration)			Design rainfall (used in rational formula for calibration)				Calibrated c-coefficient, $c_{(T)}$	Comment
						Peak (m³/s)	Vol x 10^6 (m³)	Hydro. base-length (h)	a	b	c	Intensity *i* mm/h		
1	A2M03	25.77	27.28	494	6.4	718.4	12.1	9.3	72.6	0.224	0.776	17.21	0.304	
2	A2M12	25.82	27.92	2 586	18	507.2	23.1	25.3	67.2	0.237	0.763	7.41	0.095	
3	A3M01	25.53	26.10	1 002	8.7	341.6	5.1	8.3	65.3	0.210	0.790	11.82	0.104	
4	B2M01	25.80	28.77	1 585	18.1	895.0	73.4	45.6	78.2	0.227	0.773	8.33	0.244	
5	B4M03	25.02	29.53	2 271	19.6	416.6	29.3	39.1	56.4	0.240	0.760	5.88	0.112	
6	B7M04	24.55	31.03	130	3.7	310.7	19.4	34.8	101.3	0.294	0.706	40.21	0.214	Anomalous
7	C1M01	26.95	29.27	8 254	74	2 165.0	707.8	181.6	58.9	0.228	0.772	2.13	0.444	
8	C4M01	28.48	26.67	5 504	34	2 296.0	215.5	52.1	61.7	0.200	0.800	3.68	0.409	
9	C4M02	27.85	25.90	17 550	111	1 253.8	632.2	280.1	60.2	0.218	0.782	1.51	0.170	Anomalous
10	C5M03	29.17	26.58	1 650	18.3	1 252.0	103.1	45.7	56.3	0.227	0.773	5.96	0.458	
11	C5M04	28.85	26.18	5 012	38	3 069.0	247.6	44.8	55.7	0.226	0.774	3.34	0.660	
12	C5M12	29.65	25.98	2 383	23	751.4	72.0	53.2	51.9	0.220	0.780	4.50	0.252	
13	C5M15	28.80	26.10	6 545	43	1 743.1	226.7	72.3	54.5	0.224	0.776	2.95	0.325	
14	C7M01	27.27	27.18	5 255	57	1 462.1	255.6	97.1	61.6	0.221	0.779	2.64	0.379	
15	D1M05	30.03	28.50	10 891	60	1 938.1	529.0	151.6	61.7	0.204	0.796	2.37	0.270	
16	D5M01	31.65	20.62	2 129	27	478.8	44.5	51.6	46.9	0.172	0.828	3.06	0.264	Anomalous
17	D5M04	31.65	21.77	5 799	28	574.9	59.3	57.3	44.2	0.177	0.823	2.84	0.125	Anomalous
18	E2M02	32.50	19.53	5 778	30	1 338.5	291.0	120.8	28.2	0.194	0.806	1.82	0.459	
19	H1M06	33.42	19.27	754	7.6	992.4	58.8	32.9	38.0	0.355	0.645	10.27	0.461	
20	H1M07	33.57	19.15	83	2.4	598.6	38.2	35.5	55.7	0.399	0.601	32.88	0.790	Anomalous
21	H7M04	33.92	20.72	26	2.3	78.7	5.3	37.2	58.1	0.301	0.699	32.48	0.336	
22	J2M03	33.53	21.65	17 941	42	999.6	251.6	139.8	46.9	0.186	0.814	2.24	0.090	
23	J3M04	33.48	23.03	4 330	23	900.4	28.5	17.6	45.9	0.211	0.789	3.87	0.194	
24	Q1M01	31.90	25.48	9 150	18	1 142.1	112.6	54.8	40.9	0.209	0.791	4.15	0.108	
25	Q9M10	33.22	26.87	29 376	108	4 835.3	513.5	59.0	58.6	0.289	0.711	2.10	0.282	
26	Q9M12	33.10	26.45	23 041	85	2 201.4	427.0	107.8	47.2	0.307	0.693	2.17	0.158	
27	T3M02	30.48	28.62	2 100	26	432.4	131.8	169.3	63.0	0.207	0.793	4.76	0.156	Anomalous
28	W4A03	27.42	31.52	5 843	31	2 917.6	163.6	31.1	80.8	0.259	0.741	6.33	0.284	
29	W5M05	26.83	30.73	751	17.8	404.7	39.8	54.6	84.3	0.229	0.771	9.15	0.212	

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TABLE A5
Data used for the calibration of the rational formula's *c*-coefficient for the 100-year case

No.	Station	Lat. (dec deg)	Long (dec deg)	Area (km²)	Time of conc. (h)	Runhydrograph design flood (used for calibration)			Design rainfall (used in rational formula for calibration)				Cali-brated *c*-coeff-icient, $c_{(T)}$	Comment
						Peak (m^3/s)	Vol x 10^6 (m^3)	Hydro. base-length (h)	a	b	c	Inten-sity *i* mm/h		
1	A2M03	25.77	27.28	494	6.4	815.3	14.4	9.8	82.2	0.224	0.776	19.48	0.305	
2	A2M12	25.82	27.92	2 586	18	591.8	29.2	27.4	77.3	0.237	0.763	8.51	0.097	
3	A3M01	25.53	26.10	1 002	8.7	414.4	6.0	8.0	72.8	0.210	0.790	13.19	0.113	
4	B2M01	25.80	28.77	1 585	18.1	1 074.7	95.8	49.5	90.3	0.227	0.773	9.62	0.254	
5	B4M03	25.02	29.53	2 271	19.6	500.0	33.4	37.1	60.7	0.240	0.760	6.33	0.125	
6	B7M04	24.55	31.03	130	3.7	356.2	24.3	37.9	117.5	0.240	0.760	43.45	0.227	Anomalous
7	C1M01	26.95	29.27	8 254	74	2 530.6	872.1	191.5	66.4	0.228	0.772	2.40	0.460	
8	C4M01	28.48	26.67	5 504	34	2 672.3	247.9	51.5	68.9	0.200	0.800	4.11	0.425	
9	C4M02	27.85	25.90	17 550	111	1 381.1	721.3	290.1	67.5	0.218	0.782	1.70	0.167	Anomalous
10	C5M03	29.17	26.58	1 650	18.3	1 432.5	126.9	49.2	63.0	0.227	0.773	6.66	0.469	
11	C5M04	28.85	26.18	5 012	38	3 668.3	311.4	47.2	62.3	0.226	0.774	3.73	0.706	
12	C5M12	29.65	25.98	2 383	23	876.1	88.1	55.8	58.0	0.220	0.780	5.02	0.264	
13	C5M15	28.80	26.10	6 545	43	2 040.3	285.4	77.7	60.9	0.224	0.776	3.29	0.341	
14	C7M01	27.27	27.18	5 255	57	1 899.5	331.4	96.9	69.2	0.221	0.779	2.97	0.438	
15	D1M05	30.03	28.50	10 891	60	2 218.0	601.2	150.6	70.0	0.204	0.796	2.69	0.272	
16	D5M01	31.65	20.62	2 129	27	544.4	54.0	55.1	53.4	0.172	0.828	3.49	0.264	Anomalous
17	D5M04	31.65	21.77	5 799	28	642.4	70.2	60.8	50.2	0.177	0.823	3.23	0.123	Anomalous
18	E2M02	32.50	19.53	5 778	30	1 586.0	351.6	123.2	31.4	0.194	0.806	2.03	0.487	
19	H1M06	33.42	19.27	754	7.6	1 089.1	63.9	32.6	41.4	0.355	0.645	11.20	0.464	
20	H1M07	33.57	19.15	83	2.4	654.2	44.1	37.5	61.0	0.399	0.601	36.07	0.787	Anomalous
21	H7M04	33.92	20.72	26	2.3	97.0	7.1	40.9	68.1	0.301	0.699	38.05	0.353	
22	J2M03	33.53	21.65	17 941	42	1 223.2	370.9	168.4	54.4	0.186	0.814	2.59	0.095	
23	J3M04	33.48	23.03	4 330	23	1 091.9	35.2	17.9	53.8	0.211	0.789	4.53	0.200	
24	Q1M01	31.90	25.48	9 150	18	1 373.6	153.2	62.0	45.8	0.209	0.791	4.66	0.116	
25	Q9M10	33.22	26.87	29 376	108	6 417.6	691.0	59.8	69.0	0.289	0.711	2.48	0.318	
26	Q9M12	33.10	26.45	23 041	85	2 811.4	579.7	114.5	53.5	0.307	0.693	2.47	0.178	
27	T3M02	30.48	28.62	2 100	26	456.6	163.0	198.3	71.5	0.207	0.793	5.40	0.145	Anomalous
28	W4A03	27.42	31.52	5 843	31	3 429.6	195.4	31.7	94.6	0.259	0.741	7.42	0.285	
29	W5M05	26.83	30.73	751	17.8	497.8	54.6	61.0	97.8	0.229	0.771	10.62	0.225	

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TABLE A6
Data used for the calibration of the rational formula's *c*-coefficient for the 200-year case

No.	Station	Lat. (dec deg)	Long (dec deg)	Area (km^2)	Time of conc. (h)	Runhydrograph design flood (used for calibration)			Design rainfall (used in rational formula for calibration)				Calibrated *c*-coefficient, $c_{(T)}$	Comment
						Peak (m^3/s)	Vol x 10^6 (m^3)	Hydro. base-length (h)	a	b	c	Intensity *i* mm/h		
1	A2M03	25.77	27.28	494	6.4	918.2	17.0	10.3	92.4	0.224	0.776	21.89	0.306	
2	A2M12	25.82	27.92	2 586	18	683.2	36.4	29.6	88.2	0.237	0.763	9.72	0.098	
3	A3M01	25.53	26.10	1 002	8.7	497.3	6.9	7.7	80.5	0.210	0.790	14.58	0.123	
4	B2M01	25.80	28.77	1 585	18.1	1 271.5	122.3	53.4	103.5	0.227	0.773	11.03	0.262	
5	B4M03	25.02	29.53	2 271	19.6	591.9	37.6	35.3	66.7	0.240	0.760	6.95	0.135	
6	B7M04	24.55	31.03	130	3.7	405.2	29.9	41.0	135.4	0.240	0.760	50.07	0.224	Anomalous
7	C1M01	26.95	29.27	8 254	74	2 925.1	1 058.5	201.0	74.3	0.228	0.772	2.68	0.476	
8	C4M01	28.48	26.67	5 504	34	3 080.9	282.7	51.0	76.4	0.200	0.800	4.55	0.442	
9	C4M02	27.85	25.90	17 550	111	1 512.7	816.6	299.9	75.2	0.218	0.782	1.89	0.164	Anomalous
10	C5M03	29.17	26.58	1 650	18.3	1 619.5	153.4	52.6	69.8	0.227	0.773	7.38	0.479	
11	C5M04	28.85	26.18	5 012	38	4 312.8	383.5	49.4	69.0	0.226	0.774	4.14	0.749	
12	C5M12	29.65	25.98	2 383	23	1 007.6	105.8	58.3	64.2	0.220	0.780	5.56	0.274	
13	C5M15	28.80	26.10	6 545	43	2 356.6	352.5	83.1	67.5	0.225	0.776	3.65	0.355	
14	C7M01	27.27	27.18	5 255	57	2 406.6	419.2	96.8	77.2	0.221	0.779	3.31	0.498	
15	D1M05	30.03	28.50	10 891	60	2 515.3	677.5	149.6	79.0	0.204	0.796	3.04	0.274	
16	D5M01	31.65	20.62	2 129	27	612.4	64.5	58.5	60.2	0.172	0.828	3.93	0.264	Anomalous
17	D5M04	31.65	21.77	5 799	28	711.2	82.1	64.1	56.5	0.177	0.823	3.64	0.121	Anomalous
18	E2M02	32.50	19.53	5 778	30	1 856.2	418.9	125.4	34.7	0.194	0.806	2.24	0.516	
19	H1M06	33.42	19.27	754	7.6	1 187.0	69.1	32.3	44.8	0.355	0.645	12.12	0.468	
20	H1M07	33.57	19.15	83	2.4	712.1	50.6	39.5	66.5	0.399	0.601	39.28	0.786	Anomalous
21	H7M04	33.92	20.72	26	2.3	117.3	9.4	44.7	79.1	0.301	0.699	44.17	0.368	
22	J2M03	33.53	21.65	17 941	42	1 474.0	530.9	200.1	62.4	0.186	0.814	2.98	0.099	
23	J3M04	33.48	23.03	4 330	23	1 298.7	42.5	18.2	62.4	0.211	0.789	5.26	0.205	
24	Q1M01	31.90	25.48	9 150	18	1 629.7	203.8	69.5	50.9	0.209	0.791	5.17	0.124	
25	Q9M10	33.22	26.87	29 376	108	8 251.8	899.5	60.6	80.8	0.289	0.711	2.90	0.349	
26	Q9M12	33.10	26.45	23 041	85	3 526.9	769.6	121.2	60.3	0.307	0.693	2.78	0.198	
27	T3M02	30.48	28.62	2 100	26	480.0	198.0	229.2	80.7	0.207	0.793	6.09	0.135	Anomalous
28	W4A03	27.42	31.52	5 843	31	3 977.5	230.1	32.1	110.2	0.259	0.741	8.64	0.284	
29	W5M05	26.83	30.73	751	17.8	604.2	73.6	67.7	112.7	0.228	0.771	12.23	0.237	

TABLE A7 (Part 1)
Data used for validation

Num.	Station	River	Lat. (dec deg)	Long. (dec deg)	Area (km²)	Time of Conc. (hours)	Percentage land coverage						Runoff coefficients *c* from Chow et al. (1988)				
							For-est	Bush Wood		Cult. Land	Grass	Bare	10-year	20-year	50-year	100-year	200-year
								Dense	Thin								
1	A4M02	Mokolo	24.28	28.09	1 777	18.1	0	25	39	26	7	3	0.28	0.31	0.35	0.39	0.42
2	A6M06	Klein-nyl	24.70	28.41	168	4.4	0	28	34	29	6	3	0.28	0.31	0.35	0.39	0.42
3	C3M03	Harts	27.58	24.75	10990	78	0	0	28	59	4	9	0.36	0.39	0.43	0.47	0.51
4	C5M08	Riet	29.81	26.21	593	11.9	0	0	7	25	22	46	0.36	0.39	0.43	0.47	0.51
5	C8M01	Wilge	27.27	28.32	15673	122	0	6	8	50	24	12	0.36	0.39	0.43	0.47	0.51
6	C8M03	Cornelis	27.84	28.96	806	19.2	0	3	4	42	34	17	0.36	0.39	0.43	0.47	0.51
7	D1M01	Stormbergspruit	31.00	26.34	2 397	19.9	0	0	46	13	14	27	0.28	0.31	0.35	0.39	0.42
8	D1M04	Stormbergspruit	31.40	26.37	348	9.1	0	0	50	30	7	13	0.28	0.31	0.35	0.39	0.42
9	D2M01	Caledon	29.72	26.98	13421	106	0	4	7	73	11	5	0.36	0.39	0.43	0.47	0.51
10	E2M03	Doring	31.90	18.69	24044	59	0	0	16	16	0	68	0.36	0.39	0.43	0.47	0.51
11	G1M08	Klein-berg	33.31	19.08	395	4	5	20	5	48	15	7	0.36	0.39	0.43	0.47	0.51
12	H3M01	Kingna	33.79	20.13	611	9.5	0	0	25	15	0	60	0.36	0.39	0.43	0.47	0.51
13	Q7M03	Groot-vis	32.78	25.84	18534	59	0	0	30	8	0	62	0.36	0.39	0.43	0.47	0.51
14	Q9M04	Kat	32.56	26.69	404	6.3	4	8	24	16	32	16	0.38	0.41	0.45	0.49	0.52
15	Q9M08	Kat	32.71	26.59	748	12.7	7	3	25	19	31	15	0.38	0.41	0.45	0.49	0.52
16	R1M01	Tyume	32.76	26.86	238	6.2	10	10	15	20	30	15	0.38	0.41	0.45	0.49	0.52
17	T3M04	Mzimhlava	30.57	29.43	1029	18.8	5	5	14	46	20	10	0.36	0.39	0.43	0.47	0.51
18	V2M02	Mooi	29.22	29.99	937	18.9	20	10	0	25	43	2	0.30	0.33	0.37	0.41	0.46
19	V6M02	Tugela	28.75	30.44	12862	48	1	24	0	30	42	3	0.30	0.33	0.37	0.41	0.46
20	W5M06	Swartwater	27.11	30.83	180	5	15	13	0	7	64	1	0.30	0.33	0.37	0.41	0.46
21	X2M10	Noordkaap	25.61	30.88	126	3.3	45	15	0	10	28	2	0.28	0.31	0.35	0.39	0.42

TABLE A7 (Part 2)
Design rainfall data used for validation

No.	Station	b	c	a					Rainfall Intensity, $i=ad^{-c}$ (mm/hr)				
				10-year	20-year	50-year	100-year	200-year	10-year	20-year	50-year	100-year	200-year
1	A4M02	0.218	0.783	50.19	58.40	69.88	79.13	88.86	5.21	6.06	7.25	8.21	9.22
2	A6M06	0.217	0.783	49.87	58.07	69.45	78.64	88.37	16.37	19.05	22.79	25.80	28.98
3	C3M03	0.188	0.812	48.66	56.27	66.31	73.96	81.78	1.41	1.63	1.92	2.15	2.37
4	C5M08	0.220	0.780	38.17	44.28	52.68	59.38	66.52	5.53	6.42	7.64	8.61	9.64
5	C8M01	0.234	0.766	42.93	49.42	58.35	65.43	72.77	1.08	1.25	1.47	1.65	1.84
6	C8M03	0.231	0.769	42.23	48.28	56.45	62.94	69.62	4.35	4.97	5.81	6.48	7.17
7	D1M01	0.214	0.786	35.20	40.01	46.24	50.85	55.48	3.36	3.82	4.41	4.85	5.29
8	D1M04	0.240	0.760	36.85	42.07	48.99	54.31	59.66	6.88	7.86	9.15	10.14	11.15
9	D2M01	0.226	0.774	41.02	47.57	56.61	63.85	71.49	1.11	1.29	1.53	1.73	1.94
10	E2M03	0.201	0.799	24.06	27.80	32.73	36.52	40.36	0.93	1.07	1.26	1.40	1.55
11	G1M08	0.327	0.673	26.19	29.38	33.45	36.49	39.44	10.31	11.56	13.16	14.35	15.52
12	H3M01	0.278	0.722	29.63	36.07	45.70	53.97	63.29	5.83	7.09	8.98	10.61	12.44
13	Q7M03	0.295	0.705	32.07	37.50	45.10	51.20	57.66	1.81	2.12	2.54	2.89	3.25
14	Q9M04	0.252	0.748	37.78	44.20	53.17	60.33	67.99	9.53	11.15	13.41	15.22	17.15
15	Q9M08	0.285	0.715	30.99	36.22	43.57	49.48	55.71	5.03	5.89	7.08	8.04	9.05
16	R1M01	0.258	0.742	38.84	45.44	54.62	62.00	69.88	10.03	11.74	14.11	16.02	18.05
17	T3M04	0.221	0.779	51.08	60.15	73.32	84.35	96.45	5.19	6.12	7.46	8.58	9.81
18	V2M02	0.223	0.777	51.33	59.81	71.72	81.45	91.87	5.22	6.09	7.30	8.29	9.35
19	V6M02	0.228	0.772	57.56	69.04	86.15	100.91	117.52	2.90	3.47	4.33	5.08	5.91
20	W5M06	0.227	0.773	57.15	67.96	83.76	97.13	111.94	16.48	19.59	24.15	28.01	32.28
21	X2M10	0.225	0.776	62.92	74.80	92.18	106.94	123.24	24.93	29.63	36.53	42.37	48.83

TABLE A7 (Part 3)
Flood data used for validation

No.	Station	Rational formula flood peaks, Q_{RF} (m^3/s), using design rainfall and *c*-values from Chow et al. (1988)					GEV modelled flood peaks of recorded events Q_{GEV} (m^3/s)				
		10-year	20-year	50-year	100-year	200-year	10-year	20-year	50-year	100-year	200-year
1	A4M02	719.5	913.9	1 252.1	1 579.5	1 930.1	255.0	359.0	516.0	652.0	806.0
2	A6M06	213.9	271.7	372.2	469.6	573.8	59.0	79.0	110.0	137.0	168.0
3	C3M03	1 552.6	1962.2	2 524.9	3 079.5	3 678.0	291.0	397.0	622.0	893.0	1 301.0
4	C5M08	328.2	416.3	540.9	666.6	806.4	325.0	432.0	585.0	712.0	851.0
5	C8M01	1 699.9	2 141.5	2 759.3	3 383.6	4 067.3	959.0	1 515.0	2 719.0	4 197.0	6 456.0
6	C8M03	350.4	438.2	559.7	682.0	814.6	220.0	321.0	514.0	727.0	1 024.0
7	D1M01	626.0	776.6	1 027.5	1 258.9	1 493.9	616.0	1 144.0	2 246.0	3 557.0	5 505.0
8	D1M04	186.3	232.2	309.6	382.3	457.1	101.0	156.0	279.0	437.0	687.0
9	D2M01	1 490.9	1 890.9	2 457.4	3 029.3	3 663.7	1 939.0	2 653.0	3 543.0	4 185.0	4 804.0
10	E2M03	2 227.1	2 813.1	3 616.8	4 407.9	5 261.7	1 029.0	1 389.0	1 956.0	2 470.0	3 073.0
11	G1M08	407.2	499.5	620.9	740.3	864.6	288.0	400.0	603.0	813.0	1 092.0
12	H3M01	356.0	474.0	655.5	846.6	1 072.2	304.0	436.0	661.0	881.0	1 156.0
13	Q7M03	3 351.5	4 290.2	5 630.4	6 986.4	8 505.1	1 289.0	2 022.0	3 561.0	5 399.0	8 141.0
14	Q9M04	406.3	517.2	677.0	836.8	1 007.9	245.0	340.0	493.0	635.0	805.0
15	Q9M08	397.4	505.6	661.8	818.4	985.3	305.0	401.0	539.0	652.0	775.0
16	R1M01	252.0	320.7	419.7	518.8	624.9	178.0	269.0	442.0	632.0	895.0
17	T3M04	534.5	688.8	916.5	1 152.4	1 423.8	297.0	534.0	1 083.0	1 801.0	2 963.0
18	V2M02	408.0	529.3	703.2	884.6	1 108.5	548.0	716.0	954.0	1 149.0	1 357.0
19	V6M02	3 104.3	4 144.6	5 728.4	7 435.4	9 618.5	3 096.0	3 791.0	4 790.0	5 620.0	6 523.0
20	W5M06	247.2	327.3	446.8	574.2	735.1	192.0	292.0	474.0	662.0	970.0
21	X2M10	244.3	317.0	447.4	578.3	725.0	102.0	146.0	240.0	355.0	530.0

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