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WILL ALEXANDER is an emeritus professor of the Department of Civil and Biosystems Engineering of the University of Pretoria. He has written two books and a number of papers and articles on floods. From 1994 to 2000 he was a member of the United Nations Scientific and Technical Committee on natural disasters, and in 2001 he was a member of an international team of experts appointed to advise the government of Mozambique on the development of a national flood management policy.

The standard design flood

W J R Alexander

In this paper it is demonstrated that the unacceptably frequent destruction or severe damage to bridges and other civil engineering structures by floods in South Africa is due to the wide band of uncertainty around all estimates of the flood magnitude-frequency relationship, combined with steep increases in flood magnitude with increase in return periods. These uncertainties cannot be satisfactorily accommodated in current design flood estimation procedures, and there is an urgent need for a new approach to the estimation of the design flood. A new, simple, but robust, standard design flood estimation method is described which, together with the firmly established regional maximum flood method, can be satisfactorily be used for the design of most structures vulnerable to flood damage in South Africa. The designer is relieved of having to evaluate the relative applicability of alternative methods for determining the design flood, and is encouraged to use engineering factors of safety to accommodate uncertainties in the hydrological analyses, rather than investigate, evaluate and apply alternative hydrological procedures.

INTRODUCTION

Why have more civil engineering structures been destroyed or severely damaged by floods in South Africa during the past 20 years than in the whole of Europe during the past 200 years? Is it due to public apathy (inadequate funding), inadequate hydrological analyses, or inadequate structural design standards? Why has the civil engineering profession not expressed any official interest in this subject?

Inadequate funding is always a constraint that has to be overcome in the design and construction of civil engineering works. Designers are also used to finding solutions to problems where large measures of risk and uncertainty are present. The risks are appreciably reduced by the application of design standards and codes of practice.

In the design of structures vulnerable to destruction or damage by floods there are no hydrological design standards or codes of practice, other than for dam spillway design. International guidelines and experienced South African hydrologists and designers have stressed the need for engineering judgment in the application of hydrological analyses. However, if hydrologists cannot quantify their uncertainty, how can this uncertainty be accommodated in civil engineering design?

THE DESIGNER'S DILEMMA

The designer's dilemma is illustrated by the comparison of the flood magnitude-frequency (Q-T) relationship of two rivers. The first is the Breë River in the south-western Cape and the second is the Sand River between Pietersburg and Messina in Limpopo. During the February 2000 floods, the railway bridge across the Sand River was destroyed, and two strategically important bridges on the N1 were severely damaged. Repairs to these bridges took several months to complete. Many other bridges across secondary roads within this catchment were either destroyed or severely damaged.

Table 1 is a comparison of the Q-T relationship derived from flow data recorded in the Breë River at Ceres and the Sand River at Waterpoort near Louis Trichardt. The recommended statistical distribution for application in South Africa is the log Pearson Type 3 (LP3) distribution using conventional moment estimators (the mean, standard deviation and skewness coefficient of the logarithms of the data). This is demonstrated in the accompanying paper on the statistical analysis of extreme floods. The designer of a bridge across the Breë River at Ceres could use the estimated 50-year return period flood of 856 m^3/s with confidence. The estimated values from the generalised extreme value distribution (GEV) using probability weighted moments and the generalised logistic distribution (GL) using L-moments are in close agreement with the LP3 value. Furthermore, the 200-year flood is only 18% larger than the 50-year flood. This could be accommodated comfortably in the design of a bridge at this site.

The situation in the Sand River is quite different. Based on the gauged period of record from 1969 to 1997, the 50-year flood using the LP3 distribution is 938 m^3/s. The estimates produced by the other two distributions are in reasonably close agreement with this value.

Two tropical cyclones occurred in 1958 and in February 2000. If the floods caused by these two events are included in the analysis, the estimated 50-year LP3 flood changes dramatically from 938 to 3544 m^3/s – nearly four times higher than the analysis based on the gauged period.

The second difficulty is that the 200-year LP3 flood is nearly three times greater than the 50-year flood. Thirdly – to add to the confusion – the estimates of the 50-year flood produced by the other two distributions are substantially less than the LP3 estimate. The reason is the theoretical weaknesses in the moment estimating procedures in

Table 1 Comparison of the flood magnitude-frequency relationship in two rivers (m^3/s)

Site	Distribution	Return period (years)		
		2	50	200
Breë River at Ceres	LP3	362	856	1 010
	GEV	357	827	990
	GL	352	890	1 185
Sand River at Waterpoort 1969–1997	LP3	68	938	1 517
	GEV	80	855	1 753
	GL	71	880	2 020
1958–2000	LP3	77	3 544	9 291
	GEV	115	2 491	6 743
	GL	71	2 425	7 304

these distributions. The five-parameter Wakeby distribution using probability weighted moment estimators cannot be fitted at all to either the gauged or the extended record. As far as is known, these weaknesses have not yet been reported in the hydrological literature.

In this situation, what is a reasonable estimate for the 50-year design flood in the Sand River? What criteria should the designer use to select this value? More importantly, is there another approach that could be used for specifying the design flood?

The standard design flood (SDF) described in this paper overcomes these difficulties. Although simple in concept and application, it is soundly based on theory and practical experience. It is the hydrological equivalent of the well-known Manning formula. Like the Manning formula it is not perfect, but it can be applied with confidence for most design flood applications.

The basic philosophy of the SDF rests on the need of public administrators and designers for a method that meets the requirements of robustness (it can be applied to any site in a catchment of any size in South Africa); consistency (different users will obtain the same results); simplicity (easy to understand and apply); and sufficient accuracy (of the same order of accuracy as the other design variables and unknowns).

The 150 year old rational method meets these requirements and is the most widely used method in South Africa and internationally. It is the basis of the SDF recommended in this presentation. Users familiar with the application of the rational method will have no difficulty in applying the SDF within existing computer programs or hand calculation methods, using the information provided in this paper. Details of the calculation method are provided in the appendix to this paper. The only other information required for the application of the method is access to a copy of the Department of Environment Affairs (now Department of Water Affairs and Forestry) report TR102 on Southern African storm rainfall (Adamson 1981).

DEVELOPMENT OF THE STANDARD DESIGN FLOOD METHOD

The development of the SDF method was based on an extensive study and re-processing of hydrological and meteorological data. The point of departure was the Department of Water Affairs and Forestry (DWAF) publication *Catalogue of hydrological catchment parameters* (Petras & Du Plessis 1987). This is a catalogue of 31 hydrological characteristics from each of 137 representative catchments that varied in size from less than 10 km^2 to 38 500 km^2. The purpose of the catalogue was to provide a hydrological database for projects and research.

Additional information and data from a number of sources were used in the analyses. These included annual maximum flood data from 152 flow gauging stations upgraded and extended to October 2000 by D van Bladeren of the DWAF; daily rainfall statistics published in the DWAF report TR102 (Adamson 1981); unpublished daily rainfall and other meteorological data from the South African Weather Service (SAWS); digitised short duration rainfall statistics produced by the SAWS and further processed by J Smithers of the University of Natal; monthly district rainfall data produced by the SAWS and updated by L Dyson of the University of Pretoria; and an extensive study of the properties of widespread severe rainfall reported by Alexander and Van Heerden (1990) and further discussed in Alexander (2001).

The study commenced with the direct statistical analysis of the recorded annual flood peak maxima at the 152 representative sites. Details of the methods used are provided in the accompanying paper on the statistical analysis of extreme floods. The adverse effects of low outliers were neutralised by increasing their values to coincide with the corresponding values for the statistically neutral log normal distribution. No adjustments were made to accommodate the critically important high outliers, although they had a major influence on the calculated values for the range of return periods, as demonstrated in table 1. The results of the LP3 analyses at these sites were used for the calibration and verification of the SDF.

The next step was the identification of representative drainage basins. Twenty-nine drainage basins were identified. The map showing the location of these drainage basins is included in the appendix to this paper. A representative daily rainfall station for each drainage basin was selected from TR102 (Adamson 1981). This publication provides statistical analyses of daily rainfall at some 2 500 sites in South Africa, based on the three-parameter log normal distribution. The selection criterion was that the rainfall station should be representative of the meteorological conditions in the drainage basin, and not necessarily the average conditions, bearing in mind that the interest was in the properties of the widespread rainfall events and not point rainfall per se. The selection of the rainfall station was not critical to the analyses, as it was the product of the rainfall derived from the representative station and the values of the representative runoff coefficients that was calibrated.

The rational method was used at each of the sites for which the catchment characteristics were available. Dummy runoff coefficients were used in the first round of analyses. The calculated 2-year and 100-year flood maxima were compared with the LP3 values obtained from the statistical analyses of the recorded data. The output from the rational method was adjusted to agree with the LP3 values by changing the runoff coefficients, as there is a linear relationship between the runoff coefficients and the calculated flood peak. These were the calibrated 2-year and 100-year runoff coefficients (C2 and C100) for the site.

In the second round of analyses the method was applied using the calibrated runoff coefficients C2 and C100. As before, the results were compared with the LP3 values and the revised C2 and C100 values for each site were determined.

Most of the drainage basins had more than one of the representative hydrological stations used in the analyses, each with a pair of calibrated runoff coefficients. Representative runoff coefficients for each drainage basin were derived from upper envelope values of C100 and lower envelope values for C2 – not the average values. In combination they produce a more conservative estimate of the calculated Q-T relationship than average values.

The representative runoff coefficients were used at each site during the third round of calculations with the purpose of verifying the validity of the representative runoff coefficients.

For the verification studies, 124 sites were identified that had reliable records and for which catchment characteristics were available. The 29 drainage basins were grouped into eight larger regions. The number of stations in each region were as follows: northern interior (14); central interior (24); arid areas (8); south-western Cape (20); southern midlands (28); eastern midlands (14); coastal areas (5); and lowveld (11). The basin runoff coefficients were re-adjusted to ensure consistency within each of the wider regions.

The verification studies using the final C2 and C100 values produced estimates of the design flood with values that exceeded the LP3 values by factor of 1,6 on average, and only nine of the 84 stations used for the final analysis had estimated values less than the LP3 values. Unpalatable as it may seem, values between 50% and 200% of the LP3 values are well within the range of other uncertainties inherent in all design flood estimation and application procedures, including the LP3 values them selves.

MODEL PARAMETERS

The only values required for input into the SDF are the size of the catchment, the length and slope of its main stream and the drainage basin in which the site is located. Other algorithms for the calculations are provided in the appendix to this paper and are described in Alexander (2001). They include the widely used Bransby-Williams formula for the calculation of the time of concentration; the modified Hershfield equation for the determination of the point rainfall for the required return period; and an equation for the areal reduction factor. The validity of these algorithms was confirmed in earlier studies of the properties of widespread, flood-producing rainfall.

The only components that are unique to each drainage basin are the statistical properties of the daily rainfall at the representative rainfall station for each drainage basin derived from TR102, and the representative percentage runoff coefficients C2 and C100 for return periods of two and 100 years respectively. The runoff coefficients for other return periods are derived by interpolation between these two values. The method is amenable to hand calculation.

DESIGN HYDROGRAPH

There is not a unique relationship between the flood peak and the flood volume (hydrograph shape). The recommended design hydrograph derived from the SDF has a triangular shape with the duration of the rising limb equal to the time of concentration; the peak value equal to the estimated flood peak; and the duration of the falling limb equal to twice the time of concentration. This assumption is conservative and sufficiently accurate for most applications.

APPLICATION OF THE SDF

Bearing in mind the purpose of the SDF, it is only necessary to consider adjusting the value of the estimated design flood produced by the SDF where there are conditions that are meaningfully different from those used in the calibration process. All other adjustments should be accommodated in the design of the structure, based on engineering judgement. The following are some considerations.

Large values of C2 and C100 pairs indicate that a larger proportion of the representative rainfall contributes to the flood peak. Large proportional differences between C2 and C100 indicate the presence of factors – principally antecedent soil moisture status – that introduce additional variability (coefficient of variation) into the rainfall-runoff process.

The overseas version of the rational method and some South African versions assume a single runoff coefficient for the whole range of return periods. The implication is that the statistical properties of the flood maxima are the same as those of the causative rainfall. This is clearly not the situation over most of South Africa, as demonstrated by the large proportional differences between the representative values of C2 and C100 for the different drainage basins. The use of a single runoff coefficient may be the reason for the recommendation in some presentations that the rational method is only suitable for small urban catchments. The SDF can be applied to all sizes of catchments from 10 km^2 to 40 000 km^2.

There are many factors that influence the rainfall-runoff relationship. These are described in Alexander (2001). The factors that have a large influence on the SDF and need the designer's consideration, are catchments in dolomitic areas where the flood runoff may be less than half of the SDF values, and catchments in the south-western Cape such as the Breë River in table 1 above, that have very flat growth curve, ie low coefficients of variation. In these rivers the SDF may appreciably over-estimate the flood magnitude for long return periods. Otherwise, the SDF produced satisfactorily conservative results in the representative catchments used in the development of the method.

POSSIBLE RESERVATIONS

A common misconception is that methods for the determination of the Q-T relationship such as the SDF can be tested by comparing rainfall and flood runoff produced by observed events. It must be appreciated that these are altogether different concepts. The concept used for design is that on average, based on a large number of events, over a large region, the relationship holds. The rainfall-runoff relationship at a specific site will only be the same as that produced by a probabilistic method if the site characteristics as well as the hydrological and meteorological conditions are the same as the average conditions used for the development of the method. This is unlikely. Similarly, the SDF may not be tested by comparing the results with those produced by statistical analyses or other deterministic methods at a specific site. The only valid test is by applying numerical verification procedures, based on the analyses of large, representative samples. Conversely, the values of the coefficients C2 and C100 should not be used in the application of the conventional rational method.

Another possible criticism is that the method is too simple and that the introduction of additional parameters would improve the reliability of the method. It must be appreciated that each additional parameter brings a range of uncertainty with it, with the result that the band of uncertainty around the model output increases instead of decreases. This difficulty was discussed at length in early literature on hydrological modelling, where phrases such as the curse of the dimensionality and the need for parsimony of parameters were used. There are many examples of conceptually simple, three-parameter hydrological and hydraulic models that are successfully used in practice in South Africa and elsewhere, and higher order hydrological models that have failed to meet the claims of their developers.

A general criticism of the inadequacy of the rational method that is occasionally voiced in the literature, is based on a misunderstanding of its purpose. The rational method is a design tool that has to meet a number of objectives and is not a single purpose procedure. Anyone who remains critical of the SDF should address the designer's dilemma posed in table 1 of this paper.

RECOMMENDATIONS

Design flood applications can be divided into three broad categories: important structures and structures where public safety is at risk (for example road bridges); structures where the risk of loss of life is minimal (for example minor urban drainage works); and applications where the purpose is administrative only (for example designated floodlines in urban areas).

Where public safety is at risk, the design philosophy should be the same as that applied to other civil engineering structures. Cost is not a valid reason for relaxing this requirement any more than it would be for other structures. The tried and tested regional maximum flood (RMF) method (Kovács 1988) should be obligatory for all structures in this category, other than dams where more rigid criteria have been codified. This does not imply that all bridges have to be above the RMF level. The requirement is that they should be able to withstand the RMF without public safety being endangered.

Approach embankments in the path of the floodwaters (not necessarily the whole width of the floodplain) should be protected against erosion by overtopping. The deliberate lowering of approach embankments, or the inclusion of weaker breaching sections, is neither legally nor socially acceptable.

Where public safety is not at risk, cost optimisation procedures based in the first instance on the SDF for the specified return period can be used to determine the optimum design. Utility criteria should be included in the calculations for important structures in this category, ie the value of the structure to users, and losses incurred by users if the structure fails. These will have to include judgmental estimates of the socio-economic benefits and consequences of failure.

Consistency is the dominant consideration for the location of designated floodlines. This is particularly the case where 100-year floodlines are specified, as different methods will produce substantially different results. The SDF meets this requirement.

In situations where hydrologically unusual conditions are present, the problem should be accommodated by increasing the design waterway for example. The responses should be engineering decisions which are within the knowledge and experience of the designer, and not sophisticated hydrological analyses which are beyond the knowledge and experience of most designers.

Designers therefore have a solidly based method for the determination of the design flood that meets the requirements of robustness, consistency, simplicity and sufficient accuracy, and does not require an advanced knowledge of flood hydrology for its application or interpretation.

CONCLUSIONS

Floods are the result of the combination of natural phenomena that have no upper bound. Just as there is no quantifiable upper limit to the magnitude of a flood that may occur at a site, it is also not possible to quantify the degree of uncertainty in the results of hydrological analyses. This is demonstrated in the accompanying paper on the statistical analysis of extreme floods.

The frequent failure of bridges and other civil engineering structures in South Africa indicates that the hydrological specifications used by designers are not sufficiently conservative. In this paper it is recommended that the RMF method be used for the design of important structures or structures where public safety is at risk, other than for the design of dam spillways where more stringent specifications apply. For all other structures the SDF method as described in this paper is recommended for general application.

Practitioners faced with claims that another method is more accurate than the SDF, and is therefore also superior to the SDF, should appreciate that the SDF is a design tool that has to meet the designer's requirements, only one of which is related to incremental accuracy. It must also be borne in mind that accuracy and robustness are non-commensurate objectives (one can only be achieved at the expense of the other). Any perceived shortcomings in the SDF method at a specific site should be accommodated by applying engineering factors of safety in the design of the structure, and not in attempts to apply other methods of hydrological analysis, all of which have unquantifiable bands of uncertainty about their results.

Details required for the application of the SDF method are supplied in the appendix to this paper. Further information on the development of the method, its application, and the computer program will be made available from the website of the Department of Civil and Biosystems Engineering of the University of Pretoria in due course.

References

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Alexander, W J R 2001. *Flood risk reduction measures.* Department of Civil Engineering, University of Pretoria.

Kovács, Z 1988. *Regional maximum flood peaks in Southern Africa.* Department of Water Affairs Technical Teport TR137.

Petras, V & Du Plessis, P H 1987. *Catalogue of hydrological catchment parameters.* Department of Water Affairs, Flood Studies Technical Note 6.

Additional references relevant to design flood estimation problems are listed in the accompanying paper on the statistical analysis of extreme floods.

APPENDIX

Calculation procedure for the standard design flood

The only user input required is the size of the catchment, the length and slope of the main watercourse and the identification of the drainage basin within which the site is located from figure 1. The computer program (or hand calculation) does the rest. The calculation procedure is as follows. More details are provided in Alexander 2001. The required values of the variables are listed in table 2.

The second and third columns in table 2 are the SAWS station identification numbers from TR102. This publication provides the information required for determining the point rainfall for the specified return period and the calculated time of concentration in step 6 below. *M* is the average of the annual daily maximum rainfalls, and *R* is the average number of days per year on which thunder was heard. These two values are used in the modified Hershfield equation 2. *C2* and *C100* are the runoff coefficients used in equation 4. *MAP* is the mean annual precipitation and *MAE* is the mean annual Symons Pan evaporation. These two values are supplied for information only, and are not used in the analysis. They indicate the substantial role played by antecedent evaporation in the flood rainfall-runoff process, and how annual evaporation varies inversely with annual rainfall.

Calculation procedure

1 Identify the drainage basin in which the site is located from figure 1. (A more detailed map is provided in the user's manual.) Read the values of the parameters used in the calculations from table 2.

2 Identify the site on a topographical map, preferably 1:50 000 scale, and demarcate the catchment boundary.

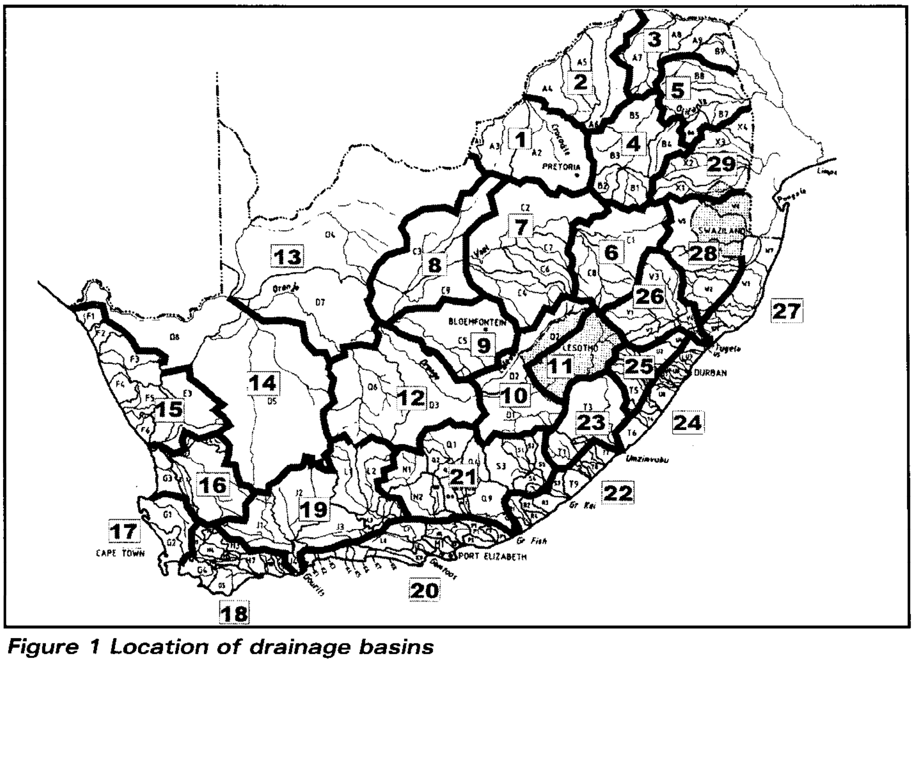

Figure 1 Location of drainage basins

Table 2 Information required for the calculation of the SDF

Basin	SAWS station number	SAWS site	M mm	R days	C2 %	C100 %	MAP mm	MAE mm
1	546 204	Struan	56	30	5	40	550	1 800
2	675 125	Autoriteit	62	44	5	30	450	1 900
3	760 324	Siloam	64	28	5	40	470	1 700
4	553 351	Waterval	58	20	10	50	630	1 600
5	680 059	Leydsdorp	78	10	10	50	620	1 700
6	369 030	Siloam	51	54	10	60	670	1 500
7	328 726	Olivine	49	39	10	60	510	1 700
8	322 071	Danielskuil	47	39	5	20	380	2 100
9	258 452	Jacobsdal	43	47	10	40	380	1 800
10	233 049	Wonderboom	54	55	10	50	560	1 600
11	236 521	Mashai	39	66	15	70	430	1 400
12	143 258	Scheurfontein	39	52	5	30	290	2 100
13	284 361	Wilgenhoutsdrif	40	55	2	15	270	2 600
14	110 385	Middelpos	25	13	2	20	140	2 400
15	157 874	Garies	22	11	4	20	130	2 100
16	160 807	Loeriesfontein	28	11	10	40	210	1 900
17	84 059	Redelinghuis	28	1	20	50	260	1 500
18	22 113	La Motte	59	4	20	40	810	1 400
19	69 483	Letjiesbos	34	16	5	30	160	2 200
20	34 762	Uitenhage	53	12	10	50	480	1 600
21	76 884	Albertvale	45	23	10	35	460	1 700
22	80 569	Umzoniana	84	26	15	60	820	1 200
23	180 439	Insizwa	60	45	10	80	890	1 200
24	240 269	Newlands	76	15	15	80	910	1 200
25	239 138	Whitson	55	9	10	80	830	1 200
26	336 283	Nqutu	61	17	10	50	760	1 500
27	339 415	Hill Farm	85	17	15	60	890	1 400
28	483 193	Maliba Ranch	75	54	5	40	740	1 400
29	556 088	Mayfern	66	11	5	40	740	1 600

Copy the boundary onto tracing paper and place over 5 mm or similar squared paper. Count the number of squares within the catchment including squares more than halfway into the catchment. Apply a factor to convert the number of squares to the catchment area A (km^2).

3 Identify the main channel on the map from the site to the catchment boundary, and measure its length using dividers set at 0,2 km (1,0 km on a 1:250 000 map). In the latter case multiply the length by a scale factor of 1,2 to compensate for the loss of resolution. Derive the length of the main channel L (km).

4 Determine the elevation of the main channel in metres at two points located at 10% and 85% of the main channel length upstream of the site. Divide the difference in elevation between these two sites by 75% of the main channel length. This is the 1085 slope S (m/km).

5 Apply the Bransby-Williams formula to determine the time of concentration tc (hours).

$$tc = \left[\frac{0{,}87L^2}{S}\right]^{0{,}385} \qquad (1)$$

6 Convert tc (hours) to t (minutes). Determine the point precipitation depth P (mm) for the time of concentration t (min) and the return period T (years). If the time of concentration is more than 24 hours use linear interpolation of the values in the reference rainfall station from TR102 listed in table 2. Otherwise use the modified Hershfield equation below. The computer implementation interpolates the values between four hours and one day.

$$P_{t,T} = 1{,}13(0{,}41 + 0{,}64\ln T)(-0{,}11 + 0{,}27\ln t)(0{,}79M^{0{,}69}R^{0{,}20}) \qquad (2)$$

where

M is the mean of the annual daily maxima from table 2

R is the average number of days per year on which thunder was heard, from table 2

7 Multiply the point precipitation depth $P_{t,T}$ (mm) by the area reduction factor ARF (%) to determine the average rainfall over the catchment for the required return period T (years). The corresponding rainfall intensity I_T (mm/h) is obtained by dividing this value by the time of concentration.

$$ARF_T = (90000 - 12800\ln A + 9830\ln t)^{0{,}4} \qquad (3)$$

8 The above steps are the standard procedure used in the conventional rational method. The SDF uses calibrated runoff coefficients C2 (2-year return period) and C100 (100-year return period) from table 2 instead of determining them from catchment characteristics. The runoff coefficients for the range of return periods T (years) are derived by applying the return period factors YT in table 3, using the relationship in equation 4:

Table 3 Return period factors

T = 2	10	20	50	100
YT= 0	1,28	1,64	2,05	2,33

$$C_T = C2/100 + (YT/2{,}33)(C100/100 - C2/100)) \qquad (4)$$

9 Finally, the flood peak Q_T (m^3/s) for the required return period T is calculated from

$$Q_T = 0{,}278C_T I_T A \qquad (5)$$

which is the standard format used in the rational method.

10 For those who already use the 1991 program DETFLOOD or the 2001 versions of the computer programs for the rational method, the SDF can be derived by using the representative rainfall station from table 2, and using the user-selectable calibration factors to adjust the runoff coefficients to the required values. This requires using dummy runoff coefficients derived from catchment characteristics in the first round and then adjusting the runoff coefficients proportionally to obtain the specified SDF runoff coefficients.

11 The new computer program SDF and user's manual will be incorporated in the revised suite of computer programs for flood magnitude-frequency analysis distributed by the Department of Civil Engineering of the University of Pretoria. The computer program includes all the rainfall and other information required for the application of the method. The only information that the user has to provide is the area of the catchment, the length and slope of the main watercourse, and the drainage basin in which the site is located.

Note

It is essential that the above procedure and equations be used to determine the SDF. Under no circumstances should alternative sources of information or equations be used, as this will invalidate the calibration and verification procedures on which the SDF is based.