MODERNISED SOUTH AFRICAN DESIGN FLOOD PRACTICE IN THE CONTEXT OF DAM SAFETY

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WRC Report No. 1420/2/07

Water Research Commission

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Report to the Water Research Commission of South Africa on the project

"Updated Guidelines and Design Flood Hydrology Techniques for Dam Safety"

WRC Report No 1420/2/07

ISBN 978-1-77005-572-8

Set No 978-1-77005-570-4

May 2007

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This report forms part of a series of reports. The other reports in the series are:

Cullis J., Görgens A.H.M, Lyons S. 2006. *Review of the Selection of Acceptable Flood Capacity for Dams in South Africa in the Context of Dam Safety* (WRC Report no 1420/1/07)

Görgens A.H.M. 2006. *Joint Peak-Volume Design Flood Hydrographs for South Africa* (WRC Report no 1420/3/07).

WRC/Ninham Shand. 2006. *Dam Safety Hydrology Toolbox*[1]

[1] The web-based Toolbox is available on the Ninham Shand web-site, or can be obtained on CD from Ninham Shand, PO Box 1347, Cape Town, 8000 (Tel. No. 021-4812400).

EXECUTIVE SUMMARY

1. INTRODUCTION

In the early 1990s the South African National Committee on Large Dams (SANCOLD) issued a set of *Guidelines on Safety in Relation to Floods* (SANCOLD, 1991), as well as a compendium of South African Design Flood determination techniques (Alexander, 1990) to provide guidance to those charged with evaluating the safety of existing dams as well as to the designers of new dams.

More than 15 years have elapsed since the publication of the SANCOLD *Guidelines*, which have by now informed safety evaluations for hundreds of registered dams. There is a distinct need to take stock of the footprint achieved by the *Guidelines* and update them in terms of international best practice. Furthermore, apart from the South African customisation of the SCS technique (Schmidt and Schulze, 1987) for small catchments, there has been no lasting establishment of new tools for the generation of complete design flood hydrographs for dam safety-related applications for more than 30 years[2]. Thus, there exists a strong argument in favour of bringing the flood hydrograph-related information contained in the streamflow records of the last three decades into South African design flood practice.

In this context the Water Research Commission of South Africa (WRC) appointed Ninham Shand to undertake flood-related research with the following original objectives:

i) *To establish updated Guidelines for the safety evaluation of dams in relation to floods*
ii) *To derive a methodology for design flood hydrograph estimation based on joint occurrence of flood peaks and flood volumes, through analysis of historically measured flood hydrographs in all regions of South Africa*
iii) *To develop a modernised set of design tools for the generation of complete flood hydrographs for dam safety evaluation or spillway design.*

To address the above objectives, the project was divided into four phases:

- Phase 1: Assessment of Local and International Practices Regarding Dam Safety in Relation to Floods
- Phase 2: Improvement of Flood Hydrograph Generation Techniques for South Africa for Dam Safety Purposes
- Phase 3: Development of a “Design Flood Hydrograph Toolbox”, the purpose of which is to support the various components of dam safety evaluation in relation to floods.
- Phase 4: Review of the *SANCOLD Guidelines on Dam Safety in Relation to Floods*.

This Report is a culmination of the Interim Research Reports written during Phase 2 of this project, and mainly addresses objectives ii) and iii) above. This Report also serves as a supporting document for the products developed out of Phases 3 and 4 of this Study, which yielded the following final outputs: a web-based "*Dam Safety Hydrology Toolbox"* and a document entitled "*The Selection of Acceptable Flood Capacity for Dams in South Africa in the Context of Dam Safety"*. It should be noted that during early 2005, upon reviewing the outcomes of a questionnaire survey and a Workshop involving about 30 dam safety practitioners, the Reference Group for the Study modified the focus of the first objective from an "updating" of the *Guidelines* to a "review" of some of the key concerns regarding the *Guidelines*.

[2] The innovative and important Runhydrograph design flood methodology by Hiemstra and Francis (1979) has unfortunately not become established in South African practice. Possible reasons for this are outlined in the sister document to this, entitled: *Joint Peak-Volume Design Flood Hydrographs for South Africa,* Görgens (2006).

This report has been divided into three separate Parts. Following the **Part 1**: "*Introduction*", **Part 2**: "*Establishment of Flood Database and Flood Hydrograph Extraction Software*", provides information on the development of the flood database, by considering the flood data sources and the screening of the primary data. The development and use of the flood hydrograph extraction software "*EX-HYD*" is also described in detail. **Part 3**: "*Modernised Perspectives on Design Flood Methodologies*" consists of a review of international and local design extreme rainfall approaches; a review of local and international approaches to design storm losses; a comparison of HRU Unitgraph-based design flood estimates with probabilistic-based estimates; and finally for gauged catchments; and a modernised approach to extreme design flood concepts in South Africa in the context of dam safety, which includes an extensive local and international literature review.

A further component of the flood hydrology research conducted under this Study is presented in a stand-alone document entitled: "*Joint Peak-Volume Design Flood Hydrographs for South Africa*", which details the derivation of a methodology for design flood hydrograph estimation based on the joint occurrence of flood peaks and flood volumes. That document also includes a design flood peak/flood volume methodology for ungauged sites based on catchment descriptors and regional/pooled probabilistic flood analysis.

2. ESTABLISHMENT OF FLOOD DATABASE AND FLOOD HYDROGRAPH EXTRACTION SOFTWARE

At the inception of this Study, the Water Research Commission recognised the need to develop a comprehensive and up-to-date flood database for South Africa, which could be used for the extraction of complete flood hydrographs for dam safety evaluation and design applications. This Study team therefore developed a comprehensive floods database as well as customised software, *EX-HYD*, for the extraction of flood hydrographs from primary stage records of the Department of Water Affairs and Forestry (DWAF).

For the development of the database, an aggregate sample of flow gauges used in previous studies of design flood estimation (Hiemstra and Francis (1979), HRU (1972) and Alexander (2002)) were considered. The flow records were initially put through two screening processes based on specific criteria such as gauge type, record length, size of catchment area, as well as reliability of the flow record in terms of obvious errors or known problems. Also, only continuous flow records were considered, i.e. flows recorded by flow gauges with automatic recorders. Following this screening process, a data set of 109 gauging stations was eventually considered for use in this study.

The next stage involved the identification and abstraction of complete flood hydrographs from the floods data set. Due to the large number of flood events to be extracted from the data set, computer software was developed, namely *EX-HYD*, to assist in identifying and extracting complete flood hydrographs. During the process of extracting the hydrographs, unreliable records were discarded for the following reasons: primary data, rating tables or annual peak information was not available; too many exceedences of the rating table beyond the acceptable criteria; large data gaps resulting in short usable periods; data errors in running the software; non-stationarity of mean peaks. Following this third and final screening process, 65 stations remained whose records were used as input into *EX-HYD*.

Input to the *EX-HYD* extraction software are primary stage records obtained from DWAF (for the selected flow gauges). The following flood characteristics are extracted for each event:

- Hydrograph peak and associated volume and duration
- Plot/graphical representation of hydrograph
- Hydrograph in text format for potential use in hydrology research and applications.

In addition to the 109 gauges considered for this study, an additional set of flood related information was obtained from the DWAF Flood Studies Section. Using the flood hydrograph information recorded at the flow gauging station immediately downstream of a reservoir, a "back-routing" approach was followed in order to determine the inflow hydrograph to the reservoir. From these inflow hydrographs, DWAF determined the hydrograph peak and volume. DWAF conducted the above exercise for 83 reservoirs across South Africa.

3. MODERNISED PERSPECTIVES ON EXISTING DESIGN FLOOD METHODOLOGIES IN SOUTH AFRICA

3.1 REVIEW OF EXTREME DESIGN RAINFALL APPROACHES IN SOUTH AFRICA

For South Africa, the only established guidelines for the estimation of PMP are those set out in the HRU 1/72 Report (HRU, 1972). Although the HRU approach represented a conservative and pragmatic approach for the estimation of PMP, it was also based on only about 30 years of rainfall data from 1932 to the 1960s. Since the 1960s, however, South Africa has experienced several large flood events, some of which caused extreme damage and loss of life. The aim of this research was to check the HRU PMP envelope curves for both large- and small-area storms against estimates based on the latest available rainfall data and, where necessary, propose improvements or sound cautions to practitioners using the HRU PMP envelope curves in design.

To assess the current applicability of the HRU PMP curves for large-area storms, six severe storms were selected based on literature information and institutional knowledge. For each storm a detailed site-specific analysis was performed where the storm isohyets were ultimately produced for critical durations. The average areal rainfall at a particular storm duration was then calculated and plotted against the appropriate HRU PMP envelope curve (identified by HRU region) of maximum probable precipitation (mm) versus area (km^2).

The results obtained in the above analysis are summarised in Table 3.1. A tick in Table 3.1 indicates that the HRU PMP envelope curves were exceeded in that instance, while a cross indicates that they were not exceeded. As can be seen, the HRU PMP envelope curves were exceeded on a number of occurrences. In particular, the KwaZulu-Natal floods of 1987 stand out. The number of crosses in Table 3.1 is somewhat deceiving, in that, although the HRU PMP curves were not exceeded, the storm rainfalls were approaching the curve. The Limpopo floods of 2000 are an example of this, as are the Orange River floods in 1988.

These results suggest that the HRU PMP envelope curves for large-area storms may be underestimating the maximum precipitation which could occur in a number of regions in South Africa, particularly considering that the HRU PMP envelope curves were developed from maximised and transposed storms, while the storms used in this research have not been maximised or transposed. The HRU PMP curves may therefore no longer be the best indicators of the maximum design rainfall that should be used in certain regions of South Africa.

For the small-area storms, the maximum envelope curve for the entire country (Figure C4 of HRU report No 1/72) was exceeded by the current data on three occasions and the regional curves approached or exceeded in four instances. This once again highlights that the HRU envelope curves may be underestimating the short duration extreme rainfall.

Table 3.1 Summary of checks on current applicability of HRU PMP envelope curves for Large-Area storms#

Flood	Year	HRU Region affected	METHOD USED TO DETERMINE STORM BOUNDARY									
			HRU Region				Proportion of peak			Storm Cells		
			1-day	2-day	3-day	4-day	1-day	2-day	3-day	1-day	2-day	3-day
Cyclone Domoina[3]	1984	Region 17 (500 – 1000 mm)				✓						
Limpopo	2000	Region 18 (250 – 500 mm)		✓		✗						
				✗								
		Region 18 (500 – 1000 mm)		✗		✗						
				✗								
Orange River Basin	1988	Region 10 (250 – 500 mm)								✓	✗	
										✗	✗	
KwaZulu-Natal	1987	Region 13 (1 000 mm+)	✓	✓	✓		✓	✓	✓	✓	✗	✗
Laingsburg	1981	Region 5 (0 – 500 mm)								✗	✗	
		Region 6 (250 – 500 mm)								✗	✗	
South Eastern Cape	1981	Region 6 (250 – 500 mm)	✓	✗	✗							

# Tick = HRU PMP curves exceeded or approached.
Cross = HRU PMP curves not exceeded.

[3] Cyclone Domoina was not analysed by covering a complete HRU region, however, the method of analysis is closest to this method.

3.2 REVIEW OF REGIONAL DESIGN STORM LOSSES IN SOUTH AFRICA

For the investigation into regional storm losses in South Africa, this study considered various methods, all of which were bench-marked against the original "average" storm losses approach undertaken by the HRU and documented in their 1972 report. The HRU approach combined design rainfall and Unitgraph-based flood details to derive flood volumes for different RIs. The HRU methodology was chosen as the benchmark methodology, given that most South African dam safety assessments deal with catchments larger than the sizes allowed by the SCS conventions.

Methods 1 to 4 considered the "average" design storm losses approach as reported in HRU 1/72, but with each Method progressively deviating from the latter approach with increased utilisation of additional data for the estimation of the flood volumes. Method 1, for example, remained true to the HRU methodology for the derivation of the design flood volumes, whereas Method 4 deviated wholly from the HRU methodology by making use of observed historical floods in representative catchments. All methods deviated from the HRU methodology with respect to the design rainfall in that the design rainfall produced by Smithers and Schulze (2002) was used throughout. Method 5 investigated the regional approach to minimum design storm losses by the consideration of extreme historical floods and their causative observed rainfall.

From the results of the analysis for Methods 1, 3 and 4, one of the overriding findings is that the existing HRU regional storm losses curves (Figure G2 of HRU Report No 1/72) can be seen to be broadly representative of mid-range values for Veld-Zone Group A (Veld-Zone 2) and Veld-Zone C (Veld-Zones 1, 3, 8 and 9). This indicates that the HRU regional storm loss curves, which are described as "average curves" in the 1972 report, might still be considered as reasonably representative for the estimation of "average" design storm losses for these two groups of Veld-Zones within South Africa.

Another overriding finding for Veld-Zone Group A was that a number of RI flood volumes exceeded 100% runoff. This outcome, which was evident in all of the Methods, indicated that the design rainfalls in these regions could be too low. This could be expected in mountainous regions, such as Veld-Zone 2, where there is a lack of representative high-elevation, high-rainfall records.

A concern brought to light by this research is that the "average" HRU storm loss curve for Veld-Zone Group B (Veld-Zones 4, 5, 6 and 7) might be under-estimating the storm runoff percentage.

For the HRU regional minimum storm loss curves (Figure G1 of HRU 1/72), the results of this research indicate that the representative HRU envelope curves may still be considered as valid for use within South Africa for Unitgraph-based PMF estimates. For this section of the research, observed historical extreme flood events were considered and it was found that for only one storm-catchment combination the inner envelope curve, labelled as "envelope of recorded floods", was exceeded, and that the outer envelope curve, labelled as "estimate of maximum runoff efficiency", was never exceeded.

3.3 COMPARISON OF UNITGRAPH- BASED DESIGN FLOOD ESTIMATES WITH PROBABILISTIC ESTIMATES

As the HRU (1972) Unitgraph-based method is still the most commonly used design hydrograph generation approach in South Africa, comparison of Unitgraph-based design flood estimates with probabilistic estimates was a necessary additional investigation in this study. To this end, the probability distributions, Log Pearson Type III (LP III) and General Extreme Value (GEV_{pwm}), which are commonly used in South Africa, were employed.

Design flood estimates were determined for 40 gauged catchments for recurrence intervals 1:2, 1:5, 1:10, 1:20, 1:50 and 1:100 years for each of the catchments and grouped according to the Veld Type Zone Groups A, B and C of HRU (1972). The comparisons of Unitgraph-based and probabilistic flood peak estimates were presented in three different ways: scatterplots of Unitgraph-based estimates versus GEV_{pwm} and LPIII probabilistic flood estimates for each Veld Zone Group; box-plots showing the quartile range values of the percentage difference between the Unitgraph and the LPIII values, arranged by RI and produced for each Veld Zone Group; and box-plots showing the quartile ranges of standardised values of Unitgraph and LPIII estimates, arranged according to RI and produced for each Veld Zone Group.

The scatterplots show that the Unitgraph-based approach generally produced higher design flood peak estimates for Veld-Zone Groups B and C than the two single-site probability analysis approaches. The quartile range of the proportional differences was seen to be wide across all RIs. The standardised quartile box-plots indicate that the LP III approach is characterised by higher variability in scaled flood peak estimates than the Unitgraph-based approach, over all RIs, for Veld-Zone Groups B and C. The lack of relative variability in the estimates by the Unitgraph-based approach across the range of RIs could seem to be a cause of concern.

3.4 REVIEW OF EXTREME DESIGN FLOOD APPROACHES IN SOUTH AFRICA

The aim of this research was to investigate the link between recurrence interval (RI), or annual exceedence probability (AEP), and the RMF and PMF for South Africa. This information would inform future applications of the *SANCOLD Guidelines*, which incorporate the use of these two extreme design flood concepts. This research seeks a better understanding of the differences between the RMF and PMF, which could have an impact on their applicability when designing a structure such as a dam, or in performing a dam safety evaluation.

As an overall finding from the foregoing analyses, we would like to propose two RI indices to help orientate hydrological practitioners in South Africa regarding the possible RI of the RMF and PMF. These are presented in Table 3.2. We have based these on the GEV_{pwm} results as these appear to generally provide more conservative RIs for the extreme floods.

Table 3.2 Recommended median and lower 95 percentile RI (years) for the RMF and PMF

Design Flood	Median	Lower 95 Percentile	Probability Distribution
RMF	3000	200	GEVpwm
PMF	30000	600	GEVpwm

An alternative approach for the estimation of the RMF RI, which used Kovaćs's original methodology from his 1988 TR 137 report, was also employed. This approach showed that the RI of the lower 95 percentile and median RMF appeared to be around 1:400 and 1:700 years, respectively. By analysis of the longer flow records than those used by Kovaćs in his research prior to 1988, we checked Kovaćs's envelope curves for the various regional K-values. The recently recorded flood peaks were found to either approach the curves or, in some cases, exceed the curves in K-regions 3.4, 5, 5,4 and 5.6. This indicates that the boundaries of the original K-regions might need to be adjusted to accommodate the more recent higher flood peaks.

In terms of the PMF, the results of the study showed that the median RI for the PMF might be greater than the median RI for the RMF by about a factor of 10. Also, for the comparison of the magnitudes of the two extreme floods on a national scale, a spatial distribution of PMF/RMF was produced. This indicated a trend in which the PMF/RMF ratios increase from the coast towards the interior, with the smaller PMF/RMF ratios of < 1 evident along the coastal area of the Western Cape and Eastern Cape, and the largest PMF/RMF ratios of > 4 evident for the northern parts of the country. For more than half of the country, the magnitude of the PMF was found to be larger than twice the magnitude of the RMF. These results are not surprising as, by definition, the PMF should be the most extreme flood that could be expected within a catchment. This is also in line with the roles these two extreme floods play in the SANCOLD Guidelines, where the PMF is clearly expected to be a larger value than the RMF. The wide spatial variation in the PMF/RMF values might indicate possible inconsistencies in the methodology used to derive the PMF, as opposed to the RMF methodology, which is considered as generally more structured.

An area of concern surrounding this research might be that very high RIs were extrapolated from flow records of only moderate length, using the "tail-end" of the various probability distributions, which might entail extensive uncertainty. For this reason, the results presented in this project are based on the application of three different probability distributions, and the values of the RI for the design floods are presented in terms of "RI bands", and percentile values.

4. JOINT PEAK-VOLUME DESIGN FLOOD HYDROGRAPH ESTIMATION METHODOLOGY[4]

While there has been continuing research interest during the past two decades in the use of probabilistic analysis for determining design flood peaks, there have been few attempts to incorporate into this research the exceedence probability of flood volumes. For dam safety assessment in relation to floods, it is important that flood volume be considered, particularly with regards to the spillway capacity design and safety evaluation for medium to large dams that require site-specific investigations.

Hiemstra and Francis (1979) conducted innovative research on the statistical relationships between flood peak and flood volume for South African rivers, which led to the so-called "Runhydrograph" design flood approach. This research showed that peak-volume pairs sampled from 43 flow gauging station records were approximately log-Normally distributed in bi-variate space. The Runhydrograph approach, however, is not in general use in the dam safety field in South Africa. This may be due to the fact that the methodology was not developed specifically for dam safety professionals and therefore tended to fall outside their "comfort zone". It appears that some design flood hydrology practitioners were also concerned about the validity of the annual exceedence probability relationships of the peak-volume pairs used in the Runhydrograph approach.

In a promising recent development the WRC has been funding a pilot study into the generation of credible design flood hydrographs for certain KwaZulu-Natal catchments by means of continuous rainfall-runoff modelling, using the ACRU agro-hydrological catchment model (Smithers, personal communication, 2006).

Against the above background, the empirical relationships between flood peak and flood volume, on a conditional basis, have been researched as part of this Study. This new approach is called the Joint Peak-Volume (JPV) Design Flood Hydrograph Methodology.

With the aid of the *EX-HYD* software, significant flood hydrographs were extracted, on a "peak-over-threshold" (POT) basis, from primary stage records provided by the DWAF for more than 200 flow-gauging stations, as well as the inflowing flood peaks and volumes for more than 80 dams across South Africa. These partial duration flood peak sequences were screened for statistical stationarity and other evidence of unacceptable upstream human impacts.

[4] It should be noted that the detailed reporting of the research to which this Section 4 of the Executive Summary refers, has been produced as a stand-alone document, entitled: "*Joint Peak-Volume (JPV) Design Flood Hydrographs for South Africa*". This was done because the JPV document was expected to be used more as a design flood estimation "manual", than as a research report. Nevertheless, given that a comprehensive research effort did underlie the JPV Methodology, the Executive Summary of the JPV Report is included here for completeness of referencing to all the research conducted under this Study.

The 12000+ joint peak-volume (JPV) pairs and 9000+ flood hydrographs that were extracted from the 139 gauging station and dam inflow records that survived the screening were appropriately standardised to facilitate examination in various alternative regionally pooled groupings. This examination broadly confirmed the log-Normal character of the POT partial duration data sets.

For analysis purposes the joint peak-volume data pairs, as well as typical standardised observed flood hydrographs, were organised in “pooling-groups” according to two alternative regionalisation schemas that are well-established in South African design flood practice. These are the Veld Type Zones proposed in HRU (1972) and the K-Value regions for the Regional Maximum Flood approach proposed by Kovaćs (1988).

Exceedence percentiles of "standardised volumes conditional on standardised POT peaks" were derived for each of the regional pooling options. The locus of each of these exceedence percentiles in joint peak-volume space displayed a fundamentally linear character. Therefore, the JPV design tools developed include, inter alia, a set of linear functions that describe the exceedence relationships of standardised flood volumes conditional on standardised POT flood peaks for two alternative sets of regionally pooled catchments. As an illustration, the exceedence percentile relationships for the "High K-Region" pooling-group are shown in Figure 4.1.

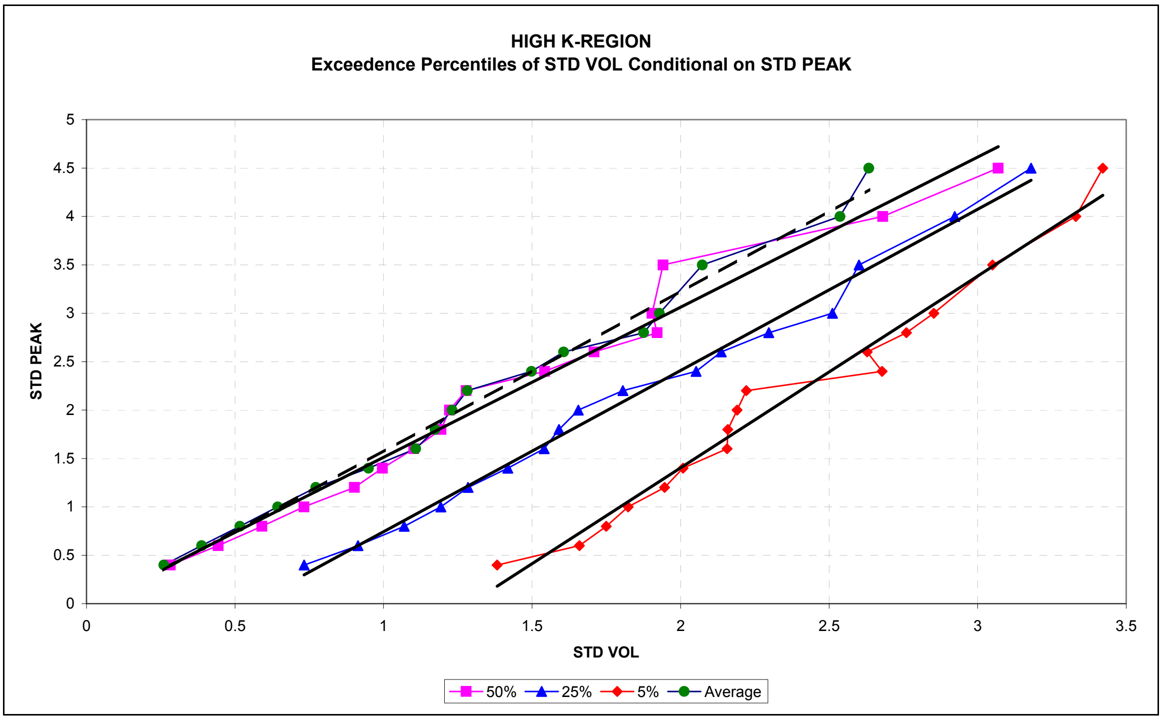

Figure 4.1: Relationship between standardised flood volumes and flood peaks for the high K-Value Regions

A design flood peak provides the entry point to the JPV methodology. Once a design flood peak for a particular catchment has been determined by any method, be it empirical, deterministic, probabilistic, or their hybrids, one or more relevant typical standardised observed hydrographs as shown in Figure 4.2 are selected and then dimensionalised via that design flood peak and the catchment's Basin Lag (as per HRU, 1972). The linear JPV exceedence percentile functions are then used to determine the "severity" or "conservativeness" of the design flood hydrographs in conditional volume terms.

The Report includes a pair of case studies for two widely differing catchments in which multiple 1:50 year design flood hydrographs are generated via the JPV approach and juxtaposed with a conventional Unitgraph-based design flood hydrograph. Analysis of the resulting flood volumes suggest that the Unitgraph-based volumes in this case have surprisingly high exceedence frequencies (>75%), i.e. they are perhaps not conservative enough, whereas the JPV-based volumes have more acceptable (in terms of conservativeness) exceedence percentiles of 50% to 20%.

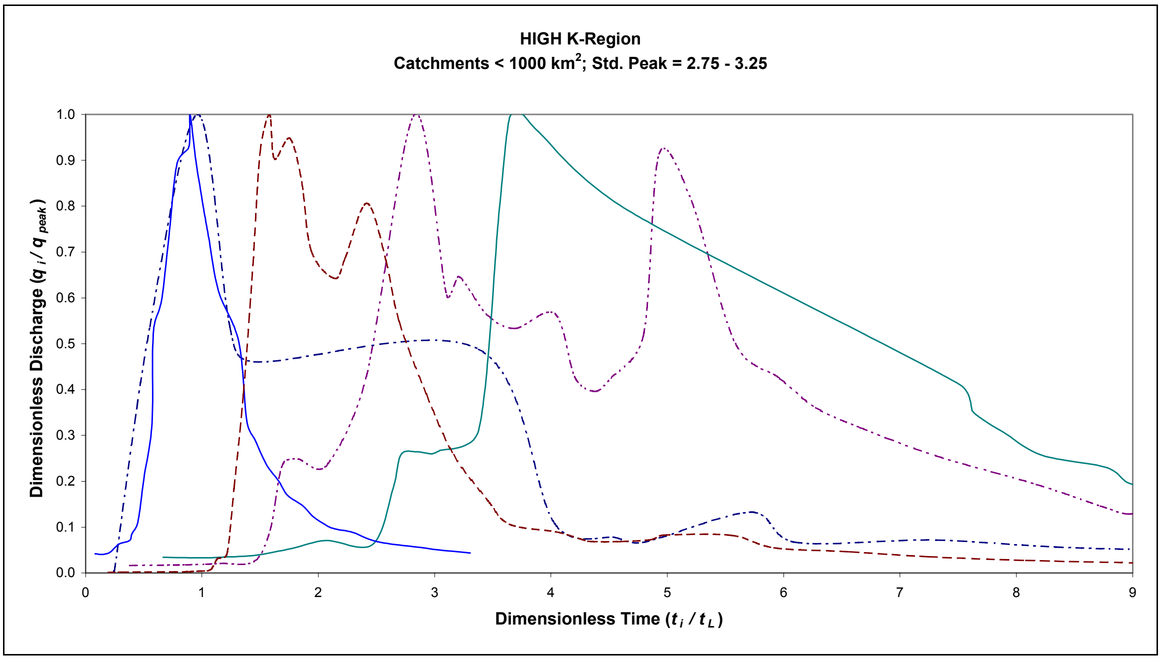

Figure 4.2 Typical standardised hydrograph shapes for the "High K-Region"

In terms of design flood peaks, specifically, this research has made us aware that a modernised alternative to the ageing design flood peak estimation methods used in South African practice would be useful to design flood practitioners. Therefore, the JPV approach also includes a regional pooling method that allows the estimation of design flood peaks and volumes at ungauged sites for any given RI, based either on the two sets of large-scale pooling-groups outlined earlier, or on customised localised groupings of "hydrologically similar" catchments. For these two pooling approaches, we coined the names of "wide pooling" and "narrow pooling", respectively.

This hybrid flood peak estimation method comprises both multi-variate regression equations for index flood estimation (in which catchment descriptor values are used), and empirically weighted pooling of the statistical parameters of observed flood records for the two sets of large-scale pooling-groups outlined earlier. The latter parameters are then used for probabilistic flood peak estimation via two alternative probability distribution functions, General Extreme Value (GEV) and the Log-Pearson III (LPIII), respectively.

As illustration of the performance of this component of the JPV approach, Figure 4.3 presents a comparison of the design flood peak estimates using the full (wide pooling) procedure outlined above with flood peak estimates via the Unitgraph method. These comparisons were performed for a representative data set of 75 catchments across all three Veld Zone pooling-groups.

In general, the wide-pooled GEV approach, combined with regression-based prediction of index flood values, performed with sound consistency relative to the single-site probabilistic estimates, as opposed to both the wide-pooled LPIII-based and the Unitgraph-based estimates, which were inconsistent and showed much greater variability than the GEV-based values.

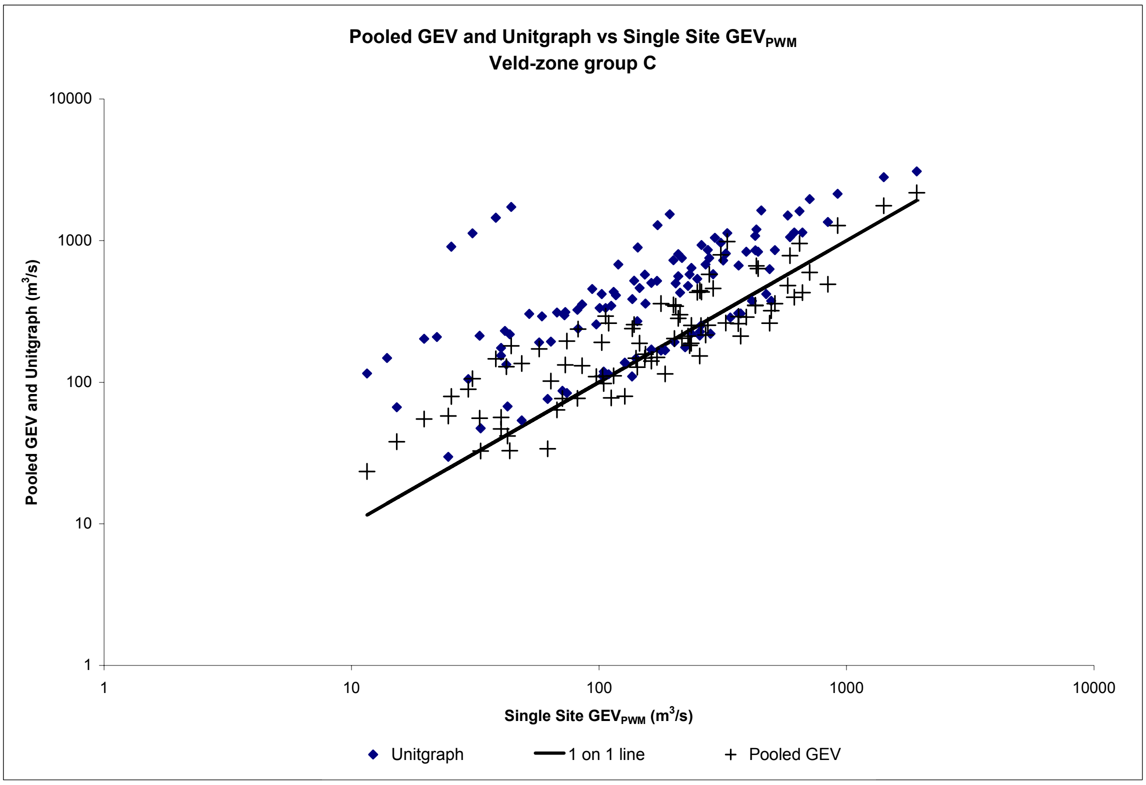

Figure 4.3 Comparison of design flood estimates using pooled GEV probability analysis for Veld-Zone Group C and Unitgraph-based estimates with single-site probabilistic estimates

5. RESEARCH RECOMMENDATIONS

5.1 DESIGN EXTREME RAINFALL

- The HRU PMP envelope curves need to be modernised with inclusion of longer and more current rainfall and meteorological records and a more extensive rainfall gauge network.
- Improved mapping of spatial rainfall during extreme storms based on point rainfall measurements.
- Further investigation into the applicability of the Extreme Point Rainfall curves for Small-Area storms needs to be undertaken.

5.2 EXTREME FLOODS

- An investigation should be undertaken to determine whether the pragmatic method used by the UK and Australia to determine design flood magnitudes between the maximum extrapolated RI value (e.g. 1:100 years) and an assigned RI value to the PMF (e.g. 1:10 000 years) can be applied to the South African situation.
- The extreme rainfall values used to calculate the PMF may need to be updated.
- An investigation should be undertaken into the use of a generally applicable probability distribution for South Africa, to promote consistency, as well as the influence of truncation in probabilistic flood peak estimations.
- The suitability of the Generalised Logistic distribution function for South African applications requires investigation.
- Inconsistencies between flood peaks measured at dam sites and flow gauging stations should be investigated.

- Research should be undertaken into the possible adjustment of K-regions for RMF calculation (Kovaćs, 1988 - Figure 7).

5.3 REGIONALISED POOLING AND REGRESSION-BASED FLOOD ESTIMATES

- Fresh perspectives are needed regarding appropriate regionalisation concepts and pooling methodologies for South African design flood practice.
- Further development of robust and physically-meaningful predictor equations for "index flood" characteristics is needed. This should include comparative uncertainty analyses.
- Further development is required of suitable approaches to standardisation of joint peak-volume data for the purposes of statistical analysis.
- Appropriate approaches to standardisation of observed flood hydrographs for the purposes of ungauged design site applications requires in-depth research.

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ACKNOWLEDGEMENTS

Ninham Shand would like to thank the Water Research Commission for funding this research as well as the DWAF Dam Safety Office and SANCOLD for their support. The members of the WRC Reference Group are as follows:

Mr A G Chemaly	Department of Water Affairs and Forestry
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Dr A Görgens	Ninham Shand, Project Leader
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Mr M Summerton:	Umgeni Water
Mr D van der Spuy:	Department of Water Affairs and Forestry

We are also deeply indebted to the following Ninham Shand researchers who made valuable contributions over the project's lifespan:

Johan Van Rensburg	Hydrologist
Nadia Nitsche	Civil Engineer
Mary Haw	Environmental Engineer
Hans Beuster	GIS Expert and Civil Engineer
Michele Warburton	Hydrologist
Tassie Clarke	GIS Scientist
Frank Denys	Civil Engineer

MODERNISED SOUTH AFRICAN DESIGN FLOOD PRACTICE IN THE CONTEXT OF DAM SAFETY

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LIST OF ABBREVIATIONS

AEP	Annual Exceedence Probability
AFC	Acceptable Design Flood
ANCOLD	Australian National Committee on Large Dams
ARC	Agricultural Research Council (South Africa)
ARR	Australian Rainfall and Runoff
CCWR	Computing Centre for Water Research (South Africa)
EAF	Elevation Adjustment Factor
DCF	Design Capacity Flood
DEFRA	Department of Environment, Food and Rural Affairs (United Kingdom)
DSO	Dam Safety Office
DWAF	Department of Water Affairs and Forestry (South Africa)
FEH	Flood Estimation Handbook (United Kingdom)
FEMA	Federal Emergency Management Agency (United States)
FSL	Full Supply Level
FSR	Flood Studies Report (United Kingdom)
GIS	Geographical Information Systems
GSAM	Generalised South East Asian Methodology
GSDM	Generalised Short Duration Methodology
HRU	Hydrological Research Unit (University of the Witwatersrand)
HMR	Hydrometeorological Report
ICOLD	International Committee on Large Dams
IDF	Incremental Design Flood
IFHC	Incremental Flood Hazard Category
JPV	Joint Peak-Volume
MAF	Moisture Adjustment Factor
MAP	Mean Annual Precipitation
MAR	Mean Annual Runoff
MR	Mean Runoff
MSL	Mean Sea Level
NEH	National Engineering Handbook (United States)
NRC	National Research Council (United Stated)
NRCS	Natural Resources Conservation Service (United States)
NWS	National Weather Service (United States)
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
POT	Peak-over-threshold
RI	Recurrence Interval
RMF	Regional Maximum Flood
SANCOLD	South African National Committee on Large Dams
SASRI	South African Sugarcane Research Institute
SAWS	South African Weather Service
SCS	Soil Conservation Service
SDF	Standard Design Flood
SED	Safety Evaluation Discharge
SEF	Safety Evaluation Flood
WMO	World Meteorological Organisation
WRC	Water Research Commission (South Africa)

PART 1

INTRODUCTION

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1. RESEARCH OBJECTIVES

More than 15 years have elapsed since the formulation of the *SANCOLD Guidelines on Safety in Relation to Floods* (SANCOLD, 1991), since which there have been a number of developments in the dam safety field both locally and internationally. Some of these developments include: increasing use of probabilistic flood determination techniques; increasing interest in risk-based dam safety assessments; and concerns raised about the applicability of certain standard historical design flood practices, such as the use of the Probable Maximum Flood (PMF) and the nominal annual exceedence probability value that could be attached to the so-called Regional Maximum Flood (RMF). These two extreme design flood concepts are imbedded in the *Guidelines*. Therefore, the need to take stock of the footprint achieved by the *Guidelines* and to update them in terms of international best practice has also increased in recent years.

Although complete design flood hydrographs play a fundamental role in dam spillway design and in dam safety assessments in relation to floods, design flood hydrograph generation has been a dormant area of research in South Africa for the past two decades. Apart from the South African customisation of the SCS technique for small catchments (Schmidt and Schulze,1987), no lasting new tools for the generation of complete design flood hydrographs for dam safety-related applications have been established for more than 30 years, since the publication of the HRU 1/72 Report (HRU, 1972)[1]. Therefore, a distinct need has existed for some time for research to bring the flood hydrograph-related information contained in the streamflow records of the last three decades into South African design flood practice.

In recognition of these needs, the Water Research Commission of South Africa (WRC), with the support of the Dam Safety Office (DSO) of the Department of Water Affairs and Forestry (DWAF) and SANCOLD, commissioned Ninham Shand in 2003 to conduct a study to undertake flood-related research in the context of dam safety. This document describes this research (Project Number: K5/1420), which has been conducted under the following original objectives:

i) *To establish updated Guidelines for the safety evaluation of dams in relation to floods*

ii) *To derive a methodology for design flood hydrograph estimation based on joint occurrence of flood peaks and flood volumes, through analysis of historically measured flood hydrographs in all regions of South Africa*

iii) *To develop a modernised set of design tools for the generation of complete flood hydrographs for dam safety evaluation or spillway design.*

To shape the research required, in March 2004 Ninham Shand conducted a systematic survey of dam safety professionals in order to develop an understanding of the perceived shortfalls of the *SANCOLD Guidelines*. The survey responses were then debated and confirmed in a Workshop held in October 2004, which was attended by over 30 dam safety practitioners. The majority of the practitioners involved in these activities felt that the *SANCOLD Guidelines* needed to be updated, but that they only required relatively minor revisions. Based on this outcome, the project's WRC Reference Group recommended that the scope of the first objective be changed from an "updating" of the *Guidelines* to a "review" of some of the key concerns regarding the *SANCOLD Guidelines*.

In response to the above objectives, as modified, this Study was conducted in the following four phases:

[1] The innovative and important Runhydrograph design flood methodology by Hiemstra and Francis (1979) has unfortunately not become established in South African practice.

- Phase 1: Assessment of Local and International Practices Regarding Dam Safety in Relation to Floods
- Phase 2: Improvement of Flood Hydrograph Generation Techniques for South Africa for Dam Safety Purposes
- Phase 3: Development of a "Design Flood Hydrograph Toolbox", the purpose of which is to support the various components of dam safety evaluation in relation to floods.
- Phase 4: Review of the *SANCOLD Guidelines on Dam Safety in Relation to Floods*.

This Report is essentially a culmination of the Interim Research Reports written during Phase 1 and Phase 2 of this project, which comprised the following tasks:

i) *International and local review of design rainfalls and storm loss algorithms used with both the PMF and Recurrence Interval (RI) flood concepts.* This research also included the "testing" of the most common South African approaches with regards to these concepts through the employment of modernised data and concepts.

ii) *Examination of the regional correspondence of estimates of RI flood peaks by the HRU (1972) Unitgraph-based approach with single-site probabilistic-based approaches.*

iii) *Investigation of the link between RI, or annual exceedence probability (AEP), and the most commonly used extreme flood concepts in South Africa, i.e. the Regional Maximum Flood (RMF) and PMF.* This information would aid in the future application of the SANCOLD *Guidelines*, which incorporates the use of these two extreme design flood concepts.

iv) *Examination of the spatial consistency in the relationship of the RMF to the PMF.*

v) *Development of customised software (EX-HYD) to extract complete flood hydrographs (peak, volume and shape) from selected flow records, as well as the analysis of hydrographs in terms of same-event flood peaks, flood volumes and hydrograph shapes.* This entailed the collection of breakpoint data (time versus stage) for selected flow gauging stations across South Africa from DWAF; the use of rating relationships to transform stage to flow rates; and the screening of flow records for errors and consistency using visual and statistical techniques. This process resulted in much larger databases than those by the HRU (1972) and Hiemstra and Francis (1979).

vi) *Derivation of a methodology for design flood hydrograph estimation based on the joint occurrence of flood peaks and flood volumes utilising the sample of extracted complete hydrographs from (v) above.* This methodology took appropriate account of some of the weaknesses and strengths of the unit hydrograph (HRU, 1972) and Runhydrograph (Hiemstra and Francis, 1979) methods.

vii) *Development of regionally pooled RI flood peak estimation tools based on catchment descriptors and statistical flood parameters.*

This Report also serves as a supporting document for the products developed in Phases 3 and 4 of this Study. From Phase 3, a web-based "*Dam Safety Hydrology Toolbox"* was developed in the form of interlinking HTML pages. It comprises a suite of reference documents and software applications to support the various components of dam safety evaluation in relation to floods. From Phase 4, a document entitled "*Review of the Selection of Acceptable Flood Capacity for Dams in South Africa in* *the Context of Dam Safety"* (Cullis *et al*, 2006) was produced. The forementioned document reviews the *Guidelines* critically and is intended to be used as a tool to assist dam safety practitioner in performing their duty to society by selecting the acceptable flood capacity (AFC) for a dam, either with regards to the spillway design for a new dam, or for the safety evaluation of an existing dam in relation to floods.

2. THIS RESEARCH IN CONTEXT

2.1 BACKGROUND

Given the unfavourable spatial and temporal distributions of rainfall over large regions of South Africa, the water supplies needed for the economic development of the country has had to be assured by storage in large numbers of dams. Currently, South Africa has over 3700 dams that are listed in the "Register of Dams", maintained by the Dam Safety Office of the Department of Water Affairs and Forestry (DWAF). Of these dams, more than half are classified as "Small Dams" (maximum wall height of between 5m and 12m) with low hazard rating in terms of potential loss of life or damage to property. At the other end of the scale there are nearly 150 "Large Dams" (maximum wall height of more than 30m) with a high hazard rating. South Africa has the highest number of large dams of all countries in Africa and is registered by ICOLD as one of the major dam-building nations of the world.

All dams large enough to warrant listing on the Register of Dams potentially pose a public safety hazard. The structural failure of any of these dams, especially the "Large Dams", would pose a significant threat not only to people and property downstream of the dam, but also to the communities and industries that depend on them for a reliable source of water, as well as the ecosystems of the rivers, wetlands and estuaries below these dams. For this reason, dam safety legislation was promulgated in 1986, which prescribed the safety evaluation of all registered dams on a five-year cycle by Approved Professional Persons specifically registered for that purpose by the Dam Safety Office. The new National Water Act of 1998 incorporated the original dam safety legislation.

Engineering design standards for dams were however not prescribed in the legislation. Aware of the necessity to provide guidance to those charged with evaluating the safety of existing dams, as well as the designers of new dams, SANCOLD issued a set of *Guidelines on Safety in Relation to Floods* (SANCOLD, 1991), as well as a compendium of South African Design Flood determination techniques, authored by Alexander (1990). These two documents and their feeder sources have been the mainstay of design flood analysis related to dam safety evaluation and dam spillway design during the past 15 years. The *Guidelines* create three categories of dams on the grounds of a combination of dam wall/embankment height and downstream hazard. The compendium by Alexander (1990) brought together deterministic design flood hydrograph estimation techniques, established in the early 1970s by the Hydrological Research Unit (HRU) of the University of the Witwatersrand under Prof Des Midgley (HRU, 1972), with empirical flood peak estimation techniques developed in DWAF during the 1980s (Kovaćs, 1988). The compendium also provided software that facilitated the probabilistic analysis of annual flood peaks extracted from streamflow records.

Thus far, South Africa has had a relatively good record regarding dam failures. Since 1987, 164 cases of failure or severe damage have been recorded. All but two of these failures were for "Small, Low Hazard" dams. In the few cases where fatalities have occurred, they have been only indirectly attributed to the failure of the dam. Of these incidents of failure, 39% have been attributed to inadequate spillway capacity. In addition 36% of the 887 dams evaluated between 1987 and 2002 were recorded as having spillway capacity less than required (Cullis *et al*, 2006).

Despite this sound track record, concern has been expressed by dam safety practitioners in relation to certain aspects of the contents of the *SANCOLD Guidelines*, especially in terms of the recommended flood hydrograph estimation methodology. It was recognised in the Preface to the *SANCOLD Guidelines* that, as more information and knowledge were assembled, they might need to be revised.

2.2 STATUS OF EXISTING DESIGN FLOOD HYDROGRAPH ESTIMATION TECHNIQUES IN RELATION TO DAM SAFETY IN SOUTH AFRICA

The design flood characteristics that are primary in dam design are the combination of flood peak, flood volume and hydrograph shape of the incoming flood; the reason being that the storage characteristics of the dam basin come into play in determining the actual outflowing flood hydrograph. The implication is that design flood estimation techniques that produce only an incoming flood peak are not adequate for use in the dam design environment, because the translation of the flood hydrograph through the dam basin cannot be assessed. The only technique in Alexander's 1990 compendium that could generate a combined flood peak, volume and hydrograph shape, was the Unitgraph-based approach of HRU (1972).

The HRU techniques were based on regionalised synthetic "unit hydrographs", derived from streamflow records measured at 96 flow gauging stations and standardised for a number of “homogeneous” hydrological response regions in South Africa. Although the flood records used to derive the HRU's techniques pre-dated 1969, Alexander’s (1990) brief from SANCOLD did not include updates or significant improvements of the HRU (1972) techniques. For more than a decade a number of concerns have been expressed about apparent shortcomings of the HRU unit hydrograph techniques (Görgens and McGill, 1990; Kleynhans, 1995; Görgens, 2001; Görgens, 2002), which could partially be attributed to the relatively limited database available at the time. Of particular concern have been the marked discrepancies country-wide between the HRU's so-called "Probable Maximum Flood" (PMF), which played a key role in both the *SANCOLD Guidelines* (1991) and Alexander's (1990) compendium, when compared with Kovacs's (1988) "Regional Maximum Flood" (RMF). The latter is a catchment area-based empirical method based on the largest observed flood peaks at observation sites in each of 9 different "homogeneous" extreme flood regions in South Africa. It should be noted that, internationally, the use of the PMF has been under critical review during the past 15 years, because of its statistical nebulousness (Graham, 2000).

An innovative South African approach to design flood determination, of importance to dam safety evaluation and dam spillway design, was developed in a WRC project by Hiemstra and Francis (1979), known as the "Runhydrograph" method. They based their technique on joint probability analyses of same-event pairs of flood peaks and flood volumes for 43 flow gauging station records across South Africa. To obtain a joint flood peak, volume and hydrograph shape at any site in South Africa, they proposed use of a standardised bi-variate Log-Normal probability distribution, combined with a particular standardised hydrograph shape. Unfortunately, practising engineering hydrologists raised certain technical and operational concerns about the Runhydrograph method, which have not been resolved, and, despite its obvious potential, it has not achieved wide-spread acceptance in practice.

The well-known SCS flood hydrograph generation technique of the US Soil Conservation Service was modified for South African conditions, after considerable local research, by Schmidt and Schulze (1987) in a WRC project. However, as its origin suggests, it is only suitable for application to relatively small catchments (8 km^2). This makes its use impractical for most dams where safety in relation to floods is a concern, as such dams by definition are relatively large and are fed by large catchments.

Alexander (2002) proposed a new regional flood peak estimation technique, which he called the "standardised design flood" (SDF). This empirical method is heavily codified and is specifically intended for use in designing road river-crossings. As such, the method was purposefully calibrated to generate, on average, flood peak estimates on the high side. The rationale here was that road designers needed a convenient, repeatable and "safe" method, as the additional costs related to over-designing some of the culverts and bridge-openings in a road system could be said to be marginal compared with the total cost of such a road system. Comparisons with standard annual exceedence probability-based estimates of the 1:50 year flood peaks across all test sites in South Africa indicate an average over-estimate of about 200%. This approach is not favoured for dam safety-related applications, for two primary reasons. Firstly, it produces only a flood peak, and no volume or hydrograph. Secondly, the sizing of the dam spillway is very sensitive to the magnitude of the design flood. As spillway costs make up a major component of total dam costs in medium to large dams, an average over-estimate of 200% would significantly undermine the economics of many dams. It should be noted that SANCOLD has not embraced the SDF for South African dam design practice.

In a promising recent development the WRC has been funding a pilot study into the generation of credible design flood hydrographs for certain KwaZulu-Natal catchments by means of continuous rainfall-runoff modelling, using the ACRU agro-hydrological catchment model (Smithers, personal communication, 2006).

3. REPORT STRUCTURE

This report has been divided into three separate Parts, following this **Part 1**: "*Introduction*". **Part 2**: "*Establishment of Flood Database and Flood Hydrograph Extraction Software*", provides information on the development of the flood database, by considering the flood data sources and the screening of the primary data. The development and use of the flood hydrograph extraction software, "*EX-HYD*", is also described in detail.

Part 3: *"Modernised Perspectives on Design Flood Methodologies"* consists of a review of international and local design extreme rainfall approaches; a review of local and international approaches to design storm losses; a comparison of HRU Unitgraph-based design flood estimates with probabilistic-based estimates for gauged catchments; and finally, a modernised approach to extreme design flood concepts in South Africa in the context of dam safety, which includes an extensive local and international literature review.

A further component of the research conducted under this Study dealing with the generation of design flood hydrographs is presented in a stand-alone Report entitled, "*Joint Peak-Volume Design Flood Hydrographs for South Africa".* That Report details the derivation of a methodology for design flood hydrograph estimation based on the joint occurrence of flood peaks and flood volumes. It also includes a design flood peak/flood volume estimation methodology for ungauged sites based on catchment descriptors and regional/pooled flood probability analysis.

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PART 2

ESTABLISHMENT OF FLOOD DATABASE AND FLOOD HYDROGRAPH EXTRACTION SOFTWARE (*EX-HYD*)

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1. INTRODUCTION

A primary consideration in dam design is that not only is the peak of the incoming flood hydrograph of importance, but equally important is the flood volume and hydrograph shape; the reason being that the storage characteristics of the dam play an important part in the determination of the outgoing hydrograph. With the publication of the SANCOLD *Guidelines on Safety in Relation to Floods* (SANCOLD, 1991), a compendium of South African Design Flood determination techniques, authored by Alexander (1990), was also released with the intention that this document be used for the estimation of the design flood hydrograph for the safety evaluation or sizing of a dam. In Alexander's compendium, the only recommended technique for the generation of a combined flood peak, volume and hydrograph shape was that developed by the Hydrological Research Unit (HRU) of the University of the Witwatersrand. This methodology was published by the HRU in 1972 (HRU Report No 1/72) and is based on streamflow records from about 1930 to 1960, which were used to derive regionalised "unit hydrographs".

Since the publication of the HRU 1/72 report, no updates or improvements have been made to this methodology and it is still commonly used today for the design and safety evaluation of dams. Studies have, however, been undertaken since 1972, which have involved the generation of combined flood peak, volume and hydrograph shape, but these are not as commonly used as the HRU methodology. For example, Hiemstra and Francis (1979) developed the "Runhydrograph method", which combined flood peaks and volumes extracted from flow records for 43 South African catchments in an innovative way in log-space. However, the Runhydrograph approach has not become established in South African practice.

In light of the above, the Water Research Commission of South Africa (WRC) recognised a need to develop a comprehensive and up-to-date flood database for South Africa, which could be used for the extraction of complete flood hydrographs for dam safety evaluation and design applications. Research was therefore undertaken as part of this Project (WRC K5/1420), which specifically addresses the following tasks stipulated under Phase 2 ("Improvement of Flood Hydrograph Generation Techniques for South Africa for Dam Safety Purposes") of the Project Proposal:

- Collect breakpoint data (time versus stage) for selected flow gauging stations across South Africa from DWAF, and use rating relationships to transform stage to flow rates. This process should conceivably result in a database of flow records with record period lengths far exceeding those used by the HRU (1972) and Hiemstra and Francis (1979).
- Screen flow records for errors and consistency, using visual and statistical techniques.
- Extract complete flood hydrographs (peak, volume and shape) from selected flow records and analyse hydrographs in terms of same-event flood peaks and flood volumes.

This Research addresses the second and third objectives of this Project, as follows:

Objective 2: To derive a methodology for design flood hydrograph estimation based on joint occurrence of flood peaks and flood volumes, through analysis of historically measured flood hydrographs in all regions of South Africa

Objective 3: To develop a modernised set of design tools for the generation of complete flood hydrographs for dam safety evaluation or spillway design.

2. FLOOD DATA SOURCES AND SCREENING OF DATA

2.1 FLOOD DATA AVAILABILITY

The Department of Water Affairs and Forestry (DWAF) executes its responsibility for streamflow monitoring in most South African river systems by building, instrumenting and maintaining streamflow gauges and stage recorders on these rivers. Due to the large number of potential gauges that needed to be examined as part of this study, it was decided, in order to economise on time and resources, to focus on an aggregate sample of flow gauges used in previous studies of design flood estimation. The main studies considered were:

- *The Runhydrograph – Theory and Application for Flood Predictions* (Hiemstra and Francis, 1979)
- *Design Flood Determination in South Africa* (HRU, 1972)
- *The Standard Design Flood – Theory and Practice* (Alexander, 2002).

Hiemstra and Francis (1979) used flow records from 43 out of a possible 123 stations considered initially for his analysis. Additional to some of the initial gauges failing the statistical tests for inclusion in the subsequent analysis, the continuous flow records at a number of these gauges were too short (less than 10-15 years) for inclusion in the analysis.

In HRU (1972) some 600 flood events from 96 gauging stations throughout South Africa were used. The main aim of the HRU was to produce unit hydrographs for 9 generalised Veld-Zone regions used to describe approximately homogeneous flood response zones across South Africa.

It was initially anticipated that the flow records of at least the 110 stations used by Alexander (2002) would be available for use in this study. Alexander (personal communication, February 2004), however, indicated that, since the SDF Method did not consider the flood hydrograph or flood volume, the database produced during development of the SDF only contained flood peaks. Extension of rating tables for those flow gauges for which the maximum flood levels exceeded the rating table limit, were not official extensions of the DWAF rating tables and were therefore not included in the DWAF hydrological database. It follows from the above that the database of flow gauges used in the SDF Analysis did not provide the specific information considered for use in this study.

A comparison of the flow gauges used by the above three studies, however, showed that a large overlap existed.

2.2 CRITERIA USED FOR SCREENING OF DATA

In order to select suitable flow gauging stations for inclusion in the flood event analysis of this study, the following first screening criteria were applied:

- **type of gauge** : stations measuring river flow or inflow to reservoirs only
- **length of record** : only streamflow records of longer than 15 years were considered
- **size of catchment areas** : gauges with catchment areas of less than 10km^2 were discarded
- **ability to record high flows** : since the focus of the study was on extreme events, it was important to obtain flow records from gauges which could record/register all or most of the high flow or flood events at the gauging site, as well as for the rating table to cover the full range of recorded flood levels

- **reliable, complete records** : streamflow records with obvious errors in the data or known problems at the gauge were discarded, inclusive of records with long periods of missing values during the wet periods
- **upstream impact on recorded high flows** : streamflow gauges which had large in-stream reservoirs upstream of the gauge were discarded due to the possible attenuation of floods through reservoirs.

The above criteria were initially applied to the flow gauges registered as part of the DWAF hydrological network. Application of only the first two evaluation criteria listed above resulted in a list of 781 gauges that could be considered for evaluation. Since the evaluation of almost 800 flow gauges and records would require considerable time and resources, it was decided to focus initially only on the flow gauges common to HRU (1972), Hiemstra and Francis (1979) and Alexander (2002) databases.

2.3 FIRST SCREENING OF GAUGES COMMON TO HRU, HIEMSTRA AND ALEXANDER

Appendix A summarises the list of gauges considered and/or used by HRU (1972), Hiemstra (1979) and Alexander (2002). The combined list of flow gauges resulted in 261 gauges that could be considered for use in this study. Selecting (screening) only gauges with rating tables, gauges with catchment areas in excess of $10km^2$ and flow records of 15 years or longer resulted in 188 flow gauges that could potentially be considered for use in this study. The fourth criterion used to evaluate the gauges from the HRU, Hiemstra and Alexander databases was the ability of the flow gauge to record high flows, considering the extreme event focus of this study. Records of annual flood peaks obtained from DWAF were used to identify gauges for which the rating tables had to be extended.

2.4 SECOND SCREENING OF FLOW GAUGE DATA

Following the initial screenings of the flow gauges described in Section 2.3 above, the second screening of the streamflow records aimed at identifying data with obvious errors or known problems. This screening included identifying records with long periods of missing values during the wet periods as well as records with upstream development that could potentially impact on recorded high flows.

Although used at a small number of stations before the 1960s, automatic recording of water levels at streamflow gauging stations generally started in the 1960s. For flow gauging stations not equipped with automatic water level recorders, the water levels had to be manually recorded. Since these manual recordings usually comprised one daily reading only, insufficient information would therefore be available to describe a flood event. The flows in most South African rivers fluctuate to such an extent that daily flows can therefore not be used to adequately describe flood events, more so for catchments for which the flood hydrograph durations are less than 24 hours. Flow gauging stations with only daily observations were therefore discarded. However, flow gauges for which automatic recorders were installed some time after the station was commissioned, were considered for further use in this study, but then only using the continuously recorded portions of the flow records.

Applying the above screening criteria resulted in an additional 79 gauges to be discarded.

Appendix A lists the gauges used or considered by the HRU, Hiemstra and Alexander, as well as the 109 gauges eventually considered for use in this study.

2.5 THIRD SCREENING OF FLOW GAUGE DATA

The first and second screenings were performed to remove the records with obvious errors and problems from the analysis. The next step involved identifying unreliable records during the process of extracting flood hydrographs via the EX-HYD flood hydrograph extraction software from the primary flow data obtained from DWAF. In the process of downloading primary data from DWAF, gauges were discarded for the following reasons: primary data, rating tables or annual peak information was not available; too many exceedences of the rating table beyond the acceptable criteria; large data gaps resulting in short usable periods; data errors in running the program; or non-stationarity of mean peaks. Appendix B shows the list of the 65 stations whose records were used as input into EX-HYD.

2.6 DWAF DAM SITE FLOOD DATA

In addition to the 109 gauges considered for this study, an additional set of flood related information was obtained from the DWAF Flood Studies Section (Van der Spuy, pers comm., 2004). Using the flood hydrograph information recorded at the flow gauging station immediately downstream of a reservoir, a "back-routing" approach was followed in order to determine the inflow hydrograph to the reservoir. From these inflow hydrographs, DWAF determined the hydrograph peak and volume. Unfortunately, DWAF registered only the annual maximum flood peak and associated volumes, i.e. the actual hydrographs were not retained.

DWAF conducted the above exercise for 83 reservoirs across South Africa.

Figure 2.1 shows the location of both the flow gauges that survived the above mentioned screening, as well as the reservoirs included in this study.

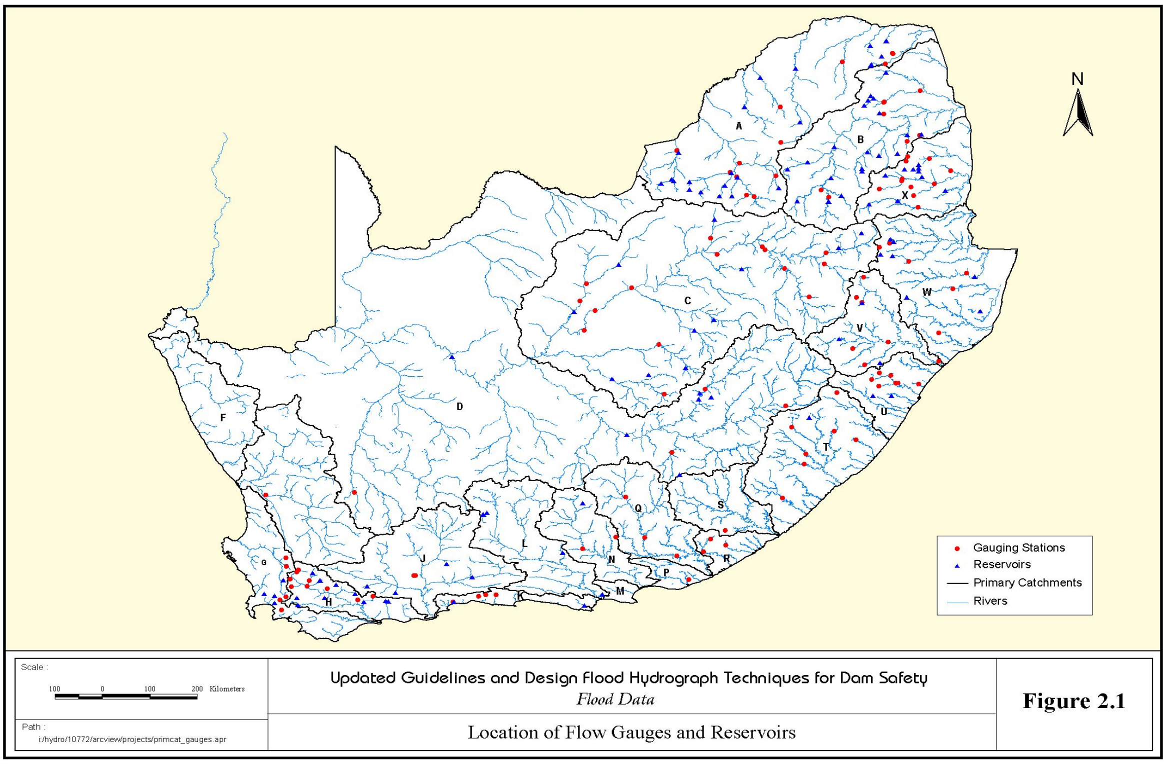

Figure 2.1 Location of Flow Gauges and Reservoirs

3. FLOOD HYDROGRAPH EXTRACTION SOFTWARE: *EX-HYD*

3.1 INTRODUCTION

Due to the large number of events to be extracted from the primary data set, a computer program, "*EX-HYD*", was developed to assist in identifying and extracting complete flood hydrographs from continuous recorded flow data. Use of this software not only speeds up the laborious task of identifying flood events, but it also ensures an objective and consistent approach in identifying events for future analysis. It should however be noted that the rules employed in the software for event identification cannot cater for all different variations in flood hydrographs, hence a measure of user intervention is required with the selection of events from some gauges. Input to the extraction program is primary records obtained from DWAF (for the selected flow gauges), extracting the following flood characteristics for each event:

- Hydrograph peak and associated volume and duration
- Plot/graphical representation of hydrograph
- Hydrograph in text format for potential use in follow-up studies.

Of importance in selecting the flood events was to ensure that only events significant enough to be classified as "flood events", should be selected.

The approach followed during the abstraction process was first to identify periods in the primary record that included significant flood events. From these record periods, referred to as "Pegram-events", individual and independent flood events were identified and extracted. The final text-based database of independent flood events contains peak, volume and duration information for each individual event.

3.2 RULES FOR FLOOD HYDROGRAPH IDENTIFICATION AND SELECTION

The three main hydrograph selection criteria are:

- the identification of significant flood events, assisted by setting "truncation levels"
- start/end time of flood hydrographs
- extrapolation of rising and recession limbs to zero flow line

3.2.1 The identification of significant flood events, by setting "truncation levels"

The first step in identifying individual flood events is to identify record periods containing extreme nature, i.e. large flood flow events only, with the basic rule that only flood events larger than the smallest annual maximum flood event on record could be selected. In order to ensure that that selected flood events are statistically independent, a threshold or "truncation level" is set to screen out minor events.

Selection of all flood events higher than the truncation level implies that more than one event can be selected for a wet year, while it is possible that no events are selected from a "dry" year. This approach results in a partial duration series of independent flood peaks above a certain level. The advantage of this series over the annual maximum series is that it potentially produces three to five times more events (Flood Estimation Handbook, FEH, 1999). Once these events have been selected, the FEH method is used to verify that events are independent.

It should further be noted that although an automated approach would best suit the interests of this study from a consistency and objectivity point of view, the difficulty was finding an approach that could cope with all the variations in hydrograph shape. Since the focus of the study is to ultimately identify flood peaks and associated volumes of flood events and not to produce unit hydrographs at each gauging site, flood hydrographs with multiple peaks could be selected, given the later peaks were not independent of the earlier ones in that specific event.

Since the aim of the flood analysis was to derive a methodology for design flood hydrograph estimation based on the joint occurrence of flood peaks and flood volumes, the complete flood hydrograph (peak, volume and shape) is required for analysis. The base flow therefore forms part of the flood hydrograph and no base flow separation is therefore conducted after selection of the flood events.

It should be noted that the database of flood events selected for this study would therefore represent the entire flood hydrograph, inclusive of base flow. Potential future studies would therefore be able to use the flood events from this study, and after applying base flow separation techniques be able to produce a database of say unit hydrographs, if so required.

3.2.2 Start/end time of flood hydrographs

In order to conduct the proposed flood analysis, same-event flood peaks and flood volumes are required. The flood peak is easily identified, while calculation of the flood volume depends on the identification of the start and end time of the specific flood event.

It is common knowledge that the duration, shape and peak of the rising limb of a hydrograph are dependent on both storm and catchment characteristics. The recession limb is however independent of storm characteristics and is controlled by the hydraulic and storage characteristics of the catchment. The start of a flood event is generally easily identified by physical inspection as the point where the hydrograph changes from near constant or declining values to rapidly increasing values. Identification of the end of the flood event, which is when the flood flow has subsided and only base flow, which is not directly related to the causative rainfall for that event, remains in the river, is however not as easily defined.

A number of approaches to determine the end the event were considered, however the eventual approach used in this study was ultimately based on the principles of the semi-log approach. The approach used in this study considered the difference in the change in angle of the recession line between two successive flow points used as the indicator. A change in angle of 25 degrees was accepted as a sound "indicator" after evaluating a range of flood events from four typical flow gauges.

The procedure for identifying the end of the flood event is:

- Identify the start of the event, as the point where the hydrograph changes from near constant or declining to showing a rapid increase
- "Draw" a horizontal line across the time-flow graph until the recession limb of the hydrograph is intersected
- Use the point where the horizontal line crosses the recession limb as the "start" point for identifying the true end point of the flood event
- From this "start" point, move backwards, i.e. "up" the recession curve, until the required change in angle is identified.

The point selected using the above approach was then assumed to be the end of the flood event. It should be noted that the base flow contribution to the total volume of the flood event is generally less than 5%, hence the error made by selecting an inaccurate end-of-flood point will have little impact on the sample statistics of the total flood volume, as long as a sample of reasonable size is used.

Although the approach mentioned above is successful in flood hydrographs with smooth recession curves, many flood hydrographs have irregular recession curves, with multiple events occurring close together. The software has additional criteria to ensure suitable end-of-flood-event values are selected and independence between events is maintained. Nevertheless, manual checks using visualisation of a number of flood events is necessary in order to verify the results of the automated process.

3.2.3 Extrapolation of rising and recession limbs to zero flow line

The preceding two steps described the approach to firstly identify significant flood events, and secondly to determine the start and end times of the flood event. The start and end points of the flood hydrograph is however not at zero flow. The rising and recession limbs therefore have to be extrapolated to the zero line in order for the total volume to be calculated.

Hiemstra and Francis (1979) used weighted average slopes to extrapolate the rising and recession limbs from the truncation level to zero flow level. As the base flow is generally low compared to the peak of the event, it was argued that this arbitrary extrapolation of the rising and recession limbs would not have a significant effect on the total flood volume.

Following on the approach by Hiemstra and Francis, it was decided to use a straight vertical line extrapolation from the start and end points of the event to the zero line, as the additional volume added to the flood hydrograph when using the weighted average slopes would be negligible.

3.3 USER GUIDE: *EX-HYD* SOFTWARE

The *EX-HYD* software can be installed out of the *Dam Safety Hydrology Toolbox*[1] that was developed by Ninham Shand as one of the final deliverables of this Study. The following sections provide a brief "User Guide" of the flood event extraction software. A more comprehensive User Guide is provided as a utility in the aforementioned Toolbox.

3.3.1 User-interface

On opening the program, a ‘splash’ screen appears (white background with the name and version of the software) after the main window opens, as shown in Figure 3.1. The labels in Figure 3.1 below indicate the various interface functions.

[1] The web-based Toolbox is available on the Ninham Shand web-site, or can be obtained on CD from Ninham Shand, PO Box 1347, Cape Town, 8000 (Tel. No. 021-4812400).

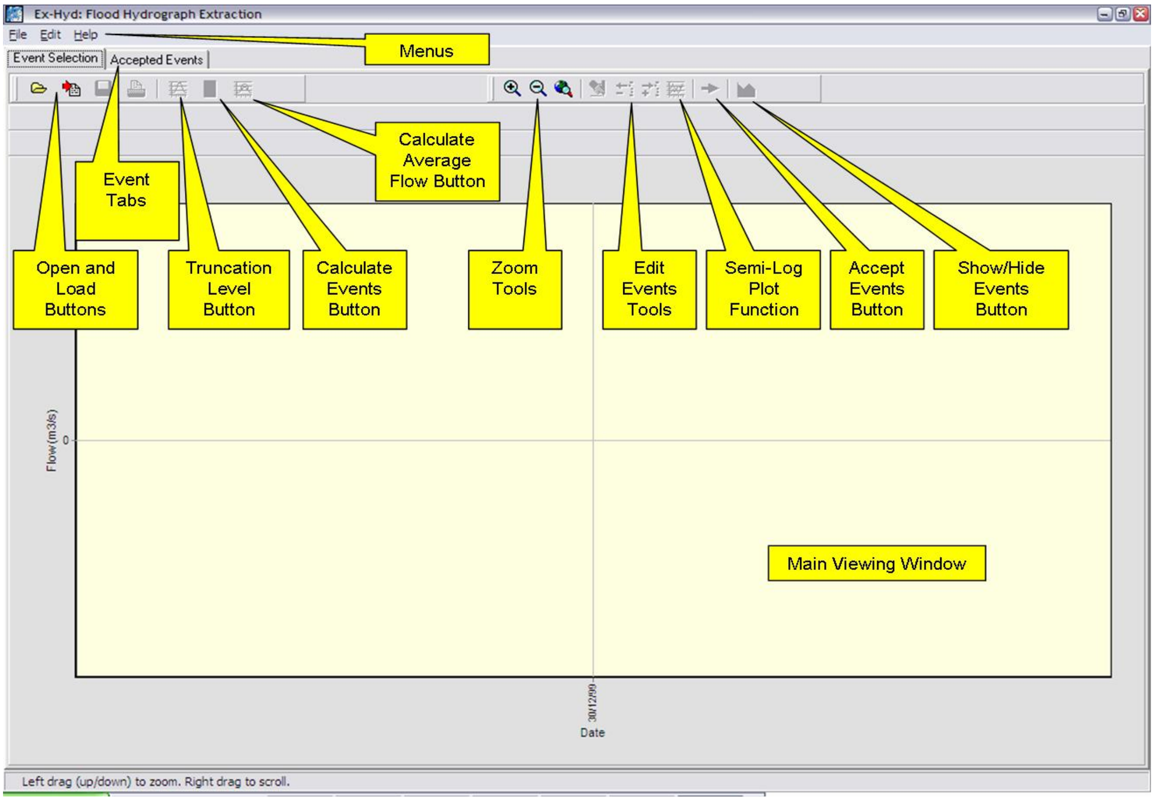

Figure 3.1 Software User Interface - Empty Window

When in use, the uploaded data is plotted in the hydrograph window (Figure 3.2) and the buttons become activated. Although most of the record appears in blue, a colour code has been applied to the data representing the five most common data quality codes defined in the dataset. These quality codes can be viewed in a legend to the right of the time series plot.

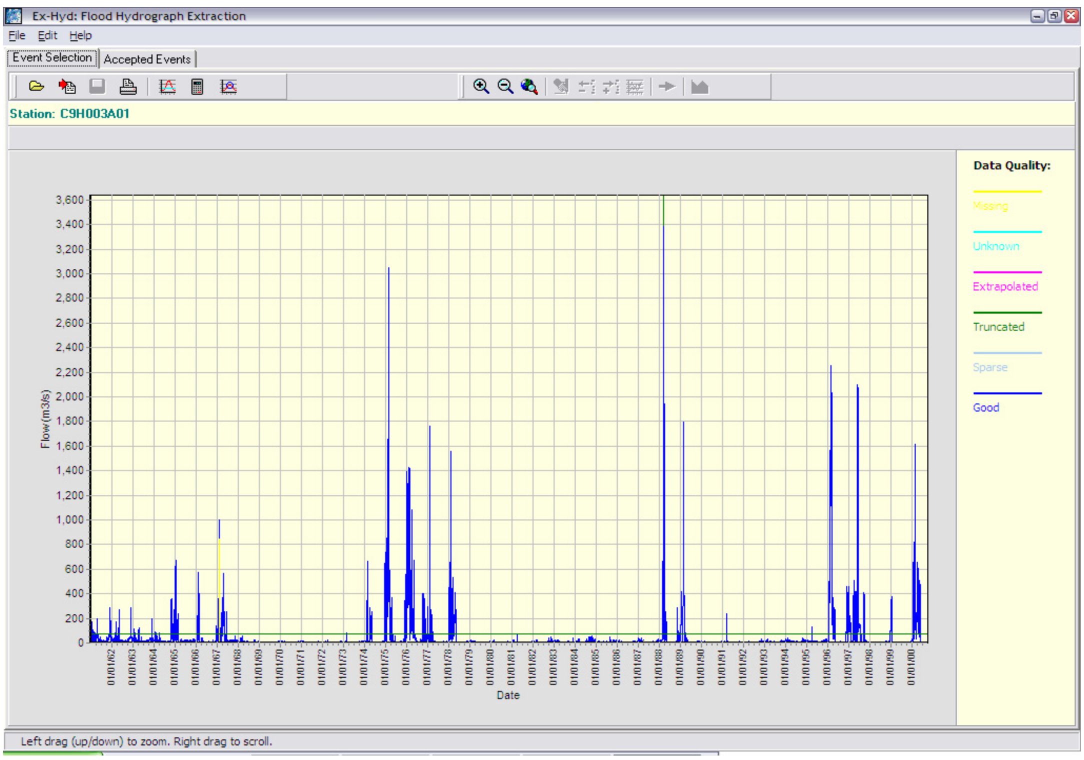

Figure 3.2 Hydrograph Window in Use

The drop down menus across the top of the window apply functions that can also be accessed through the following buttons on the screen (from the left):

- **Load Primary Data**: loads text files of primary data in DWAF (internet) format
- **Load Saved Events**: uploads events previously saved by the program. Files are text files in a format specific to this software.
- **Save events**: saves defined events. Files are saved in a text format specifically designed for use in this software.
- **Print Graph**: prints two versions of the plots on the Hydrographs sheet (one portrait orientation and one landscape orientation)
- **Set Truncation level**: sets the truncation level, which is either user defined or the default level. The default truncation level is set such that approximately five events per year exceed the level. The truncation level in drawn as a green line.
- **Calculate events**: the program calculates and defines events, plotting them in red.
- **Calculate Average Flow**: calculates the average flow rate between two user defined dates.
- **Accept Events**: accepts the events in the hydrograph as accurately defined and allows the user to view the individual event characteristics.

The buttons specific to Hydrographs are as follows:

- **Zoom in**: zooms in closer. This can also be done by holding down the right mouse button and creating a box around the portion of the hydrograph to be expanded.
- **Zoom out**: zooms out.
- **Zoom all**: zooms out to display the full record.
- **Modify event start and end date**: allows the start and end date of an event to be defined by the user.
- **Delete event**: deletes the selected event.
- **Add event**: adds event to user-specified start and end dates.
- **Plot semi-log**: plots semi-log of active event in a window just below the hydrograph window.
- **Accept Event and Post to the Database**: when the record has been checked for accurate start and end dates and the user is satisfied with the results, the data can be posted to the database. There, each event can be viewed and its individual characteristics scrutinised.
- **Show/Hide Events**: if the user would like to inspect the data quality, the red events (events that peak above the truncation level) can be hidden by clicking on this button. If this button is selected, the editing buttons are disabled.

Note: To scroll up, down or side ways within the hydrograph window, hold the right mouse button and move the mouse in the desired direction.

3.3.2 Loading data files

Check peak data and rating table information for each flow gauge to determine whether a rating table extension is necessary.

Loading primary data files that do not require rating table extensions

- Click on the 'Load Primary Data' button.
- A dialog opens which allows the user to select the correct primary data.
- Click the 'Open' button to browse and choose appropriate primary data text file.
- Ensure that the 'Extrapolate rating table' option is unselected.
- Click the 'Ok' button to load data.

Loading primary data files that require rating table extensions

For records in which the peak observed levels exceed the rating tables, an extension of the rating table is necessary to get an accurate flow reading. The WRC Flood Hydrograph Extraction software uses an extension created by the user following a log-log regression analysis and predicts flow values along the extension, given inputs regarding the rating tables and extension.

- Click the 'Load Primary Data' button.
- Select appropriate primary data file.
- Click the 'Extrapolate rating table' check box (Figure 3.3)
- The 'Open' window enlarges to the right to allow for rating table inputs.
- Select the 'plus' (insert) button to add a rating table extension
- For each rating table, enter the start and end date applicable to that particular rating table.
- Enter the Height and Flow for the start of the extension (i.e. the rating table limit) in the 'Rating' cells and the end of the extension (i.e. the maximum observed level and the user-created extension for flow) in the 'Observed' cells.
- When all information is entered, click the 'Ok' button to load data.

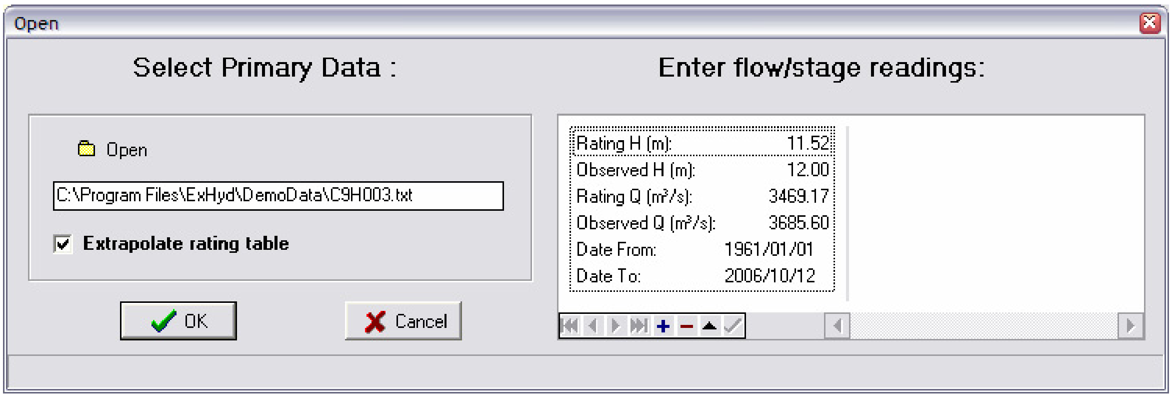

Figure 3.3 Loading data that require rating table extension

3.3.3 Defining Events

Once the primary data is loaded into the program and plotted in the hydrograph window, the program can be used to determine a truncation level and calculate flood events. The user can then view different parts of the hydrograph and alter any events that are not accurate:

- To add a truncation level to the hydrographs in the current window, click on the '**Set Truncation Level**' button. The user can either specify a truncation level or select the default level, which defines on average 5 events per year. The truncation level is plotted as a green line.
- After this the actual events can be calculated by clicking on the '**Calculate Events**' button. This process may take a little time since the software must read all the primary data and perform some fairly complex calculations to determine the start and end date of the events. Events are plotted in red over the colour coded line of the primary record. The start of an event is indicated with an asterisk.
- Due the non-uniform nature of primary data, the events defined by the software are not always correct. As indicated previously, some user intervention may be required as no two gauges or flood events are the same, which means that the general rules for choosing events do not apply accurately in all cases. It is therefore essential that the user briefly view each of the events in the context of the flow record to determine suitability.

- The **zoom tools** allow the user to get a get a closer look at specific parts of the record.
- To alter an event, click on the '**Modify event start and end dates**' button which activates the graph, showing each data point as a blue diamond. As prompted by the prompt line above the hydrograph box, click on the specific event to be changed. The chosen event will now show in yellow. As prompted, click on the preferred start date and then click on the preferred end date. To de-activate the graph re-click the 'Modify event start and end dates' button. The de-activation function is also useful if an incorrect start date is chosen, or incorrect event is activated.
- The "semi-log plot" function is a useful aid choosing the correct end dates, as the change in angle in the recession limb of the hydrograph is more defined on the semi-log plot than on the linear scale hydrograph. To use this function, activate the graph using the 'Modify event start and end dates' button and then click on the event to be plotted. Click on the '**Semi-log**' button and a window containing the plot will appear below the hydrograph window (Figure 3.4). Multiple events can be plotted on the semi-log scale by activating the graph and simply clicking the appropriate event followed by the 'Semi-log' button for each desired event. To close the semi-log plot window, click on the door icon in the top left corner of the window.

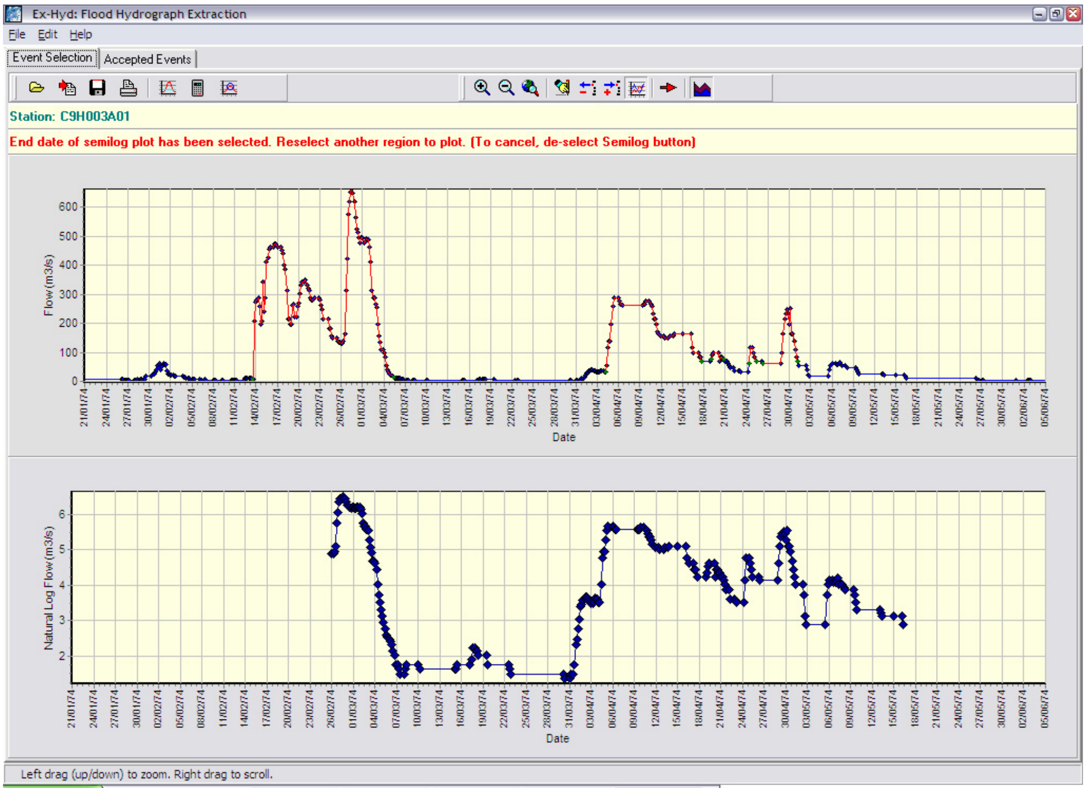

Figure 3.4 Hydrographs and Semi-log plots

- To add an event, click on the 'Modify event start and end dates' button to activate the graph. Then click the '**Add Event**' button. As prompted, click on the desired start date, and then click on the desired end date. The new event is stored in internal memory and plotted over the blue base-line in red. To de-activate, re-click the 'Modify event start and end dates' button.
- To delete an event, click on the 'Modify event start and end dates' button to activate graph. Then click the '**Delete Event**' button. As prompted, click the event to be deleted. The event is removed from internal memory and re-plotted in blue. To deactivate the graph, re-click the 'Modify event start and end dates' button.

3.3.4 Saving events

This function allows the user to save events to be recalled into the program and confirmed at a later stage. The events are saved in a text file in a format specific to this program.

- To save, click on the 'Save Events' button. A window appears allowing the user to choose an appropriate file in which to save the events.

3.3.5 Loading events

To upload files of events that have previously been defined and saved using the WRC Flood Hydrograph Extraction software, the following is required:

- Click on 'Load Saved Events' button.
- A window opens allowing the user to select the correct saved file.
- Click 'Open'.

3.3.6 Accepting events

When the user is satisfied with the accuracy of the start and end dates of the events, the function of accepting the events allows the user to view individual events and their characteristics.

- To accept events, click on the 'Accept Events and Post to Database' button. Once the significant events have been posted to the database they can be viewed on an individual basis. The database can be accessed by selecting the "Accepted Events" tab which can be found just to the right of the current tab, at the top of the screen. When the "Accepted Events" tab is selected, a database window is presented (Figure 3.5).

- The top window in the database window, labelled *Stations*, lists all the stations that have been placed in the database. The station that was added last will be at the bottom of the list. Stations can be selected by left clicking on them.

- Below this window is the box labelled *Pegram Events*. These events are basically a wetter period in the record when a series of events occurred close together. The Pegram events are listed chronologically, together with their start and end dates as well as the maximum peak of all the flood events in that particular period.

- The bottom left window displays all the individual flood events that are part of the selected Pegram event above it. This is where the peak flow, date and flood volume data are presented for each hydrograph. The hydrographs themselves are presented in a table, as well as graphically on the right hand side of the screen.

- It is possible to edit the data in the database by using a toolbar, which is located under each window. This toolbar can also be used to scroll through events by selecting the left and right arrow buttons.

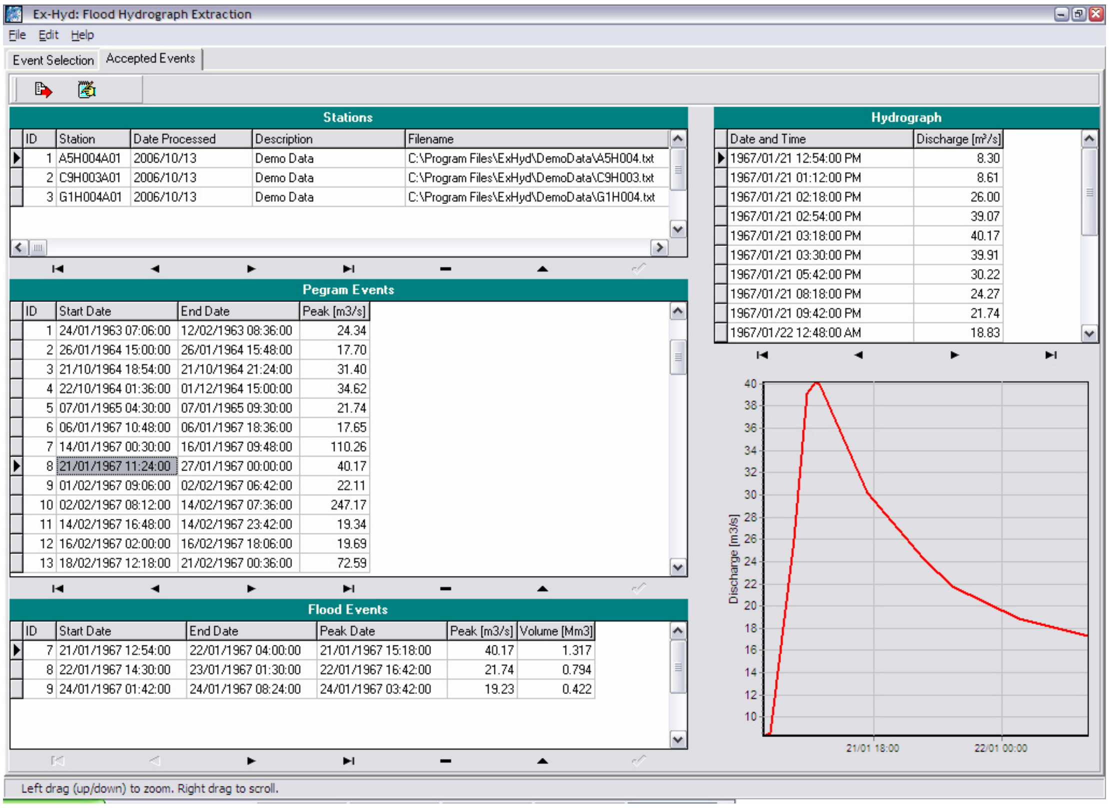

Figure 3.5 Database Window

3.3.7 Exporting events

This function writes the accepted events to a text file database.

- Click on 'Export Events' button.
- A window opens which allows the user to specify the file and location to which the events are written.
- The events are written out to a text file giving date and time of writing out, the start and end dates, peak (in m^3/s) and volume (in Mm^3) for each individual event. Below this information are the dates, times and flows recorded at each point within the event.
- The process of writing out the events may take some time as the software exports each of the points in the events as well as calculated information such as peaks and volumes.

4. REFERENCES

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HRU, 1969. *Synthetic unitgraphs for South Africa*. Report No 3/69, Hydrological Research Unit, University of the Witwatersrand, Johannesburg.

HRU, 1972. *Design flood determination in South Africa*. Report No 1/72, Hydrological Research Unit, University of the Witwatersrand, Johannesburg.

APPENDIX A
Flow gauges used or considered by HRU (1/72), Hiemstra and Francis (1979) and Alexander (2003)

Gauge Number	Catchment Area (km²)	HRU 1/72[1]	Hiemstra[2]		Alex[3]	Record Period			This Study	Reason
			Use	Discard		Start Date	End Date	Years		
A2H001	2909		Y			1904	1922	18	discard	no DT[4]
A2H002	1207	Y	Y			1904	1922	18	discard	no DT
A2H003	495	Y	Y		Y	1904	1929	25	discard	no DT
A2H004	137			Y	Y	1903	1946	43	discard	no DT
A2H005	808				Y	1904	1950	46	discard	permanent gaps
A2H006	1028	Y			Y	1905	to date	98	consider	use latter portion of record
A2H007	142				Y	1905	1951	46	discard	gauging problems to 1946
A2H009	481			Y		1918	1930	12	discard	permanent gaps and exceeded values
A2H011	308			Y		1919	1921	2	discard	short record < 15 years
A2H012	2551		Y		Y	1922	to date	81	consider	use latter portion of record
A2H013	1171	Y		Y	Y	1922	to date	81	consider	use latter portion of record
A2H015	23940			Y		1927	1931	4	discard	short record < 15 years
A2H017	70			Y		1927	1936	9	discard	short record < 15 years
A2H019	613			Y		1951	1983	32	consider	A2R015 w-component, use to 1983
A2H020	4558			Y		1951	1970	19	consider	
A2H021	7483			Y		1955	to date	48	consider	
A2H024	13			Y		1958	to date	45	discard	low peak flows, catchment areas < 15 km²
A3H001	1165	Y	Y		Y	1906	1939	33	discard	many exceedences, gaps in primary data
A3H007	8685			Y		1957	to date	46	consider	
A3H008	1213			Y		1923	1933	10	discard	short record < 15 years
A4H002	1777			Y	Y	1948	to date	55	discard	short record < 15 years
A4H003	519			Y		1954	1963	9	discard	many gaps, exceeded values, Hiemstra discarded
A5H001	619			Y		1937	1967	30	discard	no DT
A5H004	629				Y	1962	to date	41	consider	DT extension required
A6H002	984			Y	Y	1971	to date	32	discard	no DT
A6H004	521			Y		1938	1951	13	discard	short record < 15 years
A6H005	1331			Y		1946	1952	6	discard	short record < 15 years
A6H006	168			Y	Y	1949	to date	54	consider	
A7H003	6700			Y	Y	1947	to date	56	consider	
A8H001	554			Y		1932	1946	14	discard	short record < 15 years
A8H002	500			Y		1937	1947	10	discard	short record < 15 years
A9H001	912	Y		Y		1931	to date	72	consider	downstream A9R001, early part of record suspect
A9H002	96	Y		Y		1931	to date	72	consider	1931 - 1961 minimum values, use latter part of record
A9H003	62	Y				1931	to date	72	consider	
B1H001	3989	Y		Y	Y	1904	1951	47	consider	
B1H004	376			Y		1959	to date	44	consider	some gaps in record
B2H001	1594	Y	Y		Y	1904	1951	47	discard	many exceedences in primary data
B4H003	2240		Y		Y	1955	to date	48	discard	DT needs to extend from 0.79m to 2.64m
B5H002	31416			Y		1948	to date	55	discard	daily record
B6H001	508	Y		Y	Y	1909	to date	94	consider	
B6H002	97	Y				1909	1939	30	discard	daily record
B6H004	2241			Y		1950	to date	53	discard	DT needs to extend from 1.0m to 4.74m
B7H001	135			Y		1938	1950	12	discard	short record < 15 years
B7H002	58	Y			Y	1948	to date	55	discard	continuous portion of record too short
B7H003	98			Y		1948	1973	25	consider	DT extension required

Gauge Number	Catchment Area (km²)	HRU 1/72[1]	Hiemstra[2]		Alex[3]	Record Period			This Study	Reason
			Use	Discard		Start Date	End Date	Years		
B7H004	136	Y	Y		Y	1950	to date	53	consider	
B8H005	16.6			Y		1948	1956	8	discard	short record < 15 years
B8H006	38.9			Y		1948	1954	6	discard	short record < 15 years
B8H008	4716			Y		1959	to date	44	consider	gaps to 1977, consider after 1977
B8H009	851	Y				1960	to date	43	consider	downstream of dams
B8H010	477	Y				1960	to date	43	consider	
C1H001	8193		Y		Y	1905	to date	98	consider	downstream of Grootdraai Dam, verify data, use only portions ?
C1H002	4152			Y	Y	1906	to date	97	consider	DT extension required
C1R001	38505				Y	1936	to date	67	consider	Vaal Dam
C2H001	3595			Y	Y	1904	to date	99	consider	DT extension required, first part daily, small peaks
C2H002	5594			Y		1905	1922	17	discard	short record, recording problems after 1915
C2H003	38564			Y		1923	to date	80	consider	verify data, DT extension required
C2H006	1335			Y		1906	1968	62	discard	opened in 1906, but continuous record only from 1960 to 1968
C2H017	75859			Y		1929	1975	46	discard	no DT
C2H018	49120			Y		1938	to date	65	consider	verify, downstream Vaal Dam
C2H020	46.6			Y		1952	1965	13	discard	no DT
C2H021	1726			Y		1952	to date	51	consider	verify data, daily
C2H026	26			Y		1957	to date	46	discard	many gaps in primary data
C2H028	31			Y		1957	to date	46	discard	small peaks
C3H003	10990			Y	Y	1923	to date	80	consider	DT extension required
C3H004	10204		Y			1923	1947	24	discard	daily
C3H007	24097			Y		1948	to date	55	consider	verify data, use latter portion
C4H001	5504		Y		Y	1923	1947	24	discard	no DT
C4H002	17550		Y		Y	1935	1972	37	consider	verify data, daily
C4H003	5404			Y	Y	1938	1954	16	discard	no DT
C5H001	11052			Y		1912	1938	26	discard	data problems, no DT
C5H003	1650	Y		Y		1918	1954	36	discard	verify data, daily, rating table mostly exceeded
C5H004	5012	Y	Y		Y	1904	1947	43	discard	no DT
C5H007	348	Y	Y		Y	1923	to date	80	discard	DT extension too much
C5H008	593			Y	Y	1931	1986	55	consider	use latter portion , possibly from 1967
C5H010	1994		Y			1931	1948	17	discard	daily, no DT
C5H011	267			Y		1934	1949	15	discard	short record, daily
C5H012	2372	Y	Y		Y	1936	to date	67	consider	verify data from 1954, continuous from 1972
C5H015	6009		Y		Y	1949	1983	34	consider	sub-daily to 1974, verify data
C5H016	33351			Y		1953	to date	50	consider	verify data, extend DT
C6H001	5674			Y	Y	1913	to date	90	discard	DT extension too much
C7H001	5255		Y		Y	1923	1948	25	discard	no DT
C7H002	1085			Y		1930	1947	17	discard	short record, daily
C7H003	914			Y		1947	to date	56	consider	verify data, DT extension required
C8H001	15673			Y	Y	1923	to date	80	consider	use from 1961 only
C8H003	806	Y		Y	Y	1964	to date	39	consider	extend DT, use from 1966
C9H003	120902		Y			1909	to date	94	consider	sub-daily data
C9H006	108652		Y			1937	to date	66	consider	verify data, downstream Bloemhof Dam
C9H008	115057			Y		1947	to date	56	consider	few gaps
D1H001	2397	Y		Y	Y	1912	to date	91	consider	first part daily only, more than one DT extension required
D1H003	37075				Y	1914	to date	89	consider	daily record, Orange river gauge, use daily ???
D1H004	348	Y		Y	Y	1925	1981	56	discard	daily record, gaps
D1H005	10680		Y		Y	1932	to date	71	consider	Hiemstra used, data problems to verify
D1H006	3051			Y	Y	1949	to date	54	consider	DT extension required, use latter portion
D2H001	13421			Y	Y	1919	1978	59	consider	data problems to verify, use latter

Gauge Number	Catchment Area (km²)	HRU 1/72[1]	Hiemstra[2]		Alex[3]	Record Period			This Study	Reason
			Use	Discard		Start Date	End Date	Years		
										portion
D2H003	1424	Y		Y		1935	1954	19	consider	verify data, daily
D2H005	3857	Y	Y			1941	1956	15	discard	short record, DT to be extended from 2.5m to 9.8
D3H005	91994		Y			1948	to date	55	discard	DT to be extended from 0.7 m to 6.0, most years exceeded
D4H002	342	Y		Y	Y	1927	1964	37	discard	no DT
D5H001	2129		Y		Y	1927	1953	26	discard	no DT
D5H002	17154			Y	Y	1927	1948	21	discard	no DT
D5H003	1509			Y	Y	1927	to date	76	consider	use latter portion
D5H004	5799		Y		Y	1929	1979	50	discard	no DT
D5H006	414			Y		1930	1954	24	discard	no DT
D5H008	354	Y		Y		1935	1950	15	discard	no DT
D5H009	766	Y		Y		1936	1947	11	discard	no DT
D6H002	6440	Y	Y			1926	1942	16	discard	no DT
E2H002	6903	Y	Y		Y	1923	to date	80	discard	Extension of DT from 2 to 7m too much
E2H003	24044	Y		Y	Y	1927	to date	76	consider	extension of more than one DT required
E2H006	24044				Y	1927	to date	76	discard	no DT
G1H002	187	Y	Y			1951	1970	19	consider	verify middle part of record
G1H003	46	Y		Y	Y	1959	to date	44	discard	use from 1951, extend of DT required
G1H004	70	Y		Y	Y	1979	to date	24	consider	low flow gauge, low DT
G1H007	712	Y		Y	Y	1951	1979	28	discard	daily
G1H008	395	Y		Y	Y	1954	to date	49	consider	use latter portion
G2H008	121	Y			Y	1979	to date	24	consider	use latter portion
G4H003	145			Y		1950	1954	4	discard	< 15 years
G4H005	146				Y	1957	to date	46	consider	use latter portion
G5H005	658				Y	1952	1980	28	discard	no DT, short record
G5H006	3			Y	Y	1956	to date	47	discard	catchment < 10 km2
H1H003	657	Y		Y	Y	1923	to date	80	consider	downstream Ceres Dam
H1H006	753		Y			1950	to date	53	consider	exceedences in primary data, extension of DT required
H1H007	84	Y	Y		Y	1950	to date	53	consider	exceedences in primary data, extension of DT required
H1H018	113		Y			1969	to date	34	consider	gauge not used previously
H2H003	718	Y			Y	1950	to date	53	consider	"
H3H001	611			Y	Y	1925	1947	22	discard	daily
H3H003	93			Y		1934	1948	14	discard	short record, daily
H4H002	4644			Y		1911	1950	39	discard	daily record, no DT
H4H005	24			Y	Y	1950	1981	31	consider	portion daily, portion continuous
H4H006	2939				Y	1950	to date	53	consider	
H6H003	497			Y	Y	1932	1974	42	discard	short period continuous
H6H008	38				Y	1964	to date	39	consider	DT extension required, all annual maximum peaks exceeded
H7H002	16			Y		1938	1948	10	discard	short record
H7H003	451	Y		Y		1949	1967	18	consider	DT extension required, record sub-daily?
H7H004	28	Y	Y		Y	1951	to date	52	consider	
H7H005	28				Y	1951	to date	52	consider	
J1H006	319			Y	Y	1948	1977	29	discard	no DT
J2H001	10292			Y		1911	1921	10	discard	short record
J2H002	186			Y		1911	1918	7	discard	short record
J2H003	17815		Y		Y	1924	1942	18	discard	number of DT exceedences, daily values, missing values
J2H005	253			Y	Y	1955	to date	48	consider	small peaks
J2H007	25			Y		1955	to date	48	consider	small peaks
J3H001	1484	Y		Y		1912	1922	10	discard	short record
J3H003	422			Y	Y	1913	1965	52	discard	no DT
J3H004	4252		Y		Y	1923	to date	80	discard	48 of 70 years exceeded DT
J3H005	95			Y	Y	1926	1947	21	discard	daily record

Gauge Number	Catchment Area (km²)	HRU 1/72[1]	Hiemstra[2]		Alex[3]	Record Period			This Study	Reason
			Use	Discard		Start Date	End Date	Years		
K1H001	144				Y	1953	1977	24	discard	extension of DT required, short continuous record, rest daily
K1H002	3.8				Y	1958	to date	45	discard	catchment < 10 km2
K2H002	131	Y			Y	1961	to date	42	consider	
K3H001	47	Y				1961	to date	42	discard	downstream Wolwedans Dam
K4H002	22	Y			Y	1961	to date	42	consider	
K4H003	72	Y				1961	to date	42	consider	
K5H002	133	Y				1961	to date	42	consider	
L2H002	899	Y		Y		1925	1952	27	discard	daily, unstable river channel
L3H001	20339			Y		1917	1957	40	discard	gaps in primary data, downstream Beervlei Dam
L7H002	25587		Y			1928	1985	57	discard	no DT
N1H003	1040	Y				1927	1932	5	discard	no DT
N2H002	11395			Y		1923	to date	80	consider	continuous from 19'79 only, d/s dams
N2H005	13600			Y		1928	1947	19	discard	no DT
N3H001	1598	Y		Y		1928	1947	19	discard	no DT
Q1H001	9091		Y		Y	1918	to date	85	consider	
Q1H006	1577	Y		Y		1927	1948	21	discard	daily, unstable river channel
Q2H001	2445	Y				1982	to date	21	discard	no DT
Q3H001	862	Y		Y	Y	1926	1948	22	discard	no DT
Q6H001	686	Y		Y		1918	1937	19	discard	short record, daily
Q7H001	18989		Y			1906	1928	22	discard	daily, most annuals peaks exceeded DT
Q7H002	18452		Y			1922	1948	26	consider	daily, Hiemstra used, verify data
Q7H003	18503			Y	Y	1928	1948	20	discard	no DT
Q8H001	19134	Y				1972	to date	31	discard	zero DT loaded
Q8H004	808				Y	1957	1986	29	discard	daily
Q9H002	1245	Y			Y	1926	to date	77	consider	extension of DTs required, consider from 1969
Q9H004	409	Y		Y	Y	1926	1964	38	discard	no DT
Q9H007	82			Y		1928	1943	15	discard	many gaps, record incomplete
Q9H008	748	Y		Y	Y	1921	1970	49	consider	portion daily, verify data
Q9H009	78			Y		1928	1938	10	discard	short record
Q9H010	29328		Y			1930	1957	27	consider	verify data
Q9H011	539	Y		Y	Y	1931	1967	36	consider	portion daily
Q9H012	23067		Y			1935	to date	68	consider	use latter portion, DT extension required
R1H001	238	Y		Y	Y	1928	to date	75	discard	many exceedences, long DT extension required
R1H002	665	Y		Y		1938	1950	12	discard	short record, daily
R1H003	266			Y		1928	1948	20	discard	no DT
R1H005	482	Y		Y	Y	1948	to date	55	consider	long DT extension required
R1H006	100			Y		1948	1977	29	discard	daily flows, min flow problems, large gap in 60's
R1H013	1515				Y	1950	1986	36	consider	DT extension required
R1H014	70			Y		1953	to date	50	discard	many rating table exceedences
R2H005	411	Y		Y	Y	1977	1980	3	consider	gaps 1979 to 1987
R2H007	82	Y		Y	Y	1947	1981	34	consider	many exceedences, long DT extension required
R2H008	61	Y		Y	Y	1947	to date	56	consider	long DT extension required
R2H009	103			Y	Y	1947	to date	56	consider	many exceedences, long DT extension required
S2H001	500	Y				1972	to date	31	discard	no DT
S3H002	796	Y		Y	Y	1947	to date	56	discard	minimum value exceeded, long DT extension, early record daily
S6H001	90			Y	Y	1947	to date	56	consider	
S6H002	49	Y		Y		1947	to date	56	discard	DT extension required too much
T1H004	4908				Y	1953	to date	50	consider	use latter portion only
T2H002	1199			Y		1957	to date	46	discard	only consider for use to 1977, Umtata Dam, DT extension too much
T3H002	2101	Y	Y		Y	1949	to date	54	consider	use latter portion only, DT extension

Gauge Number	Catchment Area (km²)	HRU 1/72[1]	Hiemstra[2]		Alex[3]	Record Period			This Study	Reason
			Use	Discard		Start Date	End Date	Years		
										required
T3H004	1029	Y		Y	Y	1947	to date	56	consider	use latter portion only, verify DT and possibly extend DT
T3H005	2597	Y		Y		1951	to date	52	consider	DT extension required
T3H006	4268			Y		1951	to date	52	consider	DT extension required
T4H001	715	Y		Y	Y	1942	to date	61	consider	use latter portion only, DT extension required
T5H001	3643	Y		Y	Y	1931	to date	72	consider	DT extension required, check data, daily?
T5H002	867			Y		1933	to date	70	discard	use latter portion only, DT extension required
T5H004	545	Y		Y	Y	1949	to date	54	consider	
T5H006	534			Y		1949	1959	10	discard	short record
U1H001	3339			Y		1931	1936	5	discard	short record
U1H003	4375			Y		1951	1969	18	discard	short record
U2H005	2519			Y	Y	1950	to date	53	consider	upstream of Nagle Dam
U2H006	339			Y		1954	to date	49	consider	
U2H007	358			Y		1954	to date	49	discard	DT extension required, many exceedences, small peaks
U2H011	176			Y		1957	to date	46	consider	first part daily, consider latter par of record
U2H012	438	Y				1960	to date	43	consider	verify data
U2H013	299	Y				1960	to date	43	consider	verify data
U2R002	2535				Y	1969	to date	34	consider	Nagle Dam
U3H002	356			Y		1950	1977	27	consider	long DT extension required, exceedences, verify data
U4H001	2600			Y		1928	1946	18	discard	daily, no DT
U4H002	316			Y	Y	1949	to date	54	discard	DT extension required, many exceedences, small peaks
U4H003	49			Y		1956	1976	20	discard	daily, short record
U4H004	11.5			Y		1956	1976	20	discard	daily, short record
U7H001	16			Y		1949	to date	54	discard	small peaks, portion daily flows only
U7H002	938			Y		1957	1965	8	discard	daily, short record
V1H003	1689				Y	1931	to date	72	discard	catchment area < 2 km2
V1H004	441	Y				1949	1974	25	discard	short record, continuous only from 1962
V1H006	441				Y	1949	1974	25	discard	catchment area < 2 km2
V1H009	196	Y				1954	to date	49	consider	verify data
V2H001	1976	Y			Y	1934	1947	13	discard	daily record only
V2H002	937	Y		Y	Y	1950	to date	53	consider	
V2R001	152			Y		1962	to date	41	consider	verify data
V3H005	676			Y	Y	1951	to date	52	consider	use latter part only
V3H007	129			Y		1948	to date	55	consider	DT extension required, use latter portion only
V3R001	543				Y	1960	1985	25	consider	
V5H002	28920			Y	Y	1956	to date	47	consider	Tugela River, downstream dams?
V6H002	12862			Y	Y	1927	to date	76	consider	use latter part only
W1H001	570			Y		1928	1931	3	discard	short record, daily, no DT
W1H002	1276			Y		1921	1940	19	discard	short record, daily, no DT
W1H005	45			Y		1948	to date	55	consider	verify data with DWAF
W2H002	3468	Y		Y	Y	1947	1962	15	discard	short record, exceedences
W2H003	5136			Y		1947	1956	9	discard	short record
W3H001	1467	Y				1928	to date	75	consider	use latter part of record
W4H002	7081	Y		Y		1929	1968	39	consider	verify DATA, DT extension required, daily to 1950 ??
W4H003	7081				Y	1929	1968	39	discard	no DT
W4H004	948	Y		Y		1950	to date	53	discard	many gaps
W5H003	218			Y		1950	1966	16	discard	low peak flows, daily flows, short rec
W5H004	460			Y		1950	to date	53	consider	DT extension required
W5H005	804	Y	Y		Y	1950	to date	53	consider	early part daily
W5H006	180			Y	Y	1950	to date	53	consider	early part daily, extend DT from 1 to

Gauge Number	Catchment Area (km²)	HRU 1/72[1]	Hiemstra[2]		Alex[3]	Record Period			This Study	Reason
			Use	Discard		Start Date	End Date	Years		
										2.85 m??
W5H007	531	Y		Y		1951	1968	17	consider	usable, verify data
W5H008	701			Y	Y	1951	to date	52	discard	long DT extension required, exceeded values, early part daily
X1H001	5499	Y		Y	Y	1909	to date	94	consider	downstream Nooitgedacht and Vygeboom Dams
X1H007	297			Y		1958	1969	11	discard	short record, daily
X2H002	176					1904	1947	43	consider	
X2H008	180	Y		Y	Y	1948	to date	55	discard	no DT
X2H009	280	Y		Y	Y	1948	1966	18	consider	
X2H010	126			Y	Y	1948	to date	55	consider	early part daily
X2H011	402	Y		Y		1956	to date	47	consider	
X2H013	1518			Y		1959	to date	44	consider	
X2H015	1554	Y		Y		1959	to date	44	consider	gaps in record, extension of DT required
X2H018	618	Y				1960	to date	43	consider	gaps in record, extension of DT required
X2H022	1639	Y				1960	to date	43	consider	verify data
X3H001	174				Y	1916	to date	87	consider	early part daily
X3H003	52	Y				1948	to date	55	consider	small peaks
X3H006	766	Y		Y	Y	1958	to date	45	consider	

APPENDIX B
Flow gauges used in this study

Gauge No.	Dates	Area (km^2)	Record length (years)
A2H006	1949/10-2003/02	1028	54
A2H012	1953	2551	50
A2H013	1960/10- 2002/11	1171	43
A2H019	196010-1983/08	613	23
A2H021	1961-2003	7483	42
A5H004	1962-2002	629	40
6H006	1969-2002	168	33
B1H004	1960-2002	376	42
B6H001	1960-2002	518	42
B7H003	1963-1972/11	84	9
B7H004	1961-199910	136	38
B8H008	1969/03-1988/02	4710	
B8H009	1960	851	
B8H010	1960-1996/02	477	36
C1H001	1960-1989/02	8193	29
C2H003	1963-1993/12	38564	31
C2H018	1960-2002/10	49120	43
C3H007	1960-2002/05	24097	42
C8H001	1964-2002	15673	38
C8H003	1966/10-2002/07	806	36
C9H003	1961/01-2000/09	120902	39
C9H008	1965-2002	115057	37
D1H005	1987/11-1996/11	10680	10
D5H003	1987-2002	1509	16
E2H003		24044	
G1H004	1959/10-2002/09	70	43
G1H008	1962/10-2002/08	1690	40
G4H005	1962/10-2002/08	146	40
H1H006	1962/10-2002/08	753	40
H1H007	1962/10-2002/08	84	40
H1H018	1969/10-2002/08	113	33
H2H003	1962/10-1985/05	718	23
H4H005	1964/10-1981/12	20	
H4H006	1950	2939	
H7H004	1951	28	
J2H005	1967/10-2002/07	253	35
J2H007		25	
K2H002	1961/10-2002/08	131	41
K4H002		22	
K4H003	1962/10-2002/08	72	40
K5H002	1961/10-2002/08	133	41
N2H002	1979/10-1992/12	11395	14
Q9H002	1969	1245	
R1H005	1980/07-1995/08	482	16
S6H001	1960	90	
T1H004	1968/10-2001/11	4908	30
T3H005	1961/10-2002/07	2597	37
T3H006	1961/10-2002/09	4268	40
T4H001	1966/10-2002/04	715	36

Gauge No.	Dates	Area (km^2)	Record length (years)
U2H005	1962/11-2004/04	2519	
U2H006	1966/10-2002/05	339	36
U2H011	1960/10-2002/05	176	42
U2H012	1960/10-2003/06	438	43
V1H009	1963/10-2003/03	196	40
V2H002	1962/10-2002/05	937	40
V3H005	1964/10-1993/03	676	29
V3H007	1960	129	
V5H002	1959	28920	
X1H001	1958/10-2002/12	5499	45
X2H008	1963/10-2002/08	180	39
X2H010	1966	126	
X2H011	1962/10-1999/12	402	38
X2H015	1960/10-2002/08	1554	42
X3H003	1964/10-2004/07	52	40
X3H006	1960/10-2001/03	766	40

PART 3

MODERNISED PERSPECTIVES ON EXISTING DESIGN FLOOD METHODOLOGIES IN SOUTH AFRICA

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TABLES

FIGURES

1. INTRODUCTION

In SANCOLD's *Guidelines on Safety in Relation to Floods* (SANCOLD, 1991), two design flood concepts play key roles in the safety assessment of dams under "extreme flood conditions". These are the so-called "Probable Maximum Flood" (PMF), (HRU, 1/72) which is theoretically the largest possible flood peak that could occur at a structure’s location, and the so-called "Regional Maximum Flood" (RMF), (Kovaćs, 1980) which is an empirically derived flood peak based on the regional ordering of maximum observed flood peaks from 130 sites around South Africa. The calculation of the PMF and RMF is required by the SANCOLD Guidelines for the determination of the "Safety Evaluation Flood" (SEF) and the "Safety Evaluation Discharge" (SED), respectively. The use of these extreme design floods has however been an ongoing topic of debate among practitioners, especially in terms of the PMF.

The PMF is derived from the estimation of the Probable Maximum Precipitation (PMP), the procedure of which is described in detail in the HRU (1972) report. Although the HRU's approach for the estimation of PMP is still aligned with present day international practice, it is based on only about 30 years of rainfall data from 1932 to the 1960s, thus resulting in concerns over its current day application. Besides this, the PMF has also been under critical review both locally and internationally because of its statistical nebulousness (Graham, 2000). Another concern among practitioners has been the apparent discrepancies country-wide between the PMF and the RMF, which have led to confusion over the applicability of these extreme floods when using the SANCOLD Guidelines to perform a dam design or safety assessment.

In response to the aforementioned concerns, research was undertaken as part of this WRC Project to "modernise" design flood methodologies in the context of dam safety in South Africa, with the focus on "extreme" flood concepts. This research was undertaken during Phase 2 of the Project, entitled "Improvement of Flood Hydrograph Generation Techniques for South Africa for Dam Safety Purposes".

For this investigation into the modernisation of extreme flood concepts in the context of dam safety evaluation, four main areas of research were initiated. The first was a review of design extreme rainfall (PMP) approaches, both internationally and locally, in order to evaluate the HRU PMP curves provided in the HRU 1/72 report, using severe storm rainfalls that have occurred post-1960 (Part 3, Section 2). Following this, the second area of research comprised a review of design storm loss approaches, both internationally and locally, with a focus on an evaluation of the HRU (1/72) storm losses approach (Part 3, Section 3). The latter approach is recommended in the SANCOLD Guidelines for the calculation of design storm losses. Following from the storm losses study, Unitgraph-based design flood estimates were compared with probabilistic estimates (Part 3, Section 4). It was felt that the latter investigation would be valuable, seeing that the HRU Unitgraph method is still the most commonly used design flood hydrograph generation approach in South Africa.

The fourth area of research, involved a detailed investigation into modernised extreme flood concepts in the South African dam safety context (Part 3, Section 5). This research involved a detailed review of local and international approaches to dam safety evaluation under extreme flood conditions. Also, to address some of the concerns raised over the use of the RMF and PMF according to the SANCOLD Guidelines, an investigation was undertaken to attempt to link a recurrence interval (RI), or annual exceedence probability (AEP), to these extreme design floods using an updated and sanitised South African flood database.

2. REVIEW OF EXTREME DESIGN RAINFALL APPROACHES IN SOUTH AFRICA

2.1 INTRODUCTION

In the design of high-hazard dams, such as those upstream of populated areas, it has become common practice in the view of safety to design the structure for the theoretically largest possible flood that could occur at the structure's location. This flood, often known as the Probable Maximum Flood (PMF), may be estimated via probabilistic, empirical or deterministic approaches. If a deterministic approach is taken, the PMF is often calculated from the estimation of a design extreme rainfall. The World Meteorological Organisation (WMO) names this extreme design rainfall the Probable Maximum Precipitation (PMP), which it defines as 'the greatest depth of precipitation for a given duration meteorologically possible for a given size storm area at a particular location at a particular time of year, with no allowance made for long-term climate trends' (WMO, 1986). It is this theoretical precipitation phenomenon that forms the focus of this report.

The concept of a theoretical upper limit of rainfall was developed in the 1930s in the USA and was initially termed the 'maximum possible precipitation'. After 1950, however, this term was changed to the 'probable maximum precipitation' in recognition that this upper limit of rainfall could not be determined accurately and had to be based on estimation methods and scientific engineering judgement (WMO, 1986). The most common methods used worldwide for the estimation of the PMP are the storm maximisation (hydrometeorological) approach and the statistical (Hershfield) approach (WMO, 1986). The storm maximisation and transposition approach is a physical approach that requires site-specific meteorological and geographical data, and is employed extensively in many dam-building countries, such as the USA, Australia, India, Malaysia, Czech Republic, etc. Where site-specific data is not available, the statistical approach can be applied to determine an efficient estimate of the PMP based on annual maximum rainfall series. Other approaches for the estimation of the PMP include the storm model approach (Collier and Hardakar, 1996), and the more recent radar-based (Cluckie and Pessoa, 1990) and multifractal (Barros and Douglas, 2002) approaches.

For South Africa, the only established guidelines for the estimation of PMP are those set out in the HRU 1/72 Report (HRU, 1972). As HRU (1972) approach for the estimation of PMP is based only on about 30 years of rainfall data from 1932 to the 1960s, concerns have been expressed over its current day application. The objective of this report is to evaluate the HRU PMP envelope curves against severe storm rainfalls which have occurred in the last thirty years, establish whether the HRU PMP envelope curves are still applicable for design purposes today and provide guidelines in their use and possible recommendations for further research and refinement.

Prior to evaluating the HRU PMP envelope curves, an international literature review of methodologies used in the estimation of PMP is presented.

2.2 LITERATURE REVIEW

In this Section the WMO, USA, UK, Australian and South African methods of PMP estimation are reviewed.

2.2.1 Storm maximisation and transposition

Background

One of the most commonly used and accepted method for the estimation of PMP is that of storm maximisation and transposition (WMO, 1986). These concepts were first introduced in the late 1930s by the National Weather Service (NWS) of the USA and later documented in detail by the WMO in 1973.

Storm maximisation involves the upward adjustment of observed extreme storm rainfall, based on the assumption that at least one of the observed storms over the study area operates at maximum 'efficiency'. This implies that within a single storm there is a simultaneous occurrence of:

i. a maximum amount of moisture, and
ii. a maximum conversion rate of moisture to precipitation (WMO, 1986).

Maximising the moisture content of the storm is a fairly simple procedure consisting of the adjustment of the rainfall depth measured in the storm by the ratio of the maximum observed atmospheric precipitable water content in the area of the catchment to the actual precipitable water content that was observed during the storm. The precipitable water content is defined by the NWS as the total atmospheric water vapour contained in a column of air of unit cross-sectional area between two pressure levels. Ideally, atmospheric moisture content can be determined by radiosonde data, but in most cases this information is scarce and the surface dew-point temperature used instead. In some cases, rainfall is also maximised for wind speed and direction.

Maximising the rate of conversion of storm moisture to storm precipitation is more difficult and requires a large sample of storms, at least one of which is assumed to be operating at maximum efficiency. In order to obtain a sufficiently large storm sample, storm transposition is often required. Storm transposition implies the displacement of the characteristics of the storm from its original location to the location of the study area as if the storm could just as easily have occurred there (Bureau of Meteorology Australia, 2003). Storm transposition requires 'site-specific' details of the original and transposed storms.

Early estimates of the PMP were based on in-situ maximisation in which only the storms that had occurred over the study area were considered for maximisation. To increase the storm sample size, in-situ maximisation was then combined with storm transposition which generally led to higher PMP estimates. Storm transposition in this case was limited to storms that occurred near the study area in regions of similar topographic features. From the early 1960s, the US introduced a 'generalised' approach to storm maximisation and transposition. Generalised methods differ from the in-situ and transposition methods in that they use a deterministic approach to remove the 'site-specific' components of storms over a 'meteorologically compatible' region by adjusting for moisture availability and topographic effects on rainfall depth. In this way, the 'useable' transposition area can be increased significantly resulting in a much larger storm sample. Standardised depth-area-duration curves for each storm within the chosen region are derived, maximised and enveloped. For the estimation of the PMP at a particular location, the envelope rainfall depth according to the area of the catchment is determined from the depth-area-duration curves and the 'site-specific' effects of the location factored back in.

Although the generalised methods are known to provide more reliable estimates of the PMP in comparison to other methods, such as the statistical approach, these methods require considerable time to execute. Meteorological and site-specific data are required, the types and quantity of which are highly variable, depending on the study area. It has also been recommended that a skilled meteorologist be included when adopting this approach (WMO, 1986). This method does not incorporate long-term climate change.

The WMO (1986) *Manual for Estimation of Probable Maximum Precipitation* is a well-established manual for generalised storm maximisation and transposition methodology. For this reason, a simplified step-by-step approach as documented in the WMO (1986) report is presented below. In addition, a more detailed application of the methodology as developed by the US National Weather Service and the Hydrology Unit of the Australian Bureau of Meteorology for South East Australia (1996), which provides a clear example of the complex procedure that is required for generalised maximisation and transposition for large-area storms. Following these sections is an example of the generalised procedure developed by the Australian Bureau of Meteorology for the estimation of PMP resulting from small area and short duration storms.

WMO (1986) methodology for the estimation of PMP

The WMO (1986) storm maximisation and transposition method was developed in consultation with the US National Weather Service. The method differentiates between procedures for orographic and non-orographic regions and is applicable for the mid-latitudes and areas up to 50 000 km^2, although it has been applied to larger areas. The WMO manual also describes the procedure for adapting the method for tropical regions.

The basic generalised method of storm maximisation and transposition for non-orographic regions is summarised in the following steps:

1. Determine the transposition limits of storms for the study area. The transposition limit refers to the boundary of the area in which storms can be transferred with minor adjustments to their rainfall amounts and is influenced by similarity of climatic and topographic conditions within the area, concentration of rainfall gauges and frequency of occurrence of severe storms.

2. Identify the major storms in the area of transposability by surveying precipitation records.

3. Perform depth-area-duration (D-A-D) analyses of the identified storms and tabulate results.

4. The first step in moisture maximisation is to determine the representative persisting 12-hour 100 kPa dew-point for each storm. This is the highest value of dew-point that is either equalled or exceed over a 12-hour period at an atmospheric pressure of 100 kPa. Dew-point temperatures are reduced adiabatically to 100 kPa in order to normalise for difference in elevation of the various stations.

   If wind maximisation is required, the maximum 24-hour average wind speed from the moisture-inflow direction is to be determined for each storm.

5. Determine the precipitable water content corresponding to the representative dew-point for each storm. Values of precipitable water (mm) between 100 kPa and an atmospheric pressure representative of the 'top' of the column of air where the moisture content is assumed to be negligible can be read from a table in the WMO manual for a particular dew-point temperature and 'top' atmospheric pressure. Generally, the latter pressure is taken as 30 kPa.

   If wind maximisation is involved, multiply the storm precipitable water content by the 24-hour average wind speed to determine the representative storm-moisture inflow index.

6. Determine the highest maximum persisting 12-hour 100 kPa dew-point of record for the study area and surrounding regions for the storm date or within 15 days of it.

7. Determine the precipitable water content corresponding to the maximum persisting 12-hour 100 kPa dew-point for the study area in the same way as described under point (5).

   For wind maximisation, calculate the maximum moisture-inflow index by multiplying the maximum precipitable water content by the maximum 24-hour average wind speed that occurred on the same date corresponding to the maximum recorded dew-point temperature.

8. Multiply the observed storm rainfall (R_1) by the combined transposition and maximisation ratio of the precipitable water content from step 7 (W_2) to the precipitable water content of step 5 (W_1). The equation below represents the upward adjustment of the observed rainfall to the maximised value (R_2):

$$R_2 = R_1\left(\frac{W_2}{W_1}\right) \tag{2.1}$$

   If wind maximisation is required, compute the maximum moisture inflow index to the representative moisture inflow index and apply this ratio to the adjusted storm rainfall.

   Where applicable, an adjustment factor must also be applied to the transposed storm for the effect of topography (elevation adjustment), and for the effect of when there is a barrier or mountain range in the path of the moist air being fed into the storm area (barrier adjustment). These adjustment procedures are presented in detail in the manual.

9. Multiply the D-A-D data for each storm by the relevant combined ratios.

10. Plot the maximised depth-duration envelopes for each area and the area-depth envelopes for each duration. Construct Area-PMP plots for each duration.

For the estimation of PMP in orographic regions, one approach that can be employed is through the use of the 'orographic separation' method. This refers to the separation of the total precipitation in the study area into an orographic precipitation component, which results from topographical influences, and a convergence precipitation component, which results from atmospheric processes independent of topographic influences. The PMP is calculated by the addition of each precipitation component, which is individually calculated. Another approach described in the WMO manual for the estimation of PMP in orographic regions is to first estimate the non-orographic PMP for the plain areas within the study region, and then to apply factors to adjust this PMP for topographic effects. If there are no plain areas within the study region, the non-orographic PMP can be estimated as if the mountains in the region did not exist. There is no basic standard procedure presented for the modification of the non-orographic PMP. Every situation is unique in terms of differences in the topography and the effect that it will have on the rainfall. For this reason, the WMO manual only presents examples from actual case studies as a guide.

US (National Weather Service) Generalised Methodology for the Estimation of PMP

The National Weather Service (NWS) has published many Hydrometeorological Reports (HMR) for the estimation of the PMP in different regions across the USA. These reports can be viewed from the NWS web-site at http://www.nws.noaa.gov/oh/hdsc/studies/pmp.html. The NWS approach to the estimation of PMP follows much the same methodology as that set out in the WMO document, with moisture maximisation, storm transposition and envelopment of the maximised transposed depth-duration and depth-area amounts as the three main steps. Following this methodology, the NWS produced mapped values of PMP estimates for all regions across the US. The steps employed for the derivation of these maps are briefly described below. To aid in the description, HMR 51 (NWS, 1980) is referred to as an example. The scope of the latter report was to determine the average PMP for any catchment from 26 to 51,800 km^2 for durations of 6 to 72 hours in the United States east of the 105th Meridian.

Storm Database

For the United States, a large published database of storms exist from which the maximum observed areal precipitation depths for various durations have been determined by a standardised depth-area-duration analysis of point rainfall values. This database, which was started in 1945, is used as the basis for all PMP estimations across the continent. Table 2.1 is an example of the number of storms that have been analysed east of the 105th meridian, and for which areal rainfall depths have been documented for indicated area sizes and durations.

Table 2.1 Number of Analysed Storms East of the 105th Meridian, with Maximum Observed Areal Rainfall Depths for Indicated Area Sizes and Durations (table extracted from HMR 51)

Area		Duration (hr)				
mi^2	km^2	6	12	24	48	72
10	26	496	482	456	356	187
200	518	521	508	483	376	201
1,000	2,590	567	555	533	419	234
5,000	12,950	528	526	517	417	262
10,000	25,900	489	489	486	406	263
20,000	51,800	396	396	396	351	242

Moisture Maximisation

The surface dew-point temperature was used as an indicator of atmospheric moisture. Two dew-point temperatures were determined per observed storm: one representing the inflow of moisture during the storm, and the other representing the maximum value for the same location and time of year as the storm. Both dew-points were reduced pseudo-adiabatically to 100 kPa. The highest persisting 12-hour values of storm and maximum dew-points temperature were then selected for maximisation. From the surface dew-point temperatures, the corresponding storm and maximum precipitable water contents were determined from available tables. Precipitable water contents were calculated for a column of air of unit cross-section between an atmospheric pressure of 100 kPa (surface level) and 20 kPa (upper level at which moisture content is assumed negligible).

Moisture maximisation was accomplished by adjusting the observed rainfall upward by the ratio of the maximum precipitable water content to the actual storm precipitable water content.

In the case where a significant mountain barrier existed between the moisture source and the rain location, or where the rainfall occurred at a higher elevation, then the mean elevation of the mountain ridge or the rainfall elevation was used as the base of the column of air rather than the 100 kPa surface when determining the surface dew-point temperatures.

Transposition Adjustment

The transposition limits for each storm were determined. These usually coincided with either topographic features (transposition should be limited to areas of similar terrain) or differences in broadscale meteorological features. The relocated rainfall values were calculated by multiplying the maximised rainfall values by the ratio of maximum precipitable water content for the transposed location to that of the storm in place.

Generalised Maps of PMP

From the storm database, all storm rainfall values that were considered to give the highest or near highest values were selected. These values were then maximised in place and transposed to their outer limits. Separate maps of maximised storm rainfall values were then produced for the range of areas and durations of interest. In the case of HMR 51, a total of 30 maps were produced indicating maximised rainfall values for area sizes of 26 to 51,800 km² and durations of 6 to 72 hours. For each map, enveloping isohyets were then drawn between the data points. As a guide to the general shape of the isohyets, other rainfall patterns were considered. In the case of HMR 51 these were maps of regional rainfall patterns of storms plotted in place, greatest monthly precipitation, greatest weekly precipitation, maximum 1-day station rainfall, maximum persisting dew-points and the 100-year station rainfall. The isohyets enveloping the maximised rainfall data are representative of the PMP isohyets for a particular area and duration.

PMP Estimation

For a known area size and rainfall duration, the estimated value of PMP can be interpolated from the corresponding map of PMP isohyets. Generalised maps of PMP, such as Figure 2.1 below, are available for all regions across the USA.

Generalised South East Australia methodology (GSAM) for the estimation of PMP (large-area storms)

Construction of a Storm Catalogue

Storm selection
An archive of the 10 highest 1-7 day rainfall values was created for each rainfall station in South East Australia. The archive was edited so that only the dates that were common to a number of stations were kept. The station rainfall totals were then compared to the 72-hour 50-year rainfall intensity at the station location. A total of 110 storms were selected for inclusion in the storm catalogue.

Data quality control
Rainfall totals for all stations within the area were printed and scrutinised for temporal and spatial consistency. Corrections were made to the data where it was obvious what needed to be done. Original rainfall observation books were retrieved for stations at the centre of the largest storms and comments by observers of rain gauge overflow were noted.

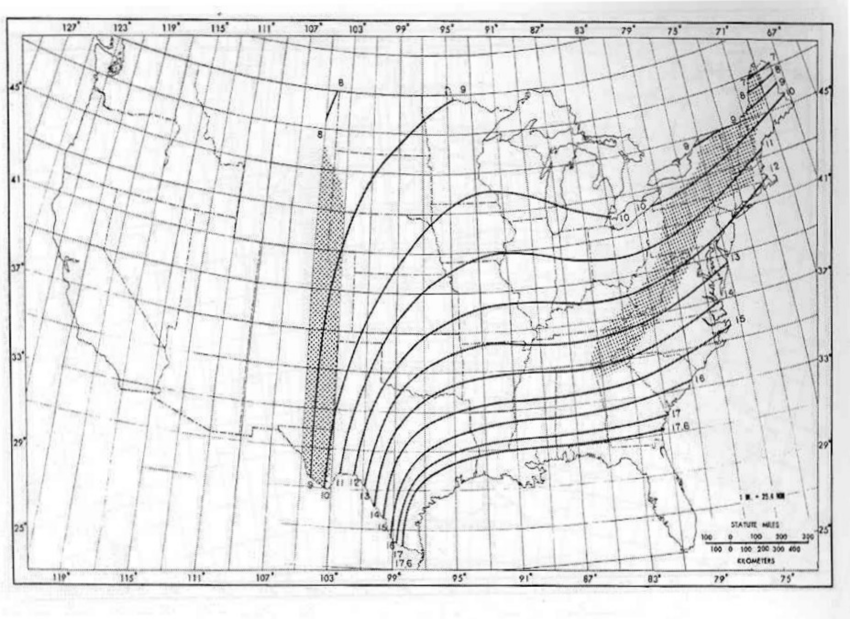

Figure 2.1 All season PMP (in.) for 24 hr 10,000 mi^2 (25,900 km^2) rainfall for the United States east of the 105th Meridian (HMR 51, Figure 40)

Storm analysis and gridding

The rainfall totals for the total storm duration were plotted and overlayed on a topographic map and analysed manually by an experienced meteorologist. The isohyets of the analysed storm were then digitised and gridded using a spline function. The gridded data was contoured and replotted to the same scale as the original analysis for comparison purposes. Where the digitised isohyets differed from that determined manually, the spline function was adjusted until a satisfactory reproduction of the original was achieved.

Storm temporal distributions

The variation in rainfall depth with time as a proportion of total storm depth was determined. The data used for establishing the temporal distribution were checked for temporal and spatial consistency and stations with anomalous data discarded. Temporal distributions were determined by the drawing of parallelograms of standard-sized areas about the centre of the storm in such a way as to enclose the maximum number of high rainfall totals (Figure 2.2). The daily rainfall depths enclosed by each parallelogram were then averaged using a technique developed by Thiessen (1961). The maximum percentages of storm rainfall that fell within standard areas within standard durations (of 6 hours) were also determined.

Depth-area-duration analysis

Depth-area-duration curves were produced from the isohyets and the percentage depths in the storm temporal distribution (Figure 2.3). Storm rainfall depths were plotted for durations from 6 hours up to a maximum of 3-5 days.

Storm dew-point temperatures

Moisture content is represented by precipitable water, which was calculated from surface dew-point temperature. Storm dew-point temperature is therefore an identifying characteristic of the storm and was included in the storm catalogue.

Due to the sparseness of the radiosonde observational network, storm dew-point temperatures were estimated using surface observations and could only be determined to an accuracy of about 2°C.

Surface dew-point was obtained from Australian region MSL charts, National Climate Centre archives and observers' logbooks. The dew-point from a number of stations was averaged for representivity. Stations in the trajectory of moisture inflow to the storm or in the area of the storm peak, and that had recorded high dew-point temperatures persisting for 6 to 24 hours were deemed suitable.

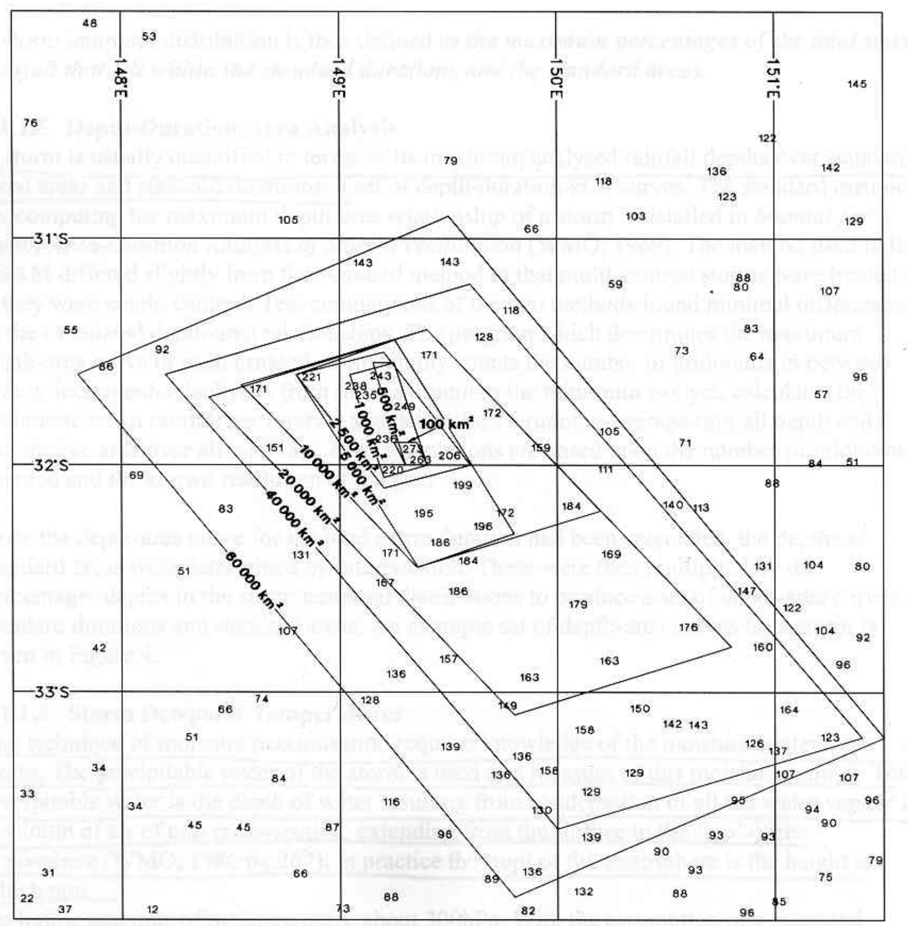

Figure 2.2 Parallelograms of standard areas about the storm centre (Bureau of Meteorology, 1996)

<u>Generalising the Storm Database</u>

This task involves identifying and removing the site-specific components of each storm, so that the storm could be transposed to other locations. The site-specific components of storm rainfall were identified as the following:

i. Storm type
ii. Storm spatial distribution
iii. Topographic influences
iv. Moisture content

<u>Storm type</u>

The storm type was removed from the database by dividing the study region into zones of different mechanisms of producing large rainfall, so that storms in a zonal database can occur anywhere within the zone (homogeneity). The GSAM region was divided into two zones, coastal and inland, which are separated by the Great Dividing Range (Figure 2.4).

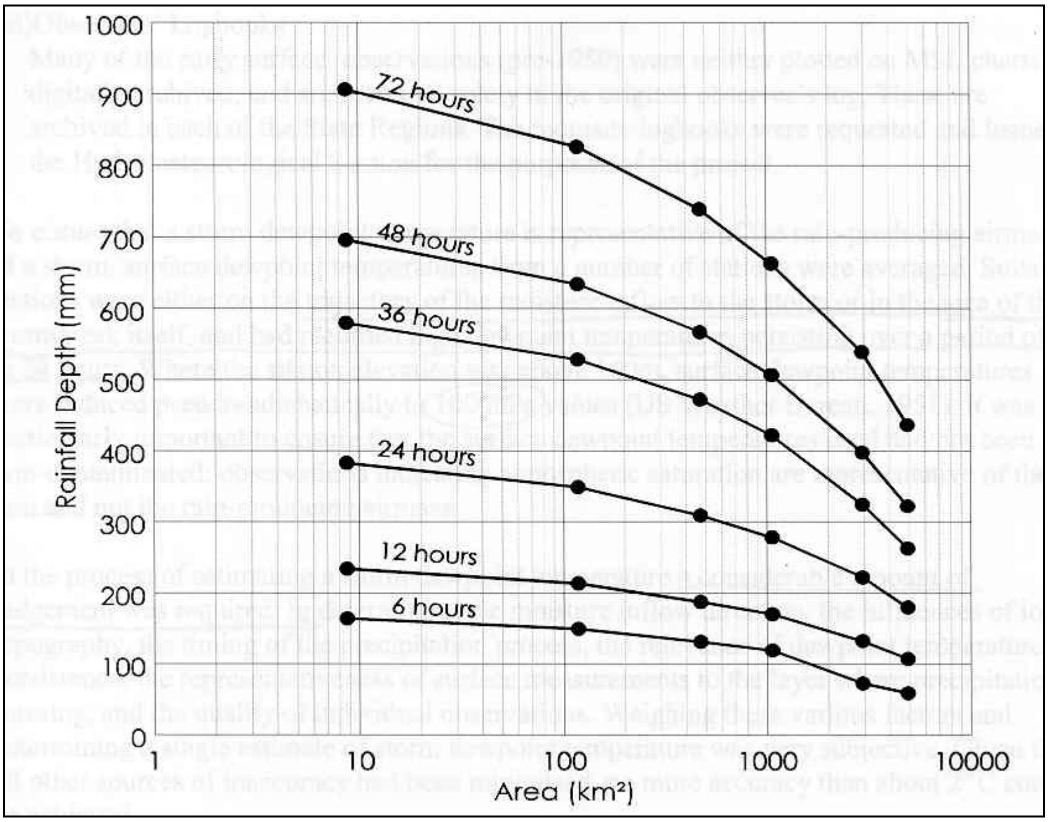

Figure 2.3 Example of depth-area curves at standard durations (Bureau of Meteorology, 1996)

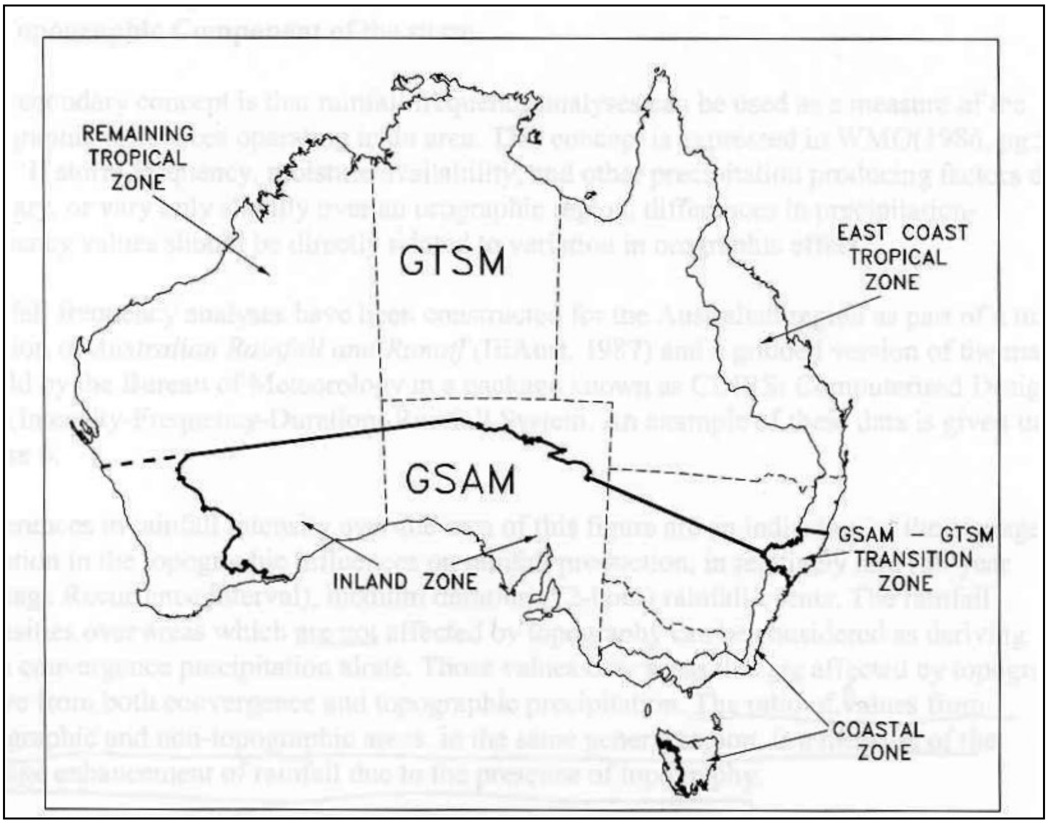

Figure 2.4 Division of the GSAM Region into zones (Bureau of Meteorology, 1996)

Spatial distribution

Specifics were removed from each storm when it was quantified in terms of a set of standardised depth-area-duration curves.

Topographic influences

For the purpose of the study, the portion of rainfall within a storm that is derived purely from atmospheric processes was termed 'convergence precipitation'. In the same way, the portion of rainfall that is attributed to topographic influences was termed 'topographic precipitation'. To determine the topographic influences of an area on storm rainfall, rainfall frequency analysis was used. This method was adopted from that expressed in WMO (1986). For the analysis, 50-year (mean of the range of average recurrence intervals for storms of the storm database) 72-hour (medium duration) rainfall events were used and gridded maps produced of rainfall intensities across Australia. Rainfall intensities read over non-orographic areas were considered to be derived from convergence precipitation alone. Rainfall intensities read over orographic areas were considered to be derived from both convergence and topographic precipitation. The average enhancement of rainfall due to the presence of topography (Topographic Enhancement Factor) could therefore be calculated by considering the ratio of rainfall intensity values from topographic and non-topographic area. To calculate the topographic enhancement factor, the following relationships have been used:

$$\frac{\text{Total Rainfall Intensity}}{\text{Conv. Rainfall Intensity}} \approx \frac{\text{Total Storm Depth}}{\text{Conv. Storm Depth}}$$

$$\text{Conv. Storm Depth} = \text{Total Storm Depth} \times \frac{\text{Conv. Rainfall Intensity}}{\text{Total Rainfall Intensity}}$$

$$\text{Topog. Storm Depth} = \text{Total Storm Depth} - \text{Conv. Storm Depth}$$

For the calculation of the convergence component of a storm, a map of the convergence component of the 50-year 72-hour rainfall intensities was constructed by pinpointing the locations where values were considered unaffected by topographic influences and manually interpolating between these points. The ratio of the total rainfall intensity field to its convergence component could then be determined point by point. The convergence component of each storm could therefore be calculated by dividing the total storm rainfall depth at each grid point by the latter ratio.

The GSAM method for separating the components of the storm was tested on a flat area and nearby area with marked topographic features. The test was successful.

Moisture content: Moisture maximisation and standardisation

The moisture content in a storm was measured by the total precipitable water content. The total precipitable water content refers to the depth of water resulting from the condensation of all the water vapour in a column of air of unit cross-section that extends from ground level to the 'top' of the atmosphere (WMO, 1996). The 'top' of the atmosphere refers to the height at which moisture content is negligible, and in this case was associated with an atmospheric pressure of 30 kPa. Assuming saturated pseudo-adiabatic conditions existed, the precipitable water content was calculated from surface dew-point temperatures that were reduced pseudo-adiabatically to a common level of 100 kPa. The storm precipitable values were then determined between this level and the top of the atmosphere.

A maximisation factor was calculated for the storm from the following equation:

$$MF = \frac{EPW_{insitu}}{SPW} \tag{2.2}$$

where EPWinsitu is the Extreme Precipitable Water associated with the storm extreme dew-point and SPW is the Storm Precipitable Water associated with the storm dew-point temperature.

The storm extreme dew-point temperature is the extreme 24-hour persisting dew-point temperature for the storm location and time of year. The location is the location of the storm peak and the time of year is the month which provided the maximum extreme dew-point temperature within ± 28 days of storm commencement.

An upper limit of 1.8 was chosen for the maximisation factor after investigations showed that higher factors were associated with storms where the assumption of an atmosphere saturated through its depth was invalid.

Moisture standardisation removes the site-specific feature of moisture content and is only valid for the convergence component of the storm. A standardisation factor is calculated as follows:

$$SF = \frac{EPW_{std}}{EPW_{insitu}} \tag{2.3}$$

where EPW_{std} is the Extreme Precipitable Water associated with the Standard Extreme Dew-point Temperature for the zone and EPW_{insitu} is the Extreme Precipitable Water associated with the Storm Extreme Dew-point Temperature.

Enveloping the depth-area-duration curves
An enveloping curve was drawn to the maximised, standardised convergence component depth-area curves. This represents the standard component of a PMP storm. Curves were plotted for eight durations for each of the four seasons in the two zones.

<u>PMP estimation technique</u>

To estimate the PMP of a catchment, the catchment-specific features of the PMP storm must be derived and combined with the standard convergence component of the PMP storm.

Catchment area and location
The standard convergence component of the PMP storm is the envelope depth at the area of the catchment.

The PMP rainfall is transposed from the standard hypothetical location by adjustment of the depth for the different moisture potentials of the two locations. The moisture adjustment factor (MAF) is calculated as follows:

$$MAF = \frac{EPW_{catchment}}{EPW_{std}} \tag{2.4}$$

where $EPW_{catchment}$ is the Extreme Precipitable Water associated with the catchment extreme dew-point temperature and EPW_{std} is the Extreme Precipitable Water associated with the standard extreme dew-point temperature.

The centroid of the catchment was taken as the catchment location and four seasonal extreme dew-point temperatures were determined for this latitude and longitude. The envelope depths for each seasonal group are multiplied by its corresponding moisture adjustment factor. The maximum of the four depths was taken as the catchment PMP convergence component.

Topographic Component

The topographic component of the PMP storm was estimated using the 72-hour 50-year rainfall intensity field. It was felt, however, that there would be less scope for topographic enhancement of rainfall in a PMP storm than there is in a storm with an average recurrence interval of about 50 years, as are the storms in the storm catalogue. The topographic enhancement factors (x) for PMP storms were therefore modified from those for the original storms in the following way:

Table 2.2 Topographic Enhancement Factors (Bureau of Meteorology, 1996)

Original	Modified
$x \leq 1.0$	$x = 1.0$
$1.0 < x < 1.5$	$x = x$
$1.5 < x < 2.5$	$x = 0.5x + 0.75$
$x > 2.5$	$x = 2.0$

Catchment PMP estimates

Finally, total PMP depths were calculated for each duration from the multiplication of the catchment PMP convergence components with the catchment PMP topographic enhancement factor. These depths were then plotted against duration and a final envelope curve drawn to these. Catchment PMP estimates are taken from the final envelope.

Generalised Australian short duration methodology for the estimation of PMP (small-area storms)

This methodology, as documented in the Bureau of Meteorology of Australia June 2003 report, is applicable for storms within Australia of durations up to 6 hours and areas up to 1000 km². Due to the lack of short duration rainfall data that has been recorded or documented in Australia, the depth-area-duration curves used in this method were derived mainly from United States data on the basis that the Australian rainfall potential was found to be similar to that of the States where the data base is much larger. One set of depth-area-duration curves was developed for durations of 1 hour or less where the PMP is considered unaffected by the type of terrain. For durations greater than one hour, two sets of curves were developed for 'smooth' and 'rough' terrain, the definitions of which are given below.

The procedure for the estimation of PMP, known as the 'Generalised Short Duration Method' (GSDM), is summarised below.

Selection of Terrain Category

The terrain of the study area can either be classified as 'rough' or 'smooth'. 'Rough' terrain is defined by the Bureau of Meteorology of Australia as catchments within which elevation changes of 50 m or more within horizontal distances of 400 m are common. If a catchment includes a flat area of land within 20 km of generally 'rough' terrain, the whole catchment is still defined as 'rough'. If there is a flat area of land further than 20 km from generally 'rough' terrain, an areally weighted factor of 'rough' (R) and 'smooth' (S) terrain should be calculated such that these factors equal one. If a catchment is difficult to classify, the catchment should be termed as 'rough'.

Adjustment for catchment elevation

For the calculation of the Elevation Adjustment Factor (EAF), the mean elevation of the catchment can be determined from a topographical map. If the latter value is less than 1500 m, the EAF is made equal to 1.

If the mean elevation is greater than 1500 m, the EAF should be reduced by 5% for every 300 m by which the mean elevation exceeds 1500 m.

Adjustment for Moisture

The moisture index used to estimate PMP is the precipitable water content, the value of which can be estimated from the surface dew-point temperature. The depth-area-duration curves used in this method have been standardised to a moisture index corresponding to a surface dew-point temperature of 28°C (Table 2.5). A percentage adjustment (Moisture Adjustment Factor) is required to reflect the potential moisture availability for a specific location. These percentages can be read from a map (Figure 2.6).

Estimation of the PMP

The PMP can be estimated from the following equation:

$$PMP = \left(S.D_S + R.D_R\right) * MAF * AEF \qquad (2.5)$$

where S and R refer to the areally weighted factors representing 'smooth' and 'rough' terrain respectively, and D_S and D_R refer to the initial rainfall depth determined from the depth-area-duration curves for 'smooth' and 'rough' terrain, respectively.

Australian Methodology for Estimated Probability of Exceedance of the PMP

The PMP is representative of the upper limit of rainfall and therefore, in theory, should be associated with a zero probability of exceedence. However, estimates made by the various PMP methods have a probability of exceedence which doesn't equal zero, thus a probability of exceedence can in fact be assigned to the PMP.

Based on the Australian methodology for the estimation of PMP, Kennedy and Hart (1984) provided notional estimates of the annual exceedence probability (AEP) of the PMP in which they took meteorological considerations into account in terms of the catchment area and the method used to estimate the PMP. Table 2.3 shows their findings.

Table 2.3 Exceedence Probability for Various Types of PMP Estimates (Kennedy and Hart, 1984)

Method of Calculation	Effective Transposition Area (km^2)	Annual Exceedence Probability			
		Catchment Area (km^2)			
		100	1000	10 000	100 000
1. Maximisation in-situ	n/a	10^{-3}	10^{-3}	10^{-3}	10^{-3}
2. Maximisation and transposition	100 000	10^{-6}	10^{-5}	10^{-4}	10^{-3}
3. Generalised method	1 000 000	10^{-7}	10^{-6}	10^{-5}	10^{-4}
4. Adjusted US data (up to 6 hours)	10 000 000	10^{-8}	10^{-7}	n/a	n/a

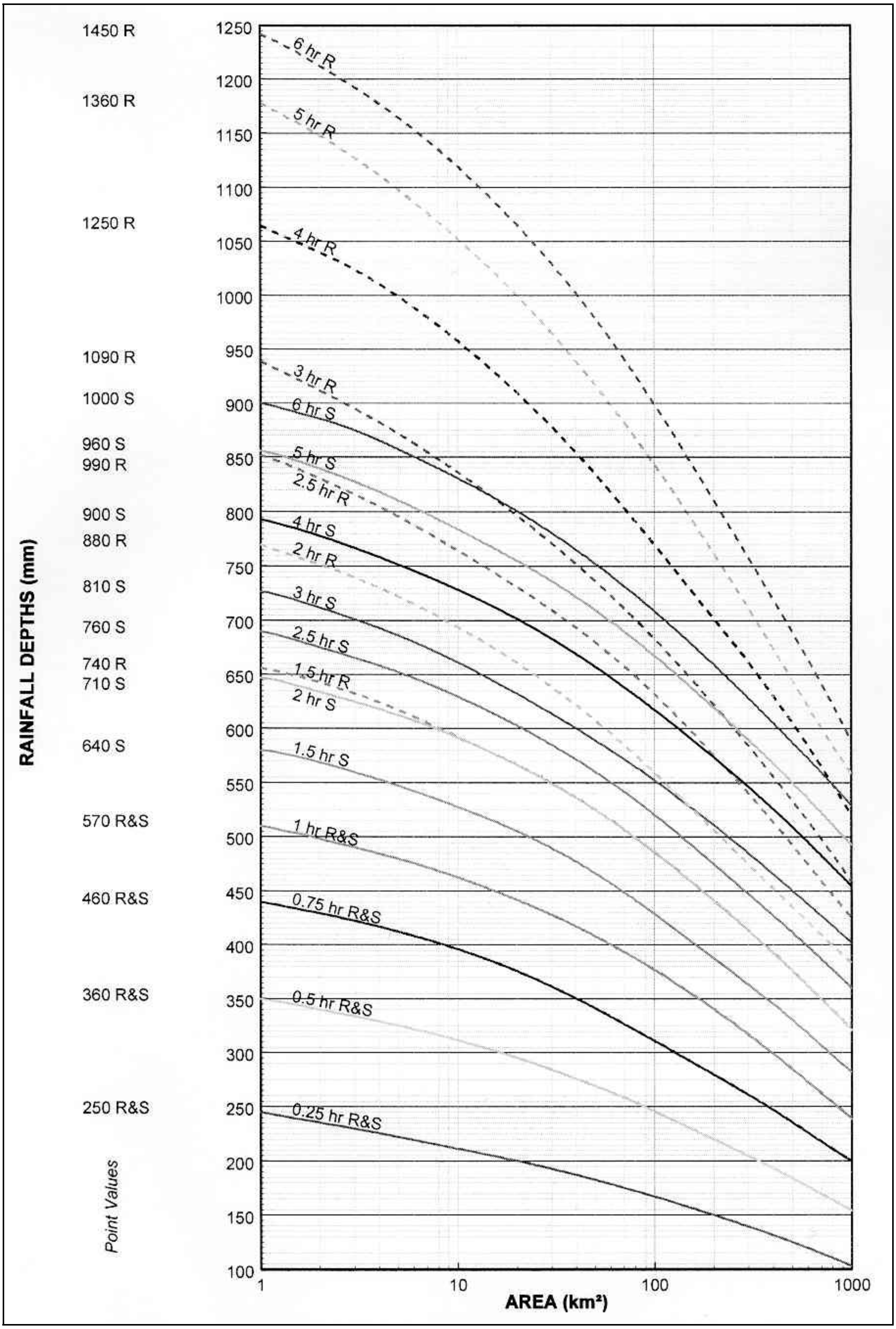

Figure 2.5 Depth-area curves for short duration rainfall (Bureau of Meteorology, 2003)

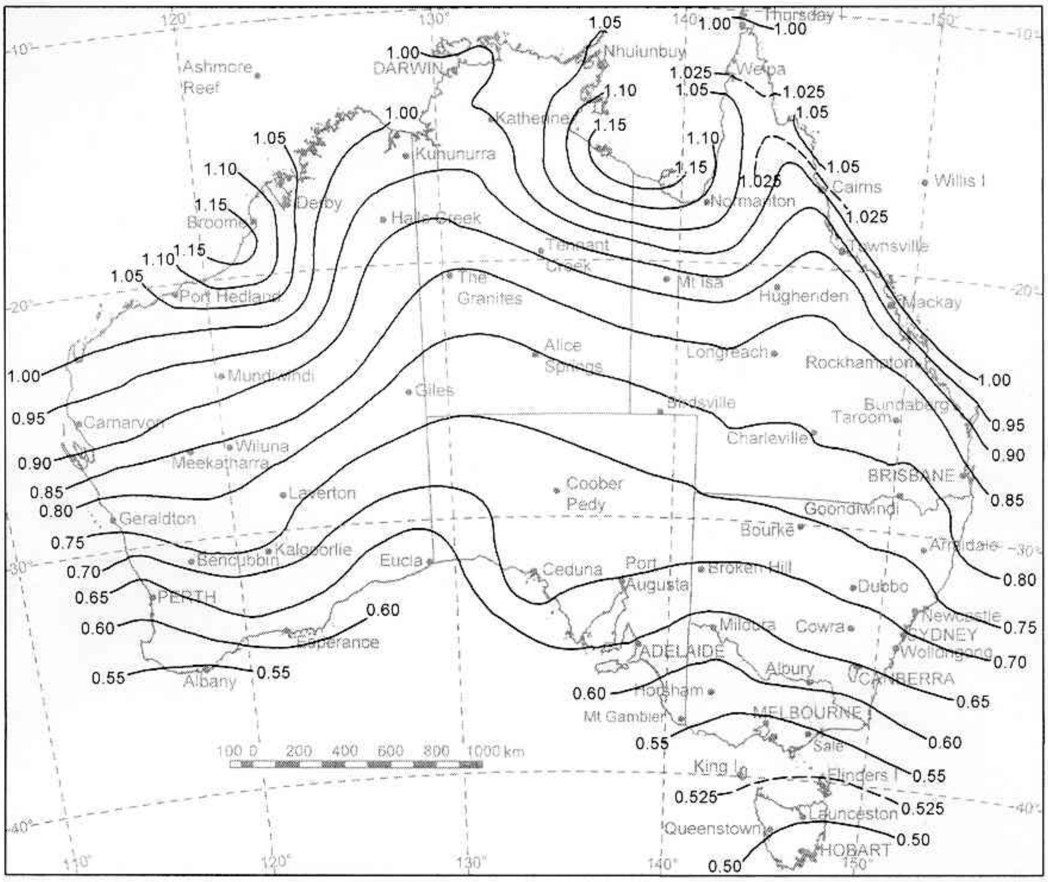

Figure 2.6 Map of moisture adjustment factors (Bureau of Meteorology, 2003)

Pearce (1994), however, disagreed that the AEP of the PMP should be based on catchment size and recommended that an AEP of 10^{-6} be adopted for all PMP estimates instead. Laurenson and Kuczera (1999) undertook an extensive study in which they set out to review all current work on the estimation of the AEP of the PMP in both Australia and internationally and found that the method as proposed by Kennedy and Hart (1984) was in fact the most appropriate. Laurenson and Kuczera (1999) also proposed estimates of the AEP of the PMP, which were modifications of the figures of Kennedy and Hart (1984). They recommended that for areas of 100 km^2 and less an AEP of 10^{-7} should be used, increasing to 10^{-6} for an area of 1000 km^2. Laurenson and Kuczera (1999), however, stated that their proposals should be viewed as interim, pending the findings of future studies, and assigned an uncertainty of two orders of magnitude to their recommended probabilities.

2.2.2 UK PMP estimation methodology

The Flood Studies Report (FSR), published in 1975, was produced for the United Kingdom with the aim of providing the first comprehensive guide to flood estimation techniques. The method for the estimation of the PMP employed in the FSR involves the examination and maximisation of observed storms across the UK for durations of 2 hours and 24 hours. This methodology is explained briefly below.

For the 2-hour storms, the maximum dew-point temperature persisting for at least 6 hours was derived for each month of record for 60 stations. From these records, the precipitable water content corresponding to the 5-year return period dew-point was calculated and mapped over the UK (Figure 2.7).

It was assumed that maximum 2-hour rainfall would follow the same pattern, being based on the same convective mechanisms and hence the same maximum storm efficiency. The storm efficiency is defined in the FSR as the ratio of rainfall to the total precipitable water in the representative air column during the storm. For the major 2-hour storms, the maximum storm efficiency was found to be 3.86; and this value was taken as the probable maximum for all regions in the UK. The multiplication of the maximum storm efficiency with each of the regional maximum values of precipitable water content gave an estimate of the maximum storm rainfall for each region. These estimates were correlated with the 5-year return period value of precipitable water and the estimated maximised 2-hour rainfall thus mapped over the UK.

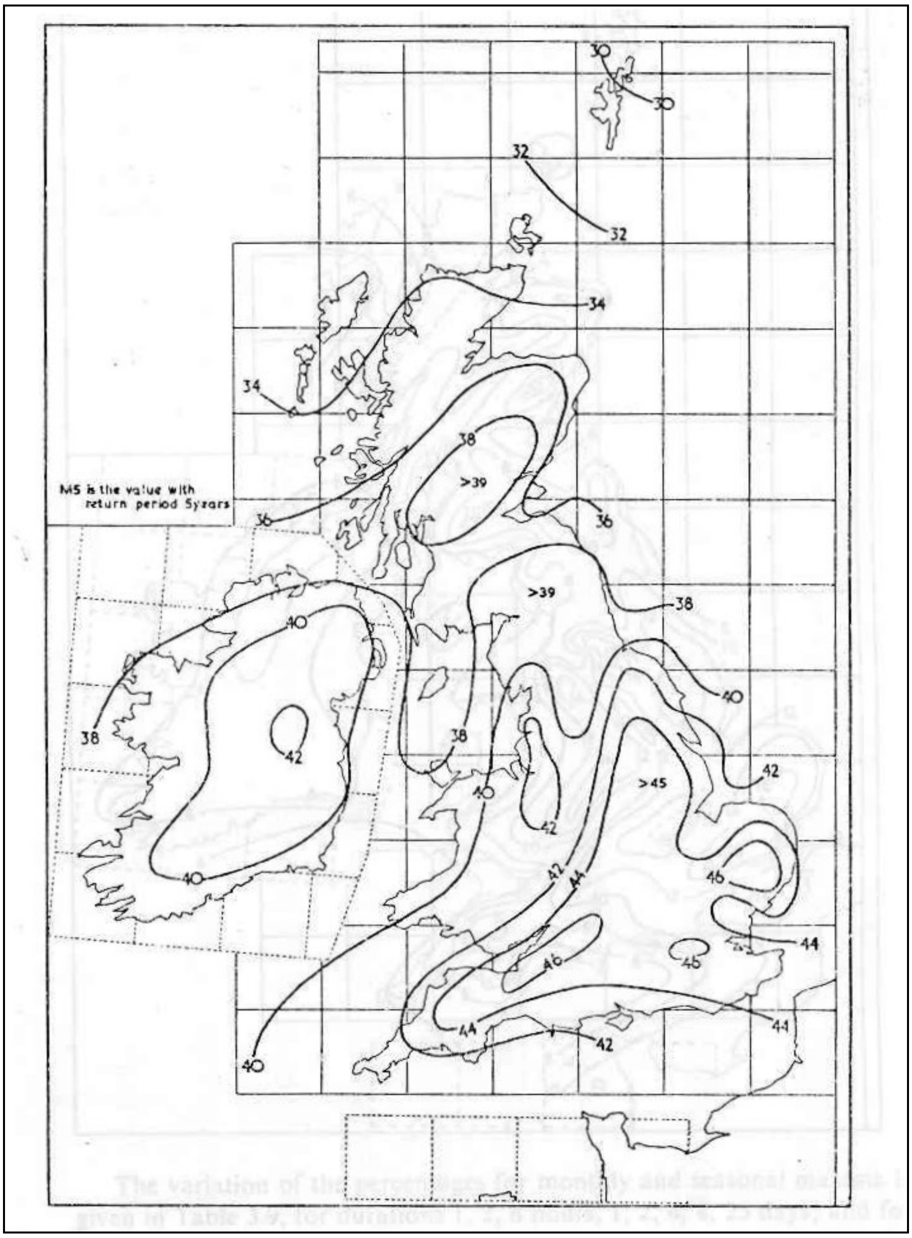

Figure 2.7 UK Map of Precipitable Water (mm) corresponding to the 5-year Return Period Dew-point Temperature (FSR, 1975)

A similar maximisation procedure was followed for the estimation of the maximum 24-hour rainfall. The maximum storm efficiencies were derived for 24-hour storms in the summer (9.3) and winter (2.2) and these values were taken as the probable maximum. These storm efficiencies were used to adjust the rainfall of the major 24-hour storms across the UK to determine the maximum 24-hour rainfall for each region.

An envelope of growth factors for all durations is presented in the Flood Studies Report (Figure 2.8) as part of the rainfall annual probability analysis. This allows for an initial quick estimation of the PMP on the basis of the 5-year return period rainfall depth for durations from 24 hours to 25 days. In Figure 2.8, 'MT' refers to the 1 in T-year rainfall depth (mm), which is representative of the maximum rainfall depth when considering enveloped rainfall values, and 'M5' refers to the 1 in 5 year rainfall depth (mm).

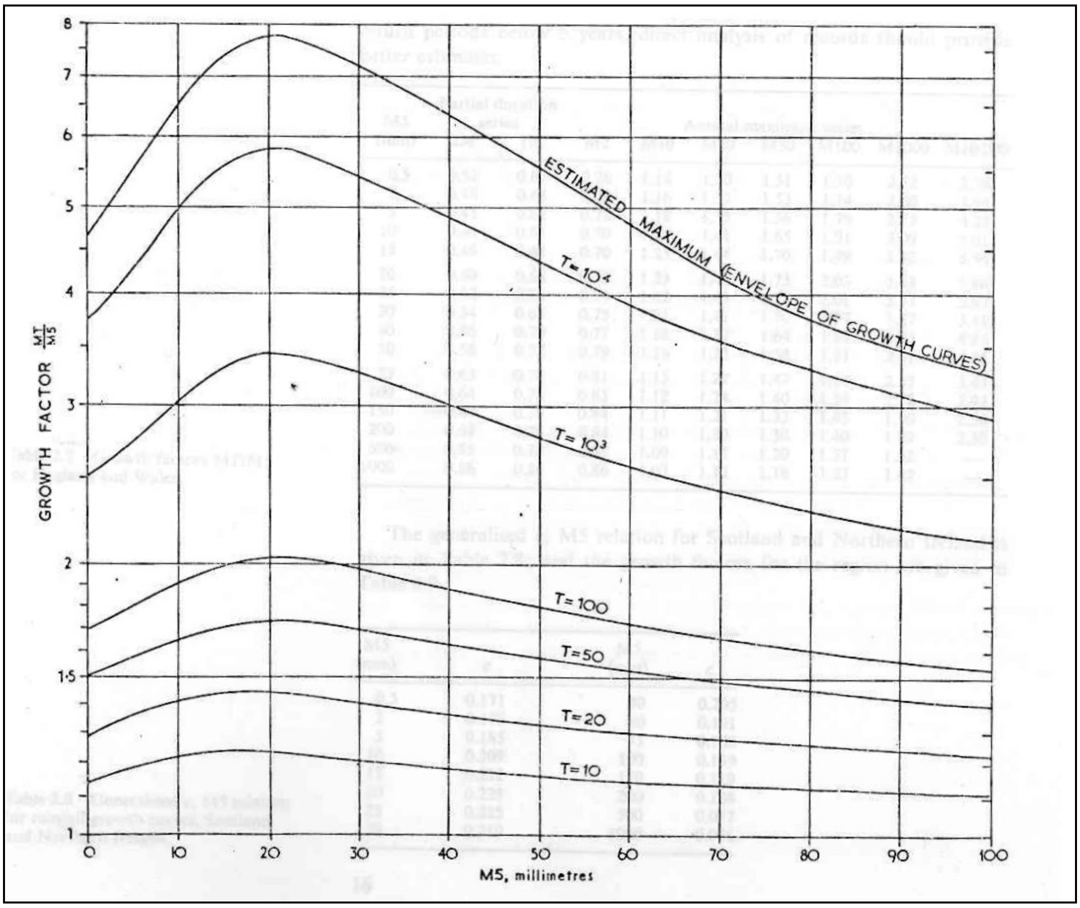

Figure 2.8 Growth Factors for Rainfall over England and Wales (FSR, 1975)

Using the envelope of growth factors, a table was generated relating estimated maximum rainfall for all durations to average annual rainfall and the 2-day 5-year return period rainfall. It was found that a good agreement existed between the rough 24-hour estimates in the table and the maximised major storms. This led to the mapping of the maximum 24-hour storm rainfall based on the map for the 2-day 5-year return period rainfall.

Values of maximum rainfall for durations 2 and 24 hours are determined by linear interpolation on a diagram of rainfall versus the logarithm of the duration. Values for durations shorter than 2 hours or longer than 24 hours can be estimated from factors related to the average annual rainfall.

The Flood Estimation Handbook (FEH), published in 1999, is the product of a five year research programme aimed at developing a new methodology for rainfall and flood estimation for the UK. The FEH is designed to estimate rainfalls up to a return period of 2000 years, but indicates that this information could be extrapolated to provide rainfall estimations up to 10 000 years. The application of the FEH has, however, caused concern among hydrologists, as it has been found that in some cases the FEH derived 1:10 000 year return period rainfall is in excess of the FSR PMP. The UK Department of Environment, Food and Rural Affairs (DEFRA) therefore commissioned a new study to investigate the latter irregularity.

2.2.3 Storm model approach

A physically based 'storm model' was formulated in the UK as a new approach to the estimation of the PMP based upon the thermodynamics of the ascent of a single parcel of air (Collier and Hardakar, 1996). In the maximisation procedure employed by the model, the maximum surface dew-point temperature is calculated based on solar heating, orographic uplift and meso-scale convergence. This is seen as an advantage of the model, as measured surface dew-point temperatures are often not available for many study areas.

The maximised surface dew-point temperature is then used to determine the maximum precipitable water content (values were read off from existing tables), which is presented at various grid points across the selected catchment. When combining the values of precipitable water (*PW*) with the storm efficiency (*E*), estimates of the PMP can be calculated at each time step ($PMP_{ij} = PW_{ij} \times E$). The storm efficiency in this case is the ratio of the total rainfall at ground level, which can estimated from radar measurements if available, to the total cloud water condensed, which can be evaluated from radiosonde data assuming that the air mass is saturated.

To determine a single PMP estimate for the whole catchment, the mean value for the grid of PMP values is simply calculated at any one particular time interval.

The storm model developed was tested against the FRS PMP. It was found that the model was able to produce the FRS PMP for storm durations from 2 to 11 hours, but for storm durations between 11 and 24 hours, the storm model PMP values that were derived exceeded the FSR values. It was however noted that the storm model PMP values were derived from mesoscale convective systems, which were not recognised in the UK at the time the FSR methodology was developed in the early 1970s. Mesoscale convective systems are organised collections of thunderstorms that last for hours (storm durations of 10 hours are not uncommon) and stretch over hundreds of kilometres. Although these storms are relatively rare, with only one or two occurring over the UK each year, they have been recognised as potential mechanisms for the generation of extreme flood events (DEFRA, 2002). Although further studies of these systems were recommended following the development of the FSR methodology, not much information has been made available (DEFRA, 2002). There are, in fact, no known hydrological studies in the UK that examine mesoscale convective systems with respect to flood generation.

2.2.4 Statistical approach

Statistical estimates of PMP may be used when sufficient precipitation data is available. The statistical procedure is useful for quick estimates of PMP when there is insufficient meteorological data available. It also takes considerably less time to apply than meteorological methods. An added advantage of statistical estimates is that they do not require a meteorologist. The method produces only point estimates of PMP and therefore requires area-reduction curves.

The WMO (1986) describes the following procedure, which was developed and later modified by Hershfield (Hershfield, 1961), as being the most widely accepted. It is usually applied to areas up to 1000 km^2, but has been used for much larger areas. It is also only applicable for storm durations up to and including 24 hours. The procedure is based on the following general frequency equation:

$$U_t = \overline{U}_n + KS_n \qquad (2.6)$$

where U_t is the rainfall at return period t, $\bar{U}_n$ and S_n are the mean and standard deviation of a series of *n* annual maxima respectively and K is a statistical variable which varies with the statistical distribution used. For the calculation of the PMP the above equation is modified by the substitution of U_t by U_m, the maximum observed rainfall (PMP), and K by K_m. The WMO manual includes a relationship between mean annual maximum rainfall (for durations of 5 minutes, 1, 6 and 24 hours) and K_m, determined from rainfall records from numerous rainfall stations, mostly in the USA. This relationship is shown in Figure 2.9, with K_m along the y-axis and the mean annual maximum rainfall along the x-axis.

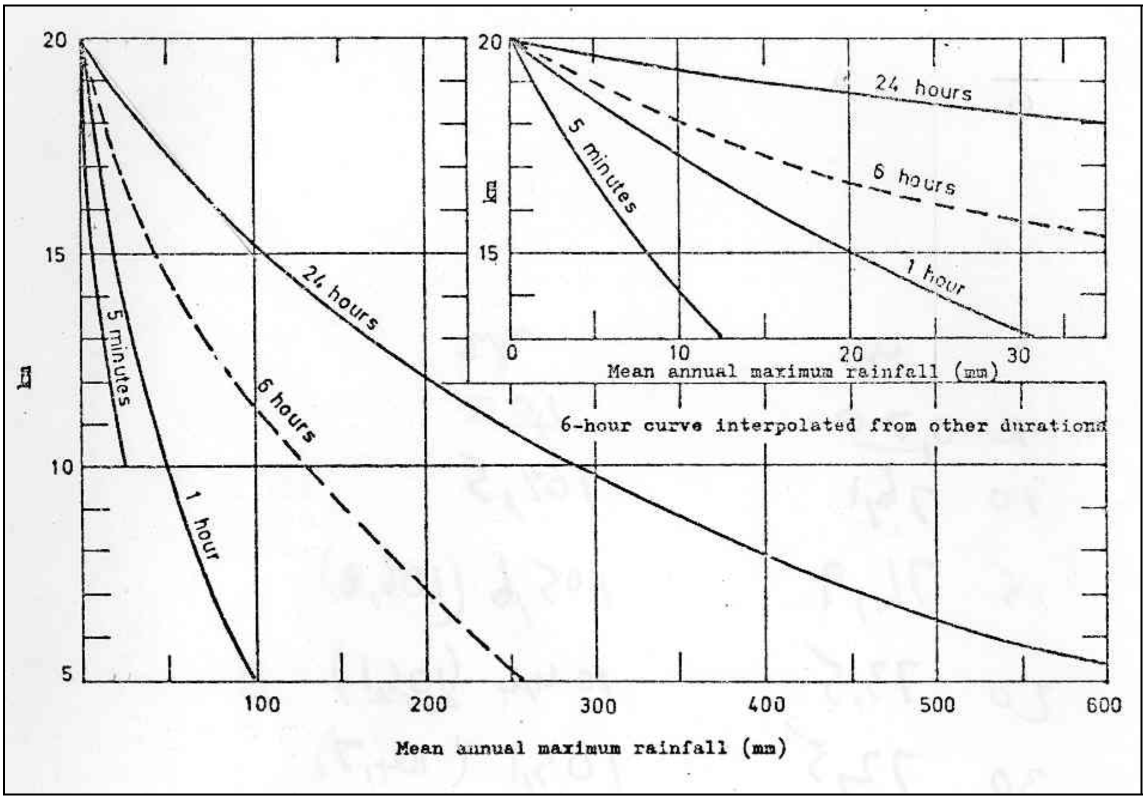

Figure 2.9 K_m as a function of rainfall duration and mean of annual series (WMO, 1973)

The statistical procedure for the estimation of the PMP is summarised as follows:

1) Series of annual maxima are obtained from the rainfall record, each series representing rainfall amounts for different time intervals. For example, series of 1-hour, 6-hour and 24-hour rainfall amounts could be formed from data observed either at hourly time intervals or at the respective fixed time intervals.

2) The mean and standard deviation of each data series are calculated and adjusted to allow for various traits of the data record. This includes the adjustment for maximum observed rainfall events, known as outliers, which may have a considerable effect on the mean and standard deviation of the rainfall series. Figure 2.10 and Figure 2.11 show this adjustment for the mean and standard deviation respectively, where $\overline{X}_{n-m}$ and $\overline{S}_{n-m}$ represent the mean and standard deviation of the rainfall series after the exclusion of the maximum observed rainfall on record.

Another adjustment made to the mean and standard deviation of the rainfall data series is for length of record (Figure 2.12).

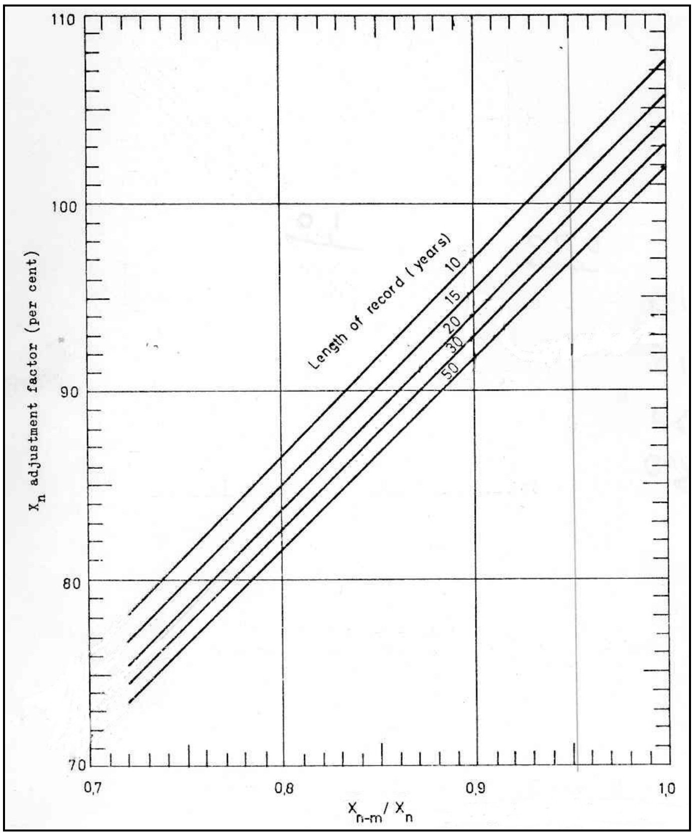

Figure 2.10 Adjustment of mean of annual series for maximum observed rainfall (WMO, 1973)

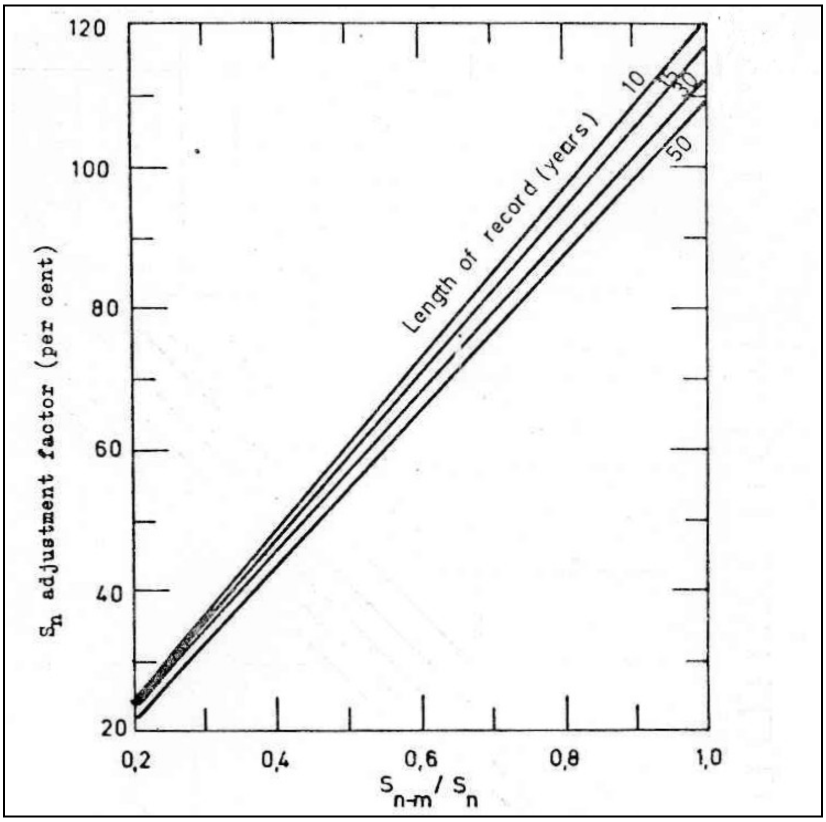

Figure 2.11 Adjustment of standard deviation of annual series for maximum observed rainfall (WMO, 1973)

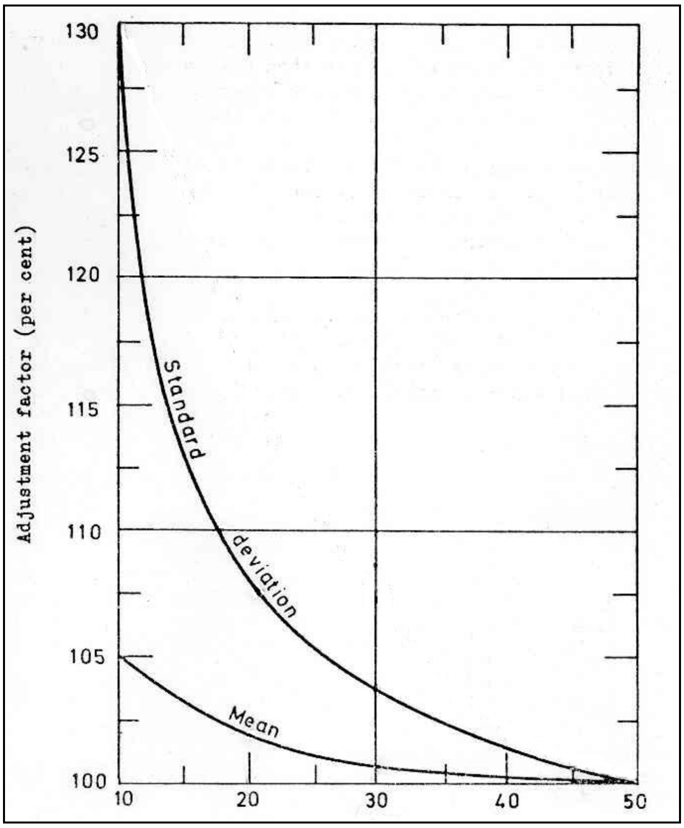

Figure 2.12 Adjustment of mean and standard deviation of annual series for length of record (WMO, 1973)

3) K_m is obtained from Figure 2.9.

4) The PMP is computed using the maximised general frequency equation, in other words,

$$U_m = \overline{U}_n + K_m S_n \tag{2.7}$$

5) The calculated PMP is then adjusted for the type of rainfall data used. If the data has been compiled from fixed observational time intervals, U_m is increased by applying the factor 1.3. If hourly rainfall data is used, U_m is increased by applying factors 1.13, 1.02 and 1.01 to 1-, 6-, and 24-hour amounts respectively.

6) U_m is a point rainfall, therefore an areal reduction factor can be applied to the PMP to determine the average maximum rainfall over a particular area. Area-reduction curves should be developed for the subject area, since each area varies according to rainfall type and geographical features.

For generalised estimates of PMP, in other words, where precipitation networks are available, a grid of PMP values can be constructed using the following equation:

$$U_m = \overline{U}_n (1 + K_m C_v) \tag{2.8}$$

In the above equation, C_v refers to the coefficient of variation and is calculated by the division of the adjusted standard deviation by the adjusted mean (see Point (2) above), in other words, S_n/X_n.

For each rainfall station, X_n and C_v can be calculated and plotted on a map and two sets of isolines drawn. A map of PMP values can therefore be obtained by the interpolation of X_n and C_v from their respective isolines at selected grid points.

It is advisable when selecting a rainfall station for the estimation of the PMP that the rainfall mean, standard deviation and coefficient of variation be determined and compared with nearby stations. In this way, the quality of the data can be checked and the data discarded if necessary. Also it is recommended that rainfall records of no less than 20 years be used for statistical analysis, and that records of less than 10 years should not be used at all.

It is stated in the WMO manual that PMP values obtained by the above Hershfield procedure tend to give lower PMP values than those obtained from the more complex meteorological methods, such as those described in Chapter 2.2.

2.2.5 Recent developments and alternative approaches

A new approach to the estimation of PMP has been through the use of radar-derived distributed rainfall measurements, which provides improved spatial and temporal information on the pattern of rainfall during extreme storm events. Cluckie and Pessoa (1990) of the UK coupled radar-derived rainfall data with storm maximisation and transposition procedure to determine estimates of the PMP. In their analysis, the RADMAX programme was devised in order to convert the radar-derived extreme rainfall data into maximised transposed storms through three steps: initial time maximisation, tracking maximisation and moisture maximisation.

Cluckie and Pessoa (1990) stated, when concluding their study, that radar calibration is a crucial element when using archived radar data. They commented that any attempt to obtain from rain gauges the spatial resolution provided by radars would not be economically viable. It was concluded that archive radar-derived rainfall data may support the PMP/PMF procedure, providing the off-line calibration of the data is satisfactory (Cluckie and Pessoa, 1990).

Another alternative approach for the estimation of PMP is through the use of multifractal analysis techniques (Barros and Douglas, 2002). Barros and Douglas (2002) applied multifractal analysis techniques to estimate extreme precipitation events from observations in the eastern United States. They referred to these estimates as the Fractal Maximum Precipitation (FMP), which represented maximum events empirically derived using the scaling behaviour of the observations. The method yields parameters for estimating both the magnitude and risk of extreme events. When comparing multifractal estimates of the 1:1 000 000 year return period rainfall to the US National Weather Service PMP value, the multifractal estimates were found to be greater. It was therefore concluded that multifractal estimates of extreme events should be viewed as an upper bound of known risk to the standard NWS PMP (Barros and Douglas, 2002).

2.2.6 South African approach to the estimation of PMP

The general South African approach for the estimation of PMP for Large-Area storms involves the method of storm maximisation and transposition. In 1964, a report was published by Gibb Hawkins & Partners for the estimation of the PMP for the Gariep and Vanderkloof Dam sub-catchments along the Orange River. Here they recognised that the moisture content of the air over the catchment study area was directly related to the moisture content of the air at the source region, which in this case was shown to be over Madagascar. For the storm maximisation procedure, surface sea temperatures around Madagascar were used together with wet bulb potential temperatures (there is a close relationship between wet bulb potential temperature and dew-point temperature) over selected radiosonde stations. The extreme surface sea temperature was calculated from the mean plus 4 standard deviations (M + 4S). The extreme surface wet bulb temperature was found to be related to the extreme sea temperature in that it was approximately 5°C lower. Using these values and taking the wind factor into account, selected storms were maximised and the PMP estimated by averaging the maximised rainfalls from the individual storms. Only the most intense storms were selected for which radiosonde data was available.

In 1969, the Hydrological Research Unit of the University of the Witwatersrand published a report (HRU 1/69) describing procedures for the calculation of design storms in South Africa, and included a detailed methodology for the calculation of the PMP. These in-depth procedures were then related to simplified guidelines which were published in a report in 1972 (HRU 1/72). Up to this day, the HRU 1/72 report provides the only established procedure for the estimation of the PMP for Large- and Small Area Storms in South Africa. Large- and Small Area Storms are analysed separately, not only due to differences in the internal mechanisms of the storms, but also due to differences in the spatial distribution of daily-observed rain gauges required by the former and autographic rain gauges required by the latter. In the case of Large-Area Storms, the estimation of PMP is based on storm maximisation and transposition, and PMP versus area curves for different durations are presented for identified sub-regions in the country. For Small-Area Storms, the PMP can be estimated from empirically derived curves that envelope the highest observed point precipitations of various durations observed in different parts of the country. Both procedures are described in detail below.

HRU (1/72) Approach for the estimation of Probable Maximum Precipitation for Large-Area Storms (> 5000 km^2)

Large-Area Storms are associated with widespread rainfall over a long duration. For the analysis of Large-Area Storms in South Africa, the country was divided into 29 meteorologically similar sub-regions (Figure D1 of HRU 1/72). For each region, the PMP can be determined from maximised curves of PMP versus area for different durations starting from a duration of 1 day to a maximum of 6 days (Figures D2 to D28 of HRU 1/72). The isohyetal pattern of the most severe storm experienced in each region is also given. This can be used as a guide when selecting a suitable shape and orientation of the design storm.

The procedure followed for the derivation of the PMP Depth-Duration-Area curves is documented in detail in the HRU 1/69 report, and is presented step-by-step below:

<u>Depth-Area-Duration Analysis</u>

Storm Selection
Daily-observed rainfall records for rain gauge stations across South Africa were obtained from the Weather Bureau of South Africa. The country was sub-divided by the Weather Bureau into 'sections' and the rain gauges numbered according to the section in which they lie. For this study, one station was selected from each section based on the reliability of the rainfall record. Records were only examined from 1932. In other words, rainfall records of only about 30 years were considered. From each rainfall station, the 12 highest rainfall events were scrutinised and the dates and approximate localities of the most severe storms identified. In all, 170 storms were selected across the country.

Isohyetal Patterns of Storm Rainfall
Because of the irregular distribution of rain gauge stations within the area of influence of the storm, it was not possible to draw isohyetal maps from individual storms with reasonable confidence. Instead, the 'isopercental' procedure was adopted, which is based on the hypothesis that the topographic features of the study area affect the areal distribution of rainfall depths during a storm in much the same way as the Mean Annual Precipitation (MAP) would be affected. The MAP isohyetal pattern can therefore be extended to indicate the spatial distribution of the storm rainfall. The general isopercental procedure adopted for the determination of the storm isohyetal pattern is summarised below:

i. The daily observed rainfall totals from the selected rain gauges are converted into percentages of the average MAP determined from the 30 years of observed data.
ii. The percentages are plotted on charts and isopercental lines drawn by interpolation.
iii. Percentage values of rainfall are then determined for defined grid points and converted to equivalent precipitation values using spot estimates of the MAP.
iv. The observed storm rainfall values and those that are calculated are then plotted on a map and the isohyetal patterns drawn.

For the HRU 1/69 study, the isopercental lines were determined by fitting a high-degree polynomial to the observed percental values at the gauging stations, in which the dependent variable (percental value) is expressed in terms of latitude and longitude. A computer program was used to fit the mathematical function to the percental values using the method of least squares. A pilot study indicated that a 6^{th} degree polynomial having one dependent and two independent variables could represent the isopercental surface of storms with reasonable accuracy. It was, however, found necessary to divide large storms into areas not exceeding 128,000 km^2.

The computed isopercental lines were visually compared with those drawn through interpolation by eye between observed values. These were found to be strikingly similar.

Conversion of isohyetal patterns to depth-area-duration graphs
The same computer program used to convert the isopercental values to isohyetal values for a selected latitude/longitude resolution was also used to determine the depth-area relationship through numerical integration. To determine the time distribution of the storm precipitation, the average mass curves at 1-day time intervals were derived. These mass curves were then used to proportion the depths of precipitation of the storm, selecting first the maximum 1-day precipitation, then the maximum 2-day precipitation and so forth up to the total storm precipitation. Through the latter processes, the depth-area-duration curves could be determined.

<u>Regional Sub-division</u>

The limit of storm transposition was taken into account when determining the regional sub-division of the country. It was decided that the boundaries of the regions would be defined according to orographic features (main mountain ranges and escarpments) and ranges of MAP, namely, 0-250, 250-500, 500-1000, 1000+ mm.

The country was divided into a total of 29 sub-regions (Figure 2.13) and the depth-area-duration curve derived for each region. It is assumed that for a given sub-region, the depth-area-duration curve would be applicable for any location within that region.

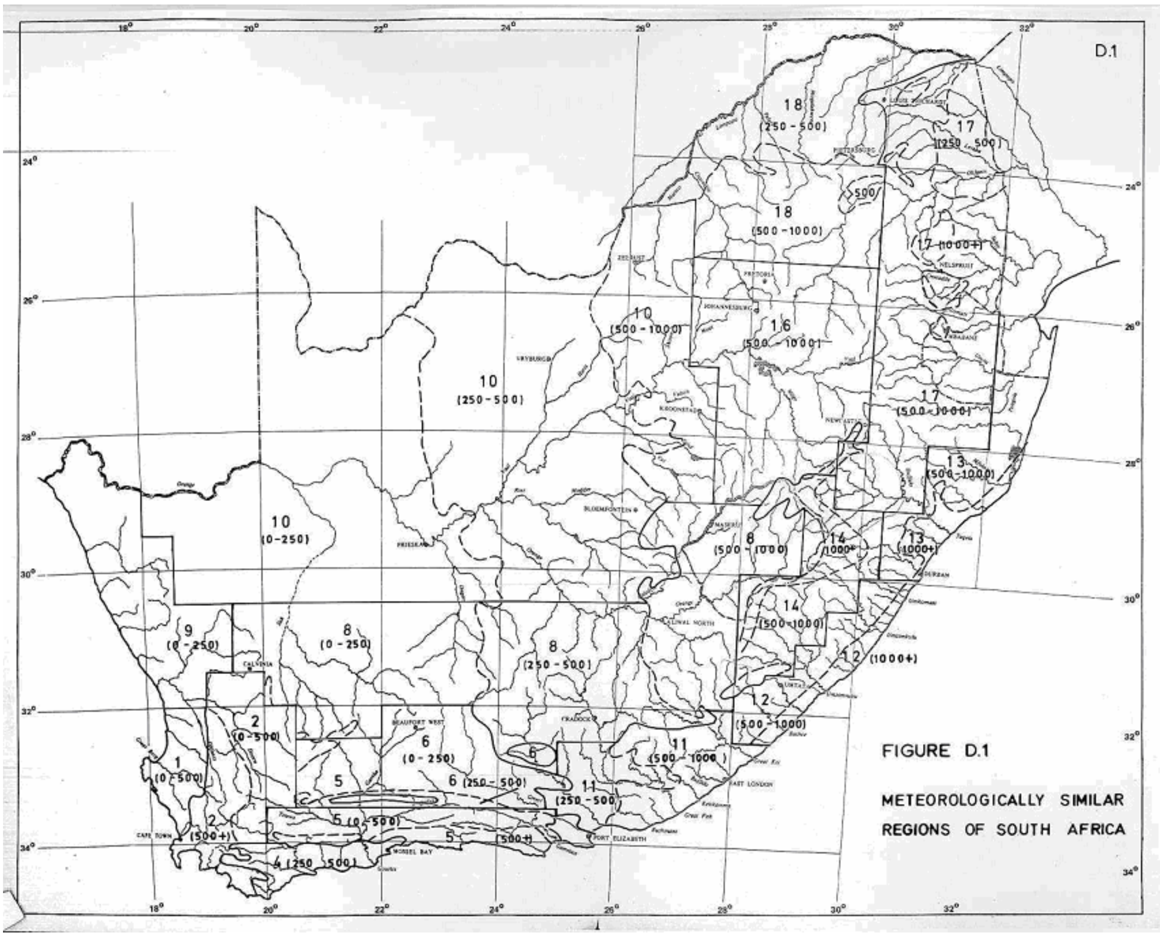

Figure 2.13 Meteorologically similar regions of South Africa (Large-Area Storms) (HRU 1/72)

<u>Storm Maximisation</u>

The approach adopted in the HRU 1/69 report for the estimation of PMP is based on the assumption that near-maximum precipitation efficiency would most probably have been attained during at least one of the storms analysed within each defined sub-region. Therefore, by maximising the moisture content that prevailed during all storms analysed, the PMP can be indicated by the envelope of the resulting maximised depth-area curves.

Calculation of moisture content of the atmosphere from upper air soundings was not possible, given the small availability of such readings. Therefore, procedures for calculating precipitable water content of a column of air from readings of surface temperature and pressure were developed. The precipitable water content refers to the total water content in a column of air of unit cross-section with its base at ground level and top at an altitude where moisture content is considered negligible; in other words, at an atmospheric pressure of 20 kPa.

Atmospheric moisture was calculated for a range of surface temperatures and pressures and plotted on a graph of moisture content, temperature and pressure (Figure 2.14). This graph showed that the moisture difference or the range of pressure experienced at recording stations is small (2%), therefore, pressure can be fixed at standard geopotential pressure and moisture content can be calculated from surface dew-point readings.

The ratio of storm moisture content to maximum moisture content for each region at the relevant time of the year was adopted as a basis for maximisation. For each sub-region, the maximum daily surface dew-point temperature was determined for a number of severe storms and the maximum storm moisture content read from Figure 2.14. The moisture contents were shown to decrease for every day of the storm. The moisture content decay pattern for each storm was determined by expressing the maximum 1-day, 2-day to 6-day moisture contents as proportions of the 1-day maximum. A typical decay pattern was then determined for each sub-region by averaging. The maximum dew-points recorded during the corresponding or adjacent months of the storms were also determined and the corresponding maximum moisture content read from Figure 2.14.

According to the before-mentioned ratio of the storm moisture content to the maximum moisture content, the 1-day precipitation depths for each severe storm were increased to the maximised value and the depth-area curves adjusted upward. The 2-day to 6-day precipitation depths were also maximised according to the following equation:

$$X_i = PR_i \left(\frac{M_x}{M_c} \right) T_i \tag{2.9}$$

where X is the maximised precipitation depth,

P = the 6-day precipitation depth,
R = the ratio of the i-th precipitation depth to the 6-day precipitation depth,
M_x = the maximum moisture content at the relevant time of year,
M_c = the average 1-day maximum moisture content, and
T_i = the ratio of the i-th day to the 1-day moisture content

PMP Estimation

The maximised depth-area curves for each duration within each sub-region were plotted on a single sheet and an upper envelope to the depths drawn. This envelope of depths was assumed to be representative of the PMP for the sub-region. PMP-area-duration curves were produced for each sub-region and presented for use in the HRU 1/72 report (Figures D2-D28). Figure 2.15 shows an example of such a curve.

Error in Maximised Depth-Area-Duration Curves

According to the HRU, error is mainly attributable to the inability to take account of possible atmospheric inversion. Because of the inaccuracies in estimation of moisture content, the PMP-area curves as drawn for each sub-region might be subject to error of the order of 25%.

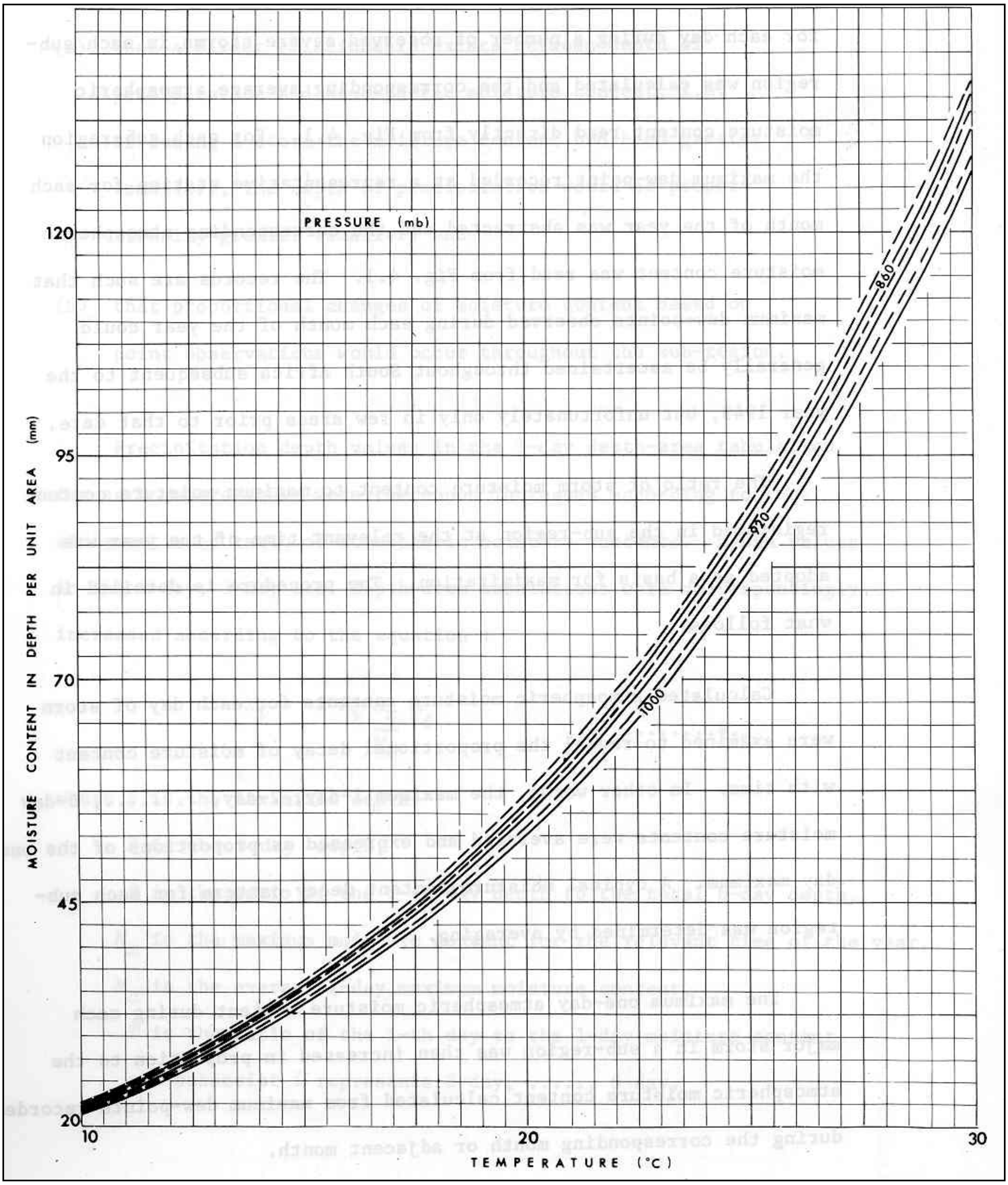

Figure 2.14 Moisture content as a function of pressure and temperature when the atmosphere is saturated (HRU 1/69)

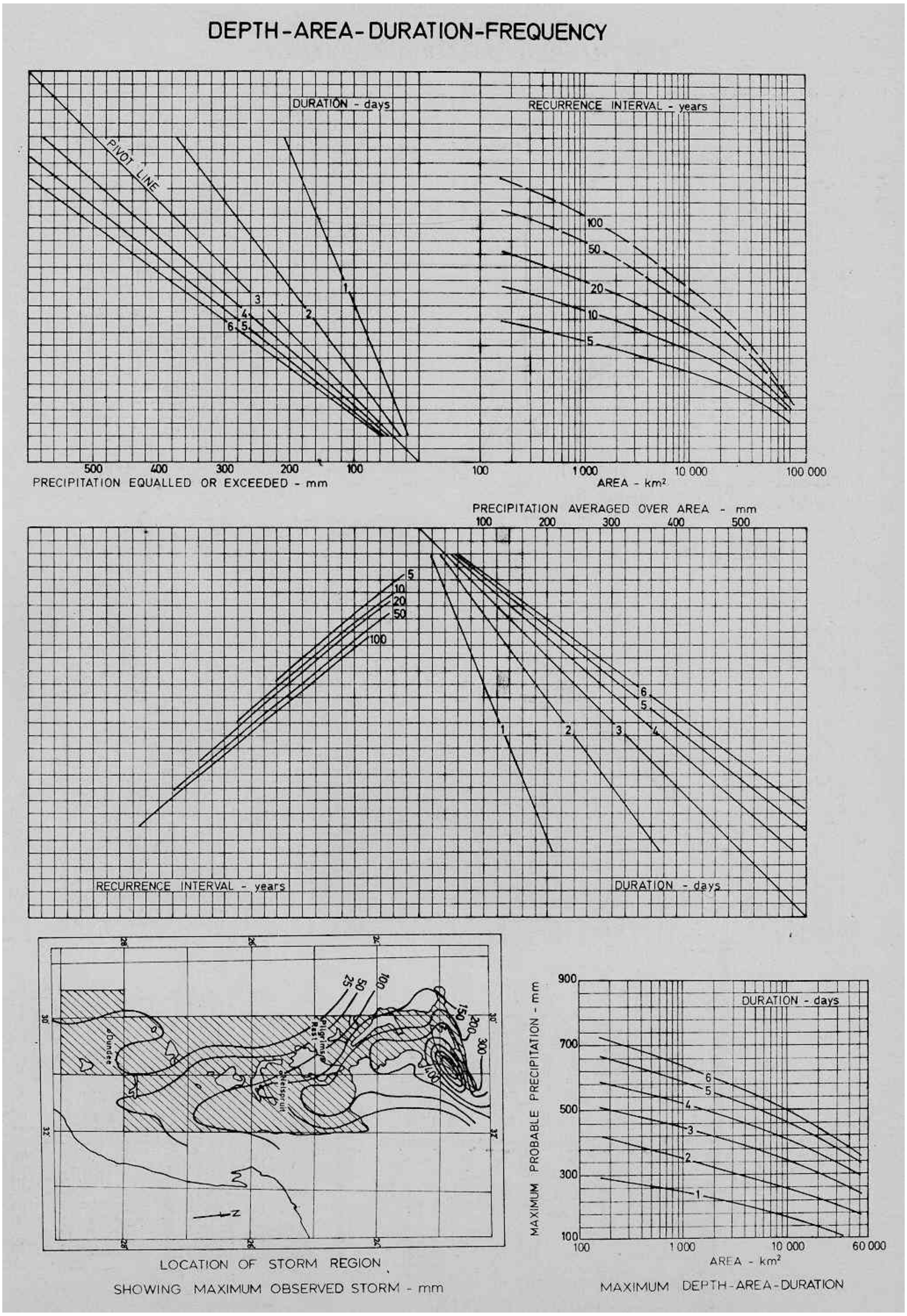

Figure 2.15 Storm Region 17 (MAP 500-1000 mm) (HRU 1/72)

HRU Approach for the Estimation of Probable Maximum Precipitation for Small-Area Storms (< 15 km^2)

Small-Area Storms imply the analysis of autographic records of point precipitation and are associated with high-intensity localised storms of short duration. An attempt was made to maximise the short-duration point rainfall through the same method employed for Large-Area Storms, but it was found to be impossible due to the lack of adequate meteorological observations during short-duration storms. Instead, an experience diagram was drawn comprising envelopes of the highest point precipitation of various durations observed in different parts of the country. The country was subdivided into a number of regions (Figure 2.16), each with its own maximum rainfall envelope (Figure 2.17). An envelope is also presented for the whole country, together with a comparative envelope of world rainfall records.

Comments on HRU Methodology

Our review of international approaches to PMP estimation has shown that the methodology followed by the HRU was credible and scientific. The HRU approach was aligned with the best conceptual approaches developed at the time and is still, in fact, aligned to present day international methodologies.

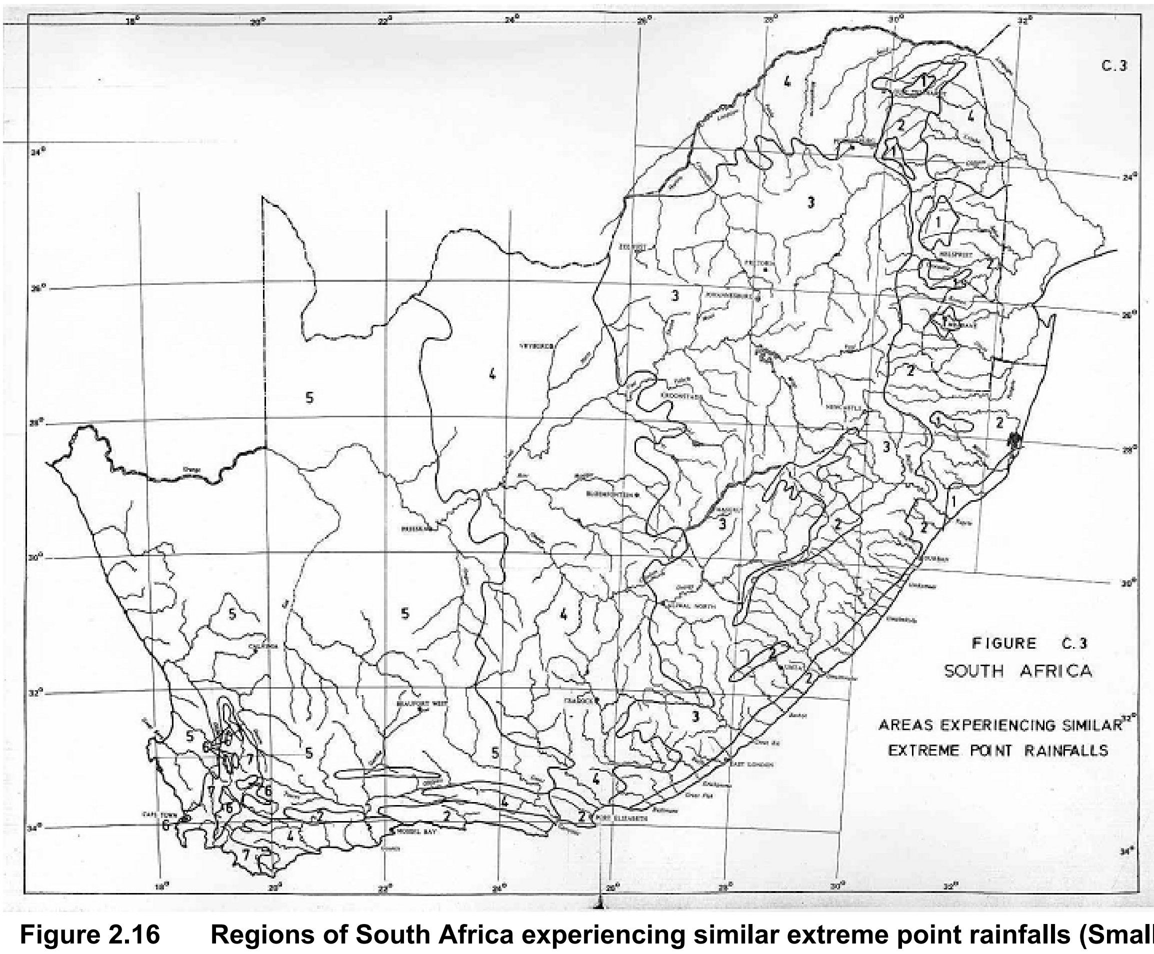

Figure 2.16 Regions of South Africa experiencing similar extreme point rainfalls (Small-Area Storms) (HRU 1/72)

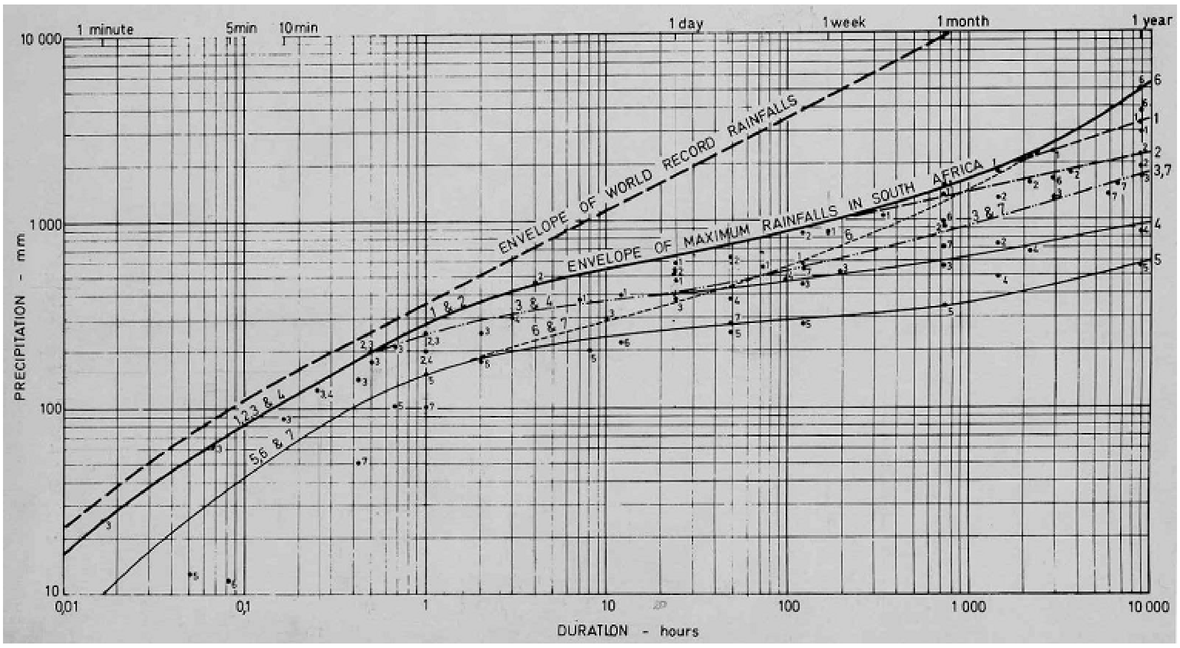

Figure 2.17 Maximum recorded point rainfalls in South Africa (HRU 1/72)

2.3 PRELIMINARY INVESTIGATION OF CURRENT APPLICABILITY OF THE HRU PMP ENVELOPE CURVES

2.3.1 Introduction

The currently established HRU 1/72 methodology presents a conservative and pragmatic approach for the estimation of the PMP in South Africa. For the estimation of both the Large-Area and Small-Area Storm PMP, the HRU considered only about 30 years of rainfall records from 1932. Since the 1960s, however, South Africa has experienced several large flood events, some of which caused extreme damage and loss of life. It is our aim to check the HRU PMP envelope curves against estimates based on the latest available rainfall data and, where necessary, propose improvements or sound cautions to practitioners using the HRU PMP envelope curves in design. Before embarking on a detailed study, two preliminary investigations were undertaken to determine if there was substance to the possibility that the HRU PMP envelope curves might have been exceeded by extreme rainfall events in the last 25 years.

2.3.2 Cyclone Domoina floods: January 1984

The tropical cyclone Domoina reached the coast of Mozambique on the 27th of January 1984. In the days that followed, Domoina was the cause of torrential rains over southern Mozambique, the Eastern Transvaal Lowveld, Swaziland and northern KwaZulu-Natal. The floods which followed directly affected tens of thousands of people, and more than 200 people died.

The report published by the Department of Water Affairs – "Documentation of the 1984 Domoina floods" (Kovacs et al., 1985) gave 4-day rainfall depths and the related area covered by that depth. Thus, these points could be plotted on the HRU PMP envelope curves for Region 17 (500 – 1000 mm), the HRU meteorological region most affected by the storm. Figure 2.18 shows that the 4-day rainfalls for the Domoina event plot well above its respective 4-day HRU PMP envelope curve for the region; in fact, they plot above the 6-day HRU PMP envelope curve. This alarming finding confirmed the need for this study to evaluate the suitability of the HRU PMP curves for other regions of the country.

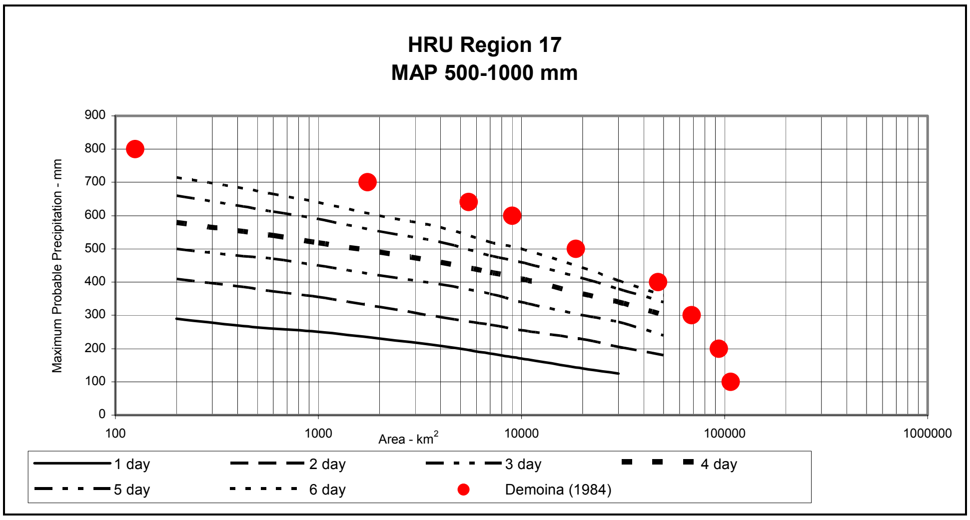

Figure 2.18 HRU PMP envelope curves for Region 17 (500 – 1000 mm) with 4-day rainfalls for Domoina (1984) plotted

2.3.3 Limpopo floods: February 2000

During February 2000 exceptionally heavy rains fell over the north-eastern parts of South Africa, Mozambique and Zimbabwe causing devastating flooding, loss of hundreds of lives and severe damage to infrastructure (Dyson, 2000, as cited by Smithers et al., 2001). The extreme rainfall was a result of tropical weather systems that moved from West to East over the subcontinent. The rainfall that fell was concentrated in two periods, 5 to 10 February and 22 to 25 February (Dyson, 2000, as cited by Smithers et al., 2001).

The methodology followed to quantify areal rainfalls for this storm for plotting on the HRU PMP envelope curves was the following:

1. Daily rainfall for gauges in the area was obtained (Lynch, 2004), and any rainfall station that had missing values on the days of interest was discarded from the study.

2. Two critical storm durations were decided upon for the first period of rainfall, a 2-day and 4-day rainfall durations, and a 2-day critical storm duration for the second period of rainfall.

3. For all chosen storm durations the rainfall gauge amounts were overlain with the topography of the area and the HRU meteorologically similar regions.

4. Rainfall isohyets of the various storms were drawn by hand. Figure 2.19 illustrates the isohyets for the 2-day rainfall of the 5th and 6th of February 2000, Figure 2.20 the 4-day rainfall isohyets of the 5th to 8th of February 2000, and lastly, Figure 2.21 shows the 2-day rainfall isohyets for the 23rd and 24th of February 2000.

5. From these, area-average rainfall was calculated and plotted on the respective HRU PMP envelope curves, as seen in Figure 2.22 and Figure 2.23.

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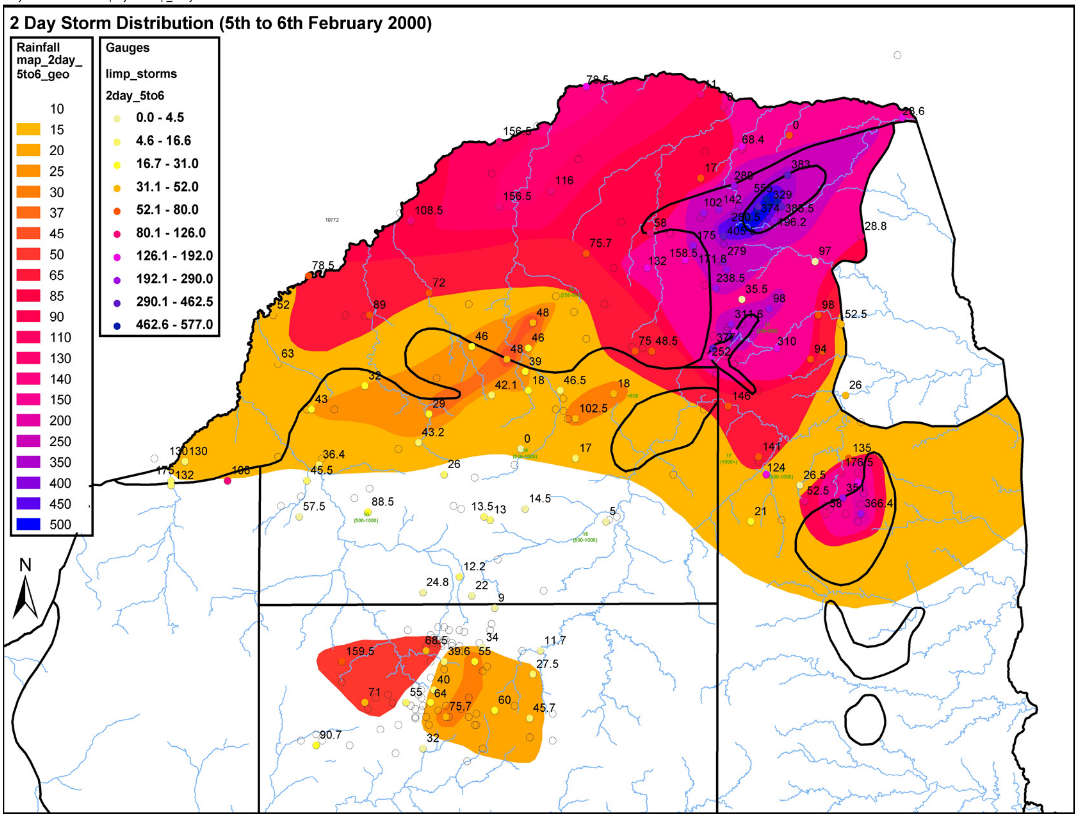

Figure 2.19 Limpopo Floods, 5th and 6th February 2000, 2-day rainfall isohyets

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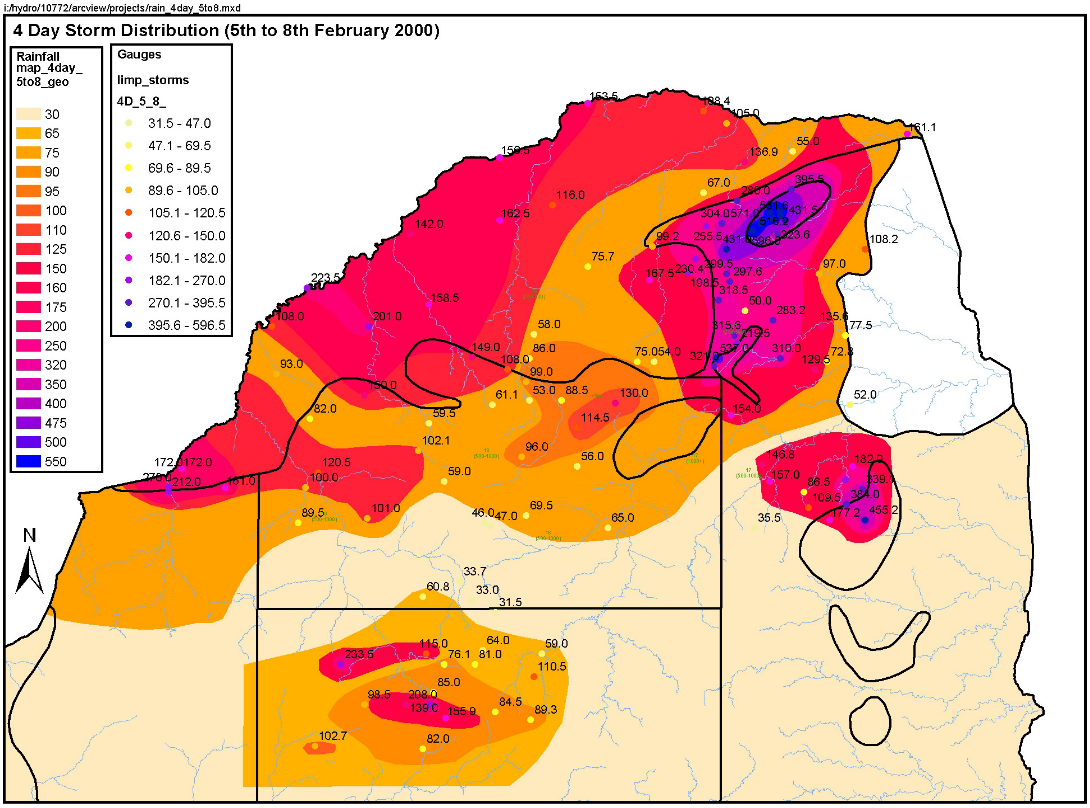

Figure 2.20 Limpopo Floods, 5th to 8th February 2000, 4-day rainfall isohyets

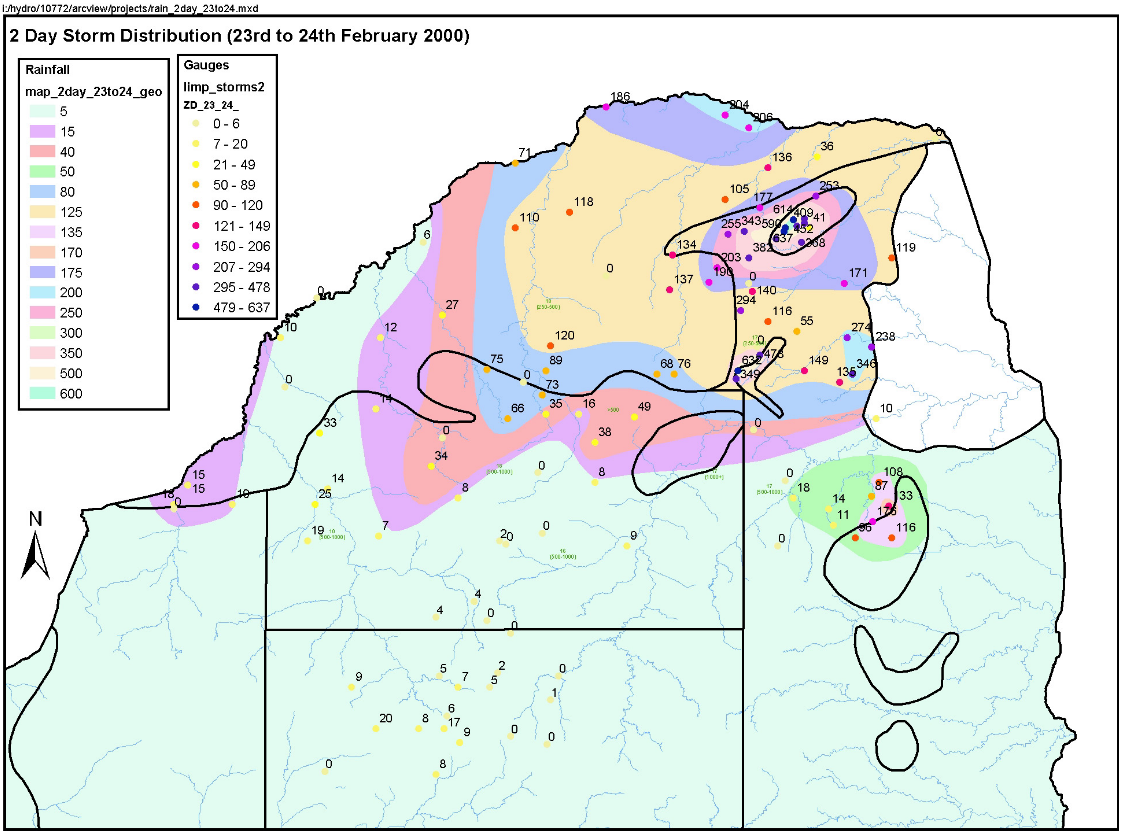

Figure 2.21 Limpopo Floods, 23rd and 24th February 2000, 2-day rainfall isohyets

It follows from Figure 2.22, the plotted storm rainfall for the 2-day duration of 23rd and 24th February, that although the value plot off the curves, it seems to be approaching the HRU PMP envelope curve for Region 18 (250 – 500 mm), as does the 4-day rainfall duration for 5 – 8 February. In Figure 2.23 none of the plotted points approach their respective HRU PMP envelope curves for Region 18 (500 – 1000 mm). Although the plotted storm rainfalls did not exceed the respective HRU PMP envelope curves, concern is still raised as the HRU curves were developed from maximised and transposed storms, while the plotted Limpopo storm rainfall of 2000 has not been maximised. By drawing the storm rainfall isohyets by hand a degree of human subjectivity is introduced, thus, making replication difficult. For further investigation a more automated method of constructing the isohyets needed to be found.

With the Domoina Floods of 1984 exceeding the HRU PMP envelope curves and the Limpopo Floods of 2000 approaching the HRU PMP envelope curves, the need to further investigate the adequacy of the HRU PMP envelope curves for present day designs was established. The similarity between the heavy rainfall events discussed above was their cyclonic nature. From the lessons learned in these preliminary analyses the following criteria were identified in for the investigation of the adequacy of the HRU PMP envelope curves:

- the method used needed to be replicable and scientifically sound,
- the storms selected needed to cover a range of different HRU meteorologically similar regions, and
- the rainfall producing mechanism needed to be different.

The following Sections outline the methods employed in investigating the adequacy of the HRU PMP envelope curves, the results obtained and recommendations.

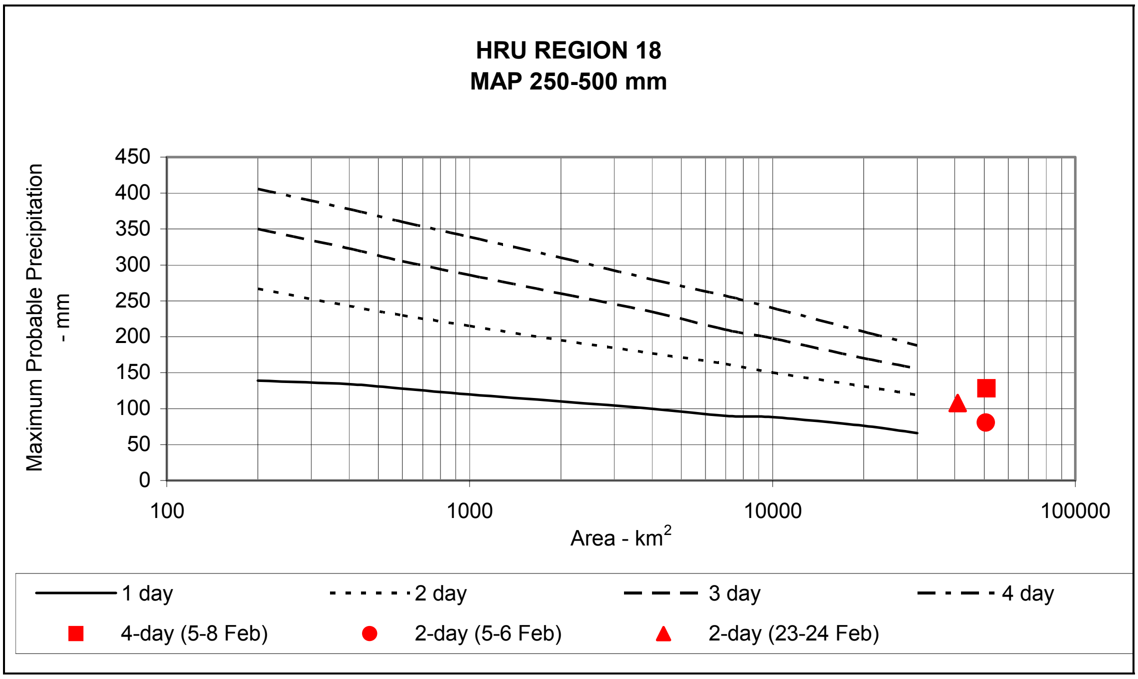

Figure 2.22 **HRU PMP envelope curves for Region 18 (250 – 500 mm) with 2-day (5 – 6 February), 4-day (5 - 8 February) and 2-day (23 – 24 February) rainfall durations for the Limpopo Floods plotted**

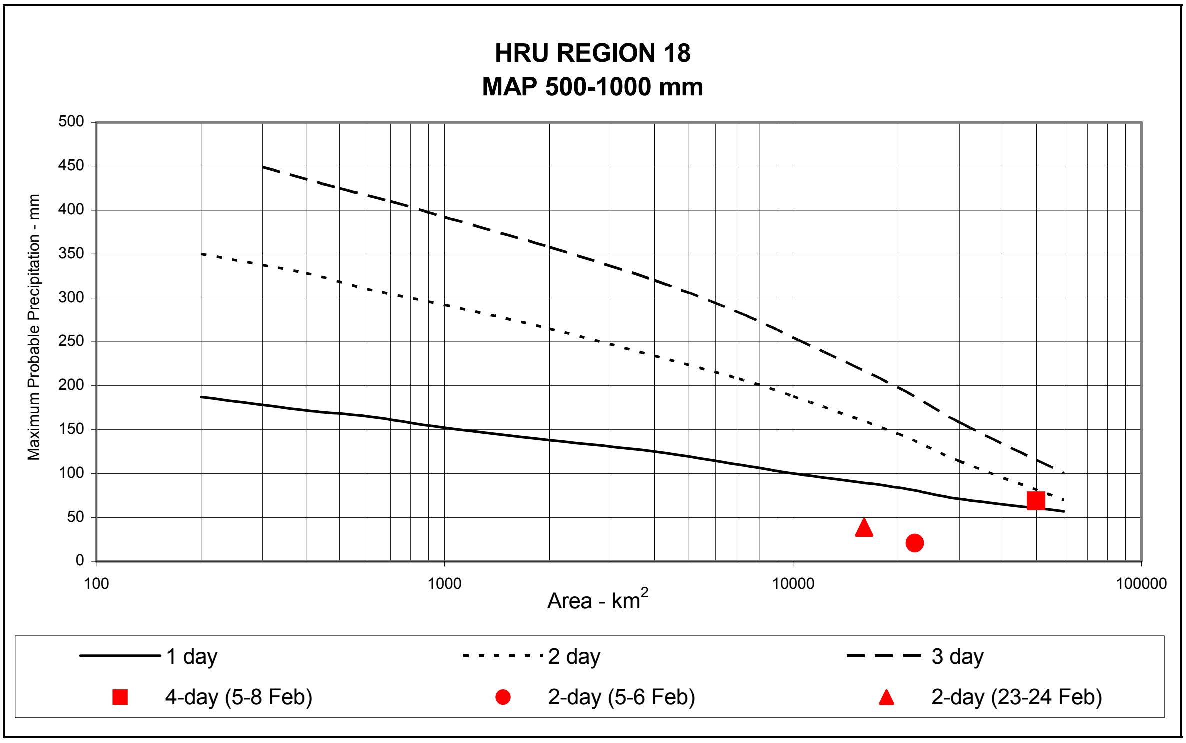

Figure 2.23 **HRU PMP envelope curves for Region 18 (500 – 1000 mm) with 2-day (5 – 6 February), 4-day (5 - 8 February) and 2-day (23 – 24 February) rainfall durations for the Limpopo Floods plotted**

2.4 DETAILED STORM ANALYSIS METHODOLOGY

2.4.1 Introduction

Our preliminary investigations showed that the HRU PMP envelope curves had been exceeded in the case of the Domina floods, and the Limpopo flood rainfalls plotted in the vicinity of the curves. However, these two extreme rainfall events were due to cyclonic rainfall which is not a common occurrence over central, western and southern regions of South Africa. The question whether rainfall events caused by commonly occurring weather patterns over these other regions of South Africa might have exceeded the established HRU PMP envelope curves, remained unanswered. This prompted the further investigations discussed hereinafter.

2.4.2 Selection of storms

Based on literature information and institutional knowledge, four large storms were selected, as shown in Table 2.4. With the storms identified, the area affected by the storms was obtained from literature. Apart from the severe nature of these storms, they were located in diverse regions in terms of MAP Figure 2.24). The Natal Floods occurred in an area of high MAP, the Laingsburg floods occurred in a dry region of the country, while both the South Eastern Cape and Orange River Basin Floods occurred over areas with a MAP ranging from reasonably high to low. The dominant rainfall-producing mechanisms of the four areas differ significantly as well. Thus, the storms selected covered a number of different meteorologically similar regions as identified by HRU (1972).

Table 2.4 Selected extreme South African floods

Flood Event	Date
Orange River Basin	1988
KwaZulu-Natal Floods	1987
South Eastern Cape	1981
Laingsburg, Karoo	1981

2.4.3 Obtaining storm rainfall data

Data for all rainfall stations within the respective storm areas was obtained from a database developed by Lynch (2004) using the Daily Rainfall Extraction Utility developed by Kunz (2004). This daily rainfall database is a comprehensive and up-to-date database. It consists of more than 300 million daily rainfall values from 12 153 rainfall stations, of the monitoring networks of the South African Weather Service (SAWS), the Agricultural Research Council (ARC), the South African Sugarcane Research Institute (SASRI) and from municipalities, private companies and individuals (Lynch, 2004). The extracted daily rainfall data was examined and rainfall stations which contained patched values during the period of interest were discarded from the study. Given the extreme nature of the rainfall under consideration in this study, it is not surprising that numerous rainfall gauges failed or had missing data over the period of interest. Shown in Table 2.5 are the number of rainfall stations present in the storm area, and the number which had recorded values for the period of interest.

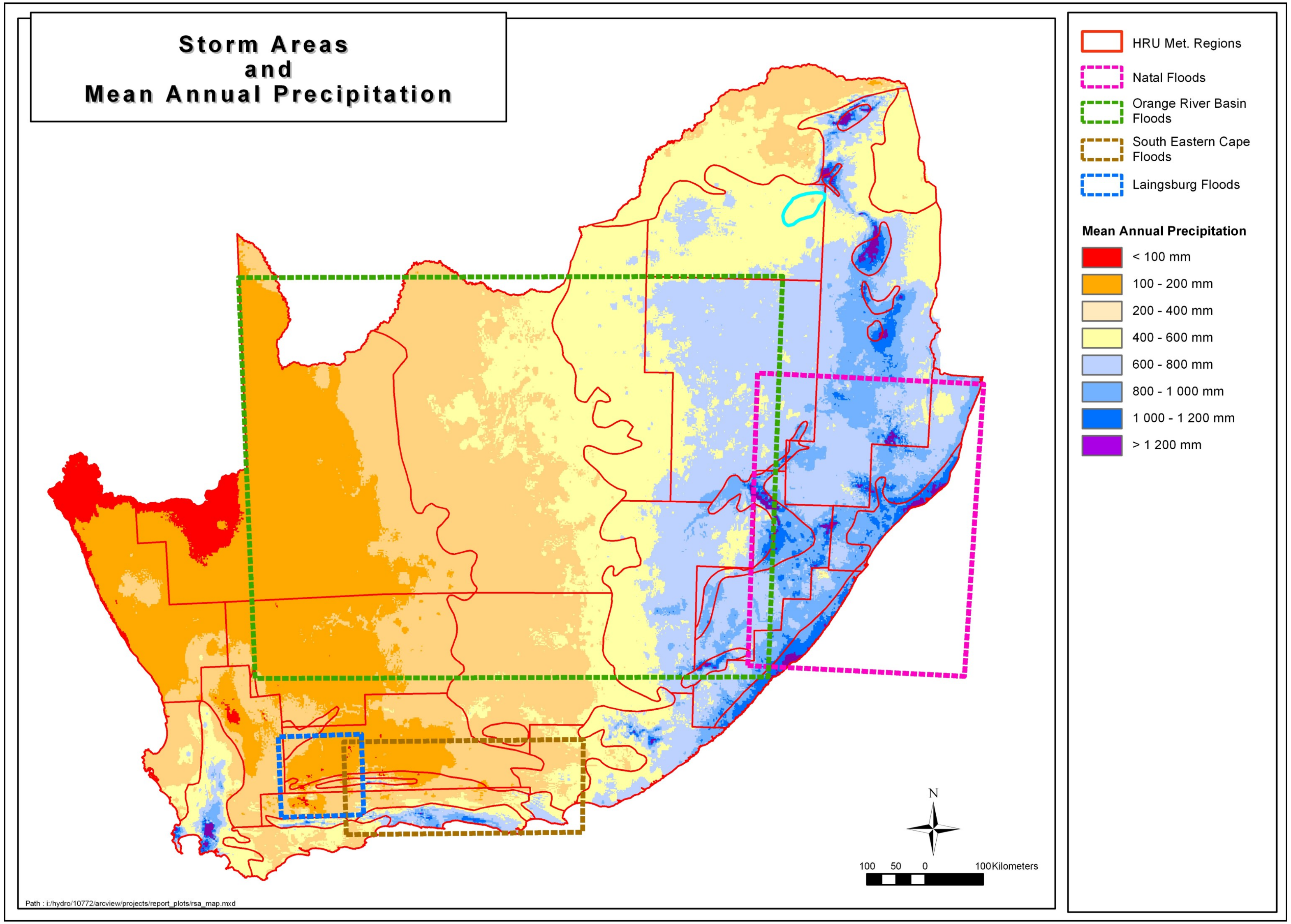

Figure 2.24 Areas affected by selected storms, MAP (after Schulze, 1997) and HRU meteorologically similar regions

Table 2.5 Reduction in rainfall stations included in study due to missing data

Flood Event	Date	Number of Rainfall stations extracted	Number of Rainfall stations with complete data for storm period
Orange River Basin	1988	3 739	1 069
KwaZulu-Natal Floods	1987	1 172	448
South Eastern Cape	1981	383	132
Laingsburg, Karoo	1981	55	25

2.4.4 Methodology used for drawing storm isohyets

The HRU 1/69 report used a 'isopercental' procedure to determine the isohyets of the storm rainfall as the coverage of rainfall gauges was irregular, the basic premise of the procedure is that the topographic features of the study area affect the areal distribution of the rainfall depths during a storm in much the same way as the MAP would be affected, thus topography and the MAP of the area were used indirectly in determining the isohyets. The procedure used is outlined in detail in Section 2.2.

In this research the spline method in ARCVIEW, which is similar to inverse distance weighting, was used to draw the isohyets. Thus, the MAP and topographic features were not used in determining the area-average rainfall in this research. The spline method and inverse distance weighting are further discussed below.

Inverse distance weighting is a common method used to estimate missing spatial data or create a surface of data. A fine mesh grid is laid over the area of concern and a precipitation value interpolated into each grid cell by using the inverse distance squared between the cell location and the rainfall gauge location as a weight for each rainfall gauge (ESRI, 2005). The inverse distance weighting method is based on the assumption that the interpolating surface should be influenced most by the nearby points and less by the more distant points. The interpolating surface is a weighted average of the scatter points and the weight assigned to each scatter point diminishes as the distance from the interpolation point to the scatter point increases (ESRI, 2005).

The spline method is similar to the inverse distance weighting method. A spline function is a polynomial function which passes through all the rainfall gauge point values and which is smoothed thereby reducing the number of peaks and pits in comparison to the inverse distance weighting method. The "roughness" of the surface is reduced by the action of forming a polynomial function through the data points (ESRI, 2005).

2.4.5 Determining the storm boundary

From the literature reviewed, no standard definition for determining the boundary of a storm could be obtained. Thus, three methods of defining the storm boundary were used. Each of the methods discussed below were applied to all four storms, however, not all methods were applicable to each storm, and thus only the results applicable are shown in Section 2.5.

Method 1: Bounded by HRU meteorologically similar regions
Once the storm isohyets had been drawn the HRU meteorologically similar regions were overlain using ARCVIEW. If the storm completely covered a HRU region, the area of that region was calculated, and the area-average rainfall for the storm was calculated for that area, for each selected storm duration.

Method 2: Defining the storm boundary by taking a proportion of the peak
It was recognised that the storms did not always cover an entire HRU meteorologically similar region, thus it was decided to determine the highest rainfall amount for each selected duration of the storm, and to determine the area and area-average rainfall which experienced rainfall in excess of one-third and two-thirds of the peak rainfall.

Method 3: Storm cells
In order to capture the intensity of the storm events, area and area-average rainfall were calculated for specific storm cells within the storm. From the storm isohyet maps, storm cells were identified, with the boundary being where the intensity changed significantly (i.e. the distance between the isohyets increased). For the area covered by the storm cell the area and area-average rainfall were calculated.

2.5 RESULTS OF DETAILED STORM ANALYSES

2.5.1 Orange River basin floods: February – March 1988

During February and March 1988, heavy and prolonged rainfalls occurred over much of the interior of South Africa, with some areas receiving rainfall equivalent to their mean annual precipitation (MAP). As a result widespread flooding occurred causing damage to many towns, farms and structures. For the purposes of this study the rainfall which resulted in two large floods in the Orange River catchment was analysed. Details of this flood event were obtained from a Department of Water Affairs (now the Department of Water Affairs and Forestry) report titled *Documentation of the February-March 1988 floods in the Orange River Basin* (Du Plessis et al., 1989).

Storm Duration

Upon examination of the rainfall data, two storm periods were selected; the first was 18 – 23 February and the second 8 – 13 March. Both events had a 2-day effective storm duration. The February storm 2-day event was on 21st and 22nd of February, and the March storm 2-day event was the 9th and 10th of March. However, after mapping the storms it was decided that the February storm was more intense and the March storm was discarded from the study.

Cause of Rainfall

On the 11th of February 1988 a low-pressure system developed over Botswana. By the 17th of February this low-pressure system had moved further southwards, causing an influx of moist tropical air, which resulted in rains over the Northern Cape Province. From the 13th to 16th of February a low-pressure system developed over the south-east Atlantic, this system moved south-eastwards to link up with the tropical low-pressure located over Botswana to form a tropical temperate trough. These prevailing conditions were further exacerbated by the passage of a cold front on the 18th of February causing heavy rainfalls over the eastern Cape interior and north-eastern Karoo. An easterly airflow, caused by a cell of high pressure located to the east of South Africa, resulted in increased surface convergence which enhanced convection within the tropical-temperate trough, thus widespread rainfall over the Free State and Cape Province ensued until the 23rd of February, after which the low over Botswana moved northwards. In subsequent days this low-pressure system caused further heavy rainfalls over the region.

Rains experienced between the 9th and 12th of March were attributable to the combined influence of several weather systems over most of South Africa. The presence of pressure systems such as those that resulted in the heavy and widespread rainfall during February and March 1988 over the interior of South Africa is not unusual; however, in this storm the magnitude and duration of the resulting rainfall was unusual.

Storm Isohyets

Figure 2.25 presents the storm isohyets for the 1-day rainfall and Figure 2.26 the storm isohyets for the 2-day rainfall. The coverage of rainfall stations recording on the days of concern in the Orange River Basin was substantial, as can be seen in both Figure 2.25 and Figure 2.26. Thus, a fairly accurate picture of the storm isohyets could be drawn.

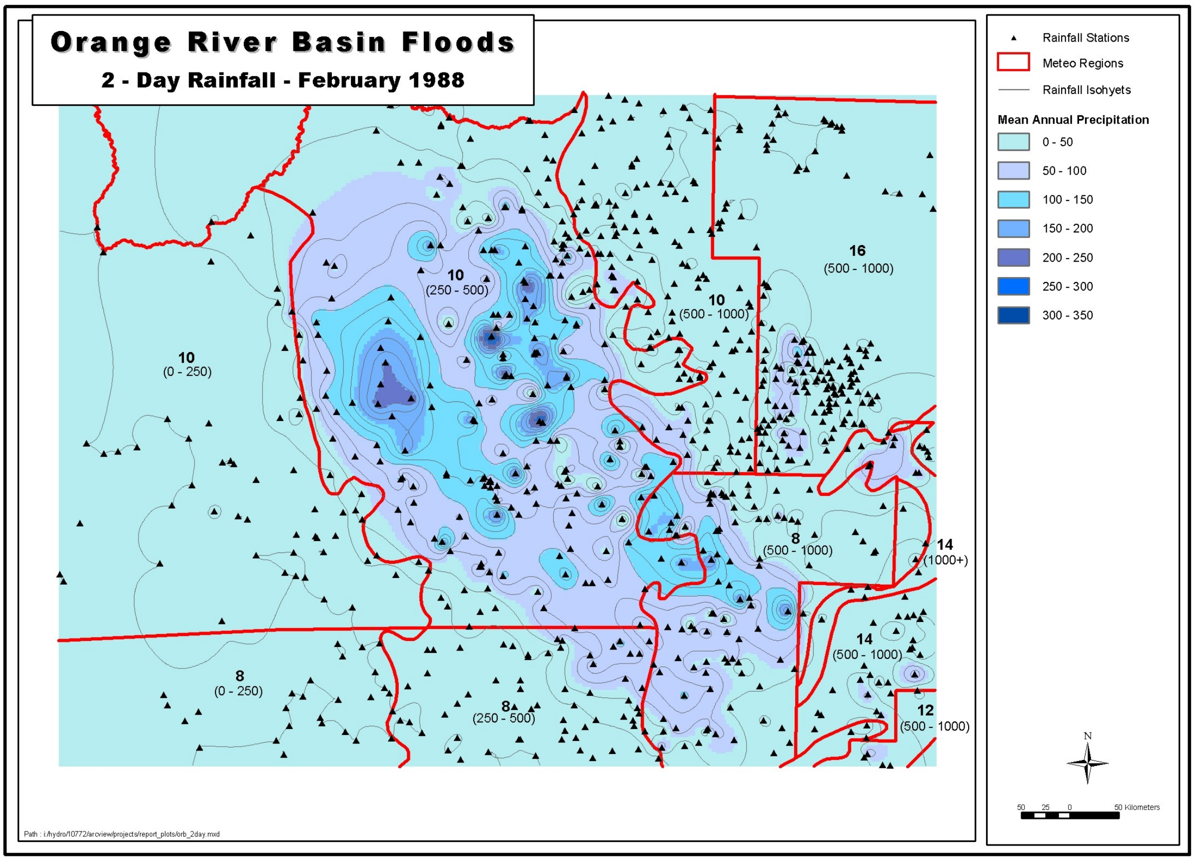

Figure 2.25 Orange River Basin Floods, February 1988, 1-day rainfall isohyets

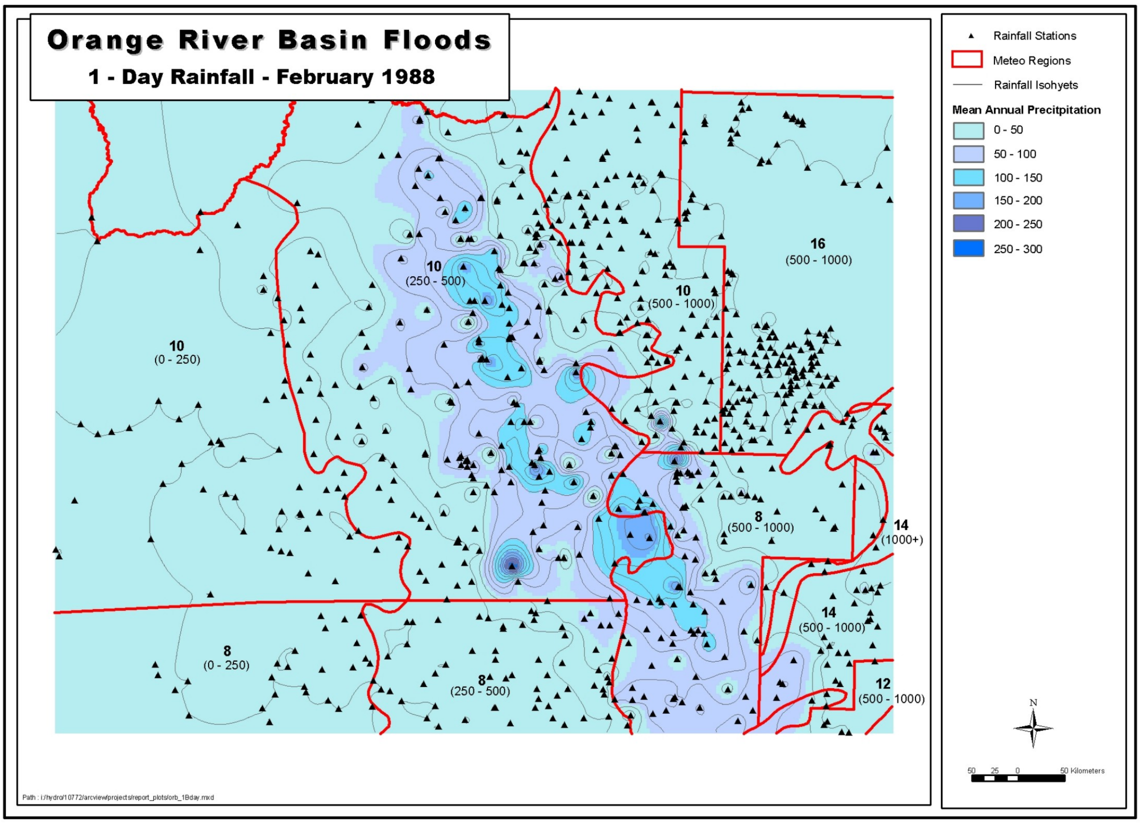

Figure 2.26 Orange River Basin Floods, February 1988, 2-day rainfall isohyets

Results

For the 1-day duration the storm occurred primarily over the HRU region 10 (250 – 500 mm), however it did not completely encompass the region (Figure 2.25). The 2-day rainfall occurred over both HRU regions 10 (250 – 500 mm) and 8 (500-1000 mm), in neither case did the heavy rainfall occur over a complete region. Taking a proportion of the peak rainfall value as the storm boundary was not applicable in this instance, as the area covered with both one-third and two-thirds of the peak rainfall value was too large to plot on the HRU PMP envelope curves. Thus, the only approach for which results are shown is that of the analysis of storm cells.

The dominant HRU meteorologically similar region over which the storm rainfall fell was HRU Region 10 (250 – 500 mm). Therefore, the area-average rainfall determined for two 1-day duration storm cells and two 2-day duration storm cells were plotted against area on the HRU PMP envelope curve for Region 10 (250 – 500 mm), as shown in Figure 2.27. Both the 2-day duration rainfalls for the storm cells plotted below the 2-day HRU curve, with the smaller storm cell plotting closer to the curve. For the 1-day duration rainfall, the larger storm cell plotted above the 1-day HRU curve, while the smaller storm cell plotted marginally below the curve.

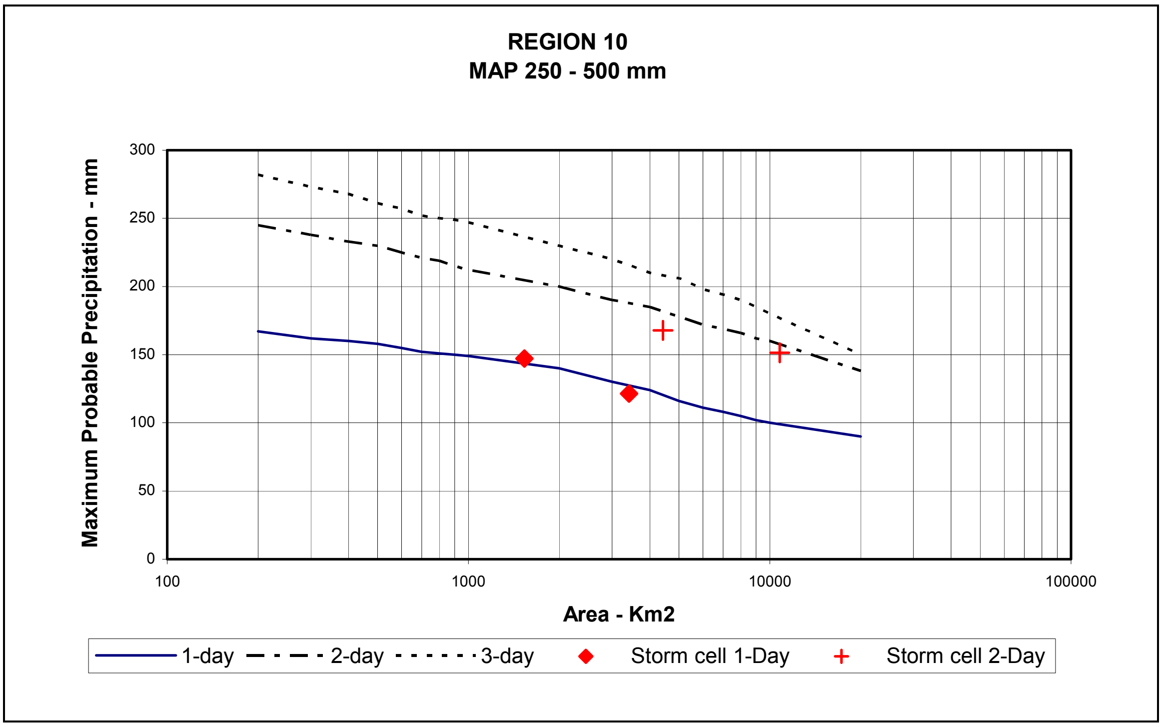

Figure 2.27 HRU PMP envelope curves for Region 10 (250 – 500 mm) with storm cells plotted

2.5.2 KwaZulu-Natal floods: September 1987

From the 28th to 30th of September 1987 devastating floods ravaged the central and southern parts of KwaZulu-Natal. The damage was catastrophic: 388 people lost their lives, 140 000 people were left homeless, damage to agriculture, communications, infrastructure and private property amounted to R 400 million. Details of the flood event were obtained from a Department of Water Affairs report, "Documentation of the September 1987 Natal Floods", which was compiled by Van Bladeren and Burger (1989).

Storm Duration

The highest 1-day rainfalls were experienced on 28 September 1987 and the highest 2-day rainfalls on the 27th and 28th September. The effective duration of the storm was 3-days, stretching from the 26th September to the 28th September.

Cause of Rainfall

The heavy and prolonged rainfall that fell during September 1987 was the result of the presence of a cut-off low and a strong onshore easterly air flow. The strong onshore flow of air was caused by the ridging of a high pressure system which originated over the Atlantic and moved to the south and then northwards east of the continent.

Storm Isohyets

Figure 2.28 illustrates the storm isohyets for the 1-day duration rainfall for the KwaZulu-Natal Floods which occurred in September 1987, Figure 2.29 the 2-day duration storm isohyets and Figure 2.30 the 3-day duration. Similar, to the Orange River Basin area, there were a large number of rainfall gauges that recorded the storm, thus improving the ability to draw reliable storm isohyets. A number of rainfall gauges included in the study were South African Sugar Research Institute rainfall gauges.

Results

The first approach taken to determine the storm boundary was to consider an HRU Region completely covered by the storm rainfall. As can be seen from Figure 2.28 to Figure 2.30, the storm rainfall fell over a number of HRU meteorologically similar regions, namely, Region 13 (1000 mm+), Region 13 (500 – 1000 mm), Region 12 (1000 mm+), Region 12 (500-1000 mm), Region 14 (500 – 1000 mm), Region 14 (1000 mm+) and Region 17 (500 – 1000 mm). However, the only region fully encompassed by the storm rainfall was HRU Region 13 (1000 mm+). Thus, the area-average storm rainfall for this region was calculated for the 1-day, 2-day and 3-day rainfall durations, and plotted on the HRU PMP envelope curves for Region HRU 13 (1000 mm+; Figure 2.31). For all considered rainfall durations the points plotted above the curves. The 1-day duration rainfall plotted marginally above the HRU PMP envelope curves. The 2-day duration plotted substantially higher (50 mm) than the 2-day HRU PMP envelope curve for Region 13 (1000 mm+), and the 3-day duration plotted well above (100 mm) the 3-day HRU PMP 3-day curve.

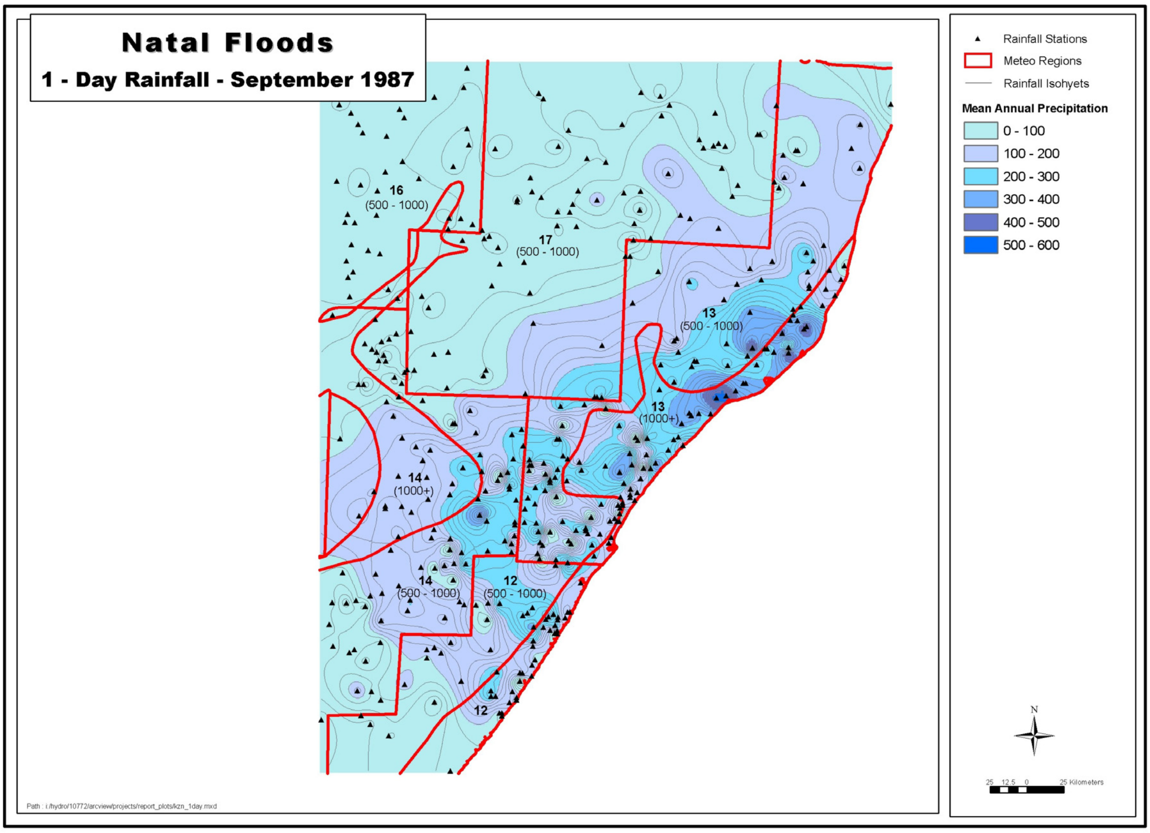

Figure 2.28 KwaZulu-Natal Floods, September 1987, 1-day rainfall isohyets

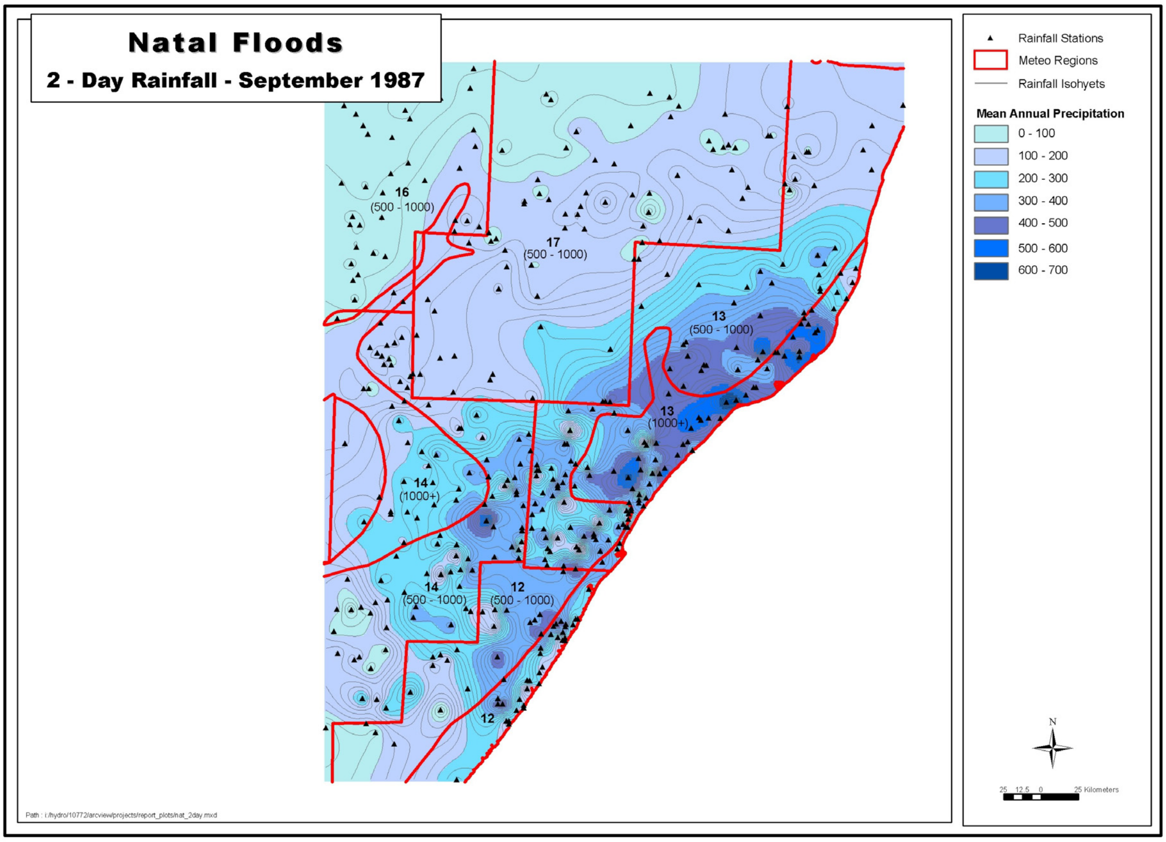

Figure 2.29 KwaZulu-Natal Floods, September 1987, 2-day rainfall isohyets

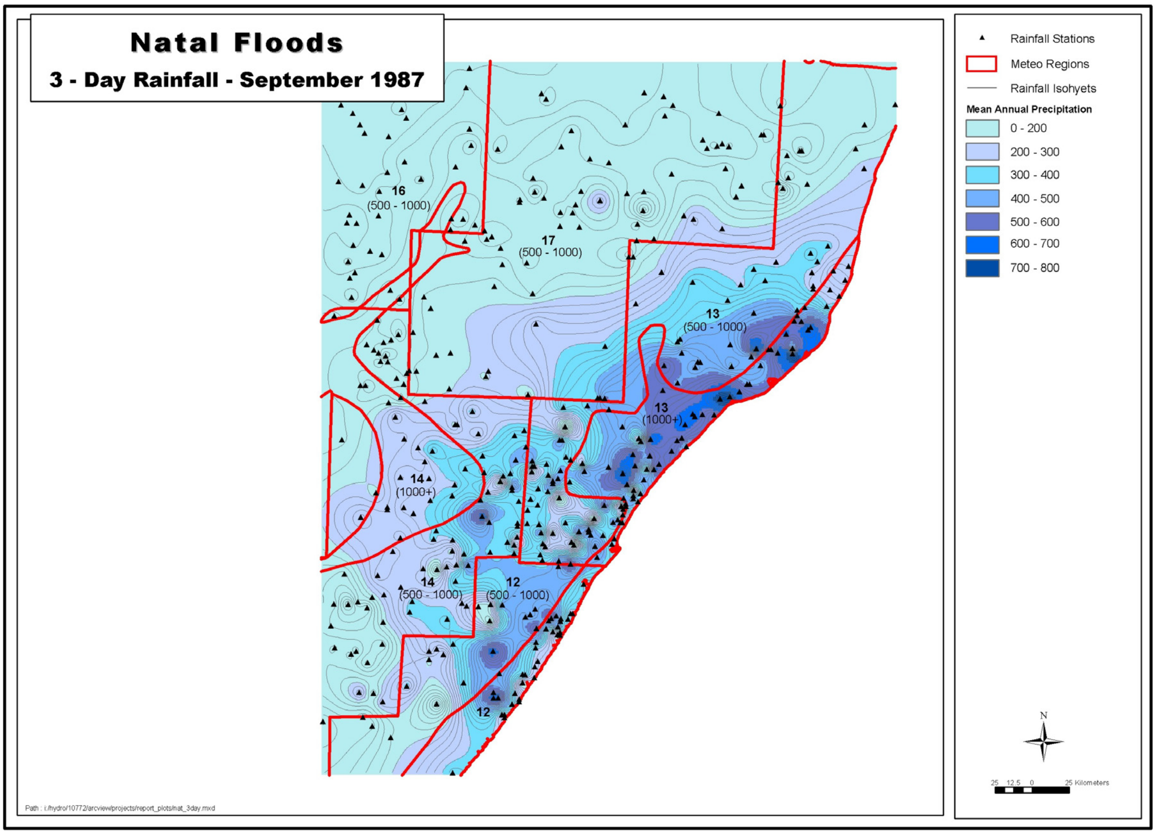

Figure 2.30 KwaZulu-Natal Floods, September 1987, 3-day rainfall isohyets

Another method of determining the storm boundary was to consider the area experiencing rainfall in excess of one-third of the peak rainfall. The area-average rainfall for the 1-day, 2-day and 3-day durations was calculated and plotted on the HRU PMP envelope curves for Region 13 (1000 mm+). The resulting plot is shown in Figure 2.32. Similar, to the previous method of determining the storm boundary, the 1-day, 2-day and 3-day duration rainfalls for the September 1987 floods in KwaZulu-Natal, plotted above their respective HRU PMP envelope curves. However, in all three instances they plotted higher above the curves than they plotted with the previous method of defining the storm boundary. In both methods, the longest duration, 3-day rainfall duration, plotted the highest above its respective HRU PMP envelope curve for Region 13 (1000 mm+).

The last method considered for defining storm boundaries was to isolate storm cells, and determine the area-average rainfall for those storm cells. The most intense storm cell was located along the coast and had a peak 1-day rainfall of 689.4 mm. As mentioned earlier the boundary of the storm cell was defined as the area where the distance between the rainfall isohyets increased. Figure 2.33 shows the plot of the area-average rainfall for all considered durations on the HRU PMP envelope curves for Region 13 (1000 mm+). Unlike, the previous two methods the 2- and 3-day duration rainfall plot below their respective curves, while only the 1-day duration rainfall plots on the respective HRU PMP envelope curve. The area of the storm cells was substantially less than the area considered in the previous two methods of defining the storm boundary, thus it seems that for isolated storms the HRU curves may be adequate to determine the maximum probable precipitation, while for widespread heavy rainfall the HRU curves on the east coast of South Africa may underestimate the maximum probable precipitation.

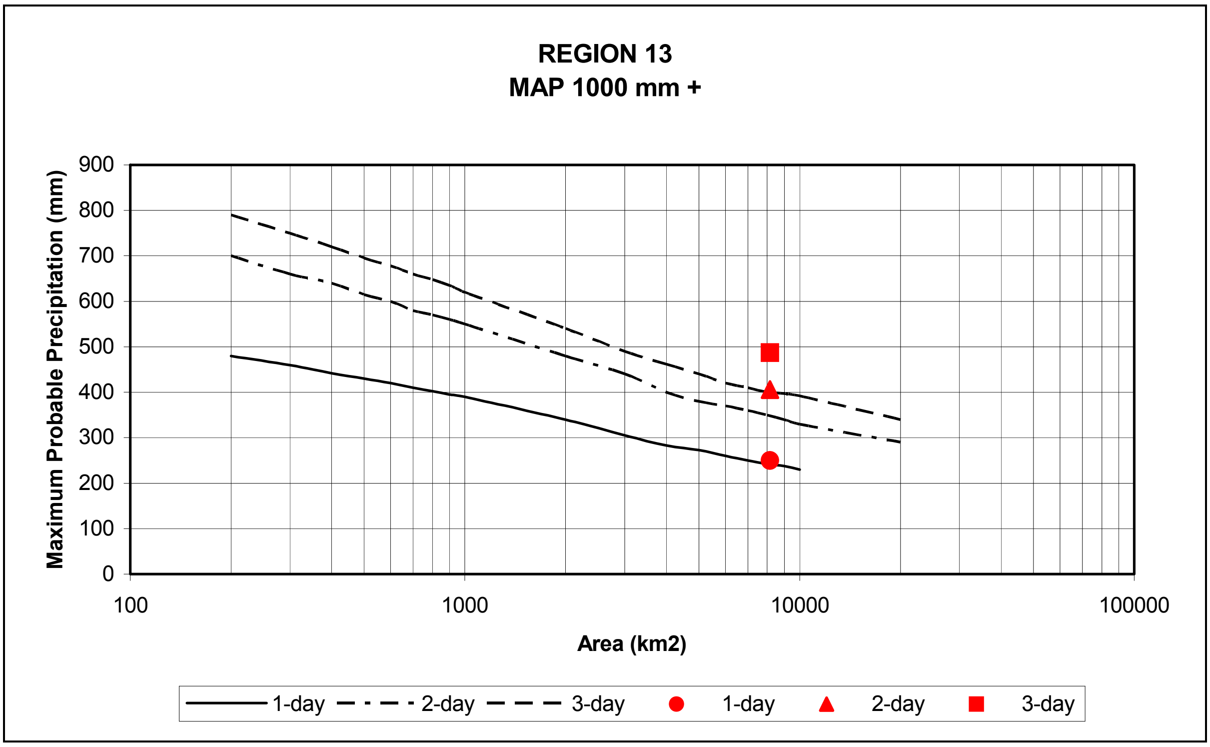

Figure 2.31 HRU PMP envelope curves for Region 13 (1000 mm+) with 1-day, 2-day and 3-day duration rainfall for the September 1987 storm

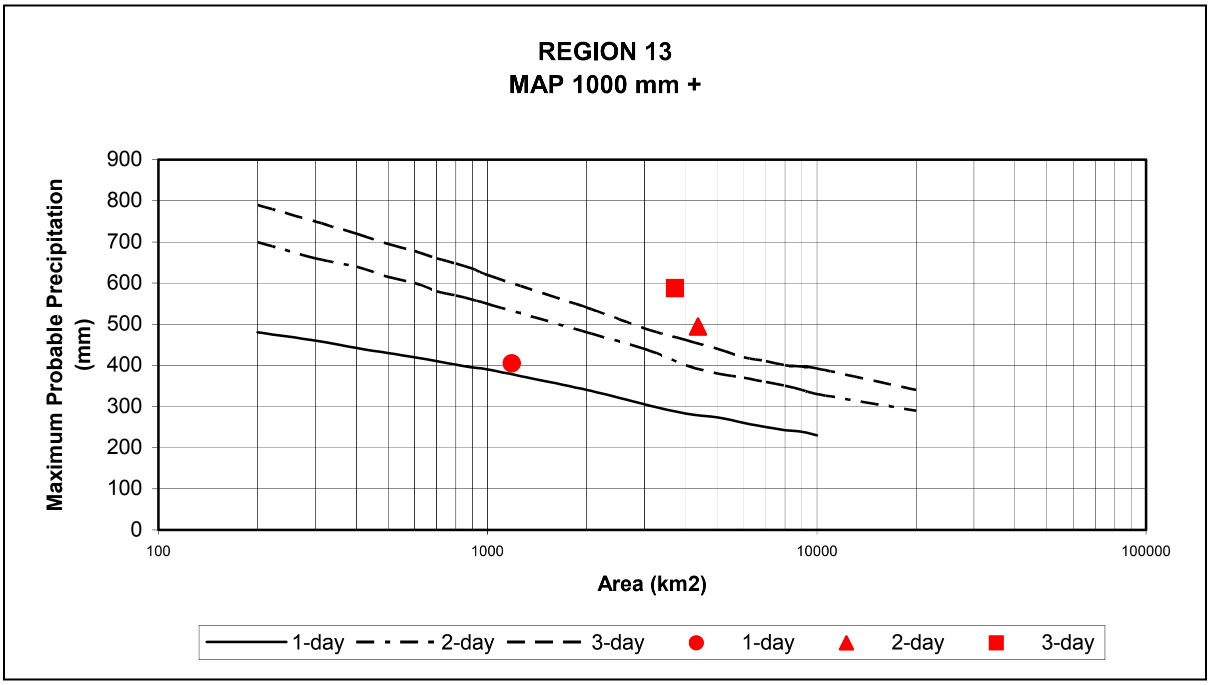

Figure 2.32 **HRU PMP envelope curves with 1-day, 2-day and 3-day duration rainfall for over the area with more than one-third of the peak rainfall for the September 1987 storm**

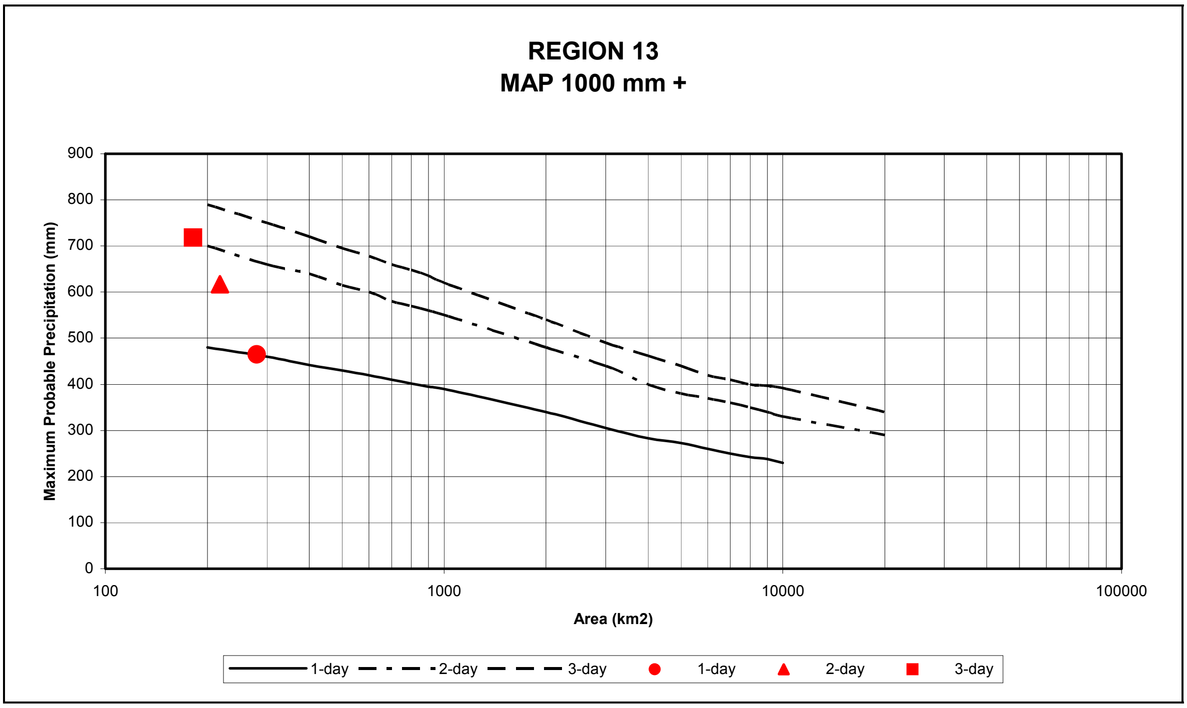

Figure 2.33 **HRU PMP envelope curves with 1-day, 2-day and 3-day duration rainfall for storm cells for the September 1987 storm**

2.5.3 Laingsburg flood: January 1981

Sunday 25 January 1981 was a disastrous day for the town of Laingsburg in the Karoo. The Buffels River, which flows past the town, started overtopping its banks at approximately 10 am and some seven hours later, 90 lives were lost and damage caused in excess of R10 million. Details of the flood event were obtained from a paper titled "Lessons learnt from the 1981 Laingsburg Flood" (Roberts and Alexander, 1982) published in the journal "The Civil Engineer in South Africa". Laingsburg, as shown in

Figure 2.34, is situated on the banks of the Buffels River, which flows in a southerly direction to join the Touws River, thus forming the Groot River which eventually turns into the Gouritz River.

Storm Duration

Although the flood event occurred on the 25th of January, the largest rainfalls occurred on the 24th of January. Thus the considered 1-day event was the 24th of January and the 2-day event the 24th and 25th of January.

Cause of Rainfall

The synoptic situation which developed over the south-western parts of South Africa during the weekend of 24 and 25 January was described as a typical black south-easter. It is this synoptic situation that was responsible for the flood producing rainfall experienced in the Laingsburg area.

Problems Encountered in Analysis

Laingsburg is situated in the Karoo, an area with a MAP of approximately 200 mm. This is an area with a sparse network of raingauges, and as is common during extreme rainfall events many of the raingauges in the area have missing data during the event. As so few stations had actual rainfall records for the storm event, the patched/infilled stations were included in the Laingsburg study.

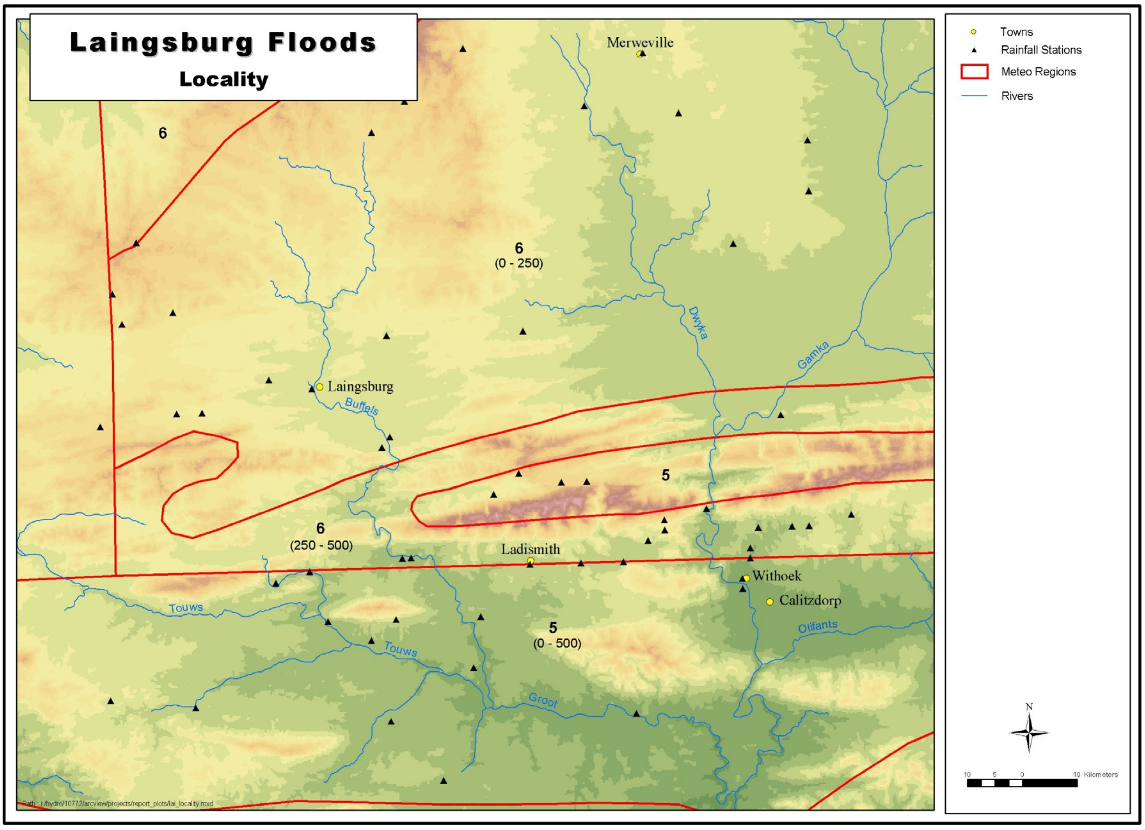

Figure 2.34 The topography and rivers of the area affected by the Laingsburg Floods

Missing or suspect data in the rainfall database used was infilled using the following hierarchy of infilling techniques:

1. Expectation maximisation algorithm;
2. Median ratio method;
3. Inverse distance weighting; and a
4. Monthly infilling technique (Lynch, 2004).

Although it is not desirable to include infilled stations, it was argued that, for the Laingsburg event, the inclusion of the infilled stations was crucial to form a more complete picture of the rainfall event.

Storm Isohyets

Illustrated in Figure 2.35 and Figure 2.36, respectively, are the 1-day and 2-day duration storm rainfall isohyets. Immediately evident is the sparse coverage of rainfall stations, even though the rainfall stations that had patched rainfall data on the days of concern are included.

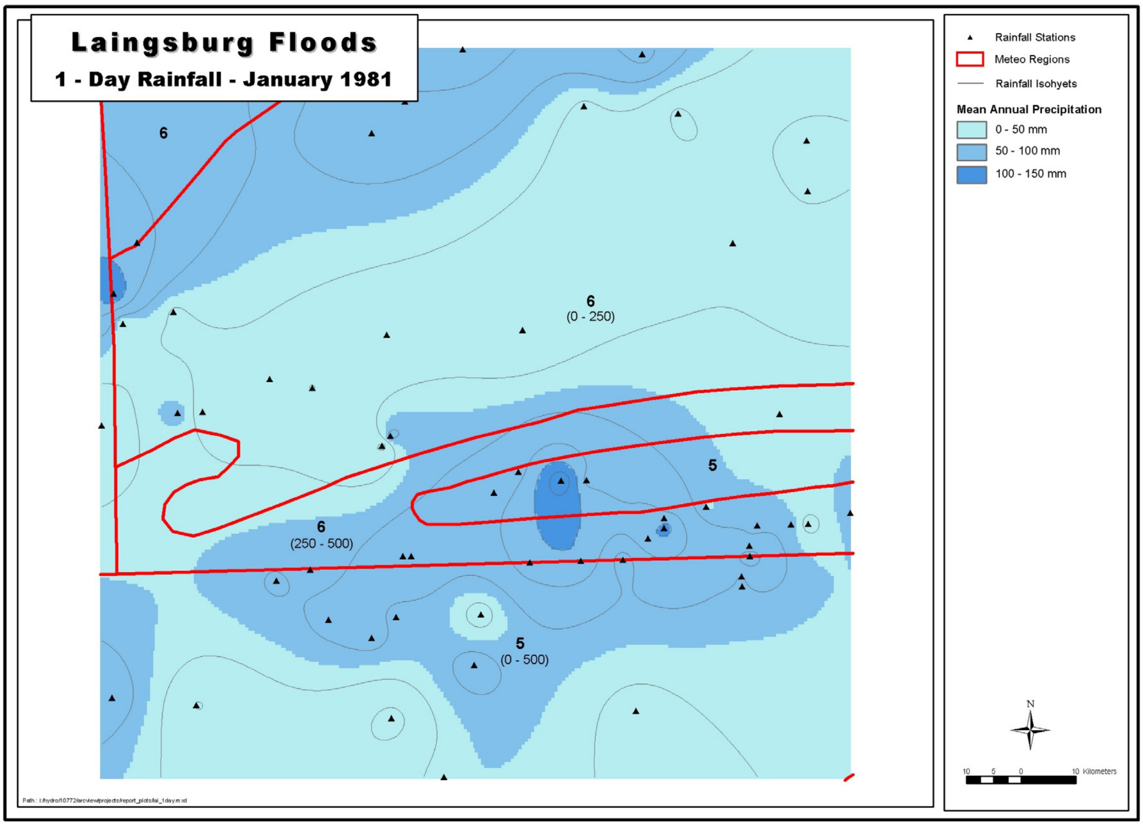

Figure 2.35 **Laingsburg, January 1981, 1-day rainfall isohyets**

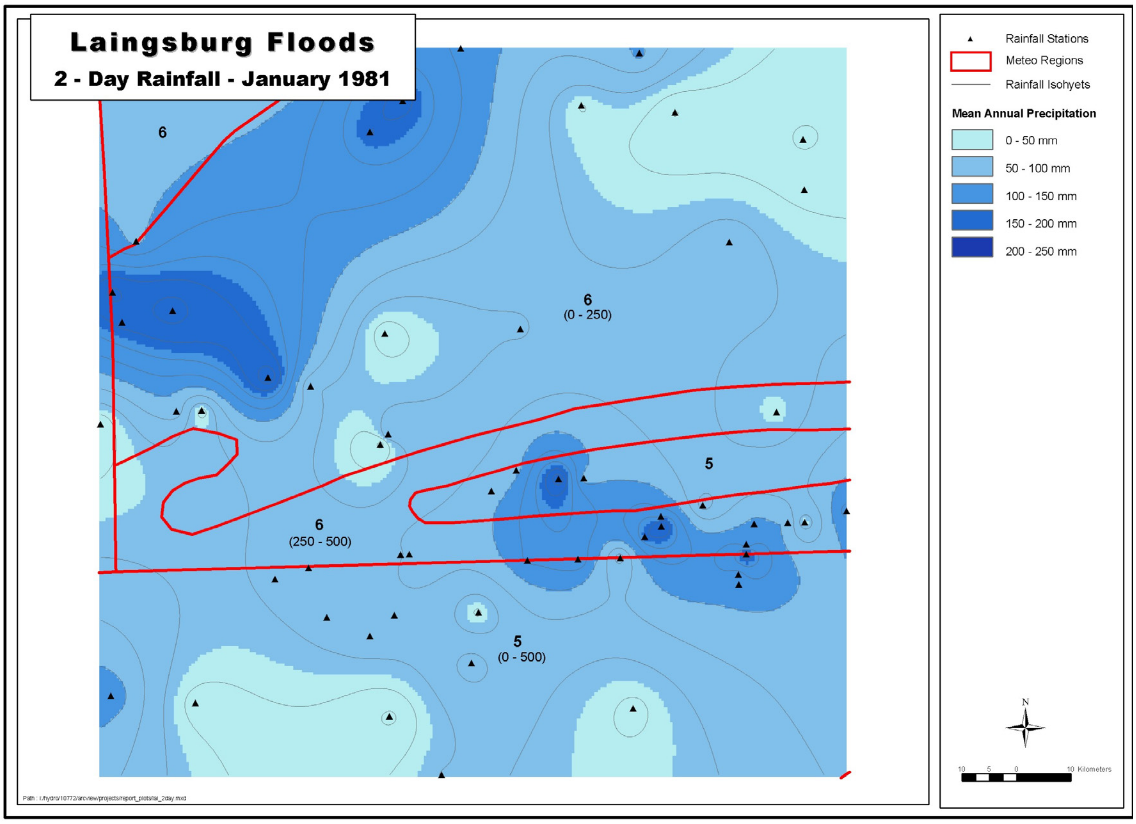

Figure 2.36 Laingsburg, January 1981, 2-day rainfall isohyets

Results

As can be seen from Figure 2.35 and Figure 2.36 the Laingsburg floods of 1981 fell over four HRU meteorologically similar regions, with no region being fully engulfed by the storm. Thus, the only suitable method of defining the boundary of the storm was to isolate storm cells and determine the area-average rainfall for the two considered durations, 1-day and 2-day rainfalls. The storm cell identified fell over HRU Region 6 (250 – 500 mm) and Region 5 (0 – 500 mm). The 1-and 2-day duration rainfalls for the Laingsburg Floods are plotted on the HRU PMP envelope curves for Region 6 (250 – 500 mm) in Figure 2.37 and on the HRU PMP envelope curves for Region 5 (0 – 500 mm) in Figure 2.38. In both instances the 1-and 2-day rainfalls plot well below their respective curves. This result was surprising, as the Laingsburg Floods of 1981 are well-known as one of the severe floods in South Africa; the extent of the flood damage was enormous, and the flood water levels high. Thus, it would have been reasonable to expect that, as the Laingsburg region has very low MAP (Figure 2.24), the HRU PMP envelope curves would be exceeded.

However, the aggravating circumstances that contributed to the Laingsburg flood need to be taken into account. Laingsburg is situated on a natural flood plain on the inside bend of the Buffels River, this flood plain formed through the deposition of sediment over a long period of geological time. It stands to reason that if the river flowed there in the past, one would expect the river, at some stage, to flow there again. Another contributing factor was that the confluence of two large tributaries, the Bobbejaans River and Wilgehout River with the Buffels River, occurs on the outskirts of the town (Roberts and Alexandra, 1982). Taking these aggravating factors into account, it is understandable that despite the severe nature of the actual flood event, the causative rainfall did not exceed the maximum probable precipitation of the area.

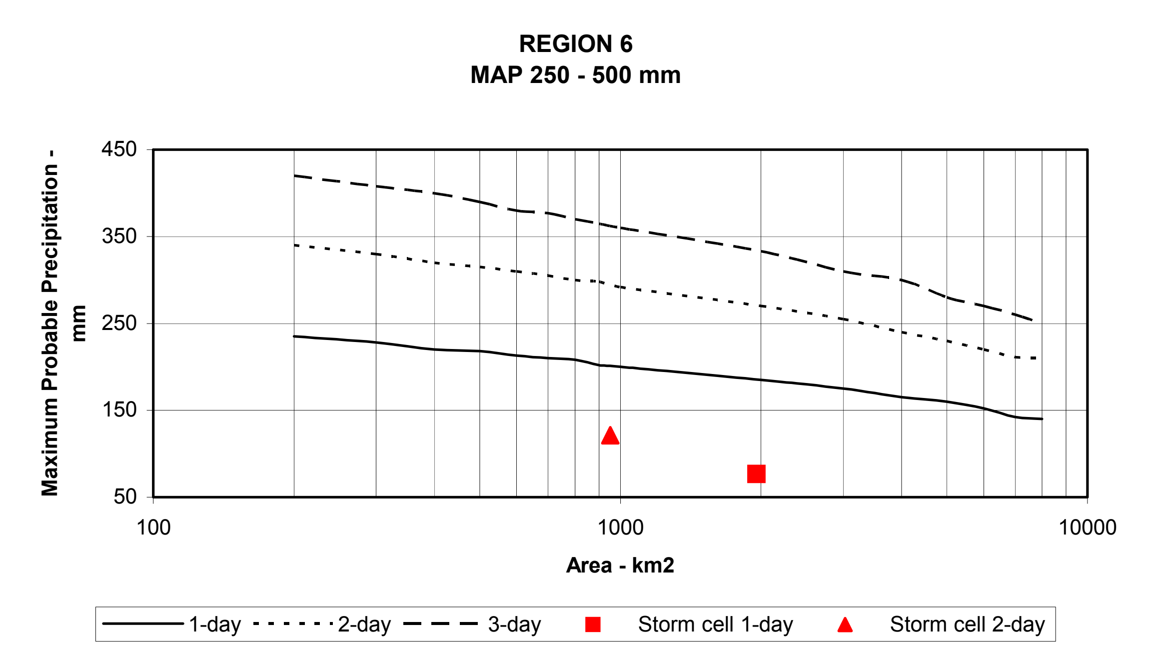

Figure 2.37 HRU PMP Region 6 (250 - 500 mm) envelope curves with 1-day, 2-day and 3-day duration storm rainfall for storm cells for the January 1981 storm

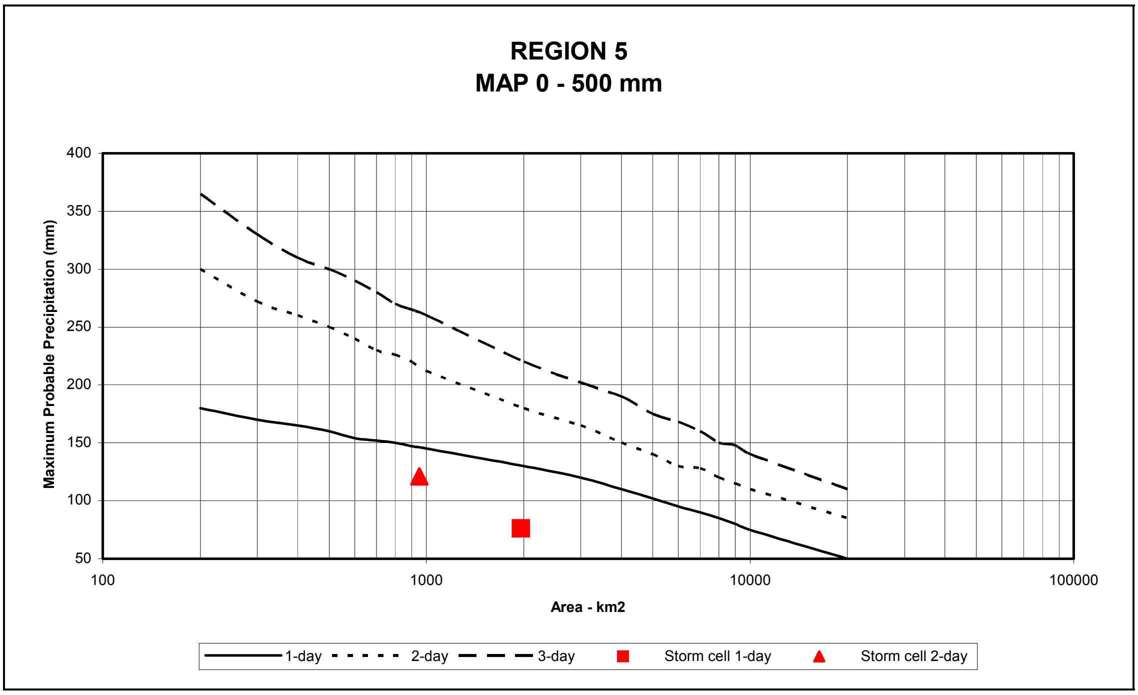

Figure 2.38 HRU PMP Region 5 (0 - 500 mm) envelope curves with 1-day, 2-day and 3-day duration storm rainfall for storm cells for the January 1981 storm

2.5.4 South Eastern Cape floods: March - May 1981

During the period January – May 1981, the south-eastern parts of the then Cape Province experienced four big storms. The March 1981 storm was the most destructive of these storms and also covered the largest area. Fourteen people lost their lives during the March storm. Details of the flood event were obtained from a Department of Water Affairs report, "Documentation of the March-May 1981 floods in the South Eastern Cape", which was compiled by Du Plessis (1984).

Storm Duration

The critical storm duration for this event was a 3-day duration, which stretched from the 24th to 26th of March. The highest 1-day rainfalls were experienced on the 25th of March, and the highest 2-day rainfalls on the 24th and 25th of March 1981.

Cause of Rainfall

The weather pattern responsible for the extreme rainfall experienced over the South Eastern Cape from 23rd to 26th March 1981 is a synoptic situation known as a cut-off low.

Storm Isohyets

Illustrated in Figure 2.39, Figure 2.40 and Figure 2.41, respectively, are the 1-day, 2-day and 3-day duration storm rainfall isohyets. Although the rainfall station network coverage is denser than the Laingsburg area, the coverage is still sparser than the KwaZulu-Natal and Orange River basin area.

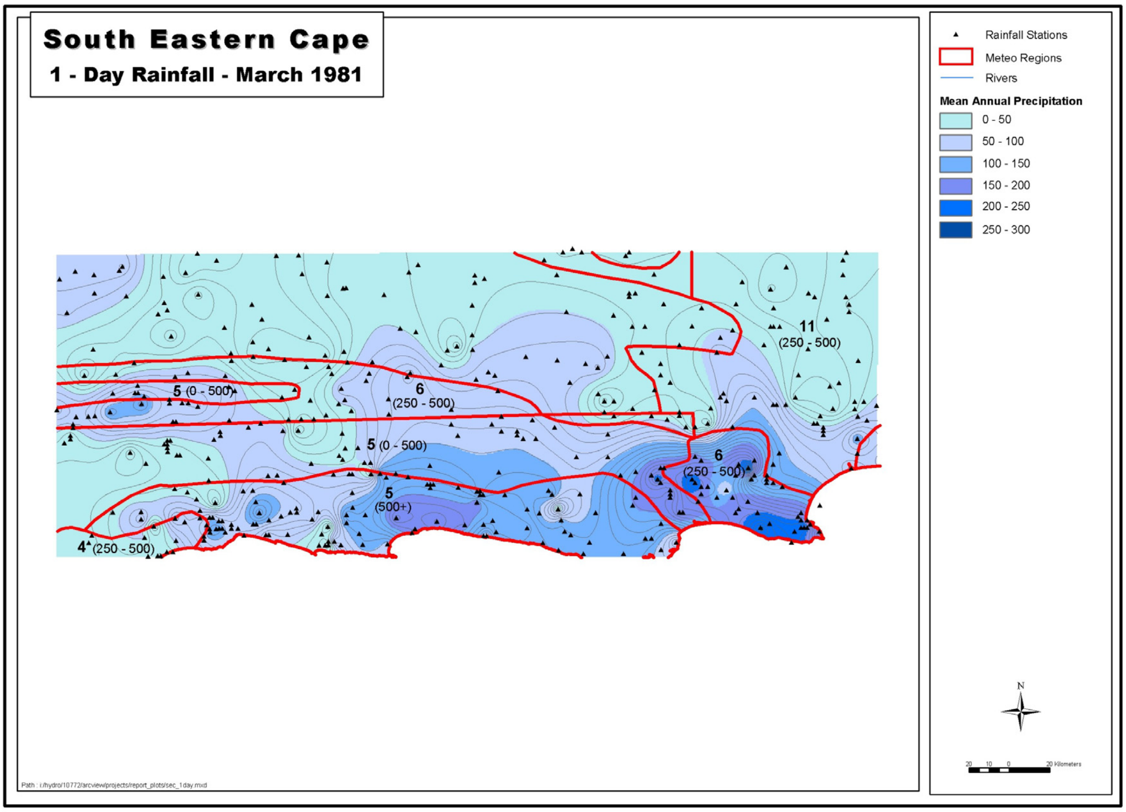

Figure 2.39 South Eastern Cape Floods, March 1981, 1-day rainfall isohyets

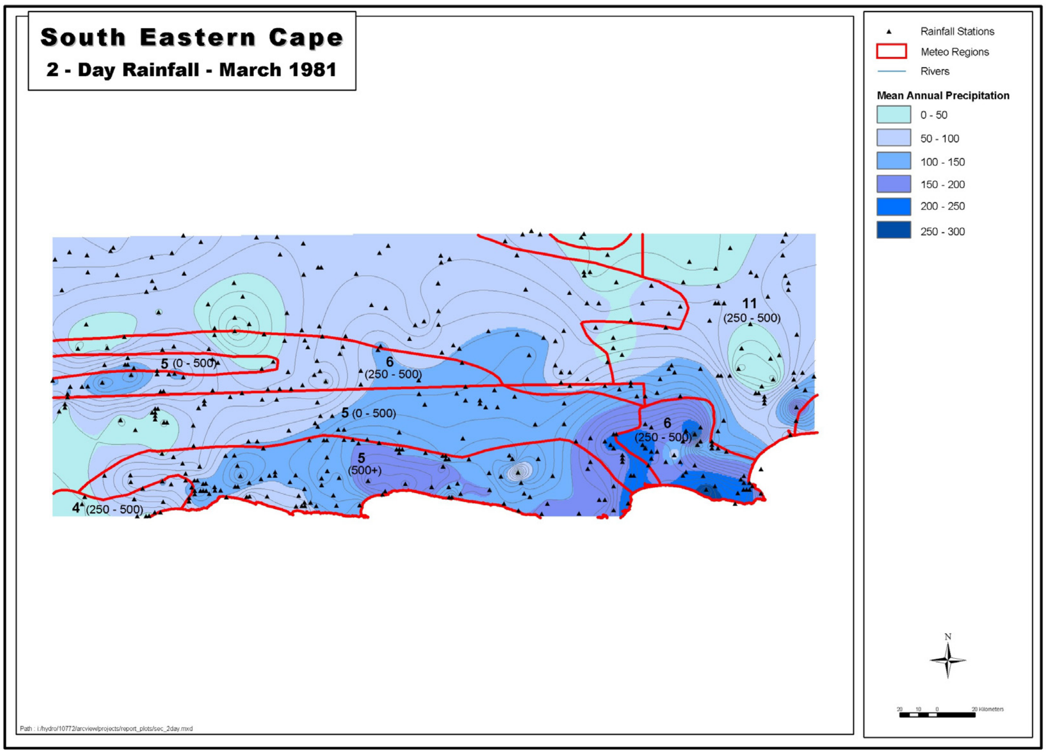

Figure 2.40 South Eastern Cape Floods, March 1981, 2-day rainfall isohyets

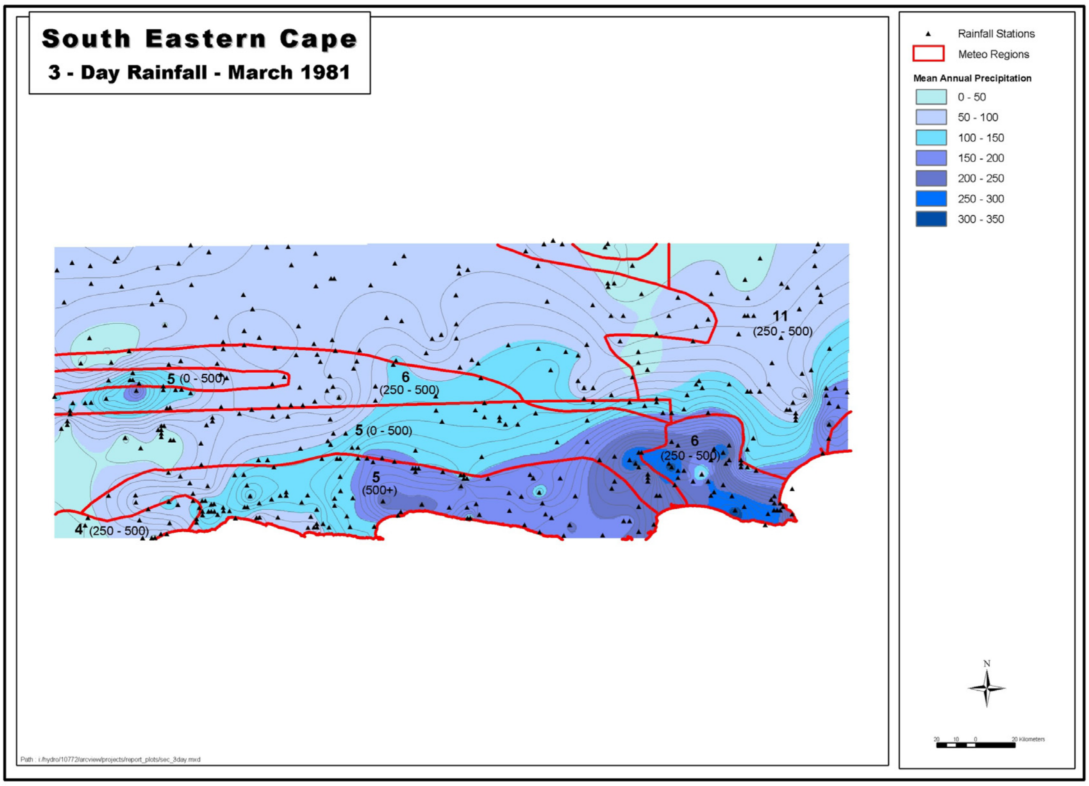

Figure 2.41 South Eastern Cape Floods, March 1981, 3-day rainfall isohyets

Results

As can be seen from Figure 2.39, Figure 2.40 and Figure 2.41 the storm rainfall covered a number of areas, completely engulfing HRU Region 6 (250 – 500 mm) situated along the coast. This region was in the centre of the storm; it experienced the peak rainfall and highest intensity rainfall. Thus it was decided that the only method of determining the storm boundary would be to consider the area of the storm bounded by HRU Region 6 (250 – 500 mm), as this would encapsulate both other methods, i.e. examining storm cells and taking a proportion of the peak storm rainfall.

Figure 2.42 shows the area-average storm rainfall for 1-day, 2-day and 3-day durations plotted on the HRU PMP envelope curves for Region 6 (250 – 500 mm). The 1-day storm rainfall plots exactly on the HRU 1-day PMP envelope curve, however, the 2- and 3-day storm rainfalls plot substantially below their respective HRU PMP envelope curves.

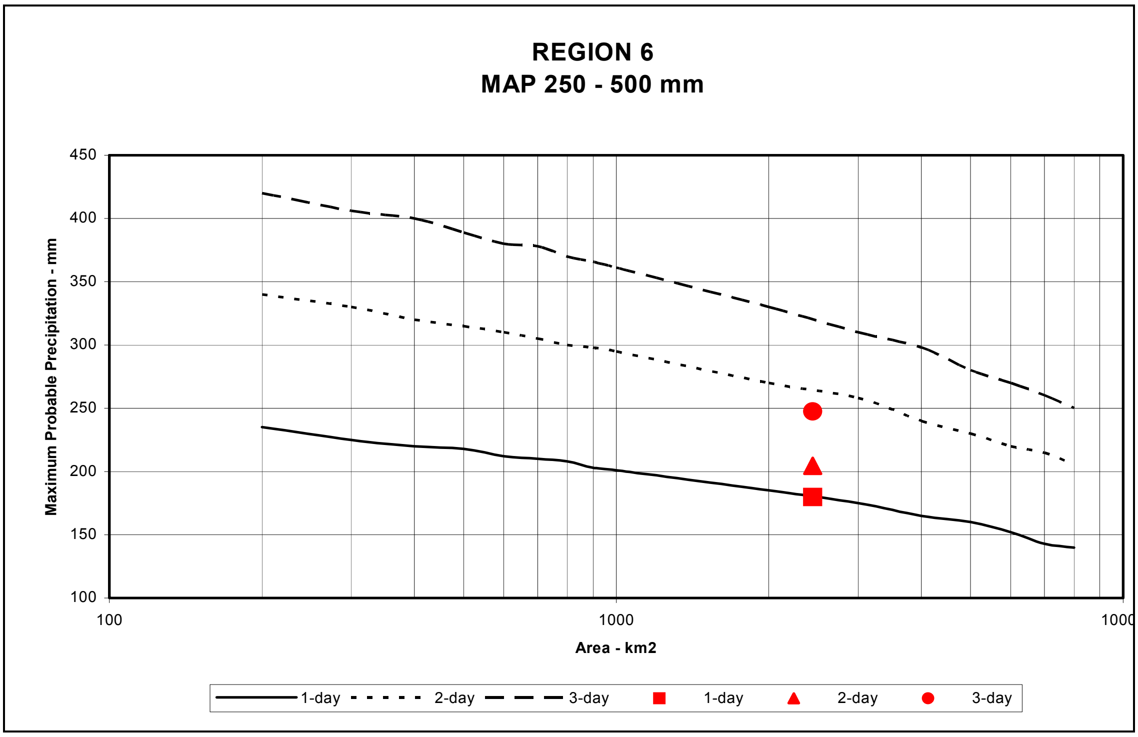

Figure 2.42 HRU PMP envelope curves with 1-day, 2-day and 3-day duration storm rainfall for Region 6 (250 - 500 mm) for the March 1981 storm

2.6 SMALL-AREA STORMS: A PRELIMINARY INVESTIGATION

The HRU Report 1/69 attempted to maximise the short-duration point rainfall through the same methods used to maximise large-area storms; however, this was found to be impossible due to the lack of adequate meteorological observations during short-duration storms. Instead, an experience diagram was drawn comprising of envelopes of the highest point precipitation for various storm durations observed in different areas of the country, and thus the country was divided into regions, each with their own maximum point rainfall envelope curve (HRU Figure C.4). An envelope curve for the entire country was also obtained, together with a comparative envelope of world rainfall records.

As a preliminary attempt of establishing the suitability of the small-area storms for current day design, the maximum point rainfall observed in the storms identified above for various durations were plotted against the respective curves. It must be noted that the durations plotted are 1-day up to 4-day rainfall, while the duration scale on the curves is in hours. The difference between daily and 24-hour rainfall is conventionally believed to be 10 – 11%; however, on the compressed scale of the HRU the difference is negligible. Table 2.6 presents the HRU extreme point rainfall regions the above-identified storms fall into, as well as the peak point rainfall that occurred. The 1-day rainfall is considered as 24-hour rainfall, the 2-day and 3-day rainfalls as 48-hour and 72-hour rainfalls, respectively, and will hereafter be referred to as such.

Table 2.6 Point rainfall values for identified storms and their corresponding extreme point rainfall regions, according to HRU Figure C.3

Flood	Year	HRU Region	1-day/24-hour Rainfall	2-day/48-hour Rainfall	3-day/72-hour Rainfall
Cyclone Domoina	1984	2	615	840	900
KwaZulu-Natal Floods	1987	2	566	698	875
Orange River Basin Floods	1988	3	300	381	
Limpopo Floods	2000	4		406	
South Eastern Cape Floods	1981	4	274	298	336
Laingsburg Floods	1981	5	146	205	221

In Figure 2.43 below, the point rainfall for Cyclone Domoina and the KwaZulu-Natal Floods are plotted against the maximum extreme point rainfall curve as well as the curve for Region 2. These curves are similar, and only after a duration of 200 hours do the curves separate. For the 72-hour rainfall, the point rainfall experienced in both Cyclone Domoina and the KwaZulu-Natal floods exceeds the maximum extreme point rainfall curve, as does the 48-hour point rainfall for Cyclone Domoina, while the 48-hour point rainfall for the KwaZulu-Natal Flood plots on the curve. The 24-hour rainfalls for both storms approach the maximum extreme point rainfall curve.

In Figure 2.44 the point rainfall for the Orange River Basin Floods are plotted against the maximum extreme point rainfall curve as well as the curve for Region 3. These curves separate at duration of 0.6 hours. The 24-hour point rainfall that occurred during the Orange River Basin Floods of 1988 plots below the extreme point rainfall curve for Region 3, whereas the 48-hour point rainfall approaches the curve.

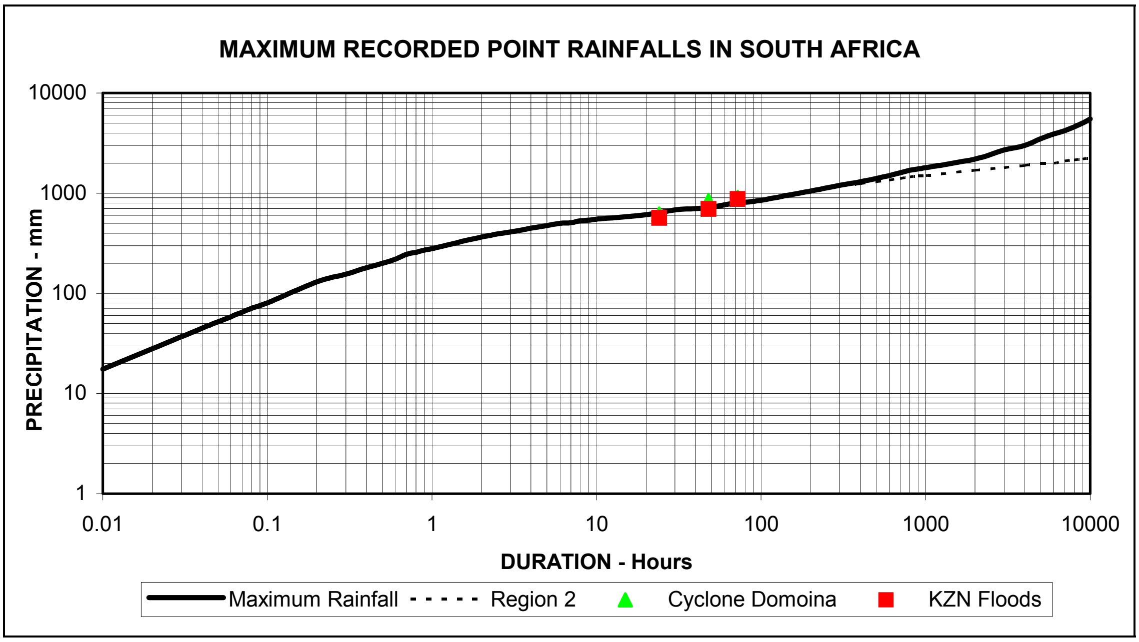

Figure 2.43 **Maximum point rainfalls for Cyclone Domoina and KwaZulu-Natal Floods plotted against Small-area storm curves for Region 2 and the Maximum Point Rainfall envelope curve**

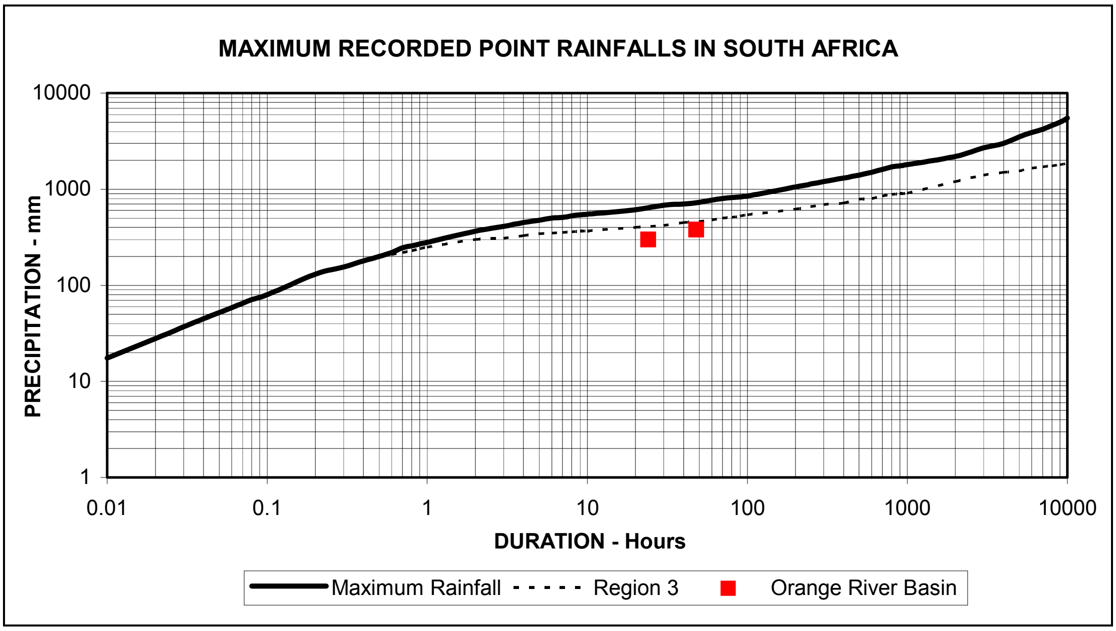

Figure 2.44 **Maximum point rainfalls for Orange River Basin Floods plotted against Small area storm curves for Region 3 and the Maximum Point Rainfall envelope curve**

Shown in Figure 2.45 are the point rainfall for the Limpopo Floods of 2000 and the 1981 South Eastern Cape Floods, with the maximum extreme point rainfall curves and the point rainfall curves for Region 4. The curve for Region 4 separates from the maximum extreme point rainfall curve at a duration of 0.6 hours, and continues with a lower point rainfall.

Only the 48-hour point rainfall for the Limpopo floods is plotted against the curves. This point rainfall slightly exceeds the curve for Region 4. For the South Eastern Cape, point rainfalls of 24-, 48- and 72-hour durations are plotted; these point rainfalls do not exceed the curve for Region 4.

Lastly, in Figure 2.46, the point rainfall experienced during the Laingsburg Floods in 1981 is shown with the maximum extreme point rainfall curve, as well as the curve for Region 5. In comparison to the other regions, the point rainfall curve for Region 5 is completely different from the maximum extreme point rainfall curve at all durations, and follows a lower point rainfall track. The three point rainfalls considered are the 24-, 48- and 72-hour durations. The 24-hour point rainfall plots well below the curve for Region 5. The 48-hour point rainfall plots below the curve, while the 72-hour point rainfall approaches the point rainfall curve for Region 5.

It seems from the above results that as the duration of the point rainfall increase, as does the possibility that the curve for the respective region may be exceeded. With the results showing the maximum extreme point rainfall curve to have been exceeded in three instances, and the regional curves approached or exceeded in four instances, the need to further investigate the curves for small-area storms has been highlighted.

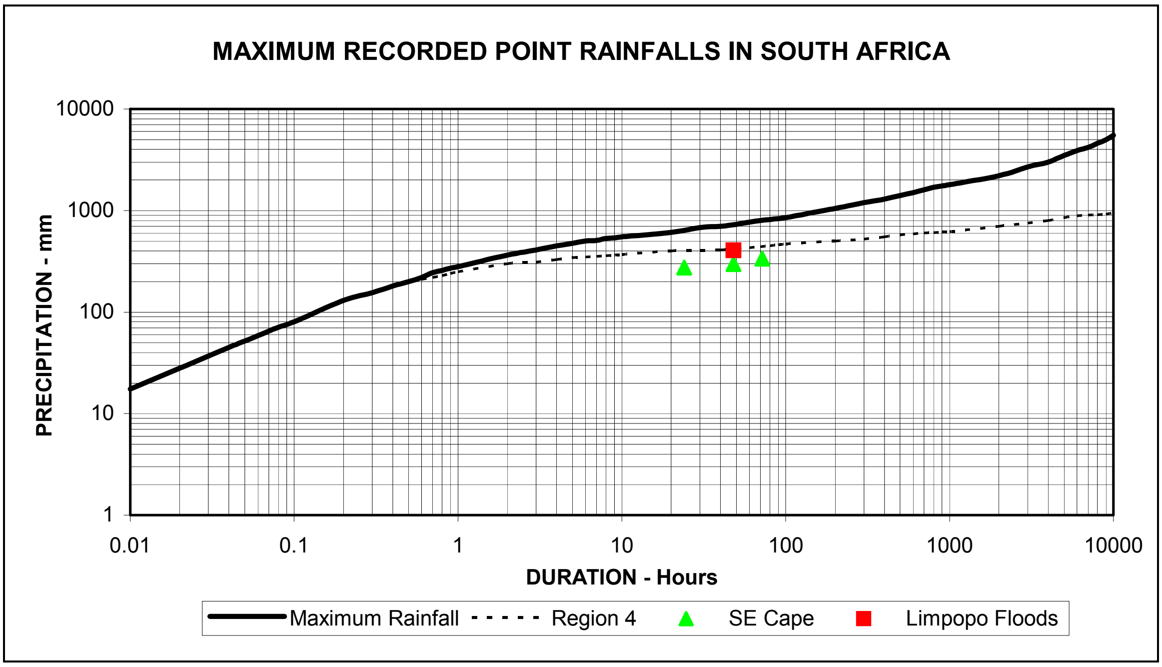

Figure 2.45 Maximum point rainfalls for South Eastern Cape Floods and Limpopo Floods plotted against Small-area storm curves for Region 4 and the Maximum Point Rainfall envelope curve

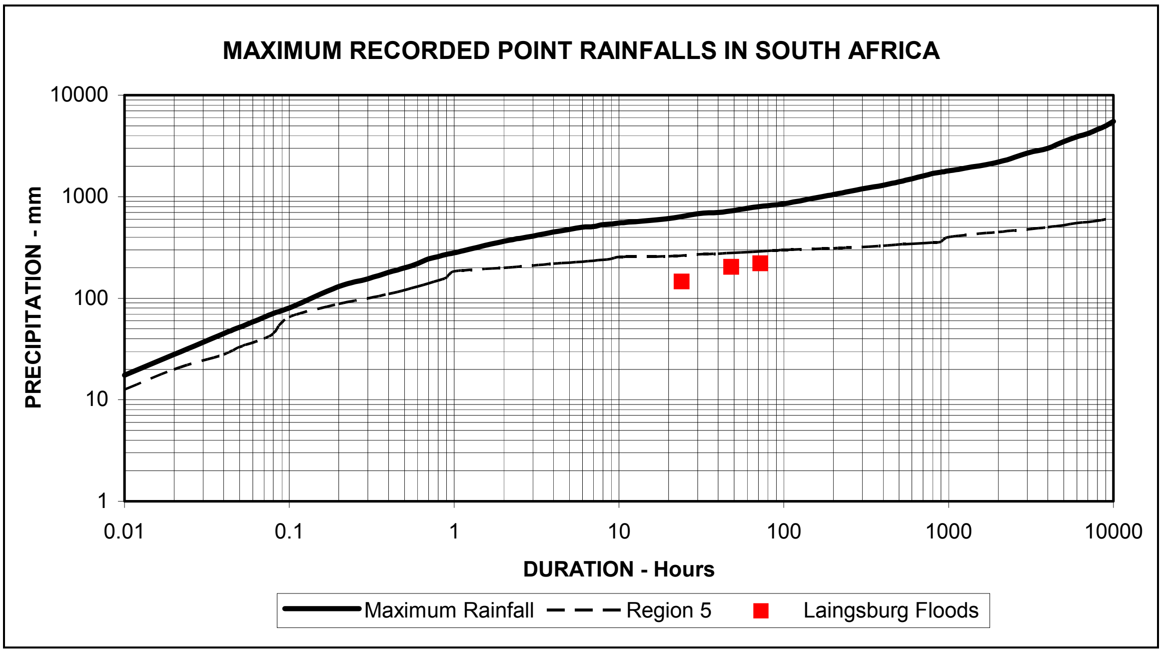

Figure 2.46 Maximum point rainfalls for Laingsburg floods plotted against Small-area storm curves for Region 5 and the Maximum Point Rainfall envelope curve

2.7 DISCUSSION AND CONCLUSION

The results obtained in the above analysis are summarized in Table 2.7. A tick in Table 2.7 indicates that the HRU PMP envelope curves were exceeded in that instance while a cross indicates that they were not exceeded. As can be seen, the HRU PMP envelope curves were exceeded on a number of occurrences. In particular, the KwaZulu-Natal floods of 1987 stand out. These floods were the result of a cut-off low, which is a fairly commonly occurring South African weather system.

The number of crosses in Table 2.7 is slightly deceiving, in that although the HRU PMP curves were not exceeded, the storm rainfalls were approaching the curve. The Limpopo floods of 2000 are an example of this, as are the Orange River basin floods in 1988.

These results suggest that the HRU PMP envelope curves for Large-Area storms may be underestimating the maximum precipitation which could occur in a number of regions in South Africa, particularly considering that the HRU PMP envelope curves were developed from maximised and transposed storms[1], while the storms used in this research have not been maximised or transposed.

[1] To recap, storm maximisation is the upward adjustment of observed extreme rainfall, based on the assumption that at least one storm over the area of concern will operate at maximum efficiency, i.e. that a maximum amount of moisture will be contained within the storm, and that the conversion of this moisture to precipitation will occur at a maximum rate (WMO, 1986). Storm transposition implies the displacement of the characteristics of the storm from its original location to the location of the study area as if the storm could have occurred there (Bureau of Meteorology Australia, 2003).

Table 2.7 Summary of results obtained from analysis to determine current applicability of HRU PMP envelope curves for Large-Area storms

Flood	Year	HRU Region affected	METHOD USED TO DETERMINE STORM BOUNDARY									
			HRU Region				Proportion of peak			Storm Cells		
			1-day	2-day	3-day	4-day	1-day	2-day	3-day	1-day	2-day	3-day
Cyclone Domoina[2]	1984	Region 17 (500 – 1000 mm)				✓						
Limpopo	2000	Region 18 (250 – 500 mm)		✓		✕						
				✕								
		Region 18 (500 – 1000 mm)		✕		✕						
				✕								
Orange River Basin	1988	Region 10 (250 – 500 mm)								✓	✕	
										✕	✕	
KwaZulu-Natal	1987	Region 13 (1 000 mm+)	✓	✓	✓		✓	✓	✓	✓	✕	✕
Laingsburg	1981	Region 5 (0 – 500 mm)								✕	✕	
		Region 6 (250 – 500 mm)								✕	✕	
South Eastern Cape	1981	Region 6 (250 – 500 mm)	✓	✕	✕							

[2] Cyclone Domoina was not analysed by covering a complete HRU region, however, the method of analysis is closest to this method.

2.8 RECOMMENDATIONS

The HRU PMP envelope curves are based on storms extracted from 30 years of rainfall record, spanning the period 1932 to 1961. Since then, there have been a number of extreme rainfall events that have occurred over South Africa. The results shown above suggest that the HRU PMP curves no longer represent the maximum rainfall that can be experienced in regions of South Africa. However, the conclusion drawn from the literature review was that the HRU methods were, at the time, aligned with the best conceptual approaches internationally, and are still aligned with sound present day methods. Our recommendation, from the research undertaken, is to modernise the HRU PMP envelope curves to include a longer and more current portion of rainfall data. New research should address the following:

i. The HRU 1/69 report states that due to inaccuracies in the estimation of moisture content, and the inability to take account of possible atmospheric inversion, the PMP-area curves as drawn for each sub-region, may be subject to errors in the order of 25%.
ii. Inclusion of a longer period of rainfall data, and a more extensive rainfall gauge network.
iii. Improved methods and replication of drawing isohyets through the use of utilities that form part of Geographical Information Systems.
iv. Further investigation into the applicability of the Extreme Point rainfall curves for Small-Area storms to be undertaken.

With warnings being sounded from the climate change community of an increasing number and magnitude of extreme events in the future, the revision of the HRU PMP envelope curves would be timeous.

3. REVIEW OF REGIONAL FUNCTIONS OF DESIGN STORM LOSSES IN SOUTH AFRICA

3.1 INTRODUCTION

Catchment characteristics and processes are important considerations for the determination of the design flood discharge. This is especially true when applying a deterministic approach for design flood estimation, such as the commonly employed Unit Hydrograph Method. The reason for this is that the catchment properties have an important effect on the rainfall-runoff process in terms of the 'storm rainfall losses' that may be incurred. Storm rainfall losses occur as the catchment experiences a change in storage, while it absorbs (dependent on the infiltration rate of the soil), retains or delays (surface, near-surface and river bank detention) and loses (evaporation and groundwater seepage) some of the rainfall. Therefore, not only will storm losses dictate the design flood magnitude and the shape of the resulting flood hydrograph, but they can also have an impact on the recurrence interval (RI) associated with the resulting design flood. In arid areas, for example, where evaporation losses are high and the antecedent catchment moisture low, one could expect the RI of the resulting flood to be lower than that of the causative rainfall, in comparison with humid areas where the RI of the flood and its causative rainfall can be expected to be similar.

A generalised South African approach for the estimation of storm rainfall losses based on regionalised functions is given in the famous "design flood handbook" (HRU Report 1/72) published in 1972 by the Hydrological Research Unit (HRU) of the University of the Witwatersrand (HRU, 1972) as part of their proposed Unit Hydrograph Methodology. Here storm rainfall losses are presented as a percentage of the total storm rainfall and expressed as a function of Veld-Zone and catchment size. The HRU methodology was based on 30 years of rainfall data from 1932 to the 1960s. The SCS method, developed by Schmidt and Schulze (1987) for South African application from the USDA Soil Conservation Services SCS methodology, also considers storm rainfall losses, but is only applicable to small catchments (originally developed for catchments < 8 km^2), although many practitioners report its satisfactory use for larger catchment of up to 80 km^2. These methods, as well as at-site investigations performed by the Department of Water Affairs and Forestry, are presented under Section 3.3 of this report. To put the South African methodologies for the estimation of design storm rainfall losses into context, various international approaches are also presented. The practices adopted in the United States, Australia and the United Kingdom are considered in detail in Section 3.4.

Section 3.5 of this document presents a preliminary review of the representativeness of the HRU regionalised storm loss functions in the light of relevant storm rainfall and flood volume data assembled in this Study, as well as in recent national studies by Smithers and Schulze (2002) and Lynch (2004).

3.2 STORM RAINFALL-RUNOFF PROCESSES

Storm rainfall losses are assessed in order to determine the excess storm rainfall or runoff that will contribute to the flood hydrograph. The classical concept of storm runoff on which all seminal deterministic design flood methodologies of the 20th century were based is that of "Hortonian flow". This concept, formulated by R. E. Horton in 1933, is described as overland runoff produced by rainfall excess which occurs at the ground surface when the rainfall intensity exceeds the soil's infiltration capacity. Horton assumed this overland flow to be the sole contributor to the production of the hydrograph peak, and that all rainfall that infiltrated would pass into groundwater and be the sole contributor to the baseflow part of the hydrograph (Amerman and McGuinness, 1967). Also in the earlier hydrological design flood practice, infiltration capacity was often perceived to be uniform across the whole catchment, so that the excess rainfall and, therefore, runoff volume is constant with area (ARR87).

Since the 1970s, an understanding developed that two other storm runoff mechanisms are as important as the Hortonian concept of runoff, namely "saturated overland flow" and "throughflow". Saturated overland flow occurs when some sub-areas in a catchment are partly saturated even though the local infiltration capacity has not been exceeded by the rainfall intensity. This can occur when the upper soil horizon on parts of the catchment becomes saturated, either as a result of a build-up of water above a soil layer of lower hydraulic capacity, or where the water table rises to the surface (ARR87). Rainfall on these saturated areas of the catchment can therefore result in 100% runoff, whereas the non-saturated parts might contribute little to the ensuing flood hydrograph. This process is naturally quite prevalent in riparian zones around streams which are nearly permanently moist due to upland subsurface flows that converge towards the stream channel. The second storm runoff mechanism, throughflow, occurs when rainfall that infiltrates into the soil moves laterally through the upper soil horizons towards the stream channel, usually due to the development of saturation above a soil layer of low hydraulic capacity. Throughflow differs from other subsurface flow in that it can reach the channel quick enough to contribute to the storm hydrograph. Another storm runoff mechanism worth mentioning at this point for completion sake is that of "channel runoff". This runoff refers to when rain falls on the flowing stream. Although this runoff appears in the hydrograph at the start of the storm and continues throughout the storm, its contribution to the hydrograph is generally negligible because of the small area involved and so is usually ignored (NEH, 2004).

In reality, we now understand that storm runoff from a catchment is a variable mix of Hortonian overland flow, saturation overland flow and throughflow that originate from an infinite number of source areas. The nature of the storm runoff that occurs within a catchment is strongly related to the total storm rainfall and the instantaneous rainfall intensity pattern in space and time, as well as the following catchment characteristics, which ultimately determine the magnitude of losses during a storm:

- Soil type/ depth distribution
- Slope of primary channels
- Slope distribution of landscape elements
- Drainage path lengths and catchment shape
- Drainage density
- Natural land cover distribution
- Land use distribution
- Antecedent soil moisture conditions
- Catchment size

3.3 SOUTH AFRICAN APPROACHES TO DETERMINATION OF STORM LOSSES

3.3.1 HRU 1/72 methodology

The HRU 1/72 document presents two approaches for the estimation of storm rainfall losses. The one approach considers "minimum losses", which would typically be deducted from extreme rainfall events, such as the Probable Maximum Precipitation (PMP), to determine the extreme flood hydrograph (Probable Maximum Flood hydrograph). The second approach considers "average losses" to be deducted from the design total storm input of a given recurrence interval to yield a runoff that would result in a flood of the same or similar recurrence interval. The two approaches are described in detail below.

Minimum losses approach

The HRU undertook an empirical study where the ratios of observed runoff to total storm rainfall for select extreme rainfall events across South Africa were plotted against catchment area. Envelope curves were then fitted to these values of observed runoff percentage and can be viewed in Figure 3.1. The outer envelope curve, labelled "Estimate of Maximum Runoff Efficiency" was included in the plot to provide the user with a more conservative estimate representative of an extreme event in terms of providing a higher runoff percentage for a particular catchment area. This was included due to the concern expressed by HRU over the small data sample available at the time of analysis.

By considering the outer envelope curve in Figure 3.1, it is evident that for catchments of about 1000 km^2 and smaller, 100% of the storm rainfall gets converted to runoff, in other words, there are zero storm rainfall losses. As the catchment area increases from 1000 km^2, the percentage of storm rainfall that is converted to runoff decreases, for example only 50% of the storm rainfall is converted to runoff for a catchment area of about 170 000 km^2.

Average losses approach

As a tool for determining storm rainfall loss values associated with design storm rainfall of a given RI, the HRU produced regional curves (Figure 3.2) based on Veld-Zones from which percentage runoff/loss values can be determined for known values of design storm areal rainfall (mm) and catchment area (km^2).

To determine these regional curves, the HRU considered the 96 catchments for which they had developed Unitgraphs of suitable critical duration. For each catchment, flood peaks corresponding to 1:5-, 1:10-, 1:20-, 1:50- and 1:100-year recurrence intervals (RIs) were determined from a co-axial regional flood peak probability diagram (our Figure 3.3, copy of Figure B2 in HRU 1/72). The flood peak for a particular RI was determined via the particular homogeneous flood region, as well as the relevant catchment area size. The HRU then estimated the corresponding runoffs in millimetres (i.e. the flood hydrograph volumes) associated with the selected RIs according to the following logic:

In line with general Unitgraph conventions, the Unitgraphs for all of the above 96 catchments were derived for their individual critical storm durations from a sample of observed large-volume hydrographs regarded by the HRU team as "representative" of each catchment. This group of hydrographs were converted into hydrographs of one mm volume each, i.e. the so-called "unit" hydrograph, by dividing their ordinates by their respective total flood volumes in mm, according to the convention of linear proportionality between design flood hydrographs and their underlying unitgraphs. By applying this convention in reverse, i.e., by expressing the value of the above RI flood peaks determined from the regional probability diagram in Figure 3.3 as ratios of their respective unitgraph peak values, the corresponding flood hydrograph volumes (in mm) associated with the selected RIs could be calculated.

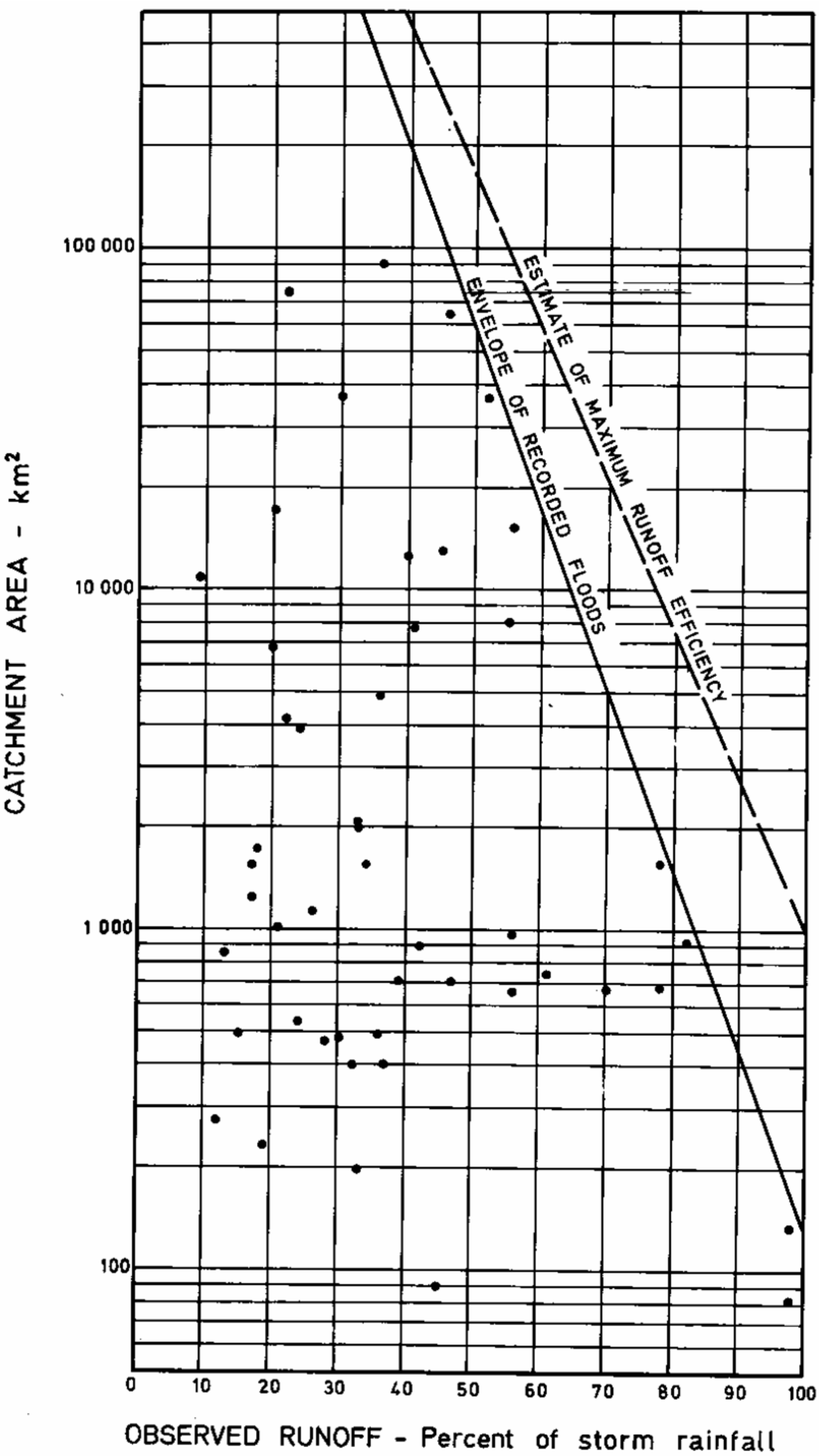

Figure 3.1 Minimum storm losses envelope curves (HRU 1/72 Figure G1)

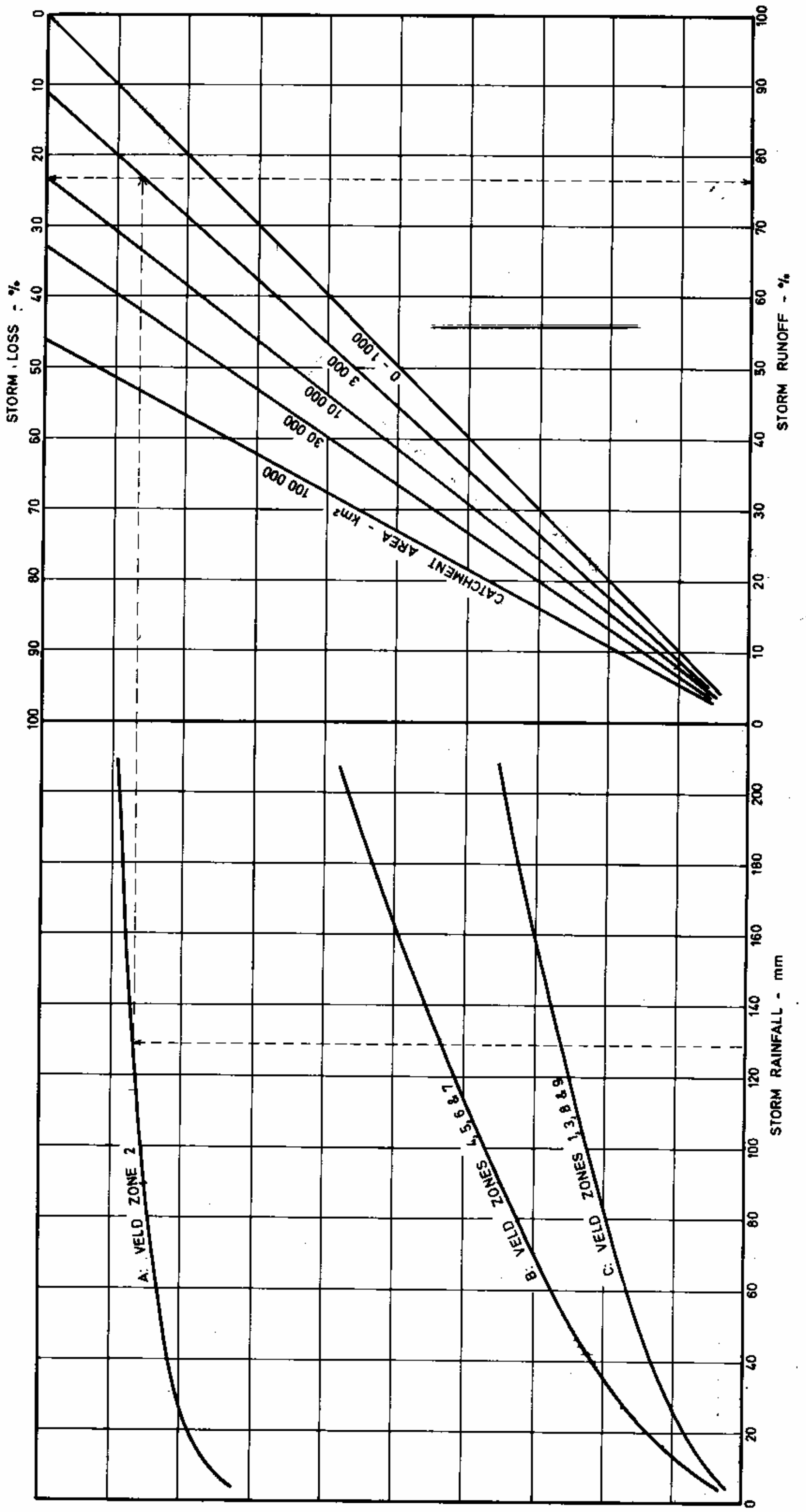

Figure 3.2 Mean Storm Losses (HRU 1/72 Figure G2)

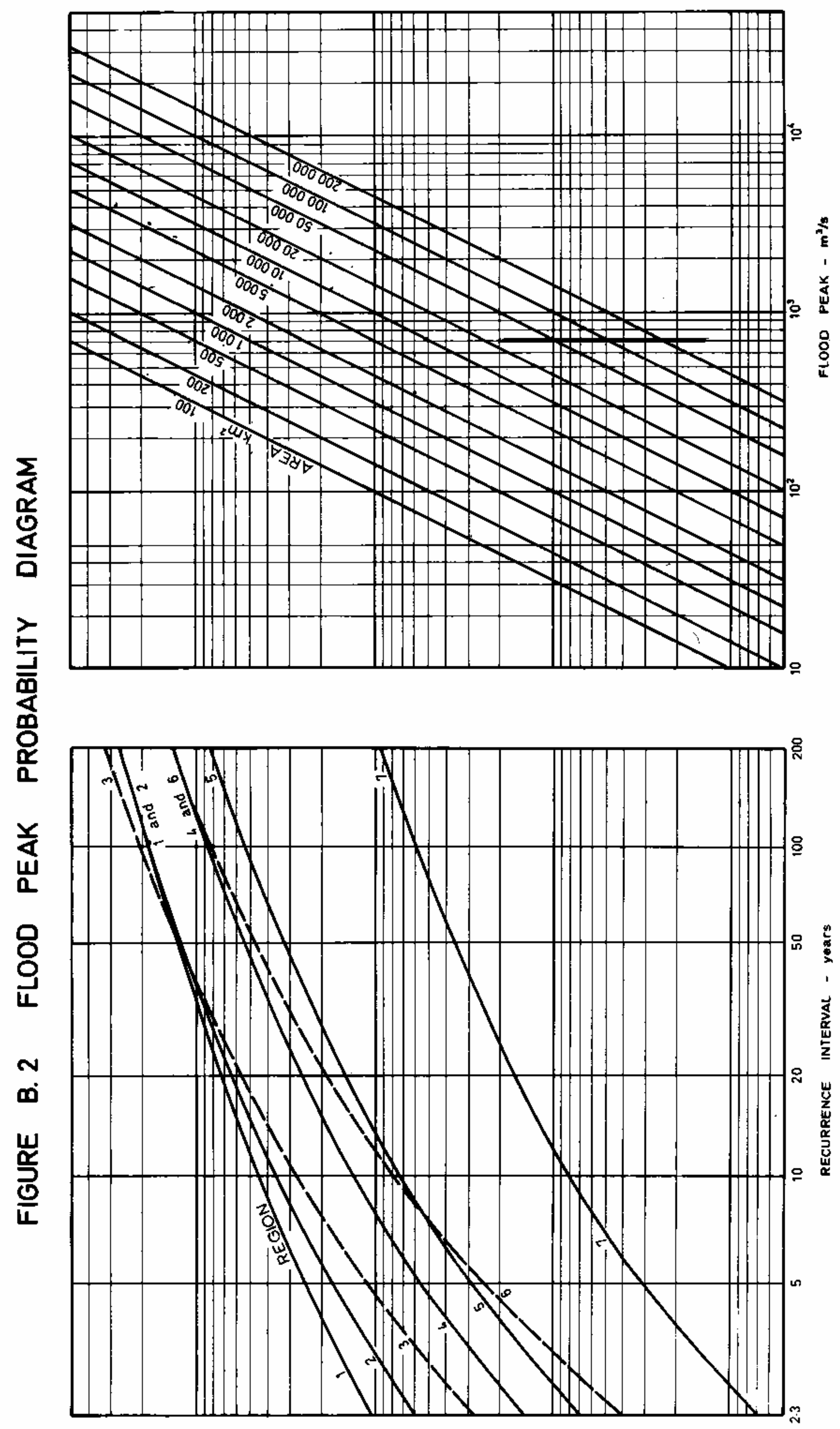

Figure 3.3 Flood Peak Probability Diagram (HRU 1/72 Figure B2)

For the derivation of the regional storm rainfall loss curves, the HRU then determined the storm rainfall associated with each catchment's set of calculated values of RI-based storm runoff. To accomplish this, diagrams C3 (depth-duration–frequency diagram for small-area storm analysis, i.e. for storm durations of up to 24 hours) and D2 to D28 (depth-area-duration-frequency curves for large-area storm analysis, i.e. for storm durations of 1 day or greater) of the HRU 1/72 report were used to determine the design rainfall associated with each of the RIs and critical unit durations. The difference between this design storm rainfall and the associated runoff volume calculated earlier therefore gave the desired catchment storm rainfall losses associated with the selected design flood events.

For each catchment, losses were plotted against storm rainfall and catchment area, which lead to the derivation of the average curves as shown in Figure 3.2.

Since the publication of HRU 1/72, further rainfall data was processed by the HRU, which lead to the update of co-axial point rainfall diagram Figure C3. This figure was published in HRU 2/78, referenced as Figure 4. In 1986, Lahmeyer Macdonald Consortium and Olivier Shand Consortium undertook a hydrological study for the Lesotho Highlands Water Project (Lahmeyer et al., 1986). As part of this study, the HRU Unitgraph methodology was adopted for the assessment of the flood risk associated with floods of lesser recurrence interval than the PMF. As part of their analysis, the validity of the HRU 1/72 storm loss curves was checked using the updated rainfall information now made available in HRU 2/78. Although this assessment was only performed for Veld-Zone 4, this study provided a warning to hydrological practitioners, as, although the scatter of points was quite large, it was clear that the loss curve for Veld-Zone 4 plotted higher than that given in Figure G2 of HRU 1/72 (see Figure 3.4). This also indicated the need to review the validity of the HRU 1/72 Figure G2 for the calculation of average storm losses for South Africa.

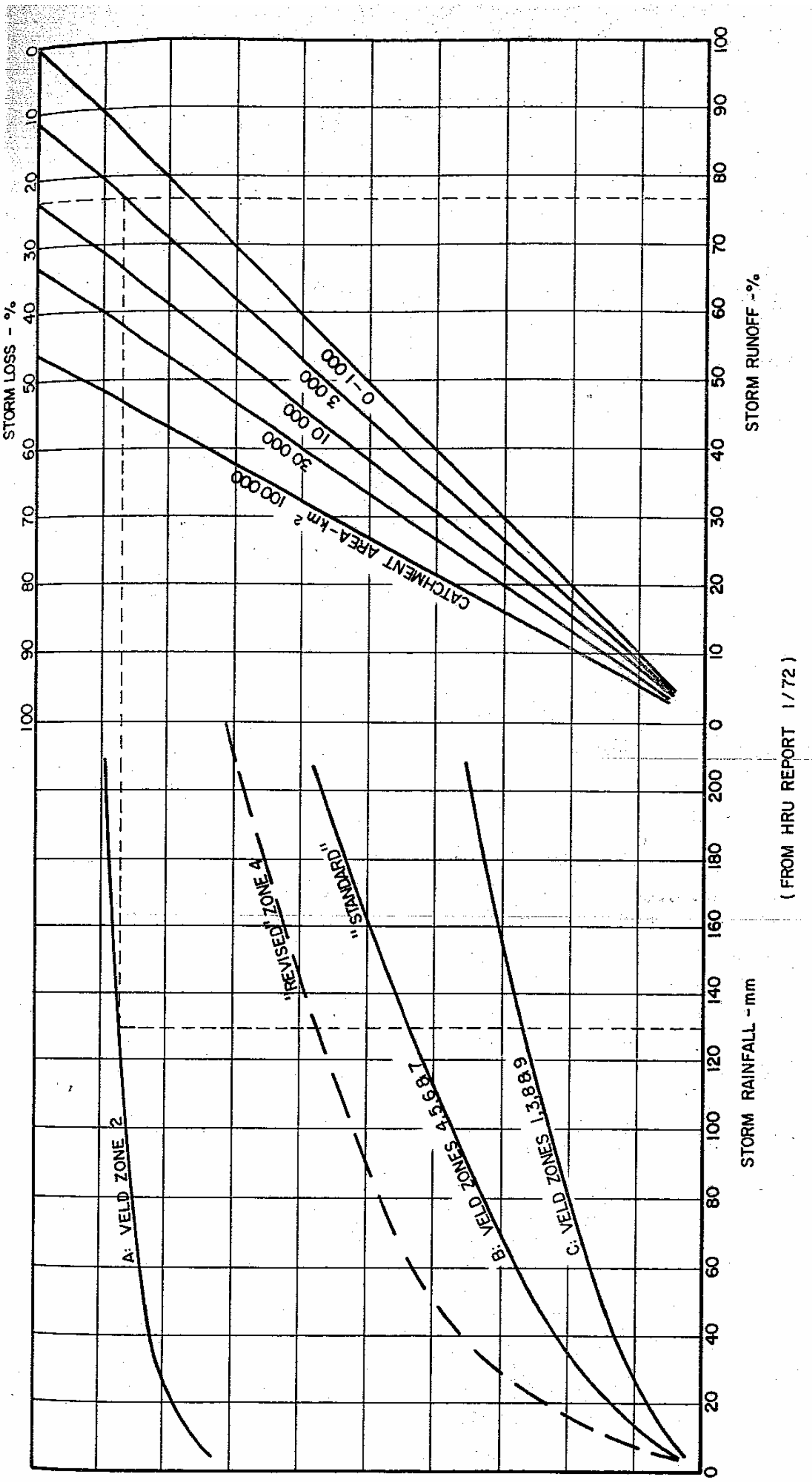

Figure 3.4 Revised and Standard HRU 1/72 Loss Curves as presented in the Lesotho Highland Water Project Hydrological Study

3.3.2 South African SCS methodology

The SCS (USDA Soil Conservation Services) model is a deterministic method for the computation of the daily runoff depth and discharge hydrograph for small catchments up to 8 km^2. This model was adapted, from the US Department of Agriculture's original methodology (see Section 3.4.1), for application in Southern Africa by Prof Roland Schulze's research team at the University of KwaZulu-Natal between the late 1970's and mid-1980s.

For South African application, the basic SCS equation is the following:

$$Q_t = (P_t - 0.1S)^2 / (P_t + 0.9S) \tag{3.1}$$

where Q_t is the runoff depth in mm for a T-year storm; P_t is the 24 hour design rainfall in mm; and S is the potential maximum retention in the catchment.

The storm rainfall losses component of the equation is incorporated within the calculation of the potential maximum retention, S, of the catchment. The term 0.1S of the above equation represents the "threshold" that the rainfall has to exceed before runoff can commence. S is expressed in terms of a "Curve Number", CN, through the relationship:

$$S = 25400/CN - 254 \tag{3.2}$$

where the CN is determined by considerations of soil properties, land use conditions and antecendent moisture conditions of the catchment. The CN transforms S (mm) into a dimensionless number between 0 and 100.

Although the SCS methodology for the calculation of storm rainfall losses is more detailed in terms of the soil properties and land-use of individual catchments than the other deterministic methods for flood hydrograph estimation, this methodology is not as widely used in dam safety hydrology as, say, the HRU Unit Hydrograph methodology owing to its limitation to small catchments. Furthermore, as for the USA SCS approach outlined in Section 3.4.1, the RSA SCS methodology does not require the development of large-scale regional functions of design storm losses.

3.3.3 Department of Water Affairs and Forestry technical reports

The Department of Water Affairs and Forestry produced a number of technical reports on severe floods that have occurred within South Africa since the publication of the HRU 1/72 report. Within each report a description of the process followed for the calculation of the storm rainfall losses is given. Two examples of these processes are described below for the following selection of severe flood events:

The 1984 Domoina Floods (Kovaćs et al., 1985, DWAF Report TR 122)

For the calculation of the storm losses associated with the Domoina floods of 1984, the authors defined the flood volume as the "total flood volume between the time of the apparent sudden rise of the hydrograph and a fixed time after the peak". This time (t), measured in days, was calculated as a function of catchment area (A), measured in km^2, through the following relationship:

$$t = 0.8A^{0.2} \tag{3.3}$$

Particular care was taken to exclude the parts of the hydrograph that could have been generated by post-Domoina rains. At 55 sites, defined as either drainage regions or flow measuring stations, the individual run-off percentage was then determined by calculating the total flood volume (expressed as a depth in millimetres) as a percentage of the average areal rainfall (in millimetres). Based on this information, a diagram of runoff percentage (%) versus total storm rainfall (mm) was produced (Figure 3.5), in which three curves were indicated, each representing a particular antecendent catchment wetness (AP) and vegetal cover. The three curves correspond to the following conditions as described in the Kovaćs 1985 report:

Line I: The 14-day antecedent rainfall (AP_{14}) was generally 50 to 100 mm. This is more or less equivalent to average January conditions in the area. The characteristic vegetation cover was grassveld. The run-off percentages along the line are slightly higher than that given by Figure 3.2.

Line II: AP14 was generally 20 to 50 mm. The vegetation was mainly bush and grassveld. This line seems to be representative for most sites. Note that under the particular catchment wetness conditions approximately 50 mm storm rainfall was needed to start run-off.

Line III: AP14 was variable but generally less than 50 mm. The predominant vegetation in catchment 66, 69, 70 and 71 which plot nearest to the line was forest plantations or orchards. The storm loss in these catchments was very high and apparently about 100 mm storm rainfall was needed to start run-off.

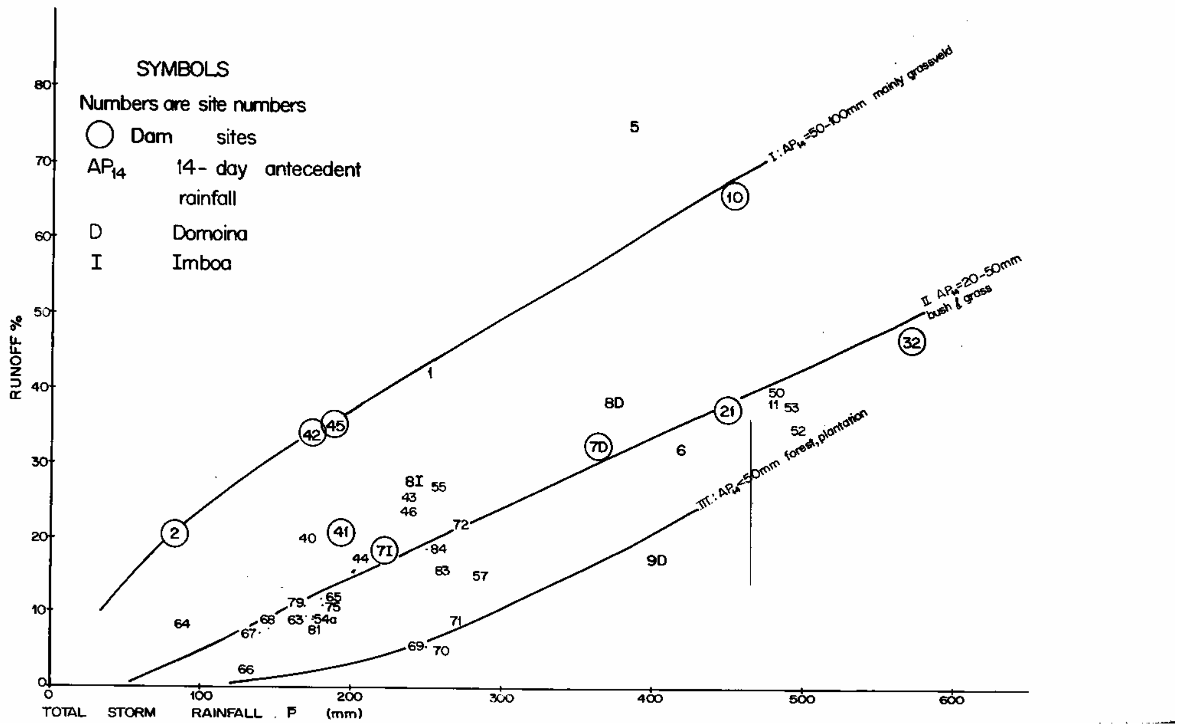

Figure 3.5 Runoff % vs. Storm Rainfall (Kovacs 1985, Figure 6.3)

Nearly the exact methodology for the assessment of storm losses was carried out by Van Bladeren et al. (1989) for the later 1987 Natal Floods, as documented in DWAF Report TR 139. The only difference being that in TR 139, the classes of antecedent catchment wetness were defined as AP_{14} < 50 mm, AP_{14} between 50 and 100 mm, and AP_{14} > 100 mm.

The March-May 1981 floods in the South Eastern Cape (Du Plessis, 1984, DWAF Report TR 120)

For the calculation of storm losses associated with the 1981 floods in the South Eastern Cape, the authors considered four "rules" to define the total flood volume associated with the flood hydrographs that were produced at 53 sites across the study area. Since the start of the flood could be easily identified by the authors by the sudden increase of flow, the four "rules" were based on the method used for the calculation of the duration of the termination of the hydrograph. The "rules" are summarised in Table 3.1 and explained in Figure 3.6, where *Tp* refers to the duration of the total net rise of a multiple-peaked hydrograph, *tc* refers to the time of concentration, T_A refers to the time of maximum curvature, Q_0 refers to the flow at "0" time and Q_m refers to the maximum peak flow value.

Table 3.1 Alternative durations of falling limb (TL)

Method	Duration	Minimum Duration	Maximum Duration
1	*2Tp*	Point of max. curvature T_A	When flow drops to the greater of Q_0 or 0.1xQ_m
2	*2tc*	"	"
3	*0.1* Q_m	-	"
4	Parabola rule	-	-

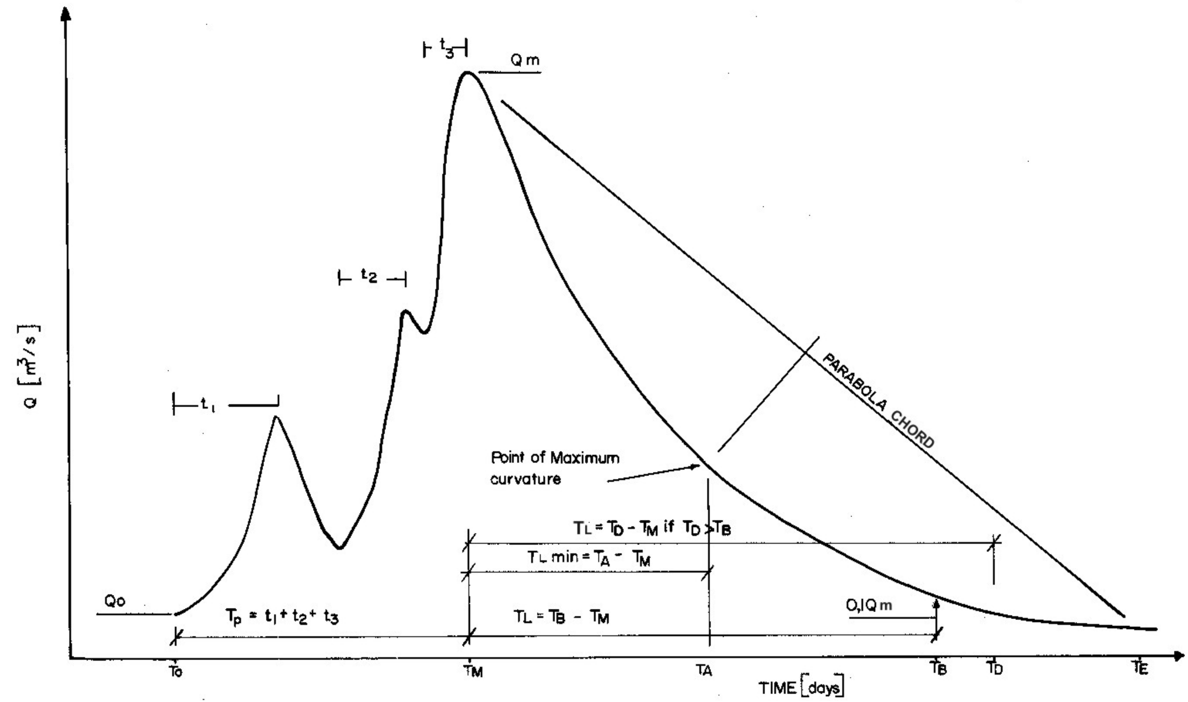

Figure 3.6 Alternative definition of the duration of a flood hydrograph (Du Plessis, Figure 5.6)

The use of the various methods to calculate the flood volume indicated that the value of flood volume was relatively insensitive to T_L, as long as there was a realistic minimum and maximum limit assigned to the duration of T_L. The authors therefore decided to define the total flow volume as the volume between the sudden increase in the flow and the time when the flow in the river has returned to 10% of the last peak flow. For smaller floods the hydrograph was terminated when the flow reached a steady value, even if this value was greater than 10% of the peak flow.

For each site, the runoff percentage was calculated by expressing the total flood volume (expressed as a depth in millimetres) as a percentage of the average areal rainfall (in millimetres). These points were then plotted on a graph of runoff percentage (%) versus storm rainfall (mm) and a "maximum" and "minimum" runoff curve was defined encasing the points (Figure 3.7).

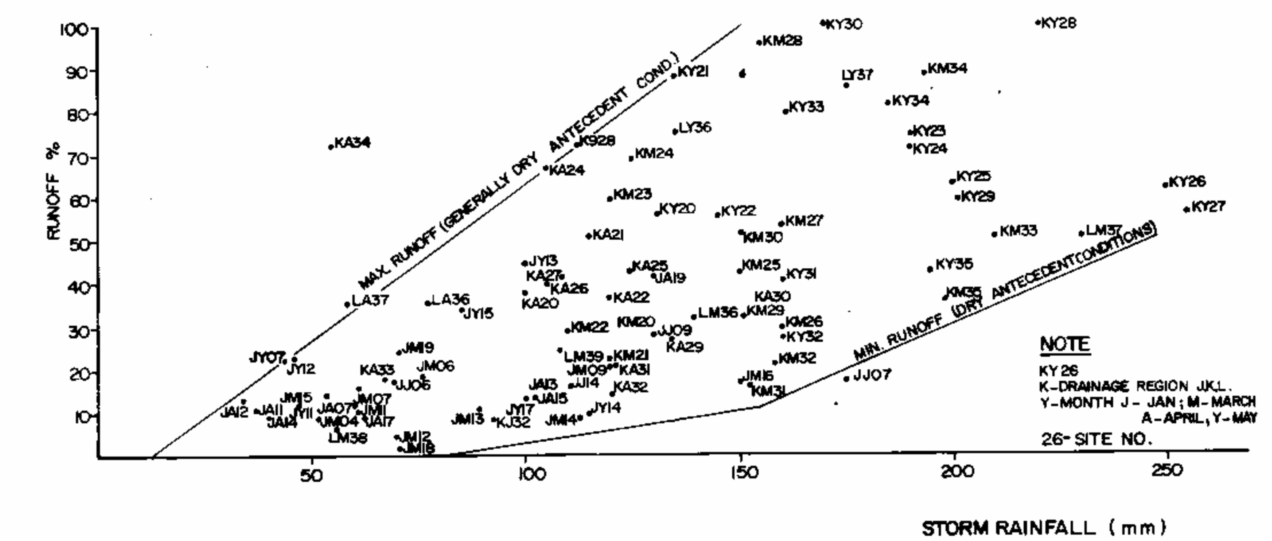

Figure 3.7 Runoff % vs. Storm Rainfall (Du Plessis, Figure 5.25)

3.4 INTERNATIONAL APPROACHES TO STORM LOSSES DETERMINATION

3.4.1 United States

Perhaps the most common method for predicting storm runoff in the United States is the SCS curve number method. This method was developed by the U.S. Soil Conservation Service (part of the U.S. Department of Agriculture), now called the U.S. Natural Resources Conservation Service (NRCS), and is set out in detail in the Chapter 9 and 10 of the National Engineering Handbook, Part 630 (NEH, 2004).

The underlying assumption on which the SCS methodology is based is that, for a single storm, the ratio of actual retention (the rain not converted into runoff) after runoff begins (*F*) to potential maximum retention (*S*) is equal to the ratio of direct runoff (*Q*) to actual rainfall (*P*), i.e.

$$\frac{F}{S} = \frac{Q}{P} \tag{3.4}$$

This relationship, however, only holds for the case where the initial abstraction (I_a), which consists of plant interception, infiltration during the early parts of the storm, and surface depression storage, is zero. To take I_a into account, the NRCS assumed the initial abstraction to be a function of the maximum potential retention S and an empirical relationship was developed based on experimental catchment data (see Figure 3.8). This relationship was determined to be

$$I_a = 0.2S \tag{3.5}$$

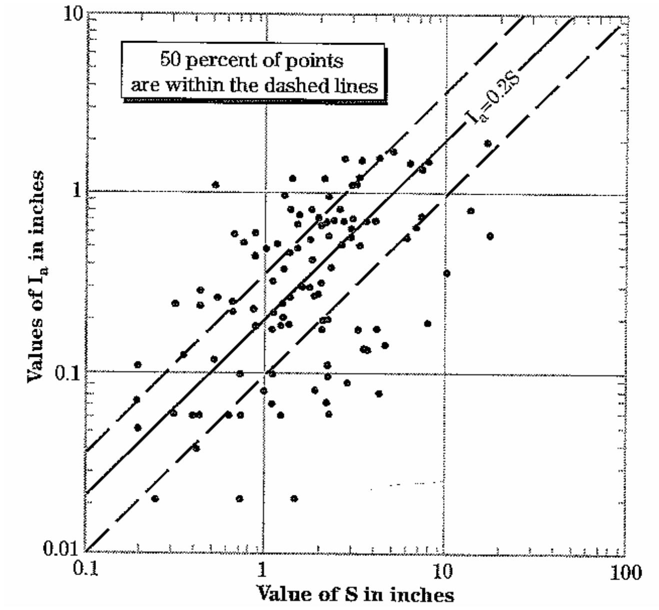

Figure 3.8 Relationship between Ia and S (NEH, 2004)

After algebraic manipulation and expansion of the above two equations, the following SCS rainfall-runoff relationship was defined:

$$Q = (P - 0.2S)^2 / (P + 0.8S) \qquad (3.6)$$

The potential maximum retention, *S*, is represented by a Curve Number, *CN*, which varies according to the hydrologic soil group and the land-use of the study area. Values of *CN* can be read off from Table 9.1 and Table 9.5 in Chapter 9 of NEH (2004) for agricultural lands and urban areas respectively. These values of CN were developed based on rainfall-runoff data available from literature for storms producing the annual flood for catchments of a particular soil group and land-use. These catchments were generally less than 1 square mile (2.59 square kilometres) and the storms were of 1 day or less duration. The potential maximum retention can be determined from *CN* through the following relationship:

$$S = (1000/CN) - 10 \text{ (where } S \text{ is in inches)} \qquad (3.7)$$

or

$$S = (25400/CN) - 254 \text{ (where } S \text{ is in millimetres)} \qquad (3.8)$$

Although the SCS methodology was developed based on relatively small catchment areas, and is therefore most applicable to such catchments, the SCS methodology can just as equally be applied to large catchment areas if the geographical variations of storm rainfall and soil cover are taken into account (NEH, 2004). This is accomplished by dividing the catchment into smaller sub-areas. The individual runoff values would be estimated using the normal SCS methodology for each sub-area, and the average runoff determined through areal weighting. As for the South African SCS approach outlined in Section 3.3.2 above, the US-SCS methodology does not require the development of large-scale regional functions of design storm losses.

3.4.2 Australia

Guidance for the calculation of storm losses within Australia is provided in the Australian Rainfall Runoff publication of 1987 (ARR87) and the more recent publication of 1999 (ARR99). For the calculation of storm losses, the ARR proposes five different loss models, the selection of which is dependent on type of problem, the data available, the likely runoff processes within the study area, as well as whether a design or site-specific study is required. A description of the five models and where they might best be applied is given below (ARR87) and illustrated in Figure 3.9:

i. Constant fraction: Here the loss is considered a constant fraction of rainfall in each time period. This fraction would be the fraction of the runoff producing portion of the catchment. This model would be applicable in the case where there is saturated overland flow occurring from a fairly constant portion of the catchment.

ii. Constant loss rate: Here the rainfall excess is the residual left after a selected constant rate of infiltration capacity is satisfied.

iii. Initial loss and continuing loss: Here no runoff is assumed to occur until a given initial loss capacity has been satisfied.

iv. Infiltration curve: Here the capacity rates of loss vary with time.

v. Standard rainfall-runoff relation, such as the US SCS approach:

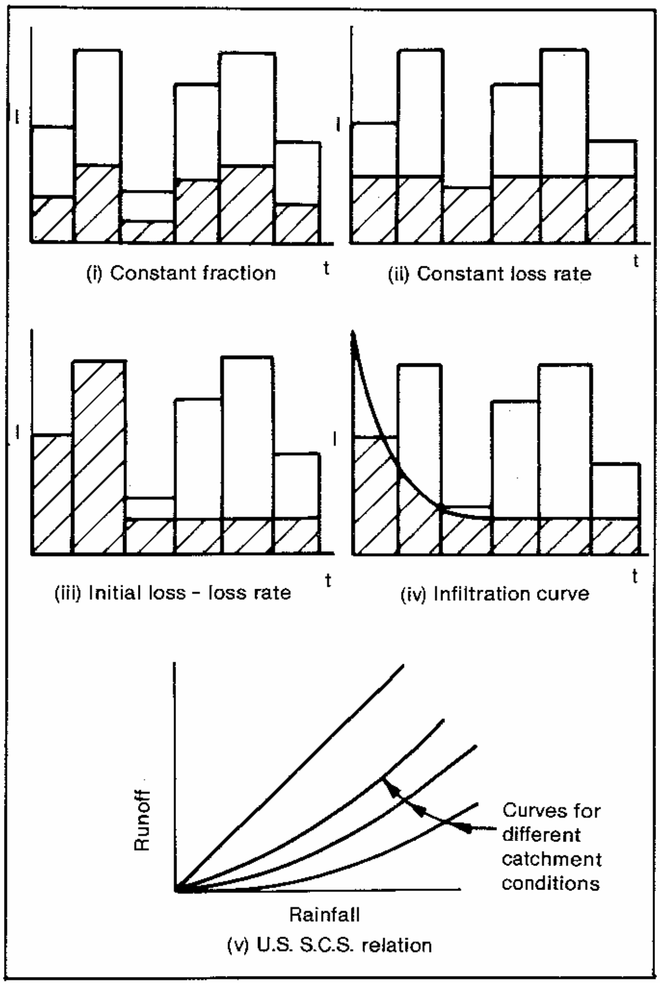

Figure 3.9 Loss models to estimate rainfall excess (ARR87, Figure 6.1)

For the case where the analysis approach is one of design, i.e. the flood is derived from some probability analysis of flood or rainfall data as opposed to actual observed storm events, the ARR recommends the use of loss models (i) and (iii). This is because design studies are usually associated with the assessment of large area storms from which runoff is produced from the whole catchment. In these cases, the behaviour of runoff is close to that of Hortonion flow.

Where the derivation of storm losses based on an observed storm event is required, the ARR recommends the use of loss model (v), as in the case of a site-specific study, it is necessary to make allowances for the moisture conditions of the catchment prior to the occurrence of the storm. If, however, a reliable rainfall-runoff relationship cannot be established due to insufficient data, the ARR recommends the use of loss model (iii) where a relationship between an initial loss and antecendent moisture conditions would have to be estimated.

Where local loss values are required for a catchment, but where data is insufficient for a site-specific analysis, the ARR recommends the calculation of a median loss rate based on five or more observed storm events that have occurred within nearby similar catchments. If any number of storm events less than five is available, then care must be taken to avoid the use of extreme events, where the resultant loss values would be biased towards wet catchment conditions. Table 6.1 in ARR87 lists 54 catchments for which the median loss rate (mm/h) has been calculated based on five or more observed storm events.

For the cases where absolutely no data is available for the calculation of local loss rates, or where the level of detail and effort is not required, the ARR provides eight tables representing appropriate loss values for different regions across Australia (covering about 40% of the country). Within each table, the loss model and parameters appropriate to that region are given, as well as the sources from which the data was obtained. If the study area lies outside of the indicated regions, the storm losses can be estimated from the table of the region that displays the closest catchment characteristics.

3.4.3 United Kingdom

The current recommended methodology in the UK for the estimation of storm losses is that reported in Volume 4 of the Flood Estimation Handbook (FEH) of 1999. This methodology is an improvement upon the original rainfall-runoff method first presented in the 1975 Flood Studies Report (FSR), the core of which is made up of the unit hydrograph model and percentage runoff model. Since the FSR publication, significant improvements have been made to the model parameter estimation equations used in the percentage runoff model, as reported in detail in FSSR16 (Wallingford, 1985) and as presented in the FEH.

The FEH percentage runoff model synthesises percentage runoff (*PR*) from the natural part of the catchment, which is then adjusted for the effects of catchment urbanisation. The "natural" runoff percentage component, PR_{RURAL}, is made up of a *standard* term, *SPR*, which represents the normal capacity of the catchment to generate runoff; and dynamic term, *DPR*, which represents the variation in runoff depending on the condition of the catchment prior to the storm (DPR_{CWI}), i.e. the antecedent moisture conditions, and the storm magnitude (DPR_{RAIN}). The value associated with DPR_{CWI} and DPR_{RAIN} is dependent on a Catchment Wetness Index (*CWI*) and the storm depth, *P*, respectively. The procedure for the calculation of percentage runoff is presented in the following equations:

$$PR = PR_{RURAL}(1.0 - 0.615\,URBEXT) + 70(0.615\,URBEXT) \tag{3.9}$$

where $$PR_{RURAL} = SPR + DPR_{CWI} + DPR_{RAIN} \tag{3.10}$$

$$DPR_{CWI} = 0.25(CWI - 125) \tag{3.11}$$

and $$DPR_{RAIN} = \begin{cases} 0 & [for\ P \leq 40\text{mm}] \\ 0.45(P-40)^{0.7} & [for\ P > 40\text{mm}] \end{cases} \tag{3.12}$$

As evident from Equation 3.9, the urban adjustment made to natural runoff component assumes that 61.5% of the urbanised area is impervious and gives 70% runoff. The other 38.5% of the urban area therefore acts as a natural catchment. The term *URBEXT* is a digitally derived catchment descriptor (constant), which is defined in the FEH as the "extent of urban and suburban land cover". The values of *URBEXT* can be read off from Table A1 of Appendix A.3 in FEH Volume 5. CWI (mm) can be determined from Appendix A of FEH Volume 4.

The standard component, *SPR*, is fixed for a particular catchment. Where rainfall and runoff records are available for a catchment, FEH Volume 4 describes the process that should be undertaken to either derive *SPR* at the same time as the Unit Hydrograph time-to-peak, or by using the catchment baseflow index (*BFI*). Where no records are available, FEH Volume 4 presents a method for the estimation of *SPR* based on catchment descriptors using a generalised model derived by regression analysis. The FEH cautions the user when estimating *SPR*, as it describes this component as the most significant of the percentage runoff model parameters.

3.5 REVIEW OF REGIONAL STORM LOSS METHODOLOGIES FOR SOUTH AFRICA

3.5.1 Orientation

Ultimately, the challenge with any deterministic design flood methodology is to be able to derive credible input data at *ungauged* sites. It can be seen from the above examples of international and historical local practice in this regard, that this challenge is met in two primary ways: either through detailed quantification of those catchment characteristics that affect losses (e.g. the SCS or UK FEH approach), or in terms of large-scale/regional design storm loss functions/models, e.g. the South African or Australian approaches. Given that most South African dam safety assessments deal with catchments larger than the SCS conventions allow, the focus of our quantified review has had to rest on the regional curves for average and minimum design storm losses, respectively, presented in HRU 1/72. For the average design storm losses review we approached the analysis through four discrete stages, each based on a progressively more intense utilisation of the raw and processed data assembled in this Project:

- Method 1: An exact repeat of the HRU Veld-Zone-based methodology, as described in Section 3.3.1 above (Average losses approach), but with the use of the 1'x1'-gridded design rainfalls for a range of RIs published by Smithers and Schulze (2002).
- Method 2: A further modification of the HRU methodology, additional to Method 1, in which the RI flood peaks produced and described elsewhere in this Project are used.
- Method 3: An alternative modification of the HRU methodology, additional to Method 1, in which average standardised flood volumes, conditional on standardised flood peak and RMF-K-Region, and produced and described elsewhere in this Project, are used.
- Method 4: Conventional storm runoff percentage calculations, based on a comparison of observed rainfall surfaces for selected observed historical floods in representative catchments.

3.5.2 Method 1: HRU methodology for *average* storm loss functions with updated design rainfall

Section 3.3.1 outlines the innovative manner in which the HRU 1/72 team derived *average* typical storm loss/runoff percentages by combining their map of regional RI-flood peaks regions with their design rainfall and unitgraph tools. In this first check, the Research Team remained true to the HRU methodology, but replace their catchment design rainfall for each selected RI with 1'x1'-gridded design rainfalls produced via the design rainfall software developed under a WRC contract by Smithers and Schulze (2002). This was done for RI-events of 1:2, 1:5, 1:10, 1:20, 1:50 and 1:100 years for each of 40 catchments (see Figure 3.10) out of the total HRU sample of 92 catchments (the catchments this

Project has in common with the HRU sample). To develop catchment areal rainfalls, a GIS tool was specifically developed to create a design rainfall surface that matched each catchment's boundaries.

Figure 3.11 depicts the outcome as a scatter-plot on an identical template to that used in HRU1/72. The scatter of points might appear quite bewildering initially, but, on closer inspection, it can be seen that the curves representing the three Veld-Zone Groups, labelled in HRU 1/72 as "average", do broadly bi-sect the three groups of plotted symbols. This is borne out by the percentiles reported in Table 3.2, which indicate that the HRU average runoff/loss curves do broadly represent mid-range values per Veld-Zone Grouping. The large number of values that exceed 100% runoff / 0% loss is surprising. For Veld-Zone 2 the most likely explanation for that outcome is that the design rainfalls are too low in a number of catchments – this is an endemic problem in mountainous regions of the country, such as Veld-Zone 2, because the terrain precludes rainfall recording in high-elevation/ high rainfall locations.

Table 3.2 Methods 1, 2 and 3: Outline of results of review of HRU average loss curves

Method	Veld-Zone	Number of Catchments	% Above the Curve	% Below the Curve
1.	1,3,8,9	22	40	60
	4,5,6,7	9	44	56
	2	9	44	56
2.	1,3,8,9	19	18	82
	4,5,6,7	12	3	97
	2	9	55	45
3.	1,3,8,9	19	59	41
	4,5,6,7	11	45	55
	2	9	83	17

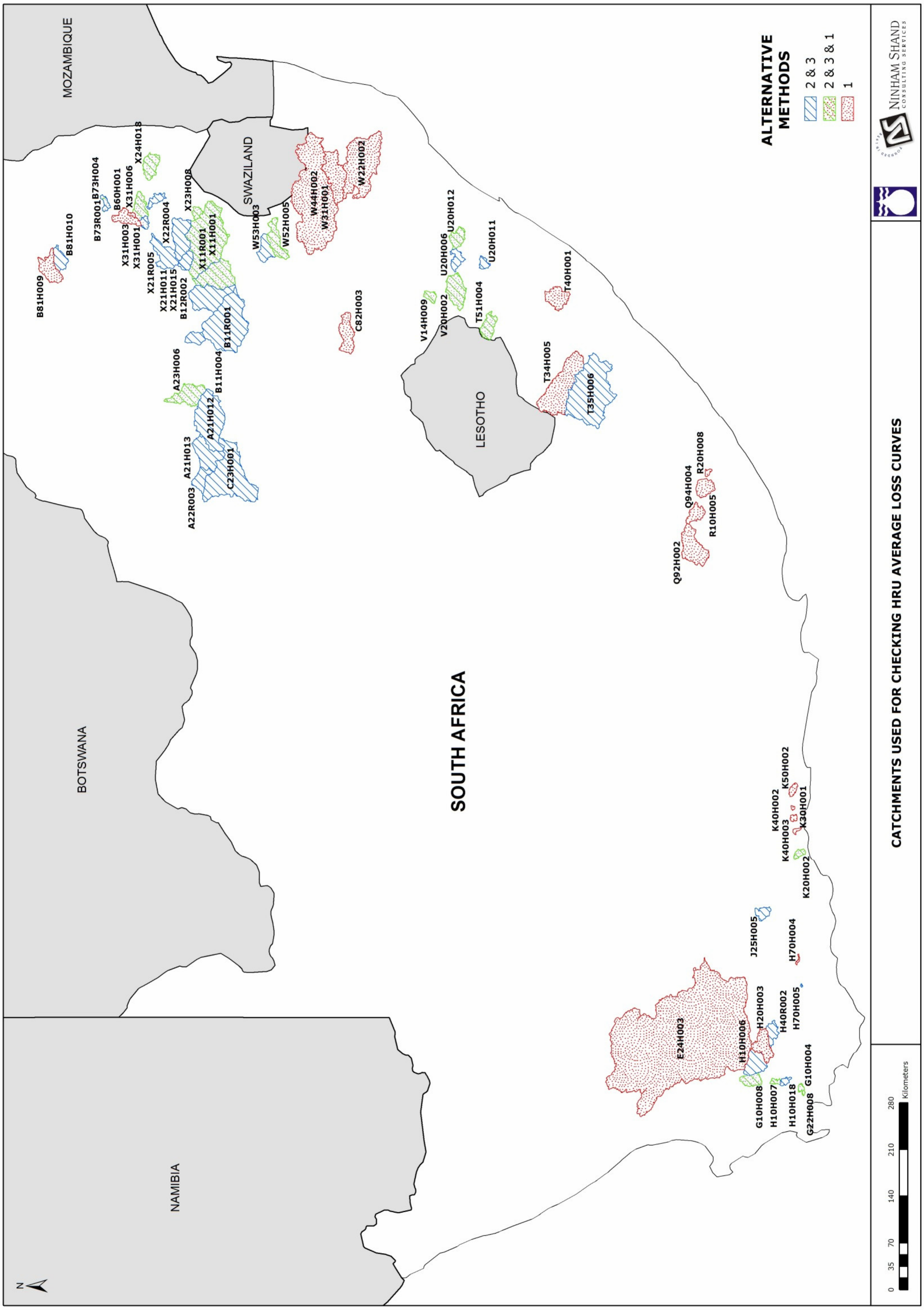

Figure 3.10 Locations of catchments included in average design storm loss analysis according to the HRU 1/72 methodology

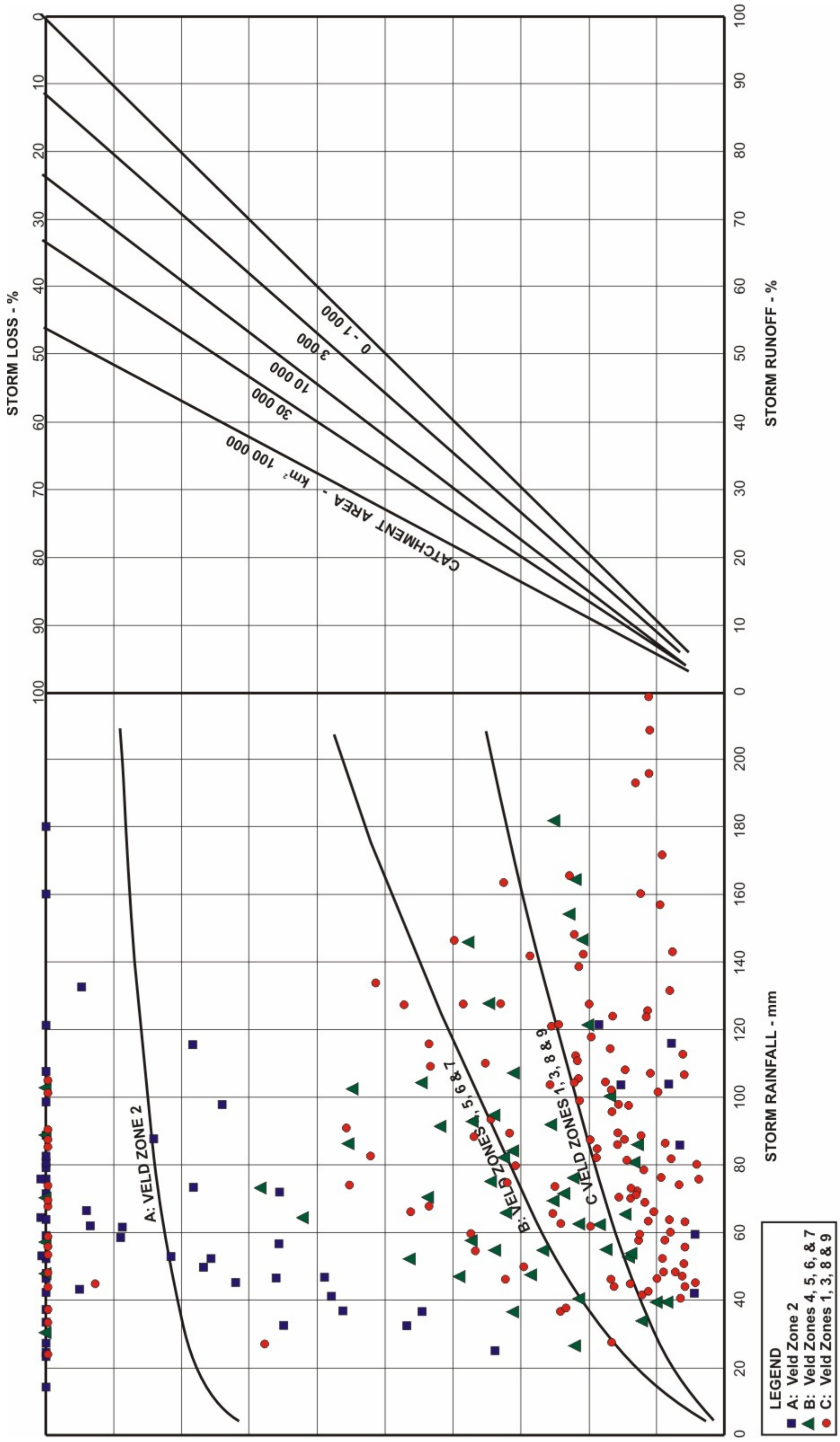

Figure 3.11 Method 1: HRU methodology for average design storm losses but with updated design rainfall

3.5.3 Method 2: HRU methodology for *average* storm loss functions with updated design rainfall, site-specific RI-flood peaks and different unitgraph peaks

In this approach the Research Team went one step further than Method 1 with the modification of the HRU methodology by introducing site-specific RI-flood peaks and corresponding unitgraph peaks, additional to the updating of the design rainfall. In this case we used 40 catchments, of which 20 catchments differed from those in Method 1, due to limitations on the availability of catchment characteristics for all HRU cases from which to derive different critical unit duration unitgraphs.

Figure 3.12 depicts the outcome of this second approach as a scatter-plot on an identical template to that used in HRU1/72. The scatter of points is bunched across a lower region than in Method 1 for all the Veld-Zones, except Veld-Zone 2. The net result is that the two curves for those two Veld-zone groupings appear to indicate losses that are too low/ runoffs too high, given this particular data set and modified approach. This is borne out by the percentiles reported in Table 3.2. A large number of values of Veld-Zone 2 exceed 100% runoff / 0% loss, but its HRU curve still represents a reasonable mid-range for this approach.

It is expected that the results for Method 2 are less reliable than those of the other methods because of the way in which the critical storm duration was defined, which was a requirement of this method. It is quite apparent that the derivation of regional storm loss curves is very sensitive to the methodology employed.

3.5.4 Method 3: HRU methodology for *average* storm loss functions with updated design rainfall and standardised average flood volumes conditioned on flood peak and RMF K-region

Review of the HRU regional storm losses curves

In this approach the Research Team went two steps further than Method 1 with the modification of the HRU methodology by introducing average flood volumes for each of our site-specific RI-flood peaks, additional to the updating of the design rainfall. These average flood volumes were extracted from the database of more than 12 000 floods developed under this Project (reported elsewhere) and which had been organised by RMF-K-region and pooled as standardised values. The conventional standardisation transformation for log-Normal space was employed. In this case we used 39 catchments, of which 19 catchments differed from those in Method 1, due to limited current availability of reliable streamflow records for all the HRU sites.

Figure 3.13 depicts the outcome of this third approach as a scatter-plot on an identical template to that used in HRU1/72. The scatter of points is more even than in Method 1 for all the Veld-Zones. The net result is that the points for all the Veld-Zones, excluding Veld-Zone 2, appear to be reasonably bi-sected by their respective HRU curves, given this particular data set and modified approach. This is borne out by the percentiles reported in Table 3.2. However, a large number of values for Veld-Zone 2 exceed the relevant HRU curve. A plausible interpretation would relate to the lack of representative high-elevation, high-rainfall records in this Zone, resulting in design rainfalls that are too low and runoff percentages that are too high.

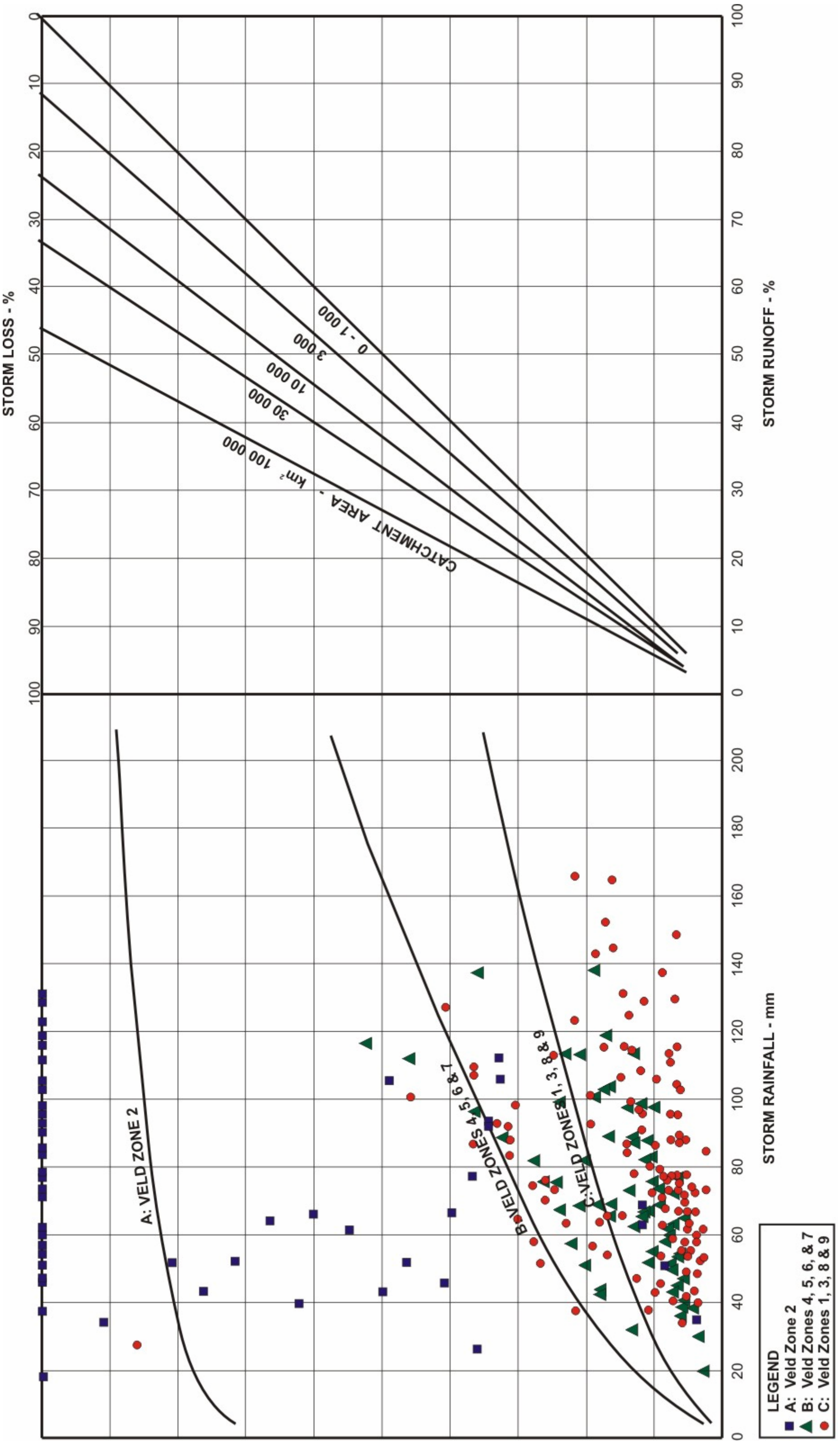

Figure 3.12 Method 2: HRU methodology for average design storm losses but with updated design rainfall, site-specific RI-flood peaks and different unitgraph peaks

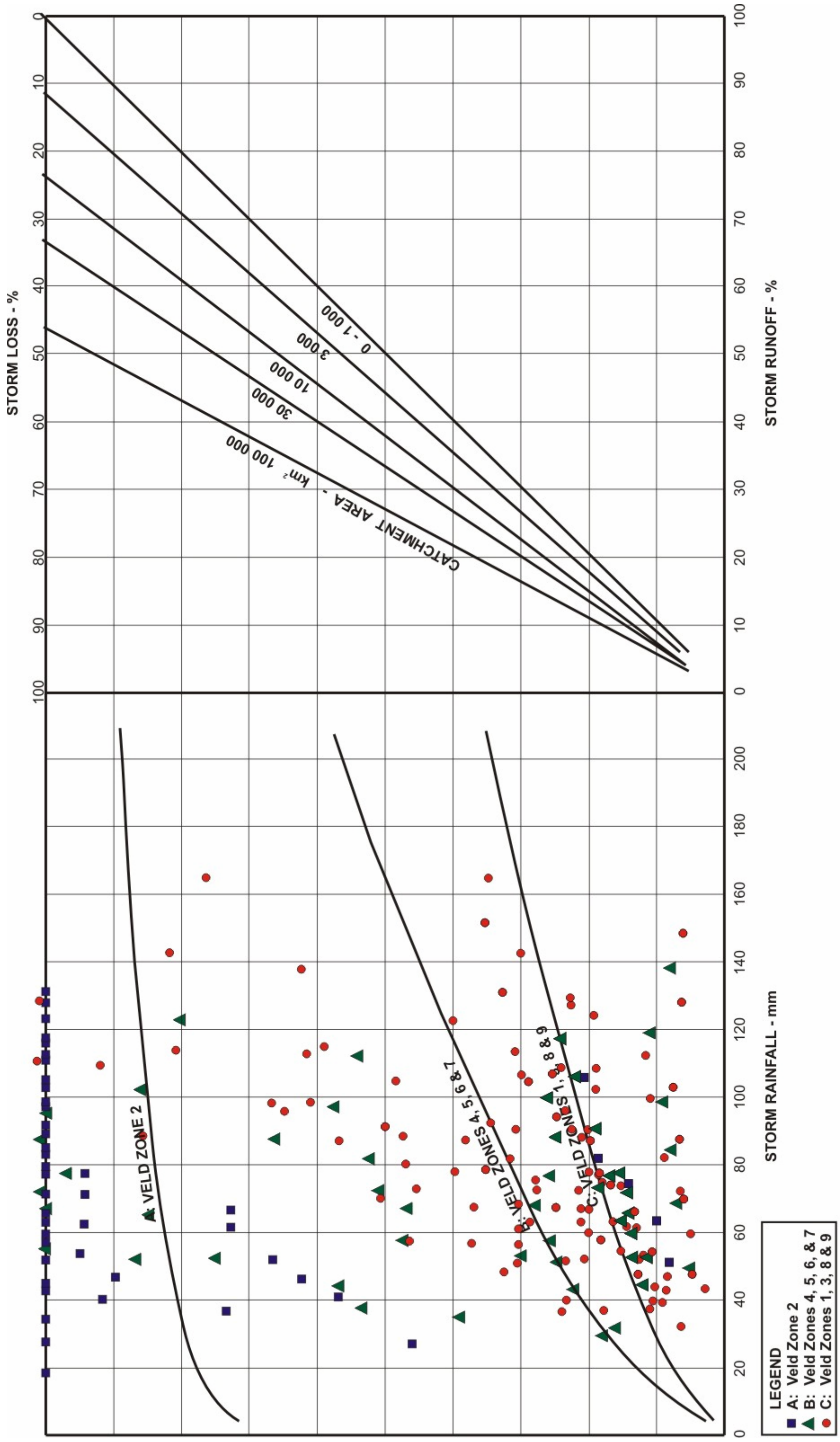

Figure 3.13 Method 3: HRU methodology for average design storm losses but with updated design rainfall, and average flood volumes for site-specific RI-flood peaks

Derivation of average regional loss curves arranged according to Kovaćs K-regions

Using the average flood volumes and associated design rainfall values determined as part of Method 3, an attempt was made to produce regional loss curves, this time based on groupings of K-region as opposed to the Veld-Zone groupings used in the HRU methodology. The three K-region groups are the following: 'Low K-values' ranging from less than 2.8 to 4.6, 'Mid K-values' containing Region 5, and 'High K-values' for K-regions greater than 5.2.

This exercise was undertaken to establish whether these types of design storm loss curves could successfully be produced, in which case, these curves could be used in conjunction with the Joint Peak-Volume Design Flood methodology, which is regionalised according to the above K-region groupings. This design flood estimation methodology was specifically developed as part of this Project and is reported elsewhere.

For the drawing of the curves, it was decided that the same axes as Figure G2 of HRU 1/72 would be used. Also, it was decided that the curves would be drawn so that they roughly represented the mid-range storm runoff/loss percentage, in other words, that they would roughly bisect the appropriate data points evenly. The resulting "average" storm loss curves arranged according to K-region are shown in Figure 3.14. The examination of these curves indicates that they indeed have promise to be used in conjunction with the standardised hydrographs derived as part of the Joint Peak-Volume Design Flood methodology. It should be noted, however, that this is a first attempt and that, if further research were to be undertaken, a larger database could improve their positioning. Also, in order for these curves to be used in conjunction with the newly developed Joint Peak-Volume Design Flood methodology, standardised unitgraphs specific to each K-region group would have to be defined. This, once again, could be addressed in further research if the opportunity arises.

3.5.5 Method 4: Analysis of losses during selected historical storms

Orientation

As catchment characteristics have an important influence on rainfall-runoff processes in terms of storm losses, they are an important consideration in the methodology described below. In order to obtain representative rainfall for storm events in catchments located in mountainous regions, it is necessary to consider the influence of elevation and the associated higher rainfall that is experienced in these areas. Generally, there are relatively few rainfall gauges located in the high-lying mountainous areas of these catchments where orographic rainfall is experienced and therefore there is no record of the rainfall in these areas. It is therefore proposed that in order to obtain representative catchment rainfall experienced in mountainous areas, daily storm rainfall would need to be factored according to a rainfall surface that is influenced by the topography of the catchment.

Gauges from each of the HRU 1/72 storm losses regions, Veld-Zone 2, Veld-Zone 5 and Veld-Zone 8, were selected as the study areas for the application of the catchment-specific approach for the calculation of storm losses. The approach involved selection of suitable flow measuring gauges and rainfall stations in each study area, identification of flood events at each flow gauge, and identification of the storm rainfall and its duration.

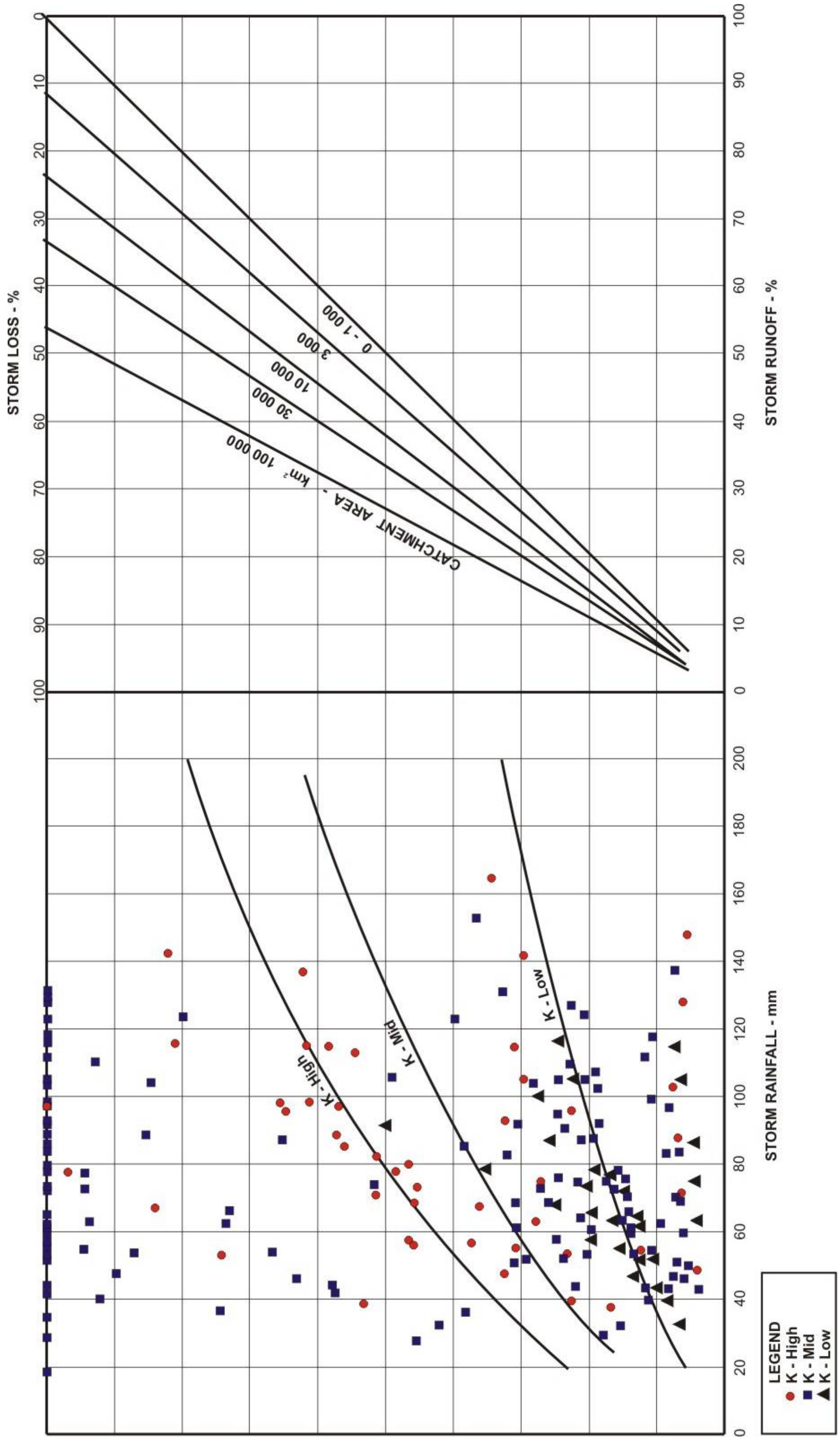

Figure 3.14 Regional "average" storm loss curves arranged according to K-region

Selection of flow gauging stations and flood data

The selection of flow gauge stations was limited to the stations that had already undergone a screening process for use in addressing the second aim of this project, which involves the derivation of a methodology for design flood hydrograph estimation. Representative flow gauges were chosen in each of the HRU regions and four observed flood events were selected from the annual maximum flood record at each gauge. The observed flood events included the largest flood, the tenth largest or 1 in 5 year event (dependent on the length of the analysed record), the median event and the largest volume event for the length of the record. The selection of flood events was based on obtaining a spread of historical values for relatively large events in each Veld-Zone in order to test the validity of the HRU Veld-Zone average loss curves.

Flood hydrographs for the selected events from each of the flow gauges were extracted from this Project's Floods Database and flood data for reservoirs in the study areas was obtained from the DWAF floods database.

Selection of rainfall stations

Daily rainfall stations were selected on the basis that they would provide a broad coverage of points across the flow measuring catchments in order to create a catchment rainfall surface using GIS tools. Once the catchment rainfall stations were selected for each study area, the daily rainfall records were extracted from a database developed by Lynch (2004) using the Daily Rainfall Extraction Utility developed by Kunz (2004). This daily rainfall database is a comprehensive and up-to-date database consisting of more than 300 million daily rainfall values from 12 153 rainfall stations, of the monitoring networks of the South African Weather Service (SAWS), the Agricultural Research Council (ARC), the South African Sugarcane Research Institute (SASRI) and from municipalities, private companies and individuals (Lynch, 2004). The daily rainfall data was extracted for a 7-day period, 3 days either side of the flood peak for all the gauges in the catchment study area. Stations with missing data were discarded, but stations with patched data were included in the analysis. Initial comparison of flood events and daily rainfall for gauges in the catchment indicated that storm rainfall occurred one to two days before the flood peak. Consequently, it was decided that the storm rainfall contributing to the flood was generally of 3-day duration, i.e. antecedent rain 2 days before the flood peak and rainfall on the day of the storm (Figure 3.15). An optimistic scenario of 4-day rainfall that included rainfall on the day after the flood peak was also compiled for comparison.

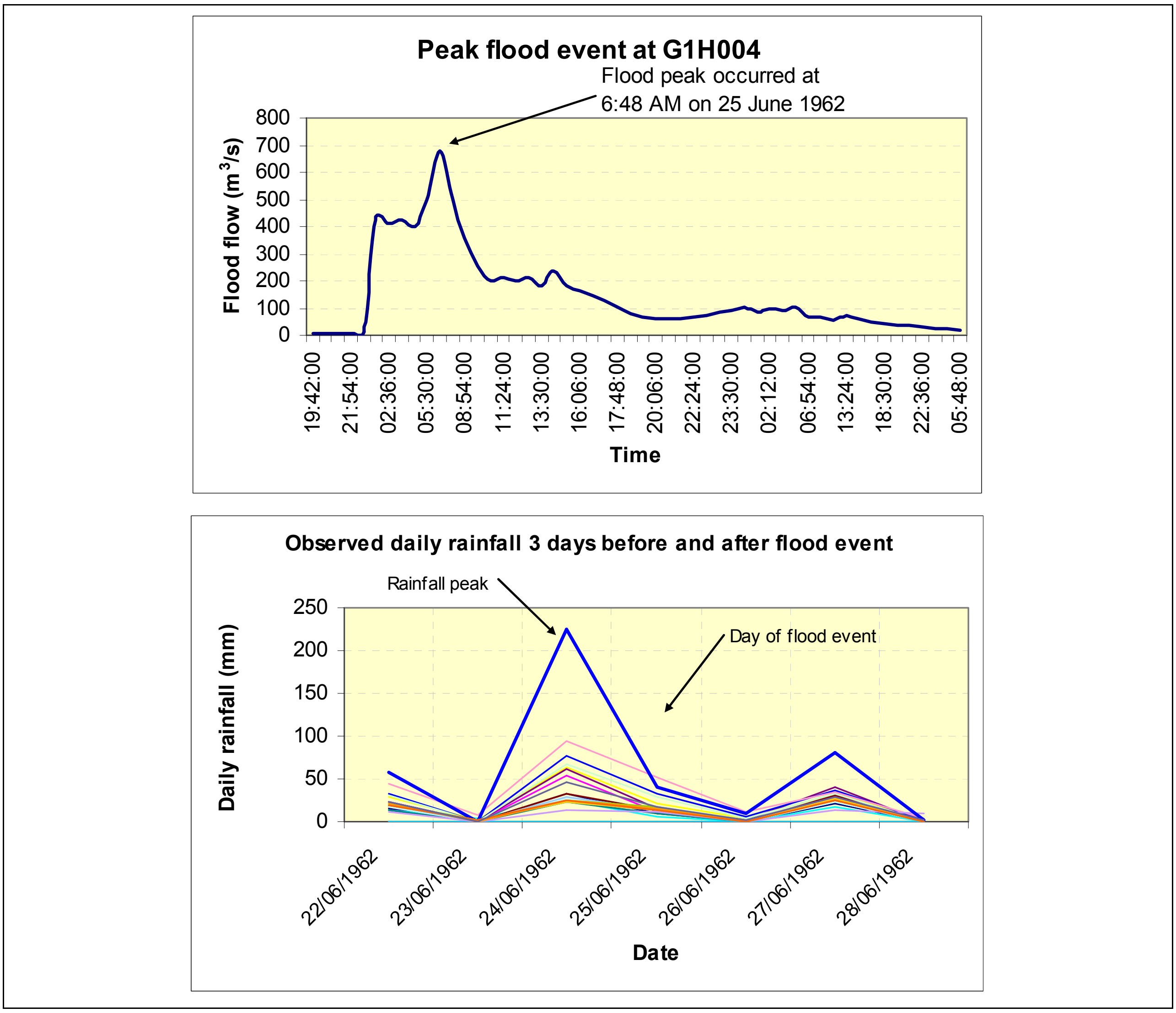

Figure 3.15 Identification of storm rainfall

Creation of a catchment rainfall surface

Using the selected catchment rainfall stations in each study area, a 3-day catchment rainfall surface for each flood event at each gauge was generated and intersected with the flow measuring catchment boundary and the mean storm rainfall was calculated for the catchment. The ArcGIS Spatial Analyst tool was used to interpolate data from rainfall stations to create a raster surface. The interpolation procedure has been designed to take advantage of the types of input data commonly available and the known characteristics of elevation surfaces. This method uses an iterative finite difference interpolation technique. It is optimised to have the computational efficiency of local interpolation methods, such as inverse distance weighted (IDW) interpolation, without losing the surface continuity of global interpolation methods, such as Kriging and Spline. The ArcGIS Spatial Analyst was used to calculate the statistics for the raster surface for each catchment.

In total, 72 surfaces for the 3-day rainfall associated with each flood event were created. Additionally, a 4-day catchment rainfall surface was generated for the largest (Event 1), the tenth largest (Event 10) and the 1 in 5 year event in Veld-Zones 2 and 8. A 4-day catchment rainfall surface was not created for the other flood events in these Veld-Zones nor in Veld-Zone 5 due to data processing constraints. An example of the resultant catchment storm rainfall surface is presented in Figure 3.16. In addition, a station Mean Annual Precipitation (MAP) surface was created for each study area to intersect with the CCWR gridded rainfall database of 2004 so as to obtain a correction factor for storm rainfall contributing to the flood events at each gauge. The CCWR gridded rainfall surface provides a minute by minute grid of point rainfall that has been spatially corrected by 4 variables: elevation, latitude, longitude and distance from a topographic barrier. It was for these reasons that this database was selected to provide a factor that considers the effect of catchment characteristics on daily storm rainfall such as in Veld-Zone 2. The CCWR, station MAP and storm rainfall surfaces were intersected with the flow gauging catchment boundaries in the selected study areas and an areal catchment rainfall estimate was obtained for the flood events at each gauge.

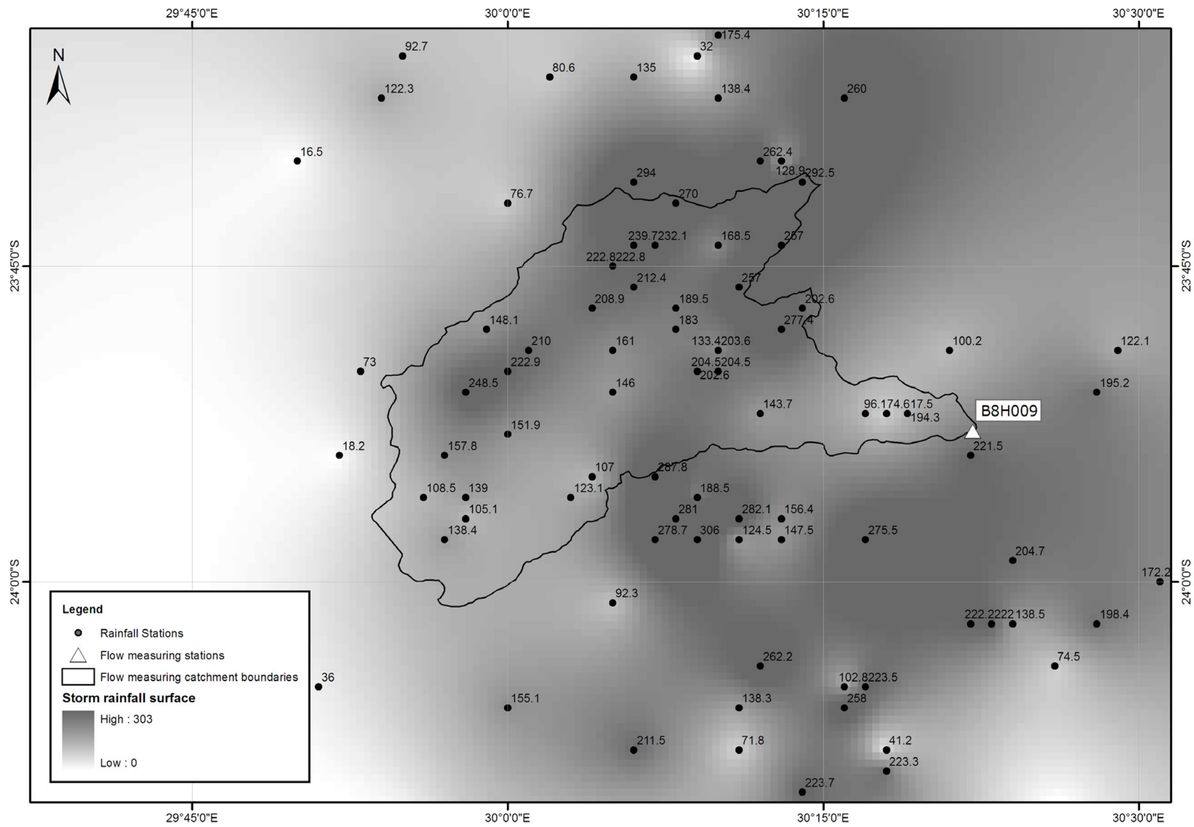

Figure 3.16 Example of catchment storm rainfall surface

Results: Veld-Zone 2

Figure 3.17 shows the selected flow gauging stations and rainfall stations located in Veld- Zone 2 in the Berg River catchment in the Western Cape. The following flow gauges were used in the analysis: G1H004, G1H008, G2H008, H1H006, H1H007 and H1H018.

Table 3.3 presents the flood events identified at each of the gauges in the analysis as well as the storm rainfall associated with that event calculated from GIS. It is noted that the flood volume for the tenth largest annual flood at G1H004 is greater than the flood volume for the largest annual flood, however its peak is approximately half the magnitude. It was found that the catchment rainfall values for the catchment MAP surface and the CCWR gridded surface were similar and thus it was not necessarily useful to apply a weighting factor to the original catchment storm rainfall surface. Another observation was that at flow gauge G1H004, the 3- and 4-day catchment rainfall were seen to be greater for the tenth largest flood than for the largest flood. This correlates well with the larger flood volume observed at the gauge for the tenth largest event.

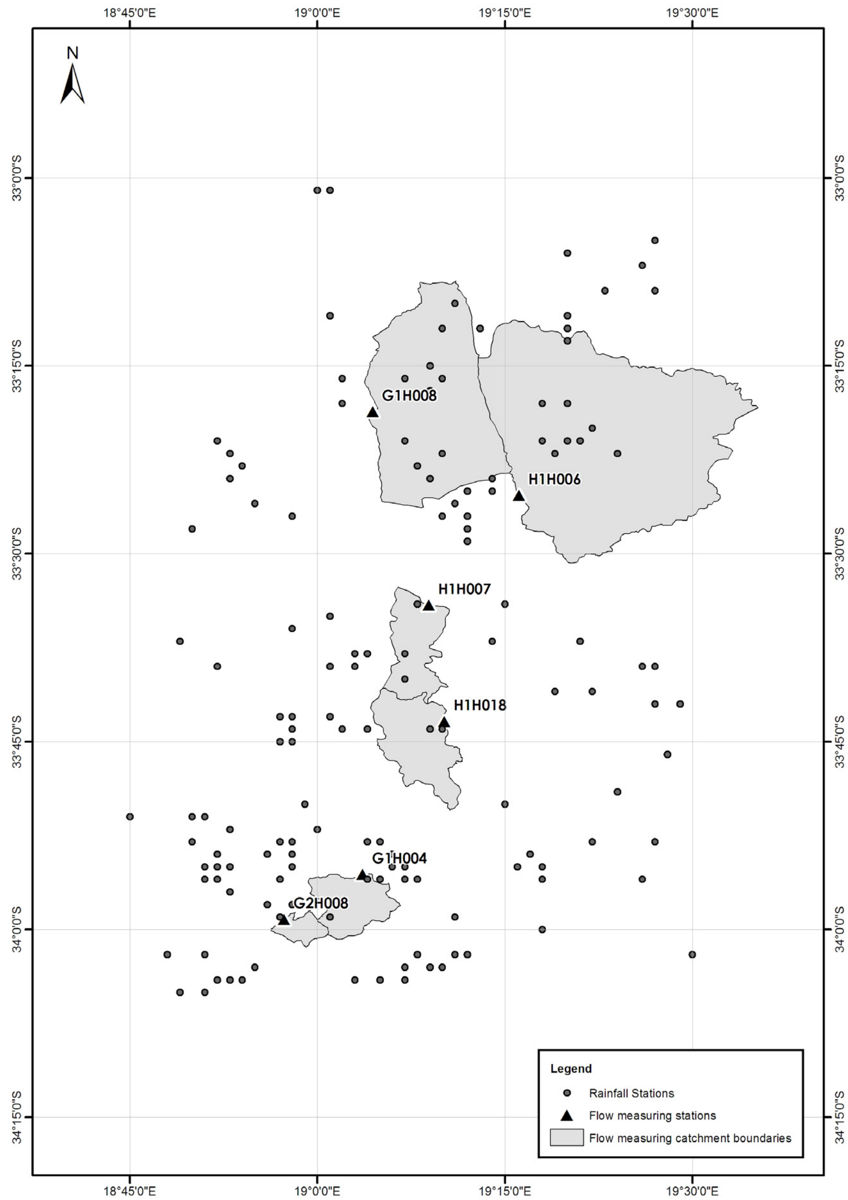

Figure 3.17 Location of flow measuring and rainfall stations in Veld-Zone 2

Table 3.3 Selected flood events and catchment rainfall for flow gauges in Veld-Zone 2

Event	Gauge	Area (km^2)	Date	Flood peak (m^3/s)	Flood volume (Mm^3)	Flood volume (mm)	3-day Rain from GIS		4-day Rain from GIS	
							Rainfall (mm)	Runoff %	Rainfall (mm)	Runoff %
Largest peak	G1H004	69	25 June 1962	679	22	325	122	**100**	127	**100**
Largest peak	G1H008	394	16 May 1984	517	16	40	146	**28**	169	**24**
Largest peak	G2H008	20	18 February 1955	46	1	72	143	**50**	160	**45**
Largest peak	H1H006	753	10 June 1967	876	39	52	102	**51**	110	**47**
Largest peak	H1H007	85	26 June 1994	448	10	116	177	**66**	206	**56**
Largest peak	H1H018	110	12 June 1985	705	22	200	107	**100**	108	**100**
Tenth largest peak	G1H004	69	24 June 1991	364	29	425	235	**100**	234	**100**
Tenth largest peak	G1H008	394	24 November 1976	196	11	29	118	**25**	123	**24**
Tenth largest peak	G2H008	20	8 August 1963	35	1	68	126	**54**	153	**45**
Tenth largest peak	H1H006	753	6 June 1976	514	30	40	24	**100**	24	**100**
Tenth largest peak	H1H007	85	24 October 1995	257	8	90	56	**100**	57	**100**
Tenth largest peak	H1H018	110	6 June 1976	481	7	62	57	**100**	58	**100**
Largest volume	G1H004	69	1 May 1990	438	59	849	63	**100**	-	-
Largest volume	G1H008	394	4 August 1986	149	17	44	77	**57**	-	-
Largest volume	G2H008	20	6 February 1952	46	4	195	Missing data	**-**	-	-
Largest volume	H1H006	753	11 July 1993	553	87	116	53	**100**	-	-
Largest volume	H1H007	85	6 June 1976	280	32	372	128	**100**	-	-
Largest volume	H1H018	110	8 July 1993	607	37	337	122	**100**	-	-
Median	G1H004	69	8 August 1974	255	19	271	100	**100**	-	-
Median	G1H008	394	7 June 1964	98	4	10	89	**11**	-	-
Median	G2H008	20	10 June 1967	30	2	120	128	**93**	-	-
Median	H1H006	753	14 May 1987	377	56	75	15	**100**	-	-
Median	H1H007	85	2 July 1966	222	9	108	104	**100**	-	-
Median	H1H018	110	22 June 1983	370	11	100	128	**78**	-	-

The results of the analysis for Veld-Zone 2 are plotted in Figure 3.18 and can be viewed in Table 3.3. By considering the positions of the data points, it appears that the HRU loss curve for Veld-Zone 2 could reasonably represent mid-range design loss values. Also, as is evident from the results for Methods 1 to 3, there are a number of events for which more than 100% runoff appears to have been generated. This once again can be explained by the apparent underestimation of rainfall from gauges located in mountainous regions, which is typical of Veld-Zone 2.

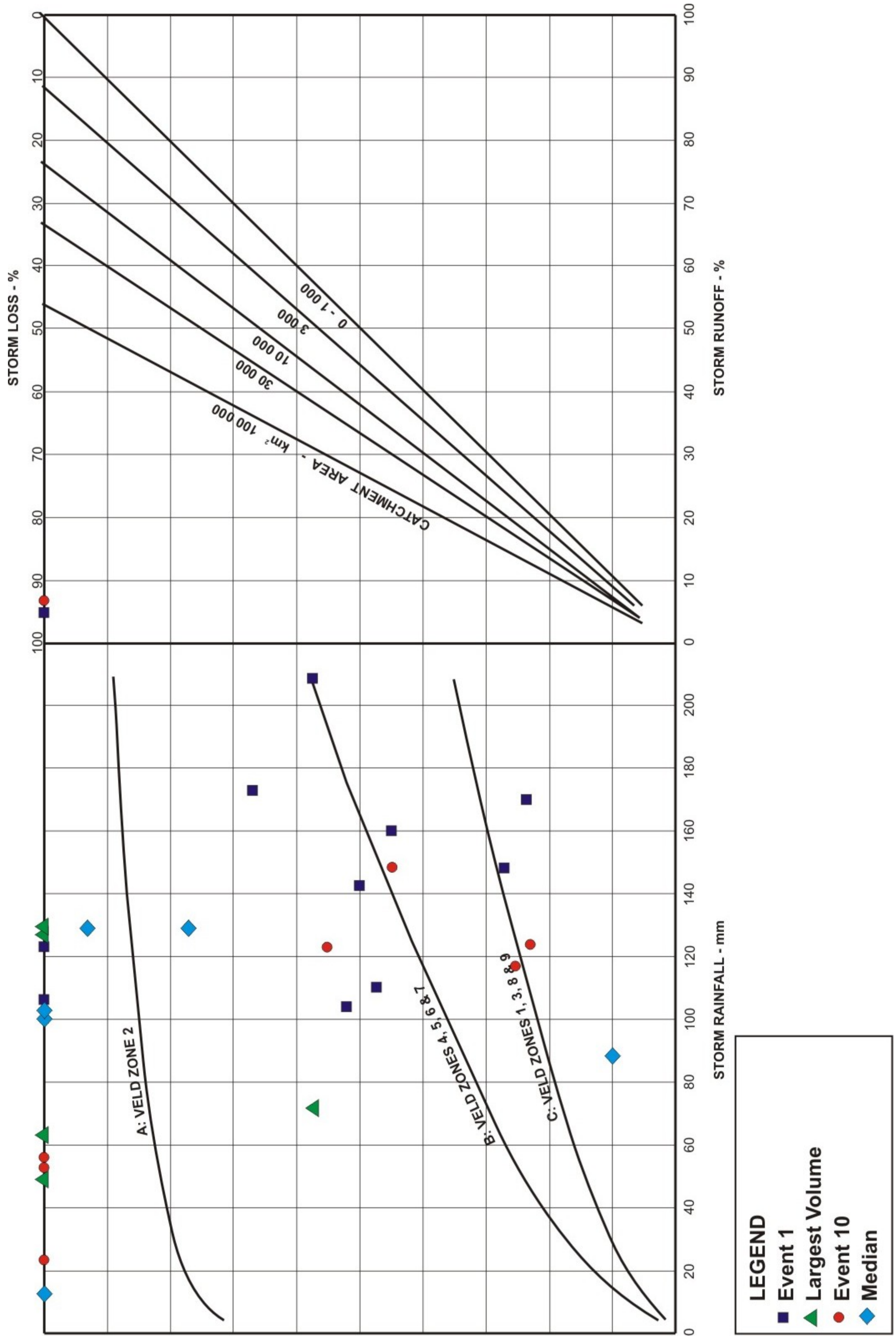

Figure 3.18 Method 4: Regional storm loss curves for recorded events in Veld-Zone 2

Results: Veld-Zone 5

Figure 3.19 shows the selected flow gauging stations and rainfall stations in Veld-Zone 5 located in the north-eastern cape and southern KwaZulu-Natal provinces, and experiences summer rainfall. The following flow gauges were used in the analysis: T3H005, T3H006, T5H004, U2H006 and V2H002.

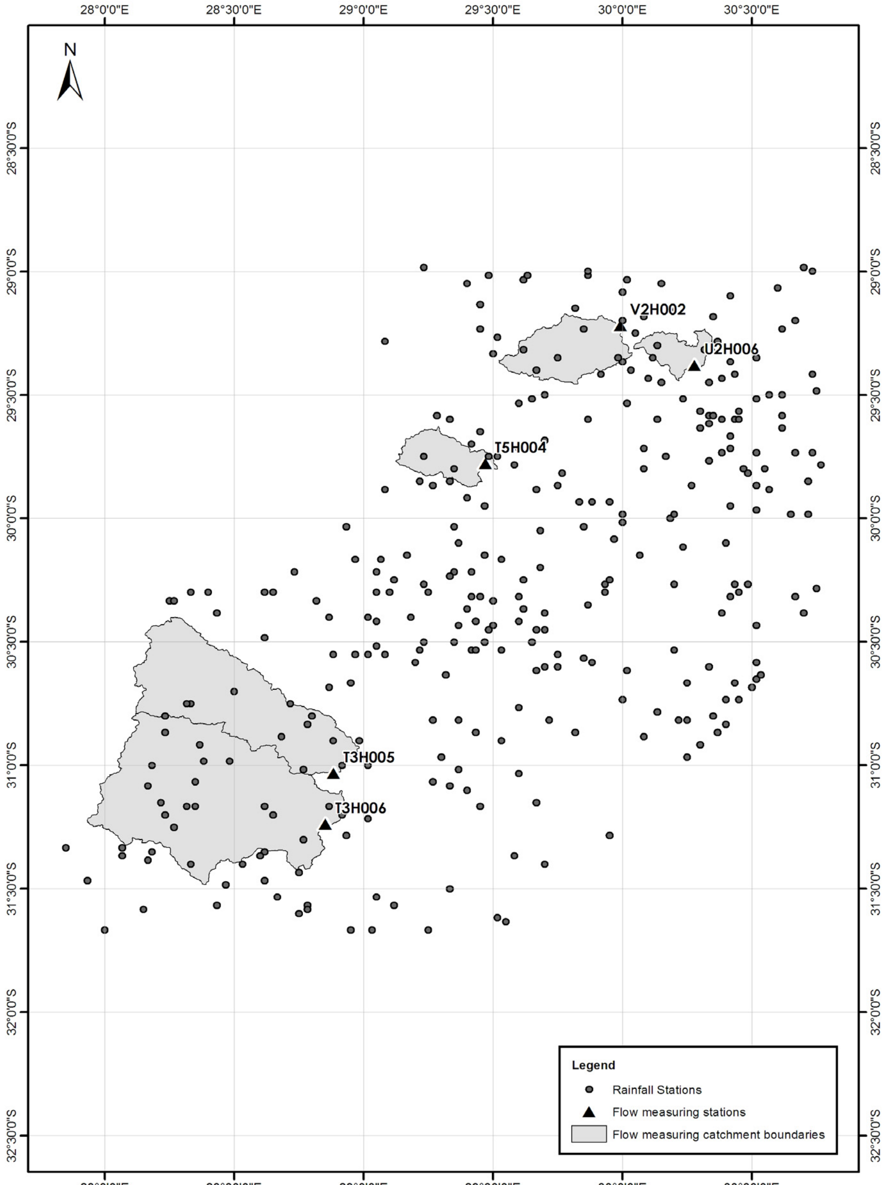

Figure 3.19 Location of flow measuring and rainfall stations in Veld-Zone 5

Table 3.4 presents the flood events identified at each of the gauges in the analysis as well as the storm rainfall associated with that event calculated from GIS. The runoff for the median flood at T5H004 could not be calculated because of a large number of missing values at the rain stations during that period.

The results of the analysis for Veld-Zone 5 are plotted in Figure 3.20. Although the scatter of points is large, it appears as though the HRU loss curve for Veld-Zone 5 might be underestimating the storm runoff percentage, i.e. that the curve plots too low. This observation would however have to be confirmed by further investigation where a greater number of flood events would need to be included.

Table 3.4 Selected flood events for gauges in Veld-Zone 5

Event	Gauge	Area km^2	Date	Flood peak (m^3/s)	Flood volume (Mm^3)	Flood volume (mm)	3-day Rainfall (GIS) (mm)	Runoff %
Largest peak	T3H005	2417	26 January 1996	1192	164	68	61	**100**
Largest peak	T3H006	4049	26 January 1996	995	44	11	39	**28**
Largest peak	T5H004	490	5 March 1976	414	25	50	112	**45**
Largest peak	U2H006	312	21 March 1976	294	59	189	119	**100**
Largest peak	V2H002	883	29 September 1987	1381	97	110	97	**100**
Tenth largest peak	T3H005	2417	10 February 1985	613	30	13	68	**18**
Tenth largest peak	T3H006	4049	1 October 1987	609	102	25	34	**74**
Tenth largest peak	T5H004	490	14 February 1977	109	2	4	75	**5**
Tenth largest peak	U2H006	312	10 April 1984	40	14	45	76	**59**
Tenth largest peak	V2H002	883	14 February 1981	172	26	29	31	**94**
Largest volume	T3H005	2417	21 March 1976	1192	162	67	91	**74**
Largest volume	T3H006	4049	11 March 1963	788	286	71	64	**100**
Largest volume	T5H004	490	25 February 1988	289	63	128	140	**91**
Largest volume	U2H006	312	15 February 1975	31	29	94	67	**100**
Largest volume	V2H002	883	22 March 1976	357	100	113	52	**100**
Median	T3H005	2417	28 January 1986	305	11	5	38	**12**
Median	T3H006	4049	06 March 1969	380	38	9	24	**40**
Median	U2H006	312	10 February 1973	24	14	45	30	**100**
Median	V2H002	883	3 November 1985	71	17	19	24	**79**

Similarly as for Veld-Zone 2, there are instances where the data plots at 100% runoff / 0% loss. This is an indication that there is a lack of representative high-elevation, high-rainfall records in the mountainous catchments situated in this Zone, resulting in design rainfalls that are too low and runoff percentages that are too high.

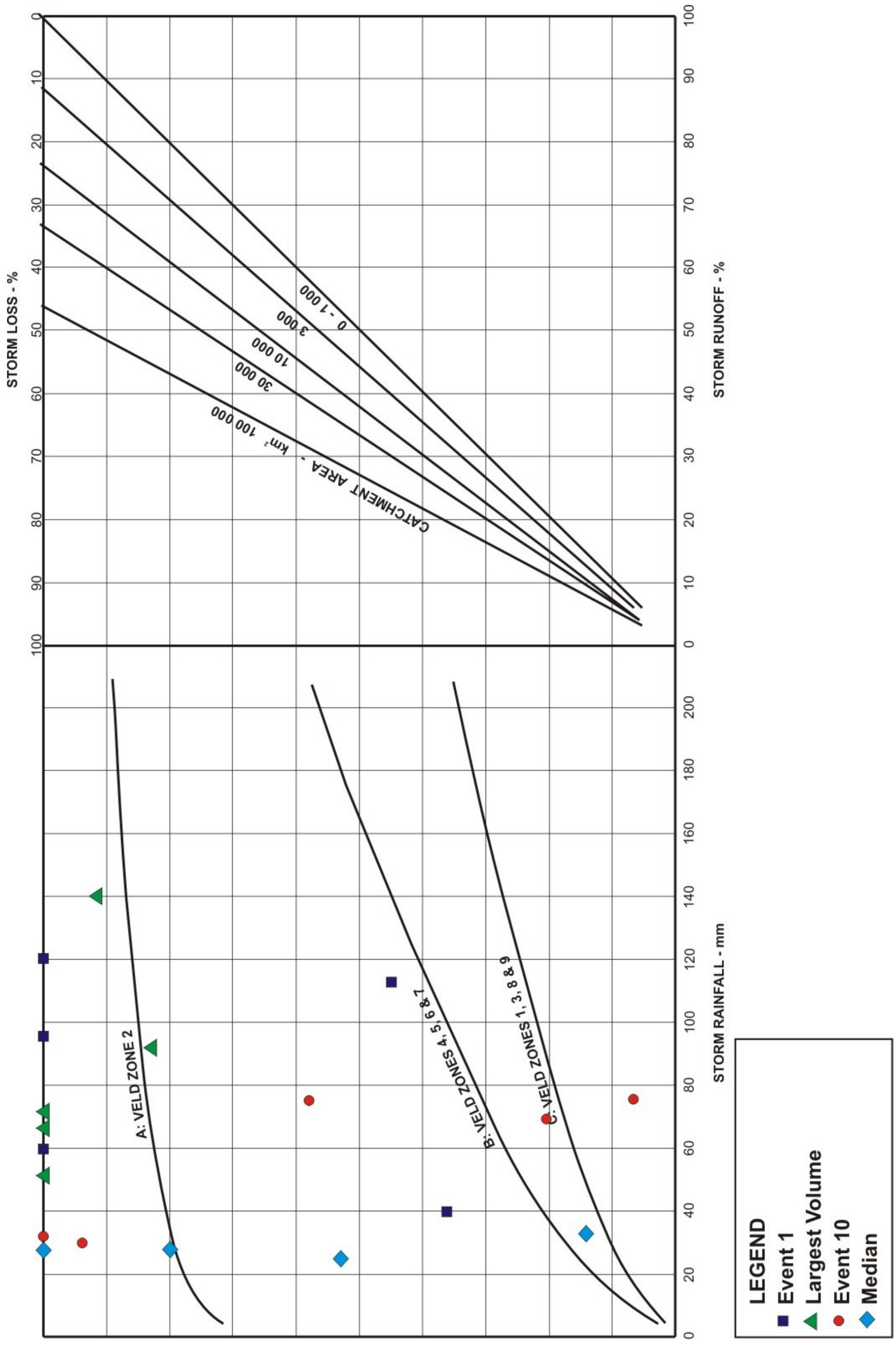

Figure 3.20 Method 4: Regional storm loss curves for recorded events in Veld-Zone 5

Results: Veld-Zone 8

Figure 3.21 shows the selected flow gauging stations and rainfall stations located in Veld-Zone 8 in the Olifants River catchment in the Limpopo Province. The following flow gauges were used in the analysis: B8H009, B8H010, B8R001, B8R002, B8R003, B8R005, B8R006 and B8R007. Following the analysis, data from B8R006 was found to be erroneous and was discarded from the study.

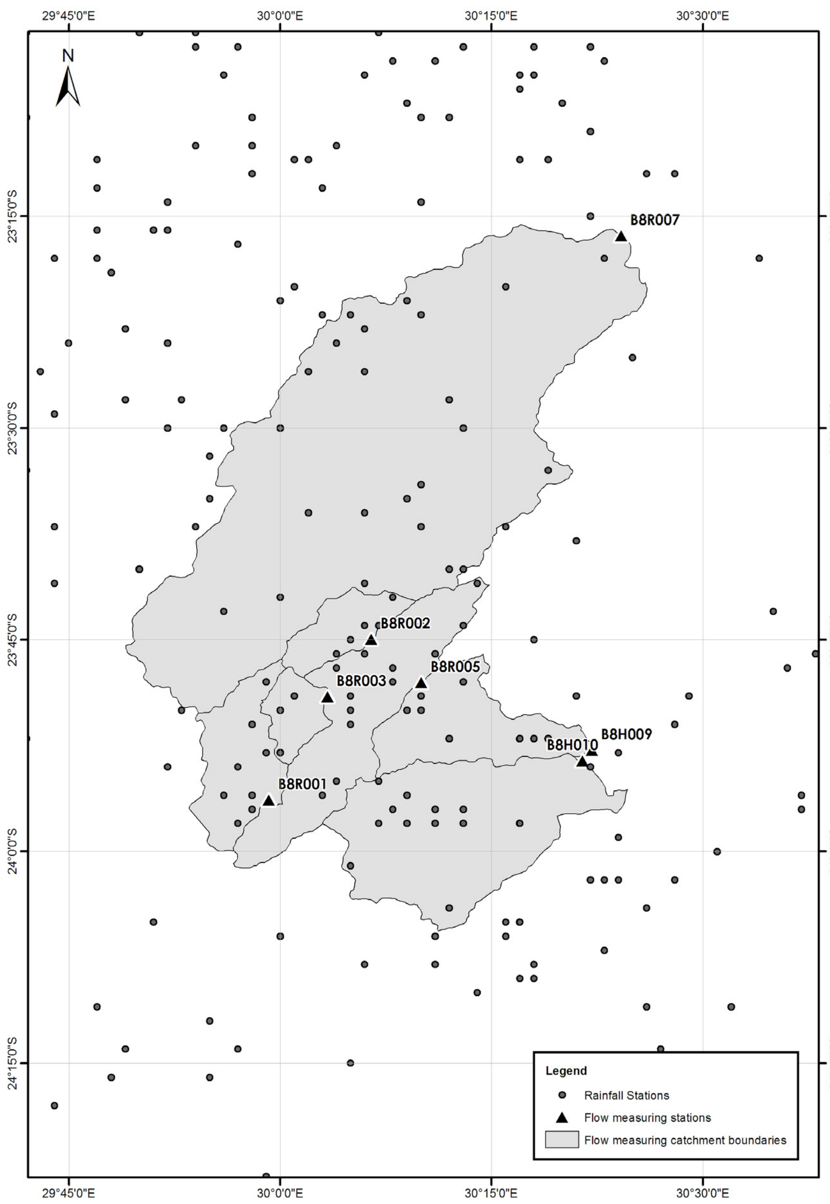

Figure 3.21 Location of flow measuring and rainfall stations in Veld-Zone 8

Table 3.5 presents the flood events identified at each of the gauges in Veld-Zone 8 as well as the storm rainfall associated with that event calculated from GIS. It is noted that the February 2000 flood was the largest at three gauges in the study area.

Table 3.5 Selected flood events for gauges in Veld-Zone 8

Event	Gauge	Area (km^2)	Date	Flood peak (m^3/s)	Flood volume (Mm^3)	Flood volume (mm)	3-day Rainfall (GIS) mm	Runoff %
Largest peak	B8H009	851	31 January 1976	564	67	79	173	**46**
Largest peak	B8H010	477	22 March 1976	526	35	74	101	**73**
Largest peak	B8R001	170	26 February 2000	196	24	140	361	**39**
Largest peak	B8R002	88	25 February 2000	316	15	166	287	**58**
Largest peak	B8R003	64	4 January 1978	165	19	302	287	**100**
Largest peak	B8R005	652	25 February 2000	1069	110	169	350	**48**
1 in 5 year peak	B8H009	851	4 January 1978	229	13	15	46	**33**
1 in 5 year peak	B8H010	477	5 February 1981	300	17	35	62	**56**
1 in 5 year peak	B8R001	170	5 January 1978	48	16	97	168	**58**
1 in 5 year peak	B8R002	88	3 February 1981	52	5	51	232	**22**
1 in 5 year peak	B8R003	64	5 February 1987	36	4	57	152	**37**
1 in 5 year peak	B8R005	652	6 March 1997	239	9	14	124	**11**
Largest volume	B8H009	851	4 January 1978	339	104	122	210	**58**
Largest volume	B8H010	477	13 February 1996	418	41	86	175	**49**
Largest volume	B8R001	170	18 March 1969	9	55	325	69	**100**
Largest volume	B8R002	88	26 February 1988	43	6	65	200	**32**
Largest volume	B8R003	64	25 February 2000	98	21	332	490	**68**
Largest volume	B8R005	652	29 December 1980	290	56	86	19	**100**
Largest volume	B8R007	1799	10 February 1996	1085	102	57	153	**37**
Median	B8H009	851	1 February 1994	92	2	2	120	**2**
Median	B8H010	477	18 January 1969	88	1	2	32	**6**
Median	B8R001	170	24 February 1989	9	2	11	69	**16**
Median	B8R002	88	30 January 1998	21	2	21	103	**20**
Median	B8R003	64	25 March 1991	14	4	60	162	**37**
Median	B8R005	652	31 December 1989	110	2	3	84	**3**
Median	B8R007	1799	18 December 1987	396	4	2	33	**7**

Figure 3.22 shows the 3- and 4-day rainfall-runoff percentage values for events in Veld-Zone 8. In the case of this zone, the x-axis (storm rainfall in mm) of HRU Figure G2 had to be extended to include storm rainfall values that exceed 200 mm.

The HRU curve for Veld-Zone 8 appears to broadly bisect the scatter of points on the graph, although it might be a bit on the low side. Two of the largest volume events have 100% runoff or more, as a result of the storm rainfall being lower than expected for events of this magnitude. This could indicate that the storm rainfall for these catchments is not representative of the true rainfall.

3.5.6 Minimum design storm losses

The HRU 1/72 document presents the minimum storm losses wherein the ratios of observed runoff to total storm rainfall for select extreme rainfall events across South Africa were plotted against catchment area (Figure G1, HRU 1/72). In the HRU analysis, a relatively small data sample was used, which necessitated the proposal of a conservative extreme flood event outer envelope in Figure 3.1.

For the purpose of this research, flood events post-1960, which post-dated the HRU analysis, were selected and plotted on HRU Figure G1 (see Figure 3.23). The following flood events were analysed:

- Laingsburg Floods of 1981
- South Eastern Cape Flood of 1981
- Domoina Floods of 1984
- KwaZulu-Natal Floods of 1987
- Orange River Basin Floods of 1988
- Limpopo Floods of 2000

The extreme rainfall events that were identified in the Veld-Zone 8 analysis, i.e. those where the rainfall is greater than 200 mm, were also included in this analysis.

It is evident from Figure 3.23, that the runoff percentages of more recent extreme floods still fall within the "envelope of recorded floods", which represent the minimum storm losses proposed by the HRU. Only one recorded flood plots out of the HRU envelope and this was recorded during the KwaZulu-Natal floods in September 1987 at flow gauge U2H005 with a recorded flood peak of 3927 m^3/s and flood volume of 689.3 million m^3. The 3-day storm rainfall for this event (28-30 September 1987) was estimated to be 350 mm.

It can also be seen from Figure 3.23 that a number of flood events have less than 20% observed runoff. This is an unexpected result for events associated with extreme rainfall and resultant extreme flood. Several of these events were investigated further and it was found that, because the flood events were so large and, in some cases, were of relatively long duration, the extraction of flood volume from this Project's database was clipped too soon, therefore resulting in lower flood volumes for a given storm. To correct this, it is necessary to manually select the beginning and end of the flood event for analysis. In addition, one must also ensure that the correct storm rainfall duration is selected for that flood event in order to obtain a better estimate of storm runoff. This was not attempted at this stage in the research due to budget constraints, and is recommended should further research be undertaken.

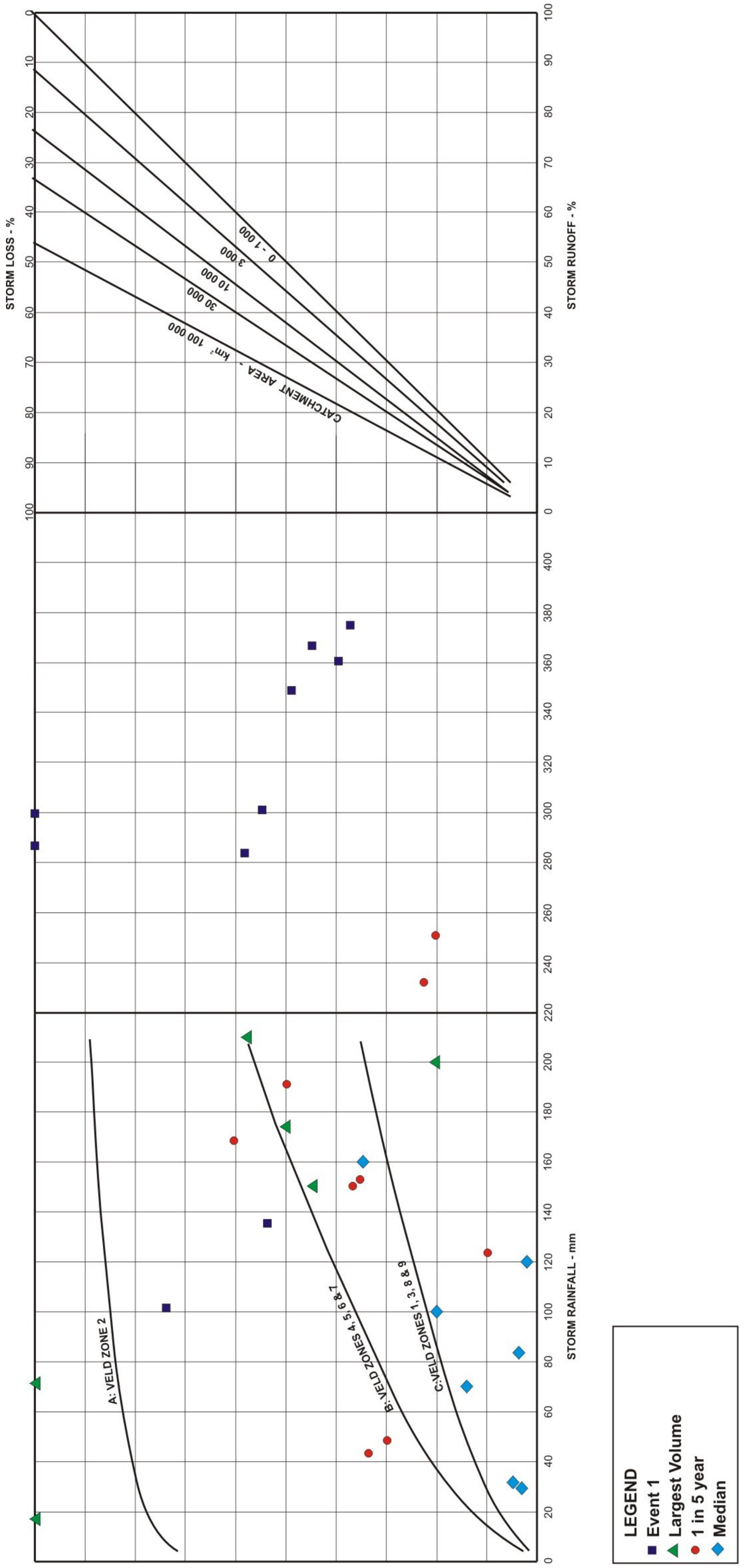

Figure 3.22 Method 4: Regional storm loss curves for recorded events in Veld-Zone 8

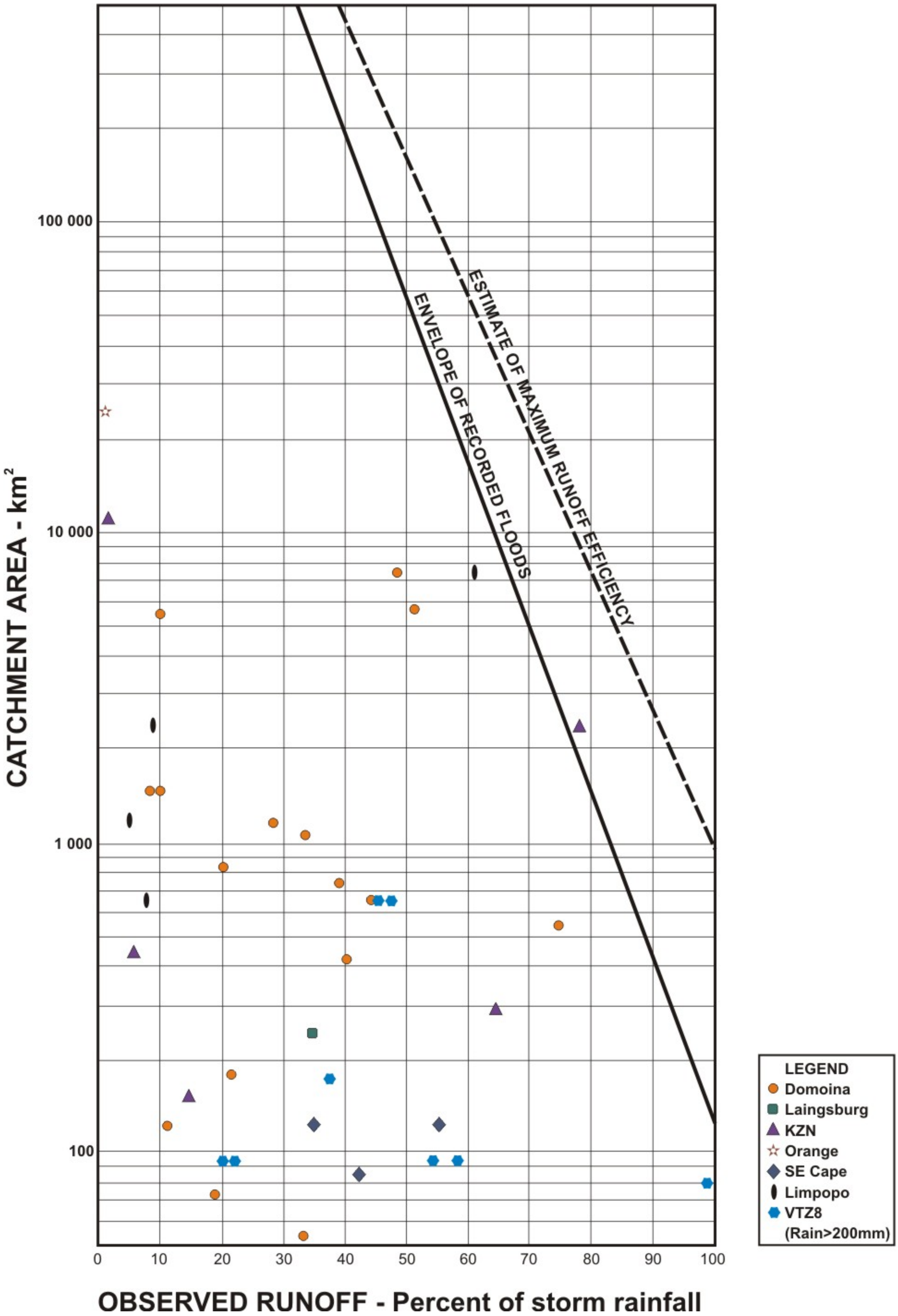

Figure 3.23 Minimum losses curve for extreme floods post-1970

3.6 CONCLUSIONS

For the investigation into regional storm losses in South Africa, this study considered five methods, all of which were bench-marked against the original regional storm losses approach undertaken by the HRU and documented in their 1972 report. Methods 1 to 4 considered the "average" design storm losses approach as reported in HRU 1/72, but with each Method progressively deviating from the latter approach with the increased utilisation of raw data for the estimation of the flood volumes. Method 1, for example, remained true to the HRU methodology for the derivation of the design flood volumes, whereas Method 4 deviated wholly from the HRU methodology by making use of observed historical floods in representative catchments. All methods deviated from the HRU methodology with respect to the design rainfall in that the design rainfall produced by Smithers and Schulze (2002) was used throughout. Method 5 investigated the regional approach to minimum design storm losses by the consideration of historical extreme floods and their causative rainfall.

From the results of the analysis for Methods 1, 3 and 4, one of the overriding observations is that the existing HRU regional storm losses curves (Figure G2 of HRU 1/72) can be seen to be broadly representative of mid-range values for Veld-Zone Group A (Veld-Zone 2) and Veld-Zone C (Veld-Zones 1, 3, 8 and 9). This indicates that the HRU regional storm loss curves, which are described as "average curves" in the 1972 report, might still be considered as reasonably representative for the estimation of "average" design storm losses for these Veld-Zones within South Africa.

Another overriding observation for Veld-Zone Group A was that there were a number of values that exceeded 100% runoff / 0% loss. This phenomenon, which was evident in all of the Methods, is indicative that the design rainfalls in these regions are too low. This can be expected in mountainous regions, such as Veld-Zone 2, where there is a lack of representative high-elevation, high-rainfall records.

A concern brought to light by this research is that the "average" HRU storm loss curve for Veld-Zone Group B (Veld-Zones 4, 5, 6 and 7) might be plotting too low, i.e. that it might be underestimating the storm runoff percentage. This was evident in the results for Veld-Zone 5 using Method 4 (refer to Section 5.5.6). Although it was commented under the latter section that more data would be required to make a solid deduction of this point, the concern lies in that this was also a finding by Lahmeyer Macdonald Consortium and Olivier Shand Consortium (Lahmeyer et al., 1986) in their Lesotho Highlands Water Project hydrological study, which only considered Veld-Zone 4 (refer last paragraph of Section 3.3.1).

For the assessment of the HRU regional minimum storm loss curves, it can be broadly deduced from the results of this research that the representative HRU envelope curves (Figure G1 of HRU 1/72) can still be considered as valid for use within South Africa. For this section of the research, historical extreme flood events were considered and it was found that on only one occasion, the inner envelope curve, labelled as "envelope of recorded floods", was exceeded, and that the outer envelope curve, labelled as "estimate of maximum runoff efficiency", was never exceeded. There were, however, some concerns about the plotting positions of some events in that their representative runoff percentages were quite low for what would be expected from an extreme event, which typically would be biased towards a wet catchment. This appears to be an artefact of the algorithm used in the extraction software used in this study that need a manual override for long-duration storms (4 days or greater). It is recommended that these concerns be addressed should the findings of this research warrant further studies.

4. COMPARISON OF UNITGRAPH-BASED DESIGN FLOOD ESTIMATES WITH PROBABILISTIC ESTIMATES

4.1 INTRODUCTION

As the HRU (1972) Unitgraph method is still the most commonly used design hydrograph generation approach in South African dam safety practice, comparison of Unitgraph-based design flood estimates with probabilistic estimates was a necessary additional investigation in this study. To this end, the probability distributions Log Pearson Type III (LP III) and General Extreme Value (GEV_{pwm}), which are commonly used in South Africa, were employed.

4.2 METHODOLOGY

In this investigation the flood records for the 40 gauged catchments chosen for the storm losses task, reported in Part 3, Section 3.5, were again used. The design flood estimates were determined for recurrence intervals of 1:2, 1:5, 1:10, 1:20, 1:50 and 1:100 years for each of the catchments and grouped according to the Veld-Zone Types of HRU (1972). The comparisons of Unitgraph-based and probabilistic estimates are presented in three different ways in the sections that follow. Table 4.1, Table 4.2 and Table 4.3 present the values used for the analysis per Veld-Zone grouping.

4.2.1 Scatterplots

The two sets of pairs of estimates were compared in scatterplots, one each for Veld-Zone Groups A, B or C as shown in Figure 4.1, Figure 4.2 and Figure 4.3.

Table 4.1 Design flood estimation information for Veld-Zone Group A

Station	Area (km^2)	Veld Zone Type	Critical duration (hrs)	RI (yrs)	Design flood peak estimates: LP III (m^3/s)	GEV_{pwm} (m^3/s)	Unitgraph (m^3/s)	% Difference Unitgraph from LP III	Ratio $Q_T/Q_{1:2}$ for LP III	Ratio $Q_T/Q_{1:2}$ for Unitgraph
G1H004	70	2	4	1:2	250	254	132	-47	1	1
				1:5	374	369	183	-51	1	1
				1:10	449	439	217	-52	2	2
				1:20	515	502	253	-51	2	2
				1:50	592	576	313	-47	2	2
				1:100	644	628	348	-46	3	3
G1H008	395	2	4	1:2	98	101	379	287	1	1
				1:5	191	185	495	159	2	1
				1:10	264	252	568	115	3	1
				1:20	342	326	641	88	3	2
				1:50	450	438	715	59	5	2
				1:100	536	536	814	52	5	2
G2H008	20	2	2	1:2	30	30	56	86	1	1
				1:5	36	36	85	139	1	2
				1:10	39	39	102	160	1	2
				1:20	43	42	119	180	1	2
				1:50	47	45	139	199	2	2
				1:100	50	47	150	200	2	3

Table 4.1 (cont.)

Station	Area (km^2)	Veld Zone Type	Critical duration (hrs)	RI (yrs)	Design flood peak estimates: LP III (m^3/s)	GEV$_{pwm}$ (m^3/s)	Unitgraph (m^3/s)	% Difference Unitgraph from LP III	Ratio $Q_T/Q_{1:2}$ for LP III	Ratio $Q_T/Q_{1:2}$ for Unitgraph
H1H006	753	2	12	1:2	363	384	302	-17	1	1
				1:5	549	546	401	-27	2	1
				1:10	662	633	461	-30	2	2
				1:20	762	704	512	-33	2	2
				1:50	880	781	601	-32	2	2
				1:100	961	829	651	-32	3	2
H1H007	84	2	4	1:2	212	216	163	-23	1	1
				1:5	270	272	215	-20	1	1
				1:10	308	305	255	-17	1	2
				1:20	344	334	287	-17	2	2
				1:50	392	368	334	-15	2	2
				1:100	428	392	365	-15	2	2
H1H018	113	2	4	1:2	350	366	203	-42	1	1
				1:5	518	517	271	-48	1	1
				1:10	621	600	311	-50	2	2
				1:20	713	669	359	-50	2	2
				1:50	824	745	411	-50	2	2
				1:100	900	794	451	-50	3	2
H4R002	377	2	8	1:2	8	7	254	3078	1	1
				1:5	30	22	367	1127	4	1
				1:10	62	43	461	643	8	2
				1:20	116	78	549	373	14	2
				1:50	239	161	678	184	30	3
				1:100	392	274	790	101	49	3
H7H005	9	2	1	1:2	20	20	13	-35	1	1
				1:5	25	25	19	-24	1	1
				1:10	29	29	24	-17	1	2
				1:20	33	33	30	-10	2	2
				1:50	39	38	39	-2	2	3
				1:100	44	42	45	3	2	4
K2H002	131	2	4	1:2	36	41	85	136	1	1
				1:5	91	91	127	40	3	1
				1:10	151	139	162	7	4	2
				1:20	232	199	200	-14	6	2
				1:50	382	305	258	-32	11	3
				1:100	536	414	304	-43	15	4

Table 4.2 Design flood estimation information for Veld-Zone Group B

Station	Area (km^2)	Veld Zone Type	Critical duration (hrs)	RI (yrs)	Design flood peak estimates: LP III (m^3/s)	GEV$_{pwm}$ (m^3/s)	Unitgraph (m^3/s)	% Difference Unitgraph from LP III	Ratio $Q_T/Q_{1:2}$ for LP III	Ratio $Q_T/Q_{1:2}$ for Unitgraph
A2H012	2551	4	7	1:2	135	146	511	279	1	1
				1:5	281	279	873	211	2	2
				1:10	412	390	1145	178	3	2
				1:20	566	516	1511	167	4	3
				1:50	811	715	2046	152	6	4
				1:100	1030	897	2629	155	8	5
B1R001	3541	4	8	1:2	145	167	518	257	1	1
				1:5	424	402	794	87	3	2
				1:10	719	633	984	37	5	2
				1:20	1093	934	1309	20	8	3
				1:50	1716	1489	1639	-4	12	3
				1:100	2294	2074	1969	-14	16	4
B1R002	1576	4	8	1:2	48	53	279	482	1	1
				1:5	103	105	434	320	2	2
				1:10	160	152	561	250	3	2
				1:20	234	209	743	218	5	3
				1:50	366	308	946	159	8	3
				1:100	498	405	1096	120	10	4
C2H001	3595	4	8	1:2	208	248	735	253	1	1
				1:5	614	578	1104	80	3	2
				1:10	1029	883	1413	37	5	2
				1:20	1535	1261	1843	20	7	3
				1:50	2343	1920	2401	2	11	3
				1:100	3058	2577	2767	-10	15	4
J2H005	253	6	2	1:2	6	8	71	1080	1	1
				1:5	19	19	142	636	3	2
				1:10	35	32	188	433	6	3
				1:20	59	50	235	297	10	3
				1:50	108	87	353	228	18	5
				1:100	162	129	447	176	27	6
T3H006	4268	5	8	1:2	396	428	554	40	1	1
				1:5	684	669	912	33	2	2
				1:10	874	813	1251	43	2	2
				1:20	1050	940	1609	53	3	3
				1:50	1263	1089	2165	71	3	4
				1:100	1413	1192	2643	87	4	5
T5H004	545	5	2	1:2	79	79	334	323	1	1
				1:5	132	127	586	345	2	2
				1:10	180	172	712	296	2	2
				1:20	237	226	963	306	3	3
				1:50	333	321	1216	265	4	4
				1:100	423	415	1550	266	5	5
U2H006	339	5	2	1:2	25	26	206	722	1	1
				1:5	53	52	360	582	2	2
				1:10	84	79	490	484	3	2
				1:20	127	116	645	406	5	3
				1:50	211	187	903	328	8	4

Station	Area (km^2)	Veld Zone Type	Critical duration (hrs)	RI (yrs)	Design flood peak estimates LP III (m^3/s)	GEV$_{pwm}$ (m^3/s)	Unitgraph (m^3/s)	% Difference Unitgraph from LP III	Ratio $Q_T/Q_{1:2}$ for LP III	Ratio $Q_T/Q_{1:2}$ for Unitgraph
				1:100	303	267	1161	283	12	6
V2H002	937	5	4	1:2	77	84	397	415	1	1
				1:5	173	176	623	259	2	2
				1:10	289	276	819	184	4	2
				1:20	462	416	1075	132	6	3
				1:50	828	696	1441	74	11	4
				1:100	1262	1017	1777	41	16	4
X1R001	1569	4	4	1:2	82	89	277	238	1	1
				1:5	167	167	415	148	2	1
				1:10	244	230	558	129	3	2
				1:20	332	301	726	119	4	3
				1:50	470	411	932	98	6	3
				1:100	592	509	1207	104	7	4
X2H011	402	4	2	1:2	100	110	143	43	1	1
				1:5	179	178	214	20	2	2
				1:10	235	221	310	32	2	2
				1:20	291	261	381	31	3	3
				1:50	364	309	500	37	4	4
				1:100	419	344	572	37	4	4
X2R005	954	4	2	1:2	74	74	235	217	1	1
				1:5	122	118	391	222	2	2
				1:10	155	151	549	254	2	2
				1:20	188	185	665	254	3	3
				1:50	231	233	824	256	3	4
				1:100	264	272	980	271	4	4

Table 4.3 Design flood estimation information for Veld-Zone Group C

Station	Area (km^2)	Veld Zone Type	Critical duration (hrs)	RI (yrs)	Design flood peak estimates			% Difference Unitgraph from LP III	Ratio QT/Q1:2 for LP III	Ratio QT/Q1:2 for Unitgraph
					LP III (m^3/s)	GEV$_{pwm}$ (m^3/s)	Unitgraph (m^3/s)			
A2H013	1171	8	3	1:2	48	68	311	549	1	1
				1:5	170	171	520	205	4	2
				1:10	322	269	679	111	7	2
				1:20	535	394	834	56	11	3
				1:50	931	615	1143	23	19	4
				1:100	1334	841	1356	2	28	4
A2R003	492	8	2	1:2	18	22	209	1060	1	1
				1:5	56	59	292	423	3	1
				1:10	110	101	335	206	6	2
				1:20	199	163	502	152	11	2
				1:50	412	294	1046	154	23	5
				1:100	688	453	1635	138	38	8
A2H006	1028	8	4	1:2	29	33	213	633	1	1
				1:5	89	85	355	300	3	2
				1:10	166	146	463	180	6	2
				1:20	282	236	642	127	10	3
				1:50	527	429	855	62	18	4
				1:100	808	663	1141	41	28	5
B1H004	376	8	2	1:2	11	12	115	950	1	1
				1:5	20	20	202	937	2	2
				1:10	27	25	908	3305	2	8
				1:20	34	31	1126	3185	3	10
				1:50	45	38	1453	3116	4	13
				1:100	54	44	1726	3096	5	15
B7H004	136	8	1	1:2	32	43	67	111	1	1
				1:5	103	105	119	16	3	2
				1:10	188	163	170	-9	6	3
				1:20	308	236	221	-28	10	3
				1:50	534	365	307	-43	17	5
				1:100	769	496	375	-51	24	6
B7R001	165	8	2	1:2	38	40	154	305	1	1
				1:5	108	98	256	137	3	2
				1:10	170	154	360	111	4	2
				1:20	237	229	479	102	6	3
				1:50	329	367	668	103	9	4
				1:100	400	513	857	114	11	6
B8H010	477	8	2	1:2	86	112	346	302	1	1
				1:5	223	232	578	159	3	2
				1:10	366	326	811	121	4	2
				1:20	549	427	1080	97	6	3
				1:50	864	579	1506	74	10	4
				1:100	1166	710	1969	69	14	6
U2H011	176	8	2	1:2	58	62	76	31	1	1
				1:5	131	127	137	5	2	2
				1:10	200	185	168	-16	3	2
				1:20	282	255	214	-24	5	3
				1:50	414	374	306	-26	7	4
				1:100	534	489	628	18	9	8

Station	Area (km^2)	Veld Zone Type	Critical duration (hrs)	RI (yrs)	Design flood peak estimates LP III (m^3/s)	GEV$_{pwm}$ (m^3/s)	Unitgraph (m^3/s)	% Difference Unitgraph from LP III	Ratio $Q_T/Q_{1:2}$ for LP III	Ratio $Q_T/Q_{1:2}$ for Unitgraph
U2H012	438	8	2	1:2	23	25	30	29	1	1
				1:5	50	49	54	8	2	2
				1:10	79	74	84	6	3	3
				1:20	120	109	114	-5	5	4
				1:50	200	178	168	-16	9	6
				1:100	289	256	228	-21	13	8
V1H009	196	9	2	1:2	131	136	110	-16	1	1
				1:5	229	223	177	-23	2	2
				1:10	295	282	220	-26	2	2
				1:20	359	339	286	-20	3	3
				1:50	438	415	375	-14	3	3
				1:100	496	474	420	-15	4	4
W5H005	804	9	4	1:2	39	40	175	349	1	1
				1:5	74	73	313	321	2	2
				1:10	107	103	419	292	3	2
				1:20	147	139	522	255	4	3
				1:50	213	200	726	240	5	4
				1:100	276	260	931	237	7	5
W5R003	548	9	3	1:2	42	44	218	418	1	1
				1:5	84	82	325	287	2	1
				1:10	121	115	435	261	3	2
				1:20	163	153	575	254	4	3
				1:50	228	216	753	231	5	3
				1:100	285	275	859	201	7	4
X1H001	5499	9	8	1:2	201	209	803	299	1	1
				1:5	454	433	1200	164	2	1
				1:10	698	647	1614	131	3	2
				1:20	996	921	2138	115	5	3
				1:50	1488	1415	2803	88	7	3
				1:100	1947	1924	3090	59	10	4
X2H015	1554	8	4	1:2	137	143	270	97	1	1
				1:5	225	213	429	91	2	2
				1:10	269	249	537	100	2	2
				1:20	300	279	753	151	2	3
				1:50	329	311	965	193	2	4
				1:100	345	330	1129	227	3	4
X2R004	263	8	2	1:2	14	14	148	957	1	1
				1:5	50	42	231	360	4	2
				1:10	95	72	297	212	7	2
				1:20	158	117	412	161	11	3
				1:50	273	209	560	105	20	4
				1:100	390	317	726	86	28	5
X2H008	180	3	4	1:2	28	33	47	69	1	1
				1:5	74	71	87	18	3	2
				1:10	117	103	111	-6	4	2
				1:20	169	141	147	-13	6	3
				1:50	251	202	192	-23	9	4
				1:100	323	258	252	-22	12	5

4.2.2 Percentage differences

The quartile range values of the percentage difference between the Unitgraph values and the LP III values were investigated by quartile box-plots arranged by RI, as depicted in Figure 4.4, Figure 4.5 and Figure 4.6.

4.2.3 Standardised ratios

The pairs of design flood values estimated via the LP III and Unitgraph-based methods were standardised by the relevant $Q_{1:2}$ median values, and the quartile ranges of the standardised values compared by box-plots arranged by RI. These comparisons appear in Figure 4.7, Figure 4.8 and Figure 4.9.

4.3 RESULTS AND DISCUSSION

4.3.1 Scatterplots

Figures 4.1, 4,2 and 4.3 show that, in general, the Unitgraph-based approach produced higher design flood peak estimates for Veld-Zone Groups B and C than the two single-site probability analysis approaches. In contrast, for Veld-Zone Group A the distribution of the sets of pairs of estimates around the 1:1 line of perfect fit is markedly better.

It is of interest to note that, in general, the LP III and GEV_{pwm} values corresponded reasonably well.

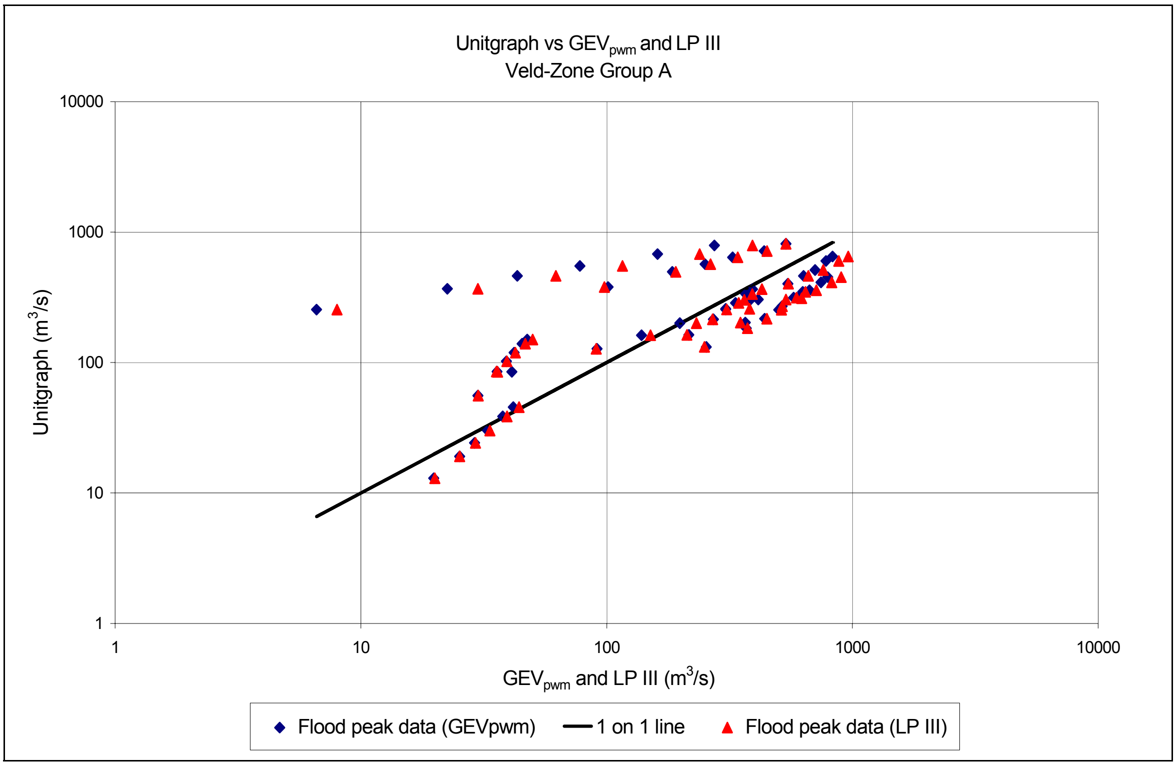

Figure 4.1 Comparison of Unitgraph with LP III and GEV_{pwm} design flood estimates for Veld-Zone Group A

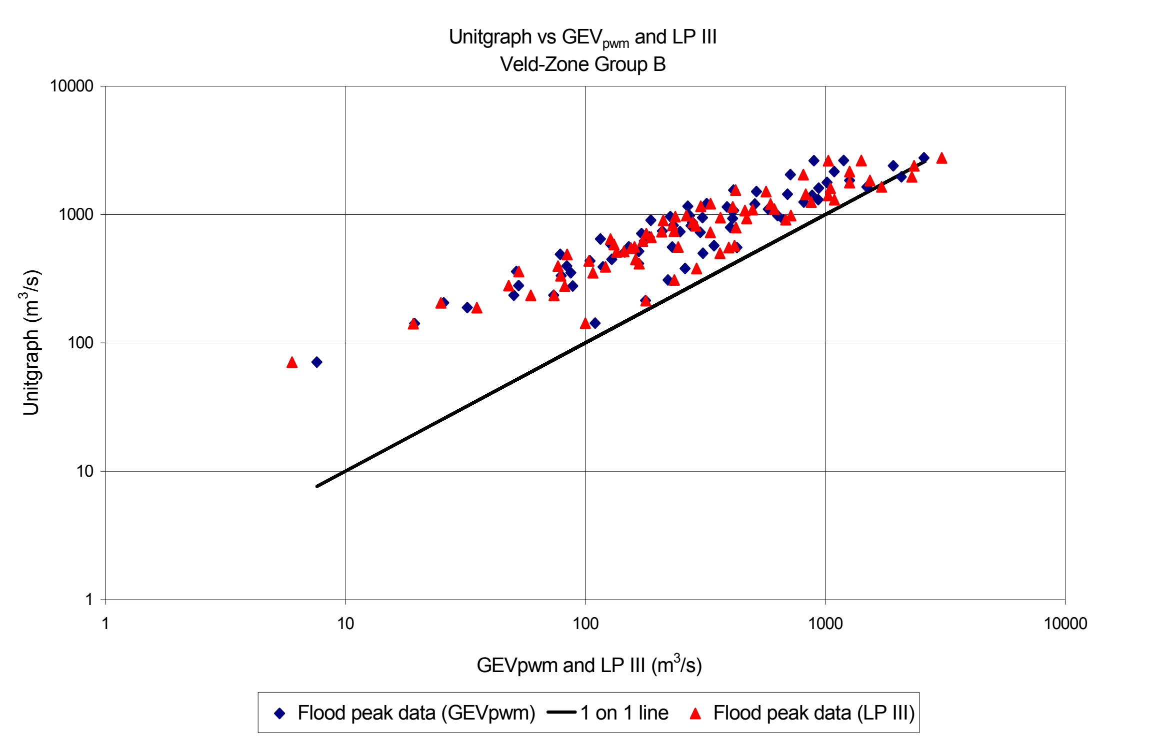

Figure 4.2 **Comparison of Unitgraph with LP III and GEV$_{pwm}$ design flood estimates for Veld-Zone Group B**

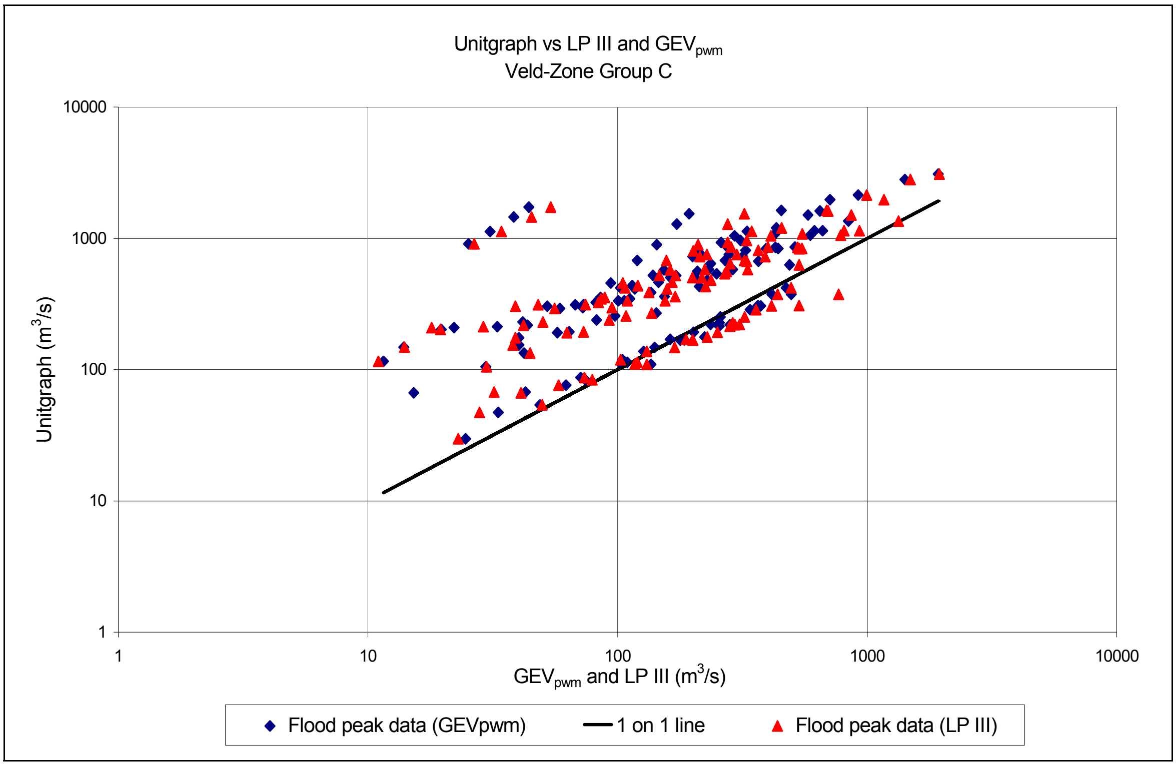

Figure 4.3 **Comparison of Unitgraph with LP III and GEV$_{pwm}$ design flood estimates for Veld-Zone Group C**

4.3.2 Quartile Box-Plots

Figures 4.4, 4.5 and 4.6 show that the Unitgraph-based estimates tend to exceed the probabilistic estimates across all the RIs. The quartile range of the proportional differences is alarmingly wide across all RIs. For Veld-Zone Groups B and C the proportional differences tend to be considerably higher for the 1:2 and 1:5 year cases, where the median over-estimation by the Unitgraph-based approach is in the region of 200%. The lowest median over-estimation for Veld-Zone Groups B and C is about 60% (for RI=100).

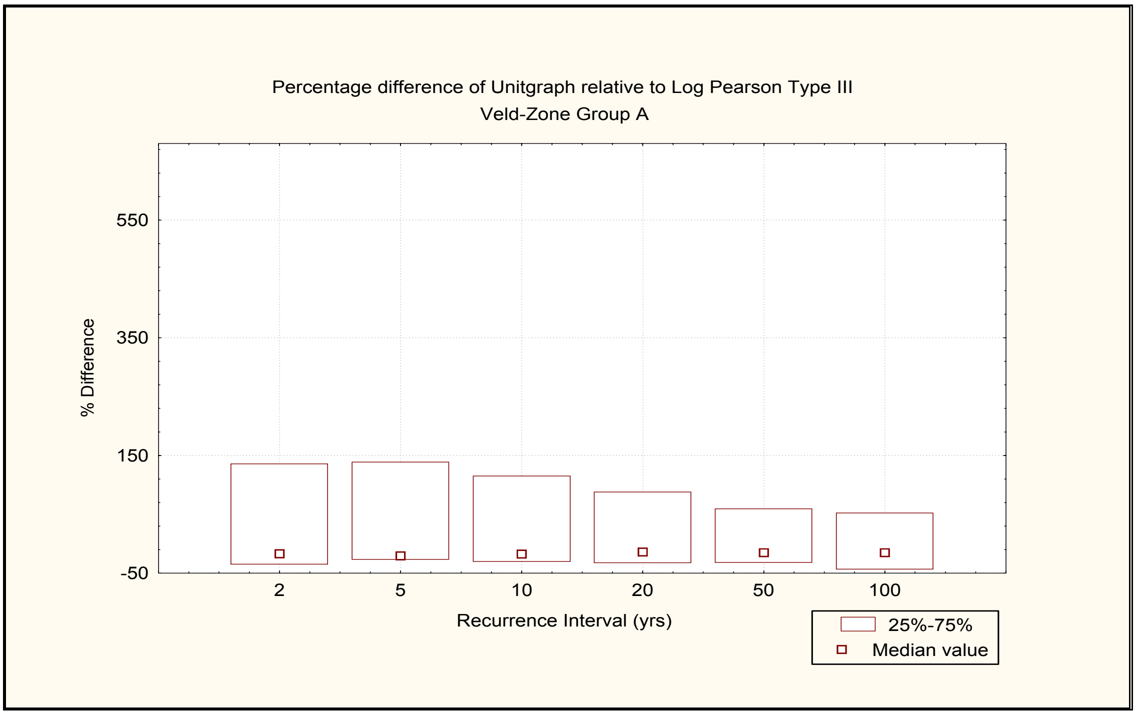

Figure 4.4 Percentage difference of Unitgraph relative to Log Pearson Type III design flood peak estimates for Veld-Zone Group A

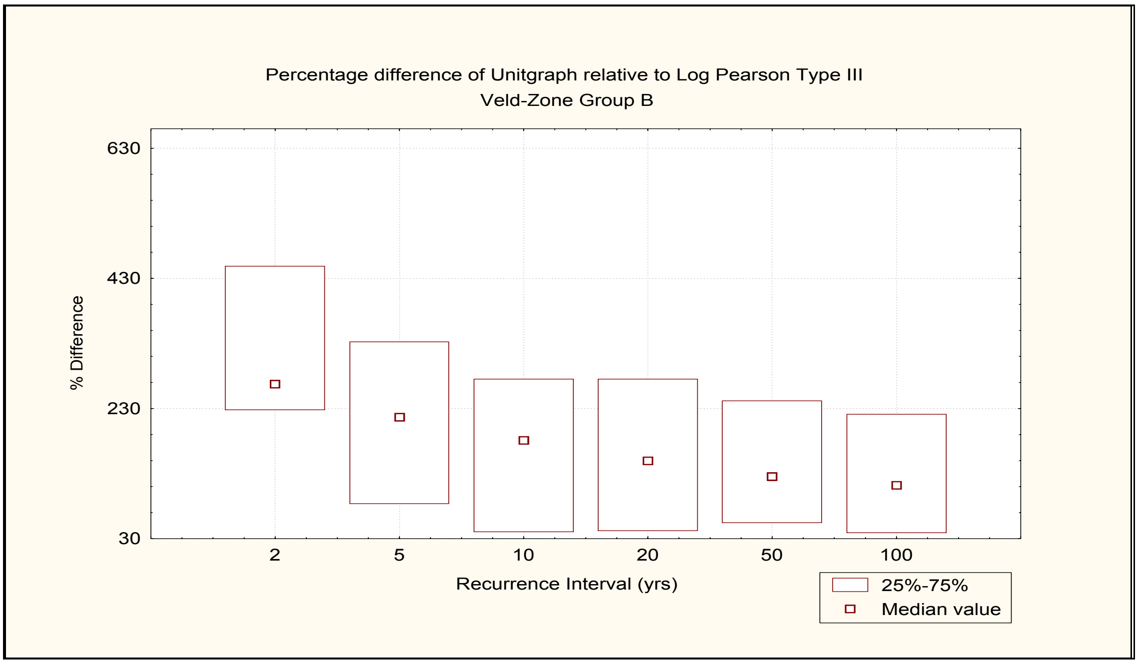

Figure 4.5 Percentage difference of Unitgraph relative to Log Pearson Type III design flood peak estimates for Veld-Zone Group B

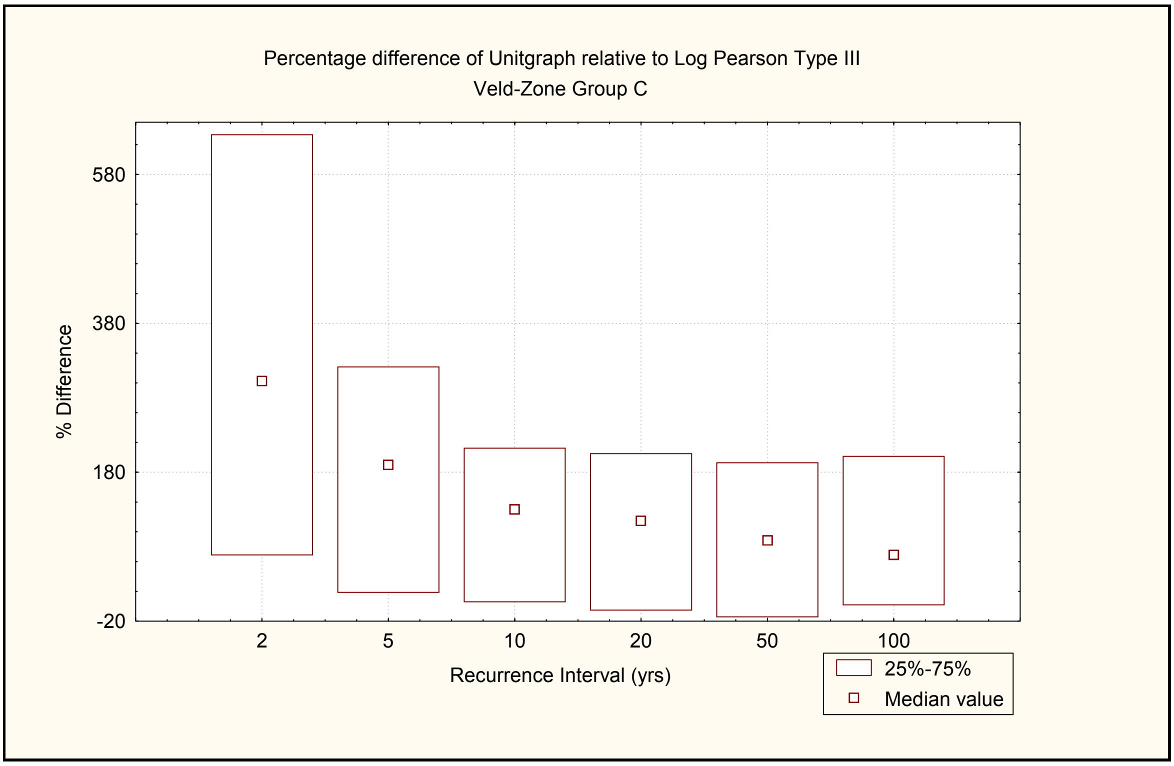

Figure 4.6 **Percentage difference of Unitgraph relative to Log Pearson Type III design flood peak estimates for Veld-Zone Group C**

4.3.3 Standardised Quartile Box-Plots

By expressing each catchment's flood peak estimates proportional to the 1:2 RI value for that catchment, a form of scaling or standardisation is achieved. This makes comparisons among the wide range of catchments in the sample more meaningful. Figures 4.7, 4.8 and 4.9 present quartile box-plots of these standardised design flood peak estimates. For Veld-Zone Groups B and C the LP III approach displays markedly higher variability in scaled flood peak estimates than the Unitgraph-based approach, over all RIs. The lack of relative variability in the Unitgraph-based approach across the range of RIs could be a cause for concern. On the other hand, the higher RIs (1:50 and 1:100) form part of the tail of the probability distribution, which is by definition very sensitive to record length. Flood peak estimates utilising the distribution tail in relatively small samples, such as those in this study, could therefore be expected to be highly variable.

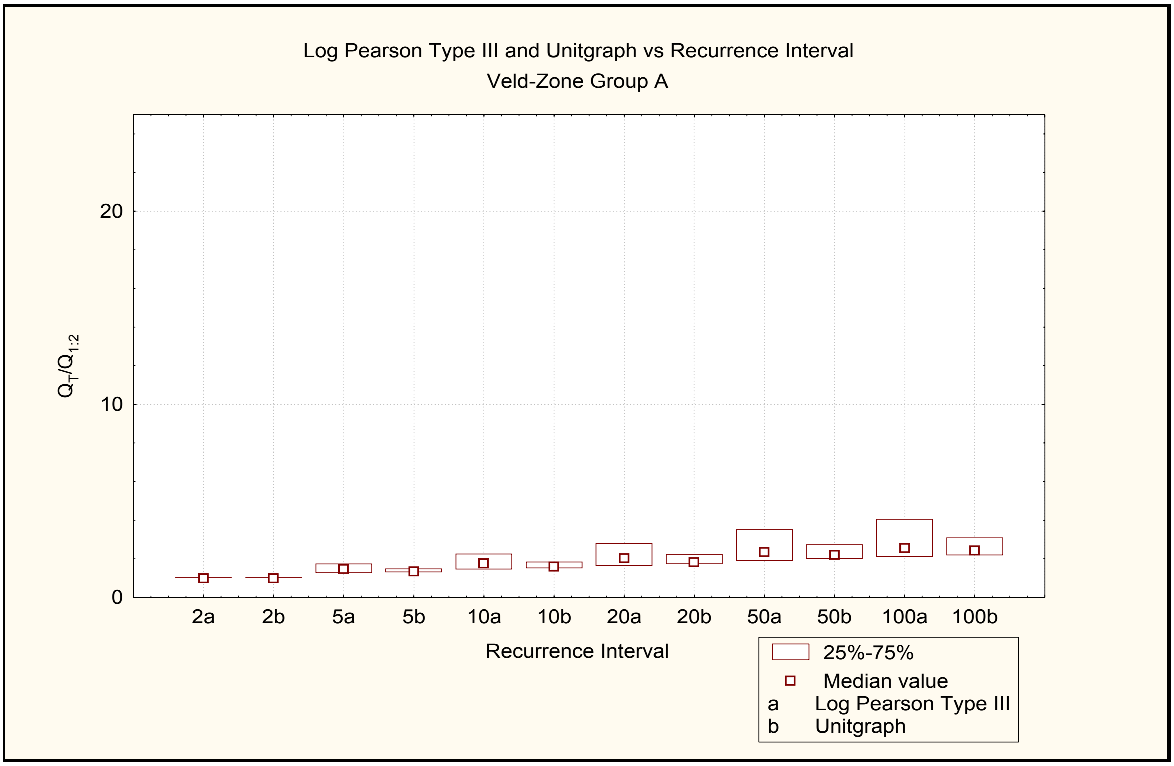

Figure 4.7 Standardised quartile range differences by RI for Veld-Zone Group A

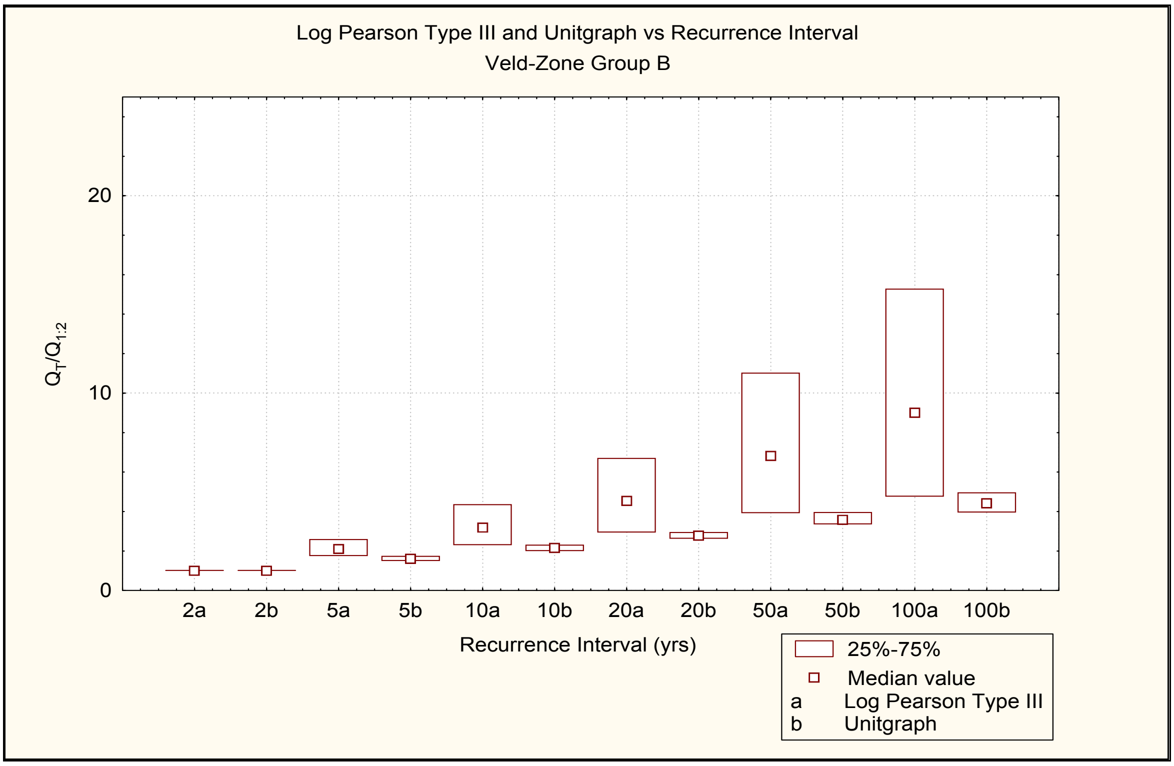

Figure 4.8 Standardised quartile range differences by RI for Veld-Zone Group B

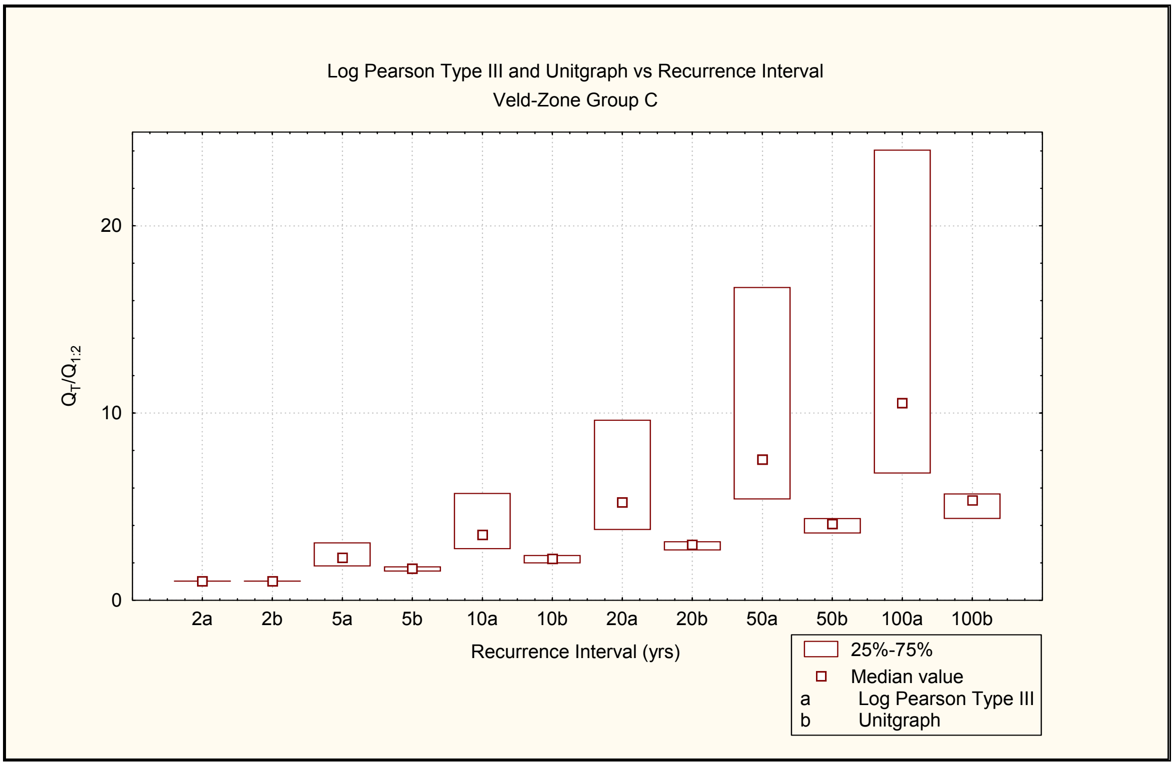

Figure 4.9 Standardised quartile range differences by RI for Veld-Zone Group C

4.4 CONCLUSION

The scatterplots show that the Unitgraph-based approach produced higher design flood peak estimates for Veld-Zone Groups B and C than the two single-site probability analysis approaches. The quartile range of the proportional differences is wide across all RIs. The standardised quartile box-plots indicates that the LP III approach is characterised by higher variability in scaled flood peak estimates than the Unitgraph-based approach, over all RIs, for Veld-Zone Groups B and C. The lack of relative variability in the estimates by the Unitgraph-based approach across the range of RIs could be a cause for concern.

5. REVIEW OF EXTREME DESIGN FLOOD APPROACHES IN SOUTH AFRICA

5.1 INTRODUCTION

The general practice employed in South Africa for the safe design of dams is to follow the guidelines set out in SANCOLD's *Guidelines on Safety in Relation to Floods* (SANCOLD, 1991). To ensure the safety of dams under extreme flood conditions, the SANCOLD guidelines make reference to the Regional Maximum Flood (RMF), an empirically derived flood based on the ordering of maximum observed flood peaks from 130 sites around South Africa, and the Probable Maximum Flood (PMF), the theoretical largest possible flood that could occur at the structure's location derived from the Probable Maximum Precipitation (PMP). The recommended use of these "extreme" design floods within the guidelines, which is dependant upon the hazard rating assigned to the dam, has raised some concern among designers who question their applicability.

In order to inform the debate surrounding the RMF and PMF and so improve understanding as to when either of these extreme design floods should be applied for dam design, this research attempts to link a recurrence interval (RI) or Annual Exceedence Probability (AEP) to the RMF and PMF. By expressing these extreme design floods in RI of AEP terms, the South African dam safety guidelines can also be viewed in the context of the design methodologies followed by other countries for the design of dams under extreme flood conditions.

Sections 2 and 3 of this report present a literature review of the approaches taken with regards to extreme floods for the safe design of dams in South Africa and abroad, respectively. Sections 4 and 5 then present the investigation, findings and conclusions with regards to linking a RI to the RMF and PMF for South Africa. Finally, Section 6 of this report presents the recommendations based on the findings of this research.

5.2 EXTREME FLOODS IN SOUTH AFRICAN DAM SAFETY GUIDELINES

5.2.1 Dam Safety Guidelines (SANCOLD)

All dams in South Africa are categorised according to their size and hazard potential. The category of dam defines the requirements in respect of spillway design in terms of the SANCOLD *Guidelines on Safety in Relation to Floods* (SANCOLD,1991). The *SANCOLD Guidelines* recommend two levels of safety evaluation: (i) generalised, and (ii) site-specific.

Small, low hazard dams, i.e. Category I dams, only require a generalised safety evaluation as this is much easier and cheaper than the more detailed site-specific requirements. The generalised safety evaluation is also used as the initial screen in the safety evaluation process to determine whether or not more detailed site-specific calculations need to be done for these and other category dams. However, it is stated "*for all new HIGH hazard dams, as well as for medium and large dams having SIGNIFICANT hazard ratings, it is obligatory that site-specific analysis shall be the bases of the safety status under extreme flood conditions"* (SANCOLD, 1990, pp 33).

Irrespective of whether generalised design criteria or site-specific methods are used, consideration has to be given for two flood scenarios:

a. "design flood conditions", during which, provided normal maintenance work is executed on a regular basis, the spillway will operate without damage to any of its components or to the associated structure, and

b. "extreme flood conditions", under which spillway operation may result in substantial damage to its components and/or to parts of the dam structure but would not result in catastrophic failure of the dam.

In terms of the safety of dams in relation to floods, the second of these flood scenarios is the most important, although the proposed methodology for determining the recommended extreme flood raises some concerns that are reflected in the recommendations for determining the design floods, particularly with regards to the use of the Regional Maximum Flood (RMF).

Under the extreme flood scenario the dam spillway must be capable of discharging the Safety Evaluation Discharge (SED) so that, although there may be extensive damage to the structure, it will not fail. The SED is expressed in terms of the Regional Maximum Flood (RMF) as in Table 5.1, where subscript (-Δ) means choose the RMF K-value region numerically one step lower and subscript (+Δ) means choose the region numerically one step higher.

Table 5.1 Recommended Safety Evaluation Discharge

Size Class	Hazard Rating		
	Low	Significant	High
Small	$RMF_{-\Delta}$	$RMF_{-\Delta}$	RMF
Medium	$RMF_{-\Delta}$	RMF	$RMF_{+\Delta}$
Large	RMF	$RMF_{+\Delta}$	$RMF_{+\Delta}$

The RMF, which in South Africa replaced the earlier "Craeger-value" method still used in some parts of the world (Liu, 2002), has been calculated by applying the Francou-Rodier (1967) empirical regional envelope method to Southern African conditions in DWA Technical Report 137 (Kovaćs, 1988). This method uses observed extreme floods in a region to determine a regional K factor that relates catchment area to maximum flood discharge according to the formula:

$$Q_{max} = 10^6 \, (\text{Area}/10^8)^{1-0.1K} \tag{5.1}$$

For the site-specific safety evaluation, which requires the consideration of a family of extreme flood hydrographs, the guidelines recommend the use of the Probable Maximum Flood (PMF). The calculation of the PMF starts with an estimation of the Probable Maximum Precipitation (PMP), the procedure of which is described in detail in the HRU (1972) report. The PMF is then calculated, based on using the PMP in some suitable flood hydrograph generation technique such as those described in Alexander (1990), and selecting the inflowing hydrograph considered to be the most significant with regards to the safety of the dam.

In the *Interim Guidelines on Safety in Relation to Floods* (SANCOLD, 1986) the recommended SEFs for different categories of dam are given in terms of a proportion of the PMF as shown in Table 5.2.

Table 5.2 Recommended Safety Evaluation Flood

Size Class	Hazard Rating		
	Low	Significant	High
Small	0.2 x PMF	0.5 x PMF	0.7 x PMF
Medium	0.5 x PMF	0.7 x PMF	PMF
Large	0.7 x PMF	PMF	PMF

This table was, however, replaced in the final SANCOLD Guidelines, by the statement given below:

> *"For MEDIUM and LARGE HIGH hazard dams and LARGE dams of SIGNIFICANT hazard, the incoming hydrograph shall be the PMF. For dams of smaller size and lesser hazard the PMF may be downrated proportionately to the interrelationship of corresponding SED values. . ." (SANCOLD 1991, pp 34)*

The requirement in the *SANCOLD Guidelines* that the SED and the SEF be determined in relation to the RMF and the PMF respectively has been a major area of concern. The consequence of this is that some individual practitioners reject the *SANCOLD Guidelines* and employ their own approaches for dam safety evaluation in relation to floods. In most cases there is sufficient capacity among the designers of the dam to ensure that the dams in South Africa are generally well designed and safe. In some cases, however, particularly with regards to smaller, privately owned dams, the capacity constraints of the individuals responsible for the design and construction of the dam could result in inconsistencies and a high degree of subjectivity in the dam safety calculations. This might lead to concerns over the safety of these dams. A selection of the concerns with the use of the RMF and PMF encountered during this research are listed below:

1. The data set of flood peaks used to calculate the RMF in TR 137 ended in 1988. Subsequent floods in certain areas have meant that the K-value has already had to be adjusted upwards in five regions (Görgens and McGill, 1990).

2. The RMF is considered to give unreliable results for small catchments or ones that are predominantly urban in nature (Görgens and McGill, 1990).

3. The K-values developed from an envelope of observed extreme floods incorporate an unknown and subjective factor of safety that may not be consistent between regions.

4. The data used to develop the PMP estimates in HRU 1/69 and 1/72 considered only about 30 years of rainfall records from 1932 to 1962. Since the 1960s, however, South Africa has experienced numerous large flood events, the rainfall for which may have exceeded these PMP curves.

5. There is a high degree of subjectivity in terms of selecting key catchment characteristics and flood hydrograph generation techniques, which, at a specific site, can lead to a wide range in estimates of the magnitude of the PMF.

6. The proportioning of the PMF or RMF for different categories of dams in the way recommended in the *SANCOLD Guidelines* is mathematically inconsistent with probability theory generally used in design flood determination methodologies.

5.2.2 Assigned RI / AEP to design extreme floods

The two extreme design flood concepts embedded in South Africa practice are the RMF and the PMF. To develop the concept of the RMF, Kovaćs's research (Kovaćs, 1988) was based on an empirical evaluation of maximum flood peaks from 130 sites around South Africa. For each flood peak, the "representative period of flood", N, was determined. Kovaćs stated that this period is not the return period, except by chance. At gauges, N may represent the length of record, and at other sites where a historical flood has occurred, N may represent the time that has elapsed between that flood and 1988, the final year of record. Where N could not be assigned to a flood peak, a provisional N was determined based on the assumption that the ratio of the 1:200 year flood (Q_{200}) and the RMF is 0.65, Q_{100}/RMF = 0.575, Q_{50}/RMF = 0.5 and Q_{20}/RMF = 0.2. Where the ratio of flood peak to RMF exceeded 0.65, Kovaćs assumed an upper limit of N = 200 years.

Kovaćs noted in his research that there would undoubtedly be several flood peaks with longer representative periods than 200 years. He also stated however that there were a number of peaks in the sample with a longer N than what should be. Kovaćs therefore concluded that in light of these two considerations, the over- and under-estimation of N should cancel each other out when the sum of the individual N-values in a region is determined, and so should not be an issue of concern when determining the mean regional N-value. Kovaćs stated that although in the case of individual peaks, N is not the return period, except by chance, the mean regional N-value can be reasonably assumed to be equal to the mean return period of the peaks for the region. Seeing that the regional RMF curves represent envelope curves, Kovaćs, by implication, estimated the return period of the RMF to be greater than 200 years, although he did not actually model their probability distribution (Kovaćs, 1988).

Pegram et al. (2004) undertook a pilot study to investigate various aspects relating to the calculation of the RMF, one of which was an investigation into the possible assignment of a return period to the RMF. Pegram had previously hypothesised that the RMF envelope curves have a recurrence interval of about 200 years (Pegram et al., 2004).

For their investigation, Pegram et al. used the original database of annual flood peaks that had been used by Kovaćs in his original study. The investigation involved the use of flood frequency analysis[3] to extrapolate the historical floods from the database for three K-regions, regions 4.6, 5 and 5.2, to the 50-, 100- and 200-year recurrence intervals using the Weibull Plotting Position and fitting a General Extreme Value (GEV) distribution to the data. The 50-, 100 and 200-year flood estimates were then plotted coaxially with the RMF estimates against catchment area for the corresponding catchments.

For all three K-regions, Pegram et al. found that the RMF line and the 200-year line estimated from the fitted GEV distribution corresponded reasonably well. The authors therefore concluded that it is reasonable to assume the RMF to have a return period of the order of 200 years.

With regards to an investigation into the possible assignment of a return period to the PMF, it appears that to date no such work has been carried out in South Africa. Some interesting work has been done by Roberts (2002), however, on trying to relate the PMF peak to the RMF peak. In his study, Roberts attempted to relate the PMF, which was calculated using the synthetic unit hydrograph method, to the RMF at 75 dam sites. He found that on average the mean ratio of PMF versus RMF was 1.82 with a minimum of 0.54 and a maximum of 4.49. Roberts also drew attention to what he termed the "inconsistency" of the PMF estimates by comparing the K-value of the RMF (K_{RMF}) to the K-value equivalent of the PMF (K_{PMF}). The results of this study are shown in Table 5.3 (ICOLD, 1992).

Table 5.3 Comparison of the K_{RMF} and K_{PMF} for six K-regions in South Africa

Number of Dams	K_{RMF}	K_{PMF}	K_{PMF}/K_{RMF}
1	4	5.34	1.34
17	4.6	5.51	1.20
41	5	5.47	1.10
8	5.2	5.60	1.08
6	5.4	5.39	1.00
2	5.6	5.90	1.05

It would appear therefore that on the whole the RMF tends to be lower than the PMF, but as has been discussed earlier, there is a high degree of variability and subjectivity involved in determining the PMF and at this stage it is unclear if the variability is on the side of the RMF or the PMF.

[3] The term "flood frequency analysis" is interchangeable with "probabilistic flood analysis", but for this document only "probabilistic flood analysis" will be used.

5.3 EXTREME FLOODS IN INTERNATIONAL DAM SAFETY GUIDELINES

In most countries dams are categorised in a similar manner to those in South Africa, i.e. according to their size and hazard potential, but the details differ particularly in regards to the hazard classification. The recommended values for the safety evaluation floods also differ. A summary of the safety evaluation floods recommended in a range of dam safety guidelines from other countries around the world is presented in this section. The quantified risk associated with these floods, whether in terms of a recurrence interval or an annual exceedence probability, is also described in detail.

5.3.1 USA

Selection of the Design Flood

In the US there is a very low level of acceptable loss of life as a result of dam failure. Dams are classified as Low, Significant or High Hazard. The hazard is determined by the incremental consequences of failure under a worst-case scenario (FEMA, 1998). The US Army Corps of Engineers' criteria for the selection of the Incremental Design Flood (IDF) are given in Table 5.4 and in most cases this is the PMF.

Table 5.4 Design Flood Criteria of the US Army Corps of Engineers (Source: Liu, 2002)

Hazard Potential Classification	Loss of Life	Economic, Environmental, Lifeline Losses
Low	None expected	Low and generally limited to owner
Significant	None expected	Yes
High	Probable. One or more expected	Yes (but not necessary for this classification)

Size Classification	Reservoir Capacity (hm^3)	Height of Dam (m)
Small	0.62 – 1.23	7.66 – 12.2
Intermediate	1.23 – 61.5	12.2 – 30.5
Large	> 61.5	> 30.5

Recommended safety standards		
Hazard	**Size**	**Design Flood Standard**
Low	Small Intermediate Large	50-yr to 100yr Frequency 100-yr to ½ PMF ½ PMF to PMF
Significant	Small Intermediate Large	100-yr to ½ PMF ½ PMF to PMF PMF
High	Small Intermediate Large	½ PMF to PMF PMF PMF

Assigned RI/AEP to Extreme Design Floods

The US Bureau of Reclamation (1981) recommended that the AEP of the PMF should be in the order of 10^{-4}. The U.S. National Research Council (1985) suggested that it was reasonable to consider the AEP of the PMF to be 10^{-6} or 10^{-4}, the latter being considered more conservative in terms of associated higher estimates of risk costs. The Nuclear Regulatory Commission suggested the AEP of the PMF to be 10^{-6} based on the assumption that the PMF has an AEP of the same order as the PMP (ICOLD, 1992). It could be argued however that the PMF would have a lower AEP than the PMP since extreme floods are often combined with wet antecedent moisture conditions and/or snow packs (ICOLD, 1992).

The National Research Council (NRC) report in 1985 also considered the problem of deriving extreme floods with return periods between the maximum flood that could be reasonably determined using probabilistic flood analysis and the assumed AEP of the PMF. This was accomplished by the linear extension of the empirical flood frequency curve from the 1:100 year event to the PMF on lognormal probability paper. This method follows that suggested by the Bureau of Reclamation (1981) that specified the frequency curve to pass through a "box" bounded vertically by 40 and 60% of the PMF and horizontally by vertical lines drawn at the 1:200 and 1:500 return periods. This procedure was however criticised by the NRC, as in practice, the curves are often drawn to just pass through the lower right-hand corner of that box resulting in 0.4PMF being the 500-year event, irrespective of the size of the 100-year flood or the AEP assigned to the PMF (NRC, 1985). The NRC also criticised the Bureau of Reclamation's methodology as the Bureau assumed the flood-frequency curve to approach the PMF asymptotically. The NRC was concerned that this reflected the mistaken belief that the PMF estimate is the maximum possible flood that can occur. The NRC suggested that although the PMF is a very large flood, it can still be expected to be exceeded.

The most widely used procedure in the USA for probabilistic flood analysis is that recommended by the U.S. Water Resources Council (1981) who formulated a set of guidelines known as Bulletin 17. The US WRC developed these guidelines to promote a correct and consistent application of flood probabilistic techniques to be used by private, local and federal agencies. In Bulletin 17, the Log-Pearson Type III distribution is recommended for defining the annual flood series. The method of moments is also recommended to determine the statistical parameters of the distribution from the data. The Bulletin 17 procedures utilise three categories of data: systematic records, historical records and regional information.

More recently, Swain et al. (1998) of the US Bureau of Reclamation presented a framework for deriving extreme floods in the USA based on a risk-based approach to dam safety. The framework considers the credible limit of data extrapolation to determine the maximum flood event (Table 5.5). It was found that through the use of paleoflood data, the credible limit of extrapolation of return period could be in the order of 40 000 years and under optimal conditions 100 000 years. Swain et al. stated that there is a limited scientific basis for assigning an AEP to the PMF and that 100 000 years represented the practical upper limit of the flood return period (DEFRA, 2002).

Table 5.5 Data types and flood extrapolation limits in the USA (Source: DEFRA, 2002)

Type of Data	Credible limit of extrapolation: return period (years)	
	Typical conditions	Optimal conditions
At-site flood data	100	200
Regional flood data	750	1,000
At-site flood and paleoflood data	4,000	10,000
Regional precipitation data	2,000	10,000
Regional flood and paleoflood data	15,000	40,000
Combinations of regional data sets and extrapolation	40,000	100,000

It was noted by DEFRA (2002) that the 1998 Framework made no reference to the procedures mentioned in the NRC 1985 report. It is therefore unclear whether the earlier NRC approach is considered to be outdated, or if it remains in use as a pragmatic solution to providing the information required to undertake a risk-based safety assessment.

Graham (2000) published a thought provoking paper challenging the use of the PMF as a US safety standard in dam design. In his paper he states that the US Army Corps of Engineers found that 27% of 8818 dams inspected between 1977 and 1981 were declared unsafe because they would fail to pass even half of the PMF. This leads Graham to conclude that most significant and high hazard dams in the US would be overtopped by the PMF, to which he assigns an AEP of 10^{-6} based on the recommendation of the NRC (1985), and he therefore rejects the use of the PMF as a safety evaluation standard. He lists four main objections to modifying existing dams to meet the PMF:

- larger spillway capacity may increase annual downstream flood losses
- benefit-cost ratios may be low
- construction accidents associated with dam modification may cause fatalities
- the dollar amount spent to save lives by making dams safer is high.

5.3.2 Australia

Selection of the Design Flood

Australia uses advanced risk based criteria for classification of dams as well as the choice of design flood and for safety evaluation purposes. They have recently updated their Guidelines in relation to floods (ANCOLD, 2000) and are due to publish new guidelines on Risk Analysis for dam safety (Mc Donald, 2003, pers. comm.). They appear to have made much progress in terms of relating design flood to the consequence of failure. Three design floods are differentiated as follows:

Table 5.6 Definitions of Australian Design Floods

AFC	Acceptable Flood Capacity	1. Overall flood capacity, including freeboard, which provides an appropriate level of safety against flood initiated dam failure to protect the community and environment to acceptable risk levels, within the total context of overall dam safety from all load cases 2. It is noted that the outflow up to the AFC, without dam failure, are likely to cause severe damage and disruption to the dam and appurtenant works, and certainly to the community and river valley 3. The assessment of AFC and spillway provision includes consideration of DCF and SDF. 4. When selecting the AFC the DCF stage which includes flood surcharge can be considered initially without additional “dry” freeboard for wind run-up and set-up 5. Risk procedures for selecting AFC set out in Fig. 6.1 of ANCOLD (2000).
DCF	Dam Crest Flood	Indicator for initial hydrological safety assessment and dambreak studies (this replaces the 1986 "imminent failure flood") – Stillwater, excluding wave effects, of lowest point of the dam crest
SDF	Spillway Design Flood	"Serviceability" flood for operational spillway hydraulic sizing and consideration of optimum overall spillway provisions to provide for the AFC

Unlike conventional design philosophy, which prescribes a design flood based on the classification of dam and then determines if the spillway capacity is sufficient to pass this flood, in the Australian guidelines the return period of the design flood is obtained from a risk study starting with an analysis of all the possible failure modes. The probability of the flood that will cause failure due to overtopping, the DCF, is then used to estimate the risk associated with that dam due to overtopping. All other modes of failure, such as piping or earthquakes, are treated in a similar manner to get an overall risk for the dam. The decision of whether or not this dam is safe is based on the acceptable level of this risk. Despite their advance in the use of risk based design criteria, the Australian Guidelines still prescribe a "deterministic fallback alternative" (ANCOLD, 2000), as shown in Table 5.7. Here, the recommended design flood standard to which a spillway for a proposed dam should be built is related solely to the 'Incremental Flood Hazard Category' (IFHC) of the dam. The significance of 'incremental' is that only the incremental hazard is assessed, which refers to the loss of life, property and services over and above the loss which would occur if the dam did not fail (Cantwell, 1988).

Table 5.7 ANCOLD "Fallback Flood Capacity" (ANCOLD, 2000 Table 8.1)

Hazard Category (IFHC) Rating	Flood AEP
Extreme	PMF
High A	PMP Design Flood
High B	10^{-4} to PMP Design Flood or 10^{-6} (ii)
High C	10^{-4} to PMP Design Flood or 10^{-3} (iii)
Significant	10^{-3} to 10^{-4}
Low/very low	10^{-2} to 10^{-3}

Notes:

(i) The IFHC should be based on the ANCOLD Guidelines on Assessment of the Consequences of Dam Failure.

(ii) Pre-Flood reservoir level to be taken as FSL.

(iii) A joint probability assessment can be made for reservoir level as appropriate.

Table 5.7 makes a distinction between the "PMF" and the "PMP Design Flood". In Australia, the ANCOLD guidelines are used in conjunction with the Australian Rainfall and Runoff (ARR) publication which provides guidelines on the estimation of extreme floods relevant to dams. The concept of the "PMP Design Flood" was introduced in Book V1 of Australian Rainfall and Runoff, 1999 (ARR99), which is viewed by ANCOLD as providing improved procedures for flood estimation in comparison to those described in Chapter 13 of the earlier 1987 version (ARR87). According to Nathan and Weinmann (co-authors of ARR99), the PMP Design Flood can be described as the flood derived from the PMP using AEP-neutral assumptions, and as such is estimated to have the same AEP as the PMP. The PMF, on the other hand, is described as the flood resulting from the PMP coupled with the worst flood-producing catchment conditions that can reasonably be expected. As AEP-neutral assumptions are rejected for the derivation of the PMF, it is recognised that an AEP cannot be assigned to the PMF.

Assigned RI/AEP to Extreme Design Floods

The assignment of an AEP to extreme floods was considered in Volume 1 (Chapter 13) of the 1987 version of ARR (ARR87). The ARR87 recognised that it was not possible to extrapolate a flood-frequency curve to determine the AEP of the PMF. The absolute limit of extrapolation for frequency analyses of extreme floods or rainfalls was rather determined to be at a return period of 1 in 500, and for 'important projects' at a return period of 1 in 100 years. Above the credible limit of extrapolation, a pragmatic method developed by Rowbottom et al. (1986) is recommended that links the maximum flood at the limit of extrapolation (e.g. the 1:100 year flood) to the PMF by a smooth curve. This methodology requires the assignment of a probability to the PMF. It also assumes that the AEP of the PMP and PMF are identical.

The method of interpolation between the maximum flood and the PMF developed by the Bureau of Reclamation (1981) was tested by Rowbottom et al. (1986) and found to be inappropriate for most of the locations in Australia. This was thought to be due to a greater variability of statistics of Australian floods compared with those in the United States (Rowbottom et al., 1986). Instead Rowbottom et al. developed a method based on the slope of the frequency curve at a return period of 1 in 100 (determined by the ratio of the 1:100 and 1:50 year events); the ratio of the difference in logarithms of the PMF and 1 in 100 year event, and the 1 in 100 and 1 in 50 year events; and the assignment of probability to the PMF. The procedure also assumes that the slope of the frequency curve at the PMF is horizontal. Based on these considerations, the frequency curves were sketched in by hand to pass through the various defined points, being tangent to a line joining the 1:50 and 1:100 events at the latter point to a horizontal at the PMF, as a mathematical function could not be found to give an appropriate curve. An example of such frequency curves is presented in Figure 5.1.

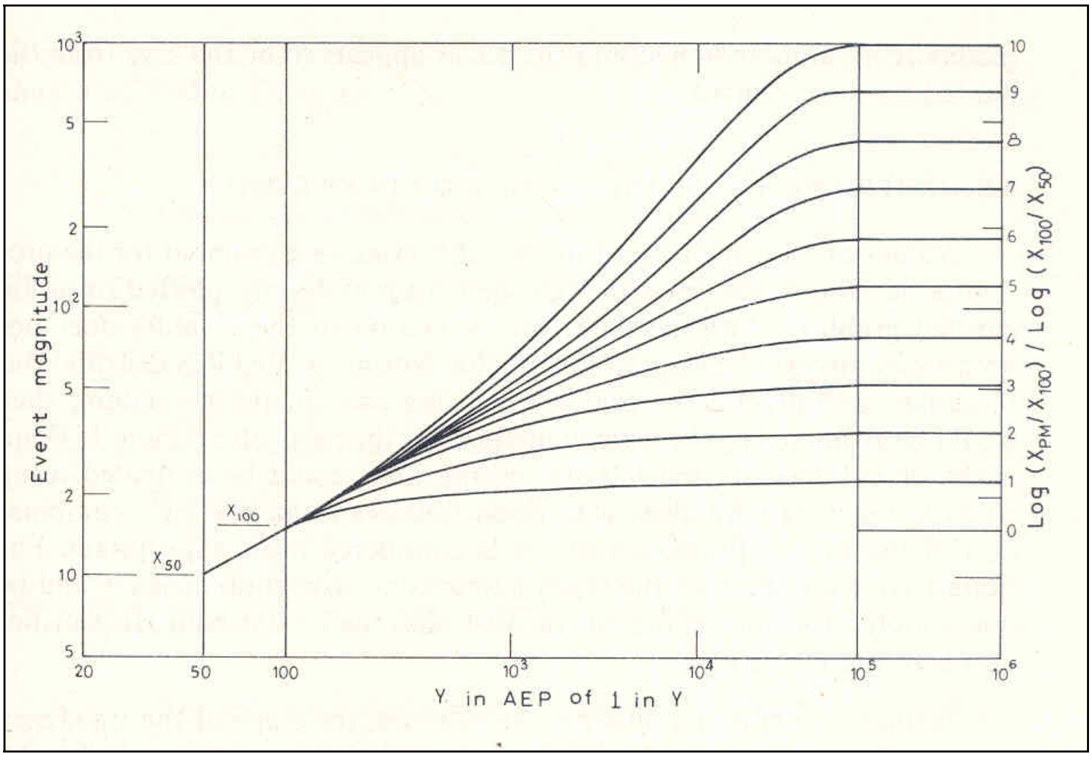

Figure 5.1 Set of frequency curves for assigned probability of 1 in 10^5 for probable maximum flood (Rowbottom et al., 1986)

Based on Rowbottom et al.'s research, ARR87 presents two criteria that can be used to select the AEP of the PMF. The first criterion is based on the exceedence probability of the PMP, adapted from Kennedy and Hart (1984) of the Australian Bureau of Meteorology as shown in Table 5.8.

Table 5.8 AEP for various types of PMP Estimates (Kennedy and Hart, 1984)

Method of Calculation	Effective Transposition Area (km^2)	Annual Exceedence Probability – Catchment Area (km^2) 100	1000	10 000	100 000
1. Maximisation in-situ	n/a	10^{-3}	10^{-3}	10^{-3}	10^{-3}
2. Maximisation and transposition	10^5	10^{-6}	10^{-5}	10^{-4}	10^{-3}
3. Generalised method	10^6	10^{-7}	10^{-6}	10^{-5}	10^{-4}
4. Adjusted US data (up to 6 hours)	10^7	10^{-8}	10^{-7}	n/a	n/a

The second criterion is based on the shape of the frequency curves. In this case, the assigned value of the PMF depends on the value of the ratio $[\log(X_{PM}/X_{100})/\log(X_{100}/X_{50})]$ and the zone (see Figure 5.2) in which the catchment is located. The values of the assigned AEP to the PMF, as published in ARR87, are shown in Table 5.9 and Table 5.10 below.

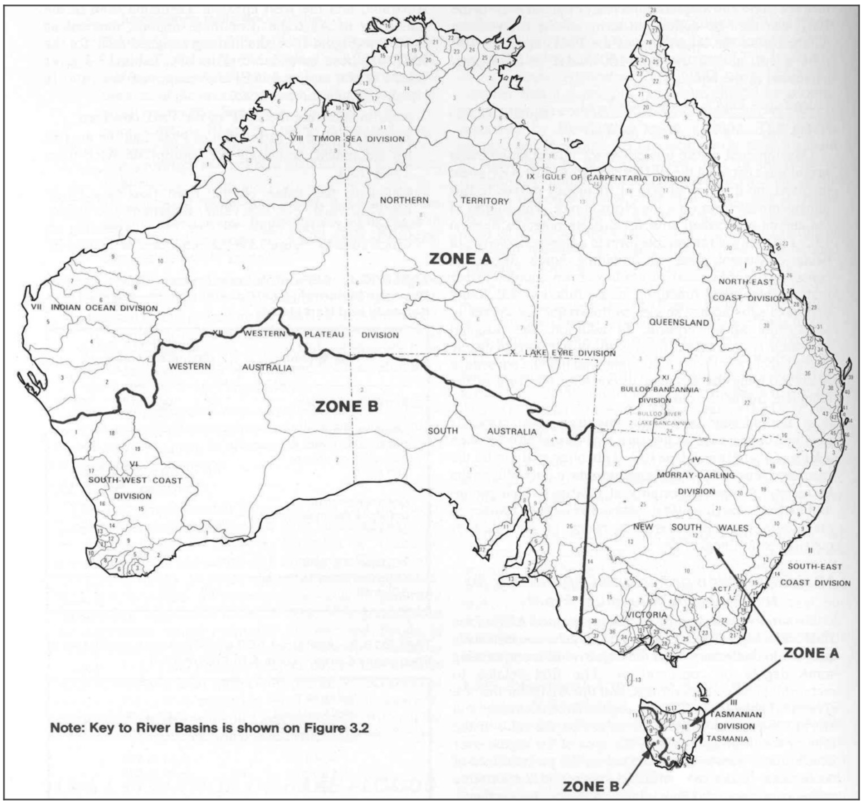

Figure 5.2 Zones for delimiting AEPs of the PMF (ARR87 Figure 13.6)

Table 5.9 Assigned AEP of PMF for Zone A

Value of ratio [log(X_{PM}/X_{100})/log(X_{100}/X_{50})]	AEP of the PMF
<4.5	1 in 10^{-4}
4.5-7.5	1 in 10^{-5}
>7.5	1 in 10^{-6}

Table 5.10 Assigned AEP of PMF for Zone B

Value of ratio [log(X_{PM}/X_{100})/log(X_{100}/X_{50})]	AEP of the PMF
<4.5	1 in 10^{-4}
4.5-7.5	1 in 10^{-5}
7.5-10.0	1 in 10^{-6}
>10.0	1 in 10^{-7}

In considering both of the above criteria, the one that gives the higher probability is chosen as the AEP of the PMF.

To enable the designer to reproduce the frequency curves for a catchment of interest, ARR87 provides a useful table (see Table 5.11) that contains ordinates of the curves for five assigned AEPs. To use this table, all that is required is the ratio [log(X_{PM}/X_{100})/log(X_{100}/X_{50})] and an assigned AEP to the PMF .

Table 5.11 Values of frequency curve ordinates for a known value of $[\log(X_{PM}/X_{100})/\log(X_{100}/X_{50})]$ and PMF (ARR87 Table 13.4)

AEP of PME	1 in 10^3		1 in 10^4		1 in 10^5		1 in 10^6		1 in 10^7 (Only for use in Zone B of Figure 13.6)	
$\frac{\log(X_{PM}/X_{100})}{\log(X_{100}/X_{50})}$ / AEP of Intermediate Points (1 in Y)	1 in 200	1 in 500	1 in 500	1 in 2000	1 in 1000	1 in 10000	1 in 2000	1 in 50000	1 in 2000	1 in 200000
1	0.735	0.960	0.877	0.976	0.928	0.989	0.948	0.994	0.956	0.995
2	0.494	0.852	0.711	0.936	0.802	0.970	0.850	0.982	0.893	0.987
3	0.380	0.777	0.585	0.880	0.690	0.944	0.752	0.967	0.814	0.975
4	0.330	0.757	0.489	0.816	0.600	0.911	0.667	0.945	0.734	0.960
5	0.305	0.749	0.419	0.783	0.527	0.862	0.597	0.915	0.663	0.941
6	0.290	0.743	0.372	0.766	0.467	0.811	0.539	0.874	0.597	0.916
7	0.279	0.740	0.345	0.756	0.420	0.784	0.488	0.827	0.539	0.882
8	0.271	0.737	0.328	0.749	0.385	0.770	0.447	0.797	0.491	0.838
9	0.265	0.735	0.316	0.745	0.360	0.762	0.413	0.781	0.451	0.808
10	0.261	0.733	0.306	0.742	0.342	0.757	0.385	0.771	0.419	0.789
11	0.258	0.732	0.298	0.740	0.330	0.753	0.363	0.766	0.394	0.778
12	0.255	0.731	0.291	0.738	0.321	0.751	0.347	0.762	0.375	0.771
13	0.252	0.730	0.286	0.737	0.314	0.749	0.335	0.759	0.360	0.765
14	0.250	0.729	0.281	0.736	0.307	0.747	0.327	0.756	0.348	0.761
15	0.248	0.728	0.276	0.735	0.301	0.746	0.320	0.754	0.338	0.759
16	0.246	0.727	0.272	0.734	0.295	0.745	0.314	0.753	0.329	0.757
18	0.243	0.726	0.265	0.733	0.286	0.744	0.304	0.752	0.314	0.755
20	0.240	0.725	0.260	0.732	0.279	0.743	0.294	0.751	0.303	0.754

In the latest revised version of the ARR, published in 1999, flood estimation procedures are summarised according to three classes of flood: large, rare and extreme. These procedures are shown in Table 5.12 below.

Table 5.12 Classification of flood events in Australia (DEFRA, 2002 Table F.6)

Event class	Large	Rare	Extreme
RI	50-200	200-2000	2000-PMP (PMP:10^4-10^7)
Method of determination	Interpolation	Extrapolation	Pragmatic
Uncertainty	Moderate	Moderate to large	Unquantifiable but notionally very large

For the estimation of "large" floods, the general method of interpolation adopted for Australia is the use of the Log-Pearson Type III distribution which is fitted to the data using the 'method of moments' procedure, based on and preserving the moments of the logarithms of flows (ARR87). It was viewed as advantageous to select one distribution for use in probabilistic flood analysis across the country as it would lead to a consistency in design practice, as opposed to fitting several different distributions to the data and selecting the distribution that gives the best fit. The Log-Pearson Type III was chosen as it was determined to perform the best out of the various distributions tested on data for many catchments in Australia, as well as it being consistent with the procedure recommended for the US in Bulletin 17B. Based on more recent research, however, this general approach is likely to be replaced by the use of the LH-Moments fitted to the GEV distribution (Wang, 1997).

For the estimation of "rare" floods, the notional credible limits of extrapolation for different types of data are shown in Table 5.13.

Table 5.13 Data types and extrapolation limits in Australia (DEFRA, 2002, Table F.7)

Type of Data	Credible limit of extrapolation: return period (years)	
	Typical conditions	Optimal conditions
At-site flood data	50	200
At-site rainfall data	100	200
At-site/Regional flood data	200	500
At-site flood and Paleoflood data	5,000	10,000
Regional precipitation data	2,000	10,000
Regional flood and paleoflood data	15,000	40,000
Regional paleoflood and rainfall data and extrapolation	40,000	100,000

Above the credible limit of extrapolation, the assessment of "extreme" floods follows the same pragmatic approach as described in ARR87 in terms of linking the maximum flood at the limit of extrapolation to the PMF by a smooth curve (Nathan et al., 2001). Following the concepts defined in the ARR99, however, the "PMP Design Flood" is representative of the upper limit of the flood frequency curve, as opposed to the PMF value adopted by ARR87 (Nathan et al., 2001).

5.3.3 The United Kingdom

Selection of the Design Flood

In the UK the classification of dams according to hazard class for floods and the corresponding return period of the design flood are given in Table 5.14. Dams are designed to pass the extreme flood under normal operating conditions and as a result there is no specified safety check flood.

Table 5.14 Classification and Associated Design Floods used in the UK (DEFRA, 2002)

Hazard Class	Consequence Class	"Breach could endanger lives in a community"	Percentage of UK dams in each class	Design Return Period (general standard)
Highest	A	> 10	48 %	PMF
Second	B	1 – 10	19 %	10,000
Third	C	Negligible	23 %	1000
Lowest	D	None	10 %	150

Assigned RI/AEP to Extreme Design Floods

In the UK the magnitude of the PMF is estimated based on the conversion of the PMP to the extreme flood value using the rainfall-runoff method recommended in the Flood Studies Report (FSR), which was published in 1975. According to DEFRA (2002), the FSR (1975) recommends that an AEP of 10^{-6} should be assigned to the PMF. The current methodology adopted in the UK for the estimation of the AEP of the PMF, however, is that reported in the more recently published Flood Estimation Handbook (1999). The methodology is based on the research of Lowing (1995), which was undertaken following the 1975 publication of the Flood Studies Report. In his methodology, Lowing suggests two approaches for the estimation of the AEP of the PMF, the lower value of which would be adopted for design purposes:

(i) The PMF is assigned an AEP of 10^{-6}, which is increased by a factor of 10 (10^{-7}) if any of the following apply (FEH, 1999):

- The PMP is derived on a catchment between 100 km^2 and 500 km^2
- The FSR all-year PMP is derived, i.e. summer PMP combined with snowmelt

- The snowmelt rate is increased to 5 mm h^{-1}

Lowing also suggests that the PMF AEP of 10^{-6} be increased by a factor of 100 (10^{-8}) if the catchment area exceeds 500 km^2.

(ii) The second approach recommended by Lowing is based on the methodology developed by Rowbottom et al. (1986), which involves an interpolation technique for producing a flood frequency curve defined up to the level of the PMF. The method developed by Lowing involves the estimation of the peak flows of the 1:100 year flood, 1:1000 year flood and the PMF using the FSR rainfall-runoff technique. An AEP is then assigned to the PMF, based on the slope of the growth curve between the 100 and 1000 years. The AEP of the PMF was found to be in the range of 10^{-6} and 10^{-9} years (Table 5.15).

Table 5.15 Assigned AEP of PMF after Lowing (1995) (FEH, 1999 Table 4.3)

Ratio (Q_{PMF}/Q_{1000}-1):(1-Q_{100}/Q_{1000})	PMF AEP (years)
< 5	10^{-6}
5-10	10^{-7}
10-15	10^{-8}
> 15	10^{-9}

For the derivation of intermediate floods between the 1:1000 year flood and the PMF, Lowing used a cubic spline function to draw smooth curves linking the 1000 year flood and the PMF, from which the flood magnitudes can be read.

Table 5.16, provided by DEFRA (2002), presents a "preliminary assessment of the credible limit of extrapolation in the UK", in which it suggests that the maximum return period up to which a flood can be extrapolated is 1:10 000 where paleoflood assessment is available. It is also evident from the table that the limit of extrapolation has fallen from a return period of 10 000 years using the FRS to 1000 years using the FEH where paleoflood data is not available. Where regional flood data is available, it is shown that statistical methods can be used to extrapolate the flood data up to a 1:1000 year flood. According to the FEH, the Generalised Logistic (GL) distribution is recommended for use in the UK when performing a probabilistic flood analysis, although the GEV distribution (recommended by the FSR) is also recognised as an important distribution with strong theoretical and historical justification.

Table 5.16 Preliminary estimate of flood extrapolation limits in the UK (DEFRA, 2002 Table F.9)

Type of Data	Credible limit of extrapolation: return period (years)	Comments
Single site data	50-100	FEH Single site method
Single site flood data and data from historical records of floods	150-500	Data from BHS flood archive and other sources
Regional flood data (FEH)	1,000	FEH Statistical method
Regional rainfall data (FEH)	1,000	Post-1999, FEH rainfall data and rainfall-runoff method reconciled with Statistical method
Regional rainfall data (FRS)	10,000	Pre-1999, FRS rainfall-runoff method
Paleoflood data, uplands	1,000	Analysis of boulder deposits
Paleoflood data, lowlands	10,000	Analysis of post-glacial slack water sediments

5.3.4 Europe

As is evident in Table 5.17 below, it is not common practice in Europe to design dams for the safe passage of the PMF. Instead the maximum flood, in many cases, is taken as the 1:10 000 year flood.

Table 5.17 Comparison of Accepted Practice in Europe in Relation to the Design of Dams for Floods

COUNTRY	Design Flood	Safety Check Flood
AUSTRIA	5,000	No
FRANCE		
- concrete	1,000	No
- fill	10,000	No
NORWAY	1,000	PMF
FINLAND		
Permanent dams		
- Cat A	500	No
- Cat B	1,000	No
- Cat C	10,000	No
Temporary dams		
- Cat A	100	No
- Cat B	500	No
- Cat C	5,000	No
SPAIN		
- Cat A	1,000	5,000 – 10,000
- Cat B	500	1,000 – 5,000
- Cat C	100	100 - 500
SWEDEN		
- High Hazard	1,000 – 10,000	No
- Low Hazard	100	No
SWITZERLAND		
- Concrete	1,000	1.5 x Qd (i)
- Fill	1,000	1.5 x Qd

Notes:
(i) Qd refers to the 1:1,000 year Design Flood

Europe, with relatively long streamflow records, tends to favour probabilistic flood analysis using annual flow maxima to estimate design floods up to as great as the 1:10 000 year flood (ICOLD, 1992). This differs from, for example, the USA where relatively short flow records enforce a bias towards the calculation of the PMF from the PMP.

The approach adopted in Finland involves the estimation of the 1:100 year flood using a Gumble probability analysis, which is normally based on 80-100 years of data. Above the 1:100 year flood, regional growth curves are then applied, for example, Q_{1000}/Q_{100} = 1.3 and $Q_{10\ 000}/Q_{100}$ = 1.6 (ICOLD, 1992). In Switzerland a similar approach is adopted where the 1:1000 year Design Flood (Qd) is determined based on probabilistic flood analysis of observed flood data and then factored up by 150% to determine the 'Safety Check Flood' (Biedermann et al., 1988).

In general, hydrological models using a rainfall-runoff process, such as HEC-1 (Finland) and HBV-3 (Norway, Sweden), are used to determine the flood hydrograph based on the estimated flood peak. An important development in France for the determination of the flood hydrograph is the GRADEX method which was developed in 1966 (Duband et al., 1988). The GRADEX method deduces extreme value discharge frequencies from extreme rainfall frequencies on the basis of several straightforward physical assumptions applicable to small and medium sized catchments (Duband, 1988). It is based on a number of simple statistical assumptions and considers the rainfall-runoff process as a stochastic process.

5.3.5 China

The Chinese design flood criteria were first developed in 1964 and revised in 1990 (Liu, 2002). Projects are ranked into categories according to their scale, benefits and importance to the national economy (see Table 5.18). Projects are then classified into 5 grades according to the rank and the type of project; main structure, less important one (auxiliary structure) or temporary structure (see Table 5.19). The design flood criteria for permanent structures are given in Table 5.20 and the safety evaluation flood values are given in Table 5.21.

Table 5.18 Project categories (Xuemin, 1989)

Category	Reservoir Capacity (10^8 m^3)	Power installation (MW)	Irrigation area (10^4 ha)	Object of flood protection
A	> 10	> 750	> 100	Major cities
B	10 – 1	750 – 250	33.33 – 100	Fairly large cities
C	1 – 0.1	250 – 25	3.33 – 33.33	Medium-sized cities
D	0.1 – 0.01	25 – 0.5	0.33 – 3.33	Ordinary cities
E	0.01 – 0.001	< 0.5	< 0.33	-

Table 5.19 Classification of hydraulic structures (Xuemin, 1989)

Category	Permanent Structures		Temporary Structures
	Main Structures	Auxiliary structures	
A	Class 1	Class 3	Class 4
B	Class 2	Class 3	Class 4
C	Class 3	Class 4	Class 5
D	Class 4	Class 5	Class 5
E	Class 5	Class 5	

Table 5.20 Design flood criteria for permanent structures in China (Xuemin, 1989)

Class	1	2	3	4	5
Return Period of flood (yr)	2000-500	500-100	100-50	50-30	30-20

Table 5.21 Return Period (yr) of Safety Evaluation flood criteria for permanent structures in China (Xuemin, 1989)

Class	1	2	3	4[(1)]	5[(2)]
Embankment Dam	10,000 or PMF	2,000	1,000	500	300
Concrete Dam	5,000	1,000	500	300	200

(1) In a publication by Liu (2002), the design flood for a Class 4 concrete dam was shown as the 1:200 year flood.

(2) In a publication by Liu (2002), the design flood for a Class 5 embankment dam and concrete dam was shown as the 1:200 and 1:300 year flood, respectively.

For an embankment dam whose failure would cause catastrophe in the downstream reaches in terms of heavy loss of life and property, it is recommended that the PMF should be considered as the safety evaluation flood; otherwise the flood should be taken as the 1:10 000 year flood. Besides this consideration given to the use of the PMF, what is interesting to note about the Chinese dam safety codes, is that in general no consideration is given to the consequences of dam failure and the corresponding threat to life. Another interesting factor is that concrete dams have a lower safety evaluation flood because they are considered to be inherently safer than embankment dams. Chinese practice therefore accepts that the type of dam should have an influence on the safety requirements.

In China the main method used for flood prediction is the probabilistic flood analysis method. Due to the variability of floods across the country, China has recommended through practice that the Log-Pearson Type III be used for determining the flood frequency curves, as this distribution was found to be the most suitable for highly variable flood sequences (Pan et al., 1988). In the case of the PMF, however, probabilistic flood analysis is not used; instead the extreme flood is calculated from the PMP design rainfall.

5.3.6 India

In India, dams are classified as small, intermediate or large according the dam's storage capacity and hydraulic head (see Table 5.22). In Table 5.22, the SPF refers to the Standard Project Flood which represents "the flood discharges that may be expected from the most severe combination of meteorological and hydrological conditions that are considered reasonably characteristic of the region, excluding extremely rare combinations" (Varma, 1998). For large dams and spillways, the PMF is used as the design flood, which is described in the same way as the SPF, but with the inclusion of extremely rare combinations of meteorological and hydrological conditions. Such as the case in China, in India no consideration is given to the consequences of dam failure and the resulting threat to life.

Table 5.22 Criteria for Design Flood in India (Varma, 1998)

Classification	Gross Storage (10^6 m^3)	Hydraulic Head (m)	Design Flood
Small	0.5 – 10	7.5 - 12	1:100
Intermediate	10 – 60	12 - 30	SPF
Large	> 60	> 30	PMF

For flood probabilistic analyses, India uses the Fisher Tippet Extreme Value Type 1 which has been shown to be most suitable for arid and semi-arid regions where the number of independent floods is small (Varma, 1998). Varma also states that distributions in India have severe limitations unless data of a few years is grouped together.

5.4 INVESTIGATION OF THE POSSIBLE RECURRENCE INTERVAL OF THE RMF AND PMF IN A SOUTH AFRICAN CONTEXT

As can be seen from Section 4.2, there are some concerns with the recommended criteria and methodologies for extreme floods used to evaluate the safety of dams in South Africa. In addition, from Section 4.3, there appears to be a wide range of standards applied for determination of the safety evaluation flood for dams, internationally. In order to put the South African recommended extreme floods in context with those of other countries, and to put into context some of the concerns raised about the use of the RMF (including its variants RMF+Δ and RMF-Δ) and the PMF, we examined the apparent recurrence interval (RI), or annual exceedence probability (AEP) of these extreme design floods, using RSA flood data. This would not only help in contextualising some of the current concerns, but would also pave the way for reviewing the recommended criteria in the SANCOLD Guidelines. The potential to move away from RMF and PMF based criteria to AEP based criteria is also in line with the increasing use internationally of probabilistic techniques for design flood determination and risk analysis in dam safety assessments. The remainder of this report therefore focuses on the methodology followed for the estimation of the RI/AEP of the RMF, RMF+Δ, RMF-Δ and PMF, as well as the relevant findings and conclusions.

5.4.1 Methodology for determining the RI of the RMF and PMF

Selection of flow measuring stations

The flood records used in the study have been observed at 48 dam sites and 32 flow gauge stations (see Table 5.23). Only flow records of 30 years or longer were selected and a sample fairly representative of the whole country was pursued. The dam inflow records considered were limited to the dams that had been included in a database developed by DWAF's Directorate of Hydrology. They were created by back-routing observed inflow hydrographs at a "pilot" sample of dam sites around the country. For the choice of the flow gauge stations, these were limited to the stations that had already undergone a screening process for use in addressing the second aim of this project, which involves the derivation of a methodology for design flood hydrograph estimation.

Table 5.23 Selected Flow Measuring Stations

Station Type	Station Number	Station Name	Length of Record (years)	Catchment Area (km^2)	Regional K-value
DAM SITES	A2R001	Hartbeespoort	99	4120	5
	A2R003	Olifantsnek	73	492	5
	A2R005	Buffelspoort	66	114	3.4
	A2R006	Bospoort	75	1078	5
	A2R007	Lindleyspoort	56	704	4.6
	A2R012	Klipvoor	48	6128	4.6
	A3R001	Marico Bosveld	68	1219	4.6
	A3R002	Klein Maricopoort	96	1180	4.6
	A3R003	Kromellemboog	48	1786	4.6
	A3R004	Molatedi	76	8703	4
	A6R001	Doorndraai	64	595	5
	A8R001	Nzhelele	70	832	5
	A9R001	Albasino Dam	55	509	5.2
	B1R001	Witbank	98	3541	4.6
	B1R002	Middelburg	45	1576	4.6
	B2R001	Bronkhorstspruit	98	1263	4.6
	B3R001	Rust de Winter	69	1145	5
	B3R002	Loskop	65	12262	3.4
	B5R002	FlagBoshielo	65	23566	4.6
	B6R001	Ohrigstad	47	84	5
	B6R003	Blyderivierspoort	51	2166	5
	B7R001	Klaserie	52	165	5.2
	B7R003	Tours	54	45	5.2
	B8R001	Ebenezer	52	169	5.2
	B8R005	Tzaneen	52	652	5
	C3R002	Spitskop	78	26922	3.4
	C5R002	Kalkfontein	86	10264	5
	C5R003	Rustfontein	80	937	5
	C7R001	Koppies	80	2154	4.6
	C9R002	Bloemhof	64	108125	4
	D2R004	Welbedacht	70	15330	4.6
	D3R002	Gariep	99	70749	5
	D7R001	Boegoeberg	90	342952	3.7
	H3R001	Poortjieskloof	47	94	5
	H4R002	Keerom	49	377	5
	H7R001	Buffeljags	53	614	5
	J1R002	Bellair	77	546	5
	J2R003	Oukloof	70	141	5
	K9R002	Impofu Dam	72	856	5

Station Type	Station Number	Station Name	Length of Record (years)	Catchment Area (km^2)	Regional K-value
	L9R001	Loerie	31	138	5.4
	N2R001	Darlington	69	16700	5
	U2R001	Midmar	49	925	5
	V2R001	Craigie Burn	37	154	5
	W4R001	Pongolapoort	58	7814	5.6
	W5R003	Morgenstond	53	548	4.6
	X1R001	Nooitgedacht	42	1569	4.6
	X2R004	Primkop	40	263	5
	X2R005	Kwena	44	954	5
FLOW GAUGES	A2H006	-	54	1028	5
	A2H012	-	45	2551	5
	A2H013	-	42	1171	5
	B1H004	-	42	376	4.6
	B7H004	-	38	136	5.2
	B8H010	-	36	477	5.2
	C2H001	-	29	3595	4
	C9H008	-	36	115057	3.4
	G1H004	-	43	70	5
	G1H008	-	41	395	5
	G2H008	-	44	20	5
	H1H006	-	40	753	5
	H1H007	-	40	84	5
	H1H018	-	33	113	5
	H7H005	-	42	9	5
	J2H005	-	35	253	5
	K2H002	-	41	131	5.2
	T3H006	-	37	4268	5.2
	T5H004	-	38	545	5
	U2H006	-	36	339	5
	U2H011	-	42	176	5.2
	U2H012	-	36	438	5.2
	V1H009	-	39	196	5
	V2H002	-	40	937	5
	W5H005	-	43	804	5
	X1H001	-	44	5499	5.2
	X2H008	-	38	180	5
	X2H011	-	37	402	5
	X2H015	-	41	1554	5
	X3H001	-	44	174	5
	X3H003	-	39	2231	5
	X3H006	-	40	766	5.2

Assembling of flow records

For the calculation of the RI, or AEP, of the RMF, RMF+Δ, RMF-Δ and PMF, flow records of annual maxima were required for each flow measuring station. The flow records for the dam sites were obtained directly from DWAF's Directorate of Hydrology's database, and were used unaltered. For the flow gauge stations, however, the flow record was compiled with the use of software, called *EX-HYD*, which was developed under this project by Ninham Shand Consulting Services with the primary purpose of identifying and extracting complete flood hydrographs from continuous recorded flow data. This software and its development are described in Part 2 of this Research Report.

Using *EX-HYD*, flood events above a set truncation level are identified and exported to a database. From the database, a text file is requested and the annual maximum floods are identified. Due to the variable nature of streamflow in South Africa, there were a few years when a flow gauge did not record an event above the truncation level. In these instances the annual maximum flood was identified by visual inspection from the flow record for the purposes of this study.

Determination of catchment characteristics

As part of the analysis, the magnitude of the RMF, RMF+Δ, RMF-Δ and PMF were required to be calculated for each of the flow measuring stations. Due to the large areas associated with many of the dam sites, the Unit Hydrograph method was adopted for the calculation of the PMF. The following catchment characteristics were therefore required as input for the analysis:

RMF:
- Catchment area
- Kovaćs regional K-value

PMF (HRU, 1972):
- Catchment area
- Generalised Veld Type Zone
- Length of the longest watercourse
- Slope of the longest watercourse (equal area slope)
- Distance to the centroid of the catchment

A spreadsheet containing the list of gauges and their catchment characteristics was supplied to the project by DWAF; however the catchment characteristics were incomplete. The approach therefore taken was to use GIS to determine the missing information. To achieve this, two GIS datasets were supplied by DWAF. The first dataset contained points representing the location of flow measuring stations across the country. The second contained the catchment boundaries corresponding to each individual station. Using GIS, a new set of flow stations and catchments were generated for the purpose of calculating the missing information, which included mainly the length of the longest watercourse, length of watercourse from centroid to outlet and the longest watercourse slope. The steps that were followed to achieve this are shown below:

- The flow measuring station dataset was filtered to remove stations that were not relevant to the project.
- By overlaying the flow measuring station and catchment datasets, the catchments were grouped to accurately represent the downstream catchment area for each station.
- The area for each catchment was calculated using standard GIS tools and a comparison was made with the areas provided by DWAF. Where large differences in area occurred, the catchments were re-examined and amended. For most stations, a close match was achieved.
- Using the Xtools extension for ArcMap, the catchment centroid for each station was generated.
- By overlaying the station catchments and the river network, the length of longest watercourse was manually determined using the "select and summary" tools in ArcMap.
- Using the catchment centroid as a guide, the river centerline was split and the length of watercourse from centroid to outlet was then manually calculated using the "select and summary" tools.
- Using ArcGIS 3D Analyst, height values were determined at 5m intervals along the river centerline using the DTM as a z source (height). A second script was written in ArcView 3.1 to plot the profile of the river and determine the equal area slope by drawing a series of lines across the profile and comparing the area above and below the cut. The equal area value was determined when the areas achieved the closest match.

Based on the information available from DWAF and that newly derived from GIS, a decision tree was then established to determine the source of information to be used for the calculation of the design floods, in particular the PMF, for each of the flow measuring station. The following was decided:

- Where DWAF data was available, this would be used unaltered.
- Where no DWAF information was available, the GIS derived characteristics would be used.
- Where some DWAF catchment characteristics were provided but where there were also some missing for a station, a combination of the DWAF and GIS information would be used.
- When neither the DWAF nor GIS data could be determined, these stations would be excluded from the analysis.

For the majority of the stations, DWAF provided the "length of the longest watercourse", but the "distance to centroid" was missing. For these cases, the GIS derived "distance to centroid" was factored by the ratio of the DWAF "length of longest water course" and the GIS derived "length of the longest water course".

Where the DWAF information was missing the "equal area slope" for a measuring station, the GIS slope was used. To check the GIS derived slope, the slopes were compared with the slopes provided by DWAF, where available. In general, the GIS derived slopes compared well with those provided by DWAF. Where there were major differences evident, the GIS derived slopes were manually checked by visually inspecting the plots of height versus distance. For all of these cases, blips were evident in the GIS profiles. These were corrected by eye and the equal area slope calculated by hand using a planimeter. It was found that the hand calculated slopes compared well with those provided by DWAF. Based on this experience, where the DWAF spreadsheet was missing an "equal area slope" for a station, the GIS slope was checked to ensure that no blips were present in the profile and corrected, if necessary, before being used in the analysis.

Calculation of the PMF

The Unit Hydrograph flood estimation method (HRU, 1972) was used for the calculation of the PMF[4]. For this calculation, an attempt was made to use the Windows version of UPFlood DETFLOOD that uses equations derived from the HRU diagrams (HRU, 1972) for the calculation of the design flood. This program, however, was found to be erroneous in that the output PMF values were, in general, very low, and the program would not allow for critical storm duration greater than 20 hours. The earlier DOS version of UPFlood was therefore used, which when checked against hand calculations, was found to produce reliable results.

Calculation of RIs for RMF and PMF using Probabilistic Flood Analysis

[4] It should be noted that in the case of those gauged catchments with areas larger than 5 000 km^2, the HRU 1/72 Unit Hydrograph approach requires subdivision of the catchment into sub-catchments followed by routing of the individual sub-hydrographs to the exit point. Given the resource constraints that inevitably face a multi-pronged research project such as this, combined with our intention to use the PMF and RMF results of this task only in terms of broad regional trends and patterns, we decided to model the larger catchments as unitary catchments, i.e. not sub-divided. This could mean that for any particular large catchment our PMF value could be either larger or smaller than one based on sub-division and routing. Our PMF value could be larger in a case where the individual sub-hydrographs are out of phase, yielding a lower aggregate flood peak. On the other hand, because the average channel slope of a unitary catchment is usually significantly less than that of sub-catchments, our PMF value could be smaller, given that a Unitgraph-based flood peak is usually indirectly dependent on the average slope of the main water course.

For the estimation of the RI or AEP of the RMF, RMF+Δ, RMF-Δ and PMF, probabilistic flood analysis was used. To perform the probabilistic analysis, individual spreadsheets were produced for each of the selected flow measuring stations based on those developed by DWAF's Directorate of Hydrology. DWAF's original spreadsheets include the fitting of five different distributions to the flow records (annual maxima) of the various dam sites. These distributions include the Log-Normal (LN), Log-Pearson Type III (LPIII), GEV using methods of moments (GEVmm), GEV using probability weighted moments (GEVpwm) and a "DWAF Proposed" distribution. The latter distribution consists of a combination of the other distributions depending on that determined to provide the best fit to the data. For the plotting of the flow data, the Cunane plotting position is used and plots of observed flood data versus AEP or RI can be viewed over which the various distributions are laid.

A check was made of the probabilistic flood peaks estimated by DWAF's spreadsheet for a number of return periods by comparing the values to those determined by the statistical component of the computer program UPFlood, namely REGFLOOD, as well as by hand calculations for the LPIII and GEVmm distributions. The flood peak estimates were found to be nearly identical for the three different methods.

For the purpose of this project, a template was produced with the following alterations made to the original DWAF spreadsheet:

- Only the LPIII, GEVmm and GEVpwm distributions were considered. These were selected, as the LPIII and GEV distributions are the most common distributions used globally. The GEVpwm distribution was included in the analysis to allow for the uncertainties in parameter estimates caused by small sample sizes.
- The chosen probability functions were extrapolated to a maximum RI of 1 in 10^9 years. This maximum RI of 10^9 was arbitrarily chosen, but was informed by the UK practice of limiting the maximum RI assigned to the PMF to 1 in 10^9. Table 5.24 presents an example of such a table, in this case for A3R004 (Molatedi Dam).

Table 5.24 Probability Distribution Functions for Molatedi Dam (A3R004)

HHP	Exceed.	LPIII	GEV_{param}		LPIII	GEV_{MM}	GEV_{PWM}
	Probab.	W_{TLP3}	WT	Par.	Q (m^3/s)	Q (m^3/s)	Q (m^3/s)
Inclusion, or not, of a distribution's results in the **"Proposed result"**			to include read in **"1"** otherwise read in **"0"**		**1**	**1**	**0**
Range in which particular distribution has to be taken into account					1.01 1E+09	10 1E+09	1.01 1E+09
1.0001	0.9999	-4.400	-1.908		0	0	0
1.002	0.998	-3.219	-1.612		2	2	2
1.005	0.995	-2.829	-1.487		3	4	3
1.010	0.99	-2.519	-1.375		5	6	5
1.020	0.98	-2.191	-1.242	k	7	10	8
1.053	0.95	-1.715	-1.017		13	18	15
1.111	0.90	-1.308	-0.787	-0.140	22	30	25
1.25	0.80	-0.832	-0.460		40	54	45
2	0.50	0.035	0.376		121	164	136
5	0.20	0.850	1.669		340	425	321
10	0.10	1.257	2.645	E(y)	570	621	504
20	0.05	1.582	3.683		862	830	748
50	0.02	1.938	5.191	1.103	1355	1134	1203
100	0.01	2.169	6.457		1817	1389	1690
200	0.005	2.376	7.849		2364	1669	2350
500	0.002	2.622	9.904		3229	2083	3597
1000	0.001	2.790	11.642	var(y)	4001	2433	4937
2000	0.0005	2.947	13.557		4883	2819	6756
5000	0.0002	3.139	16.390	0.051	6230	3390	10195
10000	0.0001	3.274	18.788		7397	3873	13895
20000	0.00005	3.402	21.430		8702	4405	18917
50000	0.00002	3.561	25.340		10652	5192	28412
100000	0.00001	3.675	28.650		12310	5859	38627
200000	0.000005	3.784	32.296		14136	6594	52494
500000	0.000002	3.921	37.693		16826	7681	78714
1E+06	0.000001	4.020	42.261		19082	8601	106918
2E+06	0.0000005	4.115	47.295		21541	9615	145210
5E+06	0.0000002	4.236	54.744		25122	11115	217608
1E+07	0.0000001	4.324	61.050		28097	12385	295487
1E+08	0.00000001	4.598	86.983		39784	17608	816172
1E+09	0.000000001	4.847	122.777		54619	24818	2253911

- A results table (see Table 5.25 for an example) was produced in which an interpolation function is used to estimate the RI and AEP of the RMF, RMF+Δ, RMF-Δ and PMF for the three probability distributions. The RMF, RMF+Δ, RMF-Δ is also calculated within the same table.

Table 5.25 Results Table showing interpolated annual probability of exceedence (P) and RI for the various design floods for Molatedi Dam

Gauge	A3R004								
Dam	Molatedi			Individual Distributions					
Area	8703 km2			Log-Pearson Type III		GEVmm		GEVpwm	
Design Flood	K-value	Zone	Q (m^3/s)	P	RI (years)	P	RI (years)	P	RI (years)
RMF	4	Flood	3663	0.001317	745	0.000132	7436	0.001899	520
RMF+	4.6	Flood	6418	0.000175	5640	0.000006	169391	0.000564	1784
RMF-	3.4	Flood	2090	0.006890	143	0.001967	508	0.006336	156
PMF			40310	0.000001	116872092	0.000034	5977749817	0.000009	110142

- For comparison purposes, plots representing the value of RMF, RMF+Δ and RMF-Δ and PMF were produced on the same set of axes as the plots of the observed flow data and the various probability functions. Figure 5.3 is an example of such a comparative plot.

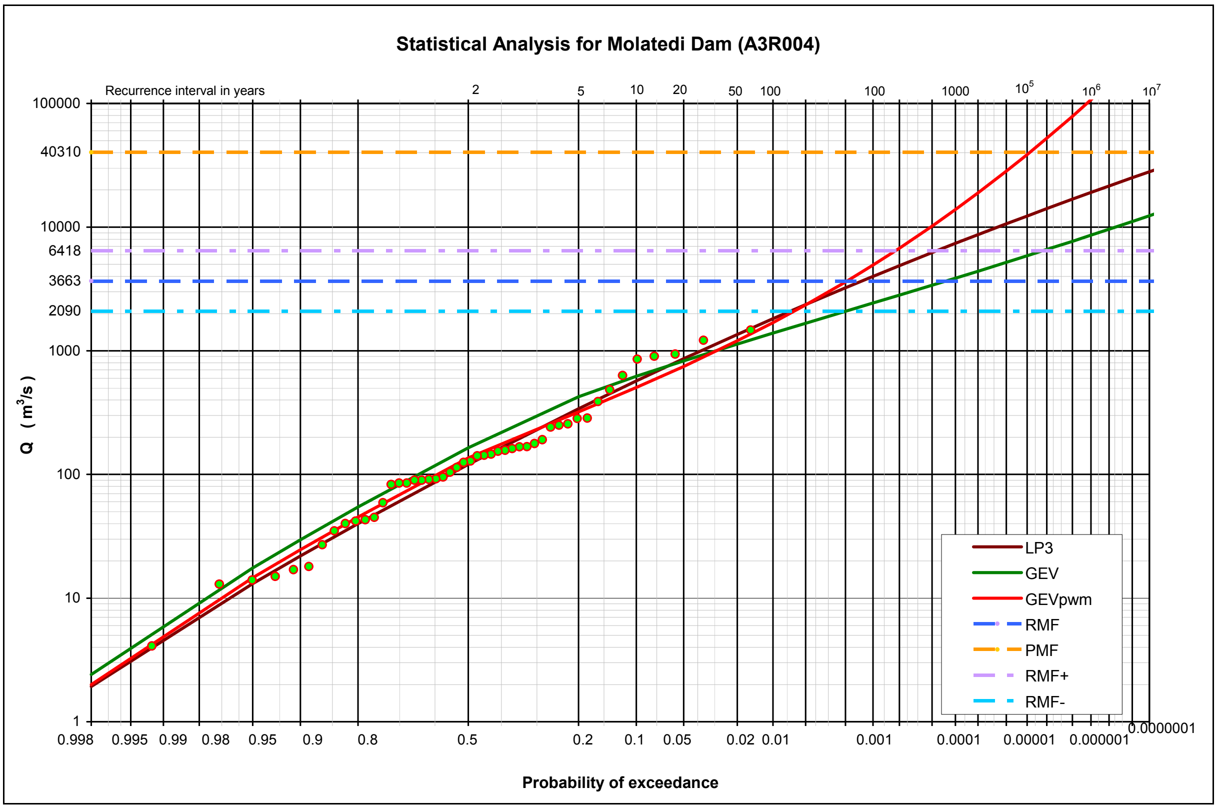

Figure 5.3 Probabilistic Flood Analysis for Molatedi Dam (A3R004)

- An "input" sheet, where the flow measuring station's catchment characteristics and PMF magnitude is entered, and an "output" sheet, where a summary table of the final results of the probabilistic analysis can be viewed, was included for convenience to the user.

5.4.2 Preparation of results

To be able to view and interpret the values of RI that were estimated using the adapted spreadsheets for each design flood, the values calculated from each individual spreadsheet were pulled into a summary spreadsheet where two plots were produced. The first plot is of RI (years) versus Catchment Area (km^2), which gives an indication of the scatter of the results. In the case of the RMF, the latter plots were also sorted according to the station's regional K-value. The second plot produced is of the stations that fall within certain defined bands of K-value versus the RI (years). Originally, the plots were produced separately for the dam sites (see Figure 5.4 for example) and the flow gauge stations (see Figure 5.5 for example) to see if their different forms of derivation yield different trends.

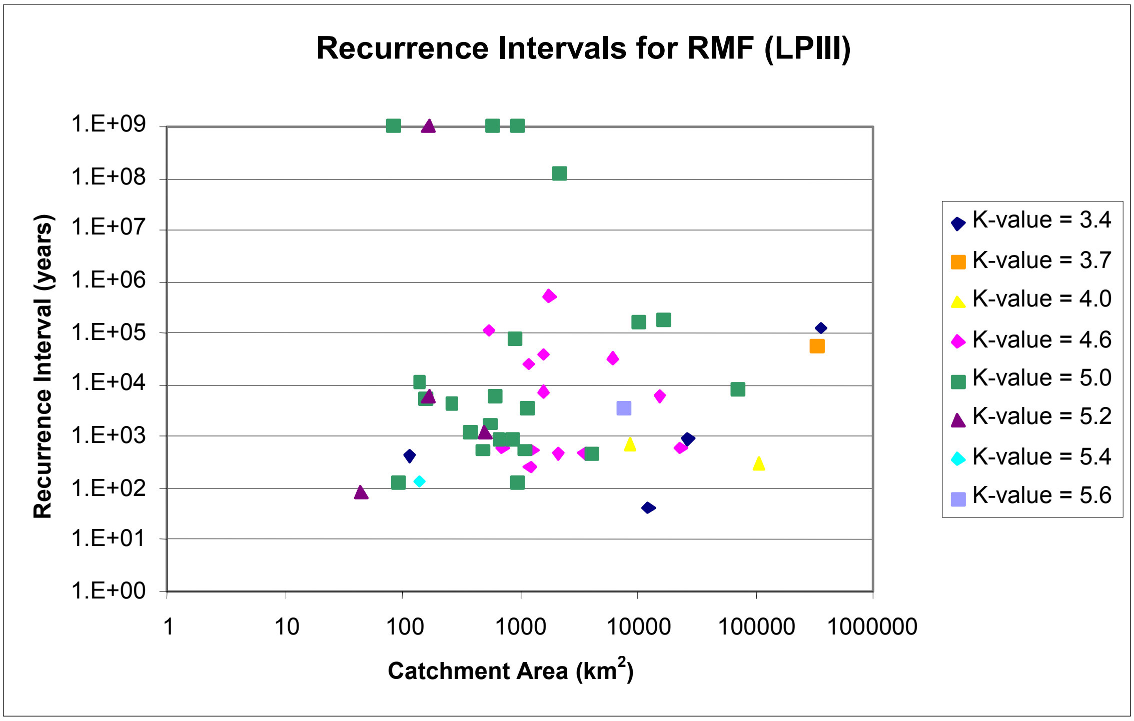

Figure 5.4 RMF results for the dam sites selected for the analysis

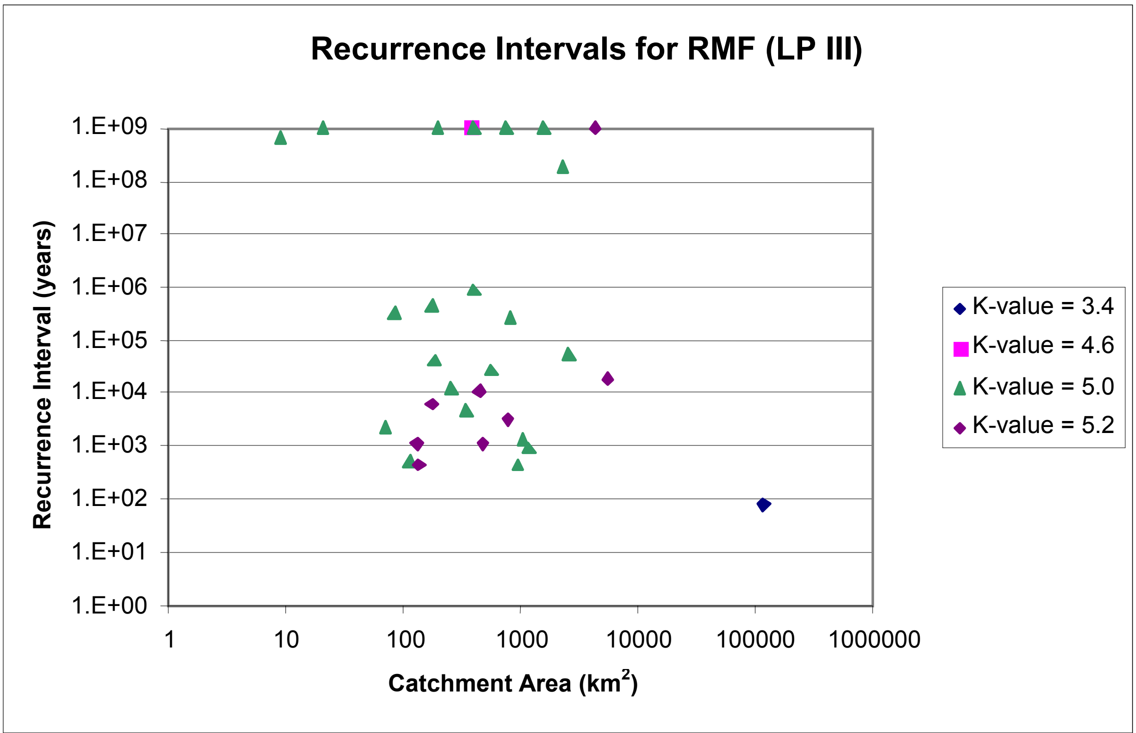

Figure 5.5 RMF results for the flow gauge stations selected for the analysis

For a relatively high proportion of stations, the RI for the RMF and PMF was estimated to be as high as $1:10^9$ years or greater. The probability distributions for these gauging station records have a flatter slope (e.g. Figure 5.6) in comparison to the stations where the RIs were estimated to be less than $1:10^9$ years (e.g. Figure 5.7). Cases of very high probabilities appeared more evident for the flow gauge stations, which in general had shorter flow records, than for the dam sites. In the case of the RMF related values, another attributing factor might be that the Kovaćs regionalisation of some of those stations might have been anomalous in the first place.

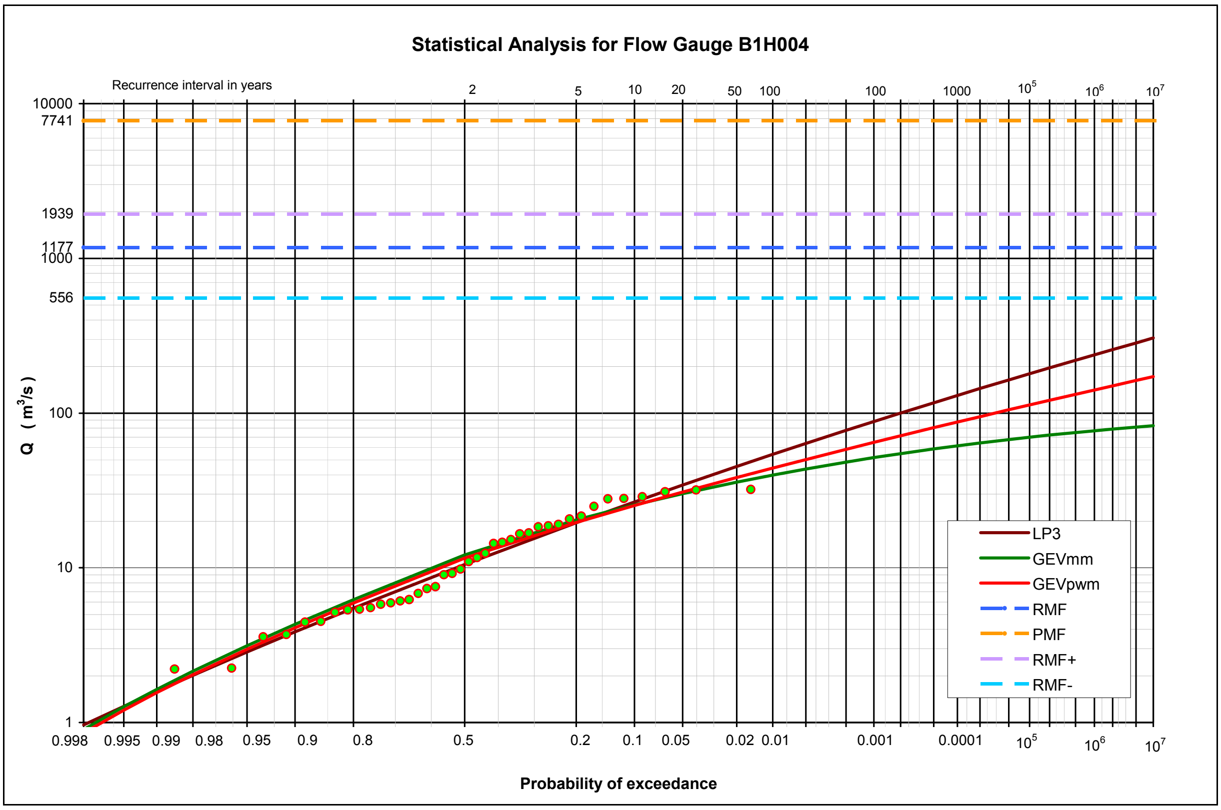

Figure 5.6 Probabilistic Flood Analysis for Flow Gauge B1H004

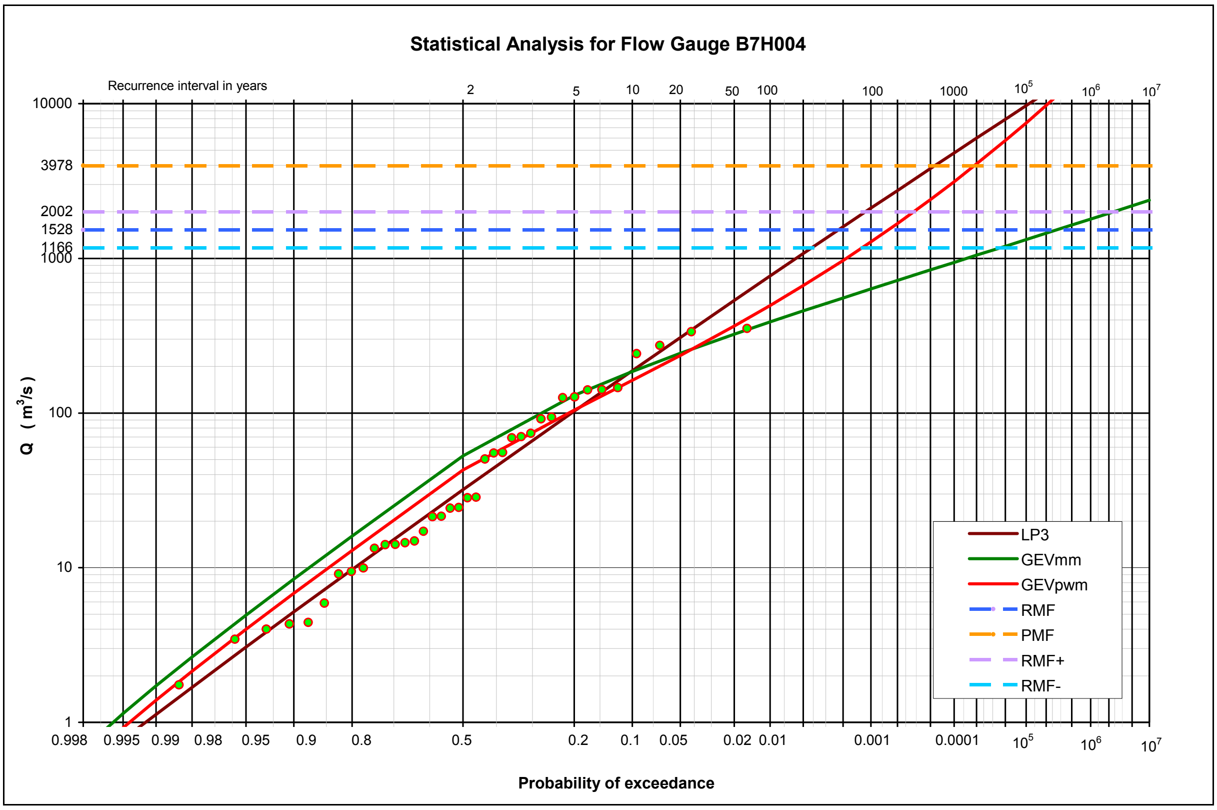

Figure 5.7 Probabilistic Flood Analysis for Flow Gauge B7H004

Given these cases of high probabilities, it was therefore decided to concentrate on the clustering trends and percentile zones evident in the aggregate RI plots for the interpretation of the results. The combined dam site and gauging station results obtained for the RMF, RMF+Δ, RMF-Δ and PMF are presented in Sections 5.4.3 and 5.4.4, respectively.

5.4.3 Results for RMF, RMF+Δ, RMF-Δ

RI Plots

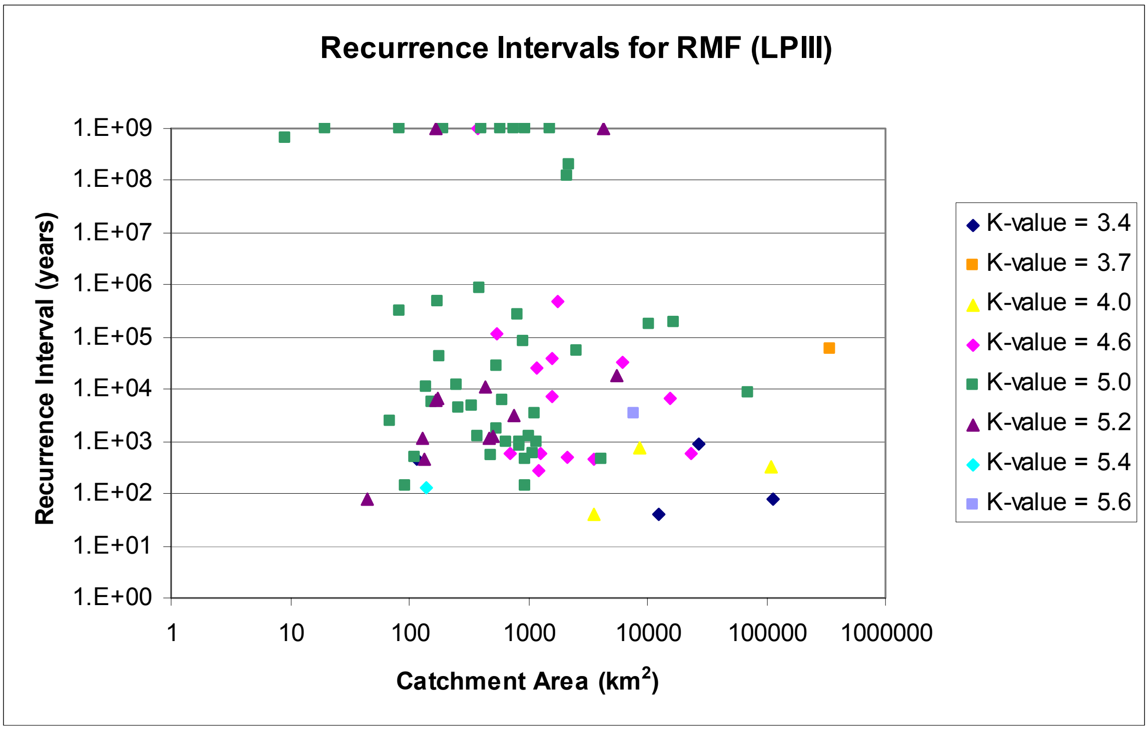

Figure 5.8 RI versus Catchment Area (km^2) for the RMF using the LPIII distribution

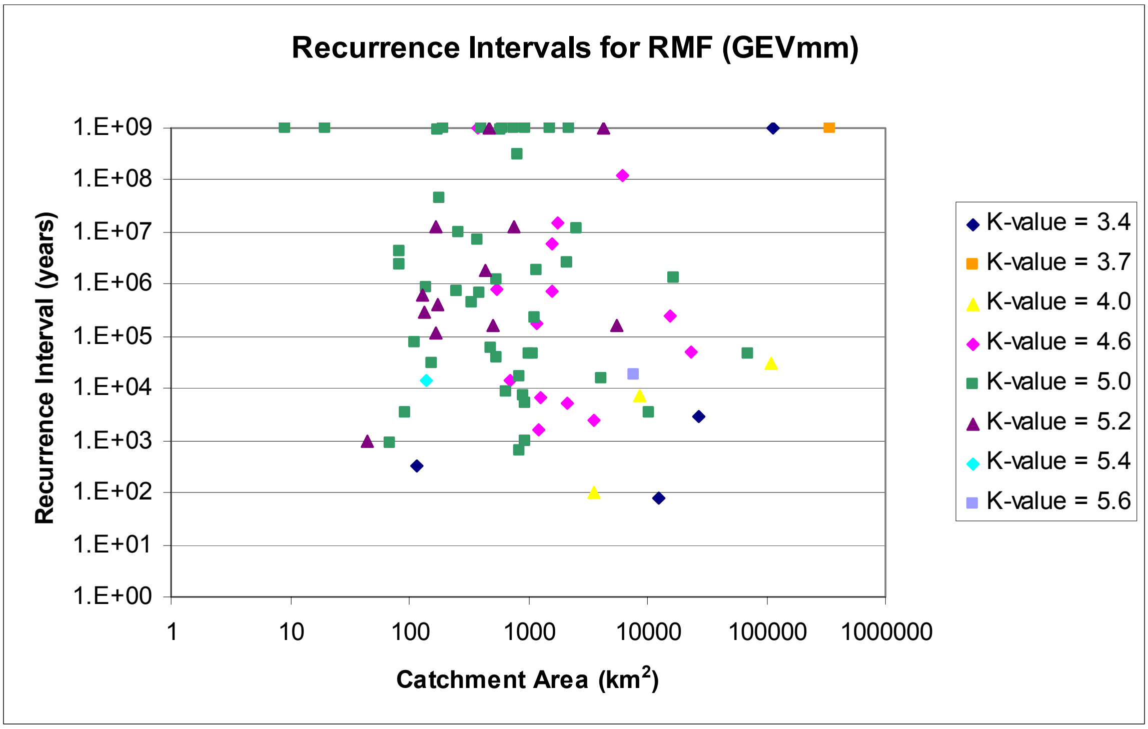

Figure 5.9 RI versus Catchment Area (km^2) for the RMF using the GEVmm distribution

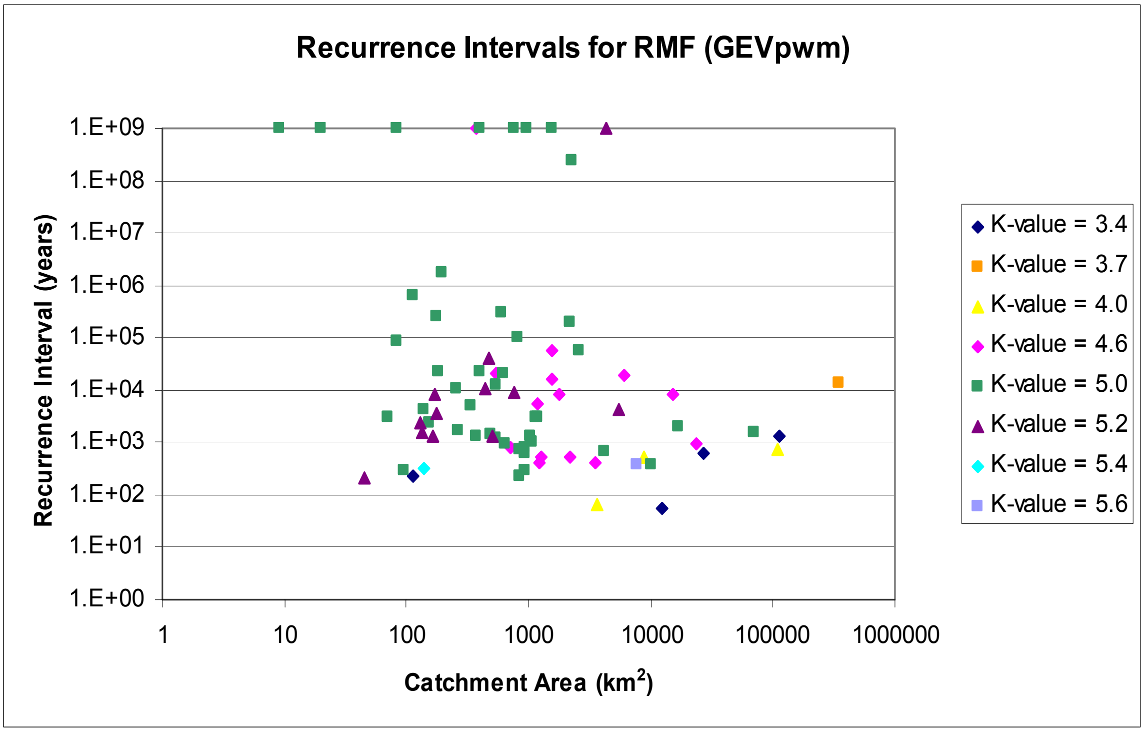

Figure 5.10 RI versus Catchment Area (km^2) for the RMF using the GEVpwm distribution

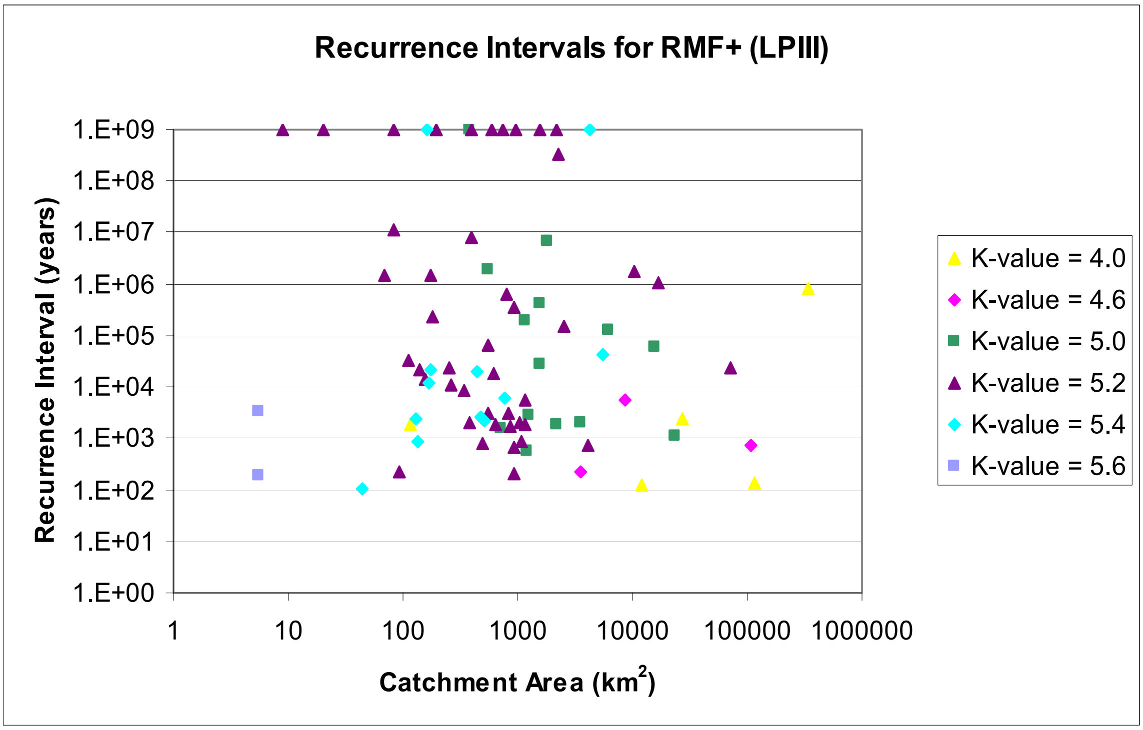

Figure 5.11 RI versus Catchment Area (km^2) for the RMF+Δ using the LPIII distribution

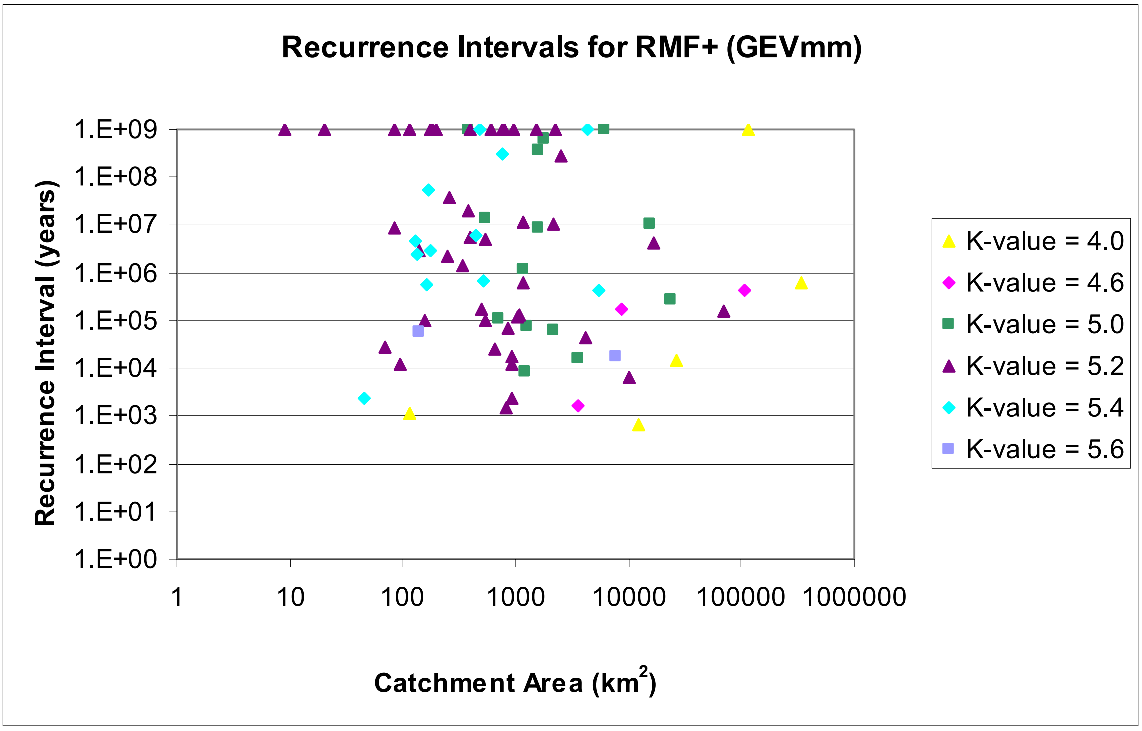

Figure 5.12 RI versus Catchment Area (km^2) for the RMF+Δ using the GEVmm distribution

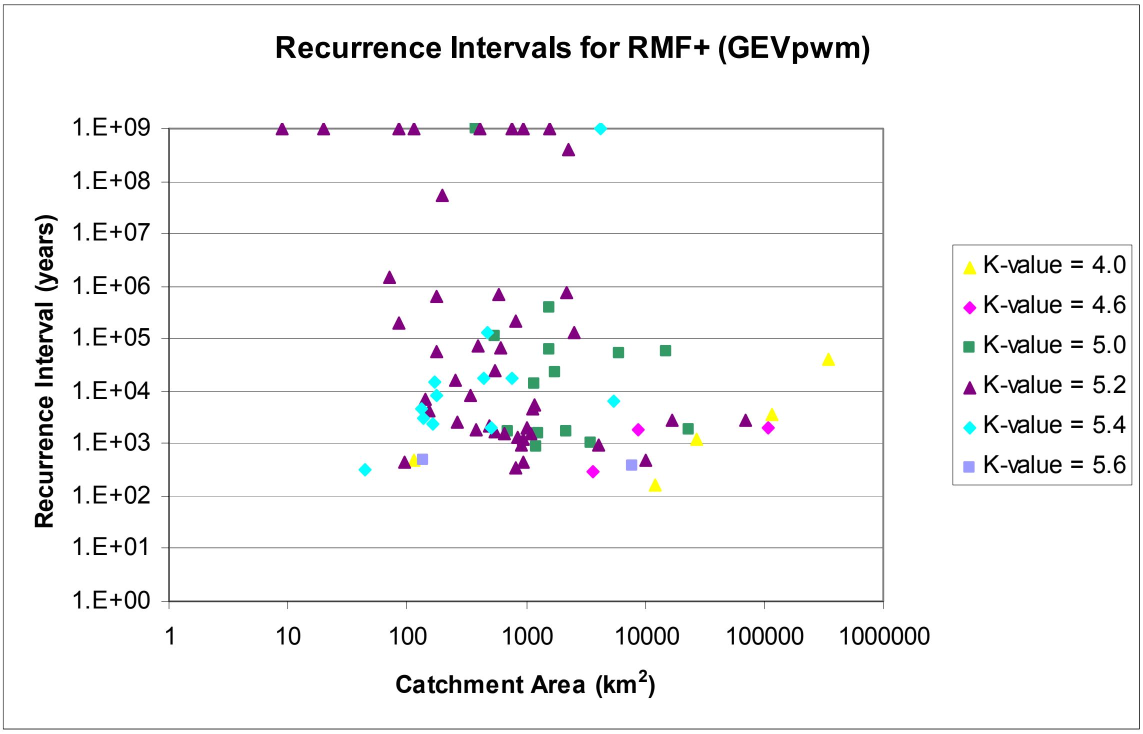

Figure 5.13 RI versus Catchment Area (km^2) for the RMF +Δ using the GEVpwm distribution

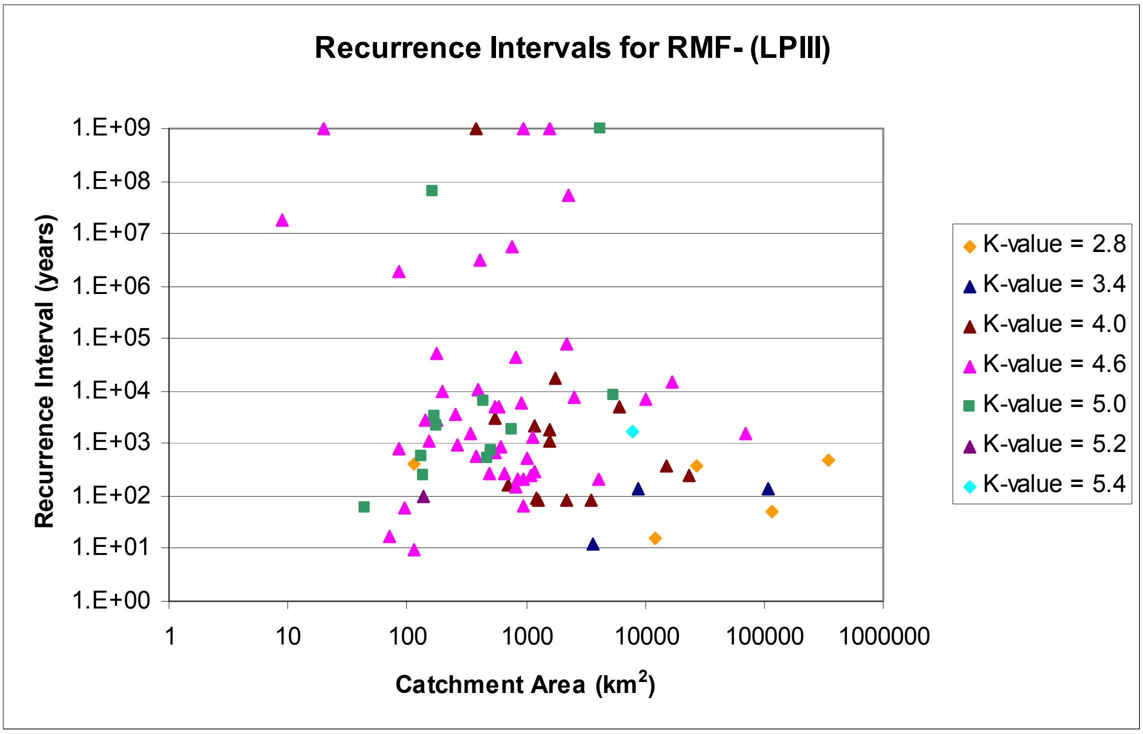

Figure 5.14 RI versus Catchment Area (km^2) for the RMF-Δ using the LPIII distribution

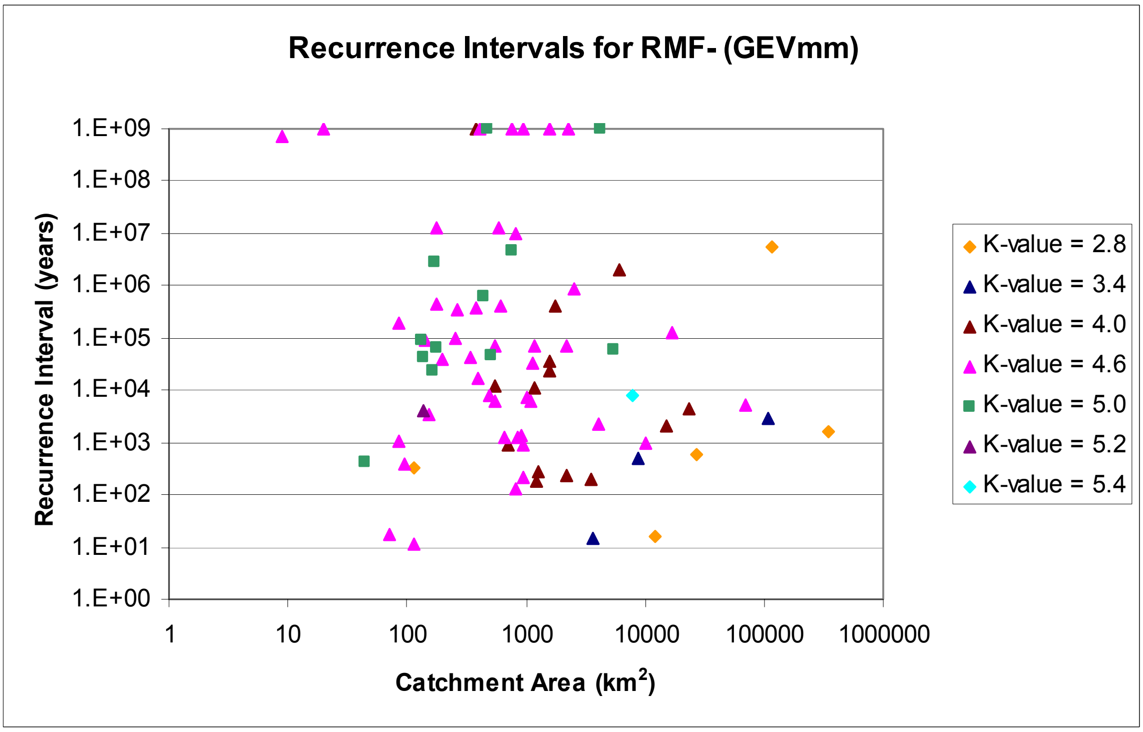

Figure 5.15 RI versus Catchment Area (km^2) for the RMF-Δ using the GEVmm distribution

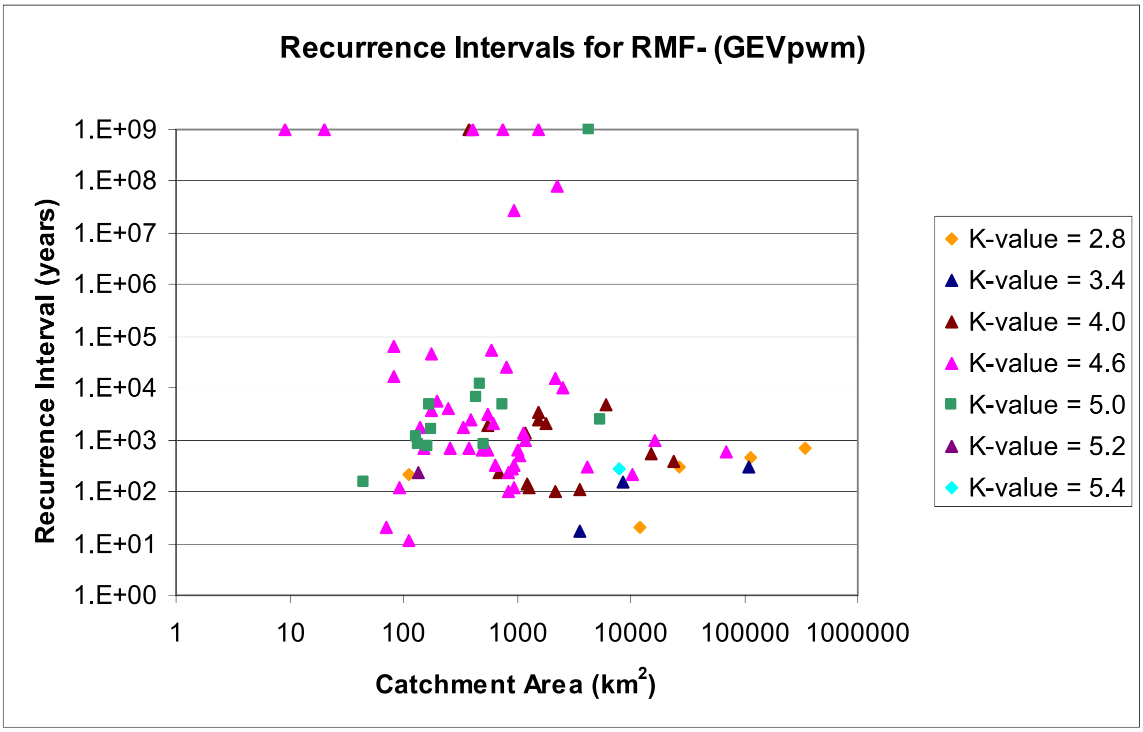

Figure 5.16 RI versus Catchment Area (km^2) for the RMF-Δ using the GEVpwm distribution

Discussion

In general, the GEVmm tails displayed much lower slopes than those of the other two distributions, leading to higher RI estimates for the RMF-related and PMF values.

Table 5.26 shows the RI bands associated with the "cluster of data" for each combination of RMF related flood and probabilistic distribution. This information was obtained from the visual inspection of the plots of Recurrence Interval versus Catchment Area. From Table 5.26 it is evident that the "data cluster" derived from the use of the LPIII and GEVpwm distribution broadly sit within the same RI band for all the RMF related floods. The results obtained using the GEVmm distribution stand out apart from the other two distributions in that the RI band sits higher by about a factor of 10 for the RMF and RMF+Δ and a factor of 100 for the RMF-Δ. This difference between the distributions can also be seen in Table 5.27 which shows the median and lower 95 percentile RI values for the RMF related floods. It is therefore worth considering that the GEVmm distribution be viewed as problematic for the purposes of this project.

In terms of relating the RI to the catchment area, there appears to be no meaningful pattern.

Table 5.26 RMF related floods Recurrence Interval bands for the various probabilistic distributions

Design Flood	Probabilistic Distribution	Data Cluster RI Band
RMF	LPIII	500 – 5 x 10^5
	GEVmm	1000 – 5 x 10^6
	GEVpwm	100 – 1 x 10^5
RMF+Δ	LPIII	500 – 1 x 10^6
	GEVmm	10 000 – 1 x 10^7
	GEVpwm	500 – 1 x 10^6
RMF-Δ	LPIII	50 – 5 x 10^4
	GEVmm	100 – 1 x 10^6
	GEVpwm	100 – 5 x 10^4

Table 5.27 RMF related floods: Median and lower 95 percentile RI value for the various probabilistic distributions

Probability Distribution	Design Flood	Median	Lower 95 Percentile
Log-Pearson Type III	RMF	6000	100
	RMF+Δ	19000	200
	RMF-Δ	1200	50
GEVmm	RMF	347000	750
	RMF+Δ	2666000	1950
	RMF-Δ	29000	100
GEVpwm	RMF	3000	200
	RMF+Δ	6000	400
	RMF-Δ	900	50

An attempt was made to determine whether any trends existed within the data clusters with regards to the Kovaćs regional K-values and the RI. Figure 5.17 is an example of the plots that were produced in which the regional K-values, in this case for the RMF using the GEVpwm distribution, were grouped according to the following groups: where K > 5.0, K = 5.0 and K < 5.0. From the figures produced, it was not immediately evident that a definite pattern existed within the clusters between the grouped K-regions and RI, which could be due to the small sample size. It was therefore decided to not pursue this investigation further for the purpose of this project.

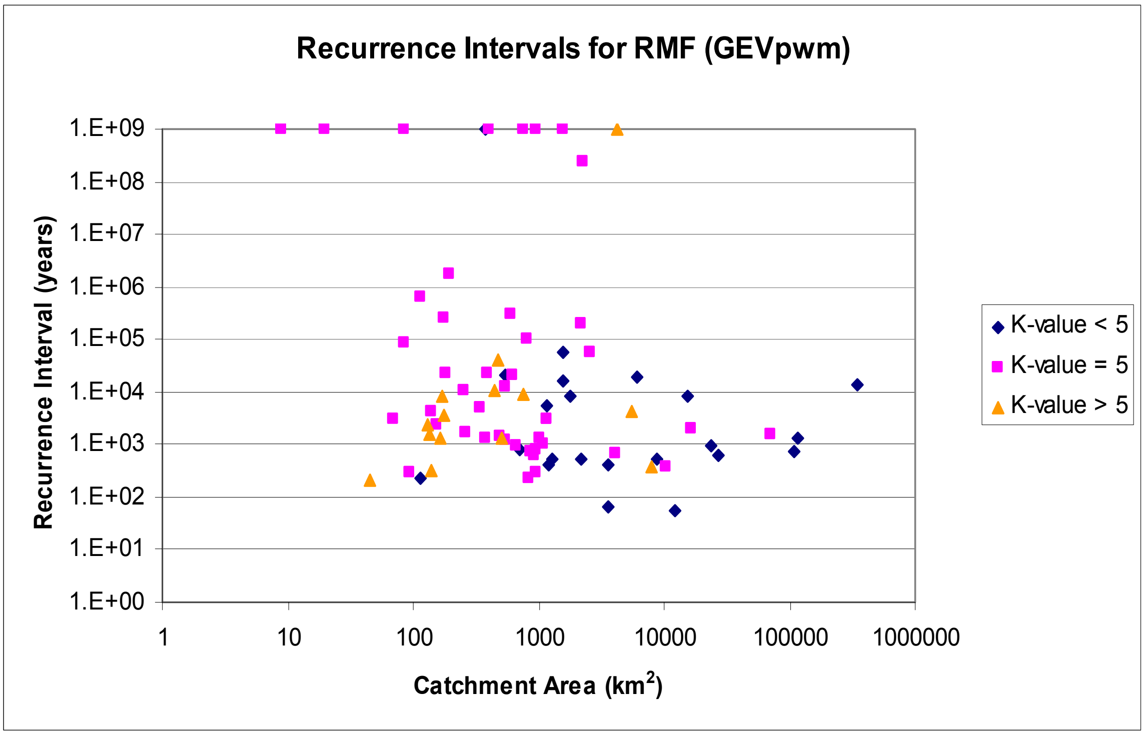

Figure 5.17 RI versus Catchment Area (km^2) for grouped regional K-values

5.4.4 Results for PMF

RI Plots

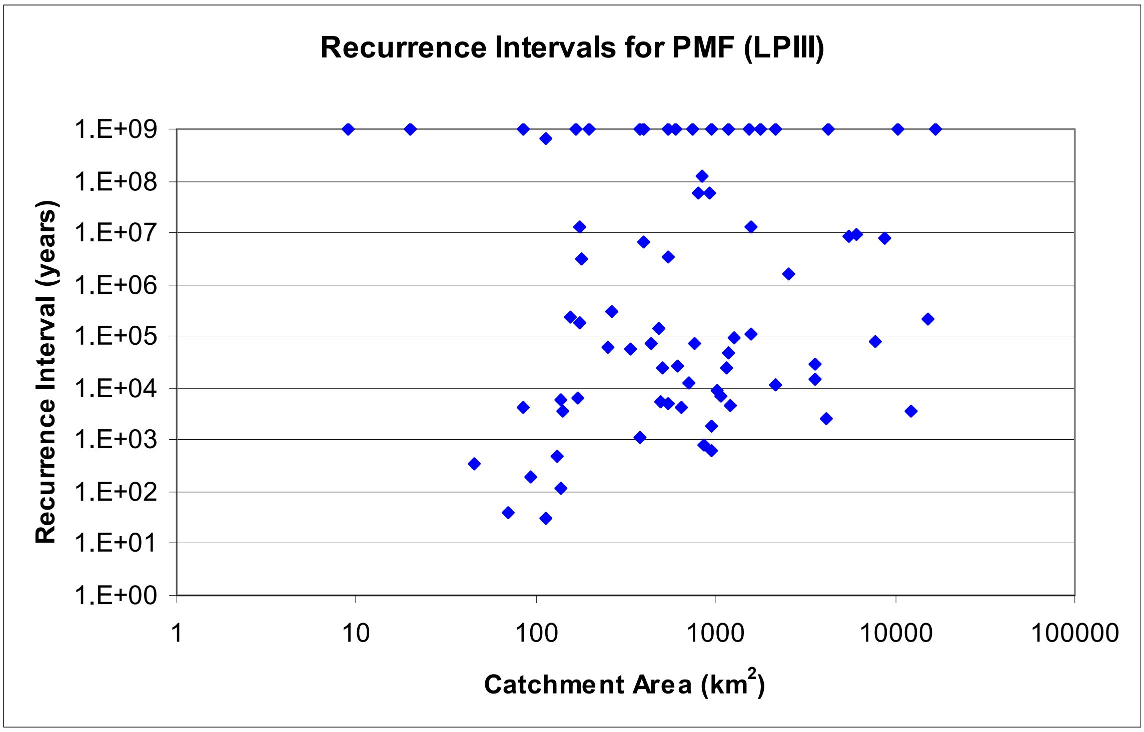

Figure 5.18 RI versus Catchment Area (km²) for the PMF using the LPIII distribution

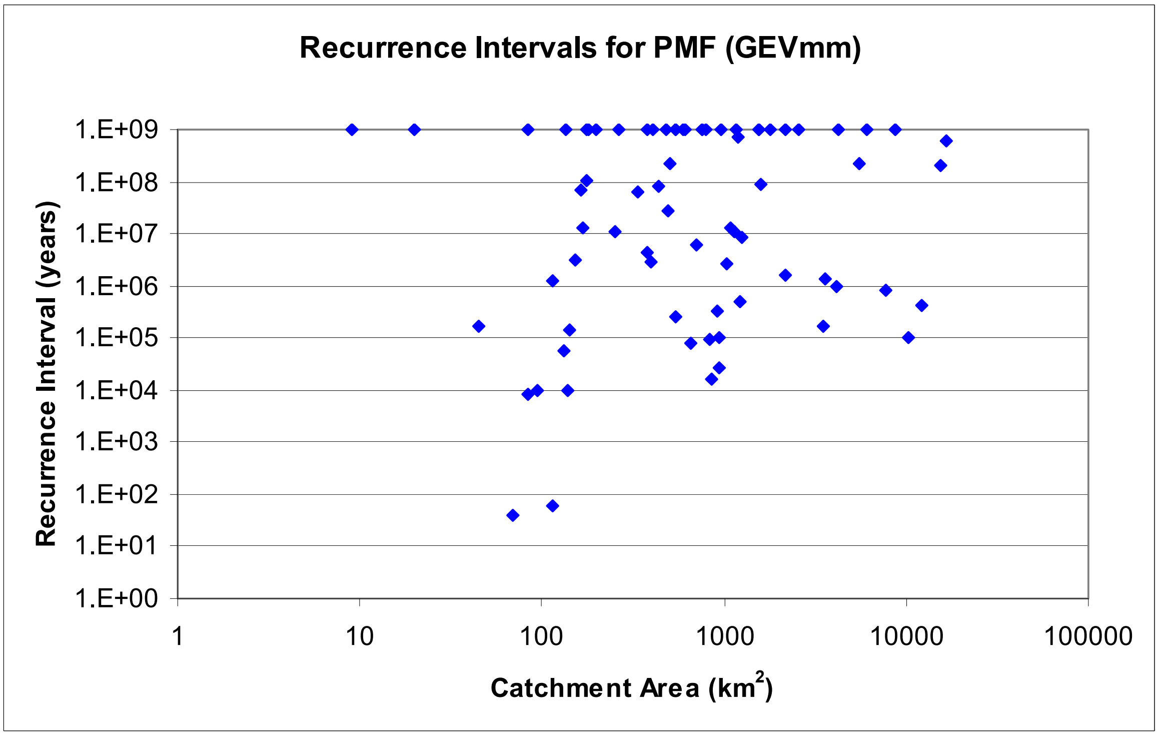

Figure 5.19 RI versus Catchment Area (km²) for the PMF using the GEVmm distribution

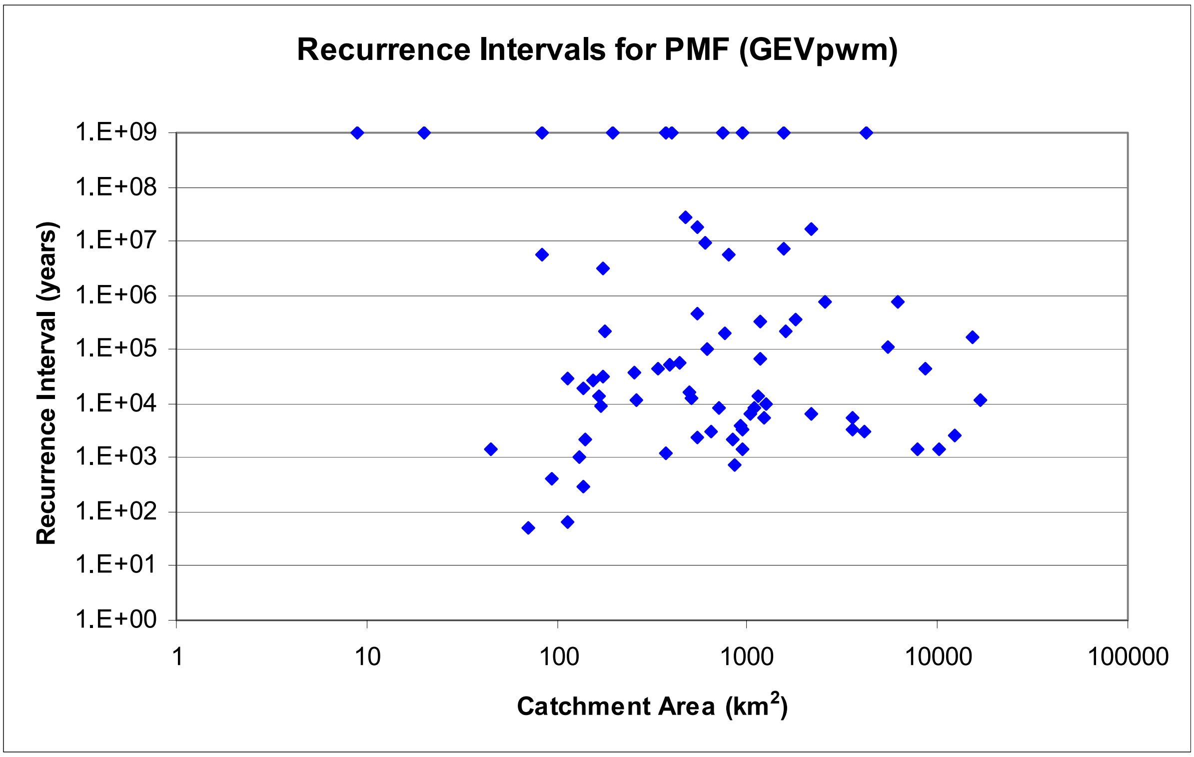

Figure 5.20 RI versus Catchment Area (km^2) for the PMF using the GEVpwm distribution

Discussion

A clustering trend within the RI estimates of the PMF is less evident in comparison to that for the RMF RI estimates. It might be said that by considering the estimates derived from the LPIII and GEVpwm distributions, the PMF RI values lie in the range of 100 to 10^8, with the majority of the values between 1000 and 10^7. As for the RMF RI estimates, the GEVmm distribution gives noticeably higher RI values in comparison to those derived from the LPIII and GEVpwm distributions. The median and 95 percentile PMF RI values are presented in Table 5.28.

Table 5.28 PMF: Median and lower 95 percentile RI value for the various probability distributions

Probability Distribution	Median	95 Percentile
Log-Pearson Type III	106000	300
GEVmm	69330000	10000
GEVpwm	30000	600

The more random pattern associated with the PMF RI estimates might indicate possible inconsistencies in the PMF methodology, in comparison with the RMF method that can be considered as more structured. Inconsistencies in the PMF methodology may arise from the use of the Probable Maximum Precipitation (PMP) estimates developed by HRU (HRU1/69 and 1/72). These estimates are based on only 30 years of rainfall records from 1932 to 1962, since which South Africa has experienced several large flood events. Other inconsistencies could be due to the subjectivity involved in the selection of the catchment characteristics as well as the flood hydrograph technique used to derive the flood.

5.4.5 PMF versus RMF

Spatial distribution of PMF versus RMF ratios

In general, given their different conceptual origins, the PMF is expected to be a much larger value than the RMF at the same site. By considering the spatial distribution of the ratio PMF/RMF for each of the flow measuring stations, some insight might be gained into the internal consistency in the methodology used to derive the two types of design floods, as well as the relative anomalies resulting from any inconsistencies. Table 5.29 presents the PMF/RMF ratios for each of the flow measuring stations and Figure 5.21 shows a map of these ratios that have for ease of viewing been arranged according to the following groups: PMF/RMF < 1, PMF/RMF between 1 and 2, PMF/RMF between 2 and 3, PMF/RMF between 3 and 4 and PMF/RMF > 4.

Table 5.29 PMF/RMF[5] ratios for the flow measuring stations selected for analysis

Station Type	Station Number	Station Name	PMF (m^3/s)	RMF (m^3/s)	PMF/RMF
DAM SITES	A2R001	Hartbeespoort	14229	6419	2.22
	A2R003	Olifantsnek	9434	2218	4.25
	A2R005	Buffelspoort	3202	351	9.13
	A2R006	Bospoort	11242	3283	3.42
	A2R007	Lindleyspoort	6653	1651	4.03
	A2R012	Klipvoor	23092	5311	4.35
	A3R001	Marico Bosveld	10487	2220	4.72
	A3R002	Klein Maricopoort	14915	2182	6.84
	A3R003	Kromellemboog	14875	2729	5.45
	A3R004	Molatedi	27238	3663	7.44
	A6R001	Doorndraai	6368	2439	2.61
	A8R001	Nzhelele	10269	2884	3.56
	A9R001	Albasino Dam	9465	2879	3.29
	B1R001	Witbank	9759	3949	2.47
	B1R002	Middelburg	6053	2551	2.37
	B2R001	Bronkhorstspruit	7848	2263	3.47
	B3R001	Rust de Winter	8014	3384	2.37
	B3R002	Loskop	16173	2621	6.17
	B6R001	Ohrigstad	3543	917	3.87
	B6R003	Blyderivierspoort	10204	4654	2.19
	B7R001	Klaserie	4608	1676	2.75
	B7R003	Tours	2873	843	3.41
	B8R001	Ebenezer	1721	1696	1.01
	B8R005	Tzaneen	4247	2553	1.66
	C5R002	Kalkfontein	25080	10131	2.48
	C5R003	Rustfontein	7273	3061	2.38
	C7R001	Koppies	7636	3020	2.53
	D2R004	Welbedacht	14767	8714	1.69
	H3R001	Poortjieskloof	1208	970	1.25
	H4R002	Keerom	1776	1942	0.91

[5] Noteworthy qualifications regarding the exact values of the PMF in Table 5.29 appear in Footnote 4.

Station Type	Station Number	Station Name	PMF (m^3/s)	RMF (m^3/s)	PMF/RMF
	H7R001	Buffeljags	3407	2478	1.37
	J1R002	Bellair	3776	2337	1.62
	J2R003	Oukloof	771	1187	0.65
	K9R002	Impofu Dam	2919	2926	1.00
	L9R001	Loerie	1846	2015	0.92
	N2R001	Darlington	29078	12923	2.25
	U2R001	Midmar	8072	3041	2.65
	V2R001	Craigie Burn	3572	1241	2.88
	W4R001	Pongolapoort	34424	15591	2.21
	W5R003	Morgenstond	9975	1442	6.92
	X1R001	Nooitgedacht	7659	2545	3.01
	X2R004	Primkop	4794	1622	2.96
	X2R005	Kwena	8658	3089	2.80
FLOW GAUGES	A2H006	-	8280	3206	2.58
	A2H012	-	9645	5051	1.91
	A2H013	-	11367	3422	3.32
	B1H004	-	6257	1177	5.32
	B7H004	-	4014	1528	2.63
	B8H010	-	10299	2791	3.69
	C2H001	-	12041	2155	5.59
	G1H004	-	577	837	0.69
	G1H008	-	2371	1987	1.19
	G2H008	-	287	447	0.64
	H1H006	-	2323	2744	0.85
	H1H007	-	634	917	0.69
	H1H018	-	764	1063	0.72
	H7H005	-	387	300	1.29
	J2H005	-	3094	1591	1.95
	K2H002	-	1077	1501	0.72
	T3H006	-	13590	7989	1.70
	T5H004	-	8385	2335	3.59
	U2H006	-	5361	1841	2.91
	U2H011	-	3515	1729	2.03
	U2H012	-	6107	2679	2.28
	V1H009	-	5134	1400	3.67
	V2H002	-	6593	3061	2.15
	W5H005	-	10733	2835	3.79
	X1H001	-	33194	9023	3.68
	X2H008	-	2490	1342	1.86
	X2H011	-	5537	2005	2.76
	X2H015	-	15589	3942	3.95
	X3H001	-	2783	1319	2.11
	X3H006	-	11202	3503	3.20

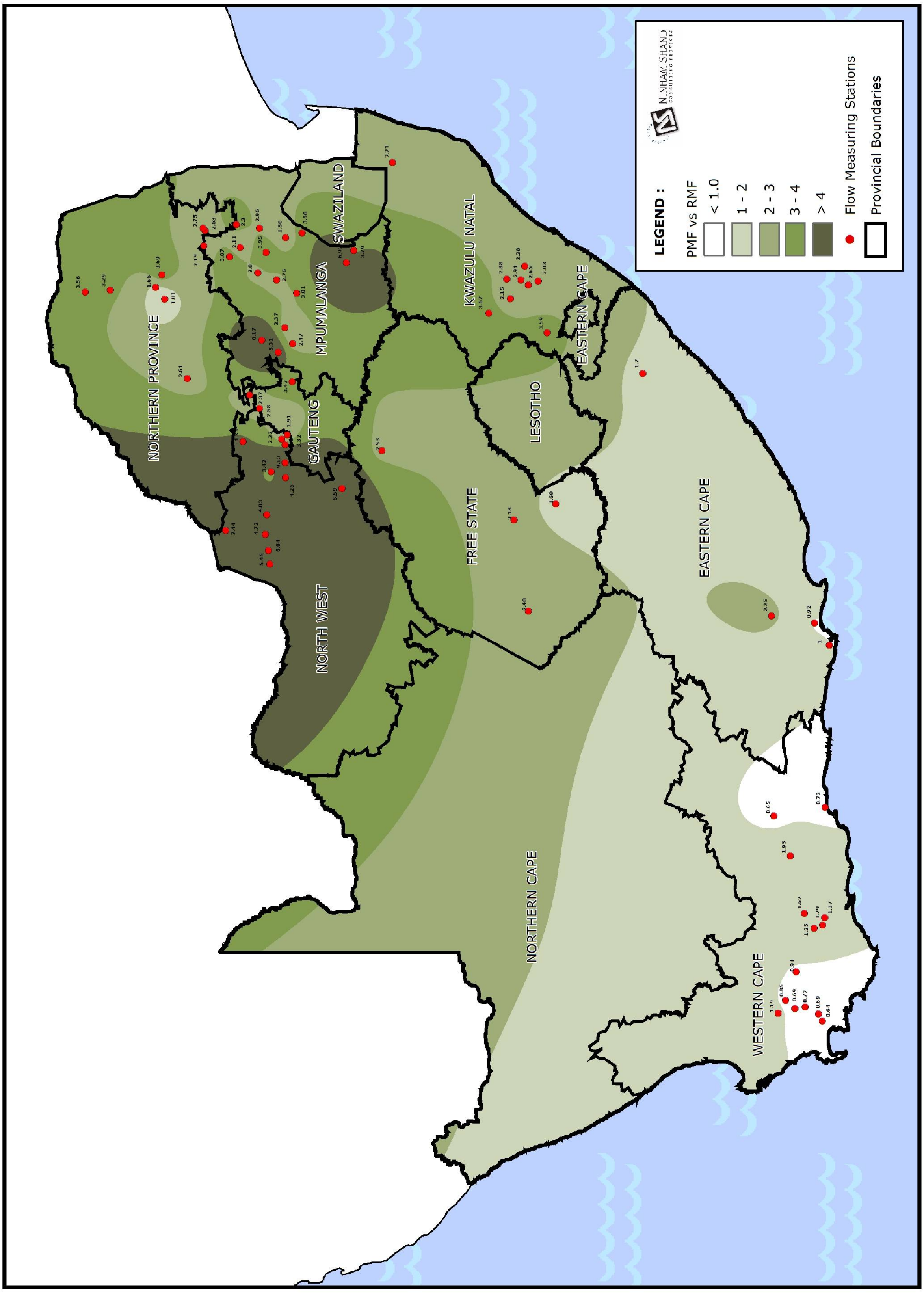

Figure 5.21 Map indicating the spatial distribution of PMF/RMF for the flow measuring stations considered in the analysis

From Figure 5.21, it is evident that a clear spatial pattern of PMF/RMF ratios exists across the country. The smaller PMF/RMF ratios of < 1 are evident along the coastal area of the Western Cape and Eastern Cape, whereas the largest PMF/RMF ratios of > 4 are evident for the northern parts of the country. There appears to be a trend that the PMF/RMF ratios decrease from the coast towards the interior.

Spatial distribution of PMF and RMF Recurrence Intervals

Following from the above analysis, where the PMF and RMF were compared in terms of their magnitudes, it was felt that a valuable next step would be to consider the spatial distribution of the PMF and RMF nationally in terms of their RI values. To accomplish this, it was decided to only consider the RI values of the PMF and RMF as derived using the GEVpwm distribution. The individual RI values for each of the flow measuring stations considered in this study were arranged into broad bands, which were then used to generate spatial maps. Figure 5.22 and Figure 5.23 represent the spatial maps of the PMF and RMF RI values, respectively.

From Figure 5.22, the spatial pattern of PMF RI values indicate that the PMF RI is greater in the north east of the country and smaller in the Western Cape with the exception of some local anomalies, such as the small areas in the Western Cape where the PMF RI is indicated as falling within the 1:10^5 to 1:10^6 range. The spatial pattern of the RMF RI values (Figure 5.23) shows a slightly different relationship in that there appears to be fairly high RI values in both the Western Cape and north eastern part of the country. In general, however, the RMF RI values across the country appear to be about an order of magnitude of 10 lower than the PMF RI values, except in the Western Cape where the RMF and PMF values are both indicated to fall within the 1:10^4 to 1:10^5 band. This latter finding is consistent with the findings from the "PMF versus RMF ratios" investigation, where the PMF/RMF ratios for the Western Cape were found to be in the proximity of 1.

5.4.6 RMF RI based on Kovaćs's methodology

Pegram et al. (2004) undertook a pilot study into the possible assignment of a recurrence interval to the RMF in South Africa. They based their research on Kovaćs's original (1988) database of annual flood peaks, and, using truncated probabilistic flood analysis, concluded that it was reasonable to propose that the RMF be assigned a return period of the order of 200 years. This result differs quite substantially from the finding in this research that shows the lower 95 percentile RI of the RMF to be in the order of 1:200 years.

Given the discrepancy between the two findings, it was therefore decided to introduce a further approach to the examination of the RMF RI. The approach was based on Kovaćs's own methodology. Kovaćs (1988) provides a Table (in Appendix 6) listing factors, Q_T/RMF, that can be applied to the RMF in order to determine the magnitude of a design flood (Q_T) of known recurrence interval. For the 1:200 year flood, the average Q_T/RMF for South Africa is 0.69. This implies that the RMF can be seen to be nearly 1.5 times (1/0.69 ≈ 3/2) the magnitude of the 1:200 year flood. This in turn could indicate the return period of the RMF to be significantly larger than 1:200 years.

To test the above interpretation, probabilistic flood analysis was preformed for the estimation of the 1:200 year flood peak magnitude, based on records of annual flow maxima for 73 flow measuring stations across South Africa. The LPIII probability distribution was chosen for use in the analysis. With the catchment area and K-region of each station known, the factor Q_T/RMF was read off from Appendix 6 of Kovaćs (1988). This factor, once inversed, was then multiplied with the 1:200 year flood peak magnitude to determine the "theoretical RMF" magnitude. Once again using probabilistic flood analysis (LPIII distribution), the RI of the "theoretical RMF" was determined using the same method as that described under Section 5.4.1. The results of this investigation are shown in Table 5.30.

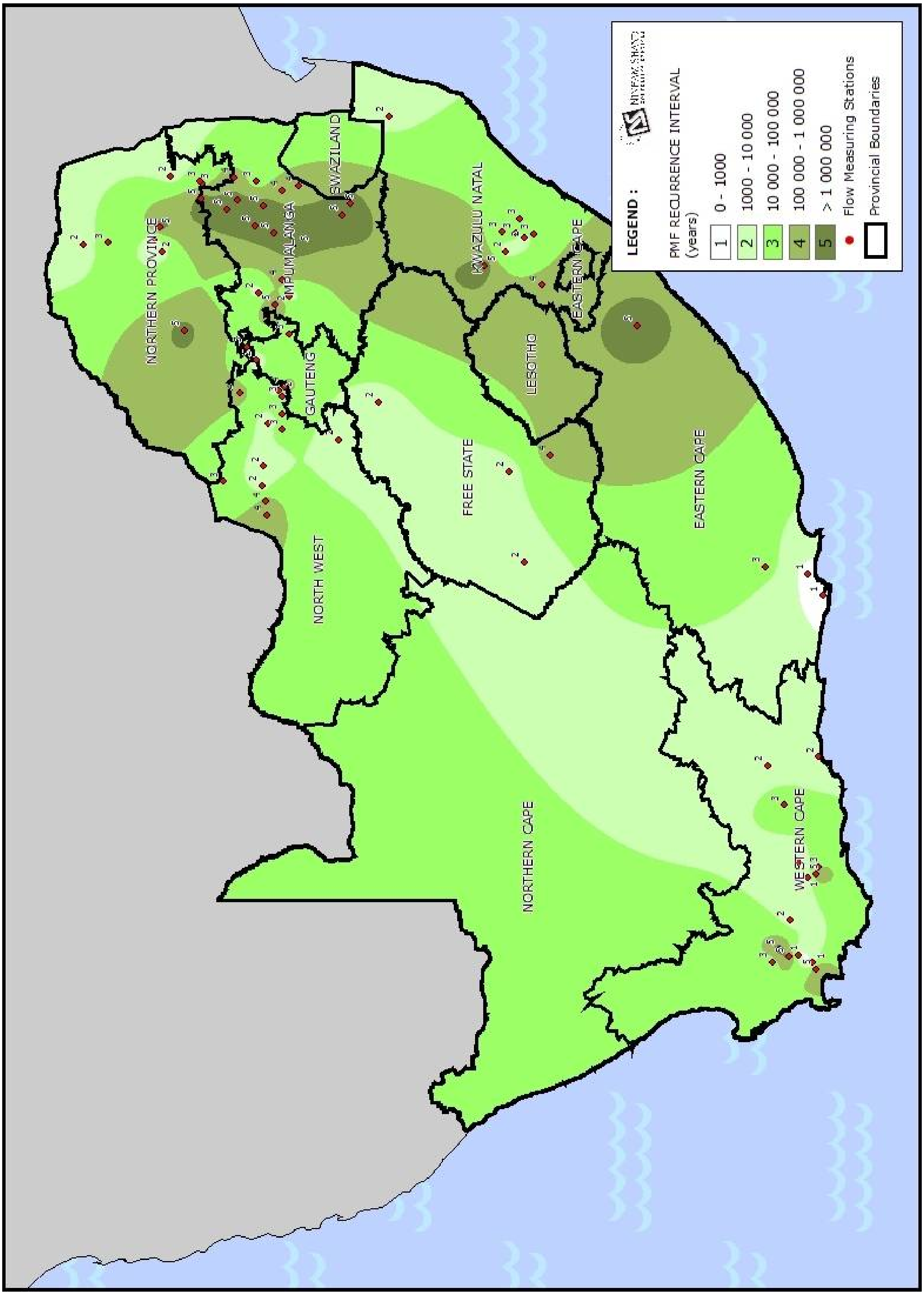

Figure 5.22 Map indicating the spatial distribution of PMF RIs for the flow measuring stations considered in the analysis

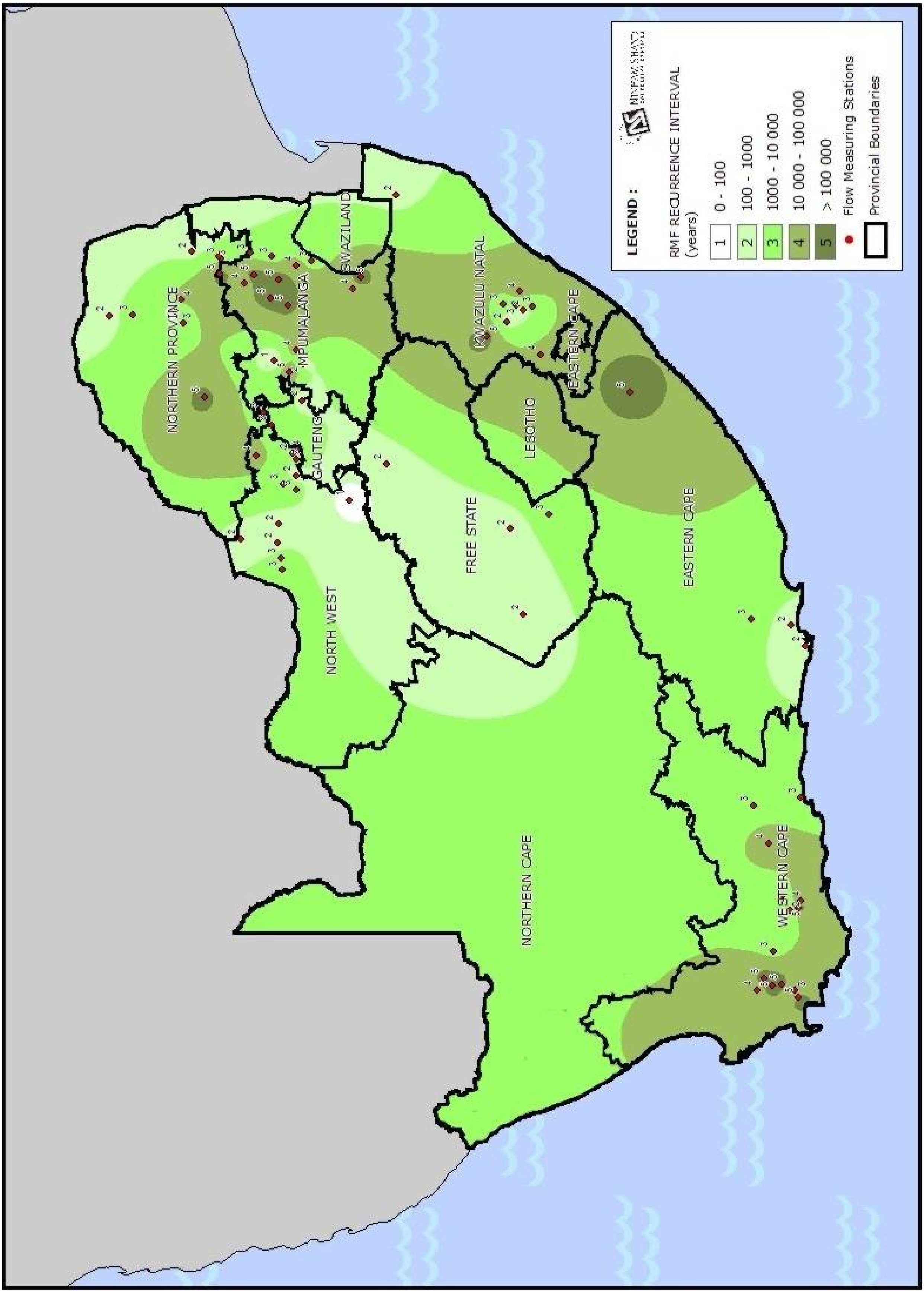

Figure 5.23 Map indicating the spatial distribution of RMF RIs for the flow measuring stations considered in the analysis

Table 5.30 "Theoretical RMF" RI values for selected flow measuring stations

Station Type	Flow Measuring Station	Area km²	K-region	Q_{200} (m^3/s)	Factor (Q_{200}/RMF)	"Theoretical RMF" (m^3/s)	"Theoretical RM" RI (LP III) (years)
DAM SITES	A2R001	4120	5	4213	0.692	6088	433
	A2R003	492	5	1123	0.641	1753	395
	A2R005	114	3.4	277	0.568	488	1684
	A2R006	1078	5	1804	0.662	2725	422
	A2R007	704	4.6	916	0.618	1483	492
	A2R012	6128	4.6	936	0.674	1387	562
	A3R001	1219	4.6	1869	0.632	2956	444
	A3R002	1180	4.6	490	0.632	776	731
	A3R003	1786	4.6	427	0.641	666	870
	A3R004	8703	4	2364	0.740	3195	488
	A6R001	595	5	279	0.645	433	1492
	A8R001	832	5	1939	0.654	2963	1106
	A9R001	509	5.2	1243	0.658	1889	483
	B1R001	3541	4.6	2968	0.662	4480	751
	B1R002	1576	4.6	667	0.638	1045	650
	B2R001	1263	4.6	1594	0.633	2517	841
	B3R001	1145	5	841	0.663	1269	451
	B3R002	12262	3.4	5527	0.642	8612	629
	B5R002	23566	4.6	5969	0.714	8364	376
	B6R001	84	5	119	0.614	193	1593
	B6R003	2166	5	1394	0.676	2062	2345
	B7R001	165	5.2	471	0.632	745	3926
	B7R003	45	5.2	1850	0.644	2870	351
	B8R001	169	5.2	377	0.633	595	531
	B8R005	652	5	1408	0.647	2176	629
	C3R002	26922	3.4	1865	0.667	2798	419
	C5R002	10264	5	3530	0.718	4914	884
	C5R003	937	5	3848	0.658	5843	428
	C7R001	2154	4.6	2229	0.647	3445	759
	C9R002	108125	4	13252	0.805	16460	331
	D2R004	15330	4.6	4438	0.701	6335	1125
	D3R002	70749	5	13213	0.767	17235	745
	H3R001	94	5	1208	0.610	1980	474
	H4R002	377	5	624	0.636	981	422
	H7R001	614	5	1033	0.646	1600	1044
	J1R002	546	5	756	0.643	1176	453
	J2R003	141	5	209	0.613	340	554
	K9R002	856	5	1790	0.655	2732	693
	L9R001	138	5.4	2816	0.612	4603	370
	N2R001	16700	5	3641	0.728	5002	693
	U2R001	925	5	877	0.658	1333	1130
	V2R001	154	5	390	0.615	635	793
	W4R001	7814	5.6	6451	0.829	7778	371

Station Type	Flow Measuring Station	Area km^2	K-region	Q_{200} (m^3/s)	Factor (Q_{200}/RMF)	"Theoretical RMF" (m^3/s)	"Theoretical RM" RI (LP III) (years)
	W5R003	548	4.6	350	0.612	572	1396
	X1R001	1569	4.6	732	0.638	1148	1076
	X2R004	263	5	534	0.628	850	667
	X2R005	954	5	297	0.659	450	4867
FLOW GAUGES	X3H006	766	5.2	1075	0.667	1611	496
	A2H006	1028	5	1208	0.661	1827	438
	A2H012	2551	5	1283	0.681	1884	801
	A2H013	1171	5	1840	0.663	2774	550
	B1H004	376	4.6	64	0.605	106	3059
	B7H004	136	5.2	1073	0.629	1707	604
	B8H010	477	5.2	1532	0.657	2333	685
	C2H001	3595	4	3860	0.720	5359	655
	C9H008	115057	3.4	23164	0.713	32477	329
	G1H008	395	5	627	0.637	985	2686
	H7H005	9	5	49	0.662	74	4000
	J2H005	253	5	237	0.627	378	378
	K2H002	131	5.2	734	0.628	1169	624
	T3H006	4268	5.2	1554	0.707	2199	9700
	T5H004	545	5	533	0.643	829	836
	U2H006	339	5	427	0.637	670	514
	U2H011	176	5.2	673	0.634	1062	955
	U2H012	438	5.2	411	0.655	627	478
	V2H002	937	5	1898	0.658	2882	425
	W5H005	804	5	353	0.653	540	758
	X1H001	5499	5.2	2491	0.712	3497	587
	X2H008	180	5	404	0.618	654	1243
	X2H011	402	5	475	0.637	746	6481
	X3H001	174	5	155	0.617	251	902
	X3H003	2231	5	139	0.677	205	698
	X3H006	766	5.2	1075	0.667	1611	496

Based on the "theoretical RMF" RI values, the median and 95 percentile RI values were calculated and these can be viewed in Table 5.31 below.

Table 5.31 Median and lower 95 percentile RI for the RMF, calculated using Kovaćs's (1988) Q_{200}/RMF ratios

Design Flood	Median RI (years)	Lower 95 Percentile RI (years)	Probability Distribution
RMF	660	370	LPIII

The above analysis illustrates that Kovaćs "expected" the RI of the RMF to be fairly extreme. On the face value of his own Q_{200}/RMF factors, our simple analysis suggests a "working hypothesis" where the RMF RI equals 1:1000 years.

5.4.7 Applicability of K-region envelope curves

Given that we had access to two decades of additional flow gaugings than Kovaćs had at the time of his research, the question arose as to whether the envelope curves developed by Kovaćs for each regional K-value could still be seen as applicable. With the availability of longer flow records, there is a possibility of larger flood events having been recorded in comparison to those selected by Kovaćs in his research prior to 1988. In order to check Kovaćs's envelope curves for the various K-regions, the flood database, which was developed as part of this Project and is reported in Part 1 of this report, was used. From this database, the primary flow record at 91 flow measuring and dam site stations was scrutinised and the largest instantaneous flood peak identified. These floods peaks were then plotted against Kovaćs's envelope curves for the relevant K-region. The envelope of world recorded flood peaks was also included in the plots for comparative purposes. The results of this analysis can be viewed in Figure 5.24 to Figure 5.30 below. It should be noted that although there was only one recorded flood peak available from the flood database for each of K-regions 5.4 and 5.6, these plots have still been included for completeness sake.

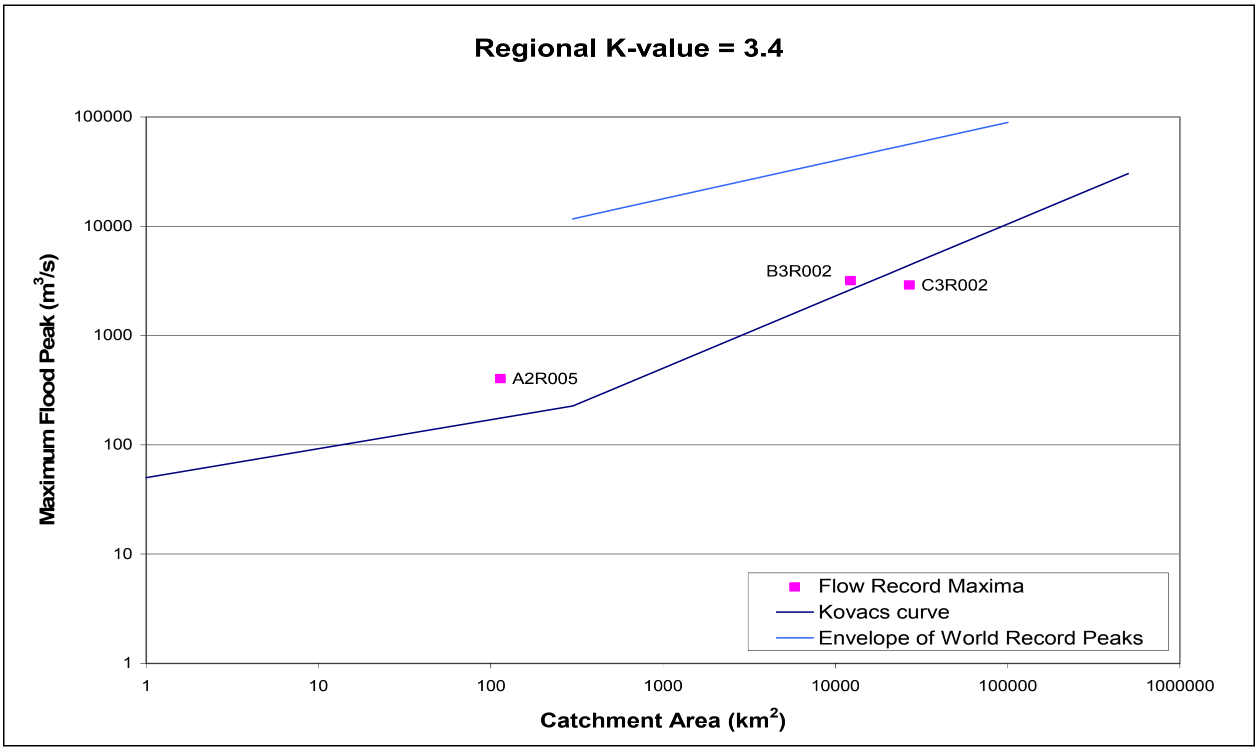

Figure 5.24 Maximum observed flood peaks for K-value 3.4

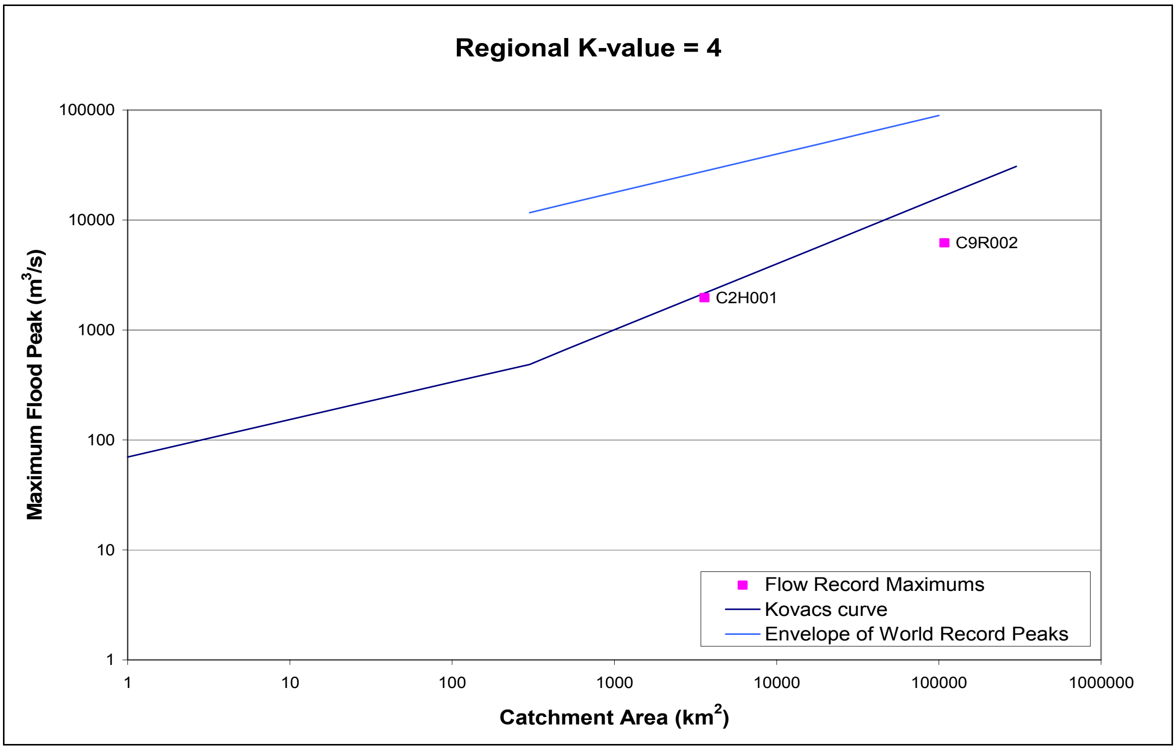

Figure 5.25 Maximum observed flood peaks for K-value 4

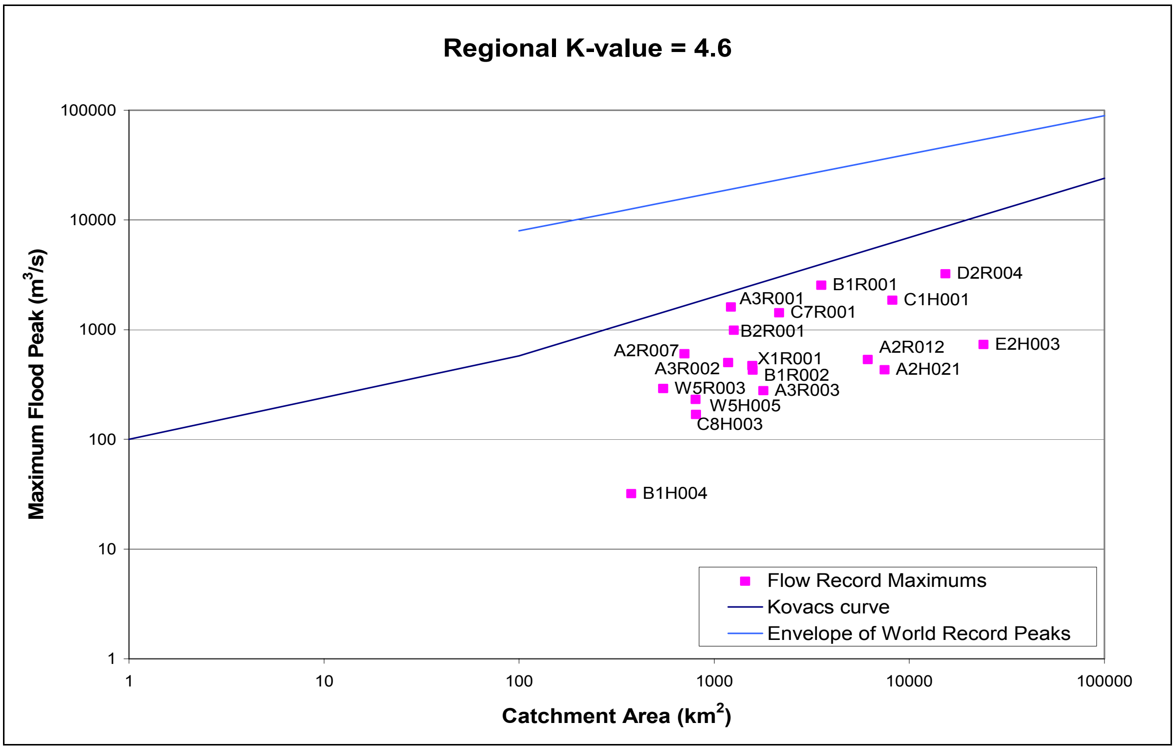

Figure 5.26 Maximum observed flood peaks for K-value 4.6

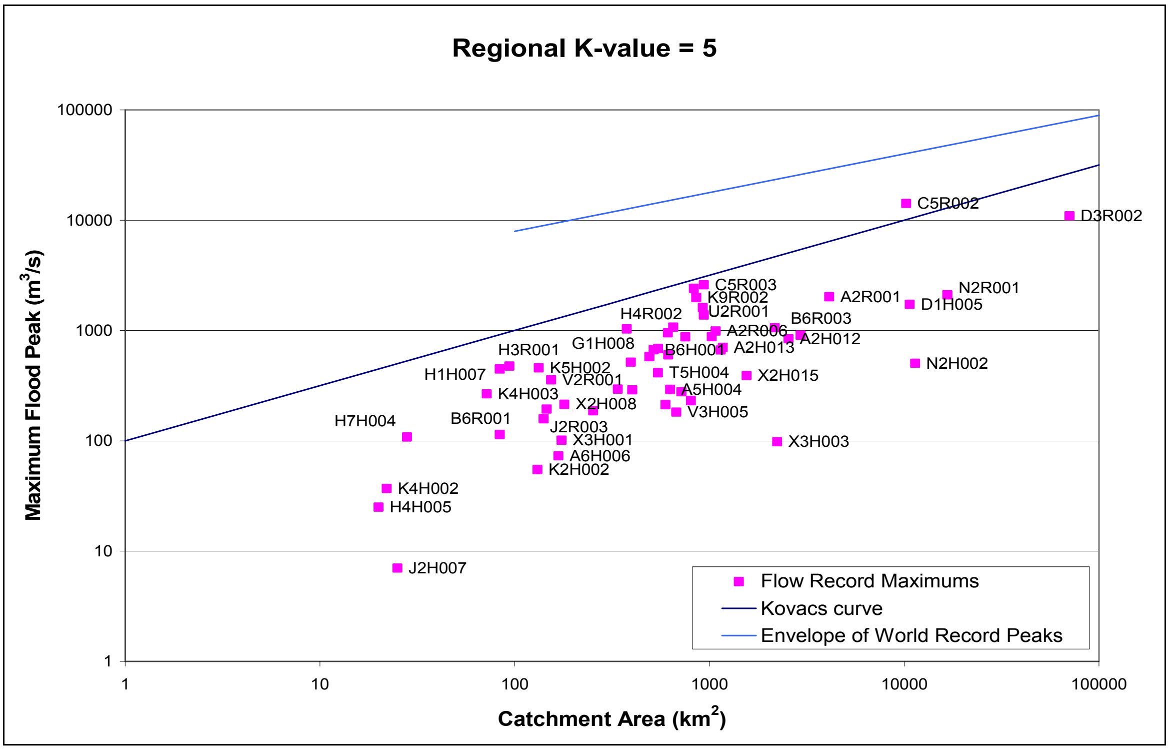

Figure 5.27 Maximum observed flood peaks for K-value 5

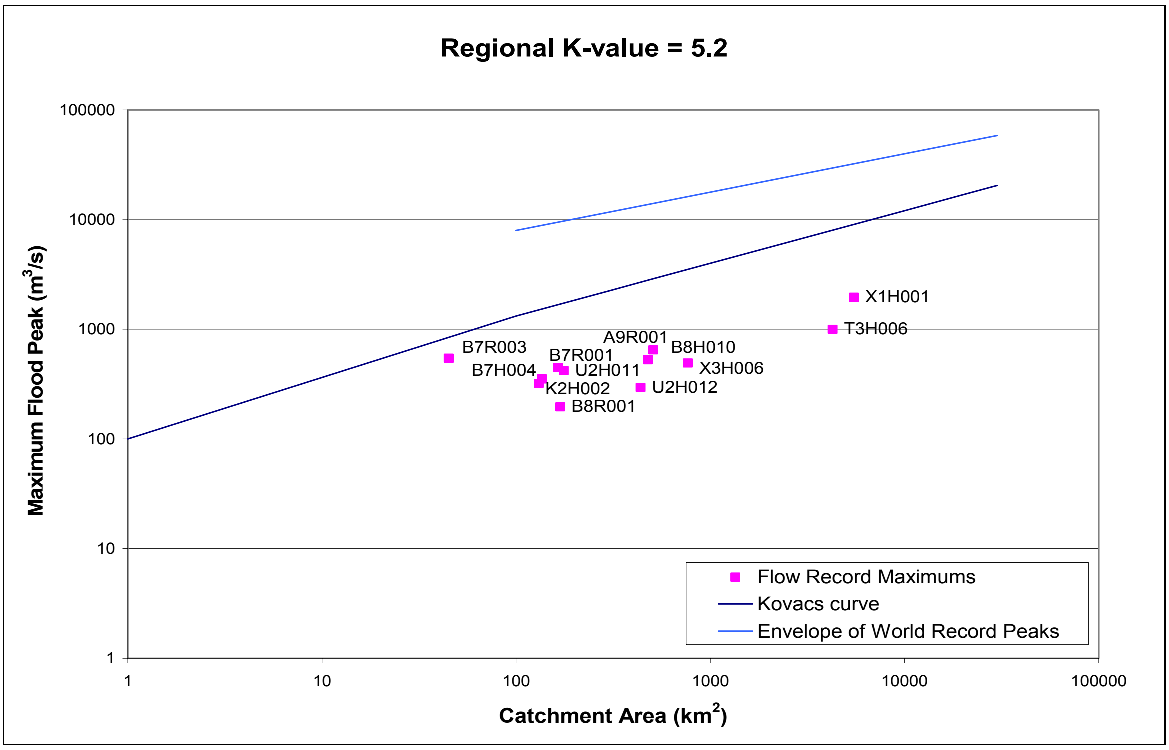

Figure 5.28 Maximum observed flood peaks for K-value 5.2

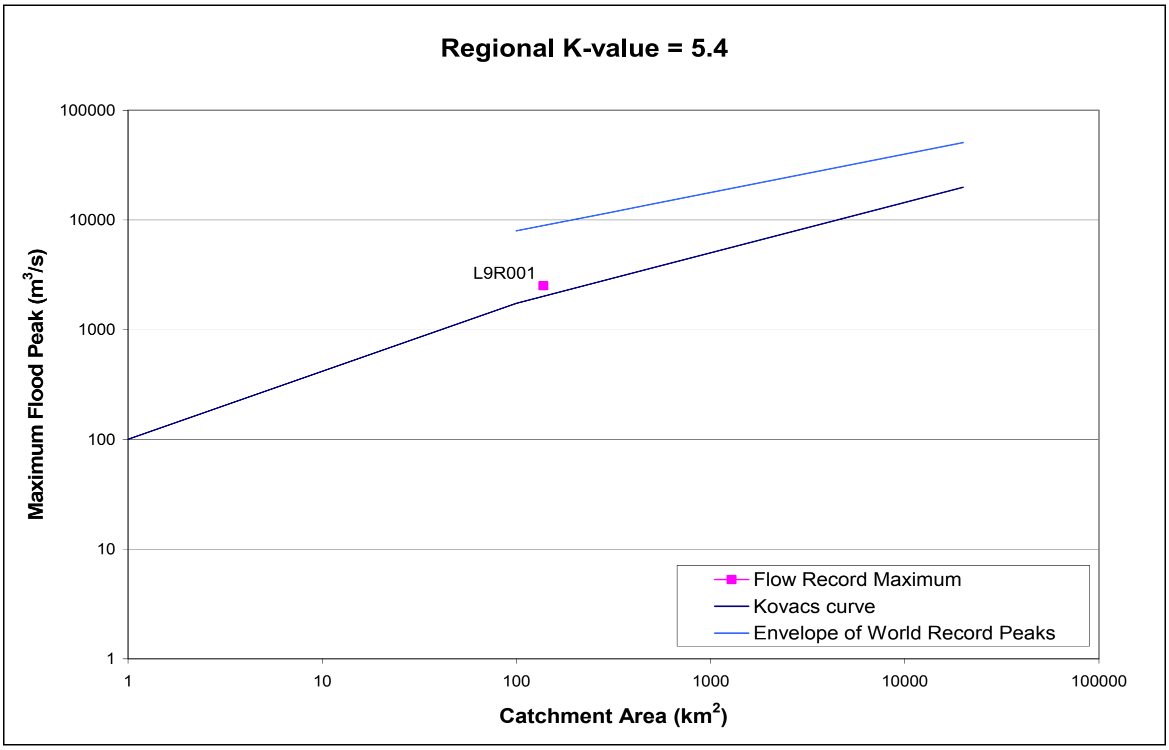

Figure 5.29 Maximum observed flood peaks for K-value 5.4

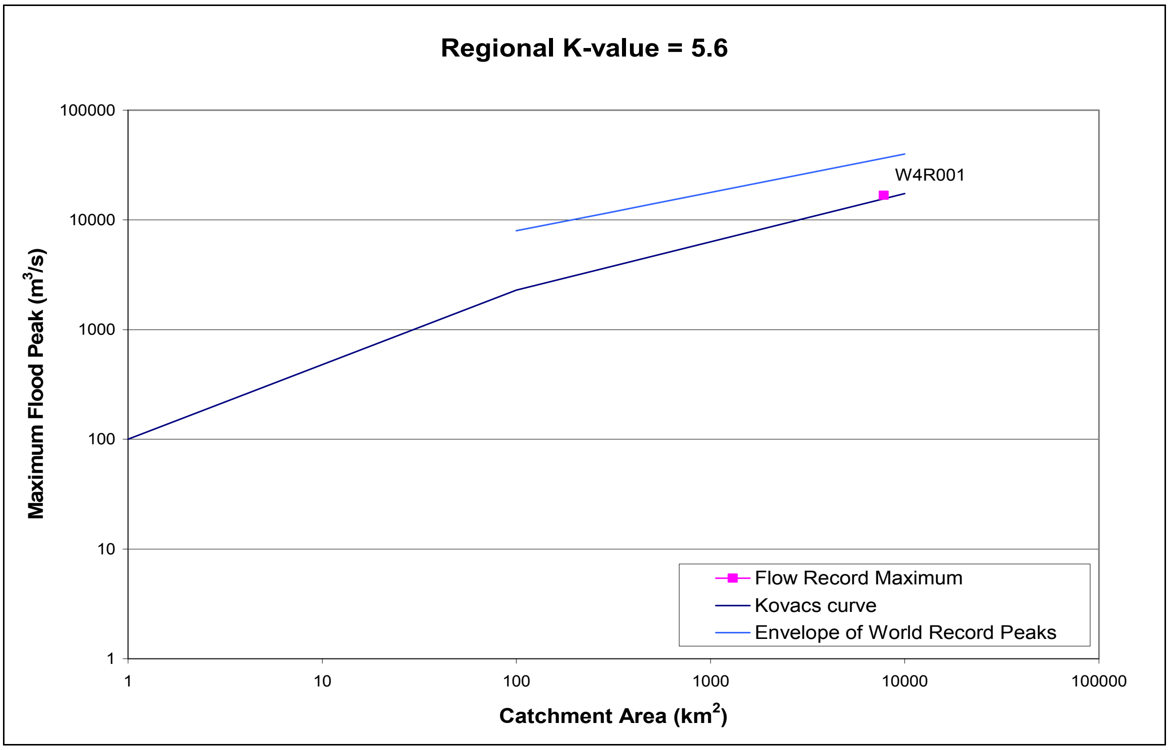

Figure 5.30 Maximum observed flood peaks for K-value 5.6

The above diagrams show recently recorded flood peaks plotting either near Kovaćs's envelope curves or exceeding the curves, i.e. K-regions 3.4, 5, 5,4 and 5.6. This result indicates that the boundaries of the original K-regions, as defined in Figure 7 of Kovaćs's 1988 TR 137 report, might need to be adjusted to accommodate these higher recorded flood peaks.

5.5 CONCLUSIONS

The aim of this research was to investigate the link between recurrence interval (RI), or annual exceedence probability (AEP), and the RMF and PMF for South Africa. This information would aid in the future application of the *SANCOLD Guidelines*, which incorporates the use of these two extreme design flood concepts. This research also aimed to bring about a better understanding of the differences between the RMF and PMF, which could have an impact on their applicability when designing a structure such as a dam, or in performing a safety evaluation.

As an overall finding from the foregoing analyses, we would like to propose two RI indices to help orientate hydrological practitioners in South Africa regarding the possible RI of the RMF and PMF. These are presented in Table 5.32. We have based these on the GEVpwm results, as these appear to generally provide more conservative RIs.

Table 5.32 Recommended median and lower 95 percentile RI for the RMF and PMF

Design Flood	Median	Lower 95 Percentile	Probability Distribution
RMF	3000	200	GEVpwm
PMF	30000	600	GEVpwm

An interesting observation from these values of RI is that for the RMF, the lower 95 percentile value is 1:200 years. This differs quite substantially from the outcome of the analysis by Pegram et al. (2004) where they deduced that the RI of the RMF could be about the 1:200 year flood. Given the discrepancy between the two findings, an alternative approach for the estimation of the RMF RI, which used Kovaćs's own methodology from his 1988 TR 137 report, was also employed. This approach showed the lower 95 median percentile and RMF RI to be around 1:700 and 1:400 years, respectively. With access to longer flow records than those used by Kovaćs in his research prior to 1988, Kovaćs's envelope curves for the various regional K-values were checked. The recently recorded flood peaks plotted either near the curves or exceeding the curves, i.e. K-regions 3.4, 5, 5,4 and 5.6. This indicates that the boundaries of the original K-regions might need to be adjusted to accommodate the more recent higher flood peaks.

In terms of the PMF, the results of the study showed the median RI for the PMF to be greater than the median RI for the RMF by about a factor of 10. Also, for the comparison of the magnitudes of the two extreme floods on a national scale, a spatial distribution of PMF/RMF was produced. This indicated a trend that the PMF/RMF ratios increase from the coast towards the interior, with the smaller PMF/RMF ratios of < 1 evident along the coastal area of the Western Cape and Eastern Cape, and the largest PMF/RMF ratios of > 4 evident for the northern parts of the country. For more than half of the country, the magnitude of the PMF was found to be larger than twice the magnitude of the RMF. These results are not surprising as, by definition, the PMF is the most extreme flood that could be expected within a catchment. This is also in line with the roles these two extreme floods play in the SANCOLD Guidelines, where the PMF is clearly expected to be a larger value than the RMF. The wide spatial variation in the PMF/RMF values might indicate possible inconsistencies in the methodology used to derive the PMF, as opposed to the RMF methodology, which is considered as more structured.

An area of concern surrounding this research might be that very high RIs were extrapolated from fairly short flow records using the less confidence "tail-end" of the various probability distributions. For this reason, the results presented in this project are based on the application of three different probability distributions, and the estimates of the RI for the extreme design floods are presented in terms of "RI bands" and percentiles instead of a single value.

It is not intended that the RIs shown in Table 5.32 be used blindly by hydrological practitioners when designing a dam; rather the intention is to raise designers' awareness so that they may be guided to make prudent decisions. The reader should also note that care was taken in this research to apply conventional methods most commonly used in South Africa, such as the use of the Unit Hydrograph method for the derivation of the PMF and the use of well established probability distribution functions. By following conventional South African practice, subjectivity is diminished which promotes consistency in findings.

5.6 RECOMMENDATIONS

Based on the lessons learnt in this study, the following recommendations for further research can be made:

- An investigation should be undertaken to determine whether the pragmatic method used by the UK and Australia to determine design flood magnitudes between the maximum extrapolated RI value (e.g. 1:10^2 years) and an assigned RI value to the PMF (e.g. 1:10^4 years) can be applied to the South African situation.
- The extreme rainfall value curves/functions used to calculate the PMF may need to be updated.
- An investigation should be undertaken into the use of a general probability distribution for South Africa, to promote consistency, as well as the influence of truncation on probabilistic flood peak estimations.
- Reasons for the apparent inconsistencies between the dam site and flow gauge flood peak statistics should be investigated.
- Further research should be undertaken into the possible adjustment of Kovaćs's K-regions, which are proposed in Figure 7 of his 1988 TR 137 report.

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