INSTITUTE OF HYDROLOGY

DRAFT
ARCHIVE

ARAB POTASH PROJECT

SUPPLEMENTARY REPORT

ON THE

DESIGN FLOOD FOR THE TRUCE LINE CHANNEL

This report is prepared for Sir Alexander
Gibb & Partners, Reading, by:

Institute of Hydrology
Wallingford
Oxon
UK

July 1978

1. INTRODUCTION

This supplementary report describes further studies of the design flood for the Truce Line channel. In particular, it is concerned with the difficult problem of estimating the likelihood of severe floods occurring in several wadis simultaneously. No new data on flood flows are available but we have analysed the pattern of heavy daily rainfalls in an attempt to define the storm size which would be associated with floods of long return period. This new approach provides a basis for the combination of flood estimates for the individual wadis in the catchment area of the channel.

The catchment area

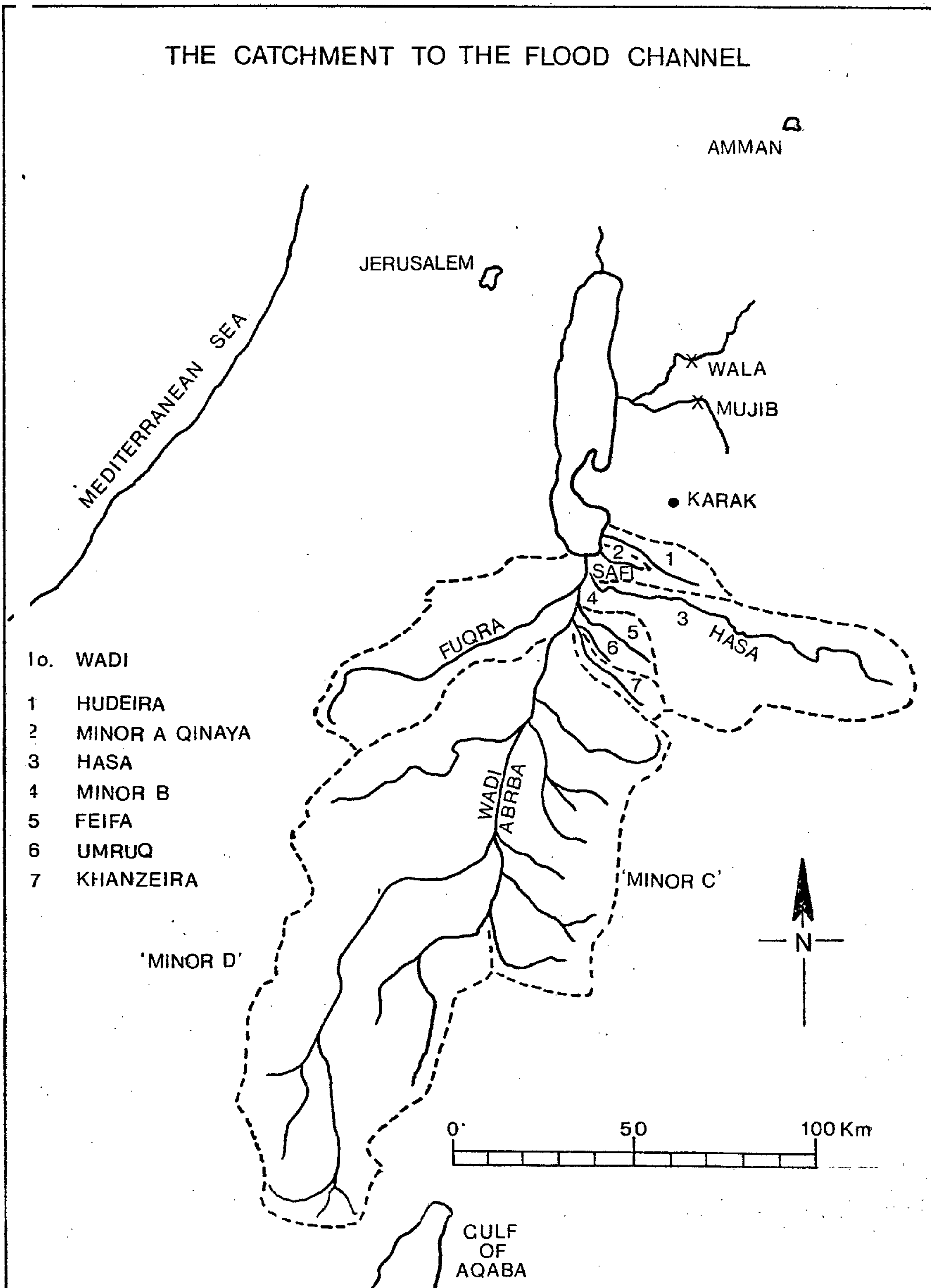
The catchment area of wadis draining through the flood channel is shown in Figure 1. The area encompasses the hills to the east of the rift valley, with moderate annual rainfall, as well as parts of the more arid Negev desert.

Rainfall occurs mainly between the months of November and April, mostly as a result of depressions moving in an easterly direction across the area. Mean annual rainfall is highest over the eastern hills, in the Karak area, with about 350 mm per annum. The rainfall is much lower in the centre of the rift valley, dropping to about 70 mm at Safi. The sparse network on the Israeli side of the rift valley indicates annual rainfall of the order of 100 mm. There is a strong tendency for rainfall to decrease to the south.

Previous studies

Flood runoff from wadis draining the hills to the east of the Dead Sea has been studied by a number of teams. The reports found to

Figure 1


THE CATCHMENT TO THE FLOOD CHANNEL
AMMAN
JERUSALEM
MEDITERRANEAN SEA
WALA
MUJIB
KARAK
1
2
SAFI
3
4
5
6
7
FUQRA
HASA
WADI ABRBA
'MINOR C'
'MINOR D'
N
0
50
100 Km
GULF OF AQABA
Io. WADI
1 HUDEIRA
2 MINOR A QINAYA
3 HASA
4 MINOR B
5 FEIFA
6 UMRUQ
7 KHANZEIRA

be of particular value were those of Sir Murdoch MacDonald and Partners,[1,2] 1965 and 1973, and Binnie and Partners et al.[3], 1977.

The 1965 MacDonald study covered the entire East Bank of the River Jordan, and was primarily a review of the water resources of the area. Storm rainfall/runoff relationships were attempted for all gauged wadis, including Wadi Hasa. The 1973 MacDonald report concentrated on the southern wadis and floods were estimated for various return periods on the Wadis Hudeira, Hasa, Feifa and Khanzeira, amongst others. The recent study by Binnie et al. continued the work started by MacDonalds. Although no new flood estimates are made in their preliminary review reports, discussions with members of their study team concerning their most recent estimates were of great assistance.

All previous investigations were concerned with estimating peak flow rates. The problems of estimating a volume of runoff associated with the peak flow rate and the possibilities of simultaneous floods in many wadis have not been examined.

Data available

Rainfall and runoff data are collected and held by the Jordanian Natural Resources Authority. Our only source of data for the catchment area west of the truce line was the British Meteorological Office.

Daily rainfall data was abstracted for the following stations: Hemud, Rabba, Ghor Mazra'a, Ghor Es Safi met. station, Ghor Es Safi police post, Qatrana Evap. station, Qatrana, Ain Bisas, Karak, Mazar, Khanzira, Tafila, Hasa police post, Jurf Ed Darawish, Buseira, Dana and Shaubak school in Jordan; Beer Sheva, Ein Hasb, Sodom and Arad in Israel. Some of these stations had very intermittent records.

1 *Sir Murdoch MacDonald & Partners, East Bank Jordan Water Resources, 1965*

2 *Sir Murdoch MacDonald & Partners, Mujib and Southern Ghors Irrigation Project, Up-dated Report, 1973*

3 *Binnie & Partners, Jouzy & Partners, Ove Arup & Partners, Booker Agriculture International. Mujib and Southern Ghors Irrigation Project, Preliminary Review Report, 1977*

Data from recording raingauges were abstracted for four stations: Mazar, Buseira, Tafila and Ain Bisas. However, the length of record that could be considered to be reliable was very short at each station.

The NRA have established a number of flow gauging stations on the eastern wadis and daily flood runoff data were abstracted for Wadi Mujib at the Karak Road, Wadi Wala at Karak road and at the new weir, Wadi Karak at Mazra'a and Wadi Hasa at Safi and at Tannour.

2. ANALYSIS OF THE FLOOD RUNOFF DATA

The first step taken in the study was to examine the available runoff data for any distinct pattern of runoff behaviour from which conclusions about the 1000 year design flood could be drawn.

Major historic floods

The principal recent flood events in the south of Jordan occurred at Ma'an in March 1966 and at Petra in April 1963. In both cases, severe flooding was caused by severe thunderstorms and torrential rain of an extremely localised nature. In neither case was the wadi in flood subject to regular gauging and no estimates of the peak flow rates and flood volumes are available.

The most extreme flood to have occurred recently on either of the three gauged wadis near the study area (Wadis Hasa, Mujib, Wala) was on Wadi Hasa during January 1965. Unfortunately, the flood was large enough to wash away the water level recorder, but later a peak flow of 706 m^3/s was estimated by slope-area techniques. It was impossible to estimate the associated flood volume. January 1965 was generally an extremely wet month, and the highest daily rainfall recorded at Safi, 43 mm, occurred on the 12th of that month.

The largest mean daily flow recorded at the Safi gauging station was on the 16 November 1967, when 23.5 million m^3 were estimated to have passed the gauge. However, examination of the rainfall records for gauges in the Hasa catchment cast some doubt on this figure.

The other notable flood event occurred on both the Mujib and Wala catchments in April 1971, when 89 and 21 million m^3 respectively were estimated to have passed the gauging stations. These volumes are equivalent to runoff of 13.5 and 12.4 mm over the catchments.

Seasonal patterns of runoff

The distribution of flood runoff in the three gauged wadis by months is summarised in Table 1. No flood runoff has been recorded between the months of June and September. Examination of these figures reveals no strongly seasonal characteristic; the design flood would seem equally likely to occur in any month from November to April.

Correlation of daily flood flows between stations

The dates of the 15 most extreme events recorded on each of the wadis were abstracted from the records, and, where possible, compared with flood flows on the same day on the other wadis. A correlation coefficient significantly different from zero was obtained only for the comparison between Wadis Mujib and Wala, where the correlation coefficient was 0.744. However, if the flood of 13 April 1971, the largest, was omitted from the analysis, this correlation coefficient was reduced to 0.014 which is not significantly different from zero. Thus we could find little evidence from the flow records alone to suggest that there is a tendency for major floods to occur on the same day in neighbouring wadis.

3. ANALYSIS OF STORM RAINFALL

The flood records available are insufficient to define the likelihood of floods occurring on several wadis simultaneously especially in the catchment of the Wadi Araba where very few records exist. Thus we have attempted to define the size of the flood-producing storm by an analysis of the rainfall records.

TABLE 1

SEASONAL PATTERN OF RUNOFF

	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY
Average rain[1] (mm)	5	15	25	30	30	25	8	5
Max. daily runoff (m^3/s)								
Wadi Hasa	42.3	272	27.9	46.5[2]	93.6	78.8	48.5	84.5
Mujib	43.4	83.3	135	28.3	137	98.9	1030	7.7
Wala	8.4	16.4	181	67.1	31.2	67.7	245	2.9
Average monthly runoff (million m^3)								
Wadi Hasa	0.89	4.01	0.36	1.00[2]	2.48	1.12	0.19	0.90
Mujib	0.77	2.11	0.96	2.19	3.10	4.01	10.5	0.20
Wala	0.29	0.54	5.00	2.45	0.35	2.38	3.97	0.04
Average monthly runoff (mm)								
Wadi Hasa	0.35	1.59	0.14	0.40[2]	0.98	0.44	0.08	0.36
Mujib	0.18	0.48	0.22	0.50	0.71	0.92	2.40	0.05
Wala	0.17	0.32	2.94	1.44	0.21	1.40	2.33	0.02

Notes: 1 estimated
2 excludes January 1965 flood.

The 1000 year storm

One of the areas of greatest uncertainty in this study is the nature of the design storm - that which produces the most extreme flood runoff into the southern end of the Dead Sea with an average recurrence interval of 1000 years.

Rainfall in the area occurs as the result of two distinct processes: firstly, during the two transition periods between the dry and wet season (approximately the months of October to November and March to May), rainfall occurs mainly as a result of localised instability in the air masses, causing short periods of intense, thundery rain. In the main part of the wet season (December to February), the rainfall is most likely to be frontal in origin. According to the Jordanian Meteorological Department the two most disastrous recent floods in Jordan - those occurring at Ma'an in 1966 and Petra in 1963 - were both a result of extremely localised thundery rainfall of short duration and great intensity.

Examination of the runoff records available for the Wadis Hasa, Wala and Mujib show that flood runoff can occur at any time during the wet season with the more extreme events occurring with almost equal likelihood in any of the months November to April. Thus, we cannot assume that the extreme flood event will be caused only by thundery rain during the transition periods.

Comparison of the rainfall and runoff records leads to the conclusion that runoff must be strongly dependent upon rainfall intensity, as daily rainfall and runoff are surprisingly poorly correlated.

The correlation of daily rainfalls between stations

Due to the nature of the data available, the most promising approach to the problem of estimating the storm size was through statistical analysis of the daily rainfall records, concentrating on days when at least 40 mm fell at one or more stations. This threshold has been chosen in order to consider only the higher rainfalls, those which we must assume are representative of severe flood-producing storms, while allowing a sufficiently large sample of storms for statistical analysis.

The correlation analysis used daily rainfall data from stations at Ain Bisas, Karak, Ghor Mazra, Mazar, Khanzeira, Ghor Safi, Tafila and Buseira. The days on which more than 40 mm of rain fell at any station were noted, and for each of these days, the rainfall at every station was listed.

A correlation coefficient was calculated between the data sets from every pair of stations, although in order to maintain consistency, only data from days when at least 40 mm fell at either of the pair of stations were used. The results, shown in Table 2, were used to find a relationship between the correlation coefficient, r, and the separation of the stations. The results, shown in Figure 2, show a large degree of scatter, some of which can be explained by anisotropic features of the rainfall distribution.

For any given station separation, there is generally a higher correlation between daily rainfalls at stations whose relative orientation is north-south than at those oriented east-west. Two lines have been sketched on Figure 2, illustrating the relationship between correlation coefficient, r, and station separation for these cases. These results suggest that the isohyets of storm rainfalls are approximately elliptical with the longer axis in the north-south direction, a conclusion which is not unreasonable given the topography and meteorological conditions.

Storm extent

Many authors have examined the distribution of rainfall through correlation analysis, but, for the most part, the objective has been to derive an optimum raingauge network design. The step from correlation analysis to an estimated storm size is rarely made.

TABLE 2

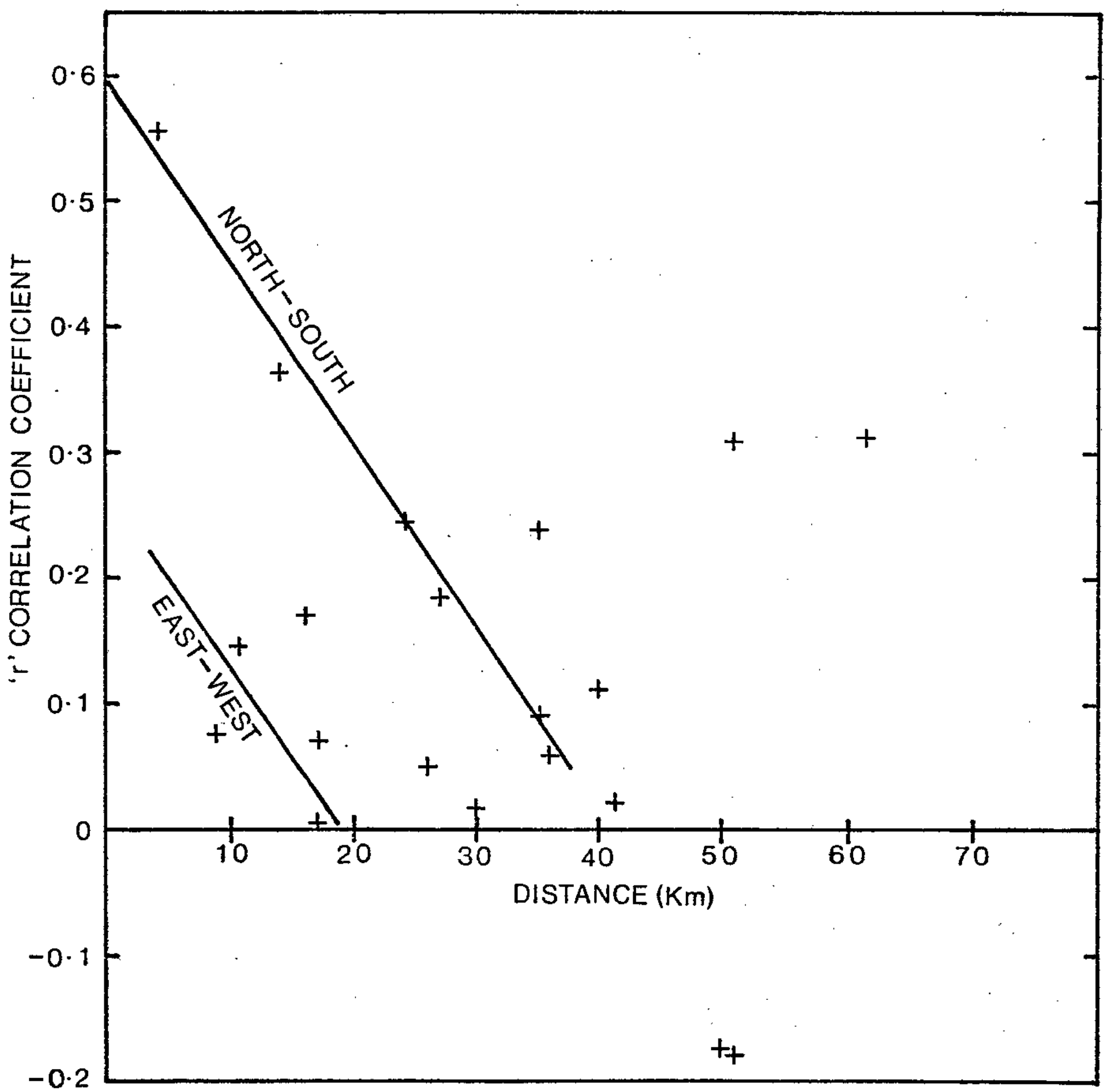
DAILY RAINFALL CORRELATION COEFFICIENTS

	1	2	3	4	5	6	7	8
1. Ain Bisas								
2. Karak	0.559							
3. Ghor Mazra	-0.369	-0.252						
4. Mazar	0.172	0.376	0.017					
5. Khanzeira	0.068	0.003	0.183	0.075				
6. Ghor Safi	-0.579	-0.432	x[1]	-0.213	-0.212			
7. Tafila	0.023	0.111	0.305	0.053	0.247	-0.231		
8. Buseira	-0.174	-0.172	0.304	0.057	0.092	0.236	0.146	

Notes: [1] x denotes inadequate data

Figure 2

RELATIONSHIP BETWEEN RAINFALL CORRELATION AND DISTANCE



A measure of the 'average' storm size can be obtained by finding the point at which the correlation becomes not significantly different from zero. In statistical tests such as this, it is usual to adopt a level of significance of 5 percent on which to base judgements. In this case, however, applying this criterion gave an estimated storm size of 15 km, which seems incautiously low. A 10 percent level of significance corresponded to a distance of 25 km in the north-south direction. To express this in a different way, there is only a one in ten chance that for a separation of gauges in a north-south direction of less than 25 km, the rainfall data will be uncorrelated.

In subsequent analysis, we have chosen a storm size of 30 km in order to allow for uncertainty in the correlation analysis. Further, we have carried out a sensitivity analysis using storm sizes of 20 and 50 km.

4. FLOOD PEAK ESTIMATION

The two main alternative approaches to the problems of estimating the design flood are to treat the catchment as a single entity subject to a design storm or to estimate the flood frequency relationships for individual wadis and combine them in some way to produce a composite flood frequency relationship.

Past work on floods in the area has concentrated on the wadis flowing from the higher ground to the east of the Dead Sea, the catchments to the south and south-west being of little interest. As a result, any extension of this work to incorporate the entire composite catchment with its large variation in rainfall and topography, would be inappropriate. The large variations throughout the catchment would also make the application of empirical methods derived for other areas of the world inappropriate if applied to the area as a single catchment.

As a result, we have adopted the alternative approach; the flood frequency relationships for the individual wadis have been based on the results of previous studies while the analysis of the extent of major storms provides the basis for combining flood flows within the whole catchment area.

Flood peaks on individual wadis

Study of the MacDonald reports and discussions with Binnie & Partners have led to the conclusion that the most appropriate method for estimating flood frequency relationships in this area is based upon the empirical method devised by Creager[1]:

$$Q = 46\ C\ A^{n}$$

where Q is the flood in cusecs, A is the drainage area in square miles,

$$n = 0.894\ A^{(-0.048)}$$

and C is a coefficient depending on basin characteristics.

This method can be extended by defining the coefficient C as dependent upon both the catchment characteristics and the selected return period of the flood. We have derived values for each wadi or group of minor wadis based on the previous studies and experience elsewhere in order to estimate the flood peaks of 100 and 1000 year return period. These are shown in Table 3.

Definition of storm zones

The rainfall analysis suggested a storm size of 30 km. Thus, over the whole catchment area, we can define a specific number of zones which can be considered to be independent in the sense that there would be no correlation between storm rainfalls on adjacent zones. In practical terms the problem of defining the zones is that of grouping the wadis listed in Table 3 in such a way that the storm size criterion is generally met. Table 4 shows how the zones have been defined for the basic 30 km storm size and for the alternative storm sizes used

[1] *Creager, Justin and Hinds Engineering for Dams, Vol. I. John Wiley & Sons, New York 1944*

FLOOD ESTIMATES FOR INDIVIDUAL WADIS

Catchment	Area (miles^2)	Creager "n"	Return period 100 year "C"	Return period 100 year Q(m^3/s)	Return period 1000 year "C"	Return period 1000 year Q(m^3/s)
Hudeira	39	0.750	15	305	20	405
Minor A	13	0.790	15	150	20	200
Hasa	975	0.642	12	1300	17	1850
Minor B	18	0.778	15	185	20	250
Feifa	60	0.734	10	260	15	400
Umruq	11	0.797	15	130	20	175
Khanzeira	70	0.729	10	290	15	430
Minor C	1040	0.641	10	1120	15	1680
Minor D	2330	0.616	8	1240	13	2010
Fuqra	630	0.656	10	890	15	1340

Notes: The minor wadis in groups labelled Minor A, Minor B, etc. above, have been treated as though they responded as single wadis. Their catchment areas are defined in Figure 1.

TABLE 4

DEFINITION OF STORM ZONES

Storm Size	Zone	Catchments
	East Bank North	Hudeira, Minor A (Quinaya), Hasa, Minor B Feifa Umruq, Khanzeira
50 km	East Bank South	Minor C
	West Bank North	Fuqra
	West Bank South	Minor D
	East Bank North	Hudeira, Minor A, Hasa
	East Bank Central	Minor B, Feifa, Umruq, Khanzeira
30 km	East Bank South	Minor C
	West Bank North	Fuqra
	West Bank South	Minor D
	East Bank North	Hudeira, Minor A
	Hasa	Hasa
	East Bank Central	Minor B, Feifa, Umruq, Khanzeira
20 km	East Bank South	Minor C
	West Bank North	Fuqra
	West Bank South	Minor D

in the sensitivity analysis.

Wadi Hasa is the single catchment posing the most serious flood risk. Thus more attention was paid to the definition of zones in this and adjacent catchments than in the lower rainfall areas of the Wadi Araba. For the 30 km storm size for example, we assumed that a single storm would cover the Hasa, Quinaya and Hudeira catchments; a second, independent storm would cover the lower Araba tributaries up to the Khanzeira and the rest of the area being of lower flood-producing potential, was split into 3 zones, each representing a fairly homogenous well-defined subcatchment of the Wadi Araba.

Zone floods

In order to estimate the zone flood peak produced by a single storm in the cases where the zone covers a number of wadis, we have assumed that the individual floods occur simultaneously and the combined flood peak is simply the sum of the individual floods. This is clearly a conservative assumption.

Other methods of estimating the combined peak flow were investigated, but they rely on assumptions made about the storm profile, the speed with which each individual wadi responds to the rainfall and other similar, poorly defined variables. Estimates based on the Wadi Hasa and the neighbouring minor wadis suggested that the overestimate of the zone flood following from the simple approach of adding flood peaks for individual wadis would not be over cautious.

Combination of zone floods

Analysis of the peak floods in each zone gives a number of flood frequency relationships which must be combined to produce a compound flood frequency relationship for the entire area. In order to produce an analytically-based solution to this problem, simplifying assumptions have to be made. The principal assumption is that the annual maximum combined flood is equal in magnitude to the largest individual zone annual maximum flood. This is tantamount to saying that the annual maxima from the zones never coincide and the annual maxima of the combined flood are never the result of combining lesser zone floods.

Two further assumptions need to be made to make the problem capable of solution. Firstly, that each zone has a flood frequency distribution of the same type (in this case it is assumed to be a Gumbel distribution), and secondly, that the flood-frequency relationships for each zone are parallel.

The cumulative probability distribution function of the Gumbel distribution can be expressed in the form:

$$F(q) = \exp\ \{ -\exp\{\frac{-q-u}{\alpha}\}\}$$

where F(q) is the probability of a flood greater than q and u, α are the parameters of the distribution: α being known as the scale parameter and u the location parameter. The second of the additional assumptions made in mathematical terms means that the scale parameters for each distribution are assumed to be equal.

It can be shown that the combined flood frequency relationship will have the same scale paramter, α, and a location parameter U where

$$U = \alpha \ln\{e^{u_1/\alpha} + e^{u_2/\alpha} + \ldots\ldots + e^{u_n/\alpha}\}$$

where u_i is the location parameter of the i^{th} zone.

The Gumbel scale and location parameters can be calculated from estimates of the 100 year (q_1) and 1000 year (q_2) floods for each zone. It can readily be shown that:

$$\alpha = \frac{q_2 - q_1}{2.31} \quad \text{and } u = 2.99q_1 - 1.99q_2.$$

The calculation of a flood for a given return period (T) and distribution paramters α and U is made easier by employing the close approximation to the true relationship:

$$Q(T) = U + \alpha\ln(T-0.5).$$

The estimation of distribution parameters for the various zones and storm sizes is summarised in Table 5.

When estimating the parameters of the combined distribution for each storm size, the shape parameter was taken to be equal to that of the dominant zone, East Bank North or Hasa in the case of the 20 km storm, and the location parameter calculated accordingly. The results are summarised in Table 6.

5. THE FLOOD HYDROGRAPH

The presence of the attenuation area which is capable of providing temporary storage for flood runoff and thus influencing the peak discharge into the flood channel means that attention must be paid to the shape of the flood hydrograph, and the volume of water contained within it.

Time of concentration

The time lag between the rainfall and the arrival of the flood peak is strongly dependent upon the time-of-concentration of a catchment which nominally corresponds to the time taken for a drop of water to travel from the extreme limit of the catchment to the gauging station or point of interest on the wadi.

Ibbitt[1] suggests that an appropriate relationship to use to calculate values for this parameter is that derived by the California Highways division, namely:

$$T = \left(\frac{0.871\ L^3}{H}\right)^{0.355}$$

where T is the time of concentration in hours, L is the length of the stream to the gauging station in kilometres and H is the difference in elevation between the highest point in the catchment and the gauging station in metres.

Steady rainfall on a hydrologically well-behaved catchment will produce a peak flow at a time equal to the time-of-concentration from

[1] *Ibbitt, M.E. Rainfall intensities in Jordan for use in engineering design. NRA Professional Paper No. 2, 1969*

TABLE 5

FLOOD FREQUENCY DISTRIBUTION PARAMETERS

20 km Storm

ZONE	1000 Yr Flood (m^3/s)	100 Yr Flood (m^3/s)	Distribution α	Parameters μ
East Bank North	605	455	65	157
Hasa	1850	1300	238	206
Central	1255	865	169	88.9
South	1680	1120	242	5.6
West Bank North	1340	890	195	- 5.5
South	2010	1240	333	- 292

30 km Storm

ZONE	1000 Yr Flood (m^3/s)	100 Yr Flood (m^3/s)	Distribution α	Parameters μ
East Bank North	2455	1755	303	362
Central	1255	865	169	88.9
South	1680	1120	242	5.6
West Bank North	1340	890	195	- 5.5
South	2010	1240	333	- 292

50 km Storm

ZONE	1000 Yr Flood (m^3/s)	100 Yr Flood (m^3/s)	Distribution α	Parameters μ
East Bank North	3710	2620	472	451
South	1680	1120	242	5.6
West Bank North	1340	890	195	- 5.5
South	2010	1240	333	- 292

TABLE 6

COMBINED FLOOD-FREQUENCY DISTRIBUTION

STORM	DISTRIBUTION α	PARAMETERS μ	1000 YR FLOOD (m^3/s)
20 km	238	496	2140
30 km	303	591	2684
50 km	472	773	4033

the start of the rainfall. Thus the time of concentration is useful in estimating the timing of the flood peak and the duration of the storm likely to produce the estimated peak flow.

The design flood into the attenuation area is dominated by the Hasa flood hydrograph. Using the method outlined previously the time-of-concentration for this catchment is of the order of 15 hours. However, this estimate is probably too high since the rainfall correlation analysis indicated the very limited extent of storms in the east-west direction. We therefore repeated the calculation for the lower half of the catchment (below Tannour gauging station) to estimate the time-of-concentration for a storm centred in the area of highest mean annual rainfall. This gave a value of 5.6 hours which we believe to be more realistic.

Storm rainfall

Ibbitt derived a regression relationship between extreme daily rainfall values X_T, at a point in a catchment and mean annual rainfall, z, in millimetres. For 1000 year return period:

$$X_{1000} = 0.197z + 66.8.$$

He also derived a relationship to determine the ratio, R, daily rainfall : 1 hour rainfall, with a similar format:

$$R = 0.0011z + 1.85$$

Thus, we can use the techniques derived by Herschfield[1] to estimate storm rainfall for any other duration.

Mean annual rainfall figures for the wadi catchments were estimated using Figure 3 of the Gibb "Truce Line Flood Channel Design" report of February 1978.

[1] *Herschfield and Wilson. Generalising of rainfall intensity frequency data. Proc. IASH, Toronto, 1957*

Areal reduction factor

It is generally recognised that rainfall at a point is more variable than the corresponding catchment rainfall which is an average rainfall over the area. Thus the extreme point rainfalls which are derived by the method described above must be reduced by an "areal reduction factor" to arrive at an estimate of the average catchment rainfall.

Ibbitt suggests that the work of the U.S. Weather Bureau would prove most appropriate for application to Jordanian catchments. This led to a reduction factor for the Hasa catchment of 0.87. Again because of the limited extent of storms in the east/west direction shown by the storm analysis, we believe this value to be unrealistically high. We therefore developed an alternative approach to the estimation of an areal reduction factor based directly on considerations of storm size.

We assumed that the Hasa catchment could be divided into 5 strips, running in a North-South direction, with a width of about 20 km, such that the rainfall on each strip can be considered to be independent of rainfall on other strips. If the strips are assumed to be homogenous, for each strip, i, we can define average values of the following parameters:

number of raindays per year = N_i

average rainfall per wet day = X_i

length of strip = L_i

Generally, for rain X_j, on strip j the areal reduction factor can be expressed as

$$\frac{\sum_{i=1}^{5} X_i L_i}{X_j \sum_{i=1}^{5} L_i}$$

as all strips are of equal width.

For the average areal reduction factor the average value of the term $\{\sum_{i=1}^{5} X_i \cdot L_i\}$ is required.

If $N_j \geq N_i$ for all i, then the average value of X_i, $\bar{X}_i$, can be expressed by:

$$\bar{X}_i = \frac{N_i}{N_j} \cdot R_i = \frac{A_i}{N_j}$$

Thus where A_i is the annual rainfall on strip i . the areal reduction factor becomes:

$$\frac{\sum_{i=1}^{j-1} \frac{A_i}{N_j} \cdot L_i + \sum_{i=j+1}^{5} \frac{A_i}{N_j} \cdot L_i + X_j \, L_j}{X_j \sum_{i=1}^{5} L_i}$$

This approach contains the implicit assumption that the strip j, which is defined as having the highest number of rain days, is the one containing the highest point rainfall in the design storm. For the Hasa catchment strip number 2 covers the highest rainfall area which also has the highest number of rain days. Thus the assumption would seem to be valid.

Table 7 shows the parameters used in evaluating the areal reduction factor. Given a value for X_j of 73 mm from the Ibbitt and Herschfield approach outlined above and for a return period of 1000 years and storm duration of 5.6 hours, the areal reduction factor is found to be 0.23.

Flood volume

The product of the design point rainfall, the catchment area and the areal reduction factor gives the rainfall volume in the design storm. This multiplied by the percentage runoff gives the volume of runoff in the design flood. Choice of an appropriate percentage runoff was based on the results of previous studies and discussions with Binnie & Partners. For the 1000 year storm we concluded that a figure of 50 per cent would be realistic.

The calculation of flood volumes corresponding to the various storm sizes are shown in Table 8. We have assumed in each case that the maximum flood volume will be derived from a storm centred over the Hasa catchment. Areal reduction factors for all catchments except the Hasa were derived by Ibbitt's method.

Finally the flood volume corresponding to the design storm on the whole catchment was derived from that for the zone containing the Wadi Hasa by ratio of the composite flood peak to the zone flood peak. The results suggest flood volumes of 24.8, 28.2 and 42.6 million m^3 respectively for the 20, 30 and 50 km storm sizes.

The recorded flood hydrographs on Wadi Hasa suggest that the design flood hydrograph is likely to be approximately triangular in shape.

TABLE 7

PARAMETER VALUES USED IN ESTIMATING THE AREAL REDUCTION FACTOR

STRIP NO.	AV. ANNUAL RAINFALL (mm)	LENGTH (km)	NO. OF RAIN-DAYS
1	118	7.5	14
2	225	25	30
3	130	35	10
4	70	30	6
5	55	25	6
		$\sum$ 122.5	

Note: Each strip is 20 km wide in the east-west direction

CALCULATION OF FLOOD VOLUMES FOR THE ZONE CONTAINING THE HASA CATCHMENT

20 km Storm

CATCHMENT	AREA	1000 YEAR RETURN PERIOD DAILY RAINFALL	1000 YEAR RETURN PERIOD 5.6 HR RAINFALL	AREAL REDUCTION FACTOR	AREAL RAINFALL	FLOOD VOLUME
	(km^2)	(mm)	(mm)		(mm)	(million m^3)
HASA	2520	113	73	0.23	17	21.4
					TOTAL	21.4

30 km Storm

CATCHMENT	AREA	DAILY RAINFALL	5.6 HR RAINFALL	AREAL REDUCTION FACTOR	AREAL RAINFALL	FLOOD VOLUME
HUDEIRA	101	111	72	0.94	68	3.4
MINOR A	34	91	59	0.96	57	1.0
HASA	2520	113	73	0.23	17	21.4
					TOTAL	25.8

50 km Storm

CATCHMENT	AREA	DAILY RAINFALL	5.6 HR RAINFALL	AREAL REDUCTION FACTOR	AREAL RAINFALL	FLOOD VOLUME
HUDEIRA	101	111	72	0.94	68	3.4
MINOR A	34	91	59	0.96	57	1.0
HASA	2520	113	73	0.23	17	21.4
MINOR B	46	87	57	0.96	55	1.3
FEIFA	155	111	72	0.93	67	5.2
UMRUQ	28	85	55	0.97	53	0.7
KHANZEIRA	181	112	73	0.93	68	6.2
					TOTAL	339.2

6. REVIEW AND CONCLUSIONS

The estimation of design floods on individual catchments is not easy when there is not a long sequence of flow data available with which to define the frequency distribution of the annual maximum floods. Some benefit can be gained by pooling data from a number of catchments to produce a regional flood frequency curve but its application to ungauged catchments is difficult since it is necessary to estimate a mean annual maximum flood. In this study the problem was complicated further by having to estimate a combined flood frequency distribution for a catchment incorporating a number of hydrologically dissimilar, ungauged sub-catchments. This necessitated the use of rainfall data to estimate an average storm size to make a solution to the problem possible.

The method of solution adopted incorporated a number of assumptions concerning the pattern of rainfall and runoff. The Creager method of estimating flood size was assumed to be appropriate when estimating individual wadi peak flows. Somewhat conservatively it was assumed that the peak flow rates on individual wadis within the zone of influence of a storm would occur simultaneously. This conservatism was counter-balanced by the assumption that combined catchment flood peak flows would occur from only one storm within the catchment. The area of greatest uncertainty is in the estimation of storm size, which is a result of the assumption that the 1000 year storm is drawn from the population of all storms in the area with a daily rainfall greater than 40 mm.

In the course of the analysis it became evident that the flood frequency distribution of the combined catchment is dominated by the contribution made by Wadi Hasa and the neighbouring catchments. The influence of storm size on the flood series is effectively to dictate the extent of this principal zone. For the 20 km storm this was reduced to a minimum of the Hasa catchment alone. The 50 km storm represents a reasonable maximum size to the zone as the Khanzeria, included in the zone, is the most southerly wadi to drain the higher rainfall area of the eastern hills. The rest of the catchment contributed up to 15 per cent of the estimated 1000 year combined catchment flood.

The correlation analysis has shown that the storm size is likely to be 30 kilometres. However, the sensitivity analysis has shown that the flood peak is broadly linearly related to storm size, and thus the uncertainty involved in the estimation of storm size should not be ignored when estimating the risk of failure of the truce line channel. This factor, along with others such as the return period of the flood, the life of the project, the cost of a breach in the dykes and the cost of raising the dykes can be incorporated into an economic analysis of the optimum design for the truce line channel dykes.
The frequency distribution of the annual maximum floods from the entire catchment can readily be deduced by extension of the present method of analysis adopted.