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Jeddah Stormwater - II

FLOOD ESTIMATES FOR JEDDAH MOUNTAIN REGION

JEDDAH STORMWATER - PHASE II

FLOOD ESTIMATES FOR

JEDDAH MOUNTAIN REGION

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FLOOD ESTIMATES FOR JEDDAH MOUNTAIN REGION

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1. INTRODUCTION

1.1 Approach to analysis

This report provides estimates of floods from 5 to 100 years return period for the mountain wadis supplying the Jeddah southern stormwater channel.

The location of the catchments is shown on Figures 1.1 and 1.2. Figure 1.1 shows the group of smaller wadis contributing to the upstream end of the stormwater channel. Figure 1.2 shows the much larger Wadi Fatima which joins the stormwater channel closer to the Red Sea.

The hydrological study commenced with a one week visit to Saudi Arabia. This comprised about five days in Jeddah involving site visits, meetings and data collection. This was followed by a two day visit to the Ministry of Agriculture and Water in Riyadh to collect additional data.

The most accurate estimates of floods of various return periods are obtained from sites which have a number of years of wadi flow data collected at a well rated gauging station. If no flow data are available at the site, flood estimates may be obtained by comparison with similar wadis for which data are available. Alternatively, techniques are available to estimate floods from rainfall records.

Unfortunately no evidence could be found of gauging stations on any of the wadis for which flood estimates were required. However sufficient data were collected from other wadis during the visit to enable both a statistical analysis and a rainfall runoff analysis for ungauged sites to be attempted.

Firstly a regional flood study was undertaken using annual peak flow data from 17 gauged wadis on the Red Sea coast (Figure 1.3). This technique relates a standard flood to catchment characteristics such as area and rainfall. The standard flood is then multiplied by an appropriate growth factor to give floods at various return periods. This approach has the advantage of using recorded flood data rather than rainfall data, thereby avoiding the many assumptions required when relating rainfall to flow. One disadvantage of this method is that only peak flows are estimated; hydrograph shape and volume are not provided.

Smaller wadis [A to G] contributing to southern stormwater channel

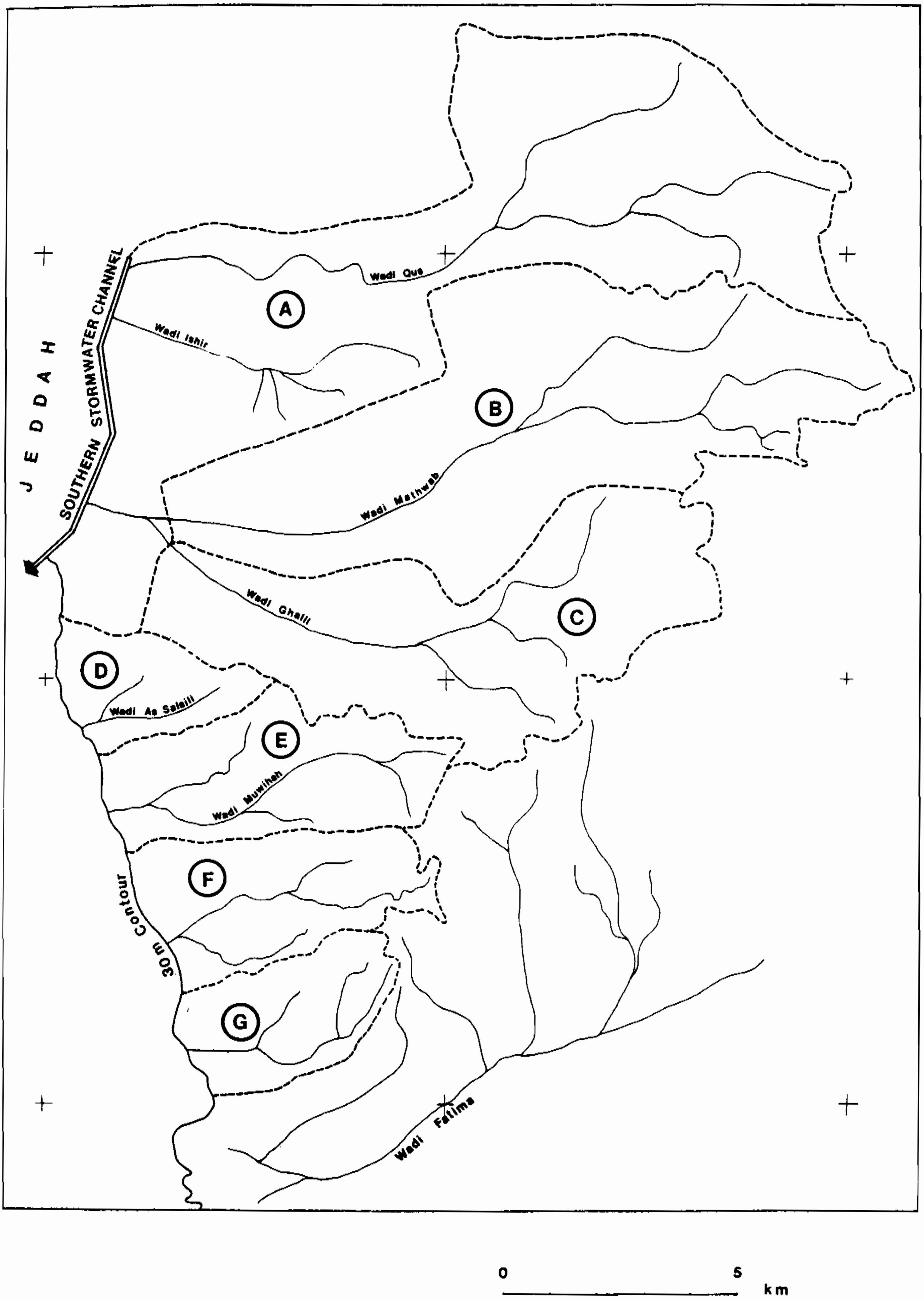


Figure 1.1

Location of catchments, gauging stations and raingauges near Jeddah

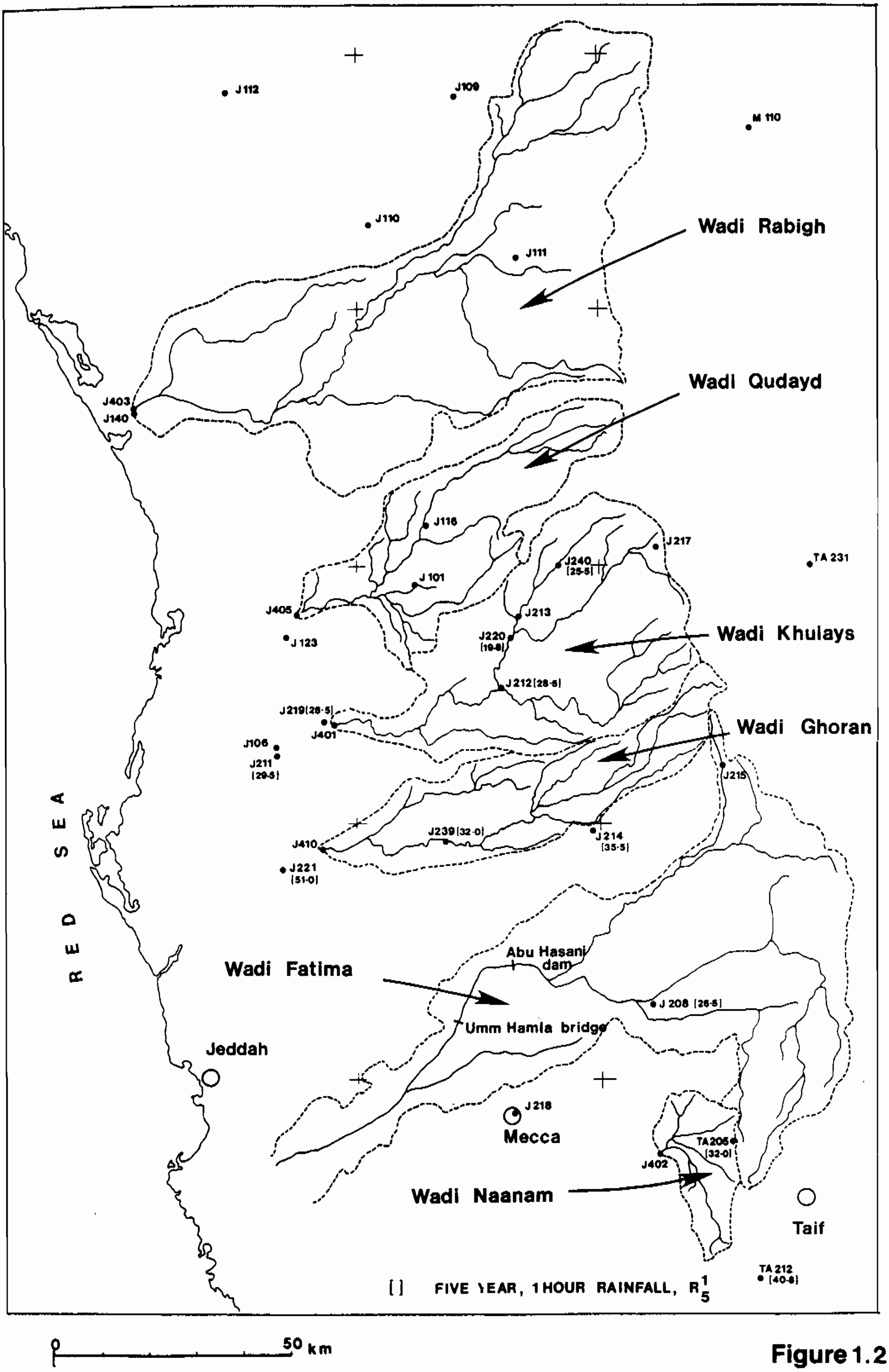


Figure 1.2

Location of peak flow stations

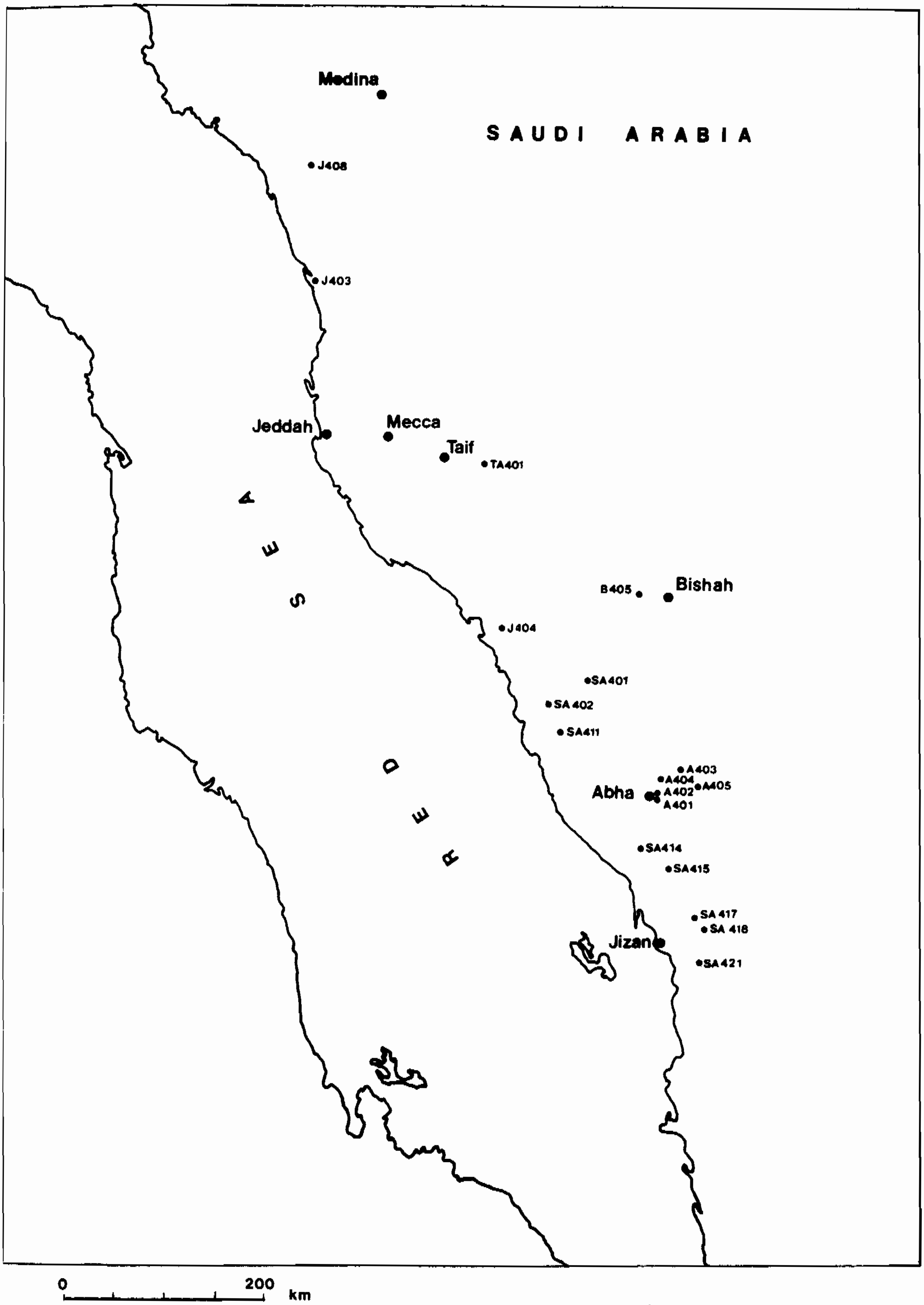


Figure 1.3

In the second approach a unit hydrograph/losses model was developed for flood prediction. The unit hydrograph method involves several stages. Firstly a rainstorm of suitable severity to produce a flood of the required return period must be constructed. This in itself requires knowledge of such factors as the local rainfall intensity/duration/frequency relationship, storm profile and areal reduction factors. The second stage requires the net rainfall to be estimated from the total rainstorm by subtraction of losses. These losses represent evaporation, interception and infiltration. A unit hydrograph is then required to translate the net rainfall over the catchment into the flood hydrograph. A unit hydrograph defines the response of the catchment to a unit input of rainfall over a certain time period and may be derived from floods and rainfall recorded on the catchment, or less accurately, from physical properties of the catchment. An advantage of this approach is that it provides the full hydrograph (peak, volume and shape) for design purposes. However, the main disadvantage is that the many assumptions and relationships are required, and particularly in a region of poor data, confidence in the results must be less than for the regional flood studies approach.

The regional flood study and unit hydrograph, are considered in more detail in the following sections.

1.2 Data available

The data used in this study fall into two categories:

(1) Wadi flow data
(2) Rainfall data

Wadi flow data were available in two forms. Firstly a series of annual maximum instantaneous peak flows for gauged wadis throughout Saudi Arabia was obtained direct from the Hydrology division of The Ministry of Agriculture and Water (MAW) in Riyadh. These records were augmented by data published in the series 'Hydrological Publications - Hydrology Division, Ministry of Agriculture and Water, Riyadh' (MAW, 1979). The second form of flow data were mean daily flows for gauged wadis in Saudi Arabia made available to the project by MAW, Riyadh. The instantaneous peak flow data were used in the regional flood study approach; the mean daily flow data were used to assess percentage runoff in the unit hydrograph approach.

Daily rainfall data were also made available in two forms. Firstly daily rainfall totals from gauges in the Jeddah-Taif-Rabigh region (from MAW) and records at Jeddah and Taif airports from the Meteorological and Environment Protection Agency in Jeddah were obtained. Secondly rainfall intensity data were provided by MAW for recording raingauges in the Jeddah-Taif region. The daily raingauge data were used both in the areal reduction factor analysis and percentage runoff calculation in the unit hydrograph method. The rainfall intensity data were used in deriving regional rainfall intensity/duration/frequency relationships for the unit hydrograph method.

1.3 Records of floods on Wadi Fatima

During the visit to Jeddah evidence was sought from local officials and residents of historic flooding on the wadis for which flood estimates were required. This investigation was concentrated on Wadi Fatima since this was the major wadi for channel design purposes and also the one most likely to have been observed flooding. It was important that any information regarding flooding depths could be translated into flow discharges. For this reason suitable sites for this information were constrained to reaches of well defined channels such as gorges or bridges where flow is confined to a known cross section.

In recent years it appears that there have been two significant floods on Wadi Fatima. The first and largest flood occurred in early April 1975 and the second flood occurred in mid January 1979. Other wadis in the Jeddah region also flooded at these times. Two useful reports of these floods were obtained for the Usfan-Mecca road crossing of Wadi Fatima near Umm Hamla (Figure 1.2):

(1) A report from an official at the MAW offices in Jeddah who witnessed the 1975 flood. He stated that the water level reached the road surface. In other words the bridge was passing its maximum discharge without being overtopped.

(2) A report from a local farmer whose house is on the bank immediately upstream of the bridge reported that in 1979 the water depth was about two metres.

1.4 Calculation of recorded floods on wadi Fatima

The Umm Hamla bridge has 10 spans of width 10.2 metres. The bridge is located on a fairly straight stretch of the wadi and is the control section of the channel at that point. In other words it is unlikely to be drowned out by constriction downstream. At the time of the visit (November 1984) no solid base to the channel was visible. Instead a fairly level sandy/fine gravel material formed the channel base under the bridge. The clearance under the bridge to this sandy material was measured as 4.2 m and the depth of bridge deck estimated as 1.2 m. Bradley (1973) provides a formula for calculating flow beneath bridges when inundated as in the 1975 flood:

$$Q = C_d \cdot b_n \cdot Z \cdot \left[2g(y_u - \frac{Z}{2} + \alpha_1 V_1^2/2g)\right]^{1/2}$$

C_d = coefficient of discharge

b_n = net width of waterway excluding piers

Z = bottom of upstream girder to mean river bed

y_u = upstream water surface to mean river bed

V_1 = upstream velocity

α = velocity head coeffient (1.5 assumed).

g = acceleration due to gavity

Substituting information from the 1975 flood a discharge of 1632 m^3s^{-1} is estimated.

During the 1979 flood the bridge was not inundated in the same way as in 1975. The 1979 flood was estimated assuming that the bridge acted as a rectangular throated flume. In this case the discharge was calculated as 583 m^3s^{-1}.

It is likely that these figures are underestimates of the true discharges because of scour of the sandy bed material during flooding. If, for example, the true depth during the 1975 flood was 5 m (originally 4.2 m) the peak discharge would be 2052 m^3s^{-1}. Similarly in 1979 (assuming 2.5 m depth) the peak discharge would be 814 m^3s^{-1}.

These estimates, however, should be used with caution, not only because of the uncertainty of the scour depth but also because of the unconfirmed evidence of the flood depths. However they do give an idea of the size of floods produced by Wadi Fatima in the last 9 years.

2. REGIONAL FLOOD STUDY

2.1 Introduction

Although river flow data are not available for Wadi Fatima or the other wadis for which flood estimates are required, some other catchments in the area are gauged. All available flood peak records from these gauged catchments were collected and analysed in order to generalise the regional flood regime of the Wadi Fatima area.

The method of analysis used is the proven technique of determining a standard reference flood and scaling this up to derive the flood peak of required return period using a flood growth curve. The standard reference flood adopted here is the 5 year return period flood peak, Q_5, rather than the more commonly utilised mean annual flood. Q_5 is believed to be a better standardising reference flood than the mean annual flood, because many wadis in this region do not experience annual flooding. Several years may separate flooding events.

2.2 A flood frequency curve for one site

A flood frequency curve relates the magnitude of a flood to the probability that a flood of that magnitude would be exceeded. The flood frequency curve enables flood magnitude corresponding to various design criteria to be estimated.

If a long flow record exists for a point on a river it is possible to construct a flood frequency curve from the recorded data as shown in Figure 2.1.

The annual maximum floods are abstracted from the N years of data and ordered so that the smallest flood is given rank 1 and the largest rank N. For each flood a probability of non-exceedence is assigned to it based on its position in the ranked series. This requires making an assumption about the form of the distribution from which the observed annual maxima are drawn. If the distribution is assumed to be a type 1 extreme value

(EV1) or Gumbel distribution, a good approximation to the non-exceedence probability is given by the Gringorten formula:-

$$F_i = \frac{i - 0.44}{N + 0.12}$$

where F_i is the non-exceedence probability (or plotting position) and i is the rank of the flood. In order to plot the frequency curve on linear graph paper, the EV1 reduced variate, y_i, is calculated from the values of F_i using the approximation

$$y_i = -\ln(-\ln F_i)$$

which is sufficiently accurate for plotting purposes.

The values of O_i are plotted against the corresponding y_i on linear graph paper. The resulting plot becomes rather more useful when the reduced variate axis is rescaled in terms of return period, T. The y values corresponding to various return periods can be calculated from

$$y = -\ln(-\ln(\frac{T-1}{T}))$$

The following table gives values of reduced variate for commonly required return periods.

Return Periopd, T (years)	Reduced Variate, y
2.0	0.37
5	1.5
10	2.25
20	2.97
50	3.90
100	4.60

A flood frequency curve

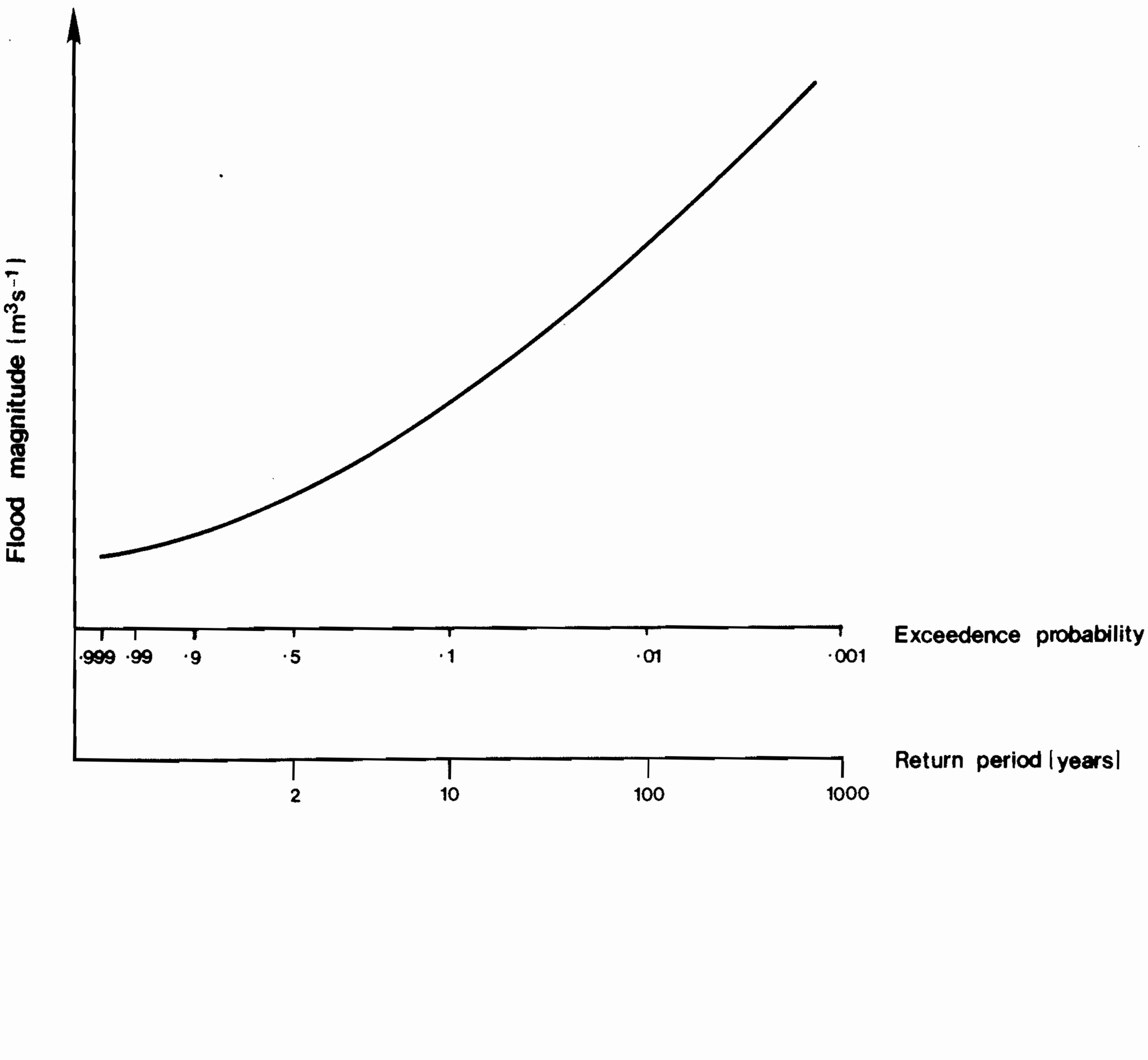


Figure 2.1

A smooth line is drawn through the plotted points but need not be constrained to pass through the highest point where this lies a considerable distance from the rest of the data. If the data plot as a straight line, then the assumption of a parent EV1 distribution appears valid. However, the plot is likely to show a slight curvature suggesting the parent distribution is something other than an EV1.

For the range covered by the curve the flood corresponding to a given return period can be estimated. The upper limit of the range will depend on the variability of the plotted data about the curve; even if the data plot on a straight line it should not be extended to return periods greater than twice the length of record.

There are several peak flow records available in this area. A summary of all records greater than 5 years in length and on the coastal side of the mountain range is shown in Table 2.1. The period of record varied between 6 and 21 years. Peak flow estimates are required for return periods of much greater than 20 years and so no one station can provide all the information required. Flood frequency information from several sites needs to be combined to extend the return period of predicted floods.

This is achieved by pooling all the data available and obtaining a consensus on the behaviour of catchments at high return periods.

2.3 Pooling of Growth Curves

As flood frequency curves differ greatly from catchment to catchment it is desirable to scale the individual curves prior to pooling. This is achieved by using non-dimensional flood frequency curves (growth curves) in which the flood magnitude scale is divided by an index flood. The index flood is then related to floods of other return periods by dimensionless multipliers or growth factors. The index flood used is normally the mean annual flood (MAF) but for Saudi Arabia we have chosen the 5 year return period flood Q_5. This is because the river flows are sporadic, the rivers do not flood each year and Q_5 may be more reliably estimated for our sites

TABLE 2.1 CATCHMENTS USED FOR REGIONAL ANALYSIS

	Catchment Name	Latitude	Longitude	Area (km^2)	MAR (mm)	Q_5 (m^3s^{-1})
SA A401	Wadi Abha at Abha	18°12'	42°29'	59	425	47.5
SA A402	Wadi Ashran at Mazma	18°18'	42°29'	80	425	78.2
SA A403	Wadi Bin Hashbal	18°28'	42°42'	2285	250	516.4
SA A404	Wadi Hani	18°24'	42°31'	146	350	60.2
SA A405	Wadi Jindahah	18°20'	42°52'	440	200	204.7
SA J403	Wadi Rabigh at Rabigh	22°48'	39°02'	4500	75	845.6
SA J404	Wadi Doqah Nr Ushaylah	19°45'	41°02'	970	285	293.0
SA J408	Wadi Safra at Dashabij	23°51'	38°54'	896	50	680.6
SAS SA401	Wadi Yiba at Thuluth	19°16'	41°48'	784	425	297.2
SA402	Wadi Yiba at Suq Juma	19°02'	41°28'	2722	425	796.5
SA411	Wadi Hali at Al Hussan	18°46'	41°35'	4576	350	1235.3
SA414	Wadi Itwad Main Station	17°46'	42°20'	1350	350	417.1
SA415	Wadi Baysh at Fatiyah	17°34'	42°36'	4713	400	1031.6
SA417	Wadi Damad Nr Damu	17°09'	42°53'	1000	450	915.3
SA418	Wadi Jizan at Malaki	17°03'	42°57'	1200	450	1129.1
SAS SA421	Wadi Khulab Nr Suq Al Ahad Masarha	16°43'	43°01'	900	500	699.5
SAT TA401	Wadi Bissel	21°11'	40°43'	236	200	125.5

17 Stations Total number of years 231

MAR = MEAN ANNUAL RAINFALL

Q_5 = FIVE YEAR RETURN PERIOD ANNUAL MAXIMUM FLOW of interest than MAF. The index flood is assumed to take into account catchment variables such as area, rainfall, slope etc.

The flood peak with a return period of five years was found by ranking and plotting the data as described in Section 2.2. From the table relating return period to reduced variate it can be seen that a five year return period is equivalent to a reduced variate of 1.5. The Q_5 can therefore be read directly from the station flood frequency curve at a reduced variate of 1.5.

For each station a non dimensional growth curve was constructed from the flood frequency curve by dividing each flood on the record by Q_5. In each case the growth curve was stored as a series of points; reduced variate and associated Q/Q_5. An example is shown in Figure 2.2 for station SA418.

An average growth curve was produced by taking the mean reduced variate and mean Q/Q_5 from all stations within each interval of reduced variate. The intervals of reduced variate used were - 1.5 to - 1.0, - 1.0 to - 0.5, - 0.5 to 0 etc. This is shown in Figure 2.3.

With the individual station record lengths ranging from 6 to 21 years, the smoothed average growth curve was well defined up to a return period of about 30 years. Because this is insufficient for many design purposes, the growth curve was extended by considering the five largest Q/Q_5 values in the data set and plotting these as the five largest values in a supposedly independent sample.

A pooled growth curve should be constructed from records describing stations in a homogeneous region. The stations summarised in Table 2.1 are all on the coastal side of the mountain range and so would be expected to be influenced by similar meteorological conditions. However there is a gradual reduction in mean annual rainfall from Jizan to Medina which may effect the shape of the growth curve. To test for the effect of mean annual rainfall on growth curve shape, the stations were divided into a northern and southern region and two separate pooled growth curves constructed. Separate growth curves were also constructed for large ($>$ 1000 km^2) and small ($<$ 1000 km^2) catchments to investigate the effects

Flood growth curve for Wadi Khulab

nr Suq Al Ahad Masarha, SA 418

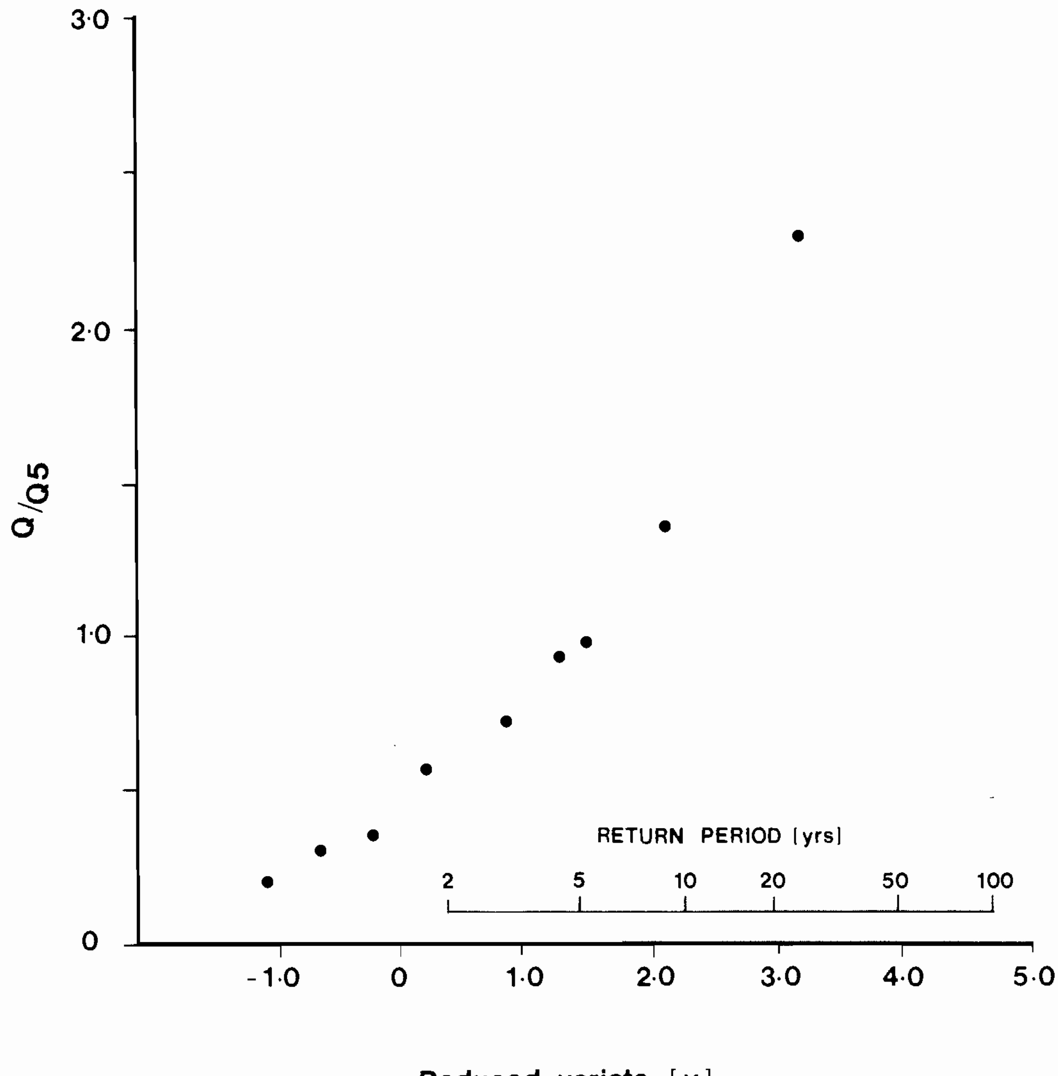


Figure 2.2

Regional growth curve

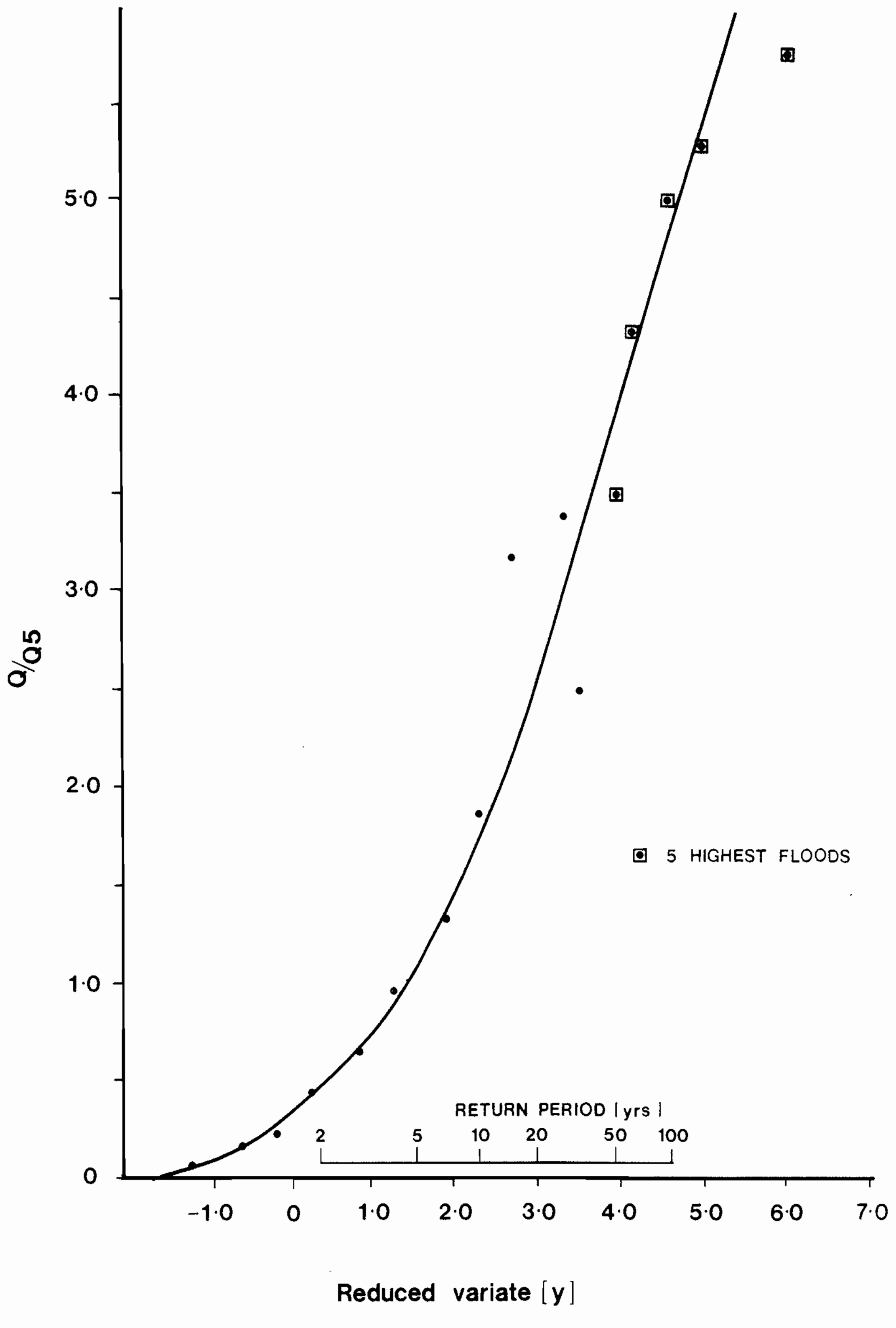


Figure 2.3

of catchment area on growth curve shape. In both cases the curves exhibited no significant difference in shape therefore the original pooled curve, constructed from all the data, was used for flood peak prediction. This curve (Figure 2.3) can be used to predict the peak flow of any return period up to approximately 100 years once the Q_5 has been established for a catchment.

Growth factors abstracted for convenient return periods from figure 2.3 are:

Return period	Growth factor
10	1.64
20	2.36
50	3.56
100	4.52

2.4 Estimation of Q_5

The catchments for which flood estimates are required are all ungauged. No flood peak information exists other than the level of two floods at the main road bridge on Wadi Fatima. In these circumstances the most effective way of estimating Q_5 is to define a relationship between Q_5 and catchment characteristics for the gauged catchments and apply this to ungauged sites. Thus Q_5 may be estimated at an ungauged site from easily measured characteristics of the catchment and the derived relationship.

The general form of the relationship between particular catchment characteristics and the magnitude of floods is often obvious; for example, bigger catchments have bigger floods. However, to be of any use it is necessary to index both the size of flood and the characteristic of the catchment and to establish a formal relationship between the two. The size of a catchment is given by its area although an alternative index would be main stream length. It is not possible to describe the relationship between Q_5 and area as a precise physical model but it is possible to develop a simple relationship based on observed values of the two indices. Values of Q_5 can be plotted against area and any observed relationship can be represented by a line on the figure. The subjectiveness of this can be removed by using regression analysis which provides an optimal line, in the least-squares sense. If the relationship appears non-linear, then it is necessary to transform the variables before analysis so that linear regression techniques are applicable. Regression analysis enables coefficients of the proposed relationship to be determined, the goodness of fit to be evaluated and a comparison of different relationships. Of course the magnitude of the Q_5 may not just depend on catchment size but also on climate, slope, geology and soil and other factors. However, experience in other parts of the world shows that floods are affected primarily by catchment area with climate being the other important secondary factor.

Regression analysis enables an equation to be developed that relates Q_5 to area and the mean annual rainfall of each gauged catchment.

Table 2.1 lists the catchments used in this regression study and includes details of catchment area and mean annual rainfall (MAR) for each catchment. Area was taken from computer listings of instantaneous flood peaks provided by MAW. The MAR was calculated by superimposing the catchment boundaries on a map of MAR produced by the Al Shalash (1973).

Two types of regression were carried out, firstly linear regression of Q_5 against catchment area, and secondly multiple regression of Q_5 against area and MAR. It was necessary to transform all the variables to logarithms to approximate a linear relationship in each case.

The regression of Q_5 with area produced a reasonable fit with a correlation coefficient of 0.92. Adding the catchment rainfall did not produce a significant improvement in the regression and so the equation involving area was chosen for Q_5 prediction. Figure 2.4 shows a plot of the results and the best fit line which is

$$\mathrm{Log}_{10}(Q_5) = 0.45 + 0.72 \log_{10}(\mathrm{Area})$$

$$\text{or } Q_5 = 2.818 \, \mathrm{Area}^{0.72}$$

other catchment characteristics could be introduced to the regression to try to improve the relationship for Q_5 estimation but as rainfall does not improve the prediction it is unlikely that any other characteristic will have any effect.

Regression results

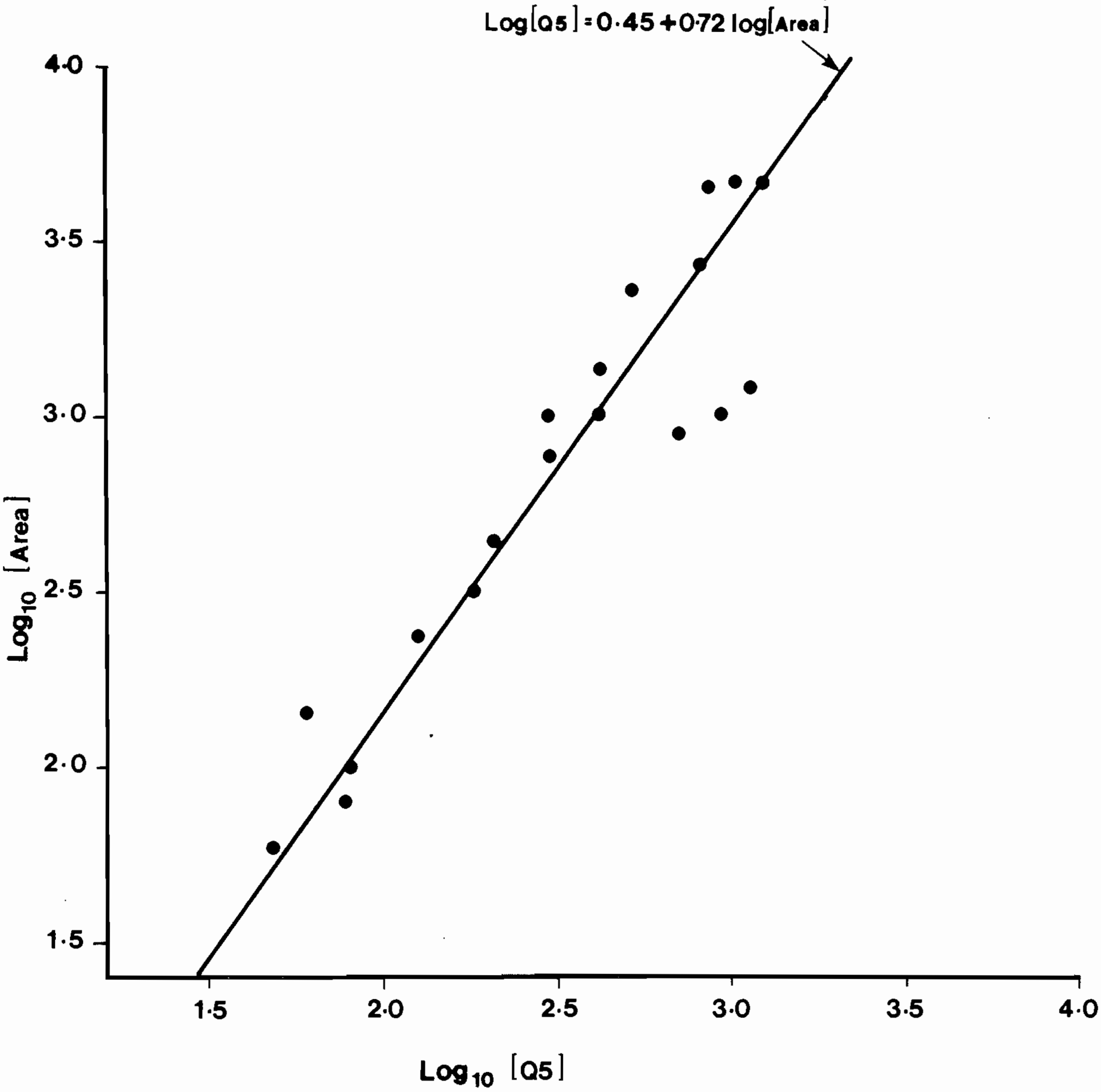


Figure 2.4

2.5 Using the method

For all catchments for which flood estimates were required, Q_5 was estimated using the regression relationship with catchment area.

Floods of return period 10, 20, 50 and 100 years were then obtained by multiplying Q_5 by the appropriate growth factors given in section 2.3.

The results of this method are presented and discussed in Section 4 below.

It is interesting to compare the flood frequency results from this study with those from other similar regions of the world for which the consultant has experience. It was shown in Section 2.3 that the 20 and 100 year return period floods are respectively 2.36 and 4.52 times larger than Q_5. For central Iran the observed ratios and 2.00 and 3.87 for 20 and 100 year return periods respectively whilst for Jordan the comparable figures are 1.89 and 3.21. Thus the Jeddah region appears to have a relatively steep flood frequency relationship and is certainly comparable with other semi-arid regions of the world such as Iran, Jordan and north-east Botswana where the ratios are 2.07 and 4.73. This agreement between the present study and results from other regions of the world with broadly similar climates increases confidence in the results presented in Tables 4.1 and 4.2.

The other component of the regional estimation procedure is the estimation of Q_5. The regression equation derived in Section 2.4 for this purpose predicts Q_5 varying from 1.6 $m^3\ s^{-1}\ km^{-2}$ on Wadi D to 0.27 $m^3\ s^{-1}\ km^{-2}$ for the entire wadi Fatimah catchment of 4600 km^2. These specific 5 year return period runoffs agree well with those from Botswana where Q_5 varies from 0.17 $m^3\ s^{-1}\ km^{-2}$ on a catchment of 5960 km^2 to 0.59 $m^3\ s^{-1}\ km^{-2}$ on a 570 km^2 catchment and for large catchments of about 4000 km^2 to 6000 km^2 in Jordan where Q_5 varies from 0.12 to 0.18 $m^3\ s^{-1}\ km^{-2}$.

Thus the present regional flood study produces flood estimats that are in broad agreement with floods from parts of the world having generally similar climates.

3. UNIT HYDROGRAPH ANALYSIS

3.1 Explanation of method

On a worldwide basis rainfall stations are more plentiful and their records longer than river gauging stations. From local rainfall records it is normally possible to derive rainfall depth/duration/frequency relationships and use the statistical properties of the rainfall to estimate floods of the required return period. For this to be possible a method of converting rainfall to river flow is required. Unit hydrographs, which define the response of a catchment to an input of unit net rainfall, have gained acceptance by most hydrologists as a useful tool in flood estimation. If possible catchment unit hydrographs should be derived from flood events recorded on the catchment together with a continuous (autographic) trace of storm rainfall. In the absence of the necessary continuous rainfall and flow data, synthetic unit hydrographs may be constructed using catchment properties such as stream length and channel slope. For the catchments in this study, synthetic unit hydrographs were used because of the lack of specific flood event data.

The unit hydrograph defines the catchment response to net rainfall. Gross storm rainfall of a given return period may be estimated from the rainfall depth/duration/frequency relationship. The difference between the gross rainfall and that running off as flood water is termed 'losses'. Rainfall losses occur as evaporation, interception and infiltration and were estimated in this study by looking at runoff from specific storm events on gauged wadis in the Jeddah area.

The three main aspects of the unit hydrograph analysis, the rainfall depth/duration/frequency relationship, the catchment losses and unit hydrograph derivation are considered more fully in the following sections.

3.2 Analysis of rainfall

3.2.1 Depth/duration/frequency relationships

Rainfall measured at recording raingauges around Jeddah was provided by MAW (Riyadh) to the project. These gauges were:

TA212, TA205, J214, J208, J240, J220,
J212, J239, J218, J219, J221, J211

From these data annual maximum rainfall for the following durations was abstracted:

10 minutes, 30 minutes, 1 hour, 3 hours, 12 hours, 1 day, 3 days

In common with other rainfall depth/duration/ frequency studies both in Saudi Arabia (Wan, 1976) and elsewhere (Bell, 1959), the rainfall data from the 12 gauges were standardised. Both Wan and Bell used the station 1 hour, 10 year return period rainfall, R^1_{10}, as the standardising factor. In this study the same duration of 1 hour was used, but of 5 years return period, R^1_5. The 5 year return period was adopted for two reasons. Firstly R^1_5 was more accurately estimated for individual gauges from the short records than the 10 year return period rainfall. The second reason was that the 5 year return period was the standardising return period flood used in the regional study (Section 2). R^1_5 is shown on Figure 1.2 for the 12 raingauges used.

The advantage of standardisation is that generalised rainfall depth/ duration/frequency relationships may be established for a region and scaled by local estimates of R^1_5. Thus to estimate, R^t_T, the t duration, T year return period point rainfall for a site, R^1_5 is estimated for the site and scaled by the appropriate factor from the depth/duration/frequency curves. This approach is similar to that adopted in the regional flood peak analysis where Q_5 is the standardising factor.

The length of each station record varied between 4 and 14 years which is too short to derive individual station rainfall depth/frequency relationships to the return periods required (up to 100 years). The record length was extended by adopting the station-year approach which assumes that in a region of similar rainfall characteristics the summation of a number of individual stations may be taken to represent a single station of longer record. For this to be true it is necessary that the rainfall regime for the durations of interest is homogeneous in the region and that the stations are sufficiently far apart for the rainfall events at these durations to be independent. In the time available no statistical tests were carried out to ascertain that these assumptions were true, however on the assumption of independence, a review of the data revealed that only on a very few occasions did the same storm produce annual maximum intensities at more than one place. In order to ensure homogeneity, only stations in the mountain range were used in the analysis. Furthermore, since the rainfall data have been standardised, it is only necessary to assume that the ratio of rainfall depths at various return periods to R_5^1 be constant over the region. There is no requirement for the absolute rainfall depth/duration/frequency relationships to be constant over the region. It was therefore concluded that the region covered by these raingauges was homogeneous and rainfall events sufficiently independent to allow the station year approach to be used. The total record length for these gauges was 107 station years. This was sufficient to define rainfalls at each duration up to the 100 year return period.

The following procedure was used to derive the regional standardised rainfall depth/duration/frequency curves.

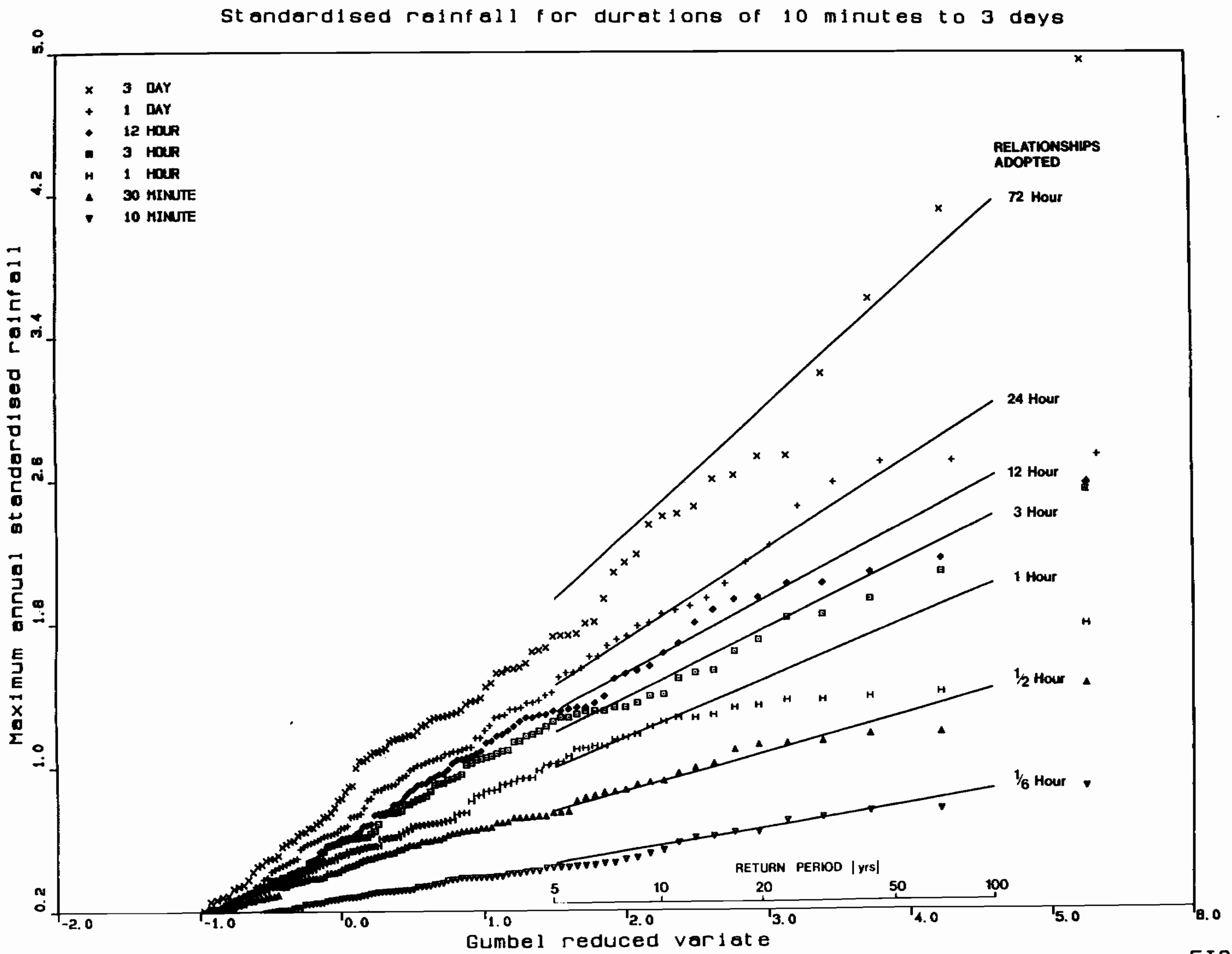
(1) Annual maximum 1 hour rainfalls for each station were ranked and plotted using the Gringorten plotting position with a Gumbel reduced variate (Section 2.2)

(2) R_5^1 was estimated for each station from the above graphs.

(3) For each of the 12 stations used, rainfalls for various durations were standardised by dividing by the appropriate station R_5^1 value.

(4) Data for each duration were combined using the station year approach outlined above and plotted using the Gringorton plotting position with a Gumbel reduced variate. These data are shown in Figure 3.1.

(5) Figure 3.1 shows that for all durations a straight line relationship is reasonable for return periods up to at least 20 years. For some durations increasing scatter and flattening above this return period makes the relationship less obvious. However there are fewer data points in this region and they are also less accurate. On several



Standardised rainfall for durations of 10 minutes to 3 days
3 DAY
1 DAY
12 HOUR
3 HOUR
1 HOUR
30 MINUTE
10 MINUTE
RELATIONSHIPS ADOPTED
72 Hour
24 Hour
12 Hour
3 Hour
1 Hour
½ Hour
⅙ Hour
RETURN PERIOD |yrs|
5
10
20
50
100
Maximum annual standardised rainfall
0.2
1.0
1.8
2.6
3.4
4.2
5.0
-2.0
-1.0
0.0
1.0
2.0
3.0
4.0
5.0
8.0
Gumbel reduced variate

FIGURE 3.1

occasions of major rainstorms, raingauge observer's notes state that the gauge had malfunctioned, either as a result of blockage by sand or failure to record. This non recording of the more severe storms may help to explain the flattening of some of the rainfall frequency relationships at high return periods. In consideration of this fact the straight line relationship was extended for all durations to 100 years return period.

(6) Standardised rainfall were abstracted from these lines for each duration and for return periods between 2 and 100 years. This information is given in Table 3.1

It might be expected that the 1 day and 3 day rainfall totals would be lower than the 24 and 72 hour totals respectively. This is because it is possible for a rainstorm to be split between two measuring days. However the times of rainstorms in the Yemen Arab Repulic were shown to between the hours 12.00 am and 7.00 pm (Green, 1982). It is believed that Saudi Arabia follows common worldwide practice of measuring rainfall in the early morning. In this case the 1 day rainfall will be equal or extremely close to the 24 hour rainfall and the 3 day rainfall have a similar relationship to the 72 hour rainfall. Therefore, in the study, the 1 day/24 hour, 3 day/72 hour rainfalls have been assumed to be the same.

Rainfall at durations between those given in Table 3.1 were obtained by logarithmic interpolation on both the R^1_5 ratio and storm duration. The rainfall depth/duration/frequency ratios given in Table 3.1 were used to derive synthetic rainstorms for input to the unit hydrograph model.

3.2.2 Standardising rainfall

The rainfall depth/duration/frequency relationships derived above are expressed as a ratio to the 1 hour, 5-year return period rainfall, R^1_5. The The problem considered here is the estimation of R^1_5 for each catchment so that absolute point rainfalls can be obtained for any location in the study area.

Figure 1.2 shows R^1_5 for the gauges used in the study. It had been hoped that an isohyetal map of R^1_5 could have been drawn from these values.

TABLE 3.1 RATIO TO 1 HOUR, 5 YEAR RETURN PERIOD RAINFALL

	Return period (years)					
Duration	2	5	10	20	50	100
3 Day	1.11	1.92	2.46	2.99	3.65	4.32
1 Day	0.88	1.46	1.84	2.21	2.68	3.04
12 hour	0.77	1.28	1.62	1.95	2.36	2.69
3 hour	0.72	1.18	1.48	1.77	2.14	2.43
1 hour	0.62	1.00	1.24	1.48	1.78	2.02
30 minute	0.52	0.76	0.91	1.07	1.26	1.41
10 minute	0.31	0.45	0.55	0.64	0.76	0.85

However there was no clearly definable trend of R_5^1 over the study region to enable this to be done.

A second approach was to correlate R_5^1 with mean annual rainfall since a mean annual rainfall map exists for the region (Al. Shalash, 1973). Figure 3.2 shows R_5^1 plotted against mean annual rainfall for the recording gauges used in the study. There is poor correlation between the two variables.

The lack of success of the above approaches led us to believe that over our area of interest, and with the data available no well defined trend in R_5^1 was apparent. For design purposes, therefore, a mean R_5^1 of the following gauges was used for all catchments in the study:

J221, J239, J214, J208, TA205, TA212

These gauges were closest to the catchments for which flood estimates were required. The mean R_5^1 of these gauges is 36.4 mm.

Point rainfall depths within all of the design catchments was estimated by multiplying the mean R_5^1, 36.4, by the appropriate factor given in Table 3.1 to give rainfalls of the required duration and return period.

3.2.3 Areal Reduction Factors

The rainfall estimates derived earlier are those that apply to any given raingauge or point within a catchment. The total storm rainfall over a large catchment area would be significantly lower than these point rainfalls. An areal reduction factor is a means of converting point rainfall statistics to catchment rainfall estimates and is normally a function of storm duration and catchment area.

The existing raingauge network around Jeddah is not ideally suited to estimation of an areal reduction factor (ARF), nevertheless the available data were analysed to obtain appropriate ARF values for the study area.

Five Year, 1 Hour Rainfall v Mean Annual Rainfall

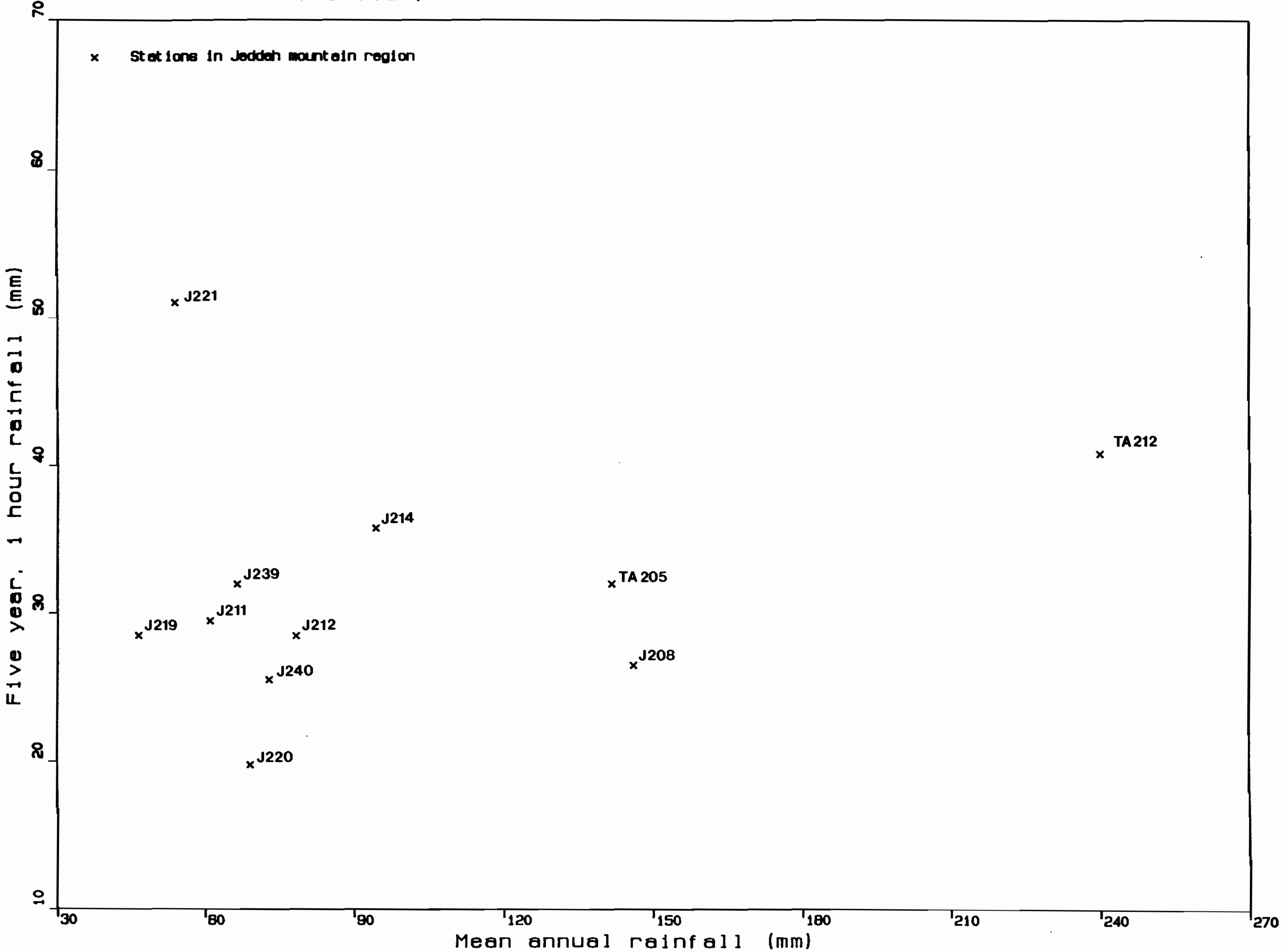


FIGURE 3.2

An examination of the available raingauge network showed two broad groupings of raingauges with suitable concurrent records. The two groups are situated north of wadi Fatima and are shown on Figure 3.3.

A number of arbitrary circular "catchment" areas were drawn onto the available raingauge network having areas varying from 500 to 11400 km^2. These synthetic catchments contained varying numbers of gauges from a minimum of three in areas N1 and S3 (where gauge J106 has only a very short record and receives very similar rainfall to J211). The larger synthetic catchments had a larger number of raingauges, although not all gauges were operational for all storms.

For each synthetic catchment area, all significant large storms were abstracted from the available daily rainfall records. For each storm, an areal rainfall was estimated as the simple mean of all recording raingauges. For each storm the duration was assumed to be 24 hours as no adequate duration data were available. However, some information on rainfall depths and durations was available and the analysis of these data is described later. The areal reduction factor for any storm is the ratio of the areal rainfall to the maximum point rainfall at any of the raingauges within the area. A large number of storms were analysed in this way. It was decided that the arithmetic mean of all computed one day ARF's be taken as the best estimate of ARF for each area. The computed ARF values are shown in Table 3.2.

The analysis was repeated for storms of two and three days duration and results are also shown in Table 3.2.

For durations of less than one day, some depth and duration data were available. However, for any given storm, it was not always possible to determine an areal rainfall as the timing of the storms on any given date is not given. For a number of storms examined, several separate rainfalls were recorded at some raingauges on any date, yet only one rainfall storm was noted at adjacent raingauges for the same day. Therefore it was not possible to determine the areal rainfall on such occasions as the single rainstorm at one site could not be associated with any particular storm at sites with several storms on the same date. Hence it was only possible

"Synthetic" catchments used for derivation of areal reduction factors

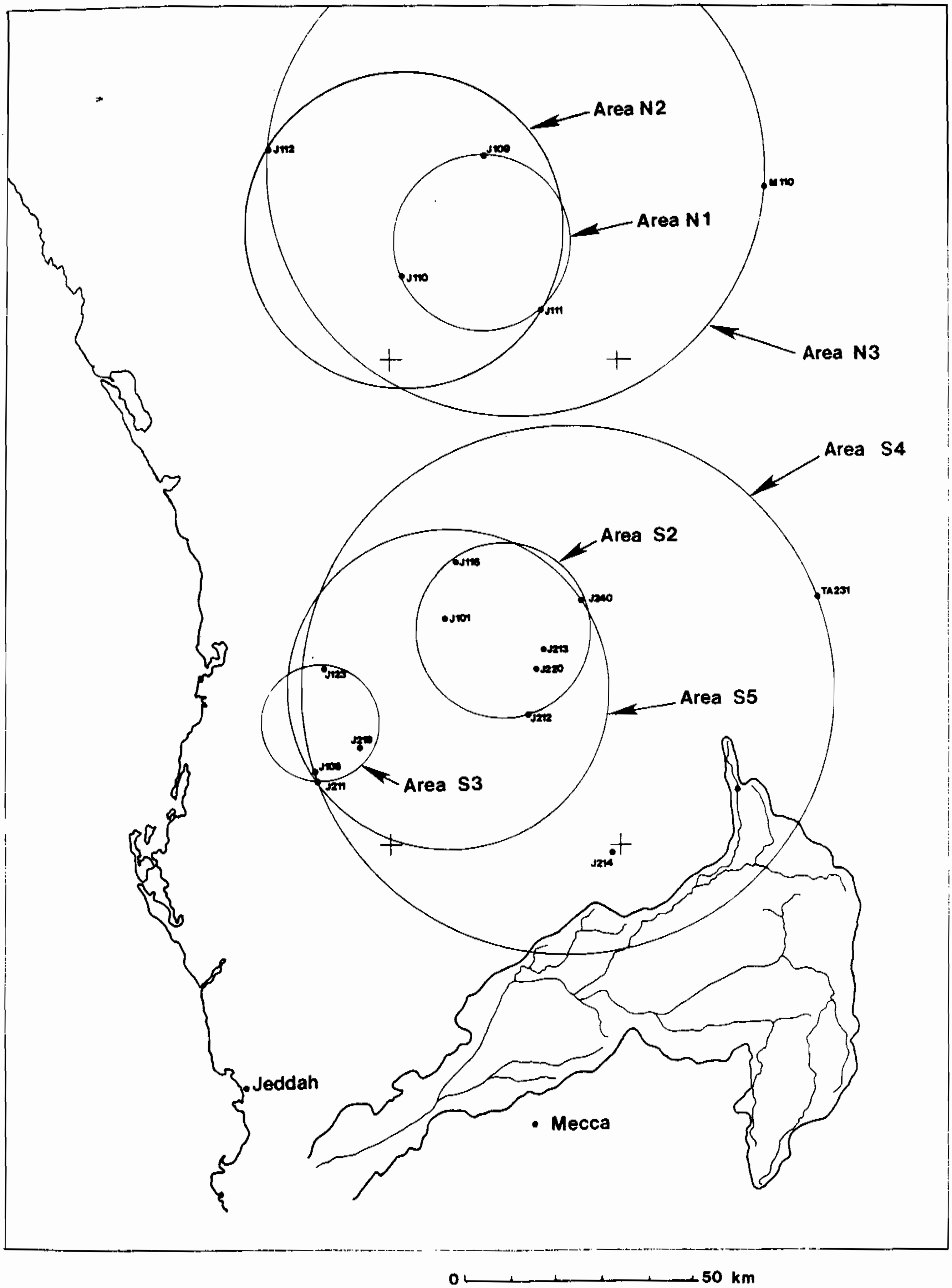


Figure 3.3

TABLE 3.2 ESTIMATED AREAL REDUCTION FACTORS

Note ARF = $\frac{\text{Catchment area rainfall}}{\text{Maximum point rainfall within catchment}}$

<table>
<tr><th></th><th></th><th colspan="4">DURATION</th></tr>
<tr><th>Area (Km^2)</th><th>Map Unit</th><th>1 hour</th><th>1 day</th><th>2 days</th><th>3 days</th></tr>
<tr><td>500</td><td>S3</td><td>0.48</td><td>0.62</td><td>0.72</td><td>0.82</td></tr>
<tr><td>1200</td><td>S2</td><td rowspan="2">0.37</td><td rowspan="2">0.55</td><td>0.65</td><td rowspan="2">0.72</td></tr>
<tr><td>1250</td><td>N1</td><td>0.645</td></tr>
<tr><td>3960</td><td>N2</td><td rowspan="2">0.30</td><td>0.43</td><td rowspan="2">0.50</td><td rowspan="2">0.61</td></tr>
<tr><td>4130</td><td>S5</td><td>0.43</td></tr>
<tr><td>9850</td><td>N3</td><td>0.255</td><td>0.41</td><td>0.45</td><td>0.51</td></tr>
<tr><td>11400</td><td>S4</td><td>0.25</td><td>0.40</td><td>0.44</td><td>0.49</td></tr>
</table>

to derive ARF's for dates where only one rainstorm was recorded at each gauge. There were few such dates on record and consequently estimation of ARF for durations of less than 24 hours is less precise than for the longer durations. The best available estimates for storms of 1 hour duration were obtained and are shown Table 3.2.

The ARF estimates for each duration were plotted against synthetic catchment area as shown on Figure 3.4. These curves were re-plotted as ARF against $\log_{10}$(AREA) and straight lines drawn through the points by eye. It was possible to fit a functional relationship to these lines of the following form:-

$$\text{ARF} = 0.9332 - 0.188 \log_{10} \text{AREA} + 0.0434 \sqrt{D}$$

where D is the storm duration in hours. The relationship is illustrated on Figure 3.5. For small areas and long durations the relationship suggests areal reduction factors of 1 or more. In order to keep ARF's reasonble, a maximum ARF of 0.98 was assumed when the relationship estimated an ARF greater than 0.98.

For any catchment area, the ARF for each duration was obtained from the fitted functional relationship given above.

3.2.4 Storm duration

Having obtained the catchment areal rainfall it is now necessary to decide upon a suitable design duration of the rainstorm. The design duration is the critical duration for the design flood. Small catchments have a shorter design durations than larger catchments because small catchments respond more quickly to rainfall than larger catchments and are therefore more sensitive to short, local intense storms. Large catchments have a higher response to generally less intense but longer duration, widespread storms.

The UK Flood Studies Report (NERC, 1975) suggests a design duration which depends both on the unit hydrograph time to peak, Tp, and catchment mean annual rainfall. Tp is a measure of how quickly a catchment responds to rainfall and is discussed later in Section 3.4.1. This formula is considered inappropriate here since it was derived for a range of Tp and mean annual rainfall untypical of the Saudi west coast.

Best estimates of areal reduction factors

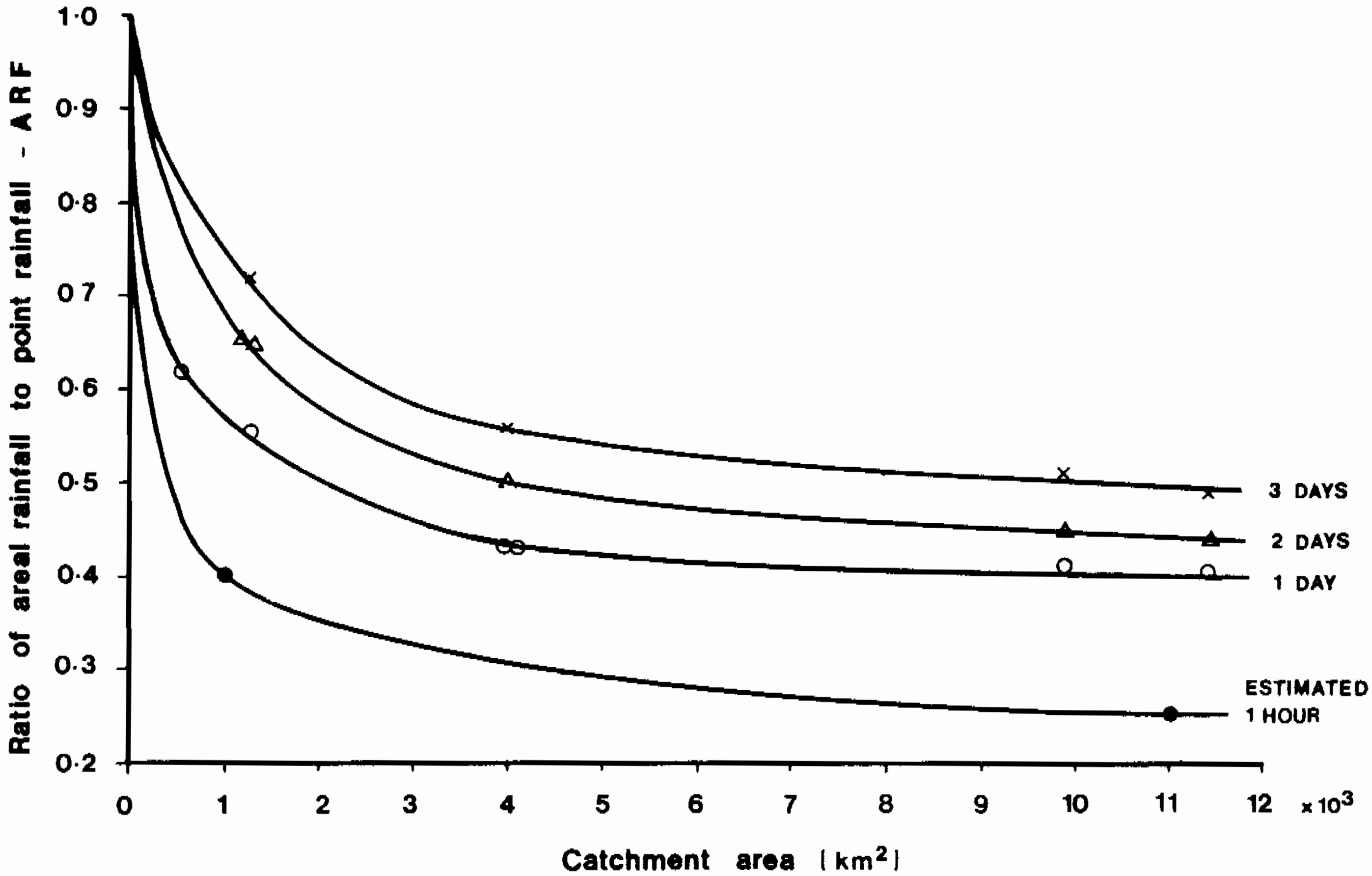


Figure 3.4

In this study the following relationship between design storm duration, D, and unit hydrograph time to peak, Tp, gave reasonable durations for the range of catchments studied:

$$D = Tp \times 12$$

3.2.5 Storm profile

So far we have information to enable us to estimate areal rainfall depths of various durations and return periods and a design duration for each catchment. A storm profile is now required to distribute the rainfall in time.

Normally storm profiles may be constructed by studying rainfall events recorded on recording raingauges in the region of interest. However the rainfall intensity data provided gave only the highest rainfall totals within each day at various durations. No information concerning the distribution of rainfall within the day was available.

In the absence of information on local storm profiles a nested rainfall profile was adopted such that for all durations the rainfall intensities of the same return period occurred within the same storm. A nested rainstorm profile is symmetrical with the highest intensity in the centre of the storm. For example the 1 in 100 year, 24 hour storm was composed of the 1 in 100 year 1 hour storm in the centre of the 1 in 100 year 3 hours fall etc. Although the average intensity during any part of the storm does not exceed the 1 in 100 year value, nesting the profile in this way tends to create a larger flood peak than a more uniform rainfall profile. However the flood peak estimated from a nested profile is not as large as that from a profile with the highest intensity shifted towards the start of rainfall. The nested profile may therefore be considered as a compromise between the two extremes.

3.2.6 Time interval

The basic time interval, dt, used to define the rainstorm (and the unit hydrograph) is not critical in the estimation procedure. In order to achieve a similar resolution of the design flood peak, a unit hydrograph with a short time to peak requires a finer time interval than does a unit hydrograph with a large time to peak. The UK Flood Study Report (NERC, 1975) suggests the following relationship:

$$dt \approx Tp/5$$

The relationship is approximate so that dt may be chosen as some convenient number of hours or fraction of an hour. This relationship has been used in this study since it is independent of local factors.

3.2.7 <u>Rainfall/Flooding return period</u>

For this study it has been assumed that the storm and flood return period are equal (ie the 100 year return period storm is used to estimate the 100 year return period flood). In practice this may or may not be the case depending on such factors as the antecedent conditions of the catchment, storm profile and storm duration. In the very dry Jeddah region, with normally dry sandy soils and bare rocky slopes the antecedent conditions for major flood events are likely to be similar. Although no information about storm profile and duration, the assumption that rainfall and corresponding flood return period are similar was considered acceptable in this situation.

3.2.8 <u>Example design storm</u>

As an example consider the 100 year design storm for the catchment comprising Wadis B & C (Figure 1.1) whose time to peak (Tp) is 1.5 hours (Section 3.4.1).

(1) <u>Time interval</u>

$$dt \approx Tp/5$$

The nearest convenient time interval, dt, is 15 minutes or a quarter of an hour

$$\underline{dt = 0.25}$$

(2) Design duration

$$D = Tp \times 12$$

$$D = 18 \text{ hours}$$

However with a nested rainfall profile it is necessary to have a design duration which is an odd multiple of the data interval. The next highest duration is used:

$$D = 18.25 \text{ hours}$$

(3) <u>Estimate of R_5^1</u>

The regional mean value of R_5^1, 36.4 mm, is used for this (and all other catchments).

(4) <u>Nested rainstorm</u>

Table 3.1 is used to estimate $R_{100}^{0.25}$, $R_{100}^{0.75}$, $R_{100}^{1.25}$ ... $R_{100}^{18.25}$. Logarithmic interpolation is necessary for durations between those given in the table.

Areal reduction factors calculated from the formula given in Section 3.2.3 are applied to each duration of rainfall in turn:

$$ARF = 0.9332 - 0.188 \times \text{Log}_{10}(\text{AREA}) + 0.0434 \sqrt{D}$$

Finally the rainstorm is constructed by firstly placing the areally reduced $R_{100}^{0.25}$ at the centre of the storm. This value of $R_{100}^{0.25}$ is then subtracted from the areally reduced $R_{100}^{0.75}$ to give the remainder of the rainfall falling 0.25 hours either side of the central 0.25 hours. This process is then repeated until the entire 18.25 hour storm has been synthesised.

Table 3.3 illustrates the process.

Time - t (hours)	R^{t}_{100}	ARF	ARF x R^{t}_{100}
0.25	37.26	0.5799	21.61
0.75	63.32	0.5958	37.73
1.25	76.33	0.6067	46.31
1.75	80.79	0.6156	49.74
.	.	.	.
.	.	.	.
18.25	105.42	0.7436	78.40

Design Rainstorm

Time (hours)	Rainfall (mm)
0.00	0.33
0.25	0.33
.	.
.	.
.	.
.	.
.	.
.	.
8.25	1.71
8.50	4.29
8.75	8.06
9.00	21.61
9.25	8.06
9.50	4.29
9.75	1.71
.	.
.	.
.	.
.	.
.	.
.	.
.	.
17.75	0.33
18.00	0.33

Table 3.3 Derivation of a nested rainstorm
(100 year event on Wadi B & C)

3.3 Percentage runoff/losses

The previous sections have described how rainstorms of different durations and return periods have been estimated for the study catchments. Not all of this rainfall will end up as flood runoff. Losses occur to evaporation, interception and infiltration. Estimating losses is a difficult task. Not only are all catchments different and hence losses different, but also losses vary from event to event on the same catchment depending on antecedent conditions and spatial and temporal distribution of rainfall.

It is unwise to transport empirical losses formulae from other regions of the world because of these differences. This is particularly true in the present study because of the arid nature of the region.

Losses were studied by looking at significant flood events measured at wadi gauging stations J401, J410, J405, J403 and J402 in the Jeddah region (Figure 1.2). Mean daily flow data were available for these stations along with daily rainfall from gauges in and close to the respective catchment areas. For significant flood events recorded at each of the above stations, flood volume was obtained by summing the mean daily flows during the event. Rainfall contributing to the event were abstracted from the daily records according to the table:

	Wadi	Raingauges
J401	Wadi Khulays near Umm Adda	J219, J101, J240, J213, J212, J217, J220
J410	Wadi Ghoran near Usfan	J239, J214, J215, J221
J405	Wadi Qudayd at Hammamah	J123, J116, J101, J213
J403	Wadi Rabigh at Rabigh	J140, J110, J109, J116
J402	Wadi Noanam near Firrain	TA205, TA212

Not all raingauges were operational for all events in each wadi. The catchment rainfall for each event was taken as the mean of all guages in operation at the time.

Total runoff was plotted against total rainfall and is shown on Figure 3.6. There is considerable scatter, but this is to be expected since it is not possible to get accurate estimates of catchment rainfall from a few

Storm rainfall v runoff

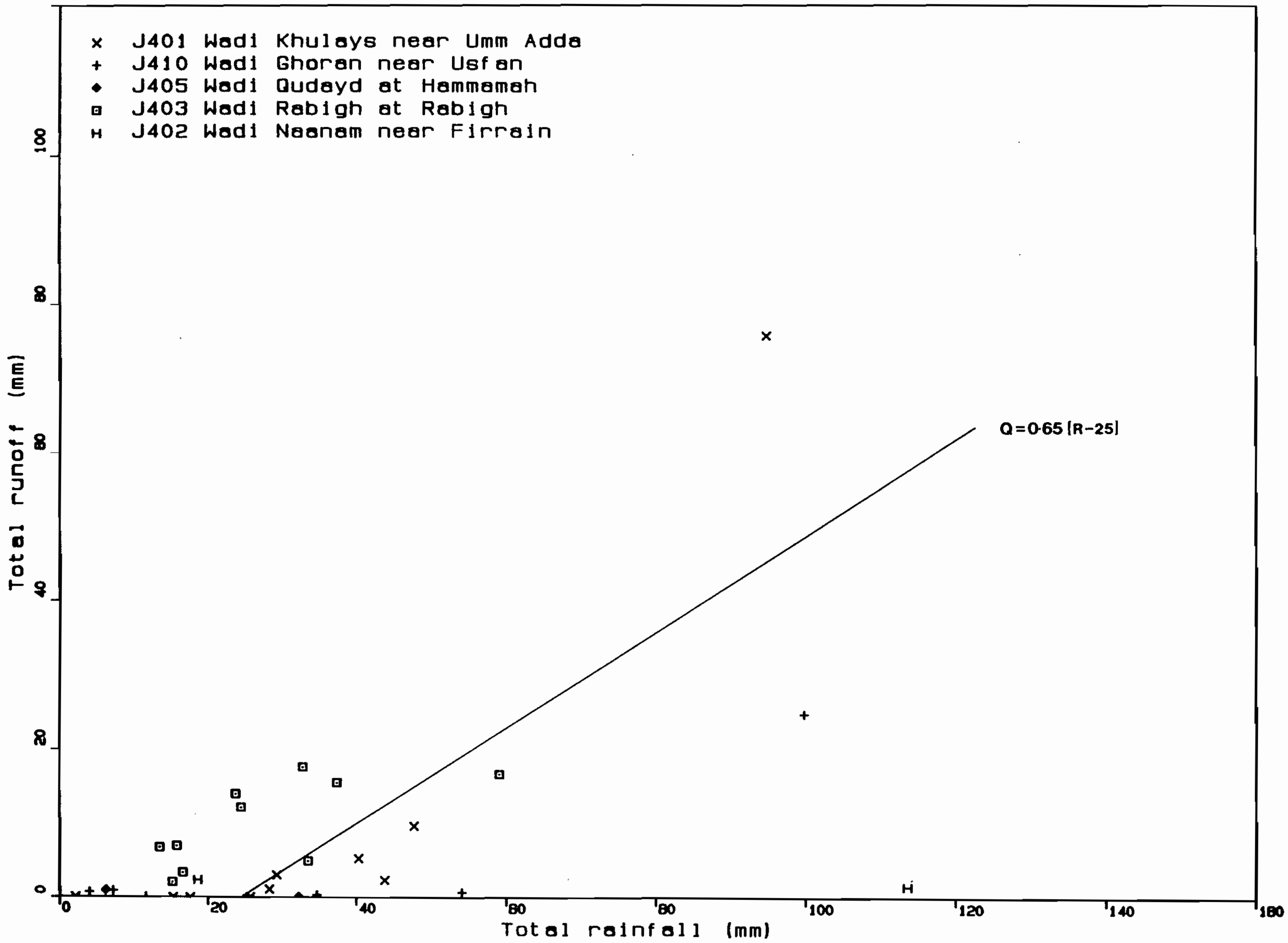


FIGURE 3.6

raingauges. However the many points close to the total rainfall axis show that some threshold rainfall must be exceeded before runoff occurs (ie certain catchment losses must be satisfied before runoff occurs). For Wadi Khulays this threshold is lower than for Wadi Ghoran but greater than that for Wadi Rabigh. The other wadis have too few events to draw any conclusions. After the threshold rainfall has been exceeded there is a tendency for an increase in runoff from an increase in rainfall. The relationship is not well defined, particularly at higher rainfall. Nevertheless a single straight line was drawn through these data, acknowledging the threshold effect, the increase in runoff with rainfall above the threshold, and the fact that data from Wadi Khulays is probably more accurate than the rest because of the greater number of raingauges contributing to the estimates of catchment rainfall. The line has the form:

$$q = 0 \qquad (r < 25)$$
$$q = 0.65 \times (r - 25) \qquad (r > 25)$$

where,

q = total storm runoff (mm)
r = total storm rainfall (mm)

The relationship implies that no runoff occurs unless there is a catchment rainfall of 25 mm or more. For the catchments studied this requires a rainfall of between 2 and 5 years return period. This is in agreement with the known fact that the wadis flood only every few years. The 25 mm threshold therefore appears reasonable on physical grounds. For the range of events considered, percentage runoff rises to a maximum of just over 50% for the most severe storms. This does not seem unreasonable when the topography is considered. A very high percentage runoff would be expected from the impervious, barren rocky mountains, combined with higher losses in the sandy gravel wadi bed.

There are various ways in which the losses could be subtracted from the total catchment rainfall; a steady percentage subtracted from all rainfall ordinates, a decreasing loss rate through the storm or an initial loss followed by a steady percentage loss thereafter. The last of the three options was considered most appropraite to the conditions in the Jeddah region.

3.4 Unit hydrograph

3.4.1 Time to peak

In the absence of any specific rainfall and flood event data to derive catchment unit hydrographs, synthetic unit hydrographs were produced for all catchments.

By definition a unit hydrograph's volume is fixed since it is equal to unit input of rainfall. An important parameter defining the shape of the unit hydrograph is the time to peak Tp. Although many formulae exist worldwide for estimating Tp from catchment characteristics (Packman, 1980), they should only be used to estimate Tp on catchments which are typical of the region from which they were derived. It is unwise to transport formulae such as these from one region to another without checking or modifying the estimate of Tp to suit local conditions. This is particularly true in this case where the catchments are some of the most barren and arid anywhere in the world.

Linsley, Kohler and Paulhus (1975), give a formula for general use which may be calibrated using local data:

$$Tp = C_t \left(\frac{L\,L_c}{\sqrt{S}}\right)^n$$

where,

L = total length of main river (miles)

L_c = stream length to centre of area of catchment (miles)

S = river slope (feet/mile)

n = exponent

C_t = constant

From a study of US catchments, some in the drier parts of the US it was reasonable to adopt a fixed exponent, n, of 0.38. The coefficient C_t did, however vary according to type of basin.

The nearest data available to estimate C_t were from a study on Wadi Zabid in North Yemen (Green, 1982). The average lag time of this catchment (ie the time between the centre of rainfall and peak flow) was shown to be 6.5 hours.

The UK Flood Study Report showed that there is a fairly good correlation between catchment lag, T_L, and unit hydrograph time to peak. The relationship is:

$$Tp = 0.9 \times T_L$$

The Wadi Zabid time to peak was therefore estimated to be:

$$Tp = 0.9 \times 6.5$$
$$Tp = 5.85 \text{ hours}$$

Catchment characteristics, L, Lc, and S were abstracted from the maps and hence C_t calculated as 0.684. Our locally calibrated formula for estimating Tp becomes:

$$Tp = 0.684 \left(\frac{L.Lc}{\sqrt{s}}\right)^{0.38}$$

Although Ct was estimated from just one catchment it was considered preferable to use the above formula to estimate Tp than use a formula derived from elsewhere in the world where assumptions would have been greater. The topography of the Wadi Zabid catchment is similar to that around Jeddah (steep barren mountains, sandy or gravel wadi bed). The increased height of mountains in Wadi Zabid is taken account of by the slope term when deriving Ct.

3.4.2 Shape of the unit hydrograph

It is usual, when using a synthetic unit hydrograph, to assume a triangular shape. This is not only for reasons of simplicity but it also approximates reasonably well with unit hydrographs derived from individual storm events.

Having fixed the volume and the time to peak, only one more dimension is required to fully define a triangular unit hydrograph. In the absence of any local information to complete the definition, the UK Flood Study relationship between the unit hydrograph time to peak and base length, T_B, was used (NERC, 1975):

$$T_B = 2.525 \times Tp$$

3.5 Using the method

Design rainstorms were derived for all catchments and for return periods 5, 10, 20, 50 and 100 years using the procedures described above.

Net rainfall for each storm was then obtained from the rainfall/runoff relationship derived in Section 3.3.

Unit hydrographs were derived for each catchment having abstracted the necessary catchment characteristics from maps.

Convolution of the net rainfall profiles with the unit hydrograph gave unit hydrograph estimates of the wadi floods. Table 3.4 shows the derivation of one such flood hydrograph; the 100 year flood on the combined Wadis B & C.

It should be noted that it has been assumed that there is no flow in the wadis before the flood event (zero baseflow). This is reasonable considering the wadis are often dry for many years at a time.

The full set of results is presented and discussed in section 4 below.

Convolution of unit hydrograph and net rainfall profile

Traingular unit hydrograph computed from Tp= 1.50

Area (sq.km)	98.80		Total rain (mm)	78.39
Data interval (hr)	0.25		Percentage runoff	44.27
Design duration (hr)	18.25		Baseflow	0.00

Time	Total Rain mm	Net Rain mm	Unit Hydrograph ordinate	Total Hydrograph cumecs
0.00	0.33	0.00	0.00	0.00
0.25	0.33	0.00	24.44	0.00
0.50	0.34	0.00	48.89	0.00
0.75	0.34	0.00	73.33	0.00
1.00	0.35	0.00	97.78	0.00
1.25	0.35	0.00	122.22	0.00
1.50	0.36	0.00	146.67	0.00
1.75	0.37	0.00	130.69	0.00
2.00	0.37	0.00	114.71	0.00
2.25	0.38	0.00	98.74	0.00
2.50	0.39	0.00	82.76	0.00
2.75	0.40	0.00	66.78	0.00
3.00	0.33	0.00	50.81	0.00
3.25	0.27	0.00	34.83	0.00
3.50	0.27	0.00	18.85	0.00
3.75	0.28	0.00	2.88	0.00
4.00	0.29	0.00		0.00
4.25	0.30	0.00		0.00
4.50	0.31	0.00		0.00
4.75	0.32	0.00		0.00
5.00	0.33	0.00		0.00
5.25	0.34	0.00		0.00
5.50	0.36	0.00		0.00
5.75	0.38	0.00		0.00
6.00	0.39	0.00		0.00
6.25	0.42	0.00		0.00
6.50	0.44	0.00		0.00
6.75	0.47	0.00		0.00
7.00	0.51	0.00		0.00
7.25	0.56	0.00		0.00
7.50	0.85	0.00		0.00
7.75	1.21	0.00		0.00
8.00	1.40	0.00		0.00
8.25	1.71	0.00		0.00
8.50	4.29	0.00		0.00
8.75	8.06	2.21		0.00
9.00	21.61	14.04		5.33
9.25	8.06	5.24		44.58
9.50	4.29	2.79		96.47
9.75	1.71	1.11		155.10
10.00	1.40	0.91		216.43
10.25	1.21	0.78		279.96
10.50	0.85	0.55		336.57
10.75	0.56	0.36		338.43 -Peak-
11.00	0.51	0.33		320.25
11.25	0.47	0.31		291.73
11.50	0.44	0.29		259.50
11.75	0.42	0.27		224.32
12.00	0.39	0.26		186.66
12.25	0.38	0.24		147.42
12.50	0.36	0.23		107.32
12.75	0.34	0.22		69.31
13.00	0.33	0.21		49.42
13.25	0.32	0.21		39.66
13.50	0.31	0.20		34.42
13.75	0.30	0.19		30.87
14.00	0.29	0.19		28.32
14.25	0.28	0.18		26.56
14.50	0.27	0.18		25.29
14.75	0.27	0.17		24.21
15.00	0.33	0.22		23.26
15.25	0.40	0.26		22.53
15.50	0.39	0.25		22.11
15.75	0.38	0.25		21.99
16.00	0.37	0.24		22.15
16.25	0.37	0.24		22.57
16.50	0.36	0.23		23.26
16.75	0.35	0.23		24.00
17.00	0.35	0.23		24.60
17.25	0.34	0.22		25.06
17.50	0.34	0.22		25.38
17.75	0.33	0.21		25.57
18.00	0.33	0.21		25.62
18.25				25.54
18.50				24.82
18.75				23.48
19.00				21.57
19.25				19.19
19.50				16.36
19.75				13.07
20.00				10.17
20.25				7.65
20.50				5.49
20.75				3.70
21.00				2.27
21.25				1.19
21.50				0.46
21.75				0.06

Total flood volume 3439137 cubic metres

Table 3.4 Convolution of rainstorm and unit hydrograph (100 year event on Wadi B & C)

4. RESULTS

4.1 Introduction

Floods of 5, 10, 20, 50 and 100 year return period were estimated on all wadis by both the regional flood study method described in Section 2 and the unit hydrograph/losses method described in Section 3. These results are summarized in Table 4.1 for the small catchments close to Jeddah, and in Table 4.2 for Wadi Fatima.

4.2 Small catchments

Considering the many assumptions and approximations made during the flood estimation procedures, particularly in the unit hydrograph analysis, the results presented in Table 4.1 for both methods show acceptable agreement.

The group of smaller wadis to the north comprise Wadi C, Wadis B & C, and Wadis A & B & C. Over the range of return periods no one method consistently produces higher or lower flood peaks. However the unit hydrograph method does give consistently higher estimates of peak flows on the southern group of small wadis (Wadis D, E, F and G). The average over prediction at 5 year return periods is 30% and 40% at 100 year return period. Nevertheless these figures are within the range of errors to be expected with both flood estimation procedures. Furthermore when these flow peaks are converted to levels for channel design, the relative differences will diminish.

It is recommended that the average of the peak discharges estimated by the regional flood study and unit hydrograph method be taken for design purposes. For example the 100 year flood peak on Wadi B & C is:

$$Q_{100} = \frac{338.4 + 347.8}{2} = 343.1 \ m^3s^{-1}$$

If flood volume is required, then the figure provided by the unit hydrograph analysis may be used (the regional flood study does not give volume). Therefore the 100 year flood volume on Wadi B & C is 3.44 million cubic metres.

JEDDAH STORMWATER PHASE II - FLOOD ESTIMATES

Catchment Name	AREA sq km	MSL km: mi	SLOPE m/km: ft/mi	S1085 m/km: ft/mi	Lca km: mi	Tp hours	D hours	Q 5	Q 10	Q 20	Q 50	Q 100
Wadi C	39.3	12.4: 7.71	5.89: 31.0	5.99: 31.6	7.2: 4.47	1.25	15	35.6 u 39.6 r 0.36 v	70.8 u 65.0 r 0.63 v	104.5 u 93.5 r 0.88 v	147.9 u 141.0 r 1.20 v	180.6 u 179.0 r 1.45 v
Wadi B & C	98.8	17.4: 10.8	8.02: 42.4	7.51: 39.7	9.1: 5.66	1.5	18	57.8 u 76.9 r 0.80 v	125.2 u 126.1 r 1.44 v	191.9 u 181.6 r 2.06 v	277.7 u 273.9 r 2.84 v	338.4 u 347.8 r 3.44 v
Wadi A & B & C	173.5	18.2: 11.3	6.22: 32.9	5.55: 29.4	9.3: 5.78	1.75	21	76.0 u 115.4 r 1.35 v	178.1 u 189.3 r 2.44 v	277.8 u 272.4 r 3.50 v	406.8 u 410.9 r 4.85 v	499.4 u 521.6 r 5.88 v
Wadi D	7.93	3.26: 2.05	9.51: 50.0	9.37: 49.3	1.35: 0.84	0.5	6	17.7 u 12.5 r 0.072 v	32.1 u 20.5 r 0.13 v	46.0 u 29.5 r 0.18 v	65.4 u 44.6 r 0.24 v	80.2 u 56.6 r 0.29 v
Wadi E	17.7	8.1: 5.03	14.2: 75.1	10.4: 55.1	4.0: 2.49	0.75	9	23.4 u 22.3 r 0.15 v	45.1 u 36.6 r 0.27 v	67.1 u 52.6 r 0.38 v	95.1 u 79.4 r 0.52 v	115.6 u 100.8 r 0.63 v
Wadi F	15.5	7.0: 4.35	21.4: 113.0	13.4: 70.5	2.6: 1.62	0.5	6	27.6 u 20.3 r 0.12 v	53.9 u 33.3 r 0.22 v	79.3 u 47.9 r 0.31 v	112.4 u 72.2 r 0.43 v	141.1 u 91.6 r 0.52 v
Wadi G	10.95	5.5: 3.42	16.4: 86.4	17.2: 90.4	2.4: 1.49	0.5	6	22.1 u 15.8 r 0.091 v	41.3 u 25.9 r 0.16 v	59.9 u 37.3 r 0.23 v	85.0 u 56.2 r 0.32 v	105.7 u 71.3 r 0.39 v

Key:
u = unit hydrograph estimate of peak (cumecs)
r = regional analysis " " " "
v = flood volume (million cubic metres)
MSL = main stream length
D = design duration
Lca = length of stream to centre of area

Table 4.1 Results for small catchments

JEDDAH STORMWATER PHASE II - FLOOD ESTIMATES

Catchment Name	AREA sq km	MSL km:mi	SLOPE m/km: ft/mi	S1085 m/km: ft/mi	Lca km: mi	Tp hours	D hours	Q 5	Q 10	Q 20	Q 50	Q 100
Fatima to Dam	3033	129.5: 80.5	13.3: 70.0	12.6: 66.3	49.4: 30.7	6.0	72	459.0 u 905.5 r 40.6 v	1052 u 1485 r 65.6 v	1647 u 2137 r 90.7 v	2281 u 3224 r 122.3 v	2632 u 4093 r 153.0 v
Fatima to Bridge	3606	150.0: 93.2	11.8: 62.3	10.9: 57.3	62.6: 38.9	7.0	84	565 u 1026 r 55.0 v	1029 u 1682 r 86.9 v	1817 u 2421 r 119.0 v	2374 u 3653 r 159.3 v	2695 u 4638 r 199.8 v
Fatima to End (Natural)	4597	197.2: 122.6	10.0: 52.5	8.9: 46.9	98.9: 61.5	10.0	120	821 u 1222 r 98.1 v	1451 u 2004 r 147.7 v	1860 u 2884 r 198.0 v	2259 u 4350 r 260.8 v	2564 u 5523 r 328.3 v
Fatima between Dam and End	1564	92.5: 57.5	6.8: 36.1	4.3: 22.9	43.6: 27.1	6.0	72	341.0 u 562.1 r 24.8 v	688.0 u 921.8 r 38.8 v	1018 u 1327 r 52.8 v	1353 u 2001 r 70.4 v	1543 u 2541 r 87.6 v

Key:
- u = unit hydrograph estimate of peak (cumecs)
- r = regional analysis " " " "
- v = flood volume (million cubic metres)
- MSL = main stream length
- D = design duration
- Lca = length of stream to centre of area

Table 4.2 Results for Wadi Fatima

Appendix A gives the full hydrographs for these wadis as obtained by unit hydrograph analyses. These hydrographs have been modified so that their volumes remain the same as the original prediction but the peak has been adjusted as described above. This was accomplished by revising the original hydrograph in one of two ways depending on whether the adjusted peak was higher or lower than the original unit hydrograph estimate:

(1) If the adjusted peak was higher than the original, ordinates may be removed from the tail or recession limb and redistributed around the peak.

(2) If the adjusted peak was lower than the original, ordinates around the peak were scaled down and the surplus volume added to the hydrograph recession.

These final design hydrographs therefore account for the revised estimate of peak, but retain their original volume.

4.3 <u>Wadi Fatima</u>

Table 4.2 summarises the flood estimates produced by both methods for various sub-divisions of Wadi Fatima:

(1) 'Wadi Fatima to Dam' is the catchment above Abu Husani Dam. Abu Hasani Dam has recently been constructed on Wadi Fatima for the purpose of groundwater recharge. The location is shown on Figure 1.2. The concrete structure is approximately 500 m long and 15 m deep. Although it is reported to have a storage capacity of 20 million cubic metres, this is likely to be reduced in time by the large sediment load carried by the mountain wadis. The effect of the dam will be to reduce flooding downstream. The amount of reduction will largely depend on the ratio of flood volume to dam storage capacity.

(2) 'Fatima to bridge' is the catchment above the road bridge at Umm Hamla (see Figure 1.2). This is the bridge for which estimates of floods in 1975 and 1979 are available (Section 1.3) and as such provides a check on flood estimates produced by both methods at this point.

(3) 'Fatima to End' is the entire Wadi Fatima catchment as far as the mountain front. The results given are for the natural catchment before the construction of Abu Hasani Dam.

(4) The results for the catchment described as 'Fatima between Dam and End' represent flooding produced by the catchment downstream of Abu Hasani dam alone.

The regional flood study gives flood estimates which are between 28% and 215% higher than those produced by the unit hydrograph method. The discrepancies are largest at higher return periods. Possible reasons for the differences are:

(1) The regional flood study considers only catchment area as the parameter in estimating flood peak. Other factors such as local climatic differences, catchment slope and geology are not included. The absence of such factors from this appraoch may contribute to the higher estimates of peak flow on Wadi Fatima that this method suggests.

(2) The unit hydrograph method requires the following assumptions or approximations which may be in error either alone or in combination:

- local depth/duration/frequency curves
- local estimate of the standardising rainfall R^1_5
- areal reduction factors
- storm profile
- storm duration
- rainfall losses
- relationship between rainfall and flooding return period.

The uncertainties associated with these elements of the unit hydrograph analysis have been discussed in Section 3.

Both methods therefore have their uncertainties, drawbacks and advantages. It is not possible to say one method is right and one method wrong.

In order to resolve these differences use was made of the floods recorded at the Umm Hamla road bridge and discussed in Section 1.4. This is the only point on Wadi Fatima for which we have any information of 'real' floods.

Figure 4.1 shows the flood frequency curve for Wadi Fatima at Umm Hamla road bridge derived from the regional flood study. The flood estimates from the unit hydrograph study are shown also, but no line drawn between them because of uneveness of increase in flood peak with return period. (The uneveness stems from the rainfall depth/duration/frequency analysis).

The estimates of the 1975 and 1979 floods calculated in Section 1.4 are also shown on Figure 4.1 and have been plotted assuming that they are the largest and second largest floods (respectively) in the 9 years 1975 to 1984. The floods have been plotted as lines representing zero to moderate scour of the sandy bed material beneath the bridge. Moderate scour was taken as 0.8 m in 1975 and 0.5 m in 1979 (Section 1.4). Although the 'moderate' scour condition has been chosen somewhat subjectively it is considered reasonable and more likely to be closer to the true situation than assuming no scour at all. These recorded floods suggest:

(1) The flood estimates produced by both methods are of the right order

(2) The general slope of the flood frequency curve is reasonable

(3) That the 'true' Wadi Fatima flood frequency curve is probably somewhere between the regional flood study and unit hydrograph estimates.

Figure 4.1 also shows an increase in discrepancy between the regional flood study and unit hydrograph estimates above 20 years return period. However it must be remembered that there is increased uncertainty in both estimation procedures at higher return periods. Record lengths of no more than 21 years were used in the regional flood study method. Extrapolation to higher return periods was based on just the five highest floods recorded in the Red Sea region. The maximum rainfall data record length used in the unit hydrograph analysis was 14 years. Although the station year approach was used to extend the effective record length, there must be some uncertainty about the high return period rainfalls, particularly as raingauges were blocked in some severe storms (Section 3.2.1). For these reasons a larger discrepancy between flood estimated by the two methods would be expected at high return periods than for the more common floods.

Flood estimates for Wadı Fatima at Umm Hamla bridge

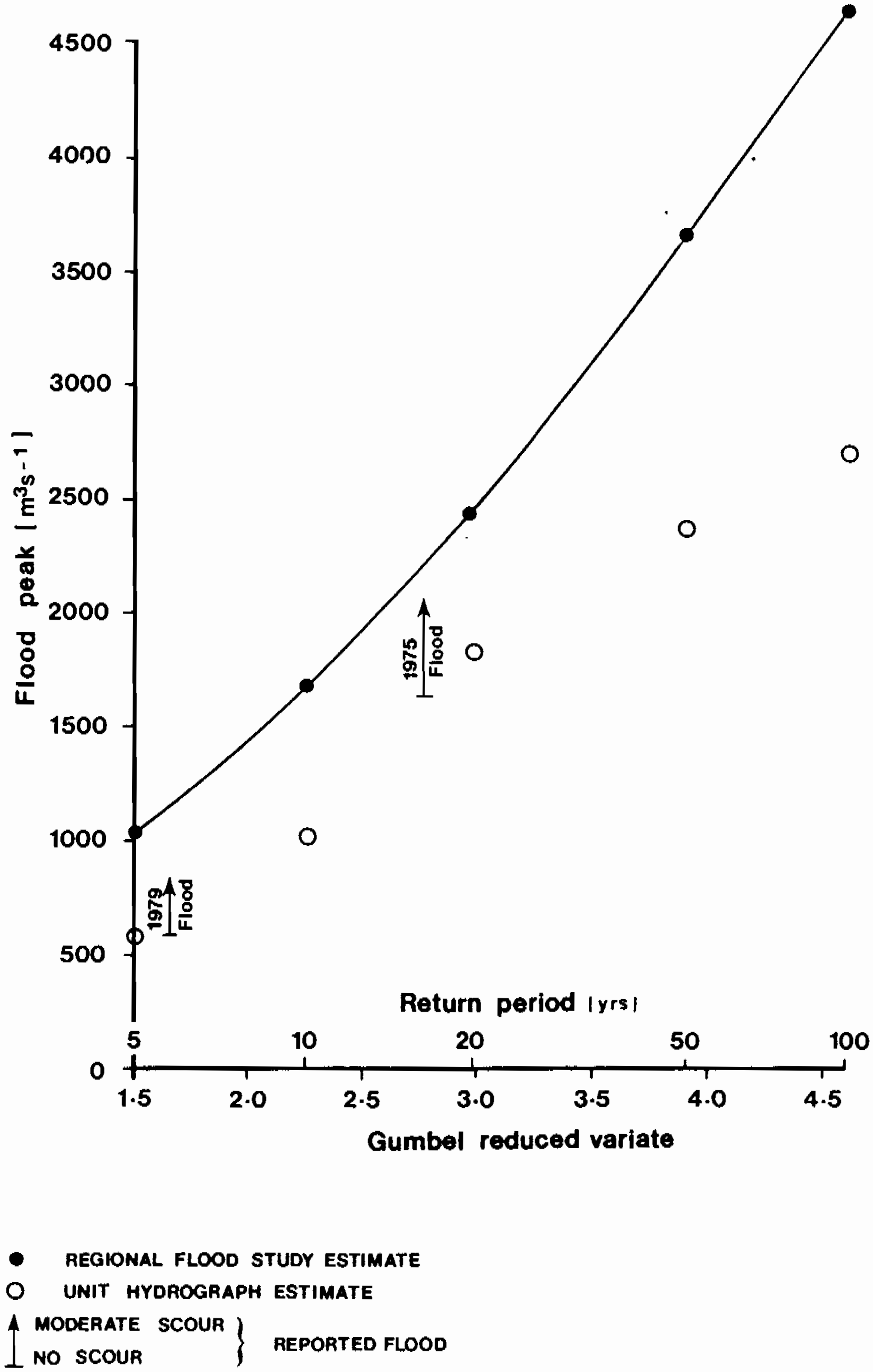


Figure 4.1

The effect of Abu Hasani dam is the one remaining problem. However its maximum storage volume of 20 million cubic metres is small compared to the total volumetric runoff of both 98 million cubic metres at 5 years return period and in particular 328 million cubic metres at 100 years return period. There is also a strong likelihood that the 20 million cubic metres storage will be reduced significantly by sedimentation. It is suggested therefore that the effect of Abu Hasani dam on the flood estimates will be small, particularly at high return periods.

Considering both the recorded floods and the small influence of the Abu Hasani dam, it is recommended that the average of the peak discharges estimated by the regional flood study and unit hydrograph method be taken for design purposes.

The 100 year design flood for the Wadi Fatima catchment is therefore

$$Q_{100} = \frac{2564 + 5523}{2} = 4044 \ m^3 s^{-1}$$

In common with the small catchments discussed earlier, flood volume provided by the unit hydrograph analysis should be used for design. The full hydrographs given in Appendix A have been modified to incorporate the adjusted peak discharge by removing ordinates from the tail or recession limb of the hydrograph and redistributing them around the peak.

5. CONCLUSIONS AND RECOMMENDATION

The flood estimates provided may be used with reasonable confidence since they are confirmed by two independent sources of data. The regional flood study uses the instantaneous peak flow data whereas the unit hydrograph uses local rainfall data. The agreement is at its best on the small mountain catchments close to Jeddah.

The regional flood study gives higher estimates of peak flow on Wadi Fatima than the unit hydrograph but independent observations of floods on the wadi show that both estimates are of the right order and that the 'true' flood frequency curve probably lies between the two.

For design purposes it is recommended that the peak discharge be taken as the average of the two estimates both on the small wadis and Wadi Fatima.

This study has advanced the understanding of the hydrology of the region considerably. Any future study may consider the following worthwhile to pursue:

(1) In the regional analysis, other factors such as various measures of rainfall (eg R_5^1), catchment slope or geology may be found significant in estimating Q_5 along side catchment area.

(2) In the unit hydrograph method, local variation of R_5^1 requires closer investigation as do areal reduction factors and storm profiles.

(3) Synthetic unit hydrographs were used in this study. A study of individual flood events and recording raingauge data would help define more realistic unit hydrograph shapes.

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APPENDIX

UNIT HYDROGRAPH FLOOD ESTIMATES

Design hydrographs for Wadi C

Time from start of rain (hours)	Q 5	Q 10	Q 20	Q 50	Q 100
7.25	0.00	0.00	0.00	0.00	0.00
7.50	0.00	0.00	0.00	0.12	3.71
7.75	1.05	7.46	13.65	21.58	31.24
8.00	6.37	20.21	33.71	50.75	67.58
8.25	13.54	35.54	56.78	83.19	108.48
8.50	21.47	51.39	79.97	116.77	149.94
8.75	30.15	65.41	99.00	143.67	180.00
9.00	36.78	68.00	98.37	144.00	177.35
9.25	38.00	65.51	93.22	134.12	162.41
9.50	34.90	59.59	83.75	118.38	141.20
9.75	30.84	51.93	72.00	100.27	117.59
10.00	26.27	43.12	58.80	79.95	92.01
10.25	21.30	33.09	43.89	57.68	64.84
10.50	15.96	22.23	27.76	34.57	38.80
10.75	11.18	15.25	18.88	22.97	25.90
11.00	8.67	11.65	14.54	17.31	19.69
11.25	7.40	9.75	12.28	14.30	16.50
11.50	6.54	8.48	10.78	12.30	14.38
11.75	5.94	7.60	9.73	10.94	12.90
12.00	5.52	7.05	9.05	10.11	11.95
12.25	5.22	6.68	8.59	9.58	11.30
12.50	4.97	6.37	8.21	9.15	10.76
12.75	4.76	6.11	7.88	8.77	10.29
13.00	4.56	5.87	7.58	8.43	9.87
13.25	4.39	5.66	7.32	8.13	9.50
13.50	4.23	5.47	7.08	7.86	9.16
13.75	4.14	5.35	6.93	7.70	8.94
14.00	4.14	5.36	6.91	7.72	8.90
14.25	4.24	5.48	7.03	7.93	9.04
14.50	4.43	5.72	7.28	8.31	9.35
14.75	4.69	6.05	7.64	8.83	9.80
15.00	4.95	6.38	7.99	9.36	10.25
15.25	5.16	6.64	8.27	9.77	10.60
15.50	5.13	6.61	8.20	9.74	10.48
15.75	4.86	6.28	7.79	9.27	9.90
16.00	4.37	5.67	7.05	8.38	8.87
16.25	3.64	4.77	5.97	7.07	7.39
16.50	0.16	3.68	4.66	5.45	5.60
16.75	0.00	2.71	3.51	4.03	4.03
17.00	0.00	1.90	2.55	2.84	2.72
17.25	0.00	1.26	1.79	1.90	1.68
17.50	0.00	0.78	1.22	1.19	0.89
17.75	0.00	0.45	0.83	0.71	0.36
18.00	0.00	0.28	0.63	0.45	0.09

Flows in cubic metres per second

See section 4 of main report for rescaling of peaks

Design hydrographs for Wadis B & C

Time from start of rain (hours)	Q 5	Q 10	Q 20	Q 50	Q 100
8.75	0.00	0.00	0.00	0.00	0.00
9.00	0.00	0.00	0.00	0.00	5.33
9.25	0.00	8.90	18.25	30.14	44.58
9.50	5.35	25.39	45.71	71.35	96.47
9.75	13.38	45.61	77.77	118.41	155.10
10.00	22.53	67.43	111.75	167.97	216.43
10.25	32.58	90.74	146.32	219.04	280.88
10.50	45.27	115.26	181.77	271.62	338.85
10.75	59.42	126.00	187.00	275.00	343.00
11.00	67.00	124.04	181.51	262.92	323.45
11.25	63.15	116.38	168.99	241.82	293.56
11.50	57.79	106.57	153.32	216.93	260.42
11.75	52.19	95.05	135.47	189.17	224.77
12.00	46.48	82.04	115.47	158.82	186.66
12.25	40.16	68.18	93.86	126.74	147.42
12.50	34.35	53.82	71.64	93.97	107.32
12.75	28.18	39.03	48.87	60.58	69.31
13.00	21.69	28.62	35.40	42.78	49.42
13.25	17.78	22.96	28.56	33.94	39.66
13.50	15.66	19.85	24.85	29.06	34.42
13.75	14.20	17.74	22.33	25.78	30.87
14.00	13.12	16.23	20.52	23.46	28.32
14.25	12.35	15.21	19.27	21.95	26.56
14.50	11.76	14.48	18.36	20.90	25.29
14.75	11.26	13.87	17.60	20.03	24.21
15.00	10.82	13.33	16.93	19.26	23.26
15.25	10.49	12.93	16.41	18.70	22.53
15.50	10.33	12.73	16.13	18.44	22.11
15.75	10.32	12.73	16.08	18.47	21.99
16.00	10.46	12.91	16.24	18.79	22.15
16.25	10.74	13.26	16.59	19.38	22.57
16.50	11.16	13.79	17.15	20.22	23.26
16.75	11.60	14.36	17.74	21.11	24.00
17.00	11.97	14.82	18.22	21.85	24.60
17.25	12.26	15.18	18.60	22.44	25.06
17.50	12.47	15.45	18.87	22.87	25.38
17.75	12.60	15.62	19.03	23.15	25.57
18.00	12.66	15.70	19.09	23.29	25.62
18.25	12.65	15.68	19.05	23.28	25.54
18.50	12.31	15.26	18.53	22.67	24.82
18.75	11.65	14.44	17.55	21.47	23.48
19.00	10.70	13.27	16.16	19.75	21.57
19.25	9.52	11.80	14.42	17.59	19.19
19.50	1.43	10.06	12.34	15.03	16.36
19.75	0.00	8.04	9.94	12.05	13.07
20.00	0.00	6.26	7.81	9.42	10.17
20.25	0.00	4.70	5.97	7.13	6.65
20.50	0.00	3.38	4.39	5.18	0.00
20.75	0.00	2.19	3.08	3.56	0.00
21.00	0.00	0.00	2.03	2.26	0.00
21.25	0.00	0.00	1.24	1.28	0.00
21.50	0.00	0.00	0.70	0.61	0.00
21.75	0.00	0.00	0.41	0.25	0.00

Flows in cubic metres per second

Design hydrographs for Wadis A & B & C

Time from start of rain (hours)	Q_{5}	Q_{10}	Q_{20}	Q_{50}	Q_{100}
10.25	0.00	0.00	0.00	0.00	0.00
10.50	0.00	0.00	0.00	0.22	7.77
10.75	0.00	10.21	22.61	38.69	58.24
11.00	5.14	30.02	56.86	91.16	124.71
11.25	13.69	54.57	96.98	151.09	199.75
11.50	23.68	81.20	139.56	214.22	278.26
11.75	34.84	109.52	184.16	279.99	361.94
12.00	47.02	140.48	229.94	348.40	446.62
12.25	63.96	169.65	269.22	405.62	511.00
12.50	81.67	184.00	275.00	409.00	505.45
12.75	96.00	179.97	266.40	390.49	476.93
13.00	89.80	169.55	249.92	361.88	438.10
13.25	81.72	156.94	229.88	329.12	395.78
13.50	74.19	142.68	207.12	293.05	349.80
13.75	67.37	126.63	182.06	254.13	301.99
14.00	59.83	110.05	155.37	213.72	252.28
14.25	53.74	92.82	127.87	172.39	201.44
14.50	47.18	75.01	99.66	130.24	149.59
14.75	40.20	56.68	70.80	87.45	100.87
15.00	32.82	43.19	53.11	63.91	74.57
15.25	27.77	35.61	43.94	52.12	61.51
15.50	24.83	31.36	38.86	45.49	54.32
15.75	22.79	28.44	35.36	40.97	49.41
16.00	21.25	26.31	32.79	37.72	45.79
16.25	20.12	24.82	30.96	35.51	43.22
16.50	19.23	23.72	29.60	33.94	41.31
16.75	18.56	22.89	28.56	32.77	39.83
17.00	18.13	22.37	27.87	32.05	38.82
17.25	17.92	22.12	27.50	31.76	38.25
17.50	17.93	22.14	27.44	31.86	38.08
17.75	18.14	22.41	27.66	32.33	38.30
18.00	18.53	22.92	28.16	33.17	38.88
18.25	19.09	23.62	28.87	34.30	39.75
18.50	19.65	24.33	29.61	35.45	40.66
18.75	20.13	24.93	30.23	36.41	41.40
19.00	20.50	25.41	30.71	37.19	41.98
19.25	20.79	25.77	31.06	37.78	42.40
19.50	20.98	26.01	31.28	38.18	42.65
19.75	21.07	26.13	31.38	38.41	42.73
20.00	21.08	26.13	31.34	38.45	42.65
20.25	20.99	26.02	31.18	38.31	42.40
20.50	20.80	25.80	30.90	38.00	41.99
20.75	20.53	25.46	30.48	37.50	41.42
21.00	20.21	25.06	30.01	36.92	40.76
21.25	19.90	24.67	29.54	36.34	40.12
21.50	19.29	23.92	28.64	35.24	38.90
21.75	18.40	22.81	27.33	33.61	37.09
22.00	17.23	21.35	25.59	31.46	34.72
22.25	15.77	19.54	23.44	28.80	31.78
22.50	0.52	17.39	20.87	25.62	28.27
22.75	0.00	14.89	17.90	21.94	24.21
23.00	0.00	12.18	14.67	17.94	19.80
23.25	0.00	9.75	11.78	14.37	15.85
23.50	0.00	5.07	9.22	11.20	1.75
23.75	0.00	0.00	6.99	8.44	0.00
24.00	0.00	0.00	5.09	6.08	0.00
24.25	0.00	0.00	3.50	1.79	0.00
24.50	0.00	0.00	2.22	0.00	0.00
24.75	0.00	0.00	1.26	0.00	0.00
25.00	0.00	0.00	0.60	0.00	0.00
25.25	0.00	0.00	0.24	0.00	0.00

Flows in cubic metres per second

See section 4 of main report for rescaling of peaks

Design hydrographs for Wadi D

Time from start of rain (hours)	Q_{5}	Q_{10}	Q_{20}	Q_{50}	Q_{100}
3.00	0.00	0.00	0.00	0.00	0.00
3.25	1.99	9.16	14.96	21.89	29.57
3.50	9.61	25.04	38.00	54.29	68.00
3.75	15.00	26.00	36.30	55.00	67.72
4.00	13.83	21.27	28.13	41.54	49.79
4.25	9.32	11.91	13.94	19.72	22.80
4.50	5.12	6.88	8.55	14.03	15.88
4.75	4.29	6.61	8.65	11.49	12.41
5.00	3.66	5.77	7.76	9.29	10.04
5.25	2.97	4.62	6.43	7.40	8.17
5.50	2.53	3.92	5.61	6.27	7.03
5.75	2.31	3.59	5.22	5.74	6.47
6.00	2.19	3.44	5.04	5.54	6.21
6.25	2.10	3.33	4.89	5.38	6.01
6.50	1.78	2.96	4.43	4.85	5.36
6.75	1.28	2.34	3.65	3.96	4.26
7.00	0.94	1.93	3.15	3.38	3.55
7.25	0.78	1.73	2.89	3.10	3.19

Flows in cubic metres per second

See section 4 of main report for rescaling of peaks

Design hydrographs for Wadi E

Time from start of rain (hours)	Q 5	Q 10	Q 20	Q 50	Q 100
4.25	0.00	0.00	0.00	0.00	0.00
4.50	0.00	0.00	0.00	0.00	1.47
4.75	1.14	7.70	13.95	21.89	29.46
5.00	7.03	20.43	33.57	50.70	65.70
5.25	14.86	35.62	56.02	83.21	104.42
5.50	21.97	41.00	60.00	87.00	108.00
5.75	23.00	40.31	56.81	80.19	97.61
6.00	20.69	34.75	47.64	65.97	78.58
6.25	17.02	26.82	35.37	47.82	55.38
6.50	12.44	17.84	22.01	28.10	31.36
6.75	7.86	11.61	14.41	18.19	20.01
7.00	5.58	8.21	10.31	12.77	14.14
7.25	4.47	6.48	8.27	9.98	11.26
7.50	3.74	5.35	6.95	8.17	9.40
7.75	3.26	4.62	6.09	7.01	8.19
8.00	2.97	4.21	5.60	6.39	7.49
8.25	2.78	3.98	5.31	6.06	7.08
8.50	2.63	3.79	5.07	5.79	6.75
8.75	2.49	3.63	4.87	5.56	6.46
9.00	2.38	3.49	4.69	5.36	6.22
9.25	2.28	3.37	4.54	5.19	6.00
9.50	2.04	3.08	4.18	4.77	5.49
9.75	1.67	2.62	3.61	4.12	4.68
10.00	1.16	2.00	2.83	3.24	3.60
10.25	0.73	1.47	2.17	2.47	2.66
10.50	0.40	1.07	1.67	1.91	1.96
10.75	0.19	0.81	1.34	1.53	1.50
11.00	0.08	0.67	1.17	1.34	1.26

Flows in cubic metres per second

See section 4 of main report for rescaling of peaks

Design hydrographs for Wadi F

Time from start of rain (hours)	Q 5	Q 10	Q 20	Q 50	Q 100
3.00	0.00	0.00	0.00	0.00	0.00
3.25	0.32	13.43	24.65	35.22	46.39
3.50	11.09	39.24	64.00	89.33	114.71
3.75	24.00	44.00	62.31	92.00	116.00
4.00	23.83	37.34	49.35	71.04	87.20
4.25	17.54	22.22	25.59	34.63	40.20
4.50	9.73	12.96	15.33	25.49	29.12
4.75	7.69	11.95	15.55	22.03	24.04
5.00	6.38	10.30	13.98	18.29	19.96
5.25	5.10	8.20	11.55	14.84	16.54
5.50	4.30	6.92	10.05	12.75	14.43
5.75	3.88	6.31	9.32	11.79	13.41
6.00	3.67	6.04	8.99	11.40	12.93
6.25	3.49	5.83	8.72	11.10	12.56
6.50	2.92	5.12	7.85	10.10	11.32
6.75	1.96	3.95	6.38	8.43	9.26
7.00	1.34	3.18	5.43	7.34	7.91
7.25	1.03	2.80	4.95	6.79	7.24

Flows in cubic metres per second

See section 4 of main report for rescaling of peaks

Design hydrographs for Wadi G

Time from start of rain (hours)	Q_5	Q_{10}	Q_{20}	Q_{50}	Q_{100}
3.00	0.00	0.00	0.00	0.00	0.00
3.25	1.55	11.22	19.11	27.84	37.03
3.50	10.70	31.48	49.00	69.63	88.75
3.75	19.00	34.00	47.22	71.00	89.00
4.00	18.09	28.22	36.96	54.10	66.05
4.25	12.68	16.25	18.70	26.00	30.30
4.50	7.00	9.42	11.35	18.68	21.25
4.75	5.66	8.71	11.47	15.61	16.82
5.00	4.76	7.52	10.31	12.77	13.70
5.25	3.83	5.98	8.52	10.24	11.20
5.50	3.24	5.04	7.43	8.72	9.66
5.75	2.94	4.60	6.90	8.02	8.91
6.00	2.78	4.40	6.66	7.74	8.57
6.25	2.66	4.25	6.47	7.52	8.30
6.50	2.25	3.75	5.84	6.80	7.41
6.75	1.56	2.90	4.78	5.60	5.92
7.00	1.11	2.35	4.09	4.81	4.95
7.25	0.88	2.08	3.75	4.42	4.47

Flows in cubic metres per second

See section 4 of main report for rescaling of peaks

Design hydrographs for Wadi Fatima to End

Time from start of rain (hours)	Q 5	Q 10	Q 20	Q 50	Q 100
30.00	0.00	0.00	0.00	0.00	0.00
32.00	0.00	0.00	0.00	0.00	0.91
34.00	0.00	0.00	0.00	0.00	25.21
36.00	0.00	0.00	0.00	0.00	73.41
38.00	0.00	0.00	0.00	0.00	146.09
40.00	0.00	0.00	0.00	11.55	243.91
42.00	0.00	0.00	0.00	44.16	366.11
44.00	0.00	0.00	0.00	98.60	476.41
46.00	0.00	0.00	0.00	175.78	574.96
48.00	0.00	0.00	17.68	276.80	662.05
50.00	0.00	0.00	53.87	381.54	734.18
52.00	0.00	0.00	108.02	473.62	788.07
54.00	0.00	0.00	182.05	554.10	824.94
56.00	0.00	12.68	275.38	620.23	842.73
58.00	0.00	41.58	359.39	670.28	846.07
60.00	0.00	99.09	447.55	724.95	869.34
62.00	131.17	359.91	748.88	1038.19	1200.21
64.00	284.22	649.35	1052.05	1355.91	1550.73
66.00	490.70	989.84	1446.25	1878.99	2199.90
68.00	729.27	1346.10	1875.93	2501.21	2989.12
70.00	1022.00	1728.00	2372.00	3305.00	4044.00
72.00	949.47	1567.17	2079.32	2820.82	3407.37
74.00	852.53	1373.90	1758.79	2303.42	2733.59
76.00	766.63	1197.79	1488.89	1893.02	2211.21
78.00	690.30	1036.79	1266.81	1582.45	1831.49
80.00	603.46	866.25	1043.95	1272.16	1455.77
82.00	527.08	721.12	871.87	1065.27	1229.81
84.00	441.07	577.52	702.08	860.64	1007.92
86.00	374.98	479.58	586.66	721.91	859.20
88.00	360.24	462.46	567.50	699.97	840.79
90.00	348.96	449.82	553.65	684.09	828.49
92.00	337.81	436.94	539.37	667.05	814.20
94.00	327.21	424.37	525.14	649.81	798.37
96.00	317.45	412.45	511.27	633.00	781.35
98.00	308.25	400.92	497.53	616.29	763.08
100.00	299.85	390.26	484.69	600.69	745.48
102.00	292.36	380.77	473.26	586.78	729.79
104.00	285.63	372.23	462.97	574.27	715.66
106.00	279.53	364.48	453.63	562.91	702.82
108.00	273.96	357.41	445.11	552.53	691.09
110.00	268.85	350.92	437.27	543.00	680.29
112.00	264.12	344.92	430.04	534.19	670.32
114.00	259.74	339.36	423.33	526.01	661.06
116.00	255.67	334.18	417.07	518.40	652.43
118.00	251.85	329.33	411.22	511.28	332.79
120.00	248.28	324.79	405.73	136.99	0.00
122.00	244.92	320.52	400.57	0.00	0.00
124.00	234.55	307.06	324.55	0.00	0.00
126.00	217.24	284.47	0.00	0.00	0.00
128.00	92.00	76.32	0.00	0.00	0.00

Flows in cubic metres per second

See section 4 of main report for rescaling of peaks

These estimates, however, should be used with caution, not only because of the uncertainty of the scour depth but also because of the unconfirmed evidence of the flood depths. However they do give an idea of the size of floods produced by Wadi Fatima in the last 9 years.