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Report on Mission to the Ministry of Agriculture and Fisheries Dubai - United Arab Emirates

During the period 10 - 24 March 1995

PRELIMINARY DESIGN OF A RECHARGE DAM ON WADI JAZIR

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The views expressed in this report are those of the author and do not necessarily reflect those of the United Nations Economic and Social Commission for Western Asia.

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PART I

1- INTRODUCTION

1-1 THE SCOPE OF WORK FOR THE MISSION

The scope of work for the mission to the Ministry of Agriculture and Fisheries during the period 10 - 24, March 1995 was:

1: To identify an adequate dam site on Wadi Jazir to store floodwater for later releases to recharge the groundwater reservoir down stream of the proposed dam site.

2: To conduct the required hydrological analysis in order to determine the design hydraulic parameters.

3: To prepare a preliminary design for the proposed dam and associated recharge facilities.

1-2 PROJECT SETTING

1-2-1 Location :

Wadi Jazir is a small stream which flows on the western slopes of the north-eastern mountain range of the United Arab Emirates (U.A.E).

The proposed dam site is located in the lower most reach of Wadi Jazir about 1 km. upstream of its outflow to the flood plain area.

A more favorable dam site, half kilometer downstream, with larger reservoir area was previously selected but presently neglected because of its interference with existing private properties.

1-2-2 The Drainage Area and Topography :

The total drainage area of Wadi Jazir to the proposed dam site is about 4.5 square kilometers (km^2) (Figures 1 and 2).

The upper part of the drainage area is relatively rugged and mountainous with steep slopes.

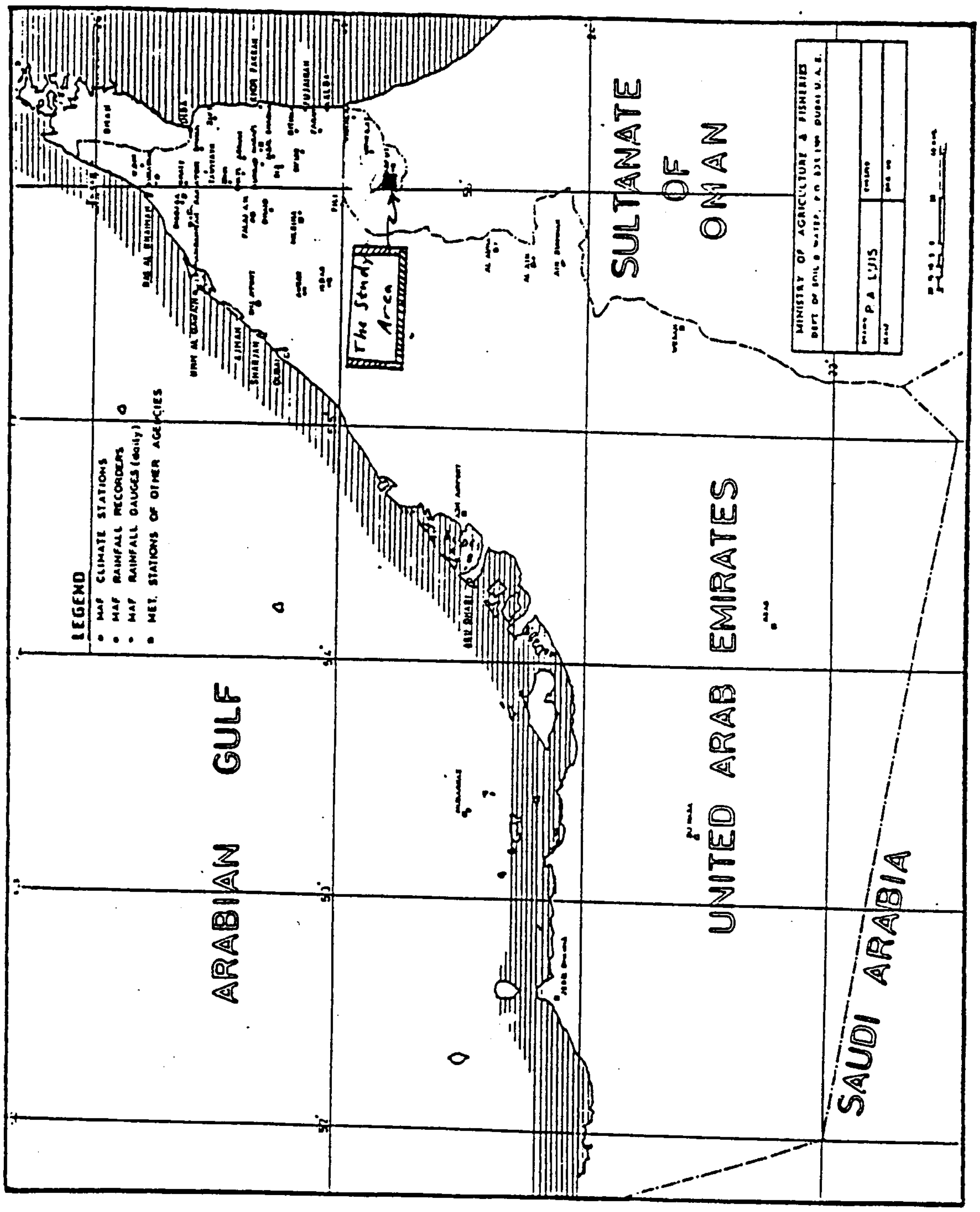


FIGURE (1) : LOCATION MAP

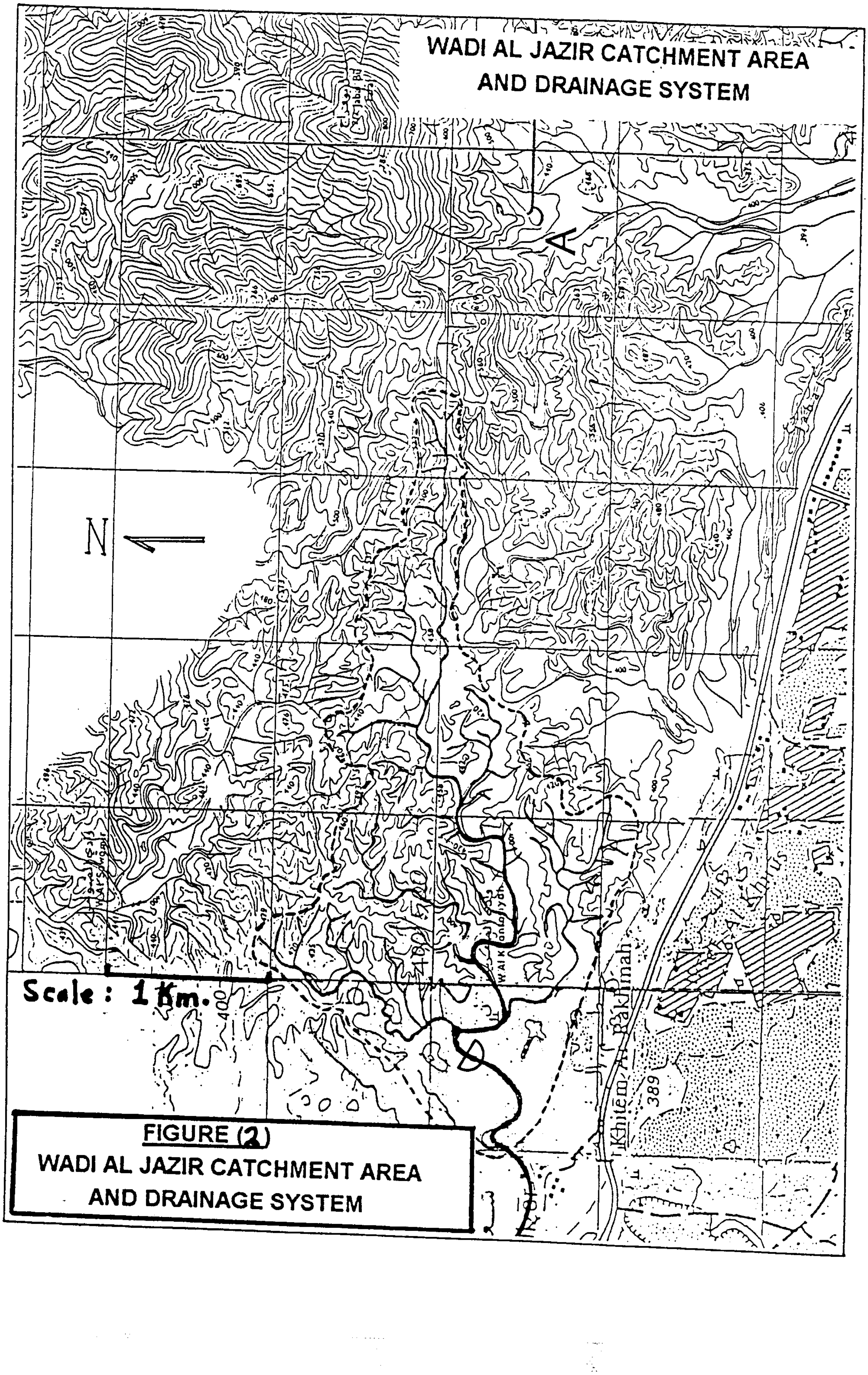


FIGURE (2)
WADI AL JAZIR CATCHMENT AREA
AND DRAINAGE SYSTEM

The main stream channel widens up gradually, and is filled with recent alluvial deposits and older terraces. Some of the topographic features for the stream and its catchment are given below.

Parameter	Measurement
Area (sq.km).	4.5
Length of main stream (km)	6.7
Highest point elevation (m.a.s.i.)	560.0
Lowest point elevation (m.a.s.i.)	+/- 320.0
Slope (%)	3.58

Wadi Jazir is an ephemeral stream where flood flows occur only in response to heavy rainstorms over its catchment. No springs or base flow exist within the catchment area. However the alluvial deposits in the channel down stream of the proposed dam site form a good but local and a shallow water table aquifer which is recharged by direct infiltration of flood water. A water supply wellfield exists in Wadi Jazir half kilometer down stream of the proposed dam site.

Rainfall over the area on the eastern mountains of the United Arab Emirate (U.A.E) is generally higher than on the rest of the country. It ranges from 120-160 mm/yr on the long term (Figure 3). Most of the rainfall in the area occurs from January through April (Figure 4). It occurs mostly as thunder storms with relatively high intensities and short durations, resulting in high flash floods.

Rainfall in this area is characterized by high variability with time and space, seasonally and annually. Figure (5) shows alternating 3 to 5 year cycle of wet and dry periods. The year 1987-88 had abnormally high rainfall and was the highest since 1967.

1-2-3 **Geology** :

The Wadi Jazir catchment lies in the pre-cambrian to Upper Cretaceous allochthonous beds of the Samuel ophiolite Suite. The rocks consist of coarse-grained gabbros and ultrabasic peridotite and serpentinized peridotite and serpentine.

No significant faults were observed in the vicinity of the proposed dam site.

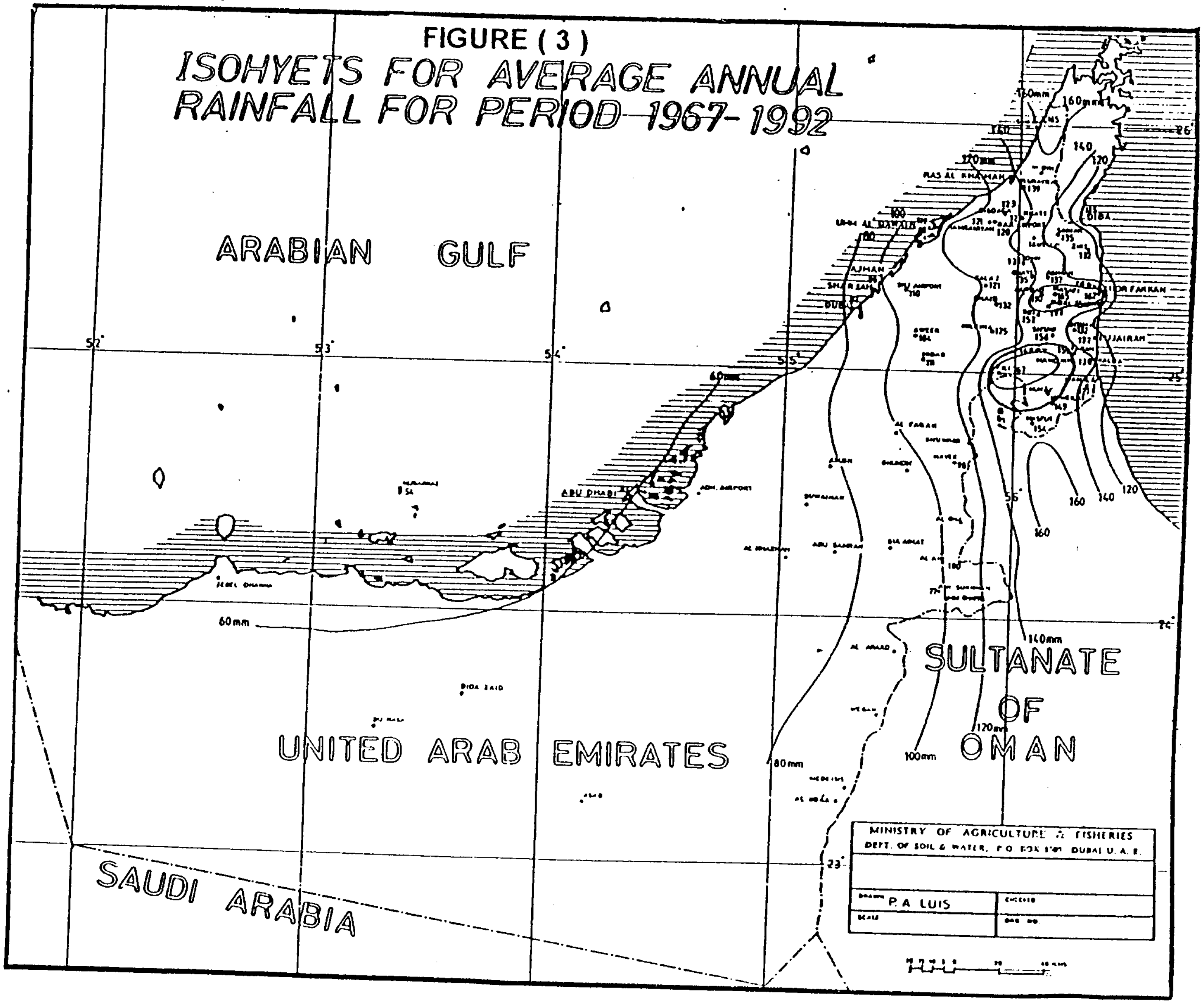


FIGURE (3)

ISOHYETS FOR AVERAGE ANNUAL RAINFALL FOR PERIOD 1967-1992

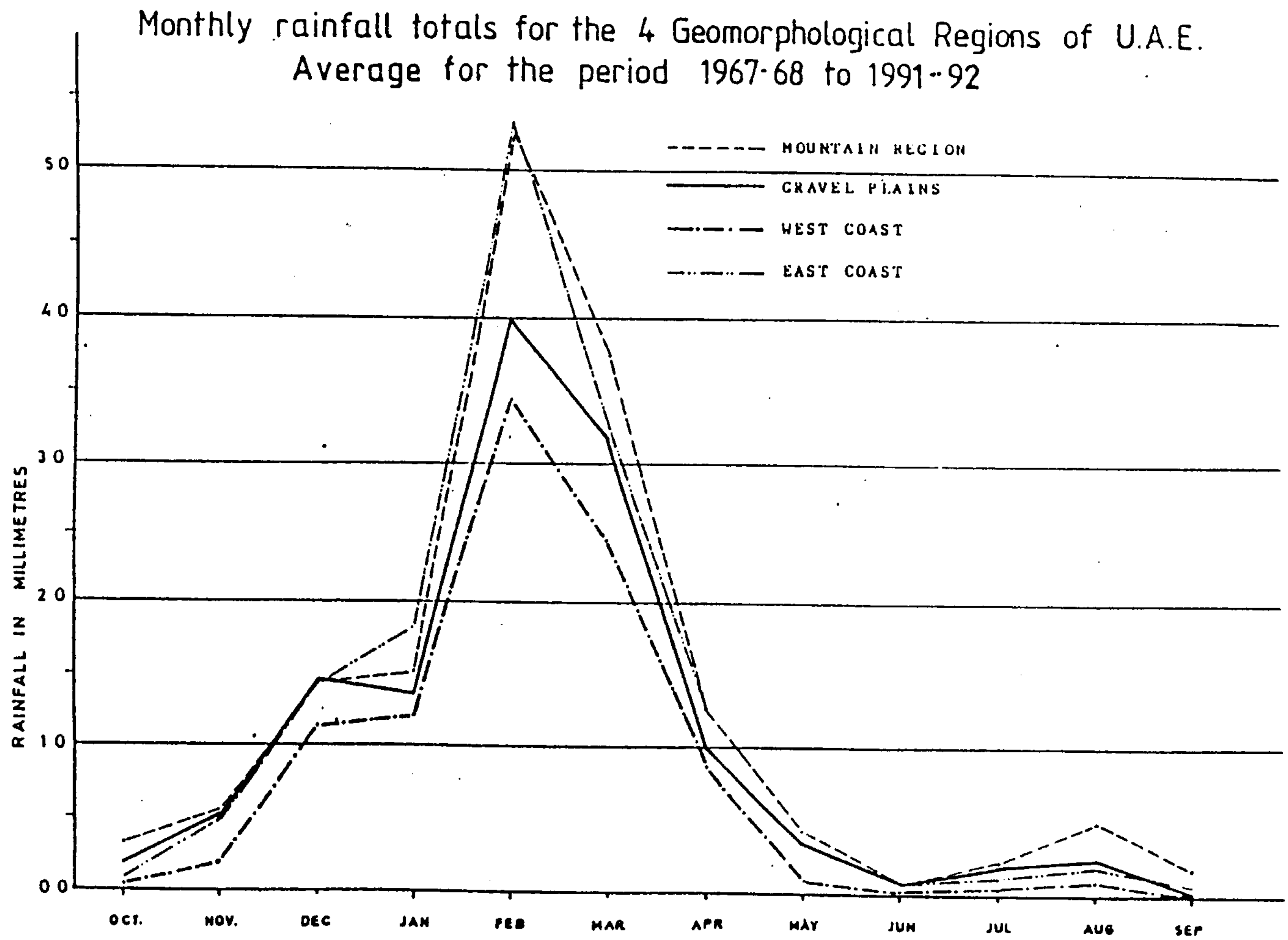

Monthly rainfall totals for the 4 Geomorphological Regions of U.A.E.
Average for the period 1967-68 to 1991-92
MOUNTAIN REGION
GRAVEL PLAINS
WEST COAST
EAST COAST
RAINFALL IN MILLIMETRES
50
40
30
20
10
00
OCT.
NOV.
DEC
JAN
FEB
MAR
APR
MAY
JUN
JUL
AUG
SEP

FIGURE (4)

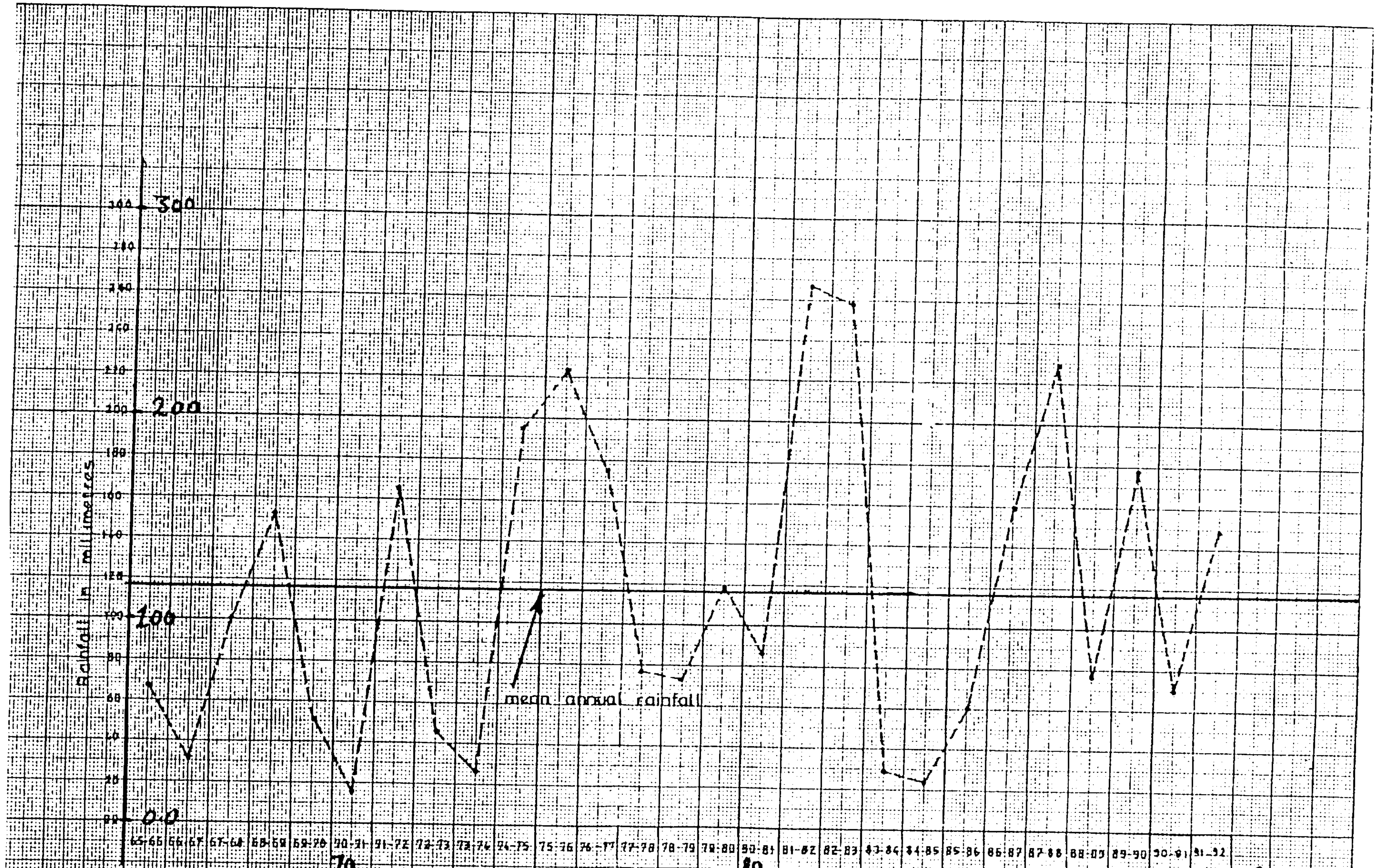


Fig 5 Average Annual Rainfall in the United Arab Emirates from 1965 to 1992

At the proposed dam site massive ultrabasic rocks exist. However, some bedding planes and joints can be seen at the surface. Quarrying of these massive rocks should yield large amounts of armor rock.

The lower most reach, of Wadi Jazir is rather flat but relatively narrow. It is filled with recent alluvium and locally some older alluvial terraces. The alluvial deposits consist of medium size gravel and course sand.

The lower terraces are moderately cemented. The recent aluminum is loose material.

The thickness of the alluvial deposits is not expected to exceed five meters at the proposed dam site in the central active channel, and increases in the wider channel down stream.

The fine portion (silt and fine sand) of these deposits does not almost exist. The percentage of fine material is expected to increase in a downstream direction.

1-2-4 **Groundwater Resources** :

Local groundwater aquifer exists in the alluvial deposits in lower Wadi Jazir.

The underlying rocks are relatively impervious, except in their upper few meters, where they have some fracturing and weathering. The degree of fracturing in these igneous rocks decreases with depth giving rise to less permeable rock. As a result no groundwater is expected at depth.

Groundwater in lower Wadi Jazir is exploited for domestic water supply in the same area.

So far the groundwater situation is still good. Building a dam in Wadi Jazir to store floods would increase the recharge into the alluvial aquifer and will augment this groundwater supply source.

The quantity as well as the quality of the groundwater will improve with the construction of such recharge dam.

Surface water resources in the study area consists mainly of the flood flows of Wadi Jazir. Because of the high variability and fluctuations of rainfall and the resulting floods, and the short durations of flood flows and their flashy nature, they are not significant for direct use. The only benefit of these floods at the present time is through recharging the groundwater in the alluvial aquifer. Utilization of this water requires controlling high floods and storing them behind dams and in the aquifer, which is the main purpose of the proposed dam.

PART II

2- *HYDROLOGICAL ANALYSIS*

2-1 GENERAL :

The purpose of the hydrologic analysis performed for this study is to provide the design parameters required for the proposed dam and its appurtenant structures. These design parameters include: The design flood (volume, peak flow and return period), storage and discharge rating relationships, reservoir sedimentation and seepage. Additional details on the hydrologic techniques applied in this study are given in the annexes at the end of this report.

The hydrologic analysis was based on 18 year rainfall records (1976-1993) for three nearby stations, Masfout, Munayi and Huwaylat and a number of nearby stations. More detailed analysis was performed for the storm of 17-18/2/1988, which was the maximum during this period.

Runoff records for Wadi Jazir are not available. Some measurements of flood flows for the nearby Wadi Qawr were used in this analysis.

2-2 RAINFALL :

The average annual area rainfall over Wadi Jazir catchment is about 150 mm. Rainfall in this area is highly variable in time and space. Figure (5) shows the variation of annual rainfall during the period 1965 to 1992.

No rainfall stations exist within the catchment of Wadi Jazir. The records from the nearby Masfout station (10 km to the east) were used in this analysis. Rainfall data from this station was used for estimating surface runoff for Wadi Jazir catchment.

The proximity of Masfout station to Wadi Jazir, and the similarity of the physiographic conditions over the area, makes rainfall records at Masfout station a good representation for Wadi Jazir catchment rainfall.

The average monthly and annual rainfalls at Masfout station are given in Annex 1. Most of the rainfall occurs during the period December to February. The probabilities of the major storms are given in Tables (1 & 2).

TABLE (1): PARTIAL SERIES OF MAXIMUM STORM RAINFALL
[MASFOUT ,MUNAYI' ,AND MASFOUT STATIONS]

MAXIMUM STORM R.F.	RANK	PROPABI	RETURN PERIOD YEARS
71	11	0.32	3.1
120	4	0.12	8.5
31	27	0.79	1.3
31	27	0.79	1.3
44	18	0.53	1.9
65	13	0.38	2.6
38	21	0.62	1.6
127	3	0.09	11.3
39	20	0.59	1.7
44	18	0.53	1.9
205	1	0.03	34.0
30	28	0.82	1.2
36	23	0.68	1.5
33	25	0.74	1.4
117	5	0.15	6.8
34	24	0.71	1.4
102	6	0.18	5.7
53	16	0.47	2.1
51	17	0.50	2.0
36	23	0.68	1.5
38	21	0.62	1.6
73	10	0.29	3.4
33	26	0.76	1.3
80	8	0.24	4.3
144	2	0.06	17.0
57	15	0.44	2.3
67	12	0.35	2.8
40	19	0.56	1.8
75	9	0.26	3.8
84	7	0.21	4.9
37	22	0.65	1.5
61	14	0.41	2.4

TABLE (2): ANNUAL SERIES OF MAXIMUM STORM RAINFALL
[MASFOUT ,MUNAYI' ,AND MASFOUT STATIONS]

YEAR	MAXIMUM STORM R.F.	RANK	PROPABI	RETURN PERIOD YEARS
1976	40.0	12	0.67	1.5
1977	84.0	7	0.39	2.6
1978	37.4	13	0.72	1.4
1979	73.0	9	0.50	2.0
1980	33.2	15	0.83	1.2
1981	80.0	8	0.44	2.3
1982	144.0	2	0.11	9.0
1983	102.0	6	0.33	3.0
1984	12.0	18	1.00	1.0
1985	20.6	17	0.94	1.1
1986	36.9	14	0.78	1.3
1987	117.0	5	0.28	3.6
1988	205.0	1	0.06	18.0
1989	65.4	11	0.61	1.6
1990	126.8	3	0.17	6.0
1991	30.8	16	0.89	1.1
1992	70.8	10	0.56	1.8
1993	120.0	4	0.22	4.5

2-3 STREAM FLOW :

Wadi Jazir is an ephemeral stream where floods occur only in response to rain storms of high intensities. No measurements of these flood flows are available for the study catchment. However, some measurements are available for the nearby Wadi Qawr. Analysis of these measurements for specific storms enabled calculations for runoff coefficients for those storms. Significant portion of flood flows infiltrate through the alluvial Wadi bed to recharge the groundwater in these sediments, particularly in the lower part of Wadi Jazir.

2-4 ESTIMATION OF SURFACE RUNOFF :

2-4-1 Methodology :

In the absence of sufficient and adequate measured surface runoff data, estimation was required. A number of methods are available. The empirical method derived by the U.S. Soil Conservation Services (SCS) is adopted for this study.

This method is widely applied in the U.S.A. and many other countries under various hydrological condition. It is also known as the surface runoff curve number method.

This method was also applied by the author in a previous similar study for Wadi Munayi in the UAE and it has been seen adequate to describe it here to present a complete report.

Rainfall excess (runoff) varies with many factors centered around the catchment and storm characteristics. The initial part of storm water is either evaporated, or absorbed by the vegetal cover and other rock surfaces, or absorbed by the soil to satisfy its moisture deficit or retained in surface depressions.

When these total losses or abstractions are fully satisfied, they approach a potential or ultimate saturation value (S'). Then subsurface runoff (infiltration and percolation) and surface runoff start. If infiltration approximates zero, then the surface runoff will be equivalent to the precipitation rate.

Rainfall excess (Q) and watershed storage (S) can be derived from:

$Q = P - S$ .... (1)

Where P is the rainfall volume. The Watershed storage (S) includes both the initial abstraction and infiltration.

At saturation, the rate of rainfall excess equals the rainfall intensity, and a proportional relationship can be developed:

$$\frac{S}{S'} = \frac{Q}{P} \quad \ldots\ldots\ldots\ldots\ldots (2)$$

Where S : Watershed storage at any time (mm)
S': Watershed storage at saturation (mm)
P : Precipitation at any time (mm)
Q : Rainfall excess (mm) (surface Run-off)

Substituting from equation (1) in equation (2) we obtain:

$$\frac{P-Q}{S'} = \frac{Q}{P} \quad \text{or}$$

$$Q = \frac{P^2}{P+S'} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The SCS method considers the potential (maximum) watershed storage of rain water as a function of the soil type. Using more than 3000 soil type, four hydrologic soil groups were obtained. Runoff curves were developed for each hydrologic group to estimate (S'), the maximum potential storage. A number (CN) was assigned for each curve which is used in the following equation:

$S' = (25400/CN) - 254$
Where S' is in mm, and CN is the curve number.

For Wadi Qawr and Wadi Jazir catchments, where urbanization is not there, and according to an empirical formula developed by the SCS, the initial abstraction (Ia) is found to equal 0.2S'. The factor (0.2) decreases for urbanized areas.

Subtracting the initial abstraction (Ia) from Precipitation (P) in equation (3) and subtracting 0.2 S' for the initial abstraction, we obtain:

$$Q = \frac{(P - I\,a)^2}{P - Ia + S'} \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

With P > Ia, S' > Ia, and F (infiltration = P - Ia - Q then,

$$Q = \frac{(P - 0.2S')^2}{P + 0.8\,S'} \quad \ldots\ldots\ldots\ldots\ldots(6)$$

This is for the case Ia = 0.2 S'.

Knowing the curve number, equations 4, 5 and 6 can be used to calculate the surface runoff.

Annex (4) gives a description of the characteristics and properties of the various hydrologic and soil groups.

Annex (5) and (6) shows the curves representing the solution to equation (6), and the runoff curve numbers for use in the SCS method.

2-5 ESTIMATION OF SURFACE RUNOFF FROM RAINFALL DATA :

In order to apply the SCS method to estimate the runoff, the following procedure was followed:

A backward solution for equation (4) and (6) was used to estimate the initial abstraction for Wadi Qawr catchment. The rainfall records and flood measurements in this catchment for the storm of 17- 18/2/1993 were used in solving these equations. The results were as follows:

* Potential initial abstraction (S') = 170 mm
* Initial Abstraction = 0.2 S' = 34 mm
* Average Curve Number = 51

These values were applied to Wadi Jazir catchment.

Before applying the equation a baseflow for Wadi Qawr of (2.3) MCM was subtracted from the total stream flow (14.3) MCM.

130 storms for 17 years (1977 to 1983) were analyzed and used in this study.

2-6 FREQUENCY ANALYSIS OF ANNUAL RUNOFF :

The frequency of storm runoff is taken as similar to the frequency of rainfall. The maximum storm rainfall for each year in Masfout, Munayi and Huwaylat and the partial series of all storms exceeding 30 mm of rain were analyzed.

The frequency of exceedance was thus calculated for the storm rainfall. The Weibull formula was used to calculate the probability of exceedance as follows:

$$P(X) = \frac{m}{n + 1}$$

Where PX is the probability of exceedance
m is the order (rank) of storm rainfall when annual Values are arranged in descending order. (i.e. m - 1 for the largest observed value)
n is the number of observations

The return period (Tr) in years is then obtained, and plotted against storm rainfall on the arithmetic scale (Figure 6). The following linear relationship was then obtained:

$$RF = 18 + 80 * \text{Log}(TR)$$

Where RF is the storm rainfall in mm
RT is the return period in years.

The results of the probability analysis are shown in Tables (1, 2 & 3).

2-7 **CALCULATION OF RUNOFF COEFFICIENTS** :

The runoff coefficient (C) represents the percentage for rainfall which appears as surface runoff. The average runoff coefficient was calculated in a previous study as 28 % for the near-by Wadi Munayi catchment.

2-8 **DESIGN FLOOD** :

2-8-1 **Determination of Design Flood** :

It is a common practice to determine a flood hydrography or peak discharge that is used as a basis for the design capacity of hydraulic structures. The design flood may be a flood with a specified return period, or it may be a probable maximum flood resulting from a probable maximum precipitation.

- The Probable Maximum Precipitation (PMP), is the theoretical greatest depth of precipitation for a given duration, meteorologically possible, for a given basin at a particular time of the year.

TABLE (3):PROBABILITY OF MAXIMUM ANNUAL STORMS ON THE EASTERN MOUNTAINS...BASED ON THE RELATION : RF=18+80*LOG (TR)

RETURN PERIOD YEARS	STORM RAINFALL M.M.
2.33	47
5	74
10	98
25	130
50	154
100	178
500	234
1000	258
10^4	338
10^5	418
10^6	498
10^7	578

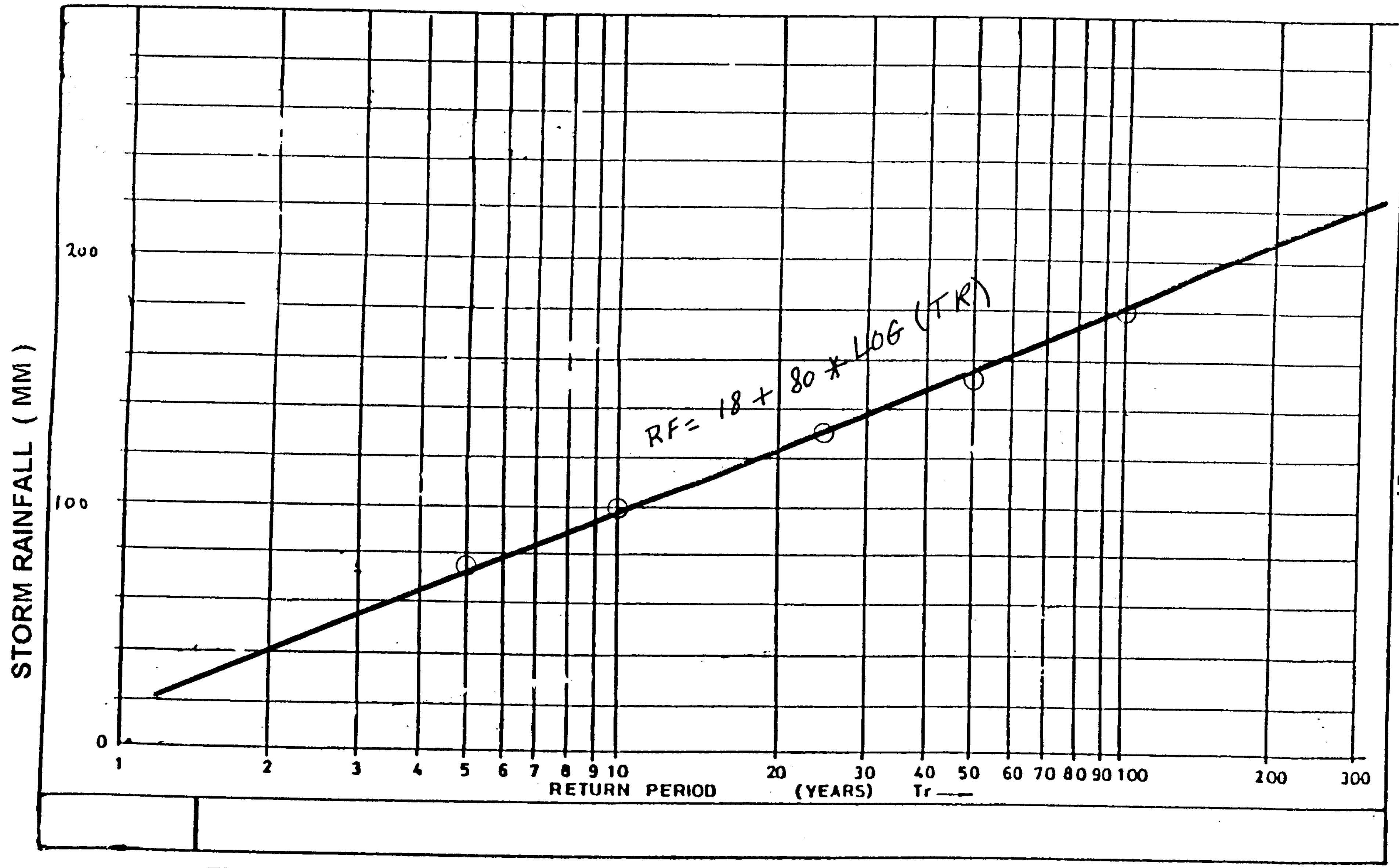


FIGURE (6) : PROBABILITY OF MAXIMUM ANNUAL STORMS
AT MASFOUT STATION

- The Probable Maximum Flood (PMF), is the flood hydrograph resulting from the PMP. coupled with the worst flood-producing catchment conditions that can be realistically expected in the prevailing meteorological conditions.

The following table (table 4) gives an example of return periods used in the design of different hydraulic structures, excluding the design of large dams. Various norms are used in different countries.

Both socio-political and economic considerations enter into the determination of optimum design probabilities of flood discharge. The selection of a design flood is in principle an assessment of the risk involved against the cost of the failure if a flood greater than the design flood is experienced.

Table (4)

Examples of return periods used in the design of different hydraulic structures

Structure	Return Period Years
Bridges on important highways where the loss of the bridge may cause excessive property damage	50 - 100
Bridges on less important roads	25
Culverts on secondary roads, side ditches	5 - 10
Storm-water inlets, gutters	1 - 2

Apart from the cost of repair of damage caused by the flood, human life may also be lost, and this is the most controversial factor. When failure of a structure will result in loss of human life or extensive property damage, a high degree of protection is required against failure. For example if a high dam is located a short distance upstream from a populated or industrialized area, a failure of the structure would be catastrophic. In such cases, the design criteria should be based on probable maximum precipitation estimates. When the consequent of failure are less severe, the level of protection is best based on economic analysis in which the cost of the structure plus flood damage is minimized.

Three alternative procedures are in common use for determining the magnitude of design floods:

a- Frequency analysis of flood flow records, including regional curves.

b- Empirical equations

c- Estimation of probable maximum flood (PAF, based on probable maximum precipitation (PMP).

$$Q_{150} = 0.2\ \text{PMF}$$
$$Q_{1000} = 0.3\ \text{PMF}$$
$$Q_{10,000} = 0.5\ \text{PMF}$$
$$Q_{10,000,000} = \text{PMF}$$

The U.S.A "Standard project flood" is often considered to be 40-60% of PMP. The following table (Table 5) illustrates criteria developed for Great Britain.

Table (5)

Criteria For Design Flood Estimation

Category		General Standard
A.	Reservoir where a breach will endanger lives in a community	Probable Maximum Flood (PMF)
B.	Reservoirs where a breach	0.5 PMF or 10 000 - year flood (take the larger)
(i)	May endanger lives not in a community	
(ii)	will result in extensive damage	
C.	Reservoirs where a breach will pose negligible risk to life and cause limited damage	0.3 PMF or 1 000-year flood (take larger)
D.	Special cases where no loss of life can be foreseen as a result of a breach and very limited additional flood damage will be caused	0.2 PMF or 150-year flood

2-8-2 <u>**Probable Maximum Precipitation (PMP)**</u> :

The probable maximum precipitation refers to the largest storm that has occurred or may occur over a climatologically homogenous region, and is considered to be reasonably typical of the region. The flood that would result from such a storm, if centered over a basin, is called the probable maximum flood (PMF) and is a characteristic of that basin.

Different methods are available to estimate the PMP. However, the best method, accuracy wise, is that which depends on actual observed data. However, the quality of such data, and the length of period of record affects the accuracy of estimation.

The return period of the PMP has not been defined in literature. Ibbit (1968), suggested a return period on (10) years for the PMP. This is an acceptable value for the PMP. This is an acceptable value for designing large reservoirs which require high safety. However, Ibbit based his analysis on annual and 24-hr rainfall data which was obtained from frequency analysis and empirical methods applied in California. This would render his estimates somewhat doubtful for direct application in the U.A.E.

2-8-3 <u>**Selection of Maximum Storm**</u> :

Rain storms which occurred between 1976 and 1993 over Wadi Jazir catchment were checked and considered. The storm of Feb. 17-18, 1989, was found to be the maximum. The isohyetal maps of the seasonal rainfall of 1987/88 over the country, and over the study area is shown in Figure (7). Continuous rainfall records indicated a duration of 27 hours for this storm over different parts of the area. A major center of this storm was located over Huwaylat station which is about (15) Km east of Wadi Jazir. The total storm rainfall in Huwaylat station was 205 mm. Detailed records for this storm at stations situated within the area of influence is given in Table (6). This storm can be considered ideal for the study of Wadi Jazir catchment, because of its proximity.

2-8-4 <u>**Analysis of the Maximum Storm**</u> :

The maximum storm of February 1988 has been analyzed for Wadi Qawr catchment, using the Theisen Polygon method. The area rainfall for this catchment was found as 175.7 mm. The iso-hyetal map for this storm was drawn (Figure 7). Rainfall over Wadi Jazir catchment during this storm can be read from this map as 160 mm.

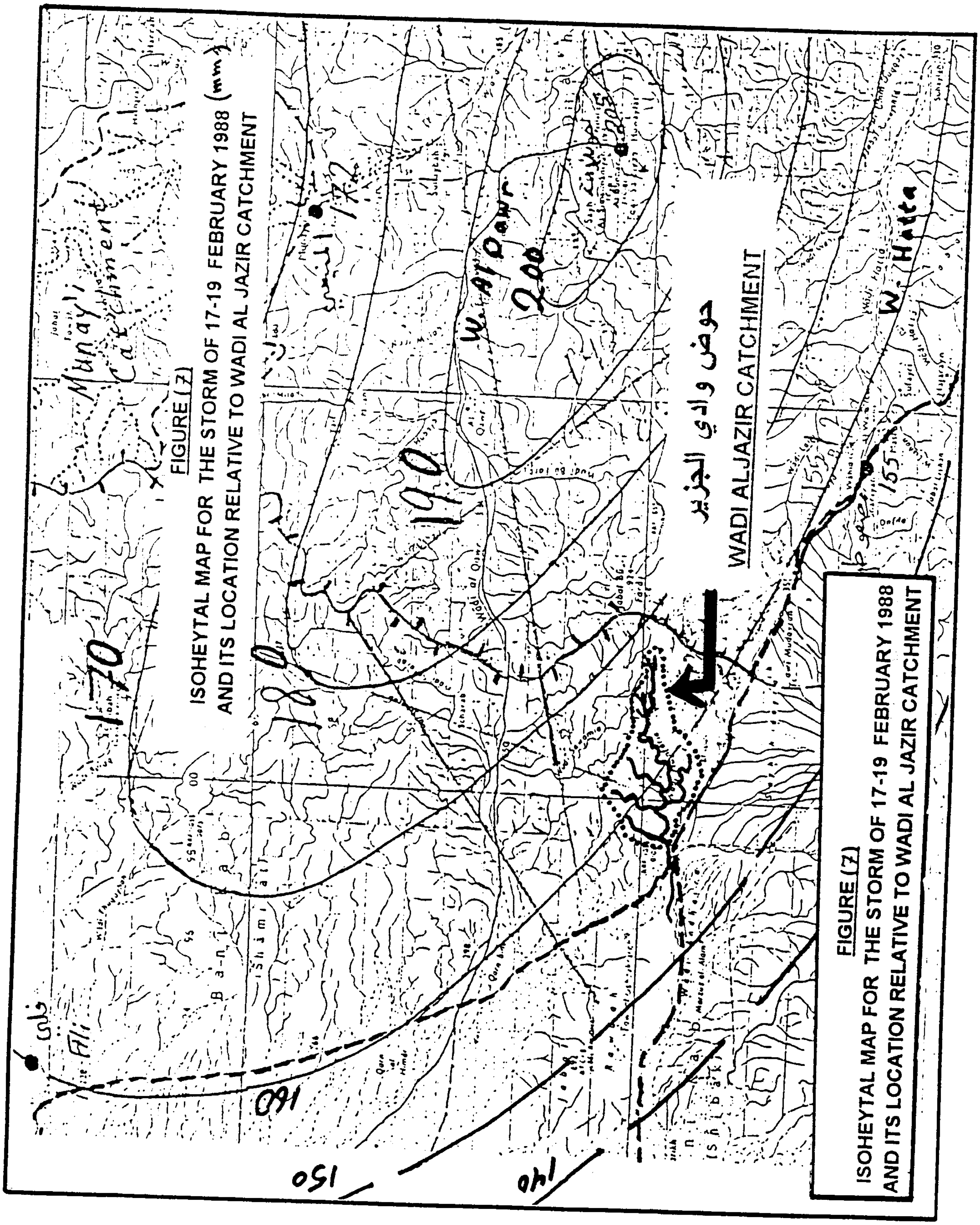


FIGURE (7)
ISOHEYTAL MAP FOR THE STORM OF 17-19 FEBRUARY 1988 (mm)
AND ITS LOCATION RELATIVE TO WADI AL JAZIR CATCHMENT

FIGURE (7)
ISOHEYTAL MAP FOR THE STORM OF 17-19 FEBRUARY 1988
AND ITS LOCATION RELATIVE TO WADI AL JAZIR CATCHMENT

2-9 STORM TRANSPOSITION :

In order to maximize the effect of the storm of Feb. 17, 1988 on generating surface runoff within Jazir catchment, the storm should be transposed over the catchment center of Wadi Jazir. This required some shifting of the storm isohyetal map slightly to the west. The resulting average rainfall over the catchment was calculated as 191.9 mm. (Table 7), compared to actual rainfall during the storm of 160 mm over Wadi Jazir catchment during this storm.

2-10 STORM MAXIMIZATION :

In areas with relatively warm temperatures and catchment areas up to a few hundreds square kilometers, the maximum precipitation, and consequently flood, would result from thunder storms. Thunder storms are characterized by an inflow of very moist air mass al low levels, and quickly reaches condensation levels. Accordingly, the precipitation yield of a thunderstorm varies with the difference in the specific humidity along the saturated adiabat between the inflow layer and the top of the cloud. The difference in the specific humidity is taken between the levels of 900 and 200 mb for relatively warm areas, and between 900 and 400 mb for relatively for colder areas. The 200 mb level is considered for Munayi catchment. The variation of the specific humidity with the wet bulb temperature is shown in (Figure 8).

Table (6)
Stations' Rainfall for the Maximum Storm
Of February 17-18, 1988

Date Stations	14/2/ 1988	15/2/ 1988	16/2/ 1988	17/2/ 1988	18/2/ 1988	19/2/ 1988	Total of
Munayi	6.6	0.8	2.0	171.6	16.4	10.8	198.8
Howeilat	7.0	0.8	5.0	205.4	23.8	34.8	264.0
Masfut	6.6	0.4	6.0	155.21	31.21	33.6	220.0
Sifuni	3.5	0.7	8.4	135.0	0.0	20.4	155.4
Wahala	6.0	0.6	1.5	165.4	20.21	15.21	200.8
Kalba	7.0	0.0	0.2	199.8	4.2	7.4	211.4
Fili	10.2	1.0	4.2	160.2	23.8	11.6	195.6
Bithna	3.0	0.6	8.0	137.0	1.4	0.6	144.4
Farah	5.2	0.0	6.8	184.0	6.2	8.6	198.8
Fujairah	6.2	14.0	14.0	170.0	0.0	20.0	190.0

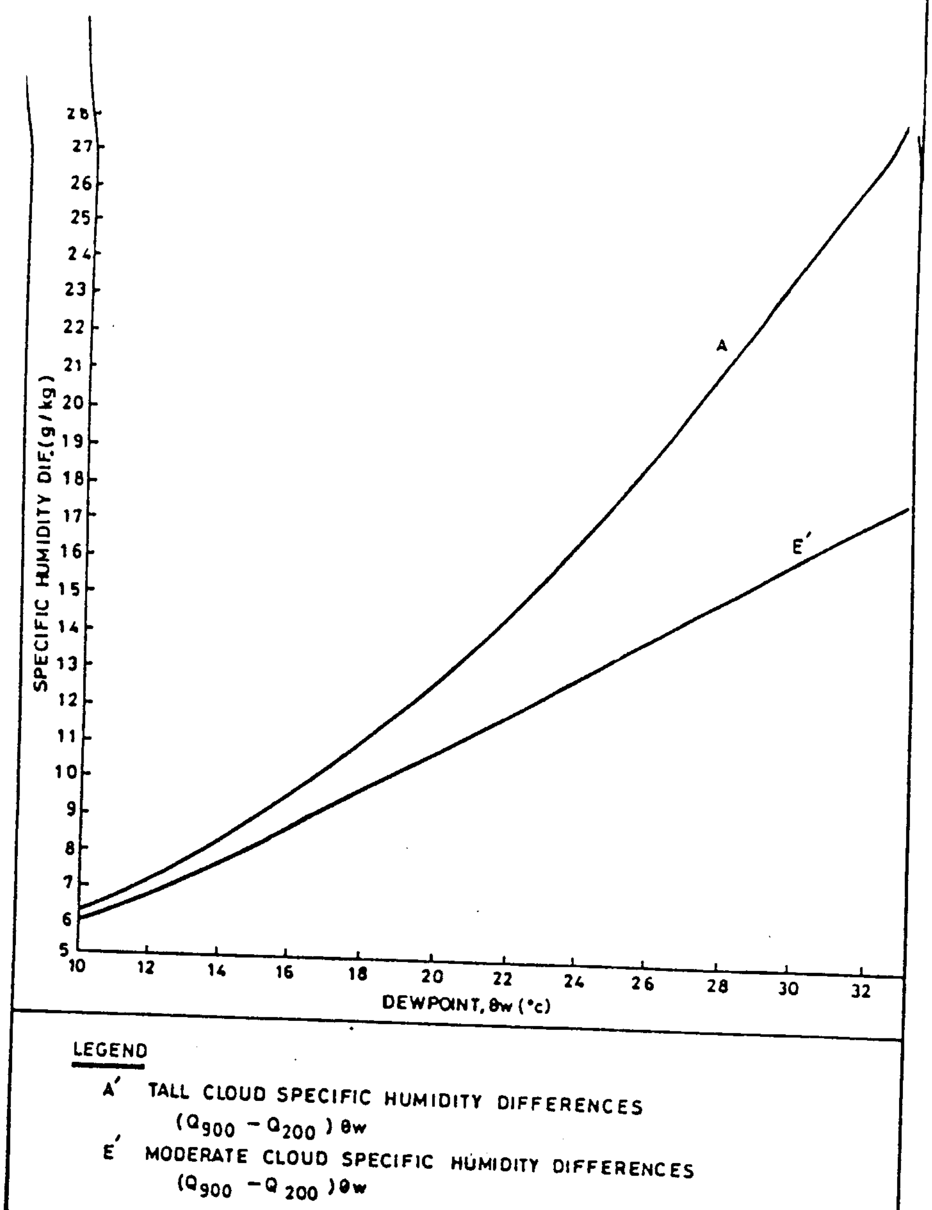


FIG. 8 . SPECIFIC HUMIDITY VARIATIONS ALONG SATURATED ADIABATS

TABLE 7: RAINFALL ANALYSIS OF THE STORM OF 17-18/2/1988
TRANSPOSED OVER UPPER WADI JAZIR CATCHMENT
(ISOHYETAL METHOD)

ISOHYET ZONE		AREA ENCLOSE WITHEN ISOHYET	NET AREA	AVERAGE PRECIPT.	PRECIPT. VOLUME	CUMMUL. PRECIPT. DEPTH	AVERAGE AREAL PRECIPT.
		SQ.KM	SQ.KM	MM	MM*KM2	MM*KM	MM
200	205	1	1	202.5	203	203	202.5
190	200	1.2	1.5	195.0	293	495	330.0
180	190	2.3	2	185.0	370	663	331.3
TOTALS			4.5		865		191.9

AVERAGE PRECIPT.OVER CATHMENT= 191.9 MM

The maximum precipitation form a thunderstorm would be obtained if the wet-bulb temperature during the storm (S), reaches the maximum (potential) wet bulb temperature recorded in the region under consideration (X). This is called the wet bulb potential temperature. Knowing these two values, the probable maximum precipitation which would have been obtained from a given storm can be calculated by multiplying the actual precipitation amount by a ratio (rm) between the saturation specific humidity difference at levels of 900 and 200 mb at the wet-bulb potential dew point temperature, and that actually measured during the specific storm, that is:

$$PMP = P * rm$$

Where P is the actual storm precipitation, and

$$rm = \frac{(Q900 - Q200)X}{(Q900 - Q200)S}$$

Where:

Q= The saturation specific humidity at the indicated level of atmosphere pressure.

X= Indicates values for the maximum potential wet bulb temperature recorded for the region, and

S= Indicates values for the wet-bulb temperature during the storm.

In our case, the storm of 17, of Feb. 1988, the top of the cumulonimbus cloud is expected to reach the 200 mb level, as the weather in February would be relatively warm.

Using the lower curve in (Figure 8), and taking the maximum recorded dew-point temperature in Masfout climatic station, which was 32 C, and the actual recorded wet-bulb temperature during this storm (16 C), the specific humidity differences at the two levels can be read as 28 and 10 for "X" and "S" conditions respectively. Consequently, the maximization factor/ratio (rm) will be equal to 2.4.

Applying this maximization factor to the actual calculated rainfall amount for the maximum recorded storm after being transposed over Wadi Jazir catchment, we can obtain the probable maximum precipitation which will be 191.2 x 2.4 = 459mm. This maximization ratio can be applied to other stream catchments within the eastern mountains of the UAE.

2-11 THE PROBABLE MAXIMUM FLOOD (PMF) :

As the storm of Feb. 17-18, 1988 was found to represent the maximum storm in the record, the time distribution of the PMP was developed from the actual recorded rainfall hydrography for Huwaylat station. The rainfall duration at this station was 27 hours, with a 9-hour delay from the start of rain at this station.

The amounts of rainfall at this station during the storm were 205 mm. The hourly incremental rainfall for this station was obtained, and its percentages from the total station rainfall were calculated (Table 8).

The one-hour incremental rainfall percentages where then converted into depths by multiplying them with the previously calculated PMP for Munayi catchment. This would represent the time distribution of the PMP.

The resulting rainfall depths were then critically rearranged so as to produce the probable maximum flood (PMF). The sequence of hourly rainfall increments which would produce the PMF can be determined by trial computations of the runoff hydrography resulting from various arrangements or sequences. In making these calculations it is necessary to subtract rainfall losses to get the rainfall excess, (surface runoff).

The initial abstraction from this storm for Wadi Jazir catchment was found to be 48 mm for the curve number (CN) of (51) which represents the dry pre-storm condition. However, for wet condition, CN is taken as (60) and the initial abstraction equals 34 mm. In addition, uniform infiltration losses of 10 mm were also subtracted. The net rainfall excess is then obtained and shown in the last column of table (8 & 9). The first (8) hours of rainfall were needed to satisfy the initial abstraction. After that, a uniform infiltration loss of 10 mm was subtracted from the remaining rainfall amounts. Rainfall excess (runoff) started after 8 hours of rain and lasted for (11) hours. These represent the hourly distribution of rainfall excess from the PMP. The total resulting runoff volume from this excess rainfall was calculated as (1.6) MCM.

2-12 DESIGN FLOOD :

As the proposed dam is relatively small, the channel width is also small, and the dam will be an over flow concrete or gabion dam, the spillway cost will not be highly sensitive to the flow rate. The Maximum probable form has been considered for designing the spillway of the proposed Dam. However, as the dam is aimed for ground water recharge, the reservoir capacity has been selected to store the 100-year flood.

TABLE (8) :RAINFALL DISTRIBUTION AND RAINFALL EXESS FOR THE MAXIMIZED PROPABLE STORM OVER W. JI

TIME HRS.	RAINFAL MM	PERCENT OF STOR RF. %	% ARRANGE TO MAX. FLOOD CONDITIO	% APPLIED TO PMP 191*2.4 MM	RAINFAL MINUS INITIAL ABSTRAC 34	RAINFAL MINUS G.W. RECHARG (10 MM/H :EXESS RAINFAL
1	0.0	0.00	0	0.00	0.00	
2	0.2	0.12	0	0.00	0.00	
3	5.4	3.15	0.12	0.55	0.00	0
4	7.4	4.31	0.35	1.60	0.00	0
5	12.2	7.11	0.47	2.15	0.00	0
6	4.0	2.33	0.70	3.21	0.00	0
7	8.2	4.78	2.33	10.68	0.00	0
8	7.0	4.08	3.15	14.44	0.00	0
9	0.0	0.00	4.78	21.91	20.55	0.00
10	24.0	13.99	7.11	32.59	32.59	10.55
11	0.8	0.47	13.99	64.13	64.13	22.59
12	35.0	20.40	20.40	93.51	93.51	54.13
13	3.4	1.98	17.13	78.52	78.52	83.51
14	4.0	2.33	10.02	45.93	45.93	68.52
15	0.0	0.00	4.31	19.76	19.76	35.93
16	1.2	0.70	4.08	18.70	18.70	9.76
17	0.8	0.47	3.50	16.04	16.04	8.70
18	29.4	17.13	2.45	11.23	11.23	6.04
19	0.4	0.23	2.33	10.68	10.68	1.23
20	0.0	0.00	1.98	9.08	9.08	0.68
21	0.2	0.12	0.47	2.15	2.15	0.00
22	6.0	3.50	0.23	1.05	1.05	0.00
23	0.0	0.00	0.12	0.55	0.55	0.00
24	4.2	2.45	0	0.00	0.00	0.00
25	17.2	10.02	0	0.00	0.00	0.00
26	0.6	0.35	0	0.00	0.00	0.00
27	0.0	0.00	0	0.00	0.00	0.00
TOTAL	171.6	100.0	100.0	458.5	424.5	301.7

TABLE (9) :RAINFALL DISTRIBUTION AND RAINFALL EXESS FOR THE 100-YR STORM OVER W. JAZIR (178 MM)

TIME HRS.	RAINFAL MM	PERCENT OF STOR RF. %	% ARRANGE TO MAX. FLOOD CONDITIO	% APPLIED TO 100-YR STOR 178 MM	RAINFAL MINUS INITIAL ABSTRAC 34 MM	RAINFALL MINUS G.W. RECHARGE (10 MM/HR :EXESS RAINFALL
1	0.0	0.00	0	0.00	0.00	
2	0.2	0.12	0	0.00	0.00	
3	5.4	3.15	0.12	0.21	0.00	0
4	7.4	4.31	0.35	0.62	0.00	0
5	12.2	7.11	0.47	0.84	0.00	0
6	4.0	2.33	0.70	1.25	0.00	0
7	8.2	4.78	2.33	4.15	0.00	0
8	7.0	4.08	3.15	5.61	0.00	0.00
9	0.0	0.00	4.78	8.51	0.00	0.00
10	24.0	13.99	7.11	12.66	0.00	0.00
11	0.8	0.47	13.99	24.90	24.74	14.74
12	35.0	20.40	20.40	36.31	36.31	26.31
13	3.4	1.98	17.13	30.49	30.49	20.49
14	4.0	2.33	10.02	17.84	17.84	7.84
15	0.0	0.00	4.31	7.67	7.67	0.00
16	1.2	0.70	4.08	7.26	7.26	0.00
17	0.8	0.47	3.50	6.23	6.23	0.00
18	29.4	17.13	2.45	4.36	4.36	0.00
19	0.4	0.23	2.33	4.15	4.15	0.00
20	0.0	0.00	1.98	3.52	3.52	0
21	0.2	0.12	0.47	0.84	0.84	0
22	6.0	3.50	0.23	0.41	0.41	0
23	0.0	0.00	0.12	0.21	0.21	0
24	4.2	2.45	0	0.00	0.00	0
25	17.2	10.02	0	0.00	0.00	0
26	0.6	0.35	0	0.00	0.00	0
27	0.0	0.00	0	0.00	0.00	0
TOTAL	171.6	100.0	100.0	178.0	144.0	69.4

2-13 UNIT HYDROGRAPH :

In the absence of measured flood hydrograph, it becomes necessary to derive a synthetic unit hydrograph. Snider's method, 1938, was used as shown in Annex (7). The derived unit hydrographs is shown in figure 9.

2-14 HYDROGRAPH OF THE DESIGN FLOOD (PMF) :

From the synthesized unit hydrograph Figure (9), and tables (10 & 11), and for the 24 hr PMP, the peak flow of the PMF and the 100-year storms were found to be 58.7, 28.3 and $16m^3$/sec. The duration of the flood would be 26, 26 and 25 hours, respectively. The total volumes would be 1.6 and 0.721 and 0.4MCM respectively. The hydrographs of the two floods MPF 2 100-yr flood are shown in Figure (10).

2-15 FREQUENCY OF PEAK FLOODS :

As no data on historical peak floods are available for Wadi Jazir the flood peaks frequency of the Wadi Qawr was transposed to Wadi Jazir catchment. The Greager formula was used in a backward manner to calculate the Greager numbers for Wadi Qawr which was then transposed to Wadi Jazir to calculate the flood peaks for various return periods (table 12). This formula is written as follows:

$$QM = C_1 * (0.386 * A)^{0.894 \# (0.386 \# A) -0.048}$$

Where :

QM = Maximum or peak flow for a given return period
A = Catchment area (Sq. Km)
C1 = Greagers number (max. 130)

Greager numbers for various return periods were calculated for Wadi Qawr and transferred to Jazir catchment. Table (12) gives the peak floods for different return periods. If the return periods of the maximum probable flood is taken as 107 years, the value of the PMF is calculated here as 64 m^3/s which is close to the previously calculated value (58.7 m^3/s).

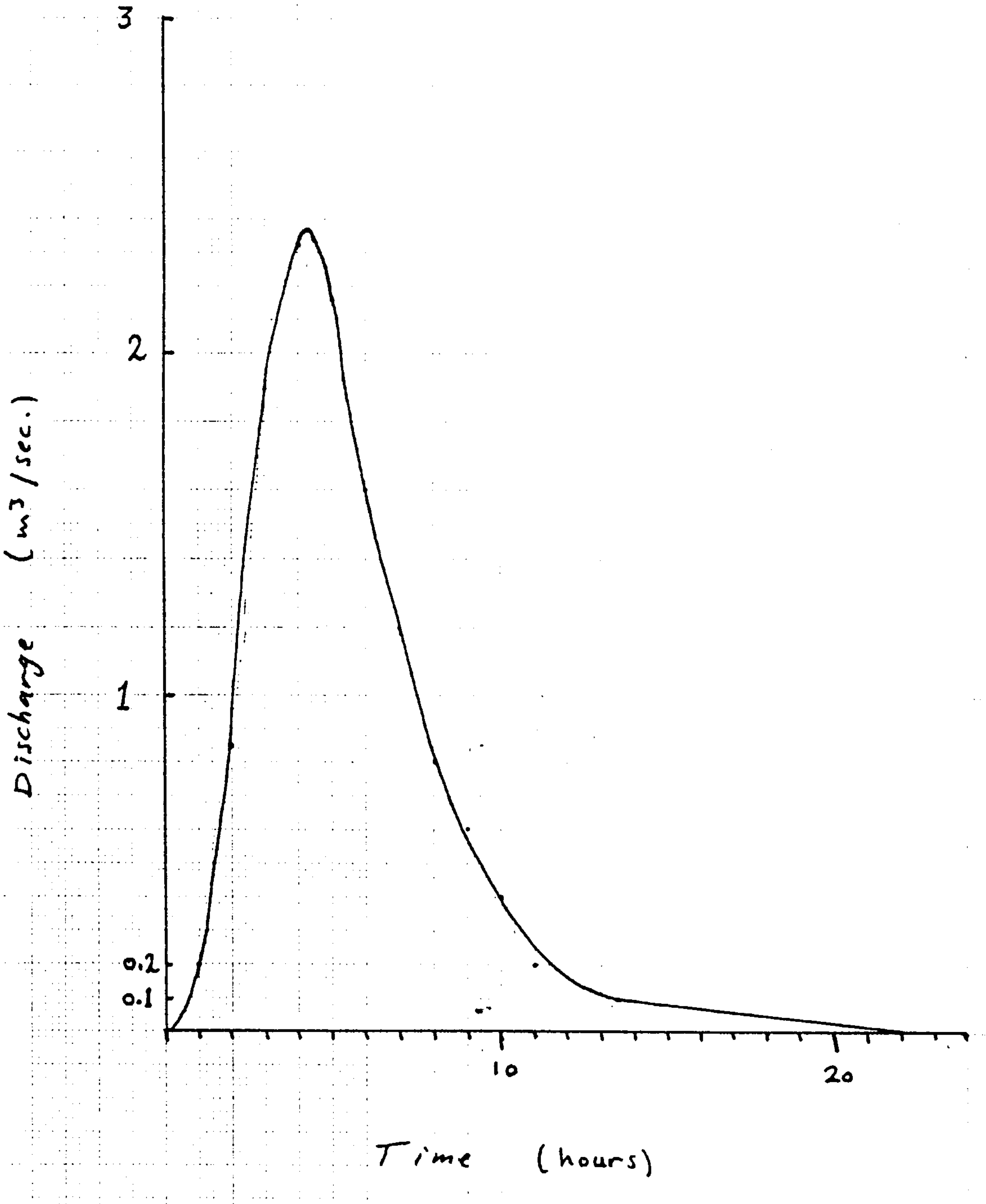


FIGURE (9) :UNIT HYDROGRAPH FOR WADI ALJAZIR CATCHMENT

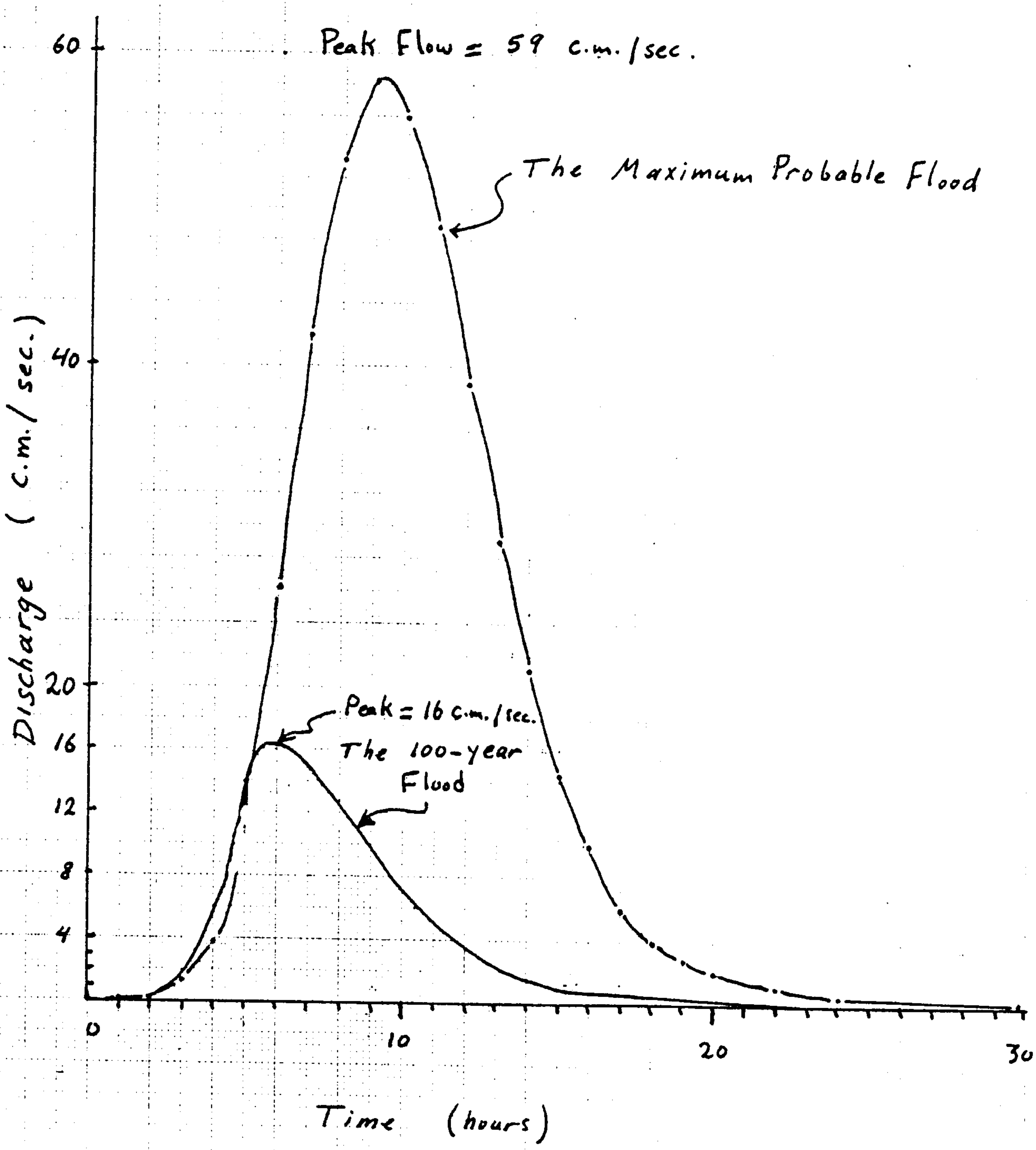


FIGURE (10) : HYDROGRAPHS FOR THE DESIGN FLOODS

TABLE (10) :FLOOD HYDROGRAPH FOR THE " PMF " - THE MAXIMIZED PROPABLE STORM
(MR=2.4,,,S=34 MM,,,INF=10 MM)

TIME HRS	DISCHARGE C.M./SEC	EXCESS RAINFALL (C.M.)										TOTAL C.M/SEC	FLOOD VOLUME C.M.
		1.055	2.259	5.413	8.351	6.852	3.593	0.976	0.870	0.123	0.068		
0	0.000											0.00	0
1	0.200	0.00										0.00	0
2	0.850	0.21	0.00									0.21	760
3	1.900	0.90	0.45									1.35	4855
4	2.330	2.00	1.92	0.00								3.92	14129
4.2	2.350	2.46	4.29	1.08	0.00							7.33	28198
5	2.160	2.48	5.26	4.60	1.67	0.00						14.01	50450
6	1.600	2.28	5.31	10.28	7.10	1.37	0.00					26.34	94827
7	1.200	1.69	4.88	12.61	15.87	5.82	0.72	0.00				41.59	149722
8	0.800	1.27	3.61	12.72	19.46	13.02	3.05	0.20	0.00			53.33	191977
9	0.600	0.84	2.71	11.69	19.62	15.97	6.83	0.83	0.17	0.00		58.67	211202
10	0.400	0.63	1.81	8.66	18.04	16.10	8.37	1.85	0.74	0.02	0.00	56.23	202434
11	0.200	0.42	1.36	6.50	13.36	14.80	8.44	2.27	1.65	0.10	0.01	48.92	176125
12	0.120	0.21	0.90	4.33	10.02	10.96	7.76	2.29	2.03	0.23	0.06	38.80	139689
13	0.100	0.13	0.45	3.25	6.68	8.22	5.75	2.11	2.04	0.29	0.13	29.05	104568
14	0.080	0.11	0.27	2.17	5.01	5.48	4.31	1.56	1.88	0.29	0.16	21.23	76442
15	0.060	0.08	0.23	1.08	3.34	4.11	2.87	1.17	1.39	0.27	0.16	14.71	52947
16	0.040	0.06	0.18	0.65	1.67	2.74	2.16	0.78	1.04	0.20	0.15	9.63	34664
17	0.030	0.04	0.14	0.54	1.00	1.37	1.44	0.59	0.70	0.15	0.11	6.07	21840
18	0.020	0.03	0.09	0.43	0.84	0.82	0.72	0.39	0.52	0.10	0.08	4.02	14484
19	0.015	0.02	0.07	0.32	0.67	0.69	0.43	0.20	0.35	0.07	0.05	2.87	10330
20	0.012	0.02	0.05	0.22	0.50	0.55	0.36	0.12	0.17	0.05	0.04	2.07	7442
21	0.010	0.01	0.03	0.16	0.33	0.41	0.29	0.10	0.10	0.02	0.03	1.50	5383
22	0.000	0.01	0.03	0.11	0.25	0.27	0.22	0.08	0.09	0.01	0.01	1.08	3886
23		0.00	0.02	0.08	0.17	0.21	0.14	0.06	0.07	0.01	0.01	0.77	2767
24			0.00	0.06	0.13	0.14	0.11	0.04	0.05	0.01	0.01	0.54	1955
25				0.05	0.10	0.10	0.07	0.03	0.03	0.01	0.01	0.41	1461
25				0.00	0.08	0.08	0.05	0.02	0.03	0.00	0.00	0.27	987
26					0.00	0.07	0.04	0.01	0.02	0.00		0.15	531
27						0.00	0.04	0.01	0.01			0.06	218
28							0.00	0.01	0.01			0.02	73
29								0.00	0.01			0.01	31
30									0.00			0.00	0
31													
32													
												TOTAL	1.604

TABLE (11) :FLOOD HYDROGRAPH FOR THE " 100-YR STORM "
(MR=2.4,,,S=34 MM,,,INF=10 MM)

TIME HRS	DISCHARGE C.M./SEC	E X C E S S		R A I N F		TOTAL C.M/SEC	FLOOD VOLUM C.M.
		1.474	2.631	2.049	0.784		
0	0.000					0.00	0
1	0.260	0.00				0.00	0
2	0.995	0.38	0.00			0.38	1378
3	1.861	1.47	0.68	0.00		2.15	7740
4	2.337	2.74	2.62	0.53	0.00	5.89	21216
PEAK 4.2	2.510	3.44	4.90	2.04	0.20	10.58	38098
5	2.164	3.70	6.15	3.81	0.78	14.44	51985
6	1.731	3.19	6.60	4.79	1.46	16.04	57741
7	1.212	2.55	5.69	5.14	1.83	15.22	54787
8	0.909	1.79	4.55	4.43	1.97	12.74	45867
9	0.649	1.34	3.19	3.55	1.70	9.77	35173
10	0.433	0.96	2.39	2.48	1.36	7.19	25874
11	0.312	0.64	1.71	1.86	0.95	5.16	18567
12	0.182	0.46	1.14	1.33	0.71	3.64	13105
13	0.121	0.27	0.82	0.89	0.51	2.48	8939
14	0.095	0.18	0.48	0.64	0.34	1.63	5884
15	0.052	0.14	0.32	0.37	0.24	1.08	3873
16	0.043	0.08	0.25	0.25	0.14	0.72	2584
17	0.035	0.06	0.14	0.20	0.09	0.49	1763
18	0.022	0.05	0.11	0.11	0.07	0.34	1241
19	0.017	0.03	0.09	0.09	0.04	0.25	905
20	0.013	0.03	0.06	0.07	0.03	0.19	673
21	0.010	0.02	0.05	0.04	0.03	0.14	490
22	0.000	0.01	0.03	0.04	0.02	0.10	362
23		0.00	0.03	0.03	0.01	0.07	235
24			0.00	0.02	0.01	0.03	107
25				0.00	0.01	0.01	27
25					0.00	0.00	0

TOTAL (MILLION C.M.) : 0.399

TABLE (12):FREQUENCY OF PEAK FLOWS IN WADI JAZIR USING CREAGER METH

A :CALCULATION OF CREAGER NUMBERS FOR WADI GHOR:

CREAGER FORMULA:

Qp=CR*(.386*AREA)^(.894*(.386*AREA)^(-.048))..OR..:Qp=CR*D..OR..:CR=Qp/D

	WADI GH	W.JAZIR
A=0.386*302 =	116.57	1.74
B=A^(-.048) =	0.80	0.97
C=0.894*B =	0.71	0.87
D= A^C =	29.53	1.62

B :CALCULATION OF CREAGER NUMBERS FOR WADI GHOR

RETURN PERIOD YEARS	PEAK FLOW C.M./SEC	CREAGER NUMBER
2.33	150	5.08
5	218	7.38
10	272	9.21
25	341	11.55
50	392	13.27
100	441	14.93
500	558	18.90
1000	609	20.62
10000	776	26.28
10^5	950	32.17
10^6	1123	38.03
10^7	1296	43.89
10^8	1461	49.48
10^9	1633	55.30

C : FREQUENCY OF PEAK FLOWS IN WADI JAZIR

RETURN PERIOD YEARS	CR.NO Qp/D	W.MUNAYI' PEAK FLOW
2.33	5.08	7
5	7.38	11
10	9.21	13
25	11.55	17
50	13.27	19
100	14.93	22
500	18.90	28
1000	20.62	30
10^4	26.28	38
10^5	32.17	47
10^6	38.03	56
10^7	43.89	64

2-16 EVAPORATION :

The closest meteorological station, which would best represent the climatic conditions in the Jazir catchment is Masfout station.

The average monthly potential evaporation for Masfout station, are given in Table (13). The highest monthly evaporation rates for the area (327-565 mm) are during the period May to September. While the lowest (129-260mm) are during the period December to January.

The average monthly evaporation for Masfout station for the period 1972-1985 are as follows in mm/day:

Table (13)
Monthly Evaporation at Masfout Station

J	F	M	A	M	J	J	A	S	O	N	D
4.3	5.7	7.5	13.1	16.3	17.9	15.1	12.7	20	8.5	6.1	4.3

The total annual evaporation is calculated as 3640 mm.

2-17 RESERVOIR SEDIMENTATION :

Normally, reservoir sedimentation is expected to reduce its useful storage capacity during the life time of the reservoir. The rate at which the capacity of the reservoir is reduced by sedimentation depends on: (1) the quantity of sediment inflow, (2) the percentage of this inflow trapped in the reservoir, and (3) the density of the deposited material.

The quantity of sediment inflow may be estimated by different methods in absence of actual measurements. The UAE water and soil year book used the following formula:

$$Qs = 292.6\ A^{-0.12}$$

Where: Qs, is the annual sediment yield in m^3/km^2, and A, in the catchment area in km^2.

Applying this formula to the Jazir catchment with an area of 4.5km would give an annual sediment inflow rate of 244 $m^3/km^2/yr$. Which gives on the average 1100 $m^3/year$. Since the stored flood water would be released within 20 days after a given storm, the trap efficiency would be relatively low. In addition the trap efficiency would decrease every year over the life of the project. Therefore an average trap efficiency of 60% would be considered over the project life. This would result in an annual reservoir sedimentation of:

1100 x 0.6 = 660 m^3/yr.

PART III

3- PRELIMINARY DESIGN

3-1 SELECTION OF DAM SITE :

The selection process of the dam site included two phases: (1) Office interpretation of topographic and geologic map, as well as areal photographs. The result of this initial phase is to prepare a hydrographic map showing the natural drainage system, the catchment boundary (water divides), and finally identifying tentative locations for potential dam sites.

The second phase included field investigation of the selected tentative sites, checking the site geology and foundation conditions and availability of construction material, the locations and type of spillway, suitable reservoir area, and the center line of the dam, and getting some idea on the possible types of dams.

Based on these investigations, one site was finally selected for detailed analysis. The selected site was also approved by the Ministry of Agriculture and Fisheries and the local Governor (Figure 11).

3-2 PROJECT COMPONENTS :

The project includes the following components:

* the main dam with an over flow spillway,
* a reservoir area,
* a downstream apron and stilling basin,
* an upstream apron,
* a bottom outlet,
* 2-3 down stream recharge basins using 2-meter high gabion walls across the stream channel.

3-3 LEFT AND RIGHT ABUTMENT :

The left and right abandments of the proposed dam consist of hard massive igneous rock which is moderately jointed and weathered at the surface and dense at depths. The abutment slope is moderate to high and the heights of the two abutment hills are 25 and 30 meters. The rock permeability is mainly secondary in the upper fractured and weathered zone and decreases with depth.

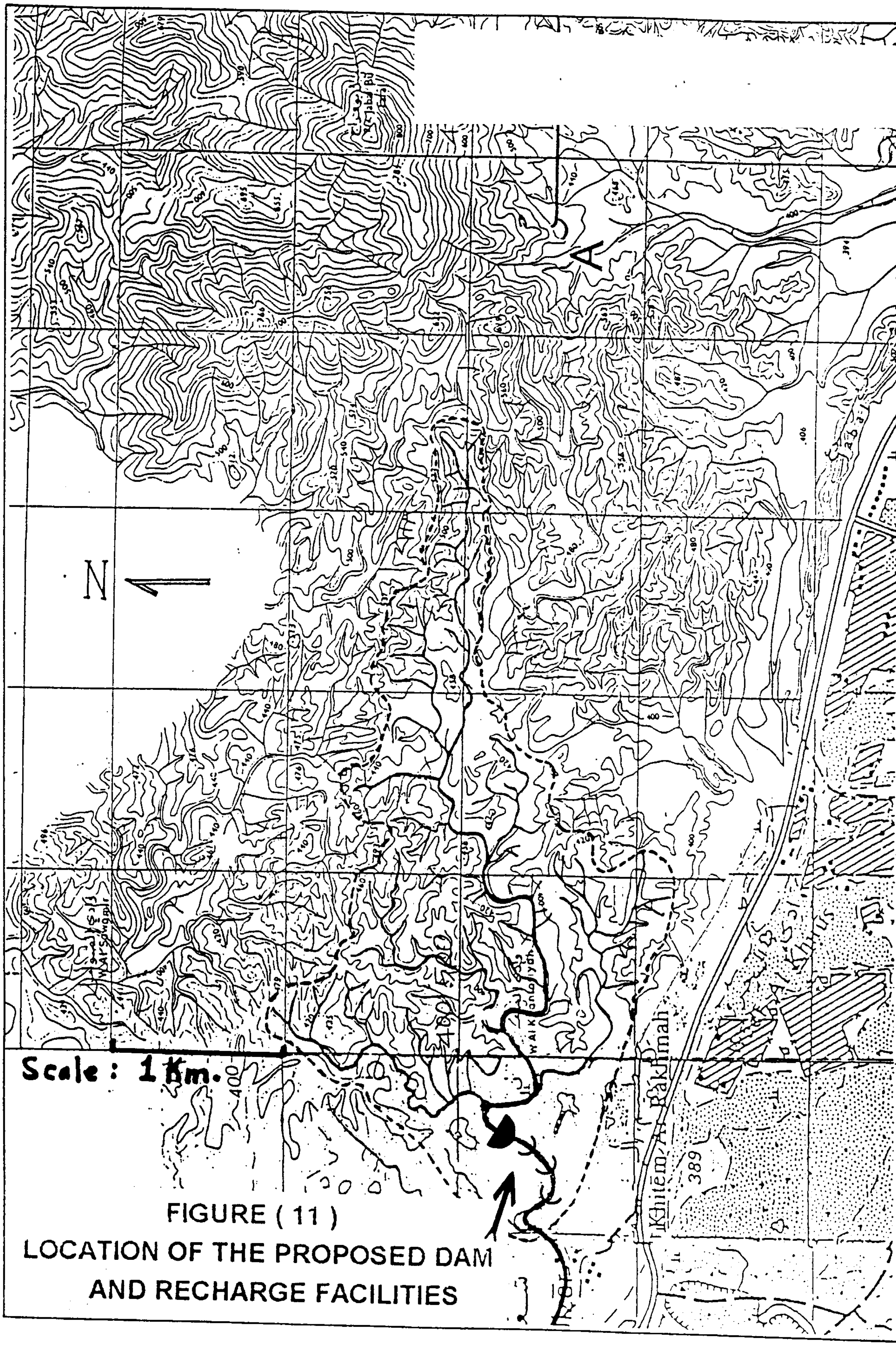


FIGURE (11)
LOCATION OF THE PROPOSED DAM
AND RECHARGE FACILITIES

The dam will tie in the solid abutment rock after excavating the surfacial weathered zone, to obtain a clean regular contact area. If fractures and open joints are still existing on the excavated abutment surfaces, adequate compacting, grouting and a concrete slap need to be done in order to control seepage.

3-4 **FOUNDATION CONDITIONS** :

The material along the dam axis consist of course-grained alluvium underlain by hard igneous rocks which are relatively weathered in the upper zone and dense and solid at greater depths (Figure 12). Therefore the rock permeability is low and expected to decrease with depth, but will remain much lower than the permeability of the over-lying alluvial deposits.

The upper recent alluvium, which mostly occurs in the central channel, is loose and more permeable than the older alluvium at depth and in the side older terraces, which usually have higher degree of cementation. The maximum depth of alluvium in the central channel is not expected to exceed 5 m. There is some saturation at shallow depth, at and downstream of the proposed dam site.

Investigation of similar alluvial deposits at some previously constructed dams in this region indicated a moderate to high bearing capacity and small compressibility of such material.

Knowledge of such properties will help determine the depth of foundation and depth of cutoff trench,and any possible treatment requirement. Stability-wise, excavations along the dam axis should reach material with properties equal to or better than the rockfill or earth fill to be placed on it.

However, if the alluvial mantle along the dam axis proves to sufficiently competent, it may be left under the embankment, and two-meter cut off trench would be sufficient.

As the purpose of this dam is flood detection and artificial recharge, see page through the foundation is allowed as long as the seepage forces at the down stream slope and in the downstream area of the dam are kept within allowable limits to minimize uplift pressure.

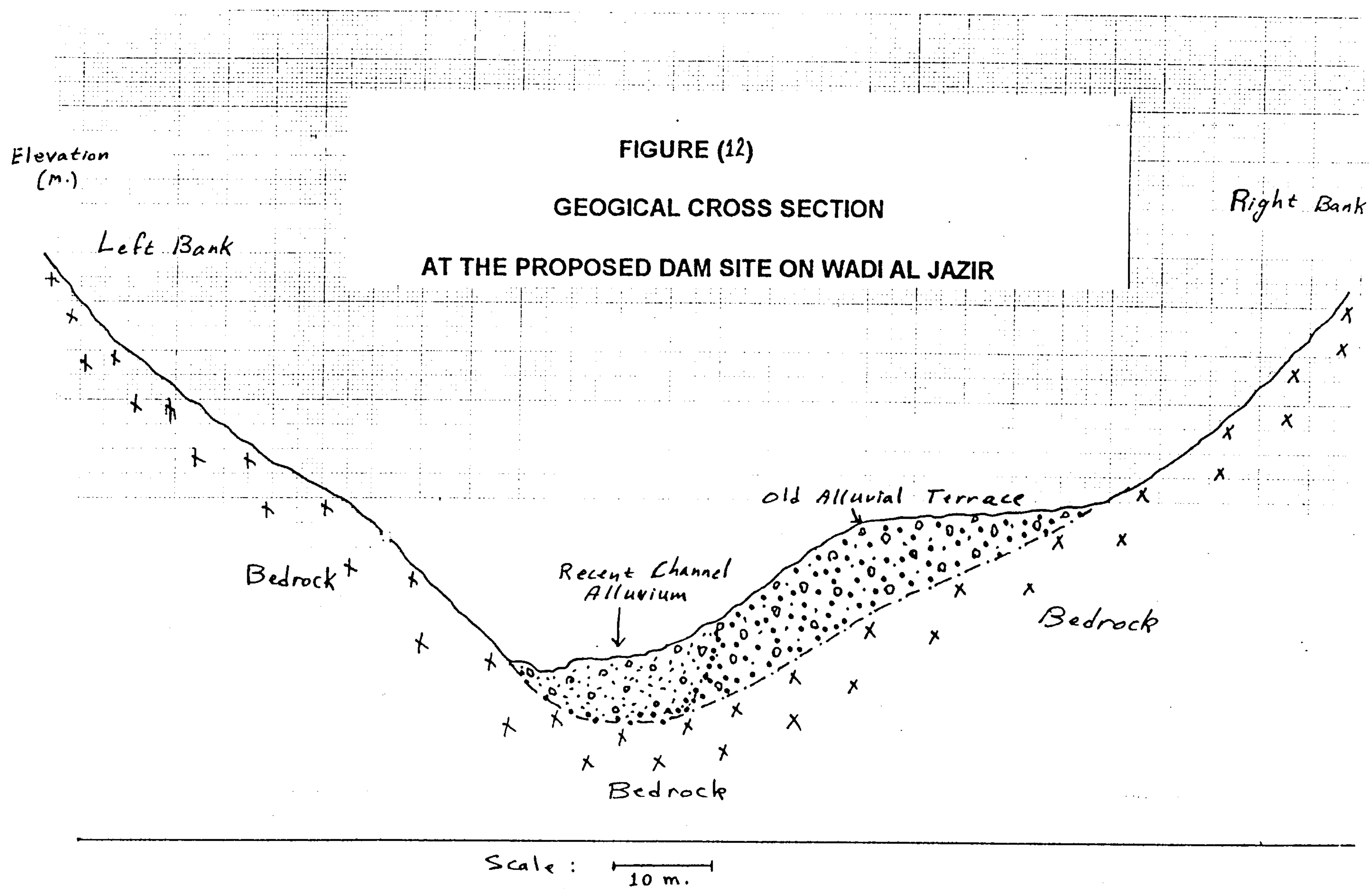
3-5 **CONSTRUCTION MATERIAL** :

Aggregates, sand and medium to coarse gravel, for concrete mix is available in the alluvial channel. However, large rocks for gabions are not available at or near the site.

FIGURE (12)

GEOGICAL CROSS SECTION

AT THE PROPOSED DAM SITE ON WADI AL JAZIR



3-6 SELECTION OF DAM TYPE & DESIGN :

A number of factors govern the selection of the type of dam: Foundation conditions, channel size and geometry, availability of construction material, economy, and sometimes the experience and preference of the designing engineer.

In our present case, the foundation and abutments' conditions, the availability of construction material, the channel width, the volume of the design flood.

1- The purpose of the dam is the detention of floods for later release for groundwater recharge in the downstream channel. Therefore water tightness of the foundation and the reservoir area would not be a problem as long as protection against seepage forces is considered.

2- The foundations of the proposed dam consist of previous alluvial deposits which are composed of mixed sand and gravel. The central parts of the alluvium are not cemented. However, the terrace on the right bank and its extension at depth is cemented.

3- The alluvial deposits have small thickness in the central channel (0 - 5 m.). The alluvial terrace on the right bank has a thickness of 5 m also.

4- The bedrock underneath this alluvium and on the hill slopes on both abutments is rather solid igneous rocks. It is moderately weathered at the surface but fracturing would be less and closed at depth. This rock type would provide very favorable foundations for a concrete dam.

5- A concrete dam is also favorable for such relatively narrow stream channel. The main existing active channel is only 17 m wide at the proposed site, and about 40 m. wide at a high of 5 m. above channel level.

6- The above conditions are favorable for a concrete dam if strongly anchored into the bedrock.

7- Another alternative is to build a gabion dam. However, the inavailability of rock material for gabion construction in the channel, is a major limitation.

8- There is no possibility to construct a side channel spillway because of the drainage, and topographic condition. Therefore the central part of the dam will be designed as overflow section. The width of this over flow section is limited by the width of the main channel at the site which is 20-25 m. only.

9- The spillway has been sized to pass the probable maximum flood safety without over topping the dam.

10- The thin alluvial deposits and shallow bedrock within the channel, as well as the type and properties of this bedrock in the main channel and on both abutments favor the construction of a concrete dam as number one alternative, or a gabion dam as a second alternative.

11- The reservoir capacity has been designed to store approximately the flood of 100-year storm. Because of the relatively narrow channel of Wadi Jazir, its storage capacity is limited to 0.34 max for a useful height of 10.5 m.

3-7 <u>FOUNDATION DEPTH</u> :

The concrete body of the dam will be sunk into the alluvial deposits for one meter in the central part of the present active channel and below the alluvial terrace on the left bank.

On the channel sides a cut-off wall will be excavated in the abutment rock for 1.0 meter (or to the depth of the weathered rock).

One-meter thick steps are also recommended at the heel and toe of the dam base to provide higher friction resistance to sliding.

Upstream and downstream aprons to decrease seepage pressure at the downstream end are also recommended.

3-8 <u>HEIGHT OF THE DAM</u> :

The total height of the dam, above the foundation will consist of the following components:

1- The thickness of the stripped weathered rock and over burden (1 m).

2- The elevation between the present ground surface to the crest level. This will determine the active usable storage behind the dam, and will be taken as the volume of the flood with 100-year return period which is about 0.4 MCM.

3- The adopted freeboard between the weir crest of the spillway and the dam crest.

3-9 HEIGHT NEEDED TO STORE THE DESIGN FLOOD :

Figure (13) shows the reservoir capacity -elevation- area rating curve at the proposed site. The flood and required usable dam heights and the corresponding crest length are listed below:

Dam Height (m)	Dam Crest Length (m)	Reservoir Capacity (m^3)
2	48	18,000
4	65	55,00
6	78	122,00
8	94	210,00
10	108	340,00
12	118	550,00

3-10 FREEBOARD :

(a) Wave Heights :
The Munayi dam is rather wide compared to the length of the reservoir area, so that the shoreline produces little drag upon the build-up of waves. This results in an effective fetch greater than the actual. This would result in higher wave height. The fetch of the reservoir would be about 300 m when it is full of water.

(b) Wave Run-up :
As the upstream face of the dam will be vertical or nearly so, this item will be negligible.

(c) Wind Tide :
Wind tide for relatively short and shallow reservoirs is usually negligible. It has been calculated for this study as 2 cm. (Annex 8).

(d) Compensation for Foundation Settlement :
The alluvial foundation at the proposed dam axis is rather shallow (0 - 5 m).

The Maximum Dam height for the 100 yr flood (0.4 MCM), will be as follows:

-	Cutoff trench	1 m
-	Usable dam height	10.5 m
-	Wave height / Free board	0.68 m
-	Surcharge storage / Free board	1.5 m
	Total Height	13.68 m

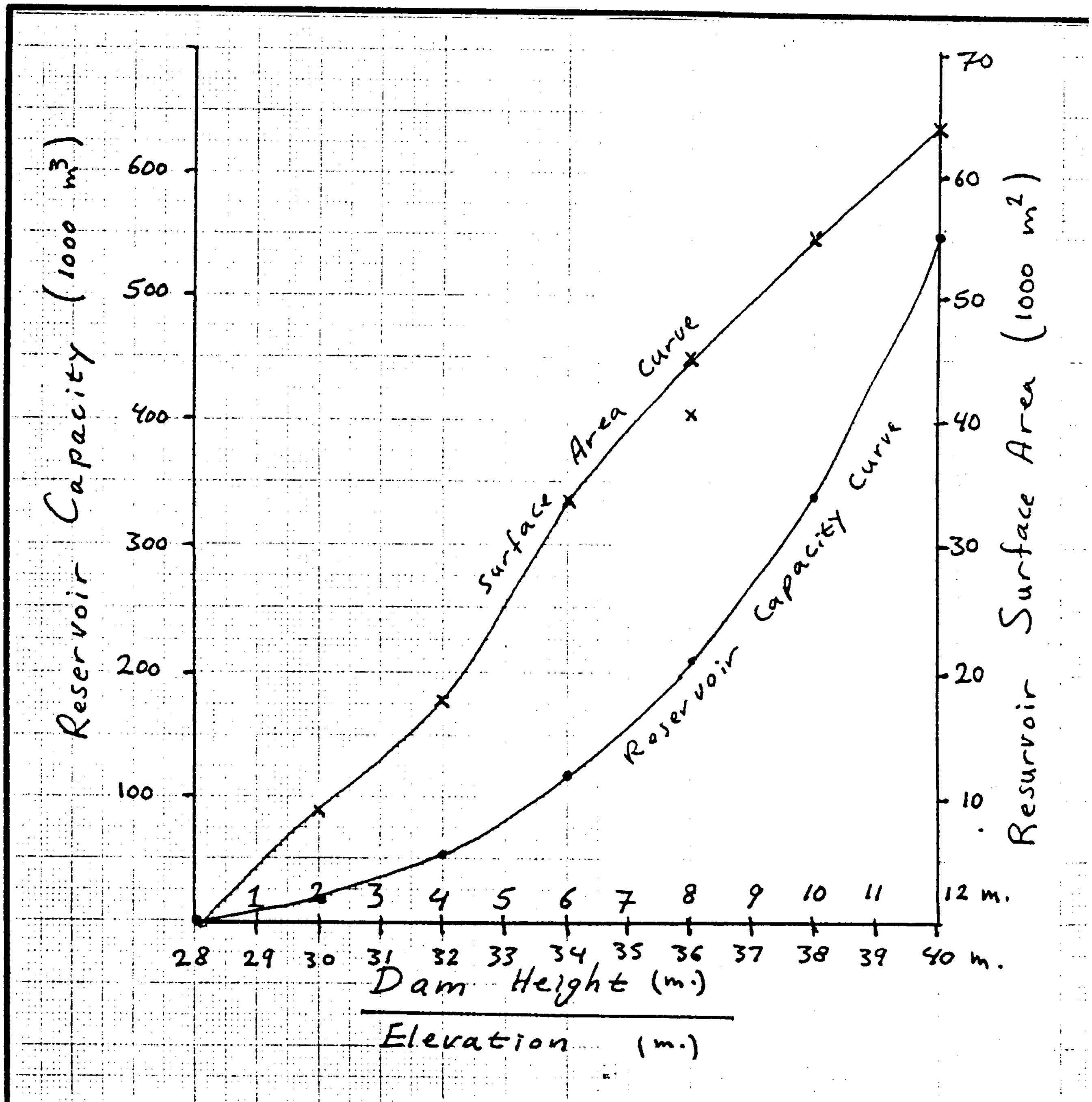


FIGURE (13)

ELEVATION-RESERVOIR CAPACITY-SURFACE AREA CURVES

FOR THE PROPOSED DAM SITE ON WADI ALJAZIR

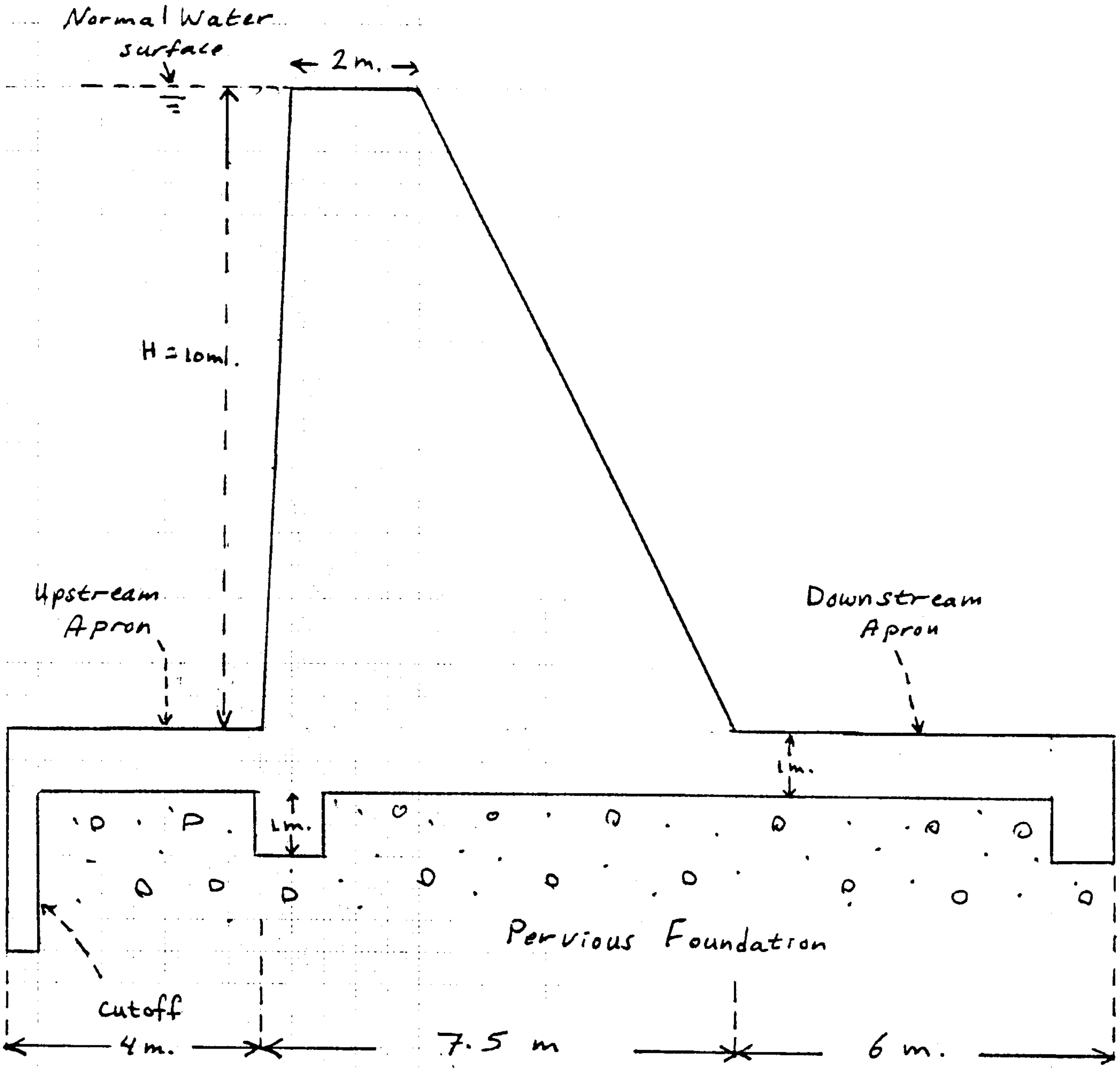


FIGURE (14) : CROSS SECTION FOR THE PROPOSED DAM

3-11 THICKNESS OF DAM CREST :

Triangular dam profiles are the most economical as they have the least volume. However the upper part of the structure, the crest, should be thick enough to resist horizontal side pressures from the silt-laden water behind the dam. The crest thickness (t) at the spillway can be determined from the following formula:

$$t = \frac{h.dw}{R.ds}$$

Where : dw and ds : are the specific gravity of water and concrete respectively; The specific gravity for muddy water is taken as 1.2 and that for the concrete as 2.4.

R : is the roughness coefficient of concrete against concrete and is taken as (0.60);

h : is the head of water overflowing the crest of the spillway, and is calculated from the discharge equation applying the design flood.

Applying this formula to the proposed dam for a crest (notch) length of (10)m, and a corresponding head above the notch crest of 2 m gives a thickness for the crest of (1.66)m, which will be taken as 2 m to provide higher safety.

3-12 THICKNESS OF THE DAM BASE AND DAM FOUNDATION :

The thickness of the dam base is determined on the basis of the stability conditions (Bearing capacity of the foundation material and seepage pressures and other uplift or overturning forces), as well as by the slope requirement of the upstream and down stream faces of the dam.

The foundation is designed so that the soils carrying capacity is not exceeded.

In the existing conditions in Wadi Jazir, where the foundation material is loose sand and gravel. The base width and surface contact area should provide the maximum spread of the weight of the structure on the foundation level. However as the load is directly transmitted to the soil level without any lateral friction, and as the alluvial channel width forms not more than 25% of the total dam width, and most of the structure weight will be loaded on the hard resistant rock on both sides of the alluvial channel, therefore the risk of foundation failure is very small.

For a 10-meter dam height (above ground surface) and a base width of 7.5 m, the load on the foundation from the concrete body of the dam only would be 1.35 kg/cm^2. The permissible load limit for loose gravel and course sand materials is (5 to 10) and (3 to 5) kg/cm^2 respectively. As the material at the dam site consists of mixed sand and gravel, an average permissible load limit would be (4-7) kg/cm^2. It is obvious that the actual load in our case (1.36 kg/cm^2) would be less than this limit, giving a safety factor more than 2.9.

For more safety the upstream sides can be sloped (upstream direction) by 1/20. This would decrease the over turning pressure around the downstream edge (the toe) particularly when the dam is full of water. Moreover, the down stream slope will be sloped at 0.5 (5/10).

Therefore, the base thickness at the dam would become 7.5 m.

In addition, the following components are recommended for the given reasons:

* Upstream apron to give more resistance against over turning around the toe when the dam is full of water, and to reduce downstream seepage pressure by increasing the length of the creep line (seepage line).
* Downstream apron o prevent scouring at the dam toe.
* Two 1-meter thick steps at the heel and toe of the dam to increase resistance against sliding at the foundation level.
* Contraction joints should be provided as necessary.
* Weeb holes as necessary.
* Bottom outlet to release the water to the downstream channel and recharge basins.

Figure (14) shows a cross section of the proposed dam main body.

3-13 **SPILLWAY** :

3-13-1 **General** :

The spillway would be a free flow ungated rectangular or trapezoidal section in the central part of the dam figure (15). Water discharging from the spillway will flow into a stilling basin leading to the main stream downstream of the dam, and then to recharge basins.

Alternatives for the length and depth of the spillway is given in tables (13 & 14). In the case of higher floods the whole dam crest would serve as a free flowing spillway without causing any damage to the solid, hard abutment bedrock.

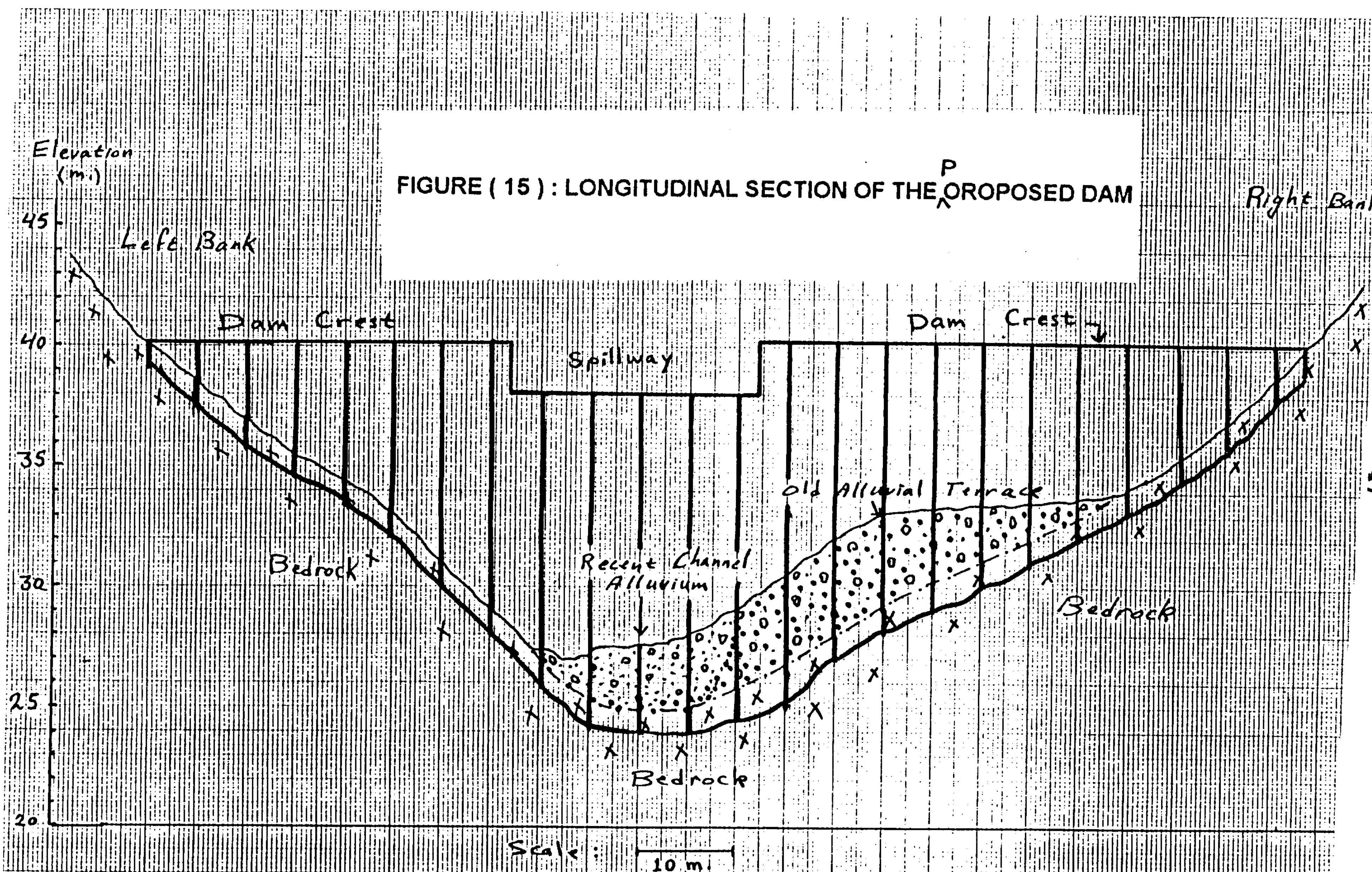


FIGURE (15) : LONGITUDINAL SECTION OF THE PROPOSED DAM

3-13-2 Spillway Hydraulic Calculations :

The spillway could be designed as rectangular or trapezoidal section. As the dam will be acting as an overflow weir, the flow rate would have little effect on the spillway cost. Therefore, the spillway will be designed to pass safely the maximum probable flood which was calculated as 59 m^3/sec.

For the rectangular section the following weir discharge formula is used to calculate the discharge capacity relations of the spillway:

$$Q = CLH$$

Where C : is the discharge coefficient, taken as 2.18,
Q : is the discharge in m^3/sec.,
L : is the width of the concrete weir,
H : is the water head over the weir crest.

For a trapezoidal section the following equation has been used:

$$H = [\ Q^2/(9*L^2)]^{1/3}$$

Where g: is the acceleration of gravity and the other terms are as described for the first equation. Tables & give the head of water flowing above the spillway crest for different alternatives of spillway lengths and for flood peaks of different return periods.

As the limiting factor is the width of the channel immediately downstream of the dam, a spillway crest width of 20 or 25 m may be selected, which would result in water heads of 1.22 and 1,05 m respectively for the rectangular section. (Tables 14 & 15).

The discharge-capacity rating of the proposed spillway is shown in tables (16 & 17) and figure (16)

3-14 LOW LEVEL OUTLET :

3-14-1 General :

The low level outlet consists of a submerged intake, a steel-lined pressure conduit and a discharge structure. The outlet works may be located in the right abutment.

The low level outlet works will serve (1) to evacuate, together with the spillway the reservoir down to Elevation of 28 m., and (2) to release up to 0.33 m^3/s for downstream groundwater recharge.

TABLE (14): HYDRAULIC CALCULATIONS FOR THE SPILLWAY
(RECTANGULAR SECTION -EQUATION :H=(Q/(2.18*L))^.666)

WIDTH O SPILLWA M.		CALCULATED HEAD OVER SPILLWAY (M.) FOR INICATED FOOD FLO (CM/SEC)			
	FLOW>>>>	16	28	42	59
	FREQUENCY>	100-YR	1000-YR	10000Y.	10^7-YR
10	***	0.81	1.18	1.55	1.94
15	***	0.62	0.90	1.18	1.48
20	***	0.51	0.74	0.98	1.22
25		0.44	0.64	0.84	1.05
30		0.39	0.57	0.74	0.93
35		0.35	0.51	0.67	0.84
40		0.32	0.47	0.61	0.77
45		0.30	0.43	0.57	0.71
50		0.28	0.40	0.53	0.66

TABLE (15): HYDRAULIC CALCULATIONS FOR THE SPILLWAY
(TRAPIZOIDAL SECTION) EQUATION:H=(Q^2/(G*L^2))^.333)

WIDTH O SPILLWA M.		CALCULATED HEAD OVER SPILLWAY (M.) FOR INICATED FOOD FLO (CM/SEC)			
	FLOW>>>>	16	28	42	59
	FREQUENCY>	100-Y	1000-	10000-	10^7-YR
10	***	0.64	0.93	1.22	1.53
15	***	0.49	0.71	0.93	1.16
20	***	0.40	0.58	0.77	0.96
25		0.35	0.50	0.66	0.83
30		0.31	0.45	0.58	0.73
35		0.28	0.40	0.53	0.66
40		0.25	0.37	0.48	0.61
45		0.23	0.34	0.45	0.56
50		0.22	0.32	0.42	0.52

TABLE (16): DISCHARGE RATING TABLE FOR RECTANGULAR SPILLWAY
(Q=2.18*L*H^1.5)

HEAD ABOVE CREST CM.	DISCARGE FOR THE GIVEN CREST WIDTHS (M)		
	15 CM/SEC	20 CM/SEC	25 M. CM/SEC
0	0.00	0.00	0.00
5	0.37	0.49	0.61
10	1.03	1.38	1.72
15	1.90	2.53	3.17
20	2.92	3.90	4.87
25	4.09	5.45	6.81
30	5.37	7.16	8.96
35	6.77	9.03	11.28
40	8.27	11.03	13.79
45	9.87	13.16	16.45
50	11.56	15.41	19.27
55	13.34	17.78	22.23
60	15.20	20.26	25.33
65	17.14	22.85	28.56
70	19.15	25.53	31.92
75	21.24	28.32	35.40
80	23.40	31.20	39.00
85	25.63	34.17	42.71
90	27.92	37.23	46.53
95	30.28	40.37	50.46
100	32.70	43.60	54.50
105	35.18	46.91	58.64
110	37.73	50.30	62.88
115	40.33	53.77	67.21
120	42.99	57.31	71.64
125	45.70	60.93	76.17
130	48.47	64.63	80.78
135	51.29	68.39	85.49
140	54.17	72.22	90.28
145	57.10	76.13	95.16
150	60.07	80.10	100.12
155	63.10	84.14	105.17
160	66.18	88.24	110.30
165	69.31	92.41	115.51
170	72.48	96.64	120.80
175	75.70	100.94	126.17
180	78.97	105.29	131.61
185	82.28	109.71	137.14
190	85.64	114.19	142.73
195	89.04	118.72	148.40
200	92.49	123.32	154.15

TABLE (17): DISCHARGE RATING TABLE FOR TRABIZOIDAL SPILLWAY
(Q=3.13*L*H^1.5)

HEAD ABOVE CREST CM.	DISCARGE FOR THE GIVEN CREST WIDTHS (M)		
	15 CM/SEC	20 CM/SEC	25 CM/SEC M.
0	0.00	0.00	0.00
5	0.52	0.70	0.87
10	1.48	1.98	2.47
15	2.73	3.64	4.55
20	4.20	5.60	7.00
25	5.87	7.83	9.78
30	7.71	10.29	12.86
35	9.72	12.96	16.20
40	11.88	15.84	19.80
45	14.17	18.90	23.62
50	16.60	22.13	27.67
55	19.15	25.53	31.92
60	21.82	29.09	36.37
65	24.60	32.81	41.01
70	27.50	36.66	45.83
75	30.49	40.66	50.82
80	33.59	44.79	55.99
85	36.79	49.06	61.32
90	40.09	53.45	66.81
95	43.47	57.96	72.46
100	46.95	62.60	78.25
105	50.51	67.35	84.19
110	54.17	72.22	90.28
115	57.90	77.20	96.50
120	61.72	82.29	102.86
125	65.61	87.49	109.36
130	69.59	92.79	115.98
135	73.64	98.19	122.74
140	77.77	103.70	129.62
145	81.98	109.30	136.63
150	86.25	115.00	143.75
155	90.60	120.80	151.00
160	95.02	126.69	158.37
165	99.51	132.68	165.85
170	104.07	138.75	173.44
175	108.69	144.92	181.15
180	113.38	151.18	188.97
185	118.14	157.52	196.90
190	122.96	163.95	204.93
195	127.85	170.46	213.08
200	132.79	177.06	221.32

The submerged intake consists of a fixed steel trashrack supported on a concrete frame. It will be located on the upstream face of the dam, and will have an intake at Elevation of 30 m. and the outlet at Elevation of 28 m.

The entrance to the low-level outlet conduit will be bell-mouth shaped to provide good entrance conditions and to minimize head losses. The bellmouth shape will change to a circular section.

An 400 mm diameter steel lined, 30 m long, pressure conduit will be constructed in a trench excavated in the rock and back filled with concrete. The conduit will extend from the intake to the downstream discharge structure located in the downstream side of the dam (20) meters from the dam toe. The conduit will lead to a rectangular 1.0 m high by 1.0 m wide section in the discharge structure. A gate value will be provided at the inlet of the conduit.

The discharge capacity of the facility at normal pool will be (0.33) m^3/s with the reservoir water surface level at Elevation of 38 m. Under maximum discharge conditions, the velocity in the conduit would be about 2.63 m/s ..

3-14-2 **Hydraulic Calculations for Bottom Outlets** :

The discharge capacity of the bottom outlet have been calculated using the Hazen williams formula:

P : $0.27853 * c * d^{2.63} * I^{0.54}$

Where Q: is the discharge (m^3/sec)
C: coefficient taken as (130)
d: diameter of pipe (m)
S: slope

Table (18) gives the discharge rate of the bottom outlet at different reservoir water levels, if a pipe line diameter of 400 mm is used. It also shows the emptying time of a full reservoir which will take about 22 days to be emptied.

3-15 ARTIFICIAL RECHARGE FACILITIES

The reservoir area of the proposed dam will constitute an important infiltration basin for groundwater recharge for the alluvial aquifer which underline the reservoir area and extends downstream for several Kilometers. As the clay and silt content of the suspended load of the flood water is almost negligible, and most of the sediment to be trapped by the reservoir would mostly consist of sand and gravel, reservoir sedimentation is not expected to significantly decrease the infiltration rate of the reservoir area, and consequently the reservoir

TABLE (18):CALCULATION OF DISCHARGE AND EMPTYING TIME FOR THE BOTTOM OUTLET IN WADI JAZIR DAM

ELEVATI	HEAD	TOTAL HEAD	SLOPE	DISCHAR	STORAGE OF 1-M. INTERVAL	EMPTYING TIME
M.	M.	M. ***		CM./SEC	C.M.	HOURS
28	0	0	0.00	0.00	0	
29	1	3	0.10	0.15	7000	12.83
30	2	4	0.13	0.18	11200	17.58
31	3	5	0.17	0.20	16000	22.26
32	4	6	0.20	0.22	20000	25.21
33	5	7	0.23	0.24	27200	31.55
34	6	8	0.27	0.26	40000	43.17
35	7	9	0.30	0.27	38200	38.69
36	8	10	0.33	0.29	50000	47.84
37	9	11	0.37	0.31	55000	49.98
38	10	12	0.40	0.32	75000	65.03
39	11	13	0.43	0.33	90000	74.74
40	12	15	0.50	0.36	120000	92.24
					549600	521 HOURS 21.7 DAYS

*** 2 METERS HEAD DUE TO PIPE SLOPE

area will continue effectively operating as an important recharge basin for several years. In addition, the occurrence of groundwater recharge within and immediately downstream of the reservoir will not adversely affect the natural recharge pattern and consequently the existing well field, but in fact will improve the situation. Particularly because the Wadi Jazir channel is the main subsurface flow zone for the recharged water, so that there wold be no other way for this water to escape away from the existing wellfield.

However, gradual emptying of the reservoir is required to provide additional storage capacity for the subsequent storm floods within the same winter season, and to reduce the time of exposure of the flood water to evaporation losses by accelerating infiltration rate.

In addition the shallow and narrow alluvial section below the dam body would have limited capacity for the infiltrating water in the reservoir area to flow downward beneath the dam to the main aquifer downstream.

Therefore additional recharge facilities will be needed immediately down stream of the reservoir. These facilities will be in the form of spreading basins across the natural channel. The larger the basins area the greater the recharge rate would be. Water released from the reservoir will flow through a series of three or more infiltration basins.

Assuming an infiltration rate of 1 meter per day (which is very reasonable), the total reservoir storage (0.4 MCM) will require an area of 20,000 m^2 to be flooded for 20 days in order to have this volume infiltrated into the alluvium.

The infiltration basins will be formed behind 2-3 meter high previous gabion dykes across the width of the natural channel. With a channel slope of 1 to 100, and average water depth in the basins of one meter, and a channel width of 25 meters, the required total basins' area would be 20,000 m^2, (20 hectares). The number of basins required will be four, with longitudinal spacing of 200 meters along the channel. However as the reservoir area would be used also for recharge, two or three additional basins will be sufficient.

The separating pervious overflow gabion dykes would be straight walls across the channel with one or two steps in the downstream direction.

3-16 SEEPAGE :

For storage reservoirs seepage means a loss of water and a source of worry on the performance of the dam which is supposed to be water tight. The proposed dam in Wadi Jazir is intended to be for flood detention and recharge purposes. The foundation is permeable alluvium. The permeadibility of such material may range from 5 to 10 meters per day in this location. The bedrock underlying the alluvial deposits is moderately weathered and jointed, and have much lower permeability than the alluvium. The rate of seepage through the foundation can be calculated using Darcy equation:

$$Q = KBIL$$

Where K : is the permeability
B : Average thickness (3m.)
I : The hydraulic gradient = highest water level divided by the seepage line (taken equal to the base width of the dam = 25 m).
L : is the width of the channel cross section, (taken as 25 m).

The rate of seepage is then calculated as 125 m^3/day and 250 m^3/day for permeabilities of 5 and 10 m/day and under fully developed seepage conditions. However, since the dam will be emptied in about 3 week period, and assuming fill-up total period of 50 days per year, fully developed seepage is not expected to occur, and a lower estimate would result. The total seepage quantity from the dam would be 100,000 m^3/season on the average.

The most serious in recharge dams are the seepage forces which if exceed the resistance of a soil particle for movement, it will be forced to move, and piping will occur.

The following precautions have been considered in the proposed Wadi Gauzier dam to control the exit gradient of seepage water:

1- An upstream apron extending for 4 meters up stream of the dam heel, with a one meter thick step at its upstream end.

2- A downstream apron, 6 m long with one-meter deep step at the downstream toe of the dam.

3-17 GABION DAM ALTERNATIVE :

A second alternative to concrete gravity dam is to build a gabion dam. It will have the same height but larger width and larger spillway.

Calculations for gabion structures should give particular importance to the apparent specific gravity of the gabion (89), which is a function of the stones specific gravity (ds), size and porosity (ds) for granite is 2600 kg/m^3 which will be considered for the igneous rocks at the dam site.

The specific gravity of silt loaded flood water is very often taken as 1100 to 1200 kg per m^3. with a porosity $^{(n)}$ of 35% for the rock filled gabions the apparat specific gravity is calculated as:

$$dg = s\,(i - n) = 2600\,(1 - 0.35)$$
$$= 1200 \text{ kg/m}^3$$

The proposed dam would be a stepped structure from both the upstream and downstream sides. This design would provide higher resistance to over-turning pressures to either side. Calculations of crest width and step widths are given in the following paragraphs.

Using the commercial gabion which is one-meter high, a 10-meter high dam would require 10 steps. In order to provide friction resistance at the foundation level, two more steps are added to the central three walls at the base, sunk in the channel alluvium, across the alluvial channel.

The central wall of the dam should be tied up with the bedrock across the channel sides and in the abutment through a one-meter deep excavated trench. Another one meter step (cutoff) should also be sunk in the alluvium below the down stream end of the embankment.

In a gabion dam it is important to have a long stilling pool apron with a cutoff, deep enough to prevent under cutting. A 10-meter long stilling basin is recommended for the gabion dam with a one meter cutoff steps and one-meter positive step as energy dissipator at the down stream end.

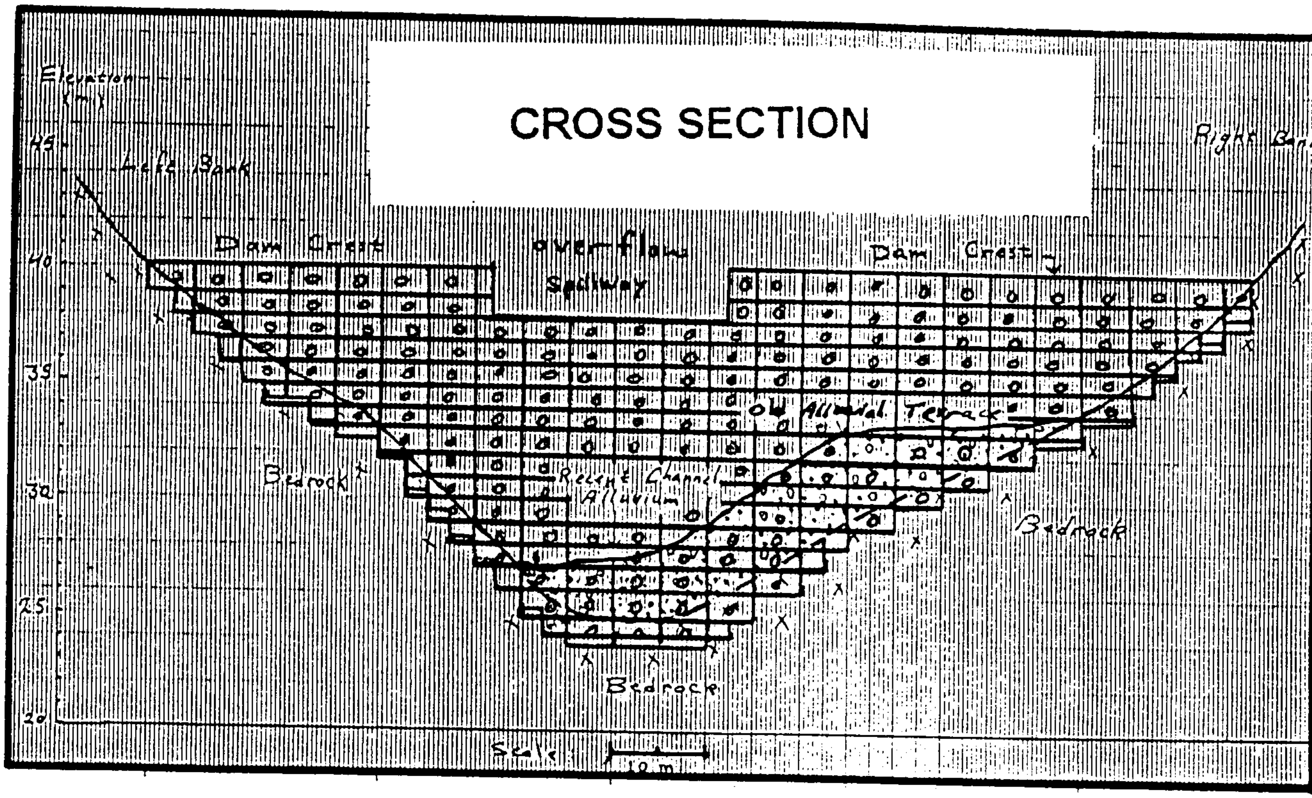
For commercial gabion dimension, the step width is taken as 1 m for the down stream face, and 1/2 m for the upstream face. The dam crest will be taken as 4 meters. Figure (16) shows the geometry of the proposed dam.

For the wings on either side of the over flow section, higher rows' width can be reduced by one meter for each row or step.

The spillway in this case should be broader than in the concrete dam as given below for the 10,000 year flood peak (59 m^3/sec):

Width of Spillway m	Water Height above Crest m.
25	1.36
80	1.59
15	1.92

(A roughness coefficient of 1.5 is taken in the discharge equation).



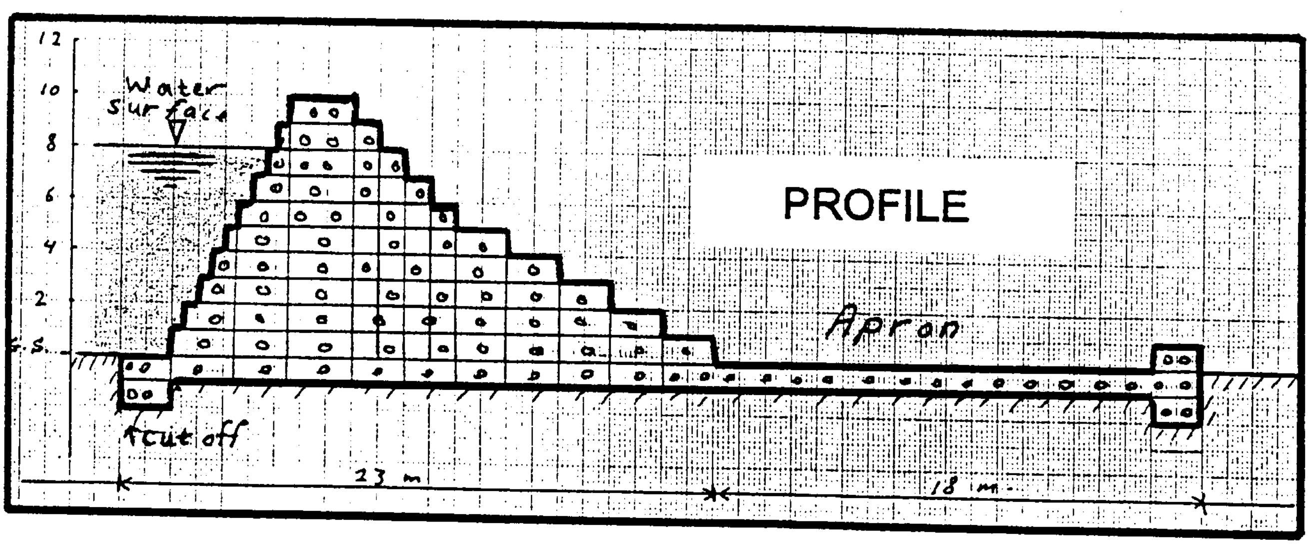


FIGURE (16) : GEOMETRY OF THE SECOND ALTERNATIVE GABION DAM

ANNEXES

ANNEX (1)

ALL THE AVAILABLE MONTHLY RAINFALL TOTALS (mm)
FOR THE PERIOD 1967/68 - 1991/92
STATION : MASFUT

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	YEARLY Total
1967-1968	0.0	0.0	1.6	17.8	48.0	0.0	25.4	0.0	0.0	0.0	9.7	6.6	[illegible]
1968-1969	0.0	0.0	19.8	96.0	14.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	130.4
1969-1970	-	-	-	-	-	-	-	-	-	-	-	-	49.8
1970-1971	-	-	-	-	-	-	-	-	-	-	-	-	6.3
1971-1972	0.0	12.2	6.6	14.4	3.2	124.3	12.9	0.0	0.0	0.0	0.0	60.2	233.8
1972-1973	3.2	0.0	1.1	0.0	Tr	0.0	0.0	0.0	0.0	0.0	0.0	0.0	4.3
1973-1974	0.0	0.0	0.0	42.0	41.3	0.0	1.3	0.0	0.0	0.0	0.0	10.2	94.8
1974-1975	62.3	0.0	15.2	103.0	229.0	0.0	0.8	27.0	11.9	4.6	25.0	0.0	478.8
1975-1976	0.0	0.0	2.4	9.0	126.2	117.6	33.4	0.0	0.0	12.2	24.3	0.0	325.1
1976-1977	43.4	27.5	30.0	29.3	29.9	1.4	26.4	4.8	4.1	0.0	0.0	0.0	196.8
1977-1978	15.0	1.2	0.8	5.6	30.0	6.0	1.8	0.0	0.0	1.8	4.6	Tr	66.8
1978-1979	0.0	0.0	0.0	4.8	0.2	22.8	2.4	0.0	3.8	0.0	12.0	0.0	46.0
1979-1980	18.0	0.0	82.8	7.2	14.2	14.6	0.0	0.0	0.0	13.4	0.0	0.0	150.2
1980-1981	0.0	0.4	0.2	12.0	2.2	18.4	21.0	58.0	0.0	0.0	0.0	0.0	112.2
1981-1982	0.6	0.4	0.4	7.8	176.2	150.6	0.2	0.0	5.4	0.0	0.0	0.0	341.6
1982-1983	4.2	4.6	16.8	20.2	68.8	54.2	62.2	7.2	0.0	0.0	26.0	0.4	264.6
1983-1984	0.0	0.4	1.8	0.8	1.4	6.2	0.0	1.2	0.0	6.8	0.0	0.0	18.6
1984-1985	0.0	0.0	4.8	7.4	0.6	1.0	0.8	0.0	0.0	0.0	0.0	0.0	14.6
1985-1986	0.6	0.2	2.4	29.2	17.8	11.2	1.2	0.0	0.0	4.6	23.6	1.6	92.4
1986-1987	0.4	0.0	40.2	1.2	16.8	175.4	18.0	0.0	2.4	0.0	38.6	0.0	293.0
1987-1988	0.0	0.0	5.8	8.4	268.6	0.0	17.0	0.0	0.0	6.2	0.0	0.0	306.0
1988-1989	0.4	0.0	0.4	0.0	21.8	45.0	11.8	0.0	0.0	0.0	0.0	0.0	79.4
1989-1990	0.4	20.4	31.8	16.8	62.2	1.8	24.0	6.8	0.0	19.6	0.0	1.2	185.0
1990-1991	0.2	0.4	0.8	10.8	19.2	68.8	0.4	0.0	0.0	0.0	0.0	0.2	100.8
1991-1992	0.4	2.0	7.8	40.3	30.4	3.2	47.0	0.0	0.0	0.4	0.0	0.0	131.1
AVERAGE	6.5	3.0	11.9	21.0	53.2	35.8	13.4	4.6	1.2	3.0	7.1	3.6	154.2

ANNEX (2)

ALL THE AVAILABLE MONTHLY RAINFALL TOTALS (mm)

FOR THE PERIOD 1971/72 - 1991/92

STATION : HOWEILAT

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	YEARLY TOTAL
1967-1968													
1968-1969													
1969-1970													
1970-1971	-	-	-	-	-	-	-	-	-	-	-	-	1.2
1971-1972	0.6	0.0	4.5	18.2	0.0	124.2	0.0	0.0	Tr	0.0	0.0	0.0	147.5
1972-1973	0.0	0.0	10.0	70.3	0.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	81.2
1973-1974	0.0	0.0	0.0	26.9	-	0.0	0.0	0.0	0.0	0.0	0.0	22.0	-
1974-1975	-	-	-	15.2	0.0	0.0	0.0	3.2	0.0	0.0	-	-	-
1975-1976	0.0	0.0	0.0	15.0	180.0	114.0	15.0	0.0	0.0	0.0	-	-	-
1976-1977	5.0	30.0	20.0	74.0	14.7	30.3	36.7	16.5	Tr	0.0	0.0	0.0	227.2
1977-1978	0.0	0.0	0.0	0.0	20.0	0.0	0.0	0.0	0.0	2.0	0.0	0.0	22.0
1978-1979	0.0	0.0	1.0	15.8	0.0	46.8	0.0	3.8	0.4	0.0	0.0	0.0	67.8
1979-1980	26.4	0.0	91.6	4.6	7.2	18.2	0.0	0.0	0.0	5.8	0.0	0.0	153.8
1980-1981	0.0	2.4	0.0	7.2	0.0	20.2	34.6	46.2	0.0	0.0	0.0	0.0	110.6
1981-1982	0.0	0.2	0.2	13.2	175.8	149.4	0.0	0.0	0.0	0.0	0.0	0.0	338.8
1982-1983	12.6	24.6	21.4	11.8	107.2	85.0	69.6	1.4	0.0	0.0	31.0	0.0	364.6
1983-1984	0.0	0.4	0.2	0.6	0.4	3.2	0.0	1.8	0.0	0.0	0.0	0.2	6.8
1984-1985	0.0	0.0	1.0	10.4	0.0	0.2	0.6	0.0	0.0	1.8	0.0	0.0	14.0
1985-1986	0.4	2.6	0.2	31.2	25.6	10.0	0.2	0.0	0.0	0.0	33.2	0.0	103.4
1986-1987	7.6	0.0	11.0	0.4	7.4	184.6	43.8	5.4	2.8	0.0	10.0	0.0	273.0
1987-1988	0.0	9.6	5.6	4.0	317.0	0.2	25.0	0.0	0.0	10.0	0.0	0.0	371.4
1988-1989	0.0	0.0	5.4	0.0	21.8	52.2	9.2	0.0	0.0	0.0	0.0	0.0	88.6
1989-1990	0.0	28.2	51.4	10.6	91.8	1.8	19.2	0.0	0.0	0.8	0.6	1.4	205.8
1990-1991	0.0	0.0	0.2	3.8	24.6	48.2	0.0	0.0	0.0	0.0	0.2	0.8	77.8
1991-1992	0.8	9.6	15.8	43.6	29.8	2.8	71.4	2.2	0.0	0.2	0.0	0.0	176.2
AVERAGE	2.7	5.4	12.0	17.9	51.2	42.4	15.5	3.8	0.2	1.0	3.9	1.3	149.0

ANNEX (3)

ALL THE AVAILABLE MONTHLY RAINFALL TOTALS (mm)

FOR THE PERIOD 1971/72 - 1991/92

STATION : MUNAI

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	YEARLY TOTAL
1967-1968													
1968-1969													
1969-1970													
1970-1971													
1971-1972	4.3	0.0	29.2	0.0	0.9	120.2	0.0	0.0	0.0	Tr	0.0	0.0	154.6
1972-1973	2.3	2.0	30.0	52.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	86.5
1973-1974	0.0	0.0	0.0	33.7	12.5	0.0	0.0	0.0	0.0	0.0	0.0	20.0	66.2
1974-1975	-	-	-	24.2	48.1	0.0	0.0	48.0	3.0	0.0	-	-	
1975-1976	3.0	0.0	0.0	7.0	85.0	65.0	-	-	-	-	-	-	
1976-1977	-	-	-	-	-	-	-	26.4	0.0	0.0	0.0	0.0	-
1977-1978	0.0	29.2	1.5	0.0	40.6	0.0	0.0	0.0	0.0	2.8	0.8	0.0	74.9
1978-1979	0.0	0.0	0.6	16.8	0.0	41.2	0.0	1.6	20.2	0.0	0.0	0.0	80.4
1979-1980	54.0	0.0	80.2	13.0	17.2	14.0	0.0	0.0	0.0	41.8	0.0	7.0	227.2
1980-1981	0.0	0.0	0.6	8.2	0.0	20.0	21.4	53.8	0.0	0.0	0.0	2.2	106.2
1981-1982	0.0	0.2	0.0	18.6	170.6	73.6	1.8	0.0	0.0	0.0	0.0	0.0	264.8
1982-1983	0.6	27.2	20.6	10.8	73.6	33.2	71.2	8.8	0.0	0.0	35.6	0.0	281.6
1983-1984	0.0	0.2	2.6	0.8	0.0	9.6	0.0	0.0	0.0	0.2	0.4	21.4	35.2
1984-1985	0.0	0.0	6.0	6.2	0.0	0.6	0.0	0.0	0.0	0.0	0.0	0.0	12.8
1985-1986	0.6	5.4	0.2	24.6	18.8	7.4	2.0	0.0	0.0	0.0	2.2	7.8	69.0
1986-1987	3.5	0.8	16.0	0.4	2.2	157.2	18.2	8.8	2.0	0.0	1.8	0.0	210.9
1987-1988	0.0	5.0	7.2	7.8	256.8	0.6	24.8	0.0	0.0	4.0	0.0	0.0	306.2
1988-1989	0.0	0.0	6.8	0.0	10.4	49.0	10.2	0.0	0.0	0.0	0.0	0.0	76.4
1989-1990	0.0	24.2	41.8	10.6	101.4	1.2	21.4	0.0	3.2	3.6	0.4	0.0	207.8
1990-1991	0.0	0.0	0.2	6.6	29.2	53.2	0.0	0.0	0.0	0.0	0.0	0.0	89.2
1991-1992	1.8	19.7	13.4	46.2	37.2	4.8	35.2	0.0	0.0	0.0	0.2	0.0	158.3
AVERAGE	3.7	6.0	13.5	14.4	45.2	32.5	10.9	7.8	1.5	2.8	2.2	3.1	139.3

Annex (4)

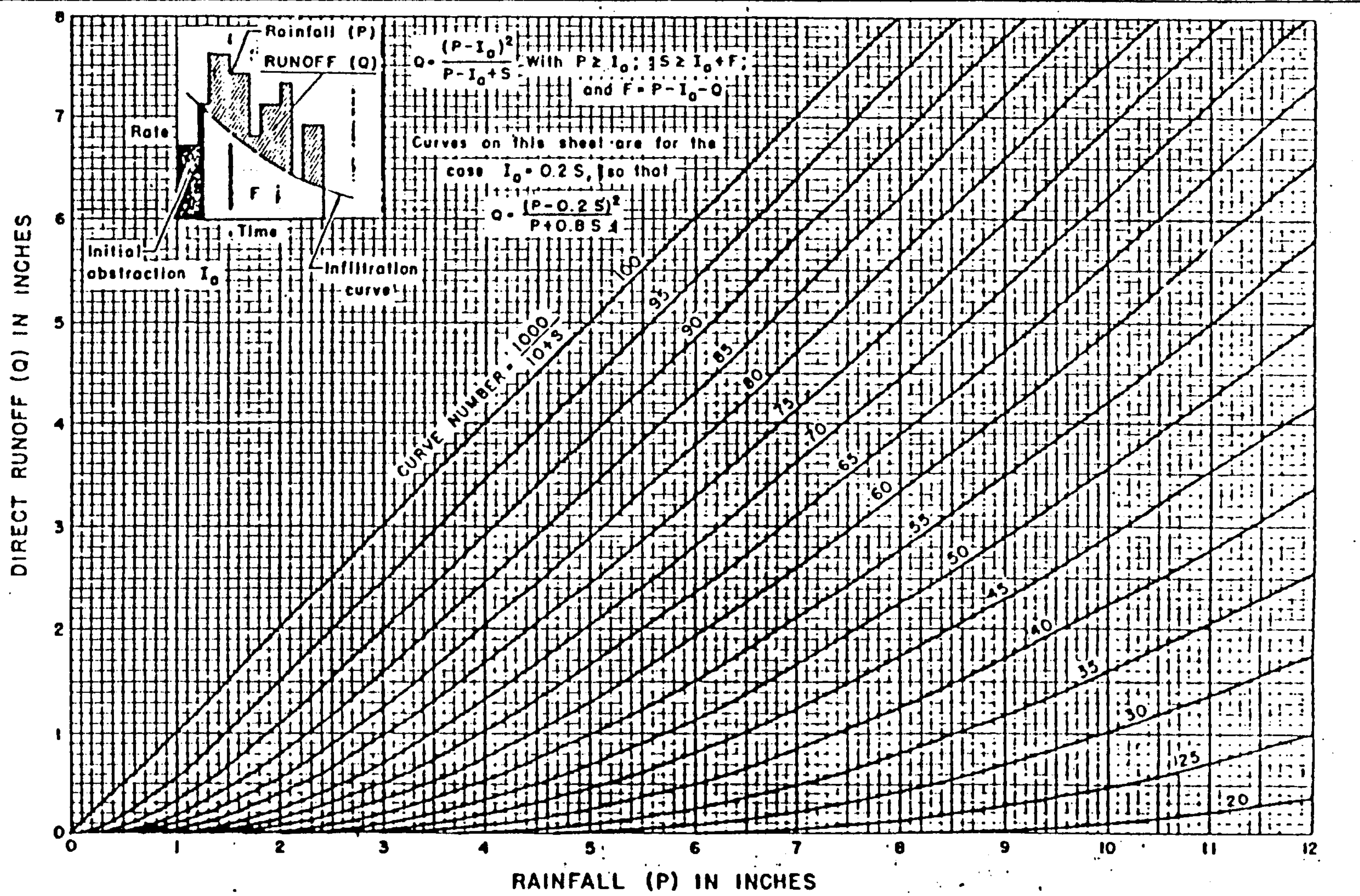
Hydrologic Soil Groups*

Group	Runoff Potential	Description
A	Low	Soils having high infiltration rates even when thoroughly wetted and consisting chiefly of deep, well to excessively drained sands or gravels. These soils have a high rate of water transmission.
B	Low-moderate	Soils having moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.
C	Moderate-high	Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture. These soils have a slow rate of water transmission.
D	High	Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

* As defined by the Soil Conservation Service (1972).

ANNEX 5. SOLUTION OF RUNOFF EQUATION $Q = \frac{(P-0.2S)^2}{P+0.8S}$

P = 0 to 12 inches
Q = 0 to 8 inches



ANNEX (6)
Curve Numbers For Different Soil Group

VEGETATION TYPE OR LAND USE	TREATMENT OR PRACTICE	HYDROLOGIC CONDITION	HYDROLOGIC SOIL GROUP* A	B	C	D
GRASSLAND		POOR	68	79	86	89
		FAIR	51	69	79	84
		GOOD	37	61	74	80
	CONTOURED	POOR	47	67	78	83
	CONTOURED	FAIR	38	61	74	80
	CONTOURED	GOOD	28	55	70	77
HERBACEOUS		POOR	68	80	87	90
		FAIR	55	71	81	85
		GOOD	42	63	74	81
DESERT BRUSH		POOR	68	80	87	90
		FAIR	52	70	80	85
		GOOD	39	62	75	80
SAGEBRUSH		POOR	47	67	78	83
		FAIR	-	48	65	74
		GOOD	-	30	53	64
PINYON-JUNIPER		POOR	60	75	83	87
		FAIR	34	58	73	78
		GOOD	-	41	61	70
CHAPARRAL(ARIZ.)		POOR	68	80	87	93
		FAIR	32	57	71	83
		GOOD	-	41	58	74
OAK-ASPEN		POOR	43	64	76	82
		FAIR	-	47	64	73
		GOOD	-	30	53	64
PONDEROSA PINE		POOR	45	66	77	83
		FAIR	29	56	70	77
		GOOD	-	46	64	72
FOREST		POOR	45	66	77	83
		FAIR	36	60	73	79
		GOOD	25	55	70	77
ROADS(DIRT)		-	72	82	90	92
BARE ROCK		-	96	96	96	96
WATER SURFACES		-	100	100	100	100

NOTE:FOR THE AGRICULTURAL PRACTICES ABOVE,POOR HYDROLOGIC CONDITION MEANS LESS THAN 20 PERCENT AND GOOD HYDROLOGIC CONDITION MEANS MORE THAN 20 PERCENT OF THE SURFACE IS COVERED WITH RESIDUE.

ANNEX (6) (cont).

VEGETATION TYPE OR LAND USE	TREATMENT OR PRACTICE	HYDROLOGIC CONDITION	HYDROLOGIC SOIL GROUP* A	B	C	D
MINED LANDS	CONTOUR FURROWING	MAX	25	57	70	73
	CONTOUR FURROWING	MIN	47	67	81	88
	IMPRINTING	MAX	30	58	71	78
	IMPRINTING	MIN	47	67	81	88
	PITTING	MAX	34	59	72	75
	PITTING	MIN	57	73	83	88
	RIPPING	MAX	33	61	74	80
	RIPPING	MIN	68	79	86	89
FALLOW	STRAIGHT ROW	-	77	86	91	94
	STRAIGHT ROW AND CONSERV.TILLGE	POOR	75	84	89	92
	STRAIGHT ROW AND CONSERV.TILLGE	GOOD	74	83	87	90
ROW CROPS	STRAIGHT ROW	POOR	72	81	88	91
	STRAIGHT ROW	GOOD	67	78	85	89
	STRAIGHT ROW AND CONSERV.TILLGE	POOR	71	79	86	89
	STRAIGHT ROW AND CONSERV.TILLGE	GOOD	64	75	82	85
	CONTOURED	POOR	70	79	84	88
	CONTOURED	GOOD	65	75	82	86
	CONTOURED AND CNOSERV.TILLAGE	POOR	69	78	83	87
	CONTOURED AND CNOSERV.TILLAGE	GOOD	64	74	80	84
	CONTOURED AND TERRACES	POOR	66	74	80	82
	CONTOURED AND TERRACES	GOOD	62	71	78	81
	CONTOURED,TERRACES,AND CONSERV.TILLAGE	POOR	65	73	79	81
	CONTOURED,TERRACES,AND CONSERV.TILLAGE	GOOD	61	70	76	79
SMALL GRAINS	STRAIGHT ROW	POOR	65	76	84	88
	STRAIGHT ROW	GOOD	63	75	83	87
	STRAIGHT ROW AND CONSERV.TILLGE	POOR	64	74	82	86
	STRAIGHT ROW AND CONSERV.TILLGE	GOOD	60	72	80	84
	CONTOURED	POOR	63	74	82	85
	CONTOURED	GOOD	61	73	81	84
	CONTOURED AND CNOSERV.TILLAGE	POOR	62	73	81	84
	CONTOURED AND CNOSERV.TILLAGE	GOOD	60	72	79	82
	CONTOURED AND TERRACES	POOR	61	72	79	82
	CONTOURED AND TERRACES	GOOD	59	70	78	81
	CONTOURED,TERRACES,AND CONSERV.TILLAGE	POOR	60	71	78	81
	CONTOURED,TERRACES,AND CONSERV.TILLAGE	GOOD	58	69	76	79

NOTE: FOR THE AGRICULTURAL PRACTICES ABOVE, POOR HYDROLOGIC CONDITION MEANS LESS THAN 20 PERCENT AND GOOD GOOD HYDROLOGIC CONDITION MEANS MORE THAN 20 PERCENT OF THE SURFACE IS COVERED WITH RESIDUE.

* SEE ANNEX 5-3 FOR DEFINITION OF HYDROLOGIC SOIL GROUPS.

ANNEX (7 : a, b, c)

DERIVATION OF SYNTHRTIC UNIT HYDROGRAPH (Snyder's Method)

a): The lag time (TL) of the basin is calculated from Synder's equation $TL = CT\ (L * Lc)^{0.3}$

Where:

L = Total length of the main stream from the divide to the proposed dam site.

LC= Distance from the dam to a point on the main stream nearest to the centroid of the basin.

CT= Constant where ($1 < CT < 2.2$) lower values for steeper slopes. For Munayi catchment it was taken as 1.2 because of relatively steep conditions.

b) The standard duration of rainfall (TR) was proposed by Synder as:

$$TR = \frac{TL}{5.5} \text{ (hours)}$$

c) The synthetic Unit hydrograph (SUH) peak (QP) for rains of (TR) duration in m^3/s may be obtained from the equation.

$$QP = \frac{CP * A}{TL}$$

Where:

A = Basin area (km^2)

CP = Coefficient ranging form 2 to 2.5 for metric system (1 cm U.H), and was accepted as 2 for Munayi catchment.

QP = Flood peak in m^3/s

For a one hour hydrograph:

$T'R = 1hr$

$$T'L = TL + \frac{T'R - TR}{4}$$

$$Q'P = \frac{CP * A}{T'L} \ (m^3/sec)$$

b: DERIVATION OF SYNTHETIC UNIT HYDROGRAPH

```
*******************************************
CT =1.8
L  =4.5 K
LC =1.75 KM
A  =4.5 SQ.KM

===============================================================================:
TIME LAG -HRS :FOR THE STANDARD DURATION =CT*(L*LC)^0.3
TL     =       3.34
===============================================================================:
STANDARD DURATION (TR) =TL/5.5555
TR =           0.60
===============================================================================:
FOR A ONE HOUR HYDROGRAPH   T'R =1 HR.
TIME LAG:
T'L =TL+(T'R-TR)/4
T'L     =       3.8
===============================================================================:
PEAK DISCHARGE FOR THE ONE-HR HYDROGRAPH
Q'P     =CP*A/T'L  (CM/SEC     2.37 C.M./SEC

===============================================================================:
TIME FROM THE BEGINNING OF THE RISING LIMB TO THE PEAK .T.=TL+T'R/2
TP=TL+T'R/2
TP      =       3.84    HRS
===============================================================================:
TIME BASE -DAYS
TB=1+1*T'L/24
TB      =       1.16 DAYS        27.84 HRS

===============================================================================:
```

C : DERIVATION OF THE UNIT HYDROGRAPHUSING
GENERALISED DIMENTIONLESS UNIT HYDROGRAPH

TP1=4.2 Q'P= 2.38

T/TP	Q/QP	T (HRS)	Q (CM/SEC)
0.00	0.000	0.000	0.000
0.10	0.015	0.384	0.036
0.20	0.075	0.768	0.178
0.30	0.160	1.152	0.379
0.40	0.290	1.536	0.687
0.50	0.430	1.920	1.019
0.60	0.600	2.304	1.422
0.70	0.770	2.688	1.825
0.80	0.890	3.072	2.109
0.90	0.970	3.456	2.299
1.00	1.000	3.840	2.370
1.10	0.980	4.224	2.323
1.20	0.920	4.608	2.180
1.30	0.840	4.992	1.991
1.40	0.750	5.376	1.778
1.50	0.650	5.760	1.541
1.60	0.550	6.144	1.304
1.80	0.420	6.912	0.995
2.00	0.320	7.680	0.758
2.20	0.240	8.448	0.569
2.40	0.180	9.216	0.427
2.60	0.130	9.984	0.308
2.80	0.088	10.752	0.209
3.00	0.055	11.520	0.130
3.50	0.036	13.440	0.085
4.00	0.018	15.360	0.043
4.50	0.009	17.280	0.021
5.00	0.004	19.200	0.009

ANNEX (8)

Methods used for Calculations for Embanlement Design.

(1) Crest Width:

Various emperical formulas have been suggested for determination of crest width. For example, the U.S. Bureau of redamation proposed the following formula

$$B = \frac{H_d}{5} + 3$$

Where, B is the top width in meters, and Hd is the dam height in meters. A minimum of 3 m. is usually required for maintenance.

(2) Width of bottom of cutoff trench (w):

w = h-d

where h = reservoir head above ground surface

d = depth of cutoff trench excavation below ground surface.

(3) Wind Tide:

Wind tides are commonly estimated using the following formula:

$$z = \frac{Vw^2 F}{1400b}$$

where, (z) is the rise of waves above still water level in feet, (Vw) is the wind speed in miles per hour, (F) is the fetch, or length of water surface over which the wind blows in miles, and (b) is the average depth of the lake along the fetch in feet. Wind-tide effects may be transferred around bends in reserviors, and the value of the fetch used may be longer than the straight-line fetch.

(4) Wave Height (Zw):

The following equation is used to determine the wave height:

$$Z_w = 0.034\ Vw^{1.06}\ F^{0.47}$$

where, (Zw) is the average height in feet, of the highest one third of the waves and is called the significant wave height, (Vw) is the wind velocity in miles per hour, about 25 feet above the water surface, and F is the fetch in miles.