INSTITUTE of HYDROLOGY

FLOOD ESTIMATES FOR
MUSHWAB DAM
JEDDAH

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CONTENTS

1. INTRODUCTION

The Institute of Hydrology were approached by Sir Alexander Gibb and Partners in December 1983 and asked to estimate the stormwater runoff from the catchment area above the existing Mushwab dam. This study is necessary because of proposed development in the catchment upstream of the dam, noteably that of the King Abdul Aziz University teachers and students compound. Further urban development of much of the flat land within the catchment seems likely and predictions of possible future stormwater runoff were required for the design and economic evaluation of stormwater protection works.

No flow data were available from the catchment or from adjacent wadi systems, but rainfall data for the original Jeddah airport are available since 1961. The Institute of Hydrology in association with the Hydraulics Research Station at Wallingford, studied the storm rainfall for Jeddah in 1982 and their work is reported on in HRS report No. EX 1103 of March 1983. We have made extensive use of that report in the present study.

The only other information available to us was in the form of the 1:25,000 scale topographic maps of the area and a part copy of Jeddah Master Plan Mapping prepared by Somait Engineering Services in 1983 which showed expected future development in the Mushwab valley.

It was decided that the most appropriate means of estimating stormwater runoff from the available information, would be use of a synthetic unit hydrograph and the storm rainfall studies contained in our previous report, EX 1103.

2. DERIVATION OF SYNTHETIC UNIT HYDROGRAPH

2.1 Present "undeveloped" case

Because no flow data were available from the Mushwab dam catchment, it was necessary to derive a synthetic unit hydrograph from topographic information. A unit hydrograph is an expression of how a catchment would be expected to respond to a "unit" input of effective rainfall in "unit" time. The unit rainfall in this study is 1 mm and the unit time chosen is 30 minutes.

Effective rainfall is that proportion of the total storm rainfall remaining once all losses such as infiltration, depression storage and evaporation have been removed. This topic is discussed later.

Where no concurrent rainfall and flow records are available, as in the present study, a unit hydrograph may be derived from the stream length and slope derived from topographic maps. A number of empirical formulae exist whereby such synthetic unit hydrographs may be estimated. These empirical formulae have been developed for countries such as the U.K. and U.S.A. However, the most important parameter is the lag time between the causative rainfall and resulting runoff, which is a function of the physical parameters of channel length, slope and to an extent channel roughness. Because these physical parameters are not unique to any particular country or climate, the formulae may, with caution, be applied to the Mushwab dam catchment.

From the available 1:25,000 scale maps, the main stream length, L, was measured as 9.5 kms. A number of slope parameters have been considered by various authors. Of these the most common is the difference in height between the point at which the main stream projected would cut the watershed and the height of the dam divided by the main stream length, L. This slope equals 12.5 m/km and is called S in the present study. The U.K. Flood Studies Report used the slope between points 10 per cent and 85 per cent of the way upstream of the dam along the stream channel. This is called S1085 in the present study and is 8.56 m/km for the Mushwab catchment.

From the U.K. Flood Studies Report, two formulae are available to estimate the time to peak of the unit hydrograph, Tp. These are;

$$Tp = 20.46\ S1085^{-0.598} = 5.67 \text{ hours}$$

$$Tp = 2.8 \left(\frac{L}{\sqrt{S1085}}\right)^{0.47} = 4.87 \text{ hours}$$

A report by Packman (1980) gives a number of other empirical formulae in an Appendix for the lag time of a catchment, T_L. The majority of these fall into two classes; those based, as is the second formula above, on $\frac{L}{\sqrt{S}}$ and those developed from the Snyder method. Snyder uses mainstream length, L, and also the length from the dam to the centre of gravity of the catchment, L_{Ca}. His method does not use slope directly, rather his multiplying coefficient C_t is an empirical function of slope which is chosen subjectively. The formula is;

$$T_L = C_t \, (LL_{Ca})^{0.3}$$

Ct is normally taken as 2.0 for mountainous regions such as the Appalachians in the U.S.A. where the method was derived. With a value of Ct = 2.0, and L_{Ca} of 3.1 miles.

$$T_L = 4.81 \text{ hours.}$$

Of the formulae using the ratio $L/\sqrt{S}$, a range of values of T_L from about 2.6 to 5.8 hours emerges with a mean of 4.1 hours from the seven formulae used. Several of these formulae are developed principally for urbanised catchments and will tend to predict rather short lag times.

The lag time of a catchment is normally rather greater than the unit hydrograph T_p and the U.K. Flood Studies Report determined a relationship for the U.K. of

$$T_p = 0.9 \, T_L$$

From the lag results above, an appropriate T_p might be some 4 to 4.5 hours whilst the two U.K. Flood Studies Report formula suggest rather higher results. A unit hydrograph time to peak for the existing undeveloped Mushwab dam catchment was subjectively chosen as 4.5 hours from the above analyses.

From the U.K. Flood Studies Report, the synthetic unit hydrograph may be approximated as a simple triangle from the single parameter T_p. The time base, TB, is determined as;

$$TB = 2.525T_p = 11.4 \text{ hours.}$$

It is convenient to adopt a time base that is a multiple of the 0.5 hour time unit chosen, so TB was taken to be 11.5 hours. The unit hydrograph peak, Qp, is determined from the knowledge that 1 mm of rainfall in 30 minutes on a catchment area of 37.9 km^2 yields 37900 m^3. For a triangular unit hydrograph with a time base of 11.5 hours,

$$Qp = \frac{37900 \times 2}{11.5 \times 3600} = 1.83 \ m^3/sec.$$

Thus the adopted 1 mm, 30 minute triangular unit hydrograph for the present, undeveloped case has been taken to have the following parameters;

$Tp = 4.5$ hours, $TB = 11.5$ hours, $Qp = 1.83 \ m^3/sec.$

2.2 Developed or urbanised case

The extent to which the Mushwab dam catchment will become developed or urbanised in the future is difficult to assess. The Jeddah Master Plan mapping prepared by Somait Engineering Services in 1983 suggests that the majority of the low lying land in the catchment will eventually be covered by relatively low density development. It has been assumed here that all land up to approximately the 75 m contour will be developed and that higher land with steeper slopes will remain largely in its present natural state. This implies that some 40 per cent of the catchment will eventually be urbanised.

The effects of this urban development will be to shorten the lag time and time to peak of the unit hydrograph and also to increase the volume of runoff from any storm rainfall.

Packman (1980) studied a large number of papers and reports to examine how urbanisation might affect both unit hydrograph shape and also runoff volume.

From his results we have assumed that if a catchment becomes 40 per cent urbanised, the unit hydrograph time to peak will be 60 per cent of that for the undeveloped case. Thus for Mushwab dam, Tp^1 for the urban case will be;

$$Tp^1 = 0.6Tp = 0.6 \times 4.5 = 2.7 \text{ hours.}$$

For convenience, this is taken as a round multiple of the time unit being used, 30 minutes. Thus Tp^1 is taken as 2.5 hours, and TB taken as 6.5 hours. As for the rural case, Qp may be determined from the known volume of runoff and the time base

$$Qp = 3.38 \ m^3/sec.$$

3. RAINFALL

3.1 *Data available*

This report draws heavily on our earlier work on rainfall for Jeddah reported on in HRS Report No. EX 1103 of March 1983. For return periods of up to 20 years and durations up to 3 hours, the results of Table 4.6 of this report have been utilised. For return periods of 50 and 100 years, the generalised point rainfall depth - frequency relationships of Bell (1969) have been used. Packman showed in Report No. EX 1103 that these generalised rainfall relationships were not particularly good at estimating rainfall depths of any specified duration and return period, when compared with estimates from observed data. Nevertheless, the generalised relationships provide an objective means of estimating rainfalls of durations greater than 3 hours and return periods of 50 and 100 years where no adequate data exist.

3.2 *Design storm rainfalls*

Report EX 1103 shows that for Jeddah, the rainfall depth-duration curves become almost flat for durations of over 3 hours and thus choice of an appropriate design storm duration is not particularly important. Flood peak will mainly depend on the high intensity short duration rainfalls and because over 95% of storm rainfall occurs within the central three hour period, any duration of over three hours should be suitable.

A design storm duration of up to twice the unit hydrograph time to peak is commonly used in both the U.K. and U.S.A. It seemed appropriate to use a somewhat shorter duration for Saudi Arabia and a storm duration of 330 minutes (5.5 hours) was adopted, this being the longest duration that could be taken from Figure 4.3 of Report EX 1103.

From Table 4.6 and Figure 4.3 of Report EX1103, rainfalls of 5, 10 and 20 year return period for durations of 30, 90, 150, 210, 270 and 330 minutes were listed. A symetrical rainfall profile was derived from these by nesting the 30 minute fall within the 90 minute fall within the 150 minute fall and so on. Because a synthetic rainfall profile was used in the absence of adequate data from Saudi Arabia, such a synthetic profile may as well be symetrical.

The generalised relationships of Bell were used in a similar way to derive the 50 and 100 year rainfalls of up to 120 minutes duration. For longer durations, a logarithmic plot of rainfall depth against duration was plotted for return periods of 5, 10 and 20 years for durations up to 330 minutes and for 50 and 100 year return periods, for durations of up to 120 minutes. The depth-duration curves for 50 and 100 year rainfalls were extrapolated by eye, from 120 to 330 minutes using the lower return period curves as a guide. Rainfall depths of the required return periods were read from this diagram which is shown as Figure 1.

The rainfall profile for return periods of 50 and 100 years were derived in exactly the same way as those of lower return periods. For all return periods, the derived rainfall depths for any duration are those that might be expected at any point in the catchment. The catchment areal rainfalls would be lower and some appropriate areal reduction factor (ARF) must be applied to the point rainfalls derived from Report EX 1103 and Bell's work. No work on ARF has been carried out in Saudi Arabia to our knowledge and consequently results from the U.K. Flood Studies Report were used. The ARF varied from 0.83 for a duration of 30 minutes to 0.95 for 330 minutes. Whilst use of an ARF derived for the U.K. may not be entirely appropriate for Saudi Arabia, the values utilised are likely to be conservatively high and thus any error will err on the side of safety.

The adopted design storm rainfalls are listed in Table 1.

4. RAINFALL LOSSES

Broadly speaking, there are two distinct approaches to estimation of losses from the design storm rainfalls computed above. The first method relies on estimating the infiltration capacity of the soil, either as a fixed rate throughout the storm or more commonly as a reducing rate with high initial losses reducing to a lower value later in the storm as soil moisture approaches saturation. The disadvantage of this method is that for a strongly peaked rainfall profile, such as that for Jeddah, the central 30 minute rainfall peak will tend to dominate the flood computation.

An alternative approach to estimation of rainfall losses is to use a constant runoff coefficient or to allow either a constant or variable percentage of each time units' rainfall to become storm runoff. This method does not give undue weight to the central peak rainfall and accords better with modern concepts of the contributing area of a catchment being the significant factor in runoff generation. Thus given a runoff of x per cent for a catchment, the implication is not that all of the catchment is generating x per cent runoff. Rather the suggestion is that in very simple terms, x per cent of the catchment close to the water courses will generate something like 100 per cent runoff and other areas of the catchment permit infiltration of rainfall and this produces either a slower baseflow component to runoff or is subsequently lost as evaporation.

The concept of a fixed percentage runoff has been adopted in this present report and the percentages chosen subjectively from knowledge of the climate and soils of the Jeddah area and based on our experience from other arid regions of the world.

For the present undeveloped catchment, a percentage runoff of 10 per cent has been adopted. Some justification for this apparently low figure is presented in section 6 of this report where we discuss the January 1979 storm. The runoff percentage that might be expected from the catchment when developed to the 40 per cent areal coverage assumed has been taken as 20 per cent.

To determine the net or effective rainfall in each 30 minute time interval, the intervals rainfall for the appropriate return period given in Table 1 is multiplied by either 10 or 20 per cent as appropriate.

5. RESULTS

The net rainfall for each return period was combined with successively the unit hydrographs for the present undeveloped catchment and then for the developed urban case. Because the Mushwab dam catchment does not sustain a permanent baseflow, no baseflow element has been added to the design floods computed. Only direct flood runoff is assumed to contribute to the reservoir.

The flood peak and total volume of runoff for each return period for both the undeveloped and urbanised case are given in Table 2. The resulting design flood hydrograph ordinates for the 5 year return period flood for both the undeveloped and urbanised catchment are given in Table 3. Because the unit hydrograph model used is linear, the design flood ordinates for any other return period may be determined by multiplying the ordinates of Table 3 by the ratio of the total storm rainfall of the required return period given in Table 1 to the 5 year rainfall total. Thus to estimate the 100 year flood from the urbanised catchment, multiply the appropriate urban flood ordinates from Table 3 by 133.1/63.7.

There is some uncertainty as to the position of the topographic watershed to the south of the catchment. The catchment area adopted, 37.9 km^2, is that of the natural catchment as shown on the 1:25,000 scale maps provided. However, extensive gravel extraction and building work has possibly modified the watershed divide and an additional area of at least 3.5 km^2 may now drain into the Mushwab dam catchment. If this area does drain into the Mushwab dam catchment, all flows and volumes computed above would have to be multiplied by a factor of 1.09 (which is (37.9 + 3.5)/37.9).

A further major wadi system drains a valley immediately to the south of the Mushwab dam catchment and has a very sizeable catchment area. It has been assumed here that runoff from this wadi system will not drain into the Mushwab dam catchment. However, if because of interference with the natural drainage channels, floodwater were to be allowed to flow into the Mushwab dam catchment, the consequences on stormwater inflows to Mushwab dam might be very serious indeed.

6. COMMENTS ON THE JANUARY 1979 STORM

Jeddah experienced five days of rainfall in January 1979 between the 15 and 19 January. The most significant rainfall was one of 80 mm on the 16 January. The duration of this 80 mm rainfall seems somewhat uncertain since Watson Saudi Arabia suggest a figure of 2 hours whilst the observer's notes give a duration of 300 minutes or 5 hours. A report on the rainfall characteristics of rainfall in Jeddah by Hassan El-Sayed and Kamal Enani of the General directorate of Meteorology in 1979 comments on the January 1979 rainfall event but does not give a duration.

The highest daily rainfall of 80 mm on 16 January 1979 was not in itself unusual. On 17 April 1968, 88 mm of rainfall was recorded in 10 hours, on 3 November 1972, 83 mm fell in only 2 hours 50 minutes, and on 22 January

1969, 79 mm fell in 3 hours 22 minutes. In terms of total rainfall therefore January 1979 was not unusual and Figure 4.3 of our previous report No. EX 1103 suggests that the return period of such a fall of 80 mm in 5 hours would be about 8 years or if the duration were 2 hours, the return period would be about 15 years.

The rainfall intensity does not look to have been exceptional either, even if the shorter duration of 2 hours is accepted. On 17 February 1978, 67 mm fell in 1 hour 15 minutes of which 57 mm fell in 45 minutes at a rate of 76 mm/hour, yet no serious flooding followed.

Hassan El-Sayed and Kamal Enani suggest that January 1979 was unusual in that it had never before in the 21 years for which records were available rained for five consecutive davs. However, since the first daily fall on 15 January 1979 was only 4 mm and the rainfalls on the 17, 18 and 19 January were only 7.0, 1.0 and 2.2 mm respectively, this does not seem to be particularly significant. Neither does their suggestion that because the sky was almost totally cloud-covered throughout the five day period, evaporation would be suppressed and the normal quick drying of the ground after heavy rainfall prevented. With a previous day's rainfall of only 4 mm this hardly seems important and the suggestion by Watson Saudi Arabia that "since the earlier storms had saturated the unpaved surfaces, a high proportion of the precipitation appeared as run-off" also seems to be giving too much weight to the 4 mm rainfall on 15 January.

One factor that Hassan and Karmal mention is the significant growth of urbanisation during recent years. In our report No. EX 1103, we pointed out that Jeddah is one of the fastest growing cities in the world. There were only some 300,000 people six years ago, but Jeddah now has a population of over 1.2 million and the land area of the old city of 20 km^2 has now expanded to cover approximately 140 km^2 today. This could explain why previous storms of over 80 mm in 1968 and 1972 failed to produce such dramatic flooding as that of January 1979. The effects of this recent storm have been felt more dramatically than those of earlier storms because of the significant growth in urban area.

It is significant that the January 1979 storm did not produce dramatic flooding of the Mushwab dam catchment. Watson Saudi Arabia point out that the main storm affected wadi Ghalil to the north and that wadi Mushwab received less intense rainfall. They observed a detained volume of water in Mushwab lake with a maximum depth of 2 metres and a surface area of about 5 hectares. This gives a volume of about 33,000 m^3 and because Watson's suggest that the culvert was blocked by windblown sand which was dislodged by the water on the day following the storm, we may assume that the total flood runoff was only 33,000 m^3. From Table 2 of this report, the 5 year return period flood has a computed volume of 296,000 m^3, very much greater than that observed in January 1979. This implies either that the percentage runoff used in this report is considerably overestimated, perhaps by a factor of as much as ten, which seems unlikely, or that the storm of January 1979 may not have been as intense over the Mushwab catchment as it was over much of central Jeddah.

It is suggested that the January 1979 storm in fact adds confidence to the percentage runoff estimates presented in this report.

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FIGURE 1 RAINFALL DEPTH - DURATION CURVE FOR JEDDAH

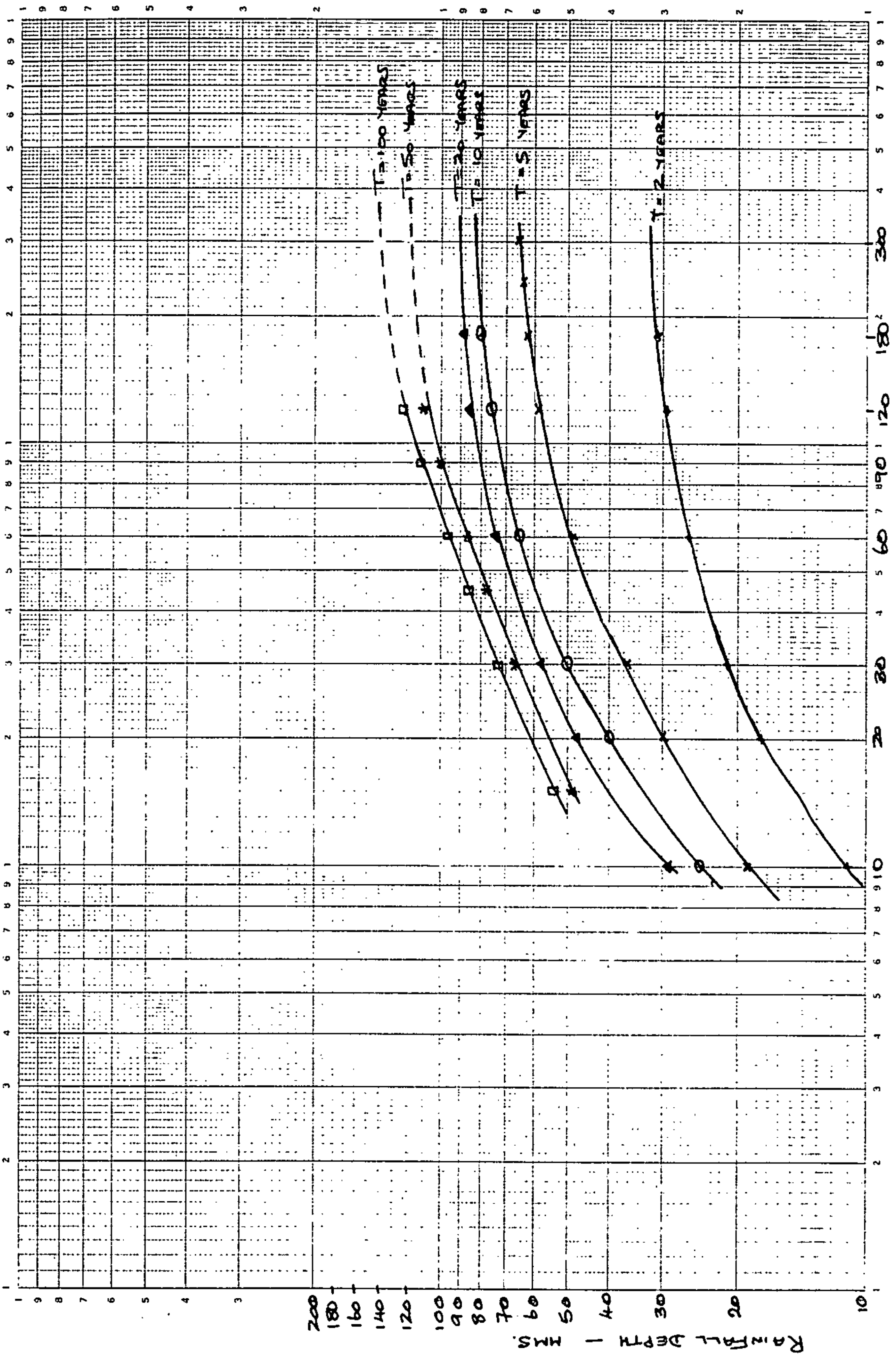

TABLE 1 DESIGN STORM RAINFALLS

	Design rainfalls in mms for 30 minute time periods				
Return Period, T = (Years)	5	10	20	50	100
Time Interval (minutes)					
0-30	0.8	0.7	0.95	1.1	1.7
30-60	1.25	0.85	0.95	1.5	2.0
60-90	1.25	1.25	1.1	2.0	2.9
90-120	3.5	3.35	3.5	7.4	8.5
120-150	9.4	12.25	13.15	18.3	20.3
150-180	30.3	41.5	48.1	55.3	61.3
180-210	9.4	12.25	13.15	18.3	20.3
210-240	3.5	3.35	3.5	7.4	8.5
240-270	1.25	1.25	1.1	2.0	2.9
270-300	1.25	0.85	0.-5	1.5	2.0
300-330	0.8	0.7	0.95	1.1	1.7
Total	63.7	78.3	87.4	115.9	133.1

TABLE 2 DESIGN FLOOD PEAKS AND VOLUMES

Results of convoluting the design storms of Table 1 with the appropriate unit hydrograph for either the present, undeveloped case with a percentage runoff of 10 or the future urbanised case with a percentage runoff of 20. Catchment area = 37.9 km^2.

(a) Present Undeveloped Case

Return Period (Years)	Design Flood Peak ($m^3\ sec^{-1}$)	Total Volume of Flood Runoff (m^3)
5	10.50	237300
10	13.32	296440
20	14.83	329000
50	19.34	433359
100	22.09	500408

(b) Future urbanised case

Return Period (Years)	Design Flood Peak ($m^3\ sec^{-1}$)	Total Volume of Flood Runoff (m^3)
5	35.98	495907
10	44.44	609570
20	49.45	680410
50	67.43	919389
100	75.79	1047900

TABLE 3 5 YEAR RETURN PERIOD DESIGN FLOOD ORDINATES

(All flows in $m^3\ sec^{-1}$)

30 Minute Time Interval	Present Undeveloped Case	Future Urbanised Case
1	0.02	0.11
2	0.06	0.39
3	0.12	0.83
4	0.26	1.75
5	0.59	3.94
6	1.53	10.05
7	2.67	17.16
8	3.88	24.47
9	5.11	31.17
10	6.34	35.98
11	7.55	34.24
12	8.71	30.44
13	9.76	25.86
14	10.49	21.08
15	10.22	16.14
16	9.62	11.12
17	8.92	6.39
18	8.17	2.46
19	7.37	1.09
20	6.55	0.52
21	5.74	0.24
22	4.92	0.07
23	4.10	0
24	3.29	
25	2.50	
26	1.72	
27	0.99	
28	0.38	
29	0.17	
30	0.08	
31	0.04	
32	0.01	
33	0.0	

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