Gode Irrigation Scheme
Nedeco, May, 1986

2. PHYSICAL RESOURCES

2.1 Hydrology of the Wabi Shebelle River

2.1.1. Introduction

The hydrology of the Wabi Shebelle river basin is determined by two rainy seasons. The first rainy season occurs in March, April and May and the second rainy season in July, August and September.

The runoff pattern in the Wabi Shebelle river at Gode, as a consequence, shows two flood seasons. The characteristics of these two flood seasons are different, which can be explained by the occurence of two types of runoff.

The first type of runoff is very gradual and is generated in the mountains which form the North-Western boundary of the river basin (Arussi and Chercher). The rainfall pattern in these mountains is regular with low coefficients of variation. The two rainy seasons generate flood-waves with modest peaks. The second flood peak is generally the highest. When the flood peaks reach Gode they have been very much attenuated.

The second type of runoff is of a more vehement character and has the form of flash floods, which are generated by heavy rainfall in the North-Eastern and central part of the catchment. These flash floods generally occur in the first rainy season during April or May. The many rivers which carry these flash floods, like the Daketa and the Fafen, are intermittent and are dry during five months of the year.

The result of this runoff pattern is that the second flood season in Gode has the largest flood volume, whereas the highest flood peaks generally occur in the first flood season.

The Wabi Shebelle river basin was extensively studied by ORSTOM from 1967 to 1972 (Wabi Shebelle Survey, Volume III, Hydrological survey of the Wabi Shebelle basin, ORSTOM, 1973). Although the survey was based on a very short period of observation (1967 to 1972), the hydrological study is still considered very valuable and of a good standard. Now that observations exist until 1985 (although in some cases with considerable gaps) it is necessary to reassess the water availability and the occurence and impacts of floods.

In the following sections this reassessment is made and the water availabilities and floods are studied in order to reach design criteria for the irrigation system, the diversion weir, the flood protection works and outfall structures.

2.1.2. Analysis of existing information

The WRDA supplied summaries of hydrometric discharge data of the stations Dodola bridge, Melka Wakena, Lega Hida, Hamaro, Imi, Gode and Kelafo. These discharge data have been calculated manually, using a rating table, recorder charts, and two daily gauge readings in case the recorder failed to register or was destroyed. The following barchart shows the data availability.

Table 2.1. Data availability

	drainage area (km2)	Year 67	68	69	70	71	72	73	74	75	76	77	78	79	80	81	82	83	84	85
Dodola bridge	1035	xx	xx	xx	xx	xx	xx	xx	xx	xx	xx	xx	xx	xx	xx	xx	xx	xx	xx	
Melka Wakana	4388	x	xx	xx	xx	xx	xx	xx	xx	xx	xx	xx	xx	xx	xx	xx	xx	xx	xx	x
Lega Hida	20473				x	xx	xx	xx	xx	x										
Wamaro	63644		xx	xx	xx	xx	xx	x	x											
Imi	91600			xx	xx	xx	xx													
Gode	127300	x	xx	xx	xx	xx	xx	xx	xx	xx	xx			x	xx	x	xx	xx		
Kelafo	139100			xx	xx	xx														

Although the data of 1967 to 1972, which were analysed in the ORSTOM report, proved to be of good quality, inconsistencies started to arise when analysing the runoff data of 1976 onwards. Comparing the annual runoffs of Dodola and Melka Wakana, one finds a good correlation from 1968 to 1975 (see Table 2.2.) but a poor correlation for the following years; the correlation coefficient for the early and the later years are 0.894 and 0.608 respectively. Furthermore it can be seen in the table that the ratios between the relative runoffs, which should be more or less unity, show considerable deviation in later years (with extremes of 0.69 and 1.30).

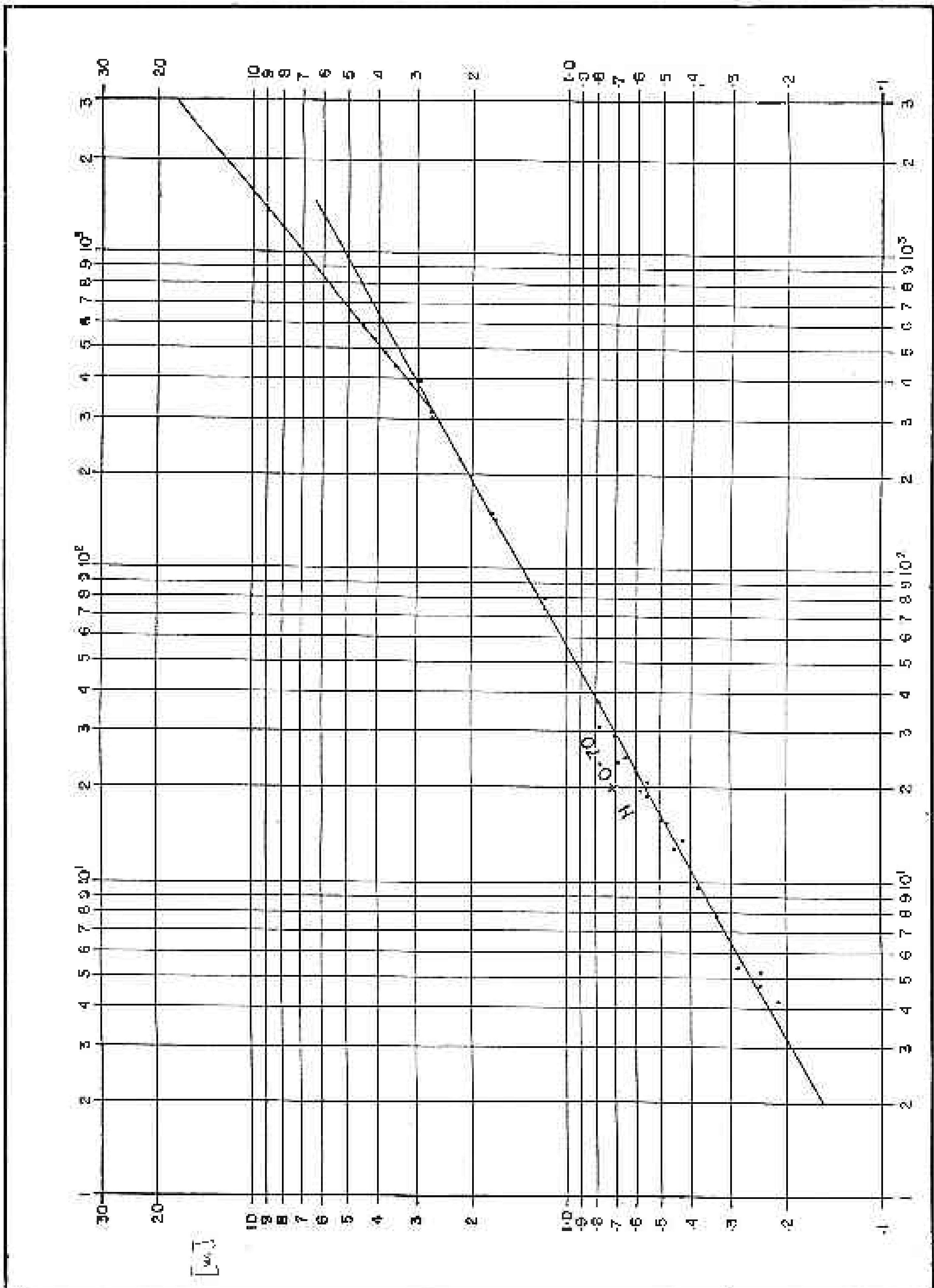
While examining the runoffs of Gode, even larger inconsistencies were found in the years from 1975 onwards. As the Gode runoff data are of crucial importance for the design of the Gode irrigation scheme, the reason of these inconsistencies had to be thoroughly analysed.

The main inconsistencies appear to be related to the rating curves which have been used. Figure 2.1. shows the discharge measurements which were used for the ORSTOM report and the rating curve which corresponds with these measurements. An anomaly which occurs in the rating curve, and which was also discussed in the ORSTOM report, is the upward inflection in the curve for the high discharges. This phenomenon can be explained by the back-water which is introduced by the narrowing of the river at Gode bridge and by the possible accumulation of debris around the pillars. Until 1975 this set of curves was used for the runoff calculations but in 1976, the runoffs were calculated with one single curve without considering the upward section. This yielded a maximum discharge for 1976 which was far too high. Because the recorder also faltered during the 1976 flood and the observer did not manage to read the maximum water level, the maximum water level had to be reassessed. The maximum gauge reading has been estimated by NEDECO to be 7.2 m corresponding with a discharge of 1,190 m3/s (see Figure 2.10).

After the upstream recorder had ceased to function in 1977, the gauging station was moved downstream of the bridge. The rating of the gauge only started in 1982, resulting in the rating curve presented in Figure 2.2. This rating curve also shows an upward inflection. This may be explained by decrease of the bottom slope downstream of the gauging site. At the gauging site the bottom slope is relatively large, which results in rapids at low flows. At high flows, however, the rapids are submerged by a back-water from the downstream reach which has a moderate slope and consequently higher water depth.

Table 2.2 Correlation between the runoffs of Dodola and Melka Wakana

	Runoff		Relative Runoff								
	$10^6 m^3$/year		Runoff/mean		Ratio	Runoff/mean		Ratio	Runoff/mean		Ratio
Year	Dodola	Melka	Dodola	Melka		Dodola	Melka		Dodola	Melka	
1967	271	976	1.227	1.212	1.01	1.227	1.212	1.01			
1968	240	966	1.087	1.187	0.92	1.087	1.187	0.92			
1969	249	864	1.127	1.078	1.05	1.127	1.073	1.05			
1970	254	924	1.149	1.148	1.00	1.149	1.148	1.00			
1971	282	905	1.277	1.124	1.14	1.277	1.124	1.14			
1972	232	697	1.052	.866	1.21	1.052	.866	1.21			
1973	160	569	.724	.706	1.03	.724	.706	1.03			
1974	128	462	.579	.573	1.01	.579	.573	1.01			
1975	208	811	.941	1.007	0.93	.941	1.007	0.93			
1976	147	622	.665	.773	0.86				.665	.773	0.86
1977	267	1411	1.209	1.753	0.69				1.209	1.753	0.69
1978	213	815	.967	1.012	0.96				.967	1.012	0.96
1979	148	679	.671	.843	0.80				.671	.843	0.80
1980	167	684	.759	.850	0.89				.758	.850	0.89
1981	187	691	.847	.858	0.99				.847	.858	0.99
1982	261	736	1.185	.914	1.30				1.185	.914	1.30
1983	387	1182	1.754	1.468	1.19				1.754	1.468	1.19
1984	172	509	.780	.632	1.23				.780	.632	1.23
Mean	221	808	1.000	1.000		1.016	.988		.980	1.011	
s	64	236	.29	.29		.23	.23		.36	.36	
	Y=A*X^B		Y=A*X^B			Y=A*EXP(B/X)			Y=A*X^B		
	A=	9.189	A=	.994		A=	2.190		A=	1.015	
	B=	.828	B=	.828		B=	-.780		B=	.771	
	R=	.706	R=	.709		R=	.894		R=	.608	



RATING CURVE OF UPSTREAM GAUGING SECTION GODE BRIDGE

FIGURE 2.1

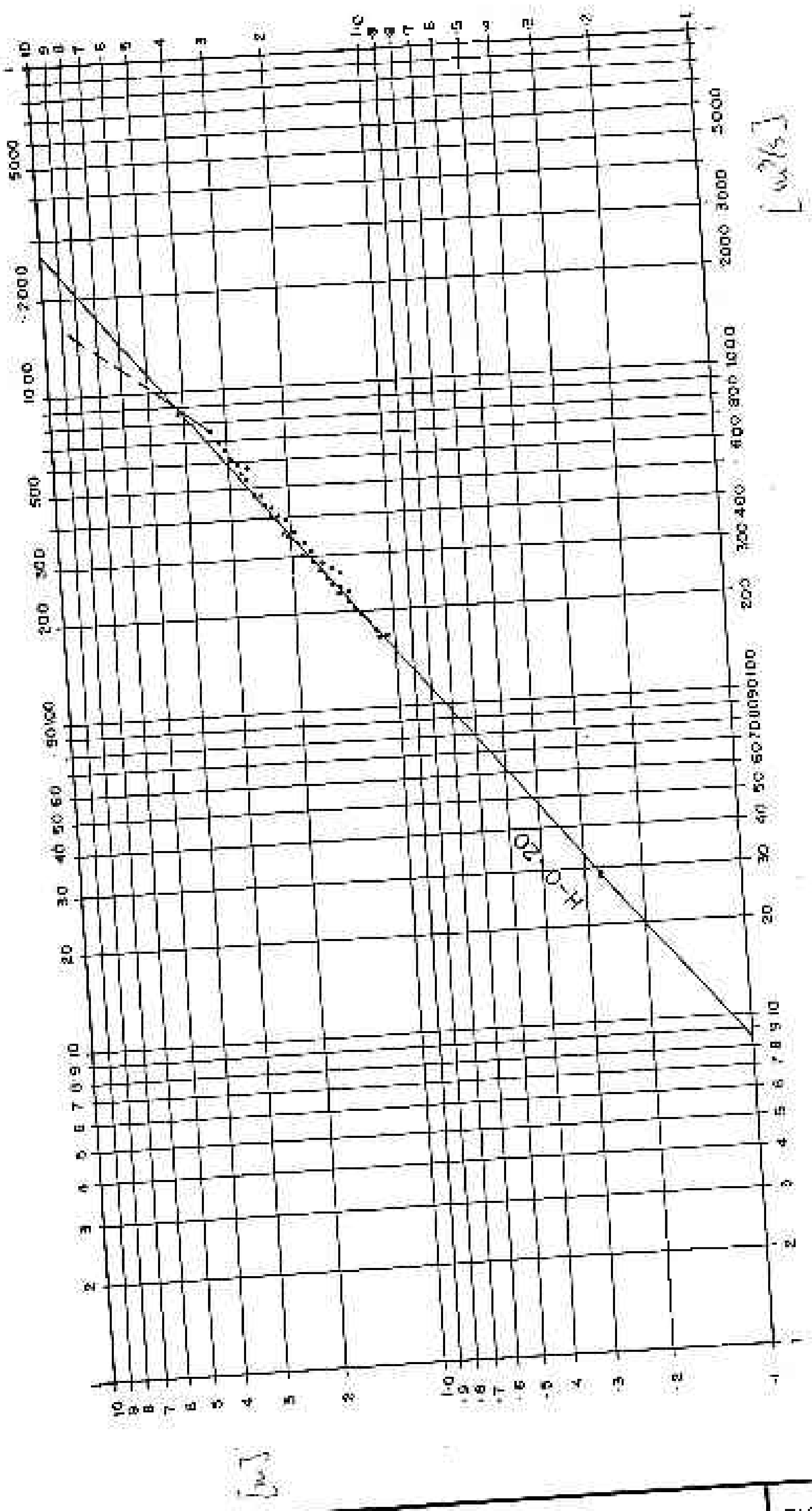


FIGURE 2.2

RATING CURVE OF DOWNSTREAM
GAUGING SECTION GODE BRIDGE

The problem with the runoff calculations from the data that have been gathered since 1979 is that the rating curve of the upstream section was continued to be used until the downstream section was rated in 1982. This yielded completely erratic runoffs for the years 1979, 1980 and 1981. Since 1981 the new rating curve has been used but the upward inflection of the rating curve for high flows has not been observed. Therefore, the high flows have been exaggerated. Furthermore, the minimum flows recorded after 1979 appear to be much higher than the ones recorded before 1977. It can be seen in Figure 2.2 that the rating curve has no measurements for low flows. In sandy river beds, bed levels may fluctuate considerably, which has a large influence on the recorded discharges, especially at low flows. In the area of the gauge, where accelerations due to rapids and decelerations due to back-waters alternately occur, the variation of the bed level may be unpredictible. Therefore it is proposed that flow measurements be carried out, at least twice monthly, throughout the dry season. The runoffs of 1979 to 1983 have to be recalculated on the basis of Figure 2.2, taking into account that the curve is not valid for low flows. Low flow runoff calculations should be based on runoff measurements at low flows if possible in the same year.

For the calculation of the water availability (Chapter 2.1.3) only the runoffs of 1967 to 1977 are used until the remaining years will be recalculated. For the flood analysis, however, the most recent period contains very important floods. Therefore the maximum flood levels have been abstracted from the original observers booklet and used for the frequency analysis of the floods (see Chapter 2.1.5.).

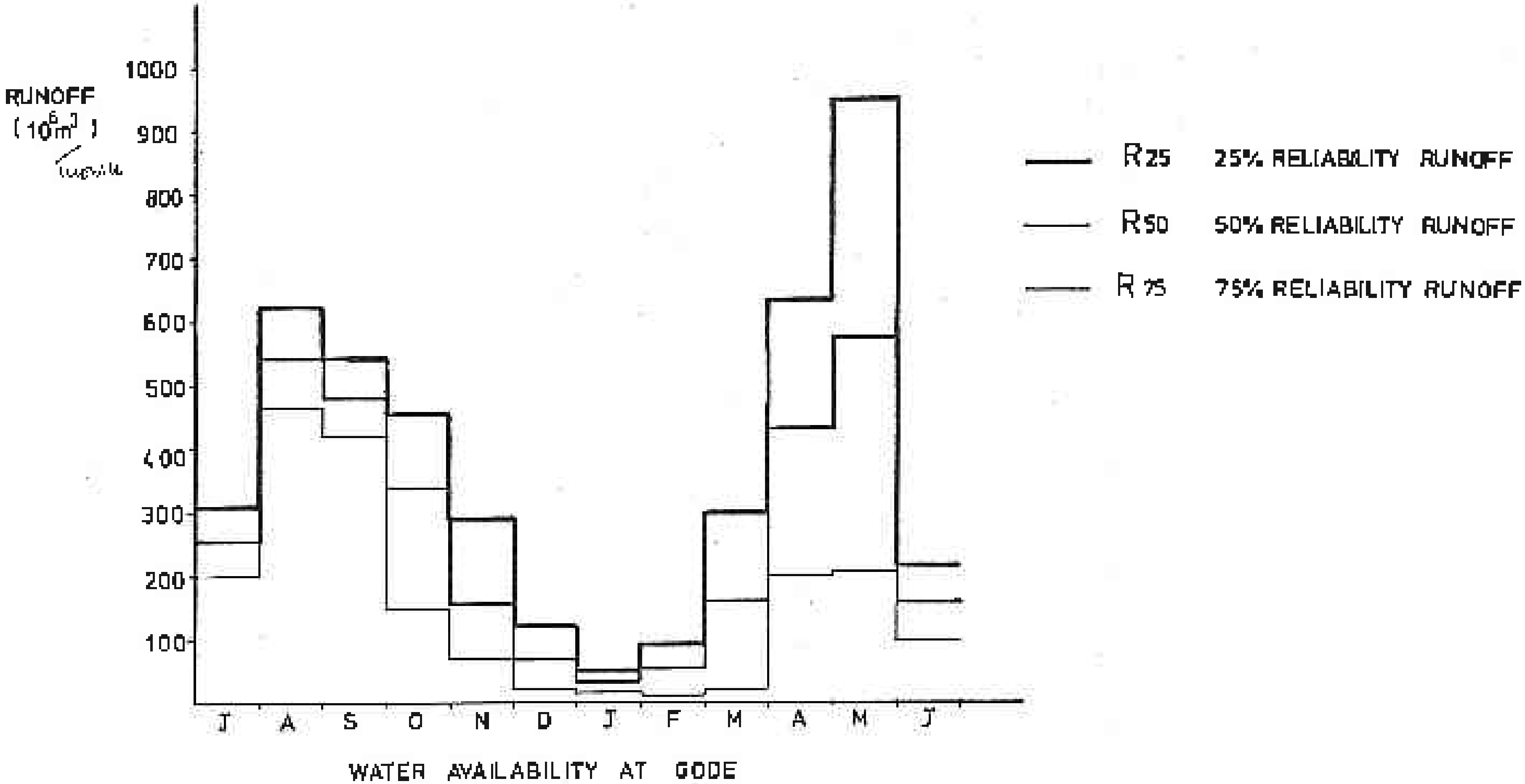
2.1.3. Monthly runoffs and water availability

Because the most recent data have been shown to be unreliable, the water availability has been studied on the basis of the data of 1967 to 1977. For the reliability of the water availability this set of 9 years is acceptable, because it is generally considered that a 70 to 80% reliability is adequate for irrigation purposes. With 9 years of data this estimate can be made with sufficient accuracy. Furthermore it has been shown in Table 2.2 that the second set of data from 1977 to 1985 has almost the same average yearly runoff as the first set at Melka Wakana, and, as will be shown furtheron, the flows at Melka Wakana have very much influence on the low flows at Gode. Therefore the use of the 9 years of records for the determination of the water availability at Gode is permissible.

In Table 2.3 the runoffs have been analysed resulting in water availabilities with probabilities based on the Gaussian distribution (normal distribution). The applicability of the normal distribution has been tested and found justified. The results have been presented in Figure 2.3. It can be seen that water availability constraints will occur in December, January, February and March. Although in most years there is sufficient water in March, in exceptional years, when the second flood season starts late, there may be considerable shortages.

Table 2.3. Water availabilities and reliabilities $10^6\ m^3/month$ $10^6\ m^3/year$

year	July	Aug.	Sep.	Oct.	Nov.	Dec.	Jan.	Feb.	March	Apr.	May	June	Total
1967/68				716	582	243	85	148	334	960	634	295	
1968/69	274	863	419	207	162	113	51	129	409	261	477	93	3308
1969/70	251	872	418	183	118	37	42	52	454	531	334	65	3057
1970/71	144	785	611	474	175	48	33	22	23	230	343	207	3098
1971/72	331	505	440	366	240	80	40	96	72	403	548	140	3256
1972/73	378	613	433	348	178	52	22	13	8	72	220	50	2286
1973-74	133	452	375	290	46	16	12	6	46	237	236	232	2080
1974/75	260	386	477	167	44	20	12	8	6	144	398	114	2033
1975/76	268	836	637	226	67	26	13	10	5	900	2000	229	5003
1976/77	267	504	536										
R 50	255	546	483	336	179	70	34	54	161	415	576	158	3016
σ	78	115	92	173	165	72	24	56	207	321	581	86	982
Cv	.31	.21	.19	.51	.92	1.02	.69	1.04	1.28	.78	.96	.54	.32
R 25	308	624	545	453	290	110	50	92	301	632	948	216	3664
R 75	202	468	421	219	68	21	18	16	21	198	204	100	2360



MONTHLY RUNOFFS WITH PROBABILITIES OF EXCEEDENCE

FIGURE 2.3

Fortunately the picture will improve when taking into account the effect of the Melka Wakana dam which will generate a minimum base flow. The Melka Wakana dam has been studied by Technopromexport and the report was published in Moscow in 1985. They recommended to build the dam with a normal high water level of 2520.7 m, which corresponds with a guaranteed outflow of 19.2 m3/s. Assuming that during the dry months the outflow of the dam will be 19.2 m3/s instead of the natural flow measured at the gauging site, Table 2.4 has been drawn up analysing the water availabilities before and after the construction of the dam. In order to determine the future flow at Gode, the natural flow of Melka Wakana has been substracted from the Gode natural flow and 19.2 m3/s has been added to the result.

It can be seen that due to losses on the way, the discharge may drop under 19.2 m3/s. These losses are the losses which under the present situation occur. With the Melka Wakana dam being operational, the low flows and consequently the losses due to evaporation and infiltration will increase. The infiltration increases due to the increase in depth and the evaporation increases due to the increase in width. The 70% natural flows in the dry period are in the order of 10 m3/s and the regulated flows are about double that value. At those flows, the increase in width and depth are generally small. At the 80% flows, however, the increase of the flow is four to five times the natural flow. Here the increase in width is no longer negligable. Assuming an average increase of 10 m, with a river length through the low lands of 700 km, this implies an increase of area with 7*10^6m2. With a monthly evaporation of 150 mm this implies a monthly increase of evaporation by 1*10^6 m3. This is a very small quantity and no reason for concern.

Still it will be necessary to maintain a base flow along the river to guarantee water uses downstream at Gode, Kelafo and Mustahil. The minimum flow recorded with 80% reliability was 2 m3/s in March. If the diversion dam is built, the same minimum flow should be maintained with 80% reliability during the critical month. It will depend on the water requirements which will be the critical month, but in that month a base flow of 2 m3/s will have to be maintained. In the remaining months, the flow which passes the dam will be more than 2 m3/s. In order to be able to select the critical month, in Table 2.4 a base flow (Qb) of 2 m3/s has been substracted from the 80% and 70% low flows. The 80% reliability minimum flow is then 15.8 m3/s in March and the 70% reliability minimum flow is 20.0 m3/s in January.

2.1.4. Extreme levels and flows

As has been pointed out before, the available data contain several inconsistencies with regard to the water levels recorded at Gode bridge, the major inconsistency is caused by the change of the gauging section from upstream to downstream of the bridge. Also the gaps in observation between 1976 and 1979 and between 1983 and 1985 cause difficulties. And finally it appeared that during the floods of 1976, 1978 and 1983 the highest water levels were not observed.

This set of difficulties does not make it easy to analyse the frequency of flooding at Gode bridge. It has been possible, however, to partly overcome these problems, either by accurate surveys and calculations or by fair estimates. In the following paragraphs the composition of the frequency distribution on the basis of different pieces of information is discussed.

Table 2.4 Water availiability after construction of Melka Wakana Dam

Runoff in 10^6 m3 /year

	December			January			February			March		
	Gode	Melka Wakana	Gode future	Gode	Melka Wakana	Gode future	Gode	Melka Wakana	Gode future	Gode	Melka Wakana	Gode future
1967	243	23	271									
1968	113	16	148	85	11	125	148	39	165	334	57	328
1969	37	13	75	61	26	77	129	43	132	499	108	442
1970	48	12	87	42	23	70	52	12	86	454	78	427
1971	80	19	112	33	12	72	22	98	59	20	12	62
1972	52	13	80	40	15	76	95	38	108	72	40	89
1973	16	12	55	22	11	62	13	87	51	8.4	8.9	50
1974	20	19	58	12	12	61	6.2	9.2	42	46	26	71
1975	25	9.1	67	12	11	52	9	10	45	5.9	9.9	47
1976	74	17	108	13	12	62	10	9.4	47	4.7	12	44
R50	71	15	107	34	16	71	64	19	81	161	39	178
s	68	4.0	64	24	5.5	23	56	35	42	207	35	173
Cv	.96	.28	.60	.69	.37	.32	1.05	.75	.52	1.28	.91	1.00
R70	36		73	22		59	25		59	53		82
R80	14		63	14		52	6.8		46	5		44

Discharge in m3/s

	December			January			February			March		
Q50	26.4		39.9	12.8		26.5	22.2		33.5	60.1		64.6
Q70	13.3		27.4	8.2		22.0	10.1		24.4	19.8		30.8
Q80	5.2		19.8	5.4		19.4	2.8		18.9	1.9		17.5
Q70-Qb			25.4			20.0			22.4			28.8
Q80-Qb			17.8			17.4			16.9			15.5

Table 2.5 shows the peak flows and maximum gauge readings and how the values were arrived at. For the years 1968, 1969, 1970 and 1971 no original gauge readings were present but the peak discharges could be obtained from the GRSTEM report. These peak flows have been converted to gauge readings using the rating curve of the downstream section (Figure 2.2). For the years 1972, 1973, 1974 and 1975 the maximum gauge readings at the upstream section were available at the WRDA. These were converted to flows, using the upstream rating curve (Figure 2.1), and then converted to gauge levels at the downstream section using the rating curve of Figure 2.2. In 1976 no peak flow was recorded because the recorder did not register water levels over 5.0 m. By plotting the hydrograph as far as it had been registered and by extrapolating the hydrograph (see Figure 2.10), an estimate of 7.20 m has been obtained which could be transformed to a downstream gauge reading in the same way as described above.

For the year 1979, 1980, 1981, 1982 and 1983 gauge readings were available at WRDA although the readings in 1983 seem rather unreliable. The 1985 flood level was accurately obtained during the field visit.

The flood peaks of the remaining years: 1977, 1978 and 1984, are mere estimates in the knowledge that no floods occured during these years. The values have been added to fill in the gaps so as to be able to attribute the proper frequency to the highest recorded floods.

Table 2.5 Maximum gauge readings and peak flows at Gode

Year	Maximum reading old gauge (m)	Peak discharge (m3/s)	Maximum reading new gauge (m)	ranking
1968		600	3.10	11
1969		480	2.75	14
1970		590	3.05	12
1971		430	2.50	16
1972	2.92	460	2.70	15
1973	2.17	350	2.20	18
1974	2.48	380	2.30	17
1975	3.78	620	3.20	10
1976	e 7.20	1190	8.20	3
1977		680	* 3.5	9
1978		830	* 4.5	8
1979		970	e 5.0	7
1980		1020	5.90	4
1981		1300	6.41	2
1982		1010	5.25	5
1983		890	e 4.5	6
1984		560	* 3.0	13
1985		1600	s 7.2	1

e : extrapolated from observations
* : estimates
s : survey

The procedure which has been followed is rather risky. Therefore two checks were made one during the field visit and one using rainfall data.

When looking for flood marks of the latest flood several people were interviewed about the occurrence of floods, their remembrance of the highest flood and the relative heights of the floods. All interviewed people agreed that the 1985 flood was the highest of them all and one person at Gode West village claimed to have lived there for 25 years and never have experienced a flood as high as the latest one. There were enough floodmarks around to obtain a clear picture of the extent of the 1985 flood (see Chapter 2.1.5). This person recalled that 8 times during these 25 years he had to evacuate the village and move his cattle to higher grounds. He also stated that the flood plain went under water during these floods. He indicated the 1985 flood as the highest one followed by the 1981 flood the 1976 flood and a flood somewhere near 1966. Although this type of information is only partly reliable, as people tend to exaggerate floods in their remembrance, it presents sufficient correspondence with the available records to merit credibility.

The rainfall of the central catchment of the Wabi Shebelle, which is responsible for the flash floods, was roughly analysed. A detailed analysis is not possible because much information is lacking and the network of raingauges is not dense enough to analyse point rainfall. The two-monthly sums appeared to give the best correlation with the floods, probably because it influences soil moisture and consequently the runoff coefficient is increased. Table 2.6 indeed shows that 1976 and 1981 had the highest rainfall in the flood season and that 1979 and 1983 also ranked high. In general the picture corresponds rather well with Table 2.5.

Table 2.6 Maximum two-monthly rainfall in the central catchment of the Wabi Shebelle (mm)

Year	Gursum	Qubradare	Degahebur	Mean	months
1969		167	88	112	April May
1970		179	147	163	April May
1971	132	178	170	160	April May
1972	217		109	163	April May
1973	116	58	167	114	April May
1974	229	26	107	120	April May
1975	136	137	147	140	April May
1976	274	280	186	247	April May
1977	247				April May
1978	241				April May
1979	352				May June
1980	164		105	135	April May
1981	458	273	302	344	March April
1982	156	131		144	April May
1983	412	139	84	198	April May

With regard to the flows, another complication arises by the fact that the flood plain is inundated during the high floods and that part of the flow passes by the Gode gauging section through the right bank flood plain. This means that at high floods a certain amount of flow should be added to the discharge obtained from the rating curves. This has been done for the 1981 and 1985 floods, the justification of which will be given in Chapter 2.1.5.

The two rankings of Table 2.5 of both the peak-flows and the maximum gauge readings have been subjected to frequency analyses and it appeared that the Gumbel-I extreme values distribution gave the best fit to both peak-flows and gauge readings. The Log-Pearson type III distribution also produced a reasonable fit to the peak-flow data (see Table 2.7, Figures 2.4 and 2.5). These frequency distributions will be used in Chapter 2.1.6 to derive the design discharges for the diversion dam and the protection of the project area.

2.1.5. Flood survey and slope area calculations

The recent occurrence of the highest flood on record, the 1985 flood, offered an excellent opportunity to survey the propagation of a major flood through the project area. Figure 2.6 shows a 1:100,000 map of the project area supplied by the WRDA. It shows contour lines with one metre interval and a set of points where the river bed level has been surveyed.

Using this map and additional surveys in the project area, a longitudinal profile of the river bed and river banks has been obtained (see Figure 2.7). Four clear flood marks have been surveyed and the maximum water level at the gauge downstream of Gode bridge was known to the responsible hydrometrist.

These permitted the drawing of the curve which links the maximum water levels reached during the 1985 flood. The interpolation of the flood marks has been done using a backwater computation model.

Looking at Figure 2.7 several peculiarities can be observed. Two rather sudden drops in the river bed exist downstream of the diversion dam site and at Gode bridge. At both sites rock outcrops can be found in the river bed which cause rapids in the flow. Also it appears that at these sites the banks are high enough to contain the flow. At the intermediate reach the water levels of the 1985 flood appear to have overtopped the banks. Another phenomenon is caused by the Gode bridge. During high flows the discharge capacity of the bridge becomes considerably less than the capacity of the original section and is possibly decreased even more by accumulated debris. This causes a considerable rise in the upstream water levels and introduces a back-water which extends several kilometers upstream. At the bridge itself high velocities must have occured due to a local drop in waterlevel of about three meters and it is probably thanks to its solid foundation in rock that no pillars were washed away during the flood.

A reason for concern is the overtopping of the right bank upstream Gode bridge. This means that the flow which is recorded at the section during floods is not the whole discharge of the Wabi Shebelle. The same happens at the diversion dam site. On the left bank of the most upstream of two very large meanders, about 12 km upstream of the diversion dam site, proof was found, during the survey, of flood water entering the flood plain through a gully. Furthermore it can be observed from the aerial photographs that the flood plain at that point probably already conveys flood waters proceeding from even further upstream. It can be clearly seen that an old branch of the river follows the left bank flood plain at Gode West, crosses the river at the main drain site and then follows the right bank flood plain of Gode South, to join the river again several kilometers downstream of Gode bridge. This branch is very old and in the terrain the relief of this river bed and its levees has been eroded to the extent that it is hardly visible.

Table 2.7. Frequency analyses of Wabi Shebelle at Gode Bridge

Gumbel-type I — Annual maximum levels

N	Yn	Sy
18	.5182	1.0396

MEAN	=	4.003
LOGMEAN	=	1.321
ST.DEV.	=	1.546
LOGST.DEV	=	.371

Gumbel-type I — Annual maximum flows

N	Yn	Sy
18	.5182	1.0396

MEAN	=	775
LOGMEAN	=	6.560
ST.DEV.	=	350
LOGST.DEV	=	.444

Log-Pearson type III — Annual maximum flows

N	=	18
MEAN	=	6.560
ST_DE	=	.444
SKEW	=	.142
ALPHA	=	108
BETA	=	.032
CUTOF	=	.334

Sorted distribution

	Gumbel-type I (levels)			Gumbel-type I (flows)			Log-Pearson type III		
Year	x	y	p	x	y	p	i	x	p
1973	2.20	-1.080	.053	350	-1.080	.053	1	350	.053
1974	2.30	-0.812	.105	380	-0.812	.105	2	380	.105
1971	2.60	-0.613	.158	430	-0.613	.158	3	430	.158
1972	2.70	-0.443	.211	460	-0.443	.211	4	460	.211
1969	2.75	-0.289	.263	480	-0.289	.263	5	480	.263
1984	3.00	-0.142	.316	560	-0.142	.316	6	560	.316
1970	3.05	.001	.368	590	.001	.368	7	590	.368
1967	3.10	.145	.421	600	.145	.421	8	600	.421
1975	3.20	.291	.474	620	.291	.474	9	620	.474
1977	3.50	.443	.526	680	.443	.526	10	680	.526
1978	4.00	.604	.579	830	.604	.579	11	830	.579
1983	4.50	.778	.632	890	.778	.632	12	890	.632
1979	5.00	.969	.684	970	.969	.684	13	970	.684
1982	5.25	1.186	.737	1010	1.186	.737	14	1010	.737
1980	5.30	1.442	.789	1020	1.442	.789	15	1020	.790
1976	6.20	1.761	.842	1190	1.761	.842	16	1190	.742
1981	6.41	2.196	.895	1300	2.196	.895	17	1300	.895
1985	7.20	2.918	.947	1600	2.918	.947	18	1600	.947

Gumbel (levels): y=a*(x-b)
a=.673
b=MODE= 3.24

Gumbel (flows): y=a*(x-b)
a= .00297
b=MODE= 601

Log-Pearson curve

x	p
250	.007
500	.220
700	.563
1000	.786
1250	.898
1500	.951
1750	.976
2000	.988
2250	.994
2500	.996

Gumbel-type I (flows):

Probability of non exceedance .990
Quantile= 2150

Probability of non exceedance .995
Quantile= 2385

Log-Pearson type III:

Probability of non exceedance .990
Quantile= 2077

Probability of non exceedance .995
Quantile= 2380

WABI SHEBELLE AT GODE
ANNUAL MAXIMUM FLOWS

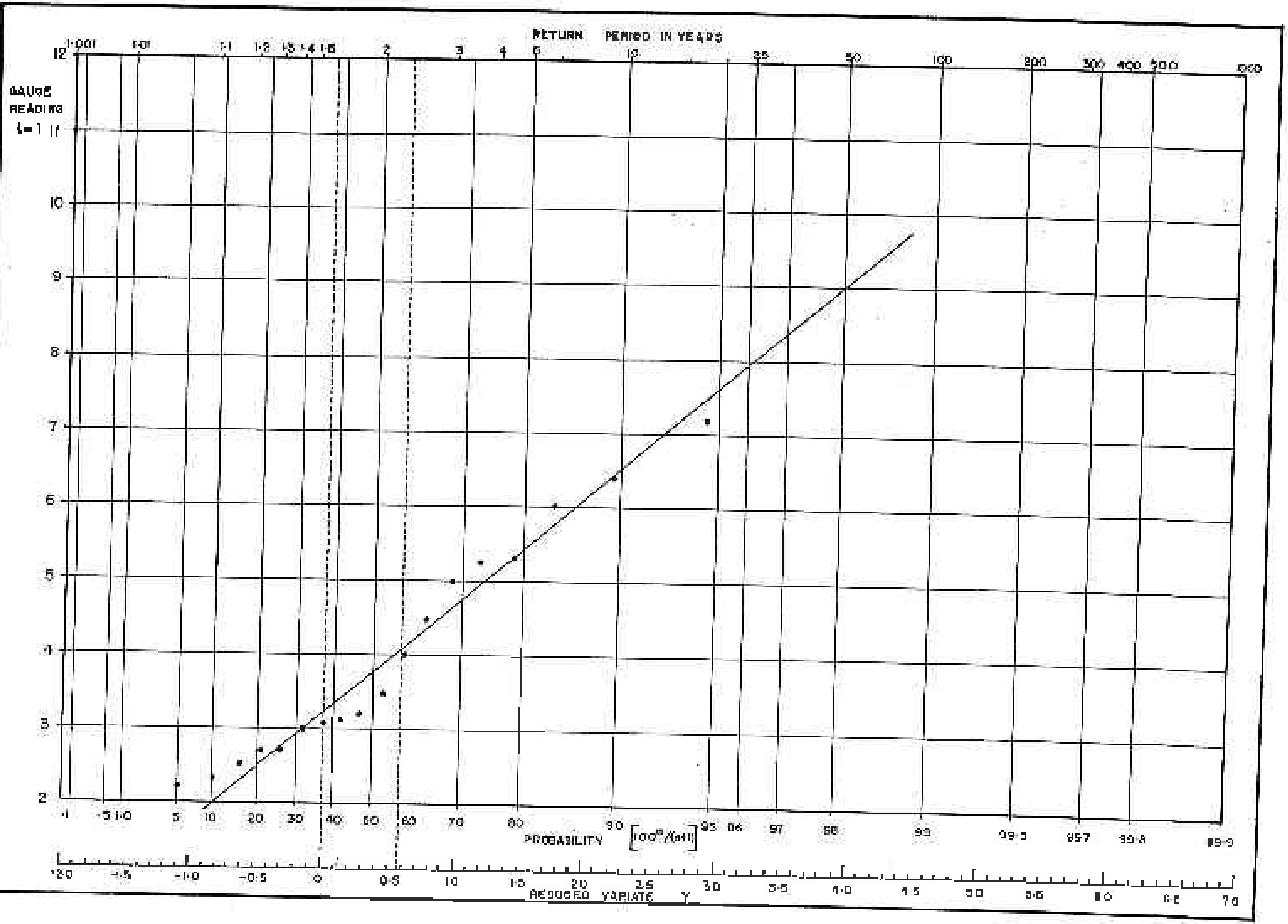
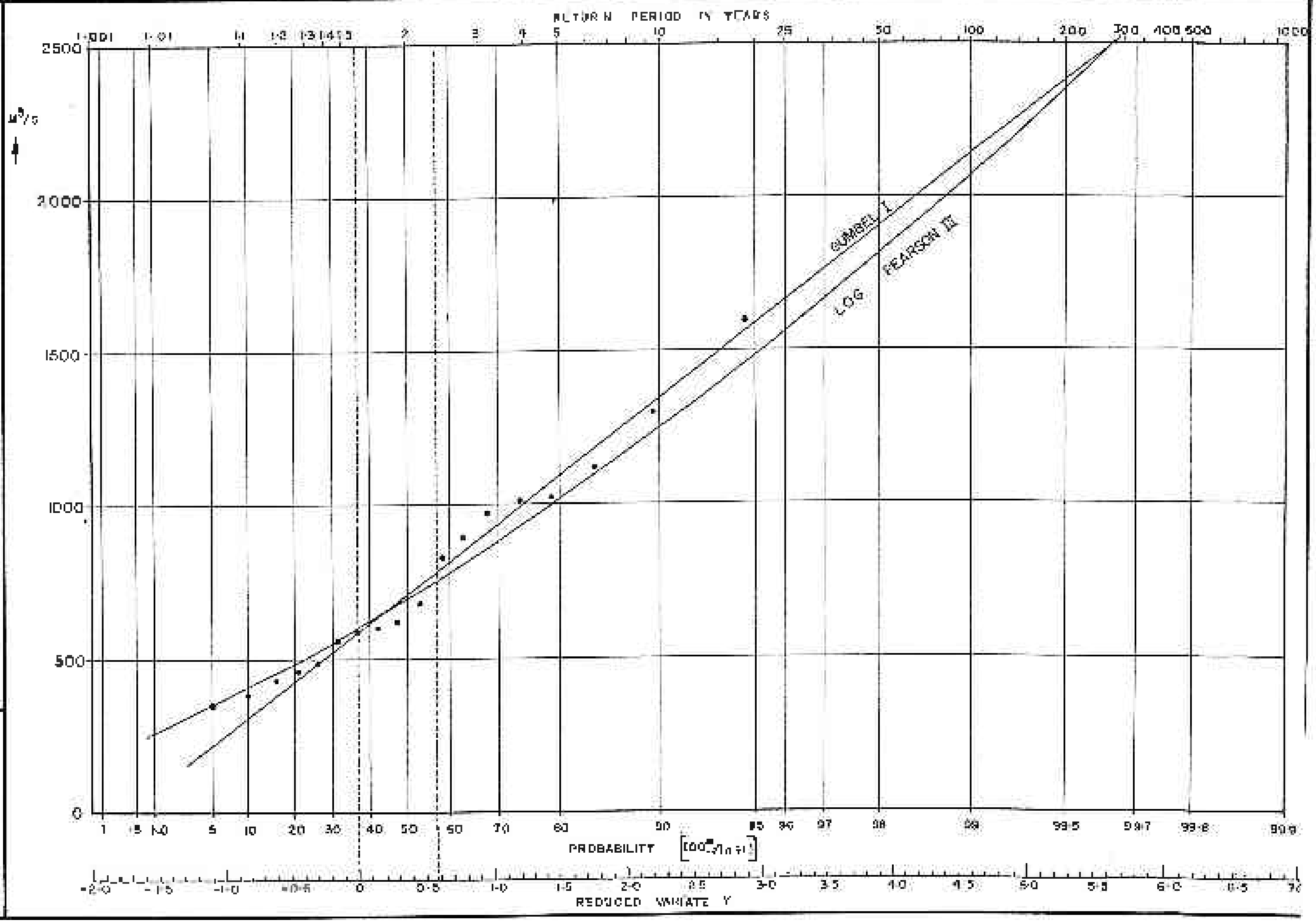
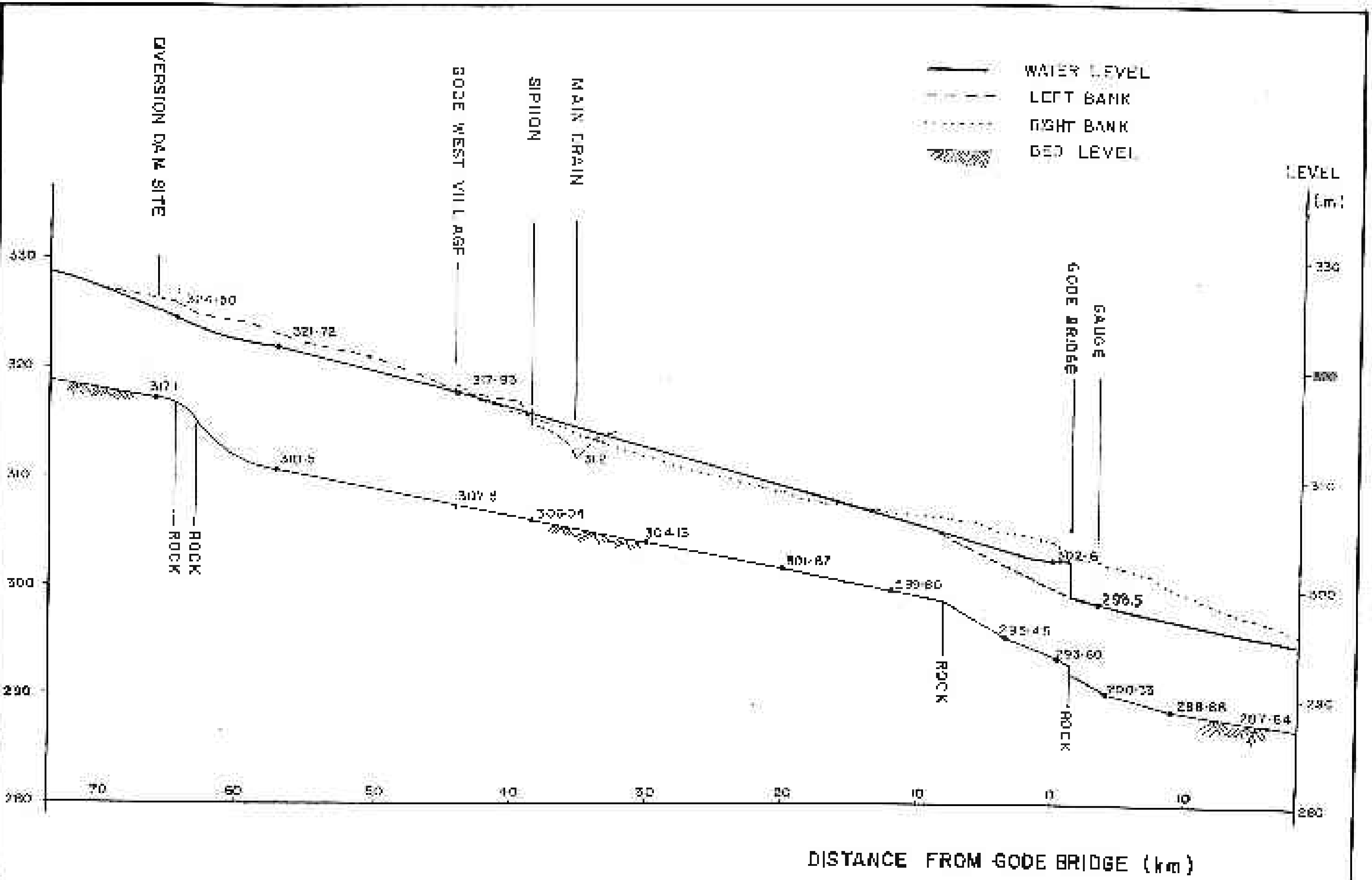


FIGURE 2.5

WABI SHEBELLE AT GODE
ANNUAL MAXIMUM FLOWS

FIGURE 2.4





1985 FLOOD IN THE PROJECT AREA

FIGURE 2.7

In purely alluvial rivers the overtopping of banks is a regularly occuring phenomenon. If a river is in a dynamic equilibrium (or regime) it has been shown by Leopold et al. (1964) that bankfull stage has a probability of once in 1.5 years. The Wabi Shebelle in the project area may not be considered purely alluvial. Its flow is dominated by the two rock outcrops and its geomorphology appears to be restricted by steep cohesive bank of an old alluvial terrace which has been deposited ages ago. Within the steep banks, especially at the inner bends, levees can be found of sandy material covered by trees which indeed are flooded almost yearly. These are the alluvial levees within the constraining banks.

The steep cohesive banks overtop with a lesser frequency and , based on local information, this frequency is estimated as once in seven years. This means that the 1985, 1981 and possibly the 1978 flood must have had higher total discharges than what would follow from the rating curve.

The quantity of overland flow can be estimated, although only through rough calculation. In 1985 it is estimated that a sheet-flow with a maximum depth of one meter by-passed the weir site. According to the contour map (Figure 2.6) this results in a width of flow of 1,500 m, resulting in a cross-sectional area of 1,500 x 1/2 = 750 m2. With a bed slope of 6 * 10^-4, a hydraulic radius of 0.7 m and a Chezy roughness of 30 $\sqrt{m}$/s this yields a discharge of 460 m3/s.

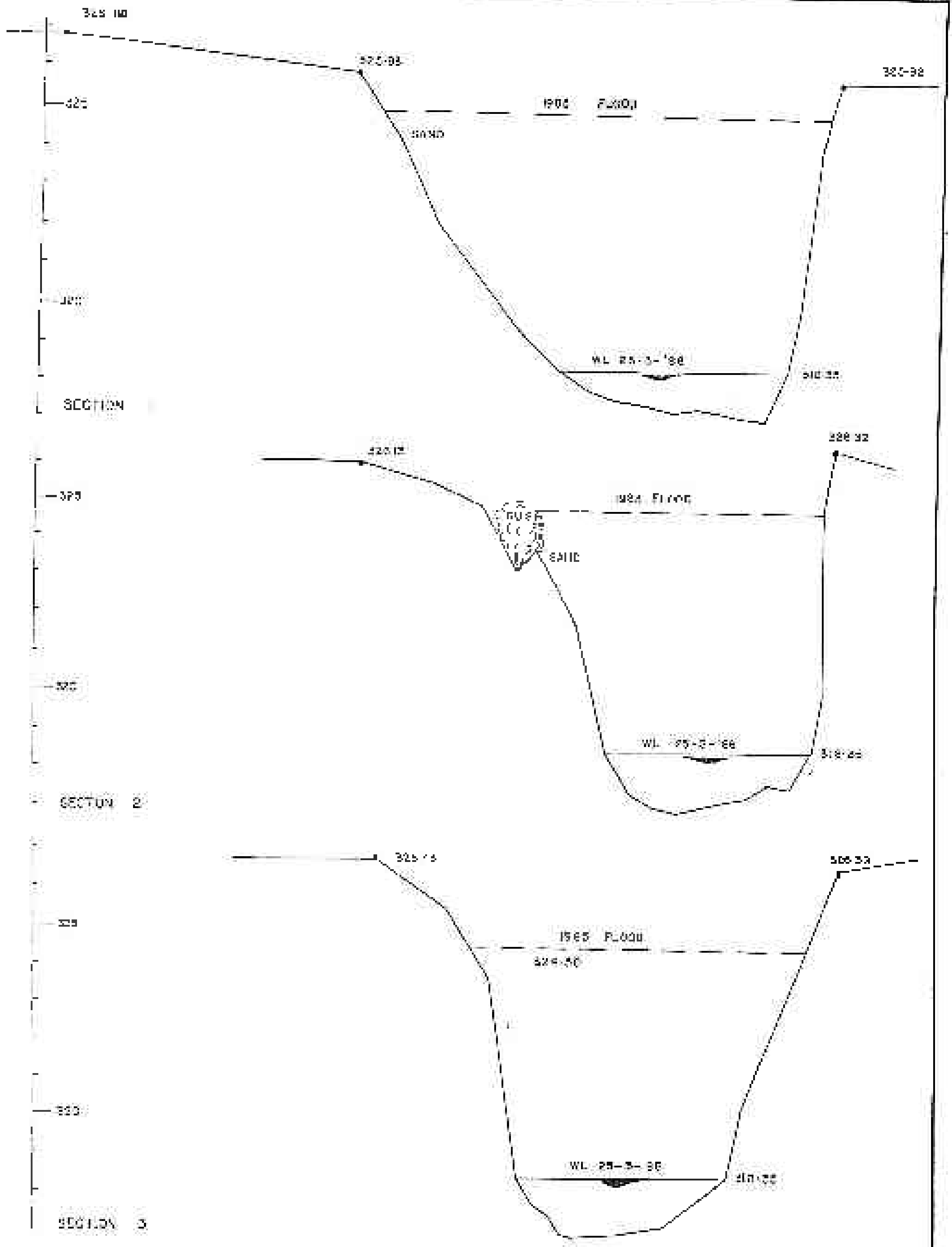
At the river section itself a slope-area calculation has been made. For that purpose three cross-sections have been surveyed downstream of the diversion dam site. These cross-sections are presented in Figure 2.8. The relation between cross-sectional area and water level is presented in Figure 2.9 At section 3, the cross-sectional area A was 480 m2 at the peak of 1985's flood. The wetted perimeter P amounts to 95 m and the Chezy roughness C has been assumed as 45$\sqrt{m}$/s. From Figure 2.7 the slope can be measured, producing I = 5.7 * 10^-4. Using Chezy's formula.

$$Q = A*C*\sqrt{A/P*I}$$

the discharge Q becomes 1160 m3/s. It can be seen in Figure 2.9 that the cross-sectional area increases in upstream direction and that the velocity decreases proportionally. Such decrease in velocity corresponds with the upstream decrease in water surface slope which is caused by the rapid downstream of cross section 3. Adding the flow through the flood plain to the above calculated flow, a total flow of 1,620 m3/s is obtained.

The rating curve of Figure 2.2 produces a discharge at Gode bridge of 1,300 m3/s. With 1,620 m3/s at the weir site, this means that a flow through the right bank flood-plain of 300 m3/s must have by-passed the Gode section. This flow enters the right bank flood-plain downstream of the projected siphon and returns to the river downstream of Gode bridge.

Concluding this section it is worth mentioning that a lot of uncertainties about the peak-flow of the 1985 flood remain. The estimate of the peak-flow, which has a high influence on the frequency analysis presented earlier, is susceptible to discussion. For the final establishment of the flood peak, professional intuition and professional experience had to be leant upon heavily. The result however is considered an acceptable estimate under the given circumstances.



CROSS – SECTION DOWNSTREAM DIVERSION DAM SITE

FIGURE 2.8

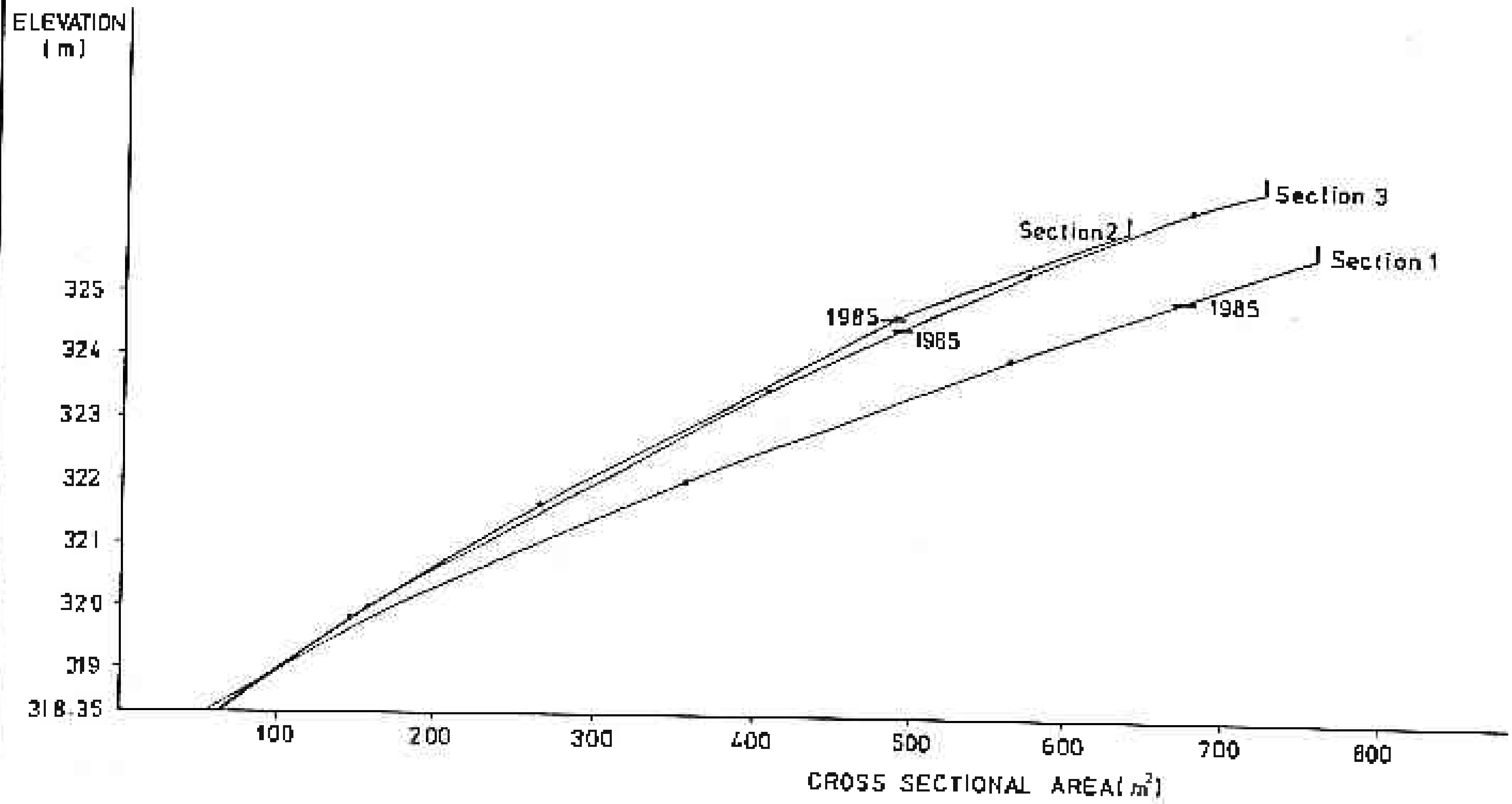


FIGURE 2.9

CROSS SECTIONAL AREA AS A FUNCTION OF WATERLEVEL

2.1.6. Design level and design hydrograph at the diversion dam site

For the design of a major river work sufficient safety has to be provided to guarantee its existence over its projected lifetime. For the diversion dam a lifetime of 50 years is projected. For the diversion dam two alternatives are being considered; a barrage equipped with movable gates and a fixed weir (See Chapter 5).

For the safety of a barrage, a design flood of once in 100 years is used. With 18 years of observations an extrapolation of the Gumbel probability distribution to a probability of 1/100 years is acceptable. Over the lifetime of the barrage this annual probability of occurence results in a guarantee of 60% that such a flood will not be exceeded. For the barrage this is an unacceptable risk. Therefore, it is recommended to construct a spillway section in the dike which links the dam to the escarpment, at a site where minimal damage is expected. The impact of such an overtopping will be discussed in Chapter 2.1.7.

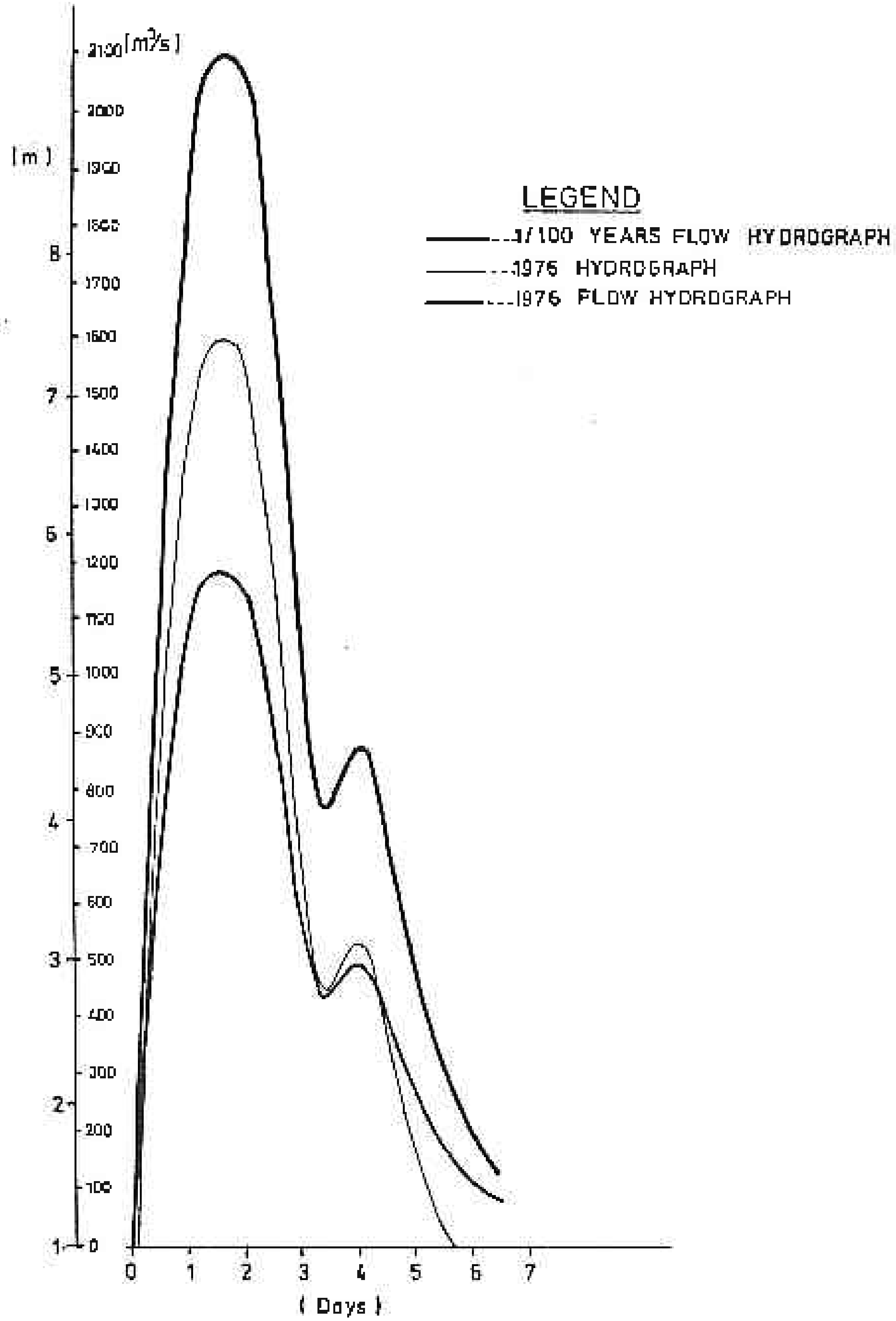
For the fixed weir option a design discharge of once in 50 years will be adequate. For a fixed weir with additional spillway provisions, the hazard for damage, in case the design flood is exceeded, is less than for a barrage.

For the flood protection dikes downstream of the diversion structure a design flood of once in 20 years is considered appropriate.

According to the Gumbel-I frequency distribution, the discharge with a return period of 100 years corresponds with a flow of 2150 m3/s and according to the Log-Pearson type III distribution with a flow of 2077 m3/s (see Table 2.7). For the 1/50 years flood these values are 1,900 ad 1,817 respectively, while for 1/20 years, values of 1,590 and 1,480 have been derived. Both frequency distributions provide a good fit. Therefore the design discharges which are recommended are:

q = 2100 m3/s with P(Q<= q)= 0.99
q = 1900 m3/s with P(Q<= q)= 0.98
q = 1600 m3/s with P(Q<= q)= 0.95

The hydrographs which correspond with these design discharges can be determined by studying the general shape of flash flood hydrographs recorded at Gode. The only hydrograph which has been recorded reasonably accurate is the April flood of 1976. In Figure 2.10 the 1976 hydrograph, of which the highest water levels had to be estimated, is presented. Using the upstream rating curve of Figure 2.1 this hydrograph has been converted to a flow hydrograph.



1976 AND DESIGN FLOOD HYDROGRAPHS

FIGURE 2.10

Table 2.8. 1976 and design flood hydrographs

(hours)	Flood of 1976 (m3/s)	Design flood of 1/100 years (m3/s)	Design flood of 1/50 years (m3/s)	Design flood of 1/20 years (m3/s)
0	0	0	0	0
12	750	1325	1200	1010
24	1150	2030	1835	1545
36	1190	2100	1900	1600
48	1130	1995	1805	1520
60	860	1520	1375	1155
72	550	970	880	740
84	450	795	720	605
96	500	880	800	670
108	390	690	620	525
120	270	475	430	365
132	180	320	290	240
144	110	195	175	150
156	80	140	130	110
Volume (10^6 m3)	164	290	260	220

In order to arrive at the design flood hydrographs with peak-flows of 2,100, 1,900 and 1,600 m3/s, the 1976 flow hydrograph has been multiplied by the ratio of the peak flows (see Table 2.8). Although this may seem a rather bold method, it is the best method that, with the available information, can be applied. There is simply not enough information available to try and develop a unit hydrograph. The method applied here is based on the assumption that the shape of the 1976 hydrograph is that of a unit hydrograph with an adequate rain duration.

To calculate the design levels of both barrage and weir alternatives and the dike which links the headworks to the escarpment, the capacity of the upstream river has to be calculated. Since no cross-sections upstream of the dam site were available at the time of drawing up of this report, use has been made of cross-section 9 (Figure 2.8), situated some 800 m downstream. With a water level of 527 m, the flood plains on both sides convey water.

The discharge capacity of the main channel at 527 m can be calculated as follows:

J = 5.0 x 10^-4
C = 45
A = 770 m2
R = 125 m

Q = 1923 m3/s

For the 1/100 years flood, this leaves 177 m3/s to be conveyed by the flood plains.

At a later stage these design levels can be simulated with a higher degree of accuracy by the Ribasi flood routing model and if necessary, minor adjustments may then be made.

2.1.7. In case the design flood is exceeded

As has been stated in the previous chapter, the probability that a flood with an annual probability of exceedance of 1% does not occur during a 50 years period is 0.99^50 = 61%. This leaves a risk of 39% that the flood is exceeded once or more times during the life-time of the barrage. Because of this risk provisions have to be made to guarantee that the safety of the barrage is not endangered at somewhat higher floods.

For the fixed weir option the annual probability of exceedance has been chosen at 2% (once in 50 years), the risk that this flood is exceeded during the life time of the weir is 64%, with the measures taken (see Chapter 5) this risk is deemed acceptable.

The solution for the problem is to provide spillways in the dikes which link the diversion structure to the high areas on both sides of the river.

For the determination of the width of the spillways, the 1/1000 years flood will be used. Although the Gumbel graph may not be extrapolated to such a high value for the design of a major work like the dam, it may be used to estimate the quantity of water which has to be conveyed over the spillway section of the dike. In case this high discharge is not correctly estimated then the stretch of dike might be destroyed, but that involves relatively low costs.

The 1/1000 years flood has a reliability of 95% that it will not be exceeded during the 50 years period. According to the Gumbel line of Figure 2.4, this flood corresponds with a peak-flow of 3,000 m3/s.

The flow which overtops the spillway on the left bank can be stored in the flood-plain between the western dike and the primary canal dike. The storage which is reserved in this flood plain to store the floods of the wadi Bu-y is 80*10^6m3 (see Chapter 4). This is found to be sufficient to store the water overtopping the western dike as well (see Figure 2.6).

Even if the wadi has a moderate flood, there still is some storage left for this eventuality. It is not correct to superimpose the design discharge (670 m3/s) of this wadi (see Chapter 4) on the design discharge of 1/1000 years, as that would correspond with a flood of a much smaller frequency.

As will be discussed furtheron (Chapter 4) the water will evacuate through two outlets, one leading through a culvert with 15 m3/s capacity towards the middle area and one along the northern boundary of the project towards the northern drain.

2.1.8. Extreme values at main drain outfall

To determine the frequencies of extreme water levels at the main drain outfall, use can be made of the Gumbel graph of Figure 2.4. The discharges in this curve can be converted to water levels if the topography of the river stretch at the main drain outfall is known. Using the slope-area method a water level can be attributed to each peak-flow.

Although it is not yet possible to make such calculations, because the required cross-sections for the slope-area calculations are not yet available, it is possible to make a preliminary estimate. The once in nineteen years flood (1985) level at the natural drain outfall is known from Figure 2.7 (314.50 m). Also the flood level which is exceeded almost every year can be estimated. At Code bridge the water depth which is exceeded almost every year is 2.0 m (10% probability in Figure 2.6) corresponding with a flow of about 400 m3/s. The gradient at Code bridge is 8.2*10^-4 and at the main drain 2.3*10^-4, a factor 3.6 smaller. Applying Chezy's equation,

$$Q = C*B*H*\sqrt{H*I}$$

and assuming Q, C and B as constants, then the water depth H should be increased by the cubic root of 3.6 which equals 1.5. The water depth then becomes 3.0 m for 10% probability of non-exceedence. This depth corresponds with a water level of 308 m.

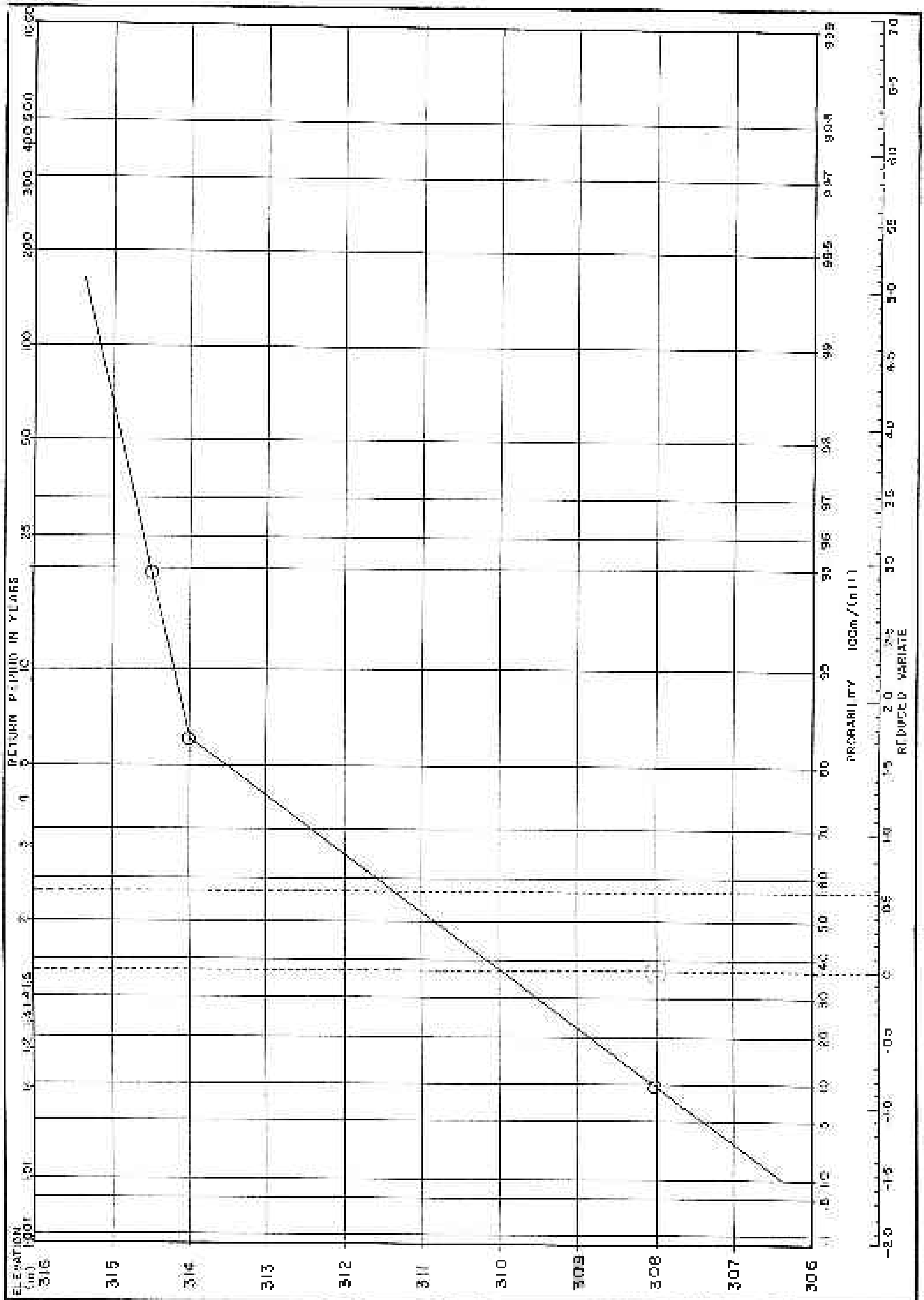
The 19 years flood and the 10% flood may not be connected by a straight line on the Gumbel paper. At 314 m the right bank of the river starts overtopping. The rate of rise of the water level will then be considerably less. The Gumbel graph, therefore, has to have an inflection point. It is estimated that the right bank starts overtopping with a frequency of once in 6 years (1976 flood). This is consistent with previous assumptions. Figure 2.11 now shows the preliminary Gumbel graph. Care should be excercised in using this graph. It may only be used as a preliminary estimate.

Considering the once in 5 and once in 3 year floods as design floods for the assessment of drainage problems, an estimate can be made of the time that certain drainage levels will be exceeded. The following table is a preliminary result. At a later stage a design flood can be routed through the project area with the Rihani model to supply more certain results.

Table 2.9 drainage levels exceeded at natural drain outfall

1/5 years		1/3 years	
Elevation (m)	Duration (days)	Elevation (m)	Duration (days)
313.5	1.0	312.0	1.0
312.5	1.5	311.0	1.5
312	2.0	310.0	2.0

As the main drain outfall will be located at one kilometer downstream of the existing drain, the water levels will still be 0.40 m lower and therefore no severe problems with drainage are expected in normal years.



PRELIMINARY GRAPH FOR EXTREME FLOOD LEVELS AT THE MAIN DRAIN OUT FALL

FIGURE 2·11

2.1.9 Recommendations for rating procedure

The lack of hydrological data and the incompleteness of existing information is always a serious problem when water resources development is planned. Although it is always possible to estimate hydrological values when information is lacking, serious efforts should be made to overcome the lack of information. In the case of this project the installation of the water level recorder near the diversion dam site was an adequate measure to make sure that from the start of the project the floods and droughts will be effectively monitored.

The section downstream of Gode bridge has to be rated during the dry season. This is essential to determine water availability. It will be necessary to measure the flows at least twice monthly during the dry periods.

The mere installation of a recorder, however, is not sufficient. The section has to be rated by regular discharge measurements, preferably by cableway, staff gauges should be installed at the recorder site.

Also sediment load measurements should be done for the dimensioning of the desilting basin. Suspended load measurements are considered sufficient for that purpose. In Appendix A recommendations for a rating procedure are summarized, followed by a procedure for the rating and analysis of suspended sediment samples.

2.2. Climate

The climate of the project area is described by the meteorological station in Gode town. This station is in operation since 1967. The available data on temperature, relative humidity, daily sunshine hours, wind speed, evaporation and rainfall are presented in Appendix B, Tables B.1 through B.6.

Temperature
The mean average temperature is around 28 oC, with average maximum and minimum temperatures respectively of 35 and 20 oC.

Broadly speaking there is a somewhat colder season with lower minimum temperatures from November till March i.e. prior to the early rainy season.

Relative Humidity and Windspeed
On the average, the relative humidity is about 56%, without any great daily or monthly variation. The mean wind speed is ranging from 4-6 m/sec in the period June-September accompanied by south-western winds, and from 2-3 m/sec in the period October-May, accompanied by north eastern winds. During day time generally strong winds are experienced.

Sunshine Duration
Sunshine durations are lower during the dry summer period, which is accompanied by relatively strong dusty winds. The highest values occur from November till April.