Chapter 5
THE POSSIBLE DAM BREAK

5.1 General

5.1.1 Possible modes of failure

The creation of a large dam and its associated reservoir represents a potential danger in terms of the catastrophic flood resulting from the extreme event of a dam-break.

Although a dam break at Kulekhani is a most unlikely event, some possible causes of failure may be due to :

- seismic activity
- soil slide/rock avalanche in the reservoir initiating a wave which overtops the dam crest and subsequently causes the failure of the dam
- differential settlement may cause leakage of water through the dam core which, if not discovered and attended to, will cause the dam to fail.

Nepal is an earthquake-prone country, and due consideration should be paid to this in the present context. In fact, the dam has been constructed along a seismic (not active) fault and requires therefore special attention.

Nepal is also prone to soil and rock avalanches, and several areas around the reservoir are of such a nature that they may slide into the reservoir and set up a wave overtopping the dam. In fact, a slide occurred during construction in the area of the intake for the power station. This area is now secured by extensive protection works including tensioned steel bars and a drainage gallery.

In 1985, a longitudinal crack was discovered in the impervious core of the dam. This was repaired by injecting bentonite and the leakage was stopped before any damage was done to the dam structure. The crack probably developed due to differential settlement of the dam.

In addition, a leakage in the headrace tunnel was discovered during the first fill-up. This was repaired by cement grouting.

5.1.2 On the calculations

The Terms of Reference. The Terms of Reference call for the carrying out of hydraulic analyses to determine:

- the limits of areas flooded at times of peak flow and the design capacity of the dam for flood control
- the water volumes, discharge rates and arrival times of flood peaks
- the areas flooded downstream of the dam for selected scenarios based on the above two activities.

Furthermore, the above analyses are to be based on existing information; i.e. drawings of the dam, maps of the downstream areas, etc.

The Terms of Reference do not define the project area. Based on communications between DHA and the Consultant, the Consultant has however, considered the project area to be the river stretch between the dam and the confluence of Kulekhani and Bagmati rivers.

Basic information. The basic information used in the present analyses were primarily found in the publications listed in Appendix B. This included drawings of the dam, hydrological data, etc, much of which is included as figures in the present report.

In addition, topographical maps, scale 1:50.000 for the area between the dam and the confluence of Kulekhani and Bagmati Rivers with contour interval of 100 ft, and maps of scale 1:500.000 between the confluence and the terai were used.

Additional information on soil properties were supplied in fax dated 23 July 1993 from

UNDP/Kathmandu to the Consultant.

The calculations. Basically, two types of calculations have to be carried out in the present context; namely :

- calculating the dam break wave, and
- the routing of the dam break wave down the valley.

The dambreak and the resulting wave were calculated by using BREACH, which is an erosion model for earthen dam failures developed by Hydrologic Research Laboratory, National Weather Service, NOAA, Maryland, USA. (Further details may be found in Appendix C).

In order to make use of routing models utilizing the complete dynamic equations (St. Vernant's equations), the longitudinal river profile and river cross-sections are inter alia required. As it is not possible to obtain such information based on the maps provided (minimum requirements are map scale 1:5.000 and 1 meter between the contour lines) it was decided to consider the valley as consisting of a number of reservoirs (with river stretches in between) and utilize standard reservoir routing procedures. Such procedures are described in standard textbooks[3].

5.2 Calculation of the Dam Break Wave

Two modes of failure have been simulated by BREACH; namely:

- failure due to overtopping, and
- failure due to piping.

These simulations are detailed in sections 5.2.1 and 5.2.2 respectively.

[3] e.g. Henderson: Open Channel Flow, MacMillan

5.2.1 Failure due to overtopping

A sufficiently high waterlevel in the reservoir to initiate a dam break due to overtopping was introduced in the BREACH model, and the associated out flow hydrogram was determined. As can be seen from Figure 5.1, the discharge rises steeply to a discharge close to the maximum discharge in the course of about 30 minutes (0,55 hrs). The maximum discharge of 2.827 cumecs is reached 2 hours after the failure process started. The discharge then starts to decrease, and 4 hours after failure, it decreases at a steady rate.

After 11.5 hrs, the discharge is down to 248 cumecs. The waterlevel is down to 1445 m; i.e. only about 2 million m^3 of water are left in the reservoir.

The maximum discharge of 2.827 cumecs is only about 50% more than the maximum design discharge of the spillway of 1.945 m^3/s.

KHULEKANI DAM BREAK STUDY.

Conversion and graphing of BREACH calculations.

Discharge cusecs		Time hrs	Discharge cumecs
=====	=====	=====	=====
13.269		0,05	376
94.422		0,55	2.674
97.328		1,05	2.756
99.146		1,55	2.807
99.850		2,05	2.827
99.443		2,55	2.816
97.945		3,05	2.773
95.358		3,55	2.700
91.795		4,05	2.599
87.217		4,55	2.470
81.935		5,05	2.320
76.203		5,55	2.158
70.146		6,05	1.986
63.880		6,55	1.809
57.511		7,05	1.629
51.128		7,55	1.448
44.805		8,05	1.269
38.513		8,55	1.093
32.728		9,05	927
27.200		9,55	770
22.024		10,05	624
17.149		10,55	486
12.530		11,05	355
8.761		11,55	248

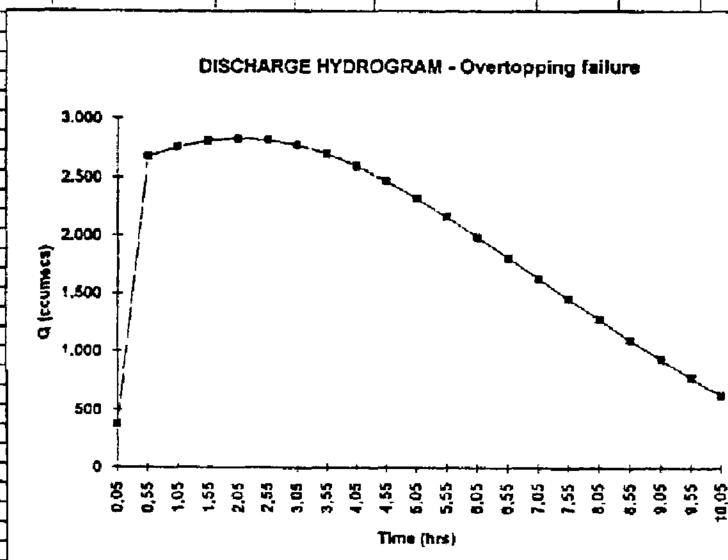

Figure 5.1: Discharge hydrogram for dambreak wave (overtopping failure)

BREACH gives the dimensions of the opening of the dam as follows:

final depth of breach	=	351,04 ft	=	107 m
final elevation of bottom of breach	=	4681,70 ft	=	1427 m
acute angle of breach with vertical	=	2,289 °		
bottom width of breach	=	5,21 ft	=	1,6 m

This means a volume equal to 161.369 m^3 or 3,65 % of the total dam volume has been washed away. Calculations for the overtopping failure case are included in Appendix D.

5.2.2 Failure due to piping

Failure due to piping was simulated by BREACH; - anticipating that the piping started at an elevation of 4700 ft (1.432 m); i.e. close to the base of the dam. The development of the breach is quite different from the overtopping case (reference Figure 5.2). At first, there is

KHULEKANI DAM BREAK STUDY - piping failure.

Conversion and graphing of BREACH calculations.

Discharge cusecs		Time hrs	Discharge cumecs
=====	=====	=====	=====
13.711		0,000	388
12.948		0,050	367
12.141		0,100	344
13.340		0,150	378
307.162		0,200	8.698
615 702		0,250	17.435
874.277		0,300	24.757
1.073 120		0,350	30.387
1.219.002		0.400	34.518
1.317 421		0.450	37.305
1.374.519		0,500	38 922
1.398.231		0,550	39 593
1.390.725		0,600	39.381
1.358.288		0,650	38.405
1.290.640		0,700	36 547
1.206 397		0,750	34.161
1.103.537		0,800	31.248
985.384		0,850	27.903
852.441		0,900	24.138
701.691		0,950	19 869
565.251		1,000	16.006
			500.748

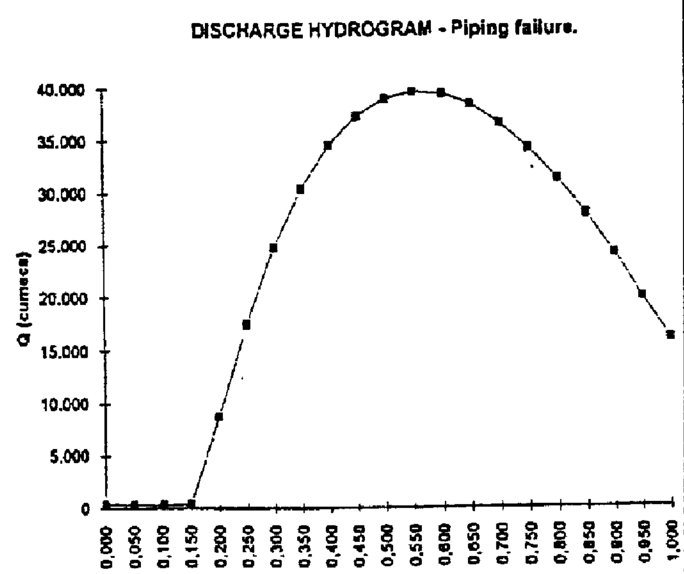

Figure 5.2: Discharge hydrogram for dambreak wave (piping failure)

an almost steady discharge of about 360 cumecs. Then, after 9 minutes the outflow increases sharply and reaches its maximum value of 39.593 cumecs after 33 minutes (0,55 hrs). It then decreases at an almost linear rate, and after 1 hour the reservoir is empty. (This development is rather similar to the failure of Teton dam. See Appendix E which also failed by piping.)

BREACH gives the dimensions of the opening of the dam as follows:

final depth of breach	=	351,04 ft	=	107 m
final elevation of bottom of breach	=	4681,70 ft	=	1427 m
acute angle of breach with vertical	=-	0,117 °		
bottom width of breach	=	128,18 ft	=	70 m

This means a volume equal to 1.737.800 m^3 or 40 % of the total dam volume has been washed away.

Calculations for the piping failure case are included in Appendix E.

5.3 Routing of dambreak wave

The dambreak waves computed by BREACH above are to be routed down the valley. As discussed previously, the maps available are not sufficiently detailed to carry out a routing utilizing the complete dynamic equations (St. Vernant's equations).

However, as noted in Chapter 4, there is a number of constrictions in the valley. Upstream of these constrictions, the valley floor was usually rather wide. Hence, in the case of a dambreak, these constrictions would serve as a series of reservoirs dampening the wave both in terms of peak magnitude and also delaying the peak discharge.

Unless the constrictions consist of sound rock, they will be subjected to erosion. Although erosion may be simulated in reservoir routing techniques, this has not been done in the present case. This because the rate of erosion is not known, and because the calculations would be based upon too many "ifs", and hence may not improve the final outcome of the study.

During the field trip, the party walked about halfway from the dam to the Kulekhani/Bagmati confluence. During this stretch, a total of three such constrictions were encountered, and hence it is logical to assume that there will be another three in the remaining half of the valley. Hence, there is a total of six constrictions (or reservoirs) between the dam and the confluence.

By doing so, one can use standard reservoir routing techniques in routing the dambreak wave down the valley. From the field trip and the maps provided, it was decided to assume that all the reservoirs had the same dimensions. These are:

length of reservoir	=	500 meters
width of reservoir	=	500 meters
breadth of constriction	=	25 meters

The sides of the reservoirs are considered steep (vertical) and critical flow conditions are assumed to prevail through the constriction (spillway).

In addition to the time delay of the peak flows due to the reservoirs, the actual travel time of the wave down the valley must be added in order to obtain the arrival time of the wave (peak). Again, one has to turn to experience and comparison with similar rivers and say that for a valley of this sort, narrow and having an average longitudinal slope of 1:42, it is natural to assume that the celerity of the wave will be about 2,0 m/s. Hence, without the six reservoirs, the travel time from the dam to the confluence with Bagmati River would be 7500 secs or roughly 2 hours.

5.3.1 Routing of wave due to overtopping failure

Figure 5.1 shows the dambreak wave due to overtopping. This wave was routed through the six reservoirs described in the preceding section in such a way that the actual dambreak wave (Figure 5.1) was used as input in the first reservoir and the (computed) wave coming out of this reservoir was used as input to the second reservoir, and so on. The result of the six routings is given in Figure 5.3.

SUM - DS - Overtopping.							
Discharge out of reservoir number ...							
Time	Q(res)	Q(1)	Q(2)	Q(3)	Q(4)	Q(5)	Q(6)
0,00	736	0	0	0	0	0	0
0,50	2.674	1.053	220	25	1	0	0
1,00	2.758	2.518	1.371	441	76	5	0
1,50	2.807	2.789	2.514	1 621	663	153	16
2,00	2.827	2.815	2.778	2 557	1 833	869	255
2.50	2.816	2.822	2.817	2.786	2.607	2.019	1.117
3,00	2.773	2.796	2.808	2.811	2.790	2.652	2.179
3,50	2.700	2.738	2.768	2.789	2.799	2.769	2.687
4,00	2.599	2.654	2.698	2.735	2.764	2.781	2.781
4,50	2.470	2.540	2.601	2.653	2.696	2.732	2.757
5,00	2.320	2.403	2.477	2.545	2 603	2.653	2.695
5,50	2.158	2.250	2.336	2.415	2.485	2.549	2.605
6,00	1.986	2.066	2.180	2.268	2.350	2.425	2.492
6,50	1.809	1.915	2 015	2.110	2 200	2.285	2.362
7.00	1.629	1.740	1 846	1.947	2.042	2 134	2 220
7,50	1.448	1.563	1.673	1.779	1 879	1.976	2.068
8,00	1 269	1.388	1.501	1.610	1 714	1.815	1.911
8,50	1.093	1.215	1.330	1.442	1.550	1.653	1.753
9,00	927	1 048	1.165	1.278	1.388	1.493	1.595
9,50	770	891	1.006	1.120	1.228	1.335	1.438
10,00	624	743	857	969	1.077	1.183	1.287
10,50	486	605	718	828	934	1.039	1.141
11,00	355	477	589	697	801	904	1.004
11,50	248	361	469	575	677	777	875

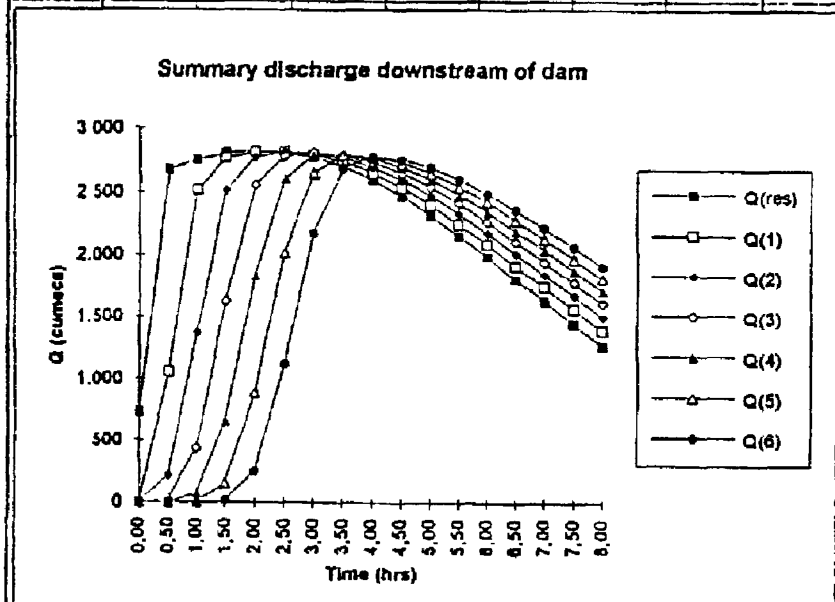

Figure 5.3: Dambreak wave (Q_{res} and the discharges out of the six reservoirs (overtopping failure)

From Figure 5.3 it is seen that there is hardly any reduction in the peak flow while there is a delay in the time occurrence of the peak. The lack of reduction of the peak flow is due to the fact that near stationary flow is established through each reservoir. This is so because the dambreak wave does not really have a single peak value, but a prolonged period with a high discharge around 2600 - 2800 cumecs. From the results included in Appendix D, it is seen that the reduction in peak flow through reservoir 1 is (2827 - 2822 =) 5 cumecs only, while the time delay of the peak is 30 minutes.

The maximum waterlevel in the first reservoir has been calculated at 16,37 meters and at 16,21 meters in the sixth (Figure 5.4).

Waterlevel summary - DS - Overtopping.

Time	WL(1)	WL(2)	WL(3)	WL(4)	WL(5)	Wl(6)
0,00	0,00	0.00	0,00	0,00	0,00	0,00
0,50	8 48	2,99	0,70	0,08	0,00	0,00
1,00	15,17	10,12	4,75	1,48	0,25	0,02
1,50	16,18	15,15	11,31	8,24	2,35	0,52
2,00	16.34	16,20	15 33	12,28	7,58	3,30
2,50	16 37	16 35	16 23	15,53	13,09	8,82
3,00	16 27	16,31	16 32	16,24	15,70	13,78
3,50	16 04	16.16	16 24	16,28	16,24	15,84
4,00	15,71	15,89	16 03	16,14	16,21	16,21
4,50	15 26	15,50	15,71	15,88	16,02	16,12
5,00	14 70	15,01	15,28	15,51	15,71	15,87
5,50	14,07	14,43	14,75	15,04	15,29	15,52
6,00	13,38	13,78	14,15	14,49	14,79	15,06
6,50	12,64	13,08	13,48	13,87	14,22	14,54
7,00	11,86	12,33	12,78	13,19	13,59	13,95
7,50	11,04	11,55	12,03	12,48	12,90	13,30
8,00	10,20	10,75	11,28	11,74	12,19	12,62
8,50	9 33	9,91	10,48	10,98	11,48	11,91
9,00	8,48	9,07	9,65	10,19	10,70	11,19
9,50	7,59	8,23	8,84	9,40	9,93	10,44
10,00	6 72	7,40	8,02	8,61	9,17	9,70
10,50	5,86	6,57	7,23	7,83	8,41	8,95
11,00	5,00	5,78	6,44	7,07	7,66	8,22
11,50	4,15	4,85	5,67	6,32	6,93	7,50

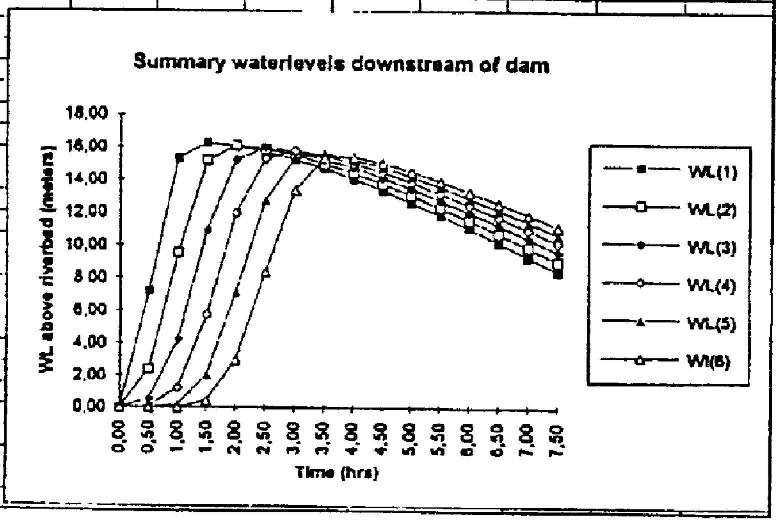

Figure 5.4: Waterlevels in the six reservoirs (overtopping failure)

Through the six reservoirs there is a total reduction of the peak discharge of (2827 - 2781 =) 46 cumecs, equalling a reduction of 1,6% and a total delay of the peak of 2 hours.

Hence, in the case of dam failure due to overtopping, it will take 4 hours for the wave to travel from the dam to the confluence, and the peak discharge will be reduced by 46 cumecs to 2781 cumecs.

Calculations for the overtopping failure case are included in Appendix D.

5.3.2 Routing of wave due to piping failure

The results are quite different when it comes to the piping failure wave.

By routing this wave through the first of the six reservoirs, we find that there is a reduction of peak flow of (39.593 - 36.477 =) 3.116 cumecs and a time delay of (0,70 - 0,55 = 0,15 =) 12 minutes (see Figure 5.5 and Table 5.1).

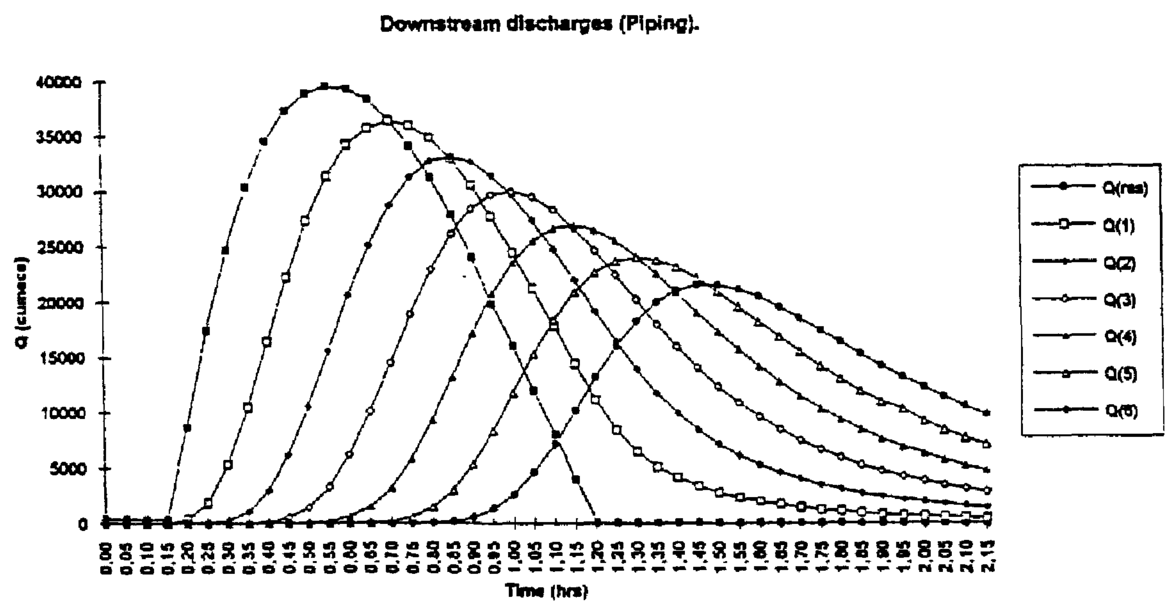

Figure 5.5: Dambreak wave (Q_{res} and the discharges out of the six reservoirs (piping failure)

Table 5.1: Tabulated values for the dambreak wave (Q_{res} and the discharges out of the six reservoirs (piping failure)

SUM - DS - Piping.							
Time	Q(res)	Q(1)	Q(2)	Q(3)	Q(4)	Q(5)	Q(6)
0,00	388	0	0	0	0	0	0,00
0,05	367	6	0	0	0	0	0,00
0,10	344	16	0	0	0	0	0,00
0,15	378	28	0	0	0	0	0,00
0,20	8698	328	3	0	0	0	0,00
0,25	17435	1886	39	0	0	0	0,00
0,30	24757	5357	271	2	0	0	0,00
0,35	30387	10481	1083	20	0	0	0,00
0,40	34518	16413	2954	121	1	0	0,00
0,45	37305	22264	6153	486	6	0	0,00
0,50	38922	27375	10534	1427	39	0	0,00
0,55	39593	31383	15584	3281	175	1	0,00
0,60	39381	34208	20656	6221	582	9	0,01
0,65	38405	35781	25174	10118	1521	49	0,16
0,70	36547	36477	28788	14565	3256	193	1,58
0,75	34161	36077	31335	19035	5913	588	10,30
0,80	31248	34874	32768	23039	9378	1455	49,30
0,85	27903	33013	33178	26235	13304	3009	181,38
0,90	24138	30595	32699	28455	17246	5350	531,66
0,95	19869	27701	31470	29678	20780	8381	1281,49
1,00	16006	24508	29637	29975	23606	11818	2610,90
1,05	12000	21199	27364	29475	25574	15282	4608,62
1,10	8000	17826	24798	28333	26683	18412	7205,38
1,15	4000	14452	22051	26704	26945	20947	10176,11
1,20	0	11131	19217	24728	26540	22749	13209,76
1,25	0	8387	16441	22534	25589	23799	16000,24
1,30	0	6478	13915	20259	24236	24155	18317,05
1,35	0	5108	11745	18033	22621	23923	20032,50
1,40	0	4100	9928	15949	20872	23231	21116,47
1,45	0	3341	8424	14056	19092	22209	21612,33
1.50	0	2758	7182	12371	17355	20973	21605,51
1,55	0	2304	6156	10890	15710	19620	21200,64
1,60	0	1944	5306	9599	14184	18224	20501,24
1,65	0	1656	4598	8477	12789	16840	19599,43
1,70	0	1422	4006	7505	11526	15502	18572,98
1,75	0	1230	3508	6664	10392	14235	17481,43
1,80	0	1071	3087	5933	9378	13052	16371,10
1,85	0	939	2729	5299	8474	11858	15273,45
1,90	0	827	2423	4747	7668	10952	14211,44
1,95	0	732	2161	4265	6951	10334	13239,15
2,00	0	652	1934	3844	6313	9198	12311,86
2,05	0	583	1737	3473	5745	8439	11399,44
2,10	0	523	1566	3148	5238	7750	10555,21
2,15	0	471	1417	2860	4765	7126	9776,61

The maximum waterlevel in the first reservoir has been calculated to 90,14 meters, and to 63,59 meters in the sixth (see Figure 5.6).

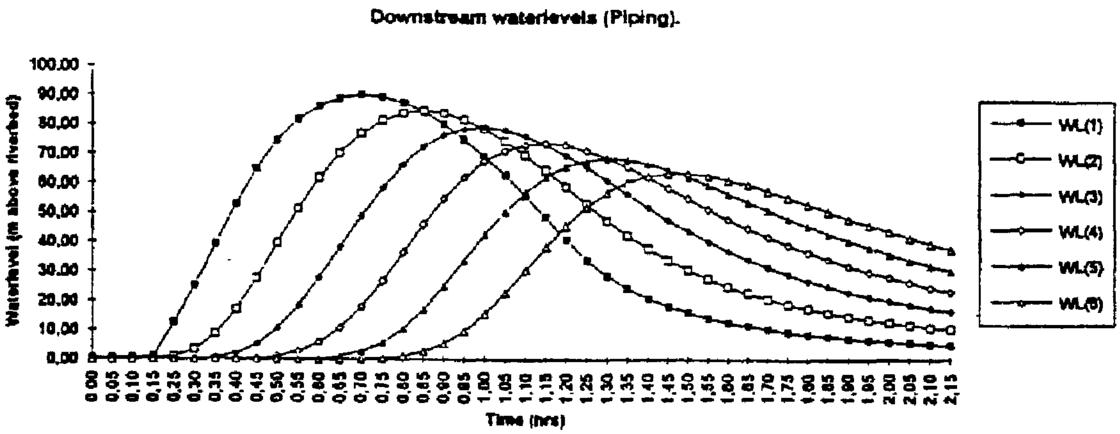

Figure 5.6: Waterlevels in the six reservoirs (piping failure)

Table 5.2: Tabulated values for the waterlevels in the six reservoirs (piping failure)

SUM - DS - Piping.						
Time	WL(1)	WL(2)	WL(3)	WL(4)	WL(5)	WL(6)
0,00	0,00	0,00	0,00	0,00	0,00	0,00
0,05	0,27	0,00	0,00	0 00	0,00	0,00
0,10	0,52	0 01	0,00	0,00	0,00	0,00
0,15	0,78	0,03	0,00	0 00	0,00	0,00
0,25	12,51	0,94	0,02	0,00	0 00	0,00
0,30	25,09	3,43	0,13	0 00	0 00	0,00
0 35	39,25	8,65	0,61	0,01	0 00	0 00
0,40	52,93	16,87	2,01	0,06	0,00	0 00
0,45	64,86	27,52	5,07	0,27	0 00	0,00
0,50	74,44	39,38	10,38	0,85	0,02	0,00
0,55	81,54	51,13	18,09	2,58	0,10	0,00
0,60	86,36	61,70	27,72	5,71	0 38	0 00
0,65	88,99	70 39	38 34	10,84	1,10	0,02
0,70	90,14	76,98	48,88	18,00	2,73	0,11
0,75	89,48	81,48	58 43	26 80	5,75	0,39
0,80	87 48	83 92	66,36	36 44	10 52	1,10
0,85	84,34	84,62	72,38	46 02	17 08	2 63
0,90	80,17	83 80	76,39	54,71	25 07	5,38
0 95	75 03	81,69	78,56	61,94	33,82	9,67
1,00	69 15	78 49	79,06	67,44	42,52	15,54
1,05	62,78	74 42	78,20	71,14	50,47	22,70
1,10	55,93	69 69	76,17	73,14	57,15	30 57
1,15	48 63	64,45	73,22	73 66	62,28	38 49
1 20	40 88	58,60	69 56	72,92	65,80	45 90
1,25	33 63	52,99	65,38	71 17	67 81	52,04
1 30	28 48	47,41	60,91	68,64	68,44	56 95
1,35	24,31	42,35	56,38	65,55	68,04	60 45
1,40	20,99	37,86	51,93	62,13	66 72	62 61
1,45	18 31	33,93	47 73	58 54	64 75	63,59
1,50	16,12	30,51	43,84	54,94	62 33	63 57
1,55	14,30	27,53	40 26	51 41	59,62	62,78
1,60	12,77	24,93	37 02	48 02	56 78	61,39
1,65	11,47	22,66	34 07	44 82	53 84	59,58
1,70	10 36	20 87	31 42	41 82	50 95	57 48
1 75	9,41	18 92	29,02	39 03	48,14	55,20
1,80	8,58	17 38	26,86	36 45	45 43	52,84
1 85	7 86	16 01	24,91	34,06	42 86	50,45
1,90	7,22	14,79	23,15	31 87	40,42	48,08
1,95	6 66	13,70	21,55	29 85	38 13	45 87
2 00	6 16	12,72	20,11	27 99	35 98	43 70
2 05	5 72	11,84	18,80	26,29	33,97	41 51
2,10	5,32	11,05	17,60	24,72	32,10	39 44
2 15	4 96	10 34	16,51	23 27	30 35	37 47

Through the six reservoirs there is a total reduction of the peak discharge of (39.593 - 21.612 =) 17.981 cumecs, equalling a reduction of 45% and a total delay of the peak of 54 minutes; say 1 hour.

Hence, in the case of dam failure due to piping, it will take 3 hrs for the wave to travel from the dam to the confluence, and the peak discharge will be reduced by 17.981 cumecs to 21.612 cumecs.

Calculations for the piping failure case are included in Appendix E.

5.4 Impacts Downstream of Dam

5.4.1 Failure due to overtopping

The impact due to overtopping failure will not be too serious. A peak discharge of about 150

Table 5.2: Tabulated values for the waterlevels in the six reservoirs (piping failure)

% of the spillway design discharge will be reached and the maximum waterlevel will be about 16 meters above the riverbed.

Erosion along the steeper parts of the river, and particularly at the constrictions, must be expected and the eroded material will be deposited along the flatter stretches of the river.

The eroded dam construction material will be deposited in the upper reaches of the river, particularly in the flatter areas. This will amount to about 161.000 m^3.

Houses in the entire valley having an elevation less than 16 meters above the riverbed will be in danger (washed away). So will also houses situated on top of ridges close to the constrictions, as these will be subjected to undercutting.

The peak of the wave will travel the distance between the dam and the confluence in four hours.

5.4.2 Failure due to piping

The impacts of the dam break wave downstream of the dam will be considerable. First of all, the wave will have a peak discharge of 38.900 m^3/s, and the valley being rather steeply sloped, this wave will have a tremendous force and erosive capability. Also, the wave will carry not only the sediments which it erodes on its way down the valley, but also the sediments carried away from the actual breach in the dam. The dam erosion will amount to about 1.75 million m^3.

As described in the field excursion report (Chapter 4), the valley is varying in width and also comprises a number of constrictions. As the wave encounters a constriction/hydraulic control section, the flow through the constriction will be reduced, and hence creating a reservoir upstream of the constriction. This in turn leads to lower water velocities and hence a reduced sediment carrying capacity and sediments will be deposited upstream of the constriction. If the area upstream of the constriction is wide and flat (as is the case of the Chakhel confluence), it is probably used as agricultural area. As quite large rocks and boulders must be expected to be deposited in such an area, it is obvious that it will be rendered useless for agricultural purposes.

Erosion however, may occur at the constriction itself if it does not consist of sound rock. Hence, undercutting may occur and the sides of the constriction may fall in. If houses are located on the flat upper areas next to the constriction (as is the case in Todke) these houses are liable to be undercut by erosion and fall into the water.

Another impact which probably will occur in the Chakhel confluence is that due to the flat areas and downstream constriction, backwater effects will be inevitable in the Chakhel river.

The waterlevel in the six reservoirs due to this wave will vary between 90 meters in the upper one and 63 meters in the lower one. With reference to Chapter 4, houses were located both at low and higher grounds. The low-lying houses will be rather exposed to the dam break wave, while the higher-lying ones remain safe unless they are undercut by erosion (as will probably be the case in several areas visited).

It is anticipated that half of the houses in the villages of Devaltar, Khanikhet and Todke will be affected (.ie. destroyed) by the dam break wave and hence, also half of the population. It is reasonable to assume that a similar scenario will apply to the lower part of the valley down to the Bagmati confluence.

The peak of the wave will travel the distance between the dam and the confluence in three hours.

If the dam break occurs during night-time, it is reasonable to anticipate that the losses, particularly in terms of human lives, will be more severe.

5.5 Summary

In the case of failure due to overtopping:

- the peak discharge of the dambreak wave will be 2.827 cumecs,
- the peak outflow will be reached 2 hours after the failure process has started,
- the reservoir is emptied in the course of 11 - 12 hours,
- a volume of 161.369 m^3 of the dam will be washed away,
- there will hardly be any reduction in the peak discharge as the wave travels down the valley,
- the maximum waterlevel in the valley will be about 16,5 meters above the riverbed,
- the peak of the wave will travel the distance between the dam and the Kulekhani/Bagmati confluence in 4 hours, and
- too serious damage is not envisaged.

In the case of failure due to piping:

- the peak discharge of the dambreak wave will be 39.593 cumecs,
- the peak outflow will be reached 33 minutes after the failure process has started,
- the reservoir is emptied in the course of 1 hour,
- a volume of 1.737.800 m^3 of the dam will be washed away,
- there will be a reduction of 45% in the peak discharge as the wave travels down the valley,
- the maximum waterlevel in the valley will be about 90 meters above the riverbed in the upper reaches and roughly 63 meters in the lower,
- the peak of the wave will travel the distance between the dam and the Kulekhani/Bagmati

confluence in 3 hours, and

- very serious damage and loss of lives are envisaged.

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Chapter 6
CONCLUSIONS - RECOMMENDATIONS

6.1 Conclusions

The findings of the Consultant may be summarized as follows:

A possible break of the Kulekhani dam will cause:

- as much as 85,3 million metric tonnes of water may pass through the damsite in a matter of hours.
- Nepal will loose a major source of electric power (probably for a prolonged period of time).
- Maybe as many as fifty percent of the houses in the valley between the dam and Bagmati river may be destroyed by the floodwave.
- Lives will be lost and considerable damage will be done in the villages downstream of the dam.
- Agricultural land will be destroyed.
- Sediments carried by the flood water will be deposited in areas having a reduced slope. These sediments may form a temporary dam which will create a reservoir. The temporary dam will eventually break and cause a second flood wave down the valley.
- If the sediments are deposited just after Kulekhani joins another river, this may cause backwater effects and flooding in the second river also.

6.2 Recommendations

The effects of a possible dam-break may be mitigated by assessing the flood hazards and vulnerability of the downstream areas and preparing proper plans for evacuation of people living in the vulnerable areas.

The following, in addition to the installation of a warning system (e.g. sirens), should be initiated/undertaken.

Assessment of Vulnerability

Taking into account the severity of the flood hazard and the population exposed in each area, a list should be made of the localities which vulnerability will be determined in order of priority. In each such locality, the study will include:

- the population distribution by age, the socio-economic and cultural level, etc.
- the physical environment, soil characteristics, types, number and quality of houses.,

This information, together with data on the flood hazard will enable the preparation of scenarios of possible flood emergencies.

Evacuation Plans

The preparation of plans for the evacuation of the population from threatened areas is one of the most important elements in flood mitigation. Such plans must be based on estimates of the peak flood level, its time and duration.

Refuge zones should be identified and delimited and arrangements must be foreseen for the reception and care of the refugees.

It will be necessary to identify escape routes, check their state of repair, and remove any obstacles to the rapid movement of evacuees into the refuge zones.

Evacuation procedures should be planned in consultation with the whole community as well as with the organization directly concerned/responsible.

Training for Evacuation

Of equally high importance is the training of the public authorities and of the population as a whole in evacuation procedures, through simulated emergency exercises.

Such exercises will help to reveal any deficiencies in the plan and to remedy them. Leaflets and other printed information should be prepared for distribution to the local authorities and to the general public.

Feasibility of moving key installations and action to mitigate the effects of floods on buildings

Economic feasibility studies should be undertaken as a basis for deciding which facilities can be moved from zones liable to flooding to safer sites.

On the basis of previous knowledge and experience of flood effects in the area, measures to reduce possible flood damage should be recommended.

Microzoning Studies and Physical Planning

It is foreseen that microzoning studies will be carried out using a simplified method with a view to encouraging the villages downstream of the dam to expand towards zones less exposed to the flood hazard.

Criteria should be formulated for the selection of safer sites for important engineering works, industrial plants, etc.

Alternative ways of supplying electric power

A study should be undertaken to establish possible ways of providing electricity in the case of a prolonged power cut. This should also encompass rationing plans for the supply of electricity as well as providing vital public authorities and institutions with emergency (diesel) generators.