El documento original no contiene las páginas 6 - 12 y 32.

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Department of Humanitarian Affairs

KULEKHANI DAM BREAK STUDY

United Nations
1994

LIST OF CONTENTS

ABBREVIATIONS and ACRONYMS

ASCE	American Society of Civil Engineers
cfs	Cubic feet per second
cumec	Cubic meter per second
cusec	Cubic feet per second
DHA	Department of Humanitarian Affairs
EL	Elevation
FAO	United Nations Food and Agriculture Organization
GLOF	Glacial Lake Outburst Flood
IBRD	International Bank for Reconstruction and Development
ICOLD	International Commission on Large Dams
IDA	International Development Association
KDPP	Kulekhani Disaster Prevention Project
MoH	Ministry of Home
MoWR	Ministry of Water Resources
NEA	Nepal Electricity Authority
OPS	Office for Project Services
PMF	Probable maximum flood
SDU	Special Disaster Unit
UNDP	United Nations Development Programme
US$	United States Dollar
USCOLD	US Commission on Large Dams
WECS	Water and Energy Commission Secretariate

LIST OF FIGURES

LIST OF TABLES

FOREWORD

DHA is pleased to present the following technical report entitled ***Kulekhani Dambreak Study***. The hundreds of large and small dams in the world are all sources of hazards in terms of the potential catastrophic floods which result from dambreaks.

The danger of dam failure has in many cases increased due to changes in runoff as a result of deforestation in the catchment basin, a common feature today in both developed and developing countries. In other words, the peak flood from a catchment may be much higher than was calculated at the time of design and construction of water control structures. Many countries have experienced the consequences of this phenomenon in recent floods which have often shown the discharge capacity of dams to be insufficient, in several cases leading to disaster. Specific projects are needed to evaluate the possible consequences of environmental changes and to work out additional monitoring systems and safety precautions. This report reviews possible dambreak scenarios for a particular dam, Kulekhani located in Nepal, and provides details on calculating the impact of the failure on the population and infrastructure downstream of the dam.

The economic consequences of floods caused by dam failure are usually extremely heavy due to the fact that they often affect large and heavily populated areas and related infrastructure. This study illustrates the necessity of the inclusion of risk analysis and pre-disaster mitigation planning into economic and social development plans.

The present study, as per the terms of reference, was focused on a hydraulic analysis to establish the limits of areas flooded at the time of peak flow under different hypotheses of dam rupture (complete, partial and minor). It is clear that the exercise of disaster mitigation planning is a long one requiring further substantial studies by specialists of differing backgrounds, to be undertaken at the next stage of the exercise, and to concentrate on:

- assessment of the impact of the dambreak on the population and relevant infrastructure down river;
- assessment of local capacities to ensure proper monitoring of the hazard;
- establishment of appropriate early warning; and
- training of the population and relevant authorities for proper response to emergency situations.

The present study was carried out by Dr. Tor Aamodt of Oceanor, Norway, who acted as the Department of Humanitarian Affairs' Consultant. All expenditures for his mission were jointly covered by Norway and UN Agencies, i.e. the United Nations Development Programme, Office for Project Support, and DHA.

The purpose of this report is to bring to the attention of technical authorities concerned, at the national and local levels, the wide range of mitigation measures which should be implemented at the pre-disaster stage. The report is being distributed to relevant authorities in other countries with similar disaster potential, in order to stimulate the wider application of appropriate mitigation methods.

Disaster Mitigation Branch
Department of Humanitarian Affairs
Geneva
September 1994

Chapter 1
INTRODUCTION

1.1 General Background

The present report comprise the findings and outputs from the analyses carried out for the UNDP/OPS - DHA assignment "**Kulekhani Dam Break Study**". The contract with UNDP/OPS is dated 18 August 1992, and was signed on 28 August 1992. The Mission consisted of one member only; namely Dr. Tor Aamodt, Hydrology/Flood Specialist. The contract included a visit to Nepal, and this was undertaken during the period April 16th -April 29th, 1993.

During his stay in Nepal, the Consultant conducted a number of meetings with Nepalese authorities as well as with other UN-organizations. In addition, a one-day field trip to Kulekhani dam and the area downstream of the dam was carried out.

The Consultancy was executed in close cooperation with UNDP/Kathmandu and the Kulekhani Disaster Prevention Project (KDPP). KDPP is mainly directing its activities towards the prevention of landslides in the area between the Kulekhani dam and power stations 1 and 2; - i.e. along the supply tunnel and the penstock.

During the execution of the assignment, the Consultant had also close contact with Nepalese Authorities; - particularly the Prime Minister's Private Secretariat, Ministry of Water Resources and Nepal Electricity Authority.

The remaining part of Chapter 1 deals with the Terms of Reference for the Consultancy and acknowledgements. General information - including examples - on dam failures may be found in Chapter 2. Chapter 3 contains relevant background information on the Kulekhani Hydropower Project, while Chapter 4 gives the findings of the field trip. Chapter 5 contains the analyses and impacts of the dambreak wave, while conclusions and recommendations may be found in Chapter 6.

There are 3 appendices. The Terms of Reference for the mission may be found in Appendix A. List of references is given in Appendix B. Appendix C contains information on the BREACH model, while the calculations for the overflow and piping failures are included in Appendices D and E respectively.

1.2 Discussion of the Terms of Reference

The Terms of Reference (included in Appendix A) calls for the carrying out of hydraulic analyses under different hypotheses of dam rupture. The hydraulic analyses are to establish the water volumes, discharge rates and arrival times of the flood peaks. Also, the areas likely to be flooded for the various scenarios are to be estimated.

The above mentioned activities are to be carried out based on **existing** information such as drawings of the dam, maps of the reservoir and of the downstream areas.

In order to estimate the areas downstream of the dam which are likely to be flooded, quite detailed maps are required; - say to a scale of 1:5.000 with an equidistance of 1 meter between the contour lines. As it has not been possible to find such maps of the area in question - maps in 1:50.000 and contour interval of 100 ft only were obtained - it is evident that an estimation of the flooded areas is not possible with any degree of reliability.

Based on assumptions and idealizations, the Consultant has - however - been able to come up with indications of areas to be flooded, and also the determination of safe areas where the population can seek refuge in the extreme event of a dambreak.

1.3 Acknowledgements

The Consultant wishes to express his thanks to all officials and individuals met for their kind support and valuable information which he received during his stay in Nepal, and which highly facilitated the work of the Consultant. In particular, his thanks are extended to UNDP, NEA and KDPP.

Particular thanks are extended to Mr Rajendra P. Hada, KDPP, who was the Consultant's "right hand" during his stay in Nepal, and Mr Hada played a major role in making the Consultant's stay in Nepal a most effective one.

Lastly, but not leastly, thanks are also directed to the Norwegian Consultancy Trust Fund (NCTF) with UNDP/OPS which funded the present study. The assistance and cooperativeness of the NCTF are highly appreciated.

Chapter 2
ON DAMS AND DAM FAILURES

2.1 General

Dams provide society with essential benefits such as water supply, flood control, recreation, hydropower, irrigation, etc. However, catastrophic flooding occurs when a dam fails and the impounded water escapes though the breach and into the downstream valley. Usually, the magnitude of the flow greatly exceeds all previous floods and the response time available for warnings is much shorter than for precipitation-runoff floods.

According to reports by the International Commission on Large Dams (ICOLD, 1973) and the United States Committee on Large Dams in cooperation with the American Society of Civil Engineers (ASCE/USCOLD, 1975) about 38% of all dam failures are caused by overtopping of the dam due to inadequate spillway capacity and by spillways being washed out during large inflows to the reservoir from heavy precipitation runoff. About 33% of the dam failures are caused by seepage or piping through the dam or along internal conduits, while 23% of the failures are associated with foundation problems, and the remaining failures are due to slope embankment slides, damage or liquefaction of earthen dams from earthquakes, and landslide-generated waves within the reservoir.

In the USA, the potential for catastrophic flooding due to a dam failure was brought to the nation's attention during the 1970s by several floods due to dam failures such as the Buffalo Creek Coal-Waste Dam, the Teton Dam, the Toccoa Dam, and the Laurel Run Dam.

Also, there are many dams that are 30 or more years old, and many of the older dams are a matter of serious concern because of increased hazard potential due to downstream development and increased risk of failure due to structural deterioration or inadequate spillway capacity. A report by the U.S. Army (1981) gives an inventory of the approximately 70,000 dams in the USA with heights greater than 25 ft or storage volumes in excess of 50 acre-ft. The report also classifies some 20,000 of these as being "... so located that failure of the dam could result in loss of human life and appreciable property damage ....".

2.2 Dam Failures

The actual failure mechanics are not well understood for either earthen or concrete dams. In previous attempts to predict downstream flooding due to dam failures, it was usually assumed that the dam failed completely and instantaneously. More recently, the need to consider partial rather than complete breaches has been recognized.

The assumptions of instantaneous and complete breaches were used for reasons of convenience when applying certain mathematical techniques for analyzing dam-break flood waves. These assumptions are somewhat appropriate for concrete arch dams, but they are not appropriate for earthen dams and concrete gravity dams. In these cases, the breach will develop over a finite interval of time and will have a final size determined by the bottom width, and the shape of the breach will be largely dependent upon the angle of repose of the compacted and wetted materials through which the breach develops.

2.2.1 Concrete Dams

Concrete gravity dams tend to have a partial breach as one or more monolith sections formed during the construction of the dam are forced apart and over-turned by the escaping water. The time for breach formation is in the range of a few minutes.

Concrete arch dams tend to fail completely and are assumed to require only a few minutes for the breach formation.

2.2.2 Earthen Dams

Earthen dams - which exceedingly outnumber all other types of dams - do not tend to fail completely, nor do they fail instantaneously. The fully formed breach in an earthen dam tend to have a width which is usually much less than the total length of the dam as measured across the valley. Also, the breach requires a finite interval of time to form through the erosion of the dam materials by the escaping water. Total time of failure (for overtopping) may be in the range of a few minutes to usually less than an hour; depending on the height of the dam, the type of materials used for construction, the extent of compaction of the materials, and the magnitude and duration of the overtopping flow of the escaping water. For overtopping failures the beginning of breach formation is after the downstream face of the dam has eroded away and the resulting crevasse has progressed back across the width of the dam crest to reach the upstream face.

Piping failures occur when initial breach formation takes place at some point below the top of the dam due to erosion of an internal channel through the dam by the escaping water. Times of failure are usually considerably longer for piping than overtopping failures, since the upstream face is slowly being eroded in the very early phase of the piping development. As the erosion proceeds, a larger and larger opening is formed; this is eventually hastened by the caving-in of the top portion of the dam.

2.3 Cases of Dam Failures

2.3.1 Teton Dam

The Teton Dam - a 300 ft high earthen dam with a 3.000 ft long crest and 250.000 acre-ft of stored water - failed on June 5th, 1976, killing 11 people, making 25,000 homeless and inflicting about US$400 million in damages to the downstream Teton-Snake River Valley. According to a report by Ray, et al (1976), the failure started as a piping failure about 10:00 am and slowly increased the rate of outflow until about 12:00 noon when the portion of the dam above the piping hole collapsed and in the next few minutes (about 12 minutes according to Blanton 1977), the breach became fully developed allowing an estimated 1,6 to 2,8 million cfs (best estimate 2,3) peak flow to be discharged into the valley below. At the time of peak flow, the breach was estimated from photographs to be trapezoidal in shape, having a top width at the original water surface elevation of about 500 ft and side slopes of about 1 (vertical) to 0,5 (horizontal). After the peak outflow, the outflow gradually decreased to a comparatively low flow in about 5 hrs as the reservoir volume was depleted and the surface elevation receded. The downstream face of the dam had a slope of 1:2 and the upstream face 1:2,5, the crest width was 35 ft and the bulk of the breach material was a D_{50} size of 0,03 mm. The inflow to the reservoir during failure was insignificant.

2.3.2 Lawn Lake Dam

The Lawn Lake Dam - a 26 ft high earthen dam with approximated 800 acre-ft of storage - failed July 15th, 1982, by piping along a bottom drain pipe. The material properties of the breach were:

D_{50} = 0,25 mm
ø = 25 °
C = 100 lb/ft^2
= 100 lb/ft^3

The downstream face of the dam had a slope of 1:1,3 and the upstream face 1:1,5. The actual peak outflow has been estimated at 18.000 cfs. The breach was trapezoidal in shape; having top and bottom widths of 97 and 55 ft respectively.

2.3.3 Mataro Landslide Dam

On April 25th, 1974, a massive landslide occurred in the valley of the Mantaro River in the mountainous area of central Peru. The slide - with a volume of approximately $5,6*10^{10}$ ft^3, dammed the Mantaro River and formed a lake which reached a depth of about 560 ft before overtopping during the period June 6-8, 1974. The overtopping flow eroded very gradually a small channel along the approximately 1 mile long downstream face of the slide during the first 2 days of overtopping. Then a dramatic increase in the breach channel occurred during the next 6-10 hrs, resulting in a final trapezoidal-shaped breach channel approximately 350 ft in depth, a top width of some 800 ft, and side slopes of about 1:1. The peak flow was estimated at 353,000 cfs (Lee and Duncan - 1975), while a later estimate arrived at 484,000 cfs (Ponce and Tsivoglou - 1981).

The breach did not erode down to the original river bed; and this caused a rather large lake to remain after the breaching had subsided some 24 hrs after the peak had occurred. The slide material was mostly a mixture of silty sand with some clay resulting in a D_{50} size of about 11 mm with some material ranging in size up to 3 ft boulders.

Chapter 3
ON THE KULEKHANI PROJECT

3.1 General Information

The Kulekhani Hydroelectric Power Plant is the major supplier of electricity to the Central Grid. It is located on and around the Mahabarat ranges about 25 to 40 km southwest of Kathmandu. It covers two different river basins, i.e. the Kulekhani River basin (which is one of the tributaries of the Bagmati River) and the upper Rapti River basin which is separated from the Kulekhani basin by a mountain ridge. The Kulekhani Hydroelectric Power Project was completed and commissioned in 1982 and consists of the following main components:

a) Kulekhani river basin including the Chakhel and Sim rivers diversions and associated watersheds
b) Kulekhani dam
c) Kulekhani reservoir
d) Kulekhani dam spillway
e) Kulekhani river outlet
f) Intake and headrace tunnel
g) Surge tank
h) Penstock
i) Power station (Kulekhani 1)
j) Tailrace tunnel
k) Transmission line and substations.

In the following, only items a) through d) from the list above will be dealt with.

With reference to Chapter 4, Figure 4.1 shows the outlet of the Chackel diversion, while Figures 4.2 and 4.3 give general views of the valley downstream of the dam.

3.1.1 Kulekhani River Basin (including Chakhel and Sim)

The Kulekhani River originates in the Mahabarat mountain ranges, and is a tributary to the Bagmati river. The catchment area at the damsite is about 126 km^2. Two downstream tributaries have been diverted into the reservoir or the headrace tunnel directly; namely the Chakhel river and the Sim river with catchment areas of 23 km^2 and 7 km^2 respectively.

The runoff of the Kulekhani river has been observed at the damsite since 1963. The readings have been checked with those of the nearby Chobhar station in the Bagmati river. In addition, the runoff of the tributaries Chakhel and Sim were measured from 1966 to 1973 at the sites of their respective intake weirs. The average annual runoff at the damsite over the observation period is about 123 million m^3 or 3,9 m^3/s in annual mean. The specific runoff is calculated at 3,1 $m^3/s/100km^2$.

The runoff records of the Chakhel and Sim rivers have been extended for the period from 1963 to 1972 by correlating the measured flow data with the corresponding records at the Kulekhani damsite. The average annual runoffs for these two tributaries are estimated at 22,4 mill m^3 and 11,8 mill m^3, or corresponding to an annual mean discharge of 0,71 m^3/s and 0,37 m^3/s, respectively.

The largest recorded flood between 1963 and 1979 at the Kulekhani damsite occurred on July 16, 1970, and it had a peak discharge of 572 m^3/s. The peak discharge of annual maximum floods for the same period is given in Table 3.1.

Mean monthly flows for the period 1963-72 for Kulekhani, Chakhel and Sim rivers are given in Table 3.2.

Table 3.1: Peak discharge (in m^3/s) of annual maximum floods at the dam site

Date	Peak Discharge
Sep 29, 1963	40,0
Jul 15, 1964	82,8
Jul 7, 1965	304,0
Aug 24, 1966	197,0
Jul 10, 1967	277,0
Oct 4, 1968	141,0
Aug 21, 1969	32,6
Jul 16, 1970	572,0
Jun 13, 1971	305,0
Jul 24, 1972	251,0
Jun 17, 1973	99,6
Aug 30, 1974	236,0
Jul 28, 1975	116,0
Jun 30, 1976	76,0
Jun 20, 1977	16,0
Jun 16, 1978	380,0
Aug 21, 1979	370,0

Table 3.2: Monthly mean discharge for Kulekhani, Chakhel and Sim Rivers. (Unit m^3/s)

A(Kulekhani) = 126 km^2, A(Chakhel) = 23 km^2,
A(Sim) = 7 km^2. A(Total) = 156 km^2.

	Month Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Yearly Mean
Kulekhani	1963	1,17	1,06	1,13	1,15	1,64	1,13	6,60	7,64	5,75	3,53	2,16	1,75	2,89
Chakhel	1963	2,21	0,19	0,21	0,21	0,30	0,21	1,20	1,38	1,04	0,64	0,39	0,32	0,52
Sim	1963	0,09	0,08	0,09	0,09	0,13	0,09	0,63	0,74	0,54	0,31	0,18	0,14	0,26
Total	1963	1,47	1,33	1,43	1,45	2,07	1,43	8,43	9,76	7,33	4,48	2,73	2,21	3,67
Kulekhani	1964	1,47	1,39	1,16	0,91	1,27	2,21	7,24	12,12	15,23	4,49	1,84	1,50	4,24
Chakhel	1964	0,27	0,25	0:21	0;16	0,23	0,40	1,31	2,19	2,76	0,81	0,33	0,27	0,77
Sim	1964	0,12	0,11	0,09	0,07	0,10	0,18	0,70	1,24	1,61	0,41	0,15	0,12	0,41
Total	1964	1,86	1,75	1,46	1,14	1,60	2,79	9,25	15,55	19,60	5,71	2,32	1,89	5,42
Kulekhani	1965	1,32	1,29	1,22	1,76	1,47	3,74	19,54	18,36	5,72	3,16	2,86	1,75	5,18
Chakhel	1965	0,24	0,23	0,22	0,32	0,27	0,68	3,45	3,32	1,04	0,57	0,52	0,32	0,94
Sim	1965	0,10	0,10	0,09	0,14	0,12	0,33	2,13	1,99	0,53	0,27	0,24	0,14	0,52
Total	1965	1,66	1,62	1,53	2,22	1,86	4,75	25,21	23,67	7,29	4,00	3,62	2,21	6,64
Kulekhani	1966	1,55	1,41	1,14	0,75	1,12	1,14	10,37	20,78	10,29	3,33	2,20	1,70	4,65
Chakhel	1966	0,28	0,26	0,21	0,14	0,20	0,21	1,88	3,76	1,86	0,60	0,40	0,31	0,84
Sim	1966	0,12	0,11	0,09	0,05	0,09	0,09	1,04	2,28	1,03	0,29	0,18	0,14	0,46
Total	1966	1,95	1,78	1,44	0,94	1,41	1,44	13,29	26,82	13,18	4,22	2,78	2,15	5,95
Kulekhani	1967	1,08	0,94	0,84	1,01	0,65	4,52	11,80	6,92	5,66	3,36	2,32	1,68	3,40
Chakhel	1967	0,20	0,17	0,15	0,18	0,12	0,82	2,14	1,25	1,02	0,61	0,42	0,30	0,62
Sim	1967	0,08	0,07	0,06	0,08	0,05	0,41	1,21	0,66	0,53	0,29	0,19	0,13	0,31
Total	1967	1,36	1,18	1,05	1,27	0,82	5,75	15,15	8,83	7,21	4,26	2,93	2,11	4,33
Kulekhani	1968	1,66	1,46	1,63	1,21	1,11	2,52	5,72	6,33	2,62	7,12	2,25	1,47	2,93
Chakhel	1968	0,30	0,26	0,30	0,22	0,20	0,46	1,04	1,15	0,47	1,29	0,41	0,27	0,53
Sim	1968	0,13	0,11	0,13	0,09	0,08	0,21	0,53	0,60	0,22	0,78	0,19	0,12	0,26
Total	1968	2,09	1,83	2,06	1,52	1,39	3,19	7,29	8,08	3,31	9,19	2,85	1,86	3,72
Kulekhani	1969	1,18	0,89	0,87	0,92	0,95	0,88	3,10	6,89	4,18	1,91	1,10	0,83	1,98
Chakhel	1969	0,21	0,16	0,16	0,17	0,17	0,16	0,56	1,25	0,76	0,35	0,20	0,15	0,36
Sim	1969	0,09	0,07	0,06	0,07	0,07	0,06	0,27	0,66	0,37	0,16	0,08	0,06	0,17
Total	1969	1,48	1,12	1,09	1,16	1,19	1,10	3,93	8,80	5,31	2,42	1,38	1,04	2,51
Kulekhani	1970	0,82	0,72	0,64	0,61	0,64	3,42	21,29	8,98	5,26	3,30	2,18	1,47	4,11
Chakhel	1970	0,15	0,13	0,12	0,11	0,12	0,62	3,85	1,61	0,95	0,60	0,39	0,27	0,74
Sim	1970	0,06	0,05	0,05	0,04	0,05	0,30	2,35	0,89	0,49	0,29	0,18	0,12	0,40
Total	1970	1,03	0,90	0,81	0,76	0,81	4,34	27,49	11,48	6,70	4,19	2,75	1,86	5,25
Kulekhani	1971	1,15	1,11	1,10	1,98	2,13	21,64	5,09	7,68	4,27	3,82	2,22	1,86	4,50
Chakhel	1971	0,21	0,20	0,20	0,36	0,39	3,92	0,92	1,39	0,77	0,69	0,40	0,34	0,81
Sim	1971	0,09	0,08	0,08	0,16	0,18	2,39	0,47	0,74	0,38	0,34	0,18	0,15	0,44
Total	1971	1,45	1,39	1,38	2,50	2,70	27,95	6,48	9,81	5,42	4,85	2,80	2,35	5,75
Kulekhani	1972	1,72	1,78	1,62	1,41	1,38	4,54	28,27	6,15	8,36	2,49	1,85	1,58	5,10
Chakhel	1972	0,31	0,32	0,29	0,26	0,25	0,82	5,12	1,11	1,51	0,45	0,33	0,29	0,92
Sim	1972	0,14	0,14	0,14	0,11	0,11	0,41	3,23	0,58	0,82	0,21	0,15	0,13	0,51
Total	1972	2,17	2,24	2,05	1,78	1,74	5,77	36,62	7,84	10,69	3,15	2,33	2,00	6,53
Kulekhani	Mean	1,31	1,21	1,14	1,17	1,24	4,57	11,90	10,19	6,73	3,65	2,10	1,56	3,90
Chakhel	Mean	0,24	0,22	0,21	0,21	0,23	0,83	2,16	1,84	1,22	0,66	0,38	0,28	0,71
Sim	Mean	0,10	0,09	0,08	0,09	0,10	0,45	1,26	1,04	0,65	0,33	0,17	0 13	0,37
Total	Mean	1,65	1,52	1,43	1,47	1 57	5,85	15,32	13,07	8,60	4,64	2,65	1,97	4,98

3.1.2 Kulekhani Dam

The Kulekhani dam is a zoned rockfill dam, consisting of an inclined core zone, filter core zones, quarry rock zones and a random rock zone. The main dam is 114 m in height and has a volume of 4,4 million m^3. Its crest is 397 m long and 10 m wide, and being located at EL 1.534 m, this leaves a freeboard of 4 m above the high water level.

The upstream slope of the dam is 1:2 between the crest elevation and EL 1.519 m and 1:2,35 below. The downstream slope is 1:1,8.

The foundation area of the impervious core embankment is excavated down to bedrock and a grout curtain is provided to eliminate seepage.

The cross-section of the dam site is shown in Figure 3.1, while a typical cross-section of the dam may be found in Figure 3.2.

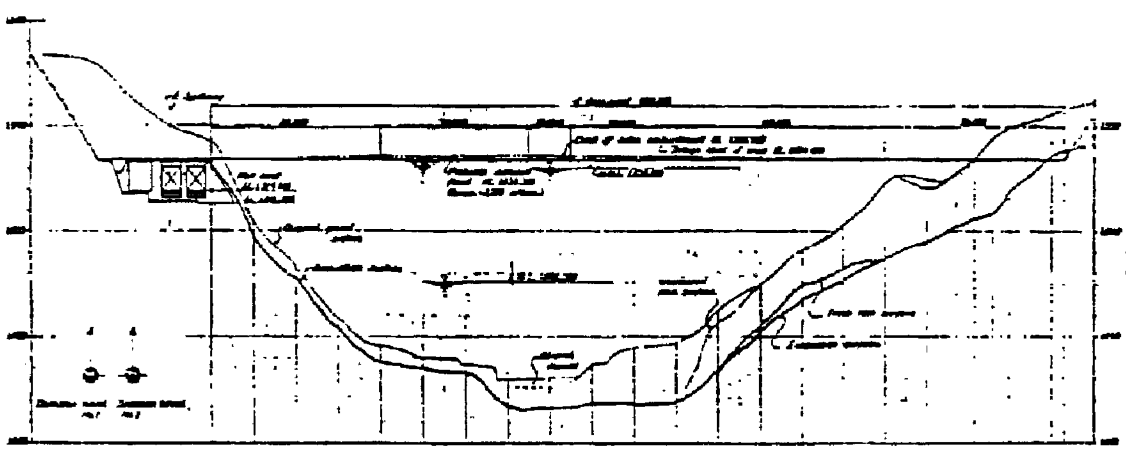


Figure 3.1: Cross-section of the dam site

Damsite geology

The bedrock of the damsite is composed of metamorphic rocks such as mica schists, sandy schists and quartzites, which are affected by the intrusion of a granite mass (batholith). The eastern end of the granite mass is exposed about 500 m southwest of the damsite. Bedding planes generally dip toward NE to ENE. Minor scale mineralizations have been observed in some places.

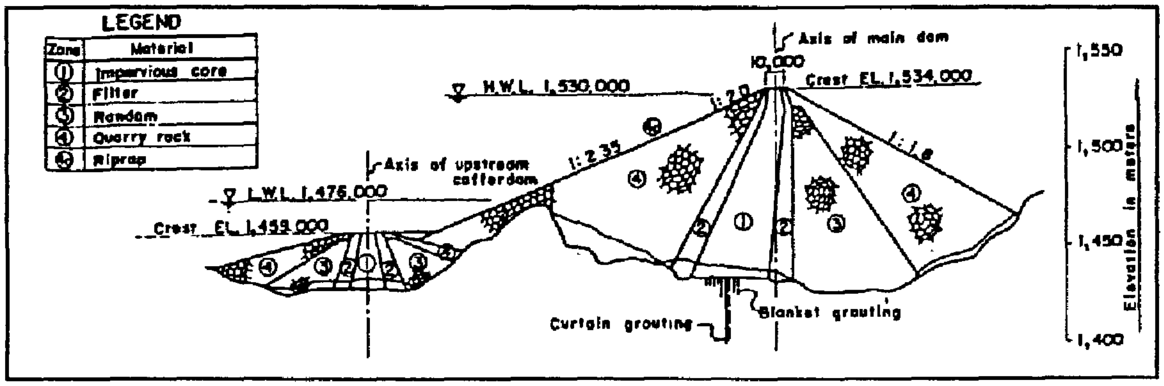


Figure 3.2: Typical cross-section of the dam

Left bank. The rocks on the left bank were generally stable showing the bedding N12°W to N21°W of strike and 30 - 40°NE of dip with slight flexures. The velocity of waves measured in the seismic refraction prospecting ranged from 4,1 to 4,3 km/s in the deepest speed layer, which geologically correlates with the basal sound rocks. As a whole, it appears that the geological conditions at the left bank were fairly good for dam foundation and diversion tunnel environment in their bearing capacity and water tightness.

A narrow terrace was formed on the left bank about 10 m above the riverbed and talus deposits covered the foot of the slope on this terrace. The thickness of terrace deposit has been measured to 4,6 m, and the thickness of the talus deposit to 11,5 m.

Riverbed. The strata exposed along the riverbed dip toward NE to ENE at around 40° on the left bank, 30 - 50° on the upstream right bank and 50° to the vertical on the downstream right bank. The bedding system in the riverbed shows monoclinal folding. A fault (see Figure 3.3) which nearly followed the river channel was inferred from the distribution of low velocity zones on the seismic refraction profiles.

Right bank. The right bank is intensively affected by the intrusion of the granite mass. Thin intrusions of mineral veins, aplites and micro-granites have been observed in places. The rocks are cracked and fractured in some places by folding and faulting. The velocity of the waves measured in the seismic refraction prospecting was 3.0 to 3,2 km/s in the deepest speed layer on the right bank. This speed is rather low, reflecting worse rock conditions compared to the velocity of 4,1 to 4,3 km/s on the left bank.

Fracturing on the right abutment of the damsite was once considered to be attributed to old landslides of 40 to 50 m deep. These depths of 40 to 50 m coincided with the bottom of the 1,1 to 1,3 km/s speed layer in the seismic refraction profiles. It was concluded however, that this rather deep fracturing had resulted from folding and faulting according to the careful study about the possibility of landslide, based on the geological observation of the test adits and geomorphological analysis. It was revealed that landslides in the damsite area were superficial and limited in scale.

Figure 3.3 shows the trend of bedding planes and faults, while the distribution of rock faces at the damsite is given in Figure 3.4.

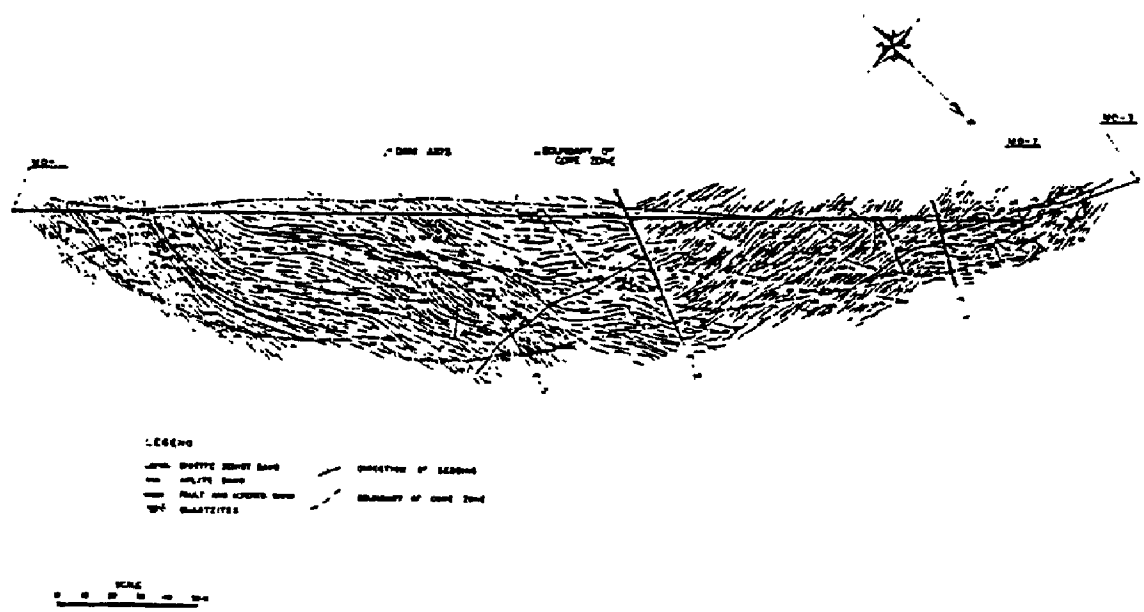


Figure 3.3: Trend of bedding planes and faults of Kulekhani Dam Foundation

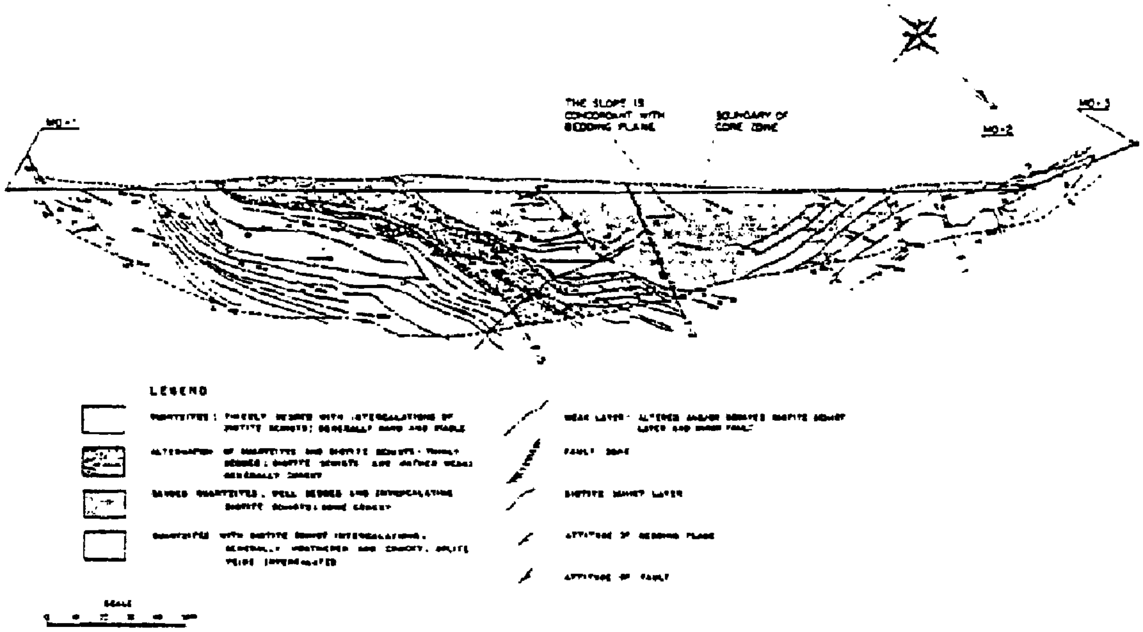


Figure 3.4: Distribution of Rock Faces of Kulekhani Dam Foundation

Measuring apparatus for dam embankment. For the checking of the safety of the dam during the construction period as well as after completion apparatus for the following measurements have been installed in and on the dam embankment:

a) Measurement of pore pressure in the impervious core zone

b) Measurement of vertical displacement of the impervious core zone

c) Measurement of vertical displacement of the rockfill zone

d) Measurement of seepage rate.

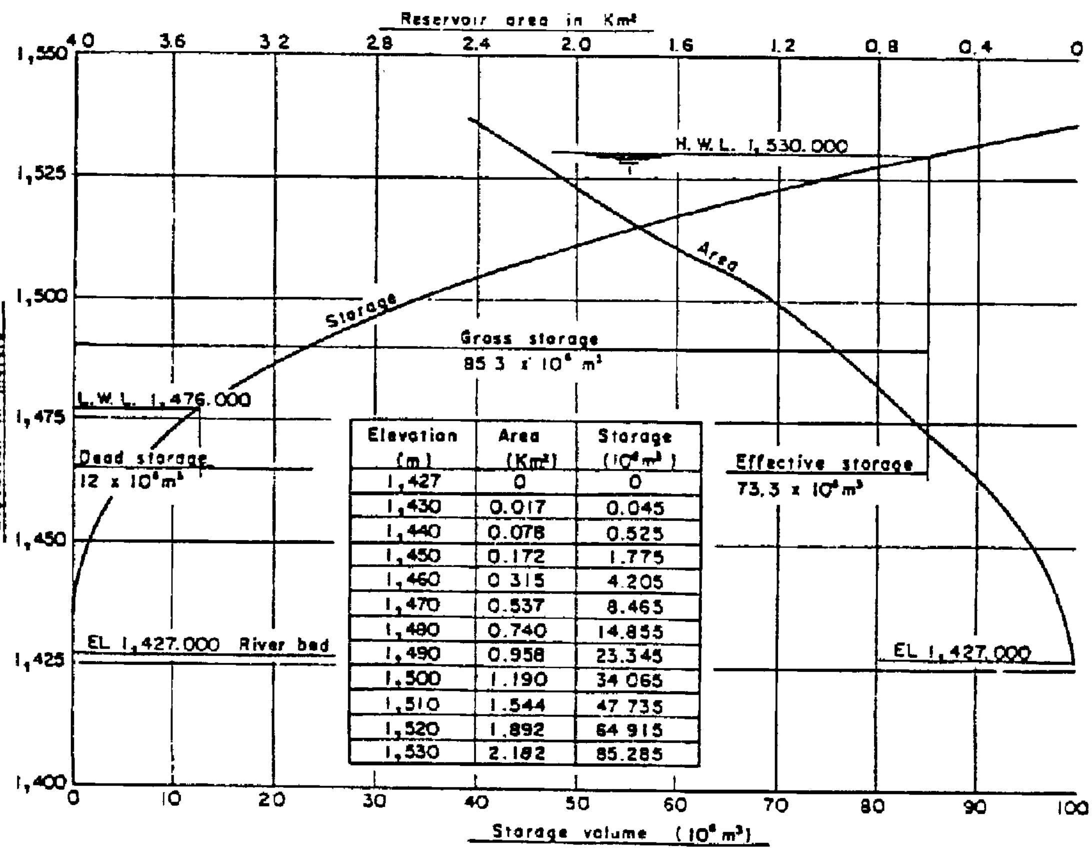
For (a) above, pore pressure meters of the strain gauge type and hydraulic type were installed in the impervious core zone and in the downstream filter zone. (b) above is checked by measuring vertical displacement of the crest settlement point installed at the dam crest. For (c) above, the surface settlement points are set on the upstream and downstream slope surfaces. The seepage, (d) above, is checked at the leakage measuring chamber located on the downstream side of the dam.

3.1.3 Kulekhani Reservoir

The Kulekhani dam creates an artificial reservoir having a surface area of 2.2 km^2 when full.

The overall storage capacity is 85,3 * 10^6 m^3, while the effective storage is 73,3 * 10^6 m^3, leaving a dead storage volume of 12,0 * 10^6 m^3. The high and low water levels are at EL 1.530 m and EL 1.476 m respectively, making a drawdown zone of 54 m. The total length of the reservoir is 7 km.

The capacity and surface curves for the reservoir may be found in Figure 3.5.



Elevation (m)	Area (Km²)	Storage ($10^6 m^3$)
1,427	0	0
1,430	0.017	0.045
1,440	0.078	0.525
1,450	0.172	1.775
1,460	0 315	4.205
1,470	0.537	8.465
1,480	0.740	14.855
1,490	0.958	23.345
1,500	1.190	34 065
1,510	1.544	47 735
1,520	1.892	64 915
1,530	2.182	85.285

Figure 3.5: Capacity and surface area curves for Kulekhani Reservoir

Reservoir sedimentation. Based on measurements carried out at the damsite during a period lasting from January - August 1975, the annual average erosion in the catchment has been estimated at 700 m^3/km^2. This corresponds to a sediment inflow of 88,200 m^3/year. With a dead storage of 12,000,000 m^3, the reservoir's live storage will not be effected during the projects' expected life of 100 years.

3.1.4 Kulekhani dam spillway

The spillway is located adjacent to the dam embankment in the left abutment, and the axis of the weir makes an angle of 22° 30' with the axis of the dam. The spillway consists of the forebay, the ungated side channel weir, the gate controlled overflow weir, the open chuteway and plunge pool.

The forebay is about a 90 m long open channel and the bottom is set at EL 1.516 m. The uncontrolled side channel weir is located on the left bank of the forebay, and its crest is set at EL 1.530 m with a length of 65 m.

The gate-controlled overflow weir is located adjacent to the main dam with its crest set at EL 1.519 m. The flow area is divided into two sections by a pier, and each section is equipped with a radial gate (9 m wide and 11,6 m high).

The chuteway is a concrete rectangular open channel with a slope of 1:2 which gradually reduces its width in the downstream direction. A flip bucket at the end of the chuteway has a flip angle of about 25°.

The plunge pool, located just downstream of the flip bucket has a length of 150 m and is 30 m wide. Its bottom is set at EL 1.418 m.

The spillway is designed for two different discharges; the design discharge of 1.150 m^3/s with a 20% allowance for the inflow discharge with a 100-year return period. The second discharge is that of the probable maximum flood (PMF); being calculated at 2.540 m^3/s.

Hydraulic calculations and models tests were carried out to determine the maximum water levels in the reservoir for the above flood discharges. The results are given in Table 3.3.

Table 3.3: Maximum reservoir water level for flood discharges

Inflow discharge (cumecs)	Outflow discharge (cumecs)	Max. WL. (EL m)
1.380	1.270	1.530,3
2.720	2.540	1.533,3

As a result of the above analyses and model tests, it was found that overtopping would not occur at any portion of the chuteway wall for the design flood discharge.

However , in the case of the PMF discharge a slight overtopping of the downstream section from 65 m point from the axis of the dam will occur on the left bank side. It is considered that this problem is not serious from the point of view of dam safety. This is because the berm which is located at the top of the chuteway wall is protected by stone pitching and the face excavated above the chuteway wall is covered with a 10 cm thick layer of shotcrete.

Key data relating to hydrology, the dam, reservoir and spillway as well as the tributary intakes of the Kulekhani Hydropower Project are given in Table 3.4.

Table 3.4: Principal Data for Kulekhani Dam and Reservoir

1. Hydrological Features

Basin	Catchment Area (km²)	Annual Total Runoff (m³)	Mean discharge (m³/s)
Kulekhani	126	122,9 * 10⁶	3,90
Chakhel	23	22,4 * 10⁶	0,71
Sim	7	11,8 * 10⁶	0,37
Total	156	157,1 * 10⁶	4,98

2. Reservoir and dam

i) **Reservoir**	
High water level	EL 1.530 m
Low water level	EL 1.476 m
Drawdown	54 m
Surface area	2,2 km²
Gross storage capacity	85,3 * 10⁶ m³
Effective storage capacity	73,3 * 10⁶ m³
ii) **Dam Type:** Zoned rockfill dam	
Crest elevation	EL 1.534 m
Dam height	114 m
Crest length	397 m
Embankment volume:	
Impervious zone	606.000 m³
Filter zone	390.000 m³
Random rock zone	632.000 m³
Quarry rock zone	2.740.000 m³
Riprap	48.000 m³
Total volume	4,416.000 m³
Upstream slope above EL 1.519 m	1:2,0
Upstream slope below EL 1.519 m	1:2,35
Downstream slope	1:1,8
iii) **Spillway Type:** Open Chuteway with gate-controlled weir and non-gate-controlled side channel weir and flip bucket and plunge pool.	
Design peak inflow	2 490 m³/s
Max design discharge at FWL 1.532,26 m	1.945 m³/s
Overflow crest elevation	
Gate controlled weir	EL 1.519 m
side spillway	EL 1.530 m
Effective overflow width	
gate-controlled weir	18 m
side spillway	65 m
Gates	Two radial gates, each 9m wide and 11,5 m high

3. Tributary intakes

i) Chakhel intake	250 m³/s
Discharge capacity	EL 1.532,5 m
Overflow crest elevation	20,0 m
Overflow width	
ii) Sim intake	140 m³/s
Discharge capacity	EL 1.548,5 m
Overflow crest elevation	9,5 m
Overflow width	

Chapter 4
FIELD EXCURSION

4.1 Field Trip to Kulekhani Area

A field trip to the project area was undertaken on Thursday April 22nd, 1993. Participants were, in addition to the Consultant:
Mr Yoshiri Kishi, Assistant Resident Representative, UNDP
Mr Y. B. Thapa, Senior Programme Officer, UNDP
Mr Rajendra Prasad Hada, Civil Engineer, KDPP.

The field trip started at 7:30 am and arrived Kulekhani at 11:00 a.m. Here the field party was joined by NEA's:
Mr Dambar Shresta
Section Incharge, Civil Section, Dam Site, Kulekhani.

After a brief introduction, the mission inspected the reservoir and dam area. The reservoir water level was down to EL 1.479,3 m; only 3,3 m above the Low Water Level. This is the lowest drawdown ever after the closure of the dam in 1982[1].

Due to the low water level in the reservoir, the field party had an excellent opportunity to view the parts of the reservoir and the dam which are usually submerged, such as the outlet of Chakhel diversion, the flood spillway, the gated spillways, the upstream side of the dam, etc.

The field party then started to walk along the Kulekhani river, and about half of the distance to the Bagmati confluence was covered. (General views of the valley downstream of the dam

[1] The low water level in the reservoir is not only due to little rain. The Kulekhani power plant was designed as a peak load plant, but has however, been used as a supplier of base load also. This accounts for the low water level in the reservoir and also for the load sharing of electricity that Nepal presently experiences (April 1993).

and the Chakhel diversion outlet in colour are available in limited editions with the Nepal technical authorities, UNDP Kathmandu, and DHA Library as Figures 4.1, 4.2 and 4.3.)

The Kulekhani river flows in a steeply sided and rather narrow valley. The slopes of the sides are between 45 - 60° to the vertical, and the bottom width ranges primarily between 100 - 500 m. As the base of the dam is at EL 1.420 m, the Kulekhani-Bagmati confluence is at EL 1.065 while the distance is roughly 16 km, the average bed slope (of the valley, not the river) is[2]

$$I_b = (1.420 - 1.0660)/15.000 = 0,0235$$

$$\text{or } 1:42$$

There is however, a number of contractions in the river course which serves as hydraulic control sections. Upstream of these contractions, the valley floor was usually rather wide and covered with sediments of varying sizes.

The Kulekhani-Chakhel confluence comprised a rather large and flat area (estimated at almost 1 km^2). Here also, there was a constriction/hydraulic control section just downstream.

The village of Todke is situated on a rather high-lying ridge area from where the field party had an excellent view of the valley further downstream. In fact, the mountain sides of the confluence area could be seen from this vantage point. The field party was told, as was also the party's impression, that the remaining part of the valley down to the confluence was similar to the section covered (i.e. between the dam and Todke).

There are a number of villages in the valley, having an (estimated) accumulated population of 950. The villages and their population is given in Table 4.1.

[2] This is the approximate slope that the dambreak wave will encounter.

Table 4.1: The villages between Kulekhani dam and the confluence with Bagmati river. Number of inhabitants is also given. (Source: Mr Dambar Shresta, NEA).

Village	Inhabitants
Devaltar	50
Khanikhet	200
Todke	150
Tekar	200
Sulikot Seshari	200
Epa	150
Total	**950**

Most of the houses in the villages were situated on relatively high grounds, but quite a few were located close to the bottom of the valley, hence making them vulnerable to (dambreak) floods.

The field party went as far as Todke. Walking back to the dam, the Chakhel river course was followed. The first 1,0 - 1,5 km of the river course upstream of the confluence were in the flat area, but thereafter the river changed to having a very steep bed slope.

The field party arrived at the Kulekhani dam at about 15:30. After a round-up meeting lasting to about 17:00, they departed for Kathmandu, arriving at the capital at about 20:30.