HIRAKUD :
ITS BACKGROUND AND PERFORMANCE

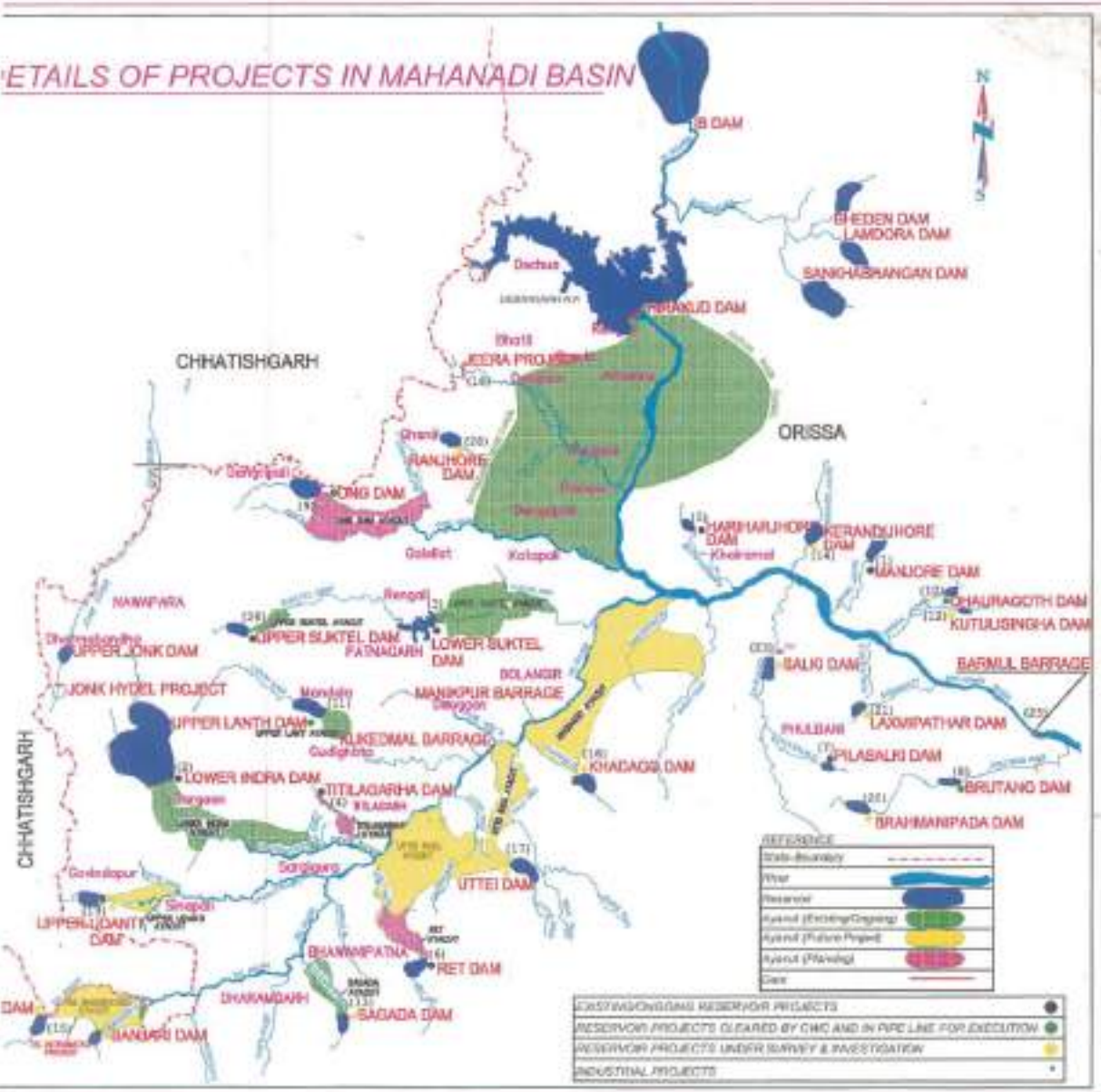


Release of water through Sluice gate of Right Spillway

by

Er. G. C. Sahu
Er. G. N. Das
(Through Walmi, Odisha)

INDIAN NATIONAL COMMITTEE
ON
IRRIGATION & DRAINAGE (INCID)



PROJECTS U/S OF HIRAKUD

SL. NO.	NAME OF THE PROJECTS	CATCHMENT AREA (SQ.KM)
1	Bhedan *	784
2	Upper Jonk *	342
3	Ib Dam *	5849
4	Sankhabanghan Dam*	488
5	Lamdora Dam *	413
6	Jonk Hydel Project *	195

PROJECTS D/S OF HIRAKUD

SL. NO.	NAME OF THE PROJECTS	CATCHMENT AREA (SQ.KM)
1	Manjore	452
2	Lower Indra	756
3	Lower Suktel	1148
4	Titlagarh	120
5	Harharjore	425
6	Rat Project	262
7	Pila Salki Dam	87
8	Bristang Dam	725
9	Ong Dam	2321
10	Dhaurugoth	66
11	Upper Lanth	169
12	Kutulsirgha	83.3
13	Sagada	443
14	Kerandijore	67
15	Tel Integrated Project	550
16	Khedago *	1960
17	Uttei Roul *	1540
18	Jeera	167
19	Upper Udant *	1927
20	Ranjore	123
21	Laxmipathar nala	86
22	Brahmanipada	109
23	Salki Hydro Electric Project *	1249
24	Upper Suktal	45
25	Barmul Barrage *	42000

HIRAKUD :
ITS BACKGROUND AND PERFORMANCE

Er. G. C. Sahu
Er. G. N. Das
(Through Walmi, Odisha)

INDIAN NATIONAL COMMITTEE
ON
IRRIGATION & DRAINAGE (INCID)
(CONSTITUTED BY MoWR, GOVERMENT OF INDIA)
NEW DELHI
2012

FOREWARD



Irrigation has been linked to the development of civilizations since the history of mankind. Irrigation development in India has a very old history. The efforts of past builders and engineers to develop innovative, safe and sound engineering designs are a fine expression of their irrepressible zeal to favourably maneuver the vagaries of nature for overall development of humanity. A study of these efforts presents a fascinating insight into the innovative techniques of harnessing the precious water resource adopted by our forefathers in their search for survival in a period, when technolbgy was still nascent and a scientific approach unheard of. Decisions were based on requirements and site selection by thumb rule and still the structures stood the test of time providing the much needed water to the inhabitants during scarcity days.

In pursuance of its objectives and functions, Indian National Committee on Irrigation and Drainage (INCID) has taken up the work of publishing the historical appreciation of development of irrigation and drainage in the country, which will disseminate relevant information related to the subject. INCID has already published four such documents "History of Irrigation Development in Tamil Nadu", "History of Irrigation Development in Andhra Pradesh", "History of Irrigation Development in Orissa" and "History of Irrigation Development in Uttar Pradesh". The present document, "Hirakud: Its Background and Performance" brings out a comprehensive History of Hirakud Project.

The task of preparing the report was entrusted to Er. G. C. Sahu, Engineer-in-Chief (Retd.), Water Resources Department, Government of Odisha and Er. G. N. Das. The report prepared by the authors was circulated to D&R Wing, Central Water Commission for their comments. INCID has finalized the report taking into account the comments received from D&R Wing , Central Water Commission.

This publication contains 8 chapters covering the background, investigation and planning aspects, project construction, instrumentation aspects, post construction issues and overall performance of Hirakud Dam Project. The publication covers the aspects like design, model study, resettlement & rehabilitation issues, construction methodology of dam, development of cracks in the spillway: its causes and remedial measures. The publication also covers chapters on instrumentation in dam & dykes, silting of Hirakud Reservior and Flood control.

INCID wishes to acknowledge the contribution of Er. G. C. Sahu and Er. G. N. Das alongwith their associates for preparing this report. The efforts put in by Shri Yogesh Paithankar, Member-Secretary, INCID; Shri Piyush Ranjan, Deputy Director, INCID and other officers concerned of Central Water Commission are very much appreciated. Thanks are also due to the Ministry of Water Resources, for providing requisite support.

It is hoped that this publication will be useful as a reference book to all concerned in the Water Resources Sector, particularly to those working in various State Irrigation Departments/ PSUs and other Agencies dealing with dams and multipurpose projects.

(R. C. Jha)
Chairman
INCID & Central Water Commission



PREFACE

Soon after independence, the Planning Commission was established in 1950 with experts of outstanding calibre in various fields of activities for formulating Five Year Plans for overall development. The broad objective of the Commission was to assess the potential of material and human resources with an eye to their optimum utilisation for development of infrastructures for nation building within a stipulated time frame. The Commission recognised the importance of exploiting the irrigation potential of the country for increasing the food grain production. The country has to support a large population. The population was 361 million in 1951 when food grain production was 51 million tonne. In 2001 population has swelled to 1027 million with food grain production of 212 million tonne. There is tremendous pressure of population on land.

After experiencing successive flood and droughts, the British Government had setup First Irrigation Commission in 1901 and report published in 1903. The report had highlighted the need to extend irrigation as a protection against future famines. The wide spread suffering caused by successive famines in 19th century led to setting up a series of Famine Commissions. The 2nd Famine Commission recommended that 'among the measures that may be adopted for giving direct protection from drought, the first place must, unquestionably be assigned to the work of irrigation'. First Irrigation Commission also fully endorsed this view.

'Irrigation is now no more looked upon merely as an artificial application of water to crops in an isolated and individual way, but it has come to have a different meaning, especially in the arid and semi arid regions of the world, most of which are undeveloped and underdeveloped. In such region irrigation provides the essential basis for an integrated all-around development and is, therefore, more than a practice and technique, it is a way of life, which embraces all the various aspects of development of a community-economic, social, cultural etc.'

Besides drought, 'flood is the scourge of the Orissa Province'. In the words of Mahatma Gandhi 'Flood is a permanent disease in Orissa'. Government constituted Orissa Flood Committees (1928 & 1938), Enquiry Committee (1940) and requested M. Visvesvaraya (1937) for his expert advice. He opined that 'on account of their cost and the difficulties faced on inter-provincial cooperation there is small prospect of the idea (i.e. storage reservoir) materialising in the near future, but it is desirable to make some preliminary investigation to see how the question stands at present, and whether there is any prospect of its being taken up later when the province is able to finance such undertakings. If a reservoir is constructed, it may prove useful generating electric power etc. Once floods come under effective control, the whole area may be transformed into a prosperous region.'

Finally the matter was referred to CWI&NC when Dr. A.N. Khosla was the Chairman. He came to the conclusion that the only cure for many troubles of Orissa, viz. floods and droughts etc.

lay in the control, construction and utilisation of the enormous water wealth of its rivers by means of a storage dam. At the Cuttack Conference (Nov. 1945) under the chairmanship of Dr. B.R. Ambedkar, it was unanimously agreed that the potentialities of Mahanadi for unified multipurpose development viz. flood control, irrigation, navigation and hydroelectric power should be thoroughly and expeditiously investigated by the Commission. This paved the way for construction of the multipurpose project at Hirakud by Government of India during 1948 and was completed in 1957.

It was a privilege that INCID entrusted us to undertake the study on Hirakud Dam a major multipurpose river valley project, which has successfully completed more than five decades of useful service in irrigation, flood control, power and related sectors. Writing a book of this complex nature and broad scope commencing from conception to completion and to record the developments that have taken place in the headworks and command area during last fifty years is an extremely difficult task due to poor documentation. Either old records are not properly preserved or whatever available is not easily accessible. Most of the datas have been collected from Hirakud Dam Project Library, Library of CWC and CBIP, several reports prepared by State Government, Govt. of India, NRSA, CWPRS, CGWB, SPCB and other institutions, and documents available with friends and colleagues both in service and retired. Further we express our gratitude to the authors of the books and journals from where we have collected valuable datas and informations and incorporated in this compilation. The book has been broadly divided into eight chapters. Those are :

I. Background

II. Project Investigation, Planning & Foundation Features.

III. Project Construction.

IV. Development of cracks in Spillways & Power Dam: Causes & Remedial measures.

V. Instrumentation in Dam & Dykes.

VI. Post Construction Issues.

VII. Performance of Hirakud Dam Project.

VIII. Conclusion.

Chapter-I describes the course of river Mahanadi, its catchment, occurrence of high floods in pre-Hirakud Dam Period, Reports of various Committees, Background leading to construction of the Project and Factor for selecting Hirakud as first Dam site.

Chapter-II deals with Geological Investigation, Hydrology, Design Flood, Foundation Treatment and Special Foundation Problem.

Chapter-III discusses on Commencement of Project Work, Design, Model Study, R&R Issue, River Diversion, Construction Methodology, Quality Control, Labour Welfare, Martyrs during Project Construction, Chief Architects associated with the construction and Land Mark Events.

Chapter-IV examines the probable causes of Distress, History of Development of Crack, Studies undertaken by Expert Organisations, Recommendation for Remedial Measures, ASR, Epoxy Treatment, Instrumentation & Monitoring of Crack Development etc.

Chapter-V details several instruments embedded to study the settlement, strain, temperature, pore pressure, seepage etc. along with development of boils and provision of relief wells.

Chapter-VI is on Post Construction Issues dealing with Flood Forecasting, Reservoir Operation, Rule Curve, PMF, Necessity of Additional Spillway, Dam Break Analysis, Silting of Reservoir, Interception in the Upstream of Hirakud and Study of Waterlogging in Hirakud Command.

Chapter-VII indicates regarding Irrigation practices in pre and post Hirakud period, Annual Irrigation, Impact of Hirakud Reservoir on Flood Moderation, Power Generation, M&I Water Supply, Pisciculture and Recreation.

Chapter-VIII sums up with discussion on Large Dams vs Small Dams, other proposed dams viz. Manibhadra and Subalaya, Inter Basin Transfer in the State, Construction of Projects in the Downstream of Hirakud, Environmental Issues, Water Quality, Research Activities, Issues and Concerns with concluding remarks.

The authors are highly indebted to Er. J.B. Mohapatra, Superintending Engineer, Hirakud Dam Circle and all other officers of the Project particularly to Er. S.K. Das and Er. P. Samantray, RDC, Sambalpur, Officers of O.H.P.C., Burla, C.R.O., Hirakud Research Station, Deputy Director (Agriculture), Sambalpur, and Assistant Director (Fisheries), Sambalpur for rendering immense help at every stage of work. Without assistance of Er. Mohapatra it would have been really a difficult task. We very much appreciate his positive attitude. We express our hearty thanks to Er. Sagar Mohanty who without any hesitation assisted us in preparing the drawings. We pray Almighty for his well-being and prosperity. Our sincere thanks to Er. A.K. Das, Er. Y. Parida of DoWR and Sri S. Prusty of SPCB for timely supplying the relevant datas. We also thank our colleague Er. P.C. Das, Retd. Chief Engineer, DoWR, who agreed to work as a Research Fellow in this project. We acknowledge the help rendered by Sri S.S. Tripathy of Graphic Art Offset Press, Cuttack for type setting and get up. We are thankful to the Director, WALMI, Odisha and his staff for their co-operation at every stage of work.

The authors express their gratitude to INCID for entrusting this research scheme.

Lastly we must acknowledge and thank the patience and understanding shown by our family members during preparation of this report.

We owe gratitude to the large number of institutions and individuals those who have helped us and also to those we have failed to mention here.

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CHAPTER-1

BACKGROUND

1.0 General:

Floods and droughts are the twin spectres haunting the people of Orissa from time immemorial. About 70% of state's population depend on agriculture for their livelihood. Contribution of agriculture & animal husbandry to the Net Domestic Product in 2007-08 is 20.9% at 1999-2000 prices (vide Economic Survey, Orissa, 2008-09, Pg. 4/1). Year after year, vast agricultural lands get inundated by floods causing extensive damage to the standing crops. Often failure of monsoon, completely or partially wither away the plants thereby smashing all the hopes of farmers. Due to vagaries of such frequent phenomena, the inhabitants of Orissa suffer from untold misery.

Almost every year, vast stretches of land in the delta and fertile areas of undivided Cuttack, Puri, Balasore and Ganjam districts are severely affected either by flood, cyclone or drought. Sometimes two or more such natural phenomena have occurred in one year in different parts of the state. The nature has become the potent enemy in the economic development of Orissa. The flood, drought and cyclone are recurring features, which have broken the backbone of its people. There is hardly a year when the State has not experienced such natural calamity. Attempt had been made by some scholars to study the intensity of natural calamities and its impact on the socio-economic condition of the people. Few of them are described as under:

"History of Orissa" written by William Hunter published in 1872 provides an extensive coverage on natural calamities like flood, famine and consequent socio-economic problems. Similarly 'A Sketch of History of Orissa' by George Toynbee published in 1873 highlights the administrative aspects of East India Company from 1803 to 1828 with a brief description of scarcity of food grains from 1806 to 1809. N.N Banerjee's Report on 'Agriculture of the districts of Cuttack' (1898) throws some insight into the history of natural calamities in undivided Cuttack district. Two other coastal districts, i.e. Puri and Balasore, which are prone to cyclone, have been ignored. But S.L.Maddox's "The Final Report on Survey and Settlement of Province of Orissa (1890-1900)" gives a brief outline of the natural calamities in the State. The autobiography written by Fakir Mohan Senapati (1917) in 'Odia' describes an eyewitness account of the terrible famine of 1866. Causes and consequences of 1866 famine have also been analysed by Dr. H.K. Mahatab in his 'History of Orissa' Vol-II, 1960. 'History of Orissa' Vol-VI (1964) written by P.Mukherjee provides an outline on natural calamities in Orissa and the infamous famine of 1866. 'The famine and some aspects of British Economic Policy in Orissa 1866-1905' written by G.C.Pattnaik (1980) reviews the socio-economic condition of the State before and after 1866 famine. He also mentions regarding the remedial measures undertaken by the British Govt. for rehabilitating the people affected due to 1866 famine. B.B Bhatta in his book 'Natural calamities in Orissa in the 19th century' (1997) focuses on historical perspective, examines the causes, nature and consequences of frequent natural calamities on Orissa and their impact on the State's economic setup. 'Irrigation, Inland Navigation and flood problems in North Orissa during British Rule' written by P.Mukheree (1967) gives a picture of the river system of Orissa, their specific problems and how these problems were dealt in the past.

All the rulers of Orissa in the past had realized the basic problems of the State. For mitigating the suffering of the people in localised patches some earthen dykes were constructed before British occupation of Orissa (i.e. 1803 AD). Few important places were also protected by stone revetments. But no systematic schemes were undertaken to provide protection against flood and insurance against drought to give relief to the millions of suffering masses.

In 1858 AD (i.e. during British Rule), Captain Harris investigated and prepared a scheme to control the floods of Mahanadi by diverting about 4 lakh cusec of excess flood water through Daltola

cut to the Chilka lake and then to the sea. Unfortunately the scheme did not see the light of the day. This could have solved the flood problem to a greater extent. The scheme can not be thought of even now as there is large scale development in the area (Source: - 'Irrigation, Inland Navigation & Flood problems, in North Orissa during British Rule' by P.Mukherjee & 'Flood Enquiry Committee - Interim Report' 1959, Pg.29.

The worst Orissa famine occurred in the year 1866 which is known as '*Na-anka Durvikshya*' as it happened during the 9th year of reign of the then Maharaja Gajapati Divya Singh Dev of Puri. Report of the Famine Commission of 1878 quoted in Buckland's "Bengal under Lieutenant Governors" (Pg.329) explains the gravity of the situation. The rainfall in 1865 was scanty and ceased prematurely. This completely damaged the food crops. It was followed by flood during 1866. Whatever was spared by drought was destroyed by flood. This brought untold miseries to the people of Orissa. Government failed to render any assistance and death toll due to starvation exceeded million. Even the British rulers were overwhelmed at the sight of this tragic calamity and human suffering. They immediately launched flood control and irrigation schemes in large scale to redress the miseries of people. The canals, harbours and regular system of communication were improved with Calcutta to break the isolation of Orissa, which was the main cause of the great famine (vide Statistical Accounts of Bengal, Vol.XVIII, Pg.148 by W.W.Hunter). This created situation to commence Delta Irrigation scheme in undivided Cuttack district comprising present Cuttack, Jajpur, Kendrapara and Jagatsingpur districts and the major portion of the work completed by 1883.

The rivers which cause flood in the State are mainly the Mahanadi and its tributaries, the Brahmani, the Baitarani, the Salandi with Kapali and the Subarnrekha. The deltaic area is about 6.02% of the catchment area of above rivers. 'Total annual precipitation in the deltas of the Mahanadi, the Brahmani along with Baitarani and the Subarnrekha are respectively 5.7, 7.7 and 2.2% of the total volume of water received in the drainage basins. During monsoons, these percentages are 4.9, 8.4 and 1.9 for an average rainy day during the season. Thus 94.3, 92.3 and 97.8% of the annual total and 95.1, 91.6 and 98.1% of the monsoon total is drained through the Mahanadi, Brahmani and Baitarani deltas (vide 'Geography of Orissa' by B.N.Sinha, Published by N.B.T, New Delhi, 3rd edition, 1999 Pg.145 and 147). It is seen from above that an enormous quantum of water is drained over the delta during monsoon which cause flood in lower reaches. But the Mahanadi and its branch Kathjori, Brahmani and Baitarani frequently create flood havoc in the deltaic region of the State. Seeing the devastation and suffering of human beings almost every year, Mahatma Gandhi rightly observed that 'Flood is a permanent disease in Orissa'. Historian William Hunter stated that 'Flood is the scourge of the Orissa Province'. Due to frequent floods, the economic background of the people got crippled or ruined. For rendering relief to the flood affected destitutes of the State, Utkalmani Gopabandhu Das sacrificed his life and life of his beloved son, which made him immortal.

1.1 Mahanadi – Its Course and Catchment:

Mahanadi, the 6th largest river of India, is a major east flowing river of the peninsular system and originates from Pharasiya village of Raipur district. It drains large area of Chhatisgarh and Orissa but relatively small area of Jharkhanda and Maharashtra. It enters Orissa near Padigan in the district of Jharsuguda. Thus the river basin is an Inter-state one. State wise drainage area is shown in the Table: 1.1

Table 1.1 : State Wise Drainage Area of Mahanadi Basin

Sl.No.	State	Area (Sq.km)	% of total basin	Remark
1.	Chhatisgarh	75136	53.24	Out of 141134 sq.km. the CA
2.	Orissa	65628	46.50	up to head of the delta is
3.	Maharashtra	238	0.17	132100 sqkm & rest 9034sqkm
4.	Jharkhanda	132	0.09	lies in deltaic plain.
	Total	141134	100.00	

The river traverses a total distance of 851 km and falls into Bay of Bengal in Orissa forming delta out of which a length of 375 km lies in Chhatisgarh and rest in Orissa. The river basin is bounded by central India hills in North, eastern ghat in the South, Maikala hill range in the west and Bay of Bengal in the east. There are 14 major tributaries of Mahanadi. Out of these, the Ong, Tel and Ib are in Orissa. Rest 11 tributaries are in Chhatisgarh. Besides three major tributaries, i.e. Ib, Ong and Tel, the other important tributaries which join the river in the State are Jira, Bagh, Salki, Kuanria, Hariharjore, Sagada, Ret, Hati, Indra, Suktel, Utei, Raul, Udanti, Lanth, Sapua and Kantia etc. The major branches and sub branches of Mahanadi are the Kathjori, Birupa, Kuakhai, Daya, Bhargavi, Kushabhadra, Surua, Biluakhai, Devi, Kandal, Chitrotpala, Luna, Karandia, Paika and Badagenguti. Excepting Daya and Bhargavi, which fall into Chilika Lake, others join Bay of Bengal with Mahanadi. Its maximum discharge in flood time is calculated to be about 45307 cumec (1.6 million cusec) or nearly as great as that of the Ganges. But in dry season however, the discharge dwindles to about 28.32 cumec (1000 cusec) and has been known to fall to so low a value as 200 cusec, while the least discharge of the Ganges is 45,000 cusec (1274 cumec). During eight months of the year the river is nothing more than a narrow and shallow channel winding through a vast expanse of sand. But after construction of Hirakud, regulated release from Power House is available all throughout the year.

The Mahanadi (the 'great river') rises in an insignificant pool, in the extreme south-east of Raipur district. In the first part of its course it flows to the north, and drains the eastern portion of Raipur, its valley during the first 50 miles (80.46 km) being not more than 500 or 600 yards broad. A little above seorinarayan, on entering Bilaspur district, it receives the water of its first great affluent, the Seonath, which in Raipur district is a more important river than the Mahanadi itself. It flows in an easterly direction through Bilaspur, its principal tributaries being the Jonk and the Hasdeo. It then enters Sambalpur, and turning south at the town of Padampur flows south and south-east through that district. Its affluent here are the Ib, the Ong, and the Tel, and numerous minor streams. In Sambalpur it has already become a river of the first magnitude with a width of more than a mile in flood time when it pours down a sheet of muddy water overflowing its submerged banks, carrying with it the boughs and trunks of trees, and occasionally the corpses of men and animals which it has swept away. The Mahanadi subsequently forms a series of rapids, until it reaches Dholpur. During the rainy season the water covers the rocks and suffices to float down huge rafts of timber. At Dholpur the rapids end, and the river rolls its unrestrained water straight towards the outermost line of the Eastern Ghats. This mountain line is pierced by a gorge 40 miles in length, over looked by hills and shaded by forests on either side. The Mahanadi finally leaves the Tributary States, and pours down upon the Orissa delta from between two hills a mile apart at Naraj, abut 7 miles west of the city of Cuttack. It traverses Cuttack district from west to east, and throwing off numerous branches falls into the Bay of Bengal, by several channel, near False Point, at 20° 18'N, 86° 43'E.

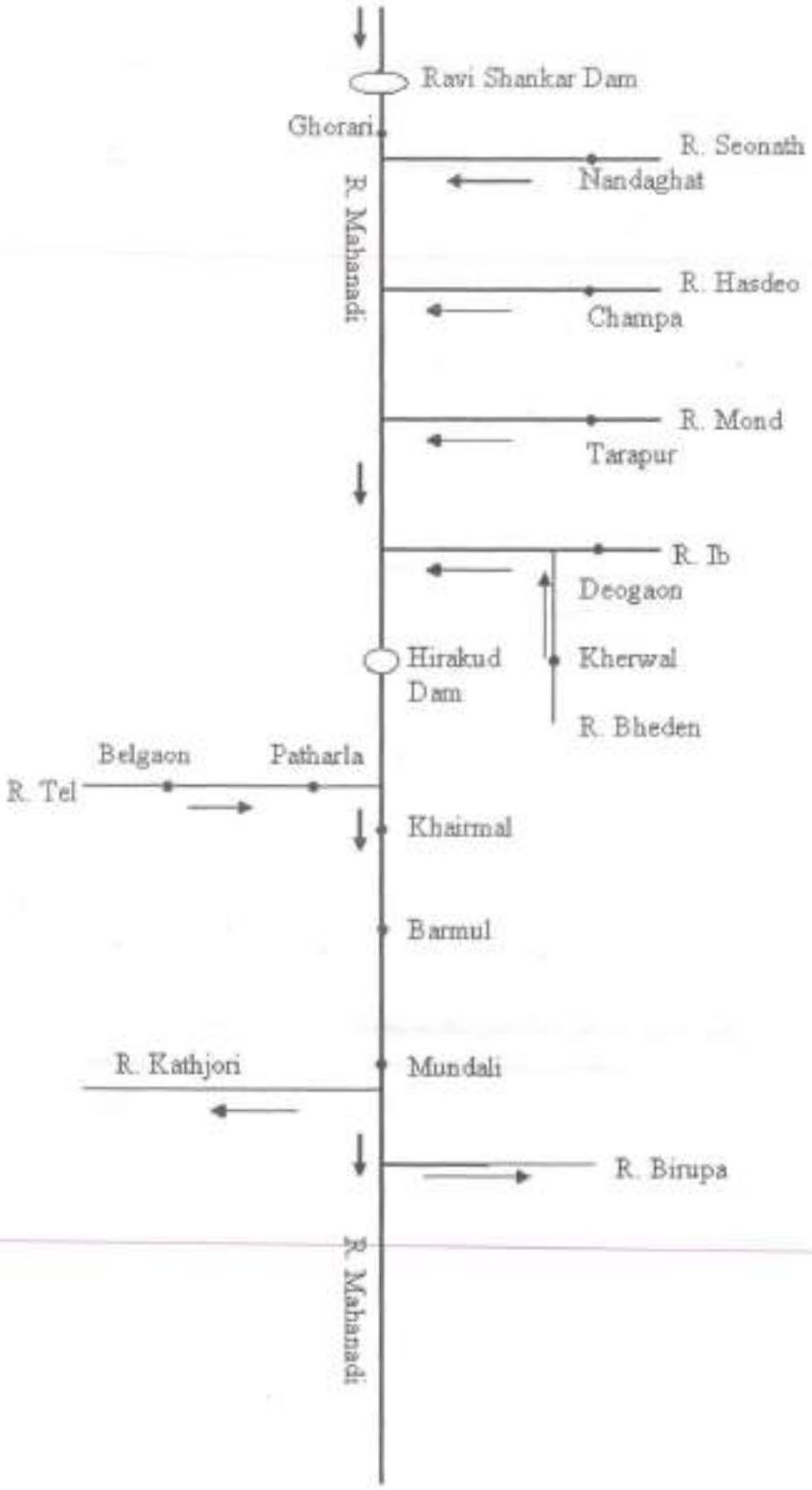
On the right of south bank, soon after entering Cuttack district, it gives off a large steam, the Kathjori, the city of Cuttack being built upon the spit which separates the two channels. The Kathjori immediately divides into two of which the southern branch, under the name of the Kuakhai passes into Puri district, and shortly afterwards throws off the Surua, which reunites with parent stream after a course of few miles. The offshoots from the left or north bank of the Mahanadi are the Birupa and the Chitrotpala. The Birupa takes off opposite the city of Cuttack, and afterwards joins the Brahmini, and its waters ultimately find their way into the Bay of Bengal by the Dhamara estuary. (Source: Mahalanabis Report, I &P Department, Government of Orissa Pg. 3-4).

Basin map of all rivers of the state with basin map of Mahanadi and Line diagram of Mahanadi and its main tributaries are shown in Drg. No.1.1, 1.2 & 1.3. The details of Drainage system in Mahanadi delta and districtwise area distribution of Mahanadi basin in the State are furnished vide Table no 1.2 & No1.3 respectively.

DRG. No 1.1


ORISSA MAP SHOWING ELEVEN RIVER BASINS
N
ORISSA
Jharkhand
West Bengal
Chhattisgarh
Andhra Pradesh
SUBERNA-REKHA
BAITARANI
BURHA BALANGA
SUBERNA-REKHA
Hirakud Dam
BRAHMANI
Rengali Dam
Samal Barrage
Mundali Weir
MAHANADI
Fore Bay Dam
RUSHIKULYA
WANSADHARA
INDRAVATI
NAGAVALI
BAHUDA
KOLAB
To Godavari
Bay of Bengal
REFERENCE
State Boundary
Basin Boundary
River
Dam
Canal


MAHANADI BASIN
Projects :
Completed :
① Hirakud
② Tikarpara
③ Sarafgarh
④ Salpala
⑤ Dumerbahal
⑥ Jharbhandh
⑦ Ong Barrage
⑧ Hati Barrage
⑨ Sundar
⑩ Uttei
⑪ Upper Sukhei
⑫ Upper Jonk
⑬ Hasthara Jore
⑭ Salki Barrage
⑮ Pilo salki
⑯ Kuamla
⑰ Budhabudhiani
⑱ Upper dahuka
⑲ Sapua Badjore
⑳ Delta Stage - I
㉑ Delta Stage - II
Ongoing :
① Lower Indra
② Telagarh
③ Bagh Barrage
④ Manjore
⑤ Lower Sukhei
⑥ Ib
⑦ Dhaurajgarh
⑧ Bolang
⑨ Hadua
⑩ Lower Kanh
JHARKHAND
CHHATISGARH
ORISSA
ORISSA
Bay of Bengal
References :
River & Nallah
Basin Boundary
State Boundary



Drg. No. 1.3 Line Diagram of River Mahanadi & its major tributaries (Not to scale)

Table 1.2 Details of Drainage System in Mahanadi Delta

Sl.no	Name of the drainage system	Total drainage area in ha
	(a) Daob-I Mahanadi-Kathjori, Devi	
1	Hansua-Barnala-Brudha-Tigiria	65918
2	Alaka	857
3	Madhusudanpur part Gobari	9290
4	Singpur-Nagpur-Alaka-Boruan	21699
5	Others	23178
	(b)Daob-II (Mahanadi-Chitrotpala-Luna-Birupa-Brahmani)	
1	Gobari Drainage system	39800
2	Other drainage system	------
	(c) Doab-III (Luna-Chitrotpala)	
1	Baghuni	15000
	(d) Daob-IV (Area to East of HLC Range-I)	
1	Chota Genguti	45000
2	Sagada	9730
3	Matagunja	11580
4	Kumaria	17900
	(e) Daob-V (Kathjori-Kushabhadra)	
1	Prachi	42553
2	Kadua	27116
3	Devi-Kandal Isalnd	7700
4	Devi-Taun Island	4727
5	Others	15837
	(f) Daob-VI (Kushabhadra-Bhargavi)	
1	Dhanua system	47173
2	Nuagaon	6916
3	Others	8664
	(g) Doab-VII (Daya-Bhargavi)	
1	Bhargavi-Ratnachira	14540
2	Luna- Ratnachira	44500
3	Others	23935
	(h) Doab-VIII	
1	Gangua system	65400

Source: - 3rd Spiral study of Mahanadi Basin Plan, Pg.32, Dept of Water Resources, Govt. of Orissa.

Table 1.3 District Wise Area Distribution of Mahanadi Basin in Odisha

Sl.No	Name of the district	Total geogr. Area (sq.km)	Basin area in district (sqkm)	% of Basin area in the district
1	Angul	6232	1305.96	20.96
2	Baragarh	5834	5834.0	100.0
3	Bolangir	6569	6569.0	100.0
4	Boudh	3444	3444.0	100.0
5	Cuttack	3733	3713.58	99.48
6	Deogarh	2784	131.16	4.71
7	Dhenkanal	4595	728.83	9.33
8	Ganjam	8706	29.97	0.34
9	Jagatsingpur	1973	1973.0	100.0
10	Jajpur	2888	128.38	4.45
11	Jharsuguda	2200	2200.0	100.0
12	Kalahandi	8364	8088.43	96.7
13	Kandhamala	7650	3436.39	44.92
14	Kendrapara	2548	1061.13	41.65
15	Khurda	2889	2109.42	73.02
16	Nawapara	3408	3408.0	100.0
17	Nawarangpur	4242	4097.3	96.16
18	Nayagarh	5294	889.78	16.81
19	Puri	3051	2780.56	91.14
20	Rayagada	7580	85.67	1.13
21	Sambalpur	6698	6111.78	91.25
22	Sonepur	2344	2344	100.0
23	Sundergarh	9712	5475.66	56.38
	Total	112738	65628	

Source:- 3rd Spiral study of Mahanadi Basin-W.R Dept. Govt. of Orissa- Annexure-2.1

1.2 Formation of Delta and Drainage Channels

Mahanadi Delta is defined as the areas in the undivided district of Cuttack and Puri bounded by the Bay of Bengal, the Chilka Lake, the Ganjam metalled road between Tangi in Puri and Barchana in Cuttack, the river Genguti and the Patamundai Canal. The total area of the delta as defined above was found by direct planimetric measurement to be about 3100 square miles of which about 1400 square miles lie in Puri district and 1700 square miles in Cuttack. In the Report of the Contour Survey of the Flooded Tract of Orissa, 1924 published by the Bihar and Orissa Secretariat the area of Mahanadi delta is given as 2525 square miles. Unfortunately in the above report the boundaries of the delta were not mentioned.

Worst flood affected areas are the deltaic region which, is traversed by a network of tributaries originating from three rivers i.e. Mahanadi, the Brahmani, and the Baitarani. Catchment area of the Mahanadi is almost twenty times larger than its delta. Similarly catchment of both the Brahmani and Baitarani together is about 25 times more than the combined delta of two rivers.

All the three rivers resemble one another in many ways. They all run nearly parallel and all originate in the hilly region. Each river enters the delta as one simple stream but immediately numbers of branches take off due to deltaic action. As these rivers originate from hills, they have high velocities and laden with large volume of silt. On reaching the plains, the velocity gets suddenly reduced. Small reduction in the velocity is accompanied by large reduction in the carrying capacity of the channel. As a result, excess material is deposited in bed and banks. Thus silt gets deposited in river beds. In this way, the river bed rises. Gradually a shallow ridge is built up, over which the rivers flows. At the sametime gradient along the river channel decreases which reduces velocity further. Consequently more silt is deposited until the bed of the river is raised very high. Due to such action, the river outflanks its banks, creating branches which flow in the valleys lying between two successive river ridges. Again new ridges

are formed by the deposit of the silt along the newly created channels. This process is repeated with the formation of new ones. During heavy flood, water spills over banks and the silt spreads over surrounding land mass. This slowly raises the level of deltaic land. But this is relatively slower than the rise in river bed. Thus formation of new branches is the characteristic feature of all the deltas. It is seen that the Mahanadi, the Brahmani and the Baitarini have been divided into about 15 main branches and innumerable sub-branches before meeting the sea. These branches are interlaced in a very intricate manner so that flood water from different rivers becomes inextricably mixed on their way to the sea.

'Deltaic action is, however still at work in the plains which causes considerably fluctuations in the relative discharge through different channels from time to time. For example, the main Mahanadi channel appears to have been considerably silted but the Kathjori is taking the increased discharge thrown on it. Again the Devi and Daya are improving while Kushabhadra and Bhargavi are becoming silted at present' (Refer Orissa Flood Committee, 1928 Pg.38 and Mahalanabis Report, Irrigation and Power Dept. Pg. 61). At present, this discharge distribution has also undergone change.

Further, along the Orissa sea coast, there exists a steady northward littoral sand drift. 'This drift tends to form bars across the river mouth from south to north. A bar of this nature is often raised in hot weather by the prevalent wind so as to form an unbroken sand dune across the mouths some 25 to 30 ft. high above sea level. x x x x The Sonamuhi for instance, rising in the 'Samanga Pat' behind Puri runs parallel with the coast behind the dunes for about thirteen miles and in some years has a common mouth with the Chilika Lake though independent mouths have been opened on occasion as high up as Harachandi' (Refer O.F.C 1928 Pg.8-9 and Mahalanabis Report, Pg.61). During high floods direct mouths are sometimes opened across the sand dunes, but such opening area are again closed soon by drift. The O.F.C of 1928 opined that the shortness of Orissa delta was probably due to this drift.

'The heads of the deltas of such rivers as the Ganga and Indus are situated about 400 miles from the sea where as the deltas of the Mahanadi, Brahmani and Baitarini are little more than fifty mile long. It is probable that these rivers are continually striving, by deposit of silt, to form new land on the sea edge but that this silt is continually being forced towards north by littoral drift. We consider it likely that this drift is due to on the one hand, the long shallow shelf stretching into the Bay of Bengal in front of the Orissa Rivers, and on the other, the progressive filling in Balasore. This rising of the sea bed in the north-west angle of the Bay has affected the estuaries of the rivers. The Subarnrekha bed has deteriorated, and the Dhamara estuary and its outer bar have several feet less water now than they had in 1885' (Refer OFC 1928, Pg.9)

The rivers of Orissa are becoming shallow and cannot contain the immense volume of flood water brought down from the catchment areas. The main channel of Brahmani has deteriorated appreciably. The flood in the Brahmani lasts longer than in the Baitarani and on the whole causes more damage than the floods in the Baitarani and Mahanadi. That is why, the Brahmani was known as 'Sorrow of Orissa' before construction of a storage reservoir across it at Rengali. The river from Alva flows in a very circuitous route upto Dhamra. The flood slope of Brahmani from upstream of Aul ring bund up to sea is only 0.38 ft per mile due to which it takes longer time for the flood discharge to pass to the sea.

The Mahanadi takes more time to rise and remains in spate for longer period. During 1896, the flood in Mahanadi lasted from 24th July to 5th August. The Chitrotpala branch receives excess volume of water from Mahanadi which its narrow tortuous channel can not pass off to the sea.

The Kuakhai, a 2nd order tributary of the Mahanadi, can not contain water from the Kathjori and even spill during low floods. The Kuakhai branches off to Daya, Bhargavi and Kushabhadra. The Kushabhadra overflows its banks even during moderate floods. "After the first mile of its course, the Kushabhadra narrows considerably and pressure of flood is greatly felt , so that breaches may occur any where, in the embankment, situated on either bank, between, the second and tenth mile." (Source:

'River of Orissa', 1905 Pg.15 by Arnott). "There is a constant struggle at the mouth of the Kushabhadra between the river current and the tide. The struggle has resulted in the formation of a sandy bar which effectively checks the discharge at the time of heavy flood. xxxx. ...The left embankment is so aligned that breaches are inevitable in the year of high flood. The embankments are funnel shaped from the head of the stream downwards". (Source: Maddox Report, 1900 Vol. I & II).

1.3 Floods in the 19th and 20th Century

In 19th century, Orissa suffered from floods almost in every two years. This can be seen from the Table furnished as under.

Table 1.4 Floods and Inundations (1803-1900)

1804, 1806 & 07, 1809, 1812, 1814, 1817, 1820, 1823, 1826, 1831, 1834 & 35, 1837 & 38, 1840 to 42, 1844 to 48, 1851 to 1859, 1862 to 64, 1866, 1868, 1870 to 72, 1874 & 75, 1877, 1879 to 86, 1888 & 89, 1892 to 96 and 1900 (Total 59 times in 98 years)

Source: Bihar and Orissa Documents, File No.NF 21/1929 Revenue Department, March 1930.

The first half of twentieth century witnessed a series of floods. In August 1907, the Brahmani, Baitarani and Mahanadi rose to floods. In august 1911, the flood was confined to the Mahanadi, Kathjori and its branches. Puri district suffered from heavy flood in November 1917 (Source: Census of India, 1951 Vol. XI, Part I, Orissa Pg.57). Another incident of major flood havoc was in August 1920 in all the rivers. Due to loss of human life and livestock, severe flood of 1933 is remembered. Again major floods occurred in 1937 in three coastal districts of Cuttack, Puri and Balasore, in 1944 in the district of Puri and in 1946 in Balasore and Cuttack district.

1.3.1 Factors for occurrence of flood:

Two factors are primarily responsible for occurrence of flood viz. (a) flow of excess water resulting from the increased runoff in the catchment and (b) reduction in the carrying capacity of channels. Others factors, which contribute for creating flood havoc, are topography, size and shape of the basin, number of tributaries joining the main stream, condition of the ground and season etc. Of all the factors, precipitation frequency and intensity are of utmost importance in determining the magnitude and extent of flood damage. According to P.C. Mahalanabis "the heaviest rainfall in the catchment area of Orissa Rivers are given by storms and depression which move inland from the Bay of Bengal during the monsoon season". Floods in the State mainly occur due to following reasons:

i) Cyclonic heavy down pour for longer duration
ii) Depression in Bay of Bengal causing wide spread rainfall.
iii) Tidal wave in Bay of Bengal and Chilika Lake preventing quick discharge of flood water into the sea.
iv) Breach in flood protective embankments
v) Deterioration of river channel due to silting and encroachment by people
vi) Inter-connection of rivers such as Birupa with Brahmani and Brahmani with Baitarini.
vii) Large scale deforestation in the upper reaches.
viii) Configuration and characteristic of basins.

1.3.2 Occurrence of high floods in Pre-Hirakud Dam Period :

1.3.2.1 Flood of 1866

During this flood, water levels at Naraj I.B in Mahanadi rose to 90.52 ft (27.6m) and at Bellevue in Kathjori to 80.80 ft (24.63m). The flood discharge in Mahanadi at Naraj I.B was

1283600 cusec (36,342 cumec) as computed from Rhinds table. The flood continued for longer duration. In the district of Puri, 300 sq mile (777 sqkm) area remained submerged from 5 to 45 days. The depth of inundation varied from 1 m to 3 m and four lakh people were rendered homeless.

In 1866, Dr, Smith Sanitary Commissioner came to Cuttack by boat from False Point. Eye witness account has been described by him in his book 'Pilgrimage to Juggernaut' (Jagnnath) He wrote "The Mahanadi was immensely flooded. It had entirely inundated its bank and high trees were half submerged in its water. xxx..... The dense foliage on either bank appeared as though it were afloat on an endless and terrible swamp. The whole face of the country was under water to depth of four or five feet. The rice fields were entirely submerged and villages were inundated. In some of these, the villagers had constructed stages inside their houses, on which they sat waiting for the subsidence of flood which was still rising".

In Cuttack district, there were 413 breaches in the embankment submerging 642 sq miles (1663 sqkm) for 3 to 60 days. Depth of inundation varied from 1 m to 5 m. All the crops in the submerged area were destroyed. About seven lakhs people lost their shelter and belongings. This led to occurrence of "Na-anka Durbhikshya' in 1866.

1.3.2.2 Flood of 1896

The Mahanadi remained above level from 24th July to 7th August with peak flood level of 92.10 ft (28.07m) at Naraj. The flood discharge in the rivers as computed from Rhinds table was 15, 02, 124 cusec (42529 cumec). The flood level at Bellevue in Kathjori rose to 82.87 ft (25.27m)

As per report of Mr. W.A. Inglish, the then Superintending Engineer, Orissa Circle, there was a total devastation in the delta due to flood. There were breaches of 300 m length with 10 m depth of scour. Besides heavy damage to crops and houses, the communication was badly affected, the irrigation system in Kendrapara and Pattamundai area suffered to a great extent.

1.3.2.3 Flood of 1926

Water level at Naraj I.B rose to 28.0 m with a flood peak of 14, 73, 233 cusec (41711 cumec). According to the letter No.725 S.G dated 19.11.1926 of Mr. J.A.Hubback Commissioner of Orissa Division addressed to the Secretary, loss of human life in Cuttack was 15. About 3000 houses were severely damaged. Crops in flood affected areas of Cuttack and Puri were completely destroyed.

1.3.2.4 Flood of 1937

The peak flood discharge in Mahanadi at Naraj was 14, 19, 746 cusec (40189 cumec). The flood level of Bellevue was 82.15 ft (25.04 m) Loss of life in Cuttack and Puri was six. Number of villages affected in Cuttack and Puri were respectively 758 and 556. Crops grown in 80531 ha (1, 98,912 ac) in the district of Cuttack, Puri and Sambalpur were damaged (Source: - File No-XIIE-9/ 38 & XIIE-7/37 of Revenue Commissioner). Frequency of high flood during Pre-Hirakud dam is furnished in table No-1.5

Table No1.5 Frequency of high flood in Pre-Hirakud period

Sl.No	Total no. of years	Peak flood discharge in cumec	No. of occurrence	% of years of occurrence of flood.
1	1855 to 1955	25485 & above	68	64.76
2	-do-	28317 & above	51	48.57
3	-do-	33980 & above	24	22.86

Source: - Delta Development Plan, Vol. VI, Hydrology & Flood Control (1986) Pg.192 E.I.C. (Irrigation)

1.4 Back ground Leading to Construction of the Project

The first serious attempt to tackle the flood problem was made in 1853. Sir Arthur Cotton, one of the foremost pioneers of modern irrigation in India was asked to report on the harnessing of the waters of the Orissa Rivers.

The menace of floods however continued and during the period from 1858 to 1896 there were four major floods occurring in 1859, 1872,1892 and 1896. In 1872 R.H Rhinds Executive Engineer, Pooree (Puri) Survey Division made an extensive survey of the Mahanadi floods and recorded reading of flood discharge of all the branches of the river and derived the final flood discharge of the river to be 15, 03, 637 cusec (42578 cumec). Flood discharge observed in 1834 and 1855 were 15, 70, 622 cusec and 15, 43, 431 cusec respectively. His observations recorded in the 'Report upon the floods of the Mahanuddy (Mahanadi) during the year 1872' standout as important fundamental hydraulic data for technical undertaking relating to flood protection.

But the idea of flood control by means of reservoirs struck even some of the earlier engineers. J.P Beadle, the Superintendent of Embankments, Lower Provinces, Bengal in his report as early as 1856 observed: - "But the flood rise is much higher than ought to be allowed and it is hoped that some large reservoirs may be formed at moderate expenses in the hills above the delta so that the flood water may be stored in the hills till the dry season and then return to the river bed for the maintenance of the navigable fair weather canals" (Source: - Floods in Orissa Rivers during 1955-56, Govt. of Orissa, Revenue, Spl. Relief Dept. Pg. 43).

1.4.1 Report of Orissa Flood Committee 1928

Between 1911 to 1927 there were six severe floods occurring in 1911, 1919,1920,1925,1926 and 1927. There was however no general enquiries into the Flood Problem during this period till the Govt. of Bihar and Orissa (a province created in 1912) in their Resolution No-4741 Xc-6-27-I dated 25-11-1927 while appointing a Committee to enquire into the flood problem of Orissa observed that "since the year 1855 many special enquiries have been made with a view to prevent or diminish the intensity of these floods. In the majority of cases attention was focused on problems of Mahanadi River which chiefly affects the Puri and Cuttack districts. But the floods of 1920 drew special attention to the Brahmani and Baitarani and other rivers which caused severe damage in Cuttack and Balasore.

The Orissa Flood Committee 1928, after a lapse of nearly 72 years and amidst more modern environments and scientific discoveries, took a very pessimistic view of the idea when they observed: -

"A proposal which has been strongly urged in several quarters is that reservoir should be built on the rivers in the country along the upper delta in which all water which cannot be passed to the sea when flooding should be stored, the water being released again as soon as the rivers fall below the flood point. In the absence of proper hydrographs we are not in a position to say exactly how large the reservoirs would have to be, but to deal with the Mahanadi alone it would be necessary to impound at least one million cubic feet and possibly more in order to reduce its discharge to an amount which its various branches can carry. It is most improbable that a site or sites could be found which would permit storage of this huge volume of water. Even should such exist, the cost would run into many crores of rupees. But the gravest difficulty in any scheme of this nature is the fact that the lives of the reservoir presuming their construction to be technically and financially possible would be extremely limited, the silt-laden water trapped by the Dams would deposit its sediment in a comparatively short period and the storage capacity of the reservoirs can be reduced to such an extent as to render the work useless. We are convinced that no solution of the flood problem of Orissa is to be found on these lines" (Source: - Floods in Orissa Rivers during 1955-56 Govt. of Orissa, Revenue Spl. Relief, Dept. Pg.43-44).

1.4.1.1 Recommendations of the Committee

The 1928 Flood Committee recommended that 8000 sq mile (20720 sq.km) of the deltaic area, fully protected by providing embankments against floods and irrigation by canal, should be maintained. In the semi-protected areas which are protected to a greater or lesser degree from floods by embankments but not irrigated by canals, all obstacles against free flow of floods to the sea should be removed as far as possible without detriment to the maintenance of the population. They recommended the abandonment of the Orissa Coast Canal, a part of Dudhei Canal of the High Level Canal System which they thought served little purpose for irrigation or transport but did obstruct free flow of flood water. They also assigned specific reasons in particular cases, suggesting opening of estuaries, cutting the sand dunes along the coast to allow free flow of water and prevention of private reclamation along he tidal basins and in the bed of the Chilka Lake. There were altogether 70 items of recommendations. Most of the proposals were investigated into and the less costly ones were carried out to some extent, while some were examined and abandoned as expensive and undesirable and on the whole there was no measure of expediency in giving effect to the said recommendations. (Source: - Floods in Orissa Rivers during 1955-56, Govt. of Orissa, Revenue (Spl. Relief) Department Pg. 41-42).

No progress was made in giving effect to any of the above proposals for want of funds and the opposition of vested interests to the removal of embankments etc. which though possibly of general benefit to the locality as a whole prejudicially affected individual interests.

1.4.2 Sir M.Visvesvaraya's Report of 1937

"On account of heavy floods in the deltaic tracts in the monsoon of 1937, which caused considerable damage, the Govt. sought the advice of Sir M.Visvesvaraya, who after the study of old records and other official documents, submitted his recommendations for framing comprehensive plan for dealing with the flood problem.

He laid particular emphasis on the investigation of the feasibility of constructing flood control reservoirs on any of the three main rivers, but particularly on the Mahanadi and stated that "reservoir if constructed will be of value to hold up the floods temporarily and release them gradually so as to run to waste at a harmless rate".

He added "The (1928) Committee considered that the proposals under this head are not practicable. The reason seems to be that such reservoirs will have to be outside the province and therefore beyond the jurisdiction of the Orissa Government. Another reason expressly stated is that on a river of the size of Mahanadi, the reservoir should be constructed only at a gigantic cost.

Unless the committee have based their conclusions on dependable data, which however are not revealed in their report, it seems necessary to investigate this question more fully.

It may be mentioned that even on such enormous river like the Mississippi and its tributary, the Ohio River (in the United States of America), the possibility of constructing flood storage reservoirs is receiving consideration.

This remedy seems to be beyond the resources of the province of Orissa at the present time. For such a remedy to succeed in the near future, it will have to be taken up by the adjoining province or provinces at the same time and treated as an inter - provincial problem. If this remedy is definitely ruled out, the investigations conducted and the reasons for abandoning the proposal should be clearly placed on record"

He further stated that "writer is well aware that on account of their cost and the difficulties faced on inter-provincial cooperation there is small prospect of the idea materialising in the near future, but it is desirable to make some preliminary investigation to see how the question stands at present, and whether there is any prospect of its being takenup later when the province is able to finance such

undertakings. If a reservoir is constructed, it may prove useful generating electric power etc. Once floods come under effective control, the whole area may be transformed into a prosperous region".

This approach of Sir M.Visvesvaraya to the flood problem of Orissa was most realistic and constructive as in pointing to the possibilities of conservation of the river water for multipurpose development, he was much ahead of his time. In his recommendation he also suggested the appointment of a Committee of experienced engineers to go into this question. (Source: - Report of Hirakud Dam Project- Revised, 1952 Vol. 1, General Report Pg.8-10, Govt. of India)

1.4.3 Flood Enquiry Committee of 1938

The Orissa Government accordingly appointed a committee in 1938 consisting of 3 engineers to investigate the flood problem confronting the province. The Committee submitted its report containing separate recommendation relating to the whole delta, and the individual deltas.

Regarding the whole delta, the committee was of the opinion that the policy to be followed for flood protection should be improvement of main rivers by

- control of discharge and silt charges at the bifurcations of the rivers.
- providing better outlets to the sea
- adequately strengthening the embankments and preventing breaches both in embankment and in the natural bank and
- provision of high level escapes.

The Committee also recommended that the question of detention or storage reservoirs be only taken up for investigation after other cheaper methods had been tried and their results known and that judging from the loss caused to the country by flood damage. Government would be justified in spending a much large amount than had been incurred in the past on flood protection.

1.4.4 Enquiry Committee Report of 1940

In 1940, after a study of the data so collected the Committee was of the opinion that:

a) To improve the river it was necessary: -

i) to improve the outfall conditions

ii) to control the distribution of water and sand entry at the heads of channels and

iii) to restrict the number of channels.

b) Without double embankments, deltaic rivers broke up and deteriorated. Double embanking in conjunction with other measures recommended retarded or prevented such deterioration.

c) Double embankments necessitated the provision of high level escapes. With the improvement of the river channels the depth of spill through these escapes should be gradually reduced.

d) Control by double embankments and escape should be combined with suitable drainage of the basins between rivers.

1.4.5 The Cuttack Conference of 1945:

Finally in 1945, the problem was referred to the newly constituted Central Waterways, Irrigation & Navigation Commission (CWI & NC). The Chairman of the Commission Shri A.N.Khosla, as a result of local inspection, studies of previous reports and discussion with the local officers came to the conclusion that the only cure for the many troubles of Orissa, viz. floods, drought etc. lay in the control, construction and utilization of the enormous water wealth of its rivers by means of a storage dam.

At a conference held at Cuttack in November 1945 under the chairmanship of Dr. B.R.Ambedkar, Member of Labour in the Govt. of India, the representatives of Orissa State and Madhya Pradesh, it was unanimously agreed that the potentialities of Mahanadi for unified multipurpose

development viz. flood control, irrigation, navigation, and hydro electric power should be thoroughly and expeditiously investigated by the commission. In this context, speech of Dr.Ambedkar is appended vide Annexure 1.1.

1.4.6 Unified Development of the Mahanadi Valley

The investigation conducted by the Central Waterways, Irrigation and Navigation Commission (renamed as Central Water & Power Commission and now bifurcated to Central Water Commission and Central Electricity Authority) showed that the only solution that would meet all the requirements of the situation lay in the basin-wise plan of the development of the Mahanadi valley, which should comprise the construction of series of dams on the main river and its major tributaries to conserve and utilize the water of Mahanadi river for purposes of irrigation, hydropower generation, flood control, navigation and other facilities such as fish culture, malaria control, soil conservation, recreation etc, with a view to raising the standard of living of the common people by banishing famine, malnutrition and disease and extending to them the necessities and amenities of modern life.

As the implementation of the above plan would involve a great deal of time in surveys, investigations, preparation and a large amount of capital investment, a more satisfactory way appeared to be to work the plan in stages in a manner that each stage would function as an independent project and yet become an integral part of the unified basin wide plan. This would comply with the urgent need for irrigation in Sambalpur district and deltaic region, cover overall problem for industrial developments, flood control in the delta and navigation facilities to the maximum extent possible. The overall development of the valley could thus be fully achieved by the construction of three dams on the main rivers at only three available good sites viz. Hirakud, Tikarpara and Naraj, each one with its own canal network and hydro electric power installations.

It was, therefore, recommended by the Commission that although the optimum development of the Mahanadi valley for multipurpose uses of industry, agriculture and transport etc. could be secured only if all the 3 proposed dams were constructed, the Hirakud dam project could be executed and operated as an independent unit, as it was uppermost on the river and was the simplest in respect of physical features, design and construction requirements and that its construction would not in anyway imply and commitment as to the construction of the Tikarpara or the Naraj projects.

If at a later date, one or both of the two projects were executed, the Hirakud dam projects would fit in as an integral unit of the overall development and the Mahanadi fully harnessed on these lines would afford complete protection against the periodical floods, provide irrigation for 20 millions acres of land, generate 4 million KW of power, and provide 380 miles navigable waterway stretching from the borders of Madhya Pradesh (now Chhatisgarh) all the way to the Bay of Bengal. The volume of traffic on this waterway would in time warrant a sea port for Orissa at Chandbali or at Dhamra, capable of handling a transport of as much as 6 million tons a year. (Source: - Report of the Hirakud Dam Project, Revised, 1952 Vol.I, General Report Pg.12-13, Govt. of India). Accordingly it was decided to undertake the construction of the Hirakud Dam Project first, as it was from Hirakud Dam only that irrigation could be provided to areas in Sambalpur district and afford a substantial and adequate measures of flood protection to the delta at relatively small cost and at a fairly early date. Power could be developed to the extent of 350,000 KW so that industrial and agricultural development could be planned and proceeded with almost at once.

In addition, the construction of the Hirakud Dam Project had some natural advantages, which were absent in the case of two lower dams. The submerged area of the reservoir would be exclusively in Orissa state.

1.4.7 Recommendation of Dr. Khosla, Chairman CWI & NC

Extract of Dr. A.N.Khosla, chairman Central Waterways, Irrigation & Navigation Commission Dated 12.06.1947 is reproduced below (vide Mahanadi Valley Development- Hirakud Dam Project, Govt. of India, June 1947, Pg. xiv) where he emphasized that the Hirakud Dam Project should be sanctioned immediately in the greater interest of people of Sambalpur and Orissa as a whole.

i) The Hirakud Dam Project should be sanctioned forthwith. The feasibility, utility and economics of the project should be judged in the light of the greater interest of the people of Sambalpur and Orissa as a whole. The greater good of community must have priority over the interest of individual or groups. Vested interests must not be allowed to stand in the way of the country's progress nor must the resource of the region be allowed to cause destruction, be wasted or lie dormant on that account.

ii) Immediate steps should be taken to decide the programme and policy of resettlement of the people whose lands will be submerged in the proposed reservoir.

iii) Immediate steps should be taken to set up a cement factory near the dam site and to place order for the necessary plant and equipment. It will take about 18 months from the date of placing of the order to the time this factory starts producing cement for the construction of the dam.

iv) Immediate action should be taken to set up an iron and steel factory in Sambalpur. With cheap power available at the site and iron ore, coal, lime-stone and fuel wood obtained within easy reach, and iron and steel factory at this place should prove to be a great economic success.

v) In addition to setting up the above two industries, immediate steps should be taken to plan and implement the programme in respect of other industries which can be set up in the neighbourhood of the Hirakud dam, e.g. aluminum, ferro alloys, fertilizers, paper, textile, chemical etc.

vi) The industrial plan for Orissa should be reviewed in the context of the all-India industrial plan with special reference to requirements of defence. This assumes special importance in view of the latest constitutional development.

vii) Special consideration should be given to the food production aspect of the project as with the cecession of the richest food producing areas, the rest of India will have to make serious efforts to solve her food problem and become self-sufficient.

viii) Special consideration should be given to agricultural, industrial, navigational and pure research by setting up agricultural farms, pilot factories and research laboratories with a view to keeping abreast of latest development and effecting new improvements. The planning of this research should take special note of defence requirements.

ix) Early inquiry should be instituted into the working of irrigation system in the delta with a view to ascertaining the reason for being unproductive and the possibilities of improvement and further extensions".

1.5 Dr. Khosla's Decade of Destiny (1963-73)

Dr. A.N.Khosla, delivered the 6th Sir M.Visvesvaraya lecture on 19.5.1963 at Bangalore on 'The Integrated Development of the River Basin of Orissa, where he stressed that Flood Control and Irrigation being the two basic needs of Orissa should be taken up for the overall development of the extremely poor but potentially rich state.'

'Orissa mirrors the paradox of all backward areas-poverty amidst plenty. Orissa has untold wealth of natural resources- land, forest, water, minerals, a long sea coast. Yet it is the poorest and most backward state of India. Any plan for development of these vast natural resources which will help transform poverty into plenty in the state of Orissa may well serve as a model for other similarly situated areas in India and elsewhere in the world'.

1.5.1 Plan of May 1945 by Dr. Khosla

In May 1945 Dr.Khosla, the then Chairman of CWI & NC (now Central Water Commission) conceived the first plan for the unified development of Mahanadi basin as an initial step for the overall development of basin of all the rivers of Orissa (now River Basin Planning). It was contemplated to construct three dams across Mahanadi at Hirakud, Tikerapara and Naraj for controlling Mahanadi floods and for power generation, irrigation, navigation and pisciculture etc. Work on Hirakud Dam commenced during 1949-50 after detailed survey and investigation and completed in 1957. Proposed Naraj Dam was substituted by a barrage at Mundali for providing irrigation to stage-II delta (in the undivided districts of Puri). Even though foundation stone for Tikerapara Dam was laid by Pt. Nehru in Jan.1964, the same could not see the light of the day due to submergence of large tracts of land including Sonepur, Boudh and Athamallik towns.

1.5.2 Plan of May 1963

This is Dr. Khosla's comprehensive integrated plan which covers the multi-purpose development of the basin of all major and small rivers of the state. It envisages a development programme under six groups that has been shown in table No.1.6.

Table 1.6 Development of the basin

Groups	Name of the project	Catchment area (sq.mile)	Submergence area (acres)	Storage Mac.ft.		Area to be irrigated '000Ac.		Power generation in million KWH
				Gross	live	Kharif	Rabi	
A	Hirakud dam project	32750	181000	6.60	4.72	380	220	1120
	Delta irrigation	–	–	–	–	964	578	
	Machhkund	755	22500	0.79	0.73	–	–	310
	Balimela	1855	44300	3.09	2.03	240	144	1210
B	Tikerapara Dam (*)	48000	600,000	47.0	38.0	–	–	7240
	Gania barrage (*)	48705	20, 000	0.40	0.10	2118	1271	–
C	Indravati	1040	28800	2.35	1.55	350	210	2050
	Upper Kolab	618	45000	1.15	1.08	200	120	693
	Tikra (Brahmani Basin) (*)	512	34000	1.90	1.15	77	46	46
D	Barakot Across Brahmani) (*)	8843	148000	5.95	3.10	637	382	2530
	Lower Kolab (*)	1200	3800	2.91	2.00	200	120	1096
E	Bhimkund (Across Baitarani) (*)	2370	80000	3.40	3.00	–	–	2000
F	Burhabalanga, Subarnrekha, Khadakhi, Bheden, Ong,Tel, Salki & Bagh	These projects are under investigation except Khadakhi, Salki and Bagh which have been completed. Subarnrekha and Ong are under construction.						

* These projects have not seen the light of the day.

All the hydrology projects taken together will involve capital expenditure of Rs.927.52 cr. Besides these projects will achieve:

i) Complete flood control
ii) Installation of 8520 MW of hydropower
iii) Irrigation to 82, 57, 000 ac (33, 41, 608 ha)
iv) About one thousand miles of navigable canal, water ways will connect Rourkela steel plant to Paradeep port and Chilika Lake.
v) Reservoir area of 12, 43800 ac (5, 03366 ha) for pisciculture, recreation and tourism.
vi) A potential fresh water protected harbour covering 150 sq.miles (388.6 sq.km) in the western part of Chilika lake with about 36 mile (57.9 km) length of fresh water canal with a perennial discharge of 37750 cusec (1069 cumec)

Dr.Khosla compared his May 1963 plan for Orissa with TVA of USA. The TVA transformed the entire economy and life of the people living in the Tennessee Valley which was one of the most poverty stricken and backward areas of USA into one of the most prosperous area in power, agriculture and industry etc. and in living standards. He advocated that the plan provided all the features similar to that incorporated in the TVA plan. "It does something more; it will provide irrigation to nearly seven million acres in addition to the present acreage under irrigation. Given the resources, what the TVA has achieved for the Tennessee Valley, the Orissa plan can achieve much more for Orissa and the adjoining states of W.B, Bihar, M.P and A.P possibly in a shorter time".

But his ambitious Tikerpara dam with Gania barrage and Bhimkund across river Baitarani could not be implemented even today. The Tikerapara proposal was modified during 1955-56 by lowering the FRL from RL 131 m (430 ft) to RL 99.0 m. But this proposal suffered from the disadvantage of submergence as the Athamallik town is situated at RL 80 m and Boudh at RL 90 m. For irrigation, it was proposed to construct a barrage at Gania with pond level at RL 55.8m. This necessitated rethinking about Tikerapara which gave birth to Manibhadra. (Source: History of Irrigation Development in Orissa, INCID, November 2009 by G.C.Sahu Pg 259)

Manibhadra proposal also met the same fate as that of Tikerapara. Due to large scale submergence, Govt. of Orissa dropped the proposal for constructing the 2nd dam across Mahanadi at Manibhadra. As both Tikerapara and Manibhadra were shelved, it was thought of to construct a barrage at Subalaya to ameliorate the sufferings of coastal Orissa from floods. The proposal contemplated flood moderation by dynamic storage which also ended in fiasco. Thus there appears to be no solution to the vexed flood problem of coastal region in the near future.

1.6 Factors for selecting Hirakud as first dam site:

Though three dams were contemplated across Mahanadi in the state of Orissa, it was preferred to undertake the construction of Hirakud Dam first, as it was only possible to provide irrigation from the dam to the land in Sambalpur district and afford a substantial and adequate measures of flood protection to the delta at a relatively cheaper cost and fairly at an early date. Power generation to the extent of 350000 KW could be achieved for setting up industries. Agricultural development will start at once bringing overall prosperity to the people of the State.

Besides, construction of Hirakud site had some natural advantages which were absent incase of other two dams site i.e. Tikerapara & Naraj. Geology of the dam site was simple and free from complex features. The foundation rock was sound. Construction materials viz. cement, aggregate and sand for concrete dam and suitable soil for the earth dam and dykes were available in the nearby vicinity. In this regard, concluding remarks of Dr. F.A.Nickell, consulting Geologist (31.03.1947) are worth noting:

i) Extremely hard bed rock composed chiefly of granite gneiss with schist forms the foundation for the concrete dam portion and is available near the surface.

ii) Sufficient material for construction are available including embankment materials.

iii) At no place, it will be difficult to secure a tight cutoff to prevent seepage from reservoir.

iv) Foundation for all components of the dam is entirely stable under load.

v) Dykes on both sides will pose no problem.

vi) As limestone, coal and other raw materials occur abundantly within 50 miles radius of Sambalpur, cement can be readily manufactured for the project.

vii) No important loss of known mineral wealth will take place in the reservoir.

(Source: Mahanadi Valley Development-Hirakud Dam Project, June 1947, CWINC, Govt. of India, Pg. 194).

Further, major submergence due to construction of reservoir was restricted exclusively in Orissa.

As per original planning, optimum development of Mahanadi valley for multi-uses i.e. agriculture, industry, flood control & navigation etc. could only be achieved if all the three proposed dams are constructed. But Hirakud can be constructed and operated as an independent unit which would fit in the integral system for unified development. Its construction in no way hinders the construction of other two dams in the lower reaches, as and when taken up.

These advantages dictated construction of Hirakud Dam in the first instance. The foundation stone of the project was laid on 15.03.1946 by his Excellency the Governor of Orissa. The Index map of the project is enclosed vide Drg. No. 1.4. Statistical data of the Project is furnished in Annexure 1.2.

1.7 Few other major projects taken up in the country during Pre-plan period:

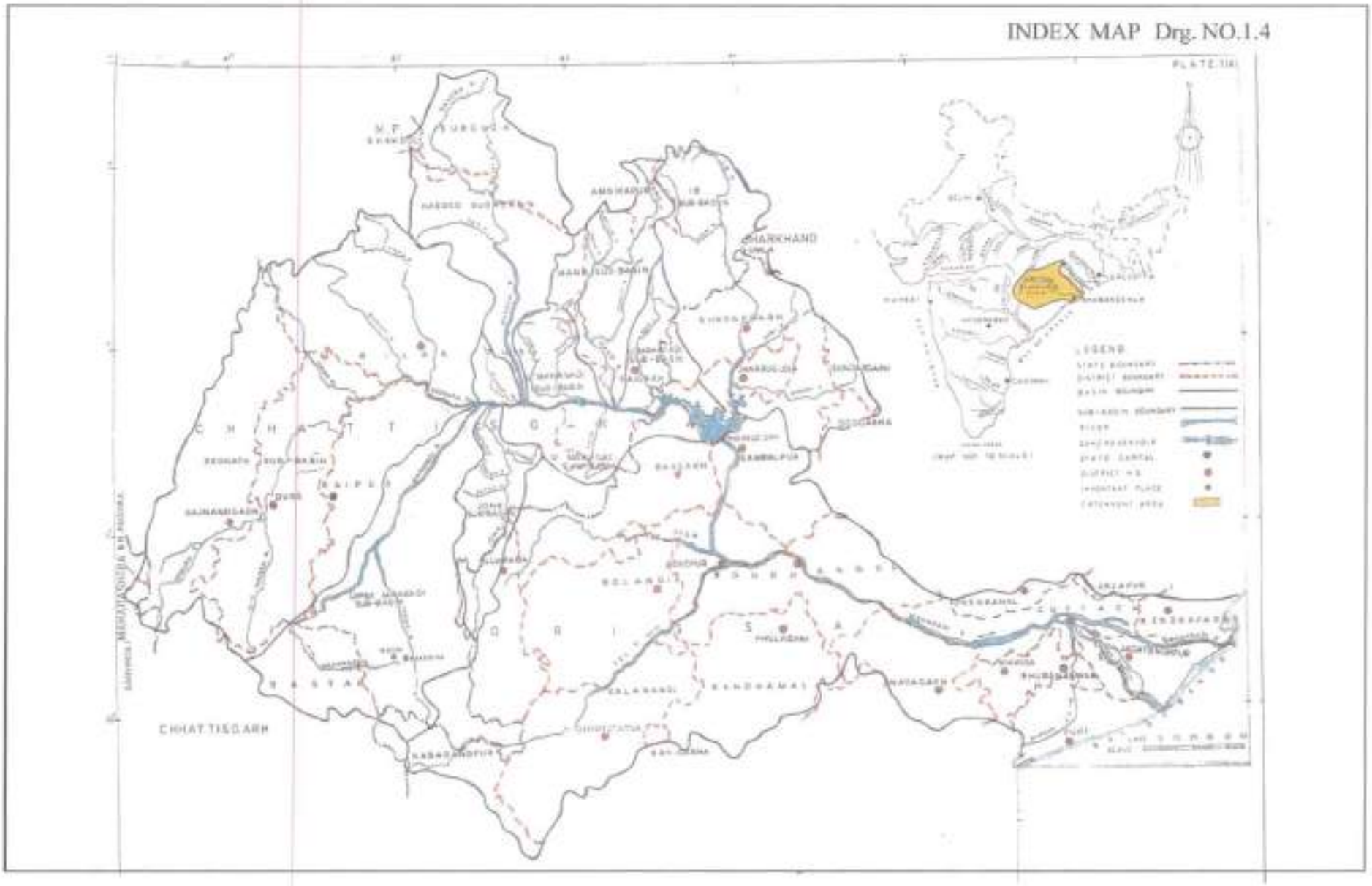
As the country was passing through acute shortage of foods grains, Irrigation projects started getting top priority in Five years plans. At the time of launching the 1st Five Year Plan in 1951, the net irrigated area was only 20.85 million ha. All over the country, major irrigation projects were takenup in right earnest. Even work on few major projects had commenced during Pre-plan and Pre Independence period, the list of which is furnished in Table 1.7.

Table 1.7 Projects during Pre-plan Period:

Sl.no	States	Name of the projects	Year
1	West Bengal	Mayurakhi	1946-56
2	Orissa	Hirakud	1947-57
3	Bihar	Domodar valley projects Kosi	1950-58
4	Karnatak	Tungabhadra	1945-54
5	Gujrat	Kakrapar	1949-55



INDEX MAP Drg. NO.1.4



Annexure 1.1

Opening Address of Dr. B.R.Ambedkar Labour Member, Government of India, at the Orissa Multi-purpose Rivers Conference at Cuttack on 8TH NOVEMBER 1945

I am glad to be here with you, the representative of the Orissa Government and the representative of the Orissa States, to lend a helping hand to solve the problem of Orissa. Let me begin by asking, "What is the problem of Orissa?" To say that the problem is one of flood is both an over simplification and an understatement of the problem. I see the problem in a somewhat different light, when I think of Orissa, the picture that comes to my mind is that of a people subject not to one affliction, but to many.

One such affliction which is on the lips of every body is the constant exposure of the people to floods, causing damage to life and property and a general sense of insecurity. The affliction is due not merely to floods but also to droughts and famines. The damage by droughts and famines can be as great as that caused by floods. It is said that in the drought of 1866, nearly 40 percent of the population of Puri District perished.

- **Not above poverty line**

Deterioration in health is another affliction of the people of Orissa. The total population of the province of Orissa is 77.5 lakhs. According to the health report of the province, the total deaths in 1944 were 2, 35, 581. Of these 1, 30, 000 died from some kind of fevers and the majority by malarial fever. This mean that 3 percent of the people died in 1944, and of these nearly half died of malaria. This is a very high figure. In 1944, 19 percent of the student population was found to show distinct signs of malnutrition and 8.7 percent showed signs of vitamin deficiency. If these facts are true, Orissa cannot be said to be above the poverty line. The third affliction of Orissa, I think it could be called affliction, is its want to internal communication. Orissa is a locked up country. Except for a solitary railway line running along the Eastern sea-cost, there is no means of communication either by railway or by canal to connect the vast hinterland with the sea coast.

- **Water Wealth**

Should Orissa continue to be in such a wretched state as it is today? It need not. It has natural resources, and they are by no means meagre. Orissa has coal. Orissa has iron, chrome, graphite, bauxite, lime stone, mica and also bamboo to mention only some of the important of its natural resources. There is another precious possession which Orissa has, namely her water wealth. The amount of water that passes through the Orissa Delta is just vast. The Delta comprising roughly the three districts of Cuttack, Puri and Balasore and covering an area of about 8, 000 sq. miles is traversed by a network of distributaries arising mainly from the three rivers, the Mahanadi, Brahmani and Baitarani. Two more rivers the Burhabalnga and Subarnarekha, of relatively lesser important also pass through delta. The three main rivers referred to above drain an area of 60, 000 sq. miles above the delta, lying in the Eastern State, Central Provinces and Bihar. Of these the Mahanadi the largest of the three drains 51, 000 sq.miles. Between them, these three rivers carry each year to the sea a discharge of about 90 M.ac.ft.

- **Not Utilised**

Given the resources, why has Orissa continued to be so poor, so backward and so wretched a Province? The only answer I can give is that Orissa has not found the best method of utilizing her water wealth. Much effort has undoubtedly been spent in inquiring into the question of floods.

As early as 1872 there was a general enquiry by Mr. Rhind. I do not know what happened to his report. Nothing seems to have been done thereafter till 1928. From the year down to 1945, there have been a series of committees appointed to tackle this problem. The Orissa Flood Enquiry Committee of 1928 was presided over by the Bengal Chief Engineer Mr. Adams Williams. In 1937, the enquiry was entrusted to the able hands of Sri.M.Visvevaraya who submitted two reports, one in 1937 and another in 1939. His work was followed by the Orissa Flood Advisory Committee. The committee submitted a preliminary report in 1938 and continued its work till 1945, during which period it submitted three interim reports. The latest effort in that direction was made as recently as the 15th March, 1945, when a Flood Conference was convened at Cuttack by the Government of Orissa. With all respect to the members of these committees, I am sorry to say that they did not bring a right approach to bear on the problem. They were influenced by the idea that water in excessive quantity was an evil. They were guided by the conception that when water comes in excessive quantity, what need to be done is to let it run to the sea in an orderly flow. Both these views are now regarded as grave misconceptions, as positively dangerous from the point of view of the people. It is wrong to think that water in excessive quantity is an evil. Water can never be so excessive as to be an evil. Man suffers more from lack of water than from excess of it. The trouble is that Nature is not only niggardly in the amount of water it gives, it is also erratic in its distribution alternating between drought and storm. But this can not alter the fact that water is wealth. Water being the wealth of the people and distribution being uncertain, the correct approach is not to complian against Nature but to conserve it. If conservation of water is mandatory from the point of view of public good, then obviously the plan of embankment is a wrong plan. It is a means who does not subserve the end, namely the conservation of water, and must therefore be abandoned.

- **Dam the Rivers**

Orissa Delta is not the only area where there is so much amount of evil proceeding from that water. The United State of America had the same problem to face. Some of their rivers the Missouri, the Miami and the Tennessee have given rise to the same problem in the United States. Orissa must therefore adopt the method which the United States adopted in dealing with the problem of its rivers. That method is to dam the rivers at various points to conserve the water permanently in reservoirs.

There are many purposes which such reservoirs can serve. They can give water for irrigation. I am told that if it were possible to store the entire runoff of the Mahanadi, it will be enough to irrigate 30 million acres provided that much area was available. The water stored in the reservoirs can be used for generating electric power. If in the midst of its natural resources, Orissa has remained an industrially undeveloped area, it is due to want of cheap power to run its factories. Here there will be abundance of electric power more than Orissa can hope to consume for a long time to come.

Another use to which this water could be put to, is navigation. Internal navigation in India has had a very chequered history. During the rule of the East India Company, the provision for internal navigation occupied a very prominent part in the Public Works budget of the Company's Government. Many of the navigational canals we have in India today, and you have one in Orissa itself, are the remnants of that policy. The Railways came in, and for a time the policy was to have both Railways and Navigation. By 1875, there arose real controversy in which the issue was Railways versus Navigation canals. The battle for canals was fought bravely by the late Sir Arthur Cotton, one of the few Engineers of the time with big ideas. Unfortunately, the supporters of Railways won. I am not quite happy about this victory of Railways over the Canals. Much more annoying is the ignorant opinion of the supporters of Railways that canals must go because they do

not pay, without knowing that if the canals do not pay, it is not because they cannot pay but because their capacity to pay has been terribly mutilated by leaving them uncompleted. I am sure that internal navigation cannot be neglected in the way in which it has been in the past. We ought to borrow a leaf from Germany and Russia in this matter and not only reconstruct our old canals but make new ones and not sacrifice them to the exigencies of Railways.

- **Storage Scheme**

The storage scheme as applied to the rivers of Orissa will have this special features, namely that it will not only give irrigation and electricity, it will also provide a long line of internal navigation. I am told that it may be possible to connect Chandbali to Sambalpur and beyond, by a navigable inland waterway by the construction of say, three dams at (1) the point of its debouch from the hills, (7 miles above Naraj), (2) Tikarpara and (3) above Sambalpur. If this plan works out, there will be a navigable channel for 350 miles which could be maintained all the year round, affording a cheap and convenient artery for passenger and goods traffic. Imagine the possibility of coastal craft or light vessels going right into the interior on a canal extending from the sea via Cuttack to Sambalpur and beyond into the Central Provinces. Here is a scheme which converts the forces of evils into powers of good. Orissa wants to get rid of malaria and other – to use American phraseology - "low income disease" causing ill-health and corroding the stamina of her people. Orissa wants to raise the standard of living of her people and to advance her prosperity by irrigation, by navigation and by producing cheap electrical power. All these purpose can fortunately be achieved by one single plan, namely to build reservoirs and store the water which is flowing in its rivers.

- **Only Method**

I am therefore glad to note that the Orissa Flood Protection Committee held in 1945 struck the right note when it said that the final solution of Orissa's water problem should be sought in the construction of multi-purpose dams. My only comment is that this should be regarded as the only method and treated as an immediate programme rather than as ultimate aim. The potentiality of this project may not be clear to every body. But it is indeed very great. It is desirable that some idea of its immensity should be conveyed to the Government of Orissa, the representatives of the State of Orissa and the people of Orissa as well, so that their interest in the project may be aroused. A comparative statement may well give such an idea. One hears of lake Mead at the Boulder Dam in the United States as the biggest man-made lake in the world.

We are also hearing of Mettur Dam in Madras and the proposed Tungabhadra Dam in Hyderabad, which are likely to be its rivals. Compared with the first the total volume of water that flows annually through Orissa Rivers will fill three time lake Mead. Compared with the Mettur Dam in Madras and the Tungabhadra Dam in Hyderabad, it may be pointed out that the Mahanadi alone carried over 55 million acre feet which would be thirty times the capacity of reservoir at the Mettur Dam in Madras and twenty times the capacity of the proposed Tungabhadra Dam. How much of this water we can economically and effectively store is a matter for investigation. The Government of Orissa, the State of Orissa and the people of Orissa should therefore regard themselves as lucky and proud of this project and determine and endeavour to make it a reality.

- **Survey on Mahanadi**

The agenda for the meeting is a very modest one. It is : -

1. To consider the desirability of undertaking forthwith the surveys and investigation of the Orissa Rivers with a view to the preparation of schemes of unified and multipurpose development comprising

(a) flood control

(b) navigation

(c) irrigation and drainage

(d) soil conservation

(e) power development.

2. To consider the desirability of concentrating in the first instance on surveys and investigations of the Mahanadi river for the control and development of that river.

3. To examine the desirability of surveys and investigations being carried out by Provincial Government in conjunction with and under the general direction of the Central Waterways, Irrigation and Navigation Commission.

Our main purpose today is to see if we can agree in the necessity for a thorough survey and investigation of the natural resources and physical features of the region with a view to prepare a comprehensive scheme for its development. For a balanced appreciation of the situation, it is necessary to carry out comprehensive surveys and investigations with a view to ascertain the possibilities of irrigation, navigation, power development and other facilities in the delta as well as in the drainage area as a whole. The investigations so far carried out have been mostly limited to the delta, although some work was done in 1862 on the survey of dam site for small reservoir on the five tributaries of the Mahanadi, viz. The Tel, Ib, Mand, Hasdeo and Jonk. The Central Waterways Irrigation and Navigation Commission constituted by the Government of India under the chairmanship of an able and well known irrigation engineer Rai Bahadur Khosla, is shortly going to initiate hydrological and other surveys of the Orissa rivers as part of their normal duties. But it will be necessary for the province and States to undertake connected surveys in conjunction with and under the general direction and supervision of the Commission.

- **Two Points**

Before closing, I would like to draw the attention of the parties which have met at this Conference to two points. They are intimately connected with the success of the project and about which they will have to make their mind at an early date. First in their readiness to consider the question of submergence of land. The reservoirs formed by the construction of dams will submerge large areas of land, both in Orissa and the Eastern States. If these dams are carried higher up the river or its tributaries, certain area in the Central Provinces will also be submerged. This question will need to be considered in detail in relation to the overall benefits of the integrated scheme. The submergence of area will be inevitable if the waters of the rivers have to be conserved for beneficial use instead of being allowed to run to waste and to work havoc enroute. The resulting benefits of unified and multipurpose development should far outweigh the loss of lands due to submergence. This project can be a success only if it is treated as a regional project. It cannot succeed with a local treatment. The Province of Orissa will fall if it were to take up a project confined to its own boundaries. The same will be the result if an Orissa State is to make similar attempt.

The project being essentially regional, it raises the question of the submergence of State and Provincial Sovereignty to the extent necessary. This is the second question to which I wish to refer. It is for the Orissa State and the Government of the Central Province to come together in order to shed a part of their sovereignty over the different sections of the rivers that are flowing through their territory so that the project could be undertaken, planned, devised and run by a single authority without disturbance either from the Province or from the state once it is established. I need hardly say that the welfare of the public is far more important both to the Government of Orissa as well as to the Rulers of the Indian States affected than sovereignty, and their sovereignty should be used for advancing welfare and not for blocking it. With a cooperative and coordinated effort on the part of the Central Government and the Provincial and State Governments, it should be possible to harness for beneficial use this perennial source of wealth, the waters of the Orissa Rivers which are at present running to waste and causing untold suffering on their way to the sea.

Annexure 1.2

Statistical Data of Hirakud Dam Project

I. General

i) Location of Mahanadi Basin
Longitude: -80°-30' to 84°-50'E Latitude: - 19°-20' to 23°-35' N

ii) Location of Dam: -
Longitude: -83°-52' E Topo sheet
Latitude: - 21°-32' N 64 0/14 & 64 0/15

II. Basin Characteristics:

i) **State- wise catchment area of Mahanadi basin in sq.mile (sq.km)**

State	Drainage Area		% of Total Basin
	Sq.mile	Sq.km	
Chhatisgarh	29021	75136	53.24
Orissa	25349	65628	46.50
Maharashtra	92	238	0.17
Jharakhanda	51	132	0.09
Total	54513	1, 41, 134	100.00

ii) Catchment area upto head of delta in sq.mile (sq.km)
i.e Naraj 51, 024 sq. mile (1,32,100 sq.km)
C.A of delta (Naraj to sea) 3489 sq. mile (9034 sq.km)

Total 54513 sq. mile (1,41,134 sq.km)

iii) Sub-basin wise catchment areas

a) Upstream of Hirakud dam

Details	Sq. mile	Sq. km
Mahanadi upto Saradihi	23, 000	58, 880
Mond up to Tarapur	2, 800	7, 170
Ib up to Deogaon	3, 200	8, 190
Ungauged	3, 200	8, 190
Total	32, 200 sq.mile	83, 365 sq.km

b) Downstream of dam upto Naraj

Dam to Khairmal	13, 024	33, 719
Khairmal to Baramul	3, 200	
Baramul to Naraj	2, 600	15, 016
Grand of total up to Naraj	51, 024	1, 32, 100
C.A of Delta	3, 489	9, 034
Total	54513	1, 41, 134

Total area of Orissa State = 60, 824 sq.mile (1, 55, 710 sq.km)

i) % of C.A of basin in State
to the total area of State 65628 ÷1, 55, 710 =42.15 %

ii) % of C.A of basin in State
to the total area of Basin 65628 ÷ 1, 41, 134 = 46.50 %

III. Hydrology

a) Rainfall in mm	Original	Revised (1958-92)	Revised (1959-2006)
Mean annual	1381.25 (1900-45)	1038(42.83")	1122 mm
Maximum annual	1808.78 (in 1919)	2518 (99.13")	in 1961
Minimum annual	940.31 (in 1902)	607 (23.89")	691.46 (in 1979)
75% dependable	1020 (40.15")	816 (32.12")	

b) Run off in M ham	Original (1900-1945)	Revised
Mean annual	6.17	3.31 (1959-06)
Maximum annual	8.62 (in 1919)	9.09 (in 1961)
Minimum annual	1.84 (in 1941)	1.13 (in 2000)
75% dependable	2.64	----

IV. Civil Structure system

A. Dam & Dykes

a) Length of main dam spanning between Lam dunguri on left & Chandli dunguri on right

i) Left masonry dam = 1640' (500.00m)

ii) Right – do- including 959 ft length of power dam = 2128' (648.78 m)

Total length of masonry dam = 3768' (1148.78 m)

iii) Right Earth dam (Zoned Earth fill) = 7540' (2298.78 m)

iv) Left Earth dam (-do-) = 4440' (1353.66m)

Total length of Earth dam = 11980 (3652.44 m)

Grand total (i+ii+iii+iv) = 15748' (4801.22 m) say = 4800 m = 4.80 km

b) Length of left dykes (zoned earth fill) spanning between hillocks & high grounds

i.	Gap I (earth)	=	5600' (1706.9 m)
ii.	Gap II (masonry)	=	750' (228.6 m)
iii.	Gap III (earth)	=	11500' (3505.2 m)
iv.	Gap IV (earth)	=	12420' (3785.6 m)
v.	Gap V (earth)	=	2000' (609.6 m)

Total length of left dyke - 32270 ft (983,841 m)

c. Length of right dykes (zoned earth fill) = 35290ft (10759 m)
spanning between Chandlidunguri on left & Barapahad on right

d. Total length of dykes (items b + item c) = 67560ft(20597m) say 20.60 km

e. Total length of dam & dykes (4.80 + 20.60) = 25.40 km

B. Maximum height (from deepest foundation level)

a. Power dam = 200' (60.98 m)

b. Right earth dam = 195' (59.45 m)

c. Left earth dam = 90' (27.44 m)

d. Right dyke = 40' (12.19m)

e. Left dykes (5 gaps) = 88' (26.83 m)

C. Top Bank level (TBL) = 642' (195.68m)

D. Reservoir

i. Reservoir spread area at F.R.L 630' =286.87 sq mile (743sqkm)
ii. Water spread area at D.S.L 590' =105.79 sq.mile (274 sqkm)
iii. MWL / FRL =630' (192.02 m)
iv. Crest level of spillway =610' (185.93 m)
v. Dead storage level (DSL) = 590' (179.83 m)
vi. Maximum fetch at FRL =52 mile (83.20 km)
vii. Reservoir storage capacity

Capacity	Unit	Original 195 7	1st survey 76-79	2nd survey 78-83	3rd 83-86	Revised 1988	4th 87-91	CWC 94-95	SPARC 2001-02
a) Gross	m.cum	8105	6934	6626	6615	7189	6210	6141	
b) Live	m.cum	5843	5288	5109	5106	5375	4947	4935	4954
c) Dead	m.cum	2262	1646	1517	1509	1814	1263	1211	

No. of villages submerged – 285 nos (Fully + Partially) (249 in Orissa + 36 in Chhatisgarh

E. Spillways & Power Dam

1. Type of spillway- Soild gravity type with ogee profile & Ski jump bucket
2. Type of P.H -R.R masonry with coursed rubble facing & R.C.C claddings in openings.
3. Length of spillway left = 1640' (499.87 m)
   Right = 2128' (648.61 m)
   Total = 3768' (1148.48 m)
   (This includes 958' long power dam blocks of 48 to 63 & transition blocks)
4. No. of blocks left = 27 (block 1 to 27)
   Right = 36 (block 28 to 63)
5. No of radial gates Left 21 + Right 13 = 34 nos
6. Size of radial gates = 20' x 51' (6.10 m x 15.545 m)
7. No. of under sluice = Left 40 +Right 24 = 64 nos
8. Size of under sluice = 20' – 4" x 12' (6.20 m x 3.66 m)
9. Sill level of under sluice = RL 510' (155.45 m)
10. Spillway design capacity = 15 lakh cusec (42450 cumec)
11. a) Size of inspection / operation gallery in both spillway=11' x 23' (3.35 x 7.01 m)
    b) Floor level of inspection gallery in both spillways = RL 557' (168.66 m)
    c) Size of bulk head gate = 41'.4" x 24'.6" (12.60 m x 7.47 m)
12. a) Size of drainage gallery / foundation gallery (right) = 5' x 7' (1.525 x 2.135 m)
    b) -do- in left spillway (left) =5'x7' (1.525 x 2.135 m) from block 23 to 27
    c) Size of header pipe in left spillway = 3' (0.91 m dia)
    d) Floor level of foundation / drainage gallery =RL 497' (151.48 m)
13. a) Size of low level drainage /foundation gallery in right spillway = 5'x7' (1.525 x 2.135 m)
    b) Floor level = RL 470' (143.25 m)
    c) Length (block 53 &54) = 98.0 ft (29.90 m)
14. Cross gallery in block 54 of right spillway floor level=470' (143.25m)

V. Irrigation & Distribution system

Construction of Hirakud dam commenced in 1948 & completed in 1957. Filling started in 1956. Power generation, irrigation & flood control started from 1957. Full irrigation potential achieved in 1966.

1. Gross command area (GCA) = 6, 17, 00 acre (242915 ha)
2. Design cultivable command area (CCA)

Khariff = 4, 04,100 Ac (159,093 ha)

Rabi = 2, 75, 298 Ac (1, 08,385 ha)

Annual irrigation = 6, 79, 398 Ac. (2, 67,478 ha)

3. Cost per acre of C.C.A at the time of construction =Rs.179.00
4. Design intensity of Irrigation - Khariff = 100 %

As per Project Report - Rabi = 48 %

5. Details of off - taking canals / channel from Hirakud reservoir.

Name of off taking canal	Off taking from	RD (ft)	Design discharge (cusec)/cumec	Sill level (RL in m)	Bed width (m)	F.S.D (m)
Baragarh Main canal	Right dyke	28000	3800 (107.6)	178.20	45.70	2.68
Sason Main canal	Gap IV	35700	630 (17.85)	178.20	16.764	1.49
Sambalpur Distributary	Gap II	484	120 (3.40)	176.36	4.57	1.06

IV. Delta Irrigation (ha) Delta stage II and Stabilisation in stage I - 251000 ha (619970 ac.)



Aerial view of Hirakud Reservoir

Salient Features of Power System

The power system of Hirakud Dam Project consists of:

- Power House no 1 accommodating 7 generating units with ancilliary structure located at the right flank of main dam adjacent to the toe of masonry power dam.
- A power channel of total length 25.45 km leading from P.H No.1 at Hirakud Dam to P.H. No.2 at Chiplima with ancilliary structures.
- Power House No.2 situated at Chiplima with 3 generating units & ancilliary structures.

The details are furnished below: -

A. Civil structures: -

1. Length of Power House (no.1) block including transition blocks

Block no	Penstock No	Width of block (ft)	
48	7	70'	
49	6	70	
50	5	70	
51	4	70	
52	3	66	Sub total = 448'
53	2	63	
54	1	79	
55	--	42	
56	--	58	
57	--	60	
58	--	85	
59	--	35	Sub Total=474'
60	--	40	
61	--	40	
62	--	40	
63	--	74	
		Total	962 ft (293.03 m)

2. Size of Power House no.1 at Burla at the toe of Hirakud Dam = 618' (L) x 132' (W) x 130' (ht.) or 188.25 m x 40.21 m x 39.60 m (ht.)

3. Detail features of Power channel with ancilliary structures are as follows:
   i) Length of the channel = 25.45 Km (83465 ft)
   ii) Length of 1st reach up to Pond-I head regulator = 9.18 Km (30100')
   iii) Length of 2nd reach from Pond-I to Chiplima P.H = 16.27 Km (53365 ft)
   iv) Detail parameters of Power Channel

Sl.no	Parameters	1st reach upto surplus escape	Beyond surplus escape	2nd reach
a	Discharge at RL 520'	18000 cusec (509.77 cumec)	25000 cusec (708.01 cumec)	13000 cusec (368.11cumec)
	at RL 525'	35000 cusec (990.50 cumec		12500 cusec (334 cumec)
b	Bed width	248' (75.5 m)	172' (52.5 m)	274' (83.5 m) unlined 172' (52.5 m) lined 126' (38.4 m) lined (in hilly region)
c	Side slope	1:1	1:1	1:1
d	Bed slope	1:10000	1:10000	1:10000 1:5000 (hilly)
e	F.S.D	23' (7.01 m)	23' (7.01 m)	23' (7.01 m)
f	Lining	Fully lined	Fully lined	Partly lined & partly unlined
g	F.S.L	525' (160.06 m)	525 ft (160.06 m)	525' (160.06m)

v. Average operating head = 67' (20.52 m)

vi. Normal tail water level = 516' (157.3 m)

vii. Minimum TWL = 512' (156.1 m)

viii. Maximum TWL = 525' (160.0 m)

ix. Storage capacity of pond I from RL 516' to 521' ft = 5764 m Ac ft (7.10 Mcum)

x. Surplus escape near P.H. No.1 on left bank of power channel

- Design discharge = 16593 cusecs (470.30 cumecs)
- No. of vents = 5
- Upstream (power channel) FSL = 525 ft (160.06 m)
- T.W.L (Mahanadi river) = 525 ft (160.06 m maximum)

xi. Head regulator at pond I of Power Channel

- Design discharge = 13500 cusec (382.82 cumec)
- No of vents = 6
- Size of vents = 25' x 10' (7.62 m x 3.048 m)
- Sill level = 505' (153.96 m)

xii. Syphon spillway at pond I Power channel

- Design discharge = 20000 cusec (566.41 cumec)
- Priming level = 521.30' (158.84 m)
- Depriming level = 519.50' (158.38 m)
- No of vents = 27 + 2 sluices on right.

xiii. Cross regulator on Power Channel

- RD = 63450 ft (19344.50 m)
- Design discharge = 2300 cusec (65.13 cumec)
- No of vents = 27 + 2 sluices on right
- Size of vent = 16' x8' (4.876 x 2.438 m)
- Sill level = RL 500' (152.44 m)

xiv. Forebay of Power Channel at Chiplima

- Normal forebay level = RL 509' (since may 1963 maintained at RL 507')
- Maximum fore bay level = RL 511' (155.79 m)
- Average tail water level = 435' (132.50 m)
- Maximum tail water level = 460' (140.10 m)

xv. Surplus escape at Chiplima

- Design discharge = 14500 cusec (410.64 cumec)
- Length of over flow section i.e. spillway = 880' (268.22m)
- Sill level = 496' (151.22 m)
- No of vents = 2
- Size of vent = 16' x 8' (4.876 x 2.438 m)

xvi. Actual Power generation from both Burla and Chiplima Power House (1960-61 to 2004-05)

Maximum = 1307 million units (90-91)

Minimum = 541M. units (1975)

Average = 1100 Million units

Power Generation & Transmission System

I) Commissioning dates of Power plant units (hydro- generation sets) of both power houses:

Sl.no	Item	Power house	Date of commissioning	Renovation / Remark
1	**Stage- 1** 37.5 MW (unit-I) Make English Electric U.K	Burla	11.05.1958	Upgraded to 49.50 MW by GEC. Alstom U.K (31.08.1998)
ii	37.5 MW (unit –II) -do-	Burla	11.11.1957	-do- to 49.50 MW by GEC (15.08.98)
iii	24 MW (unit –III) (J.M.Voith-Germany / Siemens)	Burla	18.12.1956	Renovated to 32.18MW(01.01.2006)
iv	24 MW (unit – IV) -do-	Burla	13.05.1957	-do- (01.01.2006)
	Stage-II			
a	37.5 MW (unit-V) (Hitachi-Japan)	Burla	19.04.1962	
b	37.5 MW (unit-VI) (-do-)	Burla	05.08.1963	
C	24 MW (unit-I) JM Voith Germany / Hitachi	Chiplima	15.07.1962	Renovated on 29.7.98 by L & T & BHEL
d	-do- (unit-II) –do-	Chiplima	21.11.1962	Renovated
e	-do- (unit-III), U.S.S.R	Chiplima	01.02.1964	
	Stage-III			
	37.5 MW (unit-VII) Hitachi / Japan)	Burla	13.09.1990	

Total 10 (At Burla & Chiplima) with installed capacity of 347.86 M.W

2. **Details of Burla Power House (P.H. No-1)**

A) Penstocks

- No of Penstocks - 7
- Internal diameter = 7.6 m (25') units I, II, V, VI, VII & 7.0 m(23') units III & IV
- Maximum discharge 166.6 cumec (5884 cusec) unit V,VI & VII &
  94.4 cumec (3334 cusec) unit III, IV & unit I & II
- Maximum velocity = 12' (3.7 m/sec) – Unit V, VI, VII
  = 8.2' (2.5 m/sec) - Unit III, IV, Unit I, II
- No of turbines fed by each penstock = one
- Penstock opening =7 nos x 19' (5.8m)W x 25'(7.62 m) ht. transitioned to circular shape in R.C.C & steel thereafter with dia.23' for 2 nos & 15' for 5 openings.
- Penstock gates – 7 nos x 21'.9" (6.63m) x33' (10.06 m) weighing 70 T each with hydraulic hoist.
- Hydraulic control – 7 nos x R.C.C structures with fabricated steel grill weighing 448 T
- Over head crane – 2 nos x 150 T each
- Draft tube opening: Elbow type transitioned from circular to rectangular & two opening per unit.
- Gates: Two leaves for each opening with dimension of each leaf 25'-6"x9'-0" or 7.77 m x 2.74 m
- Weight of each leaf : 16 tons
- Hoisting – Electronic gantry crane -25T capacity
- Trash rack-R.C.C semi circular opening fitted with gratings –Radius =25'-3" & height = 80'. Each structure fitted with 42 grating 11'-5" x 11'-99" each weightings about 2T. Total tonnage = 600 T.

B. Water turbine: -

- Number - 7
- Type- Vertical –Kaplan (unit 1,2,5,6,7)
  Vertical-Francis (unit 3 & 4)
- Net head = 26.5 m (87') to 35.5m (49') for Kaplan
  31.7 m (104') to 36.6 m (120') for Francis.
- Rated output = 52000 H.P (unit 1,2,5,6,7)
  = 34500 H.P (unit 3 & 4)
- Normal speed = 150 rpm – Kaplan
  138 rpm – Francis
- Run-away speed – 330 rpm – Kaplan
  - 368 rpm - Francis
- Efficiency = 90 % - Kaplan, 88% - Francis
- Weight of runner shaft – 87 T – Kaplan, 43 T – Francis

C. Generator

- Number = 7
- Voltage = 11 KV
- Capacity = 49.5 MW (unit 1 & 2), Umbrella type
  = 37.5 MW (unit 5, 6, 7) ——do———
  = 24.0 MW (unit 3 & 4), Suspended type
- Power Factor = 0.90 lagging
- Speed (normal) = 150 rpm
- Current = 2187 amp. (unit 5,6,7)
  = 1420 amp (unit 3 & 4)
- Phase = 3 Ph 50 cycle
- Weight Stator – 235 T (5nos) + 182 T (2nos)
  Rotor – 285 T (5 nos) + 211 T (2nos)
- Main exciter – 230 V (unit 1,2,5,6,7)
  316 V & 1136 amp (unit 3 & 4)
- Pilot exciter - 230 V & 30 amp (unit 1 & 2)
  - 105 V & 75 amp (unit 3 & 4)
  - 110 V & 136 amp (unit 5, 6, 7)

D. Step - up transformer

- Voltage 11 / 132 KV
- Capacity 42 MVA (unit 3,4,5,6,7) by Hitachi, Japan
  60 MVA (unit 1 & 2)
- Phase 3 phase
- Stepdown transformers
- Capacity
  - 12.5 MVA – 132 KV /66 KV – 2 no
  - 5.0 MVA – 66 KV / 11 KV – 1 no
  - 2.0 MVA – 66 KV / 11 KV – 1 no
  - 3.0 MVA – 66 KV / 11 KV – 1 no
  - 0.6 MVA – 11 KV / 400 V – 2 no
  - 0.2 MVA – 11 KV / 400 V – 4 no

	132 KV	66 KV	11 KV
No	17	6	8
Type	Bulkoil	Bulkoil	Bulkoil
Rating	400 amp	400 amp	400 amp
Rupture capacity	2500 amp	500 amp	75 MVA
Closing	Pneumatic	Solenoid	Manual

Fire protection:

Generator - Carbon - Monoxide

Transformers & circuits Breakers - Automatic water supply

E. Details of Power House No 1

- Dimension = 618' x 132' x 130 (188.25 m x 40.21 m x 39.60 m)
- Quantities
  - i. RCC = 16.69 lakh cft (0.47 lakh cum)
  - ii. R.R Masonry = 0.75 lakh cft (0.02 lakh cum)
  - iii. CR.Masonry = 0.18 lakh cft (0.005 lakh cum)
  - iv. Reinforcement = 2916 Tons
- Cost = Rs.82.47 lakh

F. Transmission lines: - (10 no 132 KV + 4 no 66 KV) in route km.

To Hirakud aluminium factory	2 circuits 132 KV	6.28 km
To Chiplima	2 circuits 132 KV	22.40 km
To Jharsuguda	2 circuits 132 KV	50.60 km
To Rourkela via Rajganggur	1 circuit 132 KV	157.30 km
To Chowdar	1 circuit 132 KV	256.00km
To Talchem	1 circuit 132 KV	180.00 km
To Rourkela - Joda	1 circuit 132 KV	82.00 km
To Baragarh	1 circuit 66 KV	32.79 km
To Hirakud	1 circuit 66 KV	6.00 km
Joda to Kuldiha	1 circuit 66 KV	94.00 km
Joda to Koira	1 circuit 66 KV	32.00 km

G. Miscellaneous:

- Lighting - Tube lights & MV lamp
- Dewatering – No of pumps - 6
  Rated total output - 430 HP
  Pumping capacity - 11000 gpm
- Air conditioner - control room – Refrigerative cooling
- Turbine & generator - Evaporative cooling
- Communication – Inter telephonic system
- Overhead cranes –
  At P.H Burla - Two overhead traveling cranes each of 150 T capacity and with a lifting beam connecting the two cranes are designed to lift composite load of 300 T. (Erected in Sept.1955)
- Sub-stations:
  - i. Chainpal on 132 KV line from Burla PH to Choudwar
  - ii. Choudwar
  - iii. Jharsuguda on 132 KV line from Burla PH to Rourkela

iv. Rourkela		
v. Aluminium factory at Hirakud		on 132 KV line from Burla PH to Hirakud
vi. Joda		on 132 KV line from Rourkela to Joda
vii. Naomundi		on 66 KV line from Joda to Kuldiha
viii. Kuldiha		
ix. Koira		on 66 KV line from Joda to Koira
x. Baragarh		on 66 KV line from Burla PH to Baragarh

3. **Details of Chiplima Power House**

a) Penstock & Scroll case (R.C.C –Unlined)

• Number	-	3
• Internal diameter	-	9.40 m(30.83 ft)
• Maximum discharge	-	136 cumecs (4803.5 cusec)
• Maximum velocity	-	2.1 m /sec (6.9 ft / sec)
• No of turbine fed by each penstock	-	One
• Size of gate	-	21' x 31' (6.40 m x 9.45 m)
• Size of trash rack	-	28' x 45 '(8.54 m x 13.72 m)
• Over head crane (travelling)	-	250 T
• Forebay gantry crane	-	75 T
• Draft tube gantry crane	-	25 T
• Size of draft tube gate	-	25'-6" x 12' (7.77 x 3.66 m)
• Penstock & scroll case –RCC- Unlined		

b) Water Turbine

• Number	-	3
• Type	-	Kaplan
• Net head	-	22 m (72.0 ft)
• Rated output	-	34100 H.P
• Normal speed	-	150 rpm
• Make	-	J.M. Voith – West German (Unit 1 & 2) USSR – (Unit 3)

c) Generator:

• Number	-	3
• Voltage	-	11 KV
• Capacity	-	24 MW
• Power factor	-	0.90 lagging
• Speed	-	150 rpm
• Current	-	1420 amp
• Type	-	Umbrella (units 1 & 2) Suspended (unit 3)
• Main excitor	-	220 V / 1000 amp (unit 1 & 2) 185 V / (950 amp) (unit 3)
• Pilot excitor	-	110 V / 130 amp (unit 1 & 2) 230 V / 65 amp (unit -3)

d) Step-up transformer

• No	-	3
• Make	-	Lepper, West Germany
• Type	-	K.D.O
• Voltage	-	11 / 132 KV

- Capacity - 27 MVA (unit 1,2&3)
- Phase - 3
- % of impedence - 9.8%

e) Transmission lines

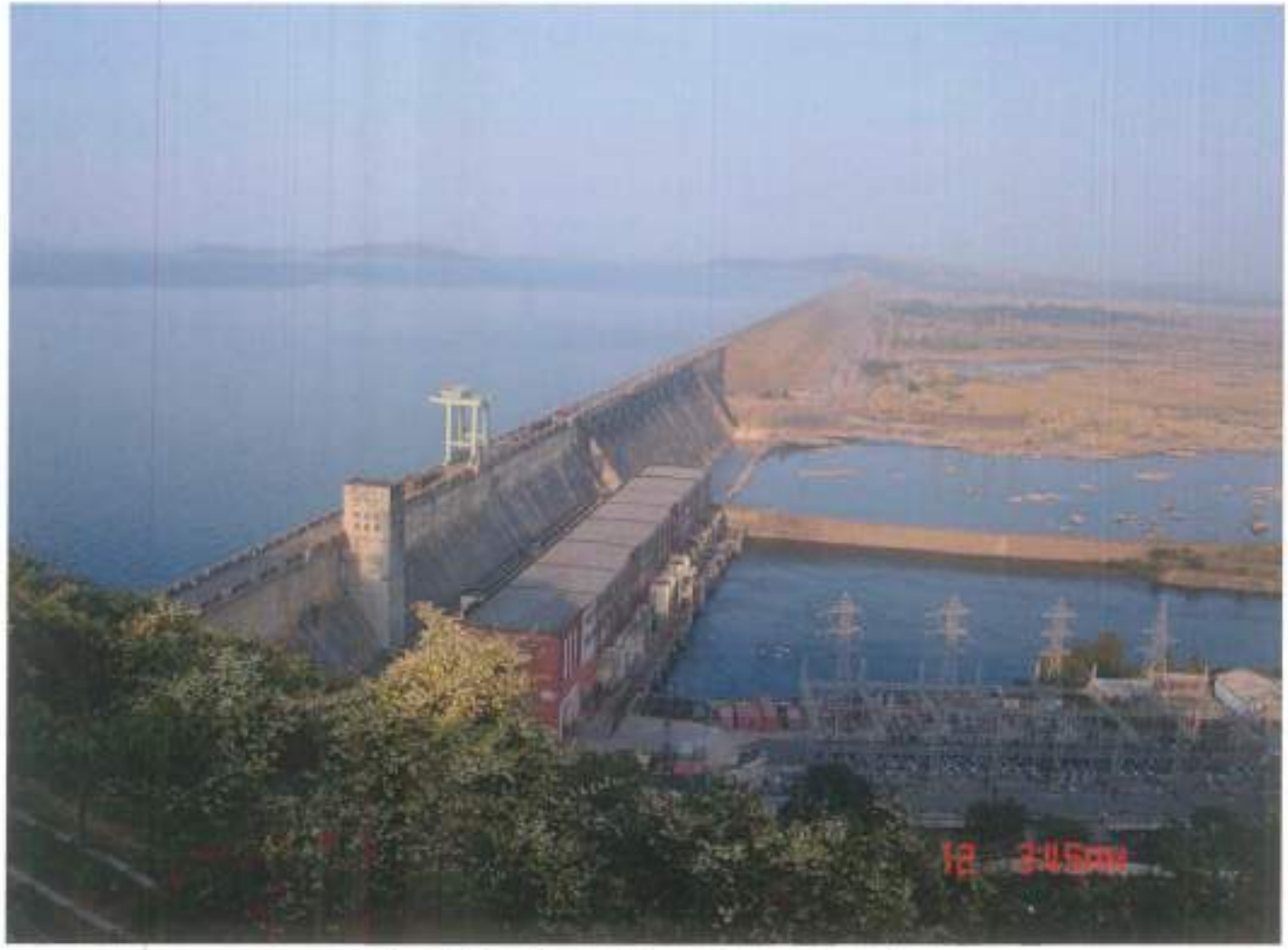
- 3 nos of 132 KV feeders
- 3 nos of 33 KV feeders
- 3 nos of 11 KV feeders

N.B: - The potential of firm generation at Hirakud power system at 100% L.F is

- Burla (P.H no 1) - 77 M.W
- Chiplima (P.H no 2) - 56 M.W

Total - 133 MW

f) Power house size - 242' x 67' (73.78 m x 20.43m)



View of Burla Power House, Power Dam and
part of Power Channel.

CHAPTER - II

PROJECT INVESTIGATION, PLANNING AND FOUNDATION FEATURES

2.0 Introduction:

The problems of construction of Water Resources Development Projects are unique in nature. Not only construction covers such varied end products like dams, power stations, tunnels, canals and flood embankment etc, but even when the end products remaining same, no two jobs are found alike. Therefore, construction planning, plant layout and equipments requirement may also differ widely from place to place depending on the topographical features. Topography, geology, climate, availability of construction materials, power, fuel, water and manpower influence the type of the structure, its magnitude, construction methodology and construction machineries etc. Transport and communication also play an important role.

The project planning initially contemplates to meet specific needs or objectives. This undergoes several modifications during detailed project formulation period and even during construction stage for optimizing the potential and to derive the maximum benefit. At the outset, reconnaissance survey and feasibility study are made basing on the available data, records and topo maps etc. If projects become techno-economically feasible, then detailed investigation is carried out and project report prepared.

2.1 Investigation

Usually for a major project of such magnitude topographic survey, geological exploration, soil survey, collection of hydrological and meteorological data, sedimentation study, water quality, drainage details are carried out. For all the three proposed dams i.e. Hirakud, Tikerapara and Naraj investigations were undertaken. In case of Hirakud extensive work has been done on hydrology, geology, topographical survey, surveys of material and machineries required for construction, soil surveys for the command areas and mineral deposits, inside the reservoir area.

Gauge, discharge, silt and meteorological observations were regularly taken in the project area and the data so collected were analysed and correlated.

The survey of the dam site and reservoir was completed by the Survey of India. Topographical surveys for 3000 sq miles for the purposes of irrigation and other works were carried out on a scale of 4" = 1 mile and the sheets printed. The entire work of topographical and contour survey of the dam sites, reservoir area and command area were done by the Survey of India by a combination of aerial and ground survey methods. The detailed surveys of the dam sites as well as the survey and location of the irrigation and drainage channels, communications etc. were carried out by the staff of CWI & NC. The Survey works for the Mahanadi Valley Development scheme allotted to Survey of India and the progress of work done by them are given in Table No-2.1.

Table No 2.1 Progress of Survey Work

Task	Scale	Survey upto RL (ft)	Contour interval (ft.)	Total area in sq. miles	Stage of Photography	Date of commencement of survey	Estimated date of completion of survey
1	2	3	4	5	6	7	8
Hirakud Dam	16"=1 mile	0.5 mile N & S of dam & upto ridge on E & W	10	7.8	completed	1.4.46	completed
Hirakud reservoir	4"=1 mile	650 ft	10	400	In complete	1.3.46	15.6.47
Command area of Hirakud Project	4"=1 mile	.....	1	2155	Complete expect for small extension	Nov.46	March 49

Number of rain gauges have been fixed in the catchment area. Discharge and silt observations have been carried out daily since 1946-47 at number of sites.

2.1.1 Geological Investigation:

Based on preliminary geological investigation conducted in the year 1945-46, the site at Hirakud was found to be quite sound for the construction of a dam. The foundation conditions at Hirakud dam presented many interesting engineering and construction problems which required careful investigation, study and proper treatment in order to consolidate the foundation and make them water-tight thereby reducing the hydrostatic uplift pressure acting on the base of the dam. The techniques of foundation treatment were modified to suit varying conditions encountered in the different parts of the foundation. Subsequently, during the years 1947 to 1950 extensive core-drilling, tunnelling and test-pitting have been carried out on the project.

The total footage of diamond core-drilling completed in the investigation of various features of the Project comes to 12122 ft, including 15 angular drill holes. Two tunnels, one on each abutment at the main dam site, totalling 270 ft in length was excavated while the footage of open test-pits completed at the main and subsidiary dam sites aggregated roughly to 1500 ft.

Other aspect of the geological investigations on the project comprised regional geological surveys of the dam site and vicinity on the scales of 4 inches and 16 inches to 1 mile, detailed study of possible sources of concrete aggregate for construction of the dam and large scale mapping of the rock-outcrops near the head reach of the various alignment of the Power Channel.

The Engineering Geology section of the Geological Survey of India under Dr. J.B.Auden, has been connected from the very beginning with all the exploratory work, from the preliminary to detailed investigation of the Hirakud main dam site and the subsidiary dam site, which is located 17 miles further downstream. Subsidiary dam proposal was subsequently dropped.

During the investigation stage, exploratory holes were drilled for the entire length of dam both along the axis of dam as well as along the toe of dam. The holes along the axis were 200 ft apart in the Concrete Dam Section and 500 ft apart in the Earth Dam Section. Those on the downstream toe were 500 ft apart in the Concrete Dam and 1,000 ft in Earth Dam Section, respectively. These gave a general indication of the probable levels of hard rock on which all the preliminary estimates were based. During the excavation stage, additional exploratory holes were drilled at closer intervals (about 3 to 4 per block) after the top overburden had been removed, in order to obtain accurate data for determining the

final level of excavation and location and extent of defective zones in the foundation which would require special treatment. Detailed investigation of the defective zones was done with the help of Diamond Drill Core holes before deciding on the method of treatment for each particular zone. Altogether, a total of 240 holes with a total footage of nearly 12, 122 ft were drilled, the depth varying from 50 to 60 ft below hard rock level. All these holes were subsequently used as grout holes. The size of holes varied from 1-7/8" to 3" diameter. The actual number of grout holes, their spacing, depth and inclination as well as the order of drilling and grouting and the pressures to be used for the injection of grout into the holes were decided from the nature of rocks as revealed by observations and core drilling explorations, the result of water pressure tests and the results of pressure washing and grouting operations. The grouting designs were modified from time to time to meet new requirements as determined in the field by the actual work in progress.

The procedures and methods employed as well as the equipments used were different for (a) concrete & masonry dam and (b) for earth dam section which have been described under Sec. 2.1.3.1.1.

2.1.2 Hydrology:

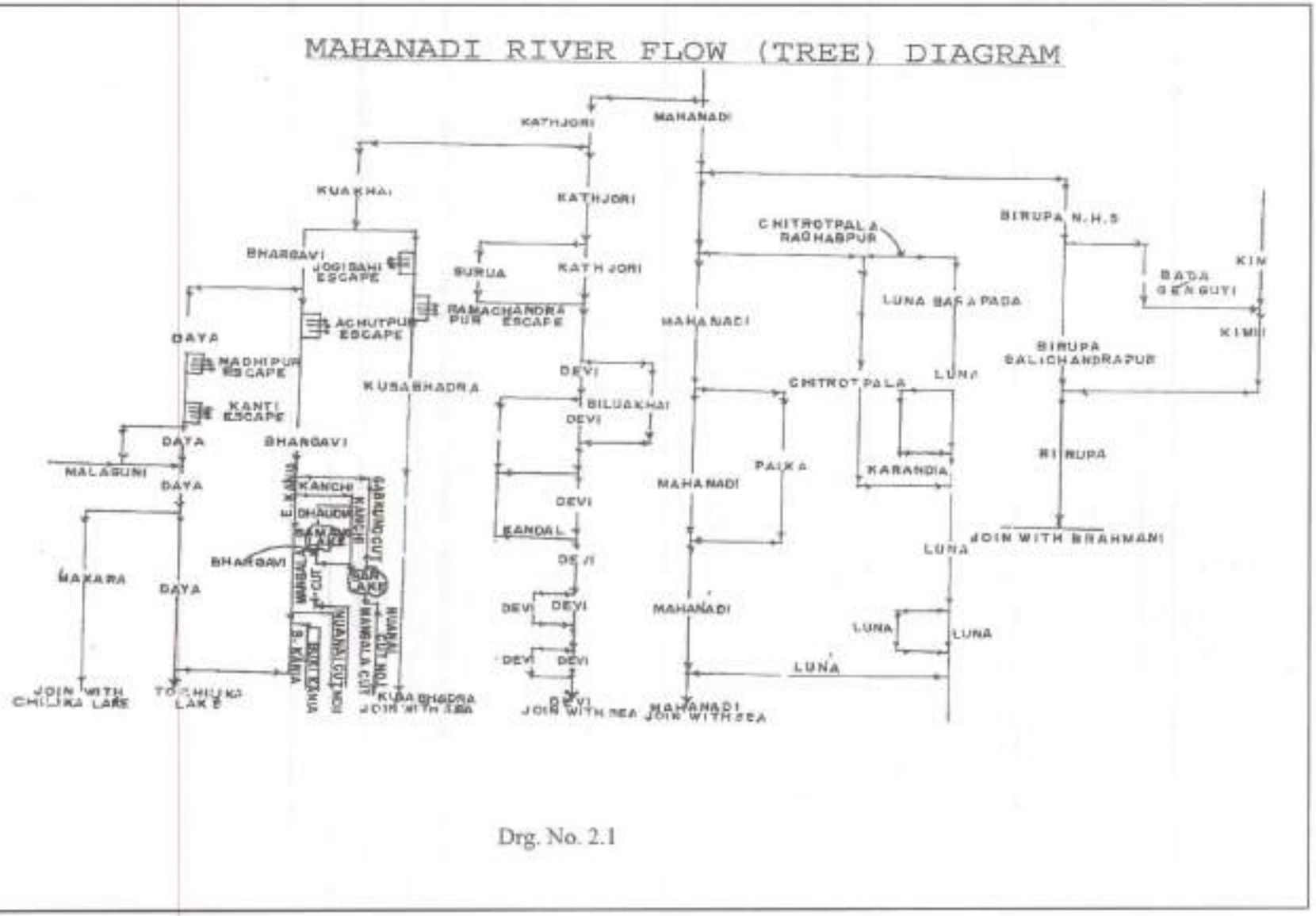
The Mahanadi Basin covers large area in Chhatisgarh and Orissa besides some small areas in Maharashtra and Jharkhanda, the details of which has been given in Table 1.1. It is bounded on the north by Central India hills, on the south and east by Eastern ghats and on the west by the Maikela range, lying in the north east of Decan Plateau. The basin is situated between 80° 39' and 84° 50' east longitude and 19° 20' and 23° 35' north latitude. The upper portion of the basin is a square shaped plateau known as Chhatisgarh. The basin is circular in shape with a diameter of about 400 km and an exit passage of about 160 km in length and 60 km breadth. The Mahanadi basin drains an area 1,41,134 sqkm (54513 sq miles) most of which lies in the States of Chhatisgarh and Orissa. The basin can be broadly divided into three distinct zones, the upland plateau, the Central hill part flanked by eastern ghats and deltaic area. Hirakud Dam across the river is located in the second zone of the catchment.

The Mahanadi rises in a pool, 6 km from Pharsiya village near Naori town in Raipur district of Chhistagarh at an elevation of 442 m above m.s.l. The total length of the east following river from its origin to its outfall into the Bay of Bengal is 851 km. After a run of about 375 km. through the districts of Raipur and Bilaspur in erstwhile Madhya Pradesh where it receives water from its major tributaries - the Seonath and Jonk, the river reaches Sambalpur town from where it takes a southern course upto Sonepur. There the tributary 'Tel' joins the river. The river flows in an easterly direction thereafter up to Naraj, the head of delta. Catchment upto Naraj is about 1,32,100 sq.km (51,000 sq.miles). Below Naraj the river divides into several branches before discharging into Bay of Bengal. Mahanadi river flow (Tree) diagram in delta is shown in Drg. No.2.1.

The Principal tributaries of the river upstream of Hirakud are Seonath, Hasdeo, Jonk, Mond, Bheden and Ib while Tel and Ong join at downstream of Hirakud.

The river passes through tropical zone and is subjected to cyclonic storms and seasonal rainfall. The cyclonic storm usually originates in the Bay of Bengal and travels generally in North-Westerly direction opposite to the flow of the river Mahanadi. Heavy rainfall first occurs in delta and then spreads inland covering in about two days. The catchment receives 75% to 90% of the total annual rainfall during south-west monsoon starting from the 3rd week of June to the end of October. The maximum precipitation is usually observed in the month of July, August and first half of September. Flood occurs when heavy rainfall continues for a few days in the catchment because of storm/depression etc.

The average rainfall in the Mahanadi basin is about 1420 mm. Owing to the topographical characteristic, the climate is variable. In the winter the mean daily minimum temperature varies from 7° C to 12.8° C. Month of May is the hottest month, in which the mean daily maximum temperature varies from 42.9° C to 45.5° C.


MAHANADI RIVER FLOW (TREE) DIAGRAM
KATHJORI
MAHANADI
KUAKHAI
KATHJORI
BHARGAVI
JOGISAHI ESCAPE
SURUA
KATHJORI
ACHUTPUR ESCAPE
RAMACHANDRA PUR ESCAPE
DAYA
MADHIPUR ESCAPE
KUSABHADRA
DEVI
KANTI ESCAPE
BILUAKHAI
DEVI
DAYA
BHARGAVI
MALAGUNI
DAYA
KANCHI
DHAUDH
BHARGAVI
MAKARA
DAYA
DEVI
DEVI
DEVI
DEVI
JOIN WITH CHILIKA LAKE
TO CHILIKA LAKE
KUAKHAIA JOIN WITH SEA
DEVI JOIN WITH SEA
MAHANADI JOIN WITH SEA
CHITROTPALA RAGHABPUR
BIRUPA N.H.5
MAHANADI
LUNA BARAPADA
BADA GENGUTI
KIM
KIMI
BIRUPA BALICHANDRAPUR
CHITROTPALA
LUNA
PAIKA
KARANDIA
BIRUPA
MAHANADI
JOIN WITH BRAHMANI
LUNA
MAHANADI
LUNA
LUNA
LUNA

Drg. No. 2.1

An enormous volume of water is brought into the delta by the river channel, and most of the inundations are caused by high floods in the rivers. In fact the total precipitation in the catchment area is nearly 30 times as large as the rainfall in the delta. A knowledge of the rainfall in each catchment is therefore of far greater importance in connection with floods than the rainfall at individual stations or in each administrative district. This aspect has been studied by Mahalanabis separately for catchments of major rivers with the help of the survey maps published by Survey of India in 1926 in a scale of 1:10, 00, 000 (i.e. 1 inch = 15.78 miles).

2.1.2.1 The Catchment of Mahanadi

The whole catchment has been subdivided into five sections i.e. M-I, M-II, M-III, M-IV and M-V to bring out the peculiar rainfall characteristics of each region.

Section V (M-V) lies between 80° 26' E. (at 20° 42' N) and 82° 33' E (at 22° 19' N) and between 22° 49' N (at 81° 55' E) and 20° 29' N (at 81° 1' E).

Section V is the area drained by the Seonath before its junction with the Mahanadi at Changori. The Seonath has its source in a region in the neighborhood and during its course is joined by a large number of tributaries. The Tendula and the Pathra join it a little above Durg. Further down it receives the Sambarsa, the Amner which drains Khairgarh and Chuikhadan, another tributary draining in Gondai the Kanrooah draining Bemetara, the Kharun draining Raipur and the Kulhan above Simga. It is next joined by the Hamp draining Kawardha. Pandaria draining Pondi-Lapha; the Agar draining Mungeli;the Maniari; the Arpa draining Pendra Bilaspur; and finally the Lilagar just before its confluence with the Mahanadi.

Section IV lies between 81° 1'E (20° 29' N) and 82° 48' E (21° 25'N) and between 21° 44'N (82° 32' E) and 19° 56' N (82° 5' E). It comprises the area on the right of the Mahanadi, the drainage water of which falls into the river between Changori (21° 44' N), (80° 46' E) and Seorinarayan (21° 44' N, 82° 35' E).

The Mahanadi is formed by a large number of small streams one of which drains Nawapara and another Kanker. It then passes through the area covered by Dhamtari, Kurid, and Rajim where the Pairi which drains Gariabund falls into it. Passing the region of Mahasamund, Arang and Kusrangi, the Mahanadi unites with the Seonath at Changori and the united river assumes the name of Mahanadi. About 10 miles further down it is joined by a large stream, the Jonk which drains Pithora.

Section III lies between 82° 4' E (22° 40' N) and 84° 48' E (21° 40' N) and between 23° 35' N (82° 28' E) and 21° 17' N (83° 8' E). It is a large area on the left (north) of the river, the drainage water from which reaches the Mahanadi between Seorinarayan and Sambalpur. Important tributaries are the Hasdeo which drains Janjgir, Korba, Katghora and Pasan, the Sone draining Sakti: a tributary from the south from near Sarangarh, the Mand on the north from the region of Dharmajaygarh and Baikunthapur; the Kelo draining Raigarh; a small tributary near Ambabhona and another draining the area on the north east near Gangpur and Jharsuguda.

Section II lies between 82° 3' E (19° 45' N) and 84° 23' E (21° 29' N) and between 21° 34' N (83° 30' E) and 19° 18' N (82° 44' E). It is the area lying practically wholly on the right (south) of the river and sending its water into the Mahanadi between Sambalpur and Sonepur. A small stream draining Baragarh joins a little below Sambalpur, and Ong which drains the region covered by Sartaipali, Padampur and Bijepur, joins a little later. The river next passes Dhama on the left, and Binka on the right and is joined by its largest tributary, the Tel, at Sonepur. The Tel is formed by the confluence of a number of small streams two of which drain the region of Deobhog and Bhawanipatna and is fed by the Barabhailat, and the Suktel which drains Bolangir and a number of other streams.

Sections I lies between 83° 48' E (20° 34' N) and 85° 42' E (20° 23' N) and between 21° 19' N (84° 13' E) and 19° 54' N (82° 3' E). It is the area drained by the Mahanadi between Sonepur and

CATCHMENT MAP Drg. No. 2.2



Naraj, 7 miles above Cuttack. Balandapara stands on a south tributary; Udaigiri and Phulbani are drained by the Salki which joins the Mahanadi from the right; Rampur stand on a left tributary; Kunjabangarh, Nayagarh and Bolgarh are drained by the Kuanria on the south. The rainfall stations Boud and Banki stand on the right (south) bank of the river and Tikerapara, Narshingpur, Baramba and Athagarh on the left.

The above division into sections is convenient in many ways, water from the near sections M-I and M-II reach Naraj practically within a short time, but there is an appreciable lag for the other sections, so that heavy rainfall in M-IV or M-V produce their maximum effect at Naraj after a considerable time. For a detailed study of the floods it is necessary to take this lag into consideration. Mr. J.Shaw, Special Officer, Floods, in his type-written note, dated 8th March 1938 states: 'As 36 hours (at 5 miles per hour) is the quickest for the Sambalpur gauge water to reach Cuttack and the longest period is about 53 hour (depending on the state of river between these points) I should say it would take about 36 hours for water from M-II to reach Cuttack. From Sonepur at the rate of 5 miles per hour it would take 26 hour. (Source: Mahalanabis Report, I & P Department, Govt. of Orissa Pg.44-46). Catchment area map is shown in Drg. No.2.2

2.1.2.2 Design Flood:

The original project report "Mahanadi valley development Hirakud Dam Project (June 1947) contemplated for construction of three Dam Projects i.e. Hirakud, Tikerpara and Naraj each one with its own canal system and hydroelectric power installation for unified development of Mahanadi valley.

The hydrological study was based on work of Prof. P.C.Mahalanobis entitled " Rain storm and river floods in Orissa". There were 39 stations in Mahanadi catchment out of which 23 were above Sambalpur.

The mean annual rainfall over Mahanadi above Sambalpur for period 1900 to 1945 was calculated as 54.38 inches (1381 mm) the minimum and maximum being 37.02 inches (940 mm) and 71.21 inches (1809 mm) in year 1902 and 1919 respectively. The mean, minimum and maximum runoff for period 1926-46 were computed as 50.00, 24.56 (1941) and 63.88 M.Ac. ft. (1936) respectively. For period 1900-46 the minimum and maximum runoff were respectively 20.61 and 69.90 M.Ac.ft. in 1902 and 1919.

The magnitude of maximum flood at Naraj was assumed to be 15.7 lakh cusec (44,459 cumecs) calculated for 1834 flood gauge based on Rhind curve. In absence of direct evidence of recorded flood at Sambalpur, maximum flood of 11.1 lakh cusec was computed indirectly, in proportion to catchment area, from maximum flood at Naraj. But for flood moderation study peak discharge of 9.42 lakhs cusec at Sambalpur and 15.7 lakh cusec at Naraj were adopted. No reason in support of reduction of flood value at Sambalpur was furnished.

Hirakud has been described by many experts as a hydrologically deficient Dam. In the project report 1947, a flood reserve of 4345 Mcum (3.51 M Acft) was contemplated between RL 600 to 630.5 ft though full reservoir level was RL 625 ft. The provision envisaged to regulate a flood of the magnitude as in 1834 AD, (Peak of 44, 459 cumec or 15, 71, 000 cusec at Naraj) within gauge of 27.432 m (90.00 ft) at Naraj with maximum flood flow of 34, 356 cumec (12, 14,000 cusec) as per Rhinds curve. For flood moderation studies flow of 26568 cumec (942000 cusec) was considered. Spillway was planned with flood release capacity of 31, 413 cumec (11, 10, 000 cusec). The regulated release from Hirakud dam was envisaged not to exceed 19, 810 cumec (7, 00, 000 cusec). This was expected to lower the maximum flood level at Sambalpur by 1.46 m (4.8 ft). The flood space was considered to be reserved upto 24th September, beyond which it could be safely filledup for power and irrigation use. It was mentioned that "there is always more than enough water in the river in late September and throughout October to fill the reservoir even in the worst year".

Dr. A.N.Khosla in 1948 analysed in detail the moderation of floods of different frequencies. Required flood reserve, reservoir level, gauge at Naraj for floods of different return periods are given in Table 2.2.

Table 2.2 Moderation of floods of different frequencies (1948 Advisory Committee Report)

Sl.No	Details	100 year flood	1000 year flood	10,000 year flood
1	Computed unregulated flood flow at Naraj in cusec	15, 70, 000	16, 50, 000	17,00,000
2	Flood inflow at Hirakud in cusec	11, 20, 000	11, 70, 000	12,05,000
3	Regulated outflow at Hirakud in cusec	7, 40, 000	7, 70, 000	8,30,000
4	Reservoir water level (ft)	630	630	630
5	Required flood reserve (M.Ac.ft.)	4.51	4.52	4.49
6	Silt reserve (M.Ac.ft.)	2.24	2.24	2.24
7	Total capacity (M.Ac.ft.)	6.75	6.76	6.73
8	Gauge at Naraj (ft.) corresponding to regulated release	89.8	90.3	90.8

In the revised project report (1953) total 105 rain gauge stations, out of which 66 nos. in catchment above Sambalpur were considered. Annual mean, maximum and minimum rainfall over the whole catchment were calculated as 1369 mm (53.9 inches), 1804 mm (71.03 inches) in 1919 and 889 mm (34.99 inches) in 1902 respectively considering 61 years rainfall series from 1891 to 1952. The annual mean, maximum and minimum ranfall over basin upto delta head is calculated as 1402 mm (55.18 inches), 1794 mm (70.65 inches) in 1933 and 1020 mm (40.17 inches) in 1899 considering 60 years rainfall series for 1891 to 1950.

Along river Mahanadi there were gauge observations at Naraj (since 1868), at Banki and Sambalpur (since 1921). But there was no discharge observation prior to 1946. Since July 1946 discharges were observed at Kaimundi (8 miles upstream of Naraj) and at Sambalpur.

In April 1951, discharge observation site at Sambalpur was shifted to Mahanadi bridge site downstream of dam site. Based on the observed, daily gauge and discharge at Sambalpur since 1946 (upto 1948), separate gauge discharge curves were developed for monsoon and non-monsoon period. Gauges were P.W.D gauge and discharges as measured by C.W & P.C during the period. The discharge series for monsoon period were derived from observed P.W.D gauge since 1926 and the gauge discharge curve for period 1946-48.

For non-monsoon period, as no data whatsoever existed at Sambalpur prior to 1946, a correlation between observed gauges at Sambalpur since 1946 and corresponding gauges at Naraj were established considering the time lag. With this corelation graph, all the Naraj gauges were converted to Sambalpur gauges and discharges corresponding to these deduced Sambalpur gauges were read off from non-monsoon rating curve for Sambalpur (1946-48) referred above. From runoff series of 27 years, 1926 to 1952 (out of which 1926 to June 1946 are deduced from developed stage discharge curve and rest actually observed), the mean annual runoff is calculated as 39.3 M.Ac.ft., highest of 53.69 M.Ac.ft. in 1936 and minimum 14.90 M.Ac.ft. in 1941. But in 1961, the actual runoff is 71 M Ac.Ft.

The non-monsoon runoff varied from 8.061 M.Ac.ft. in 1935-36 to minimum 1.339 M.Ac.ft in 1941-42, average being 3.21 M.Ac.ft. Considering the period of actual observation (1946 to 52) dependable non-monsoon yield of 2.32 M.Ac.ft was taken for reservoir planning.

In the revised report of Hirakud Dam Project (1952), the F.R.L and M.W.L was fixed at R.L 630 ft. with live storage of 4.51 M.Acft between RL 590 to 630 ft. and gross storage of 6.75 M.Acft. On preliminary assessment, the maximum flood peak was estimated 51, 789 cumec (18, 30,000 cusec) with volume of 16.9 inches of runoff (29.02 M.Ac.ft) based on envelop curve of six major rivers of world, three from U.S.A, one from Philippines and two Indian rivers, i.e. Mahanadi (at Mundali) and Godavari in India. Subsequently revised design flood was computed to be 42, 450 cumec (15, 00, 000 cusec) and volume to be 19.92 M.Ac.ft equivalent to 11.6" run off from the basin, from envelop curve considering 75 rivers of the World. As per frequency analysis this has a return period for 500 years only.

The maximum flood of 15 lakh cusecs on 500 years return period basis was considered adequately high flood and was adopted as the ***design flood*** *for Hirakud Dam.* The spillway was designed with gross capacity of 42,450 cumec (15, 00, 000 cusec) and with allowance of 1/6th of the gates inoperative. Effective discharge was calculated to be 35, 177 cumec (12, 43, 000 cusec). Full live storage capacity of 4.51 M.Ac.ft was reserved for flood absorption. The reservoir was to be maintained at D.S.L 590 ft. upto end of August. Reservoir filling was to be started around beginning of September and filled upto RL 620 ft. by 15th September, further filling the Reservoir to F.R.L by end of October.

Floods upto discharge of 40, 752 cumec (14, 40, 000 cusec) and gauge 91. 70 ft. at Naraj could be limited within safe gauge of 89 ft. with a discharge of 28, 300 cumec (10, 00, 000 cusec) by routing in Hirakud reservoir. The flood of 1833 with gauge of 91.45 ft. at Naraj, while routing required full flood reserve of 4.51 M.Ac.ft still reaching a discharge of 10, 50, 000 cusec at Naraj. The higher floods of 1834, 1855, 1872 and 1896 with gauges at Naraj 93. 60 ft., 93.36 ft., 92.10 ft. and 92.1 ft. respectively were not routed.

In the Manual of Reservoir operation (1959) it was recommended to keep the reservoir at D.S.L of R.L 590 ft. upto 1st September in normal years and to fill upto F.R.L of 630 ft. by end of September. In case of lean years the filling operation could commence from middle of August. However the reservoir is not to be raised above R.L 610 ft. till 31st August. It was expected to reduce all flood peaks to 6, 50, 000 cusec or below at Hirakud.

2.1.3 Regional Geology

The dam site and most of the reservoir basin are underlain by the Archaean formations, which include much granite gneiss intrusive into a basement series of para gneiss, schist, quartzite, phyllite etc. It is suspected that among these intrusions there may also be granites of an age younger than the Archaeans.

The predominant foliation strike of the crystalline reflected in the trend of the more resistant ridges is NW-SE, the dip is variable, but is generally very steep, either towards the north-east or south-west.

The next younger formations of the area are the Cuddapahs, consisting of sedimentary rock viz., quartzite, shales, limestone etc, which form the high Barapahar hill ranges on the western margin of the reservoir. An isolated linear ridge (Chandil Dungri) madeup of the lowest members of the Cuddpah sequence of these hills, forms the right abutment of the dam. The Cuddapah shales and limestones of the Barapahar hills strike across the Mahanadi river arm of the Hirakud reservoir on the north-western side.

In the eastern, north-eastern and south-eastern portions of the reservoir, Gondwana formations are noted, consisting primarily of Talcher shales and sandstones, while along the Ib river arm at the head of the reservoir, a section of Barakar sand -stone and coal measures crop out.

Recent laterite and flood plain deposit are the other formation units observed in the area, covering the older rocks; the laterite are found mainly over the Gondwana rocks.

2.1.3.1 Geology of Concrete & Masonry Dam :

In the left Concrete Dam the foundations consist of granitic rocks with bands of schistose rocks, mainly quartz silicate schists and chlorite schists. There were joints, parallel to the strike and dip of the rocks (N.N.W.-S.S.E. and E.N.E.-W.S.W. respectively) in the north eastern side (Block 1 to 19) while basal joints were more conspicuous over the rest of the area (Block 24 to 27). The strike of the foundations was at right angles to the axis, while the dip was about 75° to 80° to horizontal. Excavation was carried out to a depth of 10' to 15' below the bed level, for founding the concrete structure on sound and fresh rock. Deeper trenches were dug out along the chlorite schist bands, where weathering extended to greater depth. The presence of a 20 ft. wide shear zone in Block 5 necessitated trenching to a depth of about 25 ft. below the original level, along this zone for the entire width of the Dam.

In the Right Concrete Dam (Right Spillway and Power Dam Sections), the foundations consist of medium grained granitic gneiss, mylonites, slate and shales, arkoses and quartzites intersected by numerous joints, fissures, faults, shear zones and crushed zones. The right abutment (Block 45 to 63) rests mostly on shales, arkoses and quartzites. The shales varied in colour from light brown to grey, and were very hard and compact. The slates were grey to black. The quartzites were light brown to pale yellow.

The mylonite rocks were seen to lie over the Cuddapahs in the foundations of Block 53 and the contact between the two was characterised by a narrow crushed zone, about a foot in width. The mylonites formed the transition between the cuddapahs and the granitic rocks in the Power Dam Section. The mylonites were light greenish in colour near the Cuddapah contact but became gradually grey as they approached the granites.

From Block 52 to 28, the rocks were mainly granite gneiss but with bands of epidiorite and quartz of varying thicknesses. In Blocks 38, 39, 40,there was 40 ft. wide band of epidiorite cut across the granites in the north-south direction. The rock was fine grained, greyish green to grey in colour. The rock was highly sheared and fractured. In the Block 32 and 33, the presence of a 15 ft. wide epidiorite band in a highly fractured state necessitated slightly deeper excavation touching R.L 490 ft at the final foundation level.

The principal set of faults run roughly along E.N.E-W.S.W. while the other set varied in its trend from N-South to N.N.W to S.S.E. Both the sets had nearly vertical or steep dips. There were a number of shear zones remaining parallel to the water course. As the rocks were deeply weathered and altered along the planes of joints, faults and shear zones, excavation was taken deeper in these zones for founding the dam.

To reduce seepage losses through the foundation and uplift pressure under the dam, extensive grouting of the foundation was carried out. (Source: Foundation Treatment in Hirakud Dam Project, 1959, Govt. of India by M.S.Thirumale Iyengar)

2.1.3.1.1 Drilling & Grouting:

(a) Low pressure Consolidation Grouting:

As soon as the excavation in an area was completed to the final hard rock level for a length of about 50 ft on either side, the foundation treatment work was started. Drilling and washing of holes were allowed to start but no grouting was permitted until all blasting operations had been completed within 50 ft of the limits of the released area. Grout holes 30 ft. deep at 20 ft centre were drilled in 4 rows at 20 ft apart parallel to the axis of the dam by wagon drills at the required directions and angles so as to get the greatest coverage of seams and fissures. Thereafter holes were washed for 20 to 45 minutes.

Water cement ratio of the grout mix varied from 12:1 to 0.5:1 by volume at the specified pressure to prevent uplift and movement of rock mass under treatment but upto a maximum of 100 lbs / sq.inch. Grouting of the hole was continued until the hole accepted the grout of less than 1 cft in 15 minutes.

(b) High Pressure Curtain Grouting:

After the dam has come up above 610 ft. the spillway crest, drilling of the curtain holes through the embedded pipes was started, using diamond core drills. The method of washing, testing and grouting were similar to those described under Low Pressure grouting. The maximum grouting pressure varied from 150 to 200 psi.

During the progress of concreting operation 3 ft. length of 3" diameter M.S or G.I pipes were embedded in the concrete at 20' centers generally along the line of the grout curtain and at desired directions and angles. In the case of left Spillway these pipes were embedded in the sill of the sluice with the top of pipe flush with the sill and in case of Right Concrete Dam, these pipes were embedded in the floor of the foundation gallery. The top of these pipes were kept closed by means of metal plugs.

(c) Drainage Holes:

The drilling of drainage holes was started after the completion of curtain grouting. Three feet length of 4" diameter M.S or G.I pipes were embedded in the concrete at 15' centre generally along the line of drainage holes just downstream of the grout curtain during the initial concreting operations. Details of Drainage holes are furnished in Table No 2.3.

Table2.3 Particulars of drainage holes drilled in Left Spillway

Sl. No	Block no.	Total no. of holes Drilled	Total footage Drilled	Sl. No	Block no.	Total no. of holes Drilled	Total footage Drilled
1	3	2	130'-8"	13	15	2	121'-10"
2	4	2	129'-1"	14	16	2	125'-1"
3	5	2	135'-0"	15	17	2	129'-9"
4	6	2	127'-8"	16	18	2	120'-0"
5	7	2	131'-3"	17	19	2	131'-4"
6	8	3	197'-6"	18	20	2	133'-0"
7	9	3	189'-3"	19	21	2	127'-10"
8	10	3	158'-10"	20	22	2	124'-7"
9	11	2	124'-4"	21	23	4	227'-2"
10	12	2	125'-5"	22	24	4	216'-9"
11	13	2	126'-7"	23	25	4	235'-9"
12	14	2	128'-4"	24	26	4	249'-7"
				25	27	2	121'-6"

Total No. of holes drilled 61, Total footage drilled 3768'-1", Size of hole 0' 3"

(d) Test of Grouting:

To test the efficacy and penetration of grout, drill holes were drilled in the grouted area after the grouting operations were completed. The holes were tested under water pressure equal to a little less than the pressure adopted for the grouting and the amount and the rate of leakage in each holes were measured. If any holes showed excessive leakage it was grouted again under pressure.

(e) Internal Drainage of Dam:

8" diameter formed porous drains were provided in the body of the Dam for the internal drainage of the Dam structure. These were provided at intervals of 20' and 30' along a line parallel to the axis of the Dam and located at 7 ft. downstream of axis at contraction joint and 12' downstream of axis at other locations. These extended for the entire height of the Dam and discharged into the foundation gallery of Right section and into the 3 ft. diameter header on the Left section. In the blocks where penstocks are located, the spacing of these drains were slightly modified to avoid the penstocks.

(f) Drainage Sumps:

The drainage water collected from both the foundation as well as the dam structure, flowed to two drainage sumps provided at either end of each concrete dam section, from where it is pumped out by suitable pumps located in the pump well. The dimensions of the sumps are 10' x 10' x3' with 6 ft. diameter pump well over it. The rate of flow from the foundation drainage holes were carefully watched with different water levels in the reservoir. If any excessive flows were noticed in any zone, additional grout holes were drilled and grouted along the grout curtain. Details of footage drilled and grouted are given in Table No 2.4, 2.5, 2.6 &2.7.

Table 2.4 Consolidation Grouting

Abstract of Drilling & Grouting from 1951-55 (to end of September, 1955)

Sl. No	Feature	Total No. of holes grouted	Total footage drilled & grouted	Total cement placed (bags)	Average take per linear foot
1	Concrete dam -Right side	2, 034	55, 725	9, 707.574	0.174
2	Power House unit No-1 & 2	136	2, 878	894.00	0.310
3	Concrete Dam- left side	552	13, 892	5, 935.643	0.427
4	Earth Dam -left side	188	6, 458	17, 378.075	2.689
5	Earth dam- Hirakud Island	273	7, 496	6, 717.10	0.896
6	Left Dyke Gap no-1	45	1, 355	1, 229.00	0.958
7	Left Dyke Gap no-2	45	1, 356	230.25	0.130
	Grand Total	3273	89, 160	42, 281.50	-

Table 2.5 Abstract Showing the grout Consumption in high pressure grout holes in Right spillway & Power Dam

Block No.	Total holes grouted	Total footage grouted	Cement placed in rock	Cement placed in concrete	Total cement placed (bag)	Average total per linear foot
28	3	196'-9"	6.50	2.75	9.25	0.045
29	3	205'-6"	21.00	2.75	23.75	0.146
30	3	217'-0"	16.25	14.00	30.25	0.14
31	3	224-6"	58.75	7.75	66.50	0.30
32	3	308'-11"	48.75	3.25	52.00	0.25
33	3	149'-0"	7.75	2.50	10.25	0.07
34	3	195'-4"	9.25	2.00	11.50	0.06
35	3	199'-0"	68.30	2.00	70.50	0.35
36	3	194'-9"	45.00	2.25	47.25	0.29
37	3	203'-0"	64.00	2.00	67.00	0.333
38	3	204-6"	29.75	6.00	35.75	0.17
39	3	206'-6"	46.00	2.375	48.375	0.23
40	3	192'-10"	33.50	3.25	36.75	0.19
41	3	219'-6"	40.25	9.00	49.25	0.22
42	3	218'-0"	39.00	2.75	41.75	0.19
43	3	205'-8"	23.00	5.00	28.00	0.13
44	4	226'-10"	102.50	4.50	7.00	0.47
45	3	207'-9"	48.00	4.25	52.25	0.00
46	3	205'-10"	103.25	133.50	236.75	0.25
47	3	201'-6"	33.00	4.00	37.00	0.18
48	13	631'-10"	96.50	42.50	139.00	0.20
49	4	342'-0"	11.00	54.50	65.50	0.20
50	12	401-1"	26.00	17.75	43.75	0.10
51	4	266'-10"	43.75	7.50	51.25	0.20
52	4	327'-7"	172.00	17.25	189.25	0.60
53	12	920'-9"	379.50	73.00	452.50	0.50
54	15	1027'-4"	152.75	16.999	169.749	0.14
55	2	156'-7"	11.00	6.000	17.00	0.11
56	3	205'-0"	93.00	25.50	118.50	0.76
57	3	237'-4"	30.00	28.50	58.50	0.25
58	5	386'-2"	310.50	21.00	331.50	0.80
59	1	74'-0"	23.00	1.00	24.00	0.30
60	1	44'-0"	1.00	59.00	60.00	1.5
61	1	75'-0"	33.00	1.00	34.00	0.43
	TOTAL	9180'-6"	2226.8	387.374	2814.174	-

Table 2.6 Statement showing the grout consumption of High pressure holes concrete Dam-Left spillway

Sl.no	Block no.	Total holes grouted	Total footage grouted	Cement placed (bags)		Total cement placed in bags	Average take per linear foot
				In Rock	In Concrete		
1	3	6	343'-7"	25.50	128.75	154.25	0.45
2	4	4	297'-11"	168.75	21.00	189.75	0.63
3	5	6	462'-7"	41.085	82.499	123.584	0.256
4	6	6	379'-8"	13.50	37.00	50.25	0.125
5	7	6	434'-9"	34.375	9.00	43.375	0.10
6	8	6	447'-8"	528.50	36.50	565.00	1.25
7	9	6	389'-7"	63.50	118.417	181.917	0.466
8	10	6	338'-3"	12.00	8.834	20.834	0.06
9	11	6	332'-0"	46.75	86.75	133.50	0.40
10	12	7	335'-9"	6.50	145.75	152.25	0.42
11	13	4	290'-4"	35.25	41.50	76.750	0.26
12	14	4	295'-6"	107.50	57.75	165.25	0.56
13	15	4	279'-10"	25.75	69.75	95.50	0.34
14	16	4	288'-9"	40.75	45.25	86.00	0.20
15	17	5	348'-2"	65.00	36.25	101.50	0.30
16	18	4	295'-5"	140.00	23.50	163.50	0.60
17	19	4	297'-9"	107.25	88.50	195.75	0.55
18	20	5	331'-2"	230.00	103.25	333.25	1.00
19	21	4	292'-5"	124.00	60.50	184.50	0.63
20	22	4	287'-8"	175.00	34.25	210.25	0.73
21	23	3	201'-1"	324.50	27.75	352.25	1.74
22	24	3	196'-2"	70.50	26.25	96.75	0.50
23	25	3	205'-6"	31.50	26.50	58.00	0.29
24	26	3	207'-8"	28.50	26.00	54.50	0.23
25	27	2	141'-0"	18.00	36.00	54.00	0.40

Total No. of Holes grouted		115
Total footage grouted in concrete		1662' - 8"
Total footage grouted in rock		6057'- 6"
	Total	7720'-2" (2353.11m)
Total cement placed in Rock		2463.960 Bags
Total cement placed in concrete		1377.50 Bags
	Total	3841.460 Bags
Average take / linear foot in Rock		0.407
Average take / linear foot in Concrete		0.830

Table 2.7 Details of Drilling & Grouting Particulars in the Penstock No. 1, 2, 3 & 4

(Period July 1955 to August 1955)

Sl.no	Penstock no	R.D of penstock Center Line (ft)	Date of Concrete	No. of holes Grouted	Cement Used (bags)		
					Placed	Wasted	Total
1	Penstock no. 1	15758	19-11-54 to 3/55	12	27.5	0.5	28.0
2	Penstock no. 2	15697	18-11-54 to 3/55	18	200.0	2.0	202.0
3	Penstock no. 3	156 34	27-7-54 to 3/55	10	10.0	...	10.0
4	Penstock no. 4	155 35	29-10-54 to 2/55	9	50.0	1.0	51.0

Date of Commencement of Drilling	20-7-1955
Date of completion Grouting	19-9-1955
Total No. of holes drilled & grouted	49 Nos.
Total bags of cementing placed	287.5 bags
Wastage	3.5 bags
Total	291.0 bags

(g) Contact Grouting at the Exploratory Tunnel at the Right Abutment (Season 1955-56)

A tunnel of size 10' x 10' and 50' long was put into the Right abutment at R.L 618 ft for exploratory purpose. The present length is 26 ft. This gave valuable information regarding the nature of rock in the abutment as well as the extent of jointing etc. When the masonry in the dam section has come upto the level of the tunnel, the tunnel was thoroughly cleaned and all loose rock has been removed and washed and finally back filled with concrete. The tunnel had some supports for the roof. Concreting of the tunnel was started at the inner end and progressed towards the outer end, the supports being removed as the concerting progressed.

(h) Fixing of Condulets:

During the progress of concreting, condulets were fixed at a number of places along the contact of rock and concrete, especially on the upper half of the tunnel section. 1" pipes were taken out of these to the outside of the tunnel and taken up at the face of the tunnel. The end of the pipes were kept closed by means of plugs.

After the concrete in the tunnel has set and taken the final shrinkage, these condulets, were grouted in March, 1956. The interval period was about 4 months.

The statement below in Table 2.8 shows the details of grout consumption.

Table No 2.8 Condulets grouted around the Exploratory Tunnel at Right abutment.

Sl.no	Condulet No.	Pressure P.S.I	Water Cement Ratio	Cement placed (bags)	Remarks
1	C.P - 1	25	5:1	8.5	
2	C.P – 2	25	5:1	12.25	
3	C.P – 3	25	5:1	0.125	
4	C.P – 4	25	5:1	Nil	
5	C.P – 5	25	5:1	Nil	
6	C.P – 6	25	5:1	Nil	
7	C.P – 7	25	5:1	1.5	1 1/8 Bags of cement used for Back filling
8	C.P – 8	25	5:1	Nil	
9	C.P – 9	25	5:1	4.5	
10	C.P – 10	25	5:1	20.5	
11	C.P – 11	25	5:1	Nil	
12	C.P – 12	25	5:1	0.5	
13	C.P – 13	25	5:1	Nil	
14	C.P - 14	25	5:1	7.0	
				54.875 bags	
	Cement used for Backfilling			1.125 bags	
	Total cement used			56.00 bags	

(i) Contraction Joint Grouting of Left Spillway

In the Left Spillway, from Block 3 to 23, copper sealing strips were not provided around the sluice gate gallery at the contraction joints. But these copper strips were provided near the downstream face of spillway along the Ogee Curve from R.L 609 to about R.L 600 ft.

In order to prevent any possible leakage into the gallery through the contraction joints, all these joints were grouted by drilling shallow holes across the joints, both from the Spillway crest level as well as from inside the sluice gate gallery through the roof. The surface area of the joints between the roof of the gallery and the Spillway Ogee Curve were thoroughly grouted from R.L 610' to R.L 580 ft. Details are given in Table - 2.9.

While grouting the contraction joints, large number of leaks occurred on the downstream face of Spillways between R.L 610 and 580 ft along the horizontal construction joints.

Table 2.9 Grouting of Contraction Joints in Left Spillway

Construction joint between blocks	No. of holes drilled and grouted	Cement injected (bags)	Construction joint between blocks	No. of holes drilled and grouted	Cement injected (bags)
2 & 3	8	7.12	13 & 14	8	4.5
3 & 4	7	3.38	14 & 15	6	6.0
4 & 5	10	14.50	15 & 16	12	16.5
5 & 6	10	3.0	16 & 17	6	1.25
6 & 7	9	5.75	17 & 18	6	3.50
7 & 8	10	19.75	18 & 19	8	3.50
8 & 9	6	4.5	19 & 20	6	2.25
9 & 10	6	6.0	20 & 21	6	3.50
10 & 11	6	18.0	21 & 22	8	14.0
11 & 12	6	12.5	22 & 23	17	31.5
12 & 13	6	2.0	In block 23	30	66.0
					Total -249.0

Details of grouting around Penstock Liner, Draft Tube and Penstock blockouts are presented respectively in Table No.- 2.10, 2.11 & 2.22.

Table 2.10 : Details of Grouting around Penstock Liner

Sl. No	Block No.	No. of condulets grouted	Maximum pressure used in P.S.I	Total cement placed (bags)
1	Unit - 1	12	25	27.50
2	Unit - 2	18	25	200.00
3	Unit - 3	10	25	10.00
4	Unit - 4	9	25	50.00
5	Unit - 5	13	30	35.50
6	Unit - 6	17	30	7.50

Table 2.11 : Details of Grouting around Draft tube Liner Unit-3

Hole no.	Pressure applied in P.S.I	Water Cement Ratio	Total cement placed (bags)
D-T-1	0 - 5	1:1	47.0
D-T-2	0 – 5	1:1	47.0
D-T-3	0 – 5	1:1 to 5:1	1.0
D-T-4	0 – 5	1:1 to 5:1	0.5
		Total	95.5 bags

Table 2.12 Grouting at the contract of Masonry & Concrete in Penstock Block outs Condulets of Power Dam

Sl.No	Block No.	No. of condulets grouted	Maximum pressure used	Total cement placed (bags)
1	48	15	30	6.5
2	49	14	30	25.75
3	50	12	30	15.25
4	51	15	30	1.50
5	52	14	30	6.00
6	53	41	30	59.75
7	54	35	30	10.125
8	55	3	30	3.75
			Total	128.625

2.1.3.1.2 Excavation and Preparation of Foundation:

Excavation operations were conducted in such a manner as to prevent any damage to the underlying sound rock. The loose overburden was first removed by manual labour. As soon as the hard disintegrated rock was met with, drilling and blasting operations were started. Holes 2 ½" to 5' deep were drilled in a pattern and blasted. Blasting operations were strictly controlled. The exploratory core holes that were already put into the foundation both during the excavation stage as well as during initial investigation stage gave an indication of the probable level upto which the excavation was to be taken. Excavation was taken down at least 2 ft. into hard and fresh rock. Further excavation which extended for 9" to 1 ft. was done by wedging and carving operations in order to eliminate cracked and broken portions of the rock formations.

In the fissured zones, excavation was taken deeper till hard rock was met with or till the joints narrowed down, the minimum depth being twice the width of the joint or fissure. Grout holes were drilled across the fissure and joints to intersect them at lower elevations. Washing under pressure was done through these holes from the surface. The sides of the fissure were cleaned out by scaling the soft portions. After the completion of washing, these holes were grouted. Surface grout leaks that occurred through the fissures were caulked with lead wool and jute pieces. Where the leaks were too big to be caulked effectively, there the grouting was taken up after the excavated trench in the fissure had been back-filled with concrete.

In the right abutment, where the foundation was very steep, excavation profile was taken in a series of sloping benches, the elevation of the lowest bench being at R.L 480' and the top most bench at R.L 618 ft.

During the course of excavation, a plan of final shaping was followed which produced a rock surface roughly stepped in appearance that would increase the sliding resistance and bond with the

concrete. Steep downward slopes in the downstream direction were avoided and sharp re-entrant angles were prevented from being formed at the contact surface of the structure with the foundation.

After completion of excavation, all loose and shattered portions of the foundation were removed by wedging & barring with chisels, wedges and crowbars. The staining on the rock in the form of crust was removed by running the jack hammer using coromant drill bits.

The foundation rock surface, so prepared was just moistened, before concreting, if already dry and coated by brush with rich cement slurry of 1:2 or 1:3 proportions of cement and water to ensure good bond and a water tight joint. This was done just before starting concerting. Over the slurry when it was still wet a thick of cement mortar about 2" to 3" thick was immediately spread and worked into the corners and joints by trowel.

2.1.3.1.3 Special Foundation Problems :

The special foundation features encountered during execution of few blocks of Left Spillway, Right Spillway and Power Dam are briefed as under. For details, one may refer 'Foundation Treatment in Hirakud Dam Project' (1959) by M.S.Thirumale Iyenger.

(i) Block No 17/18

The block consisted of a high mound of hard granite rock in the central section of the blocks, underlaid by a horizontal layer of disintegrated gneiss of 8" thickness, facing the upstream side. The disintegrated rock between the hard rock layers was scooped out with crowbars and pick-axes for about 3 ft. distance inside (dental treatment) and a collar of concrete was laid in that portion in order to support the overlying rock. Large number of wagon drill holes 15' to 20' deep were drilled from top of the ledge in order to wash out the fissure. These holes got interconnected and much material was washed out. These were subsequently grouted after the concrete collar has been laid.

(ii) Block No. 21

This Block contained a horizontal seam of weathered rock of about 1 ½ ft. thickness below a vertical ledge of 7 ft. height. The ledge extended for the full width of the block at about 100 ft. downstream of dam axis. The upstream portion of the block was at lower level. The seam was exposed towards the upstream and nothing was seen on the downstream side of the ledge. The seam was cleaned by scooping out the weathered material for a distance of 3 ft. into the seam and washed thoroughly with air and water jets. The seam narrowed down to 6" thickness. The seam was also washed by means of eight holes drilled 15 ft. deep from top of the ledge. Washing was very effective. 1 ½" pipes were embedded in these holes and extended up for subsequent grouting after the concrete has taken the final shrinkage. After the completion of washing, the seam was washed from outside and back-filled with reinforced concrete.

(iii) Block No. 38 and 39

A band of epidiorite rock with an average width of 40 ft. and a general north-south trend traversed the foundations in Blocks 38 and 39 at an angle of 65^0 to the axis of dam. It has a very steep dip of 75^0 to 80^0 towards south west or the right abutment.

The band is highly jointed, fractured and crushed with different sets of joint planes. The rock was so much fractured that the individual pieces were about an inch in size and angular in shape. Filmy coatings of clay were found on the fracture planes. The jointing and fracturing was more intense near the contact with the adjacent rock formations. A large number of springs were noticed in this zone particularly at the contact region.

Five Diamond drill core holes were drilled in this area. The drill cores indicated that the closely spread joint pattern, continued down to R.L 485 ft in the upstream portion of this zone and to R.L 477' in the downstream portion of this zone. Excavation was accordingly done in the form of a trench and taken down to R.L 484' in the upstream section (i.e. upto 35 ft. downstream from axis) and to R.L 476' in the reaming portions of this zone, the average foundation level on either side of this zone being R.L 490'. Grouting was done in this zone with holes at 10 ft. intervals both ways and for the entire length of the zone upto the limit of the dam section, in order to consolidate the rock and seal the springs. Large number of holes 30' deep at 10' and 5' intervals were drilled, washed and grouted. All the holes were drilled inclined using wagon drill excepting a few exploratory holes which were drilled with diamond core drills. These latter were subsequently, used as grout holes. All the holes had water flows at depths varying from 10' to 24', below the top of the holes. The rate of flow varied from ½ to 3 gallons per minute.

Washing of clay joints and fracture planes was carried on through the drilled holes with air and water connected to adjacent holes and allowing the effluent to come out through the other interconnected holes.

After the completion of washing, grouting was started first in the upstream area and then proceeded towards the downstream. Since the rock was highly jointed, there were lot of surface grout leaks on the foundation during the grouting, besides interconnections. Hence grouting was temporarily suspended and started again after 5 ft. lift of concrete had been placed. This eliminated all the leaks and further grouting was carried on without much interference. All the decomposed portions of the rock on the inclined faces were removed with the help of chisels and hammers. Dowel bars were provided at intervals of about 10 ft. on both the sides of the crushed zone. Compressive strength of rock in the crushed zone is furnished in Table-2.13.

Grout condulets were embedded at the inclined contact plane of the rock and concrete in this zone. The pipe line system connecting these condulets were carried into the gallery for grouting.

Total footage grouted was 3451 ft. and the total cement consumption was 402 bags with an average of 0.12 bag per liner foot.

Table 2.13 : Result of Compressive Strength of Rock in the Crushed zone

Blocks 38 / 39

Hole No	Location	Depth of hole (ft)	R.L. of top of hole (ft)	Depth at which core is taken	L / D Ratio	Compressive strength Tons/sqft	Specific gravity
38-D-1	(In Block 38) 34 ft D/S of axis & 9 ft. from C.J of 38 and 39	50'	487.66	21'-3" to 22'-1"	2	442.3	2.791
39-D-1	(In Block 39) 69' D/S of axis & 1 ft. from C.J of 38 and 39	30'	487.40	18'-4" to 19'-1"	2	627.5	2.943

(iv) Block No 54

A major fault zone existed in this block at RD 15791 ft. This was inclined at 70° to the horizontal and extended at right angles to the axis for the entire base width of the block from the upstream edge of excavation and continued into the Power House foundation at the downstream end of the block. Specialized treatment was given for the fault plane in view of the fact that this may act as a passage of leakage from the reservoir.

The rock in the vicinity of this fault zone comprised of slates, shales, and phyllites with their bands of arkosic quartzites. These were highly crushed and sheared for a width of 15 ft. on either side of the fault plane. The fracturing was so intense that on excavation, the individual pieces came out as thin flaky material, less than an inch in thickness. The fracture planes were smooth and had a soapy feel due to the presence of talc and chloritic material. The fault zone itself was about 9 ft. wide containing crushed slate with a clay gouge of varying thickness of 2' to 5 ft. within this crushed rock. The gouge zone gradually thinned down at depth to about 1'6" wide at RL 441 ft and further to about 6" at R.L 432'. The material was quite soft and friable and could be excavated easily with picks and crowbars or by using high pressure air water jets. Water seepage was quite heavy along this and some springs were noticed near the contraction joint of Blocks 54 and 55, at a distance of 60' to 70' from the axis.

In view of highly sheared and fractured nature of the rocks and the presence of the fault zone, the foundation surface has been taken down to R.L 462' i.e. 53 ft. below the original ground level (R.L 515) for the entire block. The foundations level in the adjacent blocks on either side was R.L 483'. Along the fault zone, a trench 10' wide has been cut to R.L 450'. A shallow trench 20' wide has been provided upstream of the axis and parallel to it. At the upstream end of the fault zone, a vertical shaft of about 15' x 15' has been excavated extending down to R.L 432. The trench and shaft have been provided with a view to increase the path of percolation underneath the dam to about 3 times the hydraulic head. Examination of the fault zone with the help of diamond drill core holes revealed that this zone continued at the same direction and angle to any depth. It was originally proposed to excavate a shaft 40' deep on the upstream side along the axis for a length of about 30' and a downstream shaft at the toe of the block with 20' depth, in addition to transverse trench perpendicular to the axis and extending upto the downstream end of the block. Further examination of the foundation surface after it was cleaned of all muck and debris showed that the rock adjacent to the fault zone was sufficiently hard and it was found difficult to excavate a trench 40' deep unless blasting was resorted to. Since the foundation nearby was already in a highly crushed state, blasting would shatter the foundations further and damage the foundation. Hence it was decided to restrict the size of the shaft and provide the deep shaft of about 15' wide and 15' long only at the fault zone where the rock is soft, and a shallow cut off trench for the remaining portion of the block where the rock was hard, in addition to the transverse trench at right angles to the axis along the line of the fault zone, extending upto the downstream toe.

The deep shaft finally excavated was about 15' x 15' and 30' deep below the general foundation level in this block.

This was designed in such a way that the ratio of the path of percolation to the hydraulic head at the upstream end of the fault was equal to 3. The clay gouge was about 3" to 4" thick at the bottom of the shaft. A shallow upstream cut off trench was provided for a length of about 70 ft. The transverse trench at right angles to the axis was 12' wide and 10' deep. The thickness of clay gouge at the bottom of this trench varied from 3" to 6" which was excavated to twice the width of the gouge before backfilling with concrete.

Excavation of the deep shaft and the transverse trench presented considerable difficulties due to the high percolation, from the foundations narrow working space and the sudden rock slides that occurred often from the sides of the shaft.

Since the rock was very much crushed, and could not take high pressure, stage drilling and grouting was adopted. Three stages were adopted first. But later on, it was found that it would be more economical to do two stage grouting. The object of this method was to consolidate the upper portions of the rock at low pressure so as to enable higher pressure to be used for lower zones. The holes were drilled first to 12' depth and grouted at 30 psi. After an interval of 24 hour the holes were redrilled to the full depth of 30 ft. and then grouted at 74 psi. Levels were taken over the entire area during the grouting operation in order to see whether any uplift occurred due to the grouting operations. This served as guide for controlling the pressure.

Most of the holes were drilled with Wagon Drills with the exception of a few which were drilled with diamond core drills. Considerable difficulty was experienced while drilling with Wagon Drills, as the drill rods used to get stuck up due to the softness of the rock and caving from the sides of holes. Drilling had to be done continuously with washing of the hole at frequent intervals.

(v) Power House:

In the original design, no grouting was proposed for the Power House foundations. But subsequently examination of the foundations after final level was attained revealed softer formations in the areas below Unit 1 and 2 consisting of numerous springs all over the area. The foundation of Unit No- 1 was very soft and decomposed slate with numerous springs and consisted of fault zones containing clay gouge while Unit 2 was on disintegrated mylonites and granitic gneiss, all the other Units were on fresh, hard and massive rock. The foundation below Unit 1 was therefore excavated about 5 ft. deeper than the designed level and heavily grouted and R.C.C. Raft was provided for the entire base of this Unit, in order to prevent differential settlement. (Design level was R.L 447 and excavated level was R.L 442 ft). The fault zone noticed in Block 54 extended into the foundation of Unit No.1. This was excavated in the form of a narrow trench in the clay gouge portions the depth of the trench being as much as twice the superficial width of the zone. The deepest level attained was R.L 440', about 75 ft. below the natural ground level. Some of the drill holes in this area were flowing water as much as 6 gallons per minute. In order to reduce the uplift pressure on the base of the Power House structure, the foundations under Units 1 and 2 were grouted with holes at very close intervals (5ft) till the percolation was stopped. All the grout holes were 20' deep below the final foundations level. Altogether 873 bags (or 43.65 Ton) of cement were injected into the foundation area, the maximum grout consumption in a single hole being 96 bags. Work was started in February 1954 completed by 19-3-1954. (Source: - Foundations Treatment in Hirakud Dam Project, 1959, Govt. of India by M.S.Thirmale Iyengar, Pg. 12 to 14.).

Since the longitudinal gallery at R.L 497 is too high above the foundation level in this area, another foundation gallery was provided in this block at about R.L 470 with a transverse gallery at right angels to the axis and located over the fault zone. The lower longitudinal gallery was connected to the upper gallery by a vertical shaft 5' diameter located in Block 53. Curtain grouting and drainage hole drilling in this area were done from the lower gallery at R.L 470 ft.

A reinforced concrete shaft consisting of 18 lbs. rail at 2' c/c in both ways was laid to span over the crushed zone. This served to distribute the load and prevent unequal settlement.

Since the foundation rock is badly crushed, it was not possible to get proper size of core for testing the crushing strength. Hence the standard bearing pressure test in situ was conducted in this block as described in the "Second Field report Foundation Preparation and Treatment- Period October, 1952 to July 1953". The total test load amounted to 51.027 ton over an area of 3 sq.ft i.e. 17.09 tons per sq. ft. The net differential settlement of the bearing plate averages to 0.065" only.

Grout holes 30' deep were also drilled into upstream face of excavation at intervals of 10 ft. after every 5 ft. lift of concrete, starting from R.L 460 ft and upto R.L 480'. Large number of grout leaks appeared on the upstream face of excavation which were subsequently sealed by grouting.

Number of observation holes 30' deep were drilled in the grouted zones in the foundation, and 1" diameter pipes extended from these into the gallery for observation of percolation and measurement of uplift pressures later on.

3" diameter pipes were embedded in concrete and extended from the bottom of the upstream shaft and the transverse trench for observation as well as for subsequent drilling and grouting. The pipes from the shaft were taken upto R.L 515. From this level, deep holes 60' deep below the bottom of the shaft were drilled and grouted at intermediate pressure of 100 psi (during the months of July to September, 1954). Altogether about 15 holes at intervals of 5 ft. were drilled and grouted at intermediate pressure at the upstream end of the shaft.

High pressure grouting in Block 54 was done during the period January to June 1956 vide Table No.2.14.

Table 2.14 : High Pressure, grouting in Block 54

Type of Grouting	Footage	Total cement placed	Average per foot
Low Pressure Grouting	6, 700	1, 072 bags	0.17
Intermediate Pressure Grouting	--	--	--
High Pressure Grouting	1027	170 bags	0.14

(vi) Treatment of the Right Abutment

The right abutment comprises of block 59 to 63, with the final foundation levels varying from R.L 500 ft in Block 59 to R.L 620 in Block 63. The abutment face as originally excavated was very steep and consisted of numerous joints, fissures and fault planes, including the folding in different directions. The foundation rocks in these blocks were soft consisting mostly of shales, slates, arkoses, silt stones and quartzites intimately mixed with each other with a number of anticlines and synclines.

Numerous holes were drilled in each block in a pattern covering the entire width of the block. Samples of rock cores taken from the foundation of these blocks were tested for compressive strength, absorption and specific gravity. Strength in dry state varied from 2500 to 5000 lbs per sq.in, but shown considerable loss in strength on wetting.

The continuous steep dip in abutment blocks was flattened out with a number of ledges at R.L 507, 525, 545, 570 and 595 and 618 ft.

The foundation drainage gallery was taken at an inclination and provided with steps and suitable landing places following the rock contours. The gallery was constructed upto R.L 595' and stopped with a dead end. Pipes for curtain grout holes as well as drainage holes were embedded in the gallery floor as is done in other sections of the foundation.

There were 2 tunnels in the Right Abutment hill one at R.L 618 ft extending for a length of 26 ft. and the other at R.L 660 extending for a length of about 100 ft. The tunnel at R.L 618' was thoroughly cleaned and washed and backfilled with concrete leaving condulets at the contact planes of rock and concrete for grouting at a later stage, and the pipes from these were taken upto R.L 640. Subsequently these were grouted, the total cement consumption being 55 bags. The other tunnel being much above M.W.L no special treatment was required.

2.1.3.2 Earth Dam

2.1.3.2.1 Geology of Foundations:

The predominant rock type was granitic gneiss intersected by bands of chloride sericite schist. The principal system of jointing and strike was parallel to the river trending N 70° W and dippings 56° W. Depth of disintegrated rock varied from 5' to about 40'. Below this depth, hard rock existed for considerable depth. In the reach between R.D 9000 and 12, 000 ft in the middle of Hirakud Island, cuddapah shales and quartzites occur. In the reaches from R.D 6580 to 7300 ft and 12, 000 to 13, 000 ft the foundation consisted of hard and fresh granitic gneiss.

2.1.3.2.2 Foundation Treatment:

The general plan for grouting the Earth Dam foundation provided for one row of holes along the centre line of the cut-off trench and 2 rows of holes with staggered arrangement in certain reaches. Primary holes were drilled first 30' to 50' deep at 40' centers. After these were grouted, secondary holes 30' deep were drilled in between and grouted where the cement consumption in the primary holes has been excessive or where the geological conditions warranted. Tertiary holes were put in where the grout consumption in the secondary holes was excessive. The width of cut-off trench varied from 30' to 50' depending upon the height of the section and with side slopes of 1:1. Details of treatment given are described in the following paragraphs.

2.1.3.3 Special Foundation Features:

(a) The general procedure followed for preparing the foundations of the Earth Dam was to strip the top soil, humus and boulders if any. A cut off trench located between upstream toe and axis of dam was provided for the entire length of the Earth Section. Excavation for the cut off trench was taken 2 ft. into disintegrated rock where the disintegrated rock layer was fairly thick and 2 ft. into hard rock where it was met with a shallow depth. The disintegrated rock was not susceptible of holding cement grout. Considerable difficulty was experienced in grouting.

During the actual excavation of the cut off trench it was revealed that the soil strata at the depth of 8' to 10' below the ground surface at the majority of places was fairly impervious to effectively check under seepage. Grouting would require excavation to sound rock and as the depth of overburden was fairly deep at places, it would curtail considerable delay and expense. Permeability characteristics of the sub-soil material in the cut off trench were therefore investigated for taking decisions (i) with respect to the depth to which the trench should be excavated and (ii) the necessity or otherwise of grouting the lower solid material. Samples of soil were taken from the bottom of the cut off trench tested for permeability characteristics. Different types of soils were met within the cut off trench. In general the soil profile consisted of the top alluvium of clayey sand and gravel alluvium and this gradually changed to blocky weathered rock. Different types of rock such as schist gneiss, silt stone, arkose and phyllites and granites were met with. Although there was fair variation in the permeability characteristics of the materials tested the highly clayey disintegrated rock without open joints or root holes were found to be sufficiently impervious. Wherever the permeability was less than 1 ft. per year grouting was omitted and the cut off trench was taken 2 ft. into such soil so that the seepage underneath the dam would be depressed to a depth that would offer less probability of damage to the embankment by underseepage. There were certain reaches of the cut off trench which contained open jointed rocks with quartz veins, which were grouted. There were schist mounds near R.D 8640 ft, reach between R.D 9450 ft and 9600 ft, between R.D 10750 and 11300, and 11550 to 11950 ft.

(b) At the deep gorge section (R.D 7200 ft to 7600 ft.) hard rocks were met with which consisted of granitic gneiss, schist and phyllites. The principal system of jointing was parallel to the river trending N 65° E and dipping S 62° W.

The reach from R.D 6692 ft to 7300 ft consisted of massive granitic gneiss while the reach from R.D 7300 to 7600 ft consisted of schist and phyllites and that from R.D 7600 ft to 8100 ft consisted of disintegrated gneiss. After the excavation of the cut off trench, about 3 ft. into hard rock, one line of grout hole was drilled and grouted at 40 ft and 20 ft centers. Holes in the deep gorge took comparatively little grout while those on the left flank of the gorge took some appreciable quantities of grout. The maximum grout consumption in a single hole was 425 bags of cement. The gorge section showed a narrow shear zone with a width of 1' to 2' and extending for a distance of 80'. Seepage was observed here which was arrested by grouting. A group of 4 springs was observed on the right bank of the gorge section at about 20' downstream of axis, giving a discharge of about 3 gallons per minute. This was stopped by grouting. The total number of bags injected into the deep channel section was 3555 bags. The treatment of the deep channel section including the flanks was started in February, 1952 and completed by middle of April, 1952.

During the following season, in the month of December, 1952, when cleaning was going on in the deep gorge section, seepage was noticed on the Left Flank of the gorge on the exposed rock at R.L 480 ft to 485 ft and about 10ft. upstream of axis.

These were noticed near R.D 7150 ft and at R.L 480 to 485'. and just upstream of the axis. It was found that this seepage was from the downstream pool of water. Number of wagon drill holes were drilled and grouted and the flow stopped before the earth fill commenced. This zone took 181 bags of cement.

(c) In the Earth Dam Section (R.D 12000 ft to 13000 ft) between the Hirakud and Koliarkud Islands, where hard and sound granitic rock was met with near the surface, two cut off trenches were provided in order to intercept the flow line that might develop along the plane of contact between the earth and rock. The main trench was 50' wide and was located with its centre line at about 260' upstream of axis. The secondary cut off trench 30' wide was located downstream of the main trench and with its centre line 60' upstream of axis.

Excavation for the cut off trench was taken about 2 ft. into hard rock. Two rows of grout curtain with staggered arrangement were provided in the main trench with holes 30' deep and at 40' centers and one row of holes at 40 ft. centre and in the secondary trench. The total footage drilled and grouted in this reach is 3635 ft. (121 holes) and the total cement consumption is 1635.3 bags, with an average of 0.45 bag per foot. The maximum grout consumption in a single hole was 223 bags. *The maximum intake for a single hole in the entire earth dam is 1391 bags.*

During the season 1953-54, excavation of the main cut off trench between R.D 12, 000 and 13, 000 ft. was completed for the entire length with the exception of a portion of the trench from R.D 12190 ft to 12239 ft where a pocket of highly disintegrated rock was met with during excavation, extending to a maximum depth of 30 ft. below R.L 496 ft and forming a triangular pocket below which hard and sound rock existed. Since there was not enough time in that season for removal of the entire overburden of disintegrated rock prior to grouting, it was decided to do the grouting in that season and postpone the removal of disintegrated rock to the next season (1954-55). Accordingly, although the excavation in this zone was not completed to the hard rock level, grouting in this area was completed with grout holes 30' deep below the hard rock level which existed below an overburden of disintegrated rock 20 ft. to 30 ft. thick.

During the following season i.e. (1954-55) after the diversion of the river flow, attention was first concentrated on this area for removal of overburden of disintegrated rock for a maximum depth of 30' below R.L 495' (the ground level being R.L 500').

In order to determine the direction and extent of this zone, additional diamond drill holes were drilled in the main cut off trench and at about 50' intervals towards the secondary cut off

trench, about 150 ft. downstream of the main trench. From a study of the log of drill holes it was noticed that this zone of highly disintegrated rock extended almost at right angles to the axis from the main cut off trench towards the downstream, with reducing width and depth for a distance of 150 ft. where it met the second cut off trench. The width and depth of the second cut off trench was about 20' and 10' respectively. Beyond this second trench, the zone moved down to a main joint. The disintegrated rock in the main cut off trench when tested gave a permeability coefficient of 30' per year.

Treatment of this zone consisted of excavation of the comparatively permeable disintegrated rock within the two cut off trenches and sealing the joints in the hard rock below by means of grouting and backfilling the trench with selected impervious soil and well compacted by means of pneumatic tampers to obtain the degree of compaction specified for the overlying fill.

In the main cut off trench, the excavation of this zone was taken down to R.L 462 ft. where hard and fresh rock was met with. Since the objective was to have a regular cut off parallel to the axis, the excavation was done in such a way as to form a narrow inner trench at lower elevations within the main trench as the bottom width is about 5 ft. The disintegrated rock was removed by means of pickaxes, crowbars and pavement breakers.

After the completion of excavation, the trench was backfilled with selected impervious soil in these layers by manual labour and well compacted by means of pneumatic tampers and carried on upto the ground level after which machinery were used.

Similarly in the second cut off trench, all the disintegrated rock was excavated in this zone, upto the level of hard rock (R.L 485 ft.) which involved 8 ft. depth of excavation. The trench was grouted and backfilled by manual labour upto the ground level after which machineries were used.

As the entire earth dam in this reach is to rest on a bed of rock, number of holes were drilled within the limits of the impervious zone beyond the two cut off trenches in order to ensure that the hard rock exists for a depth of at least 30' and not as local boulders as met with on the surface, they were broken and removed and the roots sticking in between were cleared. Light blasting was resorted to whenever necessary for breaking the boulders. Where there were large rock outcrops with open joints in the base of the impervious zone, these were cleaned and washed with air and water inserted and backfilled with clay slurry by pumping. In deep joints, pipes were inserted and the slurry pumped under pressure to fill up the pockets. (Source: Ibid Pg.17-18)

2.1.3.4 Preparation of Foundation on Rock Surface :

The earth dam section between RD 12000 ft and 13000 ft were entirely on rocky bed consisting of hard granite gneiss with average height of dam being 150 ft. In this area, after the initial stripping, two cut off trenches were excavated for about 2 ft. depth in hard and fresh rock. The average depth of excavation below river bed was about 5 ft. Impervious grout curtain was effected by pressure grouting along the length of cut off trenches in order to provide the sub-surface seal.

Loosened and cracked rock mass was removed by wedging and barring. Pressure washing of the foundation rock surface was done to assure a clean surface and remove the fines from seams and joints.

Clay slurry was then applied on the cleaned surface and worked well into the foundation depressions with the help of stiff brooms and when it was still wet, the impervious fill material was placed by means of baskets and tamped well. The slurry was intended to get an effective bond between the foundation rock and the fill material. Hand tampers as well as pneumatic tampers were used to lightly ram the soil and obtain the degree of compaction specified for the overlying fill. Narrow joints and fissures in the foundation were filled with the impervious material in different layers and compacted by means of rod tamping.

2.1.3.5 Left Dyke

The length of this Dyke is 32, 275 ft in 5 Gaps. Foundation grouting was done in Gap I and II where the maximum height of the dyke section is about 80 ft. In the other gaps, no grouting was done as the height is less than 50 ft.

In gap I which is an earthen section, cut off trench was excavated in the disintegrated rock and one line of curtain grout holes were drilled and grouted 40' centres in certain places where the hard rock was close to the surface. In the other reaches no grouting was done, but the cut off trench was taken deeper into the disintegrated rock layer which was impervious.

The Gap II comprised of a masonry section about 80' high with a base width of about 55 ft. The foundation consisted of hard and massive quartz schist with the strike parallel to the axis and dipping towards the upstream at 70° to horizontal. This was very favourable feature as it served as a natural cut off and minimized leakage.

Subsurface explorations and foundation treatment were done in a systematic manner. Diamond drill core holes were drilled in 2 rows parallel to axis of dam with holes 80 ft. deep at 20' centers in each row. Grouting was done at 50 psi.

One interesting feature of the foundation was the existence of a wide fissure almost at right angles to the axis and for the full width of the dam at the lowest foundation level in the bottom of the valley. This consisted of decomposed schist and a number of cavities formed probably due to the continuous flow of water through the ages. This was excavated for about 4' depth where fresh rock was met with for the entire length. This trench was extended about 5 ft. upstream of the axis. Inclined holes were drilled along this trench and grouted.



Entrance gate at Hirakud Pavilion

2.1.3.6 Drilling & Grouting Equipments :

Equipments used for drilling and grouting operation of both earth and concrete dam are furnished in Table No. 2.15.

Table 2.15 : Equipments used for Drilling and Grouting of Hirakud Dam:

Sl. no	Name and make of the Machine	No. of machines used	Type and make of engine	Drive	Feed	Bit speed rpm	Weight of machine
1	2	3	4	5	6	7	8
1	**1. Diamood Core Drill** (1) Craelius Type XF-4	4	Albino Petrol Engine Type AL-22	Petrol 10-HP	Automatic pressure controlled screw feed	250 500 & 750	520 kp (1150-lbs)
	(2) E.J.Long Year & Co	2	Wankesha petrol engine Model FC	4 cylinder water cooled heavy duty industrial type petrol engine 20-HP	Hydraulic feed (Oil) length of feed 24"	1800	1725
	(3) Canadian longyear Ltd.	1	Buda model 4 D-1226	4 cylinder water cooled, heavy duty industrial type diesel engine 28-HP	-do-	-do-	2710
	(4) Sprague & Henwood Type G-3	1	Willys jeep engine	4 cylinder water petrol engine 27-HP	Screw feed	1200	2500
	(5) Sprague & Henwood	1	Buda model B.M No.10785-A	4 cylinder water cooled, heavy duty industrial type 27-HP	Hydraulic (water) length of feed - 20"	1200	2500
	(6) Ingersoll rand calyx core drill type PT-3	1	Hercules ZXB	2 cylinder water cooled, petrol engine	Screw feed 24"	500	600
	(7) CP-55-A Chicago pneumatic	4	Rotary air motor 6 blades	Air 18-HP	Screw feed 24"	0 to 1500 & upto 2500 optional	160

II	**Wagon Drills** (1) Ingersoll Rand	6	WDX 71 Drifter on FM-2 mounting	Air Motor percussion	-	-	-
	(2) "Thor"	4	-do-	-	-	-	-
III	**Jackhammer**	2	CPT- & Atlas	-do-	-	-	-
IV	**Grouting Pumps** (1) Gardner Denver duplex, double acting reciprocating pump (Size: 7" x3" x10")	2	Air Motor	Air	-	-	-
	(2) -do- (Size: 4" x 6")	10	Caterpillar Diesel engine model D-3400	Diesel 27 HP	-	-	-
	(3) -do- (Size: 3" x 6")	2	Buda Engine Model	Petrol	-	-	-
V	**Compressors** (1) Broomwades 250 C.F.M	6	Gardner	Diesel	-	-	-
	(2)Consolidated Pneumatic 315 C.F.M	2	Jap engine	Diesel	-	-	-
	(3) Ingersoll Rand-315 C.F.M	1	-	Diesel	-	-	-

Note: - Each Grout Pumps was equipped with a Miner and Agitator mounted on SKIDS, manufactured in the local workshop

2.1.3.7 Permeability Tests in Masonry & Concrete

In order to have uniform standard of imperviousness, permeability tests were conducted periodically at different elevations, as the masonry and concreting progressed. Test holes 2" to 3" diameter, 20' to 50' deep were drilled into the finished masonry and concrete and tested with water under pressure. These were then grouted to refusal. These holes were drilled in 2 rows one row about 8 ft. from the axis of dam, and the second in the middle third section at the particular elevation. Results of consolidation grouting are given in Table No 2.4 and high pressure grouting in Table 2.5 & 2.6

In addition to these, the high pressure grout holes dilled through concrete and masonry from the sill of sluices or from floor level of gallery as the case may be were also used for grouting the concrete and masonry. General abstract of foundation grouting (1951-55) in furnished in Table 2.16 and results of Permeability Test in Table 2.17 to 2.20.

Table 2.16 General Abstract of Foundation Grouting etc. in Hirakud Dam Project (1951-1955)

Sl.no	Particulars of grouting	No of holes grouted	Footage grouted(ft)	Cement placed (bags)	Average take per ft.	Reference to table no
1	2	3	4	5	6	7
1	Low pressure consolidation grouting	3273	89160'-0"	42281.50	0.474	2.4
2	High pressure grouting at concrete Dam left spillway including curtain grout	115	7720'-2"	3841.46	0.407 (Rock) 0.830 (conc. & masonry)	2.6
3	Drainage holes in left spillway	61	3768'-1"	Nil	--	2.3
4	High pressure grout in right spillway & power dam	141	9180'-8"	2814.174	0.35 (Rock) 0.331 (conc. & masonry)	2.5
5	Permeability test in right concrete dam	39	1236.50	248.875	0.201	2.18
6	-do- in left concrete dam	29	1069.00	485.929	0.455	2.17
7	-do- in power dam	20	837'-8"	179.100	0.213	2.19
8	-do- in right spillway	31	716.00	78.75	0.110	2.20
9	Contact grouting in penstock block out (8 block) & 149 condulets	--	--	128.625	--	2.12
10	Grouting in penstock liner (7 units- 102 condulets)	--	--	335.625	--	2.10
11	Grouting in Draft tube liner (unit-3)	4	--	95.500	--	2.11
12	Grouting in penstock (1 to 4)	49	--	291.00	--	2.7
13	Exploratory tunnel condulets	14 (condulets)	--	56.00	--	2.8
14	Grouting in contraction joint	197	--	249.00	--	2.9
	Grand total **Holes** **Condulets** **Drainage hole**	 **3898 no** **265 no** **61 no**		51064.381 Say **51065 bags**		

Table 2.17 Result of Permeability Test in Masonry and Concrete at RL 555 ft. (Left conc.Dam) (Period August 1951 to September 1954)

Sl.no	Block / Hole No.	Location of hole	Depth of hole	Period of laying masonry & concrete	Permeability Test			Grouting		
					Pressure in P.S.I	Qty. in G.P.M	Qty. in G.P.M sq.ft	Pressure P.S.I	W.C Ratio	Bags of cement placed
1	2	3	4	5	6	7	8	9	10	11
1	Flank wall / L.C.L.F 1	24'6" from C.J of Blocks 10 & 11 and 54' D/S of axis	10'-9"	3/54 to 5/54	25	3.0	0.43	20-30	10:1	1.667
2	Flank wall / L.C.L.F 2	38'3" from C.J of block 10 & 11 and 38'9" D/S of axis	15'-0"	---do---	40	2.25	0.225	30	10:1	1.00
3	3 / L.C. 3-4	30' from C.J of block 2 & 3 and 46' D/S of axis	50'-0"	4/54 & 5/54	22.5	5.5	0.17	15-30	5:1/8:1	15.125
4	3 / L.C. 3-5	30' from C.J of block 2 & 3 and 8' D/S of axis	49'-6"	---do---	20	5.5	0.16	5-30	5:1/8:1	48.75
5	4 / L.C. 4-1	30' from C.J of block 3 & 4 and 14'-7" D/S of axis	8'-6"	---do---	20	2.0	0.208	20-25	5:1	0.75
6	4 / L.C. 4-2	27'-3" from C.J of block 3 & 4 and 40'-10" D/S of axis	28'-8"	---do---	25	1.23	0.065	25	5:1	9.5
7	4 / L.C. 4-3	4' from C.J of block 4 & 5 and 40'-10" D/S of axis	17'-5"	---do---	25	1.125	0.065	25	2:1	9.5
8	4 / L.C. 4-4	30'-7" from C.J of block 3 & 4 and 15' D/S of axis	50'-0"	---do---	27.5	5.75	0.174	20-30	5:1/10:1	7.25
9	4 / L.C. 4-5	30' from C.J of block 3 & 4 and 45' D/S of axis	50'-0"	---do---	26	3.5	0.1	10-30	5:1/10:1	13.75
10	5 / L.C. 5-1	28' from C.J of block 4 & 5 and 47'-3" D/S of axis	50'-0"	---do---	25	13.0	0.4	15-30	3:1/8:1	25.75
11	6 / L.C. 6-1	24'-8" from C.J of block 5 & 6 and 8' D/S of axis	50'-0"	---do---	17.5	6.25	0.19	6-25	1:1/8:1	29.0
12	6 / L.C. 6-2	30'-6" from C.J of block 5 & 6 and 39'-9" D/S of axis	50'-0"	---do---	28	3.333	0.10	25-30	6:1/8:1	6.25
13	7 / L.C. 7-1	30' from C.J of block 6 & 7 and 39'-9" D/S of axis	50'-0"	---do---	24	3.75	0.11	50-30	6:1/10:1	10.875

contd.....

Table 17 ... continued

1	2	3	4	5	6	7	8	9	10	11
14	7 L.C. 7-2	83' from C.J of Blocks 6 & 7' D/S of axis	50'-0"	4/54 & 5/54	20	4.3	0.13	0-25	0.75:1/8:1	36.75
15	8 L.C. 8-1	33' from C.J of block 7 & 8 and 7' D/S of axis	30'-0"	----do----	19	6.0	0.30	0-25	2:1/8:1	33.75
16	9 L.C. 9-1	33' from C.J of block 8 & 9 and 7' D/S of axis	50'-0"	----do----	14	8.0	--	0-30	1:1/8:1	52.75
17	9 L.C. 9-2	29' from C.J of block 8 & 9 and 40' D/S of axis	50'-0"	----do----	10	11.0	0.33	0-30	5:1/8:1	34.50
18	10 L.C. 10-1	30' from C.J of Blocks 9 & 10 and 19' D/S of axis	49'-2"	----do----	14	7.0	0.28	0-30	4:1/8:1	29.25
19	11 L.C. 11-1	30'-2" from C.J of Blocks 10 & 11 and 25'-11" D/S of axis	23'-0"	----do----	15	5.4	0.45	0-30	0.5:1/8:1	13.282
20	12 L.C. 12-4	29'-8" from C.J of block 12 & 11 and 8' D/S of axis	50'-0"	----do----	15	7.0	0.28	0-30	1:1/8:1	36.00
21	13 L.C. 13-1	38' from C.J of block 12 & 13 and 8' D/S of axis	24'-0"	----do----	24	3.0	0.25	20-30	6:1/8:1	4.5
22	14 L.C. 14-2	34' from C.J of block 13 & 14 and 8' D/S of axis	36'-0"	----do----	27.5	1.0	0.05	25	8:1 /	0.25
23	15 L.C. 15-1	34' from C.J of block 14 & 15 and 8' D/S of axis	29'-0"	----do----	27.5	7.0	0.47	0-30	2:1/8:1	14.23
24	16 L.C. 16-1	18' from C.J of Blocks 15 & 16 and 24' D/S of axis	36'-0"	----do----	20	5.5	0.30	10-30	5:1/8:1	11.25
25	17 L.C. 17-1	28' from C.J of block 15 & 16 and 24' D/S of axis	36'-0"	----do----	20	2.5	0.14	15-30	4:1/8:1	9.5
26	18 L.C. 18-1	30' from C.J of block 17 & 18 and 16'-10" D/S of axis	30'-0"	----do----	25	3.0	0.20	25	7:1/8:1	2.25
27	19 L.C. 19-1	29'-9" from C.J of block 18 & 19 and 15'-6" D/S of axis	36'-0"	----do----	30	0.5	0.029	25	8:1	0.25
28	20 L.C. 20-1	29'-8" from C.J of block 19 & 20 and 18'-5"" D/S of axis	30'-0"	----do----	17.5	7.0	0.116	0-30	2:1/8:1	12.75
29	21-1 L.C. 21-1	29'-1" from C.J of block 20 & 21 and 18' D/S of axis	30'-0"	----do----	12	7.66	0.51	0-30	1:1/8:1	15.5

Total no of holes grouted=29, Total footage grouted=1069, Total cement placed =485.929 bags, **Av.take / linear ft =0.455**

Table 2.18 Results of Permeability Test in Masonry and Concrete at Right Concrete Dam (Period June 1954 to October 1954)

Sl.no	Block / Hole No.	Location of hole	Depth of hole	Age of masonry	Permeability Test			Grouting		
					Pressure in P.S.I	Qty. in G.P.M	Qty. in G.P.M / sq.ft	Pressure P.S.I	W.C Ratio	Bags of cement placed
1	2	3	4	5	6	7	8	9	10	11
1	48 / 48 T-4	8' 6" from C.J of Blocks 48 & 49 and 10'-6" D/S of axis	31'-1"	2/54 to 5/54	30	0.10	0.16	30	10:1	5.5
2	48 / 48 T-5	20' from C.J of Blocks 48 & 49 and 8'-0" D/S of axis	32'-4"	---do---	30	3.0	0.11	30	10:1	3.5
3	48 / 48 T-6	31' from C.J of Blocks 48 & 49 and 8'-0" D/S of axis	30'-9"	---do---	30	5.00	0.20	30	10:1	6.00
4	48 / 48 T-7	51' from C.J of Blocks 48 & 49 and 7'-6" D/S	29'-6"	---do---	30	2.75	0.11	30	10:1	4.25
5	49 / 49 T-6	10' from C.J of Blocks 49 & 50 and 8' D/S	34'-3"	---do---	30	3.00	0.11	30	10:1	4.25
6	49 / 49 T-7	12' from C.J of Blocks 48 & 49 and 7'-6" D/S	40'-8"	---do---	30	3.0	0.112	30	10:1	3.5
7	50 / 50 T-4	8' from axis towards down stream 36' from C.Joint 51 & 50	20'-4"	3/54 to 5/54	25	3.00	0.23	25	10:1	3.25
8	50 / 50 T-5	8' from axis towards down stream 36' from C.Joint 51 & 50	19'-10"	---do---	30	1.5	0.12	30	10:1	3.375
9	50 / 50 T-6	8' from axis D/S 20' from C.Joint 50 & 51	19'-1"	---do---	30	1.0	0.077	30	10:1	4.5
10	50 / 50 T-7	7'-6" from axis 45' from C.Joint 50 & 51	25'-9"	---do---	30	1.0	0.077	30	10:1	0.625
11	50 / 50 T-8	7'-6" from axis towards D/S 6" from C.Joint 50 & 51	25'-5"	---do---	30	7.0	0.54	30	10:1	6.25
12	50 / 50 T-9	28' from axis towards D/S 6" from C.Joint 50 & 51	9'-8"	---do---	30	6.0	0.46	30	10:1	2.0
13	51 / 51 T-5	7' 6" from axis towards D/S 23' from C.Joint of 51 & 52	22'-6"	4/54 to 5/54	25	8.0	0.57	25	8:1	3.5

Contd.....

Table-2.18.... continued

Sl.no	Block / Hole No.	Location of hole	Depth of hole	Age of masonry	Permeability Test			Grouting		
					Pressure in P.S.I	Qty. in G.P.M	Qty. in G.P.M / sq.ft	Pressure P.S.I	W.C Ratio	Bags of cement placed
1	2	3	4	5	6	7	8	9	10	11
14	51 / 51 T-6	6'-8" from axis towards D/S 10'-2" from C.J of 51 & 52	21'-6"	3/54 to 5/54	25	6.0	0.46	25	8:1	2.00
15	51 / 51 T-7	6'-8" from axis towards D/S 29' from C.J of 51 & 52	20'-0"	---do---	25	8.0	0.57	25	8:1	3.0
16	51 / 51 T-8	7' from axis towards D/S 29' from C.J of 51 & 52	19'-3"	---do---	25	6.00	0.46	25	8:1	2.00
17	51 / 51 T-9	5' from axis towards D/S 10' from C.J of 50 & 51	22'-9"	---do---	30	2.5	0.2	30	10:1	3.0
18	51 / 51 T-10	30' from axis towards D/S 16' from C.J of 50 & 51	7'-10"	---do---	30	4.0	0.8	30	10:1	1.0
19	52 / 52 T-3	8' from axis towards D/S 23' from C.J of 52 & 53	22'-0"	---do---	30	10	--	25	8:1	4.75
20	52 / 52 T-4	8' from axis towards D/S 44' from C.J of 52 & 53	20'-10"	---do---	25	6	0.46	25	8:1	2.25
21	52 / 52 T-5	7' from axis towards D/S 10' from C.J of 52 & 53	23'-8"	---do---	25	10	0.71	25	8:1	6.00
22	53 / 53 T-2	8' from axis towards D/S 10' from C.J of 52 & 53	35'-10"	---do---	30	4	0.18	30	10:1	5.75
23	53 / 53 T-3	7'-6"' from axis towards D/S 30' from C.J of 52 & 53	32'-3"	---do---	30	6	0.30	30	10:1	10.25
24	53 / 53 T-6	Just on the axis and 30' from C.Joint block 53 & 54	54'-0"	2/54 to 6/54	37	0.375	1.4	30	10:1	2.75
25	53 / 53 T-4	7'-6" from axis towards D/S and 50' from C.Joint of block 52 & 53	32'-2"	---do---	30	16.0	--	30	10:1	9.25
26	53 / 53 T-5	Just on the axis and 10'-6" from C.Joint block 53 & 54	54'-0"	---do---	42.5	25.0	0.9	80.30	10:1	5.25
27	54 / 54 T-3	10' from axis towards U/S and 13'-6" from C.J. of block 54 & 55	16'-9"	---do---	10	22.0	2.0	10.25	8:1/6:1	8.75

Contd.....

Table 2.18.... continued

Sl.no	Block / Hole No.	Location of hole	Depth of hole	Age of masonry	Permeability Test			Grouting		
					Pressure in P.S.I	Qty. in G.P.M	Qty. in G.P.M / sq.ft	Pressure P.S.I	W.C Ratio	Bags of cement placed
1	2	3	4	5	6	7	8	9	10	11
28	54 / 54 T-4	Just on the axis and 10' from C.J of block 54 & 55	34'-0"	2/54 to 6/54	22.5	13.0	0.565	15.30	6:1/10:1	8.25
29	54 / 54 T-5	1'-6" from axis towards U/S and 21' from C.Joint of block 54 & 55	52'-0"	---do---	27.5	5.0	0.143	25.30	8:1/10:1	4.0
30	54 / 54 T-6	5' from axis towards D/S and 5 ' from C.Joint of block 54 & 55	39'-0"	---do---	28.5	8.0	0.3	20.30	8:1/10:1	12.25
31	54 / 54 T-7	Just on the axis line and 16' from C.J of block 54 & 55	43'-0"	---do---	30	7.0	0.24	30	10:1	4.5
32	54 / 54 T-8	7' from axis towards D/S and 13 " from C.J block of 54 & 55	52'-3"	---do---	30	10.0	0.45	30.80	10:1	10.75
33	54 / 54 T-9	Just on the axis line and 31' from C.Joint of block 53 & 54	57'-0"	---do---	30	3.75	0.10	30	10:1	8.0
34	54 / 54 T-10	8'-4" from axis towards D/S and 33' from C.J of block 53 & 54	33'-2"	---do---	30	5.0	0.25	30	10:1	12.5
35	54 / 54 T-11	Just on the axis and 41' from C.Joint of block 55 & 54	51'-0"	---do---	30	5.5	0.17	30	10:1	13.0
36	54 / 54 T-12	1' from D/S and 49' from C.Joint of block 55 & 54	33-0"	---do---	35	7.5	0.37	30.50	10:1	15.0
37	54 / 54 T-13	1' from axis D/S and 51' from C.Joint of block 53 & 54	54'-0"	---do---	35	7.5	2.0	50.30	10:1	37.0
38	54 / 54 T-14	1' from axis D/S and 62' from C.Joint of block 54 & 55	54'-0"	---do---	35	3.3	1.2	50.30	10:1	7.0
39	58 / 58 T-1	—	20'-0"	---do---	30	6.0	0.46	30	10:1	3.75

Total no. of hole grouted-39, Total cement placed-248.875 bags
Total footage grouted - 1236.5, **Average take / linear ft. -0.201**

Table 2.19 Result of Permeability Test in masonry (1:3 C.M) of Power Dam (Period August 1955 to September 1955)

Sl.no	Block / Hole No.	Location of hole	Depth of hole	Compr. Strength in P.S.I	Bulk sp.gr.	Permeability Test			Grouting		
						Pressure in P.S.I	Packer depth (ft)	Qty. in G.P.M	Pressure P.S.I	W.C Ratio	Bags of cement placed
1	2	3	4	5	6	7	8	9	10	11	12
1	48 T-8 / 48	9' 5" from C.J of block 47/48 17' 0" from axis towards D/S	60' 0"	--	--	30	Cap	3.5/2.5	25	6:1	8.0
2	49 T-8 / 49	15' 0" from C.J of block 49/50 16' 0" from axis towards D/S	60' 0"	--	--	30	Cap	4.5	25	6:1	7.75
3	49 T-9 / 49	12' 0" from C.J of block 48/49 16' 8" from axis towards D/S	35' 5"	--	--	30	10'/2'-0"	5.0	25-30	5:1-6:1	5.5
4	50 T-10 / 50	40' 0" from C.J of block 51/50 16' 8" from axis towards D/S	55' 0"	2784	2.24	30	Cap	7.5	25	6:1	10.0
5	50 T-11 / 50	6' 9" from C.J of block 49/50 15' 0" from axis towards D/S	4' 0"	--	--	30	1' 0"	0.5	30	6:1	0.1
6	51 T-11 / 51	9' 8" from C.J of block 50 & 51 15' 6" from axis towards D/S	60' 0"	--	2.20	30	Cap	7.0	25	6:1	9.75
7	51 T-12 / 51	5' 0" from C.J of block 50 & 51 25' 0" from axis towards D/S	30' 0"	--	--	30	Cap	3.0	25	6:1	4.0
8	51 T-13 / 51	9' 0" from C.J of block 14' 8" from axis towards D/S	38' 3"	3977/4094	2.24	25	Cap	4.0	25	6:1	7.0
9	52 T-6 / 52	11' 0" from C.J of block of 52 15' 6" from axis towards D/S	55' 0"	--	--	25	Cap	5.0	25	6:1	11.00
10	52 T-7 / 52	29' 6" from C.J of block 53/52 28' 0" from axis	35' 10"	5096	2.25	23	Cap	5.0	25	6:1	7.0

Contd.....

Table no- 2.19 (continued)

Sl.no	Block Hole No.	Location of hole	Depth of hole	Compr. Strength in P.S.I	Bulk sp.gr.	Permeability Test			Grouting		
						Pressure in P.S.I	Packer depth (ft)	Qty. in G.P.M.	Pressure P.S.I	W.C Ratio	Bags of cement placed
1	2	3	4	5	6	7	8	9	10	11	12
11	52 T-8 / 52	13' 6" from C.J of block of 51/52 14' 0" from axis towards D/S	58'-0"	2483/2455	2.24/2.21	30	Cap	5.0	25	6:1	5.0
12	53 T-7 / 53	33' 0" from C.J of block of 53/54 5' 0" from axis towards D/S	25'-0"	--	--	25	Cap	4.0/2.0	25	6:1	11.0
13	53 T-8 / 53	10' 8" from C.J of block of 54/53 15' 6" from axis towards D/S	50'-0"	--	--	25	Cap	6.0	15-25	6:1	10.0
14	53 T-9 / 53	32' 0" from C.J of block of 54/53 29' 0" from axis towards D/S	27' 0"	--	--	20	Cap	1.3	20-25	6:1	5.0
15	53 T-10 / 53	10' 6" from C.J of block of 52/53 18' 0" from axis towards D/S	35' 9"	--	2.23	25	Cap	10.0/4.0	25-25	6:1/6:1	20.0
16	54 T-15 / 54	9' 0" from C.J of block of 53/54 17' 0" from axis towards D/S	30' 9"	5000	2.29	20/25	Cap	5.0/3.0	20-30	6:1/6:1	13.0
17	54 T-16 / 54	27' 0" from C.J of block of 53/54 29' 6" from axis towards D/S	33'-0"	--	--	15/25	Cap	8.0/3.0	15-25	6:1/6:1	14.0
18	54 T-17 / 54	28' 0" from C.J of block of 53/54 5' 6" from axis towards D/S	30'-2"	--	--	20/30	10' / Cap	6.0/6.0	20-25	6:1/6:1	16.0
19	54 T-18 / 54	28' 0" from C.J of block of 53/54 5' 6" from axis towards D/S	56' 0"	--	--	30	Cap	6.5	25	6:1	8.00
20	54 T-19 / 54	7' 9" from C.J of block of 54/55 15' 9" from axis towards D/S	58' 6"	--	--	30	Cap	4.0	25	6:1	7.0

Total no. of holes grouted = 20, Total footage grouted = 837'-8"

Total cement paced = 179.10 bag, **Av. take / linear ft = 0.213**

Table 2.20 : Result of Permeability Test in Concrete at Right Spillway (Period September 1955 to October 1955)

Sl.no	Block / Hole No.	Location of hole	Depth of hole	Permeability Test			Grouting		
				Pressure in P.S.I	Packer depth in ft.	Qty. in G.P.M	Pressure P.S.I	W.C Ratio	Bags of cement placed
1	2	3	4	5	6	7	8	9	10
1	35 T-1 / 35	8' 0" from axis towards D/S, 13' 0" form C.J of block 34/35	30' 0"	30	10'/2' 0"	1.5/3.0	30	10:1	4.0
2	35 T-2 / 35	11' 0" from C.J of block 35/36, 7' 9" from axis towards D/S	20' 0"	30	10'/1' 0"	3.5/1.5	30	6:1	1.5
3	35 T-3 / 35	10' 0" from C.J of block 35/36, 7' 0" from axis towards D/S	20' 0"	30	10'/2' 0"	4.0/1.0	30	5:1/6:1	10.5
4	36 T-2 / 36	8' 0" from axis towards D/S, 30' 0" from C.J block 35/36	30' 0"	30	10'/2' 0"	1.5/Nil	30	6:1	11.25
5	36 T-3 / 36	8' 0" from axis towards D/S, 11' 0" from C.J block 35/36	20' 0"	30	10'/2' 0"	3.0/Nil	30	6:1	2.5
6	36 T-4 / 36	50' 0" from C.J block 35/36, 7' 0" from axis towards D/S	20' 0"	30	10'/2' 0"	1.0/1.5	30	6:1	1.0
7	37 T-1 / 37	10' 0" from C.J block 38/37, 6' 0" from axis towards D/S	20' 0"	30	10'/2' 0"	2.0/2.5	30	6:1	2.25
8	37 T-2 / 37	11' 2" from C.J block 36/37, 7' 8" from axis towards D/S	24' 0"	30	10'/2' 0"	Nil/3.5	30	6:1	6.00
9	37 T-3 / 37	38' 2" from C.J block 36/37, 7' 2" from axis towards D/S	24' 0"	25	10'/1' 0"	2.0/Nil	30	6:1	4.25
10	38 T-3 / 38	30' 6" from C.J block 37/38, 7' 0" from axis towards D/S	30' 0"	30	10'/1' 6"	3.0/1.0	30	6:1	1.5
11	38 T-4 / 38	10' 0" from C.J block 39/38, 7' 0" from axis towards D/S	20' 0"	30	10'/1' 0"	3.0/1.0	30	6:1	4.5
12	38 T-5 / 38	10' 0" from C.J block 37/38, 7' 6" from axis towards D/S	20' 0"	30	2' 0"	Nil	–	–	–

Contd.....

Table no 2.20 (continued)

Sl.no	Block / Hole No.	Location of hole	Depth of hole	Permeability Test			Grouting		
				Pressure in P.S.I	Packer depth in ft.	Qty. in G.P.M	Pressure P.S.I	W.C Ratio	Bags of cement placed
1	2	3	4	5	6	7	8	9	10
13	39 T-1 39	30' 0" from C.J of block 38/39, 6' 0" form axis towards D/S	30' 0"	30	10'/2' 0"	1.0/Nil	30	6:1	0.50
14	39 T-2 39	8' 5" from C.J of block 40/39, 10' 0" form axis towards D/S	20' 0"	30	10'/2' 0"	Nil	--	--	--
15	39 T-3 39	10' 0" from C.J of block 38/39, 5' 9" form axis towards D/S	20' 0"	30	10'/2' 0"	Nil	--	--	--
16	40 T-1 40	35' 6" from C.J of block 39/40, 6' 0" form axis towards D/S	30' 0"	30	10'/1' 0"	Nil/1.0	30	6:1	0.25
17	40 T-2 40	10' 0" from C.J of block 40/41, 9' 0" form axis towards D/S	20' 0"	30	10'/1' 0"	--	--	--	--
18	40 T-3 40	10' 0" from C.J of block 39/40, 10' 6" form axis towards D/S	20' 0"	30	10'/1' 0"	2.0/Nil	30	6:1	1.0
19	41 T-1 41	29' 4" from C.J of block 40/41, 6' 0" form axis towards D/S	30' 0"	30	10'/2' 0"	Nil	--	--	--
20	41 T-2 41	9' 0" from C.J of block 42/41, 7' 4" form axis towards D/S	20' 0"	30	10'/2' 0"	2.0/Nil	30	6:1	1.5
21	41 T-3 41	10' 4" from C.J of block 40/41, 7' 0" form axis towards D/S	20' 0"	30	10'/1' 0"	2.5/Nil	30	6:1	2.5
22	42 T-4 42	32' 0" from C.J of block 41/42, 8' 0" form axis towards D/S	30' 0"	30	10'/2' 0"	3.0/Nil	30	6:1	7.5
23	42 T-5 40	10' 0" from C.J of block 43/42, 7' 8" form axis towards D/S	20' 0"	30	10'/1' 0"	Nil	--	--	--
24	42 T-6 42	12' 0" from C.J of block 41/42, 7' 8" from axis towards D/S	20' 0"	30	10'/2' 0"	Nil/1.0	30	6:1	0.5
25	43 T-1 43	30' 0" from C.J block 42/43, 7' 0" from axis towards D/s	30' 0"	30	10' 0" / 2" 0"	3.0/1.2	30	6:1	4.25

Contd.....

Table no 2.20 (continued)

Sl.no	Block / Hole No.	Location of hole	Depth of hole	Permeability Test			Grouting		
				Pressure in P.S.I	Packer depth in ft.	Qty. in G.P.M	Pressure P.S.I	W.C Ratio	Bags of cement placed
1	2	3	4	5	6	7	8	9	10
26	43 T-4 / 43	8' 6" from C.J block 44/43, 7' 0" from axis towards D/s	20' 0"	30	10' 0" / 2" 0"	2.5/Nil	30	6:1	2.25
27	43 T-5 / 43	9' 0" from C.J block 42/43, 7' 0" from axis towards D/s	20' 0"	30	10' 0" / 1" 0"	Nil/Nil	--	--	--
28	44 T-20 / 44	9' 6" from C.J block 43/44, 7' 0" from axis towards D/s	20' 0"	30	10' 0" / 2" 0"	Nil/Nil	--	--	--
29	44 T-21 / 44	30' 6" from C.J block 43/44 ,7' 0" from axis towards D/s	30' 0"	30	10' 0" / 1" 0"	3.0/2.0	30	6:1	3.25
30	44 T-22 / 44	48' 6" from C.J block 43/44, 7' 0" from axis towards D/s	20' 0"	30	10' 0" / 1" 0"	Nil/Nil	--	--	--
31	45 T-2 / 45	29' 0" from C.J block 45/44, 20' 6" from axis towards D/s	18' 0"	30	2" 0"	4.0	30	5:1/6:1	6

Total no. of holes grouted = 31, Total footage grouted = 716 ft.

Total cement paced = 78.75 bag, **Av. take / linear ft = 0.201**

CHAPTER - III

PROJECT CONSTRUCTION

3.0. Commencement of Project Work :

3.1 Laying of Foundation Stone :

Years of planning, investigation and strong political will paved way for the construction of probably the longest dam in the world then. After years of patient and persevering hard work, the engineers, planners, specialists, technicians and workers gave a gigantic and magnificent gift to the people of Orissa in particular and to the country in general in the form of the Hirakud Dam. This multipurpose river valley project heralded a new chapter in the history of development of the state by way of irrigation, flood control, power generation, industrialization, pisciculture and recreation etc. This is the story of the stupendous cooperative endeavour of man against nature for the benefit of mankind.

Government of India took keen interest to develop the vast water resources of Mahanadi for mitigating the devastating flood and drought which was occurring almost every year. The havoc caused by floods and droughts does not end with the destruction of life and property at the time, it leaves a legacy of sapped vitality of the people, of choked drainages and stagnating pool and of low income diseases to which people fall an easy prey. Thus for narrowing the regional imbalances, Government of India extended all assistance to go ahead with the project even during the pre-plan period.

Foundation stone was laid by Sir Hawthorne lewis, Governor of Orissa on 15.03 1946 in the presence of Dr. A.N. Khosla, Chairman, CWI & NC and Mr. Prior, representative of Govt. of India. Their speeches on the occasion of Foundation laying ceremony are enclosed herewith vide Annexures 3.1, 3.2 & 3.3.

The original estimated cost of the Project was Rs47.81cr. which comprises cost of (i) Dam & appurtenant works (ii) Main Canal, Branches, Distributaries & Water courses (iii) Hydro-electric installation and (iv) lump sum provision of Rs 1.0cr for navigation. The abstract of cost estimate is furnished in Annex.3.4

The cost under items Main canal & branches, Hydro-electric installation and Navigation will be a direct charge to the respective items of development. The final allocation of cost worked out as follows :

Flood Control	6.11	=	Rs 6.11 cr.
Irrigation	3.32 + 7.80	=	Rs.11.12 cr.
Power	6.73 +22.85	=	Rs 29.58 cr.
Navigation(L.S.)	1.00	=	Rs 1.00 cr.
	Total	=	**Rs 47.81 cr.**

Sub-head wise breakup has been furnished in Table-3.1.

(Source : Mahanadi Valley Development, Hirakud Dam Project, CWI & NC, June 1947, pg.46)

The revised cost of project worked out to Rs.89.09cr. and its break-up under different units is as follows (vide Project Report 1952 Chapter VIII pg.114)

(i)		Dam & Appurtenant works			Rs 34.39 cr
(ii)		Main Canal, Branches Distributaries & water courses			Rs 16.52 cr
(iii)	(a)	Generation			
		Civil works			
		Power House No.1	-	1.83cr	
		Power channel	-	2.84cr	
		Subsidiary Dam	-	Rs.3.78 cr.	
		Power House No.II	-	Rs.0.38 cr.	
		Power Plant	-	Rs.11.37 cr.	
	b)	Transmission	-	16.98cr	
		Sub total	-	Rs.37.18 cr	Rs.37.18 cr.
iv)		Navigation	L.S		Rs. 1.00 cr
		Total	-		**Rs. 89.09 cr.**

"The Revised Estimate was submitted by the Chief Engineer in January 1952. In the same month, the Government of India appointed a three-man Advisory Committee (hereafter referred to as the **Mazumdar Committee**) to review the estimates. This Committee examined the estimates and conseqently new cost estimate of Rs.92.08 cr. emerged. xxxx The Mazumdar Committee generally approved the estimates of irrigation, dam design, canal layout etc. It suggested changes in water rates which were far in excess of what the report on revised estimates had proposed." (Source: Economics of A Multiple-purpose River Dam - Report of an inquiry into the Economic benefits of Hirakud Dam by N.V. Sovani & Nilakantha Rath - Gokhale Institute of Politics & Economics, Pg.11)

But as per the brochure of Hirakud Project (13th January 1957), the cost of 3 components including Delta irrigation, Chiplima Power Scheme stands at Rs. 100.02 cr.

a) Stage I (Irrigation in Sambalpur Dist and Main Power House)		Rs. 70.78 cr
i) Dam & appurtenant works	45.50	
ii) Canal	8.12	
iii) Power generation and transmission	16.27	
iv) Others	0.89	
Sub-total	70.78	Rs 70.78 cr
b) Delta Irrigation		Rs 14.92 cr
c) Subsidiary Power House at Chiplima		Rs 14.92 cr
	Total	**Rs 100.02 cr**

3.1.1 Financial Arrangement :

The Hirakud project which was conceived in 1945 was sanctioned by the Government of Orissa on 8th April 1948 and the 1st batch of concrete of the project was laid by the Hon'ble Pt. Jawahar Lal Nehru, the first Prime Minister of India on 13th April 1948. The construction work of the project was commenced in September of the same year.

Since the Government of Orissa, so far as the state finance was concerned, was not in a position to incur any expenditure either towards the capital or interest charges, it was decided that the Government of India should advance the capital necessary for the construction of the project as a loan to the Orissa Government on certain terms of agreement, to be repaid by the State Government. The terms & conditions of the financial arrangements between the Central Government and the Government of Orissa, as agreed to by both the Governments are reproduced below :

(i) The Government of India would be prepared to advance the capital necessary for the construction of the Hirakud Dam Project as a loan to the Orissa Government on terms usually prescribed for such loans (vide tabloid of policy changes in central budgeting and loans to Provinces for development expenditure). Even though it would be legitimate for the Orissa Government to treat a considerable proportion of the capital as a permanent debt of the Province, it would not be possible for the Central Government to grant a permanent loan on rates of interest fixed in perpetuity, since the Government of India themselves cannot raise permanent loans in that manner. It would therefore be necessary for the provincial Government to approach the Govenment of India for the renewal of loans in accordance with the procedure laid down for this purpose. During the period of construction and for such further reasonable period as may be required for the project to begin to produce a return, the interest on capital may also be treated as an addition to capital expenditure. The period after construction during which interest may be charged to capital will be subject to the decision of the Government of India.

(ii) The Government of India do not consider it necessary to make an outright grant to the Provincial Government to the extent of cost allocated to flood control. Though for the purpose of preparing estimates of cost, separate estimates have been prepared of the cost of flood control aspect of the project, it is neither desirable nor necessary to separate this amount from the total cost of the project for the purpose of financing arrangements. It is not correct to say that the flood control measures will be totally unremunerative, as they will inevitably save the Government of Orissa the losses of revenue due to floods apart from saving losses to land and property. Even if this indirect but substantial return is left out of consideration, the preliminary estimates show that the project as a whole would be remunerative and no question of treating any part of the expentiture as an outright grant by the centre may arise.

(iii) The Government of India agree with the Provincial Government in considering that the present preliminary estimates of the cost of the Project cannot be treated as final. There must be periodical reviews and revisions of the financial estimates and forecasts as the work progresses. The entire financial prospects of the project should finally be reviewed about five years after its completion, when the question of any financial assistance from the centre, whether in the shape of an outright grant or a concession in the rate of interest on a part of the capital or otherwise, should be dealt with in the light of the allocation of the revenue resources between the centre and the provinces under the new constitution, the financial relations between the centre and the Orissa Government at that time and other relevant considerations.

3.1.2 Appointment of a High level Committee of Experts

It is necessary to appoint a committee consisting of Dr. J.L. Savage, Consulting Engineer and International authority on high dams, Mr. L.Nclellan, Chief Electrical and Mechanical Engineer, US Bureau of Reclamation, Mr. M.Narasimhaiya, Chief Engineer, Mysore and Mr.S.A Gadkari, Chief Engineer, Electricity, East Panjab to tender advice on the project in general and on its detailed features. Sri M. Visvesvaraya will also be requested to offer advice generally on any matters referred to him by the Committee.

3.1.3. Establishment of Central Designs Organisation under CW & PC :

The Central Water and Power Commission have not been able to prepare detailed designs on account of inadequate staff position. It is proposed to set up a Central Designs Organisation. The Hirakud Dam Project will be one of the many projects which this organisation will have to deal with. As the establishment of this organisation will take some time, part of the Designs work may have to be done by a foreign agency, as for the Bhakra, Dochni and Rampadasagar dams.

3.1.4 Allocation of Cost

The main objectives of the Dam Project are Flood Control, Irrigation and Power generation. The cost of the main dam and its appurtenant works should, therefore, be appropriately divided amongst various uses. The 1947 estimate followed the purpose basing on the capacities required for above three purposes if the dam were built exclusively for the purpose. The allocation was therefore in the ratio of 37.83 : 20.56: 41.61 among F.C., Irrigation & Power respectivly. As the scope of the project had undergone change, the ratio was no longer applicable.

Revised Estimate (1952) adopted a different principle. The basis was the ultimate utility of water for the various purposes. This eliminated the flood control as a separate entity and merged it with irrigation on the score that Delta Irrigation would be possible only with F.C. Thus the costs were divided between Irrigation & Power in the ratio of 71:29 respectively.

But the Mazumdar Committee (1952) had a different view. The flood absorptive function of the reservoir has already been worked out on the basis of availability of 3 M Ac.ft. (0.3699Mham) storage space. The water required for power & irrigation have also been calculated separately and found to be 3.2 Ac.ft (0.3946 M ham) for Irrigation and 2.4 M.Ac.ft (0.2960 M ham) for Power. Therefore, It may be reasonably assumed as 3 M.Ac.ft (0.3699 M ham) for any of the three purposes. But the irrigation requirements for Sambalpur area has been arrived at as equivalent to one-third of the total by allowing for greater seepage and transmission losses due to upland and steeply sloped country. Thus the ratios of 1:3:3: for Sambalpur area Irrigation, Power and F.C appear to be a fair allocation.

After taking up Delta Irrigation Scheme which would command 5.36 lakh acres (2.169.lakh ha.) new land and about 50% of 5.41 lakh acres (2.189 lakh ha.) of existing command area, the final allocation worked out as 1:1.5: 1.5:3 for Sambalpur Irrigation, Delta Irrigaton, Power and Flood Control. The F.C. allocated cost, if merged with Delta irrigation share of cost, would make the delta scheme thoroughly unremunerative (vide Revised Estimate of Hirakud Dam Project- 17th Nov.1953, pg 36-39)

3.1.5. Proceedings of the Foundation Laying Ceremony (on 15th March 1946).

Annexure 3.1

(Speech by the Hon'ble Mr. Prior representing the Government of India)

Your Excellency, Ladies and Gentlemen,

I take it as great honour that I have been asked to represent the Gonvernment of India at this ceremony. That honour has fallen to me because the Hon'ble Dr. Ambedkar, the labour Member, has not found it possible to leave his duties in the Legislature to come here today and I am sure, we all very much regret his absence. He has howerver, sent the following message which he has asked me to read:

Dr. Ambedkar's message

"It is a matter of deep regret to me that my legislative duties prevent my presence at the laying of the foundation stone of Hirakud dam on the Mahanadi, but I am asking the Secretary of my Department to convey personally to Your Excellency and the people of Orissa and neighbouring

states how keenly interested I am in the successful completion of the project of which the ceremony which you will shortly perform marks the formal beginning. Much hard and strenuous work over many years lies ahead of us, and as I said at the Mahanadi Conference at Cuttack in November last, the policy for the unified development of the Mahanadi River, which has been already accepted by the Government of India and Government of Orissa, can be successfully implemented in practice only by the full and continuous cooperation of the two Governments. The laying of the foundation stone of at least one dam pledges the support of the Government of India to this policy, and will, I hope, be at the same time a lasting symbol of the earnestness of the two Governments, irrespective of their future political complexion, to co-operate in a high endeavour to serve the cause of the people of Orissa and of the neighbouring States."

Government of India's assistance assured

Your Excellency, during last few years, the Government of India has been taking increasing interest in the efforts being made throughout this vast continent of India to secure the regional and multi-purpose development of our waterways and I welcome the opportunity which this ceremony gives me of affirming the Government of India's desire to give all such assistance as is required and as is possible to all Provinces and States through whose territories flow the rivers of our country, so that the development of these water resources on regional and multipurpose lines may be advanced and their waters used for the maximum benefit of all.

A step forward

Today, Sir, you are going to take a step forward towards the completion of a great scheme of harnessing the Mahanadi- this great river with capacity for so much good if controlled and for so much harm if uncontrolled. We are not here today to consider the utilization when control has been secured, of the waters and the power generated by them— that will be the subject of future negotiation; but we are here today because I think we have all one common determination to push forward by our co-operative efforts, the multipurpose development of this great river so that it may lay for ever the twin spectres of drought and famine in the areas it can be made to serve.

Three tell - tale figures.

Your excellency, I do not wish to enter on technical details as I shall in a few minutes ask the Chairman of the CWI & NC to speak briefly on those aspects. It is the business of those who will design this great scheme to ensure that in the future the waters of the Mahanadi are used day in and day out for good and given no opportunity of causing harm. That cannot be done-as we see it otherwise than by the construction of dams to create large storage basins.

The essence of the scheme

That is the essence of the scheme we are initiating today— river control by creation of storage. There will be difficulties and obstacles to overcome. We have been told in the benefit from storage schemes such as this will be short-lived because of silt difficulties and the cost of dam construction is bound, therefore, to be unremunerative. I propose to leave these matters to be dealt with by Mr. Khosla. Further planners of the scheme will have to take particular care of such problems as malaria control and maintenance of fisheries whether in the river itself or the Chilka Lake. Problems will also arise in regard to lands to be submerged. It may be argued that these areas will be large and that the damage so caused will offset the benefits to be secured from the project. xxxx But whatever be the difficulties they must be overcome so that the vast benefits derived from perennial irrigation, power, flood control and navigation are secured as early as possible.

Hirakud— a prelude to early completion of the whole project

Today we are considering only this first dam here at Hirakud. The site is one which to us laymen must seem almost ideally situated for its purpose and all here must be looking ahead into

the future when the river is here at this spot controlled so that its waters may give continuing benefit.

Your Excellency, with your permission I will now ask Rai Bahadur A.N. Khosla, Chairman of the CWI & NC which organisation is responsible for investigation of this project, to give some account of its technical aspects.

Annexure 3.2

Speech by Rai Bahadur A.N. Khosla , Chairman, CWI&NC

Your Excellency, Ladies and Gentlemen

It is a great privilege to be associated with this most momentous event in the history of Orissa and its neighboring States. In laying the foundation of the Hirakud Dam, Your Excellency will be laying foundation of a new policy in the harnessing of the vast resources of the rivers of India- a policy of close co-operation and collaboration between the centre, the provinces and the states for the control, conservation and regulation of the water and waterways of India with a view to their planned utilisation for multipurpose development and for the maximum benefit of the people irrespective of territorial boundaries.

The background—spectres of flood and drought,

Before I come to the technical details of the Mahanadi Valley Project and its most important unit the Hirakud Dam, I would like to give a connected picture of the background from which this great project has emerged so that its necessity and urgency may stand out in true perspective.

The rivers of Orissa carry enormous quantities of water each year which are allowed to run to waste and cause destruction and disease in their passage to the sea. The distribution of water supplies through the year suffers from maladjustment. There is too much water during the rains and little of it during the rest of the year with the result that "Orissa is haunted by two spectres of flood and drought and of these the later is the more terrible."

Previous to the inception of the Orissa canal system, howsoever limited its scope, droughts and famines were of frequent occurrences. Historical records show that terrible famines occurred in the 14th, 15 th and 16 th centuries. In the memorable famine of 1770, people are reported to have been dying by hundreds and thousands. Nearly a century later came the great Orissa famine of 1865-66. The rainfall in 1865 was scanty and ceased prematurely. The food crops failed and it was estimated that nearly one million people died in the district of Cuttack alone. In the district of Puri, nearly 40 percent of population perished. Then followed the flood of 1866. Crops and property were destroyed and what the drought had spared was engulfed in the wide flood waters. Hundreds of square miles were sumberged from 5 to 45 days with water standing up to 10 ft. deep. Over a million people were rendered homeless.

The district of Sambalpur which in earlier years of drought had suffered only scarcity and acute distress, was in the grip of famine of 1900.

The havoc caused by floods and droughts does not end with the destruction of life and property at the time. It leaves a legacy of sapped vitality of the people, of choked drainages and stagnating pools and of "low income" diseases to which the people fall an easy prey.

That great pioneer in irrigation engineering— Sir Arthur Cotton - remarked in 1858 in connection with the control and utilisation of the Mahanadi river "and then observe a district like Orissa one dreary alternation of unmitigated drought and flood without a single indication of improvement in any respect." The situation has not improved since. Droughts and floods still recur with sinister frequency.

The flood problem

Of recent years the flood problem in Orissa has however, engaged more attention than the problem of drought. But now with the spectre of famine haunting wide areas in India, the latter problem has again assumed great importance. Orissa continues to be a province of one crop and a precarious one at that. With the introduction of irrigation, two crops can be raised instead of one and the area under crop considerably extended.

With regard to the flood problem, various committees have been appointed and conferences held during the last 18 years. Their deliberations mainly covered two proposals; firstly, the construction of dams at suitable locations on the tributaries and possibly on the main rivers to store the peak flood flows and thus reduce their intensity within safe limits; and secondly to construct double embankments with high level escapes, along the rivers almost down to their mouths, with a view to conserve and improve the main rivers, and prevent breaches likely to lead to the deterioration of the main channels. The question of suitable drainage of the basins between the rivers was also considered. A third proposal of diverting the Mahanadi into the Chilika Lake was also considered but rejected for various reasons.

The proposal for the construction of dams was negatived for the reasons that firstly, the dams and reservoirs will at last in part be outside Orissa territory, secondly, the expenditure on their construction will be prohibitive and thirdly, their life will be short due to heavy silting up. Sir M.Visvesvaraya in his note of 1937 sounded a note of warning against this negative attitude and suggested further and adequate investigations.

From palliative to cure

The problem was referred to me as Chairman of the newly constituted Waterways, Irrigation and Navigation Commission in April 1945. I visited Orissa in May of that year. As a result of local inspections and discussions with Mr. B.K. Gokhale, Adviser and Rai Bahadur Brij Narayan, Chief Engineer, I came to the definite conclusion that the only cure for the troubles of Orissa, namely floods, drought, poverty and disease, lay in the control of its rivers by means of storage dams to provide flood control, irrigation, navigation, power generation, fish culture and recreational facilities, coupled with effective anti-malarial measures. I felt that an immediate start should and could be made on investigations for the multi-purpose development of the Mahanadi which held promise of the greatest benefits to the people of Orissa and the neighbouring States.

The proposal for the investigation of the Mahanadi for multi-purpose development, received general approval of the Central Government, Orissa and the neighbouring States at a meeting of their representatives held at Cuttack on 8th November 1945, under the chairmanship of the Hon'ble Dr.B.R. Ambedkar. At this meeting it was agreed that the CWI & NC would undertake surveys and investigations.

A picture of the whole scheme

The Mahanadi Valley Project as at present envisaged, comprises the construction of three dams on the Mahanadi river with provision for power generation and of three canal systems taking off at these dams for purposes of irrigation. The first dam will be at Hirakud, 9 miles above Sambalpur. The second dam will be located at Tikarpara some 130 miles downstream and the third near Naraj about 10 miles upstream of Cuttack.

The lake formed by the Naraj will extend to the Tikarpara dam and the lake formed by the Tikarpara dam will extend to Sonepur about 60 miles below Hirakud. The project will provide for navigation facilities from the head of the lake formed by the Hirakud dam right down to the sea — a distance of over 300 miles - by providing locks at the three dams and negotiating the fall between Hirakud dam and Sonepur by means of one or two weirs fitted with navigation locks.

The Hon'ble Mr. Prior has already told you that the Mahanadi is one of the biggest rivers of India and the biggest in Orissa, that it has a catchment area of 51,000 square miles above its debouch into the delta and that in floods it may rise to over 15 lakhs of cusec. In the dry weather the flow in this river may fall to as low as 200 cusec. This river carries an annual flow variously estimated between 60 million and 90 million acre feet. This annual flow is about as large as that of the great Indus river at Kalabagh and four times that of the Sutlej river at Bhakra or the Colorado river in U.S.A. across which stands the highest dam in the world - the Boulder dam with its largest man made lake. Out of these 60 to 90 million acre feet of annual flow it is proposed to impound roughly one sixth or just over 10 million acre feet each year. It is proposed to cosnstruct the three dams at Hirakud, Tikarpara and Naraj for a gross storage of 20 million acre feet. This storage will be divided into three parts. The one at the bottom of the reservoir, called the "dead storage" will never be depleted and will account for 5 million acre feet and provide a silt reserve which will trap almost the entire coarse silt of the Mahanadi for a period of over one hundred years without causing any diminution in the useable or live storage at these reservoirs. This provision of silt reserve will meet one of the major objectons against storage dams as a solution of the Orissa problem, namely that their life be very limited due to silting of the reservoirs. Of the remaining 15 million acre feet, 10 million will be available for use in perennial irrigation and power development and the balance of 5 million at top as flood reserve.

The shape of things to come

In this way it will be possible to trap all harmful sands at the reservoirs, afford complete flood protection to the delta areas in Orissa, extend perennial irrigation to over 2.5 million acres both in Orissa and the adjoining States; generate cheap hydro-electric power possibly 2 lakh K.W.; provide navigation facilities from the sea to the head of Sambalpur reservoir and create extensive lakes for fish culture and recreational use. With cheap hydro-electric power it will be possible to exploit the great mineral wealth of Orissa and the neighbouring States, pump out water from waterlogged areas for their reclamation and for extension of irrigation and otherwise to extend the amenities of life to the urban and rural areas. The project will include adequate provision for the effective control and eradication of malaria. With this scheme fully developed, poverty and disease should be a thing of the past and Orissa and its neighbouring States, now among the most backward will emerge wealthy and prosperous and stand as the equal of the more prosperous Provinces and States of India. The subsequent control and development of the two other major rivers, the Brahmani and the Baitarani and the two small rivers the Burhabalang and the Subarnarekha would add further to the prosperity of Orissa and its neighbouring States.

Hirakud dam project the most attractive

Having presented the picture of the Mahanadi Valley development as a whole, I now proceed to deal., in particular, with the Hirakud dam project the uppermost of the three component units. Fortunately each one of the three units is capable of independent and stage development without getting out of joint with or impairing the integrated development of the Mahanadi Valley as a whole. Of these three, the Hirakud Dam project is the most attractive as well as free from engineering and political complications. It is likely to yield the earliest results.

The dam and the reservoir area are situated exclusively within the territory of the Orissa Province. The area to be irrigated on either bank lies in Orissa in the head reach, but extends further south in the Eastern States so that the latter can join in at any stage.

Further, there is ample water in the river. Out of an annual run-off of nearly 40 million acre feet at Hirakud, only about 1.5 million acre feet will be stored for purposes of irrigation and power development. Thus no question of water rights will be involved.

A preliminary geological investigation shows that the foundation rock at Hirakud is composed of granite gneiss, quartzites and phyllites which constitute suitable foundations for high dams.

The Hirakud site is the most accessible by rail as well as by road so that there should be no difficulty about transport of materials.

Lime-stone is reported to be available in the neighbourhood. If investigations prove it to be of good quality and in sufficient quantity, it will be a good case for setting up a cement factory at the place to supply cement for the construction of the dams on the Mahanadi, their appurtenant works and works on the three canal systems. This, if possible, will effect large savings on the project as cement is one of the most expensive items of expenditure in dam construction. This cement factory will be a permanent asset to the province of Orissa and the adjoining areas.

In the post-war scheme of development, it is proposed to extend the Jharsugada-Sambalpur railway across the Mahanadi to link with the Cuttack-Madras and the Vizianagram-Raipur railways. It is also proposed to construct a national highway through Sambalpur over a combined road and railway bridge. The construction of the Hirakud dam may have an important bearing on the final location of the combined road and railway bridge. Similarly the layout of the proposed Sambalpur-Sonepur-Khurda road will be determined by the extent of the storage reservoirs at Tikarpara and Naraj.

From a consideration of the above factors and other technical details, it will be obvious that the Hirakud dam must be taken up for construction first.

The Hirakud dam will be about 100 feet above rock bed with a reservoir level of 610 feet above mean sea level. The area submerged by the reservoir will cover 130 square miles, but out of this, with the depletion of the reservoir, a great proportion will be available for raising one crop during the year.

In respect of surveys and investigations, the present position is that the aerial survey of the Hirakud dam site and reservoir area has been completed. The ground control for the aerial survey is in progress. The contour survey of the area to be irrigated will be taken up shortly by the Survey of India Department both in Sambalpur district and, with their permission, in the States. A preliminary geological investigation has been done. The detailed investigation with pits, drill holes, drifts and tunnels will be shortly undertaken as soon as the exploratory equipment becomes available.

Preliminary designs and estimates of cost will be taken in hand shortly. The programme of construction will be planned after the results of the preliminary investigations and studies become available.

After the detailed plans are ready it will be advisable to proceed with the construction of canals simultaneously with the dam and if necessary somewhat in advance so that irrigation can start as soon as the dam is built up to some height. The Project may take about five years to construct.

Annexure 3.3

Address of His Excellency Sir Hawthorne Lewis, the Governor of Orissa

To mark this most auspicious occasion His Excellency, the Viceroy has honoured us with a personal message. Let me read it to you :

His Excellency the Viceroy's message.

"The foundation stone of the Sambalpur Dam which His Excellency, the Governor of Orissa will lay today marks the beginning of a great enterprise which will, I am confident, bring inestimable benefit to Orissa. It will irrigate a large area of fertile soil and will begin the great task of transforming that headstrong giant the Mahanadi into a well-behaved benefactor of a poor and deserving province. It will also, I hope, provide lasting evidence of the desire of the Government

of India to help this province on the path towards security and prosperity."

The Viceroy's deep personal interest in Orissa.

On behalf of my Government, of the people of Orissa and of myself, I express to his excellency Lord Wavell, our deep appreciation of his kind and inspiring message. This is not the first occasion on which he has shown himself a real friend of Orissa. Within a few weeks of assuming office, he visited us in the midst of our distress. We were then at the close of 1943 when many hundreds of our people were still suffering and were with difficulty sustained against the privations to which they were exposed. I have no doubt that Lord Wavell's mind was deeply disturbed by what, with his own eyes, he then saw of our poverty. Its immediate relief was his first concern. I shall not recount all that was then considered and planned; but in our two Wavell Homes for orphan and destitute children at Berhampur and Balasore in our coastal areas, we have a lasting memorial of the Viceroy's deep personal interest in the welfare of the poorest of our people.

The impossible now within reach.

But man's will is unconquerable and engineering science and skill have not stood still. What has hitherto seemed impossible now comes within our reach. In other countries these forces of nature have been successfully overcome and great rivers have been tamed and brought under control and converted from agents of destruction into the instruments of prosperity. By the scheme we are-launching today for controlling the waters of the Mahanadi river, we hope that flood, drought and famine will be banished and that instead, we shall be secured against these terrible disasters which unendingly visit and revisit us, and that by this great creative effort our poverty in Orissa may be removed and may no longer be a byword to others and an abomination to ourselves. This, as the Viceroy has said, is indeed the beginning of a great enterprise and we shall pursue it vigorously and energetically to its successful conclusion.

Deeply indebted to the Government of India.

We are aware of the very special interest he (Dr. Ambedkar) has been taking in this scheme which was first put forward at the conference in Cuttack over which he himself presided last November. In particular we value the assurance given in Dr. Ambedkar's message that the laying of the foundation stone of this, the first of the three dams, pledges the support of the Government of India to the policy of the unified multi-purpose development of the Mahanadi by the full and continuous co-operation of the two Governments. My Government are deeply indebted to the Government of India for the most generous help they are giving us in the furtherance of this project which would otherwise be far beyond the narrow limits of our own resources.

We are greatly obliged also to Rai Bahadur A. N. Khosla, Chairman, CWI&NC for the lucid speech in which he has explained the technical aspects of this scheme. It was in Mr. Khosla's own mind that the scheme originated. He is its author and its begetter. I make hold to say that in the days to come his name will be long remembered for his great service to Orissa. He is not overstating the case when he describes the laying of the foundation stone as a momentous event in the history of Orissa and of the neighbouring States. But it will be momentous not only in the annals of Orissa, but also because it marks the beginning of a new all-India policy for harnessing the resources of the great rivers throughout the country.

To the people of Sambalpur

To the people of Sambalpur, so well represented at this gathering, I am myself no stranger. They more or less adopted me as one of themselves at the time some twenty years ago when I was their district officer. My stay here was a very happy period in my own service and I believe they are aware of the affectionate regard that I have always had for them. Within the next few days I shall be leaving Orissa. It gives me great satisfaction that in laying the foundation stone of this dam,

which will bring them so much prosperity, I may have been able before I leave to do at least something to repay them for all that they did for me when I was one among them and lived here in Sambalpur as their Deputy Commissioner and their friend.

Mr. Gokhale's contribution.

I endorse the acknowledgements paid by Mr. Khosla to the distinguished officers who have helped him in the preparation of this scheme, and who are engaged in the further investigation of its details. I would wish to add my own tribute to the part taken by Mr. Gokhale, my Adviser. His contribution has been invaluable and he has devoted himself unremittingly to the interest of Orissa not only in this scheme but in the entire five year plan which at all points bears the impress of his own far sighted understanding of our needs. Great help has been given also by our Chief Engineer, Rai Bahadur Brij Narayan, who, though on leave, has been enabled by the kindness of Mr. Prior to accompany him here by air from Delhi in order to be present on this great occasion.

His Excellency's hopes for the future of Orissa.

Now let us all remember that our work does not begin and end with laying a foundation stone. In his message Dr. Ambedkar has reminded us that much hard work is to be done before this great scheme comes into being. Today's ceremony is an important occasion, but still more important will be the next occasion, when the dam is complete and when with further due ceremony it will be brought appropriately into operation to serve the beneficent purpose for which it has been built. It will be for my successor, Sir Chandulal Trivedi, who will so shortly take over from me, to perform that ceremony and I hope it will not be long delayed.

Mr. Prior, I thank you for coming here today; I thank you for the kind words you have said. I most readily accept the invitation which you on behalf of the Government of India have conveyed to me that I should lay this foundation stone. It is my firm conviction that this dam when built will fulfil our highest hopes and will be the source of great prosperity and contentment to our people. It is my pleasure, my privilege and my pride to declare that the foundation stone of this dam will now have been well and truly laid under these most favorable auspices supported with the good wishes of each one of you assembled here at this morning's gathering. This is a great day in the history both of Sambalpur and of Orissa. May great success attend the labours of those who will now work to build this dam.

Annexure 3.4

Hirakud Dam Project : Original Estimate of Costs

General Abstract

A.	Dam and Appurtenant works	Rs.16.16 cr.
B.	Main canal, Branches, Distributaries and Water courses	Rs. 7.80 cr.
C.	Hydro Electric Installation	Rs.22.85 cr.
D.	L.S	Rs. 1.00 cr.
	Total	**Rs.47.81 crores**

Sub-head wise break up of Dam, Canal and Power system are as follows :

Table No. 3.1 Abstract of cost

Sl. No.	Subhead	Dam and Appurtenant works (Rs.)	Main Canal & Branches (Rs.)	Hydro-Electric Installation (Rs.)
I	A. Preliminary	15,00,000	10,00,000	10,000
	B. Land	5,00,00,000	16,00,000	21,00,000
	C. Works	7,44,00,000	-	4,00,00,000
	D. Spillways / Regulators	1,30,00,000	8,10,000	-
	E. Falls	-	3,60,000	-
	F. C.D. Works	-	29,60,000	-
	G. Bridges	-	21,10,000	-
	H. Escapes	-	1,60,000	-
	K. Building	25,00,000	15,00,000	70,00,000
	L. Earthwork	-	4,00,00,000	-
	M. Plantation	1,00,000	1,00,000	-
	O. Miscellaneous	37,50,000	1,06,04,000	10,000
	P. Maintenance	9,00,000	5,80,000	-
	Q. Special Tools & Plants	50,00,000	3,00,000	-
II	Establishment	87,00,000	72,12,000	29,72,000
III	Tools & Plant	9,50,000	5,80,000	4,91,000
IV	Suspense	Nil		
V	Receipts on capital investments (Subtract)	(-)16,00,000	-	-
VI(1)	Capitalisation of abatement of land revenue	8,00,000	1,86,000	-
VI(2)	Audit charges	16,00,000	5,00,000	5,17,000
	Distributaries	-	70,00,000	-
	Water Courses	-	4,38,000	-
	Unforeseen Works	-	-	49,00,000
	A. Power Generation	-	-	10,50,00,000
	B. Transmission	-	-	3,55,00,000
	C. Transmission	-	-	3,00,00,000
	Total	**16,16,00,000/-**	**7,80,00,000**	**22,85,00,000**

3.2 Design:

Building such a gigantic water resources project is a herculean task. Several years of planning, crossing hurdles and overcoming obstacles on many fronts are lost before you are rewarded with tangible progress, smooth construction and timely completion. The fact that the project could be completed was due to the sustained efforts of vast army of planners, designers, construction engineers, millions of skilled and unskilled workers and co-operation of people of th State and country as a whole.

Studies of river problems were carried out since several years prior to 1948. The project was launched in 1948 through combined efforts of CWI&NC (GoI) and Goverenment of Orissa. While CWI&NC assured the engineering feasibility, State Goverenment assured the future adminstrative and economic passibilities.

The designs and specifications for the main dam were drawn up by the Central Designs Organisation of the CWI&NC (subsequently changed to CWPC and now bifurcated as CWC and CEA). Preliminary Designs were carried out in U.S.A. Finally some details of work on the main dam and all the designs and specifications for the works of the canal system were drawn up by the Project Design Origanisation at site. But it is unfortunate that due to poor documentation, design details are neither available in CWC or with project authorities. This is the state of affair for all old projects constructed during 50's.

Hirakud, the longest dam in Asia comprises of 15748ft (4801.22m or 4.8km) long earth, masonry and concrete dam flanked by earthen dykes of 32270 ft (9838.41m) in length in five gaps spanning hillocks on left and 35500 ft (10820.40m) on the right in one stretch. The maximum height of masonry dam is 200 ft (60.98m) and earth dam is 195 ft (59.45m). In power Dam portion, there are 7 penstocks of 23 ft (7.0m) to 25 ft (7.6m) diameter which were the largest in Asia and far East then. The general Plan & Elevation of the main dam including both left and right spillways and Power dam is enclosed vide Drg. No.3.1. The quantities of the main items of work and the materials used in the main Dam are as follows:

Table 3.2 Quantities of major items in the main dam

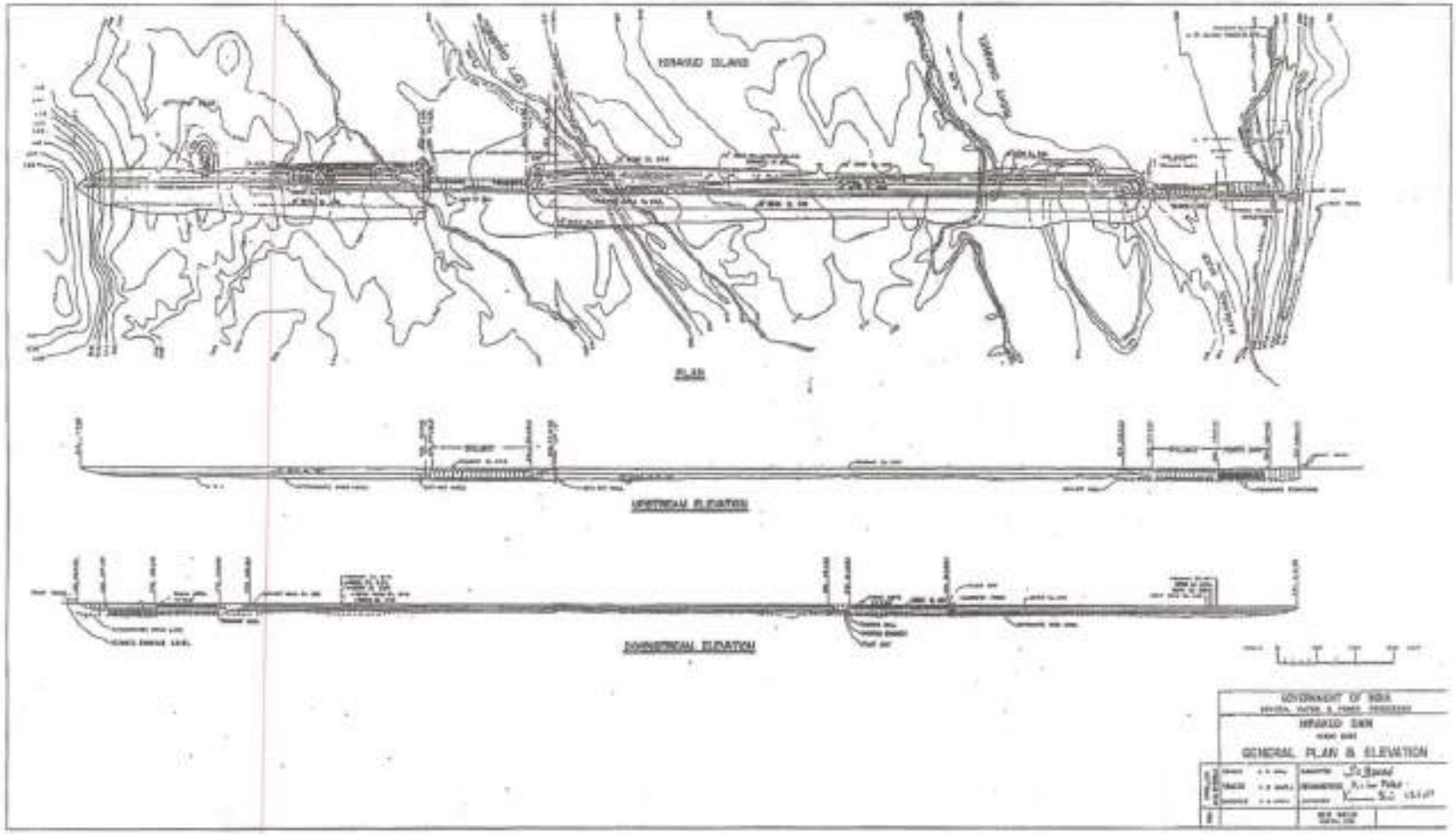
(i)	Earthfill in earth dam section	12.5 Mcum	(441.44Mcft)
(ii)	Excavation of cut-off trench in soil and rock	2.37 lakh cum	(83.70 lakh cft)
(iii)	Excavation of foundation for concrete and masonry dam in soil and rock	6.39 lakh cum	(22.57 Mcft)
(iv)	Concrete and masonry in Dam & power house	11.07 lakh cum	(39.09 Mcft)
(v)	Quantity of cement utilised	2.50 lakh ton	-
(vi)	Quantity 0f reinforcement steel	11000 M.T.	-
(vii)	Quantity of riprap	4.72 lakh cum	(16.67 Mcft)
(viii)	Quantity 0f filter (sand and shingles)	4.17 lakh cum	(14.73 Mcft)
(ix)	Quantity 0f rock fill	4.48 lakh cum	(15.82 Mcft)

(Source : Indian Journal of Power and River Valley Development, Vol.VI,No.10 pg.7)

From the above figures, the enormity of work involved can easily be gauged. Besides, it was difficult and complicated to tackle the foundation problem due to peculiar geological strata, heavy sub-soil water necessiating large scale pumping and vagaries of weather.

3.2.1 Earth Dam

Earth dam is of zoned rolled fill type with filter and rip rap on the upstream slope and filter and rockfill toe at the down stream. The top width of the dam is 25 ft (7.6m) with 3.1 slope on the upstream and 2:1 slope on the downstream. The slope of upstream semipervious zone varies from 3:1 to 2:1 and downstream semi-pervious zone varies from 1:1 or 1.5:1 to 2:1. The core or the impervious zone is sandwiched on either side with semi-pervious ones. COT has been provided in the impervious zone at the junction of upstream semipervious zone. Filter blanket at the base of downstream and rockfill at the toe with a longitudinal toe drain has been provided. Stone rip rap of 3 ft (0.91m) to 4 ft (1.20m) has been provided on the upstream slope with 18" (o.45m) to 2ft (0.6m) filter underneath. On the downstream slope above the rockfill, there is stone pitching and C.C. slab in part height, C.C. slab pitching in the top portion and turfing in between.



Drg. No. 3.1

COT of 50ft (15m) base width has been taken 2ft (0.6m) into the impervious soil or into hard rock under the seat of the dam in the impervious zone at its upstream junction which runs from one end to the other. Incase of deep channel width of COT has been increased to 30m. For the middle channel portion, two COTs one 15m wide and the other 9m (30ft) wide were provided to cut-off seepage at and below the base of the dam.

3.2.1.1 Principles of Design:

The criteria adopted for design of earthen embankments are summarised as under :

a) The discharge capacity of the undersluices and spillways be adequate enough so as to eliminate all chances of overtopping.

b) The line of saturation should lie within the downstream toe.

c) The upstream and downstream slopes must be so consructed that for the particular material used in the construction, the slopes will be stable under all conditions.

d) There must be no scope for free passage of water through the embankment.

e) Water which seeps through and under the embankment must have a velocity, at the point where it rises to the surface, so small as to be incapable of dislodging any material of the embnkment or of the foundation.

f) The free board must be sufficient enough to safe guard against overtopping due to wave action.

An earth dam designed and constructed to satisfy above criteria will obviously prove to be durable, permanent and non problematic in future.

3.2.1.2. Design assumptions :

The soil characteristics considered for stability analysis of typical section of earth dam are furnished below :

Table 3.3 : Soil Characteristics considered in Design

Properties of soil	*Impervious soil (lbs/cft)*	*Semipervious soil (lbs/cft)*	*Rock fill soil (lbs/cft)*
i) Dry weight	112	116	110
ii) Moist weight	130	154	-
iii) Saturated weight	132	137.5	-
iv) Submerged weight	69.5	75	68
v) Angle of internal friction (degree)	20	31	35
vi) Cohesion (lbs/sft)	1000	700	-

N.B.- 1lb=0.4536 kg (or 1 kg = 2.2046 lb.)

The semi-pervous zone material is asumed to drain out rapidly when reservoir level goes down.

Critical condition for upstream slope is sudden draw down condition of reservoir form 630 ft to 590 ft. Critical condition for down stream slope is maximum water level in the upstream and complete draw down of tail water to RL 532 ft.

The foundation material is assumed to have same characteristics as the embankment material over lying it.

Factor of safety:

Upstream slope 1.67 (GL EL.510ft)

Down stream slope 1.70 (GL EL 500ft)

(Refer Approved Drg.No.10 & 11 June 1951 of HDP)

3.2.2. Concret / Masonry Dam :

It is designed as straight gravity dam. The overflow section of the dam has a vertical upstream face, crest elevation RL 610 ft with parabolic curve beyond which rear face is with 0.7 (H) : I (V) slope. Sill level of the sluice is at RL 510 ft.(155.45m). Over the crest there exists road bridge with piers taken upto RL 634 ft (193.24m) and road level at an elevaton of 642 ft (195.68m). At the rear of the sluices, there is 44 ft(13.41m) wide ski-jump bucket for formation of hydraulic jump and dissipating the energy due to very high velocity through the sluices. The non-overflow section of the dam has a front batter of 0.1 (H) : 1 (V) upto elevation 550 ft and above that level, the face is vertical. The rear of the dam has a slope of 0.7 (H) : I (V) with constant width of 25ft (7.6m) above elevation 609.29 ft. (185.71m).

The NOF section of the dam including Power Dam and training walls are of granite stone masonry. About 5 ft (1.5m) thick concrete has been laid in the foundation under masonry. All the openings in the struture are R.C.C. For penstock openings, it is 5 ft (1.5m) thick and for other openings the thickness is 3 ft (o.91m) of R.C.C.

Basic data considered for design :

M W L - 630 ft.

Tail water level 535 ft

Unit weight of concerete 150 lbs/cft

Uplift - Full hydrostatic pressure at the heel decreasing uniformly towards toe and acting on 2/3rd of the area of section cosidered. The sliding factor (SF) and shear friction factor (SFF) has been computed at elevation RL 600 ft, 580 ft, 560 ft, 510 ft and 490 ft. The sliding factor for OF section varies for different elevations from 0.41 to 0.74 (safe limit not more than 0.75) and SFF from 76.4 to 10 (safe limit not less than 5). (Refer approved Drg.No.12 & Annex.IX of HDP)

3.2.2.1 Seismic Forces :

The structure site falls under zone III as per IS 1893/1994 for which seismic coefficient is 0.04. With importance factor of 3, the horizonatal seismic coefficient works out to 0.12. The corresponding vertical coefficient is 0.06. But during design earthquake forces were not considered for evaluating stability analysis of dam.

Stability Analysis of spillway section and Power Section has been carried out with Upstream water level at 630ft, 631 ft and 632 ft, (vide Report of the Evaluation Committee, HDP, 1962 pg.50). Free board for U/S water level at EL.630 & 632 ft for concrete dam works out to 10.64 ft in both the cases against which actually 12ft has been provided for EL 630 ft and 10 ft for EL.632ft. But for earth dam free board requirement is 12ft for both upstream water elevations against which 12ft provided for EL.630 ft and 10 ft for El. 632 ft. The stresses developed on upstream and downstream is enclosed vide Table No. 3.5. Stresses with FRL at 630, 631 and 632 ft developed on radial gates of spillway is appended in Table No 3.4.

Following drawings of earth dam, masonry/concrete dam (OF & NOF section, Power dam), including gates and accessories etc. are enclosed.

i) Drg. 3.2 Stability analysis of Main earth dam (Typical Section) - 2 sheets.

ii) Drg. 3.3 Stability analysis of Main earth dam (maximum section U/s slope)- 2 sheets.

iii) Drg. 3.4 Sections of earthdam for varying depth of water retention - 1 sheet.

iv) Drg. 3.5 Typical sections and details of main earth dam - 1 sheet.

v) Drg. 3.6 Typical sections of earth dykes - 1 sheet.

vi) Drg. 3.7 Stability analysis of Power Dam Section - 1 sheet.

vii) Drg. 3.8 Stability analysis of Spillway Dam section - 1 sheet.

viii) Drg. 3.9 Plan & section of hoist deck - 1 sheet.

ix) Drg. 3.10 General installation of sluice gate - 1 sheet.

x) Drg. 3.11 General installation of radical gate - 1 sheet.

xi) Drg. 3.12 Section of Spillway - 1 sheet.

xii) Drg. 3.13 Maximum Power Dam Section - 1 sheet.

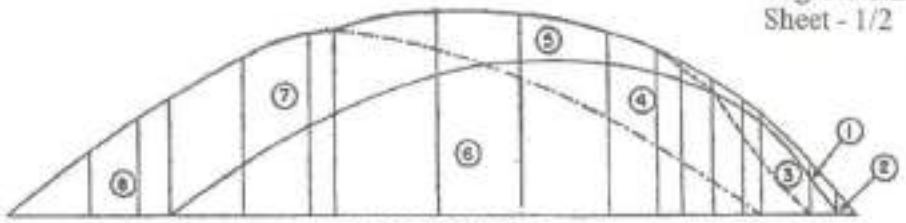
xiii) Drg. 3.14(A) to (E) Schematic diagram of canal network.

TABLE NO 3.4

Spillway Radial Gates of Hirakud (Approx, Stresses in Psi)

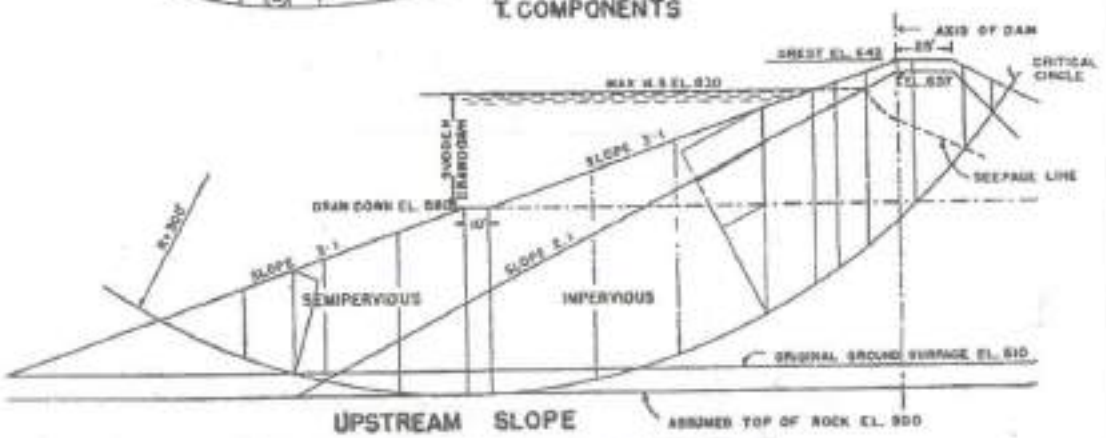
	Name of member	Stresses with F.R.L. @		
		630ft	631ft	632ft
1.	Skin plate direct bending	17,800	18,700	19,600
	combined stress	19,660	21,388	23,002
2.	Vertical stiffener bending	15,800	21,900	27,400
3.	Horizontal girder bending	17,950	20,900	23,900
4.	End Arms Compressive stress	12,950	15,900	18,200
	Compressive stress while lifting	15,000	17,900	20,200
5.	Anchor girder shear	11,800	12,980	14,160
	Bending	12,400	13,640	14,880
6.	Tie Rods-Tension	15,650	17,210	18,780
7.	Embedded girder			
	i) Bearing on concrete	284	312	350
	ii) bearing one gate open	470	517	564
	iii) bending stress	17,200	18,920	20,640
	iv) shear stress	8,130	8,943	9,756

Drg. No.3.2
Sheet - 1/2



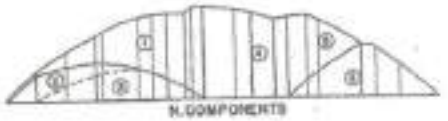






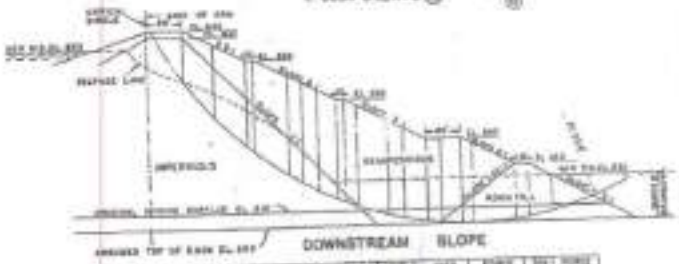
S.NO.	MATERIAL		UNIT Wt. lbs/cft	AREA SQ. FT.	FORCE KIPS	TAN Φ	FORCE KIPS
	N. FORCES						
1	SEMIPERVIOUS	MOIST	134	368	49	.364	18
2	SEMIPERVIOUS	MOIST	134	48	6	.4	4
3	IMPERVIOUS	MOIST	130	720	94	.364	34
4	IMPERVIOUS	SUBMGD.	89.3	3072	275	.364	78
5	SEMIPERVIOUS	MOIST	134	2268	307	.364	112
6	IMPERVIOUS	SUBMGD.	89.3	10144	705	.364	258
7	SEMIPERVIOUS	SUBMGD.	75	4840	548	.364	187
8	SEMIPERVIOUS	SUBMGD.	75	8960	150	.4	60 / 978
	COHESION		C lbs/sf	L feet			
	IMPERVIOUS		1000	550			550
	SEMIPERVIOUS		700	80			56 / 606
	T. FORCES						
9	SEMIPERVIOUS	MOIST	134	448	60		
10	IMPERVIOUS	MOIST	130	755	98		
11	IMPERVIOUS	SATURATED	138	2428	281		
12	SEMIPERVIOUS	MOIST	134	816	109		
13	IMPERVIOUS	SUBMGD.	89.3	2232	197		
14	SEMIPERVIOUS	SUBMGD.	75	100	15		
15	SEMIPERVIOUS	SUBMGD.	75	900	-72.-		
16	IMPERVIOUS	SUBMGD.	89.3	144	-10 / 673		

$$\text{FACTOR OF SAFETY} = \frac{\Sigma N \tan \Phi + \text{cohesion}}{\Sigma T} = \frac{1129}{675} = 1.67$$









DESIGN ASSUMPTIONS

(1) SOIL CHARACTERISTICS USED IN ANALYSISP

CHARACTERISTIC	IMPERVIOUS MATERIAL	SEMIPERVIOUS MATERIAL	ROCKFILL
DRY WEIGHT- LBS. PER CU.FT.	108	116	118
MOIST W=EIGHT-LBSPER CU.FT.	130	154	-
SATURATED WEIGHT-LBS.PER CU.FT.	130	137.5	-
SUBMERGED WEIGHT-LBS.PER CU.FT.	63.5	75	68
ANGLE OF INTERNAL FRICTION	23	32	35
TANGENT	0.384	0.8	0.7
COHESION-LBSPER SQ FT.	1000	700	-

(2) THE EFFECT OF PRESSURE HAS BEEN ALLOWED RELATIVELY FOR THE PORTION OF THE IMPERVIOUS ZONE OF EMBANKMENT WHICH IS SUBJECT TO SUDDEN DRAW DOWN.SUBMERGED UNIT WEIGHT HAS BEEN ADOPTED FOR CALCULATING NORMAL COMPONETS AND SATURATED UNITWEIGHT FOR CALCULATINGTANGENTIAL COMPONETS.

THE SEMIPERVIOUS ZONE MATERIALIS ASSUMED TO DRAIN OUT AS RAPIDLY AS RESERVOIRLEVEL IS DRAWN DOWN.

(3) CRITICAL CONDITION FOR DOWNSTREAM SLOPE SUDDEN DRAW DOWN OFRESERVOIR LEVEL FROM MAX W.S.EL. 639.00 TO EL.590.00P

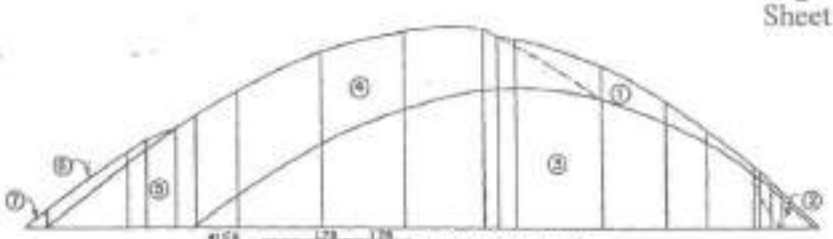
(4) CRITICAL CONDITION FOR DOWN STREAM SLOPE MAX W.S.EL. 639.00 UPSTREM COMPLETE DRAW DOWN OF TAIL WATER FROM MAX.W.L.526.00 TO RIVER BED.

(5) THE FOUNDATION MATERIAL IS ASSUMED TO HAVE THE SAME CHARECTERISTICS AS THE EMBANKMENT MATERIALIVERLAYING IT.

GOVERNMENT OF INDIA
CENTRAL WATER & POWER COMMISSION
HIRAKUD DAM
MAIN EARTH DAM
STABILITY ANALYSIS
TYPICAL SECTION

SUBMITTED [illegible]
RECOMMENDED K. L. Rao
APPROVED [illegible]

Drg. No.3.3
Sheet - 1/2



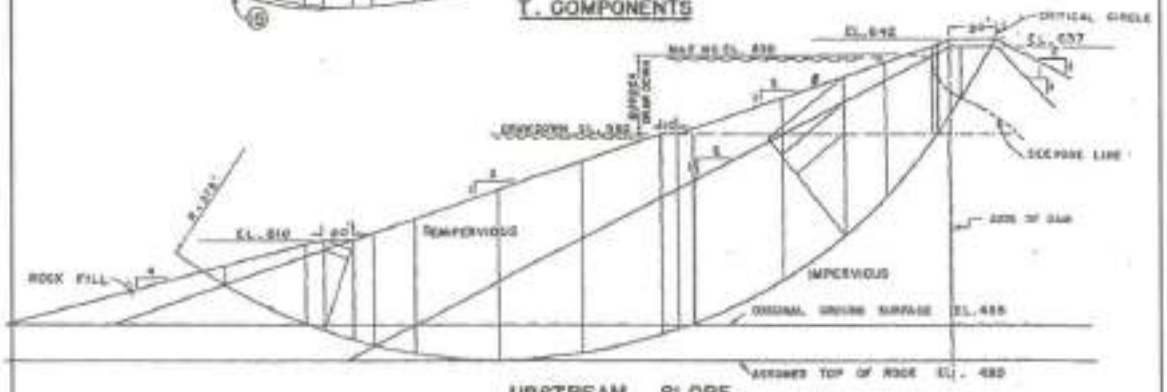
N. COMPONENTS

FACTOR OF SAFETY

AT CENTRES OF CRITICAL CIRCLES



T. COMPONENTS



UPSTREAM SLOPE

RESISTING FORCES

N. FORCES

1.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
2.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
3.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
4.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
5.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
6.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
7.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

CONCRETE

[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]

SHEARING FORCES

T. FORCES

8.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
9.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
10.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
11.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
12.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
13.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
14.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
15.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

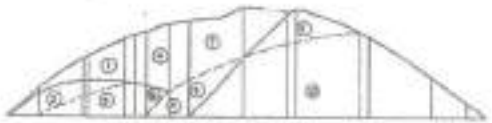
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GOVERNMENT OF INDIA
CENTRAL WATER & POWER COMMISSION

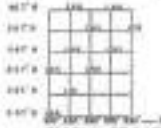
HIRAKUD DAM
MAIN EARTH DAM

STABILITY ANALYSIS
MAXIMUM SECTION

DRAWN	S.A.M.	SUBMITTED	[illegible]
TRACED	A.P.M.	RECOMMENDED	K. L. Rao.
CHECKED	R.A.I.	APPROVED	[illegible] 23.6.51
		NEW DELHI JUNE 1951	



N COMPONENTS

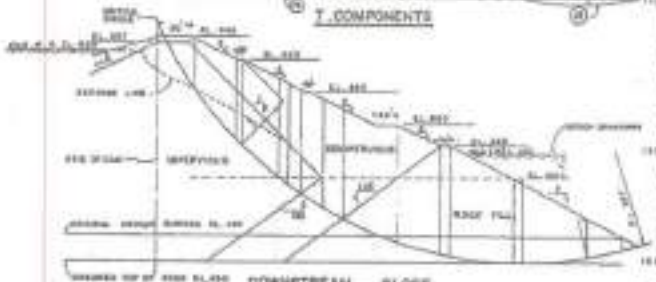


FACTOR OF SAFETY

AT CENTRES OF TRIAL CIRCLES



T COMPONENTS



DOWNSTREAM SLOPE

RESISTING FORCES

1.				
2.				
3.				
4.				
5.				
6.				
7.				
8.				
9.				
10.				
11.				
12.				
13.				
14.				
15.				
16.				

SHEARING FORCES

1.				
2.				
3a.				
3b.				
4.				
5.				
6.				

FACTOR OF SAFETY = $\frac{\text{RESISTING FORCES}}{\text{SHEARING FORCES}} = \frac{\quad}{\quad} = 1.05$

DESIGN ASSUMPTIONS

CHARACTERISTIC	IMPERVIOUS MATERIAL	SEMI-PERVIOUS MATERIAL	RESERVOIR
DRY WEIGHT - LBS. PER CU. FT.	110	95	110
BOAT WEIGHT - LBS. PER CU. FT.	120	105	-
SATURATED WEIGHT - LBS. PER CU. FT.	130	120	-
SUBMERGED WEIGHT - LBS. PER CU. FT.	63.5	58	62.5
ANGLE OF INTERNAL FRICTION - ϕ	23°	30°	0°
"COHESION" c	0.385	0.4	0.2
VOID RATIO - LBS. PER CU. FT.	0.432	0.35	-

(1) THE EFFECT OF PORE PRESSURE HAS BEEN ALLOWED FOR AS FOLLOWS:

FOR THE PORTION OF THE IMPERVIOUS ZONE OF THE CROSS-SECTION WHICH IS SUBJECT TO SEEPAGE CONDITIONS, SUBMERGED UNIT WEIGHT HAS BEEN USED FOR CALCULATING NORMAL COMPONENTS AND SATURATED UNIT WEIGHT FOR CALCULATING TANGENTIAL COMPONENTS.

THE SEMI-PERVIOUS ZONE MATERIAL IS ASSUMED TO DRAIN FULLY SO THAT NO PORE PRESSURE LOSS IS TAKEN INTO ACCOUNT.

(2) NORMAL CONDITIONS FOR UPSTREAM SLOPE: SUDDEN DRAWDOWN OF RESERVOIR LEVEL FROM MAX. RES. EL. 880 TO EL. 540

(3) NORMAL CONDITIONS FOR DOWNSTREAM SLOPE: MAX. RES. EL. FOR STEADY SEEPAGE; SATURATE DOWNSTREAM OF THE CORE TOWN AND T.W. EL. 320 TO EL. 410.

(4) THE FOUNDATION MATERIAL IS ASSUMED TO HAVE THE SAME CHARACTERISTICS AS THE IMPERVIOUS MATERIAL, OTHERWISE IT.

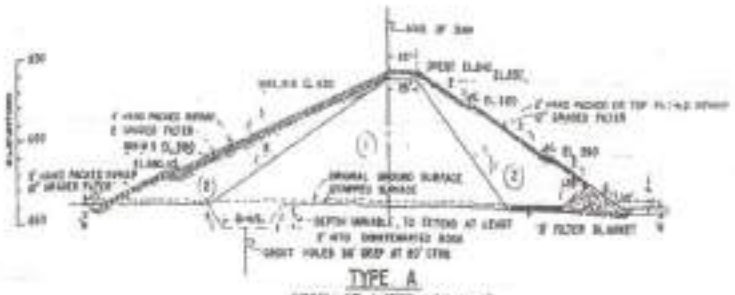
SCALE 0 50 100 FEET

GOVERNMENT OF INDIA
CENTRAL WATER & POWER COMMISSION

HIRAKUD DAM
MAIN EARTH DAM
STABILITY ANALYSIS
MAXIMUM SECTION

DRAWN S.K.M.	SUBMITTED
TRACED S.P.M.	RECOMMENDED
CHECKED S.A.I.	APPROVED

Drg. No.3.4



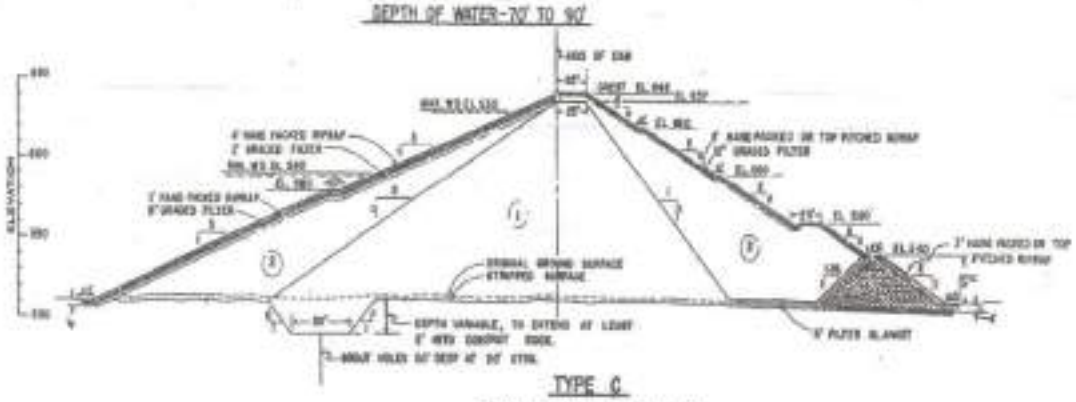
TYPE A

DEPTH OF WATER-UPTO 70'



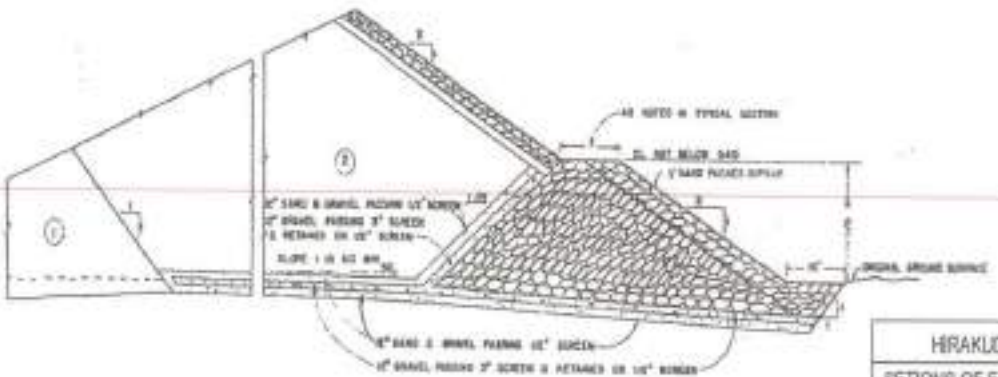
TYPE B

DEPTH OF WATER-70' TO 90'



TYPE C

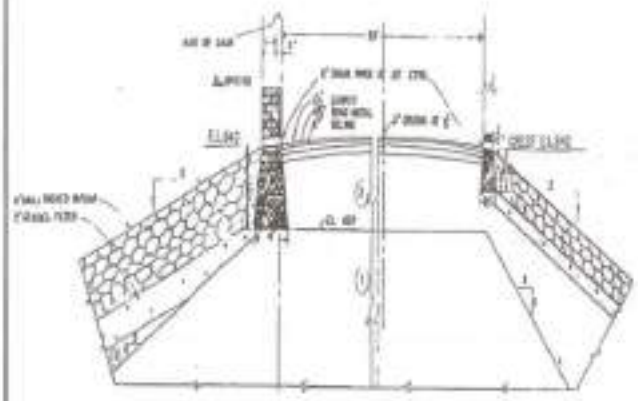
DEPTH OF WATER-90' TO 110'



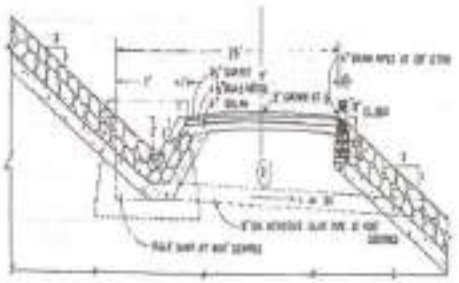
HIRAKUD DAM

SETIONS OF EARTH DAM FOR VARYING DEPTH OF WATER RETENTION

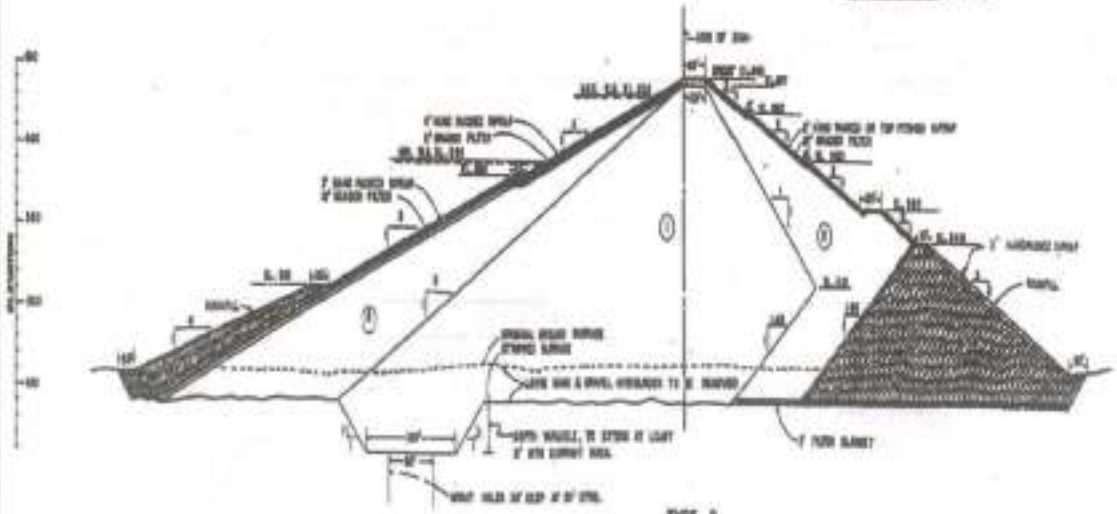
Drg. No.3.5



CREST DETAILS



DETAIL OF ROAD
ON DOWNSTREAM SLOPE

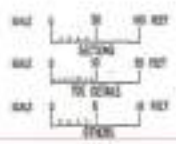


TYPE I
DEPTH OF WATER ABOVE BED

TOE DETAILS

NOTES:-

1. 'H' INDICATES THE HEIGHT OF DAM FROM ORIGINAL GROUND SURFACE TO THE TOP OF THE DAM
2. EXPLANATION OF ZONING
   ZONE (1) SELECTED IMPERVIOUS MATERIAL ROLLED IN 6" LAYERS
   ZONE (2) SELECTED & SEMIPERVIOUS MATERIAL ROLLED IN 9" LAYERS.
   FOR CHANGES IN THE ZONING OF THE EARTH DAM IN THE VICINITY OF THE RETAINING WALL (RTA 48+98) TO BACK OF WALL SEE DWG NO. 936-R-20
3. THE STRIPPING SHALL EXTEND TO MATERIALS HAVING A SHEAR STRENGTH - ESTIMATED TO BE AT LEAST EQUAL TO THAT OF THE OVERLYING ZONE OF THE EMBANKMENT
4. THE DEPTH OF THE CUT-OFF TRENCH SHALL BE DETERMINED AS EXCAVATION PROGRESSES. IT SHALL EXTEND AT LEAST 3 FT IN TO COMPACT ROCK OR OTHER IMPERVIOUS STRATUM



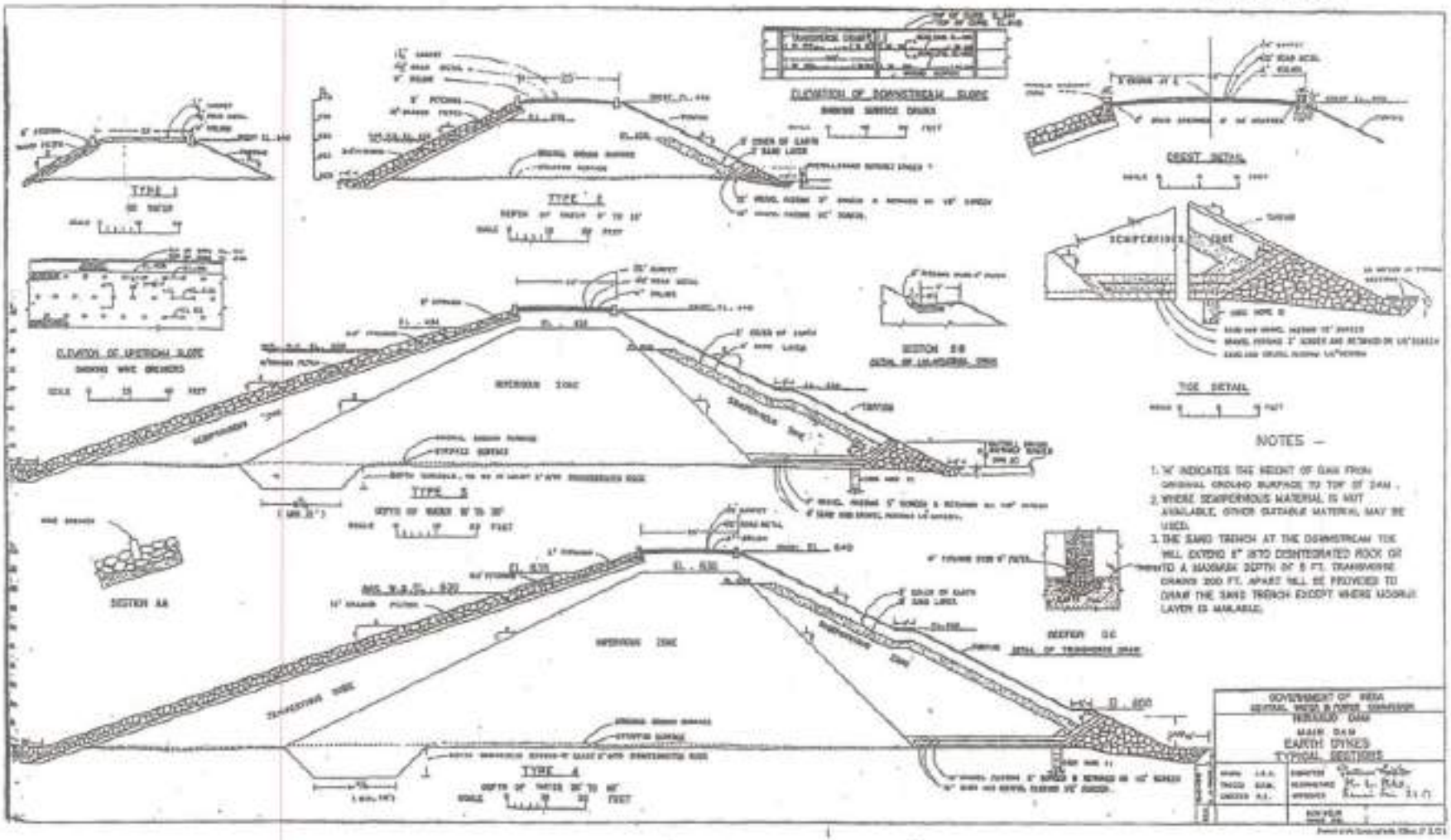
GOVERNMENT OF INDIA
CENTRAL WATER & POWER COMMISSION

HIRAKUD DAM

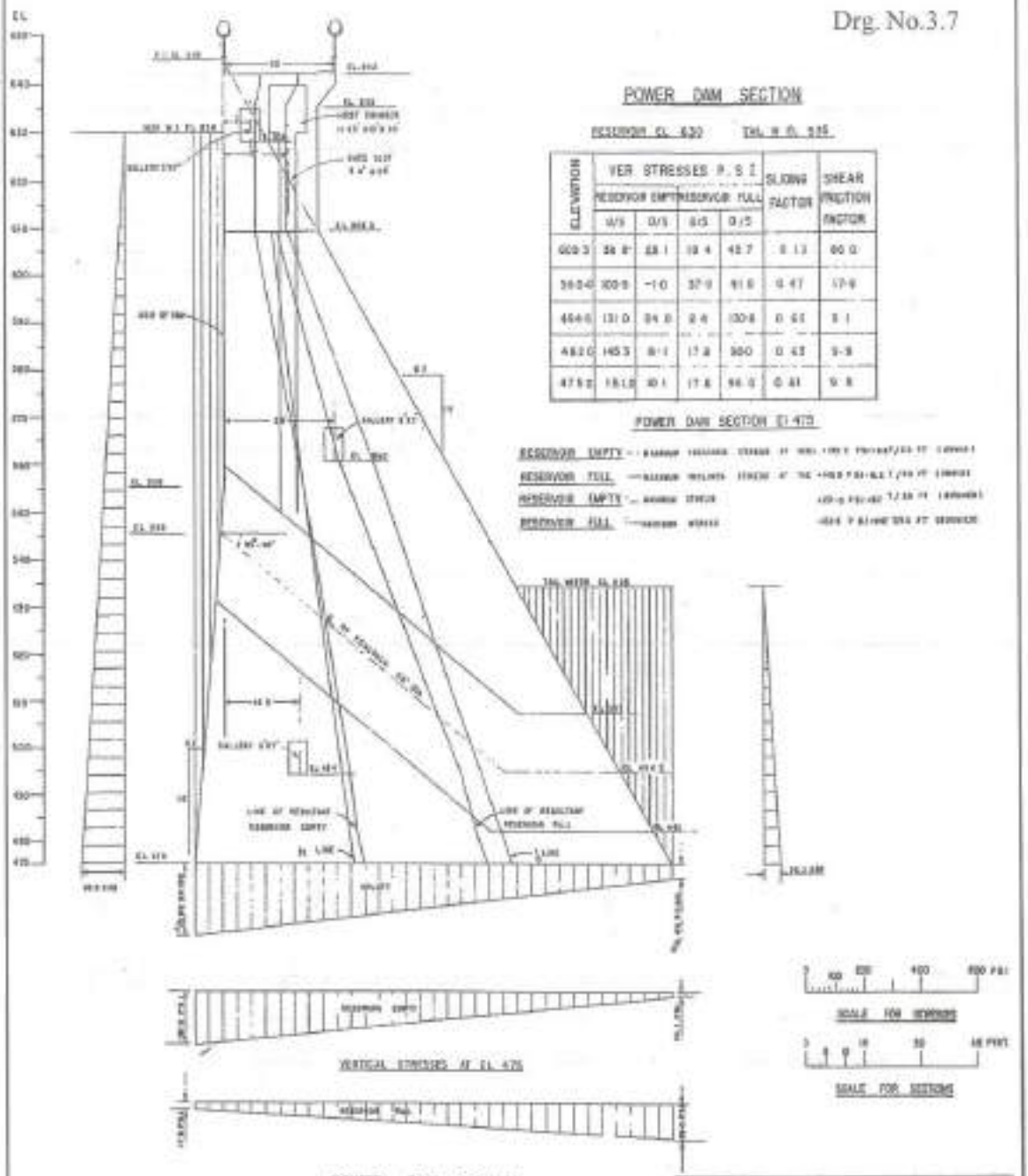
MAIN EARTH DAM

TYPICAL SECTIONS AND DETAILS

Drg. No. 3.6


TYPE 1
TYPE 2
TYPE 3
TYPE 4
ELEVATION OF DOWNSTREAM SLOPE
ELEVATION OF UPSTREAM SLOPE
CREST DETAIL
TOE DETAIL
SECTION AA
SECTION BB
SECTION CC
NOTES –
1. 'H' INDICATES THE HEIGHT OF DAM FROM ORIGINAL GROUND SURFACE TO TOP OF DAM.
2. WHERE SEMIPERVIOUS MATERIAL IS NOT AVAILABLE, OTHER SUITABLE MATERIAL MAY BE USED.
3. THE SAND TRENCH AT THE DOWNSTREAM TOE WILL EXTEND 6" INTO DISINTEGRATED ROCK OR UPTO A MAXIMUM DEPTH OF 5 FT. TRANSVERSE DRAINS 200 FT. APART WILL BE PROVIDED TO DRAW THE SAND TRENCH EXCEPT WHERE MURRUM LAYER IS AVAILABLE.
GOVERNMENT OF INDIA
CENTRAL WATER & POWER COMMISSION
PROPOSED DAM
MAIN DAM
EARTH DYKES
TYPICAL SECTIONS

Drg. No.3.7



POWER DAM SECTION

RESERVOIR EL. 630 TAIL W EL. 515

ELEVATION	VER. STRESSES P.S.I. RESERVOIR EMPTY U/S	RESERVOIR EMPTY D/S	RESERVOIR FULL U/S	RESERVOIR FULL D/S	SLIDING FACTOR	SHEAR FRICTION FACTOR
600.3	26.8	28.1	18.4	42.7	0.13	60.0
560.0	100.9	-1.0	37.9	91.5	0.47	17.9
464.0	131.0	24.0	2.6	130.8	0.65	5.1
482.0	145.3	8.1	17.2	98.0	0.63	9.9
475.0	151.0	10.1	17.6	96.0	0.61	9.9

POWER DAM SECTION EL 475

RESERVOIR EMPTY —

RESERVOIR FULL —

RESERVOIR EMPTY —

RESERVOIR FULL —

VERTICAL STRESSES AT EL 475

SCALE FOR HEADINGS

SCALE FOR SECTIONS

POWER DAM SECTION

1 BLOCK LENGTH 15 FT

2 ONE PENSTOCK 25' DIA IN EACH BLOCK

3 ONE GATE SLOT 44' X 26' IN EACH BLOCK

4 ONE HOIST CHAMBER 11.83' X 10' X 10' AT EL 630

5 THREE GALLERIES 6'-0" X 7'-0" ELEVATIONS 520.5, 560 & 600

6 U/S BATTER 0.1H:1V WITH P.I. AT EL 550

D/S BATTER 0.1H:1V WITH P.I. AT EL 540

7 ASSUMED TRANSITION EL 475

GOVERNMENT OF INDIA
CENTRAL WATER & POWER COMMISSION

HIRAKUD DAM
MAIN DAM

STABILITY ANALYSIS

DRAWN K.S. MITTAL

TRACED K.S. MITTAL
K.S. SINHAL

CHECKED V. VENKATESH

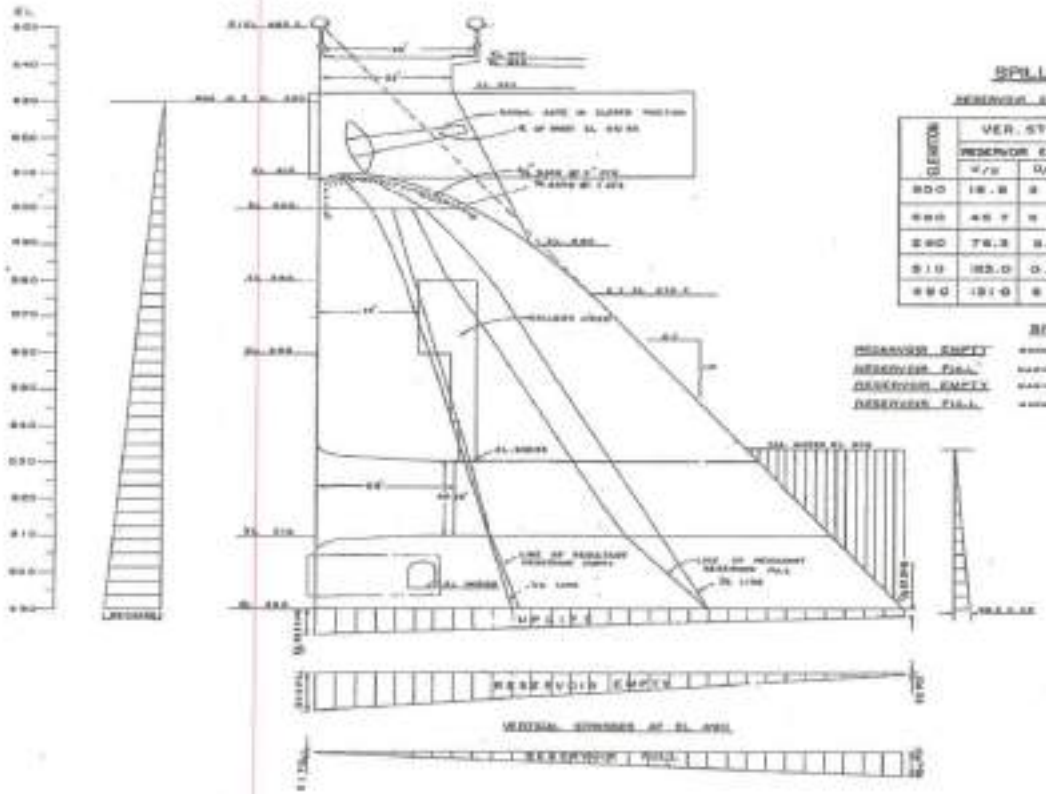
VERIFIED J.C. Sehgal

RECOMMENDED K. L. Rao.

APPROVED

K.S. SELLIN
CHIEF, WP

Drg. No.3.8



SPILLWAY DAM SECTION

RESERVOIR EL. 930. TAIL W. EL. 595.

ELEVATION	VER. STRESSES P.S.I.				SLIDING FACTOR	SHEAR FRICTION FACTOR
	RESERVOIR EMPTY		RESERVOIR FULL			
	H/U	D/U	H/U	D/U		
800	18.8	8.5	9.5	13.0	0.41	70.4
840	45.7	6.8	19.4	88.6	0.51	88.7
880	78.3	5.0	38.1	40.5	0.58	73.9
910	103.0	0.4	59.1	103.0	0.68	7.0
930	131.0	8.2	0.1	88.4	0.74	10.0

SPILLWAY SECTION EL 920

RESERVOIR EMPTY — MAXIMUM HORIZONTAL STRESS AT HORIZONTAL PLANE AT EL 910 FT. BASE

RESERVOIR FULL — MAXIMUM HORIZONTAL STRESS AT THE UPSTREAM FACE AT ELEVATION

RESERVOIR EMPTY — MAXIMUM STRESS — UPSTREAM FACE AT ELEVATION

RESERVOIR FULL — MAXIMUM STRESS — AT THE DOWNSTREAM FACE AT ELEVATION

SPILLWAY SECTION

1. SLUICE LENGTH 80 FT.
2. TWO SLUICES 10'-0 BY 8' IN EACH SLUICE
3. TWO GATE SLOTS 2'-0"x10'-0" EACH
4. ONE PARAPET WALL AT EL. 950 & ONE AT EL. 934 APPROXIMATELY.
5. U/S FACE VERTICAL
   D/S BATTER 0.70:1 WITH FILLET EL. 880.0
   REMAINING ROCK AT EL. 910

BASIC DATA AND ASSUMPTIONS

MAXIMUM RESERVOIR EL. 930.0

MAXIMUM TAIL WATER EL. 932.0

UNIT WEIGHT OF CONCRETE 150 LBS PER CUBIC FOOT

UNIT WEIGHT OF WATER 62.5 LBS PER CUBIC FOOT

UPLIFT FULL HYDROSTATIC PRESSURE AT HEEL DECREASING UNIFORMLY TO TAIL WATER PRESSURE AT THE END ACTING ON 2/3 THE AREA OF THE SECTION CONSIDERED.

WEIGHT OF CONCRETE IN OPENING DEDUCTED.

FORMULAE USED

1. SLIDING FACTOR $f = \frac{\text{RESULTANT HORIZONTAL FORCE}}{\text{RESULTANT VERTICAL FORCE}} = \frac{H}{W_{eff}}$ (NOT MORE THAN 0.75)

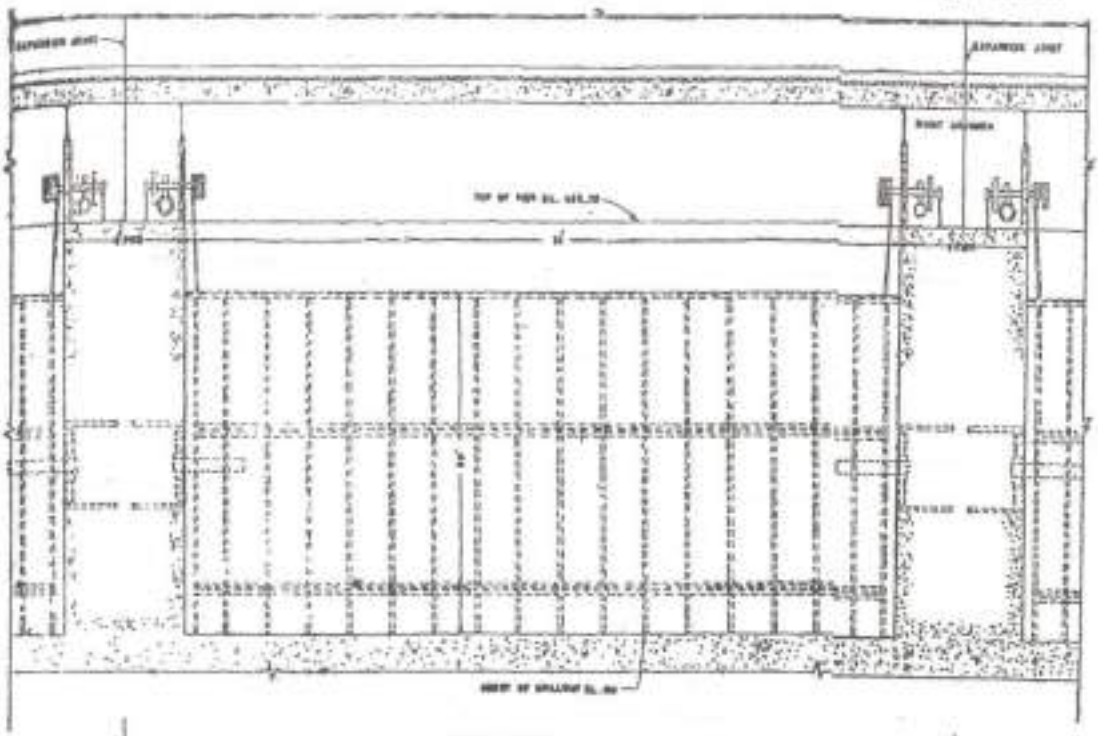
2. SHEAR FRICTION FACTOR $F = \frac{\text{RESULTANT VERTICAL FORCE}}{\text{RESULTANT HORIZONTAL FORCE}}$ (NOT LESS THAN 0.8)

3. $P_1 = P_2 \sec^2 \theta$

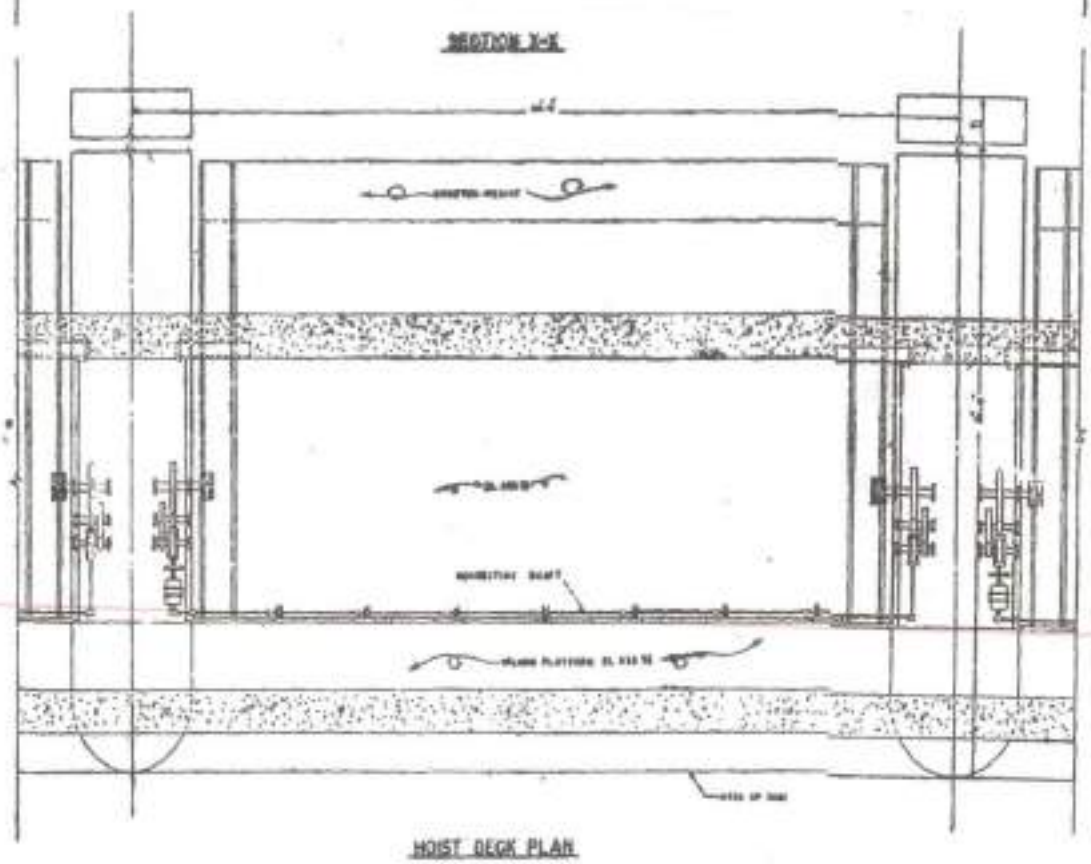
4. $P_2 = P_1 \sec^2 \alpha$

WHERE P = TOTAL HORIZONTAL FORCE; W = RESULTANT WEIGHT FORCE;
U = UPLIFT FORCE; W_{eff} = EFFECTIVE WEIGHT OF SECTION ABOVE PLANE OF U/S
A = BASE AREA OF SECTION; S = MINIMUM VALUE OF UNIT SHEARING STRENGTH (200 LBS PER SQ. IN.)
P_1 = HORIZONTAL STRESS (PSI); P_2 = VERTICAL STRESS
θ = ANGLE OF INCLINATION OF FACE WITH VERTICAL; P_1 = VERTICAL STRESS
α = ANGLE OF INCLINATION OF RESULTANT WITH VERTICAL; P_2 = MAXIMUM STRESS (PSI)

Drg. No.3.9

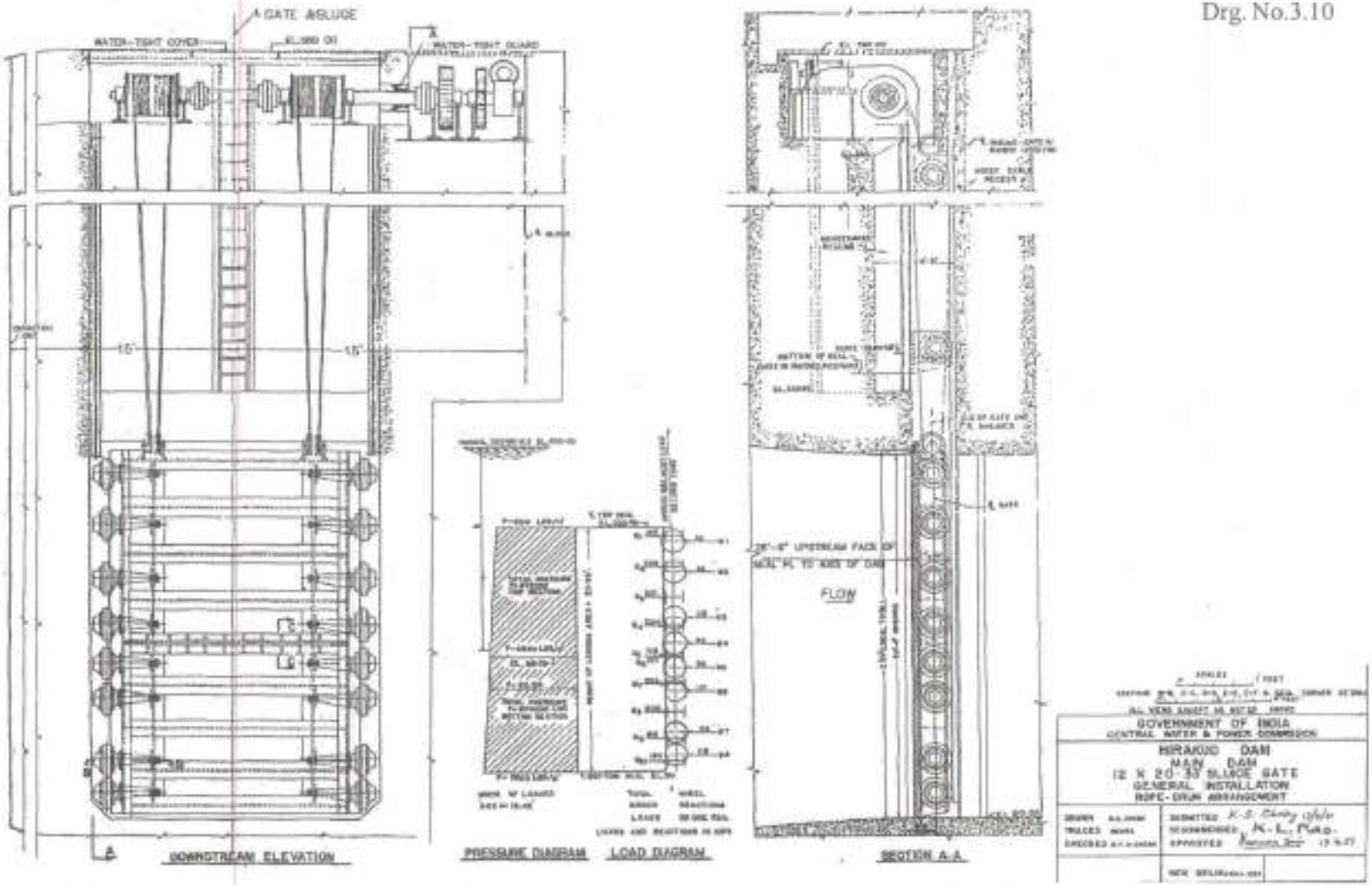


SECTION X-X



HOIST DECK PLAN

Drg. No.3.10



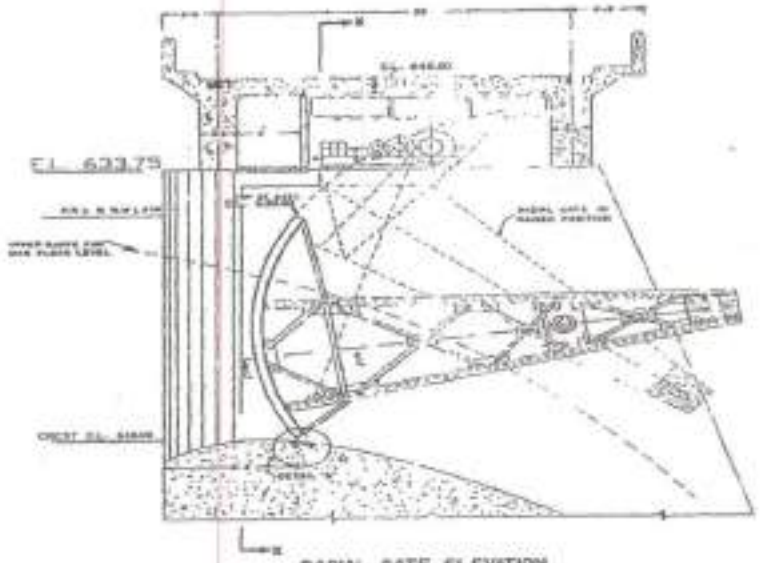
GOVERNMENT OF INDIA
CENTRAL WATER & POWER COMMISSION

HIRAKUD DAM
MAIN DAM
12 X 20'-33" SLUICE GATE
GENERAL INSTALLATION
ROPE-DRUM ARRANGEMENT

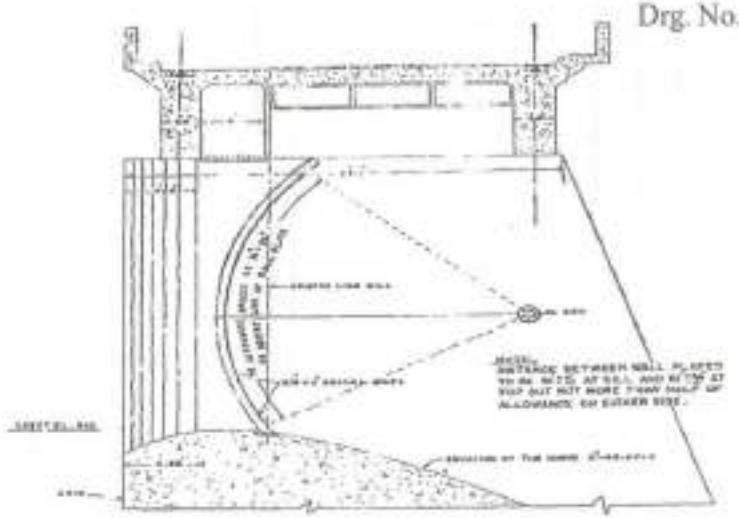
DRAWN
TRACED
CHECKED

SUBMITTED K.S. Chary Supdg
RECOMMENDED M.L. Rao
APPROVED

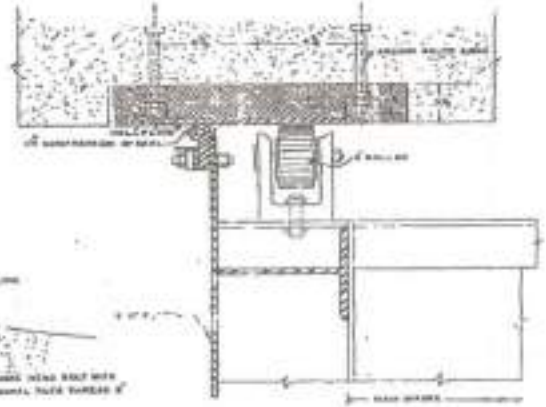
Drg. No.3.11



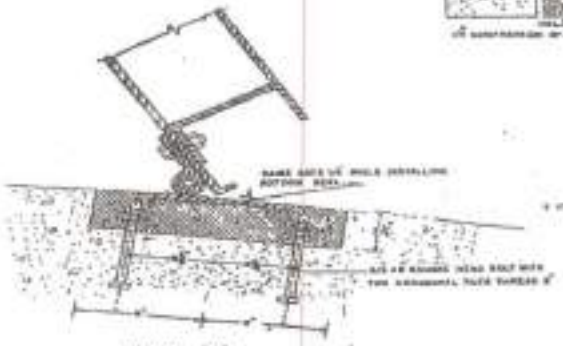
RADIAL GATE ELEVATION



WALL PLATE AND ANCHOR BOLT DETAIL



SECTION C-C



DETAIL "B"

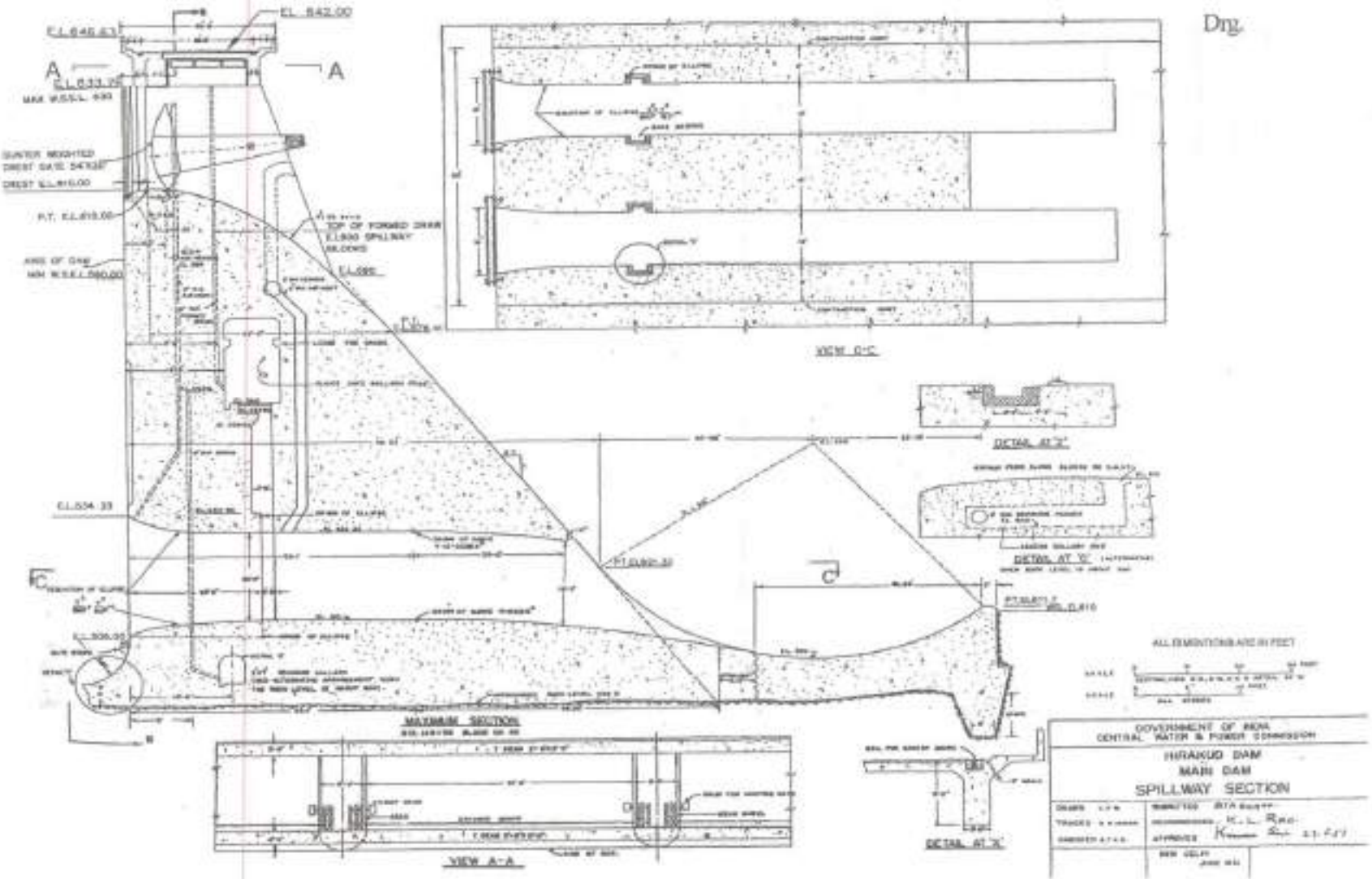
GOVERNMENT OF INDIA
CENTRAL WATER & POWER COMMISSION

HIRAKUD DAM
MAIN DAM

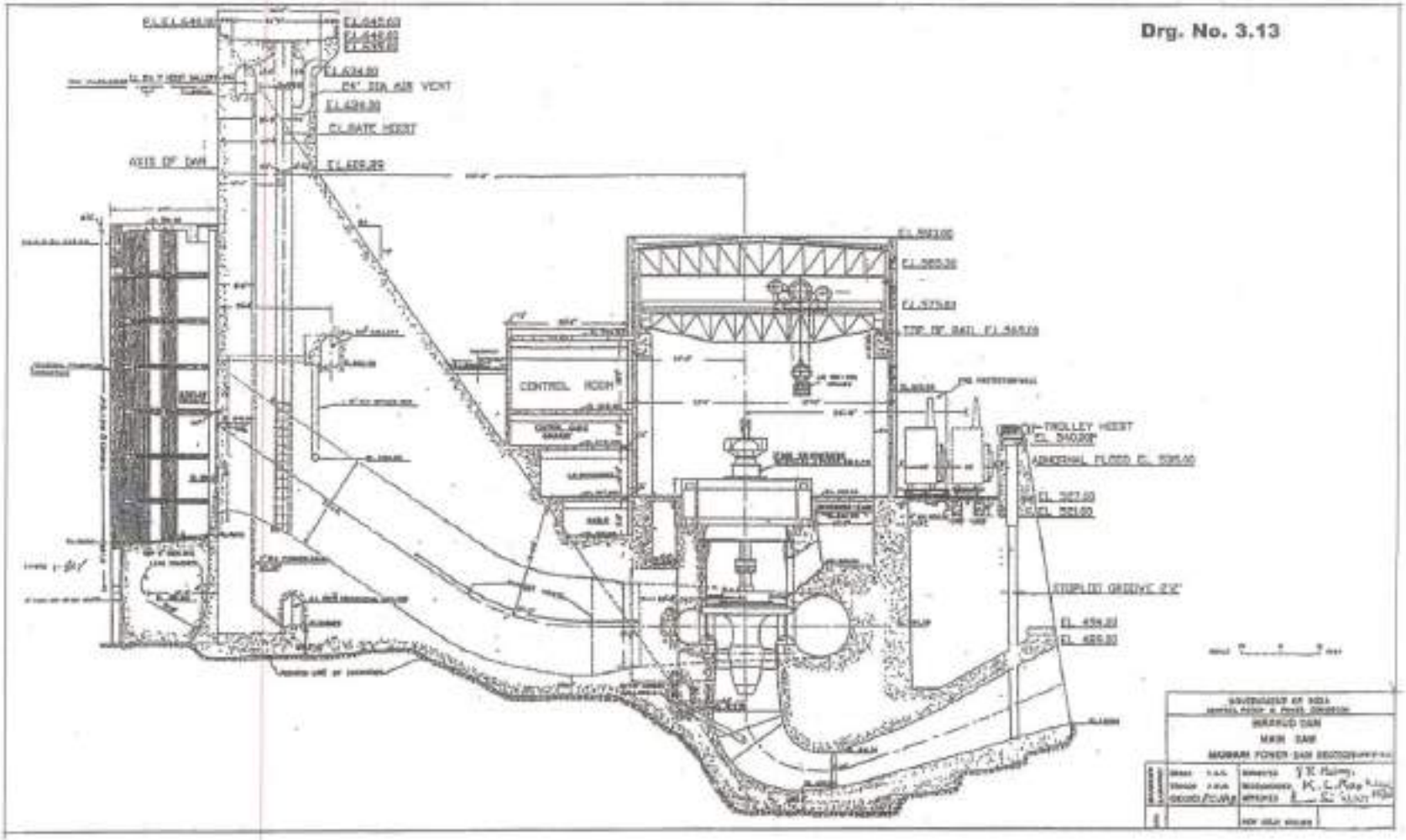
SIXTY RADIAL GATE-GENERAL INSTALLATION

	DRAWN	SUBMITTED
	TRACED	RECOMMENDED
	CHECKED	APPROVED
		NEW DELHI

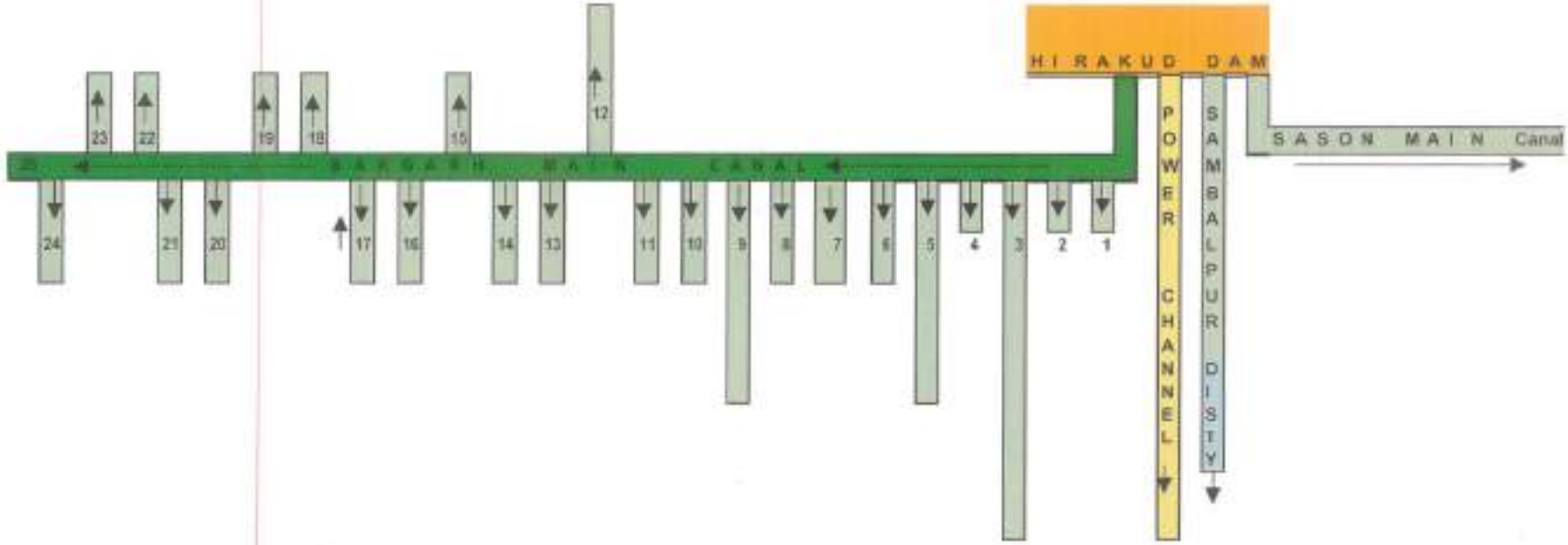
Drg.


EL. 642.00
A
A
MAX W.S.E.L. 630
GUNTER WEIGHTED CREST GATE 54'X32'
CREST E.L.615.00
P.T. E.L.610.00
AXIS OF DAM
NOR W.S.E.L. 580.00
TOP OF FORMED CREST E.L.630 SPILLWAY BEYOND
E.L.595
E.L.534.33
E.L.526.00
MAXIMUM SECTION
VIEW A-A
VIEW C-C
DETAIL AT 'B'
DETAIL AT 'C'
DETAIL AT 'D'
ALL DIMENSIONS ARE IN FEET
GOVERNMENT OF INDIA
CENTRAL WATER & POWER COMMISSION
HIRAKUD DAM
MAIN DAM
SPILLWAY SECTION

Drg. No. 3.13


EL. 645.00
EL. 634.00
24" DIA AIR VENT
EL. 629.00
AXIS OF DAM
CONTROL ROOM
EL. 583.00
EL. 575.00
TOP OF GATE EL. 565.00
TROLLEY HOIST
EL. 540.00
ABNORMAL FLOOD EL. 535.00
EL. 527.00
EL. 501.00
STOPLOG GROOVE
EL. 434.00
EL. 429.00

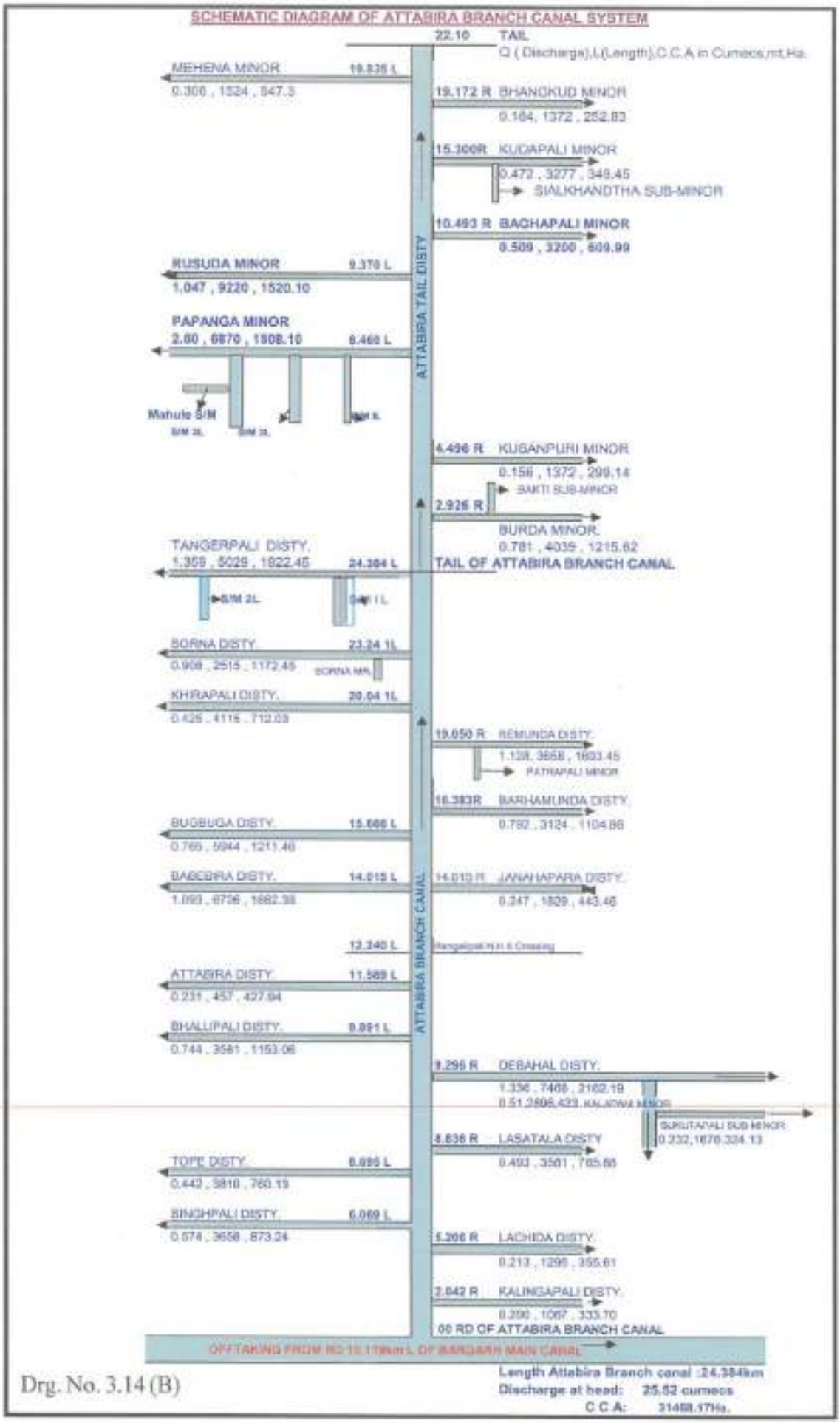
SCHEMATIC DIAGRAM OF CANAL NETWORK



SL	RD(KM)	NAME	(Q IN CUMECS, LENGTH IN KM, AREA IN HA)		
1	0.457	GODBHAGA DISTY	13.19	34.138	15957
2	2.896	TELKANI DISTY	0.284	1.524	507
3	10.119	ATTABIRA DISTY	25.52	24.384	31468
4	17.485	JARA DISTY	1.115	6.553	1553
5	26.335	BARGARH DISTY	9.393	29718	12165
6	28.590	DANG DISTY	0.242	3.505	363
7	34.138	AMBAPALI DISTY	0.322	2.286	497
8	40.965	BAIMAN DISTY	0.698	4.267	1075
9	43.769	BEHERA DISTY	2.839	13.289	3861
10	50.109	AINTHAPALI DISTY	0.449	1.783	515
11	52.730	BARPALI DISTY	1.076	6.142	1657
12	64.925	RETAMUNDA BRANCH CANAL	14.332	9.068	16760
13	68.275	BISIPALI DISTY	0.406	2.56	530

SL	RD(KM)	NAME	(Q IN CUMECS, LENGTH IN KM, AREA IN HA)		
14	69.647	RESAM DISTY	4.895	14.326	6948
15	70.211	DHANABASA DISTY	0.214	1.889	284
16	72.337	SARASMAL DISTY	0.185	1.219	380
17	74.015	RAMPUR DISTY	3.901	18.135	5270
18	74.947	PUTPUTPALI DISTY	0.864	7.239	1390
19	77.404	TELIPALI DISTY	0.204	1.143	288
20	77.990	BARKARLAY DISTY	5.514	22.814	6635
21	79.884	SULAI DISTY	0.422	2.895	607
22	80.757	PANDRAPALI DIST'	0.464	2.286	637
23	81.600	ANDHARIBANJHI D	0.449	3.352	740
24	84.277	GONIA DISTY	1.981	1.981	415
25	84.281	BHIMTIKIRA TAIL D	10.928	19.05	10792

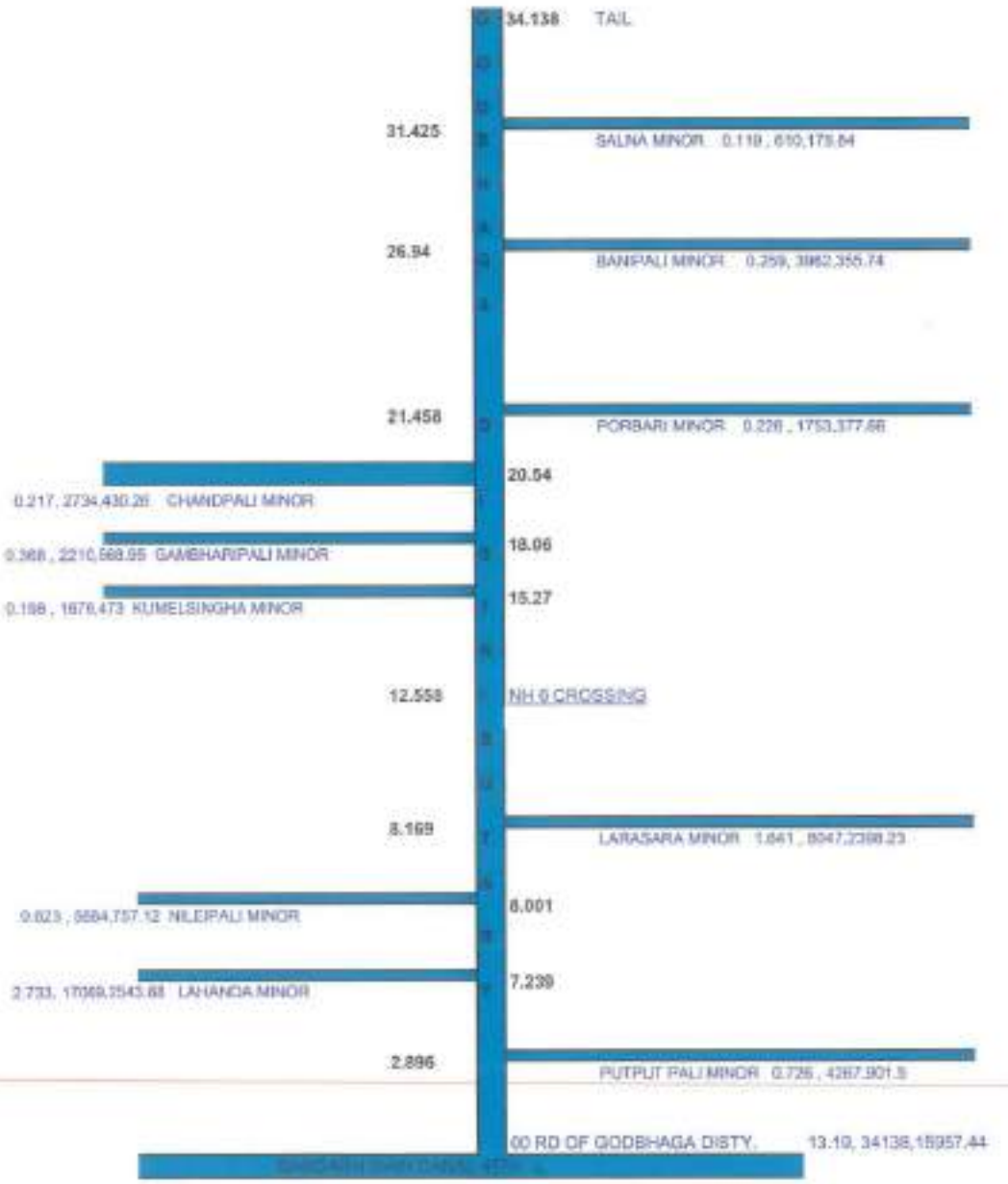
Drg. No. 3.14 (A)


SCHEMATIC DIAGRAM OF ATTABIRA BRANCH CANAL SYSTEM
22.10 TAIL
Q (Discharge),L(Length),C.C.A in Cumecs,mt,Ha.
MEHENA MINOR 19.835 L
0.306 , 1524 , 847.3
19.172 R BHANOKUD MINOR
0.164, 1372 , 252.83
15.306R KUDAPALI MINOR
0.472 , 3277 , 349.45
SIALKHANDTHA SUB-MINOR
10.493 R BAGHAPALI MINOR
0.509 , 3200 , 609.99
RUSUDA MINOR 9.370 L
1.047 , 9220 , 1520.10
PAPANGA MINOR
2.80 , 6870 , 1908.10 6.460 L
Mahule S/M
S/M 2L
S/M 2L
4.496 R KUSANPURI MINOR
0.156 , 1372 , 299.14
SAKTI SUB-MINOR
2.926 R
BURDA MINOR.
0.781 , 4039 , 1215.62
TANGERPALI DISTY.
1.359 , 5029 , 1822.45 24.384 L
TAIL OF ATTABIRA BRANCH CANAL
S/M 2L
SORNA DISTY. 23.24 1L
0.908 , 2515 , 1172.45
SORNA MR.
KHIRAPALI DISTY. 20.04 1L
0.425 , 4115 , 712.03
19.050 R REMUNDA DISTY.
1.139, 3658 , 1693.45
PATRAPALI MINOR
16.383R BARHAMUNDA DISTY.
0.792 , 3124 , 1104.86
BUGBUGA DISTY. 15.666 L
0.765 , 5944 , 1211.46
BABEBRA DISTY. 14.015 L
1.093 , 6736 , 1882.38
14.013 R JANAHAPARA DISTY.
0.247 , 1829 , 443.46
12.240 L
ATTABIRA DISTY. 11.589 L
0.231 , 457 , 427.84
BHALUPALI DISTY. 9.991 L
0.744 , 3581 , 1153.06
9.296 R DEBAHAL DISTY.
1.336 , 7469 , 2162.19
0.51,2896,423, KALADWA MINOR
SUKUTAPALI SUB MINOR
0.232,1676,324.13
8.836 R LASATALA DISTY
0.493 , 3581 , 765.66
TOPE DISTY. 8.086 L
0.442 , 3810 , 760.13
SINGHPALI DISTY. 6.069 L
0.574 , 3658 , 873.24
5.206 R LACHIDA DISTY.
0.213 , 1296 , 355.61
2.842 R KALINGAPALI DISTY.
0.290 , 1067 , 333.70
00 RD OF ATTABIRA BRANCH CANAL
ATTABIRA TAIL DISTY
ATTABIRA BRANCH CANAL
OFFTAKING FROM RD 15.119km L OF BARGARH MAIN CANAL
Length Attabira Branch canal :24.384km
Discharge at head: 25.52 cumecs
C C A: 31468.17Ha.

Drg. No. 3.14 (B)

SCHEMATIC DIAGRAM OF GODBHAGA DISTRIBUTARY CANAL SYSTEM

Q (Discharge),L(Length),C.C.A in Cumecs,mt,Ha.



Offtaking from Bargarh Main Canal at RD 0.457 Km
Length:34.138km
Discharge at head:13.19 cumecs
C C A:15957.44Ha

LEFT CANAL SYSTEM HIRAKUD DAM PROJECT.

61.798 TAIL

Q,L,C.C.A in Cumecs,mt,Ha.
RD in Km.

HUMA TAIL DISTY.

9.272 , 61798,12097.67

HUMA TAIL DISTY

55.603 BANSARA MINOR 1.00 , 2676,751.63

53.568 SALESINGA MINOR 0.44 , 2819,646.18

47.092 BADUAPALI MINOR 0.689 , 4420,905.12

41.879 CHATABAR MINOR 0.764 , 3200,668.33

32.69 RASANPUR MINOR 0.51 , 2530,748.55

22.258 KABRAPALI MINOR 1.219, 8138,1058.73

8.611 GIRIPALI MINOR 0.368 , 3581,530.05

0.795 PIPLIPALI MINOR 0.203 , 1067,350.12

21.793R PARMANPUR DISTY 3.877 , 18517,5186.31

19.833 KUTHELPALI DISTY 0.149 , 1067,293.73

18.407 BISAL KHINDA DISTY 0.295 , 1676,538.41

SASON MAIN CANAL

7.285 SASON DISTY 0.603 , 3048,858.86

2.854 KANKHINDA DISTY 0.269 , 2057,393.75

2.21 TALAB DISTY 0.175 , 1414,262.58

0.381 JAMADARPALI DISTY 1.065 , 235,520.18

IV GAP SASON MAINCANAL 17.814,21793, 21467.63

LEFT DYKE OF HIRAKUD DAM.

Offtaking from Left Dyke of Hirakud Dam IV Gap.
Length:21.793km C C A:21467.63Ha.
Discharge at head:17.814 cumecs



SAMBALPUR DISTY.
3.367,18075,3946.81

POWER CHANNEL CANAL SYSTEM

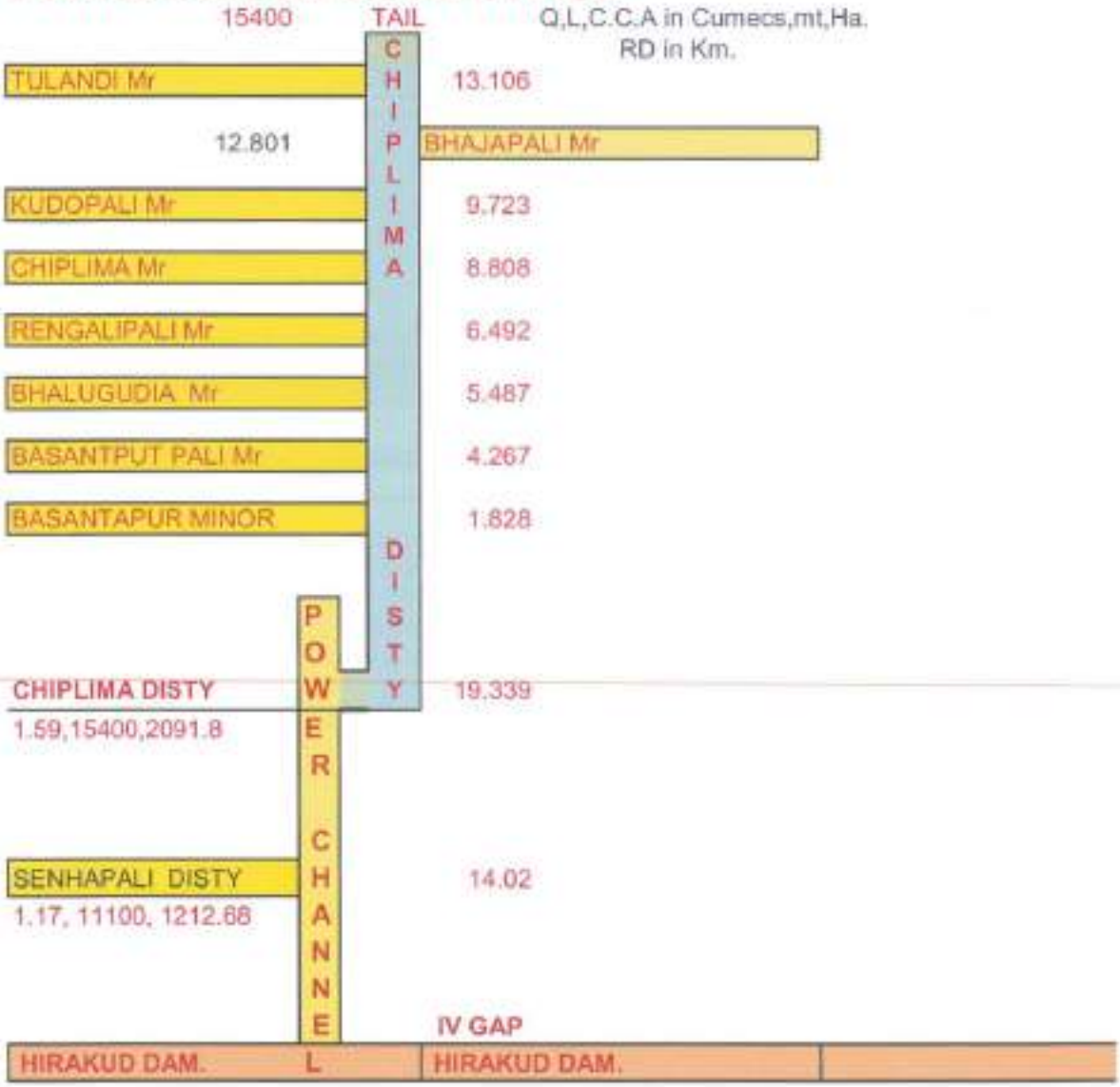


Table 3.5 : Stability Analysis of spillway & Power Dam

Section	Elevation RL in ft	U/S W.L. 630'.0" Vertical Stresses(Psi)		U/S W.L. 631.0 ft Vertical stresses(Psi)		U/S W.L. 632.0 Vertical stresses (Psi)		Data	Assumptions
		U/S	D/S	U/S	D/S	U/S	D/S		
Spillway section	490.0 (Foundation level)	0.197	98.6	(-)0.62	96.0	(-) 4.12	102.5	1. D.S.W.L. Nil 2. Spilway and sluice gates closed. 3.Sluice groove 56 4. Gallery 23' x 11' @ E.L. 557	Uplift : Full hydrostatic pressure at heel decreasing uniformly to tail water pressure at toe and acting on 2/3 the area of the section considered.
Power dam section (Block length 70') Block Nos. 49-51 Unconstructed Units Ref. Drg. 100 -IM- 118 of Nov.54	475	plus 0.3	123.5	(-)3.31	124.34	(-)5.76	128.2	1. Tail waer level 535.0 2. Batter of 0.7/1.0 from P.T. EL.609.29 and then vertical from EL.486.5 3. Gate groove-6.97' 4. Gates closed.	Uplift : Full hydrostatic pressure at heel decreasing uniformly to tail water pressure at toe and acting on 2/3 the area of the section considered.
Power Dam Section (Block length 63') Block No.54	495.5 (Section through penstock)	(-)6.85	149.75	(-) 10.8	113.5	(-) 12.1	155.4	1. Tail waer level= 535.0 2. Dia. of penstock=25' 3.Gate Groove-6.97' width	Uplift : Full head on 2/3 area and for penstock section no uplift.

N.B. - At EL 630 ft., acutal Free Board provided for Earth and Concrete Dam is 12ft and at EL. 632 ft, F.B. provided is 10ft. for both the Dams.

3.2.3 Design of Canal System

3.2.3.1 Design of Channel Sections

The parameters that influence the design of channel sections in earth are discharge (Q), bed width (b), depth (d), bed slope (s) and velocity of flow (v). In a well designed system, these will be balanced so that the channels will be in regime i.e. non-silting and non-scouring.

'Q' is computed taking command area, cropping pattern, duty and losses into consideration.

'd' is a $Q^{0.25}$

'V' is kept between $cd^{1/2}$ to $cd^{2/3}$

Kennedy's classic equation for non-silting, non-scouring velocity is $Vo = 0.84\ d^{0.64}$

Velocities ranging from 0.9 Vo to 1.2 Vo have been successfully adopted in designing canal system in South

So $V = 0.756\ d^{0.64}$ to $1.008\ d^{0.64}$..... (1)

For designing canal cross-sections for open channel flow there are several formulas which are followed. In Hirakud Project, at the initial stage, Lacey's equations for regime channels have been used in many systems, considering Lacey's silt factor varying from 0.6 to 1.60. This resulted, in many cases of channels, too little or too high velocities. Lacey's equation after slight modification stands as follows (in fps units)

$P = 5/3\ Q^{1/2}$ ..... (2)

$$V = 1.15473\ (fR)^{1/2} = 0.7937 \times Q^{\frac{1}{6}} \times f^{\frac{1}{3}} \quad \text{..... (3)}$$

$$S = 0.0003759\ f^{2/3} / R^{1/2} = 0.0005469\ f^{5/3} / Q^{\frac{1}{6}} \quad \text{..... (4)}$$

$$R = 0.47247\ Q^{\frac{1}{3}} / f^{\frac{1}{3}} \quad \text{..... (5)}$$

Where Q = Design discharge in cusec

P = Perimeter of flow area

S = bed slope

f = Lacey's silt factor

R = Hydraulic radius = A/P

A = Area of flow

The silt factor 'f' propounded by Lacey is not only function of the mean diameter of silt particles, but also it supplied design diagrams based on his original formulas. But revised design diagram/design statement worked out by the Central Designs Organisation (CWINC) on the basis of above modified formulas have been used in designing some of the canal sections.

For main contour canals, the value of 'f' is assumed as 0.85. The slopes have then been obtained from Lacey's f – Q relationship graph and bed widths as well as depths from the Regime dimension diagram.

For ridge canals as also for distributaries and minors etc. which are usually ridge channels, the value of 'f' has been adopted between 1 to 1.5. Where ground slopes rapidly 'f' is assumed 1.5 and where it slopes gently 'f' is assumed as 1.0. With 'f' thus fixed, the water surface slope is then determined from Lacey's regime slope diagram. As Lacey's section for low discharges gives a wide and shallow channel section, the sections, for these ridge channels were not taken from Lacey's dimension diagram. Instead the sections were adopted from Garret's "Hydraulic diagrams

for design of channels in earth using Kutter's formula", with value of N = 0.0225 and slopes arrived at as above. Sufficient berms have been left to allow future widening, if any, of the channel section to bed widths corresponding to Lacey's section. But choosing of a proper value of the elusive silt factor (f) was a tough problem. This value varies from 0.4 to 1.6 and fixation of a value for a particular channel is done more or less arbitrarily. This resulted in many cases of channels in developing too little or too high velocities. Parker suggests that to keep the channels free from scour, the velocities should not ordinarily exceed $1.05\sqrt{d}$. Hunter Rouse suggests that slopes in regime channels should generally vary inversely as 'd'. Taking all these factors velocities ranging from 0.9 ft/sec for a 1'0" deep canal to 3.3'/sec for a 12' deep canal have been adopted. The velocities actually adopted vary from $Q^{1/8}$ to $Q^{1/6}$.

Suitable grades increasing with reduced depth of flow were adopted to obtain the reasonable velocities as mentioned above. The grades generally vary from $Kd^{-0.8}$ to $kd^{-1.0}$

K is taken as $\frac{0.6}{1000}$ (approx)

So $S = \frac{0.6}{1000} d^{-0.8}$ .... (6)

The velocity is also a function of R and S. For convenience of applicability Manning's formula is used.

$$V = \frac{1.4858}{n} R^{2/3} S^{1/2}$$

Where n is taken as 0.0225

The channel is designed as trapezoidal with side slopes ½ : 1

$$\text{So } Q = AV = \left(b + \frac{d}{2}\right)d \times \frac{1.4858}{0.0225}\left\{\frac{\left(b + \frac{d}{2}\right)d}{b - 2.24d}\right\}^{2/3} S^{1/2} \quad \text{..... (7)}$$

Having fixed suitable depths, velocities and grades for a particular 'Q' the bed width can be found out by using equation 1, 6 & 7.

A check-up by Kutter's equation conforms the values selected and the final design is completed.

Based on above principles, K. Padmanabha Pillai, the then S.E of Hirakud Dam Project, worked out suitable sections and grades for channels with capacities from 2 to 10000 cusecs and presented it in a tabular form popularly known as "Pillai Chart" in the state of Orissa. This table shows bed width, grade, velocity and discharge from which suitable channel design has been made. All subsequent designs of channels in Hirakud Project were made on this basis. Table-3.6 shows the channel section to convey discharge from 2 cusec to 10566 cusec in Hirakud canal system.

Table 3.6 Canal Section adopted in Hirakud
(Pillai Chart in fps system)

Bed width	Depth	Grade	Velocity	Discharge
2	1.0	0.60/1000	0.89	2.2
3	1.0	0.60/1000	0.96	3.4
4	1.0	0.60/1000	1.03	4.6
3	1.25	0.50/1000	1.02	4.6
4	1.25	0.50/1000	1.07	6.2
5	1.25	0.50/1000	1.11	7.8
6	1.25	0.50/1000	1.17	9.7
4	1.50	0.44/1000	1.11	7.8
5	1.50	0.44/1000	1.16	10.0
6	1.50	0.44/1000	1.20	12.1
7	1.50	0.44/1000	1.24	14.5
8	1.50	0.44/1000	1.28	16.7
5	1.75	0.39/1000	1.22	12.6
6	1.75	0.39/1000	1.26	15.1
7	1.75	0.39/1000	1.30	17.9
8	1.75	0.39/1000	1.33	20.1
9	1.75	0.39/1000	1.37	23.7
10	1.75	0.39/1000	1.40	26.6
7	2.00	0.35/1000	1.33	21.3
8	2.00	0.35/1000	1.35	24.4
9	2.00	0.35/1000	1.37	27.4
10	2.00	0.35/1000	1.40	30.8
11	2.00	0.35/1000	1.42	34.1
12	2.00	0.35/1000	1.44	37.7
13	2.00	0.35/1000	1.47	41.2
8	2.25	0.32/1000	1.40	29.0
10	2.25	0.32/1000	1.49	37.0
12	2.25	0.32/1000	1.60	47.0
14	2.25	0.32/1000	1.70	58.0
16	2.25	0.32/1000	1.76	67.0
10	2.50	0.295/1000	1.68	47.0
12	2.50	0.295/1000	1.73	58.0
14	2.50	0.295/1000	1.79	68.0
16	2.50	0.295/1000	1.83	78.0
18	2.50	0.295/1000	1.85	89.0
20	2.50	0.295/1000	1.87	99.0
12	2.75	0.27/1000	1.73	64.0
14	2.75	0.27/1000	1.77	75.0
16	2.75	0.27/1000	1.80	86.0
18	2.75	0.27/1000	1.84	98.0

Contd...

Table 3.6 Canal Section adopted in Hirakud (Continued.....)

20	2.75	0.27/1000	1.87	110
22	2.75	0.27/1000	1.90	122
15	3.00	0.25/1000	1.80	89
18	3.00	0.25/1000	1.84	105
20	3.00	0.25/1000	1.87	121
23	3.00	0.25/1000	1.90	137
25	3.00	0.25/1000	1.94	154
20	3.50	0.22/1000	1.93	146
22	3.50	0.22/1000	1.96	162
24	3.50	0.22/1000	1.98	178
26	3.50	0.22/1000	2.00	194
28	3.50	0.22/1000	2.02	211
30	3.50	0.22/1000	2.04	227
32	3.50	0.22/1000	2.07	244
25	4.00	0.20/1000	2.05	221
30	4.00	0.20/1000	2.09	267
35	4.00	0.20/1000	2.13	316
40	4.00	0.20/1000	2.16	364
30	4.5	0.18/1000	2.15	300
35	4.5	0.18/1000	2.16	362
40	4.5	0.18/1000	2.19	418
45	4.5	0.18/1000	2.23	473
50	4.5	0.18/1000	2.25	528
40	5.0	0.16/1000	2.20	400
45	5.0	0.16/1000	2.30	546
50	5.0	0.16/1000	2.37	622
55	5.0	0.16/1000	2.44	704
60	5.0	0.16/1000	2.52	786
50	5.5	0.15/1000	2.56	742
55	5.5	0.15/1000	2.58	800
60	5.5	0.15/1000	2.59	897
65	5.5	0.15/1000	2.61	974
70	5.5	0.15/1000	2.63	1052
75	5.5	0.15/1000	2.64	1129
55	6.0	0.14/1000	2.62	913
60	6.0	0.14/1000	2.64	1009
65	6.0	0.14/1000	2.65	1088
70	6.0	0.14/1000	2.68	1175
75	6.0	0.14/1000	2.70	1263
80	6.0	0.14/1000	2.71	1305

Contd...

Table 3.6 Canal Section adopted in Hirakud

65	6.5	0.13/1000	2.69	1195
75	6.5	0.13/1000	2.73	1302
85	6.5	0.13/1000	2.78	1302
95	6.5	0.13/1000	2.82	1828
70	7.0	0.122/1000	2.75	1414
80	7.0	0.122/1000	2.78	1625
90	7.0	0.122/1000	2.81	1828
100	7.0	0.122/1000	2.81	2078
110	7.0	0.122/1000	2.87	2259
80	7.5	0.115/1000	2.81	1764
90	7.5	0.115/1000	2.83	1988
100	7.5	0.115/1000	2.85	2223
110	7.5	0.115/1000	2.87	2449
120	7.5	0.115/1000	2.89	2632
100	8.0	0.109/1000	2.88	2400
110	8.0	0.109/1000	2.90	2650
120	8.0	0.109/1000	2.91	2892
130	8.0	0.109/1000	2.93	3132
110	8.5	0.103/1000	2.92	2836
120	8.5	0.103/1000	2.94	3111
130	8.5	0.103/1000	2.96	3386
140	8.5	0.103/1000	2.08	3665
150	8.5	0.103/1000	3.00	3937
120	9.0	0.099/1000	3.01	3371
130	9.0	0.099/1000	3.02	3637
140	9.0	0.099/1000	3.03	3943
150	9.0	0.099/1000	3.04	4230
160	9.0	0.099/1000	3.05	4516
150	9.5	.0095/1000	3.08	4536
160	9.5	.0095/1000	3.10	4844
170	9.5	.0095/1000	3.11	5156
180	9.5	.0095/1000	3.12	5470
190	9.5	.0095/1000	3.13	5783
160	10.0	.090/1000	3.11	5137
170	10.0	.090/1000	3.12	5450
180	10.0	.090/1000	3.13	5790
190	10.0	.090/1000	3.13	6103
200	10.0	.090/1000	3.14	6437
230	12.00	0.078/1000	3.28	9288
240	12.00	0.078/1000	3.20	9713
250	12.00	0.078/1000	3.30	10138
260	12.00	0.078/1000	3.31	10566

It will also be seen from the table that the sections therein give enough wetted border, the ratio $P/Q^{1/2}$ varying from 2.6 to 2.7.

(Source: 'The Canal System' by K.P. Pillai & K.C. Thomas - Indian Journal of Power and River Valley Development - Vol.IV, Hirakud Dam Project No.10).

3.2.3.2 Masonry Structures :

The cost of all structures involved constitutes nearly 50% of the total cost of the system. Special and careful attention was given to design the most suitable and economical types depending on the ground considerations and keeping in view the locally available materials as well as the skill of local labours. Good granite stone is available almost throughout the area. Extensive use of this material was therefore made in masonry. Next to stone, bricks have been used in masonry works to a great extent. A quartzite variety of stone was available in plenty almost all along the canals within a radius of one mile or so. Being readily available and of a slightly softer variety, it costs considerably less than granite. Advantageous use of this type of stone has been made in mass concrete in the foundation of structures. Use of reinforced concrete work was kept to the minimum in order to restrict use of steel.

The followings are the structures provided in Hirakud distribution system.

3.2.3.3 Cross Drainages :

The main canals and some of the distributaries being contour channels are crossing a large number of drainages. Due to the rolling nature of the country even many of the ridge canals cross a number of drainages in some intermediate reaches.

For cross-drainage works, a proper assessment of maximum flood discharges expected at the site is necessary. For large catchments the observed flood discharges in the area corresponds to a co-efficient of about 500 in the Dicken's formula $Q = CM^{3/4}$. The maximum observed flood discharge of 15.7 lakh cusecs in Mahanadi at Naraj with a catchment area (C.A.) of 51000 Sq.miles gives a value of C = 490. The maximum flood discharge of 15 lakh cusec allowed for Hirakud Dam with C.A. of 32200 sq.miles corresponds to a value of 624 for C. Even allowing 50% extra over this a co-efficient of about 1000 would have an ample margin for any possible future flood of higher intensity. Flood discharges have therefore been calculated with C = 1060 in Dicken's formula as in vogue in Orissa P.W.D. then.

Creager's formula $Q = 46CA^{0.894A^{-0.048}}$ with value of C = 45 was also used. Since the vent ways so calculated was far bigger than the regime section of the drain, this was not adopted.

For very small catchments, the rate of Q per acre increases rapidly with decrease in size of catchment. For C.A. < 1 sq.mile, the Qs have been taken from a curve based on an intensity varying from 2 cusec/acre for one sq.mile to 3 cusec/acre for catchments only of a few acres in extent. For C.A. > 1 to < 5 sq.miles, the curve given values ranging from about 2 cusec/acre to 1.1 cusec/acre, the latter value being identical with that obtained by Dicken's formula for a catchment of 5 sq.miles. For C.A. more than 5 sq.miles Dicken's formula with C = 1060 is adopted.

3.2.3.4. Cross Regulator :

"Eight cross regulators have been constructed in the Bargarh canal at an average interval of about 5 miles. All are combined with bridges. Some of the earlier designs provided for controlling arrangement with needles. This was however not considered to be satisfactory arrangement to effect the necessary control and raising up water level in the upstream. Later designs have therefore been provided with gates operated by screw gears.

The cross regulators have been designed as a submerged weir having a cistern in the downstream for dissipation of energy. The needle controlled cross regulators 2 in number are having 15 ft. wide vents. In the other type width of vent is restricted to 8 ft. in order to limit the size of the gate thereby enabling easy operation by screw gears.

3.2.3.5 Aqueduct :

"These have been provided where sufficient headway is available. In case of low drainage discharge as compared to the canal discharge, under tunnel designed as an open culvert for full width of canal is provided. Where the drainage discharge is high compared to the canal discharges, the cross drainage has been designed as a flumed aqueduct.

The superstructures of the aqueducts have been designed either as R.C.C. or masonry troughs. Masonry trough is provided where there is an arch substructure. In other cases R.C.C. trough is provided. The sectional area of the trough has been fixed taking into consideration a maximum velocity of 5 ft. per Sec. through the trough. Where sufficient headway above M.F.L. of drainage is available, the bed of the trough is depressed to have maximum fluming within economic and practical limits. The flumed trough is connected to the normal section of the canal with an upstream splay of 1 : 3 and downstream splay of 1 : 5. The flumed entrances and exits have been lined with concrete or pucca stone pitching. Besides, masonry canal wing walls have been provided both at the upstream and downstream side of the trough for safety and to facilitate bank connections.

In case of costly foundations, where R.C.C. rough has been provided, the span is kept 25 ft clear and the trough divided into compartments. The side and intermediate walls of the trough are designed as beams supporting the slab at the bottom. A 12 ft. wide inspection path is carried over one of the side vents of the trough. Where better foundation conditions are available, the span is kept 10 ft. clear with a slab spanning directly and the side walls designed as R.C.C. cantilevers. In this type, the 12 ft. wide inspection path is carried by the side of the trough over concrete arches resting on the piers.

The foundations of the aqueducts fall mainly under three categories depending on foundation conditions :

(i) Well type foundation for piers and abutments have been adopted where no good strata was available at considerable low depths. The wells adopted are of plain concrete with nominal reinforcements. In such cases span adopted is 25 ft. clear.

(ii) Where good strata consisting mostly of disintegrated rock was available at shallow depths, plain concrete raft foundation from abutment to abutment with cut off walls at upstream and downstream of the drainage has been adopted. Span is kept 10 ft. clear in these cases.

(iii) Where good rock strata was available within reasonable depths, open foundation has been resorted to and superstructure designed as plain concrete arches of 25 ft. clear span.

In all these aqueducts abutments wing walls, returns etc are built in coursed rubble masonry.

In case of small channels crossing fairly wide drainages, hume pipe aqueducts resting on masonry piers have been provided."

3.2.3.6. Syphons / Superpassages

"In most of the canals and major distributaries, drainages have been syphoned and channels allowed to flow freely. In case of minor distributaries where the drainage discharge is considerably higher compared to the discharge of the distributary, the channel is syphoned within permissible limit of head loss through the structure.

Generally all syphons are designed as barrels with maximum permissible velocity of 8 to 10 feet sec. in the barrel. In the earlier designs, R.C.C. box type barrels were used but due to constructional difficulties and slow progress in execution, subsequent designs were changed to mansonry side walls with R.C.C. slab on the top.

Incidentally, a large number of 6' dia Armco pipes were available from Disposals and these were made use of in forming syphon barrels. The Armco pipes are encased in plain cement concrete with nominal reinforcements. This use helped a good deal in speedy execution.

For small discharges where the smallness of the barrel does not allow sufficient working space inside, R.C.C. hume pipes are used as barrels.

The head walls, wing walls etc of all syphons are either of stone or brick masonry.

Where the bed of the drainage is having steep slope, a fall (Drop wall) was provided at the upstream of the syphon. The clearance between the soffit of the barrel and top of the downstream curtain wall is kept to a minimum of 3' or half the height of the vent whichever is less in order to prevent choking of the barrel by the incoming sand and grit. Where the downstream ground level is higher than the downstream curtain wall, a tail channel is invariably cut for proper functioning of the syphon."

The design of superpassage is similar to that of aqueduct.

3.2.3.7. Falls :

No falls are required in the main canals and some of the distributaries, as they run on contour. The number of falls required on the channels running on ridges is however numerous due to the rapidly sloping nature of the country. At places the ground falls is as high as 20 ft. to 25 ft. per mile. In order to effect economy to the maximum extent possible, some simple and cheap type falls with locally available materials have been made use of especially for small discharges and low drops.

The following types of falls have been adopted in this Project :

- Glacis type
- Vertical falls with notches
- Well type
- Skin type
- Rock cascade type

3.2.3.7.1. Glacis type :

This type has been used for discharges over 40 cusecs and drops over 3 ft. The glacis is designed with slopes 3.4:1 & 2.3:1. Downstream baffle and cistern are provided for dissipation of energy on the principle of standing waves. Fluming to a maximum extent of 66% is resorted to when found economical.

3.2.3.7.2. Vertical Fall :

Vertical falls with notches have been adopted for discharges upto 600 cusecs for low drops. This type has the special advantage of maintaining the upsteam supply level with varying discharge in the channel.

3.2.3.7.3 Well type Fall :

This type is suited for small discharges with comparatively high drops. They have been adopted for drops from 6 ft. to 12 ft. with discharges upto 40 cusecs. This type generally provides a well in the upstream connected to the downstream by means of Hume pipe. Ordinarily a well is also provided at the downstream. Most of the energy is lost in overcoming the friction through the pipe. This is quite a cheap of fall and incidentally provides for a crossing across the channel with a nominal extra cost. The cost of construction of one 10 ft. fall with a discharge of 32 cusecs combined with a V.R.B. comes to Rs.4200/- against Rs.6100/- for a Glacis type fall for similar conditions.

3.2.3.7.4 Skin type Fall

This type is suited for discharges upto 40 cusecs and drops upto 6 ft. It is designed on the same principles as a Glacis type, difference being that the Glacis in this case is in a slope 4 to 1. Its construction is however very much simpler and it is also much more economical. The glacis and the cistern are generally constructed with one foot thick masonry or concrete. The cost of a fall of this type is Rs.1400/- against the cost of Rs.3570/- of a Glacis type for a discharge of 37 cusecs and drop 6 ft.

3.2.3.7.5 Rock Cascade type

This is yet another very cheap type of fall suitable for low discharges. This type has been adopted for discharges upto 15 cusecs and drops upto 6 ft. The principle of design is same as per Skin type fall except that the glacis is made in a slope 8 to 1. The entire structure is made with one foot dry rubble stone pitching with cement mortar grouting. This type with simply dry rubble pitching has been extensively used in the water courses.

The general guide line for deciding type of fall for different discharges & height are as follows :

Discharge (cusec)	Height (ft)	Type adopted
Q > 50	4' & below	Vertical
Q > 50	Above 4'	Glacis and notch
Q < 50	6' to 13'	Well type
Q = 30-50	4' to 6'	Glacis and notch
Q = 10-30	4' to 6'	Skin and notch
Q < 10	upto 6'	Rubble cascade

3.2.3.7.6 Bridges

"Bridges have been provided at all crossings of channels in the existing roads. They are designed to conform to the standard of the existing roads. Where only an existing track is crossed, a bridge conforming to the standard of village road is provided. All bridges have been designed as per I.R.C. standards. Where a road is crossing a channel in deep cutting and sufficient head room is available, arch bridge has been adopted. In other cases R.C.C. decking has been provided. The substructure in all cases is either of stone or brick masonry. In some of the earlier designs, cement concrete block masonry with cement concrete hearting has been adopted."

3.2.3.7.7 Escape

Two escapes have been provided in the Bargarh canal. They are designed for 2/3rd supply discharges of the canal. They have been provided with regulation gates operated by screw gear arrangements.

In Sason canal an escape weir has been provide between RD 12000 ft to 15000 ft where it skirts around the foot of a hill.

Sambalpur distributory passes through 3 tanks where 3 escapes have also been provided.

3.2.3.7.8 Head Regulator

These are designed either as a weir or an arifice. Depending upon the F.S.L. in the offtaking channel in relation to the bed level in the parent channel, the weir or the orifice has been treated for free flow or submerged conditions. The regulating arrangement in the weir type regulator is with two rows of karries and that in the orifice type is with gates with screw gear operation.

TABLE :3.6(A) DISCHARGE OF SASON MAIN CANAL H.R.

Discharge Q = 0.66 x A$\sqrt{2gh}$, A = Area of Gate opening, H = Height of Water from top of Gate opening (fps system)

Reservoir Level	0.25	0.50	0.75	1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.25	3.50	3.75	4.00	4.25	4.50	4.75	5.00 opening
590-591	20	33	58	78	93	108	123	138	147	156	165	175	176	178	180	182	-	-	-	-
591-592	21	43	65	87	104	121	139	157	169	182	196	208	214	220	227	234	236	-	-	-
592-593	23	46	70	94	114	134	154	174	189	204	219	235	245	255	266	277	284	289	292	293
593-594	25	50	75	101	123	145	167	189	205	224	242	260	273	286	300	314	325	334	341	347
594-595	27	54	81	108	131	154	178	202	222	242	262	283	296	314	330	346	361	373	384	393
595-596	28	56	85	114	139	164	189	215	237	259	281	303	321	339	358	377	394	409	422	435
596-597	30	60	90	120	147	174	201	228	251	275	299	323	343	363	383	404	424	441	456	473
597-598	31	62	93	125	153	182	211	240	265	290	316	342	364	386	408	430	453	472	490	508
598-599	32	65	98	131	161	191	221	251	278	305	332	360	384	408	432	456	479	501	521	540
599-600	33	67	101	135	166	198	230	262	290	318	347	376	402	428	454	484	504	528	550	571
600-601	35	70	105	141	173	205	238	271	301	331	362	393	420	447	474	502	526	553	577	601
601-602	36	72	108	145	179	213	247	281	312	343	374	406	435	464	494	524	551	578	604	622
602-603	37	74	112	150	185	220	256	291	323	356	384	422	452	482	512	542	574	602	629	655
603-604	38	76	115	155	191	227	263	300	334	368	402	436	467	498	530	562	595	624	653	681
604-605	39	79	119	159	196	233	271	309	344	379	415	451	483	516	549	582	615	646	676	706
605-606	41	82	123	164	202	240	279	318	354	390	427	464	498	532	566	601	635	667	699	730
606-607	42	84	126	168	207	247	287	327	364	401	439	477	512	547	583	619	654	688	721	753
607-608	43	86	129	172	212	253	294	335	374	413	452	491	527	563	599	636	673	708	742	775
608-609	43	87	131	176	217	259	301	343	388	423	463	503	540	578	616	654	691	727	763	797
609-610	44	89	134	179	222	265	308	351	392	432	474	515	553	592	631	670	709	746	783	819
610-611	45	91	137	183	227	271	315	359	400	442	484	526	566	606	646	687	726	765	802	839
611-612	46	93	140	187	231	276	320	369	409	452	495	536	579	620	661	702	743	783	821	860
612-613	46	96	144	191	236	283	330	374	417	463	509	549	590	635	670	718	760	800	840	879
613-614	49	98	147	194	241	290	337	381	425	472	519	561	602	648	694	733	776	817	858	899
614-615	50	100	150	198	245	294	343	388	433	481	529	572	613	660	707	747	792	834	876	918
615-616	51	101	152	201	249	299	349	395	440	489	538	582	625	673	721	762	807	851	894	936
616-617	52	103	155	204	253	304	355	402	449	499	549	593	636	685	734	776	823	867	911	954
617-618	53	105	158	208	257	309	361	408	456	507	558	603	648	698	748	790	837	883	928	972
618-619	53	106	159	211	262	314	366	415	463	515	567	613	659	710	761	804	852	898	944	990
619-620	54	107	162	214	266	319	372	422	472	524	576	623	669	721	773	817	867	914	961	1007
620-621	55	109	165	217	270	324	378	428	479	532	585	633	681	733	785	834	881	929	977	1024
621-622	56	111	167	221	274	329	384	434	485	539	591	644	694	744	795	845	895	944	992	1040
622-623	56	112	169	223	277	333	389	441	493	548	600	653	701	755	807	858	909	958	1008	1057
623-624	57	114	171	226	281	337	393	447	499	555	609	663	715	767	819	871	922	973	1023	1073
624-625	58	116	174	230	285	342	399	455	506	562	617	672	725	777	830	883	932	987	1038	1089
625-626	59	117	176	233	289	347	405	460	513	570	625	681	735	788	842	896	949	1001	1053	1104
626-627	59	118	178	235	292	351	410	465	520	578	634	690	744	799	853	908	962	1015	1068	1120
627-628	60	120	180	238	296	355	414	471	527	586	642	699	754	809	865	920	975	1028	1082	1135
628-629	61	121	182	241	300	360	420	477	533	592	650	708	764	820	876	932	987	1042	1096	1150
629-630	62	123	185	244	303	364	425	482	539	599	658	717	774	830	887	944	1000	1055	1110	1165

In the case of H/R of main canals taking off from the reservoir, the waterways provided is such as to enable the canal to draw full supply at the lowest water level of the reservoir. For smaller distributaries and minors, the regulators are designed to draw full supply when the parent channels are running at half full supply. The relationship $Q = 5A(h)^{0.5}$ has been adopted to determine the required waterways. Charts/tables have been developed relating reservoir level, gate opening and flow into canal in case of B.M.C./Sason/SBP distributary taking off from the reservoir. (charts attached to reservoir operation). Table showing the discharge of Sason Main Canal is enclosed vide Table No.3.6(A).

Important informations on Hirakud Distribution System

- No. of Main canal and branches 210
- No. of distributaries and minors 990
- No. of outlets (authorised) 1660
- Water was first released in Bargarh main canal on 07.09.1956.
- Length of canals:
  - Main & Branches = 91.5 miles ≃ 146.4 km
  - Distributories & minors = 460 miles ≃ 736.0 km
  - Water courses = 2000 miles ≃ 3200.0 km
- In Bargarh main canal, there are 2 nos. of branch canals of 902 and 506 cusecs capacity

  21 no. of distributaries having

  < 100 cusec - in 14 nos.

  100-200 cusec - in 4 nos.

  300-400 cusec - in 3 nos.
- Major aquaducts in Bargarh main canal are (i) Danta (ii) Girisul (iii) Jira & (iv) Ranijhor where Q > 33000 cusec in drains.
- In Sason canal, there are 8 nos. of distributaries.
  - Paramanpur - 12 mile long (ridge) with Q = 140 cusec
  - Huma tail - 40 mile long (contour) with Q = 327 cusec
  - 6 no. of small distributories.
- Total nos. of masonry structures in Hirakud canal systems are shown in Table 3.6.(B)

Table-3.6(B) Details of Masonry Structures in Canal System.

	Structure	No. of structures
A.	Main canal & branches	
	1. Head regulators	60
	2. Falls	7
	3. Aqueducts	25
	4. Syphons	78
	5. Under Tunnels	3
	6. Superpassages	2
	7. Bridges	31
	8. V.R.B. cum H.R.	2

	9. Escapes	3
	10. Flush inlet	1
	Total	**211**
	11. Outlets	144
B.	Distributaries & Minors	
	1. Regulators	95
	2. Falls	447
	3. Aqueducts	28
	4. Syphons	285
	5. Under Tunnels	4
	6. Superpassages	123
	Total	**986**
	7. Outlets	1397

Besides above there are approximately 30,000 nos. of 1'-0" high stone pitched falls, 1660 nos. of pipe syphons and 1300 nos. of road crossings in water courses.

There are many unauthorised outlets constructed by land owners at later stage to take water to their fields.

Schematic diagram of the entire canal system is shown in Drg. No.3.14(A) to (E)



Surplus in Power Channel

3.3. Model Study :

3.3.1 Introduction

"The use of hydraulic model as a tool in hydraulic design is commonly accepted. Many hydraulic phenomena which occur in nature are too complex to be described by rigorous mathematical techniques and models are used as an alternative means of obtaining the information necessary to complete an efficient and satisfactory design. Even in relatively simple situations, such as the design of spillways or river barrages, it is often impossible to predict the exact nature of the flow patterns without conducting a model study. In more complex cases, such as those involving the release of pollutants from industrial or thermal outfalls, model studies have become almost mandatory because of the necessity of satisfying regulatory boards that there will be no harmful effects on the water body.

A model may be thought of as a prediction device in which the full-sized phenomenon, the prototype, is reproduced at a small scale. Geometric models are quite familiar. In these, every length of the prototype is scaled down by the same amount so that in architectural models, for example, the public may gain an appreciation of the 'look' of a building and may understand how it will fit into its surroundings. Hydraulic models, however, although often geometrically similar to the prototype, are intended to reproduce physical phenomena and can therefore only be useful if the important physical aspects of the prototype are reproduced at certain scales which can be calculated. An incorrectly designed model cannot be used as a prediction device and the determination of the criteria which must be satisfied to ensure the validity of the model is therefore of prime importance. These criteria are represented by various model laws which must be used in the design of the model. In turn, the model laws give rise to scale ratios so that measurements of velocity and force etc.,. can be made in the model and scaled up to give the appropriate prototype value.

If the mathematical equations which govern the phenomena are known in sufficient detail it is possible to use these equations and develop modelling criteria without great difficulty. Unfortunately, however, this is not often possible because most model studies are undertaken precisely to deal with conditions which are not amenable to rigorous mathematical analysis. Under these circumstances the modelling criteria must be developed from incomplete methods of analysis." (Refer Hydraulic Modeling by J.J. Sharp)

Hirakud multipurpose project encompasses several complex hydraulic problems. Therefore model studies were conducted in respect of many important structures / structural components of Hirakud Project for refining the designs to achieve economy and for better performance.

At the initial stage of project construction there was no full-fledged research station in India to conduct the model studies. Therefore Central Water Irrigation, Navigation Commission (CWINC) of Government of India, who was in charge of project construction referred the matter to Govt. of U.S.A. Dr.John L.Savage of U.S.A. was retained as a consulting engineer for Hirakud Dam Project. On his recommendation, International Engineering company, Inc, Denver, Colorado acted as a co-ordinator and designer. Some model studies were conducted in the hydraulic laboratory of Colorado Agricultural & Mechanical college at Fort Collins. The report was prepared by Civil Engineering section, Colorado Agricultural Experiment Station, Fort Colins in January 1950.

Subsequently Governmnet of India (GoI) established a research station at Pune named as Central Water & Power Reseach Station (C W P R S) for conducting various model studies. Problems of Hirakud Project were refered to them for getting efficient solutions.

Of late a research station was started at Hirakud Island near Burla in the year 1972 to conduct model studies. It was renamed as Sailendra Narayan Engineering Research Institute (SNERI) in the year 1983 to commemorate the late S.N. Bhanjdeo, the illustrious son of Orissa for his

valuable contribution towards water resourses development in the state. Few model studies pertaining to Hirakud Project were also conducted during 1972 to 1989 in this institute. Only one division, named as Hydraulic Research Division is functioning under this institute.

3.3.2 Model Studies conducted

The following model studies (except Sl.1,32 & 33) have so far been conducted by CWPRS. Pune in connection with Hirakud Dam Project. The bracketed figures mentioned below indicates the year of CWPRS Annual Research Technical Publication.

1. Hydraulic model tests for Hirakud Dam by Agricultural & Engineering Research Station, Colorado, U.S.A. on behalf of C W I N C, Govt of India (1950).
2. Hirakud left channel spillway-Testing the designs of retaining wall and training walls (1951).
3. Discharge characteristics & dissipation of energy with conventional & upturned deep seated sluices. (1951)
4. Discharge characteristics & dissipation of energy with conventional & upturned deep seated sluices. (1952)
5. Cavitation free design of Hirakud Dam sluices (1950).
6. Discharge characteristics & dissipation of energy below sub-sidiary dam spillway (1951).
7. Peak run-off in Mahanadi by Unit Hydrograph- (1951).
8. Analysis of the minimum section between the sluice gate gallery & downstream face of Hirakud Dam (Spillway section). (1951)
9. Relative Discharge in the left & Right channels of Mahanadi at Hirakud Dam site (1952).
10. Discharge & Pressure characteristics of alternate desings of entrance bell mouth of main sluice. (1952)
11. Pressure distribution in power Penstock (1952).
12. Testing alternate crest profiles for discharge characteristics & interference with under sluice jets (upturned type) (1952).
13. Photoelastic analysis of stress concentration around the sluice gate galleries - part.1(1953)
14. H. D. P. -- Supplimentary tests on pressure distribution on the sluice invert at entry due to alternation consequent on change in design of emergency gate. (1953).
15. Hirakud Left channel spillway—Testing vortex formation on upstream left flank wall adjoining the earth dam. (1953)
16. Hirakud Right Channel spillway - Testing the design of divide wall & training walls (1953).
17. Photoelastic studies of Hirakud dam spillway section (1954).
18. Irrigation outlet in left dyke gap-2— Testing design of outlet (1954).
19. Irrigation outlet in left dyke gap-4 – Testing design of outlet (1954).
20. Hirakud Dam Project - Working of deep seated sluices (1956).
21. Silting in relation to model of Hirakud lake— (1956).
22. Silting in relation to Hirakud Reservoir — (1957).
23. Silting in relation to model of Hirakud reservoir — (1959).
24. Performance of Ski - jump energy dissipators— by C L Handa & K.C. Thomas — C.W.P.R.S-Model & Prototype conformity Vol -I - Golden Jubilee Symposia - (1961).
25. Evaluation of scour below ski-jump buckets of spillway— P.M. Dimple et al — C.W.P.R.S- Model & Prototype comformity Vol-I Golden Jubilee Symposia - (1961).

26. Behaviour of Hirakud spillway - Conformity of model & prototype by J.Tripathy & A.B. Jena - Golden Jubilee symposia (1961).

27. River diversion for construction.

28. Model study on drainage syphon.

29. Model study on canal falls.

30. Photo-elastic model study — C.W.P.R.S specific note no 1778 /16.09.1978 - vol.IV.

31. Development of Mathematical model for flood forecastings for Hirakud reservoir & downstream catchment up to Mundali by Dr. B.U. Nayak— C.W.P.R.S Technical Report No. 3409 - March 1997.

32. Mathematical model for flood forecasting— by Institute of Hydrology— Wallingford, England.

33. Hydraulic model for surplus escape of Hirakud Power channel - stage III - Expansion of Hirakud Dam site power station— Vide : 2nd Annual Report 1984 & 3rd A.R. - 1989 - by S N E R I (Hydraulic Research Division, Burla).

N.B. The above model studies excepting SL 30, 31 & 32 were conducted to support design aspects but Sl 30 was in connection with crack development and Sl 31, 32 concerned with reservoir operation. Besides above, some studies, investigations were conducted at CWPRS Pune, I.I.T. Delhi, CRII, CSMRS, HRS etc in connection with crack in Hirakud Dam Spillways & Power House which have been briefed subsequently.

3.3.3 Brief Discussion on Few Model studies

3.3.3.1 Model study at Colorado, U.S.A

The original designs of Hirakud Dam Project were prepared in 1949 by the International Engineering Co. of U.S.A. in consultation with Central Water & Power Commission, New Delhi (C W P C). Some of the hydraulic features of the spillway & sluices were studied on hydraulic models at Fort Collins, Colorado U.S.A. The tests were only preliminary in nature and were neither exhaustive nor conclusive.

Limitations in time and funds restricted the investigtation to studies on a four-half bay sectional model for spillway & sluices.

The auxilliary and service spillway aprons were investigated & developed from the tests on the sectional model. Two types of sluices— open channel & pressurised were investigated and finally a separate model of one sluice outlet was constructed & tested to develop a more satisfactory inlet shape. Because of the influence of the overfall spillway discharges on the sluice outlets, the pressurised sluice was recommended.

Fort Collins recommended for additional model tests which reads in brief as follows: :**Sluices** : Models of the sluices should be constructed and tested to be certain that excessive sub-atmospheric pressures do not exist in the inlet or the barrel of th sluice. Lack of funds and time prevented completion of these studies.

Complete River model : The entire spillway section of the project should be modelled and if possible, the river banks should be included within the model limits. Sufficient upstream and downstream topography should be modelled to provide proper approach and exit conditions and a portion of the earth embankment on the left side should be constructed.

Test should be made using the apron and bucket recommended as a result of these tests. The effect of wave heights on the earth dam, river banks and dyke may be studied. Longer aprons should be tested to determine their effect on reducing the wave action and a bucket with a slightly flatter trajectory to be investigated for the same reason. It is known from past experience that the recommended bucket provides nearly the best operation. It is possible that a bucket of the same

radius but with a slightly flatter trajectory might improve the performance. Other tests might be made on the trajectory bucket to determine whether ventilating the air pocket under the nappe leaving the bucket lip would improve the stability of the jet and result in a quieter water surface downstream. A dentated bucket lip might help to ventilate the pocket & improve the performance in other ways.

In all tests on the spillway full consideration should be given to the size of the waves. Medium & large height waves should as far as possible be avoided.

The training walls of the spillways should be investigated to determine their length and height. Erosion tendencies at the end of the walls should be studied for various combinations of open and closed gates & sluices.

A schedule for operating the prototype structure should be evloved since severe damage may be inflicted upon the trajectory bucket by improper manipulation of the sluices and the over flow spillway.

The effectiveness of the rock dyke in the right in properly directing the flow down the river should be investigated. Therefore Colorado recommended to construct a full river model of the dam site locality with inclusion of full spillwys, dam and Power House etc. for further studies.

In the mean time some further data were made available from the project regarding the discharging capacities of th left and rihght channels of hte river at Hirakud. It was proposed, therefore to modify the spillway design both in lay out and other features of the structure. It became bare necessity to initiate model experiments in India to study these proposals. It was decided to conduct all these tests at the Central Water & Power Reseacch Station Pune (C W P R S) of Govt. of India, where excellent facilities for large scale model work were available by then.

The following paras describe the results of some of the model studies in brief conducted at CWPRS, Pune.

3.3.3.2 Hirakud left channel Spillway— Testing the design of Retaining & Training walls– CWPRS - (1951).

Full witdth model of left spillway (GS.1:100) with 12 under sluices upturned type in central blocks & rest 28 nos. conventinal type provided with outside bucket. Each spillway block length was 60 ft.

Results :

(i) On upstream side the retaining walls & flank walls are satisfactory. On downstream with end sluices closed under maximum discharge condition, the height of left retaining wall (top RL 550') was sufficient. The elevation of right training wall, however, may be kept at RL 550 ft as aganest 540'designed. If the end bays and sluices were to be operated at maximum flood, the walls would have to be raised to 570' on right and 575ft on left.

(ii) For divide walls between spillway blocks having two types of sluice ways a height up to EL 530 ft is recommended, the walls being extended up to the end of bucket. (Remark-upturned sluices were finally abandoned & therefore divide walls were eliminated)

(iii) The 50' radius bucket with 40 ° lip angle as per C.D.O. design is sufficient from the point of view of a good throw of the jet overlying a strong return roller which would pile up scoured material at the toe of the dam, thus renderdeg it safe against undermining.

(iv) The protective work at the toe of the central blocks which would be unnecessary in the absence of overflow discharge can consists of a 35' long apron with 40' radius and an end sill with top R L 510 ft. For uniformity of construction, however, the 50ft radius as in (iii) can be accepted being on the safe side.

(Remark-Subsequently upturned sluices eliminated)

(v) On the Upstream side, near the left flank wall the five end sluices can be protected by trash rack to provide against sucking action of the eddy observed or they may be kept closed for upstream water elevations between RL 590 to 610 ft.

3.3.3.3 Discharge characteristics and dissipation of energy with conventional & upturned deep seated sluices (CWPRS 1952).

(Study was conducted on a $^1/_{20}$ G.S model with one sluice & on a $^1/_{40}$ G.S. model with 4½sluices).

Three types of designs were tested.

(i) Upturned sluices-Design-C (ii) C.D.O. Bell mouth-Design-A

(iii) I.E.C. Bell mouth-Design-B

As already indicated earlier upturned sluices were eliminated & convertional type sluices were adopted through out.

The co-efficient of discharge for design A & B is same i.e 0.89 for sluices . The co-efficient of discharge and discharge of sluice and overflow bay working individually and in combination is given in Fig Drg. No 3.15.

The overflow spillway coefficient is given in Fig 3.15.

It varies from about 3.3 to 4.0 in fps system

For combined overflow & undersluice discharge, the discharge was less than when they worked independently. Since, however, the discharging capacity of the overflow spillways increases much more rapidly with head than in the case of sluice ways, this lessening of the discharge can be accomodeted by a slight increase in the head only.

The sluice ways with design A (CDOtype) as well as the upturned sluice ways (design-C) can be accepted as satisfactory from the point of view of absence of sub-atmospheric presecures (N.B. In actual construction up-turned sluices have been eliminated.)

3.3.3.4 Pressure distributon in power penstock (CWPRS-1952).

The tests were undertaken in a 1:20 G.S. scale model. The tests were on the utilisation of 18000 cusec by 4 units @ 4500 cusec per unit.

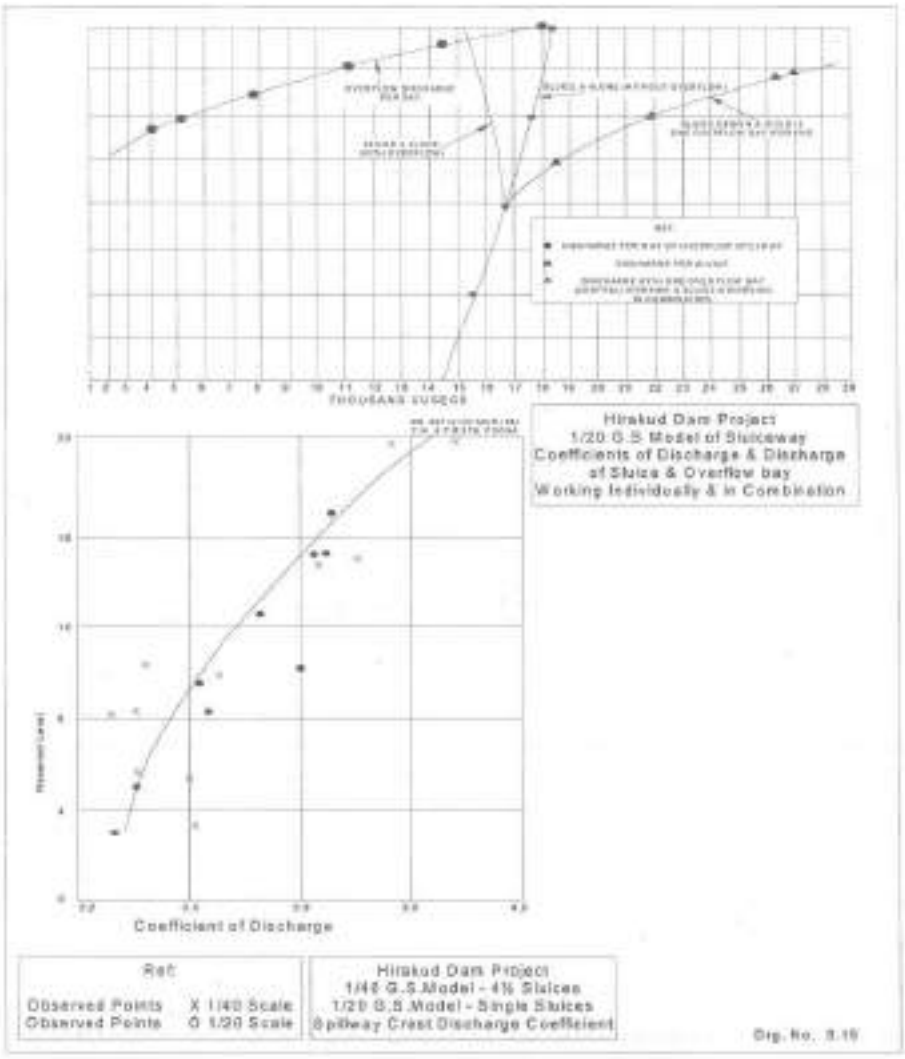
Results : The pressure conditions for the entrance to the power penstocks are satisfactory being all highly positive under different discharge conditions.

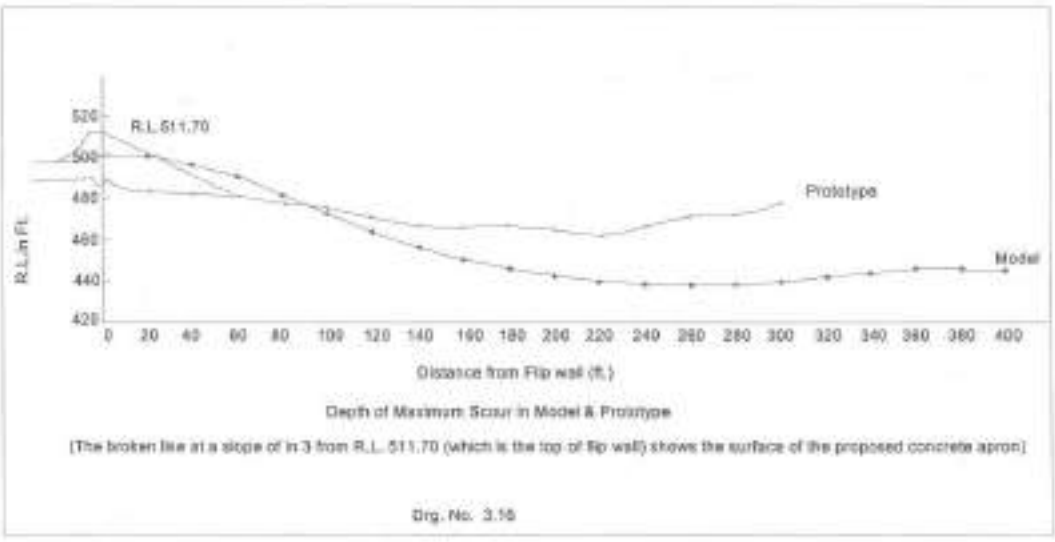
3.3.3.5. Hirakud Dam-left channel spillway -1:50 scale part width model - Testing vortex formation on u/s left flank wall adjoining the earth dam.(CWPRS - 1953)

Results : For reservoir level more than R.L. 610′, the vortices were formed only when the end over flow bays were kept closed. With opening of these bays, the higher surface velocity induced upstream of the spillway prevents the eddies over the spillway & vortice formation is eliminated.

In oder to prevent the lateral flow from the reservoir region over the sloping face of the earthdam, a divide wall was erected on the wing wall with top RL at 610′ as it was essential to eliminate the vortices at least for reservoir level up to RL 610. The optimum length of the wall was 160′. This wall effectively stopped the lateral flow & no vortices formed; a slow return flow at surface, however, was present in the shadow of the divide wall. For water level above 612′ or so, as the divide wall was submerged, the vortices reappeared again but these could be suppressed by operating the end crest gates of the overflow spillway.

If it is necessary to eliminate vortex formation without operating the end crest gates for RL more than 610′, the divide wall would have to be correspondingly raised to the required elevation. Normally such condition is not anticipated & the divide wall elevation need not be taken above RL 610′.


THOUSAND CUSECS
Coefficient of Discharge
Hirakud Dam Project
1/20 G.S Model of Sluiceway
Coefficients of Discharge & Discharge
of Sluice & Overflow bay
Working Individually & in Combination
Ref:
Observed Points X 1/40 Scale
Observed Points O 1/20 Scale
Hirakud Dam Project
1/40 G.S.Model - 4½ Sluices
1/20 G.S.Model - Single Sluices
Spillway Crest Discharge Coefficient
Drg. No. 3.15


R.L.511.70
Prototype
Model
R.L.in Ft.
Distance from Flip wall (ft.)
Depth of Maximum Scour in Model & Prototype
[The broken like at a slope of in 3 from R.L. 511.70 (which is the top of flip wall) shows the surface of the proposed concrete apron]
Drg. No. 3.16

3.3.3.6 Hirakud right channel Spillway - Testing the design of divide wall & training walls. (CWPRS - 1953)

Study was conducted on a 1:100 G.S. scale model.

Results -:

(a) Left downstream training wall :

Good improvement in the flow condition was obtained by extending the last training wall by 50' beyond the bucket. The top level has to be raised from RL 525' to RL 530' to prevent spill into the bucket from behind the training wall.

(b) River bed downstream (erodible) :

With the spillway discharging under maximum water levels both in upstream & downstream the maximum depth of scour of 55' occured at about 200' d/s of the bucket, the lowest scoured level being RL 450'. Due to deep pool formed at the point of impact of the jet, a hydraulic jump formed and the flow downstream of the jump fanned out to greater extent than in the test with non-erodible bed. The return flow on the flanks of the spillway both in the powerdam tail race and near the earth dam d/s retaining wall increased in strength.

(c) Flow conditions near the right training wall, dividing the power dam from the spillway dam :- the jet from the spillway beyond the toe swept over the bucket & while deflected upwards, a standing wave formed below the point of impact of the jets close to the d/s end of training wall. Part of the energy was transferred to the surrounding mass of water producing an undulating surface in front of the draft tube exit, the maximum range of undulation being about 5.5.'

Flow deflected by the training wall was seen to travel towards guide wall on the right bank of the river. Part of this produced a return roller over their entire tail race of the power block. A deep scour hole formed at the nose of the right training wall. To improve the situation, the training wall has to be extended by 110' followed by a sloping nose. In plan, the extended wall showed an expansion of 1:10 away from spillway block.

(d) Flow conditions near the left training wall and earthdam downstream training wall:

As the earthdam downstream of retaining wall is receded from the spillway by about 100', the training wall on the left end of the spillway does not extend beyond the bucket.

With erodible bed downstream, a strong return eddy formed in front of the pocket between the training wall & the retaining part of this was deflected by the curved retaining wall parallel to the toe of the earthdam. Scour depth in part of training wall & close to the curved retaining wall was as low as equivalent to 15' (RL 485')

3.3.3.7 Performance of ski-jump energy dissipators (CWPRS-Golden Jubilee Symposia - 1961)

The Hirakud Dam came into operation in the year 1956. It can pass a maximum flood of 41,630 cumec (14.7 lakh cusecs) with an intensity of discharge of 104.1m^3/sec/mt (1121 cusec/ft.) length of spillway.

In a composit model tests conducted under the condition of F.R.L. & with all sluices & overflow vents open, the deepest scour was seen to extend to a depth of 18.288m (60') below the river bed at a distance of 83.8m to 114.3m(277' to 375') d/s of bucket. A scour of 9.14m (30') was observed in the d/s at about 38.1 to 83.82m (125' to 275') d/s of bucket. The deepest scour so far observed in the prototype (1966) is of the order of 13.7m (45') occuring at a distance of 60.96m to 67 m (200'to 220') form bucket. Scour to a depth of 9.14m (30') has progressed u/s from this area to as near as 24.38m from bucket. Deep scour pockets have occured in front of

sluices 9,10,21,22,33,34,35 &37 because of probably more frequent operation of under sluices & spillway vents in these area & also due to the existence of minor fault there. It was also seen that, while in the 10 years of operation a scour of 6.1m to 7.6m (20′ to 25′) depth developed, a further scour of 1.83m (6') occurred during the subsequent years indicating clearly a progressive reduction of rate of scour & a tendency towards stabilisation of scour profile.

In concluding remark it can be said that with fairly good foundation rock & tail water depth inadequate for jump, the ski jump design is satisfactory & economical. All observed scours gererally are well within those anticipated on the basis of model tests and the deepest scour is sufficiently away so as not to endanger the main structure. Where however the main structure is weaker, special precautions will be required to prevent the retrogression moving upstream towards the dam.

3.3.3.8 Behaviour of Hirakud Dam spillway- Conformity of Model & Prototype results – Golden Jubillee symposia - 1961.

- **Scour below spillway : -** The scour pattern shown in the model study of right spillway is more likely to be similar to the prototype condition. The deepest points of scour as indicated in the model study have been shown in the Figure enclosed. (vide Drg. No. 3.16)

Similarly all deepest points of scour actually observed upto end of 1962 have also been plotted. It can be seen that though the deepest scour indicated in model study has not been attained in proto, the actual scour upto a distance of 100' from flip wall is much deeper (20' at places) than indicated in model study.

- **Jet Trajectory:-**

While conducting prototype observation of the spillway jets at either ends of the spillway near the training walls, it was found that, while there is less powerful jet near the training wall than the jet away from it, the tendency of the poweful jet is to lift the less powerful jet near the training wall to a much greater height than RL 556′ and to throw it towards the earth dam. Thus it establishes a definite sequence for operation of gates as given below.

(a) The end sluices are to be opened first.

(b) The sluices away from the training wall are to be opened serially towards the middle of the spillway.

(c) If crest gates are to be opened then these may be opened from the middle of the spillway towards the ends.

While closing, the reverse should be followed in case of (b)& (c)

The sequence of operation of end sluices is limited to the four end sluices and two end crest bays only since jets beyond these do not affect the training walls.

With these restrictions on the sequence of operation of end sluices & crest bays, the top of the training walls may remain at RL 557′ with 3′ parapet on it. But in case of full freedom of operation, the training walls may have to be raised to RL 588′ i.e. to the maximum R.L. of top of jet.

- **Water Level in sluice gate groove due to back pressure :-**

At maximum reservoir level of 630', when the sluice gates only are open, the back water in the sluice gate groove remains below RL 557' i.e. the floor of operation gallery. But when the corresponding crest gates are open, the discharge from the crest damps the sluice jets while falling over it. This then rises the back water in gate groove which rises to about RL 559'. Thus the gallery gets flooded. As a precaution masonry walls to RL 560' are recommended round the gate grooves.

• **Vortex Upstream of Spillway.**

The model studies have established that when there is no crest discharge, vortices are formed in front of end sluices, particularly at the left end of left spillway.

Prototype observations for various combinations of sluices & crest bays at different levels of the reservoir were made.

Observations are summerised below :-

(a) when crest bays are open-no vortices are formed.

(b) When there is no wind & reservoir is practically calm, vortices are conspicuous

(c) At RL 630' of reservoir, no vortices formed by opening sluice gates 1 & 2 with no crest gates opened. At lower reservoir levels, vortices occur.

(d) Vortices are more when larger number of sluices are open contiguously.

(e) The maximum size of vortices so far observed upto 1966 are 5′ diameterwith about 10' depth.

(f) The air column is not continuous upto sluice entrance.

(g) The vortices are not limited to end pair of sluices. These have been observed generally in front of sluices no 1 to 10 & 20 to 40 & almost any where.

3.3.3.9 Evaluation of scour below ski-jump buckets of spillways : by P.M. Dimple et al. (1961)

$ds = a\ (qH)^b$

where ds = depth of scour below TWL (ft), a & b are constants

q = Discharge of spillway in cusec per foot run (cfs/ft)

H = Fall from reservoir water level to flip top (ft)

The upper limit is $ds = 0.36\ (qH)^{0.50}$ (to evaluate ultimate scour)

The probable scour = $ds = 0.360\ (qH)^{0.50}$

Minimum Scour = $ds = 0.20\ (qH)^{0.50}$

Considerations for future designs :-

(i) Minmum radius of ski-jump bucket may be assumed as $R = \sqrt{hH}$

where - h = depth of overflow

H = Fall from crest of spillway to the invert of bucket.

This may have to be increased where sluices with large operating discharges are embodied in the spillway.

(ii) Lip angle be kept betwen 35^0 to 40^0. Lip level of bucket should as far as possible be placed near about the maximum Tail Water Level (TWL).

(iii) In order to protect the end sill form under-mining it is required to backfill the excavated portion of the bucket & to construct a sloping concrete ramp for a short distance from the lip.

(iv) In no case retrogression be allowed to travel close to bucket.

3.3.3.10 Dissipation of energy & discharge characteristics below sub-sidiary dam & spillway :- (CWPRS - 1952)

The sub-sidiary dam, intended for generating more power includes a simple overflow spillway, spanning 92' of the power channel. The maximum design head was 15'. The recommended design

of toe protection consisted of a 35' horizontal apron with a 10' high end-baffle. A row of 6' wide & 5' high blocks stream lined in plan was recommended.

3.3.3.11 Irigation outlet in left Dyke-gap-2 and gap-4, CWPRS - (1954)

The design of the irigation outlets in gap no.2 & 4 were tested on models & recommendations were made for improving the flow conditions. The provision of blocks on the apron floor downstream of the out let was found necessary to obtain uniform flow distribution in the canal downstream. The following recommendations were also made.

Irrigation outlet in Left dyke - Gap - 2

a) The apron level downstream of the main sluice outlet may be raised to RL 576' with an end sill of 1.25' high. The floor level in the stilling pool d/s of this apron can be retained at RL 568' as designed. But if ground conditions permit this floor level may be raised to 571'. A row of one foot cubical blocks with its centre line 2' d/s from the end sill may be placed on the apron. The top level of side walls between the oulet & the wave damper may be raised to RL 585'.

b) The transition in plan of the side walls just d/s of main outlet may be modified as suggested.

c) A 5.5' high wave damper in form of a 18' cross beam with bottom RL 579.5' should be constructed 43' d/s of outlet.

d) D/S of Sambalpur distributary weir, the apron level be kept at RL 576'.

e) A 6" dia air vent be povided in the bell mouth of the main sluice or between emergency gate & service gate.

Irrigation outlet in left dyke - Gap - 4

a) The transition in plan of the conduit may be modified by adopting two circular arcs forming an S-curve with R = 250ft.

b) The protective work at the outlet is to be modified as suggested.

c) The bed and sides of the conduit should be finished such that they are able to withstand velocities of the order of 50'/sec.

d) The canal bank & bed may be pitched to a length of 45' downstream of the end baffle.

3.3.3.12 : River Diversion for construction.

For river diversion, temporary low dams are put up, diverting the flow from the work-spot. The optimum & economic heights of these dams were worked out at CWPRS by model studies which were then implemented in the field for the first & second stage of construction.

3.3.3.13 : Drainage Syphon.

The design for the Baragarh canal drainage siphon provided seven barrels to pass a maximum 24000 cusecs dischage of the nala (rivulet). The transitions of guide walls for the approach were modified so as to obtain, as far as possible, a uniform distribution of flow with minimum loss of head. The transitions on the downstream side were also modified. Measures for the dissipation of residual energy of the exit were also developed.

3.3.3.14 : Canal fall.

Studies for developing economic & efficient design of canal falls were carried out at CWPRS. For the use of the Hirakud Project Authorities design of canal falls for discharges ranging from 100 to 1500 cusecs and for heights of falls below 10' were prepared.. These designs are of general applicability.

3.3.3.15 : Photoelastic model studies (Hydrostatic load) by P.C. Saxena,

- CWPRS Specific Note No 1778 dt 16.9.1978.

A composite photoelastic model of Hirkud spillway was prepared to study its structural behaviour & to find out possible reasons of cracks in concrete of spillway as refered by the Director (HCO-II-CWC). The spillways (both left & right) are having 34 nos of radial gates of size 17.4m x 6.1m (51' x 20') each & 64 nos of sluices of 3.66m x 6.20 m(12' x 20'-4") each.

3-D model of block no 44 with G.S. scale of 1:240 was prepared from stress free araldite block to study stress distribution in spillway block and to find stress concentration at operation gallery.

At the upstream boundary of spillway block tensile stresses vary linearly with maximum value of 2.5 p_o at the base, while at the downstream surface boundary stress is positive (5 p_o) at the base.

('0' at top both in upstream and down stream surface)

p_o = 0.277 kg/sq.cm.(scale 1 cm. = 2.5 p_o)

Around operation and drainage gallery :-

High boundary stresses develop around the corners of operation gallary with maximum of 2.6 p_o

In drainage gallary stresses are distributed.

Boundary stresses around the openings of the connecting passage between the operation gallery & sluice is found to be insignificant.

$$\text{Shear stress } T_{xy} = \gamma_{xy} = \frac{\delta_1 - \delta_2}{2} - \sin 2\theta, \delta_1 - \delta_2\ \eta F$$

where n = Fringe order, F = model fringe value, θ = Isoclinic angle

Shear stresses are plotted in terms of δ_0

$$\text{Where } \delta_0 = \frac{W}{L \times b}$$

W = Total load applied, L = Base width, and b = Width of block.

It was observed that shear stresses are greater at lower levels.

Shear stresses vary uniformly with value of $2.3\delta_0$ at the downstream. Higher shear stresses are observed at the region of sluices, thereby indicating the total shear at this region is much higher than in other parts.

δ_0 = W = 0.178 kg /cm ^{2} (1.cm = $2.5\delta_0$)

Stresses around sluice barrel :-

Stresses around the boundary along the vertical planes are compressive while tension occurs along horizonatal planes.

Maximum compression occurs at corner & is 5.1 p_o

Maximum tension occurs at centre of bottom surface & is 4.3 p_o

where p_0 = 0.277 kg / cm^2 (scale 1 cm = 1 p_o)

2-D photo-elastic model studies were also made in scale 1:120

Conclusion :

Maximum compressive tangential stress of the order of 2.6 p_0 occurs at the corner of operation gallery, p_0 being pressure at the base. The shear stresses along the sactions at the region close to sluice ways are generally higher than other regions.

From 2-D model studies, the tangential stresses around the sluice opening subject to tensions were obtained. The tangential stresses at the corners were of the order of $2.5\delta_0$ while at other locations, they are generally low of the order of $0.45\delta_0$

3.3.3.16 : Development of Mathematical Model for flood forecasting for Hirakud Reservoir & Downstream catchment upto Mundali (CWPRS-Technical Report No 3409 - March 1997).

The flow routing model has been calibrated for Mahanadi starting from Rajim to Mundali in the following reaches :

i) Rajim-Seorinarayan - Basantpur

ii) Basantpur - Hirakud

iii) Hirakud - Khairmal

iv) Khairmal - Tikarpara

and v) Tikarpara - Mundali

Utilising 6 to 10 years data for obtaining routing parameters K and X in Muskingum approach. Care was taken to choose such floods which are above the threshold value of 7000 cumec (2,47,240 cusec) upstream of Hirakud & 15000 cumec (5,29,800 cusec) downstream of Hirakud.

After calibration & testing models, it may be stated with confidence that the average parameters derived from several calibration runs are sturdy & give results with high level of confidence & associated low levels of errors.

A procedure has been evolved for using the flood forecasting model for real time operation & issuing forecasts for inflows to Hirakud resevoir as well as for forecasting the discharges and levels at Mundali. Considerable amount of material has been included in the report for accounting for the gauged as well as ungauged flows in the forecasing model. It is left with uncertainties about adding ungauged flows at an appropriate location in the main river upstream of Hirakud for routing down to the point of interest.

It is pertinent to caution that the flood forecasting model for Hirakud dam and downstream catchment up to Mundali have ben developed and tested for their operation in the laboratory using historical data. These models are yet to be tested real-time in their operatonal modes. Till such time, the model is tested adequately in real-time, the current method of forecasting may be used. The present mathematical forecasting model could also be run in parallel for cross checking the results and testing adequancy and appropriateness of using the model in future. After obtaining confidence in using the model, a complete switch over is recommended.

(N.B. :- This model has not been used for forecasting till end of 2009). It should be used without further delay.

3.3.3.17 : Peak runoff in Mahanadi by Unit Hydrograph Method (above Sambalpur)

Peak runoff in Mahanadi above Sambalpur was arrived by utilising data from 63 rain gauge stations spread over 32,200 sq miles of catchment above Hirakud.

The method is developed by selecting an isolated unit period's storm, presupposing thereby the total absence of precipitation in the preceding & following unit periods. For this purpose the heavy isolated storm occuring on 28th September, 1949 was selected. From the runoff records at Sambalpur during the period it is seen that, it requires 3 days from the day of storm for the peak flood to be registered at Sambalpur.

From the analysis it was concluded that any flood control measure on the river near Sambalpur will have to be designed for an expected total run-off of about 15 lakh cusec. The dam spillway has been designed with capacity of 15 lakh cusec.

3.4 Displacement, Resettlement & Rehabilitation

3.4.1 Introduction :

Every action has cause behind it. In case of R & R, it is the displacement of existing human & live stock settlement. Displacement therefore necessiates resettlement elsewhere & rehabilitation for sustainance.

Displacement is of two types :

i) Voluntary displacement & ii) Involuntary or Induced displacement.

Voluntary displacement urges the people for displacement & to accept displacement as it comes in their way. It arises without much of warning. Natural calamities like earthaquke, volcanic eruption, flood, cyclone, and political as well as racial disturbance cause voluntary displacement.

Involuntary displacement is completely man made & there is a strong element of inducement. It is conceived with a definite purpose. All types of development projects which displace population, come under this category.

Development activity is continuous process and in India, it has gained momentum soon after launching of the first five year plan. Hirakud Dam Project is one of the largest water resources project constructed during the first five year plan. Being a development project it has naturally induced involuntary displacement.

It must be recognised that no major project for the flood control, conservation and utilisation of water can be constructed without submerging some valuable property and considerable areas of culturable and other land. Further the schemes of development in the greater interest of the community can not be held up or abandoned in the interest of a group of individuals. Such schemes undoubtedly involve some hardship to the people whose land & property are submerged. In any case the interest of the community, the state & the country must take priority over the sentiments, conveniences and interests of a few. "But the problems faced by the displaced persons (D.Ps) must be solved effectively with an aim to reduce their risks of impoverishment. The goal of development should not be to develop things but to develop man." The sacrifice of the D.Ps especially the psychological trauma of displacement can not be measured in any scale. It is unfortunate that in most cases, the benefits of the project, which displaces the people, are rarely available to them. If development is required, it is equally necessary to effectively resettle and rehabilitate the D.Ps.

3.4.2 Causes of Impoverishment

As per Mr. Michael Cernea, a noted sociologist, the pobable risks of impoverishment (vide 'Development Projects and Induced Displacement' by A.K. Dalua pg. 27-28) are :

a) Landlessness : In most cases, it is seen that the displaced persons are in possession of less extent of land than what they had before displacement. Land becoming gradually scarce for allotment, many also turn to be landless.

b) Homelessness : the displaced people are at libery either to stay in the resettlement colony set up by the project authorities, or to make their own arrangements. With the house building assistance money received by them, the accommodation normally does not become adequate. In several cases, the displaced persons fritter away the money given to them and are unable to build appropriate house for shelter.

c) Joblessness : Jobs in the project are not either sufficient to meet the demand or the displaced persons are not adequately equipped to serve. The landless labourers, daily wage earners,

artisans etc. face tough competition from the host community, to earn their living. Same is also the fate of petty traders.

d) Marginalisation : Land for agriculture being unsuitable, cultivation and employment opportunities being meagre, the displaced persons depend initially on the rehabilitation cash assistance, without planning for the future. This leads to marginalisation.

e) Food insecurity : In absence of firm planning for the future and in absence of ready to depend upon vocations, the displaced persons feel diffident about assured food supply.

f) Increased morbidity : Loss of generations' old hearth and haven,separation from the near and dear ones, coupled with the problem of assured food supply, make the displaced persons mentally and psychologically sick. Inadequate health care facilities also make them inconvenient. All these, tell upon their longevity.

g) Loss of access to common poperty resources : The people before displacement used to enjoy the common property resources like forest, grazing land, water bodies without any competition. But at the new place of settlement after displacement, they have to depend on the mercy of the host community.

h) Social disarticulation : Involuntary displacement affects the social system very much. Joint family system which is the pride of Indian culture, is disrupted. They vie for exchange of kinship obligations which may not be within their affordable reach as the people after displacement move here and there as per their choice.

In order to solve the above problems of D.Ps satisfactorily, the objectives given below need be kept in mind.

3.4.3 Objectives of R & R :

The main objectives of the R&R should be the following:

(i) to give appropriate compensation for the assets lost by the affected persons.

(ii) to shift the D. Ps alongwith their belongings to their desired destination.

(iii) to relocate the displaced persons enblock as far as possible so as to preserve their socio-cultural unity.

(iv) to provide all basic minimum civic facilities in the resettlement colony as well as in the clusters set up either by the project or by the D.Ps themselves.

(v) to provide suitable infrastructures be it agricultural, or non-agricultural for the economic rehabilitation of the oustees, with particular attetion to the S.C., S.T and other weaker section of the people.

(vi) to maintain the details of the D.Ps who go on their own and resettle themselves, enquire about their well being and help them as far as practicable.

(vii) to monitor the progress of R & R activites till completion and make periodic evaluation thereafter for at least five years;

(viii) to work dispassionately so as to improve the standard of living of the resettlers; by way of restoration of their production capability;

(x) to ensure that the displaced people enjoy the benefits of the project which displaced them as far as practicable.

But experiences in the field give different picture. Hirakud is the first major multipurpose dam project executed after independence. But even after lapse of more than fifty years, hundreds of displaced families have not received their compensation money; and the only mode of R & R then in force was payment of compensation money. Several of the D.P. families are no longer

surviving and there is no information about their legal heirs. Most of these D.P. familis had complained about inadequate rate of compensation.

A study reveals that many resettlers in Orissa, deserted the project sponsored colonies as suitable and adequate economic rehabilitation facilities were not available in and around the said colonies. Land for agriculture allotted are not suitable for cultivation especially in absence of irrigation facilities.

3.4.4 Means of Economic Rehabilitation :

Keeping all these consderations in view, the planners and policy makers piously wish that the oustees should be rehabilitated economically. According to them the most viable means of economic rehabilitations are :

(i) Providing agricultural land

(ii) Forestry

(iii) setting up cottage industries

(iv) Dairy

(v) Pisciculture

(vi) Petty business under self-employment schemes

(vii) Creating employment opportunities by imparting training to become mason, carpernter, electrician, fitter, mechanic, driver etc, and

(viii) Miscellaneous activities like awarding petty contract works during project construction period to make them socially and economically stable.

3.4.5 Hirakud Land and Development Organisation :

"The Land and development organisation will be under the control of the Government of Orissa and will deal with the following matters concerning the Hirakud Dam Project.

(a) Land - Acquisition

(b) Reclamation and Resettlement, and

(c) Development.

This should be a separate Oganisation under the Orissa Governmnet. The portfolio will be under the charge of the Hon'ble premier.

The Commissioner, Northern Division, will be in charge of land acquisition, land reclamation, and resettlement while the Development Commissioner who is also the Secretary to the Government of Orissa incharge of the separate Department created to deal with the project will be in charge of Development works such as establishment of industries, etc. connected with the project.

The Development Commissioner will establish close liaision between the CWI&NC and the construction organisation on the one hand and the Government of Orissa on the other.

The entire expenditure in connection with land acquisition for the project, as provided in the Project Estimate, will be under the control of the Government of Orissa. All sanctions will be accorded by the Secretary to government of Orissa in accordance with the usual procedure after consultation with the Finance Ministry of the Government of Orissa wherever necessary. In respect of expenditure chargeable to the project esimate, he will also show the sanctions for preliminary verification and certification to the FA & CAO Hirakud dam project appointed by the Government of India, Ministry of Finance.The latter will also be the Disbursing and Accounts Officer for this organisation for all expenditure debitable to the project as well as for the construction Organisation. Any expenditure on reclamation and resettlement operations for enabling the Government to make

compensition in kind in lieu of the whole or part of the compensation otherwise payable in cash will also be sanctioned and accounted for in the same manner.

The Chief Engineer and his staff will afford the fullest cooperation and all technical advice and all reasonable assistance required by the Commissioner, Northern Division and the Development Commissioner in connection with land acqisition, resettlement operations and Development work directly connected with the project. Land plans and specifications will be prepared by Chief Engineer with the help of the Land Acquisition staff working under the Commissioner, Northern Division. The Chief Engineer will take over possession of the land and maintain a record of it. In the Land Acquisition Officer's Court, the Chief Engineer will depute his representative to verify the area etc. of land being acquired but all other interests of the project and the Government will be represented by the Commissioner, Northern Division, or one of his officers.

In regard to development schemes connected with the project, although the CWINC will be vitally concerned with policy and programme, it will not have any administrative control over the implementation of such policy or programme. The working out and implementation of the development schemes will be the responsibility of Orissa Government, the CWINC being kept fully informed. All expenditure in connection with the development schemes will be under the control of the Government of Orissa and such expenditure if it is not a charge on the project, will not be under the purview of the Financial Advisor and Chief Accounts Officer, Hirakud Dam Project, under the present arrangements."

3.4.6 Resettlement & Rehabilitation Problems of Hirakud Dam.

Before discussing R & R problems of Hirakud, similar displacement faced by major and medium dam projects of the state may be seen which are furnished in Table 3.7 (vide Development Projects and induced displacement by A.K.Dalua pg. 20 &21).

3.4.6.1 Area submerged

The area submerged under Hirakud reservoir with full reservoir level at 630' is 74300 ha, lying both in Orissa & Chhatisgarh. The area of submersion of cultivated land is nearly 1,23,303 acres (49920 ha) lying in Orissa and Forest Land is 59250 Ac (23988 ha.).

The reservoir extends from the dam site 50 river miles (80 km) upstream and has a shore line of about 400 miles (640 km) affecting Orissa & Chhatisgarh.

The following are the gist of details on villages submerged & families affected.

i) Total no, of villages submerged (both in Orissa & Chhatisgarh) = 285

ii) (a) No of Villages in Orissa (below RL.630') = 249

(b) No. of Villages in Chhatisgarh = 36

iii) No. of displaced families (Orissa) = 26501

iv) No of Tribal families = 1636

v) % of Tribal displaced families = 7.39

Table - 3.7- Displacement of families by Major & Medium Dam Projects in Orissa

A. Major Dams

S.No.	Project	River	Year of completion	No.of displaced families
01.	Machkund	Machkund	1959	2,938
02.	Hirakud	Mahanadi	1957	22,144
03.	Balimela	Sileru	1977	2,500
04.	Salandi	Salandi	1983	965
05.	Rengali Dam	Brahmani	1992-93	10,897
06.	Upper Kolab	Kolab	1991-92	3,171
07.	Upper Indravati (4 dams)	Indravati & its tributaries	1996-97	5,599
08.	Rengali Irrigation (Samal barage)	Brahmani	Completed	828
09.	Kanupur	Baitarani	ongoing	3617
10.	Subarnarekha	Haldia, Jambhira Baura & Ichha	ongoing	5398
11.	Ong Dam	Ong	ongoing	3917
12.	Lower Indra	Indra	ongoing	6181
13.	Lower Suktel	Suktel	ongoing	4160

B. Medium Project :

01.	Budhabudhiani	Duanto	1967	N.A.
02.	Saipala	Saipala	1974	9
03.	Dhanei	Dhanei	1975	N.A.
04	Pitamahal	Pitamahal	1977	68
05.	Derjang	Derjang	1978	356
06.	Ghodahad	Ghodahad	1978	68
07.	Baghua Stage I	Baghua	1978	510
08.	Khadkei	Khadkei	1979	118
09.	Salia	Salia	1980	70
10.	Nessa	Nessa	1980	78
11.	Kalo	Kalo	1981	214
12.	Gohira	Gohira	1982	143
13.	Pilasalki	Pilasalki	1986	185
14.	Sundar	Sundar	1987	Nil
15.	Sarafgarh	Sarafgarh	1987	29
16.	Talsara	Talsara	1988	85

Contd....

17.	Jharbandh	Kukrijhar	1988	126
18.	Dadraghati	Dadraghati	1988	429
19.	Daha	Daha	1988	20
20.	Kuanria	Kuanria	1988	181
21.	Ramial	Ramial	1988	414
22.	Remal	Remal	1988	4
23.	Dumerbahal	Dumerbahal	1990	253
24.	Bhaskel	Bhaskel	1990	363
25.	Kanjhari	Kanjhari	1991	197
26.	Sunei	Sunei	1990-91	353
27.	Kansbahal	Barjorinalla	1991	196
28.	Bankabahal	Bankabahal	1993	282
29.	Dhauragoth	Dhauragoth	N.A.	N.A.
30.	Satiguda	Satiguda	1995-96	162
31.	Upper Suktel	Suktel	1995-96	Nil
32.	Harbhangi	Harbhangi	N.A.	226
33.	Badnalla	Badnalla	N.A.	219
34.	Hariharjore	Hariharjore	1997.	820
35.	Upper Jonk	Jonk	N.A.	481
36.	Baghua Stage II	Baghua	N.A.	1012
37.	Deo	Deo	Ongoing	964
38.	Titilagarh	Kankarajore	Ongoing	255
39.	Manjore	Manjore	Ongoing	104
40.	Sapua	Badjore-Sapua	Ongoing	109
41.	Baghlati	Bahuda	Ongoing	123
42.	Telengiri	Telengiri	Ongoing	596
43.	Rukura	Rukura	Ongoing	132
44.	Chheligarh	Chheligarh	Ongoing	371
45.	Ret	Ret	Ongoing	583
			Total	**82,841**

For ongoing projects, the figures have been projected.

Thus excepting a couple of projects, all projects have displaced or will displace population in small or large number. Even a barrage project like Rengali irrigation, because of its location and technical necessities has displaced people. Normally the magnitude of displacement is dependent upon the size of the dam, topography and storage capacity of the reservoir.

3.4.6.2 Villages submerged & families affected :

The topography of the country on each side of the reservoir is completely different from the other. Almost all the area on the right is hilly & covered with reserve forests except for a small portion in the extreme west. The terrain being uneven, undulating and less suitable for habitation, only 43 small villages out of the total 249 villages in Orissa fall on the right. On the other hand,

the country on the left is relatively flatter and contains long stretches of agricultural fields. Communication in this area was poor. There were only a few fair weather village roads & no railway lines, no highways, no bridges nor any major structures. Most of the houses of the villages were of cheap construction built by peasants themselves. The compensations, therefore, was less compared to other large projects elsewhere.

List of villages fully acquired, partly acquired and acquired above RL 632 ft are furnished respectively in Table 3.8, 3.9(a), (b) & 3.10.

Table - 3.8 - List of Completely Acquired Villages

Name of P.S.	Sl No.	Name of the village	Name of P.S.	Sl. No.	Name of the village	Name of P.S.	Sl. No.	Name of the village
	01.	Tarda		01.	Baghra		01.	Padampur
	02.	Dasar		02.	Mahada			(Mahadevpali)
	03.	Kulsar		03.	Meripada		02.	Soda
	04.	Marang		04.	Khurigaon		03.	Pandiapali
	05.	Laxmanpur		05.	Kalmi		04.	Kotgarh
	06.	Gourpali		06.	Chenachal		05.	Jogni
	07.	Kashipali		07.	Pandapali		06.	Uchpinda
	08.	Chhadarama		08.	Urda		07.	Kadamdihi
	09.	Balanda		09.	Dewanmunda		08.	Barhampura
Rampella	10.	Budla	Mura	10.	Sahasbanga	Mahadevpali	09.	Andhari
	11.	Bhatli		11.	Ankoli		10.	Panchpudugia
	12.	Barangamal		12.	Dungipali			(Palsada)
	13.	Nimuhipali		13.	Palsapali		11.	Chichali
	14.	Jogni		14.	Patrapali		12.	Nuapara
	15.	Binka		15.	Rampaluga		13.	Thakurpali
	16.	Burla		16.	Kenapali		14.	Tilgi
	17.	Darlipali		17.	Dumerpali		15.	Panchpudugia
	18.	Petupali		18.	Gandadihi			(Soda)
	19.	Taldihi		19.	Jalhari		16.	Kusmal
	20.	Antardoi		20.	Mura		17.	Gudum
	21.	Mangalpur		21.	Malda		18.	Ghunghutpali
	22.	Tahud		22.	Haldi		19.	Mohanpara
	23.	Sartang		23.	Saralpali			
				24.	Dhamadihi			
				25.	Kharuidhi			
				26.	Deoli			
				27.	Lahhipali			

Contd...

Table - 3.8 (Contd...)

Name of P.S.	Sl No.	Name of the village	Name of P.S.	Sl. No.	Name of the village	
	01.	Nuniamhulpali		01.	Saplahar	
	02.	Resam		02.	Badtang	
	03.	Barakhandia		03.	Sartang	
	04.	Sahajbahal		04.	Amghera	
	05.	Chakarmal		05.	Gandpali	
	06.	Dumerpali	Sadar	06.	Bhulka	
	07.	Budpali		07.	Solpali	
	08.	Mundomahul		08.	Mundapali	
	09.	Pundrimal		09.	Junani	
Attabira	10.	Ramedega		10.	Hirakud	
	11.	Dhurrakusum		11.	Padharipali	
	12.	Batimunda		12.	Amdi	
	01.	Talab		13.	Parta	
	02.	Kurumkela		14.	Chilantikud	
	03.	Bhatli		15.	Solbandh	
	04.	Khairpali				
	05.	Amlipali		01.	Jogipali	
Ambhabhona	06.	Rengali		02.	Kurla	
	07.	Govindpur		03.	Kudopali	
	08.	Tamdei	Katarbaga	04.	Thuntikatarbaga	
	09.	Jarimuli		05.	Sapne	
	10.	Launsara	Total	**116**	**Villages** in	
	11.	Natsari			9 police stations	
	12.	Amguda				
Sason	01.	Tihura				
Jharsuguda	01.	Bhursund				
	02.	Balpur				

Table 3.9 (a) - No. of Villages where village Basti site submerged but cultivated land and house partly remained outside.

Name of P.S.	Sl No.	Name of the village	Name of P.S.	Sl. No.	Name of the village
	01.	Adhapara	Sason	01.	Kilasama
	02.	Remda		02.	Gourpali
	03.	Ramella		01.	Patrapali
Rampella	04.	Rampella	Jharsuguda	02.	Rampur
	05.	Batlaga		03.	Dumermunda
	06.	Bhejikud		04.	Dudelsingha
	07.	Bargar		05.	Kirarama
	01.	Kutripali		06.	Singhabaga
	02.	Benkmura		07.	Telenpali
	03.	Pandri		08.	Katabaga
	04.	Bhanrachaka		01.	Larbanga
Mura	05.	Kudabaga	Sadar	02.	Jamadarpali
	06.	Sunari		03.	Jamda
	07.	Barihapali		01.	Lapanga
	08.	Sagarpali		02.	Dantamura
	09.	Pursian		03.	Babuchakuli
	10.	Thebra		04.	Pitapali
	11.	Baghmunda	Katarbaga	05.	Khinda
	01.	Jampali		06.	Salad
	02.	Chuipali		07.	Bainsimal
Mahadevpali	03.	Chikhili		08.	Bausen
	04.	Nandapali		09.	Pudapara
	01.	Badmal	Total	48 villages in	
Ambhabona	02.	Banjhipali		8 police stations	
	03.	Kusmuda			
	04.	Chikhili			

Table - 3.9 (b) - List of Villages where only Lands have been affected.

Name of P.S.	Sl No.	Name of the village	Name of P.S.	Sl. No.	Name of the village	Name of P.S.	Sl. No.	Name of the village
	01.	Piplikani		01.	Bhikampali		01.	Ramkhol
	02.	Tilia		02.	Chikhiliapali		02.	Uttam
	03.	Singhaipali		03.	Bausenpali		03.	Lather
Rampella	04.	Bhulia		04.	Sukhasoda		04.	Bungapali
	05.	Banharpali		05.	Remta		05.	Dungri
	06.	Telenpali		06.	Semilia	Ambhabhona	06.	Sauntamal
	07.	Negipali		07.	Palsada		07.	Behera
	08.	Kumbharbandh		08.	Jharupada		08.	Antrai
	01.	Beheramal		09.	Nachenpali		09.	Bugbugi
	02.	Singharpur	Mahadevpali	10.	Muhasingh		10.	Budipali
Mura	03.	Samarbaga		11.	Bad-darha		11.	Kandeipali
	04.	Kuremal		12.	Luhabaga		12.	Ghugrapali
	05.	Telen		13.	Kanjiharan		13.	Baigapali
	06.	Jamgaon		14.	Badimal		14.	Bhainatora
	07.	Katarbaga		15.	Kanaktora		15.	Ganthaipali
Sason	01.	Balbaspur		16.	Kadamghat		16.	Barangakot
Katarbaga	01.	Pandloi		17.	Amapali		17.	Beniapali
	02.	Rengali		18.	Dhulunda		18.	Saraipali
	03.	Tarekella		19.	Pithinda		19.	Ruchida
	04.	Talabira		20.	Kandelkela		01.	Bundia
	05.	Nisanbhanga		21.	Charpali		02.	Gudhiali
	06.	Kumbhari		22.	Sarda	Jharsuguda	03.	Jurabaga
	07.	Gurupali		23.	Mahudi		04.	Darlipali
	08.	Bamaloi					05.	Katapali
	09.	Tumbekela					06.	Ainlapali
	10.	Thelkoloi					07.	Remja
							08.	Gandghora
							09.	Jamuapali
							10.	Malda
							11.	Khait
							12.	Hirma
							13.	Bhagipali
							14.	Sripura
							15.	Kherula

Total 83 villages in 7 police stations.

Table - 3.10 - List of villages in which acquisition has been made between RL 630' to 632 ft.

Name of P.S.	Sl No.	Name of the village	Name of P.S.	Sl. No.	Name of the village	Name of PS..	Sl. No.	Name of the village
	01.	Sartang		01.	Urda		01.	Gobindpur
	02.	Baliaga		02.	Sahasbaga		02.	Tamdei
	03.	Bhejkud		03.	Patrapali		03.	Jarimuli
	04.	Rampella		04.	Dumerpali	Ambhabhona	04.	Launsara
	05.	Ramella	Mura	05.	Malda		05.	Helsari
	06.	Balanda		06.	Lachhipali		06.	Arjunda
	07.	Bamgamal		07.	Pandri		07.	Banjipali
Rampella	08.	Bhatali		01.	Ghunghatpali		01.	Bhursund
	09.	Adhapara		02.	Kusmal	Jharsuguda	02.	Balpur
	10.	Binka	Mahadevpali	03.	Gudum		03.	Rampur
	11.	Bargarh		04.	Titigi		01.	Salad
	12.	Desar	Attabira	01.	Dudelkusum		02.	Khinda
	13.	Kulsar				Katarbaga	03.	Baunsen
	14.	Marang					04.	Pudapara
	15.	Budula					05.	Bansimal

Total 42 Villages in 7 Police Stations

Table No.3.11 : Abstacts of Villages acquired

Sl.No.	Name of the P.S.	No.of Completely acquired villages	No.of villages where Basti-sties submerged but cultivable land & some houses remained outside	No.of Villages where only land have been acquired	No.of Villages where land acquied between 630-632	Total	Remark
1	2	3	4	5	6	7	8
01.	Rampella	23	07	08	15	53	No. of Vilalges in Col. 3, 4 & 5 have been acquired upto RL 630 ft.
02.	Mura	27	11	07	07	52	
03.	Mahadevpali	19	04.	23	04	50	
04.	Attabira	12	0	0	01	13	
05.	Ambhabana	12	04	19	07	42	
06.	Jharsuguda	02	08	17	03	30	
07.	Katarbaga	05.	09	10	05	29	
08.	Sason	01	02	01	0	04	
09.	Sadar	15	03	0	0	18	
	Total	116	48	85	42	291	

A brief note on compensation by Mr. R.S. Swann, I.C.S. the then Deputy Commissioner, Sambalpur is appended vide Annexure 3.5.

Annexure 3.5.

"Almost useful note on the subject of compensation has been furnished by Mr. R.S. Swann, I.C.S., Deputy Commissioner, Sambalpur. This note deals with the classification of lands, the rights of Gountias and the basis for and the difficulties in the way of the assessment of compensation for land, property and other rights. Compensation based on the record of rentals, mutations and transfer of leases would appear to be inadequate for the purpose. The present market values must be given due consideration. Compensation should be equitable and even generous. Land should be given for land as far as possible and the dispossessed inhabitants settling on new lands helped and encouraged to construct and live in model villages with provision for incorporating modern amentities in due course. Mr. Swann has roughly worked out a figure of Rs.5,60,62,635 for compensation."

The summary of rough cost of acquisition including compensation is furnished below.

Table No. 3.12 : Summary of Rough Cost of Acquisition **(Rs.)**

Cultivation lands to Raiyats	....	2,31,39,319
Pasturage	....	5,00,000
House	....	1,35,40,500
Temples and mosques	....	2,25,000
Wells	....	1,11,000
Schools	....	67,000
Tanks and Irrigations Tanks	....	9,00,000
Fruit bearing trees and timber trees	....	5,00,000
Compensation to gountias, Mafidars and Zamindars including the market value of Sir. Bhogra and 15%		1,17,89,166
Government	....	50,90,650
Cost of Land Acquisition	.....	2,00,000
	Total	**5,60,62,635.00**

Areas available for Development

It is understood that about 50 per cent of the displaced people will have to be provided with land. From a preliminary investigation of the areas available for resettlemeent, it has been possible to locate about 106,400 acres for the purpose, as shown below:

Barra Jungle	...	62,800 Ac.
Village Forest	...	12,300
Govt, Reserve Forest	...	17,000
Zamindari Forest	...	7,500
Gochar lands	...	6,800
	Total	**106,400 Acs.**

"In addition to the forest lands, there will be 20,000 to 40,000 acres of marginal lands which will reappear each year with the depletion of the reservoir, and, with the newly laid rich silt deposits and moisture retained from the reservoir will be available for raising one good crop from March to September. It is possible that the cultivators whose villages are only partly submerged might prefer to make use of these marginal lands instead of moving to new areas.

A third possibility is for the Government to acquire aeas in different Zamindaris which are lying waste at pesent and which can be brought under cultivation on the introduction of irrigation."

Policy of Acquisition and Resettlemnt

"It is hoped that more than the required area of land will be available from Government and zamindari forests, the marginal lands, and acquired waste lands. Government should, however, consider the question of acquiring proportionate areas of land from those who will be benefited from the project by irrigation and otherwise and in this way minimise the hardship of land acquisition by spreading it over a wider section of the community. According to this, each land owner holding lands above a certain economic minimum may be required to give to Government on payment of compensation a part of his land, which will bear the same proportion to his total holding as the area required for the reservoir, canals and connected works on the project will bear to the total area to be benefited by the project. Such areas will necessarily be in fragments but they can be consolidated into compact blocks by readjustment between the various holders and then used for the resettlement of the dispossessed people."

Mr R.S. Swann recommended the pattern of land compensation and house construction to be followed in case of D.Ps of Hirakud Dam Project.

Model Villages

"New village sites should be located after careful consideration of the various factors that go to make community life happy and full. Government should undertake to build at public expense places of worship, schools, community centres, roads, drains, etc., and otherwise plan the villages on modern lines and reserve areas for play grounds, agricultural farms and industrial development. The area reserved for building sites should be cut up into plots. Assuming that a particular village is to accommodate 500 families, sufficient space should be reserved after allotment of the necessary number of plots to these 500 families, to accommodate a further lot of 1,000 to 1,500 families. This remaining plots so reserved can subsequently be sold at a much higher price when land values go up as a result of the general prosperity brought about by the project. In this way large revenues will accrue to the government which will more than repay the initial expenditure incured by the Government in the layout of a modern village and the construction of public places. No credit has, however, been taken in the project for this additional income. The new settlers should be encouraged by loans and grants-in-aid to build better homes than the ones they leave behind to be submerged in the reservoir area. In addition Government should provide free technical advice in the planning and construction of these homes."

3.4.6.3 Organisation for Resettlement

The Government of Orissa started a separate department called the "Hirakud Land Organisation" (HLO) with effect from 1978 to look after land acquisition, reclamation and resettlement under the administrative control of the Member, Revenue Board of Orissa, who set up his headquarters at Sambalpur.

The land acquisition department of the organisation was entrusted with the evaluation of compensation payable to the residents of the submersible area for their land, trees, houses and other properties, and for payment of compensation money to the villagers and taking possession of their lands and other immovable properties in the submerged area. H.L.O. is responsible for resettlement of D.Ps. It is responsibility of this organisation to acquire the properties submerged, to pay compensation, to reclaim sufficient lands for resettlement of the submerged-area people and to implement schemes to resettle them.

The Hirakud Land Organisation (H.L.O.) established in 1978 has since been renamed with head quarters at Sambalpur after the resettlement works of Hirakud reduced substantially.

3.4.6.4 Land Acquisition and Satisfactory Compensation

"The prices of land were fixed by the Land Acquisition Officer after taking into consideration factors like present local value, price of the crops, produce, rental terms, crop yield, etc. Such prices were paid for the lands and other properties as would enable the owners to re-establish themselves in a manner which would give them satisfaction equal to that, if not more, than they so long enjoyed for generations. The land owners were given a specified time to lodge complaints before the Acquisition Officer if they were not satisfied with the amount of compensation and every effort was made to settle the disputes by negotiation. Land owners were permitted to retain possession of their reservoir properties till such time as flooding occured and were allowed to enjoy the value of the growing crops to the last day."

This not only benefited the land owners but also prevented food shortage in this almost inaccessible area. Even though the Government took de jure posession of all the immovable properties as soon as compensation was paid, the owners were allowed to make any use they liked of their possession and in fact most of them did actually salvage large quantities of useful materials from here and used them on the construction of their new houses wherever they settled afterwards.

Provision for Homestead land for the Oustees

Govt. in revenue department vide letter No.5531/R dt.30.01.2002 have decided to settle Ac.0.10 of homestead land at the maximum with each Project Displaced Families of Hirakud Dam Project, irrespective of their annual income & other criteria stipulated in OGLS Act/Rules. Subsequently Govt. in Revenue department in letter No.41550/R dt. 30.10.2004 have clarified that each split up family headed by the major legal heirs of the original project displaced family is to be treated as separate family & eligible to get the benefit of Ac.0.10 of homestead land as prescibed in Govt.in Revenue department letter No.5531/R dt.30.01.2002.

In view of the above Govt. instruction, the Tahasildar, Sambalpur & Rengali have been requested to invite applications from the oustees of Hirakud Dam Project & allot them homestead land. As reported by Tahasildar, Rengali so far 367 applications have been received, out of which lease of house-sites has ben sanctioned to 30 oustes till date involving Ac.1.49 of land. No such application has been received in Sambalpur Tahasil.

Reclamation

"Another department of H.L.O. deals with reclamation. Reclamation goes on side by side with acquisition.

The farmers were the only residents of submerged villages and they had lived there for generations. They always desired to re-establish themselves in similar agricultual areas rather than take up some other profession. The centuries old agricultural economy of the area, based on production of paddy, had given rise to a large tenant population. The average cash income was very low, as might be expected in such case.

Construction of the dam thus provided a definite help and source of income to peasant families who were previously poor and partially employed. New camps were located as near as possible to the reservoir area and they all fell in the same district. Shifting into camps close by caused little disturbance to the resettlers in transferring their movable properties and they often supervised the construction of their houses in the resettlement camps while still living in their old homes.

The major part of the area to be reclaimed consisted of dense forest, belonging to the State Forest Department. A special forest officer was deputed from the State to see that during reclamation

proper exploitation of forest products was made by arranging the sale of these products. After the forest was cleared reclamation work began.

The reclamation areas were divided into blocks each having an area of nearly 2,000 acres. Each block was in charge of a Camp Officer who was supplied with necessary machines and tractors for reclamation work which consisted of (i) removal of tree stumps, (ii) removal of roots of trees, (iii) levelling of land, and (iv) ridging the fields. Soon after the lands were reclaimed, they were leased to displaced people or to tenants from neighbouring villages for prompt cultivation, so as to prevent forest growth coming over again. In making the settlement of reclaimed land, the policy has been to give first preference to persons displaced from the reservoir area who were given enough inducement and encouragement for taking lease of the maximum acreage of cultivable land. Only when there has been any surplus it has been settled with people of the neighbouring villages."

Facilities and concessions

"The Reclamation Department was also responsible for providing irrigation facilities to the reclaimed areas which were all beyond the command of the canals of the Project. The irrigation to the camps is done from small reservoirs called katas formed in the vicinity of the camp by making an embankment across a suitable local drainage stream. Though the average rainfall in the locality is about 60 inches, it is often irregular and ill-distributed. Storage tanks are therefore required to supplement irregular and deficient rainfall. Water supply in the camps is made from wells constructed in the colony.

As resettlement progressed, efforts were made in increasing degrees to add more and more to the convenience and comforts of the resettlers. Villagers were actively associated in laying out their own village, so that they might have due confidence in the management. All the building materials were supplied to them at cost price and all possible help has been rendered to enable them to reconstruct their new life in the new surroundings fairly quickly.

New avenues of employment as sailors and fishermen have also opened up before the villagers because of the Hirakud lake which will be stocked with fresh water fish such as Roohi, Katla and Mirgal, and the pospects of water transport arising from the control and regulation of the Mahanadi through the Hirakud dam.

The people have taken the change admirably well though nostalgic memories of the old villages remain. The first day in each new village was celebrated by the settlers by pujas and rejoicings, and they can confidently look forward to a future brighter than in their old villages. List of development works done in reclamation camps is given in Table-3.14.

In 1949 reclamation of forest area was takenup on a large scale for resettling the cultivators of the submersion area.

Resettlement

This H.L.O. deals with resettlements of villages in the nearest camps. The primary objectives of the resettlement are (i) to remove the families from reservoir area (ii) to take responsibility for an orderly evacuation within a stipulated period (iii) to cooperate with and advise the displaced farmilies during the time of evacuation and after so that they adjust themselves. In the reclaimed area, villages have ben regularly laid out and roads have been made connecting such villages with each other and all of them with the nearest main road. Wells and tanks for drinking water, schools for education of the children and Bhagbat Ghars for communal entertainment have been constructed in these villages. Grain golas have been constructed for supplying rural credit and market sheds have been constructed. In one group of camps, a hospital has been constructed and the health of the resettlers in the rest of the camps is looked after by a medical officer stationed at Sambalpur with the help of a mobile van.

House sites are given free in these reclaimed areas and timber is supplied at 60% of the normal royality for the construction of their houses. Manures and fertilisers are supplied free of cost and seed paddy is given on loan. An Agricultural Development Officer with staff constantly tours these areas and looks after the reclaimed lands and cultivation in general. Table 3.13 shows the details of only 18 rehabilitation camps and Table 3.14 gives the list of development works executed in the camps.

Periphery Settlement

"During this year's evacuation, the people living in the villages of the higher ridges preferred to shift to places above the maximum reservoir level (632 R.L) and settled there. So 62 villages were laid out and 3,150 house sites were allotted to these people during the year 1956. A programme of development works for these villages has been drawn up and will be implemented during the year, 1956-57. It is expected that about 20,000 acres of submerged land will come out of the reservoir by the months of January and February every year. It is proposed to lease out these lands to the people who have settled on the periphery. With this object in view survey operations are being taken up and all preliminary works for leasing out the periphery lands will be finalised in time to lease out this area to the people for the next cultivating season."

Cooperative farming :

"For the resettlement of landless labourers, steps are taken for forming two cooperative farming societies as pilot scheme. These schemes are under the active consideration of the Government and it is expected that they will be implemented during the next cultivating season.

The people of the submersion area who have neither settled in the reclamation centres nor on the periphery have by and large purchased lands in other old established villages and resettled by personal efforts. As far as possible, statistics of such resettlement have been maintained by this organization. Wherever such lands were available for sale, information was collected and published in the submersion area to enable the people to purchase lands. As royati land was made saleable by an amendment to the land laws in 1953, the people of the submersion area were enabled to purchase such lands and this helped resettlement substantially. It is on account of this that demand for reclaimed area was not as much as was expected."

Evacuation

"This organisation started evacuating the submersion area on the requisition of the Construction Organisation from 15th April, 1955. During the year, 33 villages fully and 13 villages party consisting of about 4.000 families were evacuated. Transport was supplied free of cost to the submerged area people for carrying all their movables excepting straw. The people were allowed to take-house building materials obtained by breaking the houses even though they were all paid for by the Government. Free transport was not however given for this purpose. During 1956, 72 villages fully and 22 partly consisting of 10.352 families have been evacuated in the same manner. The total amount of money spent for these evacutation operations by the Government of Orissa is about Rs 17 lakhs. Still.2.000 families are living in the higher reaches and evacuation of 36 villages fully or partly will be necessary during the year, 1957.

The people of the submerged area have given utmost cooperation during the very difficult period of evacuation and due to this constructive attitude of the people, evacuation could be done peacefully and in time."

Other schemes of resettlement

Besides reclaimed lands, 6,500 acres of canal irrigable and cultivable waste land have been offered to the submerged area people for resettlement. The response to this ressettlement from them is, however, poor.

Table 3.13 : Details showing 18 Rehabilitation Camps (*)

Sl. No.	Name of the Resettlement Camp	Name of the connected Rev. village	Families	Sukh-basis	Encroachers		Land settled with Settlers		Ponds	Katas	Wells	MIP	Schools	Club	Road	Remark
					No	Land (Ac.)	H.S	Agril.								
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
1.	Chhamunda	Nua-Adapada	206	9	0	0	32.69	686.01	4	5	4	1	2	-	Kachha	[1]The land Ac.213.37 stand recorded in the name of Co-operative Deptt. in MS-ROR on payment of premium. The encroachers are in possession since 1974-75. Land Ac.325.27 in vill. Deogaon stands recorded as rent free land in the name of HLO. [2]Out of 54 family settlement 42 families have left the camp since last 40 years & settled elsewhere. [3]One Veterinary Hospital [4] One P.H.C Although some lands were reclaimed for the resettlers no land oustees of HLO is staying in this camp.
2.	Basiapada 'A'	Birsinghgarh	50	5	87	235.0	6.86	318.68	1	6	1	1	1	-	Kachha	
3.	Basiapada 'B'	Nua-Mahulpali	63	1	1	0	4.43	240.27	2	4	2	1	1	-	Kachha	
4.	Badasahir	Nua-Barangmal	329	0	1	0	4.43	240.37	1	4	2	1	1	-	Kachha	
5.	Lamdunguri	Lamdunguri	84	27	1	1.82	8.15	366.33	3	9	3	0	1	4	Kachha	
6.	Hotapali	Larasara/ Deogaon/Hotapali	0	0	19	59.80	4.08	213.37[1]	1	3	3	1	1	-	Kachha	
7.	Nua-Hirakud	Nua-Hirakud	40	27	0	0	2.69	132.74	1	2	2	0	0	2	Kachha	
8.	Gaudpali-A	Gaudpali	54/42[2]	0	0	38.49	0.70	0	1	1	5	0	1	-	Kachha	
9.	Goudpali-B	Nua-Bhoghara	36/27	0	0	4.50	1.39	77.53	-	1	4	0	1	-	Kachha	
10.	San-Kundesara	Nua-Burda	153/140	0	0	43.39	29.38	433.95	2	1	4	1	1	-	Kachha	
11.	Jharghati	Nua-Rampella	254	0	0	0	+	700.71	12	5	1	2	4	5	Kachha	
12.	Attabira Colony[3]	Nua-Batimunda	62	0	5	8.25	4.52	217.75	2	-	2	-	1	1	Pucca	
13.	Sangaramal 'A'	Nua-Mura	360	0	0	0	+	1334.56	2	0	1	2	4	1	Kachha	
14.	Sangamal 'B'[4]	Tihura	346	0	0	0	+	1320.98	2	0	1	4	2	-	Kachha	
15.	Jharanapali	Nua-Gilgi	30	0	0	0	+	122.47	1	0	3	2	1	-	Kachha	
16.	Sardhapali	Lapanga	8	0	0	0	+	133.74	1	0	1	0	1	0	Kachha	
17.	Laumal	Nua-Khurigaon	142	-	-	-	-	133.74	1	0	1	0	1	0	Kachha	
18.	Udsing	0	0	0	0	0	0	0	0	0	0	0	0	-		
	Total		2300/209	69	114	396.25	128.74	7870.62	37	46	46	19	27	13	16 Kachha 1 Pucca	

Sources: Proceeding of RDC Review meeting held on 16.08.1991
(*) Note : This statement is not exhaustive.

Table 3.14 : List of Development works done in the Reclamation Camps.

Category of structure	Nos.	Category of structure	Nos.
1. Irrigation works		4. Hospital and Staff Qrs.	
i) Katas (Big-Water Reservior)	49	i) Hospital	1
ii) Munda (Small water reservior)	28	ii) Quarter for Doctor	1
2. Water Supply Projects		Quarter for Compounder	1
i) Tanks	19	5. Other Buildings	
ii) Wells.	24	i) Community Centre	1
iii) Water Tanks	3	ii) Bhagabat ghar	15
3. School Buildings		iii) Debtagudi	7
i) M.E. Schools	2	iv) Deraghar	3
ii)M.V. School for Girls	1	v) Mahima temple	1
iii) Lower Primary Schools	9	vi) Market Shed	2
iv) Temporary quarters		vii) Grain Gola	1
for M.E. School Teachers	6	viii) Roads	25 miles
v) Temporary quarters			
for mistresses of M.V.			
Schools for Girls	6		

(Source Indian Journal of Power & River Valley Development- Hirakud Project No. 10 pg. 59-60)

Press Release by M.O.W.R., Government of India.

Question of nonpayment of compensation to all the outstees of Hirakud was raised in the Parliament. Minister, Water Resources, Government of India released a Press Note which is reproduced below :



Sundar Rajan Bridge on Power Channel at Burla

PRESS RELEASES

Monday May 22,2006

Ministry of Water Resources, Government of India.

Rehabilitation Of Oustees Of Hirakud Dam

Lok Sabha

The State Government of Orissa has reported that out of 26501 displaced persons (DPs) in Hirakud Dam, 1965 DPs have been resettled in 16 State Government sponsored resettlement camps and the remaining 24536 DPs have been resettled by themselves.

As per the report of the State Government, the Deputy Commissioner had distributed patta to 1074 DPs in 1956-1958. Those pattas are being converted to regular Revenue Patta (Records of Right) and till date 954 cases involving an area of 2,247.33 acre have been regularized. The remaining cases are scheduled for regularization by the end of May, 2006. Payment of compensation to Jhankars/Chowkidars in 101 cases for an amount of Rs.2,43,284/- was pending. An amount of Rs.12,424 in 9 cases have been paid till date after verification of all records. The remaining 92 cases of Sambalpur District are awaiting settlement.

The State Government of Orissa constituted a High Power Committee in 1992 which recommended for payment of Ex-gratia at the rate of Rs.10,000/- to the remaining DPs. The State Government approved the recommendation and sanctioned Rs.1.00 crore for payment of Ex-gratia. Out of this exgratia Rs.83,78,857 has been paid till date. A special cell has been constituted in the Sambalpur Collectorate to carry out the decision of the State Government and the matter is regularly reviewed by the State Government to take necessary steps.

This information was given by Prof. Saifuddin Soz, the Union Minister for Water Resources in reply to a question by Shri Jual Oram in the Lok Sabha today.

3.4.7 Conclusion

Construction of this longest earthen dam was completed in 1957. While moderating flood in the Mahanadi delta it irrigates 1.5 lakh hectares C.C.A. in the undivided districts of Sambalpur and Bolangir and supplements irrigation in the deltaic plain. Besides it has an installed capacity of 347 MW for power generation. The vast reservoir submerged about 74000 hectares of land of different category and 249 villages inside Orissa comprising 26501 families and 36 villages in Chhatisgarh area.

There was persistent resistance to this project since 1945 when it was decided to construct the project. Anti-Hirakud dam agitation was launched by the people since vast tract of fertile agricultural land was to come under submergence. Leaders like Bharat Nayak, a Retired Deputy Collector and Bodhram Dubey, noted political leader took leading part. As per programme, compensation money for the lost immovable property was paid at the then prevalent rate and the oustees were offered reclaimed land at a nominal price of Rs.213/- per acre for agriculture. Besides arrangements were made to resettle them in colonies having drinking water wells, tanks, schools and community centres etc. But the agitation took a regional shape and the oustees were not prepared to suffer for the benefits that would accrue at their cost, to the flood affected people of coastal area. Separatist attitude grew for separation of Western Orissa from the eastern Orissa. Agitation was handled firmly by clamping 144 Cr. P.C. Further the time was not favourable to the agitators at that point of time when India was at the threshold of gaining independence. The Indian National Congress did not like that the anti-dam agitation should be encouraged leading to separation of Sambalpur area from Orissa. Thus for want of political support the resistance movement fizzled away. The situation perhaps would have been different in present days. However a meagre about 11% of the oustees moved to the Govt. sponsored resettlement colonies and the remaining shifted to places here and

there on their own. As many as 8744 persons did not receive their due compensation money, in protest. After a long gap the issue was reopened in 1992 when it was found that 3540 cases could be successfully settled and for which financial arrangements at a much higher rate (about 32.5 times) were made for payment to them. The matter is still unresolved.(Vide Development Projects and Induced Displacement by A.K. Dalua pg. 116-117). The details of 3540 cases are as under:

Sl. No.	Name of district	No. of unpaid cases	Amount of additional compensation payable (Rs.)
1.	Sambalpur	2228	2,11,97,770.00
2.	Baragarh	1087	49,66,870.00
3.	Subarnapur	224	567,500.00
4.	Bolangir	1	600.00
	Total	**3540**	**2,67,32,740.00**

Above exgratia amount has been approved by the cabinet on 14.12.1993 for payment against which Rs.2.00 cr. has been received. Payment of exgratia (upto 31.12.2008) is given in Table3.16.

It is pity that we have not learnt least from our history. Innumerable policies, guidelines. Acts and Rules, hardly work as long as they remain as paper works only without being implemented in the field. There should be effective resettlement and economic rehabilitation of the displaced persons. But field survey reveals that, if there is resettlement of some type or other, there is no rehabilitation. To cite few examples, even after more than fifty years of completion of Hirakud dam, hundreds of compensation cases on land acquisition, are still pending for disposal. At such length of time, it becomes difficult to locate the legal heirs of the deceased compensation awardees.

The erstwhile HLO which was looking after all the issues relating to Hirakud Dam Project oustees under the supervision of R.D.C., Northern Division, Sambalpur was abolished in 1978.

There was no policy decision of the Govt. from 1978 to 1990 to give tenancy right to D.C. Patta Holders. But when Govt. Notification was made for setting up IB Thermal Power Station at Banharpali under Lakhanpur Tahasil, Govt. issued instruction vide GO. No.2386/. dt.22.0.1990 to grant tenancy right to those who were in actual physical possession of land on the basis of D.C. (Deputy Commissioner) Patta. Total 5004 D.C. Patta involving an area of Ac. 9776.92 were issued to the land loosers. Out of this settlement was made in 4518 cases involving an area of Ac.8521.17 leaving a balance of 486 cases covering Ac.1455.75. The highest pendency is in Jharsuguda district-319, followed by Sambalpur-120 and Bargarh-47. Due to nonavailability of old case records, the lands are not settled in their favour. In some cases, D.C. Patta land has been occupied/acquired by other persons/Government agencies.

The up-to-date achievement in regularising the D.C. Patta cases are as follows :

Table 3.15 : Regularisation of D.C. Patta Land in favour of Outstees

Tahasil	Total No. of D.C. Patta Holders distributted during year 1956 -58		D.C. Patta settled with the allottes during settlment/ consolidation		D.C. Patta settled with the allottes by the Tahasildars		Total settlement till 22.5. 2006		Cases dropped		No. of Cases pending for regularisation	
	No.	Area (Ac.)	No.	Area	No.	Area	No.	Area	No.	Area	No.	Area
Sambalpur	355	1230.09	288	1141.07	66	86.01	354	1227.08	1	1.72	0	0
Rengali	719	1470.84	68	140.36	532	879.89	600	1020.25	0	0	119	450.50
Total	1074	2700.93	356	1281.43	598	965.90	954	2247.33	1	1.72	119	450.50

Table 3.16 : Payment of exgratia to the land outstees (upto 31.12.2008)

Name of Districts	Name of Tahasil	Total amount (Rs.)	Total no. of bene-ficiaries	Amount disbursed		Balance amount for	
				No.of families	Amount (Rs.)	No. of families	Amount (Rs.)
1	2	3	4	5	6	7	8
Sambalpur	Sambalpur	47,00,000	623	274	13,09,905	349	20,06,156
	Rengali	-	447	227+12 p.p.	15,15,320	220	9,47,775
	Total	**47,00,000**	**1070**	**501+12 p.p.**	**28,25,225**	**569**	**29,53,931**
Jharsuguda	Jharsuguda	77,228	425	37	77,228	88	-
	Lakhanpur	41,20,864	685	120	41,20,864	565	-
	Total	**41,98,092**	**810**	**157**	**44,98,982**	**553**	
Bargarh	Attabira	10,71,620	343	238	2,35,969	105	8,35,701
	Bheden	9,71,712	463	206	1,51,573	257	8,20,139
	Bhatli	11,97,210	333	273	6,33,002	60	5,64,208
	Bargarh	121,767	281	281	1,21,767	-	-
	Barpali	29,441.00	110	110	29,440.90	-	0.10
	Sohela	8,200	88	84	7,628.50	04	571.50
	Total	**34,00,000**	**1618**	**1192**	**11,79,380.40**	**426**	**22,20,619.60**
Subarnapur	Binka	2,52,000	81	17	41,600	64	2,09,400
	Rampur	2,65,000	88	44	1,89,803	44	75,797
	Total	**5,17,600**	**169**	**61**	**2,31,403**	**108**	**2,85,197**
G. Total		**1,28,15,692**	**3667**	**1911**	**84,34,100.40**	**1756**	**54,59,747.60**

(N.B. Data collected from R.D.C., Sambalpur)

3.5 : River Diversion

For facilitating execution of spillway and power house blocks, a procedure is briefly described for closing the river Mahanadi by diverting the water in phases vide Hirakud dam specification report (1953), and Drg. No.100-H-15 attached to the report.

It was contemplated to confine the flow of two channels of the river at dam site to portions of its own course in the first stage. In left channel a temporary earthen coffer dam would be raised to enclose the first eleven blocks of the left concrete dam and these blocks would be built up in the first season to RL + 505' along with the abutment, retaining wall going up to RL + 545'. Construction in succeeding seasons will be confined to the non-monsoon period for which coffer dam will not be necessary.

In right channel a coffer-dam of earth fill with sheet piles or clay-cut-off would be constructed to enclose the power dam and right abutment blocks. This will be of such construction that it will stand without failure or overtopping for first 3 years so that construction of power dam and power plant can continue without interference.

It is expected that by the end of the 3rd year the whole length of concrete dam both in right & left will be raised to RL + 540 ft. After this water will be allowed to head up behind the dam and during monsoon, it will discharge partly through undersluices and partly over the overflow (spillway) blocks.

River Diversion as actually followed during construction, is described here in nutshell.

The river at the dam site has two channels. This helped in the diversion of water during the different phases of construction.

For working in left channel every year from 1951-52 to 1953-54 a coffer dam was built across the left channel till the construction of sluices were completed. Gap between left channel and middle channel was also closed. For commencing the work in right channel, a coffer dam was built in the right channel at one location in the year 1952-53 and 1953-54, and at a diferent locatlion in the years 1954-55 and 1955-56 till the dam was completed. In the year 1951-52 left channel was kept closed . Winter supply (lean flow) passed through middle channel and deepest stream in right channel. In 1952-53 and 53-54 both left and right channels were closed and the winter flow passed through middle channel. In 1954-55 right channel was closed & the winter flow passed through left spillway. In this year earth dam was built across the middle channel to a height of about 100' which was a herculean task in one season. In 1955-56, a coffer dam was built in right channel and water was allowed to pass through the spillway.

A coffer dam was built with earth and rockfill. Stream was closed with rock fill. As rockfill proceeded, earthwork followed. The gap closing was done with crates filled with rubble and tied with thick wire ropes. The discharge through the closing gap on the day of closure was of the order of 10000 cusec (283.0 cumec). Each cofferdam every year involved a quantity of 13 lakh cft (36790 cum) of earthwork and 1.5 lakh cft (4245 .0 cum) of rockfill. It used to take a month and then further time in strengthening it.

The Power dam and Power house portion was completely encircled with a protective bund from the year 1950-51 upto 1954-55 and kept above flood level to carry the work during monsoon too.

The economics and optimum heights of the coffer dams were worked out at CWPRS Pune by model studies, which were then implemented in the field for the first and 2nd stage construction.

(Source - Bhagirath - Vol III no.9. Feb. 1957)

3.6 Construction Methodology of Dam, Dykes and Power House :

3.6.1 Introduction :

At the site of the Main Dam, the river Mahanadi has three channels, thus forming two islands in between, which are called Hirakud and Koilarkud. The dam is a masonry, concrete earth fill barrier 15748 ft long and abuts the two hills viz. Chandli Dungri on the right side and Lam Dungri on the left side . The total length of concrete and masoury dam is 3,768 feet of which 1640 feet is across the left channel called the left concrete dam and the other 2128 feet long across the right channel called the right concrete dam. There are, besides, the dykes along the saddles on the left and right sides for a total length of 13 miles.

The total quantity of concrete and masonry involved in both left concrete and right concrete dams is 3.94 crores cft. The earth dam is of zoned filled type and is in two parts, the left section being 4,440 ft. long and the Island section between the two parts of the concete dam being 7540 ft. The total quantity of earthwork involved in the case of Main Dam is 43.24 crores cft. The maximum height of the concete section is 200 ft., and that of the earth dam 195 ft. The left concrete dam is all spillway with 40 undersluices 12'x20'-4" and 21 crest bays each 51' wide which will be controlled by 20' high Tainter type gates. The right concerete dam includes Spillway and Power sctions. The right spillway contains 24 under-sluices and 13 crest bays of the same dimensions. Both the spillways are in concrete and the transitions and flank walls in masonry. The Power dam is of masonry gravity type consisting of 7 blocks with a total length of 588 ft. In each of 7 blocks is provided a partially steel lined 25' and 23 ft. diameter penstock to supply water to Power Plant In the Power House, there will be initially four generating sets, two English units of capacity 37500 KW each and two German Units of capacity of 24000 KW each. Three are kept for further extension which have been subsequently installed. The installed and firm capacity of power generated will be 1,23,000 KW and 85,500 KW respectively.

3.6.2 Some Special Features

"Among the special features may be mentioned the substitution of granite rubble stone masonry in cement mortar in place of concrete in cement mortar in certain dam locations. Apart from the fact that the rubble stone masonry is nearly Rs.50/- cheaper per 100 cft than concrete, the stone masontry is a superior material for dams. Stone masonry is our strong point in the history of constructions of our irrigation dams and other works handed over to us from generation to generation for centuries. We are sure of our masonry answering the purpose. Concrete in cement mortar for dams in this country is a new thing yet to be tested for efficiency, durability, water tightness and other essential charateristics. It is true, the American practice is concerete for dams. But construction there, is by wholly mechanized operations in every detail of work because of the dearth of skilled labour. This necessarily compelled them to use a material for the dams that can be handled in every phase of construction by machines. The Americans have perfected this technique to a very great extent for efficiency and good purpose. While it is beyond doubt that concrete has some material advantages from the point of view of rapid execution and certain constructional features in very high dams located in deep gorges of short widths, it is strange to find some of our engineers plumping for concrete for dams that are long and have ample working spaces for speedy rubble in mortar construction. It must be admitted that at best our concreting will be an inferior imitation of the American methods. It is said that the strength of stone masonry is the stength of the mortar that is in it. This obervation is generally made with reference to the ordinary building construction where the walls are thin. In a dam where these stones are placed by hand in a particular manner with interstices filled with spalls and mortar, the whole rubble stone masonry becomes a homogeneous big section not at all comparable to a section of masonry in buildings. Moreover, the 28 days crushing strength of cement mortar 1:2.75 or 1:3 is about 3,000 lbs per square inch and cores taken out by drilling through masonry have been tested for this strength in crushing. Is that not proof positive that even assuming that the strength of the masonry is the stength of the mortar in it, a 3,000 lbs crushing strength per square inch or 200 tons per square feet is a high strength normally obtainable in stone masonry ? Is it then too much to allow a safe stress of at least 24 tons per square feet? What is the justification for prescribing a factor of safety of 10 for masonry while the factor of safety for concrete is about 3. It would be perfectly justifiable if the safe inclined stress were taken the same as for concrete. A masonry dam would still be a superior construction, and when I say that the Mettur Dam in the south, which is built of stone masonry, is the dryest dam in the World, nothing more need be said about the merits of masonry for dams. Water tightness and strength are the main aims. Water will not travel though solid granite stone. It will attempt to go through the mortar joints. But this path is made tight and circuitous because it has to travel over and around, up and down every stone unlike a concrete mass with many paths of percolation. This mansonry replacement was not only economical but also labour intensive. The rate of concreting was Rs.193.00 per 100 cft where as that of stone was Rs.152.75 per 100 cft."

"The second special constructional feature is the construction of the earth dam. A 16 mile long earth bund is the Hirakud dam and the formation of this earth section, specially the main dam, needs highly mechanized operation of digging and transporting the particular kind of earth that is suitable for zoning. And all this earth is laid in layers not exceeding nine inches thick and rolled with the optimum moisture to satisfy at every stage of operation the prescribed tests for compaction and other features. At every stage, at every moment in the field as well as in the laboratory, results are correlated then and there and corrections applied either for purposes of consolidation or for purposes of moisture variation. With three loaders and shovel capacity of 20.5 cyds, the 120 and odd 16 cyd. to 20 cyd. dumpers and earth carriers along with a battery of scrapers, continuously bringing material on to the dam at the average rate of about half a cubic yard a second, meticulously planned and worked with the requisite daily field maintenance and weekly servicing staff and with a well

equipped and efficiently staffed workshop that attends to periodical overhauls and repairing of vehicles requiring larger attention, the earthwok on this huge bund was going on as per schedule.

The third special feature is the attention to the labour engaged by the Department. Nearly 7,000 workers, skilled and unskilled are engaged by the Department for the shuttering and staging for concreting operations, operating and maintaining the enormous concrete, Earth Moving and workshop equipments. Over 2,000 huts have been constructed for this labour with provision for ample protected drinking water supply, first aid posts, dispensaries, hospitals, doctors to look after their welfare, midwifery, electric lighting, sanitation and recreation facilites. This is apart from nerarly 20,000 skilled and unskilled laboureres that are engaged by the contractors, on the Main Dam whose welfare is looked after by the Project Authorities through their employers."

(Source : indian Journal of Power & River valley Development - Hirakud dam project Vol.IV: Mahanadi Valley Development - Hirakud dam project by M.S. Thirumale Iyengar pg. 1,2 & 35)

3.6.3 Left Concrete Dam

3.6.3.1 Excavation of Foundation :

"The total quantity of excavation for foundation done was 80.65 lacs cft. of which 52.07 lacs cft. was excavation in rock. Most of the excavation work for obtaining fresh and sound rock for preparation of foundations was completed by end of December 1951. The method adopted for this huge quantity of excavation work was a homogenous and economic blending of men and machinery. While Air Compressors and Jack hammers were utilized for drilling holes; the removal of muck was done entirely by manual labour. Secondary blasting was avoided as far as possible and bigger stones were broken by hammers for removal. This proved more economical in the long run. Muck was deposited mostly on the downstream side of the excavated pit leaving some space for working area. All this muck was washed away in the floods of 1952 when the deep channel was diverted through the left dam.

Excavation of foundatons did not present any major hurdle in most of the area but sound rock was not met in some blocks upto a considerable depth.

These particular shattered and crushed zones in Blocks 5,7,8,12,13,14,16,20 and 26 were taken very deep till satisfactory rock was met. Then intensive foundation grouting was resorted to for the purposes of consolidation.

Preparation of foundations was a difficult, tedious and slow work.

Where hard sheet rock was obtained the brown or yellow crust was removed with sand blasting, Jack hammers and strong air water jets. Local tiny pot holes and crevices were excavated upto a depth of more than two times the width and cleaned with jets over long periods to remove any soft material.

Shear and crushed zones were carefully cleared of all loose and weathered rock by wedging and barring.

In blocks 12 and 13 of the downstream left side flank wall, a peculiar rock was met. When fresh, it was hard as granite but left over-night under water it crumbled like clay. This removal and cleaning went on for days and a depth of 15ft. of rock was removed in this manner before good rock was met."

3.6.3.2 Dewatering of Foundation

There was heavy subsoil water seepage from sides and bed, the nature of rock being peculiar and the foundations being very deep, it was a collosal job to keep the foundations dry for working by keeping heavy pumps at main places and a number of smaller pumps feeding them;.working all-round the clock. Local shallow pits were dewatered by pneumatic sump pumps.

In the Power dam and Power house work was carried out during monsoon also by dewatering with heavy pumps. There was one Russian pump with 5000 gpm discharge, four 100 H.P. electric pumps, two diesel pumps with 2000 gpm, capacity and a number of 50 HP, 30 H.P, 20 HP, 10 H.P. and 5 H.P pumps."

3.6.3.3 Grouting :

"The steep sides on both sides of crushed zones were benched off and anchor rods were grouted on sides for better bond.

The whole foundation was granite gneiss fresh, hard and sound. In block 14 and 20 there was a huge depth of hard rock10' over a very thin layer of undesirable strata. This thin layer was completely scraped off by iron rods. Holes were drilled through the ledge to interesect this layer, and it was completely removed by washing with air water jets through these holes. Pipes were left off projecting above to be grouted later.

In addition to the curtain grouting in a standard pattern (i.e. 3 rows of 30' deep holes at 20' apart) pipes were projected from all weak zones which were subsequently grouted under pressure after some depth of concrete was placed."

3.6.3.4 Foundation Concreting

"The weak zones, local depressions, deep cuts etc. were back filled with rich concrete before mass concrete work was started in such block. Greatest attention was paid to cleaning of the foundations. The foundation were approved by the engineer-in-charge along with the resident geologists as well as the Officer-in-charge of Drilling and Grouting of foundation, before concreting was taken up.

Late arrival of concreting machiney e.g. Crushing and Batching Plants and cranes, tested the ingenuity and skill of the engineers. A Batching Plant was devised and fabricated locally which served the purpose very well."

3.6.3.5 Batching plant

"It consisted of two sets of six bins each erected in two lines opposite each other. The bins were made out of readily available pontoon plates. They had a manually operated gate and balance at the bottom. Material like aggregate of all grades and sand was dumped in these bins from ramps by dumpers or trucks. Each had a capacity of 3,000 cft. Requisite weights of different materials were weighed and discharged into tip wagons, hauled manually on 2' gauge trolly tracks laid for the purpose. This mixed material from one set of bins was hauled by mannual labour to dam site where all along the dam several diesel mixers were placed. The dry mix was fed into these mixers, cement and water was added in the hopper. Mixed concrete from these mixers was chuted down into tip wagons placed on lines in the pit. This concerete was carried, placed and vibrated.

Mixed material from the other set of bins was fed into the 1/2 cu.yd. 'Ransom' Electric Mixers arranged in two rows opposite each others. The mixed concrete from these was discharged in the 2 cu.yd. buckets placed on flat trollies. These were hauled by diesel locomotives to the flanks where concrete was placed by a single Loraine crane that was available.

While placing concrete in foundations the prepared foundations were treated with neat cement grout and a thick layer of cement sand mortar which was brushed into all crevices.

Major part of the foundations was thus covered by 24.6.1952 when the floods came. The flanks were raised above the expected flood level to protect the earth embankments on either side. As the deep channel was closed and floods were diverted through this new channel, the hazards were many but the structure was sound and withstood high flood.

3.6.3.6 Forms

"Forms at this stage were required only for the contraction joints. Light panel forms of steel plates were used for obvious reasons-convenience of handling, longer life and better surfaces. No absorptive form lining was used though cloth was tried once but was not successful.

The forms consisted of 2'- 6" x 2'- 6" square 1/8" thick M.S. Sheets with 1'- 6" x 1'- 6" angles on all sides and ribbed in one direction in the centre. The angles had slots to take specially designed cotters. These plates were assembled and for rigidity of fixture horizontal and vertical bracings were used. The horizontal bracing was fixed to the plate with a lock plate and vertical bracing with 5" bolts.

Ties and struts were used separately to align the forms and keep them rigid while placing and vibrating concrete. Concrete was placed upto 3" below the top of the forms or shuttering plates and in the top plate six numbers of tie-loops were left.

The ties consisted of 6 strands of 14 gauge G.I. wire twisted round and round as required. The struts were made out of G.I. pipes. A sharpened iron rod was welded at bottom and a nut at top. A fully threaded bolt with forked head was placed in between the forms and the strut. This permitted the adjustment as desired.

Tie loops were made by welding a $^1/_4$" iron rod loop to a nut.

For refixing the forms the bottom two plates were removed while the top plate was held in position by the tie loops. The two rows of bottom plates were refixed on top and this process was repeated.

Peparation of the old concrete surface for receiving another lift of concrete presented a great problem. Several methods detailed below were tried for treating the cold construction joints.

3.6.3.6.1 Treatment of cold joints

(a) Jack Hammer

If the surface was rather old and set, a Jack hammer with a wornout Atlas rod was moved on the surface. This was found to be quite effective in removing the laitence and when cleaned with powerful air water jets, exposed clean rock pieces. An argument, was put forth that this has a tendency of making aggregate pieces loose in their places which do not come out by air water jetting and are harmful. In practice, however, it was found that nothing of the sort happened when concrete was really set. This method was therefore adopted for cleaning very old and hard surfaces.

(b) Sand Blasting

Method of treating construction joints could not be tried with any success. The elaborate equipment, the standard nozzle and the requisite quantity of compressed air were always lacking as also the requisite size of coarse grained sand. A simple method was tried which was very successful on relatively fresh concrete. A simple 'Y' with a nozzle inserted in the air inlet so as not to force the sand back was used. Bucketful of sand was kept very near and the air rushing out sucked sand and the suface was blasted. This dry sand blasting resulted in injuries and other damages. Wet sand blasting could never be tried.

(c) Sand Treatment

It was once tied to spread a layer of sand on green concrete immediately after placement. This sand was always kept wet and cut out with air water jet before placing next lift. But the sand was found to stick in places and this method was therefore given up.

(d) Cutting

The most successful method tried and adopted was cutting green concrete. Five or six hours

(depending on air temperature) after placement of concete the surface is cut with powerful air water jets. This takes away all laitence and exposes sound aggregate particles. This, repeated periodically, leaves a beautiful, clean, wet surface to receive next lift of concrete.

3.6.3.7 Foundation Gallery

Good foundations at higher levels in most of the blocks and the level of the sluice sills did not permit a full section foundation galley. A 3 ft. diameter drainage header has therefore been provided to collect and drain seepage water. The forms for this work were made into 4 quadrants collapsible drum in 5 feet lengths.

The two semi-circles were unfastened at inclined joints and collapsed round the hinges. This made the removal and refixing extremely easy, convenient and quick. Extra bracings were provided for rigidity. Circular angles were provided every 2'6" on which battens were mounted with counter-sunk bolts, covered with G.I. plain sheet.

Copper Plates

The contraction joints left every 60 ft. were made water tight with double rows of copper plates. The plates were given three V notches for rigidity during construction and for better bond. Angles were fixed with wooden and steel plates to give the asphalt well shape automatically.

Before concreting was started in the other block, channels were used to complete the well. These channels were pulled out 8 hours after concreting and hot liquid asphalt was poured in the clean well."

3.6.3.8 Crushing Plant

"A 200 ton per hour capacity crushing plant had been procured and erected on the left bank. The running and maintenance called for constant attention as the plant covered a large area and a small breakage stopped the whole chain, causing the whole plant to be stopped. To start with a 20-ton wagon tipper tipped the wagon of quarried and blasted rubble into a hopper, an apron feeder fed this rubble into the Jaw crusher. The new jaws which were very costly got worn out in not more than 50 hours of crushing. Building and replacing of jaws itself was a time taking process. The Jaw crusher crushes stone into 9" size which was taken by 36" belt conveyor to gyratoy crusher. This gyratory crusher was an ideal thing and was working very satifactorily. The stone crushed to 6" size was fed to the stockpile. The stockpile had 2 gates which discharged stone through pneumatically operated gates and electrically operated apron feeders to horizontal conveyor belt in a tunnel under the stockpile. The 6" stone was conveyed to the seconday crushing plant. The secondary crushing plant consisted of two P/81/2 and one S-37 gyratory crushers. By a system of 4 double decker vibrating screens and return-belt, aggregate was crushed, screened and conveyed to the bins of 3 different grades of material. The chutes and screens were the other thing which tended to give trouble in the running of this Plant. Screens had to be changed quite often, the chutes got worn out very quickly and the bearings of the screens needed freequent attention.

3.6.3.9 Blow-Knox Batching Plant

Blow-Knox Batching Plant consisted of four units, Each unit is independent and was a very great advantage. One single unit going out of order could be left alone and the other three could be operated without difficulty. Each unit had a top bin with 5 compartments to store different grades of material with a centre cabin for cement. These materials were weighed in three balances and chuted down into the mixers. Each unit had two one cu. yd. mixers placed side by side discharging the mixed concrete into a single hopper. The hopper gate as well as in gates and all other gates were operated by compressed air at 100 lb per square inch. Valves that work the rams were operated electrically and needed constant watching. The capacity of each unit was 6480 cubic feet for 8 hours reckoning on 4 min. cycle time and the performance had been very satisfactory. The procurement

of spare parts had been a bottleneck. Experience of the performance of the plant indicated that with crushed rock the mixer vanes wore off quickly besides other damages. Its prformance may perhaps be better with natural gravel."

3.6.3.10 Conveyance of Concrete

Conveyance of concrete was done in 2 cubic yd. buckets designed and fabricated by Hirakud workshop. The buckets were working quite smoothly and were not much different from the buckets in use generally. They were placed on flat trollies moving on 2' gauge 20 lbs rails. Two of such buckets were hauled by a single diesel 30 HP Locomotive. Concrete was discharged from the hopper of the Batching plant into the buckets and the locos haul the concrete to the cranes. Trolly lines were laid as smoothly as possible. As they had to be removed every monsoon, maintenance of these trolly lines to avoid derailment required vigilance and attention. Yet it was experienced that the trollies for conveyance of concrete were not quite suitable for such massive works. Either due to instability of the trollies on this narrow gauge or constant breakages of trolly axles and springs, derailments were frequent, wasting some concrete and a lot of time. Double track was laid in the whole length with separate incoming and out-going trains on different tracks. The lines were interconnected with crossings and economy was effected by using lesser number of locos. A Locomotive would leave its load at a particular crane and bring back the empty buckets on the other line at that crane. This cycle would be performed regularly by the same Loco while the other cranes would be fed by other Locos from the vacant line. Locomotives were 'Schoema', Germany Locos with Merceds diesel engines.

3.6.3.11 Placing of Concrete

The cranes employed for placing concrete were level Luffing Cranes with 6 ton capacity placing concrete upto 60' from the centre of the track. The gauge of the track was 15'. Cranes themselves had electric controls with direct current diesel geneators housed in a cabin. The controls and driver's seat were in an upper cabin. The hoisting speed of these cranes was a great advantage, and made up to some extent for the short length of the boom. The track was laid parallel to the axis on the upstream of the dam with one rail at 10' from the axis. The track could not be brought nearer than the edge of excavation and then at higher levels, the counterweights were just clear of the forms with this arrangement. For placing concrete above EL 560 however a bridge to carry the crane track at EL : 560 was constructed on the down stream slope of the dam.

3.6.3.12 (a) Air Supply

Compressed air required for operating the vibrators, Jack hammers, sand blasting jets, air-water jets etc. was supplied from the central air station at 100 lbs per square inch. The station itself consisted of 2 batteries of 5 "Broom-Wade" 250 cfm compressors in each Battery. The 5 compressors were connected to one tank from which one line was taken all along the dam and tapped at various points as required. Each battey of compressors worked with a common unloader. Thus 2 lines were laid along the dam fed by the two batteries. This enabled a great saving as no air was wasted. At a time not more than 3 compressors from each battery were running and no compressor was allowed to run for more than 15 hours continuously. The arrangement proved to be extremely satisfatory.

3.6.3.12 (b) Water supply

Tanks with a total capacity of 5 lacs gallons had been erected at EL : 700.0 on the Lamdungri Hill. These tanks were supplied by 4" rising main and water was pumped by one 60 HP high head electric pump. There were 3 pumps connected in parallel out of which only one ran at a time. A part of the water in these tanks was delivered into the smaller tanks with a total capacity of 70,000 gallons by gravity.

The water was treated in these tanks and then distributed for use and drinking purposes and water from the main tanks was supplied to dam at a distance of about 1.5 miles through a 8" delivery pipe. At the dam four 4" Victualic pipes were fed from this delivery line and all points were covered properly. Another delivey line ran from the main tanks on the downstream of the earth dam to supply water to the Crushing Plant and Blaw-knox Batching Plant. Another battery of 30 HP pumps was installed in the same pumping house and when the water went low in the tanks, delivery from the tanks was stopped and this 30 HP pumps were connected to supply water to the dam and Batching Plant by direct pumping. The whole arrangement worked quite satisfactorily.

3.6.3.13 Concreting of the Bucket

Forms to be used for concreting the bucket was subject to examination for a long time. It was found that concreting the whole bucket of a complete block with the conditions actually obtaining was an impossibility. It was also decided that joints could only be left in the direction of the flow. With these in view, special templates to function as forms were made and used with great success. A length of 125 1ft. of 50' dia, bucket was formed and concreted in a short span of 64 days. Slots were made in the forms to keep the reinforcement continous over a block. Top edge was fixed with an angle to keep the shape against screeding. An added advantage was placing and vibrating concrete in the open. The form panels were held by spacers and fixed rigidly in level and direction. Then reinforcement was inserted and bound, the panels were then tightened with angles at bottom and slots choked with wooden strips. The widths for which the forms were fixed varied from 9 feet opposite a pier to 12 feet in front of the sluices. It was thought better not to make a joint in the width of the sluice way. Pathways were made in both sides and across at regular intervals by planks supported high on iron tripods. Concrete, while being placed on cleaned surface, was vibrated and screeded to required curve. It was finished first with wooden float and then with steel float.

3.6.3.14 Sluice Sills

The sluice sills were concreted first with templates. Channels were bent in 7.5 foot lengths of the required curve. Concrete in bottom layer was originally left so as to give a minimum depth of 3 feet in one pour. These channels were mounted on specially prepared simple jacks enabling final adjustment to the curve. The jacks consisted of an angle iron piece of requisite length fixed vertically and rigidly to old concrete by two bolts on the forked end of the angle and two guys.

At the top, two nuts were welded. A fully threaded bolt with two nuts carrying the inverted channel in between them completed the jack, the two nuts could be adjusted to bring the channel in requisite level at each spot and keep it rigid against all vibration. Concrete was finished to the top of the channel. The channels were connected with each other by fish plates and were removed immediately after the initial set of concrete. The clean distance between the channels was kept as 12 ft. which is the width of the sluices. Sill blockouts for sluice gate beam and emergency gate beam were made of single planks jointed by angles and mounted on similar jacks so that rigidity and correctness were ensured.

The complicated reinforcement at the blockouts was simplified by bending the bars to shape instead of fixing straight bars at all levels in all directions. At the junction of bucket and main Dam blocks, steps were left in the pier portions so that at no time a depth of less than 3 ft. of concrete was placed at one time.

3.6.3.15 Sluices

Construction of the huge sluices of an unprecedented size of 20'- 4" x 12'-0" was a problem that required considerable planning and preparations. A plan of concreting was prepared and the number of sets of sluice forms to be fabricated was decided, keeping in view the economy and progress of concreting work.

At one time it was thought of putting up form wok of whole sluice and then to concrete. This was subsequetly given up and it was decided to use the same form in lifts. The sluice form plates were fabricated in workshop on the actual size curves made in wood. Detailed drawings were made to show dimensions of each single plate. The dummy sill was made separately and the side curve was laid on floor. The top curve was inverted and laid on floor. The curves of intersection were then plotted on these full size models. Plates were separated out so that no plated was too small. For the bell mouth curve at entrance dimensions were calculated every 3 inches apart to get as accurate froms as possible and an accuracy upto 1/16 of an inch was maintaied. Plates were cut for the downstream curve in the form of a template and upto a slope of 1:1 which was obtained at EL 510.0, concrete was placed in open without forming the rear curve. This was finished by usual method of screeding and use of floats.

The forms were fabricated to requisite curves. A suitable system of struts and bracings ensured true and correct surfaces. The cross brocing was made of two pieces of angles with small welded angles at ends to be fitted to plates, with cotters. In the centre a space of $^{1}/_{2}$ inch was left and two were jointed together with bolting on another angle. This facilitated removal after concreting. Manufacture of sluice gate and emergency gate blockouts presented enormous difficulties. The main gate blockouts were made in twelve feet height of wooden planks fitted to angles bent at right angles. Yet the removal and refixing, which was required to be done once for every sluice was the most difficult wok. Emegency gate blockouts were made in one piece and were easier to handle.

A time taking and difficult process was the provision of small boxes of all shapes and sizes at different levels and fixing them to these blockouts for anchor blots etc., and then providing anchor rods through them as also erection of forms for first lift of concrete.

First a three dimensional template was prepared to mark the bottom curve of interesction. After this was plotted, small concrete slabs were cast in situ on both sides to keep the forms from moving. A first lift of 4 feet and 3 subsequent lifts of 5ft. each were poured and piers were left off about 2 feet below the roof level, so that roof concreting was done with part of piers.

The roof forms were completely fabricated in the workshop. A unique feature was the introduction of clips for tying bracing to plates instead of 5" bolts. It was considered necessay that neither there should be any hole in roof plates nor any projection like a bolt head above. The results obtained were satisfactory. Another feature was the fabrication of corner plates as one unit for side and roof. This was a great advantage in erection work and it made possible the erection of sluice roof forms along with top side forms in the least possible time. The bell mouth opening is 16' wide by almost 28' high.

3.6.3.16 Erection of Roof Forms

An attempt was made without using any supports from bottom. I beams of 8" depth were cut plain to the width of the sluice every 6 ft. at right angles to the axis.At both ends angle cleats were bolted to the I beam which had holes to be bolted to the sluice wall. While concreting the wall nuts were left in concrete to receive these nuts. Accuracy was ensured by welding 4 nuts into a frame work which was buried in concerete with the bolts. These bolts were withdrawn and retightened with girders in place. Practically however it was found that unless those bolts were constantly handled they got rusted and it became impossible to remove them.

Later, therefore, a bally grill was made to support these girders.

The usual sleeper frame was then spread and plates jointed with greatest care so that each plate finds its own place. The horizontal bracing was ultimately supported by small ballies resting on these planks at sleepers.

Then came the most difficult part of the whole form work which was fixing air vents and sluice gate and emergency gate blockouts. It was a very complicated affair. A combination of wood work and use of the steel plates helped to quicken the work. As the staff got trained, time for erection of each block roofs was curtailed and finally the roofs of one block were made ready in 10 days with binding of reinforement.

At this stage, the blockouts for trestles for the trestle bridge for cranes for concreting above EL 566 were left on rear face.

3.6.3.17 Proportioning Ingradients for Concrete

In the case of masonry portion of the Dam, concrete in foundation has been put in to a minimum depth of 5'. All the R.C.C. structures of galleries, penstocks, sump wells, stairs, etc., are in reinforced cement concrete. In masonry, hard granite stone is used with 1:3 cement mortar in face masonry upto 17 feet and 1:4 cement mortar in the hearting.

For concrete, aggregate is crushed from hard granite stone and the following mix is used with Air Entraining Agent.(AEA) vide Table No. 3.17.

Table 3.17 : Proportion of various ingradients

Material	Mass concrete per cu. yd.	Rich concrete or RCC per cu.yd.
(-) 3" + 1½" aggregate	26%	26%
(-) 1½" + ¾" aggregate	22%	22%
(-) ¾" + ⅜" aggregate	24%	24%
River Sand	28%	28%
Cement	330 to 357 lbs	385 to 417 lbs
Water Cement Ratio	0.70	0.60
If cobbles are used		
(-) 6" + 3"	23%	
(-) 3" + 1½"	19%	
(-) 1½" + 3/4"	15%	
(-) 3/4" + ⅜"	15%	
River sand	28%	
Cement	330 lbs	
Water Cement Ratio	0.70	

Natural gravel was also used as far as available within reasonable distance. The mix resulting by the use of natural gravel either wholly or partially resulted in more workable concrete. Almost the same mix as above was used. The concrete obtained was very satisfactory and workable.

Cores taken out subsequently show dense concrete with almost no water loss in either case.

Use of air entraining agent from February 1954 increased the workability considerably. A solution of "vinsol" was used as the agent.

3.6.3.18 Vibrators

For consolidation, big two men C.P.T. pneumatic vibrators were used and the resultant concrete was very dense. For consolidation in reinforced and difficult zones thinner "Per dwarf" vibrators with 3" diameter body and long handles were used.

3.6.3.19 Difficulties met with and Suggested Improvements

It would be useful to record the difficulties encountered during actual placing of concrete so that impovement could be effected in future.

The reinforcement in sluice sills upto first 26′ from axis consisted of double row of 1 $^{1}/_{4}$″ rods at 5″ C/C. With this arrangement, rate of placement of concrete was not as fast as was desired in such big works. A minimum spacing of 9″ with increased diameter of rod would facilitate more efficient placement.

After putting the blockout shuttering for sluice main gates as well as emergency gates at bellmouth, a clear space of 5′ was left between forms at the contraction joints, with reinforcement on either side the clear space left was 4′. With the diameter of a 2 cu. yd. bucket as 5′, and long rods sticking for a height of 25 feet it could well be imagined that concrete would not be placed with any degree of speed. It becomes a very slow pocess, quite undesirable in such works. The difficulty was solved by placing concrete at a slow pace all the 24 hours every day and starting at several points simultaneously.

The cranes were another problem. The boom lengths were too small that not even the half width of dam was covered. Hence the concrete delivered by the cranes was taken by manual labour in baskets to spot of placing beyond the reach of crane. Hoisting speed was another consideration. Hence cranes with electrically operated controls (Electric power may be taken from outside or a diesel generator might be housed in the crane) with a siingle rope hoist, and covering the entire width would be a suitable equipment.

It was also experienced that carrying the buckets on 12 ton semi-trailers instead of on flat trollies on 2ft. gauge would be a definite improvement. In any case, if flat trollies and small diesel locos must be used the gauge should be wider than 2 ft. making the conveyance operation stable.

The form work for blockouts particularly for the sluice gate recess was a unique thing. It was extremely difficult and slow work. Full time attention was required to fix the small complicated boxes in their right places. Such complicated form work for the huge sluice opening and for its gate blockouts does not lend itself to fast work and should be avoided in massive structures where speed was the aim.

3.6.4 Right Concrete Dam (Right spillway & Power dam)

This portion of the Dam consisted of 36 main blocks i.e. 28 to 63 and 2 cut-off blocks with necessary retaining and training walls.

Foundation Excavation

The foundation rock was mostly granite and in several reaches, Arkose, Epidiorite and granite impregnated with quartz were met with. There were many fault zones in the foundation where either the excavation had to be taken to deep level or special treatment had to be done considering economy and the nature and extent of the fault.

The foundation of blocks 28 and the cut-off blocks adjoining the earth dam was good sheet granite rock and only 5 feet of excavation was necessary to reach suitable level. The block 29 was excavated to R.L. 490 ft. where good granite rock was met with. The deepest portion of the river in the right side channel was in the blocks 30, 31 and 32 and excavation had to be taken down to R.L. 485 ft to meet suitable foundation in these blocks. There was a crushed zone in these blocks and excavation in this zone was taken down to EL 480 ft. Foundation was taken to suitable rock level in the entire area before foundation concrete was laid. Foundation in blocks 33, 34 and 35 was good granite rock with local fissures and joints. Excavation was taken to deeper levels along the fissures till good rock was met with and the joints were grouted.

Upstream half of blocks 36 & 37 had good granite rock at EL 488′ but the downstream half had to be taken down to EL 478′ to get suitable rock. The rock where the elevation of the foundation changed from 488′ to 478′ was steep. In these steep portions, rock was benched, cleaned and dowels were provided for proper bondage of the concrete with the bed rock. In blocks 38 & 39 there was a wide zone of crushed epidiorite rock between granite rock, running from upstream of the dam to the downstream. Diamond drill cores were taken and suitable rock was met with at EL 473′. Excavation in this zone was taken down to EL 473′ and hard but fissured rock was met with. The entire area was grouted and some of the downstream holes were found to be interconnected with the fissures of blocks 40 and 41. Number of grout holes were drilled in the blocks 39, 40 & 41 along a wide fault zone and grouted. All the sloping surfaces were cleaned and dowels povided before concrete was laid.. There was good granite rock in the block 42 sloping from downstream to the upstream. Excavation was taken down to good rock level and concrete was laid. The rock in the upstream of blocks 43 to 47 was good granite at average elevation of 488′ but in the downstream no suitable foundation was met with. A number of diamond drill core holes were taken in the whole area to find the level of good foundation. The entire area had to be taken down to about EL 470′ to get suitable foundation. In block 46 the upstream rock was jointed and the area was grouted. There was a fault zone from the upstream of the dam to the low area in the centre of the dam in block 45 and this area was excavated till hard rock was met with. In the sloping rock surfaces dowels were provided for proper bondage of the concrete with the rock. In block 47 the upstream rock upto about 30 ft from the axis was vey good granite at EL 493′ and this dipped nearly vertically to EL 468′. In this steep slope dowel bars 2" dia were provided for bondage of concrete with rock. In block 48 a fault zone was met with nearly parallel to the axis of the dam at about 50 feet downstream of the axis. This fault zone was dug to fairly hard rock and reinforcement of old trolley line rails were provided spanning the fault zone. No suitable foundation was met with in the upstream half of block 49. Core holes were taken and it was found that suitable rock was below EL 470′. This area was excavated to EL 468′ till good rock was mt with. The foundation rock of Blocks 50,51 & 52 is good gdranite and excavation was taken down to the minimum depth such that the foundation gallery and penstock can be constructed at the designed location. The foundation of block 53 was badly crushed and excavation had to be taken down to about 465′ to meet suitable foundation. The area was grouted before concreting. In block 54 the foundation rock even at El 462′ i.e. 53 feet below bed level of the river, was found to be crushed with local areas of soft rock. Core holes were drilled and was found that there was no improvement of the rock even at El 410. Pressure test was made on this crushed rock and settlement was observed with load of 50 tons/sq. ft. There was a completely crushed zone from the upstream to the downstream of the dam near the junction of blocks 54 and 55. At the upstream near the face of the dam this soft area was excavated to EL 432′ and in the rest of the zone a trench was dug upto El. 452′ and concreted in order to provide a cut-off. Then the block 54 was cleaned and local soft pockets were excavated deeper. Reinforcement of old tolley line rails at 2 ft. centres in both directions was provided at the base and concrete was laid. In blocks 55 & 57 suitable foundation was met with at EL 480′ but the bed rock consisted mostly of softer stone such as arkose, silt stone and slates. Blocks 58 to 63 are in the slope of the hill of Chandli Dungri and the foundation in these blocks are in rising steps fom blocks 58 to 63. The rock in this reach was of softer variety and consists mostly of arkose and silt stone. Fairly hard rock was met with in this zone at about 30 to 40 feet below ground level and the entire foundation was benched and block widths were suitably changed such that the contraction joints were located at the vertical faces of the step of foundation rock. Excavation in these blocks had to be taken down deeper as the rock at higher level was found to be soft and it was found necessary to widen the foundation pit in the upstream and downstream to get the required foundation width for the section of the dam. 15 to 20 feet deep

wagon drill holes were drilled in these vertical faces and blasted. As the rock in these faces were full of joints, there was practically no effect on blasting and it was found difficult to excavate this rock. So after drilling, the holes were filled with cement water upto the top level and within 2 to 3 hours the holes were formed again by inserting the drill rod. The holes were charged and then blasted. This gave very good result and excavation was completed quickly.

Foundation of the power unit I, located downstream of block 54 was found to be soft and crushed at EI 447′ and excavation was taken down to EL 444′ and heavy reinforcement at the base has been provided. The foundation of the remaining power units is very good, consisting of hard granite and slate.

For dewatering the foundation 5 nos. 60 H.P. 8 nos. 5 H.P., 3 Nos. 15H.P. electric pumps and two100 H.P. diesel pumps were used. Local pits had to be dewatered with air driven sump pumps for final cleaning and concreting. During the monsoon of 1954, 4 nos. 110 H,P. and 3 nos. 60 H.P. and 5 nos. 5 H.P. electric pumps were used to dewater the Power House area.

3.6.4.1 Crushing, Batching and Transporting Concrete

Stone required for manufacture of aggregate was quarried from the Laxmi Dungri hill and brought to the crushing plant in Railway wagons. The wagons are unloaded direct into the apron feeder of a Primary Jaw crusher with the help of a Wagon Tipper and the stones are fed into the crusher gradually by the apron feeder. The crushed stone delivered by the Jaw crusher was taken with a belt conveyor to a P-20 Gyratory crusher and after crushing in the P-20 crusher the stone was taken by a belt conveyor to a stock pile. Most of the stone in the stock pile was below 6 inches size and was taken to the screening plant through a tunnel underneath with the help of a horizontal and an inclined belt conveyor. The oversize stone from the screening plant passed through a P-37 or P-8$^{1}/_{2}$ Gyratory crusher and returned to the screening plant by belt conveyors. Generally 3 grades of aggregate was produced and dust below $^{3}/_{16}$″ was eliminated and the aggregate was taken to separate bins from the screening plant by belt conveyors.

Graded aggregate was also produced by individual crushers and screens and fed into the aggregate bins by trucks to supplement the requirement, of the batching plant.

Good sand was available within one mile of the batching plant and was transported by trucks and fed into the bins either direct or with the help of a bucket elevator.

Under the aggregate and sand bins there was a tunnel with hand operated gates and a horizontal conveyor for delivey of the ingredients at the main batching plant belt conveyor and the materials were supplied to the batching plant as and when required.

Cement for concrete was fed into the hoppers with a screw conveyor at the bottom and taken to a bucket elevator and lifted to the batching plant bin.

Water for concrete was pumped from the river to supply tanks located at about 100 ft. above the batching plant and supplied by a 4″ dia. pipe direct. During hot weather the water was taken into a cooler and temperature was reduced by about 10^{0} F and supplied for concrete.

The batching plant is of Winget make with 4 number of 3 cu. yds. mixers having capacity to poduce concrete at the rate of 240 cu. yd per hour.

The concrete from the batching plant was generally transported to the site of work in 2 cu. yd. buckets with flat trollies and diesel locos. So far most of the concrete had been done below bed level and suitable openings were made in the flat trollies and rail track to discharge the concrete from the buckets into chutes and platforms. From the platforms concrete was carried by manual labour to the site of placing and laid. In blocks 45 and 47 there were sudden drops in foundation away from the edge of the excavation. Concrete was brought to these blocks upto the edge of the excavation by tip wagons fom the batching plant and delivered by chute into tip wagons at lower

level and transported to the edge of the step inside the block and discharged into platforms at the low level. Thus long lead and transport of concrete in steep slope by manual labour was avoided.

For want of the required number of suitable cranes, concrete had to be done by mannual labour. Wherever possible, concrete was laid direct by cranes. In blocks 47 & 46 there was a sudden drop in the foundation rock at 50 to 60 ft. away from the edge of cutting of foundation and the crane boom was not long enough to deliver concrete at the bottom of this drop. Chutes were constructed in this slope such that the top of the chutes were within the reach of the crane. Concrete was taken by the crane to the top of the chutes by buckets and discharged into platforms at the bottom of the slope and was taken to the site of placing by mannual labour. Thus the lower portion of the foundation was concreted and the difficulty of transporting concrete in steep slope was avoided.

Shuttering for concreting

"For flat surface steel shutters 2'-6" x2'6" similar to Blow form were used for all general purpose. For foundation gallery curved plates similar to the flat forms were manufactured and used. These forms were found to be easy to handle and quick to erect in position. Where the same type were to be formed in several places, special steel shutters were manufactured and used such that the same framework could be used from time to time. Where the formed surfaces had to change in three dimensions and flat steel plates could not be used, wooden form works were manufactured and used. For special structures where steel forms were not economical, wooden forms were used.

The transition section of the penstocks was changing from a 25'dia. circle to a 25'x10'-7⅝" rectangle. The sections at intervals of 1'-0" were plotted on cement platforms and steel trusses were made. These trusses were fixed in correct location rigidly and the surface was covered with teak or deodar planks 1½ " thick with bolts and nuts. The entire transition was divided into 48 sections and these pieces of the form were manufactured with arrangement for assembling them together to give the complete framework.

The corners of the draft-tube openings of the units 1 and 2 at the vertical sides with the floor and roof were circular curves with changing radius. Formworks for these corners were manufactured with teak wood and very good and accurate surface was obtained when concrete was laid."

The foundation rock in the old concrete surface is thoroughly cleared with air water jet before fresh concrete is laid immediately after laying the concrete in place. It is vibrated with two-men type vibrators and thoroughly compacted.

Excavation of the blocks 44 to 46 took far longer time as excavation in these blocks had to be taken down to 40 feet below bed level. Concrete was transported to these blocks by dumper and placed directly and vibrated and 6 lacs cft of concrete laid by dumpers in one month.

3.6.5 Masonry

In blocks 28 to 34 and 47 to 60 after laying foundation concrete, random rubble masonry with coarsed rubble face, where necessary, was done. Very good granite stone available within 7 miles of the Dam site was quarried and transported by trucks to the site of work. Face stones were quarried by skilled workmen. It was found cheaper to quarry stone by skilled workmen but it was necessary to use compressors and Jack hammers to produce the large quantity of stone which was required to keep up the programme.

Cement mortar 1:3 for face masonry and 1:4 for hearting was mixed by individual mixers at the site of work and carried by manual labour in mortar pans. The stones were carried to the site of laying by skilled workmen mostly with the help of chains and poles.

After thoroughly cleaning the old masonry surface 80 to 300 lbs stones were laid on a bed of mortar and the gaps were packed with mortar and smaller stones. Each stone was cleaned and

surface was moistened with water before laying it in position. After laying the stone it was shaken, hammered or vibrated with a hand bar to get it properly bedded in the mortar. Extra mortar was immediately removed and the top surface of the stone was left clean. Usually one layer of masoury was done in 24 hours.

3.6.5.1 Shuttering for concrete

"For flat surface steel shutters 2'-6"x2'6" similar to Blow form were used for all general purpose. For foundation gallery curved plates similar to the flat forms were manufactured and used. These forms were found to be easy to handle and quick to erect in position. Where the same type were to be formed in several places, special steel shutters were manufactured and used such that the same framework could be used from time to time. Where the formed surfaces had to change in three dimensions and flat steel plates could not be used, wooden form works were manufactured and used. For special structures where steel forms were not economical, wooden forms were used.

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3.6.6. Main Earth Dam

The Left Earth Dam starts from the Left abutment hill, R.D. 500 and joins with the Left Flank of Left Spillway, R.D. 4940. In this reach the ground level varied from EL 559′ to 520′ with the coresponding height of embankment from 92 to 122 feet, the top of dam being R.L. 642.00.

The Hirakud Island Section started from the Right Flank of Left Sillway, RD 6300′ to the Middle Channel RD 12000. A deep gorge, called Deep Channel between 7200′ and 7600′ was found with the lowest river bed level as R.L. 445′. Therefore the height of embankment at this place was about 195 feet. In other portions of this section the natural ground level varied from R.L. 550 to 530 ft.

The middle channel was from RD 12000 to RD 13000, the river bed varying from RL 490′ to RL 500 ft.

Fom R.D. 13000 to R.D. 14400 the main dam was laid over the Koilarkud Island section where the ground level was about R.L. 535′. Thus the total length of the Eath Dam was 12540 feet including the transition zones of the spillways."

3.6.6.1 Foundation Treatment

"The area under the seat of the dam was cleared of all jungle, vegetation, and roots removed. All loose rock was removed and top layer of soil to depth of 6", or more, depending on roots and vegetation and nature of soil, is removed, so that the embankment can be laid on either rock surface or virgin soil.

In impervious zone, wherever there was pervious soil, it was removed till the impervious strata was reached. In the bed of the river special precautions were taken. In the transition zones of left and right side of left spillway and left side of right spillway which were in the bed of the river, several loose boulders had to be blasted and removed in the impervious zone and the top layer of sand and shingle had to be removed till disintegrated rock was met with The upstream semipervious zone was also similarly treated. On the downstream semi pervious zone, at most places after removing the top sand the rocky bed was exposed. For a portion of the area in rear compact shingle mixed with silt was found and this was allowed to remain as it was a good filter medium. In the

rocky portions only shingle was laid on top and in the other areas sand was laid with shingle on top. The top layer of sand was given for the semi-pervious zone.

In the impervious zone the exposed rock surface was treated with mud slury under pressure and impervious earth was laid round it in 3" to 4" layers and tamped by pneumatic tampers. In the areas where the disintegrated rock was exposed the base was moistened and all small pools of standing water drained and then rolled by the sheep foot roller. Then the first layer of impervious soil was laid and levelled and compacted as usual. The foundation trenches of the retaining walls and the cut-off wall on the earth dam side were cleared of all loose material right to the bottom and then filled with good impevious material in 3" to 4" layers and tamped by pneumatic tampers. During this backfill, the trench was kept dry by pumping out the seepage water.

On the down-stream side, the first two 9" layers of semi-pevious material was laid carefully on the filter material and then watered and tamped by hand rammers, so that the filter might not be disturbed. Subsequent layers were then laid and compacted in the usual way.

In the deep channel portion from R.D. 7200 to 7600 the foundation was treated as under.

After dewatering, all sand and loose boulders were removed from the entire base. Rock outcrops with fissures and cavities, loose seatings with base rock, or of highly weathered and decomposed nature were removed by light blasting, hammering and by crow bars. All sharp crusts of good outcrops were removed :outcrops having thin gaps between were also removed. All smooth surfaces were made rough as far as practicable, and surface was thoroughly cleared all over and specially with air water jet in the impervious section. After a layer of mud leeping was done it was immediately covered with 3" layer of impervious earth by manual labour. This layer was laid with water content of 4 to 5% above optimum mixture and tamped with specially made sheepfoot type of tampers. Crow-bars and pointed rods were used to tamp inaccessible corners. This was then considered as the base for the earth dam. Steep banks of the deep channel were flattened out roughly to a slope of 1:2 and at a few points bank was flattened to as much as 1 : 4 for better grip with the Island. Throughout the base wherever sheep foot rollers could not be used for tamping, manual labour tamping and pneumatic tamping was done till the required density was obtained.

There were too many white ant hills on Hirakud island. Pits were dug to remove such hills completely together with the main hill with queen ant. Such sites were given 1 : 2 side slopes and filled with proper earth with optimum moisture and well tamped.

3.6.6.2 Cut-off Trench

The cut-off trench was dug to a depth at least 2′ into impervious base. Such depth had been roughly known by the drill holes loggings and fairly undisturbed samples of base materials of the trench were tested in the Laboratoy to determine the permeability. Near about R.D. 3000′ where a stream was fllowing beside a hilly bank the longitudinal slope of the cut-off trench was made flat enough to pass sheep-foot rollers along the length. This was done in the deep channel too. At a few sections of the cut-off trench, side slopes of trench showed sand deposits running across dam and very near the base of the trench. Such deposits were removed by shovel and dumpers from heel of dam to the axis of dam and refilled with good impervious material under optimum moisture and to specified density.

Width of cut-off trench was made 100′ in the deep channel and base was prepared with the intention to furnish subsidiary cut-offs in the impevious section.

Grouting was done to refusal at important sections of the trench and specially in deep channel sections with steep fall, sections with stream crosings and sections showing abrupt change in the type of material and wherever the geologist advised.

3.6.6.3 Left Earth Dam (RD 500 - 4940 ft)

RD 500 - 1100 ft.

The reach from R.D. 500 - 1100 consists of weathered granitic gueiss with an exposure of black gneiss between R.D. 900 to 1000. Excavation was done to 2' to 3' into disintegrated rock. The holes on the rock mound between R.D. 980 to 1000 took some grout. The remaining holes were mostly in disintegrated rock and the grout consumption was not much. There was considerable difficulty in drilling due to caving of the holes and the drill rods getting stuck. When the hole was washed it went on widening out due to washing of the material from the sides. It was found difficult to seat packers as the grout was by-passing the packer. Hence grouting had to be done from pipe nipples fixed on top of the holes. There were too many surface leaks in the cut-off trench and caulking was done with leadwool, jute bags and cement mortar. The total footage drilled and grouted in this reach was 594 feet and the total cement consumption is 228.75 bags which works out to an average of 0.419 bags per linear foot.

Grouting was done at a pressure of 50 lbs/sq. inch

RD. 1100-2000 ft

In this reach excavation was carried 2 to 3 feet into disintegrated rock. It was found that this layer was very compact and impervious and extended to a considerable depth below. So no grouting was considered necessary.

R.D. 2000-2400 ft

In this reach excavation was taken through disintegrated rock to the bed rock. Drilling was comparatively easy. But there was some difficulty in a few holes due to caving and the drill rods getting stuck up. Additional holes were drilled and grouted in the fault zone. Grout consumtion in this reach was quite appreciable. Total footage dilled is 707 feet and total consumption of unit was 911.75 cwts and the average works out to 1.289 cwts per linear foot.

R.D. 2400 to 4827

The cut-off trench of this reach consisted of granitic gueiss with bands of schist running in between. The principal system of jointing an the strike was parallel to the river "N 70°W" and dipping "S 65° W". The section R.D. 2524 to 2686 took very heavy grout, the total number of cement bags used beig 7493. The total number of holes in this reach was 16 with a total footage of 583' - 6". The average intake worked out to 12.83 bags per linear foot. One single hole in this reach alone took 1390.75 bags of cement which was the maximum grout consumption in the entire dam foundations.

There was considerable leakage in this reach and caulking was done with lead-wool, jute bags and cement mortar.

The total quantity of cement plaed in the 1942 holes drilled and grouted was 16,239 bags. The total footage drilled was 5157' and the average intake was 3.15 bags per linear foot.

RD 6690-8100 (Deep Channel portion) Geology.

The geology of this region was considered relatively simple, the predominant rock types being granite gueiss and schist. The principal system of jointing was parallel to the river, "N65°W" and dipping "S 62° W". The reach from R.D. 6692 to 7300 consisted of massive granitic gneiss while the reach from R.D. 7300 to 7600 consisted of schist and that from R.D. 7600 to 8000 consisted of disintegrated rock.

Drilling was done by wagon drills 30 to 50 feet deep.

In the reach from R.D. 6692-7270 comprising of the angular cut-off approaching the deep channel, 18 primary holes and 12 secondary holes were drilled. Total footage dilled and grouted was 889'-9", the total grout consumption per linear foot worked out to 3.01 bags. The maximum grout consumption in a single hole was 425 bags.

Deep Gorge 7270 to 7565ft

Thee gorge section had a drop of 50 ft to 60 feet below the adjacent bed level. The gorge section showed a narrow shear zone with a width of 1' to 2' and extending for a distance of 80 feet. Seepage was observed here which was arrested by drilling and grouting some jack hammer holes across the zone.

Three grout holes were drilled in the bottom of the gorge, two primary holes and one secondary. The primary holes took $^1/_2$ bag and 2 bags respectively indicating the tightness of the rock.

On the left slope of the deep gorge there were 13 grout holes. One hole alone; took 214.5 bags and another hole took 112 bags. All the other holes took comparatively smaller quantity of grout.

Reach 7565 to 8100 : Right bank of the Gorge

This reach was comparatively tight and did not take much grout. the total footage drilled and grouted in this section was 465, the total grout consumption being 370 bags and average intake 0.8 bags per linear foot.

Reach R.D. 8100 to 8640 ft

The cut-off trench was taken 2' to 3' into compact disintegrated rock and no grouting was considered necessary in this reach. R.D. 8640-at this reach there was a schist mound across the cut-off trench.

So three holes were drilled across the trench and grouted. The three holes together took only 22.5 bags.

Based on permeability tests it was decided that no grouting would be required in the Hirakud island section and it would be difficult if the exacavation for the cut-off trench was taken about 3 feet into the disintegrated rock. But on a subsequent examination of the cut-off trench it was noticed that certain reaches contain open jointed rocks with quartz veins which require to be grouted. These reaches were from R.D. 9450 to 9650 to 10750, R.D. 110750 to 11300 and R.D. 11550 to 11950.

R. D. 9450 to 9650 ft

Five primary holes at 40' apart and four secondary holes were drilled and grouted. The average grout consumption was 1.5 cwts. per linear foot, in the primary holes and 0.61 bags in the secondary holes.

R.D 9650 to 10750

Since quartz veins were present in this reach, it was decided to drill a few holes and grout them. In all seven holes were drilled and grout consumption was low (i.e.0.36 cwt. per linear foot).

R.D. 10750 to 11300

12 Primary holes at 40' centres and one secondary hole were drilled and grouted. The average consumption was 0.4 bags per foot.

R.D. 11550 to 11650 ft

Nine primary holes and four secondary holes were dilled. The aveage grout consumption was 3.6 bags per foot in primary holes. The maximum grout consumption in a single hole was 238.75 bags.

3.6.6.3.1 Cut-off Trench

Koilarkud Island

The cut-off trench to a width of 50 ft was dug in the Koilarkud Island. At the junction with the cutoff wall loose boulders were met with and had to be removed by blasting. Finally disintegratd rock and rocky outcrops were exposed in the zone R.D. 14,000 to 13,850 at an average depth of 8 to 10 feet below the normal ground level. The junction with the cut-off wall was at R.D. 14000. The area was grouted.

Four holes were drilled into bed rock and grouted, the total consumption in all the four holes being only 50.5 bags which works out to 0.496 bags per linear foot.

In the zone R.D. 13850 to R.D. 13175 the excavation had to be taken to a depth of about 20 to 25 feet below the ground level till disntegrated rock was met with. Just above the layer of disintegratd rock a shingle layer of 1′ to 3′ thickness was found throughout this zone. In this zone, as good disintegrated rocky impervious soil was met with no grouting was done as it was not considered necessary.

Middle Channel

This section was from R.D. 12000 to R.D. 13000. At either junctions with the Koilarkud Island and the Hirakud Island, for a short length of 100 feet shattered rock was met with which took heavy grout.

In the middle channel proper good rock was met with at a depth of 5 feet to 8 feet. But along the aligmenment of the cut-off trench very many boulders had to be blasted. The work of grouting of the cut-off trench was completed. Much grout was not taken by the rock as it was quite compact. Two lines of grout holes were drilled staggered to a spacing of 40 feet between adjacent holes and all these were grouted at a pressure of 50 Lbs/sq". The average grout consumption per linear foot worked out to 0.58 bags.

For additional safety, it was decided to have a second cut-off trench of 30 feet width running parallel to the axis on the upstream side at a distance of 60 feet from it.

Koilarkud Island (Additional Precautions)

It was found that the soil of the Koilarkud Island at the dam site was very light consisting of an admixture of fine sand and silt. So, as an additional precaution a second cut-off trench was dug just at the toe of the impervious zone on the downstream side. This cut-off trench was taken to compact soil below the top light soil. The depth of excavation was varying from 10 feet to 15 feet and base width about 10 ft. This trench was filled up with selected good impevious soil and well compacted in layers 4 inches to 6 inches.

On the upsteam side, just beyond the dam, a trench 3′ deep and 30′ wide was dug and filled with impervious soil compacted in layers to provide an up-stream impervious blanket.

3.6.7 Borrow Area

The Laboratoy, after checking up the nature of soils, fixed the borrow areas as semipervious and impervious. The main borow area for semipervious mateial for the Hirakud Island section, was about 2 to 3 miles from the dam and the loader area was from 4 to 5 miles from the dam.

The stripping of the borow areas was done by the dozers before excavation was started. For the shovels and scrapers no wateing was done in the borrow area. For the loader, watering was done in the borow area from Februay to June for laying the dust for ellicient operation of the machinery.

3.6.7.1 Haul roads

The haul roads were kept about 50 feet wide with easy gradients and well lighted in the night shift. They were regularly worked with graders and kept quite smooth. Also watering of the haul roads was done continously by 6 diesel Water Lorries each of 800 gallon capacity, to lay down the dust, thereby the hauling units could run at 25 miles to 30 miles speed giving proper out-turn. The ramps connecting the haul roads to the dam were kept at an easy gradient of 1 in 30, so that the dumpers could easily get up the ramps with full load.

3.6.7.2 Receiving Materials on the Dam

The dam section was divided into three zones consisting of a central impervious core with a semipervious section both on the upstream and downstream. The material from the borrow areas was received in the respective zones. The material was dropped by the dumpers and then spread by dozers to a uniform thickness of 9". Then the requisite amount of water was sprayed on the earth by hoses connected to 4" mains. Then compaction was done by sheep foot rollers dragged by tractors. 4" water main ran along the dam from which water was taken.

3.6.7.3 Compaction

The standard density for any soil was determined by proctor's compaction test. A lot of sample was mixed thoroughly and then compacted in a cycliner 4" dia. and 4.6" in length, in three layers giving 25 strokes on each layer with standard rammer of 5.5 lb weight and 12" drop. Several tests were taken with varying moisture content. Small sections of these cores were taken and weighed in a tin and then dried in an oven, and then again weighed to find out the moisture content percentage of the dry weight. Then from the wet density and moisture content percentage, dry density of each core was calculated and a graph was drawn showing the dry densities with different percentages of moisture content. The maximum density was found out from the curve corresponding to optimum moisture.

During execution the soils were compacted to at least 98% of the maximum dry densities as determined by the Proctor's test described above and care was taken to maintain the moisture content as worked above. It had been found out by actual observation that for semi-previous soils 16 passes of sheep foot roller and 20 passes for impervious soils give the standard compaction as explained above.

For semi-pervious soils it was obsrved that the dry densities after compaction vary from 115 to 125 lbs. per cft and for impervious soils the dry densities varied from 106 to 118 lbs. per cft. One sample for about 4000 cft of earth work done was being tested. For testing, core of 4" dia, and 5" to 6" long was taken in the impervious zone by driving in a steel cylindrical casing and wet and dry densities determined to find out the actual compaction achieved. In the case of semi-previous material, the density was determined by taking out material from a hole and determining its weight before and after drying. The volume of the material was determined by ascertaining the sand fill taken by the hole. At important places like cut-off trenches and filling at junctions of cut-off and retaining walls, tests were made more frequently.

Close to the cut-off walls and retaining walls, where compaction by sheep foot rollers was not possible, the compaction was done by pneumatic tampers in layers of 3" to 4". Particular care was taken to see that the old compacted surface was watered before receiving a fresh layer of earth.

3.6.7.4 Watering and Moisture Control

The watering for the right earth dam was done by direct pumping from the river. Two electric pumps of 60 H.P. were installed in the middle channel and the pumps worked alternately. The pumping was done into two four inches mains, one on either side of the dam. From these mains the water was spread on the dam through flexible rubber hoses 3/4th inch dia, about 150' to 200' long.

The moisture content of the soil varies with the season and also the nature of the borrow area and soil. The moisture content was the lowest during the summer months April to June when about 10% of water has to be added on the dam to the soil. For the impervious soil moisture content was kept slightly below the optimum to give satisfactory results. Semipervious sections could take more water.

3.6.8 Filter Material

The filter materials used on the job were laid in two distinct layers. The gradation of each filter layer meets the following requirements with respect to the material to be protected and also with respect to adjacent filter layers.

(i) D-15 size of filter was greater than 4 times D-15 size of coarsest maferial to be protected.

(ii) D-15 size of filter was less than 4 times D-85 size of finest mateial to be protected.

In the above D-15 was the equivalent spherical diameter of the largest particle of the finest 15 percent of the material, and D-85 was the equivalent spherical diamete of the largest particle of the finest 85 percent of the material. The requirements for grading of the filters were established by the field laboratory on the basis of mechanical analysis of adjacent materials.

The following gradation was found suitable :

(a)	For filter material in contact with foundation or with earth fill materials	Well graded coarse sand and gravel passing $^1/_2$" screen
(b)	For the middle layer of the filter blanket and layers in contact with rockfill.	Coarse river shingle passing 3 inch screen and retained on $^1/_2$ inch screen.

The upstream slope of the embankment was dressed to proper slope and covered with coarse sand, then river shingle graded from 1/2" to 3" to the spcified depths were laid. Then iprap was laid on it. Care was taken to see that the filter layers were not unduly disturbed while laying the riprap.

Rockfill Toe

The rock-fill toe was constructed of hard rock fragments to the required section. Smaller rock spawls to fill in the voids of the coarser fragments were permitted. The larger rock fragments were placed on the outer slopes and the smaller fragments placed next to the earth-fill.

Pitching

The outer slope of the rock-fill toe was pitched neatly with quarried stone to a thickness of 1'3". The size of stone used was about 120 lbs.

3.6.8.1 Upstream Riprap

The riprap consists of quarried rubble of stones weighing about 120 lbs. They were laid on the slope and roughly hand-packed to proper profile filling the interstices with stone spawls.

3.6.9 Dykes

The alignments of the dykes have been fixed after thorough investigation. The dykes run over ridges so that the maximum storage is assured with the least amount of work involved in the construction. The top of the dyke is at the same level as that of the dam. The dyke on the right is in one stretch of 35,500 ft. but that on the left is made up of 5 gaps connected together by a link road 25 ft. wide on same level as the top of the dam, so that when the dam is completed, it will form one continous and pleasant long drive of 22 miles over the dykes and the dam.

3.6.9.1 Right Dyke

The dyke on right side which is in a continuous stretch, connects Chandli Dungri and the Barapahar hills. It is a stretch of 35,500 feet in length with an average height of 29.8' starting at a distance of about 3.5 miles - from the right abutment of dam. The maximum height of this dyke is 43'. The quantities of different items of work involved are :

Table 3.18 : Quantities involved in Right Dyke

1.	Earthwork in embankment	9,50,36,000 cft.	(26,89,519 cum)
2.	Earthwok in stripping	1,32,33,000 cft.	(3,74,494 cum)
3.	Rubble work in pitching and dumping on the slopes	1,08,42,000 cft.	(3,06,829 cum)
4.	Graded filter and sand	39,40,000 cft.	(1,11,502 cum)

The total estimated cost for this work is Rs.1,71,56,400.

The biggest canal of the Hirakud canal system called the Bargarh canal with a discharge of 4900 cusec takes off form the Right Dyke. The estimated cost of the head-regulator is about 16 lakhs of rupees.

3.6.9.2 Left Dyke

There are 5 gaps in the left dyke.

Left Dyke Gap I

Left dyke Gap I is a high earthen embankment and the work is done with earth moving machiney. Caterpillar DW20 scrapers, Euclid Mac & Sterling Dumpers with Lima shovel and Caterpillar D8 dozers are utilized for this work.The construction of this dyke is done mostly departmentally. The gap is about a mile long and the maximum height of the embankment here is 88'. The quantities of the different items of work involved are :

Table 3.19 Quantities involved in Left dyke Gap I

1.	Earth work in embankment	:	4,64,75,000 cft.	(13,15,243 cum)
2.	Earth work in stripping	:	84,13,100 cft.	(2,38,091 cum)
3.	Rubble work	:	50,30,000 cft.	(1,42,349 cum)
4.	Graded filter and sand	:	27,72,165 cft.	(78,452 cum)

Left Dyke Gap II

The second gap is a masonry structure, 400' long. The maximum height is 68.19 feet. The structure is of gravity type and is constructed of rubble masonry in cement mortar with face coursed and hammer dressed. The foundation contains schist with longitudinal fissures which have been thoroughly cleaned and grouted.

The quantities of work involved in the construction of this structure are as follows:

Table 3.20 Quantities involved in Left dyke Gap II

1.	Earth work	:	2,45,145 cft.	(6937.60 cum)
2.	Masonry	:	2,70,854 cft.	(7665.17 cum)
3.	Concreting	:	6,080 cft.	(172.06 cum)

The cost involved is about Rs.5,56,000.

An irrigation sluice has been provided in this dyke to feed the Sambalpur and Hirakud distributaries. The former is designed for a flow of 93 cusec and the latter for 11 cusec.

Left Dyke Gap III

The third gap is 2 miles long and is an earthern embankment. The maximum height is 29.63 feet. Most of the work has been done by manual labour. The compaction work had been carried out with the help of the sheep foot rollers and tractors. The quantities of work involved in the gap are as follows :

Table 3.21 Quantities in Left dyke Gap III

1.	Earth work in embankment	1,80,70,000 cft.	(5,11,381 cum)
2.	Earth work in stripping	18,93,500 cft.	(53,586 cum)
3.	Graded filter and sand	5,56,000 cft.	(15,735 cum)
	The cost is	**Rs.19,09,000.**	

Left Dyke Gap IV

The fourth gap of the left dyke is also 2 miles long. The average height of the dyke is 30' and the maximum height is 40.18'. The work was done on contract basis and the contractors used earth moving machines. Compaction work was carried out departmentally.

The quantities of work involved are given below :

Table 3.22 Quantities in Left dyke Gap IV

1.	Earth work in embankment	:	3,13,20,400 cft.	(8,86,367 cum)
2.	Earth wok in stripping	:	39,18,000 cft.	(1,10,879 cum)
3.	Rubble work	:	29,17,125 cft.	(82,555 cum)
4.	Graded filter and sand	:	12,00,000 cft.	(33,960 cum)
	The cost is		**Rs.42,46,000.**	

The main canal of the left bank called the Sason canal takes off from this dyke. The head-regulator is situated in this gap which is designed for a discharge of 630 cusecs.

Left Dyke Gap V

Gap V is the last of the series and is about 2,000 feet long. The maximum height of the dyke is 45.30 feet. The amount of work involved in this gap is :

Table 3.23 Quantities of work in Left dyke Gap V

1.	Earth work in embankment	:	68,89,000 cft.	(1,94,959 cum)
2.	Earth work in stripping	:	8,77,000 cft.	(24,819 cum)
3.	Graded filter	:	2,95,000 cft.	(8,349 cum)
	The total cost is Rs.8,31,000.			

Together with the manual labour which has been used for doing most of the earth work, tractors, tip wagons and even mules have been utilized for transporting mateials.

Earthen embankment of the dykes are of the rolled-fill type. The borrow areas from where the earth is brought on the embankment are selected beforehand after tests are carried out as to their suitability. The soil used in the dykes has atleast 10% of clay. Rigid laboratory and field control is held on the placement of the materials in the structure. Earth is laid in layers so as not to exceed 6" thickness often being rolled. It is watered to bring it to the optimum moisture content. Rolling is carried out next with sand filled sheep foot rollers which yields a pressure of 200 lbs per square inch. 16 to 20 passes of the sheep foot rollers bring the earth to the required compaction. The

borrow areas are generally located on the upstream side of the dykes as this will ultimately be submerged by the reservoir.

Field control of moisture content and compaction is maintained by a batch of Analyst and Embankment Inspectors who take samples and perform tests on them in various Field Laboratories located at the work site. They wok under the direct control of the Deputy Director, Research Station of the Project. The daily field tests which are conducted and the results obtained are tabulated and correlated and in this way an effective control on compaction and moisture is maintained at every stage of the work on both the Dykes.

The original design of the dykes provided zoning of the earthen embankment with an impervious core in the centre and two semi-previous zones on the two sides. As the nature of the soil in the borrow pits did not make the different types of soils available to the extent that was required, zoning was abandoned except for Gap 1 and a homogeneous section has ben provided.

The slopes of the dykes are protected both on the upstream and downstream sides. The upstream side which has a slope of 1 : 3 is protected by rip rap placed on a gravel blanket.

The upstream is pitched with $2^1/_2$' of rubble hand packed and placed over layers of 6" of gravel and 6" of sand upto R.L. 635 which is 5' above the maximum Reservoir level. Above this the thickness of the pitching is reduced to 1'-3". In the right dyke wave-breakers have been provided to minimize the effect of waves by using long stones sticking out about a foot above the pitching at intervals of 10' both ways. The placing of sand, gravel and the stones in the rip rap work have been carried out by manual labour.

The downstream side which has a slope of 1:2 is protected by a rock-toe at bottom and turfing above.

The dykes are situated at far-off places from the main colonics and have developed into independent units. Small workshops with a separate generating unit have been set up here and there. The water supply both for construction and human consumption was difficult problem. In some cases, the distance of the river is 4 to 5 miles. The difficulty was more acutely felt specially during the summer when the thermomete touched 118^0 F in the shade. As no electrical power could be available at the out-of-the-way places from where water had to be pumped to the dykes, only oil-driven pumps were used for this purpose. (Source: Indian Journal of Power & River valley Development Hirakud Dam Project Vol.IV pg.3-25)

3.6.10 Joints in Power House :

Contraction joints have been provided in between the units in the sub-structure as well as in the suprstructure of the P.H. from EL 447 to 593 ft. These joints have been provided to relieve tensile stresses induced in the structure due to shrinkage. Concrete on one side of the joints is cast first. After the form is removed from the joint face, one inch thick cork board coated with asphalt is provided to prevent adhesion of the concrete placed against it.

In English units, contraction joints are provided in the walls in the u/s and d/s sides in the substructure of the P.H. upto EL. 593 ft. Horizontal contraction joints have also been provided in the turbine floor, generator floor and roof at EL 593'. These horizontal joints are levell and terminate approximately at right angles to the faces of u/s and d/s walls of P.H.

The contraction joints betwen the units in the superstructre are made continuous through the walls, floors and roof slabs by double R.C.C. columns and double floor beams at joints.

3.6.10.1 Cork board

This is used as joint filler. It is a water proof material. Concrete on one of the joint is cast first and the form is removed from the joint face. Joint filler. i.e. cork board coated with asphalt is pressed against the surface and the concrete placing is resumed.

These joints have been made further water-tight by use of asphalt seal, copper seal rubber protective strips. Two copper seals are installed one on each side of a diamond shaped hole which is filled with asphalt. Two steam pipes have been left in diamond shaped hole for re-heating the asphalt. As an additional precaution formed drains have been povided behind rubber water stops.

3.6.10.2 Cover

Rubber protective strips have been used to protect the exposed edges of joints and to prevent the ingress of foreign material that might other wise cause spalling of the concrete edges. These strips improve the architectural beauty also of the structure. Stainless steel straps have also ben used in covering the vertical joints to hold the rubber protective strips in place.

3.6.10.3 Galleries

In the body of the dam there are galleries for various purposes :-

Foundation Galleries : This is meant for drilling and grouting operations, curtain grout and taking drainage water from the drain pipes in the foundation and from formed drain holes in the body of the dam, and inspection. In the Left Spillway there is 5′ x 7′ gallery in the Transition portion and header under the sluices 12′ downstream of axis. As there was no sufficient headway between rock foundation and the sill of the sluices 3′ drainage header was provided, which is connected with 5′x7′ gallery in the transition portions and there are 4′ diameter manway with adits along the header in block 5, 12 & 19 connecting with sluice at downstream of the gate for intermediate inspection. In the Right Spillway and Power dam, there is one continuous gallery 5′x7′ from one end to other. In block 53 and 54 , where the foundation has gone very much deeper as mentioned in the excavation, lower foundation gallery was provided in addition at 27′ below the upper foundation gallery. This lower gallery has got cross gallery also which runs over the crushed zone. In Power House 5′x7′ gallery runs at elevation 470′ with adits for inspection of draft tubes and for dewatering.

3.6.10.4 Operation Gallery

In both the Spillways, there is operation gallery 11′x20′ (the biggest in Asia) at EL 557′ for operation of the sluice gates. In the power dam in continuation of this operation gallery runs a 5′x7′ gallery at the same floor level. There is upper gallery in Power Dam at EL 628′ with hoist chambers for operation of penstock gates.

3.6.10.5 Stair Wells

There is stair well 6′-6″ in diameter at the left end of the Left Spillway (block 1 of the left flank wall) with R.C.C. spiral staircase connecting the foundation gallery with operation gallery. 6 ft. dia stair well in block 1A of left flank wall from foundation level to the top of the dam connected at the bottom and top with 10″ dia. plumb hole shaft.

3.6.10.6 Stair Case

There is stair case 3′6″ wide in block 25 of Left Spillway and block 30, and 57 of Right Spillway connecting the foundation gallery and the operation gallery.

3.6.10.7 Pump Chamber and Sump Well

To pump out water from the foundation gallery of the dam, there is a 6′ dia. sump well in the leftflank wall block 1 and block 23 of the Left Spillway block-32 and 47 of the right Spillway and in block 53 of Power Dam at EL. 470 to EL 497. In the Power House too, there are sump wells, - one between units 1 and 2, and the other in unit 3 and the third in unit 4 provided with pumps to pump out water from sumps in the foundations.

3.6.10.8 Gate

All the sluices, crest bays, penstock and draft tube openings are controlled by electrically operated gates.

Sluice Gates

There are 64 sluices. 40 in the Left Spillway and 24 in the Right Spillway. They are provided with vertical slide gates 12′ x 20.33′, each shutter weighing 26 tons. Weight of steel used in each gate including the embedded parts is 45.8 tons.

For operation of these gates, there are 5 Gantry Cranes in the Left Spillway and 3 in the Right Spillway. Each gate is operated individually by these cranes. Each crane is 50 tons capacity. The hoisting speed of the gate is 1.5ft per minute.

Emergency Gates

If any sluice gate goes out of order 2 emergency gates in each Spillway are provided. The size of the gate is 16′ x 28.3′. Weight of each shutter is 44.6 tons operated by a 100 ton over-head crane from the top of the dam. Weight of steel used in each gate including embedded parts is 307 tons.

Crest Gates

There are 34 bays over the Crest, 21 in Left Spillway and 13 in the Right. Each is provided with a radial gate 51′x 20′. The weight of the leaf is 27 tons. Weight of steel used in each gate inclusive of embedded pats and trunnions is 45 tons.

Penstock Gates

The size of penstock gates on Enghish unit is 19.60 ft x 31.05 ft. and that on German Units 17.9 ft. x 30.68 ft.

Draft Tube Gates

Size of gates on draft tubes in English units is 25′.6″ x 18′-1″ and that on German unit 23′-7.5″ x 18′-5.75″.

3.6.10.9 Cranes for Operation

There are 8 Gantry cranes for operation of sluice gates, 2 over-head cranes of 100 tons capacity for operation of emergency gates and penstock head gates and one gantry crane of 25 tons capacity for opearation of draft tube gates.

3.6.10.10 Gauge Well

For measuring elevation of water level in the reservior there is a gauge well in block 57 and a gauge well tower of R.C.C. constructed on the left flank wall, which is fitted with automitic recorder with remote control to record water levels in the reservior.

3.6.10.11 Raw Water Intake

Arrangement has been made to take raw water from the reservoir for water supply on either side of the river and for the Hirakud Island. The raw water intake on either side of the river is 3′ diameter formed hole fitted with a valve, one at EL 580 ft and the other at EL 610′. For water supply for the Hirakud island one foot dia, formed hole with a valve is provided in the Left Transition of Right Spillway.

3.6.10.12 Installation of Instruments

There are two plumb bob wells, one at either end of the dam in left flank wall of lthe Left Spillway and block 57, running from the fondations upto the top of dam for observation of deflection in the dam, cells for measuring soil pressure and hydrostatic pressure on left flank wall, strain meters and thermometers in block 41 and 57 and uplift cells in blocks 58 and 59 are also provided.

3.6.10.13 Elevator Tower

There is an elevator tower with a lift in block 56 in the Power dam for an adit leading to Power Plant at EL 527 to 32.6 ft. above the top of the main dam, connecting on the way galleries

at EL 557 and 628 and top of the dam. Between the elevator shaft and the slope of the dam, there are seven rooms, one over the other, the roof of the top one being at EL 674.63, i.e. 32.6 ft above the top of the dam.

3.6.10.14 Junctions at Abutments

Proper care has ben taken to key the concrete masonry dam into the earth dam and into the hill on the right side end. On the Left Spillway dam on the left end there is flank wall with training wall on the downstream. There are key ways at various places of the flank wall for proper bond with the earth dam behind and to obstruct creep. In continuation of the flank wall runs 100′ long cut-off wall keying into the earth dam. On the right end runs 275′ long transition with 100′ long cut-off wall keying into the earth dam, training walls upstream and downstream. In the Right Spillway, on the left end runs similar transition 360′ long with 100′ long cut-off wall keying into the earth dam and training walls upstream and downstream. On the right end, the transition is 340 ft. long, keying into the hill. There was exploratoy tunnel into the hill 26′ long at EL. 618.5 which was filled with concrete and keyed into the masonry of the last block of the dam. Proper care was taken to bond the masonry with the sides of the hill and grout the junctions at every 5′ lift.

Apart from that, earthen platform on the upstream 200 ft. long and 86 ft. wide has been constructed at the abutments with a layer of impervious earth on the hill slope and masonry pitching on the earth slope from river bed upto the top of the dam.

3.6.10.15 Road ways

The operation gallery in the Left Spillway is connectied at either end with road along the rear of the earth dam at E.L. 557. That in the Right Spillway is connected at left end with road along the rear of earth dam at E.L. 557 and at the right end with masonry road in the body of the Power dam at its rear at EL 557 ft.

On the top of the masonry concrete dam runs a 25′ road with kerbs and ornamental parapets with lamp posts on the downstream parapet.

3.6.11 Details of Power House

The work in the Power House was a colossal job with such deep foundations, 71 ft below the river bed, bad foundation rock in the first two units and heavy sub-soil water trouble, huge quantity of excavation, complicated nature of work which is mostly reinforced cement concrete with, various blockouts and with special treatment for joints. The height of the structure from the foundations to the top is as high as 148 ft.

In the Power House, there are two floors, one running at EL 510 ft called turbine floor and the other at EL 527 called generator floor. Behind the unit 2 and 3 there is an oil room with floor level at 507.5 and roof at EL 527. Betwen the Power House and the Power dam there are anxiliay rooms in triple storey structure with a lift, containing cable room, battery room, contol room, shift engineers' room, ventilation room, reception room, recreation room, store room, telephone exchange etc. The whole Power House and auxiliary rooms are air conditioned.

The entire work of the Power House has been completed in three seasons.

3.6.11.1 Hirakud Power System

It was envisaged to generate hydro-electric power in two stages; one at the main dam at Hirakud and the second at the sub-sidiary dam forming the balancing reservoir at the end of the power channel near Chiplima.

The total installed capacity proposed was :

i) At Hirakud 6 units x 25 MW, (Upper P.H)	= 150 MW
ii) At Chiplima - 8 units X 25 MW, (lower P.H.)	= 200 MW
Total 14 units	**= 350 MW**

It was proposed to have provision in the layout for 8 units in upper power house and 11 in lower power house for future expansion if required.

This was planned with F.R.L. at 625 and D.S.L. at 580 ft.

As described earlier the first power house was to be located at Hiakud Dam and the second one at Chiplima about 16.70 mile (26.72 km) downstream of upper power house.

The first power house was to be constructed within 958′ (292m) length consisting of power dam blocks of 48 to 63 & transition block adjacent to right abutting hillock of the dam called Chandli Dunguri. Subsequently Ashok Niwas, a prestigious I.B. and Jawahar Minar have been constructed over a part of this hillock.

Sir H.W. Lewis (Who was Deputy Commissioner of Sambalpur during 1920-22), the then Governor of Odisha laid the foundation stone in 1945.

The first batch of concrete in foundation was laid by Pt. Jawaharlal Nehru, Hon'ble Prime Minister of India on 13.4.1948 and the works of power house & dam went ahead.

"An Advisory Committee formed in 1948 to review the 1947 Project Report recommended as follows. The dead storage level having regard to silting condition was recommended to be 590′ R.L. instead of 580 R.L. The tail race of the Upper Power House instead of being taken through a channel would be dropped into the right arm of the river and the Power Channel would be taken off from the junction of a bund connecting the tip of the Hirakud Island with the high ground of right bank to save heavy and expensive cutting in earth and rock in the head reach of the channel. For estimating power potential the 'average year basis method was adopted and was worked out to be 152 MW. An installed capacity of 321M.W. consisting of 6 units of 37.5 M.W. in the Upper Power House and 4 units of 24 M.W. in the Lower Power House with provisions of two more penstock pipes in Upper and two in Lower Power Houses for future expansion had been proposed. The Lower Power House had to work as a base load station and the upper was to meet fluctuations in demand. A power cut of 25 to 30 percent during driest year occuring once in 20 years was envisaged and to tide over the severity of power cut, the Committee had suggested earlier filling of reservoir, reduction of water for irrigation and staggering of load etc.

Regarding programme of power development, the Committee suggested installation of two 24 MW Units in lower Power House with further installation of two 37.5M.W units in the Upper Power House in the first step. Work was started in 1949 generally following the above proposals excepting for the change that the bund across the right arm of the river below the dam was to be substituted by a masonary weir 2720' long with crest at R.L. 516′. Orders for power plants consisting of two 24 M.W. units for lower Power House and two 37.5 M.W. units for Upper Power House were placed. At this stage in December 1951 Government of India took an important decision giving higher priority for irrigation than for power generation as a consequence of which the work in progress on the lower Power House and the Power Channel was suspended temporarily and efforts were concentrated to expedite completion of main dam and irrigation works. Hence the Upper Power House had to be completed first and it was decided to install the two 24 M.W. units ordered for lower Power House with requisite modifications along with the two 37.5 M.W. units already ordered in the Upper Power House at Burla. It was found that the two 24 M.W. Kaplan turbines ordered for the Lower Power House would not be suitable of the head of the Upper Power House and hence two Francis turbines were ordered to suit the conditions obtaining in the Upper Power House and work with the 24 M.W. generators already ordered for the Lower Power House. Thus the two Kaplan turbines ordered for the Lower Power House were surplus to the immediate requirement. These turbines along with two 24 M.W. generators ordered subsequently were installed in Chiplima Power House in the second stage of Power Development. For various considerations the maximum level of reservoir was raised from 625′ R.L. to 630′ R.L.

As per the decision of Government, the original project estimate was to be reviewed after five years and accordingly a revised project estimate was submitted in 1952. In this report additional plant of two 26 M.W. units in the Upper Power House and three 31 M.W. units in the Lower Power House were contemplated. The Government of India appointed a Committee of Experts to examine the revised project estimate. The Advisory Committee recommended that the installed capacity of the Upper Power House may comprise of two 24 M.W. units and two 37.5 M.W. units in the first stage with one more 37.5 M.W. unit in the next stage of expansion. Provision had also to be made for installation of two more 37.5 M.W. units in future for generation of additional peaking power. The lower power house would consist of four 24 M.W. units. Selection of 24 M.W. units for Upper Power house was made with a view to utilising the generators already ordered for the Lower Power House. After careful consideration of water conditions it was found that the power potential would be 120 M.W. continuous for a normal year and 90% of this quantum can be obtained in a bad year. The revised Project Estimate as modified by the Advisory Committee was presented to the Hirakud Control Board as per whose decision the Chief Engineer of Hirakud Dam Project prepared a revised estimate for Hirakud Stage I works which envisaged production of power in the Upper power House at main dam only. In this form the revised Project Report was accepted by the Government and the work was executed as such. Power development was limited to an installed capacity of 123 MW. comprising two 37.5 MW. and two 24 MW. unit giving a firm capacity of 85.5 MW. and the power potential of stage-I only was 73 MW. continuous. Thus the electrical works of stage 1 consisting of installation of 123 MW. geneating capacity at Burla Powe House, constrution of 132 KV and 66 KV transmission lines totalling 578 miles and ten grid Sub-Stations at Jharsuguda, Rourkela, Joda, Noamundi, Kuldiha, Tensa, Betnoti, Bargarh, Chainpal and Choudwar were taken up and completed in all respects.

The second stage of Power development assumed great urgency in 1954 as the prospects of setting Aluminium Smelter at Hirakud, Steel Plant at Rourkela and Ferro-manganese factory at Joda began to take practical shape and the earlier scepticism regarding load prospects proved to be rather unfounded. The Development Board recommended for setting up a number of electro-metallurgical, cement, textile and paper industries in the command area of Hirakud Power System and the need of power for these prospective industrial complexes was beyond the capability of generating sets installed in Stage 1 of Hirakud Project. Hirakud Control Board and State Government took up the matter with the Govt. of India and Planning Commission for sanction of Hirakud Stage II scheme and ulitmately sanction at an estimated cost of Rs. 14.32 crores was obtained in 1956.

The earlier proposals of constructing the Lower Power House at the toe of the subsidiary dam at Chiplima, connecting the Subsidiary reservoir by a power channel to take off from the right bank of river and to direct discharge of Upper Power House let out into the river by means of a diversion weir into the Power Channel underwent certain modifications. It was decided to do away with construction of weir and to take off the Power Channel straight from the tail race of the Upper Power House by constructing a divide wall between the tail race and the river. It was further decided to exclude construction of subsidiary dam with subsidiary reservoir and to have a power channel from the Upper Power House to Lower Power House with a balancing reservoir in between. The Power Channel was decided to terminate in a small fore-bay at Chiplima Power House and to lay pipes form the forebay to the generating sets. The installed capacity in the StageII-Scheme as modified by the Hirakud Contol Board provided for three 24 M.W. units in Chiplima Powe House and extention of the Upper Power House by two 37.5 M.W. units. With the execution of this scheme the Power potential of Hirakud Stage I & II was calcuated to be 123 MW. continuous. The Stage II works consisting of about 17 mile long power channel, addition of two 37.5 M.W. units at Burla Power House, Chiplima Power House with 3 units, 24 MW. each, 126 miles of 132 KV transmission line and Railway electrification Sub-Station at Rourkela was taken

up and completed in February 1964.(N.B. Stage-II was inaugurated by A.N. Khosla, Governor of Orissa on 14.9.63) Later the Hirakud Evaluation Committee set up in 1962 found out that the power potential of Hirakud is 120 MW. instead of 130 MW.

Thus the real break-through in the field of power development was achieved with the establishment of two Hydroelectric generating stations at Burla and Chiplima having 270 M.W. installed capacity and the associated tranmission system covering northern and central Orissa. The first unit i.e. unit 3 of Burla Power House was put to commercial operation in December 1956 and till February 1968 when the first unit of Talcher Themal Station was commissioned the Power Houses at Burla and Chiplima remained as the sole source of generation to meet power demand of Northern and Central Orissa.

x x x x

Hirakud is not a carry-over reservoir. Every year it is drawn down to at least the dead storage level of 590′ in the end of June to provide for enough cushion to store the inflow during the rainy season lasting from July to October for flood moderation down stream of Hirakud dam. Thus a year starting from July and ending in next June can be taken as a convenient reference year (water year) for finding the energy generated by the system and other statistical data. The reservoir level varies from 630 ft. R.L. in November to 590 ft R.L. by end of June as a result of which the head of Burla Power House varies from 104 ft to 64′ (average 87′) from November to June and then the head rises from 64′ to 104′ during July to November. The head of Chiplima Power House remains almost constant at 71′. In the table appended the actual performance of the Hirakud Power System from the water year 1960 to 2008 is exhibited. Month wise generation from 1960 - 2008 is attached vide Table No. 7.25.

x x x x

The installation of seventh unit at Burla Power House which has been completed in 1990 has raised the installed capacity by 37.5 M.W. and has provided scope of utilisation of monsoon inflow for generation of secondary power." (Refer: Silver Jubilee Souvenir-1982, Hirakud Dam & Power System by J.M. Das Pg.37-43).

Due to more energy demand and worn out old generating sets some renovation works have been taken up as detailed below :

Burla Unit - I Generating set uprated to 49.5 MW on 31.8.98
II Uprated by GEC to 49.5 MW on 15.8.98
III Renovated to 32.18 MW on 01.01.2006
IV Renovated to 32.18 MW on 01.01.2006

Chiplima-Unit I & II Renovated by L & T & BHEL on 29.7.98

The details giving commissioning dates etc. are furnished in Table No.3.24.

Table 3.24 Commissioning Dates of the Power Plant Units

Commissioning dates of the power plan units are furnished below :

Sl. No.	Item	Name of Power station	Commissioning dates	Renovation
	Stage -I			
1.	37.5 MW hydro generating set (Unit I) Make : English,Electric,UK	Burla	11.5.58	31.8.98 uprated to 49.5 MW by GEC Alstom, UK
2.	37.5 MW hydro generating set (Unit II) Make: English Electric U.K.	Burla	11.11.57	15.8.98 Uprated to 49.5 MW by GEC, Alstom, UK
3.	24 MW hydro generating set (Unit III) Make : JM,Voith, Germany/Siemens	Burla	19.12.56	Renovated to 32.18 MW on 01.01.2006
4.	24 MW hydro generating set (Unit IV) Make : J.M. Voith, Germany/Siemens	Burla	13.5.57	Renovated to 32.18 MW on 01.01.2006
	Stage-II			
5.	37.5 MW hydro generating set (Unit V) Make : Hitachi, Japan	Burla	19.4.62	-
6.	37.5 MW hydro Generating set (Unit VI) Make:Hitachi, Japan	Burla	5.8.63	-
1.	24.MW Hydro generating set (Unit I) Make:J.M. Voith, Germany/Hitachi	Chiplima	15.7.62	29.7.98, Renovated by L&T and BHEL
2.	24.MW hydro generating set(Unit II) Make: J.M. Voith, Germany/Hitachz	Chiplima	26.11.62	Renovated
3.	24 MW hydro generating set(Unit III) Make: USSR	Chiplima	1.2.64	-
	Stage -III			
4.	37.5 MW hydro generating set(Unit VII) Make:Hitachi, Japan	Burla	13.9.90	-

3.7 Programme & Progress :

Construction Planning or Programme and monitoring progress of construction minimises time over-run and cost over-run of any project. This forms a part of value engineering whose object is to effect economy in cost of construction of a project . In River valley projects of such large magnitude, the problems are complex and manifold. Entire planning, design and execution undergo drastic change after opening of the foundation. But due to the dedication of engineers, planners and thousands of workmen, Hirakud could be completed is a short span of less than 9 yrs (i.e from April 1948 to Jan.1957). Few datas available on programme and progress are briefly described as under:

3.7.1 Main Dam :

The main dam is of 15748 feet (4.8 km) long of which 3840 feet (1170 m) will be of concrete and masonry and the rest of earth.

Progress to end of 1952-53.

Main earth dam has been raised to an aveage level of 555.0 on the left side, to 550' on the Hirakud Island portion, to 535' on Koilarkud Island portion. The total quantity of earthwork done was 12.13. crores cft (3.43 Mcum).

On the left concrete dam the 4 blocks of spillway has been raised upto sill level and the rest 4 feet below sill level. Bucket has ben completed. The left transition and cut-off wall have reached a level of 560 feet whereas the right transition and cut off wall reached a level of 540 feet.

On the right concrete dam foundation in blocks 34 to 38, 40 to 43 of spillway some parts of the foundations in the blocks have been covered with concrete. In the remaining portions of the above blocks foundations yet require completion. Upstream training wall of left transition has been raised to RL 505′ and 510′. Left cut-off wall was raised to RL 522′. Blocks 56,57 of right transition have been raised to RL 515′ and block 58 to R.L. 513 ft.

Year 1953-54

It is proposed to raise the Earth wok level on the main Dam this year to R.L. 570′ on the left earth dam, to R.L. 569′ on Hirakud Island portion, and to R.L. 550 on the Koliarkud Island portion. The quantity of Earthwork done on the Main Dam is 9.4 crores cft (2.66 Mcum).

On the left concrete dam 12 blocks of spillway are to be raised to RL 538.5ft, and the rest to R.L. 547' to 567'. Both left Transition and Right Transition are to reach RL 570'. On the right concrete dam it is proposed to raise 4 blocks of spillway to R.L. 538.5′ and 9 blocks to R.L. 508ft. The left and right transition will reach an average level of 545. Five blocks of power dam will be taken up to a level of 515' and 2 blocks to 555.0 ft.

It is proposed to construct draft tubes in units 1 to 4. Erection of embedded parts on left will be done upto elevation 538.5ft.

Year 1954-55

The earth dam at the end of this season will be raised to a Level of RL 580' right through upto Koliarkud Island. The total quantity of earthwork to be done is 9.4 crores cft (2.66 Mcum). About half of spilllway portion of left concrete dam is proposed to be raised to RL 579' and the other half of RL. 600'. The two transitions have been scheduled to reach RL 590' at the end of the working season. On the right concrete dam the spillway portion has to reach to R.L. 538.5 ft. The two transitions on either side are scheduled to go upto a level of 570. In case of power dam 4 blocks are to be raised to average RL 570 and the remaining 3 blocks to elevation of 545 ft. Except for scroll cases, superstructures will be completed. On left side main gates will be fixed and on the right side embedded parts will be raised to elevation 538.5ft.

Year 1955-56

The level of the main earth dam will reach a level of 620 ft. at the end of working season. The quantity of earthwork involved in the case of main dam is 10.3 crores Cft(2.92 Mcum). On left concrete dam, the spillway will reach a level of 610', the crest level. The piers for the bridge will be completed. The two transitions will reach final level of RL. 642'. As regards the right concrete dam the spillway portion will be taken to Rl 610 with 2 transitions at RL 620' with power dam at the same Level of 620 ft. Erection of sluice gates will be completed. Erection of main gates on right side and emergency gates on both sides will be completed.

Year 1956-57

The earth work on main dam will be completed to its final level of 642ft, the earthwork to be done during this season being 2.01 crores cft. (0.57Mcum). The left concrete dam bridge portion will be completed thus completing the work on left concrete dam. As regards the right concrete dam, the piers and the bridge to 642' level will be completed in all respects with transitions to final levels.

3.7.2 Dykes :

On both sides of the main dam there will be earthern dykes of length 35500 feet (10820 m) on the right which is in one strip and 32275 feet on the left in 5 strips spanning hilloocks. Except the 2nd gap of left dyke which will be mostly in masonry, the rest of the gaps will be of earth.

Progress to end of 1952-53

The first gap which is of length 5450 ft. and involves 4.65 crores cft.(1.316Mcum) of earthwork and 50.30 lacs cft. (0.14 Mcum) of riprap was in progress. 1.53 crores cft. (0.43 Mcum) of earthwork and 16.43 lacs cft (0.046 Mcum) of rip-rap has been done. The third gap of length 10600 feet (3231m) which involves 1.80 crores cft (0.51 Mcum) of earthwork and 18.93 lacs cft. (53596 cum) of riprap has been completed. The 4th gap of length 12500 ft. (3810 m) and which involves 13.13 crores cft (3.72 Mcum) of earthwork and 29.18 lacs cft (0.082 Mcum) of riprap is in progress and 2.49 crores cft (0.705 Mcum) of earthwork and 19.30 lacs cft (54643 M cum) of riprap has been done. The Gap 5 is of 2000 ft. long and involves 68.89 lacs cft.(0.195 Mcum) of earthwork and 5.5 lacs cft (15572 cum) of riprap out of which 49.90 lacs cft. earth and 2.16 lacs cft of rip rap has been done.

Some work on excavation of foundation of masonry work in Gap No.2 has been done.

The right dyke of length 35500 ft involves 12.31 crores cft (3.485 Mcum) of earthwork and 108.42 lacs cft. (0.307 Mcum) of riprap out which 1.97 crores cft and 18.54 lacs cft has been done respectively, thus completing almost all the work in 0 to 10,000 RD of total length of 35500, ft.

1953-54

In gap No.1, 12 crore cu.ft. of earth work and 10.4 lacs cu.ft. of rip rap are proposed to be done during this year. The remaining earth work of 88 lacs cu.ft. and 15.39 lacs cu.ft of rip rap in Gap 3,4 and 5 will be substantially completed during this year, and whatever work is left out will be completed during 1954-55.

A head regulator which has to be constructed in Gap No.4 for feeding left flow canal(Sason canal) will be started during this year.

Masonry work in gap No.2 will be completed to 1/3 of the height and earth wok will be almost completed. In the right dyke 5.52 crores cu.ft. of earth work and 44.96 lacs cu.ft. of rip rap are proposed to be done during this year, thus completing the work upto RD 20,000 ft.

1954-55

About 120 lacs cu.ft of earth work and 8 lacs cu.ft of rip rap will be done in gap No.1

during this year. In the rest of the gaps all the works will be completed including the construction of head regulator in Gap No.4.

The remaining masonry work in Gap No.2 will be completed during this lyear. On the right dyke 4.79 crores cu.ft. of earth work and 44.92 lacs cu.ft. of rip rap are proposed to be done during this year, thus completing the wok of the right dyke up to RD 35500, ft.

1955-56

30 lacs cft. (0.085 Mcum) of earth work and 8 lacs cft (0.227 lakh cum) of rip rap will be done in Gap No.1.

1956-57

42 lacs cu.ft. of earth work and 7.5 lacs cu.ft. of rip rap will be done on Gap No.1. The work in Gap No.1 will be completed in all respects this year.

3.7.3 Canals

The Irrigation Canal system consists of 2 main flow canals, one on either side and a distibutary off taking direct from the reservoir. The right side flow canal is called the Bargarh Canal. The total length of the main Canal and branches will be 93 miles and that of Distributaries and Minors 412 miles giving a total length of 505 miles of channels. The Bargarh Canal has got 2 big branches called Atabira Branch and Ratmunda Branch and no. of distributary systems. Left side flow canal is called Sason Canal taking off from left Dyke Gap No.4. Another small canal called Sambalpur Canal takes off from Gap No.2.

The above canal system will provide irrigation facilities for 448 lacs acres of land in the Sambalpur and Balangir Patna Districts.

3.7.3.1 Progress up to end of 1952-53

The work on canals was mostly confined to eath work on Bargarh Canal, right side lift canal and their off-take channels. About 70% of excavation of Bargarh Canal has been completed and 80% of excavation of Attabira Branch has also been completed. The excavation of left side flow canal has been done to the extent of 20%. About 45% of the total wok of excavation has been completed.

30 Masonry structures of different kinds have been taken up and are in progress.

3.7.3.2. Programme for 1953-54

During this working season almost all the remaining masonry structues on Bargarh Canal will be taken up and the excavation of uncompleted reaches except approaches to the masonry stuctures will be completed. On left side flow canal and Attabira Branch, about 60% masory structures will be taken up. The excavation of Bargarh, Tail of Attabira and Bhimatikra Distributary and Retamunda Branch will be completed. Excavation of masony structures on Kebar, Barkarlay, Behra, Sukha Distributaries, Sambalpur Canal and Parmanpur Distributary will be started. Due to postponment of lift fanals the work on this will be abandoned.

1954-55

The excavation of masonry structures on Bargarh Canal, Attabira Branch, Left Side flow Canal and Retamunda Branch alongwith some big distributaries will be completed. Work on small distributaries and minors will be started during this season.

1955-56

During this season Bargarh Canal, Left side flow canal, Attabira Branch, Ratmunda Branch along with their distributary systems will be completed in all respects and water will be let into them.

Estimated quantity and cumulative progress upto end of June 1954 is shown in Table 3.25.

Table No.3.25 : Progress upto June 1954

Item of wok	Estimated quantity in cum	Target date of complet-ion	Uptodate progress to end of June'54 in cum.	%of work done
Left concrete Dam				
Concreting	1,23,84,000	June'57	84,04,742	67.9
Masonry	53,19,000	June'57	36,63,122	68.9
Right concrete Dam				
Concreting	1,09,74,000	June'57	38,06,239	
Concreting in Power Dam	13,00,000	-do-	1,71,410	39.5
Masonry	93,84,000	-do-	45,55,868	
Earth Dam				
Earth work	43,24,00,000	-do-	22,48,86,064	52.3
Left Dyke Gap -I				
Earthwork	4,64,75,000	June'56	2,83,44,943	60.9
Left Dyke Gap -III, IV & V				
Earth work	5,62,79,318	June 55	5,07,87,478	90.35
Right Dyke				
Earth work	9,50,35,818	-do-	5,50,34,820	57.9
Main canal and Branches				
Excavation	43,96,85,000	June'56	37,19,75,300	84.83
Distributories and Minors				
Excavation	22,70,00,000	-do-	6,47,92,650	28.54

3.7.4 Electrical Works

3.7.4.1 Production (Electrical)

Total quantity of work involved : Erection of 2 Nos. 24000 KW Generating sets and 2 nos 37500 KW generating sets including transformer and switch gear in outdoor switch yard.

Quantity done upto 30th September, 1953 - Penstocks for set Nos. 3 and 4 have been received at site as also the draft tubes of all 4 sets. Lightening arresters for the station have also been received. Quantity of work to be done in subsequent seasons are briefed as under:

1953-54

Installation of Penstocks for sets 3 & 4 will commence in November 1953 and will be completed for all sets by June 1954. Draft tubes will be installed early 1954 and completed by March 1954. Speed ring and Scrol case for sets 3 & 4 are expected to be received in january 1954 and installed before June 1954. Generator for sets 3 and 4 will also be at site by Januay 1954. Delivery of switch gear will commence in December, 1953 and erection will commence early in 1954. 42 MVA transformers will be received at site in May 1954.

1954-55

Power house cranes are expected at site in July, 1954 and they will be erected from August, to October, 1954. Speed ring and spiral casing of set No.1 will be received about September 1954 and installed before March 1955, and for set No.2 they will be received about February 1955 and erected by June 1955.

Delivery of Turbines of sets 3 & 4 will commence in Septembe 1953 and erection of these will be completed by May 1955. Turbine for set No.1 will be delivered in January 1955 and its erection will be done by June 1955.

Delivery of Generator for set 1 will commence in June 1955 and erection of Generator for sets 3 and 4 will also commence in May 1955. Erection of switch gear will continue, throughout the year and be completed 90% by June 1955. Two 27 MVA transformers will be received at site.

1955-56

Turbine for set No.2 will be delivered in October,1955 and its erection will be complected by April 1956. Erection of Generators for set Nos. 1, 3 and 4 will be completed by September, November and December 1955 respectively. Testing of sets 1, 3 & 4 will also be carried out towards April, 1956.

1956-57

Erection of Generator of set 4 will be done by September, 1956 and the set will be commissioned by November, 1957.

Target date of completion 31.12.1956.

3.7.4.2 Transmission

Total quantity of unit involved :- erection of 90 miles of 66 KV lines, 310 miles of 132 KV lines, 5 major & 2 minor grid sub stations.

Quantity of work done upto 30th September 1953 :-

Detailed route survey of all 132 KV Lines and preliminary survey of about 30 miles 66 KV lines route has been done. Conductors and Insulators for most of the work has been received and balance is being delivered regularly. Building work at 2 major sub-station has been commenced.

Quantity of work to be done in 1953 - 1954

Delivery of 132 KV lines tower material is expected to commence in December, 1953 and erection work will start early in 1954. About 100 miles of 132 KV line and 50% of the buildings work are expected to be completed in September, 1954. Detailed route survey for the rest of the lines is also expected to be completed. Delivery of sub-station equipment is also expected to commence early in 1954 and work on 3 major sub station will commence.

1954-55

Building work will be completed 100% and 150 miles of 132 KV lines erected in this year. Major Sub-stations at Jharsuguda, Rajgangapur and Talcher will be nearing completion. Erection of 66 KV lines will commence towards middle of 1955. Delivery of sub-station equipment will be completed by September, 1955.

1955-56

Erection of all major sub stations and 132 KV lines will be complected by May 1956 and the system tested. Erection of 66 KV lines will be completed by September 1956 and the minor sub-station work will commence.

1956-57

2 Minor sub-station will be completed and energised by December 1956.

Targets and achievements from 1953-54 to 1955-56 are furnished below:

Table 3.26 Target and Achievement between 1953-54 to 1955-56.

(Quantity in Lac. cft)

Item of work	1953-54		1954-55		1955-56	
	Programme	Progress	Programme	Progress	Programme	Progress
Main Dam concrete & Masonry	113.86	134.92	86.99	122.50	31.61	33.27
Earth work	940.00	1052.05	940.00	1196.85	954.00	958.88
E.W. in Dykes	527.19	539.80	372.06	350.07	252.30	262.06
E.W. in canals	763.85	763.85	729.92	729.92	719.01	750.79

(Source : Indian Journal of Power & River Valley Development vol.VI, Hirakud Project No. Pg.123)

It is evident form the above table that the progress of work at Hirakud had in general been much ahead of the schedule. This made it possible to complete stage I of the Project ahead of the schedule date of completion. Year-wise progress achieved for dam, dykes and power house is enclosed vide Drg. No.3.17

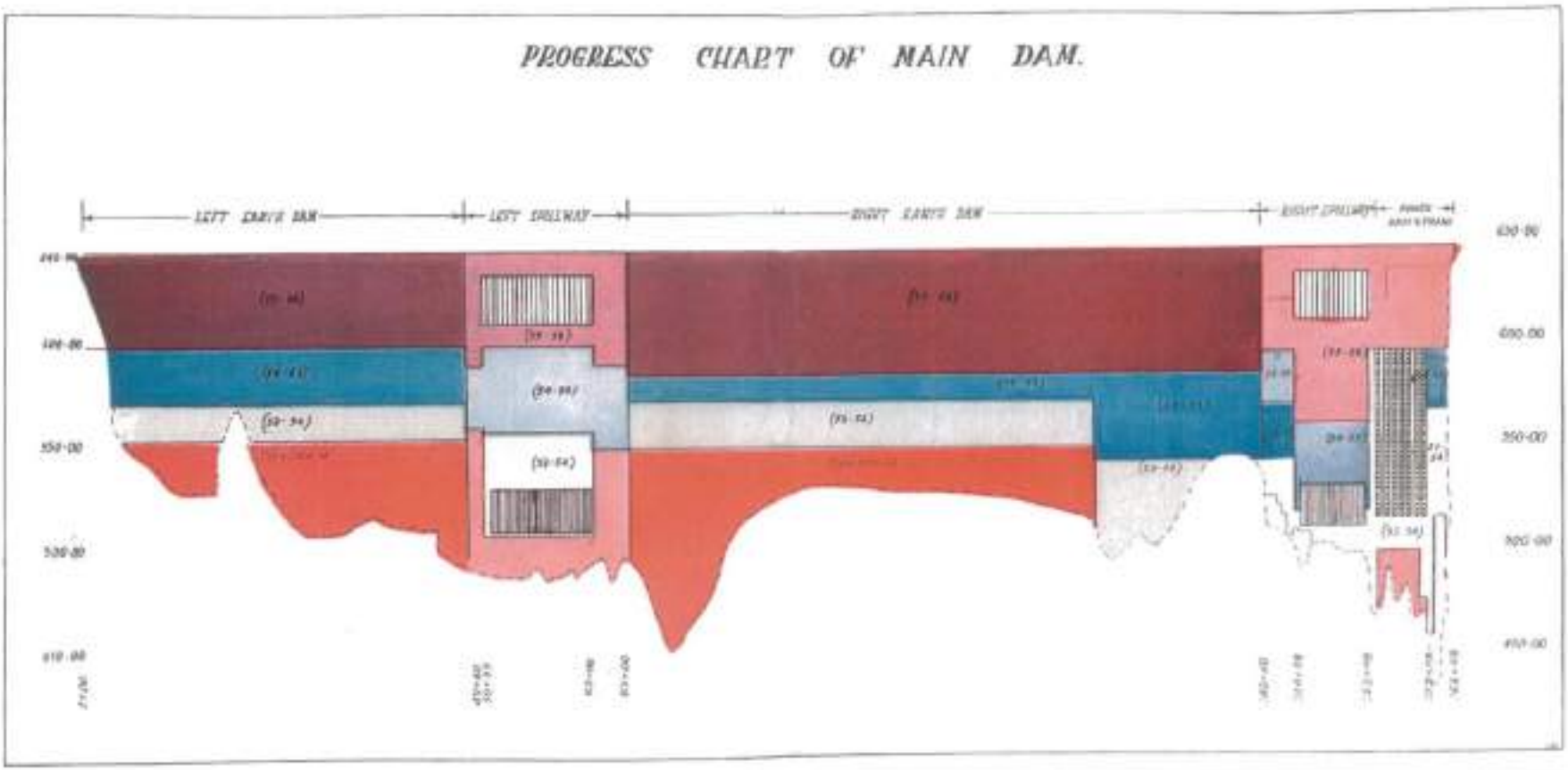
3.8 Field Testing and Quality Control

The statistical data in respect of the Hirakud dam and dykes should serve to convince one of the magnitude of the project.

Maximum height of masonry and concrete dam	: 200ft (60.96 m)
Maximum height of earth dam	: 195ft (59.44 m)
Fill for earth dam and dykes	: 64 crores cft (18.12 Mcum)
Conerete in dam	: 2.27 crores cft. (0.64 Mcum)
Masonry in dam	: 1.49 crores cft. (0.422 Mcum)

This should also serve to emphasize the imprortance of ensuring that the construction conforms to the specifications as infringement of specifications will mean undesirable incursions into the margin of safety, which is a water-storing structure of this magnitude may lead to disaster during its performance, distress calling for otherwise avoidable protective measures. Inspection, control and necessary precautions at each step during the construction insure against such frightening possibilities. Field control of the quality derives its importance also from the fact that its efficiency reflects in the economy of the design and construction. If the field control can ensure close tolerances and narrow deviations from the optimum the designer of the strutcture as well as the designer of the material like concrete with which the structure is built need allow for margin of safety less than would otherwise be necessary. In the construction of not so important structures it has been the practice so far to make the construction crew themselves responsible for the quality of the work also. In the construction of the Hirakud dam and its ancillary works testing and inspection for quality constituted a specialised function and the laboratory (the Hirakud Research station) served this functional need, in conjunction with the construction crew. It is said "A test is worth thousand expert's opinion."

The Hirakud Research Station which was responsible for the quality control had, as its assignments, investigations on borrow areas for the earth dam and dykes, determination of the types and properties of soil available prepartory to design of the earth fill section, investigations for the


PROGRESS CHART OF MAIN DAM.
LEFT EARTH DAM
LEFT SPILLWAY
RIGHT EARTH DAM
RIGHT SPILLWAY
600.00
550.00
500.00
450.00

Drg. No. 3.17

sources of rubble and aggregate for masonry and concrete, design studies on concrete for the various locations of the dam and other structures and the investigation on the silt load in the Mahanadi also. What follows here, however, restrict itself to Field Testing and Quality Control and furnishes an account of the service performed as well as the mode in which it was performed.

The Organization

The Laboratory Organization reported directly to the Chief engineer and was directly under the Chief engineer's administrative control. This arrangement made to a great degree for the efficiency and the independence in the work of the laboratory in its day- to-day testing and request of the construction engineers to effect whatever rectifications were in the laboratory's view necessary. In cases of differences of opinion which fortunately were very few, the matter was finally decided by the Chief engineer.

The laboratory was headed by an officer of the rank of an Executive Engineer designated Deputy Director, the officers in charge of the various sections of the Laboratory were designated Research Officers or A.R.O. and were of the rank of Assistant Executive Engineers or Assistant Engineers. Each section had several Reseach Assistants and Analysts working mainly in the laboratory, Inspectors and Assistant Inspectors for field work. The Personnel Chart shows the organization pattern.

One feature of the Laboratory organization deserves special mention. Field inspection and testing procedures having been standardised, it was not particulary necessary to have all the ranks of the organization manned by personnel with engineering qualifications. At any given time, only one or two of the personnel at or near the top was or were with engineering qualificators. This arrnagement helped release men with engineering qualifications for other jobs where they were essential.

3.8.1 Inspection and Field Testing

3.8.1.1 Concrete

Realising that the quality of a structure depends as much on the quality of concrete as on the quality of workmanship, inspection was organised at each stage, from the production of concrete to its final placement. Such inspection comprised :

(i) examination, field testing and acceptance of aggregates and cement.

(ii) control of proportioning and batching of materials and mixing of concrete.

(iii) examination of the adequacy of the clean-up of forms foundation or base concrete prior to placing concrete, and

(iv) sampling and testing concrete and preparation of test specimens : field tests included workability tests, unit weight and air content determination.

By observing the concrete at the plant as well as at the site and after performing necessary tests, the inspectors made any minor changes in the batching, necessitated by factors like temporary variance in the grading of aggregates or the variation in the free moisture present in them. Necessity for comparatively major changes in the mix was brought to the notice of the officers of the laboratory, who as a routine inspected the worksites and decided on necessary changes.

A report on the day's work which included particulars regarding the types of materials used, the production of concrete and its placing, improvements necessary and defects to be rectified was submitted to the Chief Engineer; this report was routed through the construction engineers so that they could furnish their comments for the Chief Engineer to issue instruction whenever necessary.

3.8.1.2 Masonry :

Masonry construction being a less mechanised operation than concrete, the work of the inspectors consisted principally of checking the quality of rubble, sand and cement used and adjustment

of the batching of the cement and sand for mortar. The inspectors particularly sought to avoid rubble with excessively smooth surfaces or skins, excessively fine or coarse sand and sand with high percentages of foreign or undesirable impurities. Batching of materials for mortar being on a volumetric basis, they determined each day and more often when necessary the bulking of the sand used and adjusted the batching suitably. They helped the construction crew in their supervision particularly to ensure avoidance by the masons of excessive use of mortar, use of flat or unduly small stones in large quantities or the tendency to dump or pile up stones on the course of masonry which had just been built and had not had time to harden to the required degree. Sampling the mortar used, making test specimens for strength tests and submission of a report on the day's masonry work, formed part of the inspectors' work.

3.8.1.3 Earth Fill :

The inspectors assigned to the earth dam reassigned amongst the borrow area, the fill and the field laboratory. The borrow area inspectors saw to it that the excavation was restricted to approved locations, routed different types of soil to their destination and collected samples representing the types of soil actually used in the fill. The embankment inspectors ensured that dumping was done only on locations previously passed, that dumping was restricted to the quantity necessary for one layer of specified thickness over the area being dumped, that necessary moisture was added if it had not already been done in the borrow area and that the layer, after proper spreading was rolled with approved types of roller. After the layer was rolled the inspector took density samples and determined the in-situ density of samples and determined the in-place density of the soil in the compacted fill and passed the area for further dumping or required further rolling to be done, depending on the degree of compaction achieved in relation to the laboratory test compaction. Records were kept of the location of the day's dumping the borrow area supplying the soil, the details of sampling done in the rolled fill and the degree of compaction achieved.

The routine functions of field inspection on the earth dam detailed above constituted only a part of the work devolving on the laboratory, for there were more interesting and more responsible assignments also. One such was the determination of the depth to which the cut-off trench should be taken for the main earth dam at its junction with the fill at the left abutment. Field permeability tests as well as permeability tests on undisturbed samples from the various depths were carried out before the final profile of the trench was specified. What made for an interesting problem was the presence of a tunnel on the abutment, parallel to the axis of the dam, located in line with the main cut-off of the fill in the immediate vicinity. After weighing all factors involved not excluding cost, the tunnel was filled wth fat clay tamped in sloping layers with pneumatic tampers. Grout pipes were left in, in case further grouting is indicated to be necessary due to the consolidation of the fill.

For carrying out the necessary tests in the field, incidental to quality control on the job, the main laboratory i.e., the Hirakud Research Station, maintained as many as were necessary of smaller field laboratories suitably equipped. The field laboratories were required to attend to such tests as were necessarily to be carried out at site and would involve no more than routine test procedures. When the interests of the job demanded, impromptu laboratories were set up at site to meet specially urgent demands. During construction, one could often see right on any important job a tent which served as the control laboratory. For the left and right concrete dams, there was one field conctrete laboratory at each site, with this difference that while the Left dam field laboratory being near the main laboratory had most of its testing carried out there, the Right dam field laboratory was more fully equipped as it was felt desirable to have as many tests as possible carried out there itself in view of the transport difficulties involved. The right dam field laboratory had its own 200 ton Compression Testing Machine, capable of testing specimens upto 12" height. These field conerete laboratories, established at the commencement of work in the concrete and masonry dam continued

till the end. The Earth Dam field laboratories were more in number at any one time, as many as five or six functioned, including three at the dykes. Each of these field laboratories, concrete as well as earth dam, was in charge of one of the inspectors, designated protem Inspector-in-charge.

Inspectors in the field were guided by the instruction manuals covering inspection of Concrete and earth dam issued by the Hirakud Research Station.

This is not out of place to record here one of the contributions of the Hirakud Research Station to the production and placement of improved concrete and use of improved mortar. The improvement referred to here is the improvement in the workability of concerte and mortar, resulting from the use of air entraining admixtures. Having proved the advantages of the use of air-entraining admixtures in concrete paricularly with the type of aggregates available at Hirakud, the laboratory initially decided on the use of one of the imported commercially available admixture. In course of time, it went further and succeeded in developing from indigenous raw materials an acceptable air entraining agent which is now known as 'Aerosin-HRS. Tests both in India and abroad proved the efficiency of 'Aerosin-HRS'. The use of the admixture was extended to masonry work also, where tests showed that the quality of the mortar used could be considerably improved. For those who would be sceptical unless precedents are pointed out, it should suffice to record that the entire quantity of conerete totalling 20 lakhs cft (56625 cum) placed in th Hirakud Dam and 10 lakhs cft (28312 cum) of masonry built during 1955-56 season had in them the admixture of 'Aerosin HRS.'

3.8.1.4 Statistical Data

The following information is of interest to those who are statistically analytical minded :

Concrete and mansonry dam :

No. of composite cement samples tested	:	3085
Number of tests on aggregates and sand	:	4944
Quantity of concrete placed	:	2.27 crores cft (0.643 Mcum)
Number of concrete cylinders tested	:	25980
Quantity of masonry in the dam	:	1.49 crores cft (0.422 Mcum)
Number of mortar cylinders tested	:	5646

Each Dam and Dykes

Quantity of earthfill	:	64 crores cft. (18.12 Mcum)
Number of field density tests	:	59,100
Number of field compaction tests	:	2,300

Generally, in the case of concrete, the scale of sampling is such that at least one set of specimens was tested for the 28 day strength out of every 400 cyds of concrete placed. The specimens were tested at the ages of 7 days, 28 days, 90 days, 6 months and 1 year. A regular testing schedule was followed according to which out of every 36 specimens moulded, 9 were tested at 7 days 18 at 28 days and 3 each at 90 days, 6 months and 1 year.

The test specimens were 6" x 12" cylinders in the case of concrete and 2" x 4" cylinders in the case of mortar.

In the case of earth dam, the scale of sampling for testing adequacy of compaction was of the order of one for every 10,000 cft. (283 cum) of fill.

The contribution of the Hirakud Research Station and its field laboratories to the construction of the Hirakud Dam and its allied works might not appear spectacular; none the less it was valuable. The laboratory derived its strength from the continued support of the Chief Engineer Shri M.S. Thirumale Iyengar who upheld the reasonable objections and sugggestions emanating from the

laboratory, even thought such objections and suggestions often had repercussions on the construction schedule. The team work of the personnel of the laboratory contributed a great deal to the laboratory's usefulness to the Project. Tests conducted at Hirakud Research Station are shown in Table 3.27.

Personnel chart of Hirakud Research Station

Deputy Director

Staff	Soil Section	Concrete Section	Silt section	chemical section
Research officer	1	1	-	-
Asst.Research officer	1	1	-	-
Research Asst.	7	6	2	2
Analysts	5	4	2	-
Inspectors	10	6	-	-
Assistant Inspectors	14	6	-	-
Laboratory Assistants	2	-	1	1

The chart represents the strength of the Station during the peak Period of work in the field

(Source : Field Testing & Quality Control by S. Balasubrahmanyan - Indian Journal of Power & River Valley Development Vol VI no.10 pg. 71-73)

Table 3.27 Details of Tests conducted on Earth, Concrete & masonry at Hirakud Research Station, Hirakud

Sl.	Description	Qty in Cft.	Qty. in Cum.
1	2	3	4
	Right Dyke		
1.	Earth work in embankment (compacted)	9,73,60,000	27,56,512
2.	Compacting of earthwork	11,68,32,000	33,07,814
3.	Earthwork in excavation	1.01,11,000	2,86,268
4.	Rubble packing	1,08,74,000	3,07,871
5.	Graded filter	20,50,000	58,041
6.	Sand spreading	18,33,000	51,897
7.	Parapet	70,000	1,982
	Left Dyke Gap 1		
1.	Earthwork in embankment	4,72,30,000	13,37,203
2.	Graded filter	27,62,000	78,200
3.	Dumping & packing	50,68,000	1,43,488
	Left Dyke Gap III		
1.	Earth work in excavation	17603000	4,98,386
2.	Compacting of earthwork	21491000	6,08,465
3.	Rubble packing	1715000	48,556
4.	Spreading graded filter	5,23,000	14,807

Contd...

1	2	3	4
	Left Dyke Gap IV		
1.	Earthwork in excavation	2,79,48,000	7,91,280
2.	Compaction of earthwork	3,35,38,000	9,49,547
3.	Earthwork in stripping	22,15,000	62,712
4.	Rubble packing	26,00,000	73,613
5.	Spreading graded filter	7,41,000	20,980
6.	Sand spreading	2,06,000	5,832
	Left Dyke Gap V		
1.	Earthwork in excavation	74,49,000	2,10,900
2.	Compaction of earthwork	89,35,000	2,52,973
3.	Earthwork in stripping	88,5000	25,057
4.	Rubble packing	5,37,000	15,204
5.	Spreading graded filter	145000	4,105
6.	Sand spreading	59000	1,670
	Bargarh Canal Head Regulator		
1.	Cement Concrete 1 : 4 : 8 in foundation	1,42,000	4,020
2.	R.C. Masonry in superstructure	1,73,000	4,898
3.	Cement Concrete 1 : 2 : 4 in foundation	25,000	708
4.	Earthwork in foundation	22,20,000	62,854
5.	R.C.C. work	46,000	1,302
6.	C.C. 1 : 3 : 6 in arching	60,000	1,699
7.	Coursed rubble work	71,000	2,010
8.	C.C. 1 : 5 : 10 in hunch filling	56,000	1,586
9.	Mild Steel reinforcement (lbs)	2,40,000	-
	Head Regulator at Left Dyke IV (Sason canal)		
1.	Earthwork in excaration and filling	3,45,000	9,768
2.	C.R.M. including archwork and chisel dressing	57,000	1,614
3.	R.C.C. work	8,000	227
4.	Cement concrete	62,000	1,755
5.	R.R. masonry	24,000	680
	Left dyke gap II & Head Regulator		
1.	Cement concrete	17,000	481
2.	R.R. masonry in cement mortar	2,30,000	6,512
3.	R.C.C. work	11,000	311
4.	C.R. masonry	47 700	1,350

3.9 Organisation, Labour Welfare and Martyrs

3.9.1 Organisation

A Chief Engineer is incharge of the Project Organisation and is directly responsible to the Government of India, through the Hirakud Control Board, for both the efficient administration and general professional control of public works on the Project. He is delegated with powers through the Hirakud Control Board to contract for works, supplies and services on behalf of th Government of India. In his day-to-day administration, he is assisted by one senior Personal Assistant (Technical) and one Administrative Officer (Non-Technical).

There is a Control Board with the Chief Minister of Orissa as Chairman which is in overall charge of the Project, whose resolutions and decisions on administrative matters are recommendatory, subject to the final orders of the Government of India in the Ministry of Irrigation and Power, as it is an agency to execute the Project on behalf of the Orissa Government.

The adminstrative set-up is centred round the Chief Engineer with great powers and responsibilities. This pattern of organisation has enabled the Chief Engineer to function with greater freedom and to take quick adminstrative decisions. He further delegated his decision making powers to the Superintending Engineers in charge of various circles and thus helped in the quick disposal.

The other imortant adminstrative features which substantially contributed to the speeding up of the works are :

(i) Establishment of a Design office attached to the Mahanadi Canal Circle (ii) Central organisation for all purchases under the control of C.E., (iii) Daily field maintenance of equipements reducing breakdown to a minimum (iv) refuelling the machines near the work sites and (v) increased attention to labour welfare.

"The Chief Engineer is assisted by 4 Superintending Engineers (Superintending Engineers, Hirakud Dam Circle, Mahanadi Canal Circle, Development Circle and Superintending Engineer, Electrial Circle) who are administrative heads of circles. They are responsible to the Chief Engineer for administration and general control of public works in charge of the officers of the department within their circles. Each of the Superintending Engineers has got one Personal Assistant in the rank of Executive Engineer or Assistant Engineer to assist him in the discharge of his duties.

Under the Superintending Engineers there are 17 Executive Engineers (i.e. 5 under Hirakud Dam Circle, 3 under Mahanadi canal circle, 5 under Electrical CIrcle and 4 under Development Circle) who are responsible for the execution and management of works in their divisions. Under the Executive Engineers, there are Sub-divisional Officers who are incharge of Sub-Divisions and are responsible for management and execution of work in their jurisdiction."

(Source : Indian Journal of Power & River Valley Development - Hirakud Dam Project Number vol.IV pg.79-80).

3.9.2. Labour Welfare

The multi-purpose projects, at once require a massive workforce, both skilled and unskilled of various trades and abilities. The mobilisation of such huge work force with varied skills being beyond the resources of a single state, they have to be recruited from length and breadth of the country and at times, beyond it. Further the nature of work being seasonal, it entails the liablilty of seasonal employement, subsequant retrenchment and re-employment. Again, there are two sets of rules governing the conditions of employment by the Government and by the contractors.

The employment position during the peak seasons of 1955 & 1956 were as follows :

Period	Agency	Unskilled	Skilled	Total nos.
March 1955	Departmental	6152	4566	10718
	contractors	24014	3102	27116
March 1956	Departmental	9037	7768	16805
	contractors	23703	2986	26689

Wages paid in the Project was very favourable, as the same was much higher with those paid by the State Government. At Project sites, unskilled male workers were earning @ Rupee one & eight anna and female Rupee one per day where as their counterparts in the State were getting twelve anna (Rs. 0.75) and eight anna (Rs 0.50) respectively per day. There are instances where female mazdoors have earned Rs.0.75 to Rs. 2.00 per day while working on piece wage-system under contractors.

The working environment was in open space except in factories. Rest sheds were provided where ever possible. Adequate drinking water was supplied. During hot summer when temperature rises to 120^0 F ice-cooled water in certain sections was made available and in winter tea was given freely. First Aid posts were placed about 100 yds. away. Toilets, where necessary, were constructed. Work sites were provided with flood lights for night work. Besides there were two self contained hospitals at Burla and Hirakud with two branch dispensaries at Chiplima and Bargarh. There was also a High school inclusive of primary classes at Burla. In addition to this, there were six Welfare Centres, creches, training facilities for workmen, marketing facilities and many other amenities to bring labour and management to a close proximity.

The working operations being hazardous viz. blasting operation in quarries, automobiles working at high altitudes, accidents were large in number. In 1955, 418 cases of accidents occured, out of which 34 proved fatal against average working strength of 30,000. As a measure of prevention, there were safety engineers both on electrical and civil side. The welfare measures taken by the project, resulted in stupendous progress in completion of the dam much ahead of the scheduled date of completion. Implementation of minimum wages Act, fair wage clause, Workmen compensation Act and Contractors labour regulation as per CPWD norms were rigidly adhered to. These ensured fair play and justice to the large number of workers employed under contractors.

3.9.3. Martyrs during Project construction

Workmen were from all parts of country. As mentioned in the previous paragraph due to hazardous nature of work, during construction 128 workmen dedicated their lives for the project Names of martyrs are given in the Annexure No 3.6.

3.9.4 Inauguration of the Project :

1st batch of concrete was laid in the dam foundation by Pt. Jawaharlar Nehru, the first Prime Minister of India on 13th April 1948 who coincidentally inaugurated the project on Jan 13, 1957. During inauguration he said, *"I dedicate this magnificent project to the well-being and prosperity of the people of Orissa."* He further added that *"Orissa a land of temples had now a new temple and in it a God for the whole country."* On this occasion the report of the then chief Engineer Sri Thirumale Iyenganr is enclosed vide Annexure 3.7.

3.9.5 Chief architects associated with Construction

History remains incomplete unless it tells of the people who have made history.

Names of Chief Engineers Superintending Engineers and Executive Engineers who served during construction of this gigantic Project are furnished in Annexure 3.8.



Martyrs Pillar



Gate from Burla Side

Annexure 3.6

Martyrs of Hirakud Dam Project

Andhra

Nakul Gour
Nakula
Uchhal Gour

Bengal

K.P. Choudhury
N.C. Dass
P.K. Chakrabarthy

Bihar

Dwarka Singh
Iteari
Mohan Prusty
Susena

Madhya Pradesh

Chilhar Satnam
Devraj Dass
Dhanu
Kali Ram
Khila Ram
Lalji
Marathi Ram
Tej Ram
Gayarthi
Ram Bai

Madras

A.K. Bhanu
A. Ettiappan
Appukuttan Nair
Bhakat Nag
Embichi Baba
Ettiappan
Govindan Nair
Jataya
John
Kanabayya
Lehur
Manuel
Manibum
P. John
P. Ponnuswamy
Vallayan
Venkayya
Amruti
Antony
M. Remi
Palani Ammal
Palliamma
Ponniammal

Nepal

Bhim Bahadur Chetty
Gangaman Sarma

Orissa

Adikanda Panda
Ailnath Pradhan
Amin
Arjun Bharita
Baia Jena
Behari Gonda
Bhima
Bibhuti
Binayak Padhi
Budwa
Budhi Sahoo
Chandra
Chakru
Chudamani Bag
Dhanu
Dhukhu
Diwakar Dass
Gangadhar Barik
Govalla Nalliah
Hari
Jagan Nath Gouda
Jataya
Jerriah Munda
Jogeswar
Kabtu Boi
Kaibal Nayak
Kali Charan
Kali Behera
Kanda Maharana
Karunakar Panda
Ketan Majhi
Khalia Behera
Krishna Sahu
Lachman Behera
Laxmaya
Mukta Patil
Nirupa Roy
Pandas Munda
Pandit
Paswatham Biswal
Payaya Panda
Phaga Uram
R.K Dass
Raghu
Raghunath Pradhan
Rama Sethi
Ramalal Sona
Satyabadi Barik
Shama Goud
Shiv Prakash
Sohera
Sudarshan Chakravarthy

Tandal Pradhan	Subhmati	Narah
Tirtha Swain	Sumitra	Nasib Singh
Udayanath Pradhan	Uma	Puran Singh
Annapuma	**Punjab**	Sadhoo Singh
Balamdin	A.N. Sharma	Shinghee
Jayabadi	Awat Ran	Sohan Lal
Jenia Juliakiran	Balwant Singh	**Sind**
Jharia	Dari Singh	Sardar Singh
Nirmal Munda	Jaspal	Trilok Chand
Panna Gundani	Kikusema	
Pennamasy	Lachman	
Rani	Nahar Singh	

N: B : Total numbers of Martyrs = 128

Annexure 3.7

Chief Engineer Sri Thirumale Iyengar's Report on the occasion of the Inauguration on 13th January, 1957

Prime Minister Sir!

Ladies and Gentlemen!

The team of engineers and builders of the Hirakud Dam Project are happy to welcome to Hirakud all, that have assembled here. We feel greatly honoured by your presence and more especially are grateful to our Prime Minister Shri Jawaharlal Nehru who has found some time to be amidst us to dedicate the Hirakud reservoir to the service of the Nation.

I believe, you are all aware that this Project has acquired a considerable reputation in this country and abroad, as the largest of its kind that has been completed substantially in the First Five Year Period of India's Plans. We hope you will take pleasure and find interest in visiting, in their final form, the varied structures of this dam and Project of which you may have read and whose progress you may have followed in the various technical and non-technical journals and newspapers. It will, however, be useful and interesting to you to know something of the history of this Project construction. I will not weary you with a long-winded account, but will just make a passing mention of certain important landmarks.

It is common knowledge that years of serious consideration to the necessity for devising measures that would protect large populated areas of the Mahanadi Valley and the delta from the ravages of floods, the construction of the large reservoirs across the main river was decided on as the best remedy for flood control. The Hirakud reservoir is the first of a series contemplated.

This great project was launched in 1948 through the combined efforts of the Central Waterways, Irrigation and Navigation Commission then styled as "CWINC" and the Government of Orissa. While the "CWINC" assured the engineering feasibility, the State Government assured the future administrative and economic possibilities.

The design and specifications for the main dam and power station were drawn up by the Central Design Organisation of the Central Water and Power Commission (CWPC). Some details of work on the main dam and the designs and specifications for most of the works in the canal

systems were drawn up by a separate Designs Organisation under the Chief Engineer's direction at Hirakud itself.

As the name indicates, the Hirakud Dam centres on and around the 'Hirakud Island' in the Mahanadi river. Here the river was flowing in four channels between two hills– three miles apart. The dam is essentially an earthen barrier across the main river with the requisite sections of concrete and masonry for the spillways, transitions, power, penstock and turbine locations, flanked by earthen dykes nearly 13 miles long in five sections spanning the hills. Thus it is a 16 mile long dam, 200 ft. high at the deepest earth and masonry sections, and perhaps the longest dam in the world. The volume of earth that has gone into the main dam and dykes is by far the largest that has ever been put in any single dam in India. The earth work at Hirakud exceeds the sum total of earth in Lower Bhavani, Gangapur, Maithon and Panchet hill dams which are the four large earth dams built in India in recent times. Even when compared with the other earth dams in the world, Hirakud ranks within the first ten. It is also the largest reservoir in its class for combined power, irrigation and flood control potential. It stores nearly 7 million acre feet of water, spreads over 288 sq. miles, which is more than four times the water spread of the great Bhakra lake, will have an installed capacity of power units that will give on annual production of one thousand two hundred million Kilo-watt hours besides irrigation over two million acres of land.

On the technical side, there are a few peculiar and noteworthy features incorported in this dam that deserve special mention. There is battery of 64 gated bed level sluices of size 12′-0″ x 20′-4″ each, in spillway length of 2160' operated from a gallery of size 11′0″ x 23′-0″ in the belly of the concrete structure. It is also capped by 34 radial crest gates of size 51′- 0″ x 20′- 0″ each. This is the biggest spill and sluiceway that has been constructed in any dam in the world. This operation gallery and the battery of that size sluices are said to be the largest of their kind in the world. These sluices can pass down nearly nine lakhs of cusec under full reservoir conditions. Aswan Dam in Egypt has a larger number of low level sluices. But they are much smaller in size and operate under a varying low head of 22' to 45' only. The Hirakud bed level sluices operate under a maximum head of 120'. These sluices and the crest gates together can pass down a colossal discharge of a million and half cusecs in a contingency of an emergency. The seven power penstock pipes embedded in the masonry section of the dam are of 25' diameter each. Some of the huge bulkhead gates of size 41′4″ x 24′- 6″ at the entrance to these sluices were manufactured at the Hirakud workshops. Not only these, the workshop at site manufactured six sets of draft tube steel stoplog gates of size 25′-6″ x 18′-1″. It also fabricated some high voltage transmission line towers.

Since a simple inspection of the structures cannot of themselves provide a true picture of what they have gone through, I will just touch very briefly on some of the important features.

One of the most outstanding facets of the present age is the rapidity with which engineering methods change, due to the scientific progress and the new possibilities offered by modern technology that have profoundly altered the design and construction methods. The effects of this evolution are apparent both in regard to the layout of construction works and in the realm of construction equipment. The economic aspect of the dam, therefore, depended on the quality and efficiency of the construction planning and equipment use. The size of the dam was undoubtedly the deciding factor that dictated the use of heavy mechanical equipment of pouring concrete, for dealing with earthwork and excavations, for the production of broken stone and the use of concrete handling plants.

The construction operations were in the beginning, planned in favour of comparatively intensive mechanised techniques. But technological considertions for intensive mechanised techniques assumes importance and significance only if the labour intensive techniques are technically unsound and unsuitable. The latter not being the case in regard to certain types of construction materials,

particular attention was gradually paid to maximizing the employment potential in order to meet the employment problem in the context of an economy with relative abundance of labour. On the Hirakud work, the traditional labour intensive techniques were, therefore, put on a more scientific and efficient basis to meet the challenge of the machine intensive technique in the construction of certain parts of structure, which resulted not only in economy but also in providing maximum employment to the skilled and unskilled labour. One such change is the substitution of rubble stone masonry for concrete in the power dam and transitions.

An idea of the vast labour employment potential that this project possessed could be had from the fact that during the periods of peak construction every year during the last three years, the total number of labour employed was well over 30,000. This consisted of various types of artisans-masons, stone dressers, fitters, riggers, carpenters and several other categories of skilled workers apart from an army of unskilled men and women labourers. A harmonious and coordinated balance was kept up between the mechanised and human labour operations through out. During the 'peak' periods we had an awe-inspiring panorama of construction operations. On the intensive mechanised front a vast array and number of machines moved about roaring like some huge pre-historic monsters. Gigantic power shovels dug lifted and disgorged soil into waiting dumpers. Gigantic loaders cut and belt conveyed the soil to pour into the dumper moving alongside. Scraper-loaders, filled their bellies with earth as they scraped the same with blades underneath and sped away roaring to dump and spread the earth on the dam at the intended place. Huge dozers dozed and fleet of sheep-foot rollers rolled to compact the earth – all combined non-stop operation. And then you had also the huge batching and mixing plants linked on one side with a crushing and convyor unit and on the delivery side, a fleet of diesel locos hauling the concrete filled buckets to the site for the lifting and placing cranes to handle.

All these soulless automations, however, produced but fleeting impressions of their performance. The everlasting impression was, however, the grand and thrilling sight of the army of workers, skilled and unskilled men and women several thousand strong drawn from all parts of India from Cape Comorin to the Himalayas, the mass movement of well-organised well-knit groups full of energy and determination – the life and soul of the construction, who were the real builders of this magnificent edifice, toiling night and day with a singleness of purpose and devotion to duty.

Building such a huge water conservation Project, however, was certainly not a simple task. Several years of planning were needed; many hurdles had to be crossed and obstacles on many fronts had to be overcome. Several times the work seemed set for required progress but some unforeseen difficulty or other turned up to upset the plans. From 1953 onwards, however, progress was steady and rapid. The most important factors that brought about this satisfactory state of affairs were an optimum arrangement of the programme of work, alterations in plant lay-out to the extent possible to ensure increased efficiency, the selection of the most advantageous methods of construction, and the shaping and adapting of men, materials and machines to the best advantage for obtaining speed of construction through maximum possible efficiency and as mentioned earlier, the substitution of rubble stone masonry in place of concrete in the power dam and transitions. There was thus practically an uninterrupted speedy progress from September, 1953, to May, 1956, except for one incident early in 1954 which, however, proved to be inconsequential so far as progress was concerned. There was then a labour strike sponsored by anti-social elements some of whom had infiltrated into the "mechanised-earth work equipment operation personnel" organisation. The strike was promptly dealt with and steps were taken to effectively prevent a recurrence of such incidents. With thousands of workers drawn from all parts of India, this magnificent undertaking has been truly national in character. Generally speaking there was comraderie and harmonious relationship between the various people and every worker was conscious of the contribution he or she was making to the common cause.

The erection of tressle bridge on the rear face of the spillway for carrying four huge six ton cranes, the track and concrete hauling equipments, the operation and maintenance there of requiring the greatest skill, courage and self-confidence, were all the work of our young Indian Engineers assisted by skilled artisans-all of Indian origin. Huge cranes were dismantled and re-erected by them. Possessing initiative and resourcefulness, these boys worked like a team and have done a magnificent job creditable to themselves and the Indian engineering profession in the country. Mechanised operations, foundation grouting and other construction techniques that are ordinarily believed to be a foreign technical expert's and technician's job, have all been, with great confidence and courage, successfully tackled by our engineers. It can truly be said that this huge construction at Hirakud has trained and turned out experts in the line who can be trusted to successfully carry out similar difficult jobs elsewhere with confidence and ease. On the electrical side the Indian engineers and technicians have tackled the jobs normally believed to be difficult and foreign in the field of operation. Hundreds of miles of high voltage transmission lines have been erected departmentally. In the erection of huge switch yard and power units and in every phase of construction in this field, our men have shown worth and courage. Thus Hirakud dam and its appurtenant works-the biggest of the completed projects in India which apart from its size has some unique technical features, has been constructed by Indian engineers in all its phases.

I wish also to place on record our grateful appreciation of the help rendered and advice tendered by Shri S. Sundararajan in his capacity as the Financial Adviser and Chief Accounts Officer. Men have come and gone but he has remained on the Project ever since its inception. His long unbroken association with the Project upto this day has been of great assistance in the solution of many financial riddles.

From the American Technical Cooperation Mission, we received assistance in the shape of earthmoving and other equipment to the value of about thirty million dollars for which we are grateful.

As is not unusual on such large constructions, death has taken its toll of lives without any differentiation. To all those large body of builders and workers that are to-day alive upto this day to see their hand-work, as well as to all those that gave up their lives in service and who are not here to-day to see the magnificent achievement, we pay homage and have erected appropriate memorials at the most important locations in the Main Dam.

If I say that we have constructed this dam economically, it sure will raise 'eye-brows' in disbelief. Yet it is true. It may be, there has been some wastage, some money spent on experiments, and some over-spent but this is not unusual in very large constructions and projects costing crores of rupees. Also it may be the estimate of 1947 is nearly doubled. The final estimates of cost of Projects in this class elsewhere even in the most technologically advanced and perfect 'cost accounting' countries, have shown this tendency to double itself-nay even treble and quadruple itself. It must be remembered that when the original estimate was framed, the framers could not anticipate the sharp rise in the prices of materials and machinery and also in wages. It was obviously impossible to make allowance for such contingencies with any degree of accuracy and if the estimate had provided for an extra 100% to cover such cases, no administation would have accepted it. But when the prices of materials and machines rose and the need for undertaking some unforeseen works during execution after exposing foundation and the like became apparent, the estimate had to be revised in 1952-53. I am, however, glad to report that the work has been completed within those estimates.

Some people thought that the Hirakud Project submerged the diamond mines that existed here in some unknown past. They will realise very soon that the completion of the project and the creation of Hirakud lake opens up the real diamond mine, rich in its variety and benefits producing

more food, turning the giant wheels of industry, and mitigating the devastation and destruction of floods that have been the root cause of famine and drudgery in this undeveloped State.

Hirakud opens up the chapter of River Valley Development in Orissa. Similar schemes on the Brahmani, Baitarani, Salandi and other rivers are on the anvil. Hirakud marks the beginning of an era of higher standards of living for the common man in Orissa, free from famines, want, drudgery and disease. It Is a gift to the Nation - a gift by thousands of workers and engineers, by those that left their hearth and home to work for the benefit of their fellowmen by those who have been uprooted from their centuries old homes in the reservoir water-spread area for the benefit of the larger community, and lastly by those that gave up their lives in this service and not lived to see and enjoy the fruits of their labours.

One word more and I am done. You, Sir, laid the foundation of this great Project and are now here to see the completion of the same. It is but in the fitness of things that you, who are the symbol of India the Nation, identifying yourself with the people and struggling hard every moment of your life for their uplift, should inaugurate the era of industrial revolution and economic prosperity of the State of Orissa which bids fair to become one of the brightest India's firmament. .

JAI HIND

Annexure 3.8

Hirakud Construction Engineers

Chief Engineers

Shri R.P. Vasishth	27-5-1948 -- 26-6-1951
Shri Kanwar Sain	27-6-1951 -- 6-6-1953
Shri M.S. Thirumale Iyengar	7-6-1953 to

Superintending Engineers

Shri M.L. Batra	10.8.1948 -- 7-6-1953
Shri M.L. Aggarwal	7-9-1948 -- 30-6-1950
Shri K.L. Vij	25-1-1951 -- 18-8-1952
Shri S.K. Palit	23-3-1951 -- 3-12-1952
Shri Indra Sen	17-7-1951 -- 2-8-1952
Shri A.R. Chellani	20-6-1952
Shri M.G. Hiranandani	23-6-1952 -- 31-10-1953
Shri N. Dharma Rajan	19-8-1952
Shri K. Padmanabha Pillai	10-10-1953 -- 29-11-1955
Shri R.P. Verma	29.12.1953
Shri S.S. Behera	30-11-1955

Deputy Director, Hirakud Research Station

Shri R.C. Hoon	27-5-1948 -- 22-12-1954

Executive Engineers

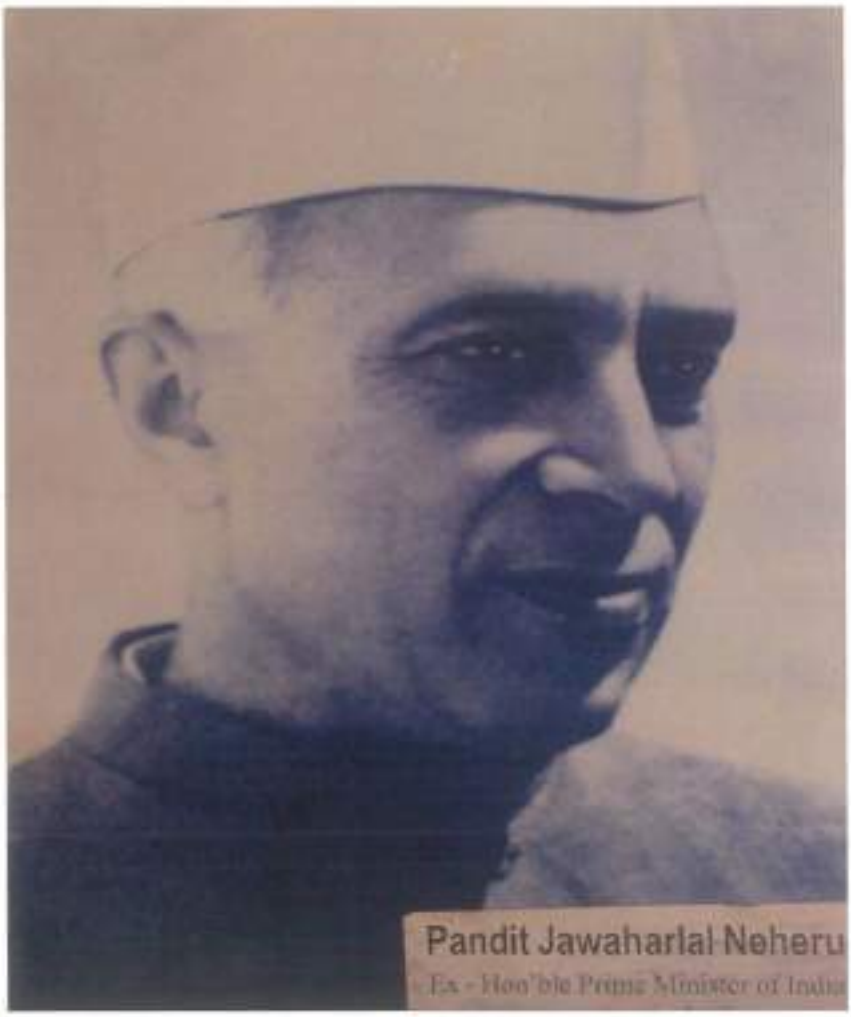
Sarvashri

Achyutam. P	Chellani, A.R.	Hari Singh	Kurpad, A.S.
Agrawal N.K.	Chopra, P.L.	Karnik, R.R.	Mehandiratta, K.R.

Babington, S	Chugh, S.P.	Karve, G.V.	Misra, B.
Balasubramanyam. S.	Dash, B.N.	Kehar, Singh	Misra, K.S.
Behera, B	Goel, J.P.	Keswani, C.K.	Misra, S.S.
Behera, S.S.	Gopaliengar, M.K.	Kewal Kishan	Mohanty, H.K
Chawala, R.N.	Gupta, Govinda	Kohli, B.S.	Murthy, K.S.S.
Murthy M.A.K.I.S.	Pritam Singh	Sahi, B.D.	Teckchandani, T.A.
Nagrani, S.D.	Raghavachari, S.	Sainbhi G.R.	Thomas, K.C.
Naphray, J.S.	Rajgopalachari, R.	Sanyal, B.C.	Thosar, G.S.
Padmanabhan, P.S	Rama Rao, V.	Sarathy, M.B.P.	Tripathy, J.
Palit, S.K.	Ramaswamy, H.	Shankaran, A.	Tripathy, S.C.
Pani, M.C.	Rangaswamy, M.B.	Tata, S.P.R.	Uke, N.G
Papa Rao, G	Rao R.C.	Teckchandani, N.D	Wadhwani, L.T
Patro, P.R.			

3.10 Land Mark Events

*	Reservoirs across Mahanadi the only remedy for Flood Control-opinion by Sir M. Visveswaraya	1937
*	Agreement on unified development of the Mahanadi Valley arrived at between the GoI, GoO and Eastern States	08.11.1945
*	Foundation stone laid by Sir Hawthorne Lewis	15.03.1946
*	Report submitted to Government	June 1947
*	Foundation - 1st batch of concrete laid by Shri Jawaharlal Nehru	13th April 1948
*	Setting up of Hirakud Control Board	March 1952
*	Project Estimate sanctioned by Govt.	May 1954
*	Delta Irrigation Scheme Sanctioned	Sept. 1955
*	Delta works transferred to Orissa P.W.D.	Nov. 1955
*	Second Stage Power Development Scheme (Chiplima Scheme) sanctioned	July 1956
*	Irrigation water let out in the Right Side Main Canal (Bargarh Canal)	7th Sept. 1956
*	Irrigation water let out in the Left Side Main Canal	2nd Oct. 1956
*	First Hydro Power plant commissioned on	19th Dec. 1956
*	Inauguration of the Project by Sri Jawaharlal Nehru, Prime Minister of India	13th Jan. 1957
*	Water let out in Power Channel to Chiplima P.H.	July 1962


Pandit Jawaharlal Neheru
Ex - Hon'ble Prime Minister of India


R.P. VASIST


M.S. THIRUMALE IYENGAR



Aerial View of Hirakud Dam



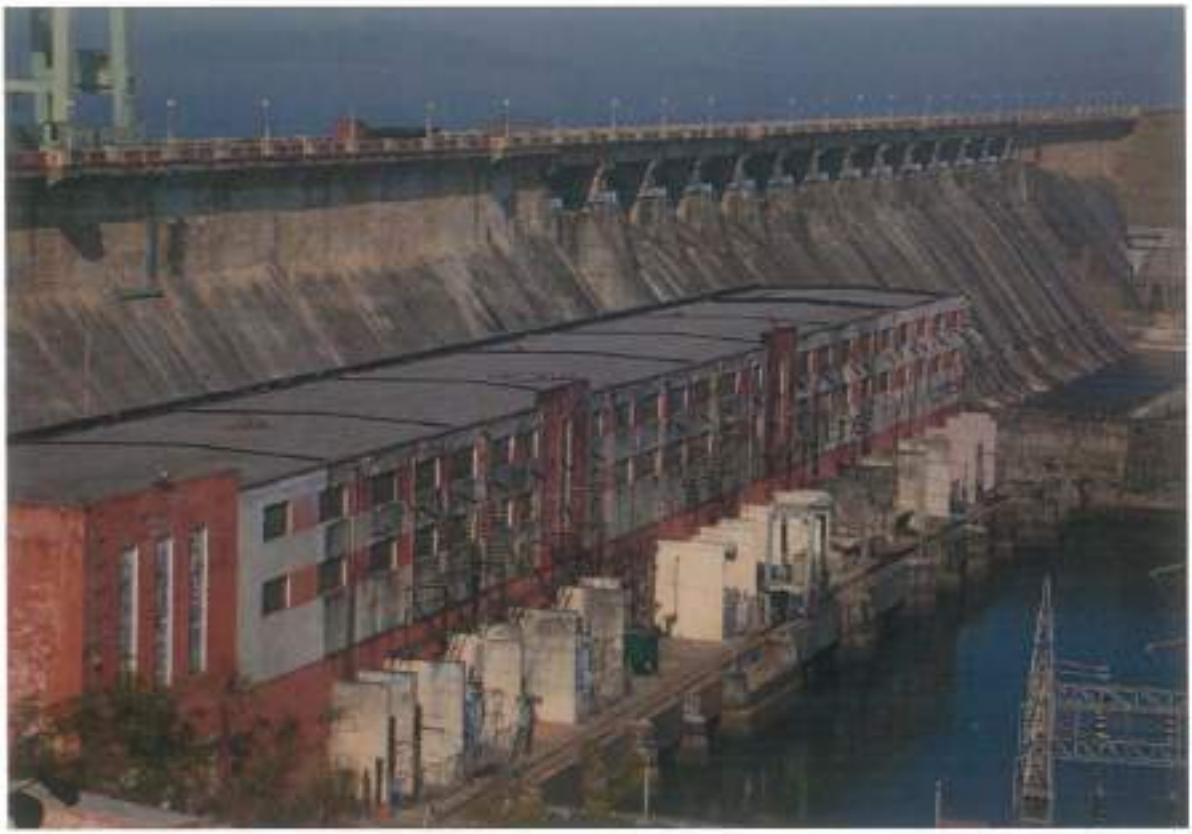
Right Spillway of Hirakud Dam



Head Regulator of Bargarh Main Canal



Head Regulator of Sason Canal



Right Spillway, Power Dam , Power House & Power Channel



Release of Water through Sluices



Bow String Girder over P C Bridge on NH - 6



Inverted Syphon on Left Bank of Power Channel

CHAPTER IV

DEVELOPMENT OF CRACKS IN SPILLWAYS, POWER DAM : CAUSES AND REMEDIAL MEASURES

4.0 Introduction :

The construction of left spillway sluices (40 nos) of Hirakud Dam was completed in June 1954 and that of right spillway sluices (24 nos) in June 1955. Flood water was released through these sluices in the monsoons of 1954 and 1955. Sluice gates were fitted in 1955-56, and the reservoir was filled up to RL 614.20ft in 1956 and upto 627.70 ft in 1957. Since 1958, reservoir is being filled up to FRL 630 ft as far as possible. The flood water is being regulated through the spillways since 1957.

The initial filling of the reservoir since 21.7.1956 was accompanied by appearance of seepage water in a number of places in the dam, conspicuous being in the operation gallery, foundation gallery, power dam and right transition blocks. The seepages were mainly observed through construction joints between lifts of concrete. The seepage in the operation gallery of left spillway was not large whereas in right spillway seepages had been through contraction joints, formed drains in the structure, pockets of concrete and through man hole of under sluices. The seepage through man holes of right spillway was 0.3045 cusec during October 1956. The formed drains located at contraction joints of blocks 34, 35, 37 were giving heavy flow of the order of 60, 75, 48 gpm respectively at R.L. 557 ft (168.38m).

Additional structural groutings were done exclusively in the right spillway and power dam portion from block No.28 to 62, during the period June 1956 to June 1958, the details of which is given vide table No. 3 and 4 respectively of the Appendix- 2 of Manual of Reservoir Operation (1959). In the spillway blocks from 28 to 47,485.5m (1593 ft) of holes were drilled mostly in foundation gallery (R.L. 497) and Operation gallery (R.L.557') and grouted with 5406.5 bags of cement. In the Power dam blocks 48 to 62 the total length drilled was 1230.6m (4037.34ft) and 6186 bags of cement was grouted.

4.1 History of Development of Crack :

In the inspection report of Sri J.Iripathy, Chief Engineer, Irrigation, Orissa on his visit to Hirakud Dam on 1.5.69 it is mentioned that 'Some horizontal cracks have developed in operation gallery of right spillway right from the time of completion of dam i.e.1957-58. The cracks are more or less near the construction joints and appear to be because of hydraulic pressure of the leakage water through the concrete dam. The cracks were grouted and appeared to have been sealed.

The cracks occurred predominantly on the walls of the operation gallery, the sluice gate shafts and the sluice barrels. Cement grouting was done since 1963, for treatment of either new cracks or extension of old cracks as and when they were occurring. Grouting was done periodically in the different zones of the dam vide Table No. 4.1

Table - 4.1 Grouting details from Dec. 63 to Nov.73

Sl. No.	Location	Period of Grouting	Total no. of joints	No. of cement bags used.
1	Sluice gate shafts	12/63 to 6/70	83	97
2	Sluice barrel	4/67 to 11/74	57	98
3	Operation gallery (Upstream wall)	1/66 to 7/69	28	13
4	Operation gallery (Downstream wall)	12/65 to 11/73	97	55

During Dec. 1973 wet patches in downstream wall of operation gallery at RL 570 ft near sluice no 51 & 52 (block no 40 of right spillway) were noticed.5½ bags of cement was grouted when wet patches disappeared.

The engineers incharge of operation of the dam were vigilant about the cracks and were resorting to cement grouting as the remedial measure upto 1974. These cracks were generally minor in extent and were considered as normal for a concrete dam.

During August 1974, difficulty was experienced while lowering the hoist beams in sluice gate shaft No.59 and 62. On close inspection, it was found that some bolts fixing the guide rails to the concrete have been sheared off. It was observed that there are some obstructions at about RL 534 ft and RL 555' respectively. This distress and the fairly large extent of cracks were considered serious enough to warrant systematic investigation for developing appropriate remedial measures.

It was found that there was a horizontal crack all round the gate shaft at R.L. 538' of the under sluice No. 59 and at R.L.552' of the under sluice No. 62 measuring nearly 3/16" and 1/16" respectively. Further it was observed that such cracks were also existing in gate shafts of 18 nos. of under sluices (out of 24 nos) fixed at RL 510 ft – each under-sluice of size 20'-4" x 12'-0", 2 under sluices in a block at 18'-0" apart. In sluice no.62, 12 nos. of bolts and one in under sluice no.59 for fixing vertical guides were found to have sheared off.

In 1980 two crest gates near the abutments of right spillway were jammed and not moving freely. Numerous additional bolts of sluice gate tracks snapped. Bulging concrete was chipped off and extra concrete was cut to refix the bolts. During 1980's generally horizontal cracking continued in operation gallery and sluice barrel walls. Only minor cracks developed in foundation gallery. No crack developed in left spillway upto 1970. By 1980 very limited crack of similar nature began to appear in left spillway and few bolts snapped.

The concrete man hole cover in spillway road bridge became jammed. Top horizontal steel frame for doors at entrance of operation gallery of right spillway buckled.

The Chief Engineer, Irrigation, Orissa, in his letter No. 12807 dated 26.9.1974 requested the Chairman, Central Water Commission for taking up investigation in the matter and suggest precautionary measures for prevention. Since 1974 Central Water Commission has been actively associated with the study of the cracks and senior engineers have inspected the dam from time to time. Discussions were also held between the state and Central Water Commission engineers to evolve remedial measures. Pending detailed analysis of the problem, epoxy grouting was recommended to seal the cracks on the right spillway which was carried out from 1975. As advised grouting was done with a compound comprising of two parts of resin Dobeckot 505c mixed with one part by weight of hardner EH 411.

Preliminary investigations were carried out by Central Water Commission, New Delhi and other expert organizations. As per letter no cited above of Sept. 74 of the Chief Engineer to the chairman C.W.C. the C.W.C. took up some preliminary investigation works as detailed hereunder.

It has however been observed in 1982 that in block no.35 and 47 on the upstream face of operation gallery, epoxy putty (epoxy mixed with portland cement) used to seal the cracks have disintegrated at places. Similar deterioration was also marked in other locations of the operation gallery. In operation gallery of block no. 35 and 47 and gate shaft no 55, 59, 60 and 61, cracks which were sealed with epoxy have opened out at same locations. In some other places in the same blocks, new cracks have appeared close (within 10-15 mm) to the cracks grouted with epoxy.

As per recommendation in 3rd meeting of Expert Committee - 1981, grouting of cracks on downstream face of the spillway has also been done in block 38, 39, 40 of the vertical cracks in block No. 46 during May-June 1983.

These compounds have properties of sound adhesive (bond strength at 7 days 38.5 kg/ cm^2), rapid curing at ambient temperature, sound tensile strength of 460 kg/cm^2, low volume shrinkage and resistance to any corrosive chemicals. It has very low modules of elasticity (6600 kg/ cm^2) compared to that of concrete. Grouting was done with maximum pressure of 50 psi. The average consumption of epoxy in the gate shaft, gallery & sluice wall were 0.56kg, 0.39kg & 0.295 kg/m of crack respectively.

The epoxy grouting was discontinued before the monsoon of 1975. Some drill holes were taken to find out the penetration of epoxy and it was found that the travel of epoxy is varying from 1½" to 1'-2".

Sri K.R. Chandrasekharan, Director, C.W.C. and Sri S.L. Mokashi C.R.O., C.W.P.R.S, Poona visited Hirakud Dam and suggested further observations to be carried out. The movement of cracks were observed from January, 1974 with the help of extensometer points fixed at 16 different locations.

As per the advice of Mr. Chandrasekharan logging of all cracks in 48 drawing sheets were supplied in January 1975 for perusal. About a dozen of drillings were made as per the advice of the Director, C.W.C. out of which four were horizontal holes in operation gallery, three horizontal holes in barrel wall of gate No.64, one horizontal hole in the gate shaft of 59, one vertical and two horizontal holes in discharge carrier in block No.46, one vertical hole in foundation gallery in block No. 46. From th above results, it was found that the cracks were not exactly horizontal. Most of them either dip down or move up. The maximum length of horizontal drilling hole was made to be 12' in block No.43, at R.L. 559. 30' at a distance of 57' from the left side of block joint 42-43.

Dial gauges were fixed in block No.44 in January 1975 and in block 38 in Aug 1991 in three mutually perpendicular directions with dummy points on the upstream wall of the operation gallery. These dial guages show the relative movement of the concrete mass on either side of the crack in X-X, Y-Y and Z-Z directions. The observations show that in block no. 44 there is no relative movement in Z-Z direction since June 75 and no movement in X-X and Y-Y directions since monsoon of 1976. (vide Table No.4.7)

From the data furnished to C.W.C. as desired by Sri Chandrasekharan the cause of crack could not be determined and certain other studies like finite element studies, photo elastic studies were required to be carried out. Those were entrusted to I.I.T., Delhi and C.W.P.R.S. Poona.

Stress analysis of the right spillway of Hirakud Dam was done by C.W.P.R.S, Poona in 1978 and in their report they have concluded that the cracking both around the gallery and sluice barrels seems to be due to thermal differential due to difference in the concrete temperature and the air temperature and due to rapid cooling of the interior concrete.

Thereafter as per the suggestion of the C.W.C. vibration study and pressure study were conducted by C.W.P.R.S. staff at site. The results of such study show that the pressure and vibration development during different conditions of gate operations are well within the permissible limits.

In March, 1980 under water photographs of the upstream face of the Right Spillway were taken by the Naval team. Nonlinear stress analysis of the dam section by finite element method was decided to be done by C.W.P.R.S. It was suggested by C.W.C. earlier to carry on certain geodetic survey by survey of India for deformation study of the Dam.

The officers of C.W.C. & CWPRS visited the project in Nov. 1974 and feb.1976 respectively. After their visit C.W.C. chairman indicated that the following factors may be kept in mind while laying down the programme of research studies.

i) Effect of Vibration

ii) General distribution of stress

iii) Effect of temperature

iv) Effect of unevenness of foundation.

M.N. venkateson of CWC after his visit in Nov.76 indicated that speed of construction in right spillway may be one cause of cracking. He also suggested to examine the upstream face of dam either by under-water T.V. camera or by divers and see if cracks have occurred in upstream face. As per his advice this work was taken up in 1980.

On the suggestion indicated above, following studies so far carried out are as under :

(a) **Stress Analysis of Hirakud Dam by Sri R. Natarajan of I.I.T. Delhi :**

Finding : The values of tension occurring near the bottom of the gallery when the temperature loads are excluded in not appreciable. When they are included, the tensile stresses are high to cause cracking. The temperature in mass concrete, however would have gone down with age. It is likely that some effect due to temperature may be present which has caused cracking of sluice barrel.

(b) **Studies on Dynamic behaviour of Hirakud Dam by C.W.P.R.S. Pune :**

It is mentioned in the report that the displacement amplitude and particle velocities generated by the operation of sluice gates under various operating conditions are very low which may not cause hazards to the structure. Dynamic strains were not in the recording range and this appeared to be very low (less than 5 micro strains). However there are indications of large static strains corresponding to pressuring and depressuring sluice barrels.

4.2 Formation of Expert Committee to study & advise Remedial measures :

In order to study and advise remedial measures on cracks in Hirakud Spillways the Government of Orissa constituted an Expert Committee in 1981 vide Resolution no 10987/ IRR/ III-HKDW-381 dt. 24.3. 1981 with following members.

- Dr.Y.K Murthy (Ex-chairman C.W.C) – Chairman
- Member, (D & R, CWC) – Member
- Sri B. Ramachandran - Director (Engg.) Geological Divn-GSI – Member
- Sri S.C. Tripathy, Adviser, Orissa – Member
- Chief Engineer (Irrigation) Orissa – Member, Secy.

Five meetings were held as detailed here under :

i) 1st meeting January 1982.

On its recommendations following members were included as members in July 1982.

(a) Dr. M.C. Chaturvedi - Prof. I.I.T. Delhi

(b) Dr. K.T.S Iyengar - Prof I.ISc. Bangalore

(c) Dr. H.C. Visvesvaraya - Director, General CRII, Delhi

(d) Dr. A.K. Mallick - G.M. CRII Delhi

In addition. Prof. CV Ramakrishna I.I.T Delhi and Sri S.L. Mokashi, Joint Director CWPRS. Pune are to be invited as and when required.

ii) 2nd meeting - August 1982 Held at I.I.T. Delhi

iii) 3rd meeting - 25th to 30th April 1983 at Hirakud- Some intermediate measures were suggested.

iv) 4th meeting - 25th to 27th July 1983 – Rough final draft report prepared.

v) 5th meeting (final) - 2th to 15th September 1983 at Bhubaneswar- Recommendations for remedial measures were incorporated in final report (Oct. 1983).

The probable causes of distress and recommendations for remedial measures basing an site inspections, study of records and investigations carried out till 1983 are furnished in the following sections of this chapter.

4.2.1. Probable causes of Distress :

The Committee after examining the cracking pattern and other manifestations discussed above, inferred that the following factors need critical study to account for the distress on the right bank spillway dam.

4.2.1.1 Geology and Foundation Treatment :

4.2.1.1.1 Geology

Preliminary and detailed geological studies for the Hirakud dam have been carried out by the personnel of Geological Survey of India between the period 1945 to 1955. These studies involve assessment of the mineral deposits liable for submergence, interpretations of the subsurface explorations for the various components of the dam, large scale mapping of the dam and the foundations and evaluation of remedial foundation treatment.

The general sequence of rock formation met within the area are as below :

(i) Pre-Cambrian - phyllites, epidiorites, granite gneisses, mica schists, quartz schists, quartz mica schist and chlorite schists, etc.

(ii) Cuddapahs - Calcareous shales and limestones, clay shales, silt-stones etc. quartzite and arkoses etc.

The younger Cuddapah sedimentaries form the right abutment hillock of the dam in series of steep folds plunging towards south. A number of faults have developed in them, parallel to the fold axis showing drag effect, shearing and minor crushing.

The Pre-Cambrians are less intensively folded with two sets faults developed in them, one set normal to the flow of the river and the second set parallel to the flow of the river. The second set is younger to the former one.

Both the formations are extensively jointed in two to three directions with alteration along joint planes and coating of secondary silica and thin emplacement of calcite.

At the foot of the right abutment hills, the cuddapahs sedimentaries have been thrust against the pre-cambrian Metamorphics, marked by a fault zone dipping steeply towards the river at 75^0 to 80^0 angle.

The fault zone is 1.5 meter wide at the river bed level with clay gouge in the middle for a width of 45 cm. The rocks are intensively sheared and fractured on either side of the fault zone for a total width of nearly 50 meters.

In the spillway dam, the sandy overburden over the granitic rocks is generally less than 3 meters, whereas along certain minor faults and joint zone, the fresh rock profile is around 6 meters, because of alteration along these plane.

In the reach lying between blocks 44 to 46 in the spillway dam, close to the power dam, fresh rock has been touched almost below the sand cover on the upstream side. But on the downstream side, of block 44, a zone of highly altered and intensively sheared rock is seen for a width of 15 meters, increasing to a width of 30 meters towards block 46. In this reach the foundation was taken down to a depth of 6 to 9 meters below average ground level with extensive consolidation grouting of the underlying rocks, for excavation to a depth of 18 meters upto fresh rock level was not considered feasible. It was opined that the massive ledge rock on the upstream side measuring about 15 m in block 46 and widening to 30 m in block 44 would take care adequately of the foundation load and possible leakage from the reservoir.

In block 38-39 a 12 m wide epidiorite band was seen cutting across the foundation transversely from upstream to downstream with sharp incised contact with the adjoining granite rocks suggesting emplacement along a fault or shear. The basic band is well fractured with three sets of joints. This band has been excavated in the form of a trench to a depth of 6 m to 9 m below the average foundation level.

In the foundation area between the heel portion of block 46 to the bucket portion of block 47 a transverse fault with vertical dip was seen. Two prominent fault zones join from upstream side of block 45 with a number of minor faults branching towards the main one aligned parallel to the dam axis. All the fault zones are around 0.3 m width and filled with fine grained chloritic material, interconnected at shallow depth. Formula depth excavation have been carried out in all these fault zones. For further details, Sec.2.1.3.1. of this book may please be referred to.

4.2.1.1.2 Foundation Treatment :

Stripping was carried out manually and with the help of mechanical equipment using explosives on touching hard strata. On reaching the foundation grade, wedging and barring was carried out for reaching the foundation level.

In all the major fractures and joint zones, as a rule, excavation was done to twice the width of the zone at the foundation level. In the excavated trench, removal of loose material was carried out manually. Wherever are seen coalescence of major joint zones and fractures resulting in intense fracturing of the rocks, steel ribs in a criss-cross manner were placed in the foundation with a view to transfer the load of the dam to the massive rock on the sides away from the joints and fracture zones. Further, on the slope of the foundation, anchor rods have been fixed drilled into the rock for obtaining support for the concrete into the fresh rock below.

Full details of the foundation treatment with the amount of cement consumed in the individual blocks of the right bank has been given in the book on "Foundation Treatment" printed by the project authorities. The total amount of cement consumed by consolidation grout holes in individual blocks are as follows:

Table No. 4.2 Consumption of Cement for Consolidation Grouting

Block	Cement in tonnes	Block	Cement in tonnes
34	2.2	48	17.00
39	20.1	50	0.06
40	5.25	51	1.1
44	7.0	52	4.6
46	20.00	53	4.8
47	12.0		

Summing up, the right spillway dam has been founded on fairly fresh and sound granites, gneisses and epidiorites, though crossed by a number of faults, shears and fractures. Along these zones, alteration of the rocks has been noticed at the foundation level, which has been extensively treated by trenching excavation to Shasta formula depth, placement of steel ribs and consolidation grouting. Hence it is surmised that development of thin sub-horizontal cracks in the right spillway dam is not in anyway related to the geology of foundation since adequate foundation treatment has been carried out during the construction stage, as reported in the publication on "Foundation Treatment in Hirakud Dam Project" 1959. This aspect has been discussed exhaustively in chapter II sec 2.12 of this book.

4.2.2 Deep Scour in Plunge Pool & Deep pit for Generating Unit No.7

Committee also observed that a deep pit was dug immediately downstream of block No. 48 for future installation of generating unit No.7 in the Power House and in some reaches there was deep scour in the plunge pool of ski-jump bucket of spillway. These features may have some contribution for development of cracks and needed to be studied in detail. This study was entrusted to the Geological Survey of India (GSI).

4.2.3 Effect of Static & Dynamic Loads on the Structural Stability (taking into consideration the large size openings of the Operation gallery, Gate shaft & Sluice barrel in each block of the Spillway Dam)

The Committee wanted a study of the distribution of the stresses in the body of each block taking into account the large size openings such as sluice barrels, gate shafts, operation gallery in order to assess the contribution, if any, to the development of cracks. Though it was found that the stability of the dam under normal loading conditions was safe, it was however considered necessary to carry out Finite Element Analysis for evaluating the stresses. This work was entrusted to Indian Institute of Technology (IIT), Delhi.

4.2.4 Vibration due to Opening of Gates & Operation of Power House

Release of flood flow through the sluice gates and crest gates induces some vibration in the body of the spillway. Operation of the power house close to the right spillway also transmits a mild vibration. The effect of these to cause distress to the right spillway was studied by the Central Water Power Research Station (CWPRS) Poona.

4.3. Studies undertaken by Expert organisations

The following studies/investigations made by different expert organisations were reviewed by the Committee before giving their recommendation for remedial measure. Those are briefly summerised here :

4.3.1 Stress Analysis of Right Spillway (C.W.P.R.S. Pune, Oct 1978)

The results have been discussed under 3 heads i .e.

i) Effect of gravity and water load.

ii) Effect of temperature changes.

iii) Effect of vibration.

The gist of conclusion is as under :

The stresses even at higher elevations are non-linear and it is not possible to interprete the behaviour of the structure in the vicinity of the openings by a linear analysis. The stresses due to the normal gravity and water loads are not excessively high although some stresses are tensile particulary near the downstream face. The internal pressure also attributes to reduction of compression in the wall of the sluice. The compressive stress at the face of the large gate operating gallery is very small. This could therefore lead very easily to crack due to thermal stresses which could be of the order of 20 kg/cm^2 for a drop of 30^0 in temperature. The major cause of the crack both around the gallery and the sluice barrels seem to be the differential thermal stress that has arisen primarily due to the difference in concrete temperature and the air temperature and secondarily due to the rapid cooling of the interior concrete.

4.3.2 Conventional Stability Analysis of Right Spillway

(As laid down in IS : 6512-1972)

The main findings of the study are :

i) The minimum factor of safety against sliding are 1.04 and 0.98 for conditions B & E (transient condition) respectivelly with no relief of uplift at the drains. This shows that **there can be no apprehension about sliding stabilty of the dam.**

ii) Dam is safe for normal loading condition.

iii) A nominal tension of 3 psi is indicated at the founding level with an earthquake acceleration 0.02g coupled with 66.7% relief of uplift at foundation level. **No tensions are indicated at higher elevations.**

iv) For extreme conditions of loading i.e F & G high tensions are indicated at the upstream heel at founding level. These are reduced at higher elevations. Howerver, **the dams is considered safe against over turning.** The maximum compresive stress during these conditions of loading is about 305 psi which is well within the permissible limit.

4.3.3 Stress Analysis conducted at I.I.T Delhi- July 1983

i) Temperature measurements :

Thermometers and strain meters have been installed in block no 41 of Right spillway and block no 57 of power dam. It has been found that the early maximum temperature rise of approximately 45^{0}F was attained after a time lapse of 40 to 100 hrs after placement of concrete. The exposed surfaces of the dam experienced a fall in temperature due to radiation. The temperature rise is rather high. No information is available regarding the actual temperature gradient. Earlier studies have indicated the existence of such steep thermal gradient over a distance of 4.5 ft. Thus in present study, thermal stresses have been computed assuming the steep thermal gradients to exist over a width of 4' and 6 ft. In case of 2-D analysis a gradient of 11.25^0 per foot has been considered.

ii) Plan of detailed stress analysis

The following action plan was finalised.

a) Use of 2-D Finite Element analysis is for stress distribution in the cross section due to:

- Reservoir empty condition – gravity load only
- Reservoir full condition – Hydrostatic + uplift + gravity forces.
- Reservoir full condition with thermal load
- Effect of presence of weak foundation on stresses

(These explain cracks in operation gallery)

b) 3D finite element analysis were used for determination of stresses on the walls of the sluice barrel for the conditions : (vide sec. 4.10.9 also)

- Hydrostatic + gravity + uplift forces for reservoir full condition.
- Hydrostatic + gravity + uplift + thermal loads (These explain cracks in sluice barrel)

c) 2-D study of a block subject to longitudinal thrust

- **Results & Conclusions :**

It is clear from 2-D and 3-D analysis that under normal loading conditions the structural design of the dam is very satisfactory with the peak stresses in tension and compression well within limits. Even large openings have not really resulted in unacceptable stress concentrations. Thus the reinforcements provided around the openings are sufficient to take care of these stress rise.

The presence of weak foundation on the downstream of spillway in block no 45, 46 & 47 and the treatment undertaken for strengthening the foundation was studied in detail and it was inferred that the settlement of the foundation on the downstream side being one of the reasons for development of crack is ruled out.

Dynamic forces being responsible for the distress have also been ruled out from the studies carried out by CWPRS, Pune.

Thus out of following six possible causes of crack enumerated by the Expert Committee, the last three are worthy of consideration which are dealt separately.

i) Effect of static and dynamic forces

ii) Effect of construction methods and rate of placemet of concrete

iii) Effect of existence of pit in downstream of block-48 & deep scour in plunge pool.

iv) Alkali - Silica reaction

v) Heat of Hydration & Thermal stresses. The effect of construction methods and the rate of placement of concrete mainly are concerned with control of temperature rise

vi) Complex geological conditions of foundation.

4.3.4 Non-Destructive studies on concrete spillway section (by ultrasonic and sonic pulse technique - CWPRS, Pune - April 1983)

Non-destructive studies were conducted by cross probing and surface probing by ultra sonic and sonic pulse technique to find out areas of deteriorated concrete, if any, so that cores could be taken in this portion for further studies. The studies were made in block no 35 and 46 of right spillway where crack was predominant and on block no 12 of Left spillway where no crack has been observed with an aim to compare the results.

Depending on velocity & attenuation of ultrasonic pulse at the receiving end, concrete was classified qualitatively. Ultrasonic pulse velocities were measured by cross probing method across the wall of sluice barrels no 41 & 42. The average velocity observed was 4.2 km /sec which was comparable to the values obtained by surface probing. The depth of crack which was visible from

the surface in barrel no 47 was estimated by measuring the transit time across the crack. It was estimated to be 80 cm.

From the velocity results, it is concluded that in-situ quality of concrete falls in grades of very good & good. However in block 35 & 46 there are a few locations where velocities were less than 3.5 km/sec. It was suggested to take cores from these areas to compare their strength characteristics. It is concluded that concrete in block-12 is more homogenous than that of block 35 and 46.

4.3.5 Vibration studies of Gates and Pressure Studies in sluice barrel - CWPRS-Sept. 1976 and Jan 1978

4.3.5.1 Vibration Studies - (In gate no 59 – Sept. 1976)

It was decided to take vibration studies in gate 59, as it was reported to be subjected to maximum vibration. Reservoir level was at RL.191.71 m (628.8ft) and the head on the centre of sluice was 33.0m.

It could be seen from the 3.D display that the amplitude of vibrations is of the order of 40 microns in the frequency range of 2 to 3 Hz. for the gate opening of about 5% of full gate opening. The vibrations are again high of the order of 40 microns in the range of gate openings of 50 to 60%.

4.3.5.2 Pressure studies (Jan. 1978)

On the day of pressure observation in the sluice barrel at site the water level in the reservoir was at RL. 190.71m (625.69'). The pressures on sides and roof of sluice-52 were observed only when the sluice gate was fully open since partial opening was not allowed. Review of piezometric pressure distribution along the sluice barrel indicates similarity of trend in both model and prototype. Magnitude wise the prototype pressures are slightly higher than that of model in respect of roof pressures. As regards side pressures, the proto values at two points are substantially higher than that of the model. This may be due to extraneous factors that might have crept into measurement during measuring period.

Between 27.8.76 to 15.9.76 some staff of CWPRS, Pune tried to

a) measure sluice gate vibration, particularly in gate no.59

b) measure vibration of bolts grouted above and below the horizontal cracks in wall upstream in front of gate-59

c) measure the air velocity in the air vent of gate -59 during its operation.

d) measure the vibration of crest gate

e) measure the pressures in the sluice barrel.

Crest gate vibration measurement was dropped as per the suggestion of the Director (Monitoring) of CWPRS. The results of other measurements were not conclusive.

4.3.6 Petrographic Analysis of Coarse aggregate from Laxmi Dunguri Quarry- (by GSI, April 1982).

The petrographic analysis & minerological composition of the aggregate source rock indicate that it is clearly free from hydrous amorphos/colloidal silicon in the form of opal and cryptocrystalline siica like chalcedony, chert, flint etc, which are considered to be deleterious materials for K_2O, Na_2O (in cement) reactivity with silica (in aggregate) producing adverse volume change and resulting crack.Thus the concrete using this aggregate is unlikely to produce any crack and this obviously rules out the alkali aggregate reactivity as one of the causes of development of cracks in the body of the dam postulated earlier.This is further corroberated by the results of the chemical tests carried out in Hirakud Research laboratory for the alkali aggregate reactivity. But it has been established later that A.S.R. is the notorious element for cracking vide sec. 4.3.6.1, 4.4 & 4.7.3 of the report.

4.3.6.1 Petrographic examination of aggregate (from river bed of Mahanadi and core drills from Hirakud Dam - G.S.I- July 1983.)

Study was made of the aggretates and concrete cores. Besides, aggregate samples from the river bed, both of fine and coarse variety were collected. These were petrographically examined. From the study it is observed that at places the aggregate pieces in the concrete are in a highly altered state as noticed in hole no 45 D_1. Thus, this high degree of alteration may induce certain deleterious features in concrete. The weathered material may not exceed more than 5% which is the permissible limit. Threfore presence of weathered material in the coarse aggretate in not considered as the main cause for the distress in concrete.

All the aggregate pieces have shown quartz in a very deformed state with undulous extinction and peripheral granulation. Felspar also exhibit alteration features as well as deformation. This highly sheared nature of the quartz may render them liable for alkali aggregate reaction. The slightly higher degree of felspars in the sand in the area, that too in a slightly altered state would induce problems of distress in concrete.

As indicated in Annual report 1950-51 of Hirakud Research station it is known that some of the riverine gravels from the reservoir in the vicinity of the dam site is reactive because of occurrence of chert, chalcedony and other forms of secondary silica. In addition to this, large amount of silicified rocks occur in the area particularly in the left bank. The extent to which those have been used in the construction of the dam is not known. Some of these deleterious materials might have been used in the concrete. This might have induced alkali reaction and caused distress in concrete.

4.4 Recommendations for Remedial Measures (1983)

The Committee in their previous meetings had stated that though there was no cause for any undue alarm about the structural stability of the dam, the various aspects of the behaviour of the affected components of the dam should be observed, investigated and studied to evolve remedial measures. Also the Committee had recommended some interim remedial measures after inspecting the dam pending the final decision on the cause of the cracks and other manifestations to be based on the various studies suggested by the Committee from time to time.

The Committee took note of the various interim remedial measures that have been executed. These remedial measures suggested were able to counter the adverse effects of the distress observed. It is important to note that the distress of cracking and other manifestations have not ceased and are continuing because of causes identified and exlpained earlier. For minimising the effect of distress obseved on the structural stability and operational facilities of the various components of the right bank spillway, the following remedial measures are recommended.

a) The main features of distress observed in right spillway are the extensive cracking that developed because of high tensile stress caused by the heat of hydration of concrete in the initial stages. This was followed by alkali aggregate reaction, which also has resulted in pattern cracking and other manifestations in the dams. It is necessary to grout the body of the dam with suitable chemicals to fill all the cracks effectivety for restoring the monolithic behaviour of the structure.

b) The cracks existing on upstream face of the dam should be filled up with epoxy mortar particularly between the elevations RL 540 to RL 585 ft where the cracking has been observed to be wide and extensive. This has to be done before grouting the body of the dam.

c) Regular monitoring of the development of further cracking should be done.

It may be relevant to mention that such treatment of the cracks by chemical grouting has been used in Libby Dam 136 m high, 684 m long concrete gravity dam which was completed by

U.S. Army Corps of Engineers in 1965. The chemical grouting was carried out in 1973 in several blocks which had developed extensive cracking.

Also Manicougan - 5 Dam in Canada, a multiple arch structure recently completed, developed extensive cracking in one of the abutment buttresses. The Project Organisation has decided to use special epoxy grout for treatment of cracks and the work of repairing completed.

d) Since chemical grouting is a highly specialised type of work, a suitable agency will have to be entrusted with this task.

e) The bulging and deflection of concrete walls or the adit gallery portal in block 47, gate grooves in the vertical shaft of sluices, and in spillway radial gate end piers, should be chipped to a depth sufficient to remove all the weak concrete that have been caused by alkali-aggregate reaction and replaced with sound concrete. It is necessary to ensure proper bonding of new concrete with the old concrete of the structures. This type of treatment may have to be repeated till alkali-aggregates reactivity ceases.

f) This Committee feels that the alkali-aggregate reactivity in Hirakud spillway dam is found to vary from 'mild' to 'moderate' and may not cause more serious distress than what has been experienced so far but may continue for some time till the reactivity ceases. As a result, the repair works to the dam described above may have to be repeated whenever the distress is experienced and at the same time exercise very close surveillance by suitable instrumentation.

g) In block 46, where cracking observed is maximum, anchor rods to act as shear pins to prevent any relative movement of cracked concrete portions of dam block should be provided as an experimental measure. These anchor rods should be long enough and grouted along their entire length and oriented in a direction as far as possible normal to the cracking in the dam. Advantage may be taken of drilled holes for dam body grouting mentioned above wherever it is feasible. Depending upon the experience gained, this treatment should be repeated in all the distressed blocks. The performance of these passive anchors should be monitored by suitable instrumentation. *(This has not been carried out by the Project).*

h) In addition to the above remedial measures suggested, it is important to watch and monitor the behavior of the dam structure by installig instruments and making systematic observations and the data collected should also be analysed regularly and interpreted. A separate note on the type of instruments to be installed in the structures will be furnished by Member (D & R) C.W.C. with necessary drawings.

i) Though it is difficult to monitor effects of alkali-aggregate reactivity continuing in the dam, it is recommended that concrete cores should be obtained from different locations suspected to have been developing alkali-aggregate reactions and where visible tendency for bulding etc. are noticed. Comparative periodic evaluation at least once a year of the mechanical properties of concrete like compressive strength and modulus of elasticity, ultrasonic pulse velocity and occurrence of alkali aggregate reaction should also be made from two comparable specimen viz. one stored in the air and another in sodium Hydroxide (NaOH) solution. In addition, observations should be made of occurrence of new cracks as well as widening of the existing cracks, if any.

j) The Committee recommends that the following precautionary measures should also be taken to ensure the proper functioning of the dam.

• Though the treatment of foundation for the right spillway dam has been satisfactorily carried out as evident from the Special Report on the subject published by the then Chief Engineer immediately after the construction of the dam, it is necessary for test check of the efficacy of curtain grouting. Whereever any deficiency is noticed drilling and grouting should be carried out.

• The drainage holes in the drainage gallery and the porous drains on the body of the dam should be redrilled, cleaned to make them function effectively.

• The stability of the dam has been rechecked for normal loading conditions and also checked for earthquake loads adopting a seismic coefficient of 0.02g. In the original design of Hirakud Dam no earthquake loads were taken for evaluating the stability of the dam.

The Commitee was informed that for the dams being designed/constructed at present in the adjoining regions in Orissa as well as in Chhatisgarh, seismic coefficientof 0.05 g has been adopted. *It is therefore suggested that the stability of the Hirakud Dam should also be checked with this seismic coefficient in consultation with the Standing Committee for Recommending Design Seismic Coefficient for Dams, Government of India.*

4.5 Formation of Standing Committee for long term surveillance of crack

In 1987, a Standing Committee for long term Surveillance of cracks of Hirakud was constituted by Govt. of Orissa in their Resulution no Irr-III HKDW-4/87-29898/BBSR dt 10.7.1987 with following members.

1. E.I.C (Irrigation) Orissa - Chairman

2. Chief Engineer, DSO, CWC - Member

3. Representative of CWPRS, Pune - Member

4. S.E. Hirakud Dam circle Burla Member, Secretary, The Committee had five meetings.

1st - on 12-12-1987 - This could not be held as members from CWC & CWPRS could not attend.

2nd - 24th to 25th May 1988 - At Burla

3rd - 17th to 18.12.1988 - At Burla

4th - 21st to 23.8.1989 - At Burla & Bhubaneswar

5th - on 7.11.1989 At Bhubaneswar

Minutes of discussion were prepared.

The terms of referrence were :

To inspect the dam at least twice a year to take a view on condition of the dam. The Committee has to submit the reports after each inspection.

4.6 Formation of Hirakud Dam Crack Review Panel (HDCRP) – 1990

A Committee styled as Hirakud Dam Crack Review Panel (HDCRP) was constituted by the GoO in their resolution no Irr.III HKDW-9-90-31985 dt 22.8.90 with the terms of referrence as stated below :

i) To review and anaylse distressed condition arising out of defects described earlier.

ii) To suggest remedial and rehabilitation measures both short term and long term.

iii) To review current measures taken by the Govt. for repair of cracks and record their opinion for guidance.

The formation of the Committee was warranted due to the following reasons:

"The underwater mapping done by the Dutch divers in the upstream face of spillway in May-June 1989 indicated presence of other concrete defects like honey combs, holes, spalling, and pop-outs etc. which necessiated treatment in addition to crack treatment."

The committee was formed with following members.

1) Sri G.N. Tondon, Adviser, Sri Lanka - Chairman

2) Sri B.M. Hukku, Geologist, Lucknow - Member

3) Dr. A.K. Mallik, Director General - NCCBM - New Delhi - Member

4) Dr. Bharat Singh - Professor, Emeritus, Roorkee - Member

5. Sri Sundar Babu ..... Member

6. Sri S.L Mokashi, CWPRS Pune Member

7. Sri A.C. Roy, CE(Medium Irrigation), Orissa-Member Secy

8. Sri J. Tripathy - Special invitee

9. Dr. C.D. Thatte - Member (D & R) CWC -do-

10. Sri M.S. Reddy - Member, (WP) CWC -do-

The Committee had 11 meetings as detailed hereunder

1st - 13 to 14.9.90 - New Delhi

2nd - 12 to 17.11.90 - Burla

3rd - 22 to 26.12.90 - New Delhi

4th - 10 to 12.8.91 - New Delhi

5th - 19 to 23.11.91 - Dam site, Bhubaneswar

6th - 28 to 30.5.92 - New Delhi

7th - 19 to 30.1.93 - Burla / Bhubaneswar/Delhi

8th - 19 to 24.3.94 - Bhubaneswar / Burla

9th - 23 to 29.6.95 - Bhubaneswar / Burla

10th - 16 to 22.8.96 - Bhubaneswar / Burla

11th - 22 to 27.2. 99 - Bhubaneswar / Burla

Their findings, comments, recommendations, suggestions and concluding remarks are briefly summarsised in the following paragraphs.

4.7 Brief Summary of Findings, Comments, Recommendations, Suggestions and Concluding remarks of HDCRP.

4.7.1. Comments on crack widths, lengths and depth etc.

The Crack as seen on the surface of openings within the body of the dam are extensive. They are generally less than 0.05 mm thick and growing at slow rate of about 2% per year.

The cracks in upstream face (under water) as measured by video recording up to 1989 are confined in a band elevation of 535 ft & 570 ft (163.0 to 174m). This is suggestive of some deficiencies in this zone which corroberates with the findings of the Expert committee headed by Dr.Y.K. Murthy (1983). Some of these cracks are more than 10 mm wide.

The crack length measured upto 1997-98 included hair cracks ignored earlier and cracks in some new areas not included in previous surveys. The Project has also given modified data excluding such hair cracks and cracks in some new areas. This modified data is more appropriate for comparison. From this it can be seen that while the absolute crack lengths as well as five years moving averages do show increase, but the rate of increase is gradually decreasing. This can be due to continuing A.S.R. activities discussed earlier in Y.K. Murthy Committee report and later in this book vide sec 4.7.3.

A post treatment under water survery of blocks 35 to 40 was carried out. Mapping of new cracks as well as video filming were also done. It was seen that where a total of 1286.0m of cracks were sealed in these blocks, fresh cracks to the extent of 1013.9m have occurred again upto 1998-99 (vide Table 4.9). Under water survey of cracks on upstream face of C.R. Masonry power Dam was done down to RL.536.80 ft (163.61m) below which survey could not be undertaken due to silting.

A total of 662m of crack was mapped out of which 274.3m was in block no.56. As reported, the cracks are along the mortar joints and most of them are fine cracks. There is no evidence of leakage or seepage through these cracks in the power house or on the downstream face. The panel does not consider that any action need to be taken to repair these fine cracks (Refer 11th meeting report of the HDCRP).

4.7.2 Comments on Foundation :

The foundation conditions are sound and stable. The grout curtain is effective and the uplift pressures in the foundation are well within acceptable limits.

4.7.3 Alkali Silica Reactivity (A.S.R.)

There is undoubted evidence of A.S.R. activity in the body of the spillway dam. There is no technique to stop this activity. It will continue till one of the two reactive constituents in cement and aggregates get exhausted. Degradation as a result of ASR is inherently a slow process and could cause no sudden failure. In Hirakud case the activity is mild to moderate. Hence the eventuality of a sudden and catastrophic failure of the dam can safely be ruled out and no risk to the continued useful performance of the dam and its appurtenances is foreseen (Refer 2nd meeting Report of HDCRP). Incase of Hirakud, the rate of expansion of concrete due to ASR seems to have passed the peak and is levelled off (Refer 5th meeting Report of HDCRP).

Test results carried out by N.C.C. B.M established the occurrence of ASR in the concrete of left spillway as well. Block no. 4 showed prominent signs of ASR. The aggregates were more of granite granodyorite & granite poriphyri type and diorite. Occurrence of river shingles containing chert or chalcedony was relatively few and less than right spillway. Tests indicated that ASR activity was essentially due to presence of strained quartz as well as alteration of felspars into clay minerals. (Refer 6th meeting report of HDCRP).

Results of periodic assessment of concrete for ASR in right spillway (3rd cycle) showed presence of ASR activity and physical properties like compressive strength and modulation of elasticity were generally in order as in earlier cycles. In as much as the potential for ASR continued to exist, some relatively lower values of modulus of elasticity in some samples of block-35 were measured. Isolated samples showing severe signs of ASR showed wide cracks or fracturing of aggregates.

Mr. David C. Stark, senior principal scientist, Material Research & Consulting of Construction Technology Laboratories Inc (CTL), USA visited Hirakud Dam during 18th to 24th Jan. 1993 in response to an enquiry from Mr. William Price of the World Bank. He also went through the comments of H.D.C.R.P & Dam Safety Review Panel (DSRP) and furnished his Investigation Report on ASR activity in 1993. Before submitting his report, he inspected the operation galleries, foundation galleries and quarry sites etc. The gist of his observations and conclusions are as follows :

• Operation gallery - Presence of light cracks in walls of both operation galleries are visible. More cracks are found in right spillway gallery than those in left spillway. Depth of cracks are not known. However observation of core hole surfaces revealed that such cracks are limited to $\pm$ 1m thick i.e only in the portion of reinforced concrete. It has not been found in mass concrete. This type of cracking is characteristic but not necessarily diagnostic of expansive ASR. The walls of operation galleries are finished with nominal 2 to 3 cm thick mortar coat. In some places map crackings have occurred.

Whether cracks have gone upto lean mix can be ascertained by taking 85 mm dia. cores through lean mix concrete.

• In foundation gallery few horizontal cracks have occurred along lift lines which may not be due to ASR.

• In Power house, there appears no sign of defects due to ASR. If at all exists, it is negligible. There is no report of misalignment of turbines or generators, which commonly is most sensitive indicator of abnormal growth of supporting concrete.

• Snapping of bolt starting from 1980 in Right spillway and that from 1990 in left spillway is attributed to ASR. Warping is also caused by ASR.

• Entrance to operation gallery of both spillways - Horizontal gate guides are bent or buckled. This is attributed to lateral expansion and is due to ASR.

• Training walls in sluice ways - Sluice no. 43 (block 36) revealed no ASR in floor. However map cracking observed on top horizontal and sloped faces of several training walls is due to ASR.

• Core Examination - 54mm dia. cores are mostly fractured. 85mm dia. cores used in petrographic examination showed apparent presence of ASR gel reaction products associated with dark coarse aggregate particles. This indicates only that ASR has occurred and not that ASR has resulted in distress.

• Petrographic Examination - Eleven nos of core samples (100mm dia from road & 85 dia. from other places) were examined petrographically both from left and right spillways at CTL as per ASTM C-295 Standard Procedure for the Examination of Hardened Concrete.

In addition, the fractured surfaces also were examined under U-V light after treatment with Uranyl acetate solution.

The results are reproduced in Table No.4.3.



T.I.M.T.S. at Laxmidunguri (2000 T capacity)

Table No 4.3 : Petrographic Examination of core samples at U.S.A.

Core section	Spillway	Block	Location	Core length (mm)	Remark
1	Left	2	Roadway at crest	300	Coarse aggregate diorite granite gneiss river shingles primarily quartzite and granite material. Maximum particle size 25mm. Fines rounded to angular quartz and feldspar. Micro cracks within river shingles and particles only in gneiss granite textured diorite granite. Over all ASR appear to have resulted in minor distress in concrete.
2.	Right	48	Roadway at crest	340	ASR appeared to have caused minor distress.
3	Left	11	Rich concrete	00.to 0.47m	No ASR get deposit. No micro-cracking.
4	Left	11	Mass concrete	1.95m to 2.45m	Predominately dark particles free from ASR gel. ASR appeared to have resulted in little if any, distress, in concrete.
5	Left	11	Mass concrete	10.81 to 11.15m	Minor ASR gel. No distress due to ASR.
6	Left	11	Mass concrete	17.35 to 17.62m	Meagre deposits of ASR gel. Overall the concrete appeared to be in excellent condition.
7	Right	46	Mass concrete	4.67 to 5.10m	ASR did not appear to have resulted distress of concrete.
8	Right	46	Rich concrete	0.10 to 0.46 m	Very fine localised glassy clear to white deposits of ASR gel in diorite granite gneiss. No evidence that ASR has resulted in cracking in concrete.
9	Right	46	Mass concrete	10.37 to 10.77m	Gels are meagre. ASR appeared to have caused minor crack in concrete.
10	Right	46	Mass concrete	20.75 to 21.15m	Gels meagre. ASR appeared to have caused no distress in concrete.
11	Left	11	Mass concrete	9.31 to 9.75 m	A few diorite grandiorite coarse aggregate particles contain localised flourocent ASR gel. Reaction rims are observed. ASR appeared to have resulted in little distress in concrete.

Mr. Stark visited several quarry sites. His visit revealed that very complex steeply inclined beds of metamorphosed rock were used as source of aggregate. Rock types included amphitolite, diorite to granite gneisses and miscellaneous quartz rich material. These are known to have reacted in concrete else where in the world (USA) when used with alkali cements.

In concluding remark Mr. stark mentioned as given below :

• ASR to date (1993) has caused expansion and has been a minor source for the observed cracking.

• Both diorite granite gneiss and river shingles have participated in ASR in both lean and rich mix concrete.

• ASR has produced little or no damage in mass concrete.

• No ASR gel has developed in power house concrete though materials from same source were used as those in spillway construction.

• Restraint to movement by the mass of concrete is a major factor suppressing or preventing expansion and cracking due to ASR.

• The dam has already achieved 35+ years of age (by 1993). ASR should cause little or no expansion in future.

• ASR will not endanger the safety of the dam nor will it limit the functional operation. *No dam in the world has ever failed solely due to ASR.*

It is believed that ASR has passed into the phase with diminishing potentiality. More over in case of Hirakud, cement containing less alkali (<10%) has been used.

Right spillways shows greater severity than left. Method of construction may be one of the factors.

The Hirakud Dam Crack Review Panel (HDCRP) in its 8th meeting in 1994 generally agrees with the comments of Mr. Stark except a few as given under :

• Mr. Stark has commented that ASR could cause little or no expansion in future. This may not be literally true anticipation. In Panel's view the ASR activity appears to have reached a plateau and should gradually decay with time in right spillway but in left spillway the ASR activity though delayed is still on a rising trend.

• Upto the time of Mr Stark's visit cracks in Power House had not been mapped and reported. Hence his comment that deleterious ASR had not developed in Power House concrete was based on incomplete information.

• NCCBM collected core samples from 14 locations in upstream face of right spillway out of which 7 nos were in tact and tested. Sample from block 35 showed prominent sign of ASR activity. Samples from block no 37, 38, 40, 41 and 43 had shown no sign or sign of feeble to moderate ASR activity. The aggregates involved in the cores were quarzite river shingles, diorites and rocks of granite family. The physical properties like density, compressive strength and ultrasonic pulse velocity were adequate (Refer 8th meeting report of HDCRP).

4.7.4 Tests on aggregate samples from Laxmi Dunguri at NCBM

Two samples of aggregate from Laxmidunguri quarry were tested at NCBM Laboratories. The first sample was dark to medium grey in colour, coarse to medium grained and was termed as granite. Nearly 55 to 60 percent of quartz grains showed strain effect and UE angle varied from 16^0 to 21^0. The potash felspars show alteration to clay minerals and sericite. In case of plagioclase feldspar, at few places the polysynthetic twin lamellae were deformed, and showed alteration to clay minerals and sericite.

• The second sample was light gray in colour, coarse to medium grained and termed as quartzite. Nearly 50 to 55 percent of the quartz grains showed strain effect and their UE angle varied from 18° to 20°. Few quartz grains showed presence of transverse cracks. Few grains of feldspar show signs of alteration. Both the rock types can be potentially reactive.

• The conclusions of Mr. David Stark in this respect are similar.

• In conclusion, the Panel was happy to note the similarity of findings of examination of aggregate and concrete core sample by Mr. Stark at CT Laboratories, which are in agreement with previous investigations carried out in India by NCBM, both in relation to detection of occurrence of ASR in spillway block concrete; in the way of characterising the ASR product.

• The Panel recommends that periodic monitoring of ASR activity by tests on concrete core samples should continue for both left and right bank spillway. NCBM may also be requested to carryout the tests for longer duration of say 3 to 5 years, both in presence of external alkali solution at 60°c as done so far, as well as by out door exposure tests in water storage.

• Sustenance of concrete quality, elastic properties and strength is a redeeming factor for the continued safety of the dam inspite of the ASR activity. Period testing of concrete quality and its evaluation is essential till ASR activity subside to insignificant limits.

• The H.D.C.R.P in its 11th sitting reiterated the occurrence of ASR activity in both spillways based on test reports on samples carried out by NCCBM, CSMRS and Dr. Stark. The members were shown two horizontal core samples extracted from the repaired portion in the face of the spillway to a depth of 6ocm (block 6 & 7 of left spillway) which showed unmistakable presence of ASR even to the naked eye.

• In so far as the current rate of reaction is concerned (upto 1999), the summary of measurements of extensometer shows a tendency of crack widths still increasing which indicate the ASR activity is continuing. Identification of fresh cracks in upstream face under water and in spillway galleries also indicates continuing ASR. On the other hand, observation like the declining trend in respect of total crack width, number of tell tale failures or number of snapped bolts indicate that the rate of activity is rather slow. It will be fair to summarise that the ASR activity is continuing at a decreasing rate (Refer 11th meeting report of HDCRP)

4.7.5 Concrete Homogeneity:

The results of water loss in 14 inclined holes made from operation gallery right spillway towards the downstream side reveal that the concrete is not uniformly dense and compact. This is further corroborated by the inflow of water in 3m (10') deep vertical holes made from the roof of each block. Existence of cavities in mass concrete was also seen directly in a 54 cm deep excavation made on downstream face of operation gallery.

From the concrete cores taken from upstream face of right spillway it could be seen that there were essentially two types of concrete samples and the one representing the secondary concrete in the rail guide was of different composition and texture. In concrete samples from the main structure, there were large cavities representing honey-comb in original construction, which contained white deposits of leached calcium hydroxide and some contained mud pellets up to 15 to 20 mm size.

4.7.6. Impermeabilisation by Epoxy grouting :

The panel in its 5th meeting had suggested a time frame and nature of remedial works which essentially would involve the impermeabilisation of the zone between the upstream face and the operation gallery in particular; but some other upgrading and rehabilitation works in general.

So far as main impermeabilisation is concerned, basically grouting and improved drainage in the body would be the main approach. This is barely necessary, as in the estimation of the Panel

about 500 Tonnes of material has been leached out of the dam in last 30 years or so. The panel feels that before taking a final view and recommending a firm action plan for impermeabilisation, it would be useful to have a second opinion on these aspects from some expert deeply involved in concrete technology. For this purpose, the panel suggests interaction with Dr. David stark, Project Manager Construction Materials Construction Technology Laboratory Inc. of USA. He could be invited to visit Hirakud for about a week in later February 1992 when panel may fix up a meeting to coincide with his visit. Subsequently Mr Stark visited the Dam and submitted his report as described earlier, vide Sec.4.7.3.

The impermeabilisation of zone upstream of drainage line is to be generally done by epoxy grouting, but where so found possible, cement fly ash grout may be used to effect economy.

One of the adverse effects of cracking is the ingress of water into the body of the dam resulting in leaching which will lead to progressive deterioration of concrete. This has to be guarded against by impermeabilising the upstream and/ or the zone between upstream face and galleries. The sealing of cracks in upstream face should be continued. Medium to long range measures are required to improve durability of concrete.

4.7.7. Effectiveness of Epoxy Grouting :

Two coring holes 60 cm deep were drilled along grouted crack in Block 6 and 7, at EL 174.61m. Cores recovered gave very good bond in the cracks concrete with intrusion of the Epoxy veins into cracks at different inclinations occurring within the concrete mass covering an aureole upto (±) 2.5 cm beyond the main crack line. The grouted depth extended upto 40 cm(=) and no cracks were visible beyond. The cores also revealed hardened gel which punched into the epoxy material in the form of micro mushrooms before the epoxy hardened. It was noticed that small pores which occur within the concrete were in general not penetrated by the epoxy. This may be due to lack of inter-connection air filling within the pores.

The results of two treated sluices no.53 and 54 on the right spillway were found to be excellent. This is very useful work and should be completed with proper quality control (Refer 8th & 11 th Report of HDCRP).

4.8 Treatment of cracks

4.8.1. Treatment on Upstream face both under-water and exposed faces

The upstream face cracks as existing have been treated by epoxy grouting from 1975 onwards. Cracks of exposed faces were treated departmentally from 1975 to 1983. Cracks in Blocks 35 to 40 and part of 41 on the right spillway were treated under water by M/s FPCC Ltd. Bombay from 3/88 to 1/94. The remaining underwater treatment of all cracks from blocks 41 to 46 and treatment of other structural defects (OSD) in Block 35 to 46 was completed by M/s THCIL from 5/95 to 4/98. The same organization has done the underwater treatment of cracks and OSD in blocks 3 to 22 on the left spillway from 1/96 to 6/98. Thus entire upstream face area exposed as well as under water has been treated. A summary of the crack lengths treated and consumption of grout, as supplied by the Department is given in Table 4.20.

4.9 Analysis of Results by HDCRP

• A two dimensional gravity analysis of the dam section on the assumption of crack and consequently 100% uplift extending upto the upstream face of the gallery and 50% relief on that plane, shows that the dam is safe against failure both in sliding and overstressing. This is so even when the bond on the test surface is entirely neglected.

• A 3-D analysis has been advised for a full-block including foundations to have a clearer picture in this respect. However the result ot 2-D analysis are so well within safe limits that there is little possibility of any surprise when 3-D results are obtained.

• Compressive strength of concrete corresponds well with the design requirements. There has been no perceptible loss of strength over the period from 1983 to 1990.

• As already commented earlier it is necessary to get stress measurements in concrete by overcoring and installation of orthogonal strain meters in the core holes as detailed in the report of 4th meeting of the panel. Swedish Power Board Stockholm was suggested to be one such agency. This is essential pre-requisite and need to be expeditiously followed. But DSRP in their 14th report (vide Sec.4.13.1-e) did not agree with this suggestion and therefore not carried out.

• NCCBM investigated 85 mm dia concrete cores from the spillways. The results indicated occurrence of Alkali Silica reaction in concrete in these cores taken from the interior of the spillways.

• Results of tests on water samples carried out by Hirakud Research station were sent to NCCBM for their analysis. It was found that in majority of samples from reservoir water and the operation gallery, the water quality in terms of pH, total alkalinity, hardness and calcium content were within permissible limit of potable water. Only the samples from formed holes no.F33-F-2 & F-58-F2 in foundation gallery were found to contain more calcium and relatively higher pH and hardness. It was concluded that the quality of reservoir water was such as not to cause or promote ASR, However, it was decided to continue monitoring the quality of water from these and other formed holes and water samples are recommended to be tested for dissolved CO_2 content and temporary hardness. (Ref. Table 4.16, 4.18 & 4.19)

4.10 Instrumentation - Comments by HDCRP :

4.10.1 Temperature Measurement :

C.W.P.R.S, Pune has been entrusted with monitoring of temperature in block no. 41. It was noted that readings have been taken after repairing the read out unit, in July, 1991 and December, 1991. Based on the advance copy of the interim report sent by C.W.P.R.S., temperatures in the body of the dam seem to have attained stable conditions. The Committee suggested that readings may be continued to be taken at least once a month for one more year. Simultaneously, readings regarding ambient temperature and temperature of lake water at different levels be taken. Temperatures have been recorded at 6 different depths from 09.1.1991 to 21.11.95 vide Sec 4.14.10 of this report.

4.10.2 Restoration of Plumb bob :

It was explained by the officers of C.W.P.R.S. who visited the site and found that the plumb bob shaft was not vertical and nothing can be done to rectify the misalignment at this stage, and therefore alternative proposal of measuring deflection of the dam with the help of tilt meters has been planned. The Committee agreed to this proposal and to drop the restoring of plumb bob.

Actually one plumb bob device in Block No. 57 has been activated and observations have been taken from 1986 to 1997 vide sec 4.14.6 of this report.

4.10.3 Inclinometer installation :

It was noted that C.S.M.R.S. who had been entrusted with this task had expressed doubt regarding the efficacy of the inclinometer they had for measuring very small deflections likely to be undergone by a gravity dam. The Committee agreed to delete this proposal especially in view of the now planned proposal of installing tiltmeters for the same purpose.

4.10.4 Relative Displacement by Convergence Measurement :

It was noted that C.S.M.R.S. had been entrusted with the task and observations were being taken regularly by them. Chief Engineer would request C.S.M.R.S to send the report on observation so-far made to be presented to the Committee at the next meeting. Observations have been recorded. For conclusion Sec.4.14.5 may please be referred to.

4.10.5 Tell tales :

The readings have been taken regularly by the Hirakud Research station. Their reports were put up to the committee during its 4th & 5th meetings. Some observations have also been taken by H.D.R. Division, Burla.

Gist of GTT and PTT has been given in Sec.4.14.11 of this report.

4.10.6 Collimation

Survey of India was entrusted with this study and two series of observrations were made one in 1986-87 and the other in 1987-88. The results showed that there were no significant differential movement observed between the two sets of readings. (vide Section 4.14.7)

4.10.7 Installation of joint meters

C.W.P.R.S. was entrusted with this task and they had taken action to procure the same. After some discussion regarding the scope and purpose of joint meter installation it was clarified that the purpose of installing joint meters at this stage is to monitor the development of compressive strains in the cross canyon direction, if any, due to continuing expansion of concrete and they are not meant for the relative displacement between adjacent blocks. Initial and final readings were taken respectively on 6.5.96 and 16.6.97. No analysis has been done vide Sec.4.14.14 of this report.

4.10.8 Installation of extensometers to monitor cracks

The Committee was informed that the Hirakud Research Station and C.W.P.R.S. had been entrusted with this task and both the Institutes had been taking observations.

Observations have been taken from 1974 to 1996 to measure the width of the crack vide Sec 4.14.1 of this report.

4.10.9 F.E.M. Studies

C.W.C. was entrusted with these studies. During 5th meeting of the Committee C.W.C. had submitted the report on 3.D F.E.M. study on block no.45. The study had shown that even with cracks extending upto 5.25m from upstream face, the stresses were within safe limits. Further, it was indicated the existence of crack extending upto 5.25m from upstream face and on the full cross sectional area of the dam seems untenable as the stress at the tip of crack is compressive. Possibly, there exist uncracked ribs or zones within the plane of suspected cracks so that compressive stresses are carried by them; or the depth of penetration of continuous cracks from upstream face towards downstream is less than the assumed 5.25m.

The Committee considers it important that efforts should be made to measure the internal stresses in the body of the dam by approaching some more institutes in this regard as outlined in the 5th report of the Panel. It is an essential data for a realistic assessment. Possibility of high compressive stresses exist in the body of the dam in the direction parallel to the axis of the dam. This will have a vital positive influence up to a certain extent till these stresses exceed the strength of concrete causing damage to it.

In this regard Sec.4.3.3 of this chapter may please be referred to.

4.11 Cavities Upstream of Sluice Barrels

The HDCRP have been conscious of the long cavity in front of block joint 39/40 in view of the large leakage entering foundation gallery. The Panel have repeatedly underscored the potentially dangerous implication of the existence of such a large cavity and leakage and emphasised the importance of treating it at the earliest. Detailed guide lines of how it might be done were given in the report of the 10th meeting of the panel.

The Department have carried out a comprehensive underwater survey of spaces upstream of all sluices and discovered the existence of many more cavities though none so prominent as that near block joint 39/40. According to the data furnished by the Department there are 23 such cavities, with a total volume of 37.86 m^3 on the left spillway, while there are 4 cavities on the right side with a total volume of 85.84 cum out of which that between block joint 39 and 40 accounts for 80.73 cum.

The cavities have been partly treated vide Sec 4.14.9.2 of this report. Balance treatment of cavities may be taken up without further delay.

4.12 Miscellaneous Items

4.12.1 Repair to Sluice Gate

The Project authorities have under taken to repair and rehabilitation of all the sluice gates which have now been under operation for over 40 years. There are 40 Sluice gates on the left Spillway and 24 sluice gates on the Right Spillway. The gates are of fixed wheel type and of size of 12' x 20' 4". Each gate has 16 wheels of 62 cm diameter with a flange of 12cm width. The wheels move on a rail track with rail of varying flange sizes, being 95 mm between El. 510 ft to 533 ft, 70mm between El. 533 ft to 554.25 ft, and 30mm between EL. 554.25 ft to 556.5ft.

The Panel inspected the workshop where the gates were under repair at Hirakud Island, in between both right & left spillway. There is large amount of pitting on the surface of the skin plate of the gate. The pitting is proposed to be repaired by back filling pitted surface with weld materials and then grinding it to the required profile. The Project Organisation has rightly decided to over-lay a layer of weld materials by sumbermissible arc welding (SAW) process on the entire affected zone and then grinding it to the required profile.

Prior to laying such welded materials over the affected area, the surface is required to be treated by extensive sand blasting. One such treated surface was seen by the Panel during inspection. The Panel suggested the following guidelines which may be followed for cleaning the surface to be treated prior to welding so as to ensure perfect bonding of the newly laid weld with the original plate.

(i) The surface of the skin plate be given a general sand-blasting treatment to remove paints, grease and rust flakes etc.

(ii) Pitted areas should be marked and identified for treatment.

(iii) Scrape the surface by mechanical chisel to remove narrow and deep pits as far as possible by minimum scraping. This seems necessary because the pits as they exist at present can't be back filled with weld.

(iv) Final Sand-blasting of the surfce either original or scraped-be carried out immediately prior to welding. Indication of satisfactory final sand blasting would be to obtain a silvery shinning surface after the sand-blast.

(v) Weld should be deposited on such freshly treated surface to ensure perfect bond.

4.12.2 Earth Dam Section

The subsidence of upstream riprap of right earth dam in the certain reach was brought to the notice of the Panel at its ninth meeting. The Panel recommended construction of 8 to 10 monuments to measure future movements if any. The completion of these monuments was noted by the Panel at its tenth meeting. The monuments have since been kept under observation; no movement either vertically or horizontally has been found. *It can be surmised, that the slope is now stable and the subsidence presumably occurred soon after construction.*

4.12.3 Snapping of Bolts in Service Gate Track Rails

Surveys of snapping of bolts during 1997 and also in June'98 show that only three bolts

of right spillway and none on left spillway were found to snap on light hammmering. This is again an improvement of previous situation.

In this connection, the attention of the Panel was drawn to the observations of Sri I.Azariah, Joint Director, CWPRS, Pune, who visited the Project on 15th and 16th Jan 99. In his opinion the main cause of bolt snapping was gate vibration. Most of the bolts have snapped above EL. 162.2m, where the track rail section has been reduced and the bolt spacing increased from 15.2cms to 45.7cms. As a remedy he proposed welding of joints between track rails at two locations where the section has been reduced, and also interconnection of side guide rails and down-stream track rails by welding steel plates or any other suitable section.

The Panel did not agree that bolt snapping has been caused by vibrations and not due to relative displacement by ASR activity. In the first place the sluice gates are never operated at partial opening and any vibration would be confined to the short duration of opening or closing of gates. Secondly the contact between the gate and track rails is through roller wheels which can hardly transmit the large shear stress needed to fracture the bolts. Further the rate of snapping of bolts definitely shows a decliling trend, while frequency of gate operation is constant.

The Panel do not favour welding of track rail joints as this may aggravate vibration effect, if any, by transmission to the upper sections. However, stiffening may be done in two gates on an experimental basis to observe its effect. (*This has not been tried*)

4.13 Dam Safety Review Panel (DSRP)

Hirakud Dam was included under Dam Safety Assurance & Rehabilitation Projects (DSARP) of Govt. of India(GoI) for which credit equivalent to US$ 130.00 million and loan of US$ 23.00 million was received from International Development Association (IDA) vide credit no.2241-IN and loan no 2325-1N. The total project cost was Rs 4560/- millions (U.S$ 196.80 million at currency equivalent to 1 US$ = Rs 23.17). The agreement was signed with IDA on 10.6.1991. Out of total project cost, provision for Orissa was Rs1273.0 millions.

Keeping this in view, GoO in their Notification no Irr-WB-630 dt 6.3.1991 and ammendment no Irr-1-WB-33/91-21311 dt 6.8.1991 constituted a Panel called D.S. R.P with following members.

1.	Dr. R.S Varshney	Chairman
2.	Sri M.G. Padhye	Ex-Chairman CWC, Member
3.	Dr Satischandra	Member
4.	Sri G.S.M Rao	Member
5.	Sri S Balasubramanium	Member

Total 14 meetings were held between 10.10 1991 to 12.10.1998 at Burla. Bhubaneswar, New Delhi and Pune after inspecting the dam. They had detailed discussion with Project engineers and Govt. of Orissa officials. They recommended for creation of 'Hirakud Dam Rehabilitation Division' which started functioning from June 1992 upto 30.4.2003 and later merged with Main Dam Division.

The DSRP was reconstituted in 2003 vide GoO no 2240/WR dt 02.07.2003 with following members.

i)	Sri S.D Mohile	Chairman
ii)	Sri Suresh Chandra	Member
iii)	Sri R.C. Tripathy	Member
iv)	Sri R.C. Rath	Member
v)	Sri P.C. Dash	Member-Secretary

The terms of reference are (a) To examine and review adequacy of investigation, design and hydroloy of the project. (b) To suggest remedial measures to ensure safety of the structure. (c) To

inspect dam site and review the distress condition along with recommendations, suggestions for necessary measures. This reconstituted Panel inspected Hirakud Dam from 28th to 30.5.2004 and submitted their report which is more or less similar to the report of DSRP for which details have not been included in this report.

The pertinent observations / comments of above two Panels are furnished as under :

(a) 3rd Report of DSRP (23-24.09.1992)

• Accepted Sri Narendra Singh's (Retd. C.E., U.P.) report on the upgrading measures of hydraulic gates and operating equipments :

• About 0.5m settlement of main dam to be made good.

• As per recommendations of HDCRP drainage holes may be redrilled/reamed.

• The Panel agreed to invite Dr. Stark from U.S.A. to give his expert views on A.S.R.

(b) 5th Report of DSRP (24-26.6.1993)

• DSRP advised for surveillance on behaviour of dam without any laxity.

• Suggested for early completion of proposed remedial measures i.e. sealing of cracks developed on the upstream by grouting.

• DSRP asked to contact the manufacturer for the drawings of main sluice gate, emergency gate and hoists. Sufficient copies of drawings may be kept in safe custody.

(c) 6th Report of DSRP (17-19.11.93)

• Damages in sluice barrel No. 53 & 54 (Block 41) which were repaired by Epoxy grouting has stood well for one flood season. Its efficacy for one more season may be observed before taking up repair to other sluices.

• Mapping of cracks in Power House and Power Dam may be done.

• TBL of right and left dykes may be restored to design level. (*This has been subsequently attended to.*)

• In 1976, P.M.F. was assessed as 81334 cumec (28.74 lakh cusec). CWC may be requested to expedite the review.

• DSRP agread to the suggestion of State Government to check the stability of Earth Dam and Spillway for higher MWL of 635 ft (193.55m), 638ft (194.46m) and 641 ft (195.38m) through CWC.

• Seismic surveillance for the reservoir was approved in principle.

• Upstream slope of Earth Dam between RD 6600ft to 14100 ft may be brought back to the designed slope.

(d) 7th Report of DSRP (04-6.4.1994)

• CWC and Govt of Orissa officers agreed for a common integrated system to be operated in future for the flood forecast, which is being followed now.

• Grouting to seal the cracks may be undertaken for all the sluice barrels including the patches where signs of pitting, potholes and erosion are noticed.

• Restoration work to slope protection to reservoir periphery at Right embankment completed.

• No problem of misalignment, vibration or uneven wear and tear experienced in Power House so far. In Power Dam portion the length of crack upto Jan.1994 is 1785m and those in Power House is 2271.3m. More than 70% of crack is less than 1mm in thickness.

Drainage problem in RL 470ft. gallery seems mainly due to leakage from turbines which needs to be minimised.

• Sealing of cracks in the upstream face of left spillway be done as that of right spillway.

• To ascertain the travel of epoxy grout in right spillway, 3 to 4 core drills be carried out upto 60cm depth.

• DSRP agreed for the proposal to improve the drainage at the toe of earth dam between RD 12210 to 14100 ft by filling the deeper portion with free draining rubble and chips and remaining portion with available material from nearby mounds to remove water pool and possible growth of grass. *(This work has been completed.)*

(c) 14th Report of DSRP (20th-25th Oct.1997)

• Sealing and grouting of cracks of Right Spillway have been completed and have withstood satisfactorily. No glass tell-tales have broken. Repair to right spillway basin with epoxy mortar has remained in tact after one flood season.

• Proposal for stress measurement by over coring may be dropped as suggested by HDCRP vide sec. 4.9 of this report. Further the Panel did not agree with the suggestion of HDCRP to grout the concrete between upstream face and ogee of right spillway.

• DSRP reaffirms that under water treatment of cracks and other structural defects may be completed for left spillway similar to that of right spillway.

• Procurement of Bathy sphere may be dropped.

• The Pannel recommended to continue the monitoring and observation of following instruments.

(a) Instrumentation —

(i) Tilt meter, (ii) Tell-tales, (iii) Extensometer, (iv) Joint meter and (v) Strain meter in roller track.

(b) Monitoring —

(i) Seepage, leakage & ambient temperature (ii) Water quality (iii) Long term behavior of epoxy (iv) logging of cracks and (v) Subsidence through monuments

• Sedimentation study to continue at regular intervals.

• Upstream face impermeabilisation - This new technique which has not been tried in a tropical climate like India may not be appropriate to be implemented (Refer Sec. 4.7.6 of this report).

• Modification of Energy Dissipating Arrangement (EDA) - Protective measures on the downstream of bucket to rectify the damages caused due to erosion may be taken up.

• Surface protection to the top of the dam and dykes was accepted.

• Seismological network as suggested by CWPRS was agreed in principle.

• Sealing of leakage between Block joints 39-40 - DSRP suggested that the leakage from the joint to the drainage gallery be reduced. Besides the flumed portion above the cavity is made integral with the foundation which have been complied.

4.14 Instrumentation & Monitoring

4.14.1 Increase / decrease of Crack widths in Spillways by Extensometer

On the recommendation of Hirakud Dam crack Review panel (HDCRP) the Hirakud Research Station, Hirakud started observation of crack widths by Extensometer in both spillways. Initially observation started from 02.01.1974 at 16 locations. Subsequently they were increased to 48 points as detailed here (Ref- Annual Report of H.R.S. 2001 compiled in Aug.2002)

a) Operation Gallery of right spillway (RL557') - 11no (during 1974) and 12 nos in 1996 totalling - 23 nos

b) Foundation Gallery of right spillway (RL 497') - 8no (since - 1977) and 4 nos. since 1996 totalling -12 nos

c) Operation gallery of Left spillway (RL557') - 13 no (since 1987)

Initially observations were taken daily and later on bi-weekly or at more intervals.

Extensometer is an instrument to measure increase / decrease in width in different directions due to developement of cracks. Extensometers were fixed with dummy points where there is no crack near the crack to be observed and the elongation in width of crack is compared. The detailed positions of extensometer points with extensometer readings from 1974 to Dec. 2008 are available.

Two types of measurements have been adopted i.e. (a) 4 point measurement for some crack and (b) 6 point measuremnet for other cracks as indicated in sketches below:(Drg. No. 4.1)

The increase / decrease in width of cracks in inch upto end of December 2001 are given in Tables 4.5, 4.6 & 4.4 for (a) operation gallery of right spillway (b) foundation gallery of right spillway & (c) operation gallery of left spillway respectively. The maximum increase of crack width up to end of 2001 are :

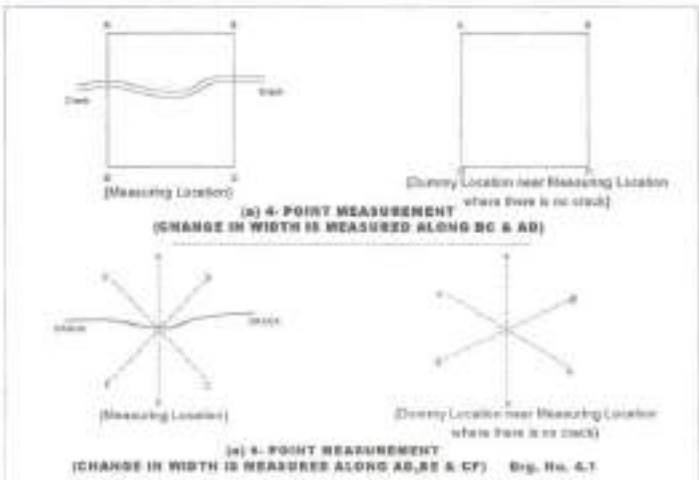
(a) crack -1, block-4 of left spillway operation gallery for AD = 0.0324 inch (0.823mm).

(b) Crack no 1, block -35 in operation gallery of right spillway for AD is 0.0460 inch (1.168mm)

(c) Crack no.4 block-40 in foundation gallery of right spillway for CF is 0.0215 inch (0.546mm)

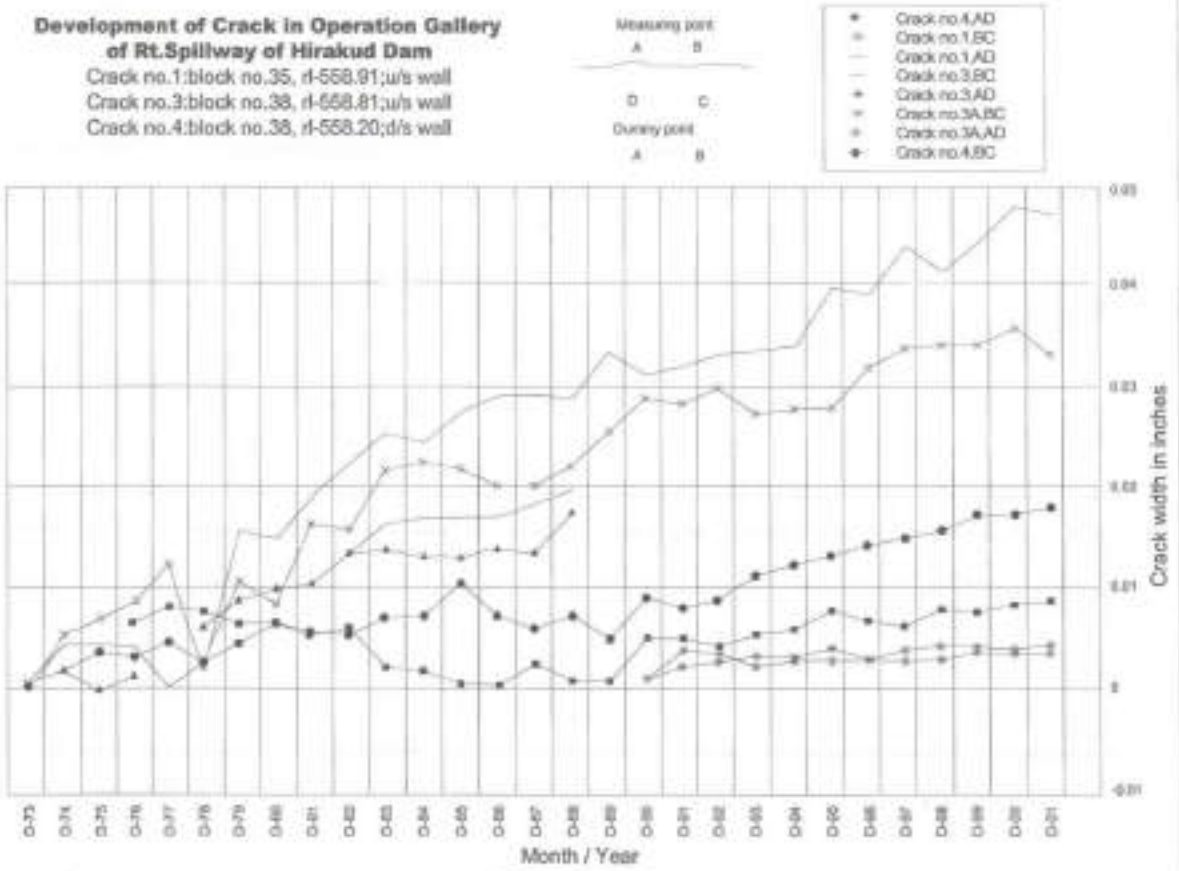
Two typical graphs (Drg. no 4.2 & 4.3) showing development of cracks in inches in Operation gallery of right spillway for cracks 1, 3, 3A, 4 and for cracks in Foundation Gallery (crack 1, 6) in block 36 & 45 of right spillway respectively from initial installation period of 1974 to end of 2001 are enclosed for reference.

N.B. - In initial stage the thickness of cracks were being measured by plastic filler gauges supplied by M/s ESBD, Holland.



Development of Crack in Operation Gallery of Rt.Spillway of Hirakud Dam

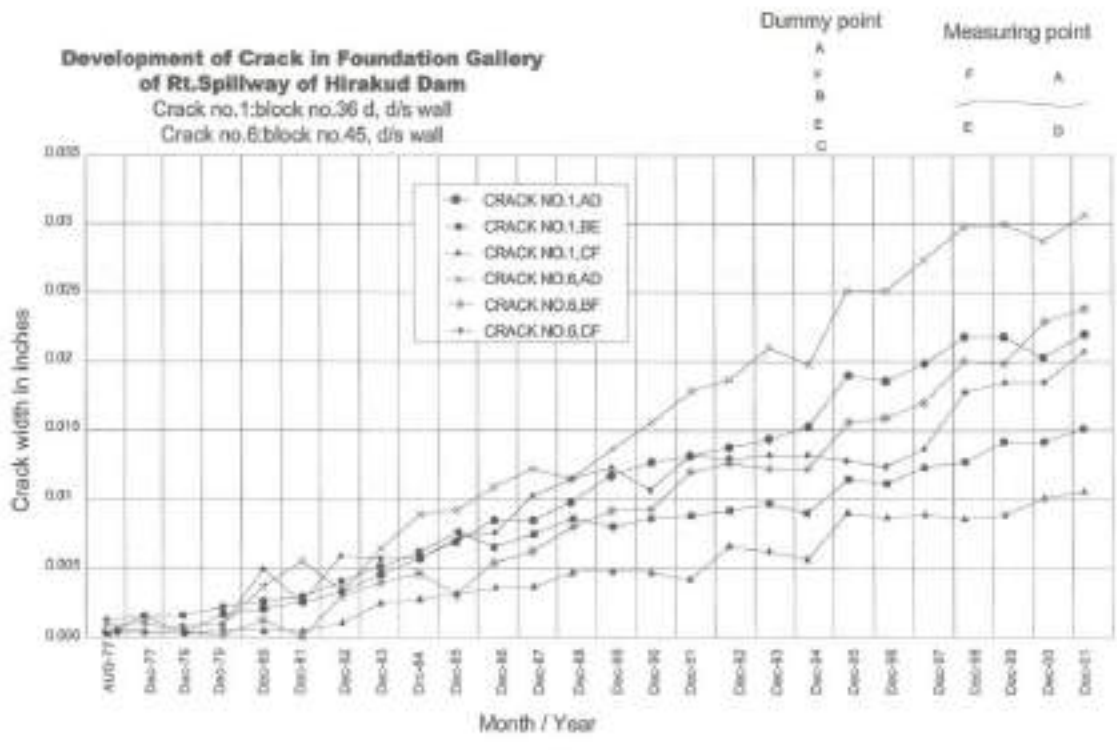
Crack no.1:block no.35, rl-558.91;u/s wall
Crack no.3:block no.38, rl-558.81;u/s wall
Crack no.4:block no.38, rl-558.20;d/s wall



DRG No. 4.2

Development of Crack in Foundation Gallery of Rt.Spillway of Hirakud Dam

Crack no.1:block no.36 d, d/s wall
Crack no.6:block no.45, d/s wall



DRG No. 4.3

Fig

Table no 4.4 : Opening & closing of cracks in operation gallery of left spillway (since initial observation)

Sl no	Crack no	Block no	RL (ft)	Location of crack	Date of initial observation	Line along Points	Increase decrease in inches upto Dec. 2001	
							Increase	decrease
1	2	3	4	5	6	7	8	9
1	1	4	564.50	D/S wall (4/5)	21.8.97	BC	0.0115	
						AD	0.0324	
2	1A	3	564.58	D/S WALL (1/2)	21.2.91	BC	0.0004	
						AD	0.0023	
3	2	6	564.30	D/S wall (6/7)		BC	0.0094	
						AD	0.0050	
4	2A	7	564.40	D/S wall (9/10)		BC	-	0.0029
						AD	0.0044	
5	3	12	562.50	D/S wall (20/21)		BC	0.0060	
						AD	0.0012	
6	4	16	564.00	D/S wall 27/28		BC	0.0078	
						AD	0.0134	
7	4A	17	564.40	D/S wall 28/29		BC	0.0101	
						AD	0.0055	
8	5	17	561.50	D/S wall (30/31)		BC	0.0122	
						AD	0.0080	-
9	6	19	564.50	D/S wall (33/34)		BC	-	0.0282
						AD	0.0178	
10	6A	20	564.25	D/S wall 35/36		BC	0.0101	
						AD	0.0074	
11	7	21	562.30	D/S wall 37/38		BC	0.0120	
						AD	0.0119	
12	7A	22	562.40	D/S wall 38/39		BC	0.0044	
						AD	0.0006	
13	8	22	564.30	D/S wall 39/40		BC	0.0053	
						AD	0.0109	

Table No. 4.5 Opening / Closing of cracks in operation gallery of right spillway (Since initial observation) from H.R.S. report - 2001

Sl No.	Crack no	Block no	RL(ft)	Location Crack	Increase decrease in inches upto Dec. 2001	
					Increase	Decrease
1.	2	3	4	5	6	7
1	1	35	558.91	U/S wall 41/42	BC 0.0333 AD 0.0460	- -
2	1A	34	560.90	U/S wall B.S.G.	BC 0.0041 AD -	- 0.0002
3	1B	31	563.00	Right wall Entrance	BC 0.0016 AD -	- 0.0015
4	1C	31	561.04	Left wall Entrance	BC 0.0011 AD 0.0006	- -
5	2	35	558.48	D/S wall 41/42		-
6	2A	35	558.48	D/S wall 41/42	BC - AD -	0.0102 0.0070
7	2B	36	558.90	D/S wall 42/43	BC - AD -	0.0022 0.0025
8	2C	36	559.70	D/S wall 44/45	BC 0.0035 AD 0.0055	- -
9	3	38	558.91			-
10	3A	38	558.91	U/S wall 47/48	BC 0.0046 AD 0.0030	- -
11	4	38	558.20	D/S wall 47/48	BC 0.0169 AD 0.0072	- -
12	4A	39	558.30	U/S wall (49/50)	BC 0.0010 AD 0.0009	- -
13	4B	41	558.20	D/S wall (52/53)	BC 0.0155 AD 0.0134	- -
14	5	42	558.89	U/S wall 54/55		- -
15	5A	42	558.89	U/S wall 54/55	BC 0.0134 AB 0.0001	- -
16	6	42	559.17	D/S wall 54/55		- -
17A	6A	42	559.17	D/S wall 54/55	BC 0.0014 AD 0.0049	- -
18	7	47	562.94	Right wall Adit	BC 0.0105 AD	- 0.0030
19	8	47	562..94 Adit	Left wall Adit	BC AD	- -

Table No. 4.6 Opening / Closing of cracks in Foundation gallery of right spillway (since initial observation) (From H.R.S. Report 2001)

Sl. No.	Crack No	Block No	RL(ft)	Location of Crack	Date of initial observation	Line along Points	Increase / decrease in inches upto Dec. 2001	
							Increase	Decrease
1.	2	3	4	5	6	7	8	9
1	1	36	506.00	D/S wall		AB	0.0202	–
						BE	0.0153	–
						CF	0.0011	–
2	1B	35 F2	501.60	U/S wall		BC	–	0.0071
						AD	–	0.0025
3	1C	34 D3	499.60	D/S wall	31.1.97	BC	0.0046	–
						AD	0.0052	–
4	2	37	501.50	U/S wall		AD	0.0121	–
						BE	0.0090	–
						CF	0.0092	–
5	3	39	501.50	U/S wall		AD	0.0179	–
						BE	0.0132	–
						CF	0.0134	–
6	4	40	501.00	U/S wall		AD	0.0030	–
						BE	0.0076	–
						CF	0.0215	–
7	5	41	501.50	U/S wall		AD	0.0073	–
						BE	0.0049	–
						CF	0.0062	–
8	7A	51	502.00	D/S wall	16.4.99	BC	0.0075	–
						AD	–	0.0007
9	8	56	502.00	D/S wall	14.1.97	BC	–	0.0024
						AD	0.0037	–
10	9	56	501.00	U/S wall	14.1.97	BC	0.0043	–
						AD	–	0.0006
11	10	57	500.60	D/S wall	14.1.97	BC	0.0025	
						AD	–	0.0020
12	10 (O.g)*	42	560.07	D/S wall		AD	0.0032	–
						BE	–	0.0080
13	10A (O.g)	44	558.90	D/S wall		BC	0.0002	–
						AD	–	0.0041
14	10B (O.g)	45	558.10	U/S wall		BC	0.0028	–
						AD	–	–
15	9 (O.g)	42	567.89	U/S wall	15.2.90	AD	0.0066	–
						BE	0.0005	–
						CF	–	0.0044
16	9A (O.g)	42	568.60	U/S wall	12.4.91	BC	0.0063	–
						AD	0.0137	–
17	9B (O.g)	42	559.50	U/S wall	29.11.96	BC	0.0045	–
						AD	0.0080	–
18	9C (O.g)	42	559.00	D/S wall		BC	–	0.0051
						AD	0.0036	–
19	9D (O.g)	43	562.10	D/S wall		BC	–	0.0009
						AD	–	0.0002

(*) O.g - Operation gallery of right spillway.

4.14.2 - 3D Gauge Observations (by dial gauge)

Three dimensional movement of some cracks in Hirakud spillways were observed by fixing dial gauges in three perpendicular directions as follows :

xx - To measure movement of crack along the axis of the dam.

yy - To measure movement of crack perpendicular to axis on horizontal plane.

zz - To measure movement of crack perpendicular to axis on vertical plane.

(Refer Drg No.4.4 attached)

The locations of 3-D gauges fixed and period of observations are given below in Table 4.7

Table 4.7 - Location of 3D Gauges

Sl No	Spillway	Block/RL (ft)	Gate No.	Distance from C.J (ft)	Period of observation
1.	Right spillway (U/S wall operation gallery)	44/ 558.5'	59	xx (pt.2) 12'-1" yy (pt.1) 12'-3" zz (pt.3) 11'-2"	03.01.1975 to 29.07.1987
2.	Left spillway (upstream wall of Operation Gallery)	16/ 560'	-		06.7.1990 to 18.01. 1992
3.	Right spillway (upstream wall of Operation Gallery)	38/ 560' at 0'6" from Block joint	-		13 08 91 to 30.04.97

i) The gauges were under repair from 12.8.80 to 19.09.90

ii) The observations were taken by Hirakud Research Station at about 1 month interval

iii) I small division of dial gauge = 0.0001 inch

(+) Reading indicates contraction

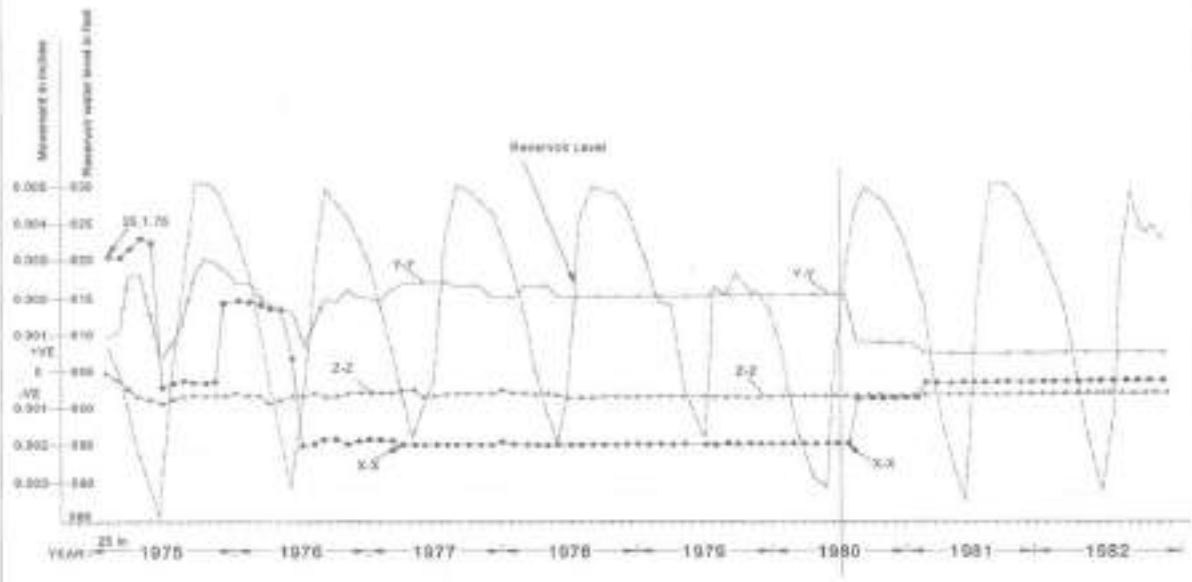
(-) Reading indicates expansion

iv) No further reading taken after 29.07.87

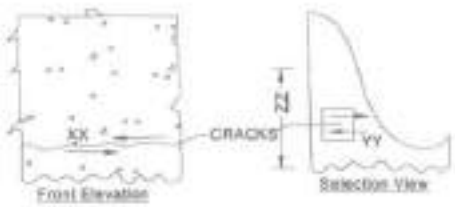
v) Dummy points were fixed in no crack zone in 3-directions for comparison from the observed values in block-44 of Right spillway. It can be inferred tht there is some movement of crack along x-x direction. There is some relative movement between concrete mass on either side of crack in y-y direction which had already been reduced and remained constant. (vide Table 4.8)

Due to non functioning of gauges and non-development of cracks in three dimensions in block No.44, the observation point was shifted to block No.16 of operation gallery of left spillway in month of April 1990 after repairing the gauges. The observation again started from July 1990.

Similarly Hirakud Dam Rehabilition Division fixed dial gauges in block-38 of right spillway as indicated above and took observation at interval of about a month.



Movement Recorded by Dial Gauge fixed on U/S Wall of O.G. (Block 44)



XX- Movement along the axis of dam.
YY- Movement of perpendicular to axis on horizontal plane.
ZZ- Movement perpendicular to axis on vertical plane.

Note: The directions marked are +ve



Fixing of 3-Dimensional Dial Gauge

Drg. No 4.4

3D gauge readings were taken in operation gallery in block 16 of left spillway as indicated earlier. The periods of observations were from 03.01.1975 to 25.07.1987 and from 06.07.1990 to 18.01.92. Last observation was taken on 27.12.1996 for one day only which showed nil reading. No further readings are available. The observations were taken approximately every forthnight.

Period of observation - Right spillway = 3.1.75 to 27.12.96 Left spillway = 6.7.90 to 18.1.92

Table 4.8 : Three Dimensional Gauge Reading :

(Dial gauge reading of Spillway operation galleries)

Location & Period	Reading (measuring point - Dummy = Difference)
Left spillway Block-44 Period 6.7.90 to 18.1.92 and 3.1.75 to 18.2.81	xx = 00 - (-0.0003) = 0.0003 yy = 00 -(0.0031) = 0.0031 zz = 0.00 - (-0.0007) = (+) 0.0007
Right spillway (block-44) Period 3.1.75 to 18.2.81 (by HRS)	xx = 0.00 - (-0.0008) = (+) 0.0008 yy = 0.00 - (0.0007) = (-) 0.0007 zz = 0.00 - (-0.0007) = (+) 0.0007
Period 20.2.81 to 27.12.96	xx = - 0.0004 - (0.0006) = (-) 0.0010 yy = + 0 0.0003 - (0.0000) = (+) 0.0003 zz = - 0.0005 - (0.0000) = (-) 0.0005

4.14.3 Measurement of Increase of crack length in Spillways & Power House

As already mentioned, cracks developed in Spillways, Power dam and Power house since beginning of its operation ie. from 1957 - 58.

These records were perused by the Committee of experts during 1981 - 83, by standing committee for long term surveillance of cracks of Hirakud between 1987 - 89, by Hirakud Dam Crack Review Panel during 1990 - 1999 & by Dam safety Review Panel between 1991 -98 & 2003. As per their recommendations regular measurement of crack lengths in different components were carried out.

The length measurements have been recorded in F.P.S. units upto end of 1991 and in MKS units from 1992 to end of march 1999. It is understood that no length measurenment is being carried out from April 1999 onwards.

The year wise development of crack lengths and cumulative lengths are recorded for following components.

(i) Left spillway operation gallery, (ii) Right spillway operation gallery, (iii) Left spillway sluice gate barrels, (iv) Right spillways sluice gate barrels, (v) Right spillway sluice gate shafts, (vi) Left spillway sluice gate shafts, (vii) Right spillway foundation gallery, (viii) Left Spillway foundation gallery, (ix) RL 628' gallery of power house, (x) Power House Penstock gate slots (at 628' gallery), and (xi) Machine shed rooms near RL 628' gallery of Power House.

Besides, crack lengths in upstream face of spillways and Power Dams have been measured by various organisations as given below :

a) U/S face of Right spillway from RL 595 ft - 610' - done departmentally.

b) U/S face of spillway under water below RL 595 ft done by F.P.C.C. in 1993

c) Under water mapping below RL 595' in Block no 35 - 40 by THCIL during 1998-99

d) U/S face of Power Dam.

The abstract of status of mapping of cracks for all above components is enclosed herewith vide Table No.4.9 Total Length of crack so far measured is 27150.77 m.

Table 4.9 : Abstract of Crack Lengths in Spillways & Power Dam etc.

Name of measuring organisatons	Sl No.	Component	Left spillway	Right spillway	Total (m)
Hirakud Dam Rehabilitation Division	1.	Foundation gallery	13.07	1303.62	1316.69
	2.	Operation gallery	1932.69	2790.70	4723.39
	3.	Sluice gate shaft	888.05	1638.52	2526.57
	4.	Sluice barrel	1804.79	4270.14	6074.93
	5.	Lower foundation gallery	-	33.95	33.95
		Total	4,638.60*	10,036.93(**)	14,675.53
Main Dam Division	6.	U/S Face (595' - 610')	320.00	103.00	423.00
Hirakud Dam Rehabilitation Division	7.	P.H exposed surfaces (628' gallery, Priming chamber,557' gallery etc)	-	1899.00	1899.00
F.P.C.C Bombay	8.	U/S face under water below RL 595'(in1993)	3795.00	4603.00	8398.00
THCIL, Calcutta	9.	Remapping underwater below RL 595' in Block 35-40 during 98-99	-	1013.86	1013.86
	10.	U/S face of Power Dam	-	741.38	741.38
Grand Total			8,753.60	18,397.17	27150.77m

* and (**) Yearwise development of crack given in Table No.4.11 & 4.12.

Analysis for moving average of total crack lengths for left and right spillways have been done from 1987 to 1999. These are annexed vide, Table no. 4.10.

Table 4.10 : Moving average of Crack Analysis

Year	Crack Increased during the year in(m)	Crack Cumulative upto end of year(m)	5 Year moving average of crack period/length in (m)	Increase over previous period (m)	% increse in moving average	Remark
1	2	3	4	5	6	7
			LEFT SPILLWAY			
1987		1535.11				
1988	242.60	1777.71				
1989	652.81	2430.52				
1990	205.45	2635.97				
1991	237.27	2873.24	1987-91/2250.51			
1992	181.06	3054.30	1988-92/2554.35	303.84	13.5	Deleting crack logged in new area and hair cracks, the percentages works out to be 4.72% & 1.88% for Left & Right Spillway respectively.
1993	238.80	3293.10	1989-93/2857.43	303.08	11.9	
1994	216.76	3509.86	1990-94/3073.29	215.86	7.6	
1995	132.03	3641.89	1991-95/3274.48	201.19	6.6	
1996	131.27	3773.16	1992-96/3454.46	179.98	5.5	
1997-98	(440.78)	(4213.94)	(1993-97/3686.39)	(231.93)	(6.71)	
	97.31	3870.47	1993-97/3617.69	163.23	4.72	
			RIGHT SPILLWAY			
1987		6986.65				
1988	315.78	7302.43				
1989	144.90	7447.33				
1990	278.13	7725.46				
1991	203.24	9728.70	1987-91 / 7478.11			
1992	81.50	8010.20	1988-92/7682.82	204.71	2.73	
1993	142.09	1852.29	1989-93/7852.80	169.98	2.21	
1994	153.42	8305.71	1990.94/8024.47	171.67	2.19	
1995	179.16	8484.87	1991-95/8176.35	151.88	1.89	
1996	145.67	8630.54	1992-96/8316.72	140.37	1.72	
1997-98	(764.75)	(9371.29)	(1993-97/8588.94)	(272.22)	(3.27)	
	161.02	8791.56	1993-97/8473.00	156.28	1.88	

Table 4.11 : Cumulative Crack length(m) upto period ending 1998-99 (Left Spillway)

Period ending	Operation gallery (RL 557')	Sluice gate barrel	Sluice gate shaft	Foundation gallery	Total (m)
1986	740.15	506.02	211.18	-	1457.35
1987	760.42	532.32	242.31	-	1535.05
1988	816.82	649.84	297.95	13.07	1777.68
1989	873.12	1084.63	459.02	13.07	2429.84
1990	947.30	1152.29	519.72	13.07	2632.38
1991	1037.11	1270.17	552.89	13.07	2873.24
1992	1099.41	1336.93	604.89	13.07	3054.30
1993	1148.24	1466.75	665.04	13.07	3293.10
1994	1202.82	1585.96	708.01	13.07	3509.86
1995	1262.74	1636.70	729.38	13.07	3641.89
1996	1349.75	1660.00	750.34	13.07	3773.16
1997-98	1680.68	1680.15	841.81	13.07	4215.71
1998-99	1932.69	1804.79	888.05	13.07	4638.60

Table : 4.12 : Cumulative Crack length (m) upto period ending 1998-99 (Left spillway)

Period ending	Operation gallery (RL 557')	Sluice gate barrel	Sluice gate shaft	Foundation gallery	Total (m)
1986	1321.91	3586.52	1236.14	569.13	6713.70
1987	1355.75	3635.10	1253.99	569.13	6813.97
1988	1384.34	3729.61	1302.79	625.84	7042.58
1989	1417.93	3795.52	1336.81	634.55	7184.81
1990	1467.63	3892.20	1369.01	726.57	7455.41
1991	1822.80	3954.22	1389.93	761.75	7928.70
1992	1858.05	3973.92	1400.18	778.05	8010.20
1993	1881.46	4060.93	1420.56	789.34	8152.29
1994	1923.64	4105.73	1435.37	840.97	8305.71
1995	2015.14	4140.07	1445.85	883.85	8484.91
1996	2092.21	4157.95	1457.35	923.03	8630.54
1997-98	2503.14	4186.99	1593.45	1049.43	9333.01
1998-99	2790.70	4270.14	1638.52	1303.62	10002.98
			Lower F.G.	33.95	10036.93

4.14.4 Periodic Assessment of Concrete in both the Spillways

Various tests were conducted periodically to ascertain the quality of hardened concrete as detailed below:

Table : 4.13 : Mechanical Tests : Comparative results

Sl. No.	Details of Test	1983 Value	1988 7(R)+4(L)=11no. Value	1990 (11 nos) Value	1992 (15 nos) Value	1992 (4 nos) Value
1.	Compressive strength kg/cm² (av)	292	279 270	357	421	406
2.	Modulus of Elasticity x 10^5 kg/cm²(av)	3.20	3.23 3.54	2.93	1.84	-
3.	Ultrasonic Pulse Velocity km/sec(av)	4.79	4.47 4.50	4.63	4.8	4.75
4.	Density kg/cum	-	-	-	-	2.31

Table : 4.14 : Chemical Analysis of Gel

Sl. No.	Constituent	1983	
		Left Spillway	Right Spillway
1.	Na2O	1.64	3.74
2.	K2O	4.33	12.88
3.	Na2O equivalent	4.49	12.22

Table 4.15 Chemical Analysis of Leached Material

Sl No	Properties	1988		1990	
		LI/12	LII/7	89(7)L	89(15)L
1.	Loss on Ignition(%)	45.04	43.83	44.65	44.34
2.	Silica (SiO_2) %	0.27	0.25	0.10	0.15
3.	Iron oxide (Fe_2O_3)%	0.10	0.05	0.09	0.10
4.	Calcium oxide (Cao)%	54.44	55.21	53.57	53.39
5.	Magnesium Oxide (MgO)%	0.06	0.40	1.06	1.37
6.	Sulphate (So_3) %	0.16	0.28	Trace	Trace
7.	Total Alkalies %				
	a) Na_2O	0.15	0.09	0.07	0.07
	b) K_2O	0.06	0.02	0.05	0.06
8.	Chlorides %	0.015	0.003	0.003	0.003
9.	pH	9.53	8.85	9.06	9.38
10.	Aluminium (Al_2O_3) oxide %	-	-	0.07	0.12

Table : 4.16 Chemical Analysis of Water Samples

Sl. No.	Properties	1983			1990			1992	
		D/s	Seepage	U/s	Left Spillway				
1.	pH	7.0	7.6	7.9	7.8	7.5	7.6	8.4	8.1
2.	Chlorides(ppm)	3.2	2.1	4.2	0.1	0.1	0.1	2.8	2.8
3.	Sulphates- ppm	5.7	Trace	0.40	Nil	Nil	Nil	9.1	12.8
4.	Total acidity for 200 ml sample (ml)	0.45	0.40	0.40	0.1	0.1	0.1	-	-
5.	Total Alkalinity for 200ml sample (ml)	3.5	3.70	3.60	2.8	3.0	2.6	3.0	3.0

Table : 4.17 : Chemical Analysis of Hardened Concrete

Sl. No.	Properties	Sample No.					
		1992		Beyond 1992			
		1991/3	1991/4	2	4	8	9
1.	pH	12.35	12.13	11.81	11.80	9.82	10.15
2.	Cement Content (% of mortar)	43.23	45.65	40.65	31.95	32.61	34.25
3.	Sulphates (% of Cement content)	1.94	0.85	2.09	1.85	1.26	0.58
4.	Chlorides (% of Cement content)	0.035	0.042	0.066	0.059	0.055	0.061

4.14.5 Convergence Measurement By Tape Extensometer

(Refer Observation / Recommendation by HDCRP in 6th meeting & DSRP in 14th meeting)

4.14.5.1. Introduction

Tape Extensometer is an instrument designed to measure the change in distance between two points which may be from 1m to 30 m apart. The measurement is made with the aid of a precison steel tape perforated at intervals and dial indicator reading with least count of 0.01mm. The measurement is made with an accuracy of (±) 0.05 mm.

The tape extensometer pins are installed at different locations by making holes in the wall with chiesel and hammer. Pins are fixed in the hole by using a mixture of araldite and sand. Measurements are taken after ensuring the setting of pins in the holes firmly.

Initially daily readings were noted which were extended to once in a week and fortnight subsequently.

4.14.5.2 Measurement of Convergence :

The convergence measurement in Hirakud Dam spillways was done in two phases. In first phase measurement was taken by the department during the period from 20.07.1985 to 04.11.89. In second phase between October 1991 to December 1993, the mesurement was taken by C.S.M.R.S New Delhi.

Both were analysed by C.S.M.R.S. and conclusion arrived, which are described below.

(a) By the Department

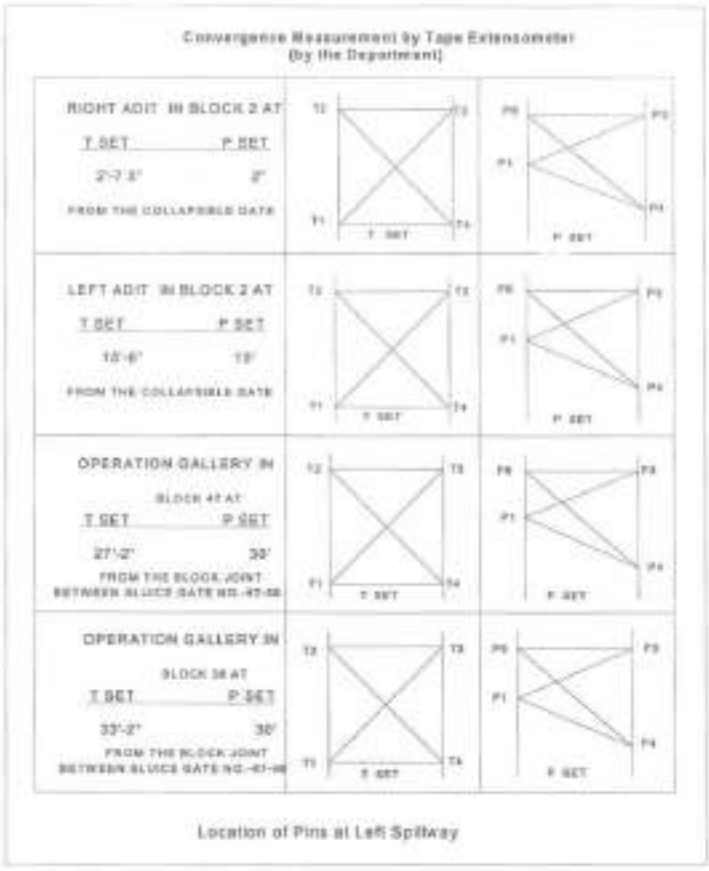
As per advice of the Expert Committee headed by Dr Y.K. Murthy arrangement was made to measure the divergence/ convergence of both the spillways by Tape Extensometers.

In 1st phase from 20.7.85 to 04.11.89, the department fixed 7 nos of pins (designated P1 to P7 on the vertical walls of the Adit & Operation galleries at following locations vide, Drg. No. 4.5

i) Right Spillway - Right Adit in block-47 at 2' from collapsible gate (20.7.85 to 2.11.89)

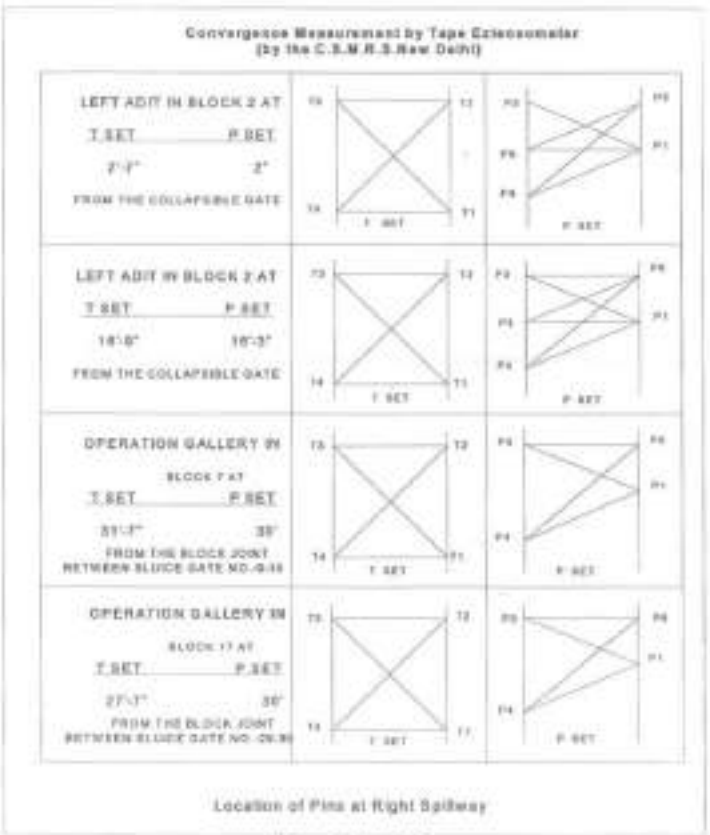
ii) - do - Right Adit in block-47 at 15' from collapsible gate. (24.7.86 to 2.11.89)

iii) - do - Operation gallery in block 43 at 30' from block joint between sluice no.57-58 (23.10.86 - 3.11.89)



Location of Pins at Left Spillway

Drg. No. 4.5



Location of Pins at Right Spillway

Drg. No. 4.6

iv)	Right Spillway	Operation gallery in block-38 at 30' from block joint between sluice gate 47-48 (24.10.86 to 3.11.89)
v)	Left Spillway	Left Adit block-2 at 2' from collapsible gate. (25.10.86 to 04.11.89)
vi)	- do -	Left Adit block-2 at 16' from collapsible gate. (25.10.86 - 3.10.89)
vii)	- do -	Operation gallery block-7 at 30' from block joint between sluice gate 9-10 (27.10.86 to 4.10.89)
viii)	- do -	Operation gallery block-17 at 30' from block joint between sluice gate 29-30. (27.10.86 - 4.10.89)

Measurements were taken approximately once in a month.

NB : Divergence shown as (-), Convergence shown as (+).

Conclusion (by CSMRS)

It is difficult to conclude about the trend from the sets of observatios. Only two sets of observation have been taken so far. However, it can be inferred that there is convergence of walls in the right spillway right adit and left spillway left adit. Observation made in the right spillway operation gallery have shown convergence at one point and divergence at the other point between the walls. Further monitoring by tape extensometer will reveal the factual data for interpratation.

(b) By Central Soil & Material Research Station, New Delhi

Executive Engineer, Hirakud Dam Project in his Lr. no 11472/19.7.1990, requested CSMRS to take up the study.

Measurements were taken at 16 locations including the 8 locations fixed earlier by the Department. The details of 8 locations fixed by CSMRS which are very near to the locations selected by the departments are given below :

i)	Right Spillway -	Right Adit in block-47 at RD2'-7.5" from collapsible gate
ii)	- do -	Right Adit in block-47 at 15'-6" from collapsible gate
iii)	- do -	Operation gallery in block - 43 at 27'-2" from block joint between sluice gate 57-58.
iv)	- do -	Operation gallery in block-38 at 33'-2" from block joint between sluice gate 47-48.
v)	Left Spillway	Left Adit in block-2 at 2'-7" from collapsible gate.
vi)	- do -	Left Adit in block-2 at 16'-9" from collapsible gate.
vii)	- do -	Operation gallery block-7 at 31'-7" from block joint between sluice gate 9-10
viii)	- do -	Operation gallery in block-7 at 27'-7" from block joint between sluice gate 29.30.

The readings of all the 16 locations were taken in 5 cycles i.e on dates 25.10.91, 25.6.92, 17.2.93, 9.7.93, & 22.11.93. The points are denoted by Tn in the attached figuges.

The conclusions as arrived by C.S.M.R.S. are as follows : (vide Report No.C-384 of CSMRS-New Delhi, June - 1995)

"From the five sets of observations taken so far it is seen that there are maximum convergence (+) of 2.05 mm and divergence (-) of 3.14mm in the right spillway. The left spillway shows the maximum convergence of 2.71 mm (+) and divergence (-) of 2.93 mm. The convergence and divergence are due to fluctuation of water level in the reservoir." No further observations have been made beyond 1993. Location of pins at Left & Right Spillways is shown in Drg no .4.6

4.14.6 Deflection Measurement of Spillway by Plumb-bob.

(Refer Observation / Recommendation of HDCRP in 6th meettings).

As per Hirakud Specification report (1951), Plumb bob devices were to be located at three places.

i) In block no.35 of Right spillway with top level at RL 637 ft and bottom level at RL 492.5ft of foundation gallery.

ii) In block no.57 of right Spillway with top at RL 637ft & bottom level at RL 502.2ft.

iii) In upstream retaining wall at RD 4940 ft of left spillway.

The plumb bob proposed to be installed in upstream retaining wall at RD 4940 ft of left spillway has not been installed, but have been installed at block no.35 & no57. The plumb bob holes are 16ft away from upstream face opening at foundation gallery. Both plumb holes were kept unused. In 1986, the right spillway plumb bob in block no 57 was activated. Probably the bob at block no 35 could not be activated due to the zig-zag hole, during construction. The observation in right spillway plumb bob of block-57 started from 16.12.86 and data were made available upto 26.03.1997.

From the observation, it is inferred that there is deflection of the spillway but the same could not be quantified as no analysis has been done.

4.14.7. Dam Deformation studies

(Refer Expert committee observation & HDCRP 6th meeting)

(a) First study

In connection with crack development in Hirakud Spillways and Power dam, the Project authority requested 'Survey of India' to study the crust movement and dam deformation in the area. For this purpose, a scheme of survey stations on either banks of river Mahanadi towards the downstream side of the dam was prepared after consulting Geological Survey of India.

It consisted of two parts -- (i) Safety zone and (ii) Deformation zone. In deformation zone, 5nos of dam deformation stations and 3 back sight stations were established. From these stations, 18 nos of dam targets were fixed on the downstream side of the dam wall at 3 different levels. The positions of deformation and back sight stations were fixed from the safety zone area by making geodetic measurements. The positions of 18 dam targets were determined by angular measurements from 5 deformation stations.

The scheme was first observed during 1986-87 at maximum water load and was repeated during 1987-88 at maximum and minimum water loads and again during 1988-89 at maximum water load of the reservoir.

A scheme of precision levelling was also made for monitoring the vertical movements. Stations in safety zone and deformation zone were connected with the precision levelling. In addition to above some more bench marks were established along the top of dam and inside the dam gallery. Levelling was done during 1986-87, 1987-1988 and 1988-89.

The compution of triangulation observation was carried out in plane rectangular system using 'variation of co-ordinates' method. The positions of the targets were computed from the deformation and back sight stations, using adjustment programme. For the final computation of cordinates of targets, the dam axis containing targets T-18 & T-2 was choosen as X-axis, and perpendicular to it as Y-axis, keeping T-18 as origin.

Result / Analysis :

(i) The maximum difference between two epochs i.e. (1987-1988) - (1986-87) was 0.97cm in Northing & 1.05 in Easting, while that between (1988-89) - (1986-87) was 3.92 cm in Northing & 1.49cm in Easting. This is for maximum water loads.

(ii) The maximum difference between two epochs i.e. (1988-89) - (1987-1988) was found to be 3.67 cm in Northing & 0.94cm in Easting.

(iii) The results at maximum water load show that there are unusual shifts or pressure on all the targets.

(iv) During 1987-1988, observations were taken at maximum and minimum water loads. The maximum and minimum difference was found to be 3.17 & 1.98cm respectively.

(v) Precision levelling was carried out during 1986-87, 1987-1988 & 1988-89 at maximum water load and during 1987-88 at minimum water load also. The results show that there are slight variations in the relatiive height of the bench marks, but these are not very significant.

(vi) It was suggested by Survey of India to repeat the observations at least for one or two more epochs at an interval of two years to indicate some confirmative trend on dam targets.

(b) Second study

As recommended during 1st study, the second study ws taken up during 1994 &1995. The observations were repeated in 1994-95 and in 1995 (summer) both at maximum and minimum water load respectively after a gap of five years. This study is based on the initiial epoch observations i.e. 1986-87 at maximum water load and 1987 -88 at minimum water load.

Result / Analysis : Horizontal movement

(i) Maximum water load

The results of 1994-95 compared with the results of 1986-87 show that the maximum difference between the two epochs i.e. in the co-ordinates of (1994-95) - (1986-87) was 0.72cm in Northing (y) and 0.37 cm in Easting (x)

Statistically tested, analysis shows singnificant changes in dam targets as under :

(1994-95) - (1986-87) - Northing (y) : T-1, T-14, T-15 & T-18 Easting (x). No significant change.

The results of (1994-95) - (1988-89) show the maximum difference between the two epochs as 4.57cm in Northing (y) and 1.78 cm in Easting (x).

Significant changes in the dam targets are :

Northing (y) - T-1 to T-10, T-14, T-15, T-17 & T-18

Easting (x) - T-4, T-6 to T-17

Conclusion

The results have shown that there is no systematic trend for the changes in position of dam targets over the period from 1986-87 to 1994-95. Overall pattern indicates the trend of most of the targets trying to keep its original position of 1986-87.

Comparison of 1994-95 at maximum water load and 1995 (summer) at minimum water load has indicated that the targets have general tendency to be shifted to downstream during maximium water load.

Vertical movement/measurment

While carrying out the observations for the Dam deformation studies to the dam targets of the Right spillway, the need of vertical movement was also emphasized.

A plan of precison levelling was prepared by Survey of India for monitoring the vertical movement in the area which emanates from the stable zone and was observed simultaneously along with the angular observations.

Precison levelling was carried out near and along dam axis and inside the gallery during 1986-87, 87-88 and 88-89 at maximum water load. During 1987-88 observations at minimum water load also have been made. The results for these observations have been indicated under first study. Observations during 1994-95 at maximum water load and 1995 (summer) at minimum water load have been taken during 2nd study. Total 37 points were installed for levelling as reported.

The comparison of 1994-95 with 1986-87 at maximum water load and 1995 (Summer) at minimum water load was done. A comparison of 1994-95 of M.W.L. with 1995 (Summer) minimum load has also been made.

Result

(i) Maximum water load : For (1994-95) - 1986-87)

* The diference (maximum) at top of dam is 0.36cm, where as inside gallery is 0.52cm

For (1994-95) - (1988-89)

* The maximum difference at top is 0.34cm and at gallery is 0.24cm.

(ii) Minimum water load :

For 1995 (summer) - (1987-88), the maximum difference at top of the dam is 0.58 cm and inside the right spillway gallery is 0.40cm

(iii) Maximum and Minimum water load :

For 1995 (summer) - (1994-95)

Maximum difference top = 0.25cm,

Maximum at Right gallery = 0.26cm, and Left gallery = 0.17cm

Conclusion :

Bench mark at the dam top have not shown any signficant changes when the values of 1994-95 were compared with those of 1986-87 & 1988-89 at maximum water load.

The difference of values of 1995 (Summer) and 1987-88 at minimum water level when compared with difference of 1995 (s) at minimum water load & 1994-95 of maximum water load, have not shown any significant changes in the B:M.s; which show that the variations in reservoir capacity (levels) have insignificant effect in vertical directions of B:Ms.

B.M. No 33 & 34 at Right gallery have shown reversible trend during 1994-95 1986-87 & 1988-89. These may be of local in nature.

Recommendation :

The Recommendation of Survey of India vide their report of 1995 is as follows :

"No systematic movement in the targets as well as B.Ms have been noticed during the above period. Significant changes in few targets / B.Ms have been noticed which appear to be of local nature. However to keep a check on any systematic charges occuring during the course of time it is felt that the observations for horizontal and vertical movement should be repeated at an interval of 4-5 years.

4.14.8 Chemical Analysis of Reservor Water (by Hirakud Research Station)

Chemical Analysis of Reservoir Water was made for

(i) pH
(ii) E.C. (micro-mhos/cm)
(iii) Total dissolved solids (mg/lt)
(iv) Phenolphthalein (alkalinity mg/lt)
(v) Total Alkalinity (mg/lt)

(vi) Hydroxyl group (OH) (mg/lt)
(vii) Carbonate group (CO_3) (mg/lt)
(viii) Bicarbonate group (HCO_3) (mg/lt)
(ix) Chloride (Cl) (mg/lt)
(x) Sulphate (SO_4) (mg/lt)
(xi) Calcium (Ca^{++}) (mg/lt)
(xii) Magnesium (Mg^{++}) (mg/lt)
(xiii) Total hardness (mg/lt)

The water samples were collected from following locations :

Table-4.18 - Location of Water Sample Collection

Sl. No.	Location	Period of Observation	Total no of samples duraing period of observation
1.	Foundation gallery of right spillway-33/F(2)	03.03.89 to 2.2.90	6
2.	Foundation gallery of right spillway-33/F(2) formed hole	28.11.90 to 24.12.92	30
3.	Operation gallery of right spillway - 35/ODH-1/U/S	28.11.90 to 22.7.91	13
4.	Operation gallery of right spillway - 35/ODH-2 U/S	28.11.90 to 22.7.91	13
5.	Operation gallery of right spillway - 35/ODH-3 D/S	28.11.90 to 22.7.91	13
6.	Operation gallery of right spillway - 35/ODH-4 U/S	28.11.90 to 22.7.91	13
7.	Operation gallery of right spillway - 36/ODH-½ U/S	28.11.90 to 22.7.91	13
8.	F.G of right spillway 39/F(2)- formed hole	28.11.90 to 24.12.92	30
9.	F.G of right spillway 53/F(1)	03.03.89 to 02.09.90	6
10.	F.G of right spillway 53/F(2) formed hole	28.11.90 to 24.12.92	28
11.	F.G of right spillway 50/F(5)	03.3.89 to 02.02.90	6
12.	F.G of right spillway 58/F(6) formed hole	28.11.90 to 24.12.92	29
13.	F.G of right spillway 58/F(8)	03.3.89 to 02.02.90	6

The chemical analysis of water is being continued at Hirakud Research station. Recently few water samples were collected on 27.08.09 from

(i) Foundation gallery formed hole F.G.$39F_2$ $33F_2$, $53F_1$ & $58F_6$
(ii) Reservoir on U/S of left spillway
(iii) Reservoir on U/S of block 35-36 at RL 580 ft
(iv) Reservoir on U/S of right spillway at a depth of 2.0m below water surface

(v) Right spillway block no 48 at RL 557'

(vi) Bucket of right spillway.

One sample test result of August 2009 has been furnished vide Table 4.19. Total 1795 nos. of water samples from various locations have been tested ending 2009.

The Experts of HDCRP & DSRP from previous test results of water samples collected between 1990-92 opined that the reservoir water has not contributed any thing to the development of cracks in Hirakud Spillways and Power dam.

4.14.9 Epoxy Treatment of Cracks

(in Spillways, Power Dam and other Structural members upto July 1998.)

4.14.9.1 Treatment of Cracks :

As recommended by Expert committee, HDCRP and DSRP from time to time cracks were grouted with epoxy compounds suitable for underwater and over water conditions.

The abstract of crack lengths treated, expoxy injected etc are shown in enclosed table No.4.20.

From the table it is clear that up to July 1998, total 14324 m of cracks have been treated with 15480 kg of epoxy. The average consumption in kg/m is 4.59. The minimum consumtion is 0.43 kg/m in exposed faces of right spillway done departmentally during 1975-1983. Maximum Consumption is 16kg/m in wider cracks of Right spillway grouted by M/s F.P.C.C. Bombay between march 88 to June 1994.

It is reported that no further grouting has been done after July 1998.

4.14.9.2 Treatment of other Structural Defects (OSD) upto July-1998.

M/S European Structural Bonding Division Holland, submitted a preliminary report through "The Freyssinet Prestresssed Concrete Company Ltd. FPCC" Bombay on cracks and other structural defects vide their Lr.No PYM :89:SC -32 :4922 dt 08.09.1989.

The gist is given below. Of course this has subsequently undergone change and revised. In this regard FPCC Report of April 1999 is appended vide Annexure 4.1.

Thereafter an estimate of cavities for left spillway was made in 1998. The total volume was 37.67 cum. In 2003, the cavities were measured, the total increaed volume became 138.695 cum. Out of this 59.01 cum has been treated and the balance is yet to be treated.

Similarly the cavities in front of right spillway sluices have been measured during 98-99 by THCIL. The treatment done is indicated in attached sheets vide Tables 4.21 and 4.22.

4.14.9.3 Tests Conducted on Epoxy system for Underwater and Surface use:

Varieties of Epoxy systems for upgradation of sluice barrels upstream face of spillways both above and under water surfaces, galleries, gate slots., Ski-jump buckets etc. were tested at: (i) C.W.P.R.S. Pune and (ii) Hirakud Research Station, Hirakud for deciding the suitability before use.

The tests were conducted for all or some of the under mentioned qualities for different mixes and proportions of mix.

* Mix Viscocity in C.P.S.
* Pot life in minutes.
* 7-days P.B.C. in kg/cm^2
* Bond strength (7 days concrete) in kg/cm^2 under tension and shear
* 7 days compressive strength in kg/cm^2
* 7 days abraison resistance in 7mm thick in gm/cm^2(Dorry's machine)

(i) Tests conducted at C.W.P.R.S Pune.

Total 18 nos of tests were conducted between May 1996 to Oct 1996.

* For injection system

a) Dr. Beck & Co (2 tests)

Dobeckot Resin 505c +Hardner EH-411 (mix 100;50)

Resin 505c+ EH-411 + Acceleratar 803/m (mix 100 :45:5)

b) CICO Product (4 tests)

Resicrete 21 (R+H); (mix 100:50)

* Epoxy mortar

a) Dr. Beck & co (5 tests)

Resin + Hardner + Accelerator 803/m + Q.sand no 10 (mix - 100:45:5:700:200)

b) CICO Product (4 tests)

Resicrete 2010 (R+H) sand, silica (mix 100:50:100)

* Sealing system

a) CICO Product (3 tests)

Armourseal (R+H) - (mix - 100:100

(ii) Tests conducted at Hirakud Research Station

Total 98 nos of tests were conducted between 27.4.96 to 18.07.1997. The gist is given in Table 4.23.

Similarly random samples of different epoxy systems were tested at Hirakud Research Station. Results are furnished in Table Nos. 4.24, 4.25, 4.26, 4.27 and 4.28.

Under water Treatment of Crack in Hirakud dam (April 1999) - Annexure 4.1

Underwater grouting has been carried out with the help of underwater injection Epoxy system. Under-water Epoxy system consists of two part, one is resin and other is hardner. This two components mix at the head. The mixing proportion of two parts is also a main factor for underwater grouting, considering the thickness of crack - fine-crack or wider crack, as viscosity and additional head generations govern the criteria.

For this Meter mix dispensing Mahine has been used. In this machine, there are two parts, one is pump and the other is tank. In Tank portion separate two tanks are used for filling resin and hardner. Two stirrer and two heater are fitted to these two tanks for continous stirring of this Epoxy and also heating as and when necessary up to a desired cut off temperature.

In pump portion, two pumps are fitted for pumping the resin and hardner separtely in a fixed ratio, which can be fixed by changing a scale which is fitted with the pump portion. After discharge from pump, these two components come separately to a two way valve through a pipe. By operating this valve these two components can be mixed at the gun head. The mixing of two components is carried out properly by static mixer. This static mixer has been fixed at the other side of valve.

In this machine any ratio can be fixed. The tank capacity is 20 kg. This machie has een designed and fabricated at Indian Institute of Technology (IIT), Delhi for the then Trafalgar House Construction India Ltd.

In Hirakud Dam Project two types of Epoxy have been used one for less than 5 mm width cracks and another for more than 5mm width. Then mixing Proportion of first type is 2:1 Comp(A):Comp. (B) and the other is 4:1. The above machine is operated pneumatically. This machine was placed at the top of dam and grouting carried out by the nipple point, deepest being at 130 ft. below.

TABLE 4.19 : CHEMICAL ANALYSIS OF WATER

Sl. No	Location	pH	E.C in micromhos/ cm	Total dissolved solid (TDS) in mg/lit.	Phenolphthalein Alkalinity (PA) in mg/lit.	Total Alkalinity in mg/ltr	Hydroxyl group (OH) in mg/ltr	Carbonate group (CO_3) in mg/ltr	Bicarbonate group (HCO_3) in mg/ltr	Chloride (CL) in mg/ltr	Sulphate (SO_4) in mg/ltr	Calcium (Ca) in mg/ltr	Magnesium (Mg) in mg/ltr	Total Hardness (TH) in mg/ltr
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1.	Foundation Gallery Formed Hole FG-39 F2	7.0	110	120	NIL	70	Nil	Nil	70	39.8	32.6	16.8	4.90	62.0
2.	Foundation Gallery formed Hole FG- 33 F2	11.0	1200	560	290	310	270	40	Nil	169.0	46.1	105.6	5.80	288.0
3.	Foundation Gallery formed Hole FG- 53 F1	11.0	220	150	60	100	20	80	Nil	49.7	32.6	20.8	4.37	70.0
4.	Foundation Gallery formed Hole FG- 58 F6	11.0	390	230	90	140	40	100	Nil	55.4	34.6	30.4	4.86	96.0
5.	Reservoir on Up-Stream of left Spillway	7.5	150	120	Nil	110	Nil	Nil	110	54.0	34.6	16.0	3.90	56.0
6.	Reservoir on Up-Stream of Block No 35-36 RL-580	7.5	120	120	Nil	60	Nil	Nil	60	52.5	32.6	18.4	3.90	62.0
7.	Reservoir on U/s of Rt. Spillway at a depth of 2m below water surface.	7.5	130	110	Nil	80	Nil	Nil	80	49.7	36.5	19.2	4.40	66.0
8.	Right Spilway at RL-557 of Block No 48	7.5	120	120	Nil	70	Nil	Nil	70	46.9	38.4	17.6	3.90	60.0
9.	Bucket of Right Spilway	7.5	120	110	Nil	70	Nil	Nil	70	51.1	36.5	19.2	3.40	62.0

N.B. : Date of collection 27.08.2009 by Hirakud Research Station.

TABLE : 4.20 STATUS OF CRACK TREATMENT UPTO JULY 1998

Sl. No.	Agency	Location	Crack length (m)	Epoxy injection (Kg)	Average consumption kg/m
1.	Departmentally (for fine crack in OG/SB) 1975-1983	Exposed face of (i) Right Spillway (ii) Left Spillway	3566 51 3617	1538 49 1587	0.432 0.96 0.44
2.	M/s Electrocoat Pvt. Ltd. Calcutta (Sluice barrel No.41,53,54, and exposed faces Feb./93-12/93 and 5/95-6/96	(i) Right Spillway under sluices (ii) U/s face of Lt. & Rt. Spillway (Rl 595' to 610')	210 254	407 409	1.94 1.99
3.	M/s S.W.C. Calcutta (61 nos of sluice barrels & few exposed faces) (1996-99)	(i) Right Spillway under sluices (ii) Left Spillway under sluices. Total	1887 1450 3337	3562 3762 7324	1.89 2.59 2.19
4.	M/s FPCC Ltd. Bombay March/88-6/94 block 35 to 40 and Part-41	U/W Right Spillway (i) Fine crack (ii) Wider crack Total	1195 91 1286	13740 1452 15192	11.50 16.00 11.81
5.	M/s THCIL Calcutta May 95-April 98 Block-41,Part 42,46 & other defects in block 35 to 46	(a) U/W Right Spillway (i) Fine crack (ii) Wider crack Total	2142 108 2250	16278 1663 17941	7.59 16.41 7.97
6.	Jan.96- June 98 Crack & other structural defects from block 3 to 22	(b) U/W Left Spillway (i) Fine crack (ii) Wider crack Total Grand Total	3302 68 3370 14324 m	14873 607 15480 58340 Kg	4.50 8.92 4.59 4.07

(Source: Memorandum to the 11th HDCRP, Feb. 1999)

Table 4.21 U/W Treatment of other structural defects (OSD) executed through Trafalgar House Construction India Limited (Right & Left Spillways)

Sl. No.	Block with surface area of 407.39 sqm each	Treatment of OSD								
		Honey comb sqm	Rebar No.	Popout No.	Spalling sqm.	Rough concrete sqm	Separation of joints (No.)	Embedded object		Holes (No)
								Metallic (No.)	Non-metallic (No.)	
1.	2	3,	4	5	6	7	8	9	10	11
1.	35	4.0050	56	7	0.1800	-	-	1	-	5
2.	36	10.4175	63	3	0.1300	12.8750	1	-	-	21
3.	37	10.1865	52	5	0.3400	1.7500	-	2	-	18
4.	38	9.0275	37	7	-	6.0000	-	-	-	12
5.	39	15.1700	29	5	1.1150	1.2600	1	-	-	2
6.	40	12.9210	34	4	0.2250	3.5000	1	-	-	10
7.	41	13.0345	125	7	0.7900	19.5900	2	35	2	16
8.	42	10.6600	89	8	0.4525	43.6400	8	28	-	36
9.	43	3.9775	139	27	1.4950	41.6900	9	25	3	32
10	44	3.2700	110	17	2.8025	67.0450	6	29	-	39
11	45	1.9975	99	20	4.8975	64.0975	8	13	1	27
12	46	4.3085	106	16	9.2600	49.1550	18	12	2	29
Total of Right spillway(35-46)		108.8755	939	126	21.6875	310.6025	54	145	8	247
Left spillway 3 to 22*		46.1900	1365	225	31.8350	213.570	187	120	47	531
Grand Total		155.0700	2304	351	53.5230	524.1730	241	265	55	778

* For details of left spillway vide Table No.4.22

Table 4.22 : Status of Under Water O.S.D. treatment work of left spillway of Hirakud Dam

Sl No	Block No	Honey Comb (in sqm)	Rebar Cutting (nos)	Popout Breaking (nos)	Spalling Area (m^2)	Rough concrete (m^2)	Seperation of joints (no)	Metallic embadded object(no)	NonMetallic embaded object(no)	Holes cavities (nos)
1	2	3	4	5	6	7	8	9	10	11
1.	22	1.9425	80	22	1.8600	3.75	3	4	1	40
2.	21	2.7050	121	26	1.5750	11.89	4	2	5	16
3.	20	0.4800	68	17	0.7700	3.30	7	1	1	18
4.	19	0.8850	67	5	0.5850	2.40	4	7	1	13
5.	18	2.4000	50	10	1.2775	3.74	7	9	3	13
6.	17	1.8350	85	11	1.0450	6.12	11	4	4	26
7.	16	1.0675	85	18	0.8700	3.09	8	5	3	22
8.	15	3.0550	99	5	1.1350	-	13	6	3	30
9.	14	1.2350	59	10	1.0450	0.80	11	9	1	27
10.	13	2.1750	54	19	1.8700	-	8	6	2	38
11.	12	0.9125	61	15	0.7650	2.60	16	12	1	32
12.	11	1.6900	67	6	1.0700	-	7	11	2	29
13.	10	4.0150	75	10	3.3000	2.46	20	6	1	36
14.	9	2.6700	84	17	6.7900	6.71	12	3	5	54
15.	8	2.450	51	8	1.8000	2.57	16	7	4	33
16.	7	3.2250	50	6	1.6225	7.25	7	7	4	10
17.	6	2.2800	53	3	1.5850	1.79	5	7	1	32
18.	5	5.2850	37	-	1.5950	5.35	15	8	1	23
19.	4	4.5750	46	5	0.5800	71.21	7	7	3	20
20.	3	1.0125	73	12	0.6950	78.54	6	6	1	19
Total Treated		46.19	1365	225	31.8350	213.57	187	120	47	531

Table 4.23 : Tests on Epoxy conduted at H.R.S.

Sl. No.	Test	Condition	Proportion		No. of tests
			Dobeckot	CICO	
1.	Shear	dry	100:50	-	2
		wet	100:45:5	-	2
		dry	-	2:1	6
		wet	-	2:1	7
		dry	-	2:1:20	1
2.	Pressure	dry	Putty (100:100)	-	1
		wet	-do- (100:100)	-	1
		dry	-	Armour seal 1:1	1
		wet	-	-do- 1:1	1
		dry	-	2:1	6
		wet	-	2:1	7
3.	Bond	dry	100:50	-	8
		wet	100:45:5	-	9
		dry	-	2:1:20	8
		wet	-	2:1:20	8
4.	Compressive strength	dry	-	2:1:20	13
		wet	-	2:1:20	5
		dry	100:50	-	3
		wet	100:45:5	-	5
5.	Sealing putty	dry	1:1	-	2
		wet	1:1	-	2
				Total	98

NB : Besides above the European Structural Bonding Division, Holland has used some epoxy System through M/s FPCC, Bombay, the test results of which are not available.

Table 4.24 : Test results of Random samples of Epoxy Sikadur 53 UF-Under water Injection grout for fine crack

Sl. No.	Testing Date	Batch No.	Viscosity	Bond in Tension (Kg/cm²)	Remark
1.	13.10.95 27.10.95	I 1975 I 1395	770	19.25	
2.	7.3.96 25.3.96	B 7596 B 8296	409	18.51	
3.	12.4.96 27.4.96	B 7596 B 8296	332	+ 11.57 + 10.64 13.42	Since failed in concrete retest done
4.	28.4.96 6.5.96 13.5.96	B 7596 B 8296	332	12.05 (7 day) 1.85 (14 day) 15.76 (14 day)	Test to be repeated
5.	10.5.96 24.5.96	B 7596 C 8296	-	16.07	-
6.	17.5.96 01.6.96	B 7596 B 8296	332	3.70, 8.33 7.40, 3.70	Rejected
7.	6.12.96 20.12.96	J 13496 J 14996		16.43	
8.	24.12.96 07.1.97	J 13296 J 14996	752	16.20	Note : Bond strength in tension to be tested at various application temperature of Epoxy i.e. at 20°c, 25°C, 30°C, 40°c and 50°c (both parts separately maintained at temperature and then mixed) and to cure under-water in usual temperature of vat.
9.	24.12.96 07.1.97	J 13496 J 15396	735	17.00	
10.	29.3.97 11.4.97	C 13097 C 14297	448	23.88	
11.	27.3.97 03.4.97	Lab.Batch	238	19.66	
12.	29.3.97 11.4.97	Lab Batch	238	16.22 Av. 18.13	

Table 4.25 : Sikadur 53 FLV-Under water injection grout for wider crack

Sl. No.	Date of Testing	Batch No.	CPS	Bond in tension (kg/sq.cm)
1.	27.10.95	I 1975/1895	910(31°C)	17.22
2.	18.10.96	I 6095/1895	665(30°C)	22.45
3.	07.1.97	J 15096/16496	Av.	24.30 21.32

Table 4.26 : Surface Grout : Dobeckot 505C (Resin) and EH 411 (Hardner)

Sl No.	Testing Date	Batch No.	Viscosity CPS	Bond in Tension D/W Kg/Cm²	Bond in shear D/W (kg/sq.cm)
1.	27.4.96	960004/6	980(at 36.5°)	+ 21.28 + 23.76	–
2.	13.5.96	960004/6	–	+ 17.46 + 21.85	–
3.	15.6.96 6.7.96	960006/9 960006/4	1522(31°c) 1522	– 27.83 28.44	+19.09 19.54 –
4.	27.6.96	960006/10	1522	+ 25.04 –	28.63 21.80
5.	6.12.96 12.12.96	960006/10 960006/10	– –	– +25.30 22.98	43.63 31.81 –
			Av.	23.38 24.07	30.45 24.38

Table : 4.27 CICO : RESICRETE 21

1.	12.2.97	162896		27.20 21.02	44.98 14.08
2.	11.3.97	051197	1326 AV.	28.38 24.38 27.79 22.70	21.02 15.69 33.00 14.88

N.B. - D = dry. W = Wet.

Table 4.28 : DR. BECK & CO : 2:1:7:2 - 7 days Test Results - Epoxy Plaster

Date of Test	Batch No.OS: Silica 778: 222=1000	Density Kg / m3	Compressive strength Kg / cm² Dry / Wet.	Bond in Tension kg/cm kg/m²	
				Dry	Wet
3.5.96	960004-4 700:200 CPS 960(36.50) 1816 (32°)	742.42 796.96			
14.6.96	960006/9 (CPS 1522)	700:200	845.33 745.18	+25.20	+26.70
18.6.96	960006/ 950010 (CPS 1522)	700:200	805.66	+ 21.46	19.92
4.12.96	960006/ 950010	700:200	675.75 721.20 AV. 767.29 754.44	+ 21.91 22.85	+ 25.92 24.18
CICO : Resicrete 2120 - 7day Test results - Epoxy Plaster (Air curing)					
12.2.97.	B'-162896 860:140	1750	690.90	+26.89	17.40
11.3.97	Bk-051197 580:420 (CPS 1336)	1850	760.22	+30.55 694.29	+29.62
11.4.97	BK - 051197 800:200(Lt.)	1728	690.90	+33.32	20.86
11.4.97	BK-051197 760:240(Rt)	1762	672.69 Av. 703.69 694.29	+ 27.77 + 29.63	19.48 21.84

4.14.10 Temperature of water in Reservoir

The observations were started on 09.01.1991 at irregular interval of about 1 to 1½ months. The temperatures were recorded at 6 different depths including bottom most and water surface points. Temperatures are recorded in °F & °C in day time between 9.30 A.M to 4.30 P.M. Observation has been discontinued after 21.11.95. Temperature observations were made as per the recommendation of the H.D.C.R.P. in their 2nd meeting.

The maximum & minimum temperatures recorded for bottom most point, surface of reservoir and atmoshphere during the above period of observation are as given below.

Point	Maximum temperture in °F	Date	Minimum temperture in °F	Date
Bottom most	92.3	20.6.91	72.68	22.2.93
Surface	92.3	21.9.95	75.20	20.1.94
Atmoshphere	97.52	20.5.94	80.60	19.2.94

4.14.11 Monitoring of Crack Development in Spillways and Power House by (a) Glass Tell-Tales (GTT) and (b) Plaster Tell - Tales (PTT)

(a) **Glass Tell-Tales** were fixed across the cracks developed in the walls of both spillways and power house at verious places to conduct qualitative study as to whether the cracks are widening or have become stable. The size of each GTT is approximately 3"x1"x0.12" (8cm x2.5 cm x 0.3 cm) and each is fixed rigidly to the wall by araldite. In total 195 GTTs were fixed by Hirakud Dam Rehabilitation Division Burla and 143 nos by C.R.O. Hirakud Research Station. The observations are taken at irregular interval of 15 days to 30 days. The condition of GTTs are recorded as follows :

NB - Not Broken, B - Broken, N.R. - Not refixed, R - Refixed

Whenever a GTT is broken another is replaced as soon as possible and observation continued. The glass pieces are broken due to :

i) Widening of cracks, ii) Mischief by miscreants, iii) Slippage due to improper fixing of glass pieces to concrete walls by Araldite, iv) High temperature etc.

The details of G.T.T. fixed by Hirakud Research station with initial dates of fixation & refixation up to December 2001 is enclosed vide Table. 4.29.

b) Plaster Tell - tales (PTT)

Those were constructed at varius places of both Spillways and Power House walls by Hirakud Research Station. The size of each P.T.T. is approximately 2" x 1" x 1" (5cm x 2.5 cm x2.5cm) made with 1:6 cement mortar.

Similar observation was made as in case of G.T.T. The P.T. Ts were fixed near to G.T.Ts to have a comparision. P.T.Ts are not also susceptible to easy damage. Probaly, they offer more resistance to breaking.

In total 142 nos were fixed in different dates and places. Out of these six were broken and replaced as early as possible. Record is available upto December 2001. vide Table 4.30

In addition to these, GTTs & PTTs have also been fixed by Hirakud Dam Rehabilitation Division (HDRD) Burla. As per available records 195 nos. of GTT & 161 nos of PTT were fixed. Observation was made aailable up to 1997 vide Table 4.31.

Table 4.29 : Abstract of GTT (Observation Upto Dec.2001 by Hirakud Research Station)

Location	Fixed		Failed		Refixed		Total Nos, now Available
	Period	Nos	Period	Nos	Period	nos	
Operation Gallery of Right spillway	18.1.91 13.10.92 02.06.95 04.08.95 05.08.95 31.10.96 Total	21 10 4 4 2 4 45	1991 1992 1994 - 	8 1 5 14	 1994 	 14 14	45
Foundation Gallery of Right spillway	25.1.91 12.10.92 6.11.92 5.9.95 5.1.96 Total	26 6 6 1 3 42	1991 1992 1993 1994 1995 1997 	2 1 11 4 3 1 26	 1994 1995 1996 1997 	 18 4 3 1 26	42
Operation Gallery of Left spillway	5.1.91 4.7.91. 21.12.95 26.12.95 Total	40 5 4 7 56	1991 1992 1993 1994 1996 	9 10 3 5 2 29	 1994 1996 	 27 2 29	56
	Grand Total	**143**		**69**		**69**	**143**

Table : 4.30 Abstract of PTT Observation
(Upto Dec. 2001 by Hirakud Research Station)

Location	Fixed		Failed		Refixed		Total Nos. now Available
	Period	Nos	Period	Nos	Period	nos	
1	2	3	4	5	6	7	8
Operation Gallery of Right spillway	7.9.93	21					
	25.8.95	6					
	26.08.95	6					
	31.08.95	9					
	31.10.95	3	13.3.96	1	30.3.96	1	
	Total	45		1		1	45
Foundation Gallery of Right spillway	8.9.93	26	14.12.93	1			
	5.9.93	5					
	15.09.93	3					
	29.5.93	4					
	10.10.95	4	27.1.95	1			
			13.9.96	1	30.9.96	3	
			16.10.97	1	29.10.97	1	
	Total	42		4		4	42
Operation Gallery of Left spillway	9.9.93	15					
	12.1.94	7					
	14.1.94	17					
	17.1.94	12	9.6.95	1			
	4.1.96	4			9.10.96	1	
	Total	55		1		1	55
	Grand Total	**142**		**6**	**6**	**142**	

Table 4.31 : Statement of Glass Tell Tales (GTT) & Plaster Tell Tales (PTT)
(by H.D.R.D. Burla)

Sl. No.	LOCATION	GTT fixed and failed (Nos)				PTT fixed and failed (Nos)			
		During 1997		Upto 1997		During 1997		Upto 1997	
		Fixed	Failed	Fixed	Failed	Fixed	Failed	Fixed	Failed
1.	Right Spillway								
	(i) Operation Gallery	5	1	78	15	3	-	66	1
	(ii) Foundation Gallery	4	2	35	9	Nil	Nil	27	Nil
2.	Left Spillway : Operation Gallery	7	3	25	11	Nil	Nil	19	2
3.	Power Dam : (i) 557' Gallery	6	Nil	12	4	4	Nil	16	Nil
	Power Dam : (ii) 628' Gallery	4	1	31	16	2	1	33	17
4.	Power House Turbine Floor	Nil	nil	14	3	Nil	Nil	Nil	Nil
	Total	26	7	195	58	9	1	161	20

4.14.12 PUNDIT (Probable Ultrasonic Non-destructive Digital Indicating Testing and electromagnetic transducers) by CWPRS :

CWPRS, Pune conducted Non-destructive studies for evaluation of in-situ quality of concrete of block no 35 & 46 of Right spillway of Hirakud Dam. This situation arose due to the fact that fine cracks developed at some places in concrete of above blocks. Accordingly CWPRS under took this work during May 1992 by sonic method.

Velocity of propagation of compressional waves through a concrete mass is an indirect measure of the soundness and quality of concrete and is widely used as non-destructive method to assess the insitu quality of concrete. Non destructive study of above two blocks had been carried out earlier during 1983 using a Portable Ultrasonic testing equipmet (PUNDIT). But this has a limitation that the ultrasonic pulse can not penetrate deep inside concrete mass due to which the conclusion arrived at can not be extrapolated to the entire mass. In such case the sonic method of generating compressional waves in concrete frequency range 500-1000 HZ and mesuring their velocities is found to be more appropriate. This is because the waves generated by this method have long wave lengths and penetrate more depth. The sonic method was deployed successfully at the foundation level in 1992.

The downstream faces of the spillway blocks were not approachable and hence the available recesses inside the inspection gallery were made use of. Compressional waves generated inside the gallery on downstream wall received on spillway slope by electro-magnetic transducers (pickups). Both the transmitted and received waves were recorded by a U.V. recorder after signal conditioning.

For an infinite, homogenous, continuous, isotropic and linearly elastic meduim, the dynamic Modulus of Elasticty for a particular concrete depends only on V.i.e. on compressional wave velocity through the medium. The velocity criteria for grading the quality of concrete as listed in the following table is widely adopted for evaluating the quality of concrete having design compressional strength between 250 & 300 kg/cm^2

Compressional wave velocity (km/sec)	Quality of concrete
4.0 & above	Very good
3.5 to 3.9	Good
3.1 to 3.4	Medium
3.0 & below	Poor

The compressional wave velocities obtained by using the sonic method during 1992 are in the range of 3.5 to 4.42 km/sec. (vide Final technical report No 3028 of Dec.1992 of CWPRS). It indicates that the quality of insitu concrete of block 35 & 46 of Hirakud Right spillway ranges from good to very good. Also the dynamic young's modulus of elasticity for block 35 and 46 is 3.473 X 10^{11} dyne / Cm^2 & 3.543 X 10^{11} dynes/Cm^2 of concrete cores obtained from these blocks.

This conclusion was arrived at by conducting 108 tests in block no.35 with 16 inlet pick-up positions inside operation gallery and 28 outlet pickup points on downstream slope, both in a grid pattern.

Similarly in block No.46, 99 observations were recorded in similar grid pattern.

The results are summerised in Table 4.32

Table 4.32 : Insitu Quality of concrete by PUNDIT/Electromagnet

Right spillway block no	Density of concrete gms/cc	Poissons ratio	Av. compressional wave velocity in km/sec	Dynamic young's Modulus
35	2.4	0.2	4.01	3.473×10^{11}
46	2.4	0.2	4.05	3.543×10^{11}

C.W.P.R.S. recommended that similar studies be made for other blocks of both Right & Left spillways to know the quality of concrete there. No further study have been done sofar.

4.14.13 Seismological Network for Seismic Surveillance of Hirakud Dam

The re-constituted DSRP (2003) in 16th meeting between 27.05.04 to 03.06.04 reviewed the position of seismological network. It was reported to them that the setting of the network is in advanced stage. This DSRP recommended to install the instruments expeditiously. No action has been taken so far.

4.14.14 Joint Meter Observation

(a) As per Hirakud Specification Report - 1951

Block joint meters to be provided as per Drg. No.100-H-56-Plate 71 are as follows :

1. Block joint	34-35	in operation gallery	- 2nos.
2. Block joint	34-35 & 35-36	in foundatin gallery (RL 495)	- 2nos.
3. Block joint	56-57	in operation gallery	- 2nos.
4. Block joint	56-57 & 57-58	in foundation gallery (RL 495)	- 2nos.
		Total	8 nos

(b) As actually provided

Joint Meters were installed at block Joints in following places to record the movement of blocks. The block joints, box nos and initial readings are indicated as under :

Sl. No.	Spillway	Block Joint	Box no.	Initial Readings		
				X	Y	Z
1.	Right	45-46	1	1343	1372	1362
2.	Right	35-36	2	1223	1448	1556
3.	Left	11-12	3	1353	1397	1440
4.	Left	2-3	4	1386	1305	1391

Observations were started on 06.05.1996 and data are available up to 16.06.1997. The intervals of observation vary from 2 days to 17 days. The movement of block joints during the period is approximetely as given in Table No.4.33.

Table 4.33 Movement of Block Joints

Joint	Box no.	Movement in direction X (10^{-4} inch)			Remark
		x	y	z	
45-46	1	13	8	4	Initial reading taken on 6.5.96 and final on 16.06.97. (-) indicates closing and (+) opening.
35-36	2	5	4	(-)8	
11-12	3	15	5	5	
2-3	4	9	5	2	

CHAPTER V

INSTRUMENTATION IN DAM & DYKES

5.0 Introduction

"The main purpose of instrumentation in a dam is to monitor the continued safety of the structure, as well as to provide a check on the design assumptions and methods of computation in vogue. Instrumentation is effective and worthwhile only if it provides sufficient reliable data from which the safety and performance of the embankment can be reasonably and reliably evaluated. Likewise mere collection of data is of no value unless a critical and timely interpretation and evaluation of recorded data is done so as to insure the safety and satisfactory performance of the structure.

Desirable features of good instrumentation are that, the measuring devices

i) Should be simple in design with minimum moving parts. The more complex the design, the greater will be the probability of mal-function;

ii) must be robust, rugged, reliable and require no maintenance as most of the instruments would be embedded; must be sufficiently durable to minimise possibility of damage due to construction activities, long term deformation of structure, and vandalism;

iii) method of observation should be easy requiring minimum skilled manpower as the observations continue for years;

iv) should have an installation procedure which causes least possible interference with construction activities and which, creates boundary conditions representative of the structure as a whole;

v) should have minimum overall cost i.e. cost of supply, installation, data acquisition etc. Often economy can be achieved by an installation for more than one purpose, e.g., some inclinometer casing installations can be used for measuring both horizontal and vertical settlements;

vi) the installation and observation procedures should be consistent with the capabilities of the available personnel;

vii) the data obtained should be meaningful and in a form in which it can be easily interpreted. As for example a knowledge of the relative movement between two points is not of much use unless it is known which point is stable.

The objective of measurements in embankment dams is to collect data on pore pressures, soil movements, soil stresses and seismic response. A knowledge of pore pressures enables a check on the stability of the dam section, proper functioning of the seepage control measures, and on assumptions made in the design. In case of significant rise or departure from design assumptions, the stability can be re-evaluated and immediate precautions as well as remedial measures adopted. Soil movements provide mainly a record of vertical settlements, horizontal displacements under water load, development of tension cracks, instability in shear and loss of free board-they could also give timely warning of an incipient *slide*. Soil stress measurements are generally less reliable, and primarily of use in improving analysis techniques. A record of seismic activity on the ground and at different levels on the dam would provide a check on the design; e.g., if minor shocks indicate that

the displacements or accelerations at the top of the dam are more than anticipated, the crest can be strengthened to withstand major shocks. Regular recording of data both during construction and performance may be of immense value in pinpointing the causes and evolving remedial measures in the event of malfunctioning or any abnormal behavior." (Vide Earth and Rock fill Dams by Bharat Singh & H.D. Sharma pg 468-469).

Details of some of the instruments relating to crack in Spillways and Power Dam have been furnished in Chapter IV. In this chapter, following devices have been described for general monitoring of health of the dam. Those are (i) settlement devices (ii) Strain meters and Thermometers (iii) Twin tube piezometers, (iv) Seepage from earth dam and dykes (v) Seepage in Foundation gallery of Right Spillway (vi) Relief wells and (vii) Stand pipes etc.

5.1 Settlement Measurement of Earth Dam

5.1.1 Introduction

Settlement of Earth Dam depends on the character of the material in the embankment, foundation and the method of construction. It is customary to construct earth dams to a some what greater height than the planned one.

For dam rolled in 6" (15cm) layers of impervious and pervious materials at optimum moisture content, there is no appreciable settlement. However for such type of dam on yielding foundation a nominal allowance for settlement is about one percent of the finished height of the dam which may go to a maximum of 6%.

For hydraulic fill dam, settlement due to consolidation of core may be upto 4%. This is exclusive of foundation settlement. Foundation settlement may vary from 0.0 to 8.0% most of which take place during construction.

Considering settlement of both foundation and embankment it may vary from 1% to 6% or even more.

5.1.2 Devices for Measurement of Settlement

In earth dams the following devices are provided during or after construction to measure the settlement.

(i) Cross arm or telescopic pipe installation

(ii) Base plate installation

(iii) Surface settlement points

(a) Embankment type.

(b) Parapet or curb type.

The method of observation of settlement in cross arm and base plate installations are done with the help of measuring device called torpedo. The reading is taken every day during construction of the dam to find out the consolidation of each segment of the cross arms. After construction is complete, the observation interval may be increased to once a week, a month or once in every three months depending on severity of settlement.

Surface settlement points are installed on the top of embankment and precision levelling done at intervals to know the settlement.

5.1.3 Devices Proposed as per Hirakud Specification Report (1951)

(Drg no 100H-70, plate -100, June 1951)

The locations of installations are :

5.1.3.1 Cross Arm

Station R.D (ft)	Left/Rt E.D	Cross arm	Location	Bottom level of cross arm (RL in ft.)
4010	LED	A	20'U/S of axis	510.00
		B	35'D/S of axis	510.00
7190	RED	C	20'U/S of axis	510.00
		D	35' D/S of axis	460.00
9500	RED	E	35' D/S of axis	540.00

5.1.3.2 Base Plate

9500	RED	4	20'U/S of axis	540.0
10,000	RED	5	- do -	-

Base plates are to be located 20ft. upstream of axis at RD 17 + 00; 33+00, 95+00, 100 +00 & 134 +00 ft.

5.1.3.3 Surface Settlement Points

(a) Upstream :

(i) On top of parapet at EL 645.63'±, @ 200' c/c from station RD 5+00 to 49+00 & from 67+00 to 139+00 ft.

(ii) On embankment slope at EL 626'±, @ 200' c/c from RD 5+00 to 49+00 & 67+00 to 139+00 ft.

(iii) On embankment slope at EL 600±, @200' c/c from RD 67+00 to 79+00 ft.

(b) Downstream

(i) On top of curb at EL 643' ±, @ 200' c/c

(ii) On embankment berm at EL 590'±, @ 200' c/c

(iii) On embankment berm at EL 561'±, @ 200' c/c

for above three from RD 6+00 to 46+00 & 66+00 to 140+00 ft.

(iv) On embankment berm at EL 620'±, @ 200' c/c

(v) On embankment berm at EL 540'±, @ 200' c/c

for above two from RD 66+00 to 80+00 ft.

Installation of instruments in retaining walls at station 49 + 40 ft of main dam & at station 226 + 69.75 of subsidiary dam is to be done as per Drg no. 100-H-71A-plate-72. At a later stage, construction of Subsidiary Dam was not taken up.

5.1.3.4 Settlement Devices actually Embedded / Installed during and after Construction of Earth Dam.

The location & other details of the installations are

5.1.3.4.1 Cross-arm installation

(Data supplied by C.R.O., Hirakud Research Station)

Sl No.	X-arm desig-nation	RD (ft)	Earth Dam	Location Dist from axis (ft)	Bottom level (ft)	Top level (ft)	Period of obser-vation	Max. settlement including construction settlement (ft)	Height of dam (ft)
1	A	4010	LED	20'U/S	-	-	-	-	-
2	B	4010	LED	35' D/S	-	-	-	-	-
3	C	7250	RED	20'U/S	439.36	634.40	19.6.52 to 27.11.58	12.07	178.24
4	D	7250	RED (39 arms)	35'D/S	445.68	643.05	1952- 12/02	14.95	178.24
5	E	9500	RED (18 arms)	35'D/S	543.75	634.08	18.12.53 to 07.10.98	4.48	83.25

Cross-Arm 'A' & 'B' are damaged during construction. Cross arm 'C' was damaged about 6 years after construction i.e on 27.11.58 due to permanent latching of a torpedo inside the settlement device. The cross Arm 'D' indicates a total settlement of 14.95 ft up to dec 1994 including construction settlement of 8.90ft. Initially the recording was started from 1952.

The cross Arm 'E' has also been recorded from 18.12.53 to 7.10.96. It shows a total settlement of 4.48 ft upto 1996 including construction settlement of 1.42 ft. This shows no further settlement.

Cross-Arm readings were taken at about 3 to 4days intervals intially which were increased to a fortnight and even to more than a month.

5.1.3.4.2 Base Plate (B.P) installations

Sl. No.	Base plate no	RD (ft)	Distance from dam axis(ft)	Bottom level (ft)	Top level RL (ft)	Date of install-ation	Period of obser-vation	Total settle-ment(ft)	Remark
1	4	9500	20 U/S	544.17	635.21	28.1.54	14.5.57 to 15.1.74	1.09	Hole blocked w.e.f. 15.1.74
2	5	10,000	20 U/S	551.00	635.76	20.1.54	22.7.58 to 17.10.96 & 2004-2008	1.25 by end of 1989	Continuing
3	6	13400	20 U/S	537.38	626.82	27.4.54	14.5.57 to 15.1.74	1.67	Hole blocked from 15.1.74

Out of above 3 nos of Base plate installatins only the plate no 5 at RD 10000 ft is functioning. Total settlement as bserved up to end of 1989 is 1.25ft. This has increased to 2.34 ft up to 17.10.96.

The details of settlements of above Base Plates are given below :

B.P no. 4 from 28.1.54 to 15.1.74 is 544.17 - 543.08 = 1.09 ft.

B.P no 5 from 20.1.54 to 17.10.96 is 551.0 - 548.66 = 2.34 ft

This was only 1.25 ft by end of 1989

B.P no 6 from 27.4.54 to 15.1.74 is 537.38 - 535.71 = 1.67 ft.

Observations were taken initially @ 3 to 4 days interval. Later on the interval was increased to about a month.

5.1.3.4.3 Surface settlement points

Some surface settlement points were fixed on center line of dam top road (design RL 642ft) of Right Earth Dam and on upstream parapet wall of right earth dam. Initial and final observation dates and settlements etc. are tabulated below :

Table 5.1 : Surface Settlement details of Right Earth Dam

Location	R.D.(ft)	Observation Details			Settlement details			Remark
		No.of points	Initial date	Final date	RD(ft)	Max. settle-ment(ft)	Final level (ft)	
At c/L of top road of RED	6600 ft 14074 ft	63	1.2.58	10/1958	11655	2.92	638.29	*
U/S parapet wall of RED	6580 to 14095	66	22.5.57	10/1962	13944	4.94	641.25	*

* This has been restored subsequently to designed level.

(Source : C.R.O., Hirakud Research Station)

5.1.4 Settlement of Right Dyke :

Total settlement of dyke at RD 23200 ft is 1.76ft (0.54m) which is 4.5% of the hieght of dam (39 ft or 11.8m) and is maximum in whole of the dyke. At RD 23600ft height of ht dam is 47 ft or 14.3 m. and settlement is 1.70 ft. (0.52 m).

(Source : C.R.O. Hirakud Research Station, Hirakud).

Instruments / devices and other facilities provided for monitoring of Hirakud Dam.

5.2 Strain meters and Thermometers

(Ref-Observations / recommendations by HDCRP in 6th meeting and DSRP in 7th & 14th meeting).

5.2.1 Introduction

To know the behaviour of the structure during and after construction, it was proposed to install some Carlson type resistance thermometers and strain meters in both concrete spillways and concrete / masonry power dam during construction period at specified levels to know the temperature and strain developments with ages inside the dam. Their effects can be studied/visualised on analysis of recorded data.

5.2.2 Provision as per Specification Report (1951) prepared by CWI&NC.

(Ref: Dr.no 100H-56-Plate-71 of April 1951)

It was proposed to embed strain meters and thermometers in block no 35 of right concrete spillway over flow section and in power house block no.57 (Non over flow section-NOF). Similar provision was proposed to be made in block no.1 (NOF) and block no.22 (OFS) of concrete left spillway.

The details in case of right spillway and Power Dam are given below vide Table No.5.2.

Table 5.2 : Strain meters & Thermometers (as per specification report)

Block No.35 (OFS)		Block No.57 (NOFS)			
RL (ft)	Group of 5 strain meters	Thermometers (no)	Group of 5 strainmeters	Thermometers (no)	Remark
1	2	3	4	5	6
At Foundation level	-	3	-	4	
500	-	6	-	6	
525	-	6	-	6	* 1st set on axis
550	-	5	-	4	2nd at 35' d/s of
575	-	4		4	axis, 3rd at 70'
600	-	1	-	4	d/s and 4th at 105'
625	-	-	1	1	d/s of axis.
495	4 sets*	-	4 sets(*)	-	
Total	4 sets	25	4 sets	29	

NB : Similar provision in block no.1 (NOF) and in block-22 (OF) of left spillway was proposed to be made.

5.2.3. Strain meters & Themometers actually provided during construction:

Strain meters & thermometers were actually embedded during cnstructon in block no.41 of right concrete spillway & in block no.57 of concrete masonry power dam. No. meters were provided in left cncrete spillway. The details are tabulated below :

Table 5.3 : Strain meters & Thermometers (actually provided)

RL ft	Block 41 (O.F.)		Block 57 (NOF)	
	Gorup of 5 Strain meters (set)	Carlson Thermometers (nos)	Group of 5 Strainmeters (set)	Carlson Thermometers (nos)
490	-	3	-	-
500	-	4	-	4
525	-	7	-	-
550	-	5	-	3
575	-	5	-	5
615	-	2	RL600'	3
			RL 625'	2
495	4 sets	-	4 sets	-
Total	4	26	4	17

5.2.4. Study of Temperature :

Concreting in block no. 41 was started in January 1954 and was completed in March 1956. No work was done during monsoon months of June to November. No arrangement was done for pre or post cooling during construction. Hirakud being a place of extreme climate, temperature during March to May rises very high.

Monthly maximum & minimum atmoshpheric temperatures in °F have been recorded during construction period of 1953-1957. Those show that the maximum temperature is generally achieved during May (about 110° F) & minimum achieved during December/January (about 55°F).

The temperatures achieved inside the Spillway/Power Dam were obseved by the help of Carlson type resistance thermometers placed inside. Maximum rise in temperature took place within 40 to 100 hours after placement. These are 46° F & 19° F for block 41& 57 respectively. Two typical graphs are appended herewith as Drg. No 5. 1 & 5. 2 one for block 41 centre and other for block-57 (centre).

Location of thermometers and strainmeters provided in both the blocks are given in sectional elevation in Drg. No 5.3 & Drg. No.5.5 and in plan in Drg. No. 5.4 & Drg. No. 5.6. Details on locations are also given in Table No.5.4 & 5.5.

Typical figures showing isothermals during March-1957 have been given in Drg. No. 5.7 & Drg. No. 5.8 for block-41 & 57 respectively. Most of the thermometers are not working since 1965.

After cooling, steady temperatures have been attained which vary with variation of atmospheric temperatures.

5.2.5 Study of strain

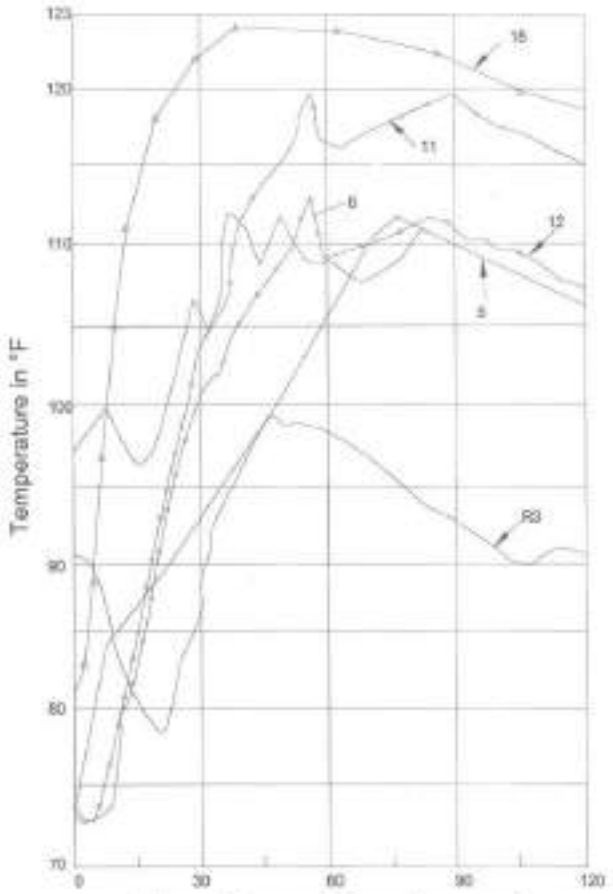
Total 28 nos of strainmeters have been embedded in block no 41 & 57 at foundation level (RL495′). Drg. no.5.3 & Drg. no. 5.5 indicate the sections and Drg no. 5.4 & Drg. no. 5.6 the key plans showing installations in block 41 & 57. In each block 4 sets of strain meters have been embedded at distances of 0′, 35′, 70′ & 105′ form the axis of dam. The locations of strain meters are indicated in Table 5.6 & 5.7 for block No.41 & 57 respectively. These meters record the total strain developed along their axis, as such they are installed in vertical, horizontal, parallel and horizontal - normal positions with respect to axis of the dam. They enable the measurement of 3-D strains due to combined effect of load, tempeature and hydrostatic pressure. A set of no stress vertical meters are also installed which measures the strain developed at the location due to causes other than load like temperature change, growth and moisture change etc. The load bearing strain is obtained by deducting the 'no stress' meter readings from the total strin recorded.

A typical fig. (Drg. No. 5.9) showing the length change of vertical strain meter no 17 in block 41 embeded at RL 495′ on 22.4.1954 is appended. This meter did not function since 25.8.56.

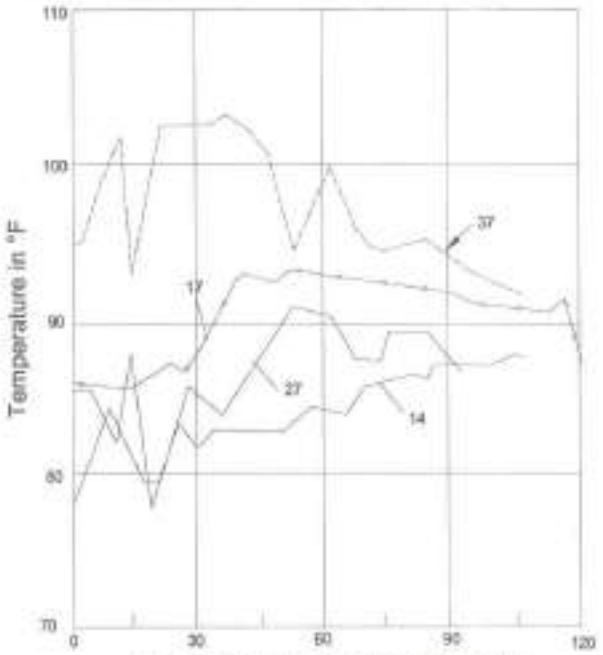
From the readings of strain meters, it is observed that with the initial filling of reservoir the meters showed a marked rise in tensile strain. With the fall in reservoir level there is a corresponding decrease in tensile strain & in a steady reservoir condition some of them recorded a decrease in tensile strain.

Further it could be observed that the horizontal strainmeters recorded much higher strain than the verical ones under similar conditions of loading & temperatures. The strain meters in horizontal-paralled to axis position recorded much higher strain than that of meter in horizontal normal to axis position. Cause of this can be attributed to difference in "restrain & growth" condition in two directions at that paricular location. The stress study did not consider the creep correction.

The strainmeters have gone out of order since 1965.



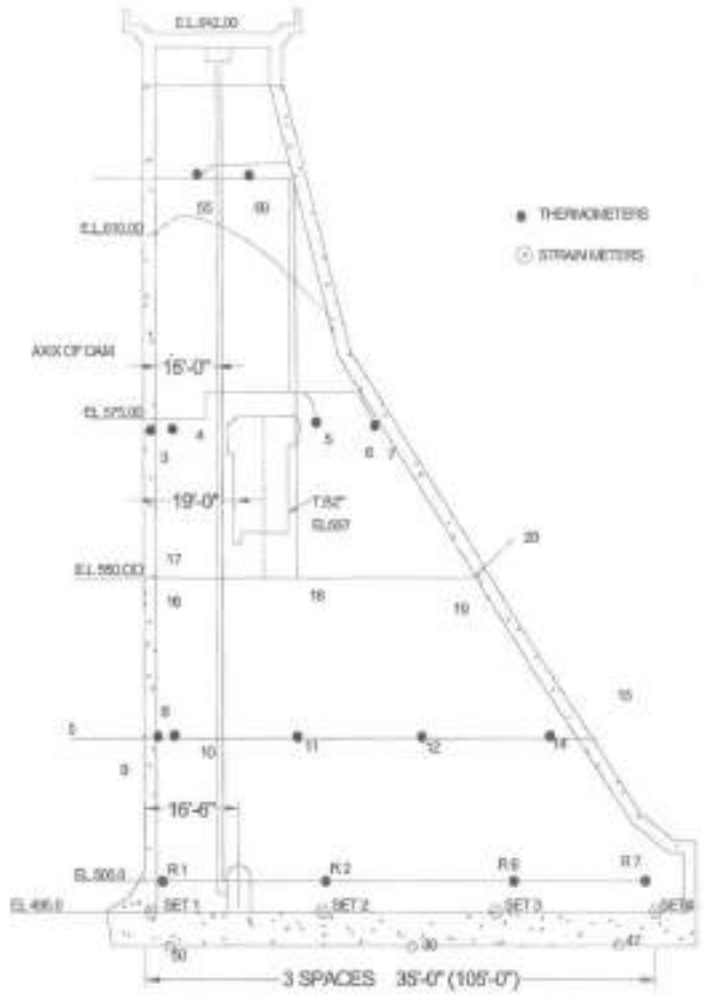
Time in hours (after placement)
showing early temp.rise in block no.41. center



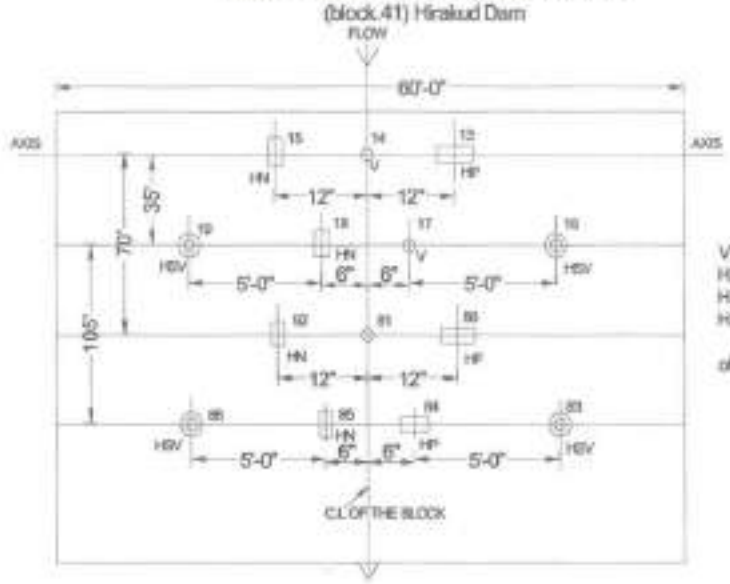
Time in hours after placement
showing early temp.rise in block no.57. center

N.B: No.s indicated in the graph represent Thermometer nos.

Drg. No. 5.1 & 5.2



Section showing strain meters & thermometers
(block 41) Hirakud Dam

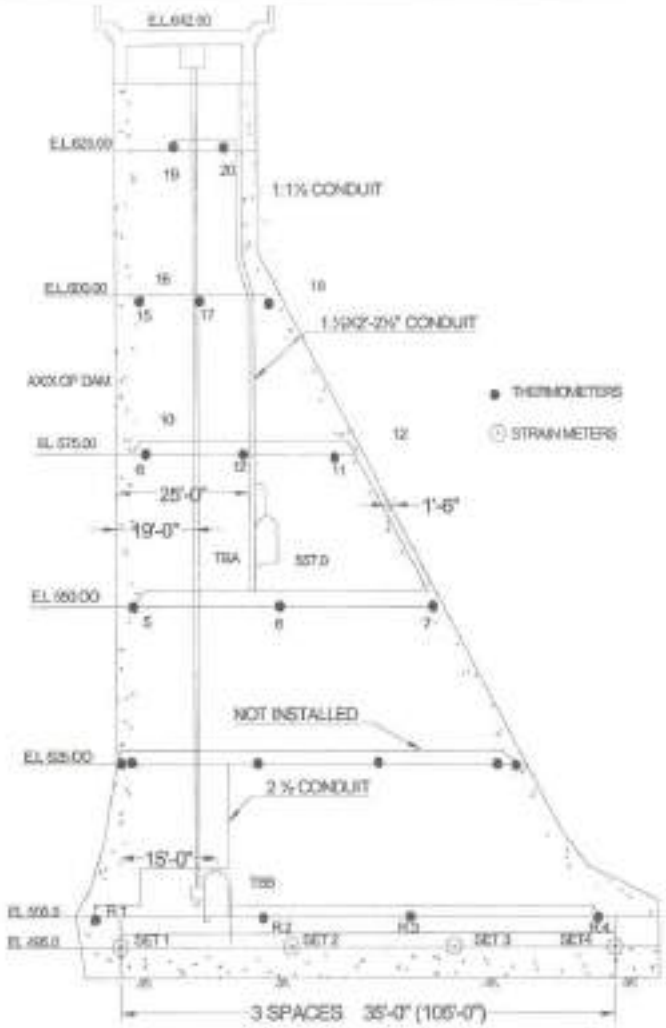


NOTE

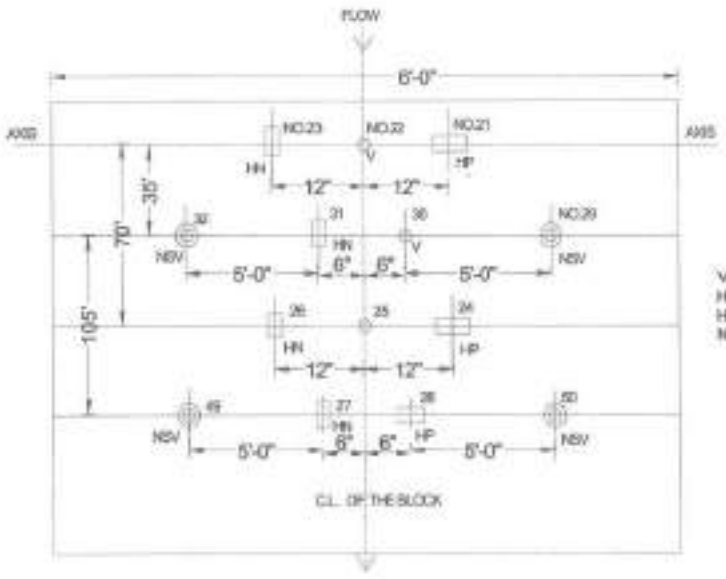
V- Vertical to axis.
HN- Horizontal & normal to axis.
HP- Horizontal & parallel to axis.
HSV- not stress - vertical to axis.
(the symbols indicate position of the strainmeters)

RIGHT SPILLWAY BLOCK 41 EL 495 INSTALLATION PLAN OF STRAIN METERS
(NOT TO SCALE)

Drg. No. 5.3 & 5.4



Section Showing Strain Meters & Thermometers
(BLOCK 57) Hirakud Dam

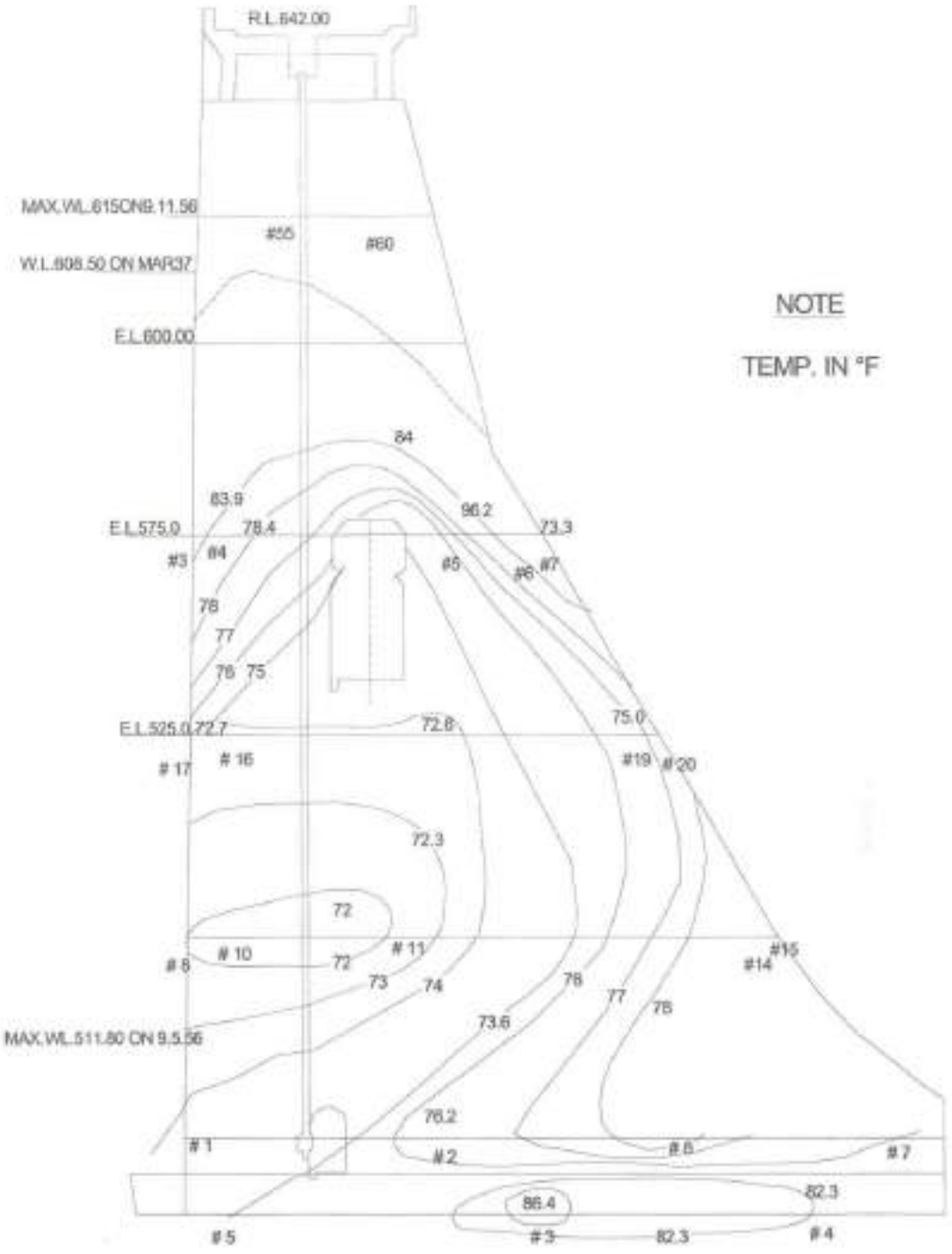


NOTE

V- Vertical to axis.
HN- Horizontal & normal to axis.
HP- Horizontal & parallel to axis.
NSV- not stress - vertical to axis.
(the symbols indicate position of the strainmeters)

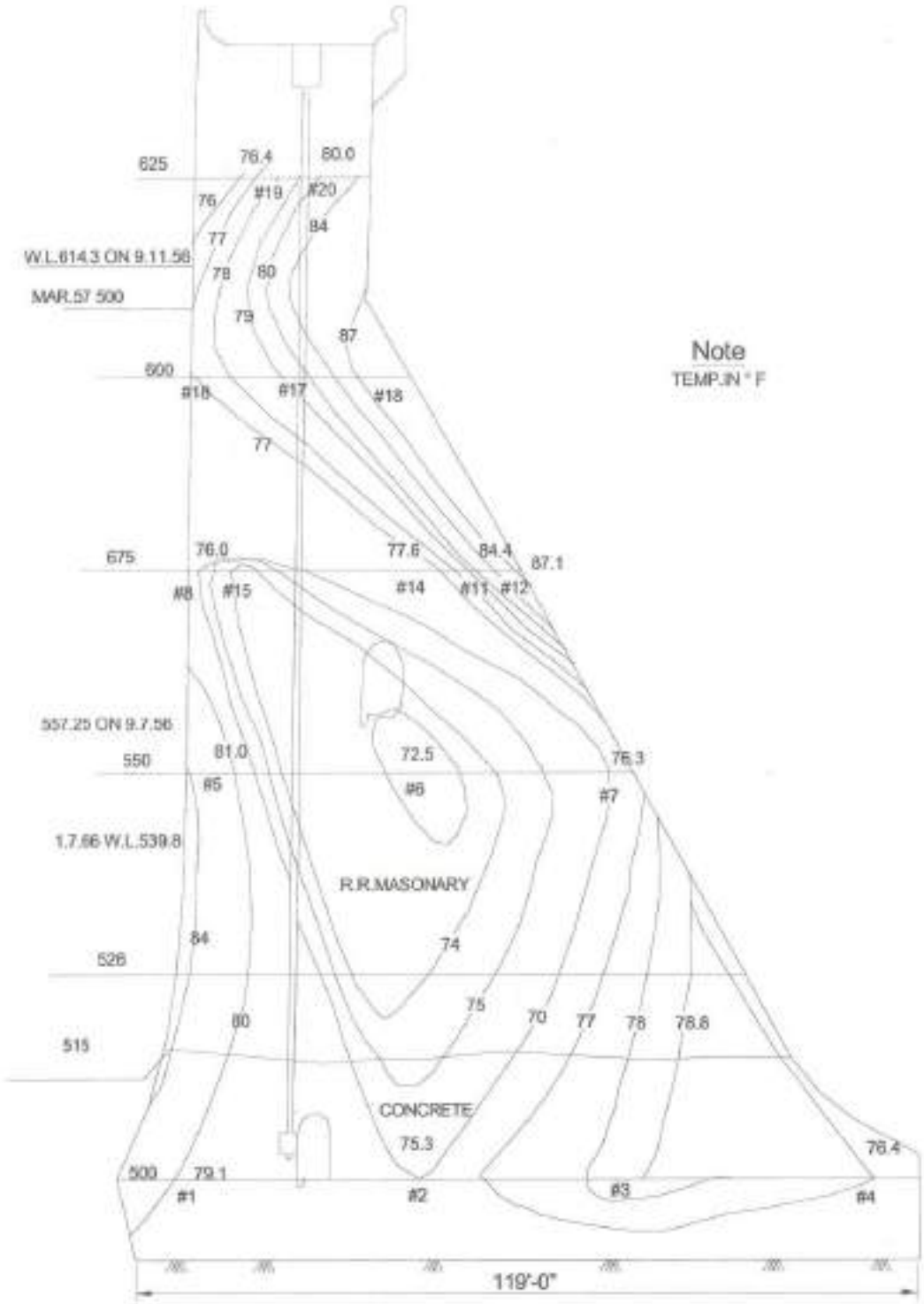
POWER DAM BLOCK 57 E.L.496.0 INSTALLATION PLAN (NOT TO SCALE)

Drg. No. 5.5 & 5.6



Hirakud Dam
Isotherm in block 41
March 1957

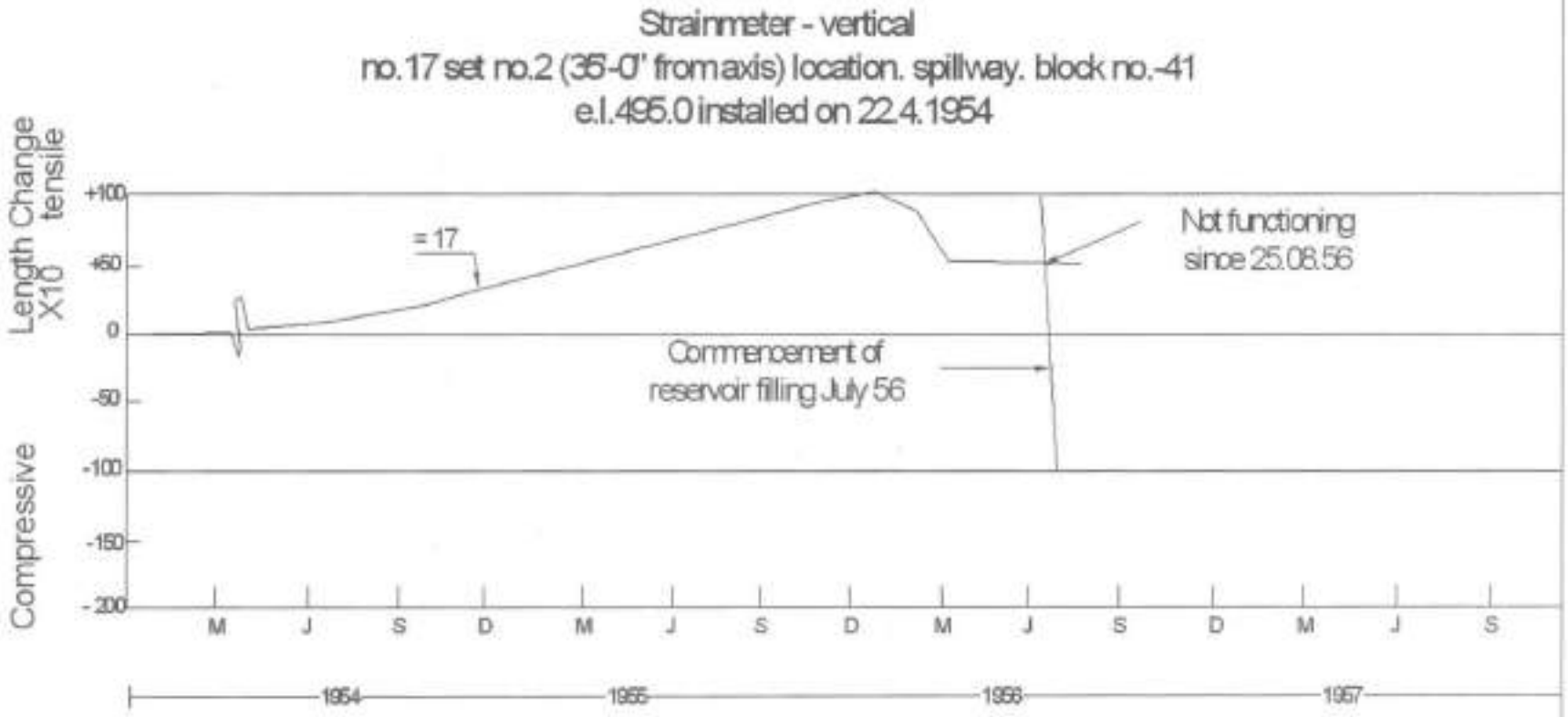
Drg. No. 5.7



Hirakud Dam
Isotherm in block 57
March 1957

Drg. No. 5.8

Right Spillway block- 41 strain - meter



Drg. No. 5.9

Table 5.4 : Location of Carlson - Resistance Thermometers embeded in block No.41

Elevation (ft)	Instrument no	Distance (ft) from axis	Date of installation	Date of out of order
1	2	3	4	5
481.58	03	50	6.4.1954	29.5.63
481.36	04	95	14.4.54	27.2.70
489.09	05	1'-0"	9.4.54	31.1.56
500	01	-	20.5.56	25.8.56
	02	35'(approx)	20.5.54	30.11.59
	06	70'(approx)	26.5.54	30.11.59
	07	105'(approx)	29.5.54	15. 6. 61
525	08	0'-6"	13.01.55	29.5.63
	09	2'-0"	- do-	27.2.70
	10	3'-6"	- do -	23. 9. 58
	11	29'-5"	- do -	11.7.61
	12	55'-5"	- do -	24. 5. 63
	14	6'-0" from d/s face	- do -	23. 9.58
	15	4'-5" -do-	- do -	9.11.56
550	17	1'- 6" -do-	23.02.55	27.02.70
	16	4'-6" -do-	- do -	30.06.59
	18	35'-0"	- do -	- do -
	19	5'-6" from d/s face	- do -	-do -
	20	1'-6" -do-	- do -	28.5.56

Table 5.5 : Location of Carlson Thermometers embedded in block-57

Elevation (ft) 1	Instrument no 2	Distance (ft) from axis 3	Date of installation 4	Date of out of order 5
500	1	U/S end	-	-
	2	middle of(U/S)	-	-
	3	middle (D/s)	-	-
	4	103'-6"	17.03.53	23.9.58
525	(5nos)	Not Installed		
	5	1-6" approx	30.11.59	27.2.70
	6	at centre(approx)	29.3.59	21.3.65
	7	1-6" approx (from D/s face)	12.12.58	27.2.70
575	8	1'-6"	2.2.55	8.9.65
	10	4'-5"	- do -	27.2.70
	14	35'-0"	- do -	27.1.70
	11	5'-6" (from D/S)	2.2.55	23.9.58
	12	1'-6" (- do -)	2.2.55	27.2.70
600	16	4'- 6"	21.7.55	27.2.70
	17	Centre of section	20.7.55	- do -
	18	1"6 approx from Ds	- do -	30.6.59
	19	-	22.10.55	2.3.65
	20	-	13.1.56	27.2.70

Table 5.6 : Location of Strainmeters installed in Block no 41 at RL 495′

Sl no	Category of instrument	Location (ft)		Instrument no	Period of of observation
		Distance from Dam axis (ft)	Distance from C.L of block(ft)		
1	2	3	4	5	6
1	H.N	0	1′-0″ (R)	15	16.1.58 to
	V	0	0′-0″	14	31.1.61
	H.P	0	1′-0″(L)	13	
2	N.S.V	35	5′-0″(R)	19	
	H.N	35	0′-6″ (R)	18	16.1 58 to
	V	35	0′-6″ (L)	17	21.4..58
	NSV	35	5′-0″ (L)	16	
3	H.N	70	1′-0″ (R)	82	
	V	70	0′-0″	81	
	H.P.	70	1′-0″ (L)	20	
4	NSV	105	5′-0″ (R)	86	16.1.58 to
	H.N.	105	0′-6″ (R)	85	28.2.62
	H.P	105	0′-6″ (L)	84	16. 1.58 to 29.7.60
	NSV	105	5′-0″ (L)	83	1.58 to 30.4.59

Table 5.7 : Locations of Strainmeters installed in Block no 57 at RL 495′

Sl no	Category of instrument	Location (ft)		Instrument no
		Distance from Dam axis	Distance from C.L of block	
1	2	3	4	5
1	HN	0	1′-0″(R)	23
	V	0	0	22
	HP	0	1′-0″	21
2	NSV	35′	5′-0″(R)	32
	H.N	35′	0′-6″(R)	31
	V	35′	1′-6″(L)	30
	NSV	35′	5′-0″(L)	29
3	HN	70′	1′-0″(R)	26
	V	70′	0′-0″	25
	HP	70′	1′-0″ (L)	24
4	NSV	105′	5′-0″(R)	49
	HN	105′	0′-6″(R)	22
	HP	105′	0′-6″ (L)	28
	NSV	105′	5′.0″(L)	50

N.B:

HN =	Horizontal - normal to axis	- 4 no
NSV =	No stress - vertical to axis	- 4 no
HP =	Horizont - Parallel to axis	- 3 no
V =	Vertical to axis	- 3 no
	Total	- 14 no

(The symbols indicate position of strain metrers both for block 41 & 57)

5.3 Installation of Twin Tube Piezometers for Pore Pressure Measurement of Earth Dam

5.3.1 Initial Proposal

The specification Report of Hirakud Dam prepared by the then CWPC, now CWC styled as "Hirakud Main and Subsidiary Dams and Power Plants Specification - general 1951 describes the Piezometer installation.

It was proposed to place tips at different levels of embankment at 4 sections i.e. at RD 40+00, 49+10, 72+00 and 95+00 (Refer drg no 100-H-71 June 1951, Plate 101)

The abstract of tip details are given below.

Section	RD of terminal well(ft)	F.P Foundatin type(no)	E P Embank-ment type(no)	Total Nos. of tips
A-A	40+00	6	25	31
B-B	49+10	2	21	23
C-C	72+10	5	40	45
D-D	95+10	8+9	15	32
Total		30	101	131

5.3.2 Actual field Installations

These works were carried out during execution of Earth dam under the supervisoin of C.R.O Hirakud Research Staton, Hirakud.

4 nos. of terminal wells were built in downstream berm/slopes of th dam wherein Bourden type pressure gauges and master gauges were fixed for taking pressure measuremetnts. The piezometer tips were connected by plastic tubings from inside the dam to the gauges in each section. The R.D. of terminal wells are :

(1) L.E.D - R.D - 4900', (2) L.E.D. - RD 7250' (Drainage channel)

(3) R.E.D. - R.D. 9500' (Hirakud island), (4). R.E.D. - R.D. 12200' (Middle channel)

The abstract of piezometer tips actually installed during construction of earth dam is given below : (Source - C.R.O. Hirakud Research Station)

Table 5.8 : Abstract of Piezometer tips actually installed

Sl No	R.D of Terminal well (ft)	Total no of tips installed	No. of Tips damaged/ not installed	No of functional tips	Period of observation
1	2	3	4	5	6
1	4900(LED)	24	3	21	3.9.58-31.1.96
2	7250 (Drainage channel)	45	Not installed=5 Damaged=6	34	9.4.57-18.12.95
3	9500 (Island)	32	4	28	9.4.57-18.12.95
4	12200 (Middle channel)	22	0	22	9.4,57-18.12.95
	Total	123	18	105	

Details of above piezometer tips actually installed in 4 sections are given in Tables 5.9. to 5.12.

Table 5.9 : Details of Twin Tube Piezometer tips actually installed inside the Left Earth Dam at RD 4900 ft during construction

Tip No.	R.L. of Tip (ft)	R.D. of Tip (ft)	Distance of Tip from dam axis (ft)		Type of Tip	Gauge Constant	Type of surrounding material
			U/S	D/S			
1	484.07	4927	13.5	-	F	57.9	-
2	492.00	4928	-	49.0	F	49.3	-
3	524.70	4927	149.0	-	E	16.0	Impervious
3A	525.09	4928	253.5	-	E	15.0	Semi-pervious
4	524.29	4925	135.0	-	E	15.2	Impervious
5	525.12	4922	74.0	-	E	13.9	Impervious
6	525.12	4927	16.5	-	E	16.8	Impervious
7	Not installed	-	-	-	-	-	-
8	Not installed	-	-	-	-	-	-
9	Not installed	-	-	-	-	-	-
10	549.47	4920	130.0	-	E	(-) 8.2	Semi-pervious
11	548.77	4920	70.0	-	E	(-) 8.1	Impervious
12	550.48	4910	16.0	-	E	(-) 10.4	Impervious
13	550.11	4921	-	50	E	(-) 10.4	Impervious
14	549.87	4921	-	105	E	(-)8.0	Impervious
15	549.34	4921	-	160	E	8.0	Semi-pervious
16	579.49	4920.7	140	-	E	(-) 38.7	Semi-pervious
17	579.56	4924.5	20	-	E	(-) 39.7	Imperivious
18	579.53	4925	83.1	-	E	(-) 40.0	Impervious
19	579.92	4911	-	43.5	E	(-) 38.0	Impervious
20	579.67	4920	-	105	E	(-) 38.4	Semi-pervious
21	612.02	4935.5	-	59	E	(-) 71.3	Impervious
22	611.30	4906	-	58	E	(-) 71.2	Impervious
23	610.44	4906	-	81	E	(-) 70.9	Semi-pervious
24	625.03	4860	11.50	-	E	14.0	Impervious

NB : E = Embankment type = 22 no., F = Foundation type = 2 no.

Total nos of tips Provided = 24 no.

Not installed/Not functioning = 3 no., Functioning = 21 no.

Observations discontinued since 31.01.1996 due to damage & theft of some pressure gauges and pipe lines. Last observation was made on 31.01.1996.

Table 5.10 : Details of Twin tube Piezometer tips actually installed in side the Earth Dam at RD 7250' during construction

Tip no.	R.L. of Tip (ft)	R.D. of Tip (ft)	Distance of Tip From dam axis (ft)		Type of Tip	Gauge Constant	Type of surrounding material
			U/S	D/S.			
1	2	3	4	5	6	7	8
1	422.36	7240	2.83	-	F	-	
2	437.29	7239.5	70.00	-	F	104.1	
3	428.01	7240	-	85.00	F	-	
4	461.41	7240	258.75		F	-	
5	455.05	7240	136.00	-	F	84.8	
6	456,96	7250	20.00	-	E	82.4	Impervious
7	455.12	7240	-	32.00	E	83.7	Impervious
8	464.61	7240	215.00	-	F	73.7	
9	462.50	7240	-	190.00	F	75.3	
10	Not Installed		-	-	-	-	
11	492.81	7240	215.00	-	E	49.1	Impervious
12	494.68	7240	-	170.00	E	46.7	Impervious
13	Not Installed		-	-	-	-	
14	495.10	7240	120.00	-	E	45.7	Impervious
15	505.16	7243	69.60	-	E	35.2	Impervious
16	505.09	7241	20.00	-	E	34.7	Impervious
17	504.73	7240	-	31.50	E	34.60	Impervious
18	504.90	7240	-	85.00	E	33.60	Impervious
19	504.39	7241	-	131.50	E	33.9	Semi-pervious
20	Not Installed		-	-	-	-	
21	Not Installed		-	-	-	-	
22	Not Installed		-	-	-	-	
23	534.70	7240	65.00	-	E	3.1	Impervious

contd.....

24	534.79	7240	15.00	-	E	-	Impervious
25	534.90	7240	-	35.0	E	6.4	Impervious
26	535.07	7240	-	90.00	E	5.8	Impervious
27	535.25	7240	-	145.00	E	5.1	Semi-pervious
28	535.38	7240	-	203.00	E	4.5	Semi-pervious
29	570.02	7240	-	140.00	E	(-) 30.7	Semi-pervious
30	569.67	7240	80.00	-	E	(-) 30.9	Impervious
31	569.65	7240	25.00	-	E	(-) 31.2	Impervious
32	569.53	7240	-	30.00	E	(-) 31.7	Impervious
33	569.11	7240	-	85.00	E	(-) 27.3	Semi-pervious
34	568.68	7240	-	140.00	E	(-) 27.3	Semi-pervious
35	611.00	7240	(Axis)	-	E	(-) 70.2	Impervious
36	610.75	7240	-	45.00	E	(-) 70.4	Impervious
37	610.82	7240	-	85.00	E	(-) 70.5	Impervious
38	609.87	7240	-	125.00	E	(-) 70.6	Semi-pervious
39	489.19	7469.8	399.15	-	E	49.6	Impervious
40	490.01	7428	319.30	-	E	-	Impervious
41	490.33	7387	239.50	-	E	-	Impervious
42	491.22	7347	155.22	-	E	46.1	Impervious
43	494.42	7290	75.00	-	E	42.9	Impervious
44	500.70	7190	150.00	-	E	36.6	Semi-pervious
45	497.15	7140	225.00	-	E	40.1	Semi-pervious

NB : Total no of Tips = 45
Not installed = 05
Damaged = 06
Functioning = 34

F = Foundation type = 7 no.
E = Embankment type = 33 no.

The observations discontinued due to damage of old pressre gauge & pipe lines & also due to theft of some pressure gauges & pipe lines since January 1996. Last observation was made on 18.12.1995.

Table 5.11 : Details of Twin tube piezometer tips actually installed inside the earth dam at RD 9500 ft. during construction.

Tip No.	R.L. of Tip (ft)	R.D. of Tip (ft)	Distance of Tip From dam axis (ft)		Type of Tip	Type of surrounding material	Gauge Constant
			U/S	D/S.			
1	2	3	4	5	6	7	8
1	450.01	9510	115	-	F		116.3
2	471.09	9510	20	-	F		94.6
3	470.31	9510	-	70	F		94.8
4	502.71	9510	215	-	E		61.8
5	501.51	9510	117	-	E		62.0
6	521.40	9510	22	-	E		41.5
7	519.43	9510	-	72	E		42.9
8	Not Installed	-	-	-			-
9	540.51	9510	175	-	E	Impervious	25.8
10	540.31	9510	140.3	-	E	Impervious	25.4
11	547.53	9510	215	-	E	Semipervious	17.6
12	549.64	9510	170	-	E	Impervious	14.8
13	549.72	9510	115	-	E	Impervious	13.8
14	549.90	9510	65	-	E	Impervious	13.0
15	550.02	9510	15	-	E	Impervious	12.3
16	550.11	9510	-	35	E	Impervious	11.6
17	550.28	9510	-	90	E	Impervious	16.6
18	550.59	9510	-	145	E	Semipervious	15.1
19	579.54	9510	135		E	Semipervious	(-)14.5
20	579.49	9510	79.75		E	Impervious	(-)15.0
21	579.44	9510	25		E		(-)16.0
22	579.38	9510	Not Installed				
23	580.00	9510	-	95	-do-		-
24	464.60	9510	0(Axis)	-	-do-		-
25	474.25	9510	0	-	F		92.0
26	470.63	9510	0	-	F		95.0
27	461.23	9810	0	-	F		103.90
28	529.20	9310	0	-	E		41.30
29	519.97	9410	2.0	-	E		43.50
30	519.83	9610	2.0	-	E	Impervious	43.0
31	520.96	9710	2.0	-	E	Impervious	41.3
32	520.95	9810	2.0	-	E	Impervious	40.8

NB : Total no of Tips = 32
Damaged = 04
Functional = 28

F = Foundation type = 6
E = Embankment type = 22

Observation discountinued since January 1996 due to damage & theft of pressure gauges & pipe lines. Last observation made on 18.12.1995

Hints: (1) Gauge constant + AV Bourden gauge reading = pore pressure
(2) Tip elevation + pore pressure = R.L. of phreatic line.

Table - 5.12 : Details of Twin tube Piezometer tips actually installed inside the earth dam at RD 12200 ft. during construction.

Tip no.	R.L. of Tip (ft)	R.D. of Tip (ft)	Distance of Tip From dam axis (ft)		Type of Tip	Gauge Constant	Type of surrounding material
			U/S	D/S.			
1	2	3	4	5	6	7	8
1	464.16	12200	325	-	F	77.6	
2	452.86	12200	253	-	F	88.2	
3	467.12	12200	145	-	F	73.3	
4	465.20	12200	55	-	F	74.5	
5	523.32	12236	315	-	E	15.7	Semi-pervious
6	523.39	12230	275	-	E	18.3	Semi-pervious
7	524.95	12222	125	-	E	16.1	Impervious
8	525.94	12223.4	172	-	E	14.4	Impervious
9	525.81	12227	135	-	E	13.9	Impervious
10	526.01	12232.5	75	-	E	13.0	Impervious
11	529.00	12238.3	15	-	E	9.3	Impervious
12	531.10	12235.5	-	45	E	10.6	Impervious
13	534.29	12230	-	105	E	6.8	Impervious
14	534.69	12228.5	-	169	E	5.7	Impervious
15	562.58	12230	-	98	E	(-) 22.9	Impervious
16	562.04	12230	-	107	E	(-) 23.0	Impervious
17	595.29	12230	-	71.50	E	(-) 57.0	Impervious
18	593.43	12230	-	111.50	E	(-) 51.7	Semi-pervious
19	591.37	12230	-	142.00	E	(-) 50.3	Semi-pervious
20	625.44	12230	17.5	-	E	(-) 85.1	Impervious
21	624.88	12230	-	17.50	E	(-) 85.2	Impervious
22	623.10	12230	-	44.25	E	(-) 84.1	Semi-pervious

NB : Total no of Tips = 22 no. F = Foundation type = 4 no.

Functional = 22 no. E = Embankment type = 18 no.

The observations discontinued since Jan-1996, due to damage & theft of some pressure gauges & pipe lines. Last observation made on 18.12.95

N.B: Procedure for Pore -Pressue calculation

(i) Mean gauge reading = (Gauge reading at inlet + Gauge reading at outlet) x 0.5

(ii) Pore pressure = Gauge constant + mean gauge reading

(iii) Top level of phreatic line = Tip elevation + Pore Pressure.

5.3.3 Details of Piezometer Installed :

5.3.3.1. Introduction :

Twin Tube piezometer tip installations have been provided in Hirakud earth dam for recording the pore pressure variations in the foundations and embankment. They are of two types :

(a) Foundation type tips

(b) Embankment type tips

The tips are connected to a central observation well fitted with necessary valves and gauges through plastic tubings embedded in earth dam.

The informations obtained from them is helpful not only in determining the safety and performance of the structure but also in the future design of earth dams.

(i) Foundation type - This is used to measure pore pressure (p.p) through a pipe extending vertically downwards into the foundation. This is moulded by the fabricator with 7" to 8" dia by 14" alundum disc sealed in place and two plastic tubes are cemented to the body. Below the porous disc, the section is extended to form a coupling.

(ii) Embankment type - This tip is used with disc down at any point in embankment where pore pressure observations are desired. This is moulded similarly as foundation type except that the extension below the porous disc is not required. The disc will come in contact directly with clay.

5.3.3.2. Process of installation of the apparatus :

To avoid interfering with embankment placing operations all the tips and tubes are carried in trenches. To carry the tubes one main trench is excavated but for connecting the tips offset trenches are made at right angles to main trench at the proper location. These trenches are kept on level grades or on ascending or descending grades relative to the entrance pipe into the terminal well. This process is strictly followed to avoid air getting trapped inside the piezometer lines.

After the trenches are excavated and levelled a two inch protective cushion of selected fine material typical of the adjacent embankment in the bottom of trench has been compacted for laying the tubes.

At the time of back filling care is exercised to prevent injury to the tubing. Individual tubes are separated at least ½". The tubes are numbered for identifying the location of tips. Three inch of selected materials is compacted over the tubings and then the adjacent embankment material is compacted in 4" layers.

Specification of U.S.B.R. for installation, flushing and observation of Bourden type gauges (as given in USBR earth manual) have been followed strictly except that pipes were not filled with water during construction. No reading could be taken during construction since Bourden gauges could not be made available. Pore pressure could be observed from 1956.

5.3.3.3 Terminal wells

These are constructed of R.C.C. for installing the terminal well apparatus to take pore pressure readings. These are designed for 40-60 tips. As soon as the apparatus are fitted, the piezometer tips are put into operation to record construction pressure as embankment placing operations are in progress. The foundation and embankment pressure is trausmitted from piezometer tips through the liquid filled tubes and observed on the compound gauges mounted in the terminal well.

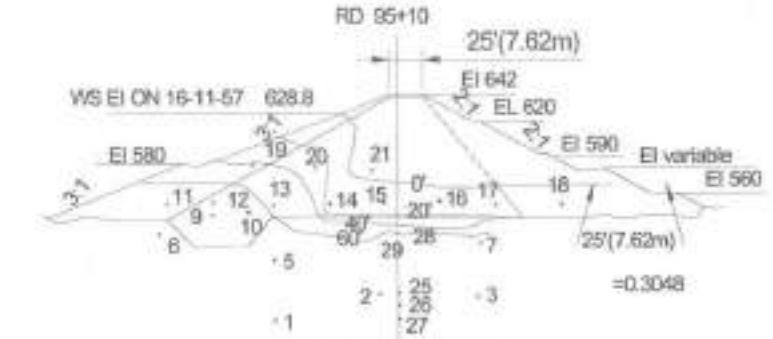
5.3.4. Pore Pressure observation and analysis :

Pore pressures are observed by Bourden gauges placed inside the terminal wells.

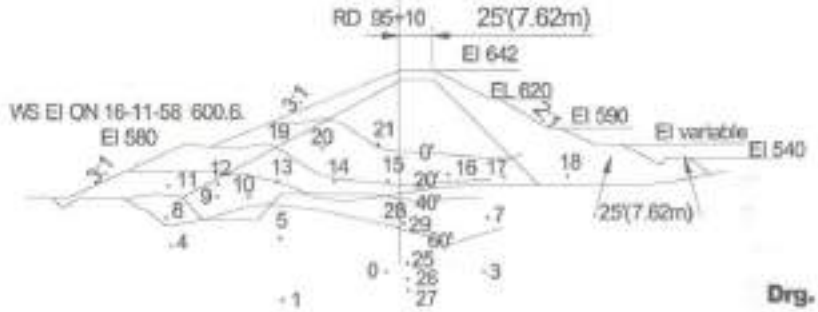
Pore pressure contours were being drawn initially at the end of every month and subsequently at the end of every year quarterly. Typical contours for RD 9510 for two days i.e. for 16.11.57 & 16.11.58 are as shown Drg. No.5.10.

PORE PRESSURE OBSERVATION & ANALYSIS

Pore Pressure Study

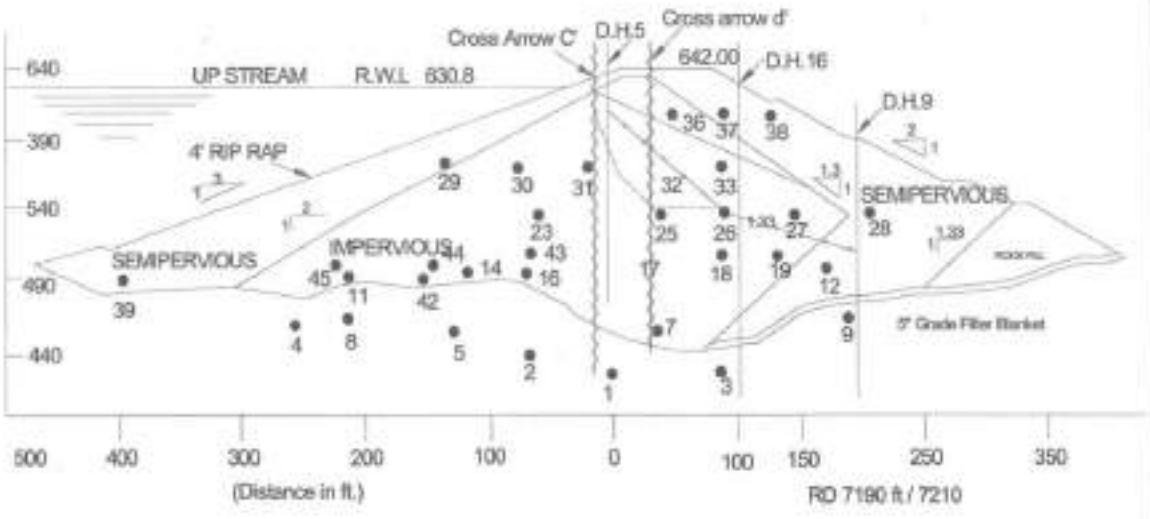


Pore Pressure Contour



Drg. No. 5.10

Pore Pressure Contour



Line of saturation observed from stand pipe

Theoretical line of saturation as per a.casagrade

Actual phreatic line got from piezometer observation

Drg. No. 5.11

5.3.4.1 Discussion on Observation of Pore Pressure :

It is to state that pore pressure during construction has not been observed due to unavailability of Bourden gauges. So the results of observation show only seepage pore pressure, which increases with rise of reservoir level and decreases with draw-down. Rate of increase does not appear to have any relation with rate of rise of reservoir level.

Pore pressure in upstream semi-pervious zone indicated more or less the reservoir water level with some lag depending on thickness of cover on the tip and due to different percentage of clay content in the soil. Pore pressure of most of piezometer tips in this zone increases along with reservoir with a lag of few inches but falls at a comparatively slower rate.

Pore pressure decreases from u/s to d/s from foundation to top of the dam, being negative above phreatic line. High negative value is attributed to the fact that water was not filled in the tube during construction.

Pore Pressure in u/s impervious zone (i.e impervious soil u/s of axis) is slightly different from that in d/s impevious zone. Foundation tips in the u/s semipervious zone behave in the same fashion as those in u/s semipervious material. In the first two years of observation, rate of rise of pore pressure in d/s impervious with rise of reservoir water was slow and now it is almost constant (within a range of 4 inches).

Phreatic lines drawn quartelry show slight seasonal variation in the u/s zone due to change of reservoir level. Correctness of phreatic line has been checkd in deep channel and middle channel by observing secpage line in stand pipes. The observed phreatic line is lower than the theoretical phreatic line drawn in design as per A. Casagrande. These lines droop down in the boundary of impervious and u/s semipervious zone, proceed downwards in a curve and again tend to bend towards d/s semipervious and meet d/s rock toe.

From the observation of pore pressure and phreatic line at R.D. 4900, it is found that cut-off wall is effectively functioning in bringing down path of percolation.

In the impervious zone, the ratio of pore pressure to reservoir level falls off rapidly with distance from the u/s face of the impervious zone, but in the lower part of the embankment and foundation some pressure is noted at points near the d/s of the embankment.

5.3.5 Comparision of prototype study with Design assumptions

Initially pore pressures in 4 terminal wells were observed daily. Subsequently the period of observation was increased to a week or more. Water levels in drill holes embedded at or near these sections were also being observed to know the phreatic line. Theoretical phreatic lines at these sections as given in design of earth dam (vide stability analysis chart given in plates 103 & 104 of book of spacifications and drawings of Hirakud Dam Project-1951) have also been drawn for comparison.

It can be observed that the actual phreatic line observed by pore pressure is below the drill hole water line and theoretical line. This indicates that the dam is behaving well as per the design assumptions and there in no danger to the safety of the dam (vide Drg. No. 5.11).

5.4 Seepage Measurement in Earth Dam & Dykes :

5.4.1. Measurement of Seepage :

Earth dam is not waterproof. Water from the reservoir (because of potential head) seeps through and underneath the dam and oozes out at different valley points beyond the toe of the earth dam in the country side. Sometimes water seeps throught the dam to make the downstrem slope sweaty or wet. Specially this becomes apparent soon after the reservoir is initially filled up. Generally the seepage reduces as time passes on and the soil below the dam in foundation as well as in the body gets consolidated due to settlement of dam & its foundation. This is because of the self weight of soil and surcharge load. If seepage shows any abnormal effect, it is not a healthy situation. The causes may be properly investigated and appropriate remedial measures taken immediately to save the dam from failure which would have caused devastation to life and property in the downstream. The seepage quantity should not exceed the design value, which is normally considered as 0.07 lt/sec/mt. Length of dam.

5.4.2 Measurement of seepage in Hirakud Dam Project :

Immediately after completion of Dam and Dykes in 1957 seepage measuring devices were installed for seepage measurement as given below :

Table 5.13 : Details of seepage measurement device

Sl No	Location	R.D (ft)		No of measuring points	Method of observation
		from	to		
1	2	3	4	5	6
1	Left Earth Dam	0	3600	4	V-notch
2	Right Earth dam	6500	7300	1	Graduated cylinder
	- do -	7300	12200	4	V.notch
	- do -	12200	14200	1	Graduated cylinder
3	Gap-I (left dyke)	0	900	2	V-notch
	- do -	900	1900	2	Graduated cylinder
	- do -	1900	2500	3	V-notch
	- do -	2500	4100	4	Graduated cylinder
	- do -	4100	4800	2	V - notch
	- do -	4800	5250	2	Graduated cylinder
4	Gap-II-L.D.	-	-	Nil	Masonry dyke
5	Gap-III L.D	15000	25897	21	Graduated cylinder
6	Gap-iv-L.D	31450	39750	17	- do -
7	Gap-v-L.D	43300	44800	3	- do -
8	Right dyke	0	35500	85	- do -
			Total	**151**	

(Source : Journal of the Indian National Society of Soil Mechanics & Foundation Engineering Vol-5, no.3 July 1966- seepage from Hirakud Dam by A.B. Jena -pg 292.)

Though initially, observations were taken as mentioned above, subsequently some installations were damaged & new devices were fixed in their places - such as sharp crested V.notches, rectangular notches, masonry V-notches etc. Whenever and wherever a device became difunct volumetric measurement by graduated cylnder was resorted to till new measuring devices were installed.

5.4.3. Details of seepage measuring points :

The following chart gives the Dam component wise details - such as length of component, initial date of measurement, number of measuring points installed & R.D (ft) of measuring points etc.

Table 5.14 : Details of Seepage Measuring Points

S.N.	Dam component	Length (Ft)	No.of devices or points installed	RD. of measuring devices or points in ft.
1(*)	L.E.D	7540	4	1100, 1600, 2700, 3500
2(*)	R.E.D.	4440	6	7000, 7400, 9100, 10800, 12000 & 12400
3 (*)	Gap-I	5600	15	200 to 500, 800, 1200, 1900, 1950, 2200, 2400, 2800, 3100, 3700, 4000, 4400, 4700, 4800, 5200
4 (*)	Gap-III	11500	21	15700, 16700, 17350, 17650, 19150, 19350, 19500, 19700, 19950, 20150, 20800, 21500, 21800, 21950, 22250, 22400,22600, 23300, 23900, 24750, 24950,
5 (*)	Gap- IV	14420	17	32400, 32800, 33200, 33600, 33900, 34200, 34600, 34950, 35300, 36800, 37050, 37400, 37900, 38900, 39200, 39500, 38600
6 (*)	Gap-V	2000	3	43700, 43900, 44200
7 (*)	Right Dyke	35550		
	(a) 1st reach		26	500, 1100, 1500, 1750, 2200, 2400, 2600, 2750, 3200, 3750, 4200, 4600, 4800, 5500, 5800, 6200, 6700, 6910, 8200, 8850, 9300, 9400, 9700, 9800, 11100, 11400
	(b) 2nd reach		28	11900,12290, 12950, 13300, 13500, 14300, 14800, 15400, 16200, 16850, 17050, 17300, 17600, 17700, 18100, 18300, 18760, 18900, 20000, 20600, 20700, 21000, 21300, 21500, 21700, 21900, 22100, 22500
	(c) 3rd reach		25	23000, 23500, 23800, 24200, 24300, 24500, 24700, 24950, 25300, 25700, 26900, 27200, 27400, 27800, 29400, 30500, 30600, 31290, 31560, 31800, 32160, 32400,32650 ,33000, 33400
	Total		**145**	

(*) The date of inttial measurement is serially indicated below :

(1) 01.09.1958, (2) 01.09.1958, (3) 06.05.1961, (4) 03.09. 1958, (5) 02.09. 1958, (6) 02.12.1958, (7) 04.09. 1958

5.4.4 Analysis of seepage data

Voluminous seepage date have been collected and recorded by the C.R.O. Hirakud Reseach Station, Hirakud from 1958 till now. Initially the observations were taken at an interval of about a week. Presently the interval has increased, specifically during summer seasons.

The following preliminary analysis have been made to know the behaviour of seepage in Hirakud Dam Project.

5.4.4.1 Relation of Seepage flow with time & Reservoir level. (1959-67)

The variation of total seepage in cusec from dam and dykes and reservoir level with time has been plotted from 1959 to 1967 (Drg. No. 5.12).

The plot shows that the maximum observed average monthly seepage from entire dam and dykes has decreased form 3.8 cuses in 1960-61 to about 2.5 cusec during 1967-68. This amount is very low in comaprison with total inflow into the reservoir and depth of storage. The quantity of seepage is found to vary with the reservoir level. The seepage is found to increase at higher rate after the reservoir level rises above RL 620' (186.09m) approximately beyond which a number of seepage observation points (which were usually dry earlier) show percolation.

The maximum amount of seepage between 1957-1965 is observed to be 0.35 cusec at about RD 4700 ft of gap-1, where earth dyke abuts against a hillock. A Pervious downstream filter blanket has been provided on a rocky mound almost up to central line of the dyke. If impervious soil blanket would have been provided in the upstream and a small amount of grouting would have been done, seepage might have reduced. This quantity is however not increasing.

5.4.4.2 Study of reduction of seepage with time

(Source - Journal of I.N.S. of S.M. & F. Engg- Pg-293 - by A.B. Jena)

A plot of cumulative seepage in cusec against reservoir level (RL. in ft) is given for the years 1958, 1960 & 1964. The same show that the seepage is reduced with time.

(vide Drg. No.5.13) Seepage reduces generally as time passed on.

5.4.4.3 Present study on relation of seepage flow with reservoir level & time.

As already pointed out voluminous data on seepage flow have been collected and recorded by C.R.O. Hirakud Research Station, Hirakud. The observation on seepage flow and data collection are continuing.

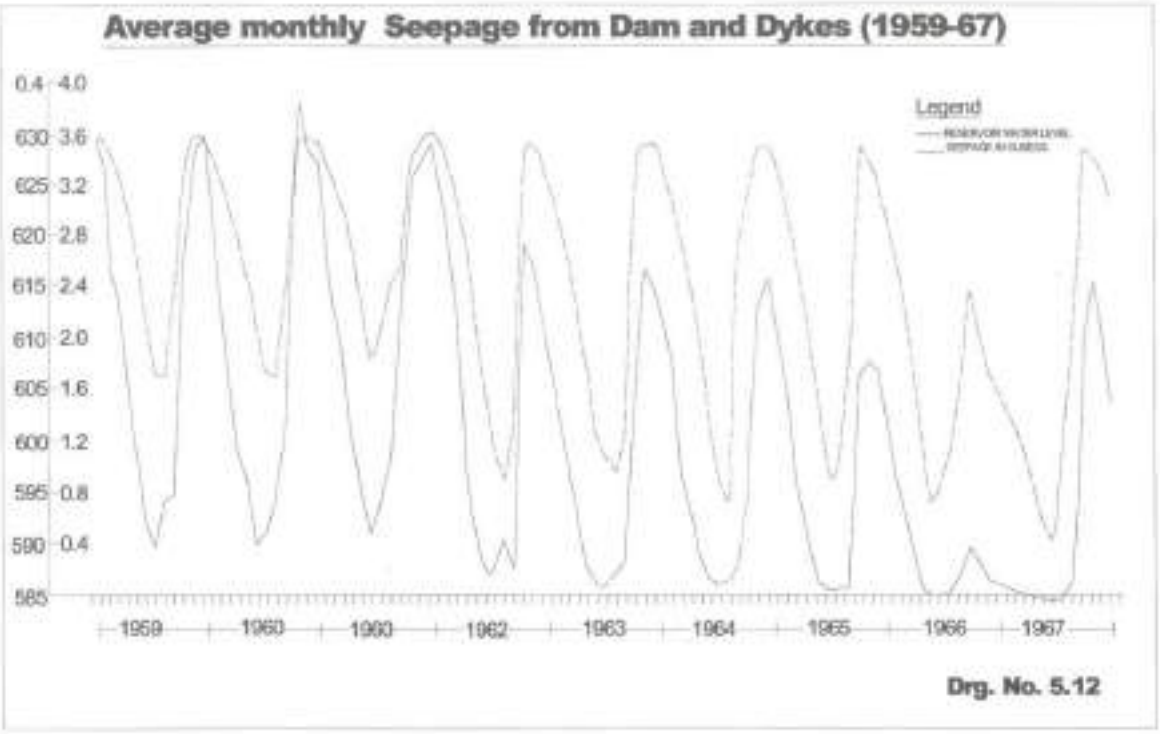
The maximum cumulative discharge from dam and dykes as observed in three exemplary years are produced below :

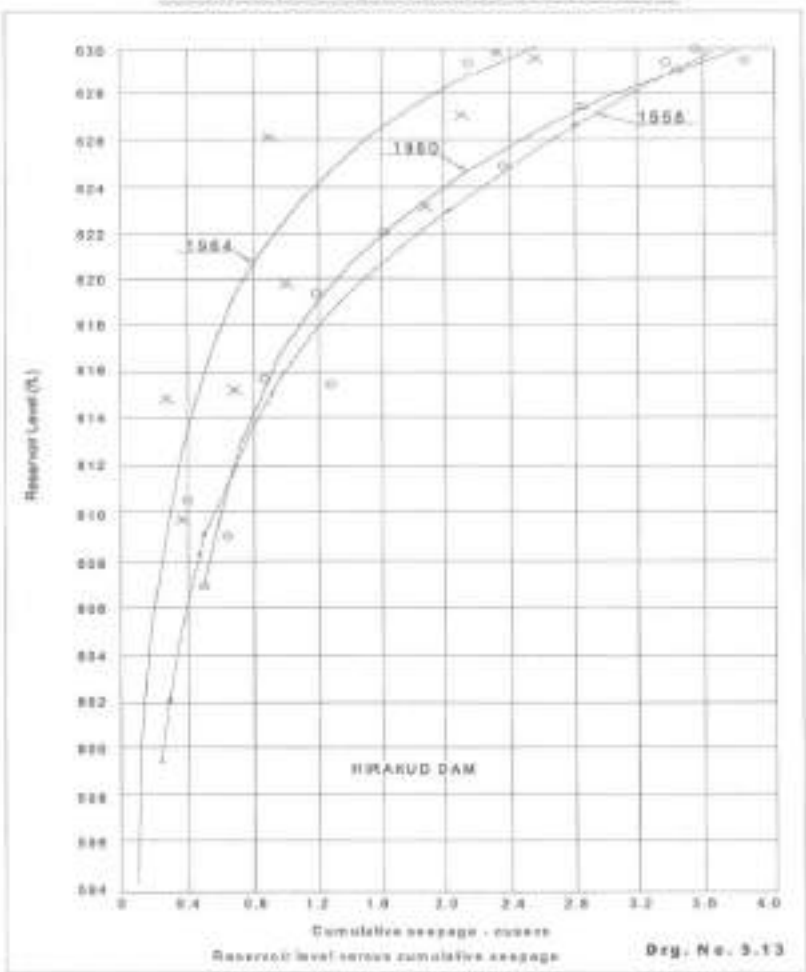
(i)	For the year 1960-61	3.7236 cusec (28.9.60)
(ii)	Year 2000	1.7462 cusec (3.1.00)
(iii)	Year 2008	- 1.4582 cusec (29.9.08)

As indicated above (in 5.4.4.1) the maximum seepage during 1967-68 is 2.50 cusec and during 1960-61 is 3.8 cusecs (approx).

Besides above, seepage data for some years are scanned through and maximum cumulative discharge for all dam and dykes are observed to be as follows in cusecs:

1959	-	3.5988 (28.11.59)
1961	-	3.6176 (18.12.61)
1970	-	2.8547 (16.11.70)
1990	-	2.1823 (22.10.90)


Average monthly Seepage from Dam and Dykes (1959-67)
0.4 4.0
630 3.6
625 3.2
620 2.8
615 2.4
610 2.0
605 1.6
600 1.2
595 0.8
590 0.4
585
Legend
1959
1960
1960
1962
1963
1964
1965
1966
1967
Drg. No. 5.12


YEAR WISE CUMULATIVE SEEPAGE
1964
1960
1958
HIRAKUD DAM
Reservoir Level (ft.)
Cumulative seepage - cusecs
Reservoir level versus cumulative seepage
Drg. No. 5.13

This cleary indicates that the seepage is reducing as time passes on. Hence as regards seepage there is no danger to the Dam and Dykes.

5.5 Seepage in foundation galleries of right spillway

Concrete is not completely impervious. Water even seeps through the concrete spillways depending on reservoir water level in upsteram side. For monitoring, seepage was measured inside the foundation galleries i.e. at RL497′ & 470′ ft coming out from different sources as given below.

(i) Total discharge through old drainage holes, (ii) Total discharge through new drainage holes, (iii) Seepage through formed hole $39F_2$ (iv) Total seepage through other formed holes, (v) Total discharge from pump well, (vi) Total discharge from uplift cells, (vii) Total seepage through additional holes, (viii) Total seepage from 470 gallery, (ix) Seepage through 42D2A, (x) Grand total discharge (from i to ix).

Interval of observation is approximately 15 days. The observation started from 15.5.1960 & is continuing. From the data available between 1960-1996, the maximum and minimum yearly grand total discharges are given below.

Table 5.15 : Yearly Maximum & Minimum discharges (grand total) in gpm/lpm

Year	Maximum discharge			Minimum discharge		
	Date	Max.Q.	RWL(Ft)	Date	MinmQ	RWL(ft)
1	2	3	4	5	6	7
		GPM			GPM	
1960	31.10.60	307.25	630.00	30.7.60	238.375	606.00
1961	31.10.61	262.00	630.20	31.5.61	233.000	607.20
1962	31.01.62	260.875	626.40	15.8.62	225.875	594.45
1963	15.01.63	237.250	623.00	15.8.63	201.500	597.17
1964	31.1.64	239.125	622.89	15.7.64	210.000	592.00
		LPM			LPM	
1964	31.12.64	990.92	626.85	15.8.64	940.18	605.60
1965	15..3.65	995.020	616.81	15.8.65	930.61	600.08
1966	15.1.66	929.16	620.52	15.6.66	795.91	-
1967	15.1.67	945.500	-	15.8.67	657.66	-
1968	15.1.68	790.160	-	31.8.68	525.20	-
1969	15.11.69	787.800	627.45	15.7.69	453.30	598.00
1970	15.11.70	881.000	629.79	31.7.70	463.50	600.83
1971	31.12.71	964.50	625.27	30.6.71	533.50	600.45
1972	15.1.72	964.75	625.27	15.8.72	500.00	607.05
1973	15.10.73	1026.50	630.30	30.6.73	536.000	584.58
1974	15.1.74	1009.50	625.70	15.6.74	650.00	589.80
1975	15.10.75	1176.00	629.95	31.5.75	558.00	589.00
1976	15.1.76	1176.00	624.81	30.6.76	708.80	588.56
1977	31.12.77	1145.10	625.91	15.7.77	485.25	600.45
1978	15.1.78	1225.00	628.79	15.7.78	556.38	608.23
1979	15.1.79	1151.25	624.29	15.7.79	691.75	593.67
1980	30.9.80	1276.95	629.19	15.6.80	609.05	588.45
1981	15.10.81	1359.10	629.62	15.6.81	588.86	603.81
1982	15.1.82	1096.52	622.96	317.82	409.25	600.39
1983	15.10.83	1326.60	630.00	30.6.83	612.45	587.89

Contd....

1984	15.11.84	1344.35	624.79	317.84	758.30	608.00
1985	15.11.85	1332.15	628.65	31.8.85	641.65	623.29
1986	10.12.86	1338.81	624.17	31.5.86	782.28	594.39
1987	14.10.87	1330.07	630.08	30.7.87	640.18	610.71
1988	6.10.88	**1596.25**	630.08	27.6.88	1047.86	589.60
1989	17.01.89	1581.61	603.54	26.7.89	**383.94**	592.10
1990	11.10.90	1222.68	629.91	6.7.90	579.41	600.21
1991	11.10.91	1293.80	630.06	31.7.91	677.70	610.87
1992	6.01.92	1049.76	624.10	31.7.92	556.37	616.83
1993	29.10.93	1035.78	630.10	30.7.93	436.30	601.75
1994	19.1.94	1044.66	622.77	30.7.94	561.61	626.10
1995	31.10.95	1056.22	630.04	22.8.95	429.00	621.93
1996	27.1.96	953.32	625.19	267.96	401.39	609.41

From the above table it can be observed that the maximum seepage was 1596.25 lpm on 6.10.1988 at reservoir level of 630.08ft. and the minimum seepage was 383.94 lpm on 20.07.1989 at reservoir level of 592.10ft.

NB: There was heavy leakage through $39F_2$ block joint. After under water sealing of cavities in the upstream face of spillway blocks in the area adjacent to the joint the leakage has reduced approximately from 1400 lpm to about 300 lpm.

5.6 Relief wells in the Dam & Right Dyke of Hirakud Dam.

5.6.1 Boils & Relief wells in Right dyke.

In the initial period of dam maintenance (1957), there appeared a number of boils a few feet away from the toe of the dyke. This is an abnormal feature. Downstream side heaved up. The top soil cracked up and water with colloidal white clay oozed out. A cow straying into the area stepped over a big boil and got trapped. Her large belly floated in that fluid and she could neither walk nor sink. Later she was pulled out safe.

Inspection, soil exploration, worry and anxiety all got mixed up and some dumpers carried loads of muck from Burla side to load it up. But diagonis was on different line. Pockets of Kaoline existed in areas of decomposed granite and those pockets when constantly charged with seepage water from the reservoir, the volume increased. The top soil cracked and the colloidal clay oozed out. But there was no continuous piping. When the reservoir level went down, the heaving subsided. These boils were very neatly 'plastered' by an inverted filter above and around it. The filter was trapped, loaded to position and the clean seepage water flowed out without endangering the safety of the dam.

5.6.2 History of Development of boils in Right Dyke:

"In October 1957, the reservoir was filled up to R.L. 628.80 and was soon reduced to R.L. 627.90 (the maximum water-level being at RL. 630.00). At this level of water on 20th October 1957, a small boil was noticed at R.D. 24550 on the downstream of service road at a distance of about 120ft.(35.5m.) from the toe. The water was coming upwards carrying very fine cream coloured clayey particles. These boils gradually multiplied over an area of about 40ft. (12.1m.) square in patches. When poked with rod in these soft patches, it was observed that slurry-like material extends to a depth of about 10 ft (3.0m). Subsequently a number of penetrometer tests were made on the area between boils and toe of dyke. Results of penetration tests revealed the existence of soft strata belw ground level all over the vicinity.

The immediate remedial measure that was adopted in this particular case was to weigh down soft patch areas with 5 to 6 ft. (1.5 to 1.8m) of excavated muck containing heavy stones from power

channel. This proved successful in arresting the boils and no further boils in vicinity developed. The berm between the boil area and toe was also weighted down with about 5 ft (1.5m.) of semi-pervious material.

On 12 November 1957, when reservoir level was at R.L.628.80 several boils were noticed between R.D. 23,400 and 23,450 at a distance of 400 to 600 ft. (121.9 to 182.8m) from the toe in a valley at four places. The average ground level in the rear of the dyke at this place is about R.L. 602.10. These boils were 3 to 5 ft.(0.9 to 1.5m.) in diameter and depth varying from 8 to 12 ft. (2.4 to 3.6m.). The soil in this area consists of a crust of laterite 2 to 3 ft.(0.6 to 0.9m.) thick with yellowish clay (more white than yellow) below, which is quite impervious. It was found that the top crust cracked and the boils appeared at the surface. The boils were enclosed in ring bunds [2 to 3 ft.(0.6 to 0.9m.)] high to head up water to counteract the uplift pressure of the vertically piping water thereby reducing the movement of soil particles. This proved succesful.

When the reservoir water-level went down to R.L. 614 in 1958, it was observed that the mud boils had completely disappeared. Soft materials at the site of boils dried up and hardened to the extent of enabling people to walk over it. Even during the monsoon when the reservoir was kept low, the boils did not appear. At the end of the monsoon when the reservoir was filled up, the boils again appeared thereby confirming that the boils were due to seepage from the reservoir through the substratum of residual soil under the cut-off-trench.

During October and November 1958 wet patches were noticed on the downstream slope of dyke from R.D 23,900 to 23,930 between R.L.620.00 - 623.00, when the reservoir was at about R.L. 630.00. In this period there were occasional rains and it could not be dicided beyond doubt if wetness was due to percolation from reservoir. However, as a measure of abundant precaution extra stone pitching was done with filter underneath (from R.D. 23,300 to 24,700). Observation pipes were drilled in the body of dyke designated as P_1, P_2, P_3 . Table 5.16 shows water-levels in these pipes and gradient of saturation line from time to time.

Table 5.16 : Water-level in observation pipes at R.D. 24,500 of Right dyke

Date	Reservoir level in ft.	Observation of W.L. in pipes, R.L. in feet			Hydraulic gradient
		P1	P2	P3	
15.10.1962	630.04	624.43	605.58	600.02	1:4.1
19.11.1962	628.10	-	605.68	599.74	1:4.5
15.12.1962	625.80	-	605.77	599.74	1:5.1
5-1-1963	624.00	-	605.76	599.32	1:5.7

On 22 September, 1959, some boils were noticed in between R.D. 23,400 to 24,000 at a distance of 350 to 500 ft. (106.6 to 152.4m.) from the toe of the dyke, when the reservoir level was 627.70. The whitish materials were coming up by disturbing the top murrum layer. This was found in an area of 3 to 4 ft. (0.9 to 1.2m.) diameter and the ground was soft to a depth of 8 to 10 ft. (2.4 to 3.0m.). The boil portions were enclosed with earthen ring bunds to a height of 2 to 3 ft.(0.6 to 0.9m.) and was allowed to stand over it thereby counteracting uplift pressure of boils. Further remedial measure was taken by providing relief wells upstream of the area towards toe of dyke for easy drainage without carring soil particles. Such relief wells were made by inserting a 3 in (7.6 cm.) diameter perforated pipe in 12 in. (30.5 cm.) diameter perforated pipe (made out of G.I. sheet) containing shingle inside and sand outside. Position of such relief wells are shown in Table no 5.16. Water-level in these wells are being observed from time to time which is found to rise and fall with reservoir level.

In the year 1960 some sand boils were noticed on the drain by the side of service road where fine sand particles were moving up and down. After covering by filter materials of 3 in. (7.6 cm) Shingle over 6 in.(15.2cm) sand and 12 in (30.5cm.) stone,these boils disappeared. But such sand boils appeared in the toe drain of the dyke between R.D 23,000 to 24900 when the reservoir level was about 630.00. This was attributed to the fact that downstream derains were completely choked up with earth slipped from the berm between the toe of the dyke and service road which was made up in 1958 by putting excavated earth of power channal. As a remedial measure the top layer of pitching was removed and was cleared from the surface. Pitching was again done on either slopes and bottom of drains with graded filter of 6 in. (15.2cm.) shingle and 9 in. (22.8cm.) sand for proper drainage without carrying soil particles. Boils which appeared on downstream side of service road at a distance of about 350ft.(106.6m.) from toe between R.D. 23,400 and 24,000 previously also came up in 1960. These boil portions were covered up with semi-pervious earth to a height of 5 to 6 ft.(1.5 to 1.8m.) as a result of which boils subsided. But at R.D. 24,400 a line of boils appeared over the filled up earth at a distance of 129ft.(39-3m.) away from the toe of dyke at about R.L. 598.00 on 19 September 1960. A T-shaped drain was provided for the easy drainage of seepage water without fine particles, as a consequence of which all these boils disappeared and did not occur again.

In 1961 no appreciable change was noticed except re-appearance of some boils in the area which was not covered up by semi-pervious earth. The entire downstream area between R.D.23,000 to 25,000 has been covered up by semi-pervious earth to an average depth of 3 to 4ft.(0.9 to 1.2m.) to a distance of about 300ft. (91.4m.) from toe of dyke.

In 1962 sand boil in the upstream side drain along service road appeared on 25 September 1962 between R.D. 23,800 to 23,900 when reservoir level was 628.65 and subsequently stopped. On 5 October 1962 small boils appeared between R.D. 23,900 to 24,000 on an area of 200ft. x 50 ft. (60.9m. x 15.2m.) at a distance of about 100ft.(30.5m.) downstream of service road when reservoir level was 630.00. As the area was wet, no remedial measure was taken.

A proposal for grouting was considered but was not carried out, as it was too expensive. However, the upstream portion of this reach of right dyke (i.e. from R.D. 23,000 to 25,000) was covered up by an upstream impervious blanket to a depth of about 1ft.(0.3m.) for a distance of 100ft.(30.5m.) upstream from toe and protected by 2ft.(0.6m.) high rubble bund with suitable filter all around the impervious blanket and with three more cross bunds perpendicular to axis of dyke to protect against wave erosion and for assisting deposition of silt. Some stone bunds were also made in the downstream gulleys to arrest further soil erosion.

On 18 September 1963 when reservoir level was at R.L. 623.15 some boils appeared at R.D.23,900 to 24,050 with negligible quantity of whitish water at a distance of 250ft. (76.2m.) from toe and sand soil at R.D.23,750 on the upstream drain of the service road at about 110ft.(33.5m.) from toe.

On 30 September 1963 sand boil appeared at R.D. 24,000 at 146ft. (44.5m.) downstream of toe when reservoir was at R.L. 628.50 and on 4 October 1963 at R.D. 24,680 at edge of dumped earth 175ft.(53.3m.) downstream of the toe when reservoir was at R.L. 629.90. Such sand boils work intermittently and are cosidered to be of local origin (since they die out after a few days of rains in September and October)."

(Source : Behaviour of Right Dyke of Hirakud Dam Project by A.B.Jena - Indian National Society of Soil Mechanics & Foundation Engineering, April 1967, Pg.187-200)

On detailed inspection, it was seen that fine particles were coming out with the water between RD 18100 to 18300ft. at 308ft downstream of toe in country side. Two numbers of relief wells were installed to release pressure and boulders were placed over sand filling in that area. Relief wells were

also provided at RD 23300 to 23400, 23800 and 24500 to 24600 ft. in similar boiling area. Earth was placed in wet patch areas.

The details of relief wells are tabulated below in Table No.5.17.

Fifteen nos of relief wells have been constructed beyond the right dyke toe in country side as detailed below: (Source : Post construction behaviour of Hirakud Dam & Dyke prepared by C.R.O. in 1989)

Table 5.17 : Relief Wells in Right Dyke

Sl. No.	R.D. (ft)	Location-downstream (ft)
1	18100	308
2	18150	308
3	23300	133
4	23300	208
5	23300	203
6	23300	283
7	23300	313
8	23300	343
9	23300	373
10	23800	313
11	24550	38
12	24550	79
13	24550	113
14	24550	178
15	24550	231

5.6.3 Boils & Relief wells in main dam.

After the reservoir was built up some swampy areas developed in the downstream of dam at many places. Earth was dumped and relief wells were installed to release the pressure as detailed in Table 5.18. In total 10 nos of wells were constructed. Now the land is dry and stable.



H.R. of Hirakud Distributary

Table 5.18 : Details of Relief wells at Hirakud Dam :

(Source - Post costruction behaviur of Hirakud dam and dykes-1989 & Data Book).

Sl. No.	R.well No.	R.D in ft.	Location of well (ft) (downstream)	Top level of Well (ft)	Period of observation
1	1	10847	642	554.70	11.5.59 to 31.10.72
2	1A	10940	641	543.11	-
3	2	10678	271	555.96	11.5.59 to 14.2.95
4	2A	10678	369	559.96	1.1.79 to 14.2.95
5	3	10732	369	552.82	- do -
6	3A	10740	369	559.46	11.5.59 to 4.4.78
7	4	8439	380	541.76	11.5.59 to 14.2.85
8	5	10948	374	551.10	- do -
9	6	10943	637	554.46	1.4.68 to 31.10.72
10	6A	10941	631	543.10	- do -
11	6B	10936	610	542.73	-
12	7	10943	592	545.50	-
13	8	8436	368	542.91	11.5.59 to 14.2.95
14	9	8477	335	540.11	-
15	10	8478	380	542.45	1.4.60 to 14.2.95
16.	11	8475	348	539.06	11.5.59 to 20.5.60

The relief wells were provided to release the hydrostatic pressure of water seeping through the earth dam foundation. This will prohibit quick sand phenomenon (Daldali). The Observations are taken initially once in a week. It is seen that the water levels in the relief wells are now below the ground surface at the well site. This indicates that the path of seepage is below the N.S.L in D/s of dam & therefore the dam is safe. There is no swampy area now at these places.

The observations have been discontinued since February-1995.

Discussion on boiling in right dyke:

"In the light of above, it may be stated that sufficient precautions have been taken in the dyke in this reach and it is still kept under observation. The length and thickness of upstream impervious blanket may be increased in future if so required. However from the experiences gained on the behaviour of right dyke in this reach and expenditure incurred in the maintenance, a design engineer has to consider whether he should bring about economy in construction of a dam by choosing the alignment of dam on ridge or he should shift it a little to upstream to avoid uncertainties of geological characteristics of foundation. If ridge alignment is adopted, special precaution should be taken in investigation of in situ permeability of cut-off trench to considerable depth (at least equal to height of dam) and sufficient

impervious natural blanket (at least 10 times height of dam) must be left in the upstream and downstream. In other words borrow areas should be made far away from upstream toe so that water will not form flow channels below the dam giving scope for piping and heavy seepage downstream of the dam. Downstream filter blanket should never be omitted in any earth dam and adequate drainage should be provided outside downstream toe."

5.7 Observation of Water Level in Stand Pipes placed in Drill holes in dam & dykes :

Eleven nos. of drill holes with 4" dia stand pipes have been provided during construction at the dam sections at RD 4900' (L.E.D), at RD 7250' (deep in channel), at RD 9500 (Hirakud Island & at RD 12200 ft (Middle channel portion). Details are given below.

Table 5.19 : Details of drill holes in main dam

Sl. No.	Drill Hole no	RD (ft)	Location in ft	Top R.L. (ft)	Date of 1st observation	Remark
	(a) Left Earth Dam (LED)					
1	1	4923	53D/S	637.99	14.7.56	Functioning
2	2	4852	7 U/S	643.46	14.7.56	- do -
	(b) Deep Channel					
3	1	6575	2U/S	643.01	14.7.56	Blocked
4	5	7265	6U/S	637.70	14.7.56	Functioning
5	6	7250	17U/S	640.37	- do -	- do -
6	6A	7236	7U/S	642.48	- do -	- do -
7	9	7149	195 D/S	598.80	- do -	- do -
8	16	7250	100D/S	630.64	- do -	- do -
	(c) Hirakud Island					
9	10	10750	D/S toe	554.27	- do -	Functioning
	(d) Middle Channel					
10	3	13500	60 D/S	629.23	- do -	- do -
11	5	14100	14U/S	639.53	- do -	- do -
12	1	14050	69 D/s	624.35		Functioning
13	2	14050	134 D/s	594.20		No water
14	3	13800	134 D/s	593.80		No water
15	4	14050	205 D/s	560.55		No water
16	5	13800	69 D/s	622.25		Functioning
17	6	14080	69 D/s	621.15		Functioning
18	7	13800	205 D/s	560.15		No water

N.B. - Drill holes from Sl.12 to 18 (7 nos) are made during 1987 (Source : C.R.O. Hirakud)

Observations are continuing once in every fortnight to know the position of phreatic line.

The above 7 drill holes were made at middle channel due to the reasons given below:

"In 1987, the top road & some patches in downstream slope settled between RD.13800 ft. & 14080 ft. Seven holes were drilled & stand pipes were fixed to know whether the area has been connected to the reservoir. After 3 years it was seen that water level observed in the first year was due to accumulated rain water. The settlement was due to improper compaction of soil near the key wall." Further 48 nos (Sl. 19 to 66) of drill holes have been made in dam & dykes.

Table 5.20 : Statement Showing Further Drainage Holes made in main dam & dykes (Source - Post construction behaviour of Hirakud Dam & Dykes, 1989 & Data book)

Sl. No.	Drill Hole no	RD (ft)	location in ft	Top R.L. (ft)	Date of 1st observation	Remark
	Deep Channel					
19	C	7250	210 D/S	590.43	14.7.56	Blocked
20	D	7250	165 D/S	617.60	-do-	-do-
21	-	7263	6 U/S	639.90	-do-	
22	-	7236	5 U/S	638.85	-do-	
23	-	7250	17 U/S	635.45	-do-	
24	-	7425	26 U/S	560.15	-do-	
25	-	7325	0.0	638.25	-do-	
26	-	7300	190 U/S	578.80	-do-	
27	-	7250	110 D/S	-	-do-	
28	-	7500	355 U/S	533.40	-do-	
29	-	7350	148 U/S	593.25	-do-	
30	-	7400	110 U/S	606.25	-do-	
31	-	7300	75 U/S	618.86	-do-	
32	-	7125	0-axis	638.25	-do-	
33	-	7200	0-axis	637.15	-do-	
34	-	7150	42.5 D/S	643.25	-do-	
35	-	7200	75 D/S	642.25	14.7.56	
36	-	7160	130 D/S	-	-do-	
37	-	6690	22 D/S	645.40	-do-	
38	-	7391	227.5 U/S	570.87	-do-	
	Hirakud Island					
39	7	10750	71.5 D/S	621.61	-do-	Blocked
40	13	10750	136.5 D/S	591.35	-do-	-do-
41	14	8475	73.5 D/S	620.43	-do-	-do-
42	15	8475	165.5 D/S	592.01	-do-	-do-
43	-	8475	348 D/S	589.74	-do-	-do-
	Middle Channel					
44	1	12200	47 D/S	643.80	6.9.57	Blocked
45	4(D)	14112	50 D/S	633.66	6.9.57	No water
46	5	14100	14 U/S	639.53	-	Functioning
47	A	12198	25 D/S	565.60	147.56	-
48	B	12200	60 U/S	613.91	14.7.56	-
49	3	13500	60 D/S	629.23	14.7.56	-

Contd...

	Gap-I - Left dyke					
50	1	1050	50 D/S	608.00	-	Blocked
51	2	1050	52 D/S	619.56	-	Blocked
52	3	1050	64 D/S	630.00	-	-do-
53	4	4250	50 D/S	611.00	-	-do-
54	5	4250	51 D/S	621.41	-	-do-
55	6	4250	51 D/S	631.08	-	-do-
	Gap-IV - Left dyke					
56	1	35597	56 D/S	621.61	25.7.63 to 9.6.67	Blocked
57	2	35500	55.75 D/S	621.13	23.7.64 to 6.8.64	Blocked
58	3	35489	55 D/S	621.47	19.7.62 to 9.2.67	Blocked
59	4	35510	56.4 D/s	621.12	2.4.64-9.2.67	-do-
60	5	36510	35.5 D/s	630.83	25.6.64-26.8.65	-do-
61	6	35489	55.33 D/s	631.51	11.6.64-26.8.65	-do-
62	7	35949	55.33 D/s	621.26	2.5.61-9.2.67	-do-
63	8	36057	56 D/s	621.29	2.4.64-9.2.67	-do-
64	9	36000	56 D/s	630.42	2.4.64-9.2.67	-do-
65	10	34650	56 D/s	632.97	2.5.64-13.8.64	-do-
66	11	32520	56 D/s	633.93	11.7.63-9.2.67	-do-

11 nos. of drill holes as given above were drilled in downstream side of gap-IV beyond toe in countryside to monitor the water levels inside them. Datas were available upto 09.06.1967. As reported all the holes are blocked & no observation has been made beyond 9.6.67.

Table 5.21 : Abstract of Drill Holes

Sl.	Place	Nos.	
1.	Left Earth Dam	2	
2.	Deep Channel	26	
3.	Hirakud Island	6	49 in main dam.
4.	Middle Channel	15	
5.	Gap-I	6	
6.	Gap-IV	11	17 in dykes
	Total	**66**	

From the observations of water level inside 66 nos. of holes (vide Table No. 5.21), it is reported that the level is always below NSL or dam surface. Hence there is no danger of NSL becoming slushy. From water levels in pipes embedded in dam it is reported that the phreatic line is below the theoretical line drawn as per method given by A. Cassagrande. Hence in this respect the performance of dam & dykes is very satisfactory.

CHAPTER VI

POST CONSTRUCTION ISSUES

6.0 Flood Forecasting and Reservoir Operation

6.1 Introduction:

Any reservoir built across a river for the benefit of mankind, animal and plant kingdom needs judicious and planned operation as regards storage, release and utilisation of water commensurate with time, climatic conditions, the purpose for which it is built, keeping in view, the safety of lives and properties both in upstream and downstream of the dam.

Some reservoirs are built for single purpose viz. flood control, irrigation, hydro-power, navigation, pisciculture or drinking water. Any reservoir built for catering for more than one purpose is classified as multipurpose reservoir. Hirakud reservoir was built primarily for three purposes in order of preference i.e. flood control, irrigation & hydro-power (vide Recommendation of flood advisory committee 1938-42). Subsequently as per revised project report (1952), the priorities were changed to (i) Irrigation (ii) Flood control, (iii) Power generation & (iv) Navigation. Pisciculture was a subsidiary aim. Navigation was finally dropped.

Operation of uni-purpose reservoir is relatively easier. Hirakud multipurpose reservoir aims to serve three conflicting requirements as stated above. Its operation is complex in nature as none of the possible uses of reservoir are entirely compatible. Consumptive uses like irrigation and navigation require clean allocation of storage space where as power generation is least compatible. In Hirakud reservoir no clear storage space has been allocated for Flood Control. Any erratic operation may not only reduce the benefit but may create disaster mainly in the downstream of the reservoir.

"According to WCD 2000 (World Commission on Dams) many large storage projects world wide are failing to produce the level of benefits that provided the economic justification for their development. This may be due in some instances to an inordinate focus on project design and construction, with inadequate consideration on more mundane operations and maintenance issues once the project is completed. Performance related to original project purposes may also be undermined when new unplanned uses arise that were not originally considered in the project authorization and development. These might include municipal or industrial water supply, minimum streamflow requirements for environmental or ecological concerns, recreational enhancement, and accommodating shore line encroachment and development. Although there may exist some degree of commensurability among these diverse project purposes, there is more often conflict and competition, particularly during pervasive drought conditions." (Source : Optimal Operation of Multireservoir Systems: State of the Art Review by John W. Labadie - Journal of Water Resources Planning and Management ASCE, March/ April 2004) Besides, performance of publicly owned reservoir systems is often restricted by complex legal agreements, interstate issues and pressures from various special interest groups.

6.1.1 Flood Forecasting :

6.1.1.1 Flood Control Room and Hydrological Setup :

Before reservoir operation is effected, engineers in charge must assess (i) the flood magnitude, amplitude and frequency of the approaching storm well ahead. For this sufficient hydrometeorological datas are required. (ii) They should also be conversant with the basin characteristics of the river system. Feeling of the morphology of the river system starting with its origin to its outfall point with sea or with any other water body is a bare necessity. (iii) The engineers in charge must have a clear knowledge about the existing controlling structures at the dam site and on structures and properties both in the upper and lower reaches of the dam. They should collect the salient features of important storage structures located both in upstream and downstream catchments. The dam operators must also have prior knowledge regarding releases from upstream reservoirs. The controlling flood levels and flood flows at different important locations along the river system should also be known.

For effective and efficient reservoir operation a temporary flood control cell is opened at Burla, the headquarters of Hirakud Dam Project every year from 1st June to atleast 31st October i.e. during the monsoon period. Superintending Engineer, Hirakud Dam circle, Burla remains in over all charge of the cell, who is assisted by EE, AE, JE and other officials for smooth operation. It functions round the clock during the monsoon.

6.1.1.2 Collection & Grasping of Hydrometeorological Data:

Field metrological office (F.M.O) of I.M.D., Bhubaneswar, transmits hydrological data including synoptic situation prevailing in upstream and downstream basins ending 08.30 hrs. and 17.30 hrs. daily to flood control cell, Burla during monsoon period.

A long term monsoon forecast is also issued by F.M.O. in the month of May/June every year prior to approaching monsoon basing on various climatological factors numbering about 18. The long term prediction provides an overall idea about percentage of rainfall expected during the monsoon compared to the normal value. This gives a rough indication whether the monsoon rainfall will be approximately normal, heavy or scanty.

The daily I.M.D message in addition to specifying its origin and time of prediction also specifies.

- Intensity of rainfall
- Rainfall conditions
- Weather conditions
- Synoptic situation
- Rainfall which has occured during last 24 hrs. over some stations.
- Q.P.F (Quantitative Probable Forecast) values if expected

Details of weather forecast are indicated below :

Weather Forecast

a) Rainfall intensity

(i) Drizzle	-	<0.1cm / hr
(ii) Light	-	0-0.28cm / hr
(iii) Moderate	-	0.28-0.75 cm / hr
(iv) Heavy	-	>0.75 cm / hr.

b) Rain drop characteristics-

i) Drizzle drops	-	<0.05 cm dia.
ii) Rain drops	-	0.05cm - 0.63cm dia.

c) Rain Strength -

i) Heavy when point rainfall exceeds 50mm/day.

ii) Very heavy when point rainfall exceeds 130 mm/day

d) Rainfall conditions-

(i) Dry	-	No rainfall
(ii) Mainly dry	-	When rainfall is <2.4mm
(iii) Isolated	-	When rain occurs in 1 to 25 % of stations.
(iv) Scattered	-	When rain occurs in 26-50 % of stations
(v) Fairly wide spread	-	When rain occurs in 51-75% of stations.
(vi) Wide spread	-	When rain occurs in ≥76% of stations.

e) Weather Conditions- (As per WMO) Circulatory speed of cloud

(i) Low pressure area	-	<17 knots (32 km/hr.)
(ii) Depression	-	17-27 knots (32-50 km/hr.)
(iii) Deep Depression	-	28-33knots (50-61 km/hr.)
(iv) Cyclonic storm	-	34-47knots (61-87 km/hr.)
(v) Severe Cyclonic storm	-	48-63knots (87-117 km/hr.)
(vi) Severe Cyclonic storm with core of hurricane wind.	-	>64 knots (>117 km/hr.)

where 1 knot = 1.1 mies/hr = 1.85 km/hr.

Flood producing rainfall in the Mahanadi catchment is associated with cyclone that originates in Bay of Bengal and moves inland generally in Westerly or North Westerly direction. The time of travel of cyclone to the farthest tip of catchment is generally two days. Rain is thus caused first over the delta and lower reach and then moves in a direction opposite to the flow of the main river for the greater part of the river length. This is expected to reduce the chance of synchronisation of flood waters from the upper and lower regions of the catchment. It may however be noted here that amount and sequence of heavy rainfall spells within the storm and their location over different parts of the catchment, which are the chief factors in the formation of peak discharge from large areas and vary from storm to storm and result in large deviation occasionally from the normal pattern expected from the typical movement of cyclones.

It may be noted here that cyclonic rain bearing cloud cover even upto 5000 sq.km and move with translatory speed of 30 km/hr. and circulatory speed up-to 200 km/hr. Usually the cyclone covers the whole catchment within two days.

The Sub-basin wise and Statewise catchment area of Mahanadi Basin is given in Table 6.1 & 6.2

Table 6.1 : Sub-basin details of Mahanadi Catchment

Details of Catchment	Area in sq.km	Area in sq.mile
Total catchment upto Bay of Bengal	141134	54513
Total catchment upto Naraj/Mundali (the Delta head)	132100	51000
Total catchment upto Hirakud	83365	32200
Catchment below Naraj	9034	3489
Catchment from Hirakud to Khairmal	33700	13000
Catchment from Khairmal to Barmul	8300	3200
Catchment from Barmul to Naraj	6700	2600

Table 6.2 : Statewise details of Basin :

State	Area in sq.km / (sq mile) and % of total	
	Catchment upto dam	Total catchment
Madhya Pradesh (now Chhatishgarh)	75136 (29021)	75136 (29021)
Orissa (now Odisha)	7859 (3036)	65628 (25349)
Bihar (now Jharkhand)	132 (51)	132 (51)
Maharashtra	238(92)	238 (92)
Total	**83365(32200) 100%**	**141134(54513) 100%**

Average Monthly Rainall & Run off of Catchment during post construction period are given in Table No 6.3

Table 6.3 : Average Monthly Rainfall and Run off of Catchment

(Post construction period)

Month	Average monthly (u/s) rainfall (mm)	Average monthly runoff (M.Ac.ft.)
January	N.A	0.158
February	N.A.	0.113
March	N.A.	0.084
April	N.A.	0.046
May	N.A.	0.020
June	159.689	1.106
July	379.304	6.286
August	364.743	10.357
September	192.623	6.761
October	45.324	1.888
November	N.A.	0.497
December	N.A.	0.211
Total	1141.683	27.527

Rainfall Series (1959 to 1996)
Avg. monsoon-1141.683 mm.
Max. monsoon -1928mm. in 1961
Min monsoon - 691 mm. in 1979

Runoff Series (1958 to 1996)
Avg. annual-27.527 M.Ac.ft.
Max. annual - 73.629 M.Ac.ft.in 1961
Min.annual - 9.244 M.Ac.ft.in 1979

Design discharge of Spillway:

a) As apportioned from gauge discharge curve at Naraj 11.5 lakh cusec. (1947)

b) As per International Engineering Company = 18.3 lakh cusec (Volume-29 Macft.)

c) As per Revised P.R. in 1952 (from flood frequency).

= 15.0lakh cusec (500 yr. Flood) i.e. **Design Flood.**

d) As per Expert Committee Report 1976 (Unit Hydrograph) = 28.74 lakh cusec (17.11 macft)

e) As per Expert Committee Flood frequency method (10,000 year Flood)

(i) By maximum likelihood method =26.85 lakh cusec.

(ii) By Gumbel's frequency factor (Method of moments) =23.53 lakh cusec.

Discharge through (40+24) =64 sluice = 9 lakh cusec (each sluice 10'x20'4")at FRL 630'

Mahanadi carries a maximum of 74 Macft of water per year which can cover whole of Orissa by 2' of water (1957 Jan 13 CBIP Report).

Peak discharge generated in one flood event (at dam site)

(i) 16 lakh cusec as in 1961 (weighted av.rainfall -192 mm)

(ii) 13 lakh cusec as in 1980 (weighted av.rainfall -139 mm)

Safe gauge lavel at Naraj =RL 89.00 ft corresponding to discharge of 10 lakh cusec

6.1.1.3 Structures in the Upstream and Downstream of Dam

6.1.1.3.1 Structures in the Upstream

At the time of completion of Hirakud Dam in 1956 and for some years there after there were no major or medium Irrigation projects existing in upper reaches of Hirakud dam. Therefore inflow to the resorvoir was mainly dependant on rainfall over the catchment. In the meantime the state Goverment of M.P (now Chhatishgarh) have constructed many Irrigation Projects in upper reaches of Hirakud Dam which adversely affect the flow condition in the catchment. They impound and release flood water as per their need without informing the Hirakud authorities in real time. This creates difficulty in operating Hirakud reservoir. In absence of real time data the old practice of assessing the inflow from rainfall is followed.

Irrigation projects constructed and proposed in upper reaches of Hirakud Dam by Chhatishgarh Govt. as per available information is furnished below.

Table 6.4 : Projects upstream of Hirakud Dam

Type of Project	No. of Projects	Total C.A. (sq.mile)	Total live storage (Macft)	Spillway (design) discharge (lakh cusec)
Dam (Major)	8	5277	4.058	21.644
Dam (Medium)	22	1260	0.547	3.000
Barrage & weir	10	-	-	-
Total (a)	40	6537	4.605	24.644
b) Projects under Construction :				
Dam (Medium)	4	184	0.125	1.285
Barrage & weir	1	-	-	-
Total (b)	5	184	0.125	1.285
c) Proposed Projects :				
Dam	5	2605	0.394	N.A.
Barrage & weir	1	-	-	-
Total (c)	6	2605	0.394	N.A.
Grand Total (a+b+c)	51	9263	5.124	25.929

6.1.1.3.2 Features in the Downstream of Dam

There are few important structures like bridges and barrages etc. in the downstream of the dam. The dam engineers should have fair knowledge regarding hydraulic parameters of those structures such as (i) Bridges at Burla, Sonepur, Boudha, Sidhamula (Narasinghpur), Subarnapur (under construction) (ii) Barrages across undivided Mahanadi at Mundali, across Mahanadi at Jobra, across Birupa at Jagatpur and (iii) NH-5 road bridges at Cuttack.

The release from the reservoir is to be so regulated that such release combined with uncontrolled discharge from downstream (such as river Tel etc)remains ordinarily within the safe limit of 9.0 lakh cusec at Naraj/Mundali, the delta head. The safe gauge at Naraj is 89.0ft. The travel time of discharge from Hirakud to Mundali varies from 36 hours to 48 hrs. It therefore follows that the discharge of downstream catchment has to be assessed nearly 48 hours in advance so that release from dam can be regulated accordingly.

The Hirakud release in normal times should be limted to 4 lakhs cusec in order to avoid ingress of water into Sambalpur town. It has been observed from past floods that the peak discharge of zone Hirakud to Khairmal usually produces flood peak at Mundali. Due to contribution from catchment between Khairmal & Mundali, maximium of 2.0lakh cusec is normally considered to be added to peak at Khairmal, though maximum peak contribution from this zone was of the order of 6.80 lakhs cusec in 1991.

As the safe flood discharge at Naraj has been considered as 9.0 lakh cusecs, the safe flood discharge at Khairmal is therefore 7.00 lakh cusec. Since the travel time of flood discharge from Hirakud to Khairmal is 12-16 hrs. the peak discharge from this zone is to be predicted atleast 12 hrs. in advance to regulate release from reservoir so as to limit the flood to 7 lakh cusec at Khairmal (vide Table 6.5)

The safe discharge at the head of delta i.e Naraj was earlier taken as 9.00 lakh cusec as the flood embankments along Mahanadi and its branches were weak. In the mean time, the embankments in delta have been strenghtened. A scheme has also heen submitted to Central Water Commission for technical appraisal to strenghthen all the flood embankments to withstand flood upto 14.0 lakh cusec in undivided Mahanadi. Table 6.6 shows when Flood gauge at Naraj exceeded 89 ft.

Table 6.5 : Travel Time & Time lag for different Gauging Stations

Sub-basin/ zone/gauge	Time of concentration from origin to gauging site (in hours)	Travel time to reach the dam/between gauges(hr.)	Total time lag (in hr)
Saradihi	24	12	36
Tarapur (on Mond)	09	15	24
Deogaon(on Ib)	24	09	33
Ungagued sub basin	03	03	06
Dam to Khairmal	-	12 to 16	-
Khairmal to Barmul	-	12 to 16	-
Barmul to Mundali	-	12 to 16	-

Table 6.6 : Year when flood gauge at Naraj was higher than 89.00ft (1868-1957)

Year	July	August	September
(A) Pre-construction			
1872	92.00	-	-
1874	-	90.00	-
1879	-	90.90	-
1880	89.10	-	-
1892	91.74	-	-
1894	89.40	-	-
1895	90.40	89.22	-
1896	91.95	91.48	-
1907	-	-	89.70
1909	-	90.09	-
1910	-	91.25	-
1914	-	89.90	-
1917	-	89.25	-
1919	-	90.28	90.17
1920	91.88	89.62	-
1925	91.05	-	-
1926	-	91.65	-
1927	-	-	89.49
1929	89.02	91.70	-
1931	-	-	-
1933	89.70	91.45	-
1934	-	89.60	89.60
1935	-	-	89.60
1937	-	91.20	-
1939	-	89.90	91.10
1940	91.30	90.10	-
1943	-	-	89.70
1944	90.30	91.10	-
1946	-	90.40	-
1947	-	-	91.15

(B) Post Construction (1958-2009)

Year	July			August			September		
	Gauge at Mundali in ft.	Discharge at Mundali in lakh Cusecs.	Maxm gauge at Naraj in ft	Gauge at Mundali in ft.	Discharge at Mundali in lakh Cusecs.	Maxm gauge at Naraj in ft	Gauge at Mundali in ft.	Discharge at Mundali in lakh Cusecs.	Maxm gauge at Naraj in ft
1	2	3	4	5	6	7	8	9	10
1980	-	-	-	-	-	-	98.20	12.70	91.14
1982	-	-	-	99.70	15.84	n.a	-	-	-
1991	-	-	-	95.90	12.71	89.04	-	-	-
1992	95.2	11.34	89.24	95.10	11.26	89.76	-	-	-
1993	-	-	-	89.00	7.80	-	-	-	-
1994	93.6	10.25	-	89.90	7.78	-	93.90	10.81	89.30
1998	-	-	-	-	-	-	90.50	8.21	-
2001	97.50	14.08	-	-	-	-	-	-	-
2003	-	-	-	97.20	13.50	-	95.80	12.34	-
2005	-	-	-	91.60	9.03	-	-	-	-
2006	-	-	-	96.70	13.08	-	-	-	-
2008	-	-	-	-	-	-	99.90	15.81	-
2009	-	-	-	-	-	-	-	-	-

6.1.1.4 Procedure for Flood Forecasting

Computation of inflow into reservoir at dam site:

The inflow is computed from the relation Inflow (I) = Out flow (0) + change of storage (S). Out flow comprises of (i) spillway release (ii) Power house release (iii) Release of water for Irrigation and (iv) Evaporation loss

Out flows are calculated by usual hydraulic formula with co-efficients found on the basis of model study.

Discharge per sluice at different reservoir levels for free flow condition and for crest gate are given in a chart which can be used convenienty. Similarly chart has been developed to find out the discharge per bay of crest gates (vide Table No 6.7 & 6.8)

Release through canals for Irrigation is calculated from regulator opening & differential head on the basis of rating curves/charts of individual head regulator. The rating chart for Sason Canal H.R. is enclosed vide Table No3.6(a).

Evaporation loss is calculated for a particular period from average surface area of reservoir multiplied with expected evaporation depth during the period. For this purpose the revised Area-Capacity curve (1988) is used. The average monthly pan evaporation loss as found out by Hirakud Research Station from observed data between (1974-2000) is now used (Refer Table No. 6.9)

Change of storage is calculated by using the revised Area capacity curve (1988).

Table N o.6.7 Discharge through sluice at different levels

Formula $Q=0.8\,A\,(2gH)^{0.5}$

All levels in ft. All discharge figures in cusec

RL(ft)	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
585	12650	12658	12666	12674	12682	12690	12698	12706	12714	12722
586	12730	12739	12748	12757	12766	12775	12784	12793	12802	12811
587	12820	12830	12840	12850	12860	12870	12880	12890	12900	12910
588	12920	12929	12938	12947	12956	12965	12974	12983	12992	13001
589	13010	13020	13030	13040	13050	13060	13070	13080	13090	13100
590	13110	13119	13128	13137	13146	13155	13164	13173	13182	13191
591	13200	13209	13218	13227	13236	13245	13254	13263	13272	13281
592	13290	13300	13310	13320	13330	13340	13350	13360	13370	13380
593	13390	13399	13408	13417	13426	13435	13444	13453	13462	13471
594	13480	13489	13498	13507	13516	13525	13534	13543	13552	13561
595	13570	13579	13588	13597	13606	13615	13624	13633	13642	13651
596	13660	13669	13678	13687	13696	13705	13714	13723	13732	13741
597	13750	13759	13768	13777	13786	13795	13804	13813	13822	13831
598	13840	13849	13858	13867	13876	13885	13894	13903	13912	13921
599	13930	13939	13948	13957	13966	13975	13984	13993	14002	14011
600	14020	14028	14036	14044	14052	14060	14068	14076	14084	14092
601	14100	14109	14118	14127	14136	14145	14154	14163	14172	14181
602	14190	14199	14208	14217	14226	14235	14244	14253	14262	14271
603	14280	14288	14296	14304	14312	14320	14328	14336	14344	14352
604	14360	14368	14376	14384	14392	14400	14408	14416	14424	14432
605	14440	14449	14458	14467	14476	14485	14494	14503	14512	14521
606	14530	14539	14548	14557	14566	14575	14584	14593	14602	14611
607	14620	14628	14636	14644	14652	14660	14668	14676	14684	14692
608	14700	14708	14716	14724	14732	14740	14748	14756	14764	14772
609	14780	14788	14796	14804	14812	14820	14828	14836	14844	14852
610	14860	14869	14878	14887	14896	14905	14914	14923	14932	14941
611	14950	14958	14966	14874	14982	14990	14998	15006	15014	15022
612	15030	15038	15046	15054	15062	15070	15078	15086	15094	15102
613	15110	15118	15126	15134	15142	15150	15158	15166	15174	15182
614	15190	15198	15206	15214	15222	15230	15238	15246	15254	15262
615	15270	15277	15284	15291	15298	15305	15312	15319	15326	15333
616	15340	15349	15358	15367	15376	15385	15394	15403	15412	15421
617	15430	15438	15446	15454	15462	15470	15478	15486	15494	15502
618	15510	15518	15526	15534	15542	15550	15558	15566	15574	15582
619	15590	15598	15606	15614	15622	15630	15638	15646	15654	15662
620	15670	15678	15686	15694	15702	15710	15718	15726	15734	15742
621	15750	15757	15764	15771	15778	15785	15792	15799	15806	15813
622	15820	15828	15836	15844	15852	15860	15868	15876	15884	15892
623	15900	15908	15916	15924	15932	15940	15948	15956	15964	15972
624	15980	15987	15994	16001	16008	16015	16022	16029	16036	16043

Contd...

625	16050	16059	16068	16077	16086	16095	16104	16113	16122	16131
626	16140	16147	16154	16161	16168	16175	16182	16189	16196	16203
627	16210	16218	16226	16234	16242	16250	16258	16266	16274	16282
628	16290	16297	16304	16311	16318	16325	16332	16339	16346	16353
629	16360	16368	16376	16384	16392	16400	16408	16416	16424	16432
630	16440	16443	16447	16450	16454	16457	16460	16464	16467	16471
631	16474	16479	16483	16488	16492	16497	16502	16506	16511	16515
632	16520	16528	16536	16544	16552	16560	16568	16576	16584	16592
633	16600	16606	16612	16619	16625	16631	16637	16643	16650	16656
634	16662									

(Source : Data on Flood Control Operation of Hirakud Dam, Sept.1993, pg. 29-30)

Table No 6.8 gives the discharge over crest bays at different levels.

No.of Crest-bays on right Side -13 Nos

No.of Crest-bays on left Side -21 Nos

Table 6.8 : Discharge over crest bays (Per bay in cusec)

R.L. (ft.)	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	Co-efficient of discharge
610	0	17	35	52	70	87	104	122	139	157	3.40
611	174	206	237	269	300	332	364	395	427	458	3.41
612	490	531	572	614	655	696	737	778	820	861	3.41
613	902	951	1000	1048	1097	1146	1195	1244	1292	1341	3.42
614	1390	1445	1501	1556	1612	1667	1722	1778	1833	1889	3.43
615	1944	2006	2067	2129	2190	2252	2313	2375	2436	2498	3.44
616	2559	2626	2693	2760	2827	2894	2960	3027	3094	3161	3.45
617	3228	3300	3372	3444	3516	3588	3660	3732	3804	3876	3.46
618	3948	4025	4102	4178	4255	4332	4409	4486	4562	4639	3.47
619	4716	4797	4879	4960	5041	5123	5204	5285	5366	5448	3.48
620	5529	5618	5708	5797	5886	5976	6065	6154	6243	6333	3.49
621	6422	6514	6607	6699	6792	6884	6976	7069	7161	7254	3.52
622	7346	7443	7540	7637	7734	7830	7927	8024	8121	8218	3.54
623	8315	8414	8512	8611	8709	8808	8907	9005	9104	9202	3.56
624	9301	9403	9506	9608	9711	9813	9915	10018	10120	10223	3.57
625	10325	10441	10556	10672	10788	10904	11019	11135	11251	11366	3.58
626	11482	11596	11710	11824	11938	12052	12165	12279	12393	12507	3.62
627	12621	12739	12857	12975	13093	13212	13330	13448	13566	13684	3.64
628	13802	13924	14046	14168	14290	14413	14535	14657	14779	14901	3.66
629	15023	15145	15266	15388	15509	15631	15752	15874	15995	16117	3.68
630	16238		-	-	-	-	-		-		3.69

Crest discharge calculated by formula $Q=CLH^{3/2}$, Where C is obtained from the graph in plate II, Drawing 100.H.10 - Schedule of specifications and drawings of Hirakud Dam Project, L per bay = 510 -0.09 H and H= E.L.610' (Crest elevation.)

(Source : Data on Flood Control Operation of Hirakud Dam, sept 1997 pg.96)

Table No.6.9 Pan Evaporation loss of Hirakud from 1974-2000

Month	As provided in Project Report (cm)	From 1974-95 (22 yrs) in cm	From 1974 to 2000 (27 yrs) inch (cm)
1	2	3	4
January	10.16	5.700	2.09 (5.31)
February	10.16	8.182	3.13 (7.95)
March	17.78	13.297	5.21 (13.23)
April	22.86	17.650	6.62 (16.81)
May	25.40	20.446	7.98 (20.27)
June	17.78	14.592	5.71 (14.50)
July	15.24	7.849	3.18 (8.08)
August	15.24	7.298	2.85 (7.24)
September	15.24	8.490	3.31 (8.41)
October	12.70	8.949	3.42 (8.69)
November	10.16	6.668	2.49 (6.32)
December	10.16	4.889	1.80 (4.57)
Total	182.88	124.010	47.79 (121.38)

(Source : Hirakud Research Station, Data on Flood Control operation of Hirakud Dam, Sept. 1997 Pg.104)

6.1.1.5. Computation of Average Rainfall

There are more than 200 raingauge stations in whole of Mahanadi basin of which about 100 are in Upper basin. These are maintained by different agencies. But real time data are received from 13 rain & river gauge stations (8 upstream + 5 downstream) maintained by Hirakud Dam Project. Hirakud Dam Project has already installed its own frequency wireless system at all G & D sites and Master Control Station (MCS) at Burla since 2004 and all G-D stations have been connected with MCS with wireless network. Datas are collected round the clock.

Similarly Mahanadi Division, Burla under C.W.C. which was established in September 1983 maintains 27 gauging sites spread over Mahanadi basin (20 upstream +7 downstream), Eastern River Division, Bhubaneswar & Brahmani Subrnrekha Division under CWC maintain 5 gauging stations in Mahanadi basin. Recently C.W.C has installed telemetry system for observation and transmission of major hydrological parameters of above stations on real time basis.

The details of gauging net works maintained by Hirakud Dam Project and CWC are enclosed in tabular form (Table No 6.10 & No 6.11)

Daily rainfall ending 08.30 hrs of 13 H.D.P. stations is received at Burla Control room at about 9.30 hrs through wireless network. The data of 27 stations are collected from CWC by 10.30 hrs.

Average rainfall of entire upstream catchment upto Hirakud (32,200 sq.miles) and downstream catchment upto Mundali (18800 sqm.) is calculated by Theiessen polygon method. Theiessen polygon coefficients with area of influence of individual rainfall stations (27 in upstream & 12 in down stream) have been established which are now used. Average rainfall can also be found out by Arithmatic average and by Isohyetal method.

Table : 6.10 : Gauging Network maintained by Hirakud Dam Project

Upstream

Sl.No	Name of Station	River/Basin	Location Longitude /Latitude (in degree, minute, and sec.)	Length of River upto gauging site(km)	Length of river from site to Dam (Km)	Catchment upto site (sq. mile Sq.Km.	Date of function -ing	Zero value of gauge (ft./m)
1	2	3	4	5	6	7	8	9
01.	Ghorari	Mahanadi	82° – 1'–43" / 21° – 8'–25"	193	256	5100 / 13,209	1963	853.0 (260.0)
02.	Nandaghat	Seonath	81° – 47'–46" / 21° – 45'–8"	194	262	9300 / 24,087	1975	777.6 (237.0)
03.	Seorinarayan	Mahanadi	82° – 35'–30" / 21° – 43'–0"	280	159	18000 / 46,620	1961	701.0 (213.66)
04.	Champa	Hasdeo	82° – 39'–36" / 21° – 1'–36"	280	174	4530 / 11,733	1963	845.04 (257.57)
05.	Sarandhi	Mahanadi	83° – 9'–19" / 21° – 43'–16"	334	96	23,564 / 61,031	1957	641.44 (195.5)
06.	Tarapur	Mond	83° – 15'–44" / 21° – 51'–30"	182	208	1530	1975	663.42 (202.2)
07.	Deogaon	Ib	83° – 55'–20" / 21° – 47'–20"	208	69	5830 / 15,100	1957	663.23 (193.0)
08.	Kheruwal	Bheden	84° – 00'–42" / 21° – 46'–50"	-	-	1200 / 3108	1995	621.06 (189.30)
Downstream								
09.	Belgaon	Tel	83° – 19'–37" / 20° – 18'–22"	162	113	4500 / 11,655	1963	506.0 (154.23)
10.	Khairmal	Mahanadi	83° – 57'–00" / 20° – 49'–00"	-	95	45,200 / 117,068	1957	322.55 (98.3)
11.	Barmul	Mahanadi	84° – 45'–43" / 20° – 34'–17"	-	214	48,400 / 125,356	1947	163.05 (49.7)
12.	Mundali	Mahanadi	85° – 43'–6" / 20° – 30'–32"	-	304	51,000 / 132,100	1968	65.00 (19.81)
13.	Patharla	Tel	83° – 48'–20" / 20° – 48'–30"	250	-	9,170 / 23,750	1994	0.00 (Arbitrary)

For Table. 6.11, (i) All ordinary raingauges. (ii) In high flood period hourly gauges observed and transmitted. (iii) Zero R.L. of gauge are connected to G.T.S. Bench marks. (iv) Stations operative only in monsoon period. (v) Only gauges observed, used to calculate discharge. (vi) Frequency of (a) Rainfall at 8.30 & 17.30 hrs (b) Gauge at every 3 hour interval.

Table 6.11 Rain Gauge Network (CWC)

Sl.No	Name of sites	Basin/ Subbasin	Location Long. / Lati. in Deg. Min. Sec	Ordinary/ S.R. Rain Guage	Frequency of Observation	Date of function ing	Mode of communi cation
01.	Baranda	Parri	81 – 52 – 56 / 20 – 55 – 06	ORG	Once daily	3.6.80	By post
02.	Rajim	Mahanadi	81 – 52 – 30 / 20 – 58 – 00	ORG/SRRG	3-hourly	1.7.79/1.6.80	W/L
03.	Kotni	Seonath	84 – 14 – 19 / 21 – 13 – 02	ORG	Once daily	25.6.80	By post
04.	Simga	Seonath	81 – 41 – 36 / 21 – 37 – 33	ORG/SRRG	3-hourly	5.6.80/13.7.87	W/L
05.	Andhiyarkore	Hamp	81 – 36 – 30 / 21 – 47 – 00	ORG/SRRG	3-hourly	1.7.80/26.6.87	W/L
06.	Ghatora	Arpa	82 – 13 – 34 / 22 – 02 – 04	ORG	3-hourly	16.6.80	W/L
07.	Jondhra	Seonath	82 – 29 – 34 / 21 – 43 – 00	ORG	3-hourly	1.6.79	By post
08.	Rampur	Jonk	82 – 31 – 30 / 20 – 39 – 57	ORG	Once daily	1.7.80	By post
09.	Bamidih	Hasdeo	82 – 43 – 12 / 21 – 53 – 39	ORG	3-hourly	1.6.80	W/L
10.	Basantapur	Mahanadi	82 – 47 – 27 / 21 – 43 – 18	ORG/SRRG	3-hourly	24.7.77/2.6.80	W/L
11.	Kurubhata	Mond	83 – 12 – 15 / 21 – 59 – 15	ORG/SRRG	3-hourly	21.6.80/23.9.87	W/L
12.	Sundergarh	Ib	84 – 03 – 00 / 22 – 15 – 00	ORG/SRRG	3-hourly	11.8.80/5.7.88	W/L
13.	Salebhata	Ong	83 – 32 – 30 / 20 – 59 – 00	ORG	3-hourly	1.1.81	W/L
14.	Kesinga	Tel	83 – 13 – 30 / 20 – 11 – 15	ORG/SRRG	3-hourly	25.6.80/30.3.88	W/L
15.	Kantamal	Tel	83 – 43 – 45 / 20 – 38 – 40	ORG/SRRG	3-hourly	22.6.80/8.8.87	W/L
16.	Scorinarayan	Mahanadi	82 – 35 – 30 / 21 – 43 – 00	ORG	3-hourly	17.3.85	W/L
17.	Phulbani	Salki	84 – 12 – 00 / 20 – 30 – 00	ORG	3-hourly	14.9.84	W/L
18.	Mahendragarh	Hasdeo	82 – 12 – 54 / 23 – 06 – 06	ORG	3-hourly	21.6.86	W/L
19.	Bango Dam	Hasdeo	82 – 35 – 00 / 22 – 34 – 00	ORG	3-hourly	1.6.85	W/L
20.	Korba	Hasdeo	82 – 41 – 30 / 22 – 23 – 00	ORG	Once daily		W/L
21.	Khairmal	Mahanadi	83 – 57 – 00 / 20 – 49 – 00	ORG	3-hourly	20.06.85	W/L
22.	Sankara	Jonk	82 – 51 – 00 / 21 – 16 – 00	ORG	3-hourly	26.4.87	W/L
23.	Dharmjaigarh	Mond	83 – 12 – 52 / 22 – 28 – 20	ORG	3-hourly	8.6.87	W/L
24.	Thettatang	Ib	83 – 54 – 43 / 22 – 40 – 32	ORG	3-hourly	28.5.87	W/L
25.	Raipur	Kharun	81 – 38 – 15 / 21 – 14 – 15	ORG	3-hourly	18.7.84	W/L
26.	Baikunthapur	Hasdeo	82 – 34 – 00 / 23 – 14 – 30	ORG	3-hourly	30.6.85	W/L
27.	Pandra Road	Arpa	81 – 54 – 30 / 22 – 45 – 30	-	3-hourly	I.M.D.	W/L
28.	Burla	Mahanadi	83 – 52 – 30 / 21 – 32 – 45	ORG/SRRG	3-hourly	10.9.84/4.6.87	W/L
29.	Padampur	Bhoden	84 – 13 – 12 / 21 – 45 – 45	ORG	3-hourly	15.6.89	W/L
30.	Surajgarh	Mahanadi	83 – 22 – 17 / 21 – 41 – 35	ORG/SRRG	3-hourly	20.6.89/22.6.89	W/L
31.	Tikarpara	Mahanadi	87 – 30 – 08 / 20 – 38 – 00	ORG	Twice daily	21.5.72	W/L
32.	Naraj	Mahanadi	85 – 41 – 00 / 20 – 25 – 00	ORG/SRRG	Twice FMO	3/75	-
33.	Alipingal	Mahanadi	86 – 10 – 00 / 20 – 10 – 34	ORG	-do-	9.5.88	W/L
34.	Nimapara	Kushabhadra	86 – 00 – 00 / 20 – 06 – 00	ORG	-do-	8.11.89	W/L

Note: Except Alipingal, Nimapara, Baronda, Kotni, Jondhra & Rampur real time data are received from other stations.

6.1.1.6 Forecast of Inflow to Reservoir

6.1.1.6.1 By Unit Hydrograph Method

The upstream catchment is divided into four sub basins.

i) Saradihi on river Mahanadi (23564 sqmile)

ii) Tarpur on Mond (1830 sq mile)

iii) Deogan on river Ib (3181 sq mile)

iv) Ungauged catchment (3625 sq mile)

One day duration unit hydrographs in fps units have been developed for each sub-basin. Similarly a 24 hr unit hydrograph has been developed for Khairmal (13000 sq mile) in downstream vide Table 6.13 & 6.14.

Contribution of the sub-basin is computed by multiplying unit hydrograph ordinates with average effective rainfall over it. Effective rainfall is calculated by multiplying the average rainfall with recommended run-off percentage, given below in Table 6.12.

The inflow hydrograph at the dam site is then derived by adding the hydrographs of all four upstream sub-basins with corresponding time lag given earlier and adding the base flow. After computing the total discharge by hydrograph 5 to 30% of (observing the meteorolgical condition around the area) of the above discharge is added to get the anticipated inflow during next 24 hrs.

Similar calculation is made for Khairmal.

Table 6.12 Recommended Run-off %

Weighted average Rainfall (mm)	June	July	August & September
20-50	-	10	20
50-75	10	25	30
75-100	20	30	40
100-150	25	40	50
150-200	40	50	60
200-300	50	60	70
above 300	-	70-80	80-90

Table 6.13: Unit Hydrograph ordinates in lakh cusec for different upstream sub-basins

			Duration 24 hr	Depth - 1 inch
Time in hr	Saradihi C.A.-23564 sq.mile	Tarapur C.A.-1830 sq.mile	Deogaon C.A.-3181 sq.mile	Ungauged C.A.-3625 sq.mile
0	0.00	0.00	0.00	0.00
6	0.68	0.45	0.265	0.265
12	1.14	6.65	0.46	0.46
18	1.82	0.41	0.60	0.60
24	2.74	0.18	0.605	0.605

Contd...

30	2.96	0.09	0.50	0.50
36	3.19	0.05	0.36	0.36
42	3.19	0.01	0.24	0.24
48	2.69	0.00	0.15	0.15
54	2.80	-	0.09	0.09
60	1.80	-	0.06	0.06
66	1.14	-	0.04	0.04
72	0.73	-	0.02	0.02
78	0.46	-	0.01	0.01
84	0.34	-	0.00	0.00
90	0.23	-	-	-
96	0.11	-	-	-
102	0.00	-	-	-

Table No.6.14 Details of Unit Hydrograph at Khairmal

Catchment area : 13000 sq. mile, Unit time : 24 hrs, Depth 1 Inch.

Time in Hours (cumulative)	Ordinate of U.H. in lakh cusec	Time in Hours (cumulative)	Ordinate of U.H.in lakh cusecs
00	0.00	114	0.14
06	0.08	120	0.14
12	0.18	126	0.10
18	0.30	132	0.10
24	0.46	138	0.08
30	0.64	144	0.08
36	1.02	150	0.08
42	1.70	156	0.08
48	1.82	162	0.08
54	1.92	168	0.08
60	1.62	174	0.07
66	1.16	180	0.06
72	0.76	186	0.05
78	0.50	192	0.04
84	0.38	198	0.03
90	0.30	204	0.02
96	0.24	210	0.01
102	0.20	216	0.01
108	0.16	222	0.00

6.1.1.6.2 By C.W.C (Mahanadi Division, Burla)

Mahanadi Division, Burla forecasts avarage inflow to Hirakud reservoir in next 24 hrs.(9Am to 9 Am) by mike-11 method and Advisory forecast of average inflow for subsequent 24 hrs. The 24 hrs forecast is revised if required at 21.30 hrs thus giving only about 12 hrs average forecast.

The proceedure is based on :

i) Routing from the base qauging stations Basantpur on Mahanadi, Sundargarh on Ib, and Kurubhatta on Mond with corresponding travel time and computation of average flow for next 24 hrs for each base stations.

ii) Contribution from free catchment assessed from weighted average rainfall over it and unit hydrograph developed for the purpose.

iii) Contribution from immediate catchment (1205 sq.km) around reservoir by direct run-off method using representative rainfall stations at Burla and Hirakud.

iv) Evaluation of total average inflow to reservoir from summation of above.

6.1.1.6.3 Forecast of flood discharge at Mundali.

Following steps are followed.

- Compution of flood discharge by unit hydrograph method at Khairmal.
- Computation of contribution discharge between Khairmal & Barmul by deducting the corresponding discharge at Khairmal with time lag from discharge at Mundali.
- During very high flood contribution hydrographs of above 3-zones downstream of Hirakud are plotted separately and the probable discharge of each zone up to 12 hrs in advance are predicted.
- The dam release and contributions from 3 lower zones are superimposed with time lag to give discharge hydrograph at Mundali.

6.1.1.6.4 Wallingford model -computation of discharge at Saradihi.

The mathematical model for flood forecasting developed by Institute of Hydrology, Walling ford, Oxan, in 1985 in collaboration with Hirakud Dam Project, is also adopted during the flood period to forecast discharge at Seorinarayan and Saradih stations on Mahanadi upstream of dam. This model gives a short period forecast of 3 hourly discharge upto 9 hours ahead of Saradihi site and 12 hours ahead for Seorinarayan gauge site. The model structure is given below.

(a) Saradihi gauge site prediction for next 3 units (9) hours

Day	Time in units (3 hourly)	Discharge at		
		Seorinarayan (SN)	Champa (C)	Saradihi (S)
1st	00 hrs(t_0)	qt_0(SN)	qt_0(C)	qt_0(S)
	03 hr(t_1)	qt_1(SN)	qt_1(C)	qt_1(S)
	06 hr(t_2)	qt_2(SN)	qt_2(C)	qt_2(S)
	09 hr(t_3)	qt_3 (SN)	qt_3 (C)	qt_3(S)

Predicted Discharge at Saradihi

On 1st day at 12 hrs (t_4)

qt_4 (S) =1.402xqt_3 (S)- 0.481 qt_2 (S) +0.794xqt_1(SN+C)-0.702qt_0 (SN+C)

on 1st day 15 hrs(t_5)

qt_5 (S) = 1.402 x qt_4 (S) - 0.481 qt_3(S) +0.794xqt_2 (SN+C)-0.702qt_1 (SN+C)

on 1st day 18 hrs(t_6)

qt_6 (S) = 1.402 x qt_5(S) - 0.481 qt_4(S) +0.794xqt_3 (SN+C)-0.702qt_2(SN+C)

B) Seorinarayan discharge can be predicted for the next 4 units(12 hours) as per the model given below.

Day	Time in units (3 hourly)	Discharge at		
		Ghorari (G)	Nandaghat (N)	Seorinarayan (SN)
Previous of 1st day	21hrs (t_{-1})	(qt_{-1}) (G)	qt_{-1}(N)	(qt_{-1}) (SN)
	00 hr(to)	qt_0 (G)	qt_0(N)	qt_0(N)
	03hr(t_1)	qt_1 (G)	qt_1(N)	qt_1(SN)
	06 hr (t_2)	qt_2 (G)	qt_2(N)	qt_2(SN)
	09 hr(t_3)	qt_3 (G)	qt_3 (N)	qt_3(SN)

Predicted Discharge at Seorinarayan

On 1st day at 12 hrs (t_4)

qt_4 (SN) =1.579xqt_3 (SN)- 0.62qt_2 (SN) +0.1982xqt_0(G+N)-0.1617qt-1 (G+N)

On 1st day 15 hrs(t_5)

qt_5 (SN) =1.579xqt_4 (SN)- 0.62qt_3 (SN) +0.1982xqt_1(G+N)-0.1617qt_0 (G+N)

Similarly qt_6(SN) and qt_7 (SN) can be calculated and prediction for the discharge of Saradihi can be calculated for further 4 units taking above predicted value of Seorinarayan and assuming the expected discharge of Champa.

6.1.2 History of Reservoir Operation including Flood Studies - Development of Rule Curves:

As discussed earlier, different Committees were formed to review the rules for operation of the Hirakud Reservoir keeping in view the serviciability and safety of the structure. They are given in a tabular form vide table 6.15.

Table 6.15 Development of Rule Curve

Year	Committee	Reserrvoir RL in ft. on										
		1st July	1st Aug.	11th Aug.	16th Aug.	21st Aug.	26th Aug.	1st Sept.	11th Sept.	16th Sept.	21st Sept.	1st Oct
1953	Revised Project Report (Vol.VI)	590	590	-	-	-	590	590	-	620	-	630
1959	Manual of Reservoir operation i) Normal years.	590	590	-	-	-	590	590	-	-	-	630
	ii) Lean Years	590	590	-	-	-	-	610	-	-	-	630
1962	Evaluation Committee.	592	590	-	-	-	590	600	615	-	625	630

Contd...

1966	National Planning Council (study Group)	600	600	-	-	-	600	605	-	-	-	630
1970	Hirakud co-ordination Committee. i) Good Av. Yrs. (>14Macft)	600	600	-	600	600	600	610	615	-	625	630
	ii) Bad (9 to 14Macft)	600	600	-	600	600	605	612	618	-	626	630
	iii) Very bad <9Macft)	600	600	-	600	605	610	615	622	-	628	630
1976	Expert Committee	-	-	600	-	605	-	617	623	-	627	630
1988	System Engg. (CWC)	595	590	605	-	610	-	619	624	-	629.5	630
1988	3rd meeting of rule curve revision Committee	595 to 595	590 to 590	605 to 615	-	610 to 622	-	619 to 627	624 to 629	-	629.5 to 630	630 to 630

At the time of preparation of first project report in 1947 there was no flow data available at Dam site, except for daily gauge observations taken during the monsoon period at Sambalpur from 1921 onwards. In 1953 when the revised project Report was prepared the data collected from actual discharge measurements started from 1946 were made use of in developing a gauge discharge curve for Sambalpur gauge and convertng the earlier gauge observation at Sambalpur using this G-Q curve.

The Reservoir filling studies made in 1953 with above inflow data revealed that net inflow in months of September and October will be sufficient to fill the Reservoir from RL 590 to 630 ft in all years except 1930, 1940 and 1941 when the final elevation at the end will be 628.45, 627.84 and 628.86 ft. Considering small number of failures and deficits, it was recommended that filling may commence from 1st September to reach RL.620 ft by 15th Sept. and thereafter it may be increased to 630 ft by Ist October. In poor rainfall years filling was to commence late in August. Flood studies were ignored from analysis.

The manual of Reservoir operations 1959, based on above studies and other data available then recommended the following rules:

(i) There should be no full impounding till 1st of September in normal years. The entire inflow should be let out. Care being exercised that the combined discharge of reservoir outflow and the run off from the catchment below does not exceed 10lakh cusec at Naraj. Generally 5 lakh cusec can be released without hesitation.

(ii) Beyond RL.625 ft no restriction can be imposed.

In 1962 Evaluation Committee based their study on 10 daily run-off data from 1921 to 1961 and following schedule was recommended. The reservoir level should be maintained at RL595ft during June. Thereafter it should be brought down to RL.592 ft and maintained at that level for whole of July. In August it should be maintained at 590 and then it should be maintained as given in above table. With above filling schedule, it was found that the Reservoir will be filled to RL.630 ft by end of October except 1940, 1941, 1951 and 1953.

The Evaluation Committee based their flood control studies on the routing of design floods of 15.0 and 18.3 lakh cusec peak dischrage with and without flood moderation and recommended following:

"The flood peak at Naraj is to be controlled to 9.0 lakh cusecs till the reservoir level reaches RL.615. From RL.615-620 ft. the peak at Naraj should be controlled to 10 lakh cusec (Naraj gauge 88'). Above RL 620ft. regulation is to be done from consideration of safety of Dam and there will be no limit to the outflow from reservoir."

The committee considered 9 lakh cusec corresponding to a gauge of 87' to be safest flood for Delta and 11.2 lakh cusec (89' gauge) to be the danger limit. It also suggested the lowering of the reservoir in September on receipt of heavy rains in catchment.

The main assumptions of their study were

(i) a constant discharge of 2 lakh cusec from the uncontrolled catchment for design flood conditions based on an envelope curve.

(ii) Initial reservoir levels of 590' & 615 ft.

(iii) for flood moderation out-flow was restricted to 7 lakh cusec up to RL 615 ft and to 8 lakh cusec upto RL 620' & full discharge above 620 ft.

But actual flood studies showed that though the dam will be safe for 15 lakh cusec the discharge at Naraj would go upto 13.96 lakh cusec should this flood occur in July & August (i.e. at RL590') and to 15.25 lakh cusec if flood were to occur around 10th sept (i.e. reservoir level at 615'). Thus no flood control was anticipated for the design flood of 15 lakh cusec.

The study Groups of N.P.C. in 1966 did not make any flood control studies but by studying data from 1947-1965 recommended filling up the reservoir to RL.600 on the out-break of monsoon to eliminate the maintenance of relatively low operating heads on the turbines of upper power house which resulted in considerable reduction in efficiency of two generating units for which actual design head is 104ft. They also recommended to set up a weather forecasting office with the help of IMD, on the lines of the Damodar valley corporation. The study groups of National Planning Council (N.P.C.) did not make any flood control studies but suggested early depletion of the reservoir to as low as RL 590 ft. on the basis of warning of approaching flood.

In 1970, Hirakud Co-ordination Committee analysed further Hydrological Data and recommended a variable rule curve as given in above table. This rule curve, it is stated would ensure filling in all years except in 1940, 1941, 1966 and 1968. The levels attained at the end of September in 1968, 1941 and 1966 would be respectively 622.60,629.70 and 627.50ft.

It also studied in some detail the effect of the filling schedules on flood control aspect of the project. The floods were assumed to occur on the rule curve level for the respetive date and year in which they occured. Discharge from the uncontrolled catchment were assumed to be known accurately and 36 hours in advance and regulation directed to limit the total discharge to 9.0 lakh cusec at Naraj. Contribution from downstream catchment was taken as the difference between the discharge at Naraj and Hirakud set apart by travel time of 36 hrs. The entire computations were made on daily basis. It showed that the maximum discharge at Naraj can be moderated to 9 lakh cusec in all the years except 1933 which was exceptional in nature. The maximum regulated flood discharge for this flood was worked out to 12.4 lakh cusec with the recommended rule curve level of 600 ft. The Committee omitted several major floods from its study as that of 1946, 1947, 1959, 1961 & 1964.

It may be noted that the duration of flooding in many floods far exceeded the desired limit of 3 to 4 days. For floods of 1925, 1929, 1937 and 1961, the duration of flooding was 11 days and for floods of 1933, 1940, 1953 was 12 days and for floods of 1944 as long as 19 days.

In 1976 a full-fledged analysis was made by an Expert Committee not only for operational aspects but also from the aspects of flood control, conservation, safety of the dam and delta as well.

They studied about design floods. Originally in 1947, the magnitude of probable maximum flood was estimated to be 15.7 lakh cusec at Naraj based on long term flood gauge observation at the site. From this the design discharge was fixed to be 11.51 lakh cusec at the dam site. Later International Engineering Company estimated the spill way design flood from study of the flows in the adjoining basin of Godavari and the flood data at Naraj and their experience of other major rivers of the World to be 18.3 lakh cusec with a volume of 29Macft. Further frequency studies were made in 1952 which showed that the 500 year flood would have a peak discharge of 15 lakh cusec which was adopted as final design flood.

The expert Committee took up detailed investigaion of past hydrometeorlogical data and by transposition of severe storm (flood of June 1925) they found out the probable maximum strom to produce a maximum priobable flood with a peak of 28.74 lakh cusec and volume of 17.11 Macft. They also made some frequency analysis and found the maximum flood to be 26.85 lakh cusec and 23.57 lakh cusec respectively by maximum likelihood method and by Gumbel's method for a return period of 10000 years. They finally recommended the maximum design flood to be 28.74 lakh cusec and suggested to increase the spillway capacity by 5.5 lakh cusec at RL.630 ft in addition to raising and strengthening of the dam and spillways. This has not yet been done. In this context, the views of DSARP may be referred to (Oct. 1991). They suggested to regulate the flood by advance release.

Routing of this flood hydrograph with inital reservoir level of RL 610′ gave the MWL of 634.75 ft with maximum out flow of 68627 cumecs (24.25 lakh cusec). For higher levels of 615′ & 620′ the corresponding MWLs were computed be 635.2 & 635.8′ respectively.

The committee concluded that with the existing spillway facility, even with initial level of RL 600′, the reservoir level would rise as high as 641.20′. The report of the team of Experts was accepted by Govt. of Orissa (I & P Dept.) in their Lr no HKD (W) -19/76-25504 dt 19.7.1996.

Subsequently the severe storm of August 1982 in Mahanadi basin downstream of Hirakud Dam was transposed to Hirakud catchment and computation showed that this was likely to produce a flood peak of 26.94 lakh cusec with a flood volume of 11.60 Macft. But in this study the D.A.D was not considered, storm ordinates were not arranged in critical sequence and maximisation for moisture content was ignored.

The Team of Expert also recommended strengthening of the dam to withstand expected MWL of 634.75 ft, to strenghten the embankment in delta for a discharge of 12 lakh cusec (With 2 lakh capacity for enchroachment) and considerable strengthening of flood forcasting set up.

When Hirakud Dam was taken up under DSARP it was decided to recalculate the PMF and CWC was entrusted with the study. The P.M.F was calculated by them and after interaction with the state engineers it was finally accepted as 69632 cumec (24.60 lakh cusec) & volume of 16800 M.cum (13.59 Macft.)

The views of DSARP (10th october.1991) on this reads as follows :

"There are likely to be serious problem in providing additional spillway capacity as suggested by Team of Expert in 1976.

Apart from high cost of the propasal the spill channel which will not be in operation frequently is likely to be enchroached upon in due course and may not be availale when required (as has happened in Ukai Dam, Gujrat). It was therefore suggested to regulate the flood by advance release from the reservoir by use of flood forecasting and flood warning system."

In 1987, the Basin Planning & Management organisation of C.W.C (formerly known as System Engineering Unit)suggested a rule curve as given in the Table No.6.15 after investing past flood data. This is an upward revision compared to all other previous rule curves. Govt of Orissa in Irrigation & power Dept. vide their resolution no 24567 dt 6.6.85 contstituted a committee under the chairmanship

of commissioner-cum-secy. I & P Dept. Dr. Y.K. Murthy, retired chairman C.W.C & other members for revision of rule curve in connection with operation of Hirakud reservoir. The committee is known as "Rule Curve Revision Committee."

In1988 the committee tried to develop a variable rule curve. They dared to do so as the hydro-meteorological forecasting methods have by then developed due to installation of sufficient rain gauge stations about 40 in upstream and 31 in downstream catchment, due to setting up of wireless networks for transmitting rainfall and run off messages to Control Room at Burla and due to transmission of cloud pictures and temperatures by Television Networks through INSAT. Moreover the IMD also is giving accurate weather forecast once or twice daily.

The Committee fixed the rule curve developed by system Engineering Unit as lower limit. To find the upper limit, backward calculation was made to find out the different reservoir levels that should have been kept on different dates (at 10 days' intervals) so that the reservoir could be filled up by 1st October. From this study an envelope curve was drawn to find out the levels of reservoir to be kept on different dates to fill up the reservoir in about 90% of the years.

The above flexibility of operation evolved by statistical analysis was recommended by the committee to be used to keep reservoir level between the two limits depending upon hydro-meteorological conditions of the year. No flood control study was made. (vide Table in Annexure 6.1).

6.1.3 Operation of Hirakud Reservoir as per recommendation of 'Rule curve Revision Committee 1988.'

Prediction of cumulative monsoon inflow graphs correlaing percentage of duration of monsoon (y) with cumulative inflow in Macft (x) have been plotted since 1961. This figure gives a good idea of the patterns of inflows in different years. By comparing the upto date inflow of a particular year with past inflow pattern a fair assumption can be made beyond middle of monsoon period about balance quanitity of inflow available that would be expected during the rest of the monsoon duration of that year.

Prediction of duration of monsoon : A correlation between the starting date of monsoon (taken as the date of 1st rise of reservoir level) and the duration of monsoon flow has been developed analysing past flow data from 1961 with Reasonable coefficient of correlation of 0.8947.A straight line fit is obtained as

$$Y=138.5 - 1.121 X$$

where Y = duration of monsoon in days

X = date of 1st rise of reservoir level

From this it is possibe to predict approximately the duration of monsoon.

Prediction of total monsoon run-off from actual run-off flow during a fraction period of monsoon:-

Such graphs & equations have been developed for 20 to 90 % at 10 % interval. The equations are

a) for 20 % $Y = 3.11147 X + 14.3709$

b) for 30 % $Y = 1.97376 X + 12.8302$

c) for 40 % $Y = 1.76817 X + 8.42996$

d) for 50 % $Y = 1.51873 X + 4.76862$

e) for 60 % $Y = 1.40473 X + 0.977546$

f) for 70 % $Y = 1.14069 X + 1.72978$

g) for 80% $Y = 1.04317 X + 1.12407$

h) for 90% $Y = 1.0234 X + 0.25993$

Where X = observed run-off in Macft for corresponding percentage period of monsoon.

Y = Total monsoon run-off in Macft.

Since adoption of Rule Curve 1988 Hirakud reservoir has been filled up in all the years except the years 2000, 2002, 2004, 2006, 2008 & 2009 (Vide Table 6.15, a). These years are low rainfall years. The reservoir filling of the project from 1958-2009 is therefore considered as satisfactory.

6.1.4 Manual of Reservoir Operation

The Hirakud Dam Project was inaugurated on 13.01.1957 by Pandit Jawaharlal Nehru, the then Prime Minister of India.

In 1959 Chief Engineer of the Project Sri M.S Thirumale Iyengar brought out Manual of Reservoir operations for Hirakud containing some useful table for reservoir management based on experience of three monsoons. Behaviour of the dam and dykes as observed during the 1st 3 years of operation were also enclosed as appendix. The same manual is being followed even now.

Since then there has been a number of ammendments to the operational rules based on revised hydrology & operational demands. A committee of technical personnel was constituted by Govt. of Orissa (DOWR) vide Lr. No.45453 dt. 22.11.2009.

The committee has recommended for revised reservoir operation manual based on the experience gained during the last five decades.

Some of the important events given in these "Manuals of Reservoir Operation" are

i) Must -Which must be followed scrupulously

ii) DON'T - The points noted under this heading should be strictly followed.

iii) Assignment and responsibility of officers involved with reservoir operation.

iv) Revision of rule curve from time to time

v) Flood forecasting

vi) Operation during non-monsoon & monsoon periods.

vii) Utilisation of storage space.

viii) Fore warning during monsoon Period.

ix) Forewarning during non-monsoon period.

For details Manual of Reservoir operation may be referred to.

The officials who are involved with the operation of Hirakud reservoir should be well conversant with the contents of the manual as well as with the technical and administrative directives issued and published from time to time.

The following main principles for operation of Hirakud reservoir may also be kept in view.

- The regulated flood release from the reservoir coupled with run off from uncontrolled catchment below dam is aimed to be within safe permissible limit at Mundali.
- During flood moderation in monsoon period the reservoir should not be allowed to rise beyond RL.625'. Above RL 625',the safety of the dam is of prime importance.
- No release shall ordinarily be made through spillways during non-monsoon period.

6.1.5. Rule Curve vis-a-vis Flood Storage Fapacity :

Hirakud Reservoir is envisaged primarily to cater for conservational uses of irrigation and power generation and as flood control reservoir for moderationg floods in the deltaic region. There are set rule curves for the operation which have been revised on various occassions as stated earlier from

Table 6.15 (a) : Yearwise Maximum and Minimum Levels of Reservoir (1958-2009) 1st October-November

(All levels in ft. at 10 hr.)

Year	Minimum		R.L. On 1ST OCT	R.L On 1ST OCT	MAXIMUM		Remark
	R.L.	DATE			R.L.	DATE	
1958	600.00	30.06.1958	626.40	629.90	629.90	25.11.1958	(Failure)
1959	601.10	15.07.1959	628.00	630.00	630.00	25.10.1959	
1960	605.60	18.06.1960	629.40	630.10	630.10	28.10.1960	
1961	605.20	09.06.1961	629.50	630.60	630.60	20.11.1961	
1962	597.39	11.07.1962	630.08	630.39	630.39	08.10.1962	
1963	597.35	14.06.1963	628.69	630.50	630.50	25.10.1963	
1964	590.87	21.07.1964	629.89	630.00	630.00	12.10.1964	
1965	595.69	19.07.1965	630.47	630.47	630.47	03.10.1965	
1966	590.83	19.06.1966	614.58	615.64	615.64	17.09.1966	(Failure)
1967	590.04	20.06.1967	629.87	629.95	629.95	10.10.1967	(Failure)
1968	590.69	27.06.1968	623.91	626.23	626.23	17.10.1968	(Failure)
1969	590.19	01.07.1969	629.95	630.00	630.00	03.10.1969	
1970	592.54	09.06.1970	629.95	630.35	630.35	11.10.1970	
1971	592.31	05.06.1971	623.45	630.50	630.50	31.10.1971	
1972	584.23	30.06.1972	630.29	630.47	630.47	06.10.1972	
1973	584.00	28.06.1973	630.17	630.52	630.52	12.11.1973	
1974	587.75	02.07.1974	612.04	613.89	613.89	05.11.1974	(F)(Lowest)
1975	585.10	22.06.1975	630.00	630.15	630.15	08.11.1975	
1976	588.50	30.06.1979	629.41	629.52	629.52	29.09.1976	(Failure)
1977	590.50	20.06.1977	621.33	630.10	630.10	29.09.1977	
1978	587.71	24.06.1978	629.97	630.00	630.00	02.10.1978	
1979	587.17	29.06.1979	616.19	618.58	618.58	12.10.1979	(Failure)
1980	588.52	15.06.1980	629.95	630.00	630.00	27.09.1980	
1981	586.87	01.07.1981	629.98	630.11	630.11	05.10.1981	
1982	58R04	08.07.1982	624.97	628.52	628.52	31.08.1982	(Failure)
1983	587.79	30.06.1983	629.95	630.10	630.10	27.09.1983	
1984	591.35	14.06.1984	627.10	627.79	627.79	20.09.1984	(Failure)
1985	586.41	30.06.1985	630.23	630.31	630.31	28.09.1985	
1986	588.79	20.06.1986	624.19	626.67	626.67	16.10.1986	(Failure)
1987	585.50	04.07.1987	630.02	630.25	630.25	24.11.1987	
1988	588.71	23.06.1988	630.12	630.17	630.17	05.10.1988	
1989	591.56	27.07.1989	630.02	630.17	630.17	03.10.1989	
1990	594.87	15/06/1990	629.04	630.12	630.12	28.09.1990	
1991	589.73	03/06/1991	630.02	630.23	630.23	16.09.1991	
1992	591.35	23/06/1992	629.31	630.00	630.00	22.09.1992	
1993	591.58	16/06/1993	630.08	630.00	630.15	30.10.1993	
1994	593.02	14/06/1994	630.19	629.97	630.19	25.09.1994	
1995	590.81	23/06/1995	630.00	630.04	630.29	21.09.1995	
1996	591.00	20/06/1996	630.10	628.27	630.21	17.09.1996	
1997	589.45	26/06/1997	630.19	629.19	630.19	23.09.1997	
1998	601.21	25/06/1998	630.00	629.02	630.89	30.01.1998	(Highest)
1999	590.77	18/06/1999	630.06	630.39	630.43	21.10.1999	
2000	588.79	01/07/2000	617.00	614.62	617.02	01.10.2000	(Failure)
2001	591.91	15/06/2001	629.06	628.71	630.10	06.10.2001	
2002	591.67	25/06/2002	628.10	625.37	629.39	17.09.2002	(Failure)
2003	592.47	20/06/2003	629.27	629.95	630.15	20.10.2003	
2004	592.54	12106/2004	624.69	623.77	625.85	11.10.2004	(Failure)
2005	591.91	28/06/2005	630.08	629.81	630.10	24.10.2005	
2006	591.04	04/07/2006	629.75	627.65	630.10	25/09/2006	(Failure)
2007	595.10	29/06/2007	630.08	627.35	630.25	22/09/2007	
2008	594.85	29/06/2008	629.27	626.77	630.00	19/09/2008	(Failure)
2009	594.89	01/07/2009	623:02	626.00	627.33	15/10/2009	(Failure)
Max(1958-2009)	605.60	18.06.1'960	630.47	630.60	630.89	30.01.1998	
Min(1958-2009)	584.00	28.06.1973	612.04	613.89	613.89	05.11.1974	

consideration of conservation and have reduced flood storage capacity systematically the details of which are furnished in Table 6.16.

Table 6.16 : Reduction of Flood Storage capacity of reservoir

Sl.No	Name of the Report/ Committee	Flood-storage capacity in %
A	Revised Project Report (1955)	100%
B	Manual of Reservoir Oeration (1959)	95%
C	Evaluation Committee (1962)	89.3%
D	National Planning Council (1967)	74.8%
E	Hirakud Co-Ordination Committee (1970)	74.5%
F	Expert Committee (1976)	68.86%
G	Rule curve Revision Committee (1988)	67.4% to 55.4%

(Source : Data on Flood Control Operation of Hirakud Dam, Sept. 1997 Pg.17)

The Proceeding of the 3rd meeting of the Rule Curve Revision Committee (1988) held on 05.01.1988 at Burla is appended vide Annexure 6.1.

Annexure 6.1

Proceeding of the 3rd Meeting of Rule Curve Revision Committee held on 05.01.1988

Chairman, while welcoming the members, requested the Member-Secretary to give a resume of the points to be discussed. The Member-Secretary briefed the Committee on the various aspects of the decision to be taken by the Committee.

1) The Team of Experts (1976) had given fixed Rule curve for operation of Hirakud reservoir during the monsoon. Studies have shown that if these guidelines are strictly followed, the reservoir would not have filled-up to RL. 630 by the 1st October in about 50% of the years.

The Member-Secretary explained in detail how a correlation between the starting date of monsoon (taking this date as first rise of reservoir level) and the duration of monsoon flow has been developed with a reasonable correlation coefficicnt of 0.8947. From this study it is possible to know approximately the duration of monsoon inflow immediately after the first date of rise of reservoir level is known. A graph correlating percentage duration of monsoon vrs. Monsoon inflow in m.ac.ft. were plotted for the last 26 years 1961-86. This graph gives a good idea of the pattern of inflows in different years. By comparing the inflows of a particular year with past inflow patterns a fair assumption can be made from middle of monsoon about balance quantity of inflow available that could be expected during the rest of the "monsoon duration" of that year.

The reservoir has not filled up to FRL for number of years. Subsequent study was carried out for the non-filling years to find out the different reservoir levels that should have been kept on different dates so that the reservoir could have filled up by 1st October. From this study an enveloping curve was drawn to find out the level of reservoir to be kept on different dates to fill up the reservoir in 90% of the years. In the same graph the reservoir levels as stipulated by SEU trial II (CWC) on different dates were also plotted.

The ruling levels are given below :

Date	SEU trial-II Levels R.L.	Suggested upper limit R.L. (ft)
Up to 1st July	595	595 *
Up to 1st August	590	590 **
Up to 11th August	605	615
Up to 21st August	610	622
Up to 1st September	619	627
Up to 11th September	624	629
Up to 21st September	629.5	630
Up to 1st October	630	--

* To meet any intermediate dry spell, during the month of June, if possible, the reservoir should be raised to R.L. 595 up to 1st July.

** In this upper limit, the level during July would be lower than SEU trial-II level. This level has heen proposed with a view to flush out silt deposited in the live storage to dead storage zone by repeatedly raising and lowering the reservoir level. These lower levels do not affect the filling up of the reservoir, but power generation would be affected to some extent.

A co-relationship between monsoon percentage duration with the total monsoon runoff gives a quantitative idea regarding minimum inflow available during the balance period of the monsoon, This will help in assessing the quantity of run off available for filling up the reservoir and help in taking a decision at a particular point of time whether highter levels than SEU trial-II levels are to be maintained in order to fill up the reservoir.

2) **Reservoir Level-capacity curve for Live storage:**

The reservoir at present is being operated basing on the original capacity curve. It was seen that the actual reservoir level is differing by about 4 to 5 feet as compared to the levels calculated from the reservoir working table for the nonmonsoon period. This was analysed and it was found that the difference was because of change in storage capacity of the reservoir due to silting.. In order to know the actual effective capacity of the reservoir the upstream discharge data observed by current meter, out-flow from the reservoir through the power house and canals and reservoir levels of non-monsoon period for 3 years i.e. 1983-84,1984-85 and 1985-86 were considered to arrive at a new capacity curve.

3) **Pre-depletion Plan :**

It is now possible to know better, well in advance the likely flood flow in the basin. The accuracy of such knowledge increases with time from the moment of formation of a storm till the flood flow reaches the reservoir which can be utilized with increasing certainty for chalking out a pre-depletion plan. This is possible from the following information.

a) Satellite Image of cloud formation

b) I.M.D. forecast indicating general monsoon trough, storm track, translation velocity, strength of circulation and its location.

c) Topography of the basin and its surrounding area with respect to storm characteristics and I.M.D. prediction of quantitative precipitation.

d) The average rainfall calculated from daily /3 hourly rainfall records of 40 stations in the upper catchment and 31 stations of lower catchment.

e) The runoff records from gauging stations in upper catchment give inflow records, though approximate, 48 hours in advance of its reaching the dam site.

f) The runoff records from the gauging stations in upper catchment give inflow records 24 hours in advance of reaching the dam site.

The runoff from the lower catchment at delta head can now be predicted approximately from the rainfall records about 60 hours in advance.

g) The releases from upstream reservoirs, if intimated, in advance will further help the reservoir operation.

Suggested schedule of pre-releases from the reservoir during storm is given below.

After knowing the characteristics of a particular storm	Pre-release over and above the inflow (subject to notes under remarks)	Remark
Likely storm track out side the catchment	1.0 lac cusec	(i) Consider the location and time of storm hitting the coast.
Likely storm track in the upper catchment after the storm spread has reached the upper catchment.	2.5 lac cusec	(ii) The maximum out flow of 4,50,000 cusecs over and above the inflow (as per guideline suggested by Team of Experts).
After average rainfall in upper and lower catchment is known till 24 hour advance flood runoff prediction is available.	To give a total discharge upto 6.5 to 9 lac. cusec at delta head.	(iii) Rainfall condition in downstream catchment. (iv) The month of occurrence of the storm.
With in 48 hour to 24 hour flood runoff prediction.	To give a maximum discharge of 9 lac. cusec at delta head.	(v) The information mentioned above (1 to vii)
During 24 hour flood runoff prediction.	To give a maximum discharge of 9 lac. cusec a more at delta head.	(vi) Above remarks are to be continuously monitored till pre-release period is over.

4) Additional spillway arrangement

Before formal meeting of Committee the members visited various alternative sites for locating the additional discharging arrangements.

The Committee members visited the Gandhi-minar site first, possibility of locating a 800 feet long ogee chute spillway with crest level at 590 and fitted with 40'x 40' gates was discussed. It was considered costly because it involves heavy cutting in adjacent hillock.

Shri J. Tripathy, member discussed about the possibility of two curved spillways to be located in side the reservoir upstream of Gandhi Minar and excavating the Gandhi Minar saddle to a greater depth so as to allow lesser width of cutting for passing an additional discharge of about 5.5. lakh cusecs at RL. 630.ft.

Then the Committee proceeded to Gap-I of left dyke. It was suggested to have a fuseplug 5000' long in this gap. The toe of the dyke at Gap-I could be cut down to RL.630 so that while passing the flood over the dyke it would breach keeping all other components of the dam safe. After the prolonged discussion it was decided to abandon this idea of locating the fuse plug in Gap-I considering the socio-economic factors and disadvantages of fuse-plug in deeper reaches where possibility of loosing storage for a few seasons in reconstructing the dyke to the full height in case the fuse-plug giving way.

The Committee proceeded to Gap III. The maximum height of the dyke at Gap III is about 35 feet. The length is about 10,000 feet. The Committee then considered the feasibility of locating the fuse-plug there. Again this idea was dropped.

Dr.Y.K Murthy proposed another alternative for providing a low un-gated weir with crest level at 631.00 ft. since space is available between Gaps II and III with appropriate approach and spill channel conditions. If necessary a part of the dyke may also be used. The weir should be as long as possible to reduce the intensity of discharge(cusec/ft run) so that the erosion at the toe is minimized. Comparative estimate taking into account the various depths flow over the weir and corresponding lengths may be worked out to limit the rise in reservoir level within 1.0m. Adequate free board above the revised M.W.L may be worked out for the main dam. The existing free board may be increased considering the new M.W.L. and the free board calculated basing on recent I.S.S. applicable to old dams.

After prolonged discussion the Committee arrived at the following decisions:

(a) Flexibility of operation evolved by the statistical study was decided to be used by the Co-ordination Committee to keep the reservoir levels between these two limits depending upon the hydro-meteorological conditions of the year. The upper limit of the reservoir level would be used as a guide line during low inflow years subject to the condition that additional spillway capacity would be provided. This was discussed in detail and finally the Committee agreed to this approach.

(b) As the effect in power generation would be about 10% of July out put due to keeping the reservoir level at RL.590 ft. this approach of the flushing out the silt from live storage to dead storage zone was agreed to.

(c) It was decided to use revised capacity curve. The capacity curve is to be updated and approved by Co-ordination Committee every year.

(d) It was decided to tentatively adopt th pre-release plan given above. The pre release plan in the CWC calculations may also be considered by Co-ordination Committee.

(e) The suggestion given by Dr.Y.K. Murthy to provide a weir between Gaps-II and III by keeping its crest at RL.631-00ft. and providing sufficient length to limit the rise in reservoir level within 1.00m adopting the predepletion plan was agreed to.

(f) It was decided that appropriate studies given below would be done by the State Government in co-ordination with Central Water Commission.

(i) Analysis of flood moderation and other similar aspects in line with the studies so far conducted as suggested by Shri Rangachari, Member.

(ii) Means of improving the practicability of obtaining parameters and their sensitivity as suggested by Shri Rangachari, Member.

(iii) The studies suggested by Dr.Y.K. Murthy are :

(a) To calculate normal and minimum free-board as per I.S.S taking PMF of 28.74 lakh cusecs with a flood volume of 17.11 M.ac.ft. and MWL at 631 considering the present and additional spillway capacities.

(b) To determine statistically the occurrence of flood with peaks of 15,18,20,22,24,26 and 28.84 lakh cusecs for 10 day periods starting from 1st August to 30th September.

Sd/- F.A. Moses

6.2 Review of PMF, Provision of Additional Spillway and Dam Break Analysis.

6.2.1. Review of Probable Maximum Flood (PMF).

In the Revised report of Hirakud Dam Project-1952, the design flood was computed as 42450 cumec (15 lakh cusecs) having a 500 year return period, with a flood volume of 19.9 M.Ac.ft. equivalent to 11.60 inch of rainfall excess from the catchment over a 12 days base period. No design hydrograph has been attached to this report. The spillway was designed and constructed for gross capacity of 15 lakh cusecs and with allowance of 1/6th of gates not functioning, the effective discharge becomes 12.34 lakh cusecs only. The spillway was contemplated for regulated discharge of 12 lakh cusecs. As the project experienced flood of much greater magnititude during August 1982, it was felt necessary to review the design flood for the safety of the structure. The Spillway Design Flood has been discussed under Sec. 2.1.1.2 and further studies in Sec. 6.1.2.

Under Dam safety Assurance and Rehabilitation Project (D.S.A.R.P.), it is required to check the hydrological distress condition of the dam. Central Water Commission (C.W.C.), Hydrology Directorate was entrusted with the work of hydrological review, which includes reassessing of P.M.F. with upto date data.

The following analysis has been carried out by C.W.C. for review of P.M.F. of Hirakud Dam.

Hirakud Dam was designed for an inflow design flood (IDF) of 42450 cumec (15 lakh cusec). The team of experts has recalculated IDF as 81369 cumec (28.75 lakh cusec) in 1976. The team suggested additional spillway to pass the revised flood. When Hirakud Dam was taken under DSARP, it was decided to recalculate the IDF and Central Water Commission was entrusted with the work. Design flood was calculated by CWC with a peak discharge of 78,125 cumec (27.59 lakh cusec) and volume of 20,395 mcum (16.5 MAF). The Government of Orissa (GoO) had some reservations in accepting the estimated flood and Engineers of GoO calculated the PMF with the same rainfall, unit hydrograph and channel routing constants adopted by Central Water Commission which resulted inflow peak of 64018 cumec (22.61 lakh cusec) and volume of 15602 mcum (12.62 MAF). The calculations were submitted by GoO to CWC on 23.5.97. The Central Commission studied the calculation and adjusted UG volume of some of the subcatchment and loss rate of the earlier computation and PMF thus obtained was communicated to Government of Orissa. The peak of the PMF hydrograph is 69632 cumec (24.59 lakh cusec) and volume of 16800 mcum (13.59 MAF). Design flood hydrograph ordinates are given in Table No.6.17.

This flood was routed with initial reservoir level at 620', 624' and 630 ft. Peak outflow and M.W.L. attained are furnished in Table 6.18 with the assumption that all gates are open and 24 hours advance warning available.

Table 6.17 Inflow Design Flood Hydrograph

Time (hrs)	Discharge (cumec)	Time (hrs)	Discharge (cumec)	Time (hrs)	Discharge (cumec)
0	870.0	78	51097.0	156	1982.0
6	2490.0	84	44325.0	162	1544.0
12	7522.0	90	36948.0	168	1283.0
18	13810.0	96	29813.0	174	1133.0
24	24972.0	102	23816.0	180	1037.0
30	34208.0	108	18668.0	186	973.0
36	34208.0	114	14645.0	192	935.0
42	54466.0	120	11186.0	198	912.0
48	65483.0	126	8547.0	204	896.0
54	**69632.0**	132	6053.0	210	885.0
60	69298.0	138	4357.0	216	878.0
66	65014.0	144	3287.0		
72	57907.0	150	2540.0		

(Source: Dam Break Analysis by CWC - July 1999)

6.2.1.1. Restriction Imposed in Operation of Existing Spillway :

(a) As discussed in the report of the Evaluation Committee , model experiments conducted at Central Water and Power Research Station, Poona (Ref. Poona Research Station Annual Report-1 953) revealed that if the end gates are operated, powerful vortices would be formed in the reservoir close to the upstream face of the spillway in the end bays. Floating trees once caught in the vortex are likely to be sucked through the sluices, consequently might damage them. Hence it was instructed not to open the end pair of sluice gates at left and right spillway except under grave emergencies. Also in the annual Technical Report (1951) of Central Water and Power Research Station at page-63, it is mentioned, "End sluices may have to be closed during the maximum flood only so that water will not overtop downstream training walls".

However the Evaluation Committee (1962) did not feel the necessity for keeping sluice no.39 closed, but instructed that, as sluice no.41 is very close to the earth dam, this sluice is to be operated sparingly. The Committee also remarked that sluice no.41 and 42 can be operated normally.

(c) The Manual of Reservoir operation(1959) instructs not to open the crest gates for which corresponding sluice gates below are not open for the following observation. "If the crest gates are opened, the flow of water over the sluice exit creates a deep draft in the operation gallery. Last year it was observed that this phenomenon was so severe that wind velocity at the entrance of the sluice operation gallery was nearly 80 miles/hour. If such currents are set up it will be difficult for people to move about in the operation gallery and attend to urgent operation. Such drafts may also set up vibration in the structure which may be detrimental."

As it seems the Evaluation committee has not taken this into consideration because they have recommended restriction of operation of sluice gates no. 1 , 2, 39 and 40 without restriction on operation of crest gates No.1, 2, 20 and 21 over these sluice gates. Also at page-19(b) the Committee has recommended opening of the crest gates 31, 33, 34 prior to opening of the sluice gates 61 to 64.

(d) The three nos of spurs on the right bank in downstream of right spillway were damaged in 1958-59 and in the floods of 1961. Due to this damage in 1958 it was felt that the right spillway cannot be operated fully without endangering the right bank of the river and in case of a breach of this bank the water will flow into the power channel with consequent damage to PH.No.I and II. Poona Research Station conducted necessary model studies and in their recommendation (Annual Research Memoir of 1961) mentioned that "for safety of the right bank and training works it would always be advisable to operate the sluices an overflow spans starting from the left side of the spillway. The five sluices and overflow spans starting from the left side of the spillway. The five sluices and three crest spans adjacent to the right guide wall should be opened only under the remotest possibility of the spillway required to discharge to its highest capacity of 16980 cusecs/sec(61akh cfts). In no case the spillway be operated with only last 5 sluices and 3 spans."

From the above the Evaluation Committee recommended that the sluice Nos. 1, 2, 40, 60, 61, 62, 63 and 64 i.e. 8 nos. in all and crest gates 32, 33, 34 will not be operated except in a grave emergency. This reduces the gross discharging capacity of the spillway to 13 lakh cusecs. Allowing 1/6th of the balance 56 nos of sluice gates for repairs etc. the net discharging capacity will come to 11.671 lakh cusecs. The committee observed (in page-18) that in case a 18.3 lakh cusecs flood is experienced, the dam will be in danger".

But considering the restrictions imposed by the Manual of reservoir operation as well as the additional restrictions as recommended by the Poona Research Station in Annual Research Memoir 1961 the following gates are not be operated except in case of grave emergency.

Sluice gates	1,2,39,4,41,42,60,51,62,63 & 64	= 11 nos.
Crest gates	1,2,20,21,22,23,32,33 & 34	= 9 nos.

The gross capacity of the spillway with reservoir at R.L.630 then works out to :

Sluice gates	3 nox x 16,440 cusec	= 49,320 cusec	(1396 cumec)
	50 nos x 14,249 cusec	= 7,12,450 cusec	(20171 cumec)
Crest gates	25 nos x 16,236 cusec	= 4,05,950 cusec	(11493 cumec)
	Total	**= 11,67,720 cusec**	**(33061 cumec)**

Thus the safe release capacity of the existing spillway is 11,67,720 cusecs, with no provision for inoperative gates.

Table 6.18 : Routing of P.M.F.

Time in hrs	PMF in cumec	Initial Level FRL-10'=620ft. (192.03-3.05)=188.98m		Initial Level FRL-6'=624ft. (192.03-1.83)=190.20m		Initial Level FRL-0'=630ft. /192.03m	
		Outflow in cumec	W.L. in m	Outflow in cumec	W.L. in m	Outflow in cumec	W.L. in m
0	0	0	-	0	190.18	0	188.01
6	0	995	-	870	190.18	870	192.01
12	0	2765	-	2480	190.12	2480	191.88
18	0	8358	-	7522	189.83	7522	191.76
24	0	15139	-	13180	189.52	13810	191.86
30	870	26122	-	24072	188.72	24972	190.84
36	2490	27160	-	28209	187.78	33404	189.59
42	7522	26638	-	27606	186.89	-	188.84
48	13810	26287	-	26504	186.18	-	187.78
54	24072	26088	-	26034	185.81	-	187.42
60	34208	26203	-	26894	185.94	-	187.48
66	44395	26577	-	27224	186.08	-	187.80
72	54466	27139	-	28421	187.27	-	186.61
78	65483	29052	-	30762	189.55	-	
84	**69632**	32360	188.07	34501	190.90	-	
90	69298	46308	190.43	36345	191.16		
96	65014	40122	191.54	42070	192.18		
102	57907	42636	192.51	42127	192.87		
108	51097	43262	192.74	-	192.49	**47946**	**193.78**
114	44325	**44060**	**192.90**	**45866**	**193.26**	47913	
120	36948	43060	-	-	193.10	46874	
126	29813	42570	-	-	192.72	45099	
132	23816	40538	-	41859	192.14	42080	-
138	18668	36040	191.17	39217	191.41	40181	
144	14645	35472	-	36423	190.81	-	-
150	11186	32024	-	-	189.78		

N.B.: (Sources: CWC-Dam Safety Monitorig Directorate, New Delhi No.3/10/A/DSM/96/NO.7 dt.10.6.1997 on conceptual action plan for passing PMF through Hirakud Dam)

6.2.1.2 Routing of PMF (Peak 69632 cumec)

The Probable Maximum Flood (PMF) with peak of 69,632 cumec (24.59 cusec) was routed with Initial reservoir level at 620 ft. (188.98m), 624' (190.20m) and 630' (192.03m). Outflow discharges with corresponding reservoir levels are reproduced in the following table.

Time in hrs	Initial reservoir level (m)/ft	Maximum outflow (cumec)	Maximum reservoir level (m)
1	2	3	4
114	188.98m/620 ft	44060 (15.56 lakh cusec)	192.90
114	190.20m/624 ft	45866 (16.20 lakh cusec)	193.26
108	192.03 m/630 ft	47946 (16.93 lakh cusec)	193.78

N.B. For the routing the following assumptions were made.

i) 24 hours advance warning is available.

ii) No restriction on either rate of increase or in value of spillway discharge.

iii) All gates are open without imposing any restriction.

"Predepletion of the reservoir at level 620 ft. (139.02 m) for 36 hours with sluices and crest gates fully open brings down the level to 596.60 ft. (181.89 m) as against the level of 600.90 ft. (183.20M) obtained by the project authorities. The maximum water level attained due to impingement of PMF at this level of 596.60 ft. is 628.45 ft. (191.60 m). The comparison of the results are given in Table-6.19 below. It is seen that the project authorities have taken the impingement of hydrograph at 24972 cumecs of PMF corresponding to the ordinate of 24 hour, the reason for which is not known. For the studies in CWC, the impingement has been taken at the start of the PMF."

Table 6.19 Initial Reservoir Level 620 ft.(189.02 m) Warning Time : 36 Hours.

Impingment Level (m/ft)		Max. Water Level obtained (m/ft)		Max. Outflow (cumec/cusec)	
CWC	Orissa	CWC	Orissa	CWC	Orissa
181.89/596.6	183.20/600.9	191.60/628.45	192.61/631.8	39893/ 1408821	40017/ 1412000

Predepletion of the reservoir at 620 ft. (189.02m) with 24 hours warning time brings down the reservoir level to 605.26 ft. (184.53 m). The maximum level attained due to impingment of PMF at this level is 629.20 ft. (191.83 m) and the maximum outflow will be 14.09 lakh cusecs (39893 cumecs).

Table-6.20 : Routing of PMF

Warning Time (hrs)	Initial Reservoir Level		Impingment Level		Max. Water Level		Max. Outflow	
	(ft)	(m)	(ft)	(m)	(ft)	(m)	(cusecs)	(cumecs)
24	625.00	190.50	610.21	166.04	630.42	192.20	1497533	42405
24	630.00	192.07	615.00	167.50	631.60	192.56	1567527	44367
36	625.00	190.50	602.37	183.65	628.55	191.63	1413765	40033
36	630.00	192.07	607.68	165.27	629.8	192.01	1465007	41484

Source: CWC-Dam Safety Monitoring Directorate No.3/10/A/DSM/96 No.7 dt.10.6.1997 on conceptual action plan for passing PMF through Hirakud Dam.

6.2.2 Necessity of Additional Spillway :

The project experienced a peak inflow of 45,280 cumecs (16.00 lakh cusecs) into the reservoir on 10th July 1961 raising the reservoir level from R.L.615.06 to FRL 630ft. with peak release of 31,300 cumec (11.66 lakh cusecs) through Hirakud spillway. But the volume of this flood was only 10.864 M.Acft considerably less than the design flood volume of 19.90 M.Ac.ft.

The Evaluation committee (1962) considered the design flood peak of 18.31akh cusecs (as suggested by the International Engineering Co.) reasonable for Hirakud. For this the routed spillway peak discharge of the order of 15 to 15.6 lakh cusecs for initial reservoir level varying from 590.0 to 615.0 ft, discharging capacity to the extent available is assured right from the start of the flood. i.e. no restriction on releases to be imposed for flood control. Even with this the reservoir level was to rise uptoRL 635.75ft. No comments regarding how the dam would withstand such flood were offered nor any measures for safeguarding the dam was suggested.

The team of Experts on operation of Hirakud reservoir. reviewed the design floods computed in the past and remarked. "It would thus appear that no clear cut decision has been taken with regard to spillway design flood and the dam constructed is unsafe for any higher flood than about 15.0 lakh cusecs ifit occurs during the filling period. It may be of interest to note here that a flood of this order has already been received in July 1961".

The Probable Maximum Flood (PMF) hydrograph computed by the Committee by storm transposition and P.M.P. technique has a peak of 81,334 cumecs (28.74 lakh cusecs) and total volume of 17,155 M.cum. (17.11 million Acft.). The Committee assumed availability of an additional spillway capacity of 5.5 lakh cusecs at R.L.630ft. and routing of this flood hydrograph with initial reservoir level of R.L. 610ft. gave the maximum water level of R.L. 634.75ft. with maximum outflow of 68,627 cumecs (24.25 lakh cusecs). For higher initial reservoir levels of 615 and 620ft. the corresponding maximum water levels were computed to be R.L.635.20 and 635.80 respectively.

The Committee concluded that "with the existing spillway facilities, even with an initial level of 600ft., the reservoir level would rise as high as 641.20ft. As the strengthening of the dam for levels higher than 635 would be very expensive and difficult, the additional spillway is inevitable". The report ofthe Team of Experts was accepted by the Government of Orissa, vide letter No.HKD(W)-19176-25504 Dt.19.7.76 of I & P Deptt.

The severe storm of August 1982 in Mahanadi basin downstream of Hirakud dam was transposed to the Hirakud catchment and computations showed that this was likely to produce a flood peak of 26.94 lakh cusecs with a flood volume of 11.6 M.Ac.ft. In this derivation, the depth area duration analysis was not considered, storm ordinates were not arranged in Critical sequence, maximisation for moisture content etc. were not done as should be done in case of Probable Maximum Precipitation. Hence Probable Maximum Flood by storm transposition technique will be higher than this flood.

From above considerations at necessity and importance of providing additional spill capacity is well established.

The Expert Committee recommended strengthening of the dam to withstand above expected M.W.L. of 634.75 ft. to strengthen the embankment in delta for discharge of 12 lakh cusecs (with 2 lakh cusecs capacity for encroachments) and considerable strengthening of flood forecasting set up.

6.2.2.1 Recommended capacity of Additional Spillway

Expert Committee while routing the PMF, considered use of the full capacity of existing spillway and additional spillway capacity of 5.51akh cusecs at F.R.L.630 ft. Even with utilisation of this release capacity of 20.14 lakh cusecs at R.L.630 ft., the M.W.L. is expected to rise to R.L.634.75ft.,

with initial reservoir level of R.L.610ft. At this M.W.L. the total release capacity of spillway will be 24.25 lakh cusecs.

Even if no provision be kept for in-operative gates or gates under maintenance, the additional spillway capacity suggested by the committee is 5.5 lakh cusecs at R.L.630 ft. The total discharge of additional - spillway at M.W.L. of 634.75 ft. works out to 21425 cumecs (7.55 lakh cusecs).

6.2.2.2 Proposed location & Additional Spillway

Following alternate sites have been suggested for the additional spillway and preliminary survey was conducted.

1. Saddle close to Gandhi Minar.

2. 3rd gap of left dyke.

3. 5th gap of left dyke with link to Haradajore.

4. 6th gap of left dyke with link to Tikra reservoir.

Each site has its merit and de-merits. discussion of which is considered beyond the scope of this report.

In the proceedings of the third meeting of Rule Curve, Revision Committee, 1988, it was recommended to provide a low ungated weir with crest level at R.L.631.00ft. between gap No. II and III of left dyke and utilising part of the dyke if necessary. The length provided should be sufficient to limit the rise in reservoir level within 1.0m. For this proposal, tentative estimate of Rs.1167.50 lakhs based on 1986 Rates) has been prepared, which does not include the downstream channel.

This may be mentioned that there is no definite channel for leading the spill to the river. The spill channel will be developed on irrigated land passing between Sambalpur and Hirakud Towns, to cross Railway line and National Highway (N.H.6). The capacity of main river Mahanadi is also considered not sufficient to carry outflow of 24.25 lakh cusecs. Most of the populated towns on banks of river Mahanadi will be affected with such release.

6.2.2.3 Views of D.S.R.P.

The Dam Safety Review Panel for Orissa (under Dam Safety Assurance and Rehabilitation Project) in para-3:04 for their first/second report dated 10th October 1991 observed that there are likely to be serious problem in providing additional spillway capacity as apart from the high cost of the proposal, the spill channel which will be in operation frequently is likely to be encroached upon in due course and may not be available when required. It was therefore suggested to plan to regulate the floods by advance release from the reservoir by use of flood forecasting and flood warning system.

In this regard the routing studies conducted by CWC with (i) initial reservoir level 620' (189.02m) & warning time 36 hours (ii) initial levels of 625' (190.50m) and 630' (192.07m) with warning times of 24 and 36 hours are attached as Table No.6.19 & Table No.6.20 respectively. The table No.6.20 shows that with 36 hours warning period, initial reservoir level of 630' & impingement level of 607.68' the maximum water level reaches is 629.8' and maximum outflow is 14.65 lakh cusecs. Hence with advance warning system of minimum 36 hours no additional spillway is required.

Even if considering 24 hrs warning period with initial reservoir level of 630 ft. the maximum water level reaches is 631.50 i.e. the free board is encroached by 1.5 ft out of total 12'. The maximum outflow is calculated to be 15.67 lakh cusecs. These occurences being remote can be tolerated.

6.2.2.4 Conceptual Action Plan for Passing PMF

In the conceptual action plan for passing of PMF in respect of Hirakud dam prepared by the Water Resources Department, Orissa, the Initial level of reservoir has been chosen 'as 620 ft, (189.02 m) based on the analysis of past storms for passing the PMF. Though only 24 hour advance warning is

only possible now with a fair degree or accuracy, it is suggested that through efficient telemetry and flood forecasting system, 36 hour pre-depletion of reservoir is possible, Accordingly, the studies are carried for a warning time of 36 hours. It is also assumed that the end gates of crest and sluice may not be operative and hence, 60 nos. or sluice gates and 30 nos. of crest gates are only considered open out of 64 and 34 respectively.

The studies have been reviewed using the one dimensional mathematical model MIKE 11. The discharge through spillway and sluices used in the studies is given below in Table No.6.21 which is based on the data supplied earlier and restricted to 60 sluices and 30 crest gates. The reservoir area at different levels used in the studies are given in Table No.6.22.

Table 6.21 Discharging through sluices & spillway

Water Level (m)	Discharge (cumecs)	Water Level (m)	Discharge (cumecs)
179.88	22261	190.24	35031
181.40	23041	190.85	37154
182.62	23653	191.86	39378
184.45	24519	192.07	41701
185.37	24961	193.29	48337
185.98	25232	194.51	51816
186.59	25937	195.73	54856
187.20	26972	196.04	55576
188.41	29688	196.34	56281
189.02	31302	196.95	57656

Table 6.22 Reservoir area at different levels

Level (m)	Reservoir Area (m^2)	Reservoir Area (sft.)
179.83	27766×10^4	298873×10^4
181.36	32209×10^4	346697×10^4
182.86	36658×10^4	394586×10^4
184.40	41649×10^4	448310×10^4
185.92	46646×10^4	502097×10^4
187.45	52548×10^4	565626×10^4
190.50	65191×10^4	701716×10^4
192.02	72731×10^4	782876×10^4
195.07	83568×10^4	899526×10^4

6.2.3 Dam Break Analysis

6.2.3.1 Introduction

"The study of catastrophic flooding that may occur in the event of a dam failure is of great concern and importance because of the risk of life and property in the potentially inundated reaches below the structure Some of the recent disasters have focussed attention on the requirement of conducting

needful analysis of such cases, however remote they may be, in order to assess probable damage and also to plan necessary measures for mitigating the losses. For assessing the flood damage due to dam break, it is necessary to predict not only the possibility and mode of a dam failure, but also the flood hydrograph of discharge from the dam break and the propagation of the flood waves. The studies will have to identify the inundated area, flood depth, flow velocity and travel time of the flood waves etc.

Dam breach flood wave analysis is a classic problem of unsteady open channel flow. However, the flood caused the failure of dam will be of very high magnitude than the floods expected from the heavy rainfall. It is very essential to predict the magnitude and timing of the dam breach flood for the purposes of dam safety, evacuation planning and flood forecasting."

Among nearly 3600 large dams in India, many are embankment dams which carry a higher risk of failure. With an unreliable and fluctuating power supply system, the possibility of an earthen dam to fall by overtopping due to mal-operation / non-operation of the spillway gates can not be ruled out. Dams built decades ago with the know how of that period, are required to be monitored carefully and dam break study has to be resorted to in most cases.

The problem of the evaluation of the effects caused by the breaking of a dam has attracted considerable attention in the world only in the recent few years. In India, the importance of dam break studies was recognised only after the collapse of Machhu II dam in Gujarat in the year 1979, completely submerging the town Morvi and nearby villages resulting in the death of thousands of people and heavy loss of property. Some of the other dams that failed are Kaddam, Panshet, Khadakwasla, Chikhole, Kodaganar and Chandora.

6.2.3.2 Dam Break Modelling :

Generally, dam break studies can be carried out by either scaled physical hydraulic models or(ii) mathematical simulation using computer. A modern tool to deal with this problem is the mathematical model which is most cost effective and approximately solve the governing flow equations of continuity and momentum by computer simulation. Sophisticated computer programs have been developed in the recent few years that can simulate the dam break hydrographs, and route these hydrographs downstream so that inundated areas, flow depths, flow velocities and travel time of flood waves can be estimated. However, these computer programs are dependent on certain inputs regarding the geometric and temporal characteristics of the dam breach. The state of the art in estimating these breach characteristics is not as advanced as the computer techniques they are used with and, therefore, they are limiting factors in dam break analysis.

The model applied with the fully dynamic equation solves the vertically integrated equations of conservation of volume and momentum (Saint Venant equations), which are derived on the basis of the following assumptions.

- Water is incompressible and homogeneous; i.e. without significant variations in density.
- Bottom slope is small.
- Wavelengths are large compared to the water depth.
- Flow is sub-critical.

By virtue of its general formulation, this model is suitable for wide range of applications as detailed below.

- Flood forecasting and reservoir operation.
- Simulation of flood and evaluation of flood control measures.
- Operation of irrigation and surface drainage system.
- Design of channel systems.

- Studies of tides and storm surges in rivers and estuaries.
- Sedimentation studies.
- Dam break studies.

The dam break model set up consists of a single or several channels, reservoirs, dam break structures, and other auxiliary dam structures such as spillways sluices etc. The river is represented in the model by cross section at regular intervals. However, due to the highly unsteady nature of dam break flood propagation, it is advisable that the river course be described as accurately as possible through the use of closely spaced cross sections, particularly where the cross section is changing rapidly. Further, the cross sections should extend as far as the highest modelled water level, which normally will be in excess of the highest recorded flood level.

The reservoir is normally modelled as a single 'h' point to describe the storage characteristics by the use of storage area at different levels. This point will often also be the upstream boundary of the model, where inflow hydrograph may be specified. However, in case of very long and wide reservoirs the routing of the inflow floods has to be carried out and hence the reservoir itself will have to be also represented by cross sections at regular intervals. The downstream boundary will be either a discharge water level relation or time series water level as in case of tidal waves etc.

6.2.3.3 Utility of Dam Break Models

The various purposes to which dam break model studies can be used are as follows:

- Preparedness to tackle the disaster.
- Preparation of inundation map.
- Preparation of evacuation plans.
- Evaluation of the risk of subsequent dam failure.
- Emergency plans for reinstatement of infrastructure.
- Design of protection measures.

6.2.3.4 Emergency Action Plan

Apart from the inherent hazard potential of dam, the risk because of its presence gets compounded due to the occurrence of natural events like an unprecendented runoff, earthquake etc. Records have shown that few dam failures are on account of earthquakes whereas large number of partial or complete failure of dams are due to unprecendented floods. Therefore, there exist needs for emergency evacuation plan to lessen or mitigate its impact upon human habitation and property. Dam break analysis helps in preparing the emergency action plan (EAP).

6.2.3.4.1 Assessment for EAP

Emergency action plan call for the assessment of following items:

- Evaluation of the disaster, like advance knowledge of the likely occurrence of flood due to dam break etc.
- To identify the likely effects on property and life.
- Vulnerability analysis and hazard area mapping to identify the most common area prone to such disaster etc.

6.2.3.4.2 Inundation Plan

After completion of the dam break studies which evaluates the flow variables, it is essential to prepare the inundation plan. The preparation of inundation maps for the area downstream of the dam primarily requires a detailed and updated contour map drawn to a suitable scale usually ranging from

1:15,000 to 1:25,000 depending upon the area covered. It must provide essential information on the following:

- The entire area prone to flood-risk.
- The route of the flood peak shown as the contour line of the maximum water level.
- Computed peak water levels at differnet cross sections in the downstream and their timing from the start of failure of the dam.
- Location of important structures/places etc.
- Details of structures and/or embankments if any.

6.2.3.4.3 Extraction of Results

For the preparation of emergency action plan, the results need to be extracted along the following lines:

- Prepare plan showing inundated area.
- Mark cross sections used for study in the plan.
- Mark HFL at the cross sections.
- Mark important towns, structures etc.
- Mark ground levels at important locations.
- Prepare a statement showing the maximum depth of water at important locations.
- Prepare a statement showing the time taken by the wave front to travel from the dam site to the various locations.
- Prepare a statement showing the time taken to reach the maximum water level at the locations.
- Prepare stage hydrographs and discharge hydrographs (This is already achieved through MIKE 11 graphics).
- Prepare a statement showing the total duration of inundation.
- Indicate clearly the areas where the flood velocities are likely to be high.

6.2.3.4.4 Model Set up

The river course below the dam is represented in the model by the cross section of the river at regular intervals. A total length of 310 km reach of the river Mahanadi below the dam with 39 cross sections available in this reach has been considered for the studies. The two major tributaries of river Mahanadi, Ong and Tel rivers have been represented by 4 cross sections each starting from confluence with river Mahanadi and upto 40 km upstream of the confluence. These rivers have been connected to Mahanadi river at Ch.87.5 km and 92.5 km. downstream of dam respectively. A constant Manning's roughness coefficient of 0.033 has been considered for the entire reach of rivers. The MIKE 11 Model set up is used to solve the problem.

The reservoir and the spillway have been represented by separate branches in the model. The reservoir is represented by the area at different levels. The dam is placed in the reservoir branch and the details of the dam are provided in the dam break structure. The spillway & sluices have been specified as special weir in the spillway branch.

The upstream boundary will be time varying discharge in the form of inflow into the reservoir and will be applied at the reservoir branch. The design flood hydrograph (PMF) as adopted for the studies is given in Table-6.17. For Ong and Tel rivers since no flood hydrographs were available, hydrographs corresponding to 10% of PMF of Hirakud Dam have been considered for each in the study. The downstream boundary to be specified at Ch.31 0 km of river Mahanadi will be the rating

curve (Discharge Vs Water levels) at that location. Since no such data was available, the rating curve has been worked out at this location using Manning's equation.

6.2.3.5 Dam Break Analysis by CWC

The critical condition of dam failure will be when the reservoir is at FRL and the flood is impinged. Accordingly in the present studies the reservoir level is maintained at FRL of 192.07 m (630') and the flood hydrograph applied. The maximum water level reached in the reservoir due to application of the above flood hydrograph is 193.49 m (634.75) and this occurs at 79 hours after the application of the flood hydrograph. As the top of the dam is at EL 195.68 (642 ft) overtopping of the dam will not take place. Hence the dam is considered to fail when the water level in the reservoir reaches 193.45 m (634.5 ft) which occurs at 76 hours after the application of the flood hydrograph.

In the first instance, sensitivity analysis has been carried out by changing one parameter at a time to know the effect of the various parameters under study on dam break flood. Two different failure modes viz. linear failure and erosion based failure have been considered. Breach parameters such as breach width, breach development time, breach slope and construction materials properties such as grain size, porosity and size erosion index have been changed one at a time to study the effect of these parameters on dam break flood. In all 17 simulations have been carried out by changing one parameter at a time. Table 6.23 & 6.24 gives details. of the failure modes and parameters used for the sensitivity analysis.

As the dam is a composite structure of earth, concrete and masonry and the earthen portion is more prone to breaches, the sensitivity analysis has been carried out on the earthen dam only. All the 17 simulations have been carried out on the earthen portion between the two spillways, which is most vulnerable to breach. For all the 17 simulations the starting breach level is 195.68 m (642 ft) corresponding to the top of the dam and final breach level is 155.58 m (510 ft) which is the bed level of the river between the two spillways.

In the linear failure mode, sensitivity analysis has been carried out by changing the final breach width, breach development time and breach slope. Total eight simulations have been carried out. The analysis shows the breach width has significant effect on the dam breach flood and subsequently on the water levels at the different cross sections. The dam breach flood increases from 69030 cumecs to 84700 cumecs when the breach width is increased from 200 m to 250 m (case 1 and case 5). The breach development time has small effect on dam breach flood and water levels (case 1,2,3 and 4). However the time of occurrence of maximum discharge and water level will get delayed. The breach slope (case 6 and case 8) has a small effect on the dam breach flood and water levels. Hence it is very clear that the breach width plays the critical role in the dam breach flood.

In the erosion based failure mode, sensitivity analysis has been carried out by changing the grain diameter, porosity and side erosion index one at a time. In the erosion based failure, the dam is considered to fail when the reservoir level reaches 193.40 m (634.35 ft) which occurs at 74 hours 30 minutes after the impingement of the PMF. This has been done so as to allow some depth of water to initiate the erosion. The analysis shows the side erosion index has significant effect on the time of occurrence of maximum discharge and water level at different cross sections. By increasing side erosion index from 0.5 to 1.0 (Case 9,10 and 11) the dam breach flood increases from 65345 cumecs to 67351 cumecs for the same final breach width and the time of occurrence of maximum discharge and water levels is advanced by almost more than 3 hours.

The increase in porosity increases the dam breach flood moderately, but the time of occurrence of maximum discharge and water level at different cross sections advances significantly. The increase in porosity from 0.3 to 0.5 (Case 12 and 13), increases the discharge from 65899 cumecs to 67333 cumecs and the time of occurrence of maximum discharge and water level advances by almost 5 hour.

The increase in grain diameter has almost no effect on the the dam breach flood (Case 14 and 15) and time of occurrence of the maximum discharge.

6.2.3.6 Project Studies

The sensitivity analysis clearly indicates that in case of linear failure, the breach width and in the case of erosion based failure, the side erosion index and porosity are the most significant factors. In the erosion based failure, the breach development time for the breach width of 200 m and 250 m and breach depth of 40m is around 14 to 18 hours. In all the cases of study the increase in breach development time will automatically delay the peak.

Taking in to account all those factors, for detailed analysis the linear failure with breach width of 200 m, breach development time 4.00 hours and breach slope 0.75 (Case 4), which will provide a fairly critical condition has been considered. In this condition it has been considered that the dam will breach when water level in the reservoir reaches 193.45m (634.5 ft) which occurs at 76 hours after the application of the PMF.

The maximum discharge through the dam with the above breach parameters is 68577 cumecs which occurs about 3 hours 20 minutes after start of breach. The maximum discharge and the time of occurrence of the breach flood at all cross sections are given in Table 6.25.

The maximum water level and its time of occurrence at different cross sections are given in Table 6.26. It can be seen from this table that the maximum water level varies between 167.43m at chainage 0.0 km to 38.56m at chainage 310km downstream of the dam. The time of occurrence varies between 83-30 hours at chainage 0.0 km to 107.00 hours at chainage 310km after the impingement of the flood hydrograph. The knowledge of maximum water levels and time to peak would be of help in planning the evacuation operations, common arrangements and identifying shelter areas. Both the right and left banks levels based on the cross sections supplied are also given in Table 6.26 which will give an approximate idea of the depth of water on banks.

It has been found that the most of the river banks at different cross sections down stream of the dam are getting inundated, even for the outflow of 41609 cumecs corresponding to the capacity of the spillways and sluices at FRL even without the dam break. Hence the corresponding water levels as obtained at the different cross sections for a constant outflow of 41609 cumecs through the spillways and sluices is also given in Table 6.27.

In order to analyse the effect of the PMF in natural conditions of river Mahanadi, ie., without the Hirakud dam; a study has been conducted to rout the PMF through the river Mahanadi for the full reach of 310 Km. without the dam and spillway. For this, the initial condition in the entire river reach was assumed to be base flow of the PMF when the actual PMF was impinged. The maximum water levels as obtained at all these cross sections are given in Table 6.28. A comparison of this with the maximum water levels attained due to dam breach shows that the inundation is about 2.0m to 4.0m more due to dam breach at different cross sections.

As already mentioned, in the absence of actual inflows from Ong and Tel rivers, 10% the PMF of Hirakud reservoir has been assumed as inflow for each Ong and Tel rivers. To analyse the effect of Ong and Tel rivers on maximum water level at different reaches of Mahanadi, one more dam break simulation has been carried out without considering the Ong and Tel rivers in the model set up and taking the-breach parameters as per Case 4, the final one selected for final studies. The maximum water level and its time of occurrence as obtained a different reaches of the Mahanadi river has been compared with the results obtained with Ong and Tel rivers in Table 6.29. The difference in water level with and without considering the Ong and Tel rivers has also been worked out and is given in the same table. As can be seen, the, maximum water levels decrease from 0.3 m to 2.8 m around the confluence and down stream.

6.2.3.7 Assumptions and Limitations

These studies are based on the cross section data of rivers Mahanadi, Ong and Tel and other data as supplied by the Dam Safety Organisation, Orissa. The data made available were limited and as the studies were to be finalised within the specified time frame, suitable assumptions have to be made where the data were not made available or the supplied data were not found to be in order The main assumptions that have been made are given below.

The reservoir has been assumed to be a single point in the model represented by the area at different elevations. This may be conveniently used for projects having small reservoirs where the propagation of the flood through the reservoir can be neglected. However, in case of Hirakud dam, the reservoir is a very large one extending for a length more than 80 km. Hence it is not correct to represent the reservoir just by the area at different elevation. The reservoir should be represented by suitable number of cross sections at least at 5 km intervals or closer so that the propagation of the flood hydrograph is taken into account to arrive at the correct flood at the dam site. Though some cross sections of the reservoir were made available, the data did not include the cross sections near the dam site for a reach of 25 km which is the most crucial data where the effect of siltation will be high.

The area at different elevations of the reservoir supplied are very old taken before the construction of the project and would have considerably changed now due to siltation.

In the absence of any calculated floods from Ong and Tel rivers, the flows in Ong and Tel rivers have been arbitrarily assumed as 10% of the PMF of river Mahanadi at the Hirakud dam site.

The river is overflowing in most of the cross sections due to dam breach. It is essential that all such cross sections are extended at least using the topo sheets so as to describe the actual topography of the sections in the model. In the absence of these extended cross sections, the model will assume vertical wall beyond the extreme points on both sides of the banks.

6.2.3.8 Recommendations and Conclusions

It is very clear that while the banks are inundated at all the locations up to 310 km. due to the dam breach, the banks up to 180 km. downstream of the dam get inundated even for the release of discharge at full capacity from the spillway. Hence it is essential that proper evacuation measures are planned in advance and kept ready so as to use them at the time of emergency. Further, it is also advisable not to allow any further encroachments in these areas including construction of important structures etc.

The present studies are up to 310 km down stream of the dam only. This should be extended further to know the effect of dam breach flood near the Cuttack city. Since no contour map of the area downstream of the dam has been provided, inundation map has not been prepared. However, the maximum water level at different cross sections downstream of the dam after the dam break has been given in Table 6.29, from which the inundation map can be prepared and based on this inundation map the Emergency Preparedness Plan can be formulated.

As already pointed out, the river is overflowing in most of the cross sections. In MIKE11 model, the cross sections are assumed as vertical beyond the extreme bank levels specified and hence, the water levels obtained above the bank levels may not be realistic. As such, it is essential that all these cross sections be extended beyond the present bank levels up to the maximum water level in order to describe the topography correctly in the model.

The reservoir of Hirakud dam is a very large one with a stretch of about 80 km. It is not correct to model this huge reservoir by a single point with the area elevation curve as done now in the present studies, as the moderation of the flood hydrograph through the reservoir will not be reflected in the model. Hence it is essential that the reservoir is also represented in the model by the cross sections at regular intervals so that the moderated flood is applied at the dam site. This can be done if the cross sections of the reservoir for the full reach are supplied.

The rise in maximum water level after considering 10% contribution of Ong and Tel rivers is of the order of about 2 m. This underlines the importance of contribution of these two rivers. It is suggested that the observations of Ong and Tel rivers should be incorporated into the model before preparing the inundation map.

(Source: Hirakud Dam, Orissa - Dam Break Analysis, Central Water Commission - July 1999 and Data on Flood Control Operation of Hirakud Dam - Published by Govt. of Orissa September 1993).

Failure Modes and Parameters adopted in different simulations are shown in Table 6.23 & 6.24

Table 6.23 Linear Failure

Case No.	Breach Level (m)		Breach Width (m)		Slope	Breach Development Time (Hr)
	Initial	Final	Initial	Final		
1.	195.68	155.48	1	200	0.75	1
2.	195.68	155.48	1	200	0.75	2
3.	195.68	155.48	1	200	0.75	3
4.	195.68	155.48	1	200	0.75	4
5.	195.68	155.48	1	250	0.75	1
6.	195.68	155.48	1	250	0.50	2
7.	195.68	155.48	1	250	0.75	3
8.	195.68	155.48	1	250	0.75	2

Table 6.24 Erosion Failure

Upstream slope : 3.00
Critical Shear Stress : 0.030
Breach Slope : 0.75
Downstream Slope : 2.00
Specific Gravity : 2.70
Intial Breach Level : 195.68 m

Case No.	Final Breach Width (m)	Grain Diameter (mm)	Porosity	Side Erosion Index
9.	200	0.075	0.4	0.50
10.	200	0.075	0.4	0.75
11.	200	0.075	0.4	1.00
12.	200	0.075	0.3	0.75
13.	200	0.075	0.5	0.75
14.	200	0.050	0.4	0.75
15.	200	0.100	0.4	0.75
16.	250	0.075	0.4	0.75
17.	250	0.075	0.4	1.00

Source : GOI-CWC-Dam Break Analysis Group – Foundation Engineering & Special Analysis Directorate (DSO), July 1999.

Table 6.25 : Maximum Discharge due to dam breach and its Time of Occurrence after the impingement of the flood hydrograph at different locations of the river reach.

(Breach parameters are as per Case No. 4)

Chainage from Dam (km)	Maximum Discharge (Cumecs)	Time of Occurrence Hrs-Mts
1.5	108618	83-30
4.5	108594	83-40
8.5	108552	83-50
13.0	108477	84-00
17.5	108440	84-20
22.5	108422	84-20
27.5	108310	84-40
32.5	108178	85-00
42.5	107892	85-40
57.5	107583	86-20
73.8	106833	88-10
78.4	106296	88-10
82.8	111551	89-00
90.7	110995	90-30
93.8	115419	91-10
102.5	114155	89-50
111.0	113854	90-20
123.0	113541	91-00
135.0	112335	91-50
143.0	112335	91-50
143.0	110736	93-10
152.5	109645	94-40
161.0	109168	95-30
168.2	108860	96-10
177.5	108608	97-00
187.5	108500	97-40
187.5	108468	98-00
211.7	108451	98-20
218.5	108430	98-30
222.5	108398	98-40
225.5	108357	98-50
236.0	108263	99-30
248.0	108039	100-10
257.5	107624	101-00
266.7	107078	101-50
281.5	105722	103-40
298.0	104775	106-20
307.5	104666	107-10

Table 6.26 : Maxm. Water Level due to dam breach & its Time of Occurrence after the impingement of the flood hydrograph at different locations of the river reach
(Breach parameters are as per Case No. 4)

Chainage from Dam (km)	Maximum Water Level (m)	Time of Occurrence (Hrs-Min)	Left Bank Level (m)	Right Bank Level (m)
0	167.43	83-30	196.00	196.00
3	163.28	83-40	153.44	155.29
6	160.48	83-50	164.03	157.30
11	158.15	84-10	150.16	149.68
15	156.29	84-00	142.80	141.12
20	151.72	84-20	140.33	139.64
25	145.63	85-00	136.53	136.25
30	141.96	85-30	133.61	132.70
35	140.62	85-30	130.90	135.36
40	138.92	85-50	129.60	131.68
45	136.84	86-10	129.61	131.68
60	126.49	87-50	124.65	124.65
76	124.63	88-20	117.32	115.20
80.75	122.53	88-50	118.40	116.70
85	120.10	89-10	111.34	112.70
92.68	116.76	89-20	110.65	109.68
95	115.23	89-30	106.23	108.95
105	108.89	90-40	100.32	102.48
113	106.66	90-50	95.61	98.20
125	101.86	92-10	96.10	95.83
137	96.40	95-50	87.59	84.39
145	95.48	96-30	85.68	84.06
155	94.18	96-40	83.89	83.44
163	92.99	97-00	80.99	79.56
170	91.86	97-20	78.21	78.04
180	89.67	98-00	76.98	77.03
190	86.05	98-10	76.14	75.59
200	80.69	98-30	70.45	73.26
214	67.31	99-00	64.45	67.50
220	65.07	99-10	97.45	101.80
225	63.48	99-20	95.97	123.40
226	62.92	99-20	96.14	97.85
238	58.02	99-50	69.32	65.68
250	53.08	101-20	50.70	49.10
260	50.17	103-10	49.98	45.01
269	47.67	105-00	44.91	47.44
284	44.98	106-00	38.29	37.86
300	41.97	106-40	33.91	37.10
310	38.56	107-00	38.78	30.99

Table 6.27 : Maximum Water Level at different locations of the river reach downstream of the dam due to release at full capacity of 41609 cumecs from spillways and sluices without the dam break

Chainage from Dam (km)	Maximum Water Level (m)	Left Bank Level (m)	Right Bank Level (m)
0	161.87	196.00	196.00
3	155.50	153.44	155.29
6	152.92	164.03	157.30
11	150.22	150.16	149.68
15	148.54	142.80	141.12
20	144.72	140.33	139.64
25	138.37	136.53	136.25
30	134.38	133.61	132.70
35	132.53	130.90	135.36
40	131.03	129.60	131.68
45	129.34	129.61	131.68
60	119.89	124.65	124.65
76	118.02	117.32	115.20
80.75	116.30	118.40	116.70
85	113.79	111.34	112.70
92.68	110.65	110.65	109.68
95	109.32	106.23	108.95
105	102.81	100.32	102.48
113	100.34	95.61	98.20
125	96.76	96.10	95.83
137	88.18	87.59	84.39
145	86.60	85.68	84.06
155	85.29	83.89	83.44
163	84.13	80.99	79.56
170	83.76	78.21	78.04
180	79.75	76.98	77.03
190	75.18	76.14	75.59
200	70.09	70.45	73.26
214	60.63	64.45	67.50
220	59.18	97.45	101.80
225	57.51	95.97	123.40
226	57.12	96.14	97.85
238	52.76	69.32	65.68
250	47.84	50.70	49.10
260	44.43	49.98	45.01
269	41.41	44.91	47.44
284	37.52	38.29	37.86
300	32.46	33.91	37.10
310	24.50	38.78	30.99

Table 6.28 : Maximum Water Level at different locations of the river reach without the dam due to impingement of PMF

Chainage from Dam (km)	Maximum Water Level (m)	Left Bank Level (m)	Right Bank Level (m)
0	164.45	196.00	196.00
3	159.20	153.44	155.29
6	156.47	164.03	157.30
11	153.92	150.16	149.68
15	152.18	142.80	141.12
20	147.97	140.33	139.64
25	141.75	136.53	136.25
30	138.05	133.61	132.70
35	136.50	130.90	135.36
40	134.97	129.60	131.68
45	133.14	129.61	131.68
60	123.32	124.65	124.65
76	121.57	117.32	115.20
80.75	120.00	118.40	116.70
85	117.37	111.34	112.70
92.68	114.55	110.65	109.68
95	112.95	106.23	108.95
105	106.55	100.32	102.48
113	104.27	95.61	98.20
125	99.86	96.10	95.83
137	93.25	87.59	84.39
145	92.16	85.68	84.06
155	90.85	83.89	83.44
163	89.63	80.99	79.56
170	88.39	78.21	78.04
180	85.96	76.98	77.03
190	82.19	76.14	75.59
200	77.05	70.45	73.26
214	64.94	64.45	67.50
220	63.00	97.45	101.80
225	61.38	95.97	123.40
226	60.89	96.14	97.85
238	56.18	69.32	65.68
250	51.23	50.70	49.10
260	48.09	49.98	45.01
269	45.30	44.91	47.44
284	41.99	38.29	37.86
300	38.04	33.91	37.10
310	33.18	38.78	30.99

Table 6.29 : Maxm. Water Level due to dam breach & its Time of Occurrence after the impingement of the flood hydrograph at different locations of the river reach
(Breach parameters are as per Case No. 4)

	With Ong and Tel Rivers		Without Ong and Tel Rivers		
Chainage from Dam (km)	Maximum Water Level (m)	Time of Occurrence (Hrs-Min)	Maximum Water Level (m)	Time of Occurrence (Hrs-Min)	Difference in Water Level Col2-Col4(m)
0	167.43	83-30	167.39	83-00	0.04
3	163.28	83-40	163.23	83-10	0.05
6	160.48	83-50	160.42	83-20	0.06
11	158.15	84-10	158.08	83-30	0.07
15	156.29	84-00	156.23	83-45	0.06
20	151.72	84-20	151.65	83-55	0.07
25	145.63	85-00	145.53	84-15	0.10
30	141.96	85-30	141.84	84-40	0.12
35	140.62	85-30	140.50	85-05	0.12
40	138.92	85-50	138.80	85-15	0.12
45	136.84	86-10	136.71	85-25	0.13
60	126.49	87-50	126.19	86-50	0.30
76	124.63	88-20	124.18	87-20	0.45
80.75	122.53	88-50	121.93	87-35	0.60
85	120.10	89-10	119.47	87-50	0.63
92.68	116.76	89-20	116.11	88-15	0.65
95	115.23	89-30	114.50	88-20	0.73
105	108.89	90-40	108.12	89-15	0.77
113	106.66	90-50	105.87	89-45	0.79
125	101.86	92-10	101.13	90-20	0.73
137	96.40	95-50	95.00	94-25	1.40
145	95.48	96-30	93.99	94-45	1.49
155	94.18	96-40	92.68	95-10	1.50
163	92.99	97-00	91.47	95-20	1.52
170	91.86	97-20	90.29	95-45	1.57
180	89.67	98-00	88.00	96-00	1.67
190	86.05	98-10	84.33	96-10	1.72
200	80.69	98-30	79.08	96-30	1.61
214	67.31	99-00	66.25	97-05	1.06
220	65.07	99-10	64.14	97-15	0.93
225	63.48	99-20	62.54	97-35	0.94
226	62.92	99-20	62.01	97-40	0.91
238	58.02	99-50	57.17	98-05	0.85
250	53.08	101-20	52.18	99-35	0.90
260	50.17	103-10	49.11	101-00	1.06
269	47.67	105-00	44.41	102-30	1.26
284	44.98	106-00	43.34	104-10	1.64
300	41.97	106-40	39.82	105-00	2.15
310	38.56	107-00	35.74	105-20	2.82

6.3 Silting of Hirakud Reservoir :

6.3.1 Introduction :

All artificial reservoirs created by construction of dams across natural streams are subjected to varying degree of sedimentation which is a natural phenomenon. The engineers and planners are confronted with the problem of prediction of the rate of sedimentation and the probable time when the useful function of the reservoir would be affected.

"Soil erosion, its transportation and subsequent deposition in reservoirs is a universal phenomenon. Uncontrolled deforestation, forest-fires, overgrazing, improper method of tillage, unwise agriculture practices and other man's activities are mainly responsible for accelerated soil erosion. It is estimated that about 6000 Million tones of soil are eroded every year in India as a result of sheet erosion. Besides, gully and ravine erosion ravages 8000 ha of useful land annually.

All eroded material does not get into a stream. Some particles travel for a short distance and get deposited before reaching a stream for want of sufficient velocity of water. Some may travel into the river system and get lodged in the vegetation on the banks. Some others may be carried downstream only to be deposited in the plains and finally only a portion of eroded particles enters the reservoir. Thus at a given control point on a river all the sediment produced by the upstream watershed does not get delivered. In a cascade system of reservoirs, upstream reservoirs intercept a part of the transported material and the sediment inflow into the dowstream reservoir reduces.

The channel and flow characteristics change considerably with the obstruction to the flow of water in a natural stream, when a dam is constructed. The depth of flow increases along a reservoir limb and the velocity decreases progressively. This results in reduction of transporting capacity of flow. The sediment brought by the stream into the reservoir starts settling down and gets deposited on the bed of the reservoir at all levels. The coarser particles settle first. The finer particles are carried in suspension and may finally settle down on the reservoir floor while some of these are passed over the spillway or through the outlets to the downstream of a dam. If the concentration is high, density current occur along the floor of the reservoir. At higher reservoir levels finer particle initially get deposited in the live storage space but successively move into the dead storage space depending on inflows due to successive floods, the draw down of the reservoir and operation of outlets. Thus the deposit of sediment can affect the storage volume available at all levels. Loss of storage space is estimated at the planning stage and provisions are so made that the benefits of the reservoir are not adversely affected." (Source: Compendium on Silting of Reservoirs in India - Central Water Commission, Jan. 2001 Pg.1).

Estimation of life of reservoir is a complex one in absence of long term silt observation data distribution of sediment, its consolidation, density and trap efficiency etc. Life of the reservoir is generally planned on the basis of sediment criterion. The useful life of reservoir is generally thought of only in terms of life but not in terms its service value to the community. Further, there is not uniformity in the method of assessment. In many cases assumptions are made or empirical formula is used in estimating the total silt load (both suspended sediment and bed load).

6.3.2 Sedimentation Planning & Practices

The problem of sedimentation of reservoirs and its effect on the useful life of reservoir is complex. The phenomenon of sedimentation and the concept of life of a reservoir and its estimation have been studied in depth over many years.

Strictly speaking reservoirs do not have a defined 'life' which denotes two functional states- 'ON' and 'OFF'. They belong to one of those systems which show gradual degradation of performance without any sudden non-functional stage. Sedimentation and consequent reduction of capacity is a gradual process, which can be classified in following phases i.e. (i) the reservoir shows no adverse effect and is able to deliver full planned benefits. (ii) the reservoir delivers progressively smaller benefits, but its continued operation for the reduced benefits is economically beneficial, (iii) the sedimentation

causes difficulties in operation such as jamming the passage of flow in channels or flow in canals or through turbines, (iv) the phase difficulties become so serious that the operation become impossible; the benefits reduce to such an extent that it is no longer beneficial to operate the reservoir.

However, there are no instances of last two phases having been reached amongst the modern projects.

6.3.2.1 Distribution of Sediment :

Initially, life related concepts of a reservoir were vague. The life has been estimated differently from project to project with different assumptions. This is due to the fact that clear knowledge and data regarding the inflow of sediment into the reservoir, its distribution and the consolidation and density aspects of sediment etc, were not available in earlier periods. For instance, it was frequently assumed that all the sediment would first be deposited in the dead storage zone and would then only encroach on the live storage zone. Under this assumption the period taken for the hypothetical total sedimentation of the dead storage was considered as the life. Such an assumption has been made in the DVC projects as also in most of the projects planned upto the middle of sixties including Hirakud Dam Project. Thus it can be taken as the normal practice of the earlier period. This assumption was proved wrong by sediment surveys of several reservoirs in the country, conducted since late fifties. Two notable examples are mentioned below.

In earlier periods, the life of certain reservoirs like Krishnarajsagar and Hirakud has been worked out by the deltaic theory enunciated by Col. Ellis of Madras PWD which assumed silting up of reservoirs in three stages- i.e., (i) Forming the silt up to sill of sluices. (ii) Intermediate stage is the formation of channel by river flow. (iii) Slow deposit of silt thereafter. For computing the quantity of silt deposited, it had been assumed that the silt deposit in area is proportional to the water standing or flowing over the area.

In case of Bhakra Reservoir, silt load was estimated by suspended silt measurement and its distribution was done assuming that all the coarse silt would be deposited in head reach of reservoir, forming delta below the full reservoir level encroaching on the live storage and that the delta would gradually progress towards the dam. Various methods adopted incalculating life of 46 Indian reservoirs having gross capacity of 0.5 M.A.cft (0.062 M.ham) and above is enclosed herewith as Annexure 6.1.

6.3.2.2 Sedimentation Planning :

"Dr. A.N. Khosla, the then chairman, Central Water Commission, had reviewed the work of reservoir sedimentation in fifties based on data available for 200 reservoirs all over the world including U.S.A., China and Africa and developed enveloping curves for annual sedimentation rate for major and minor catchments above and below 1000 sq. miles (2600 sq. kms) respectively. He concluded that the sediment rate for major catchments varies from 0.357 to 0.476 mm./year (3.57 to 4.76 Ha.m./ 100 sq.km./yr.) and for minor catchments from 0.38 to 1.28 mm./year (3.80 to 12.80 Ha.m./100 sq.km/yr.).

Up to 1965, the above recommendations were adopted in the design of reservoirs and the sediment was assumed to get deposited at the lowest level and "life": was taken as the period required for complete sedimentation of the dead storage.

The assumption that sediment would settle within the dead storage was not supported by the experience in other countries or India. The experience of USA that sedimentation takes place throughout the reservoir and the development of methods for sediment distribution were published around early fifties. It was also realised that the sediment inflow rates need to be checked up through reservoir resurveys. Hence resurveys in a number of projects were taken up through research schemes of CBI&P. The results indicating a considerable difference from the initial assumption, started becoming available by 1965.

After 1965, CWC started insisting that the sediment inflow rates be based on the basis of reservoir survey data. It also brought out the need for distributing the sediment throughout the reservoir. Although the earlier practices also continued, atleast the more important major projects had to adopt this new approach.

Around 1974, CWC decided that the 50 year sedimented position of the reservoir should be used in the simulation or 'working table' studies for the project. Also by this time the observed suspended sediment data from the key hydrologic network of CWC had become available in considerable volume. CWC therefore started insisting on the use of this measured sediment transport data also to firm up the assumption of the inflow rates of sediment, in addition to the reservoir resurvey information.

In 1982, the report of the working group on the guidelines for the preparation of detailed project report of major and medium irrigation projects was published. In this report, CWC had incorporated the above mentioned practices to make this mandatory on the state governments. Also in this report the detailing of the sediment studies was linked with the expected seriousness of the sediment problem, For very serious cases, redistribution and reestimation of trapping efficiency in 10 year block was indicated.

In 1987, CWC got this practice incorporated in the IS code 12182 (1987) "Guidelines for Determination of Effects of Sedimentation in Planning and Performance of Reservoirs" to make this the national practice. In these guidelines the general philosophy and concept of multiple life related term was also spelt out. Also these guidelines indicated that the full service time for hydroelectric projects can be reduced to 25 years against 50 years for irrigation projects. These IS guidelines also include notes on the need for periodic resurveys and give guidance to determine their frequency."

(Source: Compedium of Silting of Reservoirs in India - Central Water Commission, Jan. 2001 Pg.4-5).

6.3.2.3 Measurement of silt in Hirakud Reservoir

Measurement of silt for assessing rate of siltation in Hirakud reservoir started in 1947 in the light of C.B.I.&P Publication No.51 entitled as "Silting of reservoirs". Calculation of life of Hirakud reservoir has been given at page 96-97 of the said book. (Copy enclosed as Annexure 6.2) As per the methodology given there in, silt loads are divided into three grades.

Coarse silt - diameter more than 0.20 mm.
Medium silt - dia 0.20 mm to 0.075 mm
Fine silt - diameter less than 0.075 mm

10% of total suspended is considered as bed load. It is assumed that 100% of coarse silt, 80% of medium silt, 20% of fine silt and 100% of bed silt would be retained in the reservoir. Mean annual silt deposit (observed between 1947 to 1951 by inflow outflow method by using Punjab type bottle sampler) was calculated to be 16,897 acre ft. With dead storage of 1.88 M.A.cft the life of the reservoir was found to be 112 years. This type of observation continued upto the year 1973. In the upstream, observation was carried out at 3 sites beyond the maximum water spread area i.e. Saradihi across Mahanadi, Deogaon on Ib and Sripura on Bheden and in the downstream at Mahanadi bridge near Burla besides at all canal outlets.

The data from 1957 to 1965 showed that the average annual rate of silting is 24800 ac.ft. The life of the reservoir worked out to 76 years commencing from 1957.

As the technology developed, other better methods were deployed to find the total silt trap in the Hirakud reservoir. The trap efficiency of Hirakud Reservoir from 1957 to 1973 is appended vide Annexure-6.3 and 6.4. To know the exact amount of siltation in different zones reservoir survey by echosounding as detailed by B.N. Murthy and Albert Fry vide (i) A monogram on Reservoir capacity survey and (ii) Reservoir sedimentation survey is adopted. Radioisotope method or photographic survey can also be employed to know the silt trap. But these two methods have not been tried in case of Hirakud reservoir. The details of silt survey by Inflow-outflow method and echo-sounding method done for different projects in India and in USA upto 1965-66 are given in Table-6.30 & Table-6.31 respectively for comparison.

Table 6.30 : Details of Siltation of some Indian Reservoirs

Sl. No.	Name of Project	Catchment area in sq. miles	Mean Annual Run-off in Million Acre feet	Reservoir Storage capacity in Million acre feet	Area of Water spread in Sq.miles	Length of water spread in miles	Silt deposited in acre feet per 100 sq.miles of catchment per year.
1	2	3	4	5	6	7	8
1.	Rihand Dam Project U.P L1-3085'H-306'	5,148	5.138	L 7.28 D 1.32 G 8.60	180 sq.mile	-	-
2.	Machkund, Orissa L1-1420'H-199'	755	1.43	L 0.777 D 0.003 G 0.78	35	20	51 by inflow and outflow method (AV.1958-1966)
3.	Bhakra on the river Sutlej, Punjab (Govind Sagar Reservoir) L1-1700'H-740'	21,960	16.0	L 6.335 D 1.665 G 8.00	64	56	160
4.	Beas, Punjab	3,320	9.0	G 3.75	-	-	-
5.	Dhupdal-River Ghataprava (Belgaum,Bombay) H-27'	1,080	2.754	G 0.034	-	-	-
6.	Thungabhadra, Bellary Dt. Mysore L1-5712'H-162'	10,880	10.5	L 3.001 D 0.050 G 3.051	146	37 miles at right angles and 50 along the course	377
7.	Chambal Valley-Gandhisagar Dam (M.P) L-1685' H-212'	8,700	3.8	L 5.61 D 0.67 G 6.28	255.2	70	-
8.	Koyna (Maharashtra) L1-2200'/H-207.5'	-	-	G 0.83	21	-	-
9.	a) Panchet Hill (D.V.C.) L1-22115'H-134'	4,234	-	L 0.185+0.881 D 0.148 G 1.214	23	18	260
	b) Maithon (D.V.C.) L1-15712'H-162'	2,430	-	L 0.496 D 0.608 G 1.104	28	-	325
10.	Nagarjuna Sagar (A.P) L1-15526'H-489'	83,087	34.34	5.44	73.66	-	-

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Sl. No.	Name of Project	Catchment area in sq. miles	Mean Annual Run-off in Million Acre feet	Reservoir Storage capacity in Million acre feet	Area of Water spread in Sq.miles	Length of water spread in miles	Silt deposited in acre feet per 100 sq.miles of catchment per year.
1	2	3	4	5	6	7	8
11.	Maswad Tank (Sholapur, Bombay) H-80'	480	-	G : 0.06	-	-	-
12.	Lake Fife Khadakwasla Dam Poona (Bombay) H-102.5'	196	1.0	0.09	-	-	-
13.	Bhadra-Mysore L1-1400'+275'	-	-	L : 1.44	80	-	-
14.	Sharavati-Mysore a) Linganmakki Dam (Main) b) Talakalala Dam (subsidiary area for 2nd Power house) L1-15840' (with Dyke) H-199'	769	-	G : 3.5 L : 3.4	126	-	-
15.	Mayurakshi, West Bengal L1-2100' H-155'	718	-	L : 0.445 D : 0.055 G : 0.500	26	-	274
16.	Matatila, U.P. L1-20920' (with Dyke)	8,000	-	G : 0.75	55	-	107
17.	Lower Bhawani, Madras L-28882' H-204' (Masonry) & 120' (Earth)	-	-	G : 0.64	30.4	16 miles along Bhabani and 4 miles along Mayur	-
18.	Hirakud L1-16 miles with Dykes H-200'	32,000	33.1	L : 4.72 D : 1.88 G : 6.60	288	36 miles along the Mahanadi, 23 miles along the Ib.	76 obtained by inflow & out flow method (Av.1957-1965)

L=Live storage, D=Dead storage, G=Gross storage, L1=Length of Dam in ft. H=Height of dam in ft.
(Source: Indian Journal of Power & River Valley Development - April, 1967).

Table : 6.31 : Details of Silting of Few American Reservoirs

Sl. No.	Name	Catchment area drainage area (sq.mile)	Ratio of Reservoir capacity (gross) to Average Annual Runoff	Reservoir Storage capacity in acre feet	Mean Annual Run off in Acre feet	Area of water spread (sq. mile)	Length of water spread (in mile)	Silt diposited per 100 sq.miles of catchment per annum
1.	Hungry Horse Dam, Montana L1-2100' H-564'	1,640	-	G: 3,500.00 L: 2,960,000	-	24,000 acres (37.5 sq.miles)	35	-
2.	Grand Coulee Dam, Washington L-1-4173' H-550'	74,100	12.5	G: 9,517,000 L: 5,220,000	80,000,000	82,000 acres (128 sq.miles)	600 miles of shore line	6.2
3.	Hoover Dam (Boulder Canyon Project), Arzona-California Nevada L-1-1282' H-726.4'	167,800	20.3	G: 31,047,000	14,945,000	157,70 acres (245 sq.miles)	115	36
4.	Shasta Dam C.V.P. California L1-3500' H-602'	6,600	-	G: 4,493,000 D: 104.000	-	46 sq.miles	35	79.5
5.	Roosevelt Dam Project L1-1125' H-280'	5,760	18.1	G: 1,398,000	840,000	-	-	-
6.	Arrow Wrock Dam, Boise Project; Idaho L1-1100' H-354'	2,200	-	L: 284,000	-	3,000 acres 4.7 sq.miles	17	-
7.	Anderson Ranch Dam, Boise Project, Idaho L1-1400' H-456'	-	-	500,000	-	-	13	-
8.	Davis Dam: Davis Dam Project Arizona, Nevada L1-1600' H-200'	-	-	G: 1,820,000 L: 1,600,000	-	28,500 acres 44.5 sq.miles	67	-
9.	Green Mountain Dam, Colorado-Big Thompson Project Colorado. H-309'	-	-	G: 152,000 L: 100,000	-	2,125 acres 3.3 sq.miles	7	-
10.	Imperial Dam and Desilting Works, Boulder Canyon Project L1-3430' H-46'	187,000	-	85,000	-	7,500 areas (11.7 sq.miles)	-	-

L = Live storage, D = Dead storage, G = Gross storage, L1 - Length of dam in ft., H1 = Height of dam in ft.
(Source : Indian Journal of Power & River Valley Development - April, 1967).

Annexure 6.1

Statement showing methods adopted in calculating Life of Reservoirs having Gross Capacity 0.062 million ha m (0.5 million acre-ft) and above in India

Sl. No.	Name of Dam (State)	River	Height		Length of Crest (m)	Gross Capacity of Reservoir		Capacity in Mcum		Method adopted in calculating the life of Reservoirs
			Above Lowest Foundation (m)	Above Ground Level (m)		(Mcum)	(Acre-ft)	Dead Storage	Live Storage	
1	2	3	4	5	6	7	8	9	10	11
1.	Nagarjunasagar (A.P.)	Krishna	123.7	112.77	4650.00	11,315.4	9177000	476.1	6919.8	No Method has been adopted for calculating the life of reservoir. However, the life of reservoir has been estimated to be 377 years.
2.	Srisailam (A.P.)	Krishna	143.2	121.9	512.0	8723.0	7072000	2766.0	5958.0	Based on the silt observations in past on River Krishna the life of reservoir has been worked out to 450 years.
3.	Nizamsagar (A.P.)	Manjira	–	–	–	715	580000	94	485	Not Available
4.	Pochampad (A.P.)	Godavari	43.5	–	–	3170	2570000	–	2319	Not Available
5.	Machkund (Orissa)	Machkund	50.6	–	409.6	770	624000	–	752	Not Available
6.	Panchet (Bihar)	Damodar	44.81C 40.84E	–	–	1497	1210000	–	1307	Not Available
7.	Maithon (Bihar)	Barakar	56.0	50.0	4847.0	820.0	664800	207.2	611.8	For the purpose of estimating life of reservoir the dead storage capacity is divided by quantity of silt annually deposited 84.13 ha m (684 acre-ft). The estimated life of reservoir is 246 years.
8.	Kadana (Gujrat)	Mahi	58.0	24.0	1398.0	1714.2	1388502	339.6	1202.7	Moody's modified empirical area reduction method has been used. The life of reservoir has been worked out as 100 years.

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Annexure 6.1

1	2	3	4	5	6	7	8	9	10	11
9.	Ukai (Gujrat)	Tapti	67.4C 68.6E	44.2G 68.6E	4927.0	8511.0	6900000	1171.8	7462.6	Assuming that half of the total silt, i.e., 1778.22 ha m (14457.1 acre-ft) (average observed at Kakrapar for 3 years) is carried away long with floods passing over the spilway and through the undersluices, the sedimentation rate works out to 0.014 ha m/sq km (0.31 acre-ft/sq miles) per year. For catchment area of 60515.11 sqkm (23365 sqm) the life of reservoir is estimated to be 131.5 years with a silt reserve of 0.117 million ha m (0.95 million acre-ft).
10.	Vanivilas Sagar (Karnatak)	Vedavati	49.8	43.3	405.4	849.6	688705	38.2	812.9	Not available
11.	Krisharajasagar (Karnatak)	Cauvery	42.7	39.6	2621.3	1368.7	1100000	123.4	1247.0	Not available
12.	Tungabhadra (Karnatak)	Tungabhadra	49.4	35.35	2440.2	4040.2	305400	442.8	3330.4	Based on the observation of two anicuts above the dam site, the yearly silt deposit has been estimated at 3.64 million cu m (128.6 million cu ft), i.e., 363.81 ha m (2957.8 acre-ft). On this basis life of reservoir as a whole will be several hundred years.
13.	Bhadra (Karnatak)	Bhadra	71.6	59.1	1190.2	2022.0	1641640	236.8	1788.6	Not available
14.	Almatti (Karnatak)	Krishna	38.7	34.7	457.2	2348.0	1905000	–	–	The rate of silting is worked out by empirical formula given by Inglis ($S = cm^{3/4}$) and thus life of reservoir has been worked out as 122 years.
15.	Hidkal (Karnatak)	Ghataprabha	59.4	53.9	4267.2	1455.5	1179390	45.3	–	The average annual silt load of 0.42 million cu m (15.00 million cu ft) per year has been considered by using the empirical formula, viz, Inglis and Khosla's formulae. The life of reservoir has been worked out as 100 years.

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Annexure 6.1

1	2	3	4	5	6	7	8	9	10	11
16.	Siddapur (Karnatak)	Krishna	25.5	23.6	512.0	790.4	641200	–	–	Not available
17.	Linganamakki (Karnatak)	Sharavathi	61.3	55.2	2362.1	4417.0	3582000	141.8	4276.5	Based on Khosla's formula for silting, i.e., 0.035 ha m/sq km (0.75 acre-ft per sq mile) per year, for catchment area of 1991.70 sq km (769 sq miles) the dead storage provided will give life of 200 years.
18.	Malaprabha (Karnatak)	Malaprabha	43.3	37.2	154.0	879.8	713300	93.9	–	Assuming the rate of silting as 3.56 ha m/ year/100 sq km (75 acre-ft/year/100 sq miles) for catchment according to Khosla's and Inglis formulae. A provision of 100 years silt load has been made.
19.	Hemavathi (Karnatak)	–	–	–	–	654	530000	–	–	Not available
20.	Ghataprabha (Karnatak)	Ghataprabha	45.00	–	1300	661	540000	–	618	Dead storage designed on the basis of Khosla's formula of 3.75 ha m/100 sq km (75 acre-ft/100 sdq miles) gives annual silt load of 42.4 ha m (345 acre-ft) for a period of 100 years.
21.	Idikki (Kerala)	Periyar	171.0	168.0	366.0	1459.53	1182219	–	–	Not available
22.	Bhatghar (Maharashtra)	Yalvandi	59.0	51.2	1625.0	682.6	552900	–	–	Not available
23.	Mulshi (Maharashtra)	Mula	50.6	44.5	1555.4	761.1	611300	217.1	536.6	Not available
24.	Koyna (Shivaji Sagar) (Maharashtra)	Koyna	103.0	–	804.6	2780.0	2250000	141.9	2499.0	Some silt data were collected in Koyna River during monsoon of 1949 but this was not considered conclusive. Some similar data were collected for the Khadakwasla reservoir near Poona. This reservoir has a catchment similar to the Koyna and the results are to some extent applicable.

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Annexure 6.1

1	2	3	4	5	6	7	8	9	10	11
	Koyna (contd.)									Judged by these two observations it is anticipated that the probable annual silt load in Koyna reservoir would be of the order of 5.034 to 7.88 ha m/100 sq km/ year (106 to 166 acre-ft per 100 sq miles/ year). These figures are much higher than Shri Khosla's. It is, therefore, expected that the silting life of the reservoir as far as drawn off for power is concerned, will not be less than 150-200 years.
25.	Bhima (Maharashtra)	Bhima	43.28	–	2330	3040	2470000	–	1342	Not available
26.	Jayakawadi-I (Maharashtra)	Godavari	36.50	–	–	1456	1177000	–	1036	Not available
27.	Warna (Maharashtra)	Warna	58.83	–	–	2442	1980000	–	1068	Not available
28.	Girna (Maharashtra)	Girna	53.3	40.5	964.0	678.0	549600	28.4	652.1	Dead storage designed for a slit rate of 0.0056 ha m/sq km/year (0.12 acre-ft/sq mile/year) for a life 100 years.
29.	Mula (Maharashtra)	Mula	55.5	50.5	2820.0	1017.0	824500	131.9	604.4	Rate of silting has been taken as 0.035 ha m/sq km/year (0.75 acre-ft per sq mile/year)
30.	Yeldari (Maharashtra)	Purna	51.40	42.70	3270.0	964.0	781600	152.9	811.6	Dead storage designed for a silt rate of 2/3 i.e. 0.031 ham/sq km/year (0.66 acre-ft/sq mile/year) for a life of 75 years.
31.	Gandhisagar (Madhyapradesh)	Chambal	–	62.2	513.6	8450.0	6850000	832.6	6911.2	Dead storage has been provided on the basis of Shri Khosla's recommendation, viz. a silt reserve of 0.035 ha m/sq km (0.75 acre-ft per sq mile) of catchment per year for 100 years.
32.	Tawa (Madhyapradesh)	Tawa	55.2	–	1330	3645	2955000	–	2764	Not available

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Annexure 6.1

1	2	3	4	5	6	7	8	9	10	11
33.	Jalaput (Orissa)	Machkund	55.5	44.8	634.0	970.5	786800	–	–	–
34.	Balimela (Orissa)	Sileru	70.0	–	4633.0	3823.0	3096630	–	–	Not Available
35.	Hirakud (Orissa)	Mahanadi	58.2G +61.3E	43.3G +61.3E	1148.5G +3651.5E	8141.0	6600000	2319.0	5822.0	The life of the reservoir has been originally worked out on the basis of theory of deltaic formation. For this purpose the bed width of the stabilised river has been taken as 0.80 km (1/2 mile), the critical slope as 1/4000 and the coefficient of rugosity as 0.025. The critical discharge to establish a regime has been taken as 11326.72 cumecs (400000 cusecs).
36.	Bhakra (Govindsagar) (Punjab)	Sutlej	225.5	167.5	518.2	9868.0	8000000	2053.7	7814.1	For Bhakra Reservoir, the sediment volume has been computed on the basis of densities of 1041.3 kg/cu m and 1441.8 kg/cu m (65 lb/cu ft and 90 lb/cu ft) separately and average values have been taken to represent the volume of silt brought into the reservoir annually. This annual incoming silt has been worked out at 3936.0 ha m (32,000 acre-ft) and 2829.0 ha m (23,000 acre-ft) respectively for assumed densities. The average of these volumes, 3382.5 ha m (27,500 acre-ft) has been taken to represent the annual incoming silt. This gives a rate of 0.013 cu m/sq km (1.2 cu ft/sq mile) per year.
37.	Beas (Punjab)	Beas	115.8	100.6	1524.0	8140.0	6600000	–	7290.0	Mile by mile capacity method and Moody;s modified empirical area reduction method have been used and it is estimated that about 15 percent of dead storage and 18 percent of live storage may cost in 100 years.

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Annexure 6.1

1	2	3	4	5	6	7	8	9	10	11
38.	Ranapratap Sagar (Rajasthan)	Chambal	54.0	39.2	1144.0	2890.0	2340000	764.8	2133.9	Based on Shri Khosla's formula of 0.035 ha m per 100 sq km/year (0.75 acre-ft per 100 sq miles per year).
39.	Bajajsagar (Rajasthan)	Mahi	74.0	62.5	944.9	2038.8	1653000	345.0	1715.0	Moody's modified empirical area reduction method has been used. The life of reservoir has been worked out as 100 years.
40.	Mettur (Stanley Dam) (Tamilnadu)	Cauvery	64.9	53.6	1615.4	2708.0	2197000	62.9	2647.0	Not available
41.	Lower Bhawani (Tamilnadu)	Bhawani	67.18	42.67G	8791.0	792.8	642880	0.4	930.0	Not available
42.	Rihand (Gobind Balapant sagar) (U.P.)	Rihand	92.9	80.5	934.2	10630.0	8600000	1726.9	9214.1	Assuming silt deposit of 1/500 (this corresponds to something like 1/800 as per Khosla's formula of 0.092 ha m/year (0.75 acre-ft/year) of the average annual run-off and taking life of reservoirs as 100 years provision necessary to silting amounts to 0.169 million ha m (1.38 million acre-ft) as against 0.172 million ha m (1.4 million acre-ft) provided.
43.	Matatila (U.P.)	Betwa	36.6	24.4	6315.5	1132.3	918000	111.0	874.5	In the project report of 1951, provision was made for dead storage for 100 years at the rate of 0.013 million cu m/sq km (1.2 million cu ft per sq mile) for 100 years (on the basis of rate of silting observed at Dhukwan). In the later (revise) project report this has been reduced to 92.85 M cu m (3279 million cu ft) on 9276.29 ha m (75417 acre-ft) which is nearly 1/3 of the first provision). This dead storage provision is also reduced for 30 years instead of 100 years. The new assumption made is that 30 percent of the silt will get deposited in the silt pocket and the remaining silt will get deposited in the live storage.

Contd...

Annexure 6.1

1	2	3	4	5	6	7	8	9	10	11
44.	Ramganga (U.P.)	Ramganga	125.6	108.8	557.8	2369.3	1920000	246.7	1948.9	Based on silt load data collected for other reservoirs like Bhakra, Hirakud, etc. a silting rate of 4.27 ha m/100 sq km (90 acre-ft/ 100 sq miles) gives life of 185 years before reservoir will get silted up to dead storage elevation.
45.	Canada (Masanjor) (West Bengal)	Mayurakshi	47.2	37.5	640.0	616.7	500000	67.8	540.3	Some rough silt experiments were made at the site of the dam during 1937 and 1938. The average quantity of silt in river discharge during flood in these years is found to be 0.028 cu m in 33.98 cu m (1 cu ft in 1200 cu ft) of water. During 1939-40 silt observations were carried out on scientific basis from July to September and the average proportion of silt is 0.114 percent or part in 900 parts [0.92 million cu m (34 million cu ft)]. Taking this rate of silting life has been calculated as 200 years.
46.	Kangsabati (West Bengal)	Kangsabati	41.0	38.0	10400.0	1135.0	920000	145.5	989.0	A silting rate of 0.07 ha m/sq km (1.50 acre-ft/sq mile) per year for 100 years has been assumed.

(Source : Life of Reservoir, CBIP, Technical Report No. 19, New Delhi, (March, 1977), Pg 101-105)

Annexure-6.2

Sedimentation and Period of Useful Life of Hirakud Reservoir -

Extract from (i) "Silt in the Mahanadi and the Silting of the Hirakud Reservoir" by Hirakud Research Station (ii) Fourth Irrigation and Power Seminar held at Hirakud from 14-16 January 1957, Vol.II, CWPC, New Delhi.

(i) Among the other theories put forth in regard to silting, that relating to deltaic formation merits description. The life of the Hirakud Reservoir has been worked in the Hirakud Dam Project Report Vol.II on this basis.

In working out the life of the Hirakud Reservoir on the basis of the theory of deltaic formation the bed width of the stabilised river has been taken as 0.80 km (½ mile), the critical slope as 1/4000 and the co-efficient of rugosity as 0025. The critical discharge or discharge which would be of sufficient duration to establish a regime has been taken as 11326.72 cumecs (4,00,000 cusecs), based on the hydrograph of the river. The level of margins restraining between them a discharge of 18,405.92 cumecs (6,50,000 cusecs) has been taken as the permanent level for the new margins; in other words, silting in the diversion period will not raise the margins to any greater level, once they have been silted up to such a level as will enable them to contain between them a discharge of 18,405.92 cumecs (6,50,000 cusecs). Any further silting of the margins can be effected only during the impounding period. The silt calculations have been made for the following three definite stages.

Initial Stage: From the time the dam is brought first into use till the formation of the permanent bed for the regime channel for 11,326.72 cusecs) (4,00,000 cusecs).

Intermediate Stage: This is the period from the completion of the initial stage to the time which marks the cessation of silt deposition over margins during diversion periods for the flows of 18,405.92 cumecs (6,50,000 cusecs) or less. For purposes of calculation the intermediate stage has been further sub-divided into 7079.2 cumecs (2,50,000 cusecs) stage, 14,158.4 cumecs (5,00,000 cusecs) stage and the 18,405.92 cumecs (6.50,000 cusecs) stage.

Final Stage : This is the period during which the permanent margins get silted up during the impounding period and for flows above 18,405.92 cumecs (6,50,000 cusecs).

For computing the quantity of silt deposited, it has been assumed that the silt deposit on an area is proportional to the water standing or flowing over the area; also the silt deposited in the bed of the permanent river bed will be removed by the following freshets, as the channel is a regime channel.

The estimation of silt load for a year of mean annual runoff [taken as 6.15 million ham (50 million acre-ft)] has been based on the data of 1950's. The silt charge has been assumed proportional to runoff and as such a silt load of 4797.0 ha m (39,000 acre-ft) for 6.15 million ha m (50 million acre-ft) runoff has been worked out, the total silt load for the year 1950 being 2904.27 ha m (23,612 acre-ft) and the total runoff 4.29 million ha m (34.92 million acre-ft).

In the absence of any reliable bed load data, it has been taken as 15 percent of the total suspended load. As against this, it has been assumed that 15 percent of the total suspended load will be removed by density currents, the net annual sediment load remaining at 4797.0 ha m (39,000 acre-ft) as above.

The period for the completion of the first sub-stage of 7079.2 cumecs (2,50,000 cusecs) has been worked out at 100 years on the above basis and the sub-stage of 14158.4 cumecs (5,00,000 cusecs) will be required a further period of 260 years. In view of such lengthy periods for the first two sub-stages, it has been considered unnecessary to calculate the periods for further sub-stages and final stage.

Calculations regarding effective capacity of the reservoir have shown that even at the end of 400 years the capacity will still be sufficient to meet fully the irrigation demand of the project.

(ii) Extract from Project Report Vol.II.

General Theory of Silt Deposition (Hirakud Dam Project Report, Vol. II, Main Dam, Reservoir and Power Plant 1, pp. 54-55).

"As the rapidly flowing waters of the river reach the reservoir, the velocity suddenly gets reduced and the water moves on to the spillway considerably deadened. The coarse particles get settled at the head reach and the rest of silt moves on and begins to fill the deep river portion and the flanks, proportional to the depth of water overlying the place. The bed load will continue itself to the old deep river bed. The result will be that after a few years, the original river cross-section at the bottom of the reservoir will become almost a plain.

In the next step, phenomenon similar to what occurs in the formation of a natural delta takes place. This is the formation of a natural delta, the river deposits silt, before it empties itself into the sea; but it always keeps open a suitable passage for its own discharge. When the level of the silt deposit at the out-fall end becomes so high as to offer an obstruction to flow, it will cease rising in level for a width sufficient to satisfy the requirements of river flow but the deposit of silt will continue on the areas outside of this width. The periodic deposition of silt, if any, in the new bed of the river will be scoured and carried during the subsequent diversion. Thus, the deltaic river forms gradually its own margins and gradually pushes out a deltaic tract through which it flows into the sea.

It may be assumed that the existing spurs and features, which can be seen on the contour map of the country above the dam site will direct the course, and limit the cross-section of flow when that flow becomes appreciable, and that a cross-section of flow will be formed the width and direction of which can be foreseen within reasonable limits of accuracy, from the study of the existing contours of the country.

As the current becomes appreciable much of the silt in suspension will be carried through the section of flow, and get deposited further down, until gradually, a permanent regime is reached in the section of flow below, while marginal flats will be formed and gradually attain such *height* as not to be submerged in ordinary floods.

These marginal areas will be gradually forming and pushing themselves farther and farther into the reservoir, as in natural deltaic formation river deltas push themselves out towards the sea.

This process will go from the head of the water-spread at diversion level downwards, forming gradually a new deltaic river, with marginal deltaic flats pushing forward into the waterspread area until stopped by the dam.

The river channel will keep itself open, free to discharge the normal yearly flow, but the margins will form flat plains on either side; and when a permanent delta has been formed there will be a river of deltaic character, flowing through a deltaic plain formed from its own deposits and confined in margins similarly formed up. The process of formation will be in many respects similar to that found in nature where, as already stated, river highly charged with silt discharges into tideless estuary.

As this new deltaic river bed and margins are formed commencing at the upstream end of the reservoir, velocity in the waterspread area gradually appears and increases, and with velocity, surface slope must be present to give motion to the water.

The amount of surface slope depends on the velocity which in turn depends on the cross-sectional area of the 'Sections of flow'.

The widths of the sections of flow will be limited by the present existing contours of the country while the depth will probably approximate to that due to the velocity under Kennedy's theory regarding silt deposit and scour, and may be denominated, the 'critical depths' due to the velocities.

The gradually increasing surface slopes of the newly formed, and forming deltaic river will have the effect of gradually increasing the levels of water up stream of the points of original silt deposit for many miles and the marginal bed silt deposits will take place not only below but above the site of original deposit. The fixed tail level which in natural deltaic rivers is 'mean sea level' will here be the 'diversion level' artificially fixed and the surface slope necessary to produce flow of the river through the section of the flow will be upwards from this level and the new deltaic river surface levels will gradually increase during the process of formation.

As the silting up of the marginal deltaic flats must gradually conform to the surface levels of the silt carrying water of the river in its various cross-sections, it follows that the deltaic flat on each margin of the section of the flow will gradually conform in level to the river levels and the fall of the marginal plains along the new river will conform to the river surface fall."

6.3.2.4 Trap Efficiency of Reservoirs :

The actual rate of silting of a reservoir depends on many other factors, in addition to the rate of sediment production in the catchment area: they ate trap efficiency of the reservoir, ratio of reservoir capacity to total run-off, gradation of silt, method of reservoir operation etc.

The reservoir trap efficiency will decrease continuously once storage is begun. The dominent factors which control the rate of silting in any storage reservoirs are :

i) the relation of capacity to inflow, and

ii) the content of sediment in the inflow.

These two principal factors have a complete range of interplay, that is, a reservoir having a small capacity-inflow ratio and a small sediment content in the inflow might have the same average percentage loss of usual capacity as reservoir having a large capacity-inflow ratio and a large sediment content in the flow.

The laws of sediment deposition are the same for all types of reservoirs including stock ponds, and the factors influencing the trap efficiency of the reservoir are the same irrespective of the size of the reservoir. When the capacity of reservoir is mote than its annual runoff, i.e., its capacity-inflow ratio being more than 'I', and also taking the effect of seepage and evaporation losses, it is obvious that water is rarely spilled over the dam, and hence the trap efficiency of such reservoirs must be close to 100 percent. The capacity-inflow ratio defines the reservoir' into classification that whose ratio is less than 1 as seasonal reservoir and, those exceeding, as hold over storage reservoir.

Some of the observation made on reservoir has shown that it appears probable that reservoirs having a very low capacity-inflow ratio may alternately fill and scour, depending upon stream flow conditions, and may have' a trap efficiency of zero or less during periods of scour.

Even in hold-over storage reservoirs, if the operation is so adjusted' as to allow a major portion of the inflow through the outlets,' the trap efficiency may be brought to 60 percent with the capacity-inflow ratio of 1.7, while the trap efficiency would be in the order of 90 percent under normal operating conditions.

The trap efficiency of some of the reservoirs according to Brune's trap efficiency curve is given below (vide Table 6.32).

Table No.6.32 : Trap Efficiency of Few Reservoirs.

Sl. No.	Name of reservoir (state)	Original trap efficiency (year)	Latest trap efficiency (year)
1.	Konar (Bihar)	97% (1955)	92% (1996)
2.	Idukki (Kerala)	98.4% (1974)	97% (1999)
3.	Kakki (Kerala)	98% (1966)	96% (1999)
4.	Balimela (Orissa)	96% (1972)	95% (1999)
5.	Bhakra (Punjab)	99.8% (1996)	95.5% (1996)
6.	Matatila (UP)	90% (1956)	88% (1994)
7.	Maithon (Bihar)	96% (1955)	95% (1994)

(Source: Compendium on Silting of Reservoirs in India CWC, Jan. 2001, Pg.10)

Trap Efficiency of Hirakud Reservoir :

While calculating the life of Hirakud Reservoir, it was assumed that 100 percent of coarse silt, 80 percent of medium silt, and 20 percent of fine silt would be retained in the reservoir. Similar type of assumptions have also been made in Bhakra and other reservoirs in the country while estimating the life of reservoirs. Actual observations carried out at Hirakud, as indicated in Table 6.33 / Annexure 6.3 shows that most of the fine sediment ranging from 60 to 90 percent is retained in the reservoir. The overall trap efficiency of the reservoir is found to vary from 65 to 90 percent. Such variation is due to the inadequacy of sampling frequency. From this, it is seen that all the incoming sediment is trapped in the reservoir and nothing substantial flows out of the reservoir through the outlets and over the spillways. The capacity inflow ratio of Hirakud is 0.2 and according to Brune's curve a trap of efficiency of about 92 percent is indicated for this reservoir. Table No.6.34 / Annexure 6.4 shows the Trap efficiency of different particle sizes of silt from the year 1957 to 1973.

Trap efficiency of Gandhi Sagar reservoir (M.P.) is almost 100%, Matatila reservoir (U.P) is 88%, Bhakra reservoir (Punjab) is 95.5% and Idukki (Kerala) 97%.

Table 6.33 Trap efficiency of Hirakud Reservoir (For total sediment) Annexure-6.3

Catchment area 82879.68 sq.km (32,000 sq.miles); Capacity of Reservoir 0.81 M ha.m

(6.6 M. acre-ft.) Annual average Inflow : 4.059 M. ha m (33 M. acre.ft): Capacity/Inflow Ratio = 6.6/33=0.2

Year	Total Sediment inflow ha-m (acre.ft.)	Total sediment outflow ha.m (acre.ft)	Sediment retain in reservoir ha.m (acre.ft)	Observation trap efficiency percent	Observation inflow in reservoir M.ha.m (acre.ft)	Capacity/ inflow ratio each year	Trap efficiency as per Brune's curve percent
1957	2332.20(18961)	414.63(3371)	1917.57(15590)	82.2	2.43(19.8)	0.33	94
1958	4676.59(38021)	1299.25(10663)	3377.33(27458)	72.2	4.23(34.4)	0.19	91
1959	5149.15(41863)	1381.41(11231)	3767.73(30632)	73.2	4.39(35.7)	0.18	91
1960	3927.02(31927)	1402.07(11399)	2524.94(20528)	64.3	3.98(32.4)	0.20	92
1961	12162.73(98884)	3157.90(25674)	9004.83(73210)	74.0	8.64(70.3)	0.09	87
1962	1655.45(13459)	511.80(4161)	1143.65(9298)	69.1	1.83(14.9)	0.44	95
1963	4391.22(37701)	736.03(5984)	3655.19(29717)	83.2	3.63(29.5)	0.22	93
1964	5447.18(44286)	1481.16(12042)	3966.01(32244)	72.8	5.15(41.9)	0.16	90
1965	1863.61(15883)	174.17(1416)	1779.44(14467)	91.1	1.39(11.3)	0.58	97
1966	1924.82(15649)	464.57(3777)	1460.25(11872)	75.9	2.03(16.5)	0.40	95
1967	4166.99(33878)	1176.37(9564)	2990.62(24314)	71.8	3.75(30.5)	0.22	93
1968	3324.32(27027)	626.31(5092)	2698.00(21935)	81.1	2.44(19.9)	0.33	94
1969	2275.99(18504)	411.19(3343)	1864.80(15161)	81.9	2.42(19.7)	0.33	94
1970	3363.80(27348)	583.51(4744)	2780.29(22604)	82.8	4.30(35.0)	0.19	91
1971	2851.01(23179)	586.95(4772)	2264.06(18407)	79.4	4.66(37.9)	0.17	90
1972	2361.23(19197)	274.66(2233)	2086.57(16964)	88.4	2.25(18.3)	0.35	94
1973	2829.74(23006)	852.26(6929)	1977.47(16077)	69.9	5.20(42.3)	0.16	90

Table 6.34 : Trap Efficiency of Different Particle sizes of Sediment in Hirakud Reservoir (1957-73) **Annexure - 6.4**

Year	Coarse Grain size : 0.2 mm and above				Medium Grain size : 0.075 to 0.2 mm				Fine Grain size : Less than 0.075 mm			
	Sediment load inflow ha-m (acre-ft)	Sediment load outflow ha-m (acre-ft)	Retention of sediment load ha-m) (acre-ft)	Obser-vation trap efficiency in percent	Sediment load Inflow ha-m (acre-ft)	Sediment load outflow ha-m (acre-ft)	Retention of sediment load ha-m) (acre-ft)	Obser-vation trap efficiency in percent	Sediment load inflow ha-m (acre-ft)	Sediment load outflow ha-m (acre-ft)	Retention of sediment load ha-m) (acre-ft)	Obser-vation trap efficiency in percent
1	2	3	4	5	6	7	8	9	10	11	12	13
1957	142.92(1162)	6.27(51)	136.77(1112)	96.0	275.15(2237)	10.45(85)	264.69(2152)	96.0	1914.12(15562)	398.03(3236)	1516.09(12326)	79.0
1958	257.44(2093)	16.94(154)	238.49(1939)	92.0	560.05(4472)	35.05(285)	515.00(4187)	93.6	3869.21(31457)	1245.25(10124)	2623.96(21333)	67.8
1959	265.18(2969)	14.88(121)	350.30(2848)	95.9	617.95(5024)	20.91(170)	597.04(4854)	96.6	4166.01(33870)	1345.62(10940)	2820.39(22930)	67.7
1960	281.54(2289)	17.96(146)	263.59(2143)	93.6	489.54(3980)	21.89(178)	467.64(3802)	95.5	3155.93(25658)	1362.22(11075)	1793.71(14583)	56.8
1961	830.86(6755)	51.66(420)	781.66(6355)	93.8	1458.29(11856)	74.54(606)	1383.75(11250)	94.9	9873.58(80273)	3098.74(25193)	6774.84(55080)	68.6
1962	56.82(462)	6.84(54)	50.18(408)	88.3	107.75(876)	6.27(51)	101.47(825)	94.2	1490.88(12121)	498.89(4056)	991.99(8065)	66.5
1963	166.91(1357)	6.39(52)	161.37(1312)	96.7	339.72(2762)	10.21(83)	330.99(2691)	97.4	3884.71(31583)	721.76(5868)	3162.94(25715)	81.4
1964	232.34(1889)	12.669(103)	219.80(1787)	94.6	563.71(4583)	22.75(185)	540.95(4398)	96.0	4651.12(37814)	1445.74(11754)	3205.38(26060)	68.9
1965	48.71(396)	1.10(9)	47.60(387)	97.7	82.41(670)	1.23(10)	81.18(660)	98.5	1822.49(14817)	169.51(1379)	1650.66(13420)	90.6
1966	55.84(454)	0.61(5)	55.22(449)	98.9	92.74(754)	1.23(10)	91.51(744)	98.7	1776.24(14441)	462.72(3762)	1313.51(10679)	73.9
1967	52.16(424)	0.12(1)	52.03(423)	99.8	104.06(846)	0.62(5)	103.44(841)	99.4	4010.90(32609)	1175.63(9558)	2835.27(23051)	70.7
1968	42.43(345)	-	42.43(345)	100.0	71.83(584)	-	71.83(584)	100	3210.05(26098)	626.31(5092)	2583.74(21006)	80.5
1969	32.96(268)	-	32.96(268)	100.0	53.75(437)	-	53.75(437)	100	2189.27(17799)	411.19(3343)	1778.09(14456)	81.2
1970	62.36(507)	-	62.36(507)	100.0	117.09(952)	-	117.09(952)	100	3184.22(25888)	583.51(4744)	2600.83(21145)	81.7
1971	215.74(1754)	-	215.74(1754)	100.0	381.17(3099)	-	381.17(3099)	100	2254.09(18326)	586.95(4772)	1667.14(13554)	74.0
1972	143.66(1168)	-	143.66(1168)	100.0	185.97(1512)	-	185.97(1512)	100	2031.59(16517)	274.66(2233)	1756.93(14284)	86.5
1973	296.55(2411)	-	296.55(2411)	100.0	384.86(3129)	-	384.86(3129)	100	2148.32(17466)	852.29(6929)	1296.05(10537)	60.3

6.3.2.5 Trend in Sedimentation of Reservoirs

"Dr. A.N. Khosla had observed that in the reservoirs which have small slucing capacity with respect to normal floods and which have no reservoirs above them, the siltation rate is comparatively high in the first 15-20 years and thereafter it falls off and may ultimately become negligible. This is because the obstruction by the dam causes the dips and flanks of the storage basin to fill up with silt in early years. A stage comes when the river section is adjusted to carry the normal discharge and disposal of suspended load in the area of the reservoir is harmonised with the condition of the flow. Besides, the progressive development of deltas above reservoir helps in trapping of some of the silt load. Further, shrinkage and settlement of deposited silt also takes place due to weathering action and superimposed loads of additional silt load. This results in reduction in silt volume thereby reducing the sedimentation rate.

To verify the above phonemenon, 10 reservoirs were selected where more number of capacity surveys have been conducted. Analysis of results of capacity surveys of these reservoirs considering average rate of siltation at the end of every 10 years from the year of impoundment indicate that the rate of sedimentation is higher in initial period of operation of reservoir and decreases with the passage of time. For these reservoirs sedimentation surveys have been carried out 3 to 25 times spread over 2 to 7 decades. The percentage decrease in rate of siltation in respect of these reservoirs is given vide Table 6.35(a).

Table 6.35(a) : Rate of Siltation

Sl. No.	Name of reservoir (State)	Rate of sedimentation in Th.Cu.m/sq.km./year				Total period between mid points of two blocks in years	Percentage decrease in siltation in the total period	Percentage decrease in siltation per year
		First period of 10 years	Rate of siltation	Last period of 10 years	Rate of siltation			
1.	Panchet Hill (Bihar)	1956-66	0.973	1986-96	0.343	30	67.83	2.261
2.	Maithon (Bihar)	1955-65	1.170	1984-94	1.123	29	03.24	0.117
3.	Pong (H.P)	1974-84	2.558	1988-98	1.350	14	47.22	3.383
4.	Tungabhadra (Karnataka)	1953-63	0.602	1983-93	0.226	30	62.46	2.082
5.	Hirakud (Orissa)	1967-67	0.657	1984-94	0.562	17	14.46	0.851
6.	Bhakra (Punjab)	1958-68	0.633	1988-98	0.663	30	-	-
7.	Lower Bhavani(T.N)	1953-63	0.306	1973-83	0.246	20	19.60	0.980
8.	Vaigai (T.N)	1958-68	0.409	1973-83	0.380	15	07.10	0.473
9.	Matatila (U.P)	1956-66	0.849	1984-94	0.340	28	59.95	2.141
10.	Dhukwan (U.P)	1907-17	0.042	1970-80	0.012	63	71.43	1.138

On an average there is a reduction of 35.4% in 28 years i.e. 1.28% per annum. The observed average percentage loss of gross capacity in respect of these reservoir in each 10 years block is given in Table 6.35(b).

Table 6.35(b) : Loss of Gross Capacity in 10 years Block

Period	Percentage loss
Upto 10 years	6.31
10 to 20 years	4.71
20 to 30 years	4.22
30 to 40 years	3.84

(Source : Compendium on Silting of Reservoirs in India CWC, January 2001, Pg.8).

Average rate of silting of Hirakud and Balimela reservoirs from the year of impounding is furnished in Table 6.36 and 6.37.

Table 6.36 : Average Rate of Silting of Hirakud

Name of Reservoir : Hirakud
Name of river/State : Mahanadi/Orissa
Assumed rate of silting : 0.25 Th.Cu.m/sq.km./yr
Catchment Area : 83395 sq.km.
Reservoir Area at FRL : 727 sq.km.

Sl. No.	Year of Survey	Capacity in M.Cu.m.	Silt deposited between two consecutive surveys in M.Cu.m.	Period in years	Rate of silt deposited between two consecutive surveys		Loss of Capacity (Cumulative)	
					M.Cum.m/ year	Th.Cu.m/ sq.km/year	M.Cu.m	Percen-tage to original capacity
1	2	3	4	5	6	7	8	9
1	1957	8105.00						
2.	1979	6900.00	1205.00	22	54.77	0.657	1205.00	14.87
3.	1982	6626.41	273.59	3	91.19	1.093	1478.59	18.24
4.	1984	6614.7	11.71	2	5.855	0.070	1490.30	18.39
5	1994	6145.74	468.96	10	46.90	0.562	1959.26	24.17

Year of impounding : 1957
Average rate of silting in 37 years (1957-94) = 1959.26/37 = 52.95 M.Cu.m./year
= 52.95x1000/83395 = 0.636 Th.Cu.m/sq.km/year = 0.635 mm/year
= 6.35 ha.m/100 sq.km/year.

Table 6.37 : Average Rate of Silting of Balimela

Name of Reservoir : Balimela
Name of river/State : Machkund/Orissa
Assumed rate of silting : N.A.
Catchment area : 4908 sq.km.
Reservoir Area at FRL : 169.08 sq.km.

Sl. No.	Year of Survey	Capacity in M.Cu.m.	Silt deposited between two consecutive surveys in M.Cu.m.	Period in years	Rate of silt deposited between two consecutive surveys		Loss of Capacity (Cumulative)	
					M.Cum.m/ year	Th.Cu.m/ sq.km/year	M.Cu.m	Percen-tage to original capacity
1	2	3	4	5	6	7	8	9
1	1972	3610.53	-	-	-	-	-	-
2.	1999	3327.98	282.55	27	10.46	2.131	282.55	7.83

Year of impounding : 1972
Average rate of silting in 27 years (1972-99) = 282.55/27 = 10.46 M.Cu.m/year
= 10.46x1000/4908 = 2.131 Th.Cum.m/sq.km/year = 2.131 mm/year

(Source: Compendium of Silting of Reservoirs in India - CWC, Jan. 2001 Pg.125-126).

6.3.3 Sedimentation Study of Hirakud Reservoir

Periodic assessment of sedimentation rate is essential to ascertain the present live storage capacity of reservoir for efficient and productive management of water resources. The information so obtained is also necessary for upstream catchment treatment for controlling rate of siltation.

Conventionally assessment of sediment in the reservoir is carried out either by inflow-outflow measurement method (vide Table 6.33 & 6.34) or by hydrographic survey (4 cycles in case of Hirakud from 1979 to upto 1991), and by Remote Sensing method (vide Table 6.40).

With the availability of high resolution satellites, capacity surveys of reservoirs by Remote Sensing are gaining much popularity. Further, the data sets generated by Remote Sensing techniques relating to reservoir water spread area are analysed by GIS for management and decision making.

6.3.3.1 Sedimentation Assessment by Remote Sensing.

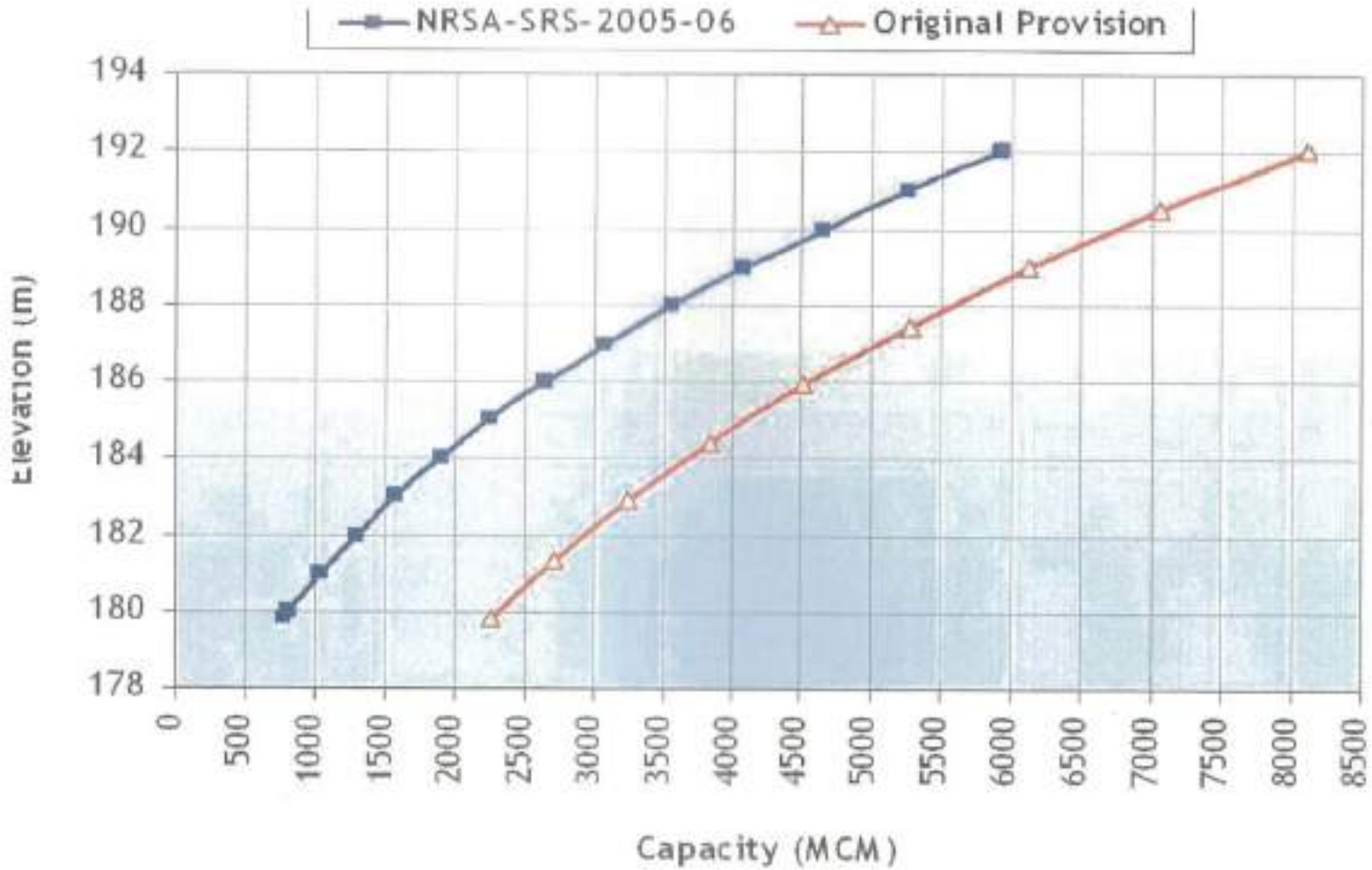
6.3.3.1.1 Study by NRSA

This study was entrusted to National Remote Sensing Agency, Hyderabad under Department of Space, Government of India. The water year selected was 2005-06. The objective of the study was to derive water spread of Hirakud at various operating levels extending between MDDL and FRL from multiple satellite observations and to derive corresponding contours to estimate live storage capacity between these observed levels. Result of study is presented in Table 6.38.

Table 6.38 : Loss of Storage Capacity

Sl. No.	Res. Elev. (m)	Water spread Area (sq.km)		Storage capacity(M.cum)		Loss in capacity	
		Original (1957)	As per present survey 12/07	Original (1957)	As per present survey 12/07	M.cum	%
1	2	3	4	5	6	7	8
1.	179	-	180.22	-	-	-	-
2.	179.83 MDDL	207.70	203.18	2262.11	776.32	1485.79	65.68
3.	180	282.60	208.04	2312.44	811.28	1501.17	64.92
4.	181	311.70	237.73	2612.04	1034.00	1578.05	60.41
5.	182	340.90	269.29	2940.28	1287.34	1652.94	56.22
6.	183	370.50	302.72	3290.01	1573.19	1716.82	52.18
7.	184	403.30	338.02	3681.26	1893.39	1787.86	48.57
8.	185	436.00	375.19	4102.30	2249.83	1852.47	45.16
9.	186	469.20	414.22	4547.42	2644.37	1903.05	41.85
10.	187	508.00	455.13	5042.61	3078.89	1963.72	38.94
11.	188	546.00	497.90	5569.60	3555.24	2014.26	36.17
12.	189	583.60	542.55	6124.40	4075.31	2049.09	33.46
13.	190	629.10	589.06	6742.75	4640.95	2101.80	31.17
14.	191	676.60	637.44	7397.42	5254.04	2143.38	28.97
15.	192	726.10	687.69	8088.41	5914.45	2171.95	26.85
16.	192.02 FRL	727.30	688.72	8104.99	5930.22	2174.77	26.83
17.	Live storage			5842.88	5153.89	688.99	11.79

(Source : Satellite Data based updation of Elevation - Area - Capacity curves and Sedimentation Assessment of Hirakud Reservoir for 2005-06, (Dec. 2007) - NRSA, Govt. of India, Hyderabad).



Drg. No.6.1 Comparision of original Live storage capacity with NRSA data of 2005-06

6.3.3.1.2 Study of SPARC

Spatial Planning & Analysis Research Centre Pvt. Ltd. (SPARC), Bhubaneswar was entrusted the task of conducting sedimentation study for revision of Area Capacity Curve as a part of Re-assessment of Design energy of 5 nos. of Hydro Electrical Power Stations of Orissa Hydro Power Corporation (OHPC) vide their letter No.17/2006/11620 dt.30.10.2006. SPARC conducted Remote Sensing Survey twice first time in 2002 and second time in 2005. The outcome of the study is given in Table no.6.39.

Table-6.39 Reservoir Live Storage Capacity Estimation from SRS Study 2005

Reservoir Elevation in m.	Reservoir Spread Area (Million m^2)	Capacity Between Elevation (Million m^3)	Cumulative Capacity (Million m^3)
179.83 DSL	179.172	0	0
180.00	183.268	30.44	30.44
181.00	209.099	196.04	226.49
182.00	237.413	223.11	449.59
183.00	268.210	252.66	702.25
184.00	301.489	284.69	986.94
185.00	337.252	319.20	1306.14
186.00	375.497	356.20	1662.34
187.00	416.225	395.69	2058.03
188.00	459.436	437.65	2495.68
189.00	505.129	482.10	2977.78
190.00	553.306	529.03	3506.82
191.00	603.965	578.45	4085.27
192.02 FRL	658.413	646.14	4731.41

* Considering Live Storage Capacity at DSL to be Zero.

The Elevation-Live Storage capacity curve is shown in Fig.6.1

6.3.4 Abstract of Result

A comparative statement of results showing reduction of storages as observed from various studies at different times are presented in Table 6.40.

Table-6.40 : Comparison of results of different silt studies

(capacity & reduction in M.cum.)

Survey	Dead storage capacity	Live storage capacity	Gross storage capacity	Reduction in Storage(%loss)		
				Dead storage	Live storage	Gross storage
Original Project Report (1957)	2262.12	5842.88	8105.00	-	-	-
1st Cycle Hydrographic Survey-1979	1646.455	5287.802	6934.257	615.665 (27.22%)	555.078 (9.50%)	1170.743 (14.44%)
2nd Cycle Hydrographic Survey-1981	1517.22	5109.19	6626.41	744.900 (32.93%)	733.69 (12.56%)	1478.590 (18.24%)
3rd Cycle Hydrographic Survey-1986	1508.229	5105.534	6613.763	753.891 (33.33%)	737.346 (12.62%)	1491.237 (18.40)
Recommedation of Rule curve Revision Committee-1988	1815.301	5376.843	7192.144	446.819 (19.75%)	466.037 (7.98%)	912.856 (11.26%)

4th Cycle Hydrographic Survey-1991	1263.28	4946.80	6210.00	998.840 (44.15%)	896.080 (15.34%)	1895.000 (28.38%)
Remote Sensing survey by CWC-1995	1208.558	4932.883	6141.440	1053.564 (46.57%)	909.997 (15.57%)	1963.560 (24.23%)
Hydrographic Survey-2000	1072.902	4821.893	5894.795	1189.218 (52.57%)	1020.987 (17.47%)	2210.20 (27.27%)
Remote Sensing Survey by SPARC 2002		4953.69	-	-	889.190 (15.22%)	-
Remote Sensing Survey by SPARC-2005	-	4731.41	-	-	1111.470 (19.02%)	-
NRSA Study 2005-06	776.32	5153.89	5930.22	1485.800 (66.12%)	688.990 (11.79%)	2174.780 (26.83%)

6.3.4.1 Future Sediment Projection of Hirakud Reservoir

"With the available study results it is found that the present reduction of live storage of Hirakud reservoir is about 17.16%. A graph of reduction of Live Storage verses the time (year) has been plotted and a trend line for logarithm curve has been fitted. The results of the Rule Curve Revision Committee, 1988, have not been considered as it did not conform to the trend. The same is extrapolated up to the year 2056 (i.e., 100 years after impoundment) and shown in Fig.6.2(a) and Fig.6.2(b). It is found that by 2056 the reduction of live storage would be about 32%, gross storage 62%, if this trend continued. Though the projected figure for the reduction of dead storage touches cent percent by the year 2050, it is practically not feasible in case of Hirakud Reservoir due to provision of the under sluices, through which a part of sediments deposited below the dead storage level automatically gets flushed.

The sill level of the 64 numbers of under sluices is RL.155.55m against the dead storage level reservoir of RL 179.83m. When the sedimentation reserve would tend to be filled up, the silt removing capacity of the sluices would increase many folds as the entire flood discharge would pass over the silted bed. The rate of silting would thereafter reduce rapidly and the live storage may become more than the projected figure. At the end, the dam may not serve flood protection purpose due to reduction of storage, but irrigation and power generation would be there. Ultimately the power house may serve as a run-of-river scheme."

(Source: Reservoir Sedimentation and Desedimentation Measures with Special Reference to Hirakud Reservoir by H.C. Behera and Dr. J. Jena - Development & Management of Water and Energy Resources - CBIP, R&D Session, 4-6 Feb. 2009).

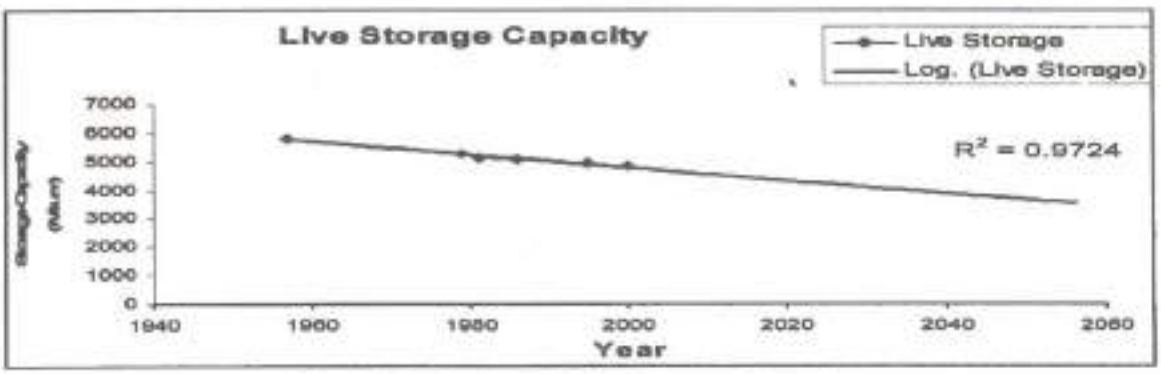


Fig.6.2(a) : Projected Live Storage Capacity

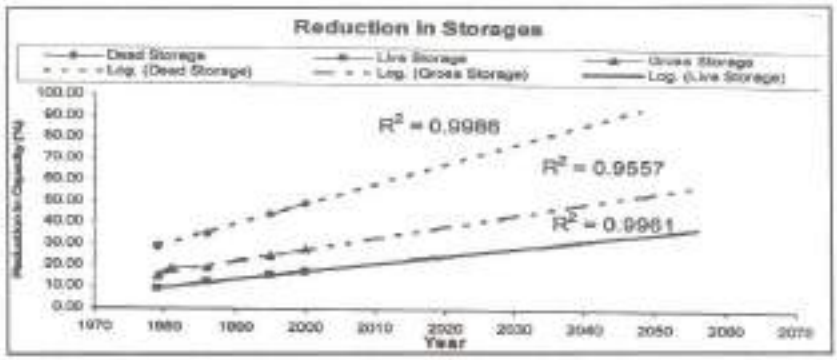


Fig.6.2(b) : Reduction in Reservoir Storages

6.3.5 Conclusion

"Storage reservoirs will continue to play an important role in the future development of water resources in the country. In some of the reservoirs which have been constructed, the rate of sedimentation has been higher than what was considered at the planning stage. Some reservoirs in the world have been silted up so fast that they have become useless. For example the Yasuka Resevoir in Japan has lost 85 percentage capacity in less than 13 years. Many of the reservoirs in India are losing capacity at the rate of 0.2 to 1.0 percent annually. About 40,000 minor tanks in Karnataka have lost more than 50 percent of their capacities. It has, therefore, been considered necessary to take steps to plan the future projects on a sound basis so that the sedimentation of the reservoir's will not reduce the benifits faster than envisaged."

"The surveys conducted during last three decades have indicated that the sedimentation rates in some of the Indian reservoirs are higher than that envisaged at the planning stage.

The variation in actual sedimentation rate with the rate assumed at the time of design is due to the fact that enough reliable data on Indian reservoirs was not available earlier at their planning stage. Further, the earlier assumption that sediment would settle within the dead storage is no longer supported by the experience gained in India as well as other countries .The hydrographic surveys have clearly indicated that the sedimentation takes place not only in dead storage but also in live storage of the reservoirs.

The present design practice (followed progressively since 1965) incorporates that the sedimentation inflow rates be based on the basis of reservoir survey data as well as actual observed sediment inflow data available from key hydrological station/network of CWC. This practice has already been incorporated in the IS: 12182 (1987) "Guidelines for determination of effects of sedimentation on planning and performance of reservoirs", to make this, a national practice.

In recent times there have been apprehension from certain quarters about the higher rates of sedimentation in reservoirs and that they will not last for their planned life. But the analysis of data collected for various reservoirs show that the sedimentation rates are not alarming. Further, it has been experienced that the sedimentation rate in reservoirs is higher during the initial period of their operation and thereafter it falls-off significantly. Even some of the reservoirs having completed their planned life are still continuing to serve and provide substantial benefits. Thus, the apprehension about reduction in the storage of reservoirs due to excessive sedimentation is unfounded.

Since sedimentation study of reservoirs using remote - sensing technique is fast and economical and considering its limitation that sedimentation taking place below' MDDL cannot be measured, it would be appropriate to conduct hydrographic surveys at longer intervals and remote sensing based sedimentation surveys are carried out at shorter intervals to make both surveys complementary to each other.

It is observed that the rate of sedimentation is maximum in the reservoirs in the Himalayan region. Median value of rate of sedimentation observed in this region is 2.11 mm/year (21.1 ha.m. per 100 sq.km./year). In the plains of Indus and Ganga only 0.89 mm/year (8.9 ha.m./100sq.km./year) is observed. The rate of 0.635 mm/year (6.35 ha.m./ 100sq.km.lyear) observed in the east flowing rivers up to Godavari may not be correct since data on only one reservoir is available. Second highest rate of sedimentation is observed in the West flowing rivers beyond Tapi and South Indian rivers (1. 79 mm/ year ie.17.9 ha.m./100sq.km./year). Rate of sedimentation in the reservoirs constructed in the other west flowing rivers is comparatively much less ie. 0.84 mm/year (8.4 ha.m/100 sq.km/year) in the reservoirs on west flowing rivers of Gujurat and Maharashtra up to Narmada and 0.750 mm/year (7.5 ha.m/100sq.km./year) is observed in the decan peninsular region." In case of Hirakud, the rate of sedimentation is of the order of 6.35 ha.m/100 sq.km/year vide Table 6.36.

6.4 Interception in the Upstream of Hirakud Dam :

Mahanadi being an interstate river, the state-wise drainage area has been furnished in Table 1.1. There exists no interstate agreement between co-basin states for sharing water of Mahanadi but such agreements have been concluded between Orissa and undivided Madhya Pradesh for utilising water of Ib, Jonk and Ong which are tributaries of Mahanadi. Table No.6.42 gives brief contents of the agreement. The details of the agreement is appended herewith as Annexure 6.5.

As the Hirakud Dam Project was constructed by Government of India during pre-plan period and there was no such heavy demand for utilising the water resources as it is today, there was no problem. After construction of Hirakud, a good number of projects have been constructed, (Ravisankar, Bango and Hasdeo etc) under construction and proposed in the upstream of dam by Government of Chhatisgarh, (vide Table 6.41) it will have adverse impact on Orissa both from irrigation and flood control point of view. In this regard, Government of Orissa letter dt.7.7.2000 addressed to the Ministry of Water Resources is enclosed as Annexure 6.6. Before it is too late, both the State Governments and Government of India must amicably solve this vexed problem in the interest of the nation as a whole. The abstract of such structures have been furnished under Sec.6.1.1.3, Table 6.4.

Interception by completed projects is 25223 sq.km. which is 30.24% of total catchment of 83400 sq.kms. upto Hirakud Dam. Out of this 16922 sq.km. (20.29%) is by storage projects affecting total yield to Hirakud reservoir.

On completion of the projects under construction and in planning stage, the interception will be around 45.5% of Hirakud catchment, out of which 28.94% will be by storage projects.

The total storage capacity works out to around 5 M.Ac.ft. which affects specifically the runoff of the base flow to Hirakud dam in months of September, October and onwards upto mid June.

In addition to the interruption stated above there are nearly 1431 nos. (919 nos. completed and 512 nos. under construction) minor Irrigation projects in Mahanadi basin in M.P. Also further 937 nos. of projects are in planning stage). Though interception of minor Irrigation projects are individually small, considering the total numbers the interception is substantial to affect runoff to Hirakud reservoir. Agreement should be made with Government of Chhatisgarh at an early date to limit interception of Mahanadi catchment.

Table 6.41 : List of Projects in Chhatisgarh in Mahanadi Basin

Note : 1. List based on data collected by personal approach.
2. Includes details of major & medium project only.

A. Completed project.

Sl. No.	Name of the Projects	Year of completion	Sub-basin	Catchment area		Live storage capacity M.Ac.ft.	Spillway flood discharge in lakh cusecs
				sq.miles	sq.km.		
1	2	3	4	5	6	7	8
(1)	**Dams**						
	a) Major Projects						
	1. Tandula Tank	1920	Seonath	319.4	827	0.246	0.697
	2. Murumsily Dam	1923	Mahanadi	187.0	484	0.132	0.500
	3. Dhudhwa Dam	1962	Mahanadi	240.0	622	0.161	0.795
	4. Siksar Dam	1976	Mahanadi	192.0	497	0.161	1.434
	5. Ravi Sankar Dam	1979	Mahanadi	1417.0	3670	0.621	8.300
	6. Kodamallah	1983	Mahanadi	122.0	316	0.121	0.220
	7. Bango Dam	1994	Hasdeo	2602.0	6730	2.470	7.682
	8. Sondur	-	Mahanadi	198.0	513	0.146	2.016
				5277.4	**13659**	**4.058**	**21.644**
	b) Medium Projects						
	1. Khapari Projects	1911	Seonath	15.4	39.90	0.009	0.119
	2. Madyan Tank	1930	Seonath	11.3	29.26	0.009	0.932
	3. Godnil Tank	1956	Seonath	75.0	194.175	0.079	0.232
	4. Dhara Tank	1961	Seonath	5.0	13.09	0.004	0.026
	5. Saroda Tank	1964	Seonath (Sakri)	36.37	94.21	0.024	0.097
	6. Kodamala Tank	1968	Mahanadi	17.5	45.18	0.013	n.a.
	7. Surhi Tank	1969	Seonath (Dottu)	37.0	95.46	0.018	0.119
	8. Kharkhara Project	1974	Seonath	143.5	371.52	0.015	0.400
	9. Mayana Tank	1977	Mahanadi	5.7	14.835	0.005	0.037
	10. Ruse Tank	1977	Seonath	16.5	42.72	0.007	0.051'
	11. Putka Tank	1978	Mahanadi	6.0	15.53	0.005	n.a.
	12. Kinkari Tank	1978	Mahanadi	16.6	42.90	0.012	-do-
	13. Ghonge Reservoir	1982	Seonath (Maniari)	111.0	287.38	0.024	-do-
	14. Khamar Pakut Tank	1882	Ib	19.5	50.53	0.015	-do-
	15. Naisajaon Tank	n.a.	10.7	27.83	n.a.	n.a.	
	16. Kurung Tank (Kharung)	1931	Seonath (Kurung)	360.0	932.0	0.156	0.351
	17. Maniari Tank	1931	Seonath	310.0	803.00	0.120	0.348
	18. Pindraban Tank	n.a.	Seonath	20.18	25.27	0.009	0.084
	19. Kumbhari Tank	-do-	Seonath (Arpa)	20.18	30.34	0.009	0.089
	20. Keswa Tank	-do-	Mahanadi (Sukha)	11.90	30.81	0.011	0.0035
	21. Nawagaon Tank	-do-	Seonath	10.75	27.83	0.003	0.080
	22. Maroda Tank	-do-	-do-	n.a.	n.a.	n.a.	n.a.
				1260.08	**3262.70**	**0.547**	**3.000**

A-2	**Barrages & Weirs**						
	1. Jonk Diversion	-	Mahanadi (Jonk)	1223	3131	-	0.450
	2. Ghongha Weir	1901	Seonath	92	235	-	0.450
	3. Rudri Weir	1937	Mahanadi	1430	3703	-	-
	4. Hasdeo Barrage	1967	Hasdeo	2900	7511 Area includes catchment of Bango Dam 6730 sqm.	-	-
	5. Jaspurnagar Weir	n.a.	Ib	174	451	-	-
	6. Mahanadi Barrage	n.a.	Mahanadi	1417	3670 Area includes catchment of Ravi Sankar Dam 3670	-	-
	7. Jhironadi Diversion schemes			51	132	-	-
				7287	18701		
	Deduct interception by Ravisankar Dam & Bango Dam U/S of Barrages			4019	10701		
				3268	8301		
	8. Shivnath Diversion	n.a.	Seonath	198	505.95	-	1.520
	9. Bilaspur Diversion	n.a.	Seonath (Appa)	n.a.	n.a.	-	-
	10. Mond Diversion	n.a.	Mond	-do-	-do-	-	
				3466	**8807**		
B.	**Dams under construction**						
a)	**Medium Projects**						
	1. Balar Tank (H/W completed)	n.a.	Mahanadi (Jonk)	32	81.42	0.030	0.213
	2. Chirpani Tank (H/W completed)	n.a.	Seonath (Sakri)	64	163.30	0.041	0.530
	3. Matiamoti Tank	n.a.	Seonath	32	82.38	0.021	0.256
	4. Piparjya Tank (H/W completed)	n.a.	Seonath	56	143.60	0.033	0.286
				184	**470.70**	**0.125**	**1.285**
b)	**Barrages and Weirs**						
	1. Mond Projects	n.a.	Mond	**2000**	**5180**	-	-
c)	**Proposed Projects**						
	a) Dams						
	1. Piari High Dam	-	Mahanadi	1180	3056	0.316	-
	2. Arpa Dam	-	Seonath	664	1720	-	-
	3. Hamp Dam	-	Seonath	180	466	-	-
	4. Gondali Project	-	Seonath	75	194	0.078	-
	5. Chando Project	-	Seonath	506	1310	-	-
			Total	**2605**	**6746**	**0.394**	
	(2)Barrages and Weirs						
	1. Agar Weir	-	Seonath	126	326		
			Total	**126**	**326**		

(Source: Data on Flood Control operation of Hirakud Dam, Govt. of Orissa, Sept. 1997 Pg.196-198).

Table - 6.42 : Inter-State Agreement between Odisha & M.P. (now Chhatisgarh)

Sl. No.	State Agrement made with	Name of the Project	River	Date of Agreement	Agreement Contents (Agreement was reached between C.M.s of both the States)
1	2	3	4	5	6
1.	Madhya Pradesh	Ib dam Project	Ib	28.4.83	M.P. will spare 25% runoff from the catchment from the area inside M.P. Orissa will fix the F.R.L. at 272.50 M. Land in M.P. will be acquired upto R.L. 273. M. for back water impact. Incase of damage caused to the area laying above R.L. 273 due to floods, Orissa will pay compensation. M.P. may generate hydro power at the head works of Ib Project at its own cost without paying for the cost of storage.
2.	Madhya Pradesh	Kumalla Project	Kumalla	28.4.83	This will be executed as a joint scheme by both the states. Cost of head works will be shared in proportion to the ultimate irrigation benefits derived by each state.
3.	Madhya Pradesh	Upper Jonk	Jonk	28.4.83	Orissa will supply water through the left bank canal of the project at Orissa-M.P. border to irrigate 2000 acres in M.P. on charges to be determined. M.P. may meet its requirement by utilising the waters of the tributaries of Jonk river Upstream of Lower Jonk Project draining that area subject to a maximum utilisation of 40 sq.miles of catchment.
4.	Madhya Pradesh	Lower Jonk Project	Jonk	28.4.83	It will be jointly executed near Girina. The cost of the dam, reservoirs & available runoff will be shared in the ratio of 30% by Orissa & 70% by M.P.
5.	Madhya Pradesh	Ong Project	Ong	28.4.83	M.P. will spare runoff from its catchment area within M.P. for use by Orissa. Orissa will fix the F.R.L. at 219 m.
6.	Madhya Pradesh	Jeera Project	Jeera	28.4.83	M.P. will spare run off from 14.25 sq.miles of its catchment for use by Orissa with F.R.L. 697.5 ft.

(Source: 3rd spiral study of Mahanadi Basin, OWPO, Department of Water Resources, Govt. of Orissa, Annexure-3.19).

Orissa-Madhya Pradesh Inter-State Matters - Irrigation and Power Projects - Memorandum of Agreement entered into between the state of Madhya Pradesh and Orissa on 28.4.1983 at Bhubaneswar.

In pursuance of discussion held on 2nd April, 1993 at Bhubaneswar attended by Sri Arjun Singh, Chief Minister, Madhya Pradesh, Sri Janaki Ballav Pattnaik, Chief Minister Orissa and Niranjan Pattnaik, Minister of State for irrigation and Power, Orissa -along with the Officials of the two State (as per list enclosed), the following agreement was reached.

1. Ib Project (Orissa Project)

Madhya Pradesh agrees to spare 25 percent run off from the catchment area drained by Ib River and lying in its territory under the proposed Ib dam project (Approximately Lat 23° - 12' N, Long 84° - 06' E). Orissa agrees to fix full reservoir level of Ib dam at R.L 272.50m. The land in Madhya Pradesh would, however, be acquired up to R.L 273.0 meters to allow for backwater impact. In exceptional cases, if areas lying above R.L 273.0, meters in the foreshore area of the reservoir in Madhya Pradesh are damaged due to floods, the damage being solely attributable to the construction of Ib dam, suitable compensation will be paid by Orissa. Madhya Pradesh may generate hydro-power at the headworks of Ib project entirely at its own cost without paying for the cost of storage, but bearing the full cost of all additional civil and electrical works required for generation of hydro-power. Madhya Pradesh will make its own arrangement for operation and transmission of power generated. Project report for power part will be prepared by Madhya Pradesh for which Orissa will extend necessary co-operation. However, the regulation of Ib reservoir shall be done by Orissa as per operational and irrigation requirements.

2. Sapnai Project (Madhya Pradesh)

Orissa agrees to spare 70 percent of run off from its catchment area for use in Madhya Pradesh at Sapnai dam (Approximately Lat 21° - 53 - 45" N, Long 83° - 30' -05" E). Madhya Pradesh may formulate Sapnai Project leaving an upstream reserve to the extent of 30 percent of run off from. Orissa catchment for use in Orissa State. Submergence due to project is limited to Madhya Pradesh territory only.

3. Kurnala Project (Joint Project)

Madhya Pradesh and Orissa agree to implement the Kurnala Project as a joint irrigation scheme, where the catchment area of Kurnala is about 45 sq.miles (Approximate Lat 21° - 55' N, Long 83° - 33' E) to irrigate about 3000 acres in Madhya Pradesh and about 9000 acres in Orissa. The cost of head work of the project shall be shared in proportion of the ultimate irrigation benefits derived by each state.

4. Upper Jonk Project (Orissa Project)

Orissa agrees to supply water through left bank canal of upper Jonk Project at Orissa- Madhya Pradesh border to support irrigation in an area of 2000 acres in Madhya Pradesh territory on charges to be determined. The requirement of rest of the areas in Madhya Pradesh may be met by Madhya Pradesh by utilising the waters of the tributaries of Jonk river upstream of lower Jonk Project (Orissa) draining that area subject to a maximum utilization of 40 square miles of catchment. Orissa agrees to consider request by Madhya Pradesh for submerging land for above project(s) if any.

5. Lower Jonk Project (Joint Project)

Orissa and Madhya Pradesh agree that a dam across Jonk river near Girina (Approximate Lat 21° - 10' N,. Long 82° - 37'-30" E) should be constructed as a Joint Project and share the cost of

dam, reservoir and available run off in the ratio of 30 percent (Orissa) and 70 percent (Madhya Pradesh). The cost of head regulator and canal system will be borne by the respective beneficiary states. The surveys and investigation of Girina dam will be carried out by Madhya Pradesh.

6. Ong Project (Orissa Project)

Madhya Pradesh agrees to spare 20 percent of runoff from its catchment area lying in Ong Sub-basin for use by Orissa at the proposed dam site across Ong river near its confluence with Surangi river (Approximate Lat 21° - 05' - 29" N, Long 83° - 03' -06" E). Orissa agrees to fix the Full Reservoir Level of Ong reservoir at R.L 219.0 meters. Orissa may however fix the full reservoir level at R.L 220 meters provided the reservoir does not submerge existing irrigated area in Madhya Pradesh for which necessary field verification will be carried out jointly.

7. Jira Project (Orissa Project)

Madhya Pradesh agrees to spare run off from 14.25 sq. miles of its catchment lying in Jira sub basin for use by Orissa at proposed Jira dam project (Approximate Lat 21 °-23' N, Long 83° - 26' E) with full reservoir level fixed at R.L 697.5 feet.

8. Sahajbahal Project (Joint Project)

Orissa and Madhya Pradesh agree to implement Sahajbahal Project as a joint irrigation scheme where the catchment area of the river is about 22 sq. miles (Approximate Lat 20° -21'_ 15" N, Long 83° - 21'_15" E), in Madhya Pradesh. The cost of joint works and irrigation benefit shall be shared in the ratio of 70% (Madhya Pradesh) and 30% (Orissa), Surveys and investigations of this joint project will be carried out by Madhya Pradesh.

9. Lower Kolab Project (Joint Project)

Orissa and Madhya Pradesh agree to re-investigate the lower Kolab Project with alternative possibilities of irrigation and hydro- power generation. If found feasible, the project shall be taken up' as a joint project. Survey and investigation will be carried out by Orissa in consultation with Madhya Pradesh and the first preliminary report will be finalised by June, 1984.

10. Back water studies for Hirakud Dam

Both the States agree to carry out fresh back water studies for the Hirakud reservoir with a view to ascertaining the extent of submergence caused by Raigarh District of Madhya Pradesh at the times of high flood. The Central Water Power Research Station, Pune may, if necessary, be associated with these studies.

11. Establishment of Joint Control Board

Madhya Pradesh & Orissa agree to establish a joint control Board to review the progress, from time to time of survey, investigation, planning, execution and operation of Joint Inter State Irrigation and or Power Project (s) and to discuss and resolve any issues.

12. Supply of Power:

It is agreed that supply of 5 MW powers which was committed by the Govt. of Orissa previously will be resumed. As the M.P Electricity Board is keen to have power from Hirakud, this will be supplied at Hirakud Bus bar. The cost of power will be the cost of generation at Hirakud Power House. The cost would be reviewed annually by the representative of the two Boards. The question of payment of Electricity Duty by M.P Electricity Board for future supply by Orissa State Electricity Board of 5 MW power would be referred to the Department of Power, Govt. of India for their guide lines.

Sd/-	Sd/-
(J.B. Patnaik)	(Arjun Singh)
Chief Minister, Orissa	Chief Minister, Madhya Pradesh
28th Apri, 1983	28th Apri, 1983

(Source: Documentation of Interstate Agreements, WALMI, Odisha, 1993 Pg.95-97).

Annexure-6.6

NAVEEN PATNAIK
Chief Minister,
Orissa

D.O.No.UM35/2000/444-CM
Bhubaneswar
Date : 7.7.2000

Dear Arjun Babu,

Sub: Problems of Hirakud Multipurpose Project.

Hirakud Dam was completed in the year 1957 by Government of India. The catchment area of the Dam spreads 83,398 sq.km. out of which about 10% is situated in the State of Orissa and the balance 90% in the State of Madhya Pradesh. Since the Dam was constructed purely by Government of India, there was no bipartite agreement between the Government of Orissa and Government of Madhya Pradesh at that point of time.

Before completion of Hirakud Dam, 7 Irrigation Projects were existing on the river Mahanadi within Madhya Pradesh having a catchment area of 3310.26 sq.km. and therefore, did not have much impact on the Hirakud Reservoir. In the meanwhile, Government of Madhya Pradesh have constructed many Irrigation Projects intercepting the flow of water to the Hirakud Reservoir. A list of such Projects indicating their catchment area, live storage capacity, spillway flood discharge informations are indicated in the Annexure 1 to this letter. By now, 8 Major Irrigation Projects, 22 Medium Irrigation Projects, 10 Barrages/Weirs have already been completed by Government of Madhya Pradesh. In addition, 4 Medium Irrigation Projects, having catchment area of 470.70 sq.km. are under construction and we understand that there are proposals for construction of 5 new Dams and 1 Barrage having an aggregate catchment area of 11,926 sq.kms.

These Projects will certainly affect the base flow to the Hirakud Reservoir in the months of September and October and particularly during lean months i.e. from November to June. Such a situation will create operational problems for power generation and will also substantially effect the proposed Hydro Power Project at Sindol and at other places down stream of Mahanadi river in the State of Orissa.

Hirakud Dam provides irrigation to 1,55,635 ha. during Khariff and 1,08,385. Ha during Rabi in the districts of Sambalpur, Bargarh, Subarnapur and Bolangir. The installed capacity for Hydro Power Generation of this project is 307.5 M.W. Besides, it provides flood protection to 9500 sq.km and irrigation to 3,23000 Ha. in the Mahanadi delta area of Cuttack and Puri districts.

Government of Orissa therefore, make the following suggestions before Government of India for co-ordinating the matter and issuing appropriate directions to M.P. Government :

(A) M.P. Government should allow a flow of 2.0 M.Acft during non-monsoon period for irrigation and generation of power.

(B) M.P. Government should make arrangements to intimate in advance if there is contemplation to release 1.00 lakh cusecs of water or more to the Hirakud Dam indicating the exact quantity and the time of release so that appropriate action can be taken at the dam site.

(C) Till construction of Ib Dam Project is completed, M.P. Government may release water in Ib river for drinking water purpose in non-monsoon period.

(D) Any new projects proposed to be taken up by M.P. Government should be cleared by Government of India only after concurrence of Government of Orissa.

The riparian rights of the State of Orissa should be protected through Inter State agreement on allocation of water between the State of Madhya Pradesh and Orissa.

Yours sincerely,
(Naveen Patnaik)

Shri Arjun Sethi
Minister of Water Resources, Government of India,

6.5 Waterlogging in Command Area

6.5.1 Introduction

Waterlogging is the "state of land in which the subsoil watertable is located at or near the surface with the result that the yeild of crops commonly grown on it is reduced well below the normal for the land or if the land is not cultivated, it can not be put to its normal use because of high subsoil water table." (Vide Dictionary on Irrigation & Drainage by ICID-CIID, 1996 Pg.602).

In waterlogged area 'water stands near, at, or about the land surface so that the roots of all plants except, hydrophytes are drowned and the plants die' (vide NCIWRD Vol.I, Sept.99, Pg.492).

6.5.1.1 Waterlogging and Salinity/Alkalinity

"Irrigation is not an unmixed blessing, particularly where the farmers have a tendency to over-irrigate through excessive water applications and where improper water management practices prevail. Inadequate drainage often causes waterlogging problems, whether the lands are irrigated or not. An agricultural land is said to be waterlogged when the soil pores in the crop root zone get saturated with water. Problems of waterlogging and soil salinity/alkalinity have been universally recognized as predominantly adverse effects of unscientific irrigation. Even well-managed irrigation projects could be affected in varying degrees, depending upon the manner of water distribution, soil and crop types, and the water management practices. Waterlogging owing to periodic flooding, over-flow by run-off, seepage and impeded drainage, are common. Other activities in the project area which interfere with the drainage flow could aggravate the situation."

(Refer : Bhakra-Nangal Project, Socio-economic and Environmental Impacts by R. Rangachari, Oxford University Press 2006, Pg.88).

Plant roots require a favorable environment to be able to extract water and soluble nutrients. Excess of water or of salts into root zone or at the land surface does not allow the plant roots to function normally. As a result, the plant grown and yield are adversely affected. In the extreme cases of water logging and salinity, the seeds may not germinate and the plants may wilt permanently. This results in loss of agricultural production. Adoption of agricultural drainage intime prevents physical and chemical degradation of soil.

6.5.1.2 Causes of Waterlogging and Salinity

The causes are either natural or man-made, main cause being supply and application of irrigation water to crops much beyond the evapo-transpiration (ET) requirement. Other causes include:

- Drainage congestion due to development activities like roads, bridges, canals, flood levees, railway embankments, buildings causing impediment to natural drainage.
- Lack of conjunctive use of canal water and groundwater
- Poor on-farm water management.
- Inadequate drainage system and deficient maintenance.

The salinity/ alkalinity may occur due to one or a combination of following factors:

- Accumulation of salts on surface (from soils or ground water).
- Upward movement of salts with capillary rise from lower layers.
- Inadequate dose of freshwater, to counter upward movement of salts.

Extent of Waterlogging and Salinisation

Water table rises steadily since advent of canal irrigation of the last century, resulting in waterlogging and salinity. The problem is acute in several irrigation schemes. The areas most affected are in the Indo-Gangetic plains. Assessment of such affected area has been carried out at different times. Reported figures vary widely as each agency adopted own norms for defining waterlogging and salinity / alkalinity. Some reports also included un-irrigated and / or coastal saline areas in their assessments.

The Working Group set up by Ministry of Water Resources (MOWR) in 1991 reported an area of 2.46 Mha as water logged and 3.30 Mha as salt affected in commands of major and medium irrigation schemes. The affected area was due to rise in water table. It did not include area affected due to surface water stagnation. The extents of water logging as assessed and reported by different agencies are given in Table 6.43.

Table 6.43 : Extent of Area Affected by Waterlogging (Area in Th. ha.)

State	National Commission (1972) (Both under irrigated and unirrigated areas)	National Commission on agriculture 1976 (For irrigated & un-irrigated area)	Ministry of Agriculture 1984-85 (For Irrigated & un-irrigated areas)	Working Group of MoWR-1991 Under Irrigated Commands)	CWC-based on Remote sensing Study RRSC Jodhpur 2003-05	
					Surface inundated water logged area*	Salt affected area
Andhra Pradesh	N.R.	339.00	339.00	266.40	28.267	12.933
Assam	N.R.	-	450.00	-	Covered in NE States	
Bihar	117.00	707.00	707.00	619.70	627.88	156.887
Goa	-	-	-	-	Nil	Nil
Gujarat	N.R.	484.00	484.00	172.59	265.26	307.32
Haryana	650.00	620.00	620.00	249.00	16.459	19.393
Himachal Pradesh	-	-	-	-	0.261	Nil
Jammu & Kashmir	N.R.	10.00	10.00	1.50	3.97	0.075
Jharkhand	-	-	-	-	Nil	Nil
Karnataka	7.00	10.00	10.00	24.54	-	5.78
Kerala	N.R.	61.00	61.00	11.61	12.33	3.997
Madhya Pradesh	57.00	57.00	57.00	73.12	0.54	4.41
Chhatisgarh	-	-	-	-	Nil	Nil
Maharashtra	28.00	111.00	111.00	15.35	426.40	34.54
Orissa	N.R.	60.00	60.00	196.26	85.99	34.78
Punjab	1090.00	1090.00	1090.00	200.00	34.97	131.998
Rajasthan	348.00	348.00	348.00	179.50	8.409	2.053
Tamil Nadu	N.R.	18.00	18.00	16.19	32.518	30.696
Uttar Pradesh	810.00	810.00	1980.00	430.00	126.68	283.146
Uttarakhand	-	-	-	-	0.225	0.013
West Bengal	1850.00	1850.00	2180.00	-	46.397	6.465
Delhi	N.R.	01.00	01.00	-	Study not taken up	
N.E. States	-			NR		
U.T. (Puducheri)	-					
Total (in Th.ha)	**4840.00**	**5986.00**	**8526.00**	**2450.00**	**1719.297**	**1034.541**

Blanks / NR - Not reported by concerned States* The area reported by satellite imageries that is perennial (May) and seasonal (November) Surface inundated area

Another assessment appeared in ICAR publication (March 1997) titled "Vision 2020 - CSSRI Perspective Plan". The state-wise affected area was different, but total affected area was 2.19 Mha which almost tallied. To reconcile the information, the National Commission for Integrated Water Resources Development (1999) recommended that uniform definitions considering diversity in soil characteristics and dominant crops grown be developed and criteria based on actual field work and soil tests, loss of productivity etc, be evolved. The CWC later undertook a study "Mapping of waterlogging and affected areas in all the major and medium irrigation commands of the country using satellite remote sensing". The remote sensing based mapping of waterlogging refers to both surface ponding (excluding ponds, lakes, reservoirs, canals and river) and saturated surface soil visible on the satellite image. The survey has classified water-logging situations into two categories: seasonal

waterlogged areas, which exhibit waterlogging only in one season, i.e. in April or November, and perennial water-logging which persists in all seasons. However, salinity is not identified on satellite data. Latest information indicates that India's water-logged area is about 4.96 Mha, comprising 2.66 Mha in canal commands and 2.3 Mha outside. Similarly, the salt-affected area is around 6.73 Mha, comprising 2.96 Mha as saline and 3.77 Mha as sodic. Severely water logged saline soils occur in about 2 Mha area in the arid/semi-arid parts in the north western states of Haryana, Punjab, Rajasthan and Gujarat. Similarly more than I Mha salt affected soils each in coastal and black cotton heavy soils (vertisol) regions of India have specific moisture stress and salinity problems which need special preventive and curative interventions.

(Source: Water Resources Development in India 2009 by INCID, Pg.126-128).

6.5.1.3 Tolerance of crops to Waterlogging

Tolerance to waterlogging varies from crop to crop. Wheat and sugarcane are affected when watertable is within 0.6m (2 ft). Maize, Bajra and cotton are sensitive to watertable within 1.2m and grain and barley within 0.9m. where as for rice it is zero. (Refer 'Waterlogging and its remedy by conjunctive use' - Edited by D.K. Chadha and S.P. Sinha Ray, CGWB Pg.7).

CGWB has suggested certain norms for waterlogged areas. A low water table at pre-monsoon is essential to avoid salinity build up in the root zone depth and successful crop production of kharif crops. Equally important in an optimal low water table in post monsoon keeping in view timely plantation and successful crop yield of Rabi crops. Following norms are suggested for the classification of different categories waterlogged areas.

Table 6.44 : Classification of waterlogged area

Classification	Depth of Water table from Soil Surface (m)	
	Premonsoon (April-May)	Post-monsoon (Oct-Nov)
Waterlogged area	Within 2 m	Within 1m
Critical area for waterlogging	2-3	1-2
Potential area for waterlogging	3-5	2-3
Safe Area	Above 5m	Above 3m

(Source: Water logging and its remedy by conjunctive use, CGWB Pg.9)

6.5.2 Study of Waterlogging in Hirakud Command by CGWB & NRSA

6.5.2.1 Study carried out by CGWB

The design CCA of Hirakud command is 159093 ha and present irrigation intensity is 170% (approx.) (100% in Kharif and 70% in Rabi). The study area is bounded by North Latitude 20° 53 to 21 ° 36' and East Longitude 83° 25' to 84° 10'. The Hirakud command area is covered by thirteen blocks of the Bolangir and Samablpur dist., physiographycially the area can be divided into high, middle and low lands. The surface drainage of the command area is mainly controlled by the Mahanadi river and its tributaries. The area has been divided into four sectors with ridge canals and major rivers, viz. Mahanadi and Ong as boundaries of the sectors in order to determine and represent the different components of the study into smaller unit. The average annual rainfall in the area is 1169.2 mm.

6.5.2.1.1 Problems in the area

Based on the monthly depth to water level data of nearly 130 monitoring wells (Dug Wells), it has been found that water logging conditions (DTW <2m) exist in an area of 174 sq.km during Pre-monsoon (May'94) and it was 1494 sq.km during Post-monsoon period (Nov'94). The water logged

areas under different depth to water levels on monthly basis is provided' in Table 6.45. Topographic set up, unlined canals, over irrigation and predominant paddy cultivation during kharif and rabi are mainly responsible for water logging conditions in the command area.

Table-6.45: Extent of Water logging and Prone to Water logging areas in Hirakud Command (Sq.Km)

Depth to Water Level (m bgl)	June 1993	July 1993	Aug 1993	Sept 1993	Oct 1993	Nov 1993	Dec 1993
< 1	16.65	100.00	534.00	1910.20	462.68	104.14	7.36
1 - 2	128.86	1396.00	1119.20	284.00	1655.32	1404.50	295.97
2 - 3	1211.35	194.67	100.00	8.79	80.17	523.00	1545.00
Depth to Water Level (m bgl)	Jan 1994	Feb 1994	Mar 1994	Apr 1994	Mey 1994	Jun 1994	July 1994
< 1	6.17	47.00	51.00	44.12	11.00	96.44	120.00
1 - 2	238.28	257.00	213.00	193.12	152.00	194.36	1494.00
2 - 3	1062.84	1344.64	1479.62	1399.00	1097.00	1413.44	594.00

The long term behaviour of water levels (1989-96) in majority of wells within the command area indicates a rising trend.

6.5.2.1.2 Hydrogeology

The weathered residuum (average thickness 15m) and fracture zones form the principal repository of ground water which occur under phreatic condition in weathered residuum and under semi confined to confined conditions in the fracture zones. Ground Water exploration to a depth of 200m has revealed the existence of 3 to 4 water bearing fracture zones with cumulative discharges varying from 0.88 to 11 lps. Aquifer parameters of the phreatic aquifer reveals that specific yield varies from 0.7 to 11 lps average being 5 lps. The specific capacity value ranges from 2.317 lpm per meter of drawn down to 142.91pm per mt of draw down. Permeability ranges from 0.472 m/day to 20.19 m/day average being 2.5 m/day. The discharge of deeper borewells tapping the fractured aquifer varies from 2.7 to 10 lps for drawdown varying from 3.89 to 21 m. The transmissivity values (T) ranges from 5.8 m^2/d to 125 m^2/d. Similarly the storage coefficient (S) ranges from 1.96 x 10-4 to 6.9 X 10-4.

Depth to Water Level :

Depth to water level monitored through 129 key wells varied from 0.8 to 9.7 m below ground level (bgl) during pre-monsoon period and 0.3 to 4.03 m bgl during post-monsoon period. During the month of August major part of the command area remains within a depth to water level range of zero to 2m bgl. Seasonal fluctuation of water level in major parts of the command is around 2 m water table elevation varies from 120 to 170 m above MSL in pre-monsoon and 130 to 175 m amsl in post monsoon. The ground water flow direction in general is towards southeast .Hydraulic gradients varied from 1/300 to 1/500. Rivers of the areas are effluent in nature.

6.5.2.1.3 Availability And Demand Of Water

Ground Water resource of the command area has been assessed on seasonal and monthly basis. As per GEC norms, 1984, the utilisable ground water resources during Kharif season have been

estimated as 508.44 Mcum and net draft through existing structure worked out to be 2.35 Mcum leaving a balance of 506.09 Mcum for further development. Similarly utilisable ground water resources during Rabi season have been estimated on 764.44 Mcum with a net draft of 4.35 Mcum leaving a balance of 760.09 MCM for future development.

Availability of surface water during the Kharif season have been estimated as 1360.8 M.cum and during the Rabi season it is 1498.8 Mcum. The monthly canal water used for irrigation during 1991-92 varies from 45.89 Mcum in November to 339.21 Mcum in October.

The Projected annual domestic water demand for the year 2025 was computed as 49.89 Mcum, similarly the projected water demand for industrial purposes have been computed as 261 Mcum. A linear programming model has been formulated to optimise and allocate the surface and ground water as per the cropping pattern for meeting the irrigation demand.

6.5.2.1.4 Conjunctive Use Planning

During the Kharif season paddy coverage is 98% and sugarcane is 2%. During Rabi season, five major crops like paddy, pulses, oil seeds, vegetable and sugarcane have been considered. Demand of water for 200% irrigation intensity can be met from surface water (90%) and ground water (10%) for both the seasons. The surface water irrigation being cheaper an attempt has been made to determine optimal cropping pattern for the maximum use of available surface water in conjunction with ground water to get maximum return. Various possible conjunctive use strategies have been tested with the ground water simulation model and it has been found that the above scenario of 90% surface water and 10% ground water use appears to be the most viable one. In Hirakud command area development of ground water is feasible through dug wells and borewells. Dugwells are the most suitable ground water structure in the area. For meeting the additional demand of water a total of 17,526 dug wells are required which should be installed in the identified area within a period of two years. The simulation studies also indicate that water logging reduces with increase in the use of ground water. The financial analysis of the development plan show that with an additional investment of Rs.953.99 million the B.C. ratio worked out to 1.66 and the internal rate of return (IRR) of 55%.

(Source: Waterlogging and its remedy by conjunctive use - Edited by D.K. Chadha & S.P. Sinha Ray, CGWB Pg.38-40).

6.5.2.2 Study by NRSA, Hyderabad

The Water Resources Division of Natioal Remote Sensing Agency was entrusted to evaluate the performance of Hirakud command area ending with Rabi 2001-02 using Satellite Remote Sensing Technique during 2002. The report was received in April 2004 in which one of the objectives was to identify the status and extent of surface water logging and salt affected soils. Section-wise waterlogged areas of the project is furnished in Table 6.46.

Table 6.46 : Section-wise waterlogged areas in Hirakud irrigation project

	Name of Division/Sub-division	Area (ha)
A.	**Sambalpur Division**	
	Sason sub-division	
	Sason	13.08
	Upper Huma	5.53
	Sub-division Total	18.61
	Sambalpur sub-division	
	Senhapali	3.46
	Godbagha sub-division	
	Gosala	11.46
	Larasara	55.36
	Kadobahal	14.44
	Chiplima	10.80
	Sub-division Total	92.06

	Attabira sub-division	
	Lachida	26.51
	Attabira	19.18
	Remunda	45.07
	Papanga	9.11
	Sub-division Total	99.87
B.	**Bargarh Division**	
	Bargarh sub-division	
	Bargarh	79.62
	Adgaon	19.38
	Kamgaon	114.22
	Behera	27.77
	Sub-division Total	240.99
	Barpali sub-division	
	Barpali	12.94
	Resam	29.80
	Retamunda	5.62
	Sukha	10.27
	Sub-divsion Total	58.63
	Rampur sub-division	
	Barkarley	20.25
	Bhimtikra	26.29
	Andharibanji	7.64
	Koibahal	24.61
	Sub-division Total	78.79
	Total Area	**592.41**

(Source: Performance Evaluation of Hirakud Command Area using Satellite Remote Sensing Technique by NRSA, Hyderabad April 2004).

6.5.3 Conclusion

From the results of study reported earlier, the following inference can be drawn, in respect of Hirakud Command.

i) There are 3 to 4 water bearing fracture zones within 200m depth with cumulative discharge varying from 0.88 to 11 lps.

- Specific yield varies from 0.7 to 11 lps.
- Specific capacity value ranges from 2.317 lpm per meter of draw down to 142.9 lpm/m of drawn down.
- Permeability ranges from 0.472 m/day to 20.19 m/day with average of 2.5 m/day.

ii) The depth of water level below ground level varies from 0.8 to 9.7m in pre-monsoon period and 0.3 to 4.03 m in post-monsoon period.

iii) The direction of ground water flow is generally towards south-east.

iv) Hydraulic gradient of ground water flow varies from 1/300 to 1/500 and rivers are affluent.

v) The availability of utilisable ground water, presently used and balance available for future use are given below in M.cum (Refer sec.6.5.2.1.3).

Purpose	Utilisable	Used	Balance (M.cum)
During Kharif	508.44	2.35	506.09
During Rabi	764.44	4.35	760.09
Domestic by 2025)	-	49.89	
Industrial use	-	261.00	
Total	**1272.88**	**317.59**	**955.29**

As can be visualized, even after meeting demand for domestic and industrial uses (projected for 2025) there is surplus of 955.29 M.cum of ground water for utilisation which can be utilised in conjunction for irrigation with surface water, extension of irrigated area etc.

This will reduce water logging and salinity as well.

- For conjunctive use it has been found that it is most viable to use 90% of surface water with 10% of ground water.
- For meeting additional demand of ground water 17526 nos. of dug wells are required.
- It has been assessed that in Hirakud Command the surface water available during Kharifl is 1360.8 M.cusecs and during Rabi i.e. 1494.8 Mcum. Not all are used for irrigation & other purposes.
- From the long term observation of water levels in 129 dug wells in Hirakud command (1989-96), majority of walls indicate rising trend in water levels.
- The minimum and maximum water logged areas (in sq.km) in Hirakud command are indicated for 1993-94 water year (vide Table 6.45).

Ground water depth below G.L.(m)	Minimum water logged area (sq.km)	Maximum water logged area (sq.km)
< 1	6.17	534.00
1-2	128.86	1655.32
2-3	8.79	1545.00

But as per the study by NRSA (2004) vide Table 6.46, surface water logged area is 592.41 ha. This wide variation in waterlogged area may be due to :

(i) Assessment is done by different agencies. Each agency adopted its own norms for defining water logging and salinity/alkalinity.

(ii) Assessment is done in different periods.

(iii) In some assessment un-irrigated and/or coastal saline areas are included.

(iv) Diversity of soil characteristics, dominant crops grown etc. are not considered.

"The degree of variation in the estimates of physically and chemically degraded agricultural lands indicates lack of effort and proper criteria to assess the condition of land resources, relation to these conditions with their agricultural production potential and finding out the causes of their deterioration to enable planning for suitable ameliorative measures."

Various measures to minimise the problems of waterlogging and salinity such as preparation of drainage master plan, lining of channels, provision of appropriate control structures and escapes etc. should be incorporated in the planning and design stage of the project prior to implementation. A detailed soil survey of the command area is essential to assess the soil capabilities for sustained irrigation. Irrigation intensities and types of crops grown also play a vital role.

These provide a strong deterrent to development of waterlogging and salinity. Hirakud being an old project, it is difficult to implement other structural measure except C.C. lining (Refer Sec 7.4.3). As a rule, less water input for crops helps in preventing waterlogging. Infact, providing 80 to 85% of the optimal dose of water does not significantly reduce the productivity while it enhances project benefits by increasing the area under irrigation and prevents waterlogging. This can only be achieved by creating awareness amongst cultivators.

CHAPTER - VII

PERFORMANCE OF HIRAKUD DAM PROJECT

7.0 Introduction :

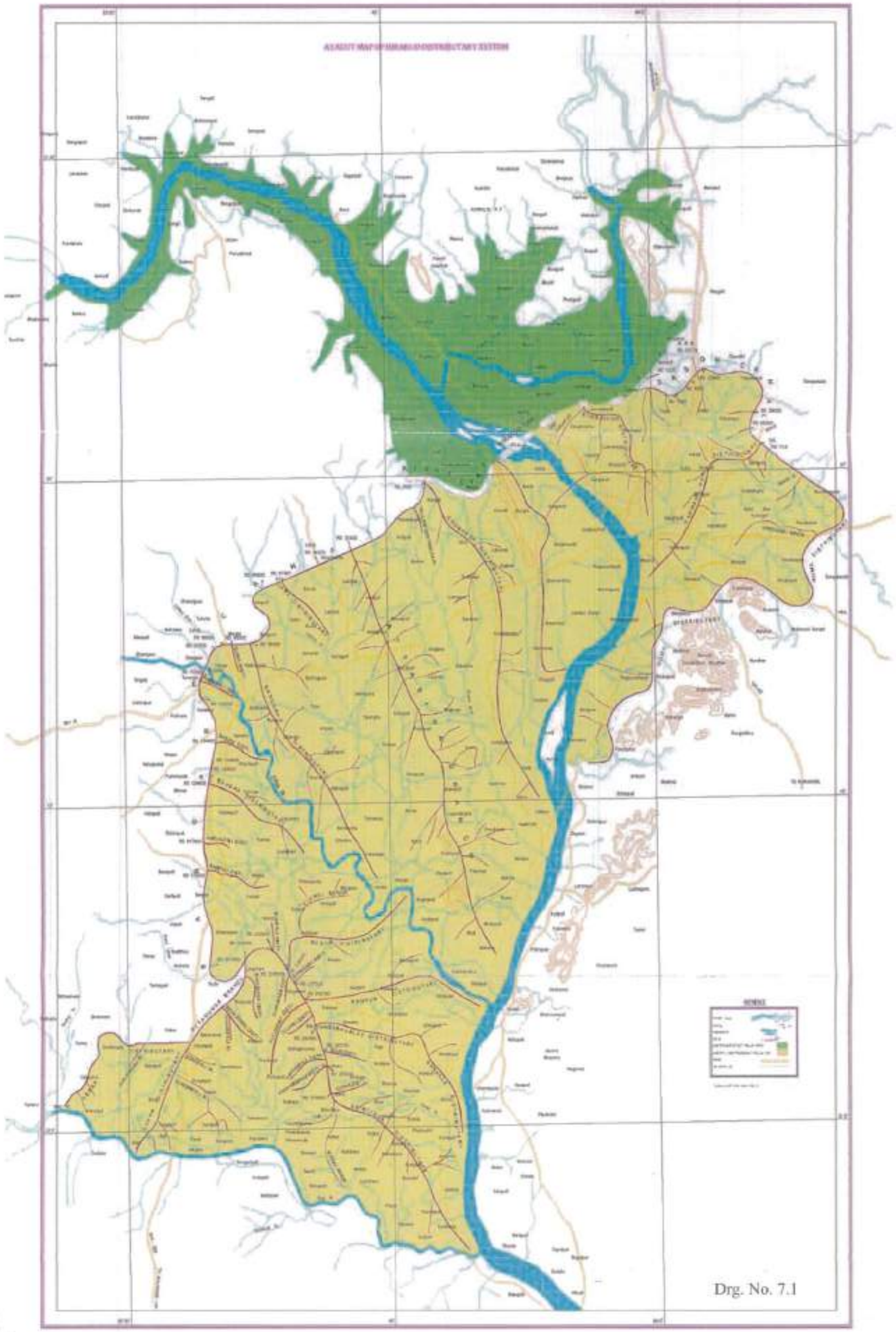
Large and wide spread programme of implementation of irrigation development taken up during the planned era has created some problems concerning drainage, lag in potential created and utilised, low irrigation efficiencies etc. The past experience shows that though the failure can be attributed to lack of management skill and inter departmental co-ordination at various stages, major part of failure can be attributable to the deficiencies in water delivery system, large losses during conveyance and distribution, inequitable and untimely delivery of water to the fields, poor on-farm development and management, inappropriate methods of field applications etc.

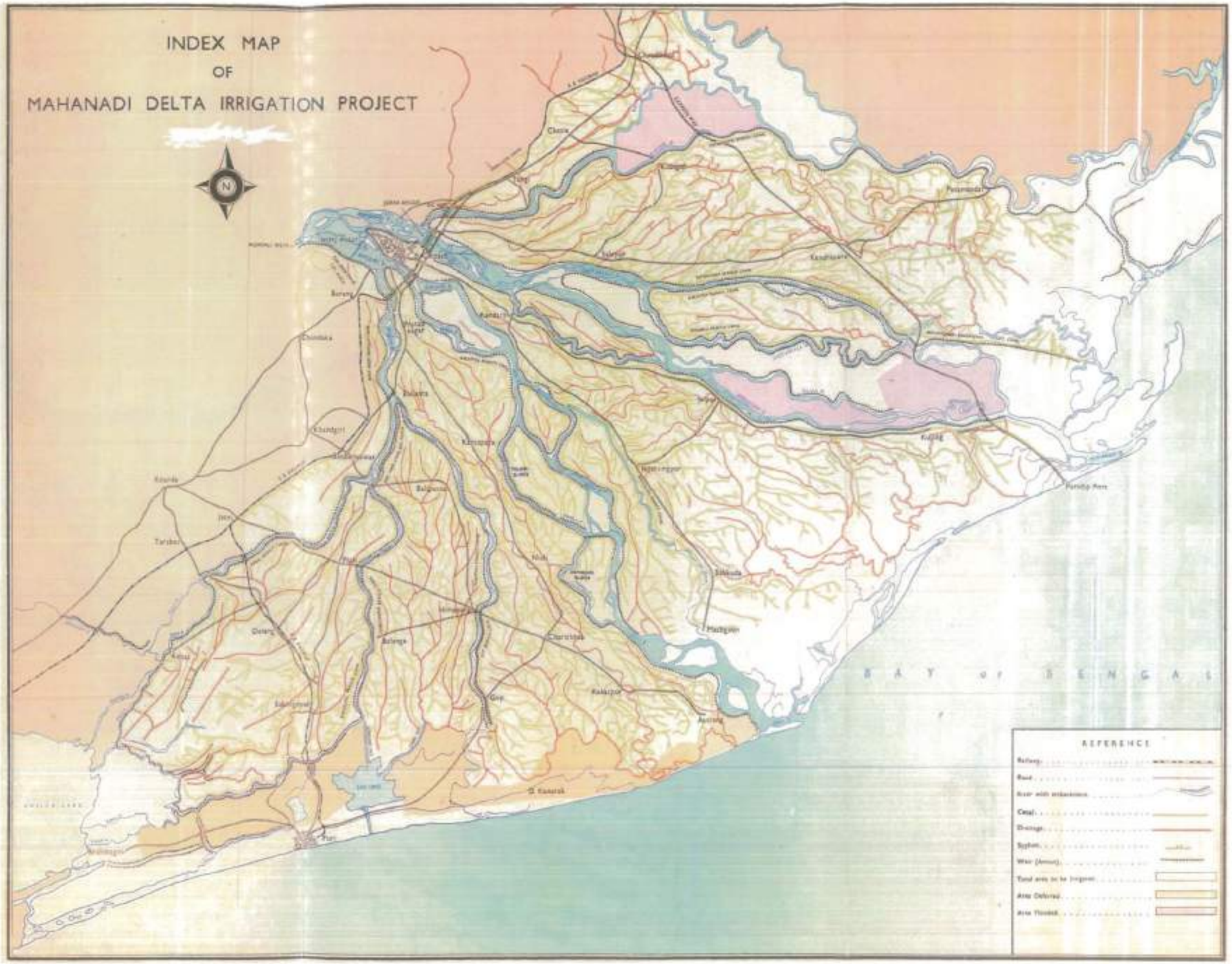
With rise in cost of construction of water resources projects and pressure on land due to increase in population, attention is to be focussed towards better water management with a view to find ways and means to raise productivity while minimizing the wastage of scarce water. This necessitates significant improvements in efficiency and flexibility of irrigation system. With lack of flexibility in delivery, the farmers have insufficient freedom in deciding the cropping pattern.

Although no project ever functions exactly as planned at the designing stage, improving performance of an irrigation system could result in agricultural productivity thereby increasing the socio-economic conditions of the farmers. It is also required to know the causes of low performance so that further deterioration of existing systems and improvement in future designing of new systems could be suitably effected.

'Worldwide, dams have played a key role in development. They were built to irrigate agricultural lands, supply water to meet domestic and industrial needs, generate power, control floods and enable navigation'. Hirakud Dam Project is of no exception. This happens to be the first major multipurpose river valley project in independent India. It not only provides irrigation to 1,59,093 ha in Kharif and 1,08,385 ha. in Rabi in Bargarh, Sambalpur, Sonepur and Bolangir districts of Western Orissa but also irrigates 3,03,000 ha. in Stage-I & Stage-II delta due to controlled power house releases. Besides, the project protects 9500 sq.km. of deltaic area in the undivided districts of Cuttack and Puri from ravages of flood. Maps showing command area directly from Hirakud dam and Stage I & II delta are enclosed vide Drg. No. 7.1 and Drg. No. 7.2

In the following sections, how far the Hirakud Dam Project has performed in fulfilling its major objectives viz. Irrigation, Power generation, Flood control and Municipal and Industrial water supply etc. have been discussed. Besides, discussion has also been made regarding pisciculture and tourism.


Drg. No. 7.1


INDEX MAP
OF
MAHANADI DELTA IRRIGATION PROJECT
N
BAY OF BENGAL
REFERENCE

Drg. No. 7.2

7.1 Irrigation in Hirakud Command :

"Irrigation is now no more looked upon merely as an artificial application of water to crops in an isolated and individual way, but it has come to have a different meaning, especially in the vast arid and semiarid regions of the world, most of which are undeveloped and under developed. In such regions irrigation provides the essential basis for an integrated all round development and is, therefore, more than a practice and technique: it is a way of life, which embraces all the various aspects of development of a community - economic, social, cultural etc." (vide 'History of Irrigation Development in Orissa' by G.C. Sahu, INCID 2009 Pg.1). 'Artificial irrigation not only acts as an insurance against drought and scarcity but also augments crop yields which are more than the mere interest on the capital outlay of irrigation projects.' (Ibid Pg.2)

7.1.1 Irrigation Practices in Sambalpur during Pre-Hirakud Period :

Water is a far more important factor than soil, and an ample and well-distributed rainfall is a matter of vital importance to cultivators. In most years the amount of rainfall is sufficient, the average being about 58" per annum, but it is often unevenly distributed, and deficient in critical months. So, artificial irrigation is absolutely necessary. Before the completion of the Hirakud Dam Project, the main source of irrigation was the tank. The tanks were of three kinds - Kata, Munda and Bandh.

(i) Kata

'An ordinary irrigation tank, which is known as a Kata, is constructed by throwing a strong earthen embankment, slightly curved at either end, across a drainage line, so as to hold up an irregularly shaped sheet of water. The undulations of that country usually determine its shape as that of a long isosceles triangle of which the dam is the base. It commands a valley, the bottom of which is the Bahal land and the sides of which are the Mal terrace. As a rule, there is a cutting high up the slope near one end of the embankment. From this the water is led either by a small channel or Tal, or from field to field along the terraces, down which it finds its way to the lower land. In ordinary years, irrigation may be entirely unnecessary and in that case the superfluous water is passed along until it falls into the nallah in which the small valley ends. In years of short rainfall the centre of the tank is sometimes cut through, when the bottom lands need irrigation, but in ordinary years such an expedient would be dangerous, for the water is deepest at the centre and no sluices are used. Such tanks supply water to at least 5 acres (2.025 hectares) and usually to an area of 30 to 300 acres (12 to 121 ha).'

(ii) Munda

'The Munda is an embankment of smaller size across a drainage channel. Embankments of this sort are very common, as they can easily be constructed by the Raiyats themselves for the benefit of their own holdings. These men have perhaps a few fields commanded by the main village tank, but have built Mundas to protect their outlying fields, more recently acquired from others or reclaimed from the waste. For its purpose the Munda is useful, for if a failure of rain is not very serious it may provide water enough in the later months of growth to save the crop. But it is necessarily shallow and cannot give more than a certain supply'.

(iii) Bandh

'The Bandh is four-sided tank excavated below the Kata, from which it derives its water by percolation. They are almost invariably used for drinking purposes only, are properly regarded as suitable monuments of piety or charity, and are invariably consecrate or married to a God.

Apart from their obvious sanitary advantages, they add to the irrigated area by spreading percolation and by rendering it possible in years of drought to empty the irrigation tank completely'.

(iv) Wells

'Apart from tanks, the district has special irrigational advantages in the wells. Its sandy soil holds in most places a plentiful store of sub-soil water at no great depth (15 to 29 feet) from the surface. Such wells hold water through the hot weather and are largely used for the irrigation of sugarcane plots. Temporary wells are also sometimes used for the irrigation of rice in the tracts near the Mahanadi where water is found close to the surface'.

(v) Other means of Irrigation

'Other means of irrigation are of little importance. At places, irrigation by nallahs is taken recourses to, by means of which the water is diverted and carried into the fields. For raising, water from a lower to a higher level the common lever lift called 'tenda' is used. This consists of a long pole poised between two uprights and weighted at its lower end, and is used invariably whether water is required form a well or from a tank. If there is only a small difference of level, baskets (Sena) worked by two men are often used'.

(vi) Methods of cultivation adopted by different communities

In the past, irrigation in the district was done mostly by tanks. This was due to suitability of the district for tank-making. The Imperial Gazetteer (1908) mentions it in these words: "In 1903-04 the irrigated area was only 31 square miles, but in the previous year it had been over 196, being the maximum recorded. With the exception of 12 square miles under surface and garden produce, the only crop irrigated is rice. It is not too much to say the very existence of villages over a large portion of the area is dependent on the tanks which have been constructed near them. There are 9, 500 irrigation tanks, or between three and four to every village in the district on an average. The ordinary Sambalpur tank is constructed by throwing a strong embankment across a drainage line, so as to hold up an irregularly shaped sheet of water. Below the embankment a four-sided tank is excavated, which constitutes the drinking supply of the village. Irrigation is generally effected by leading channels from the ends of the embankment, but in the years of short rainfall the centre of the tank is sometime cut through. Embankments of small size are frequently thrown across drainage channels by tenants for the benefit of their individual holdings. The Jambor and Sarsuitia Nallaha near Machida are perennial streams, and the water is diverted from them by temporary dams and carried into the fields. In certain tracts near the Mahanadi where water is very close to the surface, temporary wells are also sometimes constructed for the irrigation of rice. Irrigation from permanent well is insignificant".

"By 1931, the position further improved. An area of about 221,347 acres (89,646 hectares) was brought under irrigation from tanks and well. The number of tanks swelled to 12,282, The increases in the irrigated land was most remarkable in Baragarh ex-Zamindari areas where 47, 784 acres (19, 353 hectares) of rice land could get irrigation compared with 28,205 acres (11,423 hectares) twenty years back,"

The following is an interesting account from the Sambalpur District Gazetteer (1932) which highlights the difference in the methods of irrigation adopted by the Agharia and Kulta cultivators.

"There is a great difference between the methods of irrigation practised by the Agharia immigrants from Chhatisgarsh, who have settled in the flatter riparian tracts to the north, and by the Oriya Kultas, who prefer a comparatively undulating county. The Agharia works only on rich soil, and this he finds in the level tracts which the Kulta avoids. The latter depends almost entirely on

his water supply, and likes rolling country with surface drainage and shallow sub-soil water. The Agharia is a poor tank builder and constructed only the shallow square tank commonly used in Raipur and Bilaspur which gathers no surface drainage, depends altogether on direct rainfall, and fails in a year of short rainfall. One glance at a stretch of rice fields suffices to distinguish Kulta from Agharia cultivation, The former builds only low and narrow banks between his fields, seldom more than two feet high, because he has frequently to cut them in order to pass his irrigation water from plot to plot. The Agharia builds high field boundaries, making a tank of each field, because each must catch and keep its rainfall. The two systems suit the tracts to which they are applied. In any ordinarily favourable year the Agharia reaps the heavier crop, but in a bad year he loses more the Kulta does".

(vii) Administration of tanks

'The construction of tanks is of such vital importance in this district that special concessions have been made to encourage it. Land made irrigable by tank construction is secured against assessment at irrigated rates at the ensuing settlement; and in addition to this, it has been ruled for raiyatwari villages that a gaontia or raiyat who makes a tank on his land is entitled to remission of the revenue on the area submerged from the date on which the tank is completed. The distribution of water from the public tanks has hitherto been left in the hands of the panchayat or village committee, and though this gives rise to much contention in a year of drought, no more impartial and expert agency is available. Such an arrangement is necessary; for it is impossible to state definitely for every year what blocks of fields should first be irrigated from the public tanks. This is a question of each year's rainfall, on the state of each tank and on the state of the crops'.

As regards the maintenance of tanks, Mr. Dewar, in his Settlement Report, writes as follows: "At last settlement all the old tanks not constructed on the proprietary land of gaontias were regarded as public property and were recorded as the property of Govt. This step, intended to prevent selfish or short sighted misuse and encroachment, has probably been of real service. But encroachment on the beds of public tanks has been the rule rather than the exception. Many cases have come to light in which parts of the beds of the old tanks, temporarily cropped at last settlement, were then entered in private holdings. It has been found to be impossible to legally dispossess the encroachers, and as they can now claim damages if their crops are submerged, many gaontias who wish to restore old tanks to their former level are deterred from doing so. The abuse is a result of the imperfect system of repair adopted by the villagers. As they have pressing need for irrigation only once or twice in 8 to 10 years, they do not annually repair embankments, but allow them to lose a few inches every year by the wash of rain, until the water level has fallen two or three feet, when a subscription is called for and the earthwork made up. But in the intervening years, the falling water exposes round the upper edge of the tank a strip of rich land into which the nearest cultivators are tempted to turn their ploughs, the gaontia himself being not infrequently a transgressor".

"It may be added that the famine of 1900 taught two lessons, viz, the need of extending irrigation tanks and of adopting a system of closer cultivation. There was scarcely a single gaontia in the famine stricken tracts who did not double his desire for an irrigation tank and do his best to obtain it: - indeed, the gaontia who handled the money valued earth work more than silver. The shortness of the grain supply, again, made the cultivators adopt a system of closer cultivation. They abandoned, for the time being, light and comparatively useless land which they had been in the habit of scratching, and devoted their attention to the proper embanking of the better fields.

They had no seed to waste, and consequently they were willing to abandon the wasteful system of broadcasting and bihura, and transplanted their rice wherever they could".

(Source: Bengal Districts Gazetteer, Sambalpur - published by Bengal Secretariat Book Depot, 1909).

7.1.2 Soil Classification in the Command :

The cultivated land of this region have been conveniently classified into four types according to their location with respect to the watershed. They are Att (uplands), Mal (the slopes), Berna and Bahal (the low lands). The upland generally receives direct rainfall. The Mal lands are terraced to catch surface water, but their water holding capacity is relatively lower than Berna or Bahal lands. The Berna lands are of good quality and their water holding capacity is high.

The soil fertility of these different types of lands varies according to their location. The Att and Mal lands are regarded as less fertile than Berna and Bahal lands. This is because the fine clay particles as well as the soluble nutrients are washed out from higher levels and tend to increase the fertility of the lower lands.

Att, Mal, Berna and Bahal lands respectively comprise about 15%, 50%, 20% and 25% of the total area.

"Different soil types are found in different classes of land. Broadly speaking, three main types of soil are found in this region: sandyloam, loam and clayJoam. Heavy clay soils are also found in some places.

The soils also vary with the topography of the land. The Att lands have the lightest soil, mostly sandyloam and their water holding capacity is low. These soils have neither sufficient humus nor nitrogen to supply necessary nutrients to the plants.

Therefore only shallow rooted crops are grown in this type of soil. Before the introduction of canals, farmers were reluctant to grow crops in Att lands because of the high degree of risk and uncertainty.

The soils of Mal land are mostly loamy in nature, fit for growing any crop under irrigated conditions.

The soils of Berna land are clayloam and are slightly more fertile than that of Mal lands because water from Mal lands deposits soluble nutrients in Berna lands as it flows down.

The soils of Bahal land are regarded as the most fertile of the region. With good water holding capacity, these soils have some inherent fertility. The lands have deeper soil crust and are comparatively heavier than the other three types discussed above. These types of soils are fit for long duration paddy and almost all the lands are put under paddy during kharif and rabi (subject to availability of water)."

(Source: P. Kumar, 'Economics of Water Management', Heritage Publishers, New Delhi, 1991 Pg.9).

7.1.3 Agricultural Scenario :

7.1.3.1 Crops Grown in Pre-Hirakud Period :

As per June 1947 Project Report, the most important crop cultivated in Sambalpur district was paddy. The distribution of crops was as follows (vide Appendix VIII, Pg.21 of the report) :

Table 7.1 : Distribution of crops

Sl.No.	Name of crops	Percentage
1.	Autumn Paddy	20.00
2.	Winter Paddy	72.00
3.	Wheat	0.65
4.	Pulses	4.80
5.	Oil seeds	1.25
6.	Vegetables	0.80
7.	Sugar cane	0.50
8.	Tobaco	0.02
9.	Other crops	0.58

As more than 70% of the area in the district is cultivated with paddy (Khariff), the feasibility of raising two paddy crops on the tract was referred to the Director, Central Rice Research Institute, Cuttack. An extract of his No.Agr./1773 dt.26.05.1951 is reproduced below:

"Ever since this institute was started, experiments have been in progress to explore the possibilities of growing two crops of paddy one after the other. It has been found possible to grow two crops of rice under canal irrigated conditions. While the first crop would be of early, medium or late variety and may be removed from the land anytime from October-December, the second crop would necessarily be of an early variety grown from January to April. The two crops together give a yield of about 3500 lbs of paddy per acre. Thus by growing two crops of rice, the production can be tangibly increased."

The crops were selected as per geo-hydrological situation of land but rice was the predominant crop (vide Table 7.1) and was grown in all types of land though yield was more stable in Berna and Bahal which yielded about 1200 kg/ha. Farmers' wisdom was the basis of crop selection then.

7.1.3.2 Post Hirakud Canal Period :

Particulars of canal system are furnished in Table No.7.2.

Table 7.2 : Details of Canal System

Particulars	Baragarh Main canal	Sason Main canal	Sambalpur Distributory
Design Discharge(cumec)	107.59	17.836	3.397
Designed C.C.A. (ha)	1,32,170	22,663	5,597
Main Canal (km)	84.28	21.80	-
Branch Canal (km)	33.45	-	-
Distributaries (km)	330.83	95.58	21.34
Minor & S.M. (km)	322.83	43.74	13.06
Minor Distributaries (km)	1879.86	324.14	63.46
Outlet (Nos)	2637	402	128

The project was formulated with designed irrigation intensity of 148% (100% in Khariff and 48% in Rabi). It was envisaged that major crop would be paddy i.e. about 70% in Khariff and 33% in Rabi. But the actual area under paddy steadily increased and is much higher i.e. more than 90%. The cropping pattern adopted after providing irrigation is given in Table 7.3.

Table 7.3 : Distribution of Crop area in different years

Crop	% of area under crop to net area sown					
	1964-65	1975-76	1985-86	1995-96	2005-06	2008-09
Khariff						
Paddy	89.50	94.96	92.60	91.56	91.44	91.20
Mung	0.43	0.69	0.60	0.51	0.11	0.35
Biri(Blockgram)	2.08	1.50	0.32	0.38	0.38	0.28
Groundnut	0.80	2.03	0.45	0.42	0.14	0.45
Til	0.05	0.35	0.16	0.18	0.30	0.22
Sugarcane	0.57	0.14	0.65	0.63	0.03	0.18
Vegetable	0.23	0.20	6.04	6.48	7.74	7.73
Rabi						
Summer paddy	95.07	98.48	74.12	73.84	60.93	68.18
Mung(Greengram)	4.42	0.44	3.62	3.65	3.94	2.28
Biri	0.07	0.02	0.60	0.55	3.94	0.72
Groundnut	0.01	0.04	4.38	4.31	3.70	3.75

N.B. : For the years 1964-65 and 1975-76, the data has been collected from the 'Report on Socio-Economic Study in Hirakud Ayacut Area' by Bureau of Statistics & Economics, Orissa (Dec. 1981) and other datas from the office of the Dy. Director, Agriculture, Sambalpur.

7.1.3.2.1. Causes for dominance of rice :

Rice crop dominated, and is dominating continuously as can be seen from Table 7.3 because of the following reasons:

i) Plentiful available water, almost free near the farm.

ii) Availability of high yielding varieties of rice.

iii) Favourable response of rice to applied water during early years of canal system due to existing natural drainage.

iv) Increase in productivity of rice due to intensive use of inputs.

v) Wet-wet canal schedule did not permit growing of non-paddy crops.

vi) Increased number of rice mills helped not only to decrease post harvest losses but also to provide assured market of rice just at harvest.

vii) Social involvement of mill owners with farming community made the system sustainable for a long period of time.

7.1.3.2.2 Effect of dominance of rice :

"Extensive rice cultivation under intensive irrigation and other inputs with fairly long duration rice varieties in all types of land along with mismanagement of water ultimately led to several environmental consequences. The soil has degraded in terms of physical, chemical and biological properties. The physical changes were the outcome of use of cage wheel for puddling in both the seasons without allowing the soil to dry. The deterioration in soil quality is the outcome of depletive human activities and their interaction with natural environment. Continuous cropping with use of N fertilizer along without the use of adequate organic matter has brought in secondary and micronutrient deficiencies. Indiscriminate use of pesticides resulted in resurgence of pests. Soil biological activity has undergone tremendous change. The situation not only halted the crop diversification programme but also plateaued rice yield despite an increase in fertilizer and pesticide consumption. How can a sick substrate produce a healthy crop?" (Source: Sustainable agricultural development in the Hirakud Command - Issues & Strategies by JML Gulati, M.M. Panda & P.K. Mahapatra - Unpublished).

"Despite this, farmers were/are inclined to grow rice continuously overcoming production constraints through technological interventions as rice was the best choice under prevailing soil-water situation with assured market available for rice. Cropping up of a large number of rice mills in the private sector and their social involvement with rice farmers encouraged continuous rice cropping too. Rice mills used to absorb most of the rice produced without letting farmer incur any post harvest loss, thus providing assured market for rice. The above described equilibrium has been upset due to distress sale of paddy and it has become a matter of serious concern for planners, scientists, administrators and so on as it has adversely affected the socio-economic condition of the entire farming community. The situation warrants for an immediate shift from growing rice to non-rice crops (crop diversification) in larger proportions. Even though crop diversification was attempted in the past, a very limited success could be achieved in uplands. Now massive crop diversification seems to be the only option for *Mal* and *Berna* lands to improve soil health for sustainable crop productivity and to overcome emerging socio-economic problems in Hirakud command.

In order to restore soil health, make optimum and profitable use of land, soil and water, research studies on crop-water related aspects were carried out in the All India Coordinated Project on Water Management Research at Regional Research & Technology Transfer Station, Chiplima. It has been observed that key to success of crop diversification in irrigation commands lies with the method of conveyance of water. It is ideally to be from field channel to field rather than from field to field which is in vogue. The approach has been successfully demonstrated in farmers' field at Barahaguda in Bargarh district."

(Source : Problems & Prospects of Crop Diversification in Hirakud Command by J.M.L. Gulati & M.M. Panda, RR and T Transfer Station, Chiplima, Souvenir, January 2004 Pg.25).

7.1.4 Annual Irrigation

Irrigation achieved during Kharif and Rabi from 1960-61 upto 2008-09 is shown in Table 7.4. Table 7.5 gives the quantum of water released monthwise for irrigation purposes from 1958 onwards. From the table, it can be seen that maximum supplimentation is needed in the month of October for Kharif as rainfall virtually ceases by then. Similarly for Rabi crops more water has been utilised during the month of March. Average yield of crops, water rate and profit & loss in irrigated area are furnished in Table 7.6, Table 7.7 and Table 7.8 respectively.

Table 7.4 : Annual Irrigation from Hirakud Dam

Year	Area of Irrigation achieved in Acres			Annual Irrigation in ha.
	Khariff	Rabi	Annual	
1960-61	300,000	97,000	397,000	160,660
1961-62	320,000	110,000	430,000	174,019
1962-63	320,000	130,000	450,000	190,726
1963-64	320,000	151,285	471,285	190,871
1964-65	330,000	141,644	471,644	190,871
1965-66	340,000	181,240	521,240	210,943
1966-67	340,000	44,528	384,528	155,616
1967-68	340,000	232,308	572,308	236,610
1968-69	360,000	221,845	581,845	235,470
1969-70	368,000	213,047	581,047	235,146
1970-71	368,000	212,561	580,561	234,950
1971-72	367,977	225,190	593,167	240,051
1972-73	377,163	242,646	619,809	250,828
1973-74	377,163	242,005	619,168	250,569
1974-75	377,163	210,531	587,694	237,831
1975-76	377,410	264,740	642,150	259,878
1976-77	378,654	249,551	628,205	254,225
1977-78	378,654	270,863	649,517	262,850
1978-79	380,579	272,769	653,348	264,400
1979-80	383,227	242,612	525,839	253,268
1980-81	383,392	275,386	658,778	266,598
1981-82	383,392	264,704	648,096	262,275
1982-83	392,625	294,694	687,319	278,148
1983-84	392,625	279,316	671,941	271,925
1984-85	392,625	269,327	661,952	267,883
1985-86	392,625	260,912	653,537	264,477
1986-87	392,625	272,988	665,613	269,364
1987-88	392,625	272,988	665,613	269,364
1988-89	392,625	272,988	665,613	269,364
1989-90	392,625	272,988	665,613	269,364
1990-91	392,625	272,988	665,613	269,364
1991-92	393,043	257,068	650,111	263,090
1992-93	393,004	257,070	650,074	263,076
1993-94	393,004	243,500	636,504	257,584
1994-95	393,043	257,952	650,995	263,448
1995-96	393,043	255,997	649,040	262,657
1996-97	393,043	255,997	649,040	2,62,657
1997-98	393,121	239,409	632,530	255,985
1998-99	393,121	273,784	666,905	269,896
1999-00	393,120	2,74,812	667,932	270,312
2000-01	393,121	2,25,830	618,951	250,489
2001-02	393,121	2,68.536	661,657	267,773
2002-03	393,121	2,46,085	639,206	258,687
2003-04	393,121	2,56,200	649,321	262,780
2004-05	393,121	2,86,936	680,057	275,219
2005-06	393,121	2,67,220	660,341	267,240
2006-07	393,121	2,99,758	692,879	280,408
2007-08	393,121	308,452	701,573	283,927
2008-09	393,121	3,09,176	702,297	284,220

TABLE 7.5 : Monthwise Water drawal for Irrigation (Upto 2009)

All figures in M.Ac.ft.

Year	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Total
1958	0.078	0.078	0.077	0.079	0.029	0.004	0.039	0.044	0.074	0.076	0.078	0.068	0.724
1959	0.076	0.067	0.059	0.067	0.066	0.004	0.056	0.069	0.103	0.115	0.083	0.086	0.851
1960	0.069	0.073	0.082	0.083	0.048	0.016	0.042	0.047	0.086	0.110	0.085	0.081	0.822
1961	0.102	0.094	0.112	0.113	0.101	0.000	0.026	0.118	0.042	0.068	0.057	0.065	0.898
1962	0.061	0.070	0.105	0.099	0.048	0.016	0.067	0.058	0.117	0.185	0.070	0.061	0.957
1963	0.129	0.097	0.123	0.105	0.055	0.009	0.096	0.109	0.097	0.124	0.079	0.118	1.141
1964	0.143	0.147	0.173	0.139	0.009	0.052	0.051	0.107	0.132	0.172	0.116	0.124	1.365
1965	0.140	0.133	0.153	0.145	0.002	0.061	0.133	0.212	0.241	0.236	0.071	0.093	1.620
1966	0.156	0.179	0.207	0.169	0.051	0.008	0.125	0.136	0.197	0.199	0.061	0.022	1.510
1967	0.043	0.044	0.034	0.055	0.025	0.000	0.126	0.135	0.156	0.203	0.078	0.111	1.010
Av.58-67	**0.100**	**0.098**	**0.113**	**0.105**	**0.043**	**0.017**	**0.076**	**0.104**	**0.125**	**0.149**	**0.078**	**0.083**	**1.090**
1968	0.181	0.173	0.197	0.202	0.022	0.010	0.115	0.132	0.217	0.204	0.093	0.096	1.642
1969	0.203	0.198	0.259	0.249	0.017	0.089	0.198	0.134	0.255	0.243	0.113	0.132	2.090
1970	0.205	0.187	0.223	0.221	0.000	0.079	0.152	0.170	0.146	0.213	0.095	0.147	1.838
1971	0.186	0.180	0.223	0.224	0.092	0.026	0.141	0.118	0.186	0.222	0.093	0.110	1.801
1972	0.181	0.192	0.225	0.229	0.052	0.027	0.142	0.223	0.237	0.253	0.123	0.084	1.968
1973	0.198	0.194	0.235	0.236	0.023	0.000	0.107	0.158	0.156	0.211	0.100	0.093	1.711
1974	0.199	0.194	0.244	0.251	0.060	0.000	0.146	0.196	0.236	0.190	0.073	0.094	1.883
1975	0.190	0.177	0.228	0.176	0.000	0.000	0.144	0.173	0.215	0.212	0.064	0.125	1.704
1976	0.184	0.211	0.261	0.266	0.080	0.031	0.155	0.167	0.193	0.240	0.061	0.072	1.921
1977	0.192	0.191	0.238	0.178	0.001	0.095	0.165	0.177	0.184	0.227	0.042	0.076	1.766
Av.68-77	**0.192**	**0.190**	**0.233**	**0.223**	**0.035**	**0.036**	**0.147**	**0.165**	**0.203**	**0.222**	**0.086**	**0.103**	**1.832**
1978	0.184	0.189	0.241	0.260	0.047	0.101	0.164	0.161	0.177	0.222	0.043	0.097	1.886
1979	0.188	0.181	0.238	0.249	0.033	0.098	0.172	0.209	0.257	0.249	0.022	0.075	1.971
1980	0.171	0.190	0.231	0.125	0.000	0.011	0.118	0.158	0.164	0.252	0.051	0.053	1.524
1981	0.193	0.202	0.256	0.232	0.000	0.080	0.157	0.189	0.205	0.045	0.013	0.113	1.685
1982	0.208	0.211	0.250	0.203	0.000	0.094	0.175	0.188	0.210	0.269	0.008	0.112	1.928

Contd....

Table 7.5 (Continued)

1983	0.174	0.214	0.273	0.232	0.000	0.058	0.210	0.199	0.178	0.186	0.001	0.075	1.800
1984	0.199	0.229	0.286	0.288	0.045	0.088	0.198	0.189	0.217	0.197	0.077	0.099	2.112
1985	0.204	0.215	0.278	0.251	0.014	0.063	0.178	0.105	0.119	0.228	0.027	0.085	1.767
1986	0.210	0.211	0.257	0.217	0.052	0.029	0.160	0.188	0.251	0.242	0.006	0.018	1.841
1987	0.221	0.218	0.249	0.255	0.083	0.043	0.178	0.264	0.264	0.278	0.005	0.073	2.131
Av.78-87	**0.195**	**0.206**	**0.256**	**0.231**	**0.027**	**0.067**	**0.171**	**0.185**	**0.204**	**0.217**	**0.025**	**0.080**	**1.865**
1988	0.191	0.195	0.232	0.241	0.012	0.110	0.211	0.218	0.232	0.279	0.036	0.149	2.106
1989	0.232	0.224	0.270	0.274	0.092	0.093	0.231	0.211	0.230	0.244	0.102	0.109	2.312
1990	0.130	0.221	0.253	0.262	0.043	0.091	0.214	0.241	0.234	0.198	0.038	0.113	2.038
1991	0.181	0.230	0.260	0.260	0.011	0.192	0.210	0.185	0.218	0.261	0.032	0.147	2.187
1992	0.122	0.222	0.262	0.262	0.113	0.107	0.235	0.173	0.211	0.282	0.041	0.082	2.112
1993	0.192	0.227	0.274	0.272	0.070	0.106	0.217	0.213	0.231	0.216	0.108	0.105	2.231
1994	0.120	0.222	0.263	0.261	0.068	0.103	0.189	0.201	0.210	0.271	0.103	0.148	2.159
1995	0.121	0.195	0.236	0.245	0.092	0.043	0.158	0.203	0.247	0.258	0.075	0.093	1.966
1996	0.116	0.227	0.253	0.262	0.031	0.103	0.214	0.216	0.246	0.275	0.072	0.135	2.150
Av.88-95	**0.156**	**0.218**	**0.256**	**0.260**	**0.059**	**0.105**	**0.209**	**0.207**	**0.229**	**0.254**	**0.067**	**0.120**	**2.140**
1997	0.134	0.211	0.261	0.266	0.129	0.095	0.196	0.171	0.194	0.257	0.060	0.053	2.027
1998	0.165	0.200	0.242	0.254	0.113	0.085	0.234	0.241	0.223	0.257	0.001	0.090	2.105
1999	0.228	0.224	0.259	0.259	0.109	0.037	0.223	0.215	0.208	0.241	0.019	0.081	2.103
2000	0.221	0.226	0.261	0.268	0.108	0.059	0.175	0.229	0.232	0.260	0.091	0.064	2.194
2001	0.153	0.189	0.225	0.233	0.006	0.002	0.102	0.159	0.208	0.260	0.011	0.019	1.567
2002	0.179	0.199	0.244	0.246	0.084	0.031	0.221	0.200	0.208	0.259	0.103	0.051	2.025
2003	0.161	0.164	0.210	0.212	0.075	0.052	0.205	0.159	0.150	0.234	0.013	0.051	1.686
2004	0.177	0.179	0.218	0.218	0.048	0.082	0.234	0.180	0.257	0.269	0.079	0.078	2.019
2005	0.234	0.232	0.290	0.290	0.059	0.086	0.237	0.213	0.262	0.277	0.041	0.083	2.304
2006	0.241	0.248	0.287	0.294	0.088	0.096	0.256	0.177	0.254	0.297	0.077	0.076	2.391
2007	0.249	0.236	0.287	0.291	0.089	0.078	0.263	0.251	0.243	0.283	0.085	0.243	2.598
2008	0.248	0.243	0.293	0.302	0.107	0.067	0.256	0.198	0.253	0.293	0.071	0.070	2.401
2009	0.263	0.251	0.293	0.293	0.066	0.089	0.221	0.254	0.278	0.305	0.074	0.045	2.432

Table 7.6(a) : Average yield of Paddy (1966-76)

Year	Autumn Paddy (qntl/ha)			Winter Paddy (qntl/ha)			Summer Paddy (qntl/ha)		
	Hirakud ayacut	Sambalpur Dist.	Bolangir Dist.	Hirakud ayacut	Sambalpur Dist.	Bolangir Dist.	Hirakud ayacut	Sambalpur Dist.	Bolangir Dist.
1966-67	6.82	3.46	2.72	15.91	13.07	12.55	9.49	15.15	N.A.
1967-68	-	5.66	4.42	-	14.16	15.10	-	14.83	11.22
1968-69	-	3.11	3.66	-	12.33	14.43	-	18.24	13.66
1969-70	-	5.53	4.40	-	13.76	13.56	-	19.02	14.78
1970-71	-	9.81	8.87	-	15.86	15.00	-	23.87	15.69
1971-72	-	9.71	8.43	-	16.14	14.21	-	25.25	21.25
1972-73	-	8.77	6.92	-	18.11	14.50	-	21.52	16.46
1973-74	-	11.39	8.60	-	17.82	16.75	-	22.48	15.59
1974-75	-	6.27	4.45	-	13.94	9.44	-	20.21	16.68
1975-76	13.79	10.38	8.75	20.58	18.16	15.54	27.40	21.45	21.08

(Source: 'Report on Socio-Economic Study in Hirakud Ayacut area' - Bureau of Statistics & Economics, Orissa, Dec. 1981 Pg.16 to 18).

Table 7.6(b) : Average yield of Paddy in Hirakud Ayacut (1985-2010)

Year	Winter Paddy (qntl/ha)		Summer Paddy (qntl/ha)	
	Sambalpur district	Bargarh district	Sambalpur district	Bargarh district
1985-86	31.37	33.20	35.70	36.72
1995-96	35.20	37.45	38.40	39.95
2005-06	37.40	40.05	39.42	41.05
2006-07	37.45	38.20	47.22	43.52
2007-08	38.52	40.15	48.52	49.50
2008-09	39.45	41.32	50.14	51.25
2009-10	40.10	45.70	N.A.	N.A.

Table 7.6(c) : Average yield of other crops (qntl / ha.)

Year	Mung		Groundnut		Mustard		Sugarcane	
	Sambalpur	Bargarh	Sambalpur	Bargarh	Sambalpur	Bargarh	Sambalpur	Bargarh
1985-86	4.32	4.35	10.86	12.68	9.12	9.30	592.00	634.25
1995-96	4.08	4.40	11.75	12.85	9.40	9.45	797.33	824.00
2005-06	4.62	4.75	12.88	12.95	11.72	12.78	820.00	888.57
2006-07	4.95	4.97	13.08	13.10	12.20	12.80	865.32	879.12
2007-08	4.87	4.89	13.44	13.70	12.51	12.77	875.15	890.50
2008-09	5.02	5.10	13.65	13.76	12.89	12.85	892.15	896.35
2009-10	5.00	5.12	13.80	13.95	N.A.	N.A.	N.A.	N.A.

N.B. Yield figures presented in Table 7.6, (b) & (c) have been collected from Dy. Director (Agr.), Sambalpur.

The rates of water charges levied in 1975-76 and in 1960-61 per acre of cultivation of different crops in Hirakud ayacut area are given in the Table No.7.7.

Table 7.7 : Water-rate charged (Rs.) in Hirakud Ayacut srea / Ac.

Sl. No.	Name of the crops	Rates charged in 1960-61		Rates charged in 1975-76	
		Kharif	Rabi	Kharif	Rabi
1.	Paddy	4.00	8.00	8.00	24.00
2.	Groundnut	-	5.00	-	7.50
3.	Sugarcane	10.00	-	27.00	-
4.	a) Ganja	-	-	-	50.00
	b) Mung	-	1.0	-	1.50
5.	Arhar	5.00	-	7.50	-
6.	Wheat	-	2.00	-	9.00
7.	Palato	-	10.00	-	15.00
8.	Cotton	10.00	-	15.00	-
9.	Mustard	-	1.50	-	3.00
10.	Til	-	2.00	-	3.00
11.	Maize	-	5.00	-	3.75
12.	Onion	-	5.00	-	15.00
13.	Chilly	-	5.00	-	7.50
14.	Vegetable	-	8.00	-	12.00

Source: Report on Socio-Economic Study in Hirakud ayacut area, Bureau of Statistics & Economic, Dec. 1981.

Table 7.8(a) : Comparative Analysis on Profit & Loss for Cultivation of different crops in Irrigated Villages (as per 1975-76 survey)

Crop	Yield qntl./ac.	Rate qntl./ac.	Gross Income per acre	Cost/ac. (Rs.)	Net Income/Loss (+/-)	
					Per acre	Per ha.
Autumn Paddy	5.58	76.33	425.92	430.01	(-) 14.09	(-) 10.10
Winter Paddy	8.33	76.33	635.83	429.99	(+) 205.84	(+) 508.63
Summer Paddy	11.09	76.33	846.50	589.85	(+) 256.65	(+) 634.18
G.Nut (Kharif)	4.09	238.71	976.32	484.62	(+) 491.70	(+) 1215.00
G.Nut (Rabi)	7.31	104.30	762.43	1118.75	(-) 356.32	(-) 880.46
Mung (Rabi)	1.56	176.87	275.92	217.18	(+) 58.74	(+) 145.15
Sugarcane (Gur)	42.23	210.00	8868.30	1629.60	(+) 7238.70	(+) 17886.80

Source : Report on Socio-Economic Study in Hirakud Ayacut Area, Dec. 1981 Pg.29.

Table 7.8(b) : Comparative analysis on profit and loss (ha) for cultivation of different crops in irrigated villages (Survey result 1985-86)

Crop	Yield per acre (qntl.)	Rate per qntl.(Rs.)	Gross Income per acre (Rs.)	Cost of Production per Acre (Rs.)	Net Income per acre. loss (-) profit (+)	Net Profit/ Loss per ha.(Rs.)
Winter paddy	12.91	300.00	3873	1900	1973	4875.00
Summer paddy	14.48	300.00	4344	1900	2444	6039.00
G.Nut (Kharif)	5.11	800.00	4088	2220	1868	4616.00
G.Nut (Rabi)	6.20	800.00	4960	2220	2740	6770.00
Mustard	3.68	750.00	2760	1250	1510	3731.00
Sugarcane (Gur)	245.24	35.50	8706	5760	2946	7280.00

Table 7.8(c) : Comparative analysis on profit and loss (ha.) for cultivation of different crops in irrigated villages (Survey result 1995-96)

Crop	Yield per acre (qntl.)	Rate per qntl.(Rs.)	Gross Income per acre (Rs.)	Cost of Production per Acre (Rs.)	Net Income per acre loss (-) profit (+)	Net Profit/ Loss per ha.(Rs.)
Winter paddy	14.53	360.00	5230	2800	2430	6004.00
Summer paddy	15.59	360.00	5616	3020	2596	6415.00
G.Nut (Kharif)	5.68	900.00	5110	2000	3112	7690.00
G.Nut (Rabi)	8.80	900.00	7920	2000	5920	14628.00
Mustard	3.77	860.00	3225	1250	1975	4880.00
Sugarcane (Gur)	324.26	42.50	13812	9500	4312	10655.00

Table 7.8(d) : Comparative analysis on profit and loss (ha.) for cultivation of different crops in irrigated villages (Survey result 2005-06)

Crop	Yield per acre (qntl.)	Rate per qntl.(Rs.)	Gross Income per acre (Rs.)	Cost of Production per Acre (Rs.)	Net Income per acre loss (-) profit (+)	Net Profit/ Loss per ha.(Rs.)
Winter paddy	15.50	650.00	10075	4420	5655	13973.0
Summer paddy	16.09	650.00	10460	4820	5640	13936.0
G.Nut (Kharif)	5.16	1400.00	7224	4750	2474	6113.0
G.Nut (Rabi)	9.80	1400.00	13720	4750	8970	22165.0
Mustard	4.90	1500.00	7350	1370	5980	14777.0
Sugarcane (Gur)	341.71	55.00	18794	16450	2344	5792.0

Table 7.8(e) : Comparative analysis on profit and loss (ha.) for cultivation of different crops in irrigated villages (Survey result 2006-07)

Crop	Yield per acre (qntl.)	Rate per qntl. (Rs.)	Gross income per acre (Rs.)	Cost per acre (Rs.)	Net income/ per acre loss (-) profit (+)
Winter paddy	15.13	650.00	9835	4570	5265
Summer paddy	18.15	650.00	11798	4570	7228
G.Nut (Kharif)	5.24	1400.00	7336	5950	1386
G.Nut (Rabi)	10.70	1400.00	14980	5950	9030
Mustard	5.00	1700.00	8500	1370	7130
Sugarcane (Gur)	348.89	55.00	19189	16950	2239

Table 7.8(f) : Comparative analysis on profit and loss (ha.) for cultivation of different crops in irrigated villages (Survey result 2007-08)

Crop	Yield per acre (qntl.)	Rate per qntl. (Rs.)	Gross income per acre (Rs.)	Cost per acre (Rs.)	Net income per acre loss (-) profit (+)
Winter paddy	15.73	750.00	11798	5070	6728
Summer paddy	19.60	750.00	14700	5100	9600
G.Nut (Kharif)	5.42	1500.00	8130	5900	2230
G.Nut (Rabi)	10.80	1500.00	16200	5900	10300
Mustard	5.11	1800.00	9198	1500	7698
Sugarcane (Gur)	353.13	60.00	21188	16950	4238

Table 7.8(g) : Comparative analysis on profit and loss (ha.) for cultivation of different crops in irrigated villages (Survey result 2008-09)

Crop	Yield per acre (qntl.)	Rate per qntl.(Rs.)	Gross Income per acre (Rs.)	Cost of Production per Acre (Rs.)*	Net Income per acre loss (-) profit (+)	Net Profit/ Loss per ha.(Rs.)
Winter paddy	16.15	850.00	13727	5070	8657	21391.0
Summer paddy	20.27	850.00	17230	5100	12130	29973.0
G.Nut (Kharif)	5.48	1600.00	8768	5900	2868	7087.0
G.Nut (Rabi)	10.90	1600.00	17440	5900	11540	28515.0
Mustard	5.15	1800.00	9270	1730	7540	18631.0
Sugarcane (Gur)	357.70	60.00	21462	16950	4512	11149.0

* While calculating the cost of production, hire charges of tools & implements, land revenue and water charges have not been taken into consideration vide Tables 7.8(h) & (i) which will bring down the net income. The data on profit and loss shown in Tables 7.8(b) to 7.8(g) have been obtained from the office of the Dy. Director (Agr.), Sambalpur.

Table 7.8(h) : Cost of production per acre of different crops (in Rupees) : 2007-08

Sl. No.	Name of the Crop	Cost per acre								
		Seed	Manure & Fertilizers including transportation charges	Water charges	Pesticides & fungicides	Land revenue	Human labour	Animal labour	Misc. charges (hired charges in tools & implements, watchman charges etc.)	Total cost per acre
	Kharif									
1.	W. paddy	270	1050	NA	700	NA	1800	1250	NA	5070
2.	Mung	450	950	NA	200	NA	1350	400	NA	3350
3.	G.Nut	850	1100	NA	500	NA	2900	600	NA	5900
4.	Til	60	650	NA	200	NA	1200	450	NA	2560
5.	Biri	450	950	NA	200	NA	1350	400	NA	3350
6.	Sugarcane	2450	2900	NA	2200	NA	8000	1400	NA	16950

Table 7.8(i) : Cost of production per acre of different crops (in Rupees) : 2008-09

Sl. No.	Name of the crop	Cost per acre								
		Seed	Manure & Fertilizers including transportation charges	Water charges	Pesticides & fungicides	Land revenue	Human labour	Animal labour	Misc. charges (hired charges in tools & implements, watchman charges etc.)	Total cost per acre
	Rabi									
1.	S.Paddy	300	1050	NA	700	NA	1800	1250	NA	5100
2.	Mung	450	950	NA	200	NA	1350	400	NA	3350
3.	G.Nut	850	1100	NA	500	NA	2900	600	NA	5900
4.	Til	60	650	NA	200	NA	1200	450	NA	2560
5.	Biri	450	950	NA	200	NA	1200	450	NA	2560
6.	Sugarcane	2450	2900	NA	2200	NA	8000	1400	NA	16950
7.	Vegetables									
8.	Wheat	450	1100	NA	350	NA	1200	250	NA	3350
9.	Mustard	80	550	NA	250	NA	600	250	NA	1730

7.1.5 Maintenance cost of Hirakud Canals :

Average annual maintenance cost of Hirakud Dam over a period of time from 1960-61 to 1975-76 reveals a steadily increasing trend over the years 1960-61 to 1973-74 but has substantially declined in the year 1974-75 and 1975-76. The details are set out in the table below.

Table 7.9 : Average Annual Maintenance cost of Hirakud Canals

Year	Rupees	Index
1960-61	3,73,240.85	100
1961-62	6,10,659.67	163.61
1962-63	9,00,105.83	241.16
1963-64	11,07,932.28	296.84
1964-65	14,00,181.01	375.14
1965-66	14,50,153.59	388.52
1966-67	14,88,921.56	398.91
1967-68	14,43,386.44	386.71
1968-69	15,22,232.71	407.84
1969-70	19,99,320.33	535.66
1970-71	19,99,838.48	535.80
1971-72	24,58,184.64	658.60
1972-73	28,91,305.48	774.64
1973-74	33,21,442.93	889.88
1974-75	23,10,418.08	619.01
1975-76	14,17,272.79	379.71

(Source: Report on Socio-Economic Study in Hirakud Ayacut area, Dec. 1981).

Table 7.10 shows the annual R/M expenditure of Hirakud canal system from 2001-2002 to 2009-2010.

Table 7.10 : Yearwise R/M (Non-plan) expenditure (lakh Rs.)

Year	Bargarh canal Division	Sambalpur Irrigation Division	Total Rs. in lakh
2001-02	273.42	187.54	460.96
2002-03	151.21	231.77	382.98
2003-04	63.13	181.12	244.25
2004-05	160.54	240.55	401.09
2005-06	150.92	248.38	399.30
2006-07	167.03	253.77	420.80
2007-08	196.73	202.03	398.76
2008-09	207.95	392.21	600.16
2009-10	380.23	674.58	1054.81

Above figures are exclusive of plan provision. Data have been collected from respective Divisions.

7.1.6 Income from water rates :

The income from water rates charged for Kharif has increased from Rs.6.96 lakhs in 1970-71 to Rs.28.98 lakhs in 1975-76. The income from water rates charged for Rabi crops ranged from Rs.17.76 lakhs in 1970-71 to Rs.40.84 lakhs in 1975-76. The total income from the water rates charged for both Kharif and Rabi crops, raised in Hirakud ayacut, however, increased from Rs.24.73 lakhs in 1970-71 to Rs.69.82 lakhs in 1975-76 with a substantial annual increase in this five year period of time as may be seen from the Table No.7.11 below.

Table 7.11 : Income from Water Tax from 1970-71 to 1975-76 (Rs. in lakhs)

Sl. No.	Year	Kharif	Rabi			Total	Total Kharif and Rabi (Col.3+7)
			Heavy duty crop	Medium duty crop	Light duty crop		
1	2	3	4	5	6	7	8
1.	1970-71	6.96	15.73	1.61	0.42	17.76	24.73
2.	1971-72	8.28	25.46	1.48	0.65	27.59	35.87
3.	1972-73	8.30	27.82	1.60	0.74	30.16	38.46
4.	1973-74	8.32	29.37	1.60	0.67	31.64	39.96
5.	1974-75	8.33	30.18	1.92	0.66	32.76	41.09
6.	1975-76	28.98	37.93	2.49	0.42	40.84	69.82

(Source: Report on Socio-Economic Study in Hirakud Ayacut Area - Bureau of Statistics & Economics, Orissa Dec. 1981 Pg.11)

Irrigation water rate for different crops have been revised from time to time which is furnished in Table 7.12 & 7.13.

7.1.7 Irrigation Water Levy

Water rates in major & medium Projects in the State are assessed on basis of area irrigated and the type of crop grown. The charges are levied per unit area (i.e., ha) basis separately for Khariff and Rabi crops. For Kharif crops, it is compulsory basic water rate on the basis of depth of water (or the class of irrigation) that the project is designed to supply which is to be paid irrespective of its use. But, during Rabi, it is not compulsory and payable on use. Irrigation water rate for Kharif is shown in Table 7.12.

Table 7.12 : Compulsory-Basic Irrigation Water Rate

Sl. No.	Class of Irrigation	Depth of water supply (inches/mm)	Water Rate for Flow Irrigation (Rs/ha)					
			1961-62	68-69	75-76	81-82	98-99	2002-03 (*)
1.	Class-I	28/710	19.77	9.88	19.77	39.54	100.00	250.00
2.	Class-II	23/585	14.83	7.41	14.83	29.65	75.00	188.00
3.	Class-III	18/460	9.88	4.94	9.88	19.77	50.00	125.00
4.	Class-IV	9/230	4.94	2.47	4.94	9.88	25.00	63.00

* (Source: Govt. of Orissa, Revenue Department Notification 5th April 2002.

The Water charges for Rabi crop have been fixed according to the water requirement of crops. Water requirement is considerably high for water intensive crops like Ganja, betel leaf and 'Saru' (Arum). The water charges for different crops as revised from time to time are furnished in Table 7.13.

Table No.7.13 : Rabi Water Rate for Crops (Rs/ha)

Sl. No.	Name of crops	1968-69	73-74	74-75	81-82	98-99(*)	2002-03 (**)
1.	Paddy (Dalua)	19.77	39.54	59.30	88.96	225.00	450.00
2.	Tabaco	37.07	37.07	55.60	83.40	210.00	420.00
3.	Potato	24.71	24.71	37.07	55.60	140.00	280.00
4.	Vegetables	19.77	19.77	29.65	44.48	115.00	230.00
5.	Onion	24.71	24.71	37.07	54.36	140.00	280.00
6.	Wheat	4.94	14.83	22.24	32.12	85.00	170.00
7.	Maize	12.36	12.36	18.53	27.80	70.00	140.00
8.	Mung	2.47	2.47	3.71	5.56	14.00	28.00
9.	Groundnut	12.36	12.36	18.53	27.80	85.00	170.00
10.	Orchards	29.65	29.65	44.48	66.72	167.00	334.00
11.	Sugarcane	34.59	44.48	66.72	100.08	250.00	500.00
12.	Jute	7.41	7.41	11.12	16.68	42.00	84.00
13.	Fodder	12.36	12.36	18.53	27.80	85.00	170.00
14.	Pulses	4.94	4.94	7.41	11.12	30.00	60.00
15.	Cotton	24.71	24.71	37.07	55.60	140.00	280.00
16.	Til (Oil seed)	4.94	4.94	7.41	11.12	30.00	60.00
17.	Betel Leaf	74.13	74.13	111.20	166.79	420.00	840.00
18.	Arhar	12.36	12.36	18.53	27.80	85.00	170.00
19.	Sunhemp	17.30	17.30	25.95	38.91	100.00	200.00
20.	Chilly	12.36	12.36	18.53	27.80	85.00	170.00
21.	Saru (Arum)	74.13	74.13	111.20	166.79	420.00	840.00
22.	Ragi	6.18	6.18	9.27	13.99	35.00	70.00
23.	Mustard	2.47	4.94	7.41	11.12	30.00	60.00
24.	Ganja	-	123.55	123.55	185.33	465.00	930.00

(*) Govt. of Orissa, Revenue & Excise Department Notification dt.18th July 1998.
(**) Govt. of Orissa, Revenue Department Notification dt.5th April 2002.

Water rates for various crops in Lift Irrigation projects are different from flow irrigation. The water charges are given in Table 7.14.

Table 7.14 : Water Rate for Different Crops in Lift Ayacut

Sl. No.	Name of crops	Water Rate Rs. per acre				
		Upto Oct'92	From Nov'92 to Oct'96	From Nov'96 to June 97	From July'97	Revised rate from 2000
1.	Kharif Paddy	28.80	76.92	149.04	209.16	269.28
2.	-do- (Rabi)	120.00	320.50	621.00	871.50	1122.00
3.	Tabaco	57.60	153.84	298.08	418.32	538.56
4.	Potato	57.60	153.84	298.08	418.32	538.56
5.	Vegetables	72.00	192.30	372.60	522.90	673.20
6.	Onion	48.00	128.20	248.40	348.60	448.80
7.	Wheat	36.00	96.15	186.30	261.45	336.60
8.	Hy. Maize	57.60	153.84	298.08	418.32	538.56
9.	Mung	9.60	25.64	49.68	69.72	89.76
10.	Groundnut	36.00	96.15	186.30	261.45	336.60
11.	Orchards	72.00	192.30	372.60	522.90	673.20
12.	Sugarcane	108.00	288.45	558.90	784.35	1109.80
13.	Jute	24.00	64.10	124.20	174.30	224.40
14.	Fodder	48.00	128.20	248.40	348.60	448.80
15.	Pulses	12.00	32.05	62.10	87.15	112.20
16.	Cotton	48.00	128.20	248.40	348.60	448.80
17.	Til (oilseed)	19.20	51.28	99.36	139.44	179.52
18.	Betel leaf	192.00	512.80	993.60	1394.40	1795.20
19.	Arhar	36.00	96.15	186.30	261.45	336.60
20.	Sunhemp	24.00	64.10	124.20	174.30	224.40
21.	Chilly	115.20	307.68	596.16	836.64	1077.12
22.	Saru (Arum)	216.00	576.90	1117.80	1568.70	2019.60
23.	Ragi (Rabi)	52.80	141.02	273.24	383.46	493.68
24.	Ragi (Kharif)	14.40	38.46	74.52	104.58	134.64
25.	Mustard	7.20	19.23	37.26	52.29	67.32

7.1.8 Value of Land

The value of owned land per acre was reported on an average to be Rs.680.00; 'Att' lands @ Rs.500/acre and Bahal land @ Rs.852/acre (vide Economics of a Multiple-purpose river Dam by N.V. Sovani & Nilakantha Rath, Gokhale Institute of Politics & Economics, 1960 Pg.241).

The average value per acre of different types of land has increased conspicuously over a period of 10 years 1965 to 1975 in both ayacut and control areas. The value of land depends on its average yield rate which, to a great extent, is dependent on irrigation. The rate of increase appears to be slightly higher in irrigated area than in the control area. The following table gives the comparison in detail.

Table 7.15 : Variation in the value of land after coming of Hirakud canals (in ayacut and control area)

Types of land	1964-65 Average value per acre (in Rs.)		1975-76 Average value per acre (in Rs.)		Percentage Increase	
	Irrigated	Control	Irrigated	Control	Irrigated	Control
Att	713	475	2,350	1,073	229.6	125.9
Mal	1,088	725	3,826	1,813	251.6	150.0
Berna	1,430	850	4,381	2,420	206.4	184.7
Bahal	1,740	1,100	5,172	3,235	197.2	195.0
Barchha	4,500	4,350	10,767	8,119	139.3	86.6

(Source: Report on Socio-Economic Study in Hirakud ayacut area, Dec. 1981)
N.B. Villages without field channels are control villages.

The present cost of land has gone very high. In Bargarh, Bheden and Barapali Tahasil, the average value of land/ac. is Rs.1.50 lakh.

7.1.9 Participatory Irrigation Management

7.1.9.1 Introduction

Inequitable distribution of water amongst beneficiaries over space and time leads to injudicious use of precious water in irrigation commands. Farmers, whose lands are located in the head reach of canal system, get an advantage over their fellow farmers whose lands are in the tail reach in terms of water allocation. Anticipating unpredictable water supply, the head reach farmers draw more than their actual requirement as a measure against future uncertainty. This not only results in wastage of water but tail-enders are deprived of getting their due share. This leads to inequality within the same command.

This fact was realised, when it was observed that even after huge investments made in the water resources sector, there remains a wide gap between the potentials created and potentials utilized. By the end of 9th Five year plan (1997-2002), the potentials created in the country was 94 million ha against which, there remained substantial gap of 14 million ha un/ under utilised. Thus for better irrigation management Govt. of India lunched CADA programme in 1977 through State Govt. But through CADA, the desired objective of water distribution and maintenance of canal system could not be achieved. This led to the involvement of beneficiaries for better operation and maintenance of the system as they will gradually develop a sense of belongingness. With this in view, PIM programme was implemented by Govt. with formation of Water Users Association (WUA) or Pani Panchayat (PP) for participation of farmers in water management, irrigation scheduling, distribution and maintenance of canal system at micro level, economic water use, selection of crop, cropping sequence and timely water supply. It will further delineate responsibility in water distribution and maintenance of system between the end user and the department for attaining better serviceable standards and will create a congenial atmosphere between the managers and the users in the entire operation.

7.1.9.2 Management Transfer Process:

According to Chambers, "Irrigation system are not physical entities constructed to blue prints: they are more like organisms which grow, spreading over time, fitting the physical terrain pulled here and there by human activity, decaying and being restored". With Colebrook-Cameron reforms in 1832, the people's management of irrigation system in India got a set back which abolished the compulsory labour and hereditary headmanship.

An ordinance was enacted in 1856 which entrusted the Govt. responsibility of irrigation development to Govt. Agent who was the head of the administration of the province. The colonial Govt. assumed responsibility for development of irrigation removing local systems. The 1856 ordinance was revised in 1867 which provided selection of one or more headmen by the owner to ensure maintenance of rights and prevention of any action that may sabotage the ancient custom and damage the system. But the basics remained that the owners would manage the system. The Provincial Director of Irrigation was created by transferring some functions of Govt. Agent. Towards 1930s changes in national perspective regarding development of irrigation ushered. During 1948, the central amendment through irrigation ordinance emphasized four main issues. Those are (a) effective participation, (b) removal of irrigation headman (c) conflict resolution and (d) maintenance of irrigation. The 1958 Paddy land Act of Central Govt. increased its control over irrigation management and cultivation committee replaced irrigation headman (Refer: Irrigation Management Policies and Practices, 20-21 June 2005, Bhubaneswar, pp 33). The village committee expected to resolve conflicts but they had no punitive power. The introduction of modern technology alienated farmers from decision making process.

The participatory approach towards development and management of irrigation is not new in our country. Prior to British rule, the kings and rulers of different regions used to provide assistance for capital costs involved for construction of water sources viz, tanks, check dams and diversion weirs while 0 & M of canal system and distribution of water was entrusted to farmers. Such examples are found in Maharashtra near Nasik and Dhule as Phad system, in Vijaynagar Canal of Karnatak, in several tanks of Tamilnadu and A.P and also in neighboring countries like Nepal and Srilanka. Throughout the world, for irrigation reforms, now the stress is on PIM. PIM has been refined to Irrigation Management Transfer (IMT) and is being practised in Turkey, Mexico, Phillipines and the states like Maharashtra, M.P., A.P., Tamilnadu, Karnatak and Orissa.

Under USAID assisted Water Resources Management and Training (WRM & T), PIM has been introduced in several states of the country from 1980s. Towards later part of nineties, A.P., M.P., and Karnatak enacted legislation making PIM a statutory requirement to access irrigation water. Orissa adopted the 'Pani Panchayat Act' in 2002.

In A.P, a big-bang approach was followed by forming about ten thousand WUAs within a year of implementation of the programme by legislating 'Farmers Management of Irrigation system Act' in 1997. At present the performance of WUA is poor due to unwarranted interference from outside. On the contrary, Maharashtra and Gujrat have adopted motivational approach. Though they have not achieved quick success, it is progressing steadily.

7.1.9.3 PIM scenario in Orissa

PIM is being implemented in the state in the name of 'Pani Panchayat' programme. The principal objectives of Pani Panchayat are 'to promote and secure distribution of water among its users, adequate maintenance of the irrigation, system, efficient and economical utilization of water to optimize agricultural production, to protect the environment and to ensure ecological balance by involving the farmers, inculcating the sense of ownership of the irrigation system in accordance with the water budget and the operational plan'.

Initially it was a difficult task to motivate the reluctant farmers to forego their traditional attitude and to accept the PIM concept. The concept was introduced in the State during 1996 under the World Bank assisted Orissa Water Resources Consolidation Project (OWRCP) in four flow irrigation projects i.e Derjang, Aunli, Ghodahado and Rushikulya Distributory No.II covering an area of 0.21 lakh ha. on pilot basis. WALMI, Orissa along with two NGOs were entrusted to motivate the farmers to form WUA by registering the Associations under the Societies Registration Act, 1860. With a lot of persuasion, it was possible to form only 50 nos of WUAs by 1997 and another 113 nos. by August 2000. Further a decision was taken for providing financial assistance of Rs.35.00 per ha initially in 2002 as grant in aid to the PPs which was subsequently enhanced to Rs.1,00.00/ha in 2003-04.

7.1.9.3.1 Pani Panchayat in Hirakud Command :

The details of 'Pani Panchayats' formed upto 31.3.2010 under Sambalpur Irrigation Division, and Bargarh Canal Division of Hirakud Dam Project are furnished in Table 7.16(a) & 7.16(b) respectively.

Distributary wise detail informations are given below.

Table 7.16 (a) Formation of W.U.A. (P.P) under Sambalpur Irrigation Division, Burla.

Sl. No.	Name of the distributaries	Area in ha.	No. of distributaries committee delineated	No. of pp
1.	Sason Main Canal	5269.34	1	9
2.	Sambalpur Disty.	3836.03	1	10
3.	Paramanpur Disty.	4606.81	1	11
4.	Huma Tail Disty.	10713.12	1	23
5.	Chiplima Disty.	2074.00	1	5
6.	Senhapali Disty.	1413.15	1	3
7.	Godhbaga Disty.	15771.65	2	28
8.	Attabira Branch Canal	31468.68	5	71
	Total	**75152.78**	**13**	**160**

Table 7.16 (b) Formation of W.U.A. (P.P) under Baragarh Canal Division (as on 31.3.2010)

Sl. No.	Name of the distributaries	Area in ha.	No. of distributaries committee delineated	No. of pp
1.	Bargarh Disty.	12166.00	1	24
2.	Resam Disty.	6985.50	1	12
3.	Bhimtikra Disty.	10692.00	1	21
4.	Rampur Disty.	6545.00	1	12
5.	Barkarley Disty.	6352.00	1	12
6.	B.M.C. 0-35 km	6296.00	1	12
7.	B.M.C. 35-47 km	6111.44	1	13
8.	B.M.C. 47-69.65 km	8381.56	1	18
9.	B.M.C. 69.65-84.28 km	5803.20	1	12
10.	Sukha Disty.	8661.00	1	21
11.	Kebar Disty.	4816.00	1	11
	Total	**82809.70**	**11**	**168**

7.2 Flood Control

At present, there is only one multipurpose reservoir across Mahanadi at Hirakud though initially it was contemplated to have two more reservoirs in the downstream i.e. at Tikarpara and Naraj. These two projects could not see the light of the day due to various reasons (Refer sec.1.5.2 of the book). Hirakud reservoir is gradually built up to reach the F.R.L. at the end of the monsoon keeping in view the various demands viz. irrigation, power generation and flood control as per the Rule Curve (vide Sec.6.1.2 to 6.1.5 of the book). There is no specific flood storage space in the reservoir. Though the reservoir has moderated the flood to a great extent it has not been able to completely control the flood particularly when the flood of longer magnitude occurs towards the later part of the monsoon. Secondly yield from the uncontrolled catchment below Hirakud upto Naraj (48735 sq.km) is very large (i.e. from river Tel.).

Some of the worst devastating floods and damages caused are discussed in following paragraphs in nutshell.

Flood of 1955

In the year 1954-55 acute distress situation prevailed in certain parts of the State on account of drought. The intensity of drought and distress aggravated due to failure of rains in the month of July & August of 1955. On 2nd and 3rd September 1955, there was 8 inches of rainfall in Cuttack city with continuous downpour for 26 hrs. In Sambalpur, which is on the upper reach of Mahanadi, there was 14.5 inches of rainfall within 48hrs ending at 8 AM of 04.9.55. Titilagarh also received a rainfall of 5.5 inches within 24hrs during the same period. Similarly in the mountainous region of Madhya Pradesh, there was heavy rainfall. What has been unusual this year is that in addition to very heavy rainfall spread over a space of about 48 hours in the catchment area of rivers, there was wide spread rain throughout the State including the delta simultaneously, adding to the volume of water all along the course of the Mahanadi and its branches and the Brahmani and Baitarani. The Mahanadi gauge at Cuttack rose up to RL 76.20ft in the early hours of 06.9.1955, the highest recorded so far in the history of Mahanadi in about a hundred years. Within 42 hours up to midnight of 5th/6th Sept.1955, the Mahanadi rose by a height of 5.02 ft.

The devastation caused by the unexpected flood has been relatively very high because of the breach for a width of 610 m (2000 ft) at 'Daleighai' in the Sadar sub-division of Cuttack district (now Jagatsinghpur district) which inundated a large tract not only thickly populated but of which 200 sq miles (518 sq. km) were considered protected and was grannary of Cuttack district. Equally intensive devastation was caused in Binjharpur where the breaches near Kamalpur completely washed off some villages and sand cast a large area. Like wise Pattamundai and Kendrapara suffered from heavy losses where irrigated land was inundated and damaged by several breaches in Brahmani. At Chandanpur near Puri, the Bhargavi embankments also gave way.

Out of cultivated area of 15,35,290 acres about 5,56,290 acres (36%) in the undivided district of Cuttack and 1,55,846 acres out of cultivated area of 10,70,338 acres i.e. 14.6% in undivided Puri district were damaged. Communication for four days from 6th to 10th September was completely paralysed in the flood affected areas of Cuttack, Puri and Balasore. Roads were submerged under water. The portion of the National Highway No.5 between Cuttack and Bhubaneswar was under more than 2 ft of water for 48 hours. The boat communication could not be established for 3 to 4 days due to swiftness in the current of flood water.

Movement of trains was suspended on Calcutta -Madras line due to floods in Brahmani. Number of breaches in the river embankment of Stage I and Stage II delta area were 263. About 14.15 lakhs of population were affected. Loss of human life was 43 in Cuttack, 8 in Dhenkanal, 13 in Puri and 5 in Keonjhar and one in Ganjam district (Source: Preliminary Report on 1955 Orissa Flood by Revenue Department, Govt. of Orissa).

Flood of 1980

This year an unprecedented flood occurred during the 2nd half of September which created havoc at lower part of Mahanadi system. The maximum flood height rose to 27.80m at Naraj Railway Bridge on 22.9.80 against danger level of 26.52m. Peak flood discharge was measured as 12, 27, 476 cusec (34753 cumec). Kathjori system took a major share of 64% and Mahanadi system only 36%. This caused sever damages mainly in Devi river embankments.

There were 92 breaches in the river embankments of Cuttack district. The worst breach occurred at Birabarpatna in the left bank of river Biluakhai for a width of about 400 m. In Stage I & II deltas of Cuttack & Puri districts 54 blocks comprising 3140 villages were affected. Human life lost was 5 (five). Total 1,35,726 houses were collapsed, swept away and partly damaged.

This flood was mainly from the catchment lying upstream of Hirakud dam. The contribution from the free catchment below the dam was negligible. Even with partial routing effect of Hirakud dam, the magnitude of flood was so high. This shows the weakness of Hirakud reservoir to route late monsoon floods when operated according to the rule curve. As there is no reserved flood space in Hirakud, the storage capacity gets gradually reduced due to proportionate storage of water with an aim to fill up the reservoir by 31st October.

Flood of 1982

The flood was more serious in nature than any recorded in history. The damages resulting from this flood were more severe than the damages caused by floods in 1955 and 1980 taken together. Special feature of this flood is that the peak discharge of 15.84 Lakh cusec at Mundali was mainly due to the rainfall over uncontrolled catchment down stream of Hirakud dam as there was only 0.091akh cusec release of water from the reservoir since the gates were kept closed from 16.0 hrs of 29.8.82 to 11.0 hrs. of 31.8.82. Flood situation would have become more alarming, had there been release of water from the reservoir. Downstream of Hirakud, the river started rising rapidly on 28.8.82. The peak at the head of Mahanadi delta at Naraj Railway Bridge attained a level of 28.53 m at 17 hrs. of 31st August against danger level of 26.52 m. The peak level in Kathjori at Bellevue was 83.65 ft (25.50m) against danger level of 75.9 ft (23.14m). The flood discharge at Naraj railway bridge was 44827cumec (15.84 lakh cusec) of which Mahanadi share was 42% where as Kathjori system carried 58%. There were severe damages in Devi River embankments. The National Highway No.5 breached neat Balikuda due to breaches in Kathjori right embankment thereby dislocating the traffic for about a month.

The flood intensity was so severe that historic river such as Prachi (Silted up about 100 years back) and Alaka (silted about 200 years back) opened up during this flood.

Total no. of breaches and damages caused due to this flood are as follows:

No. of districts affected	8
Total population affected	54 lakhs
Number of human casualty	127+49 (missing)

Number of livestock lost	26359
Principal crops damaged	Paddy, Pulses, Jute, Oil seed, Vegetables
Cropped area affected	12 lakh ha.
Sand cast area	75000 ha.
No.of houses swept away	26500
No.of houses fully collapsed	1,58,322
No.of houses partially collapsed	3,25,227
No. of breaches in river embankments	500
No. of M.I. projects damaged	733
No. of L.I. projects damaged	1677
Length of roads damaged P.W.D.road	5000 km
Gram Panchayat road	30,000 km
Forest road	720 km
Canal road	4214 km

(Source: Memorandum on Floods in Orissa, 1982, Revenue Department)

Flood of 2001

Before the bitter experience of 1999 super cyclone disappeared from memory, the State had to bear brunt of severe drought during 2001. With poor economic condition of the State when the restoration works were still incomplete, grave flood situation prevailed almost in all the river systems.

Hirakud reservoir rose from RL 600 ft to 628.54 ft on 18.7.2001 within a period of 18 days consequent upon intense rainfall over upper catchment in Chhatisgarh and Orissa. For safety of the dam, 51 gates were opened on 18.7.01. With downstream contribution, from the tributaries viz, Ib, Ong, Tel, Hariharjore, Salki, Manjore, Brutang, Kusumi and Sapua, the flood peak in undivided Mahanadi at the head of the delta, i.e., Naraj was 14.0 lakh cusec (39638 cumec) from 17.7.01 to 20.7.01. Consequently there was heavy flood in all the branches, i.e., Kathjori, Kuakhai, Kusabhadra, Daya, Bhargavi, Devi, Kandal, Tanla, Sorua, Biluakhai, Birupa, Genguti, Gobari, Chitrotpala, Luna and Karandia. All these rivers were in spate.

Brahmani, the 2nd largest river of the State experienced very high flood even before flood in Mahanadi system completely receded. Due to intense rainfall in the upper catchment, the Rengali reservoir attained FRL 123.50 m on 23.7.0 I when five gates were opened to release flood water. The outflow of 10328 cumec (3,64,785 cusec) from Rengali reservoir had adverse effect on the deltaic area of Jajpur and Kendrapara. Due to interconnection of Brahmani with Birupa, a minor arm of Mahanadi, the flood situation became grave and inundated the head quarter town of Kendrapara. There were several breaches in the embankments in Jajpur, Kendrapara and Bhadrak. Almost during the same period i.e. from 4.7.01 to 6.7.01 and 11.7.01 to 13.07.01, the river Baitarani also remained in spate. On 13.7.01, gauge at Akhuapada was at 18.86 m against danger level of 17.83 m i.e. river flowing 1 m higher.

Hirakud reservoir level and downstream discharge from 16.7.01 to 21.7.01 is shown in Table 7.17. FRL of Hirakud is RL 630 ft.

Table 7.17 : Hirakud Reservoir Elevation and Down Stream Flood Discharge

Date	Time	Res. Level (in ft)	Inflow (Lakh cusec)	Outflow (Lac cusec)	No. of gates opened (sluice)	Flood peak at Khairmal (lakh cusec)	Flood discharge at Barmul (lakh cusec)	Flood discharge at Mundali (lakh cusec)
16.7.01	09 hrs	617.97	3.58	4.58	28	8.30	9.60	5.92
16.7.01	15 hrs	618.95	4.23	0.23	Nil	8.50	10.86	7.72
17.7.01	11 hrs	624.50	9.20	0.23	Nil	6.78	12.10	12.99
17.7.01	20 hrs	627.04	7.82	5.09	30	6.54	10.70	14.00
18.7.01	08 hrs	628.54	8.72	8.04	48	10.44	9.28	12.75
18.7.01	09 hrs	628.54	8.69	8.53	51	10.76	9.28	12.58
18.7.01	19 hrs	627.93	6.90	8.41	51	11.90	11.22	11.29
19.7.01	09 hrs	626.97	5.00	6.02	36	10.60	14.92	12.18
20.7.01	03 hrs	625.79	3.15	4.52	27	7.10	12.38	14.09
21.7.01	10 hrs	623.0	3.01	4.54	27	5.90	8.07	10.58

Maximum recorded gauge and discharge of river Mahanadi at Naraj Railway Bridge from 1964 to 2006 is given in Table 7.20. Impact of 2001 flood on life and livelihood of people is furnished in Table 7.18(a). Losses due to flood from 1972 to 2001 is shown in Table 7.18(b)

Table 7.18(a) : Impact of 2001 Flood

Sl. No.	Indicators	Unit	Loss
1.	Villages affected	Nos.	15584
2.	G.P. affected	Nos.	2429
3.	C.D. Blocks affected	Nos.	199
4.	District affected	Nos.	22
5.	ULBs affected	Nos.	84
6.	People affected	Lakh	85
7.	Crop area lost (Rs)	Lakhs	7.0
8.	Cost of crop loss (Rs)	Crore	60.0
9.	Houses affected	Lakhs	3.0
10.	Cost of affected houses (Rs)	Crore	40.0
11.	PWD roads affected	Km	5000
12.	N.H. affected	Km	700
13.	PWD, CD works damaged	Nos.	1102
14.	N.H., CD works damaged	Nos.	265
15.	Breaches on PWD road	Nos.	102
16.	Breaches on N.H. road	Nos.	13
17.	Breaches on canal embankment	Nos.	223
18.	Breaches on river embankment	Nos.	346
19.	Pipe water supply		
	In urban areas	Nos.	61
	In rural areas	Nos.	82
20.	Tube well damaged	Nos.	25555
21.	Person dead	Nos.	93

(Source : Orissa Development Report, Planning Commission GoI, 2002 Pg 431, Table 14.8)

Table 7.18(b) : Flood Damage from 1972 to 2001

Sl. No.	Years/ Months of occurrence	No. of district affected	No. of village affected	Population affected	Cultivated area affected (ha)	No. of House damaged/ collapsed/ washed away	No. of human casualties	No. of livestock casualties
1.	1972 (July)	5	3,514	1,738,400	NA	18,754	8	3,506
2.	1975	8	7,527	3,140,524	1,048,05	1,44,153	74	4,996
3.	1976	6	4,358	2,53,910	40,375	2,448	8	68
4.	1977 (Nov)	10	4,680	2,161,000	458,612	18,179	41	202
5.	1978	12	3,727	2,601,379	627,076	19,965	21	262
6.	1980	10	3,620	2,639,200	305,466	163,526	73	8,280
7.	1981 (Jun)	1	1,017	NA	26,655	NA	NA	NA
8.	1981 (Aug)	4	NA	NA	3,690,45	1,717	15	NA
9.	1982 (Aug/Sept.)	8	NA	5,400,000	1,200,000	510,049	127	26,359
10.	1984	8	6,960	3,511,177	392,448	19,394	27	459
11.	1985 (Sept.)	7	7,609	1,937,979	102,272	69,831	2,401	NA
12.	1985 (Aug)	11	17,955	3,779,506	310,499	12,974	10	2,880
13.	1985 (Oct)	5	2,571	5,700,000	326,608	40,029	47	1,073
14.	1990 (May)	1		361,868	4,984,88	9,379	1	NA
15.	1991 (July)	11	9,649	3,786,544	123,000	32,389	29	449
16.	1991 (Aug)	7	12,572	6,997,637	539,000	64,381	23	694
17.	1992 (June)	5	3,396	1,835,529	47,394	14,845	7	107
18.	1992 (July)	10	9,281	2,716,000	168,676	72,479	25	222
19.	1992 (Aug)	7	7,866	3,048,000	201,000	73,224	11	1,068
20.	1994 (July)	16	4,540	2,832,247	527,696	NA	29	NA
21.	1994 (Aug)	5	1,336	1,097,361	138,432	NA	24	NA
22.	1994 (Sept.)	18	5,368	3,116,364	351,531	NA	20	NA
23.	1995 (May)	23	31,796	7,184,264	222,840	152,542	50	372
24.	1995 (Nov.)	20	23,945	11,346,000	1,386,800	35,683	26	NA
25.	1997 (June)	4	800	594,646	NA	7,765	-	14
26.	1997 (Aug)	13	5,652	2,140,648	NA	35,955	17	16
27.	1999 (July/Aug)	7	2,486	1,772,788	149,848	7,488	10	NA
28.	2001 (July/Aug)	24	18,790	9,678,000	8,087,000	212,296	102	18,149

Source : Board of Revenue, Cuttack and Special Relief Commissioner, Government of Orissa, Revenue Department Secretariat, Bhubaneswar.

The disasters viz. flood, drought and cyclone have not only taken human lives but also resulted in damage to and loss of property. The value of properties lost and damaged due to these natural disasters has been increasing over decades. These have led to serious fiscal imbalance by placing heavy demands on revenue expenditure. Properties damaged due to disasters and amount spent on relief are given in Table 7.19.

Table 7.19 : Properties Damaged and Relief Expenditure (1972 to 1999)

Year	No. of persons affected	No. of human lives lost	No. of human beings injured	Value of properties damaged (lakh)	Financial year	Relief expenditure (lakh)	Grant (Rs. Lakh)
1972	474780	139	2	1289	-	-	-
1973	434877	17	11	291	-	-	-
1974	1072160	24	5	1289	-	-	-
1975	432902	45	7	344	1974-75	385	0
1976	2859402	65	14	1791	1975-76	388	0
1977	1194550	71	8	922	1976-77	369	0
1978	1530368	101	383	959	1977-78	409	2
1979	6323461	51	11	3257	1978-79	403	0
1980	2913000	82	53	3649	1979-80	871	0
1981	616834	109	966	1256	1980-81	1688	56.55
1982	7323000	245	493	10711	1981-82	1480	-
1983	2103578	119	29	4939	1982-83	17180	12158
1984	3511000	27	75	11889	1983-84	1798	1526
1985	2782289	194	558	13888	1984-85	3024	0
1986	7348781	81	59	9381	1985-86	3066	0
1987	6932347	171	192	6646	1986-87	2775	59.36
1988	2785765	357	105	2701	1987-88	1450	500
1989	6369689	285	75	3710	1988-89	1806	1313
1990	15499135	455	82	19949	1989-90	4598	0
1991	7610509	363	62	22925	1990-91	4714	5713
1992	10817455	218	172	154970	1991-92	4919	0
1993	6079666	353	46	77908	1992-93	4691	2978
1994	10775481	NA	NA	15326	1993-94	7742	2978
1995	3941351	197	142	10995	1994-95	3906	30.75
1996	6331974	226	80	14323	1995-96	7946	7708
1997	3486481	985	140	6855	1996-97	7528	50.0
1998	9822975	943	240	203219	1997-98	8991	4.0
1999	12569000	9885	2507	-	1998-99	4558	828.15
					1999-00	82764	49.62

(Source : Human Development Report, 2004, Govt. of Orissa, Table 7.2 & 7.3, Pg. 164-165)

Table 7.20 : Year wise peak Gauge and Discharge at Naraj Rly. Bridge of River Mahanadi

River Basin : Mahanadi

River : Mahanadi Danger Level : 88.00 ft (26.82m)

Gauge Station : Naraj Railway Bridge Zero Value : 0.00m

Year	Date	Gauge (ft)	Gauge (m)	Discharge (cumec)	Discharge (cusecs)
1	2	3	4	5	6
1964	08.07.64	96.50	29.41	19833	700501
1965	31.07.65	89.00	27.13	8864	313076
1966	31.07.66	94.30	28.74	16562	584970
1967	07.08.67	88.83	27.08	22145	782161
1968	16.08.68	85.80	26.15	21342	753799
1979	01.08.69	88.65	27.02	27664	977092
1970	28.08.70	86.90	26.49	18520	654126
1971	10.08.71	86.50	26.37	21227	749738
1972	15.09.72	85.60	26.09	20210	713817
1973	28.09.73	88.15	26.87	26245	926973
1974	19.08.74	86.50	26.37	21577	762100
1975	24.08.75	86.80	26.46	23146	817517
1976	15.08.76	87.70	26.73	26427	933402
1977	14.09.77	87.70	26.73	26464	934708
1978	29.08.78	88.45	26.96	27865	984192
1979	10.08.79	84.20	25.66	17716	625729
1980	22.09.80	91.20	27.80	34748	1227299
1981	11.08.81	84.20	25.66	17532	619230
1982	31.08.82	93.58	28.52	44750	1580570
1983	09.09.83	87.31	26.61	25088	886108
1984	17.08.84	86.78	26.45	23383	825888
1985	07.08.85	87.35	26.62	25213	890523
1986	22.07.86	87.38	26.63	25493	900413
1987	24.07.87	80.43	24.52	9813	346595
1988	09.08.88	79.90	24.35	8951	316150
1989	19.08.89	78.10	23.80	6563	231805
1990	06.09.90	85.94	26.19	20623	728398
1991	14.08.91	89.02	27.13	33028	1166549
1992	21.08.92	89.80	27.37	34219	1208608
1993	28.08.93	85.28	25.99	23065	814638
1994	06.09.94	89.30	27.22	31576	1115264
1995	25.07.95	87.66	26.72	26776	945732
1996	23.08.96	81.63	24.88	13213	466686
1997	06.08.97	87.01	26.52	24127	852168
1998	13.09.98	86.15	26.26	22907	809088
1999	11.08.99	84.12	25.64	17972	634775
2000	28.07.00	76.92	23.45	5049	178349
2001	20.07.01	89.28	27.21	39887	1408798
2002	14.09.02	89.53	25.46	16632	587478
2003	30.08.03	91.01	27.74	38223	1349844
2004	12.08.04	85.07	25.93	21695	766139
2005	31.07.05	–	–	25577	903266
2006	31.08.06	–	–	36340	1283528

7.2.1 Impact of Hirakud Reservoir on Flood Moderation

Hirakud has performed to its best to moderate the floods in delta. Most of the high floods during post period (1958 to 2009) are due to flood flow from the free catchments of 48735 sqkm downstream of Hirakud dam on which Hirakud has no role. Highest recorded post – construction flood at Naraj is 44,827 cumec (15.8 lakh cusec) at 15 hrs of 31st August 1982, which was mainly from the downstream catchment only without any significant contribution from the dam. Without the dam the possible flood would have been around 48,110 cumecs (17.0 lakh cusec). Similarly the flood peak of 39,638 cumec (14.00 lakh cusecs) in July 2001 would have been 53,794 cumecs (19.00 lakh cusec) without control from Hirakud reservoir.

During pre-construction period (i.e., from 1868-1946) there were 63 floods in 79 years exceeding 10.0 lakh cusec (35,300 cumec) at Naraj, which works out to about 8 floods in 10 years. The highest recorded peak flood was 15.71 lakh cusec (44,431 cumec) in September 1934.

During post construction period of 46 years, i.e., from 1958 to 2008, there are 19 flood events of magnitude exceeding 10 lakh cusecs i.e. average 3.5 floods in 10 years. It has been experienced that most of post flood are due to contribution from downstream catchments on which Hirakud has no control. Table 7.21 gives the clear picture.

Table 7.21 : Peak discharge at Mundali showing Contribution from d/s Catchments

Sl. No.	Flood Period	Peak discharges at Mundali (lakh cusec)	Contribution from downstream catchments (lakh cusec)			Release from Dam in lakh cusec
			Dam to Khairmal	Khairmal to Mundali	Total Dam to Mundali	
1.	29/08/82	15.84	12.81	2.94	15.75	0.09
2.	12/08 to 21/08/91	12.71	6.74	4.81	11.55	1.16
3.	16/08 to 26/08/92	11.05	9.80	0.85	10.65	0.40
4.	14/07 to 22/07/01	14.0	-	-	9.48	4.52
5.	29/08 to 05/09/03	13.50	11.04	1.62	12.66	0.84
6.	30/8 to 4.9.06	13.08	-	-	12.52	0.56
7.	15/9 to 25.9.08	15.81	-	-	13.18	2.63

It is not possible to moderate flood of 16 lakh cusecs by more than 2 lakh cusecs. When the reservoir is full and another storm approaches in quick succession it will not be possible to hold water in the Dam. It may be noted that the Hirakud reservoir is small compared to its catchments.

"There were unprecedented floods over large areas of the United States mid-west in the summer of 1993, with the Mississippi and Missouri overflowing their banks. Extraordinary and persistent rains brought on the deluge. *Dam builders have never aspired to playing God.*" (Source: Winning the Future by B.G. Verghese, Konark Publishers Pvt. Ltd. 1994, Pg. 24).

In the present scenario it may not be possible to construct a second flood control reservoir (at Tikarapara as was envisaged in the Original Project Report or at Manibhadra) due to submergence and R&R problem. To moderate flood upto its safe limit at the head of delta, it may be prudent to follow lower band of Rule Curve, which will provide some flood space in the reservoir. By adopting this, we may loose some power but irrigation is not affected. This aspect needs detailed examination and if required, rule curve may be revised.

Details of peak inflow and outflow from and to Hirakud Reservoir from 1961 to 2009 are furnished vide Table 7.22.

Table 7.22 : Details of Peak Inflow & Outflow

Year	Month	Duration	Peak inflow to Hirakud Reservoir			Maximum release from dam		
			Date	Time	Inflow (lakh cusec)	Date	Time	Spillway discharge (lakh cusec)
1	2	3	4	5	6	7	8	9
1961	July	4/7 to 13/7	10/7	03	15.25	10/7	15	11.56
	July	16/7 to 26/7	17/7	15	8.16	17/7	21	06.33
	August	23 to 27/8	26/8	06	5.55	26/8	18	04.32
	Sept.	1/9 to 11/9	8/9	21	9.10	09/9	00	7.34
	Sept.	14/9 to 11/9	16/9	12	6.95	14/9	15	6.22
1962	August	3/8 to 6/8	4/8	06	2.78	04/8	06	2.18
1963	August	10/8 to 13/8	11/8	00	6.50	11/8	01	5.55
1964	July	30/6 to 8/7	8/7	08	7.60	06/7	23	6.16
	August	16/8 to 19/8	18/8	08	8.61	20/8	01	5.82
1965	August	22/8 to 26/8	25/8	06	10.52	27/8	10	6.05
	Sept.	23 to 29/9	24/9	03	5.95	24/9	00	5.87
1966	July	29/7 to 2/8	31/7	03	5.09	02/8	00	3.57
1967	August	2 to 5/8	3/8	12	8.30	05/8	01	5.79
	August	20 to 26/8	24/8	06	6.38	24/8	18	4.14
1968	August	11 to 23/8	14/8	03	7.29	14/8	18	5.87
1969	August	1 to 6/8	02/8	12	2.79	02/8	09	3.05
1970	July	30/6 to 11/7	03/7	21	6.66	03/7	22	4.93
	August	23/8 to 18/9	26/8	06	4.92	26/8	16	4.65
1971	July	27 to 30/7	28/7	21	8.25	29/7	16	5.09
	August	3 to 5/8	03/08	12	5.71	03/8	21	3.63
	August	30/8 to 2/9	31/8	00	6.29	31/8	20	4.54
1972	Sept.	12 to 18/9	13/09	15	4.15	13/9	14	5.12
1973	July	9 to 17/7	10/7	12	6.48	13/7	06	4.73
	August	19 to 22/8	20/8	06	6.37	21/8	00	4.00
	Sept.	2 to 6/9	03/9	06	6.61	04/9	23	6.54
	October	28 to 31/10	29/10	15	5.10	29/10	08	5.11
1974	August	17 to 21/8	17/8	19	6.14	19/8	12	4.75
1975	August	20 to 30/8	21/8	13	7.98	21/8	17	6.18
1976	August	4 to 8/8	6/8	08	4.46	06/8	08	1.94
	August	11 to 18/8	15/8	15	7.54	16/8	15	4.95
1977	July	27/7 to 02/08	30/7	08	4.40	30/7	08	2.50
	August	7 to 11/8	08/8	06	6.35	10/8	08	3.88
1978	August	17 to 21/8	18/8	03	5.87	20/8	09	3.70
	August	26/8 to 03/9	30/8	08	6.53	27/8	12	4.88
1979	August	9 to 13/8	10/8	03	5.40	10/8	00	3.13
1980	Sept.	12 to 16/9	15/9	03	4.64	15/9	03	3.82
	Sept.	19 to 30/9	20/9	12	13.32	20/9	14	11.79
1981	August	16 to 26/8	23/9	15	4.74	22/8	12	3.10
1982	August	7 to 28/8	19/8	06	6.00	14/8	18	3.65
	August	30/8 to 4/9	31/8	09	9.50	01/9	03	5.54
1983	Sept	1 to 13/9	07/9	03	5.6	07/3	03	3.73
1984	August	9 to 11/8	10/8	06	5.55	11/8	18	4.38
	August	16 to 25/8	18/8	06	5.64	20/8	12	5.57
1985	August	30/7 to 14/8	09/8	03	5.20	11/8	09	3.40
1986	June	25 to 30/6	29/6	19	8.60	30/6	18	4.29
	August	21 to 22/8	22/8	00	4.60	22/8	12	3.71
1987	August	28 to 31/8	28/8	06	5.20	–	–	NIL
1988	August	03 to 10/8	04/8	14	4.96	06/8	10	3.10
1989	July	27/7 to 01/8	29/7	03	2.85	–	–	NIL
1990	August	29/8 to 10/9	06/9	21	4.46	07/9	03	3.41
	Sept.	10 to 21/9	16/9	00	5.40	16/9	00	5.38
1991	July	20/7 to 04/8	24/7	12	5.37	02/8	03	2.14
	August	12 to 21/8	14/8	03	6.76	14/8	18	2.80
	August	22/8 to 01/9	25/8	03	4.94	25/8	15	3.40
1992	July	28/7 to 02/8	30/7	06	4.76	01/8	15	2.29
	August	16 to 30/8	22/8	15	5.85	23/8	06	5.22

Contd...

Table 7.22 contd..

1	2	3	4	5	6	7	8	9
1993	August	14/8 to 26/8	21/8	15	4.17	21/8	21	3.40
1994	June	19/6 to 25/6	21/6	18	7.27	23/6	21	3.76
	July	8/7 to 19/7	10/7	12	9.09	14/7	3	7.33
	July	20/7 to 25/7	21/7	06	7.05	21/7	3	6.19
	August	3/8 to 9/8	04/8	17	6.50	07/8	0	3.89
	August	18/8 to 26/8	20/8	21	4.66	21/8	15	3.91
	Aug-Sept.	27/8 to 10/9	01/9	06	6.08	01/9	18	5.73
1995	July	18/7 to 29/7	25/7	18	5.84	26/7	18	4.60
1996	July	21/7 to 30/7	27/7	09	4.75	27/7	6	3.26
1997	August	17/8 to 28/8	23/8	18	4.75	23/8	18	4.75
1998	Sept.	7/9 to 17/9	11/9	21	6.85	12/9	3	6.85
1999	August	4/8 to 15/8	09/8	15	4.92	9/8	21	4.53
2000	August	15/8 to 23/8	18/8	09	3.16	–	–	–
2001	July	4/7 to 31/7	18/7	07	9.54	18/7	12	8.54
	August	19/8 to 28/8	22/8	21	4.79	22/8	21	4.22
2002	Sept.	7/9 to 15/9	12/9	15	3.65	12/9	18	3.43
2003	Aug-Sept.	26/8 to 15/9	30/8	12	8.72	31/8	0	8.85
2004	August	07/8 to 17/8	13/8	06	5.96	13/8	21	5.57
2005	August	30/7 to 10/8	05/8	21	4.87	06/8	21	4.67
	Sept.	14/9 to 20/9	15/9	21	4.79	16/9	09	4.71
2006	August	13/8 to 26/8	14/8	18	5.29	15/8	12	3.50
	Aug-Sep	29/8 to 04/9	01/9	24	4.45	01/9	24	4.45
2007	August	06/8 to 15/8	09/8	09	2.77	09/8	21	1.85
	August	19/8 to 27/8	20/8	24	2.88	21/8	15	2.22
2008	Sep.	15/09 to 25/09	20/09	15	7.51	20/09	12	7.91
2009	July	20/07 to 29/07	23/07	06	5.93	23/07	09	5.12

7.3 Power Generation

7.3.1 Introduction

Power development in India commenced at the end of 19th century wth the commissioning of Hydro power plant rated at 130 kw at Sidrapong in Darjeeling during 1897 followed by first steam driven power plant rated 1000 kw two years later at Kolkata in 1899 by CESC. In the year 1902, a hydro electric project was also commissioned at Deogarh in Orissa for supplying power to the palace and some neighbouring areas.

The details of installed capacity at the end of 10th plan (31.12.2008) from all sources of generation (i.e. Hydro, Thermal, Renewable source of energy sector wise i.e., state, private and Central are furnished in Table 7.23.

Table 7.23 : Installed capacity at the end of 10th plan

Sector	Hydro	Thermal	Renewable	Total (MW)
State	1935	880	1.3	2816.3
Private	0	0	0	0.0
Central	49	656	0	705.0
Total	1984	1536	1.3	3521.3

The yearwise generation of individual hydropower stations of the state from 1996-97 to 2007-08 is given in Table 7.24 (Vide Power Scenario of Orissa – An overview, jointly prepared by Energy Dept., Government of Orissa and CBIP, New Delhi, 2009 Pg. XV)

Table 7.24 : Yearwise Hydro Power Generation in MU

Year	Hirakud	Balimela	Rengali	Upper Kolab	Upper Indravati	Machhakund (Orissa drawal)	Total (MU)
1996-97	975	1185	783	727	–	362	4032
1997-98	937	919	988	469	–	297	3610
1998-99	1218	807	934	434	–	245	3638
1999-00	1115	1215	899	808	490	338	4865
2000-01	586	1007	718	520	1769	318	4918
2001-02	962	1071	793	657	2965	332	6780
2002-03	648	546	644	497	807	266	3408
2003-04	956	1145	1052	657	2141	199	6149
2004-05	845	1526	750	896	2851	366	7234
2005-06	909	1055	679	624	1763	324	5354
2006-07	862	1621	670	1026	3019	342	7540
2007-08	980	1834	983	1108	2979	175	8059

Table 7.25 shows monthwise power generation (from 1960-2008) separately from Power Houses located at Burla (BPH) and Chiplima (CPH). Table 7.26 gives the quantum of water drawn for the above purpose, which is non-consumptive one. The physical quantity of water which passes through the generating turbines even though does not diminish but at the same time the water is used in the sense that it is no longer available at the upstream level (vide Integrated River Basin Development, Report by Panel of Experts, United Nations, Dept. of Economic and Social affairs, N.Y. 1958, Pg.4). The tail water is utilised mostly for irrigation in Delta when it becomes consumptive.



Danta Aqueduct on Bargarh Main Canal

Table 7.25 : Monthly Power Generation from Hirakud (1960-2008)

(All figures in Million Units)

Years	Jan		Feb		Mar		Apr		May		Jun	
	BPH	CPH	BPH	CPH	BPH	CPH	BPH	CPH	BPH	CPH	BPH	CPH
1	2	3	4	5	6	7	8	9	10	11	12	13
1960	38.7000		35.5300		39.1100		35.0700		37.2600		41.8700	
1961	44.5600		39.7800		46.4200		40.2300		48.7700		48.8100	
1962	59.7574		56.9294		62.2852		60.8911		51.6840		49.3208	
1963	43.4618	32.4852	40.9038	31.0809	47.5837	34.7879	42.4135	33.7700	15.1480	30.9600	7.0930	29.0800
1964	46.000	31.4900	37.8400	26.3200	42.2100	30.1300	39.0000	29.6100	48.3200	33.7200	39.9000	32.9300
1965	50.9500	33.6400	45.5900	31.7400	54.1800	38.8700	46.7400	35.9100	43.6500	35.7800	39.0700	35.3200
1966	43.1869	29.7105	39.8396	28.8375	40.3843	31.2025	33.9348	28.7805	26.7331	25.3830	24.5605	24.0720
1967	11.0435	11.1305	12.6393	12.5059	15.9535	15.6284	16.5275	15.7080	11.6743	12.5482	13.2967	14.2220
1968	51.9946	35.1948	41.2438	29.2346	35.8252	26.8539	47.9258	34.0413	40.9073	33.5115	35.1735	33.6809
1969	45.5739	31.7107	34.0967	25.5487	27.8017	22.3458	21.4134	18.1454	22.2134	20.2786	21.7644	21.2279
1970	41.0697	28.0815	28.9982	20.2385	42.1497	31.1568	36.0723	28.9453	43.9746	38.5400	44.5830	32.4549
1971	44.2199	29.6820	38.3802	26.4325	40.8324	29.9935	39.9010	31.1785	54.1122	38.9528	45.4820	37.7030
1972	81.4343	35.5228	45.3865	32.0055	54.0362	35.9365	43.0282	33.9000	28.1815	24.8520	21.0132	22.2690
1973	60.5000	41.0770	46.5105	31.3487	50.9352	36.7488	44.5310	31.4990	15.9063	14.9469	12.8970	14.4450
1974	65.3478	44.0460	57.1335	38.6404	47.6685	34.8370	40.8900	31.9956	31.9280	29.9334	31.4080	31.0025
1975	16.4399	14.3512	17.2101	14.7584	15.4420	14.4035	16.2872	15.6363	11.8239	11.8064	12.6815	8.5720
1976	50.0100	32.0860	39.6080	27.5570	45.3370	33.1696	40.4600	31.9719	37.0225	31.3895	29.7545	29.4945
1977	29.7339	20.6341	43.5218	32.6486	44.5255	35.5803	31.2695	27.9695	17.3745	16.9400	30.7085	24.1875
1978	41.9800	29.8713	41.0770	29.9031	51.8010	40.0025	44.7600	37.2790	37.2970	34.0221	30.5280	30.7154
1979	48.7230	33.4595	38.3780	28.7125	42.8300	32.5650	47.4050	39.8275	37.1020	34.8530	23.0270	24.3415
1980	25.2900	19.2600	18.1300	15.0900	29.2500	25.2300	16.7500	16.2100	6.0600	6.5300	22.9800	14.5000
1981	42.2000	28.9900	45.7800	33.7400	47.0700	37.3000	36.9500	32.4800	30.6700	30.8100	23.8400	25.6700
1982	40.6800	29.7745	37.6820	27.5255	43.7920	33.4224	48.8310	41.9345	42.1390	41.0355	17.1660	19.2825
1983	33.4500	24.1720	35.0500	26.7977	50.0250	42.0060	36.5380	35.4032	16.8810	18.8356	2.8570	2.6423
1984	44.0120	30.1040	42.5500	30.8234	46.2220	35.9258	41.1890	35.2850	35.0870	33.5686	43.8620	34.0354
1985	37.3500	27.2998	32.0040	24.0592	41.3610	32.9435	31.5890	28.5100	17.6810	18.5225	15.2000	18.1560
1986	52.5750	37.2320	42.7720	32.1030	38.8540	30.5698	38.8270	32.4782	34.2950	31.7410	36.2710	29.3410
1987	44.9960	31.8816	37.2490	26.8770	39.2350	30.2257	31.0120	28.1340	26.3820	25.2875	21.3310	23.3040
1988	42.9270	29.2490	37.1180	25.0720	46.0000	32.2450	54.0790	42.3210	49.1180	45.1590	32.6820	36.5577
1989	0.0000	0.0000	3.3530	1.3927	21.8290	14.4210	38.6450	18.9500	34.4490	27.7260	60.8920	27.0060
1990	36.8680	26.9740	31.4060	23.4410	33.2970	25.8310	27.8340	22.9350	27.2440	23.6680	57.8930	25.9350
1991	57.9500	39.2549	33.4380	20.1505	54.4810	32.8441	67.3580	28.5390	55.8310	42.3530	47.0470	32.7170
1992	60.0470	42.1090	42.3580	30.9685	33.4590	26.0625	29.9340	25.6190	31.5110	29.5020	29.2090	29.6060
1993	47.2040	33.3724	27.8790	20.7840	28.6270	22.1225	29.8140	23.0070	35.3610	31.5166	25.1160	20.3564
1994	45.2660	31.1720	40.6400	28.6920	42.4740	30.9050	29.6510	23.7530	34.0960	30.2220	49.7970	26.8980
1995	45.8020	32.3390	42.8940	30.3270	60.5250	31.0070	66.0850	26.9540	56.4130	26.3450	32.6430	27.8280
1996	41.9670	28.2780	41.7590	24.9340	42.5040	22.1560	57.3430	20.4200	50.5610	18.6210	32.5570	14.1780
1997	47.5280	24.0730	35.7610	16.8530	32.8120	18.7690	43.7740	15.3260	38.9580	6.0794	22.4580	12.6246
1998	57.6530	18.8980	67.4740	19.8230	68.0385	16.6540	66.3280	17.6950	83.6950	19.9350	96.1400	21.0420
1999	76.4240	13.1960	51.8860	10.3060	43.3950	10.9636	38.2680	7.9770	23.3960	10.9005	51.4100	12.1905
2000	61.5610	16.4925	46.8110	15.5100	49.8800	15.5120	51.4140	14.9390	26.7700	13.6460	38.1340	3.3180
2001	14.9570	11.5465	10.6720	7.9055	11.0190	8.7736	9.1740	7.0594	20.8310	11.2590	52.9130	13.5350
2002	55.6440	11.7900	37.0280	12.4315	24.4370	12.1900	25.2790	10.3860	35.7380	9.6620	55.7110	7.0510
2003	32.6570	9.3080	18.2060	8.4690	26.0760	9.5110	29.2040	8.8120	21.2600	7.1910	29.7750	6.1920
2004	46.6940	12.1850	42.3780	14.8250	57.1880	13.0540	57.1800	5.9550	41.6640	2.4640	43.5800	15.4910
2005	27.0060	9.1670	26.5810	8.5090	34.8600	10.6410	41.9360	10.5090	33.2670	10.8080	20.7820	8.8280
2006	41.4000	9.4850	29.3070	8.0750	31.6970	10.1290	43.2650	9.3570	53.3560	9.7640	41.7200	9.0780
2007	22.1030	9.6910	15.2820	8.0340	19.7080	9.8340	28.3550	8.4940	46.1110	8.4950	58.8940	6.7730
2008	21.6820	7.5720	23.5290	6.4510	25.3540	8.6330	21.0890	8.5320	26.9530	8.3340	55.0080	9.0870
2009	37.017	14.308	26.422	14.874	19.247	12.166	18.593	7.573	7.722	4.671	8.346	6.426

Contd...

Table 7.25 (continued)

Jul		Aug		Sep		Oct		Nov		Dec		Year
BPH	CPH	BPH	CPH	BPH	CPH	BPH	CPH	BPH	CPH	BPH	CPH	
14	15	16	17	18	19	20	21	22	23	24	25	
37.5200	-	34.8100	-	39.5700	-	42.6300	-	40.4000	-	43.8400	-	1960
47.4700	-	43.8800	-	42.2777	-	40.3714	-	51.8247	-	52.3972	-	1961
55.2652	9.9800	51.4076	17.4400	55.4000	16.8500	55.9225	17.8900	55.1604	19.8600	44.7927	32.2625	1962
49.4600	32.8000	44.8000	32.0600	48.3000	30.5900	47.2500	31.1900	54.2700	32.4900	49.3600	32.5200	1963
44.8900	36.2900	50.8100	35.7700	52.8900	37.3500	56.9900	33.9200	50.2600	32.4900	47.9700	30.8800	1964
45.5000	40.6900	46.8200	40.1600	49.6000	33.4200	52.8500	33.8800	47.6300	30.9500	45.4200	30.6400	1965
42.7377	33.4058	46.6662	39.4010	42.9007	32.5550	42.3438	32.2220	32.7826	26.0550	16.9639	15.7445	1966
40.7881	34.0807	54.2461	32.6969	49.7205	32.0535	47.7135	29.8700	51.8953	32.9267	54.9316	36.1555	1967
36.8279	32.3567	41.1936	31.8225	41.3174	29.9559	46.5645	30.7231	42.4251	28.1973	37.4911	25.4271	1968
38.6463	34.3560	51.5071	34.8823	56.6500	36.1935	58.3556	36.8210	41.5532	27.1982	48.1218	29.1200	1969
54.3804	38.5220	63.7759	39.4112	62.9968	39.2968	68.4361	41.4320	52.4757	33.9715	42.0265	28.4150	1970
49.8820	38.1928	56.4310	41.2732	69.3890	41.7995	69.0984	39.4255	65.0172	41.1977	63.3619	40.6863	1971
45.9653	31.7230	63.1460	42.1092	64.3742	39.2415	71.8750	43.2085	62.8740	39.7180	64.8080	42.3670	1972
49.4198	31.5262	70.0463	35.0433	70.0994	35.2889	67.5782	41.0458	58.7671	37.7530	62.0428	38.9750	1973
36.2380	30.8275	60.1213	36.7766	65.8110	39.2970	29.9991	24.7755	13.2730	11.4697	12.5709	10.5306	1974
54.2966	22.8280	67.8122	32.8402	72.3888	22.8244	64.0400	37.8956	58.4022	36.1667	44.4850	29.0521	1975
45.3925	28.6350	73.9520	33.0000	77.7750	29.3700	41.5780	28.2755	29.0245	20.9392	22.6337	4.5242	1976
69.2600	47.0016	82.2400	26.2190	95.0220	37.5095	78.3140	43.2481	51.8260	34.8200	40.3870	28.4385	1977
59.2520	40.1988	83.9170	40.4162	89.0500	41.3847	67.3010	41.4762	41.9730	28.3046	43.7150	29.7040	1978
40.5000	32.9500	80.1200	40.0100	25.6400	19.7400	17.0700	11.6700	13.2500	10.3100	23.0900	23.3400	1979
99.0600	39.9300	121.860	41.620	111.240	38.8600	73.1500	45.2200	47.6600	33.4100	38.6600	26.5200	1980
75.4390	40.2530	117.411	42.338	131.360	43.6810	88.1590	44.0300	49.5720	34.0856	41.1870	29.6960	1981
22.5850	21.5285	59.3970	31.9360	86.0840	42.4650	51.5155	37.3165	48.3175	33.9215	25.4340	19.1343	1982
70.4050	39.7489	120.142	44.5150	127.2380	36.3889	116.7880	31.5220	47.9040	29.8510	43.8580	30.4534	1983
85.1960	43.7820	108.203	43.2030	94.1670	44.0050	83.2800	42.5650	44.3170	31.3110	49.6360	35.0380	1984
82.1350	29.9175	111.722	29.4910	120.0580	30.6260	117.2540	34.7120	51.0200	34.4605	54.0920	38.9230	1985
93.3710	39.2770	110.961	42.7170	82.0620	44.1380	84.0070	42.9190	57.5820	39.3871	45.8880	31.9745	1986
79.0540	34.8470	73.8680	39.7520	104.1410	42.0310	72.8690	45.2010	60.5300	35.6590	44.1210	29.2360	1987
91.2350	26.6940	94.2140	29.0320	99.7130	28.7490	94.1420	29.5300	35.1190	23.8420	0.2370	0.9100	1988
93.0110	26.5620	122.453	27.9370	112.7250	26.9020	79.7460	27.3300	38.0900	25.9420	37.7230	27.0380	1989
84.2720	35.2523	96.7100	32.0060	106.8410	38.2760	118.7080	43.0525	90.4520	39.3335	67.1480	43.1475	1990
52.8610	31.3915	123.3390	37.5000	109.9400	37.4910	82.0640	42.4860	63.1540	41.9740	63.3250	41.8640	1991
57.5510	32.4500	139.4530	41.0160	113.6830	39.6280	56.6740	37.1370	54.1800	36.0070	56.5180	10.2410	1992
94.4850	33.8940	106.8160	41.8750	108.3060	40.2320	115.6520	42.8990	66.2780	41.4930	64.7830	42.9130	1993
87.3630	27.9980	116.1500	20.8180	88.0060	21.6980	113.2840	31.4320	92.1510	31.2510	59.7280	31.1610	1994
65.5130	26.9880	122.4620	30.2500	111.1650	29.7550	86.6610	31.1830	65.2760	26.1510	47.6960	23.4690	1995
94.0070	19.6970	106.4490	24.7840	97.3120	23.7320	76.2540	24.8550	46.1680	25.5100	43.9920	23.9070	1996
42.2830	22.1090	78.5700	26.1340	76.1850	27.3690	56.6950	30.3280	55.4250	27.1390	76.1480	30.8410	1997
118.961	15.5560	92.4330	16.6390	105.597	14.4530	130.9920	10.2490	108.9500	1.6897	84.5110	12.2145	1998
83.6760	20.4310	137.093	18.7390	135.298	16.2890	135.1000	10.7815	100.7160	15.0425	72.5670	19.5385	1999
70.1280	10.2800	82.9380	17.1520	99.1090	22.0050	24.3910	11.2566	11.9260	8.5782	8.8260	6.0800	2000
116.289	12.2510	116.591	13.7500	135.4240	13.8200	0.0000	15.1010	70.4670	15.2190	61.0280	14.5330	2001
41.5820	7.5050	83.9450	1.9920	137.8930	1.4475	56.2730	0.0000	26.6250	5.2720	28.2810	8.6530	2002
80.0080	6.5340	103.866	6.2260	117.7290	7.3290	120.2340	8.6410	110.6920	9.2760	84.3740	11.8715	2003
73.4150	14.6080	119.504	13.8490	128.4990	14.3040	108.2580	12.6350	18.9440	12.0380	23.7630	10.7760	2004
87.9940	10.4500	104.4470	11.0710	136.8310	9.3600	129.7330	11.0600	79.9340	11.0640	49.6590	11.0650	2005
62.5690	12.3460	144.3910	11.7920	151.2300	6.9220	93.2340	9.0010	50.0470	9.0830	49.8270	10.7100	2006
126.9590	9.8010	142.6400	7.9410	143.5110	6.2920	144.2290	7.4660	83.0280	7.3250	45.3900	7.5490	2007
141.3580	12.8490	152.3040	15.5140	152.5830	14.6820	102.0950	15.9370	43.4890	9.3450	40.3530	4.2780	2008
103.978	14.442	140.341	14.495	132.449	15.490	63.526	19.022	26.560	10.885	14.559	3.143	2009

N.B. : B.P.H.=Burla Power House, C.P.H.=Chiplima Power House

Table 7.26 : Monthwise Drawal of Water for Power Generation from 1958 - 2009

(All figures in M.Ac.ft.)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
1958	0.098	0.118	0.145	0.155	0.212	0.241	0.277	0.271	0.226	0.242	0.215	0.241	2.441
1959	0.414	0.421	0.515	0.418	0.539	0.557	0.600	0.592	0.522	0.468	0.403	0.416	5.865
1960	0.444	0.419	0.473	0.440	0.507	0.598	0.535	0.454	0.465	0.464	0.418	0.452	5.669
1961	0.471	0.414	0.523	0.483	0.602	0.615	0.559	0.533	0.469	0.426	0.534	0.556	6.185
1962	0.627	0.605	0.683	0.733	0.717	0.731	0.848	0.780	0.647	0.599	0.616	0.473	8.059
1963	0.471	0.465	0.583	0.563	0.520	0.544	0.766	0.682	0.560	0.485	0.562	0.525	6.726
1964	0.503	0.425	0.463	0.482	0.663	0.592	0.732	0.686	0.612	0.613	0.529	0.525	6.864
1965	0.561	0.518	0.626	0.586	0.583	0.567	0.681	0.666	0.545	0.549	0.487	0.495	6.864
1966	0.490	0.472	0.508	0.435	0.387	0.371	0.625	0.650	0.516	0.519	0.416	0.221	5.610
1967	0.147	0.171	0.229	0.240	0.171	0.208	0.596	0.754	0.551	0.487	0.553	0.582	4.689
1968	0.581	0.479	0.428	0.636	0.594	0.558	0.528	0.562	0.494	0.501	0.468	0.416	6.245
1969	0.530	0.421	0.370	0.297	0.329	0.342	0.567	0.663	0.646	0.617	0.436	0.522	5.740
1970	0.457	0.338	0.515	0.477	0.649	0.671	0.798	0.866	0.707	0.729	0.556	0.448	7.211
1971	0.489	0.439	0.486	0.517	0.823	0.673	0.718	0.780	0.822	0.647	0.684	0.685	7.763
1972	0.576	0.535	0.692	0.610	0.422	0.328	0.667	0.845	0.686	0.750	0.667	0.706	7.484
1973	0.689	0.563	0.664	0.643	0.229	0.208	0.698	0.880	0.770	0.710	0.611	0.654	7.319
1974	0.716	0.663	0.595	0.541	0.453	0.509	0.544	0.814	0.844	0.366	0.156	0.141	6.342
1975	0.201	0.219	0.207	0.245	0.178	0.209	0.809	0.886	0.820	0.679	0.635	0.490	5.578
1976	0.561	0.467	0.545	0.526	0.535	0.467	0.696	0.949	0.940	0.466	0.305	0.250	6.707
1977	0.318	0.499	0.544	0.412	0.245	0.474	1.017	1.051	1.045	0.837	0.538	0.417	7.397
1978	0.449	0.462	0.619	0.581	0.532	0.485	0.846	1.072	0.989	0.709	0.430	0.460	7.634
1979	0.523	0.437	0.510	0.617	0.528	0.352	0.620	1.028	0.299	0.195	0.155	0.026	5.290
1980	0.303	0.222	0.391	0.244	0.091	0.338	1.370	1.488	1.266	0.778	0.494	0.306	7.291
1981	0.460	0.524	0.565	0.488	0.441	0.391	1.142	1.514	1.501	0.951	0.514	0.432	8.923
1982	0.441	0.429	0.531	0.649	0.627	0.267	0.347	0.741	0.978	0.568	0.525	0.281	6.384
1983	0.383	0.420	0.654	0.532	0.264	0.045	1.134	1.527	1.433	1.339	0.509	0.469	8.709
1984	0.483	0.492	0.559	0.543	0.510	0.680	1.199	1.380	1.117	0.687	0.482	0.554	8.686
1985	0.443	0.377	0.526	0.442	0.268	0.249	1.195	1.357	1.334	1.339	0.538	0.620	8.688
1986	0.595	0.505	0.478	0.518	0.501	0.534	1.176	1.347	1.238	0.696	0.626	0.503	8.717
1987	0.508	0.438	0.487	0.420	0.391	0.344	1.095	0.955	1.149	0.771	0.639	0.462	7.659
1988	0.461	0.410	0.535	0.691	0.701	0.527	1.259	1.189	1.101	1.026	0.391	0.002	8.293
1989	0.000	0.036	0.252	0.493	0.474	0.973	1.497	1.544	1.247	0.756	0.402	0.410	8.084
1990	0.408	0.363	0.402	0.359	0.382	0.911	1.123	1.137	1.152	1.257	0.976	0.704	9.174
1991	0.640	0.358	0.631	0.834	0.780	0.721	0.754	1.402	1.158	0.859	0.659	0.682	9.478
1992	0.668	0.495	0.404	0.386	0.448	0.445	0.778	1.626	1.239	0.597	0.581	0.627	8.294
1993	0.526	0.311	0.331	0.379	0.501	0.348	1.326	1.260	1.134	1.194	0.702	0.693	8.705
1994	0.517	0.467	0.516	0.383	0.486	0.681	1.050	1.275	0.937	1.172	0.983	0.648	9.115
1995	0.498	0.483	0.716	0.847	0.780	0.524	0.926	1.447	1.178	0.927	0.707	0.507	9.540
1996	0.448	0.473	0.511	0.725	0.758	0.433	1.287	1.257	1.020	0.824	0.498	0.470	8.704
1997	0.537	0.426	0.398	0.561	0.564	0.372	0.593	0.900	0.808	0.594	0.587	0.796	7.136
1998	0.593	0.703	0.739	0.707	1.008	1.298	1.751	1.237	1.105	1.394	1.150	0.891	12.576
1999	0.822	0.606	0.518	0.504	0.335	0.784	1.238	1.627	1.424	1.427	1.063	0.759	11.107
2000	0.674	0.538	0.600	0.671	0.402	0.605	1.024	1.175	1.228	0.296	0.133	0.100	7.446
2001	0.173	0.124	0.137	0.122	0.298	0.761	1.416	1.328	1.458	1.133	0.761	0.656	8.365
2002	0.637	0.450	0.288	0.342	0.488	0.856	0.686	1.003	1.490	0.634	0.289	0.304	7.467
2003	0.379	0.211	0.316	0.399	0.288	0.437	1.128	1.212	1.251	1.265	1.173	0.928	8.987
2004	0.521	0.483	0.687	0.828	0.622	0.622	1.072	1.454	1.454	1.228	0.207	0.265	9.443
2005	0.308	0.316	0.422	0.567	0.486	0.296	1.221	1.231	1.502	1.328	0.890	0.546	9.113
2006	0.464	0.343	0.374	0.545	0.745	0.658	0.857	1.700	1.656	1.036	0.533	0.545	9.458
2007	0.262	0.179	0.233	0.359	0.600	0.862	1.729	1.739	1.538	1.570	0.926	0.487	10.484
2008	0.238	0.274	0.305	0.256	0.346	0.789	1.972	1.871	1.668	1.150	0.465	0.437	9.771
2009	0.422	0.310	0.248	0.234	0.119	0.120	1.347	1.937	1.510	0.707	0.311	0.155	7.420

7.3.2 Average Tariff and Sale of Power

At present, tariff of power from all six hydropower projects under the control of Orissa Hydropower Corporation (OHPC) is being fixed by Orissa Energy Regulatory Commission (OERC) every year through public hearing on the Aggregated Revenue Requirement (ARR) and tariff application of OHPC at the beginning of each financial year. For sale of power to GRIDCO (Grid Corporation of Orissa), long term PPA for Rengali Power Station has been signed between OHPC & GRIDCO and the signing of Power Purchase Agreements (PPAs) of other power stations is under process. Average tariff of OHPC Power stations in paise/Kwh from 1996-97 and yearwise sale of power (in MU) are furnished in Table 7.27 & Table 7.28.

Table 7.27 : Average Tariff of OHPC Power Stations (Paise/Kwh)

Year	Hirakud	Balimela	Rengali	Upper Kolab	Upper Indravati
1996-97	38.00	38.00	38.00	38.00	
1997-98	49.00	49.00	49.00	49.00	
1998-99	48.11	48.11	48.11	48.11	
1999-00	49.64	49.64	49.64	49.64	59.07*
					44.70*
2000-01	50.02	50.02	50.02	50.02	57.83**
					63.05**
2001-02	24.99	24.99	24.99	24.99	65.40
2002-03	24.73	24.73	24.73	24.73	63.82
2003-04	27.35	27.35	27.35	27.35	64.96
2004-05	28.67	28.67	28.67	28.67	62.86
2005-06	52.96	19.50	31.42	13.72	64.53
2006-07	57.10	21.82	35.56	16.35	65.50
2007-08	58.07	57.01	62.06	40.39	67.81
2008-09	52.11	52.61	49.40	25.82	67.28

Sources: OERC

* One Unit Operation and Two Units Operation.

** Two Units Operation and Three Units Operation.

Table 7.28 : Year-wise Sale of Power (in MU) from OHPC Power Stations

Year	Hirakud	Rengali	Upper Kolab	Balimela	Upper Indravati	Sub-Total (MU)	Machhkund (Orissa drawal) (MU)	Total sale to GRIDCO (MU)	MP/Chhatis-garh SEB (*)
1996-97	955	761	719	1149	-	3585	362	3947	-
1997-98	919	949	457	884	-	3210	297	3507	-
1998-99	1185	890	427	779	-	3280	245	3525	-
1999-00	1078	863	777	1174	486	4379	310	4689	-
2000-01	537	699	502	967	1736	4441	318	4759	-
2001-02	925	772	640	1049	2920	6307	325	6632	-
2002-03	616	621	473	526	790	3025	368	3293	-
2003-04	903	1028	640	1118	2110	5799	199	5998	-
2004-05	804	731	867	1495	2826	6723	367	7090	3
2005-06	859	668	611	1024	1751	4913	324	5237	17
2006-07	802	649	1003	1589	2973	7016	342	7358	17
2007-08	939	956	1074	1800	2948	7717	175	7892	17

Source: OHPC

The transaction of energy with GRIDCO for the FY 2007-08 has not been reconciled. The year-wise sale of power (in MU) from OHPC power stations rounded to the whole numbers is illustrated at Table 7.28

(*) Tariff for selling power to MP/Chhatisgarh for the year 2004-05 is Rs.0.48, 2005-06 and 2006-07 is Rs.0.627 per Kwh. Yearwise revenue earned from Hirakud Power System (i.e. Burla + Chiplima) from 2004-05 to 2008-09 is as follows:

Year	Revenue (Rs. Cr)
2004-05	23.21
2005-06	46.52
2006-07	47.06
2007-08	61.20
2008-09	59.11

7.4 Municipal & Industrial Water Supply :

7.4.1 Introduction :

Orissa has developed a State Water Plan for a period covering up to 2051 AD when the population of the State is expected to stabilise. With competing demands for water from the same sources, it is necessary to lay down the priorities in its allocation. The State of Orissa adopts the following order of priority in water allocation in tune with the National Policy.

- Drinking water and domestic use (human and animal consumption).
- Ecology.
- Irrigation, Agriculture and other related activities including Fisheries.
- Hydro Power.
- Industries including Agro Industries.
- Navigation and other uses such as tourism.

Any alteration in the above-mentioned priorities will demand formulation of a new policy.

(Source: State Water Policy, March 2007, D.O.W.R., Government of Orissa, Pg.8).

The State Water Plan will not only allocate the water resource to different sectors of priority, it will also have a perspective plan for development of these resources in important areas like drinking water, irrigation, hydro-power, etc. While developing these resources, people's needs, preservation of the ecological balance and enrichment of the ecosystem would receive adequate attention. The plan will be holistic, participatory and environmentally sustainable.

The present and projected future water requirement for all purposes has been assessed by the Department of Water Resources (DOWR) vide Table No.7.29.

Table 7.29 : Water requirement for different uses (in BCM) for Orissa

Uses	Year 2001			Year 2051		
	Surface	Ground	Total	Surface	Ground	Total
Domestic	0.798(1.97)	1.198	1.996(3.63)	1.202(1.87)	1.803	3.006(3.56)
Agriculture	18.000(44.4)	4.688	22.688(41.25)	40.000(62.4)	9.408	49.408(58.5)
Industry	0.606(1.50)	0.1000.706((1.28)	1.750(2.73)	0.200	1.950(2.31)	
Environment	21.000(51.8)	8.400	29.400(53.5)	21.000(32.7)	8.400	29.400(34.8)
Others	0.100(0.25)	0.100	0.200(0.36)	0.200(0.31)	0.200	0.400(0.47)
Total	40.504(100)	14.486	54.990(100)	64.152(100) (58.3)*	20.010 (38.1)*	84.463(100) (53.59)*

* Percentage by which present level of requirement gets increased by 2051.

Figures in brackets indicate percentage of the total. Taken from Annual Report 2006-2007, DoWR, Govt. of Orissa : Table 2.4

(Source: Report of High Level Technical Committee to study various aspects of water usage from Hirakud Reservoir, Govt. of Orissa, Aug. 2007, Pg.11).

The demand pattern for water from various sectors is changing fast. It has been estimated that total water demand may increase from 54.99 BCM at present to 84.463 BCM (53.59% more) by 2051 vide Table 7.29. Keeping in view the constraints regarding availability of water, priority of uses, sensitivity and complexity of water issues, water allocation needs to be addressed judiciously.

7.4.2 Municipal Water Supply from Hirakud Reservoir

Municipal water supply is a very small component of total water use. Part of Sambalpur town, Burla and Hirakud are getting supply from Hirakud reservoir through the existing outlets provided in the dam. Towns like Brajaraj Nagar and Belpahar situated in the reservoir periphery also draw their demand from the reservoir. Present use is about 0.006 MAc.ft. during non-monsoon (Oct. to May) period.

7.4.3 Industrial Water Supply

In the original project report, it was not contemplated to supply water to the industries from the reservoir. Its primary objectives were irrigation, flood control and power generation. But the recommendation made by Dr. A.N. Khosla (the then Chairman of CWINC) in June 1947 contained provision for water to industries. Eminent engineer M.S. Thirumale Iyengar, the C.E. of the project stated in his paper presented "To the Nation" that '... one such in Orissa across river Mahanadi formulatd and executed by the Central Government on behalf of the State is the Hirakud project that will provide water and power for irrigation and industries all the year round." Subsequently the situation demanded to supply water to the industries. Table 7.30 shows the quantum of water allocated and to be allocated to the industries.

Table 7.30 : List of Industries allocated/to be allocated to draw water from Hirakud Reservoir

(A) Already Allocated

Sl. No.	Name of Industries	Location	Allocated		Present Drawal
			In cusecs	in MAF/Yr	Drawal in MAF/yr
1.	ITPS Ltd. (IB Thermal)	Bandhabahal	12.353	0.0090	0.0090
2.	MCL	Brajrajnagar	3.710	0.0030	0.0030
3.	Orient Paper Mill	Brajrajnagar	1.840	0.0010	0.0010
4.	S.E. Railway	Jharsuguda	2.470	0.0020	0.0020
5.	HINDALCO (Aditya Aluminium)	Lapanga, Jharsuguda	52.730	0.0380	0.0026
6.	SMC Power Generation	Hirma, Jharsuguda	2.450	0.0020	0.0020
7.	Shyam DRI Power Ltd.	Rengali(Sambalpur)	11.000	0.0080	0.0080
8.	Rathi Steel & Power Ltd.	Pottapali	9.810	0.0070	0.0000
9.	SPS Sponge Iron Ltd.	Barmel, Jharsuguda	9.810	0.0070	0.0070
10.	Bhusan Ltd.	Lapanga, Jharsuguda	1,00.000	0.0720	0.0087
11.	INDAL	Hirakud	10.000	0.0070	0.0072
12.	Action Ispat & Power Ltd.	Marakutta, Jharsuguda	1.512	0.0010	0.0010
13.	Viraj Steel & Energy Ltd.	Rengali, Sambalpur	3.630	0.0030	0.0030
14.	NTPC	Darlipali, Sundargarh	160.000	0.1160	0.0000
15.	VEDANTA Aluminium Ltd.	Bhurkhamunda, Jharsuguda	54.900	0.0400	0.0000
	Total		**436.215**	**0.3160**	**0.0545**

(B) Pending for Allocation

1.	Bharat Electric & Industries Ltd.	Lapanga, Sambalpur	2.040	0.0010	
2.	Sterlite Energy Ltd.	Jharsuguda	73.200	0.0530	
3.	Ma Samaleswar Industries Ltd.	Lapanga, Sambalpur	3.430	0.0020	
4.	Ultra Mega Power Project	Bhasma, Jharsuguda	160.000	0.1160	
5.	Aryan Ispat and Power	Rengali, Sambalpur	14.720	0.0110	
	Total		253.390	0.1830	
	Grand Total		**689.605**	**0.4990**	**0.0545**

(Source : Report of High Level Technical Committee, August 2007, Pg.29, Govt. of Orissa)

For the water supplied to the industries the State is collecting water tax @ Rs.250.00 per one lakh gallon. Amount of tax collected by the Bargarh Canal Division, Bagarh and Main Dam Division, Burla is furnished in Table 7.31 and 7.32.

Table 7.31 : Collection of Water Tax by Bargarh Canal Division, Bargarh

Year	Amount collected (Rs.)	
	IDCOL/ACC	Sugar Factory, Bargarh
1999-2000	367193	-
2000-2001	361887	-
2001-2002	521801	-
2002-2003	1479923	175250
2003-2004	836268	41750
2004-2005	774716	41750
2005-2006	774716	41750
2006-2007	715286	41750
2007-2008	836269	41750
2008-2009	583690	41750
2009-2010	965741	41750
Total	**80,92,262**	**4,67,500**

The beneficiaries of the command area are apprehensive that by supplying water to the industries, there may be water shortage for irrigation. Further due to siltation the reservoir capacity is also reducing. To dispel this notion from the minds of cultivators that there is no scarcity of water for irrigation, Government in Department of Water Resources had constituted a committee to look into this aspect. The committee opined that the reasonability of the water demanded by the Industries need to be checked up and confirmed by relevant department. Industries generally have over 80% regenerated water for reuse by recycling, which shall be made compulsory and, only consumptive use and evaporation loss needs to be drawn from the reservoir. Disposal of wastes, environmental safeguards, acceptable quality of any water returned to the reservoir shall be monitored by the Pollution Control Department/Concerned Agencies. Water actually drawn shall be charged at the rates fixed along with compensation for the loss of power generation on the water consumed during the period October to May.

Another study was made by G.C. Sahu and J. Jena to assess the quantum of water that will be saved by providing C.C. lining (vide Institution of Engineers, India - Orissa State

Table 7.32 :Water Tax Collected by Main Dam Division, Burla

Sl. No.	Name of the Industry	1999-00	2000-01	2001-02	2002-03	2003-04	2004-05	2005-06	2006-07	2007-08	2008-09	2009-10	2010-11	Total (1999-00 to 2010-11 (upto 30th June)
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1.	Orient Paper Mill, Brajrajnagar	0	97516	55000	60000	0	80000	160000	0	160000	80000	71646	21464	785626
2.	S.E. Railway, Jharsuguda	0	0	0	0	0	0	0	0	0	0	0	0	0
3.	M.C.L., Brajrajnagar	0	251912	528697	361495	291943	351360	320640	380160	320640	292800	380160	58640	3526447
4.	B.I.T.S. Bandhabahal	0	328521	163616	324223	294957	336373	290389	354451	332290	243856	586776	205585	3461037
5.	I.T.P.S.,Banharpalli	0	0	0	4082375	5228134	5619852	5773495	5641757	4081613	5431770	8378362	807185	45044543
6.	Lilari Open Cast Project	0	24000	0	44726	1658	14896	33656	1952	18434	45000	33000	0	217322
7.	Lakhanpur Open Cast Project	0	39996	46884	187264	104996	61982	141454	76122	70400	200000	260000	40000	1231118
8.	Tata Refractories Ltd.	0	269429	188013	193851	208022	232288	189818	213941	170238	159261	196860	31394	2053115
9.	O.H.P.C. Burla	0	0	0	0	0	0	0	0	0	0	0	0	0
10.	P.H.D. Sambalpur	0	0	0	0	0	0	3000000	3000000	0	964674	500000	0	7464674
11.	M/s Bhusan Ltd.	-	-	-	-	-	0	868300	1700949	2378572	3332436	4325750	1436201	14042208
12.	M/s S.M.C. Power Gen.Ltd.	-	-	-	-	-	0	0	1256753	1200000	4567632	919777	293878	8240040
13.	M/s Shyam DRI Power Ltd.	-	-	-	-	-	0	0	0	1178145	1513322	2534904	676513	5902884
14.	M/s Action Ispat & Power Ltd.	-	-	-	-	-	-	-	0	1064083	745816	1544618	187473	3561990
15.	M/s F.S.P.L. Ltd. Jharsuguda	-	-	-	-	-	-	0	0	0	400000	600000	200000	1200000
16.	M/s Vedanta Alumina Ltd.	-	-	-	-	-	-	-	-	0	12200000	44779703	8952759	65932462
17.	M/s Viraj Steel & Energy Ltd.	-	-	-	-	-	-	-	0	0	838696	1162029	117247	2117971
18.	Global Coal & Mines P.Ltd.	-	-	-	-	-	-	-	0	0	120520	1182687	206751	1509958
19.	Aditya Aluminium Ltd.	-	-	-	-	-	-	-	-	-	-	27790580	0	27790580
	Total	0	1011374	984210	5243954	6129710	6696751	10777752	12628085	10994415	31135782	95246852	13232120	166280425

N.B. Industries at Sl.1 to 10 are drawing water prior to 1999, Sl.11 and 12 from 2004, Sl.13 to 15 & 17, 18 from 2006, Sl.16 from 2007 and Sl.19 yet to draw.

Centre, Golden Jubilee Session, 8th Feb. 2009 Pg. 158-165). It has been estimated that the saving of water by providing C.C. lining would be about 146 Mcum which is equivalent to the live storage capacities of Harabhangi and Derjang reservoirs together. Lining will also minimise waterlogging thereby inceasing crop yield.

Funds may be a constraint to take up C.C. lining of entire canal network but this could be done in phases under ERM scheme as the project is more than fifty years old.

7.5 Pisciculture

7.5.1 Introduction

The artificial reservoirs or lakes created by constructing a dam across a river or stream have their importance as potential resources of Inland fish production. They constitute the single largest inland fisheries resource both in terms of resource size and production potential of the source of animal protein as fish.

The National Commission on Agriculture (NCA) has estimated the total area under Reservoirs at 3.00 Million Ha. during the mid- sixties and projected its growth to 6.00 million Ha. by 2000 AD (Anon, 1976). Distributed under divergent geoclimatic and edaphic environments, the Planning Commission put the total magnitude of countries reservoir area as 3.2 million Ha. classified into small (< 1000 Ha.) medium (1000-5000 Ha.) and large (> 5000 Ha.) catagories. Fish yield from these water bowls is frustratingly low being on an average 12 kg. / Ha. in case of large and medium Reservoir and 50 kg. in case of small Reservoir against 80 kg. / Ha. (large Reservoirs) in Russia and 100 kg/ Ha. (small Reservoirs) in Sri Lanka

Not withstanding the low level of yield ,there are few Reservoirs in the country where good management and involvement of the Stake holders has resulted in fast forwarding the yield. Record production level of 122 kg. / hac. in case of Govind Sagar Reservoir (Himachal Pradesh, Area - 16000 Ha., the highest yield from large Reservoir in the country) and 316 kg. / Ha. in case of Chullar Reservoir (Kerala) are typical examples of success.

Compared to the National Scenario, Orissa is bestowed with total water spread of 1.97 lakhs Ha. area in the form of Major (3 nos) Medium (3 nos) and small or Minor (1406 nos) Reservoirs which account for country's 8.0% of the total Reservoir area. This total Reservoir area also constitute 90 % of the total Inland Fisheries resource of the State (6.66 Lakhs Ha.).

7.5.2 Fish Fauna

The fish fauna of the Hiriakud Reservoir, including that of the parent Rivers comprises of both plain and hill river species with sizeable representation of Carps and Cat fishes Survey conducted in 1950s recorded 86 species belonging to 20 families of which 24 were of economic significance. Before empoundment a comprehensive survey in a stretch of Rrver Mahanadi during 1955 recorded 103 species of which the same 24 species recorded earlier were of commercial importance. *Puntius serana, Tor mossal, Labeo fimbriatus, Labeo Calbasu* and Gangetic Major Carps, viz- *Catla catla and Labio rohita* were the most common carp. The major species caught from the Reservoirs were as follows :-

- Carps: *Catla catla, Labeo rohita, Cirrhinus mrigala, Labeo calbasu, Labeo fimbriatus, Cirrhina reba, Labeo bata, Tor mossal mahanadicus.*
- Cat fishes: *Silonia silondia, Mystus aor, Mystus singhaln, Wallago attu Pangassius pangassius, Etropichthys vacha Bagarius bagarius Rita ritu, Rita, chrysea.*
- Feather back: *Notopterus notopterus, Notopterus chitala.*

- Live fishes: *Heteropneustes fossilis, Clarias batrachus*
- Perches: *Anabas testudinius Tilapia nilotica Nandus nandus*
- Clupeids: *Gadusia chapra*
- Mugilids: *Rhinomugil corsula*
- Murrels: *Channa punctatus, C striatus, C marulius, C gachua*
- Belones: *Xenetodon cancila, Hemirhampus georgii*
- Scianids: *Sciana coitor*
- Lepidocephalids: *Lepidocephalus thermalis, Lepedocephalichthys guntea*
- Minor Carps: *Chella gora, Cbacaila, Puntius sp. Amblypharyngodon mola, Rohtee cotio, Laubuca laubuca, Esomus denricus, Rasbora deniconius.*
- Miscellanus Species: *Mastacembalus armatus, M. panculus, M. aculeatus.*
- Prawns : *Macrobrachium malcomsonii*

7.5.3 Fish Production Trend

An appraisal of the commercial fisheries of the Reservoir since inception (ie. 1959) indicates progressive increase of fish catch from 2.97 MT. in 1958-59 to 63.90 MT in 1968-1969 registering an upward trend during the next 10 years recorded catch of 779.3 MT. during 1979-80 with marginal shortfall during the same year due to varied reasons. The highest catch was recorded in 1980-81 ie. 843.0 MT. with average catch of 15.6 kg./ha. There have been a sharp fall in catch during the following years and the annual exploited fisheries now fluctuates at 234.93 MT, the highest yeild being 428.68 MT during 1986-87 in the last 20 years. The fish catch has again dropped to a level of 151.54 MT during 2004-05 vide Table 7.33. Thus the yeild from the Reservoir has reached a level of 4.5 Kg/ha. which is abysmally low compared to the State average of 12 Kg/ha./Year and the national average is 28 Kg./ha/Year. However, the catch accounted of estimation of the present yeild level excludes a sizeble quantity harvested through illegal fishing.

Table 7.33 : Yearwise Fish yield from the reservoir

Year	Qty. in MT	Year	Qty. in MT	Year	Qty. in MT	Year	Qty. in MT	Year	Qty. in MT	Year	Qty. in MT
1958-59	2.976	1968-69	63.906	1978-79	690.900	1988-89	368.385	1998-99	274.781	2008-09	245.150
1959-60	5.911	1969-70	65.676	1979-80	779.300	1989-90	263.510	1999-00	259.236		
1960-61	32.736	1970-71	220.245	1980-81	843.080	1990-91	332.837	2000-01	234.931		
1961-62	34.535	1971-72	214.370	1981-82	302.018	1991-92	380.238	2001-02	186.310		
1962-63	32.400	1972-73	181.148	1982-83	287.639	1992-93	336.940	2002-03	210.200		
1963-64	14.401	1973-74	197.856	1983-84	321.160	1993-94	426.287	2003-04	292.900		
1964-65	15.092	1974-75	148.934	1984-85	207.757	1994-95	400.289	2004-05	151.540		
1965-66	4.315	1975-76	137.100	1985-86	330.134	1995-96	368.600	2005-06	182.000		
1966-67	15.791	1976-77	178.882	1986-87	408.681	1996-97	267.301	2006-07	205.000		
1967-68	54.700	1977-78	576.621	1987-88	183.478	1997-98	260.174	2007-08	217.530		

7.5.3.1 Catch Composition

The commercial Fishery of the Hirakud Reservoir can be broadly classified in to five categories based on average composition stated as follows :

Group	Average % of composition
Major Carp	25%
Minor Carp	15%
Big cat fish	24%
Small cat fish	29%
Others	07%

7.5.4 Problems :

The diminishing trend in fish production from a vast water body like Hirakud Reservoir of the state warrants judicious management plan and intervention for adequate conservation measures for augmenting fish yield to a sustainable level and develop plan for alternative livelihood for 4000 fishery folk families dwelling around the Reservoir.

- Indiscriminate killing of juveniles and under sized fish (less than 250 mm.)
- Indiscriminate fishing of brood fishes affecting auto breeding.
- Use of the most destructive fishing gears, the zero mesh size shore seine locally called Dullungi Jal.
- Indiscriminate use of under prescribed mesh size nets.
- Non observance of close season.
- Low catch per unit effort.
- Lack of alternative livelihood option for ban period / ban season.
- Lack of awareness.
- Indiscriminate and illegal fishing by Poachers / Maffias.
- Exploitation by middle men both in harvesting and post harvesting operation.
- Inaccurate and nonreliable data on total fish catch, efforts, catch per unit effort and size composition.
- Inadequate study of soil and water parameters alongwith the sustainable fish stock.
- Lack of education and high grade of illiteracy among fishermen & women.
- Poor socio economic status of fisher folk majority of whom are Below Poverty Line and are unable to understand the need for development and conservation.
- Lack of legislation to prosecute poachers & mafias.
- Pollution hazards due to industrial effluents from Paper mill, refractories, cement factories, waste from colliaries, thermal wastes from power plant, river bed modification through removal of gravel and pesticides residues from agricultural operation in the peripheral area of the Reservoir. Industries along the periphery of the reservoir are pouring untreated effluents in the reservoir.

7.5.5 Co-operatives & their management :

With the inception of fishing activities, 5 Primary Fishermen Coop. Societies have been organised and inducted to fishing activities with demarcation and allotment of 5 Sectors in their favour as detailed below :

Table 7.34 - Details of Co-operative Societies.

Sl. No.	Name of the PFCS	Sector allotted	Fishing area allotted	Membership (Nos)
1.	Ibb PFCS	Sector-I	6106.00	663
2.	Mahammadpur PFCS	Sector-II	11656.06	879
3.	Thebra PFCS	Sector-IV	11137.00	657
4.	Lachhipali PFCS	Sector-V	4144.00	379
5.	Tamdei PFCS	Sector-VI	5255.00	555
6.	Fisheries Deptt.	Sector-III	31080.00	--

7.6 Recreation

7.6.1 Introduction

The lake formed by the Hirakud dam will afford great recreational facilities and will attract holiday-makers and tourists from all over India and abroad. The experience in other countries, notably the U.S.A., shows that these artificial lake rapidly become centres of attraction. Large revenues accrue to the local people from tourist traffic. At Norris reservoir of the T.V.A. some idea of the popularity of boarding may be obtained from the fact that two years after the completion of the dam, approximately 1,200 boats were in use. Along the shoreline of the lake extensive parks will spring up, with picnic grounds, cabins and other facilities, adding to the attraction of the place.

7.6.2 Scope for recreational facility due to formation of Hirakud reservoir.

The Hirakud dam, one of the longest dams of Asia has a water spread area of 286.87 sq.miles (743 sq.km) at F.R.L. of 630 ft. which maximum fetch of 80 km and about 400 km. of shore line. At D.S.L. of RL 590 ft. the water spread area is 274 sq.km. The right side of the reservoir is bounded by hills full of jungles and inhabitated by wild animals and birds. There are scenic spots in this area which can be developed as picnic spots. The reservoir at its D.S.L. is so deep that tourist boats and laundry can easily ply from one place to other throughout the year. There are some hillocks inside the reservoir (such as beacon No.8) where cottages and small shops, gardens, fountains etc. can be built up there for comfortable stay of tourists for short periods. Nearby hillocks can be connected by ropeways, hanging bridges where small gardens can be developed. There is further scope of developing zoo at few places. Aqua-culture is another possibility. But nothing was developed during construction period except developing.

7.6.3 Developing Jawahar Udyan at Hirakud

Downstream plains of artificial large reservoirs are ideal places for developing beautiful gardens as holiday resorts. Hirakud reservoir is not an exception.

In a state level meeting held on 16.11.1988 at Bhubaneswar under the chairmanship of Chief Minister of Orissa a proposal was initiated for constructing a magnificent park at the toe of left earthdam of Hirakud Project at par with Brindaban garden of Mysore on the event of birth centenary of late Pt. Jawaharlal Nehru.

The original estimated cost (at 1986 base price) was Rs.372.5 lakhs which was subsequently revised to Rs.769 lakhs in 1995 as per advice of State Tourism Department.

The work of the project commenced during 1988-89. Due to paucity of fund however, the work of the garden could not make any headway except completion of installation of a life size statue of Jawaharlal Nehru. The garden was to be developed in an area of 49.2 ha. (123 ac.).

Preliminary planning was done to have the following facilities in the first stage.

(i) Terraced garden - Five terraces with 1.2 m. Level difference. Each terrace will be of 60mx131 m size.

(ii) Fountains of various designs i.e. Crown, Agarbati, Inverted basket, Arch, Pyramid, Umbrella, Vertical/Stepped cascade, Vertical jets and Floral etc.

(iii) Lake-with spread area of about 5.5 acres.

(iv) Rose garden.

(v) Other flower garden.

(vi) Green fencing shading trees.

(vii) Statue of Pt. Nehru.

(viii) Roads and Footpaths.

(ix) Buildings, gates etc.

In 2nd stage development, following facilities are to be provided.

(i) Cactus garden (ii) Orchid house (iii) Creeper house (iv) Apu ghar (v) Floral clock (vi) Medieval decorative statues (vii) Rock garden and (viii) Deer park.

This site has unique advantage of being integrated into a bigger concept of a combined resort, which would comprise broadly followings :

(a) Jawahar Udyan

(b) Gandhi Minar

(c) Water Spots in Hirakud reservoir.

(d) Rope way from Gandhi minor to islands inside the reservoir.

(e) Development of nearby offshore islands and periphery.

(f) Development of cottage inside islands.

(g) Development of specific islands with garden and zoo.

The Hirakud reservoir is attracting migratory birds in winter. The partly developed Jawahar Udyan has already become a tourist spot. This needs further development with public private partnership such that it would fetch some revenue for the project in particular and the state in general besides providing employment opportunities for the locals.



Gandhi Minar on left side hill of Hirakud



Nehru Minar on right side hill of Hirakud

CHAPTER - VIII

CONCLUSION

8.0 Large Dams Vs Small Dams

As long as the world's population continues to increase, additional water resources must be developed to feed and clothe the masses. The only way to achieve this is a judicious combination of small and medium dams as well as large dams along with rain water harvesting, watershed management and ground water development. Advantages and disadvantages of large and small dams are discussed as under :

8.1 Views of Proponents & Opponents

"Large dams have become very controversial in recent years. Proponents of such structures claim that they deliver numerous benefits, among which are increased water availability for domestic and industrial purposes, increased agricultural production because of reliable availability of water for irrigation, protection from floods and droughts, generation of hydroelectric power, navigation, and overall regional development which improves the quality of life of the people, including women. They argue that like any other large infrastructure development project or national policy, dams have both benefits and costs. However, the net benefits of dams far outweigh their total costs, and thus the society as a whole is much better served by their construction.

In contrast, the opponents argue that dams bring catastrophic losses to the society and the resulting societal and environmental costs far outweigh the benefits they may bring. They claim that dams further accentuate the inequalities in terms of income distribution in the society, since benefits accrue primarily to the rich, and the poor people slide further down the economic ladder. They further claim that the main beneficiaries of large dams are construction companies, consulting engineers, and corrupt politicians and government officials, who work in tandem to promote them. Poor seldom benefit from the dam 'gravy train': they mostly suffer.

The overall views of the proponents and the opponents of the large dams have been polar opposite. Scientifically and logically, these two views cannot be correct. Most unfortunately, however, there has never been a real and sustained dialogue between the two camps. For example, during the Second World Water Forum in the Hague in 2000, the pro-dam sessions mostly discussed their benefits, while the anti-dam sessions blamed all the societal ills on them, irrespective of how they were caused. The proponents and opponents of dams did not attend each other's sessions, and thus there was no debate, dialogue, and interchange of views between the two groups. Both the camps went home thinking that the Forum basically agreed with their views."

(Source : Bhakra-Nangal Project - Socio-economic and Environmental Impacts by R. Rangachari, Published by Oxford University Press, 2006, Pg.vii).

The construction of large dams in India and elsewhere is being criticised mainly on the grounds that it involves.

- Displacement of people from their traditional homes due to submergence caused by large storages created by the dams.
- Loss of traditional means of livelihood of the displaced people.
- Change in the flow regime in the river channels downstream of the dams.
- Environmental and ecological effects of large storages created by the dams.

"There is an impression in the minds of the people that large dams cause large submergence and advocate construction of small dams with smaller storages instead. This they do in mistaken belief that small dams cause less submergence. But the area inundated by many small dams to achieve the storage capacity equivalent to the capacity of a large project would be far more. Table 8.1 shows submerged area and storage capacity of few dams of the country.

Table 8.1 : Submerged Area Vs Storage Capacity

Name of Dam	Height (m)	Reservoir Area (sq.km)	Storage capacity (Mcum)	Area submerged in m^2 per 1000 cum of storage
Tehri Dam	260.50	42.00	3540	11.864
Bhakra Dam	225.55	168.35	9621	17.498
Idukki Dam	169.00	60.00	1996	30.060
Beas Dam	132.59	260.00	8570	30.338
Ramganga	84.60	78.31	2443	32.060
Ukai Dam	81.00	601.00	8511	70.615
Hirakud Dam	**59.00**	**743.00**	**8105**	**91.672**
Maithan Dam	56.00	107.00	1369	78.148
Panchet Hill Dam	49.00	153.00	1498	102.170
Talaiya Dam	45.00	75.00	556	134.819

'Moreover, small storages are unable to provide assured supplies during lean periods so very necessary for mass food production to feed the growing population in the developing world. While expressing concern about submergence the beneficial ecological changes that these projects bring should not be forgotten. The sustained irrigation supplies made through the canal system fed by Bhakra and Beas waters has not only helped in checking the process of desertification but also bringing in green revolution in the states of Punjab and Haryana apart from greening of the countryside and moderating the climate of the area." (Source: Large Dams - Gains & Losses, IWRS Seminar, Delhi, 10.12.1999, Pg.63).

The internationally famous environmentalist Patrick Mccully narrates in 'Silenced Rivers' why large dams have become the most controversial of technologies, in the last decade. In fact, 'Silenced Rivers' is one of the most scathing attacks against large dams in recent times. But in a thought-provoking article (The Hindu - 1st Dec. 1999), Prof. P.V. Indiresan has raised a pertinent question.: "Which is more important - providing food and drinking water to a crore of people or preventing the displacement of ten thousand tribals?"

8.1 Impacts of Water Storage Projects

"There are numerous incidental benefits from the construction of large dams and canal systems such as improving environment and health, afforestation, fisheries development, tourism and recreational facilities, development of agro-based industries, network of roads in command areas, development of land and improving general socio-economic standards of the people. It has been established in numerous case studies that significant improvement occurs in the food and nutritional level of the people with higher per capita food availability and diversification of crop production especially cash crops after introduction of irrigation in the area that are quite remunerative to the farmers. New employment opportunities generated by intensification of agricultural and the associated activities further improve the financial condition of the people including landless labourers. As a multiplier effect, large river valley projects tremendously improve the health of rural population by significantly enhancing the education, health care, transport facilities and the lifestyle particularly of women.

Water resource project may submerge forests, reduce downstream flows in rivers and at times lead to bio-diversity. It may be noted that the loss of forest area due to submergence is less than five percent of the total forest area lost in the country in the last five decades. Loss of bio-mass through submergence is, far smaller than the biomass generated on account of the irrigation. Notably, it has been observed that a forest far superior to the original, sans the original bio-diversity, comes up after the creation of the reservoir. Adverse effects like water logging and salinity are being prevented through conjunctive use of groundwater, prevention of canal water leakage, reduction of seepage losses from water carrying bodies, implementation of adequate drainage and adoption of efficient irrigation methods along with water conservation.

Reservoir may create new condition for the growth of organism and ultimately as adjustment are made, foster new eco-systems. Varieties of new organism thrive on this new lake system. Additional water made available for the dry period of the year, when the environment tends to be harsh making area inhospitable supports the growth of life around. Water resources projects provide a dependable source of drinking water. People from the irrigated areas enjoy better health and sanitation facilities, thus reducing the incidences of disease. General decline in incidences of diseases has been reported from different irrigation commands. The very availability of water leads to improvement in the level of sanitation. The improved economic status also makes people health conscious and capable of availing requisite health care.

Substantial increase in the numbers of tigers, panthers, elephants and Cheetals have been observed in National Corbett Park with the availability of green fodder and clean water through out the year and improved climatic conditions after construction of the Ramganga Multipurpose Dam Project. It is also observed that rare species of birds flock there, after the reservoir construction. Similar phenomenon of increased count in birds and wildlife has also been observed around the Rihand and Matatila reservoirs, which were previously barren lands. It is worth noting that some of the best tourist places of India like Ukai tourist resort, Periyar wildlife sanctuary, Shalimar Garden, Brindavan Garden, Pinjore and Bagalkot New Township Garden are highly attractive bye products of river valley projects in India.

New water supply facilities sourced from large dams have really improved sanitary conditions, leading to significant improvement in general health conditions. Water resources projects also provide a dependable source for drinking water. People from irrigated areas, in general enjoy better health and sanitation facilities, thus reducing the incidence of disease. Large dams help in conversion of wasteland into agricultural land and making the area greener. Indira Gandhi canal has not only

transformed Western Rajasthan into vast green area but also checked the spread of Thar Desert in the adjoining areas of Punjab and Haryana. Bhakra Dam is a shining example, which has changed backward area of erstwhile undivided Punjab into the granary of India with improved environment. The availability of water from Sardar Sarovar Project will benefit about 1.91 lakh of people residing in 124 villages in arid and drought-prone border areas of Jalore and Barmer Districts of Rajasthan which have been suffering grave hardship on account of scarcity of water, besides checking the advancement of Thar Desert.

Employment benefits of river valley projects have been widely experienced. Apart from employment benefits during construction and maintenance of projects, sizeable recurring on farm employment benefits are generated, because labour use in irrigated farming is more than in un-irrigated farming. Thus, irrigation development in a tract stems out migration of job seekers from that tract to distant centres and also brings a chain of multiplier socio-economic benefits.

India is predominantly an agricultural country, depending on monsoons of four months during the year. Available monsoon flows are to be used judiciously to cater to various needs for the balance period of the year. This necessarily warrants construction of reservoirs to store surplus flood flows to meet various demands during lean season. Water resource development requires a judicious mix of large, medium or small reservoirs, which are location specific. In fact, the scenario of water resources development in India depicts a discrete combination of all the sizes of projects; based on the integration of techno-economic feasibility and environmental capability along with regional demands. Sustainable management of water resources with due respect to ecological, economic and ethical sustainability blended with technical feasibility requires a holistic and integrated approach involving engineering, socio-economic and environmental aspects in Indian context" (Refer: Theme paper of IWRS 2002- Pg.4-5).

This is not only applicable to Hirakud but to all major projects of the country. "Dams are the means to an end, the planned end objective being economic and human development, leading to an improved lifestyle for the common people. Unlike countries in the temperate zone which receive rains round the year, India, under the influence of the monsoon, gets its annual rainfall in a limited period of three months, while water requirements are round the year. When fresh water resources are limited and very unevenly distributed over space and time, storage dams become necessary. India has, therefore, been building dams-small, medium, and large—for centuries. The pace of such activities accelerated after Independence from colonial rule. Increased agricultural production became inescapable for India because of rapid population expansion. Irrigation development on a greater scale became necessary to meet the requirements of the expanding population. The generation of hydropower, a relatively cleaner form of energy also became necessary." (Source: BNP Socio-Economic & Environmental Impacts by R. Rangachari Pg.5).

The recommendations of the seminar on "Large Dams - Gains & Losses" organised by Indian Water Resources Society is furnished in Sec.8.1.2.

8.1.2 Recommendations of Seminar

Extract of Recommendations of seminar (held on 10.11.1999) organised by IWRS, Roorkee is reproduced below :

(i) "The existing water resources projects with large dams like Bhakra, Hirakud, Nagarjuna Sagar, Ukai, Mula Periyar and Iddukki, constructed more than 20 to 30 years earlier have not only fulfilled their economic objectives but have also improved the ecosystems to a large extent. The overall benefits seem to far overweighed the loss of resources, including those due to submergence.

(ii) The progress in water resources development during the last two decades has slowed down below acceptable level. To meet the requirements of drinking water, food, fiber and energy for the growing population in the near future ongoing projects like Sardar Sarovar, Narmada Sagar and Tehri need to be completed at the earliest.

(iii) Small dams have their advantages but cannot substitute the large dams. Both have to coexist.

(iv) The Environmental Impact Assessment (EIA) process needs to be strengthened and streamlined. The EIA studies need to be based on the affordable technologies and conform to EIA notification 1999. Such studies need to be carried out within a fixed time frame. The decisions about the economical, social and environmental acceptability of the project need to have a finality and reviews beyond reasonable extent need to be done.

(v) Project planning and implementation needs to be transparent and needs to involve the beneficiaries as well as the adversely affected persons. Guidelines about Resettlement and Rehabilitation (R&R) need to be standardized. The conflicts related to R&R would have to be resolved through a conflict Resolution Mechanism to be provided in the National R&R Policy. This mechanism may include the representatives of the affected people apart from the social scientists, economists, people's representatives, the government officials and the project authorities.

(vi) For scientific assessment of benefits and disbenefits of the project, proper bench mark surveys need to be undertaken before the commencement of the project. The socio-economic impact assessment needs to be carried out at regular intervals during and after the implementation.

(vii) Well conceived water resources projects including the large dams shall be treated as green projects by the government."

8.1.3 Performance Analysis :

"The World Commission on Dams (WCD) Report questioned the utility of dams and caused acrimonious debates on their impact. What got sidetracked were the scientific and rational evaluations of such projects and their performance evaluation over a long-enough period of service to enable appropriate conclusions to be drawn. There are hardly any objective ex-post-facto analyses of the various impacts of large dams. Unfortunately, there has also been little systematic collection of relevant data about dam projects in the past and in their absence, definite conclusions regarding their performance and impacts are difficult. Much data remain unavailable to the public. To be credible, post-project evaluations should be made through independent professional analyses, rather than leaving those to either the same agency that built and operated them or to lobbyists. There have been precious few, if any comprehensive independent analyses on how dam projects were selected for execution, how they performed over time, and on the returns we are getting from the investments. Instead, issues relating to dams, their benefits and impact, have become one of the battlegrounds in the sustainable development arena, as pointed out by Nelson Mandela while launching the WCD Report (on 16 November 2000 at London on the occasion of the release of the Report); he correctly noted that the **'problem, though, is not the dams. It is the hunger. It is the thirst. It is the darkness of a township. It is townships and rural huts without running water, lights, or sanitation."** (Source: Socio-Economic and Environmental Impacts of Bhakra-Nangal Project by R. Rangachari Pg.5).

8.2 Present position of Manibhadra and Subalaya across Mahanadi :

Sec. 1.4.6, 1.4.7 and 1.5 of the book may be referred to wherein it was contemplated to construct two more dams i.e. Tikerpara and Naraj besides Hirakud. Naraj dam was replaced by Mundali barrage and Tikerpara was abandoned due to submergence of Boudh, Sonepur and Athamalik towns. This gave birth to Manibhadra and Subalaya project proposals.

8.2.1 Manibhadra Dam Project

The proposal envisaged construction of 2070m length of earth dam with maximum height of 49 m above ground level. It will have six earthen dykes of 2030 m long and a 350 m long main spillway having 16nos x 15 m x 11 m radial gates. The auxiliary spillway will be 650 m long and two surface Power houses having installed capacity of 960 MW. The project will generate firm power of 297 MW and protect an area of 657432 ha from ravages of flood. The project parameters were fixed on the basis of water availability and minimum submergence.

Due to problem of submergence, it was not possible to provide exclusive storage space for flood moderation above FRL 86.0 m. The highest recorded flood of August, 1982 with a peak of 44750 cumec at delta head Naraj carried a flood volume of 1513 thousand hectare meter. In order to reduce the flood peak to the level of safe carrying capacity of the channels in Mahanadi Delta (i.e., 25,500 cumec) a storage of 445 th. ham is required. To provide exclusive storage space of 445 th. ham for flood moderation above FRL, would affect Boudh town as the MWL may rise up to RL 93.10 m. Therefore, it was planned to maintain the reservoir at RL 82.30 m during monsoon so that impingement of a flood of August, 1982 magnitude or higher (200 year return period, having routed out flow 25,500 cumec) will raise the MWL to RL 91.50m and still regulate the peak to 25,500 cumec at Delta head.

This proposal also met the same fate as that of Tikerpara. Government of Orissa dropped the proposal of dam at Manibhdra mainly on the consideration of large scale submergence due to proposed structure. Moreover, the presence of felspar deficient Khondolite on the river facing slope of Manibhadra and Subalaya hills, abutment conditions are not favourable. The foundation of the proposed dam was also considered critical as at the final site location, the river bed consisted of a deposit of about 30 m depth of sand below which rock strata was met. The proposal was to provide deep concrete cut off extending into rock. It was apprehended that the dam constructed on sand foundation may suffer from damage due to liquefaction under earthquake forces.

The proposal remained in the cold storage, and the danger of flood continued to haunt the mind of politicians, administrators, engineers and particularly the inhabitants as the coastal belt of the State continued to remain exposed to the danger of flooding. This state of affair still remains unsolved.

8.2.2 Subalaya Barrage

As implementation of both Tikerpara and Manibhadra Dams across Mahanadi were shelved due to large scale submergence and agitation by people, it was thought of constructing a barrage at Subalaya to ameliorate the sufferings of coastal Orissa from floods. In the first project proposal (1964), it was aimed at utilizing fully the available water resources by submerging 1200 villages, three towns and thick forest growth. Submergence and R&R problems became main bottleneck.

In the second proposal i.e. Manibhadra Dam (1985) power potential was reduced. Though the submergence was considerably reduced to 273 villages, two towns and relatively less

forest growth, this proposal was also aborted for similar reasons after the foundation stone for the project was laid by the Chief Minister.

After premature death of above two proposals, another alternative was thought of for flood moderation by dynamic storage with conservation level of RL 54.90 m and MWL of 77.0 m. In case of 100 year maximum flood peak of 49264 cumec (17.4 lakh cusec), the proposed barrage structure will moderate the flood to 24632 cumec (8.7 lakh cusec) at the head of delta. Between the conservation level of RL 54.9 m to MWL of 77.0 m, about 73 villages will remain below the flood level for a period of six and half days, The villages will be protected by embankments with road connections at higher levels. This preliminary proposal based on topo-sheet study also had similar death as the flood control by a barrage structure is an impossible proposition.

It is ridiculous 'to have the cake and eat it too'. To give relief to the people of coastal region, it is necessary to construct another storage reservoir across Mahanadi. Simultaneously, it is also needed that there will no submergence and R&R issues. Both are contradictory. Thus there appears to be no solution to the vexed flood problem of coastal region in the near future.

8.3 Interlinking of Rivers

The primary objective of Interlinking of rivers is for diversion of surplus water from water rich zones to deficit areas for multi-uses i.e. irrigated agriculture, municipal & industrial water supply, hydropower generation, navigation, recreation, recharging ground water table and maintaining ecological balance. These coupled with flood control and drought mitigation with regulated environmental flow in rivers will accelerate economic growth. This practice was followed in India and other countries in earlier ages.

8.3.1 Study by NWDA

To give a concrete shape to Peninsular and Himalayan Rivers - Development components of National Perspective Plan (NPP) for diversion of water, National Water Development Agency (NWDA) was set up in 1982 as an Autonomous Society under MOWR to carry out the water balance and other studies on scientific and realistic basis and prepare Pre-Feasibility & Feasibility Reports for link schemes. Total 30 links have been identified. Himalayan component will have 14 links & Peninsular component with 16 links. Benefits from Peninsular component are as follows :

(i) 13 million ha of additional irrigation.

(ii) 4000 MW of hydropower.

(iii) Drought mitigation to some extent in M.P., Karnatak, Tamilnadu and Madhya Pradesh.

(iv) Flood control to certain extent in Mahanadi and Godavari basin.

(v) Facilitate inland navigation as the link canals will be 50 to 100m wide and more than 6m deep.

As there may be delay in concluding agreements with neighbouring countries for Himalayan links, emphasis was attributed more on peninsular links.

8.3.2 Interbasin Transfer in the State

There are two bright examples of interbasin transfer in the state i.e. Upper Indravati Multipurpose Project and Harabhangi Project. The former project comprises of four dams across rivers Indravati, Podagad, Kapur and Muran with eight dykes, two link channels and 4 km long tunnel to provide irrigation to more than one lakh ha. in drought prone areas of Kalahandi and

generate 600 MW power by transfer of water from Godavari basin to Mahanadi basin. Tail race release from Power House is picked up at Hati barrage from where two canals take off i.e. Left Main Canal and Right Main Canal for irrigation. Kalahandi which was once synonymous with starvation is now self sufficient in foodgrain production,

Another is Harabhangi Project in Adva village of Gajapati district. The water is diverted from Vansadhara basin to Rushikulya basin through a water conductor system leading from Harabhangi reservoir into existing natural drainage called river Padma where it is picked up through a weir at Gokulpur to irrigate 9150 ha. in Khariff and 8500 ha in Rabi in Ganjam district. (Refer: Two Inter Basin Transfer Projects in Drought Prone Areas of Orissa - A Success Story by G.C. Sahu - 11th National Water Convention, New Delhi, 11 May 2005, pp 335- 339).

8.3.3 Mahanadi-Godavari Link

In the peninsular link Mahanadi (Manibhadra)-Godavari (Dowlaiswaram) link is a major link, which is critical for Orissa. The details of the scheme based on Pre-feasibility Report of NWDA are as follows.

This link envisages diversion of about 11176 Mcum of water annually from the proposed Manibhadra Dam on river Mahanadi to Godavari to serve the demands of Manibhadra Right Bank Canal, provide irrigation enroute and transfer 6500 Mcum of water to southern Peninsula to meet their deficit requirement. The water conductor system is 932 km. lined canal (including 6.30 km. long tunnel) lying in Orissa and A.P. The capacity of canal is 627 cumecs. The canal would cross major rivers like Rushikulya. Bahuda, Vansdhara, Nagavali, Champawati before joining Godavari at Dowlaiswaram barrage.

The link will irrigate 3.5 lakh ha. in Orissa in the districts of Nayagarh, Khurda, Puri and Ganjam and one lakh ha. in A.P. in Srikakulam, Vijayanagaram and Vishakhapatnum district, utilizing 3854 Mcum of water. It will discharge 6500 Mcum into Godavari after losing 822 M cum in transmission.

It may be noted that Manibhadra Dam Project which was contemplated earlier as an alternative to Tikerpara (vide Sec.8.2.1) could not be implemented due to large scale submergence and R&R problem.

8.3.4 Proposed Inter-Basin Transfer in the State

It is envisaged to transfer flood water of river Mahanadi to Brahmani and Rushikulya basins. The proposals are virtually in inception stage. After completion of left and right canals off taking from Samal Barrage to irrigate more than two lakh ha and for providing industrial water to the mega plants coming up at 'Kalinganagar', there will be scarcity of water for meeting irrigation, industrial, domestic and environmental needs. Through a flood flow canal of about 600cumec capacity, the spill waters of Hirakud dam reservoir is to be transferred to Rengali reservoir in Brahmani basin and to construct another reservoir across Garada nallah enroute where a Power House is also contemplated. The benefits likely to accrue from this link is (i) additional irrigation (ii) power generation (ii) partial flood control (iv) industrial water supply and (v) maintaining minimum environmental flow in Brahmani river.

Besides above, it is proposed to augment flow in river Rushikulya to meet growing demands of irrigation, industry and domestic water supply by transferring waters of Mahanadi either by constructing a barrage at Barmul and/or from proposed Salki hydel project. Above proposals are only at conceptual stage.

8.4 Construction of Projects in the downstream of Hirakud (on the tributaries of Mahanadi)

8.4.1 River Basin Planning :

To prepare an integrated state water plan comprising all the river basins of the state for estimating the water resources and to utilise the available water optimally by encouraging consumptive use of water through judicious multi-sectoral uses to meet the growing demand of this scanty resource with time and increased population, a separate organisation styled as Orissa Water Planning Organisation (OWPO) under Department of Water Resources (DOWR) was created. This organisation formulated the basin planning of individual river basins.

Basin planning is a component of State Water Plan which is the corner stone for future water resources development of the State. Planning process of a river basin can be described as an orderly procedure to optimize the development of water and related land resources. This would result in investigating the alternatives, programming of potential projects with respect to a set of objectives, taking into account physical and managerial opportunities and limitations.

River basin planning concentrates the planning effort in the natural hydrological unit, the river basin. The basin level approach is based on the broad set of interrelationships amongst the domains of natural phenomena classified into physical, biological and human systems. It offers a framework for bringing out integration in planning consistence with overall economic, social and environmental policies of the state and that of the country.

Primary objectives of river basin plans of the State are as follows: -

i) To prepare long-term perspective plan for the development of Basin's Water Resources.

ii) To develop a comprehensive and integrated approach to the development of water and other natural resources using water, within the constraints of water availability.

iii) To review the management of existing water resources project and incorporate necessary changes on technical and administrative aspect for making the project sustainable.

iv) To identify and set priorities for promoting water resource development projects.

v) To formulate a long term Integrated State Water Plan for water resources - development.

vi) To create a data bank for posterity.

This is a continuous process for upgrading the State Water Plan. Accordingly 3 spiral studies for Mahanadi Basin have been completed and projects identified are furnished in Table 8.2.

Table 8.2 : Identified Projects in Mahanadi Basin

Sl. No.	Name of the Project	C.A. sqkm	C.C.A (ha)	Irrigationn (ha)		Irrigation demand (M.cum)	Total demand including evapo. losses (M.cum)
				Kharif	Rabi		
1.	Jeera	115	4600	4140	1150	43.95	58.33
2.	Lower Jonk	458	7500	6825	1275	68.44	--
3.	Karandijore	67	2100	1890	525	18.78	24.77
4.	Kutulsinga	83	2540	2032	889	23.26	25.48
5.	Upper Bheden	311	11000	9900	4180	100.26	127.28
6.	Lower Bheden	414	19000	17100	7220	171.63	224.75
7.	Lambodara	328	11200	8960	--	89.61	102.81
8.	Sankhabagan	279	4650	4185	2093	49.76	61.33
9.	Upper Tel Dam	597	13240	10592	5296	162.47	177.66
10.	Banjari nallah	240	3400	3060	1394	27.54	39.00
11.	Sagada	443	18750	16875	7500	170.18	228.57
12.	Upper Indra	337	7200	6480	3096	75.18	83.39
13.	Lower Lanth	1131	30000	27000	9000	289.8	357.99
14.	Upper Udanti	302	24000	21360	--	172.74	117.87
15.	Lower Udanti	672	14000	12600	--	119.58	248.90
16.	Khadago	1980	37140	34912	17827	479.66	524.56
17.	Surabalijore	381	18000	16200	8100	177.2	213.21
18.	Upper Tel barrage	6061	9610	7688	2403	92.76	92.76
19.	Lower Tel barrage	1127	61000	54900	18300	551.82	551.82
20.	Uttei-Raul	1438	26474	23827	11384	283.31	393.85
	Inter-Basin Water Transfer						
1.	Export to Brahmani - Baitarani	--	84986	76487	--	669.30	669.30
2.	Export to Rushikulya	--	71631	50142	--	508.79	508.79

(Source: Mahanadi Basin Plan, 3rd spiral study, O.W.P.O., Dept. of Water Resources, Govt. of Orissa, Annex.3.1)

When detailed planning for above projects are taken up and project reports prepared, flood space (cushion) may be provided in each dam. This will go a longway to mitigate the flood hazards in coastal region of the state as flood is mainly due to downstream contribution below Hirakud dam.

8.5 Environmental Issues and Mahanadi Water Quality

8.5.1 Introduction

Irrigation is the single largest consumptive use of fresh water in the world. It is linked to food production and food security. About 20% of world's agricultural land is irrigated. Irrigated agriculture accounts for nearly 40% of the world's agricultural production. Along with livelihood enhancement, the broader impacts of irrigation projects on rural and regional development are often not quantified. Beyond economic impacts, irrigation schemes also produce a series of indirect or intangible social benefits derived from the multiple use of irrigation water. Project benefits are estimated relying simply on expected crop output and not considering the use of water for horticulture, livestock, fish production and tourism etc. as well as domestic, municipal and industrial water supply.

In terms of generating employment the principal impact of dam projects-aside from the construction jobs -arises from the new productive enterprises allowed by provision of

water. As with any indirect economic impact, it is important to consider not just the gross number of jobs created by a project but also whether alternative uses of project resources would have generated similar gains.

When Aswan High Dam (Egypt) was constructed, there was hue and cry regarding environmental, disaster. All manner of dreadful consequences were forecast that spelt the doom of Egypt. A quarter century after commissioning of the dam, the Executive Director of UNEP said, "There is no argument: the dam's contributions to substantially increasing food production and hydropower generation has made an enormous difference to the quality of life of the Egyptian people.... And in this particular case the adverse impacts were all anticipated and plans set for addressing them, but international political situations delayed the implementation." Another opinion by Dixon et al in 1990: "The evaluation of the last 21 years since the completion of the high dam at Aswan has found that the project has had a profound impact on Egypt and the Nile. Irrigation, Power and flood control benefits have all been achieved and are large. The dam gave important protection to Egypt during the African drought and in particular, during the critically dry year of 1983. There are some who believe the dam paid for itself in that one year that was almost as bad as 1913." (Dixon, Talbot, Le Moigne,. 1990). A joint American-Egyptian team studied over seven years and assessed the positive and negative impacts of the project. The conclusion was unambiguous "the Aswan High Dam is not an ecological disaster but an economic blessing."

"It is evident that the benefits accrued from large river valley projects are so immense that they substantially outweigh the costs of immediate human and environmental disruptions. On the other hand, long-term adverse effects of not utilising the water resources would be catastrophic, due to recurrence of floods, droughts and the resulting unemployment, which further increases the backwardness in highly populated developing countries, like India. Controversy of the large versus small dams is highly unfortunate. Basic fact remains that medium and small water projects cannot substitute large water storages but can at best complement larger projects. This, too, depends upon the hydrological, geological, topographical, meteorological and regional conditions, which are entirely different in India and developed countries due to our monsoonic climate.

It is to be underlined that catchment area treatment and watershed management mainly degraded due to jhoom cultivation, over grazing by unlimited cattles and unscientific and farming practices. Very large number of hotels at hill slopes are, no doubt, development projects in their own right but those should be planned and executed as such independently without putting undue financial burden on river valley projects. At best, treatment of directly draining degraded sub-watersheds along the reservoir rim could be charged to the cost of the reservoir project.

In the planning, implementation and operation of projects preservation of quality of environment and the ecological balance are primary considerations. Adverse impact, if any, on the environment are minimized and off-set by adequate compensatory measures with built-in mechanism for emergency preparedness. The focus by large number of critics, media and environmental activities as it is construed at present, concentrates on what it is not, and then tries to ameliorate the negative impacts. Maximisation of positive environmental impacts should be an equally important consideration. If due care is taken right from initial stages of planning, water resources projects can contribute greatly to enhancement of environmental quality with minimal ill effects.

It is desirable that environmental aspects and the process of planning and operation of water resources projects are rightly taught at different levels of education as well as to the

experts of different disciplines. Another important fact remains that participation of people is a must in the management of water. Unlike in other branches of engineering, the people are an integral part of the water management system. The community is to be made not only water conscious, but also to be integrated to participate in the planning and management of such projects and pollution prevention programmes".

(Source: IWRS, Theme Paper on Integrated Water Resources Development and Management, 2002 Pg.51).

8.5.2 Water Quality

Construction of an artificial reservoir to provide perennial irrigation has both short term and long term impact on the eco-system and socio-economic condition of the area and the people. The output being regulated release of stored water from the reservoir, the impacts are generally complex resulting in chain actions, as the project is a water storage one, that affects human beings, animals and flora and fauna. Impoundment of water modifies the water quality both in the upstream and downstream. There are quite good number of industries coal mines and towns located both in the upper reaches and lower reaches of the dam which pollute not only the reservoir water but river water also. The list of such industries and coal mines contributing to water pollution is furnished in Table 8.3 & 8.4. The water quality trend of Mahanadi has been described in the following section (i.e. Sec. 8.5.3).

Waste water is generated from various sources i.e. from industries, mines, cities and towns and agricultural fields. These pollute the river water. Major industries and coal mines in Mahanadi basin are furnished in Table 8.3 & 8.4.

Table 8.3 : Major Industries in Mahanadi Basin

	Name of the industries and Location	Product
(a)	**Mahanadi**	
	Indian Aluminium Co.Ltd. (INDAL), Hirakud	Aluminium
	INDAL-Captive Power Plant (CPP), Hirakud	Power
	ACC Cement, Bargarh	Cement
	Indian Farmers Fertilizer Cooperative Ltd. (IFFCO), Paradeep	Phosphatic Fertiliser
	SKOL Breweries, Paradeep	Beer
	Paradeep Phosphates Ltd. (PPL), Paradeep	Phosphatic Fertilizer
	Central Orissa Straw Board, Jagatpur	Paper
	Shaktisugar, Badamba	Sugar
	Arati Steel Limited, Athagarh	Iron and Steel
(b)	**Ib**	
	Orissa Power Generation Corporation (OPGC), Brajarajnagar	Power
	Tata Refractories, Belpahar	Refractories
	L&T Cement, Jharsuguda	Cement
	Bhuban Steel and Power Ltd., Jharsuguda	Iron & Steel, Power
(c)	**Birupa**	
	Indian Chargechrome Ltd. (CPP), Choudwar	Power
	Indian Chargechrome Ltd., Choudwar	Chargechrome

There are 12 coal mines operating in the basin. All the coal mines are located in Ib valley. Together they discharge about 14,000 M^3 of mine water per day during the non-monsoon months, which increase to about 33,000 M^3 per day during the monsoon. The details of Open Cast (OC) and Under Ground (UG) mines are given in Table 8.4.

Table 8.4 : Coal Mines in Mahanadi Basin

Sl. No.	Mine	Type
1.	Lajkura, Brajarajnagar	OC
2.	Belpahar, Brajarajnagar	OC
3.	Samaleswari, Jharsuguda	OC
4.	Lilari, Belpahar	OC
5.	Lakhanpur, Belpahar	OC
6.	Hirakhand Bundia, Ib Valley	UG
7.	Himgiri Rampur, Rampur	UG
8.	Orient Colliery I & II, Brajarajnagar	UG
9.	Orient Colliery III, Brajarajnagar	UG
10.	Orient Colliery IV, Ib Valley	UG
11.	Basundhara (W), Sundargarh	OC
12.	Talabira (INDALCO), Sankarma, Sambalpur	OC

(Source: Water Quality of Major Rivers of Orissa - State Pollution Control Board, 2007 Pg.6 & 7).

8.5.2.1 Pollution from Domestic Sources

"There are about 34 cities, towns and urban conglomerates in the basin area. They include some of the very important cities and towns of Orissa like Bhubaneswar, Cuttack, Sambalpur, Puri, Paradeep etc. The estimated waste water discharge from the urban settlements in Mahanadi basin is about 345000 cum per day. None of these places has any organized sewerage system with treatment of sewage. Consequently, most of the untreated domestic wastewater finds its way to the riverine system, contributing a BOD load of about 68.6 tons per day (assuming the BOD concentration of sanitary wastewater is 200 mg/l). Since the villages do not have any organized water supply or drainage system, the wastewater generated is normally absorbed in the soil. But during rains, the sewage from the soil is usually washed into the streams and rivers, though the load is diluted by the storm water."

8.5.2.2. Pollution from Agricultural Sources

"The basin has 21 completed, 7 ongoing and 38 numbers of proposed major and medium irrigation projects. The quantity of water used for irrigation in the Mahanadi basin is quite large. About 88% of the water used for irrigation is used by crops, held back as soil moisture and lost through evaporation and transpiration. The remaining 12% of the total water used for irrigation constitutes the run-offs. The wastewater flow from agricultural sector in Mahanadi basin is of the order of 2160 million cum per year, of which the return water from Orissa portion is about 1564 million cum per year."

A part of pesticides and nutrients applied as fertilizers gets washed or leached with agricultural return waters. The amount of nutrients so leached increase with the intensity of application of pesticides and fertilizers. Assuming the loss of N,P,K and pesticides as 10%, 5% (due to lower solubility), 10% and 5% respectively, the estimated pollution load due to application of fertilisers and pesticides is given in Table 8.5.

Table 8.5 : Pollution due to Fertilizers and Pesticides in Mahanadi Basin

Component	Consumption (T) / Year	Estimated concentration in return water (1564.34 million M^3/year) (mg/l)
N	47171.7	3.02
P	14932.8	0.48
K	8999.4	0.58
Pesticides	983.2	0.03

8.5.3 Water Quality Trend of Mahanadi

"About 86% of the catchment (72,691 sq. km. out of a total of 84,372 sq. km.) and tributaries of Mahanadi (Seonath, Jonk, Hosdeo and Mond) above the dam are in Madhya Pradesh/ Chhatisgarh. Since several large towns and industries (Rajnandagoan, Bhillai, Durg, Shimoga, Raipur, Bilaspur, Korba etc.) are located on the banks of these tributaries, they carry considerable pollution load to the reservoir. In Orissa, the river Ib with its share of pollutants drains into the reservoir. But in spite of this, the reservoir water almost conforms to Class B, except for TC values.

Sambalpur is the major urban area (population about 1.5 lakhs, district and division headquarters) immediately downstream of Hirakud reservoir (about 5 km.). Apart from being a source of water supply, Mahanadi at Sambalpur is used for bathing and wastewater (untreated) disposal. Hence the expected deterioration of water quality at Sambalpur downstream. From Sambalpur downstream to Sonepur (about 60 km. along the river course), the river travels through a region with no major urban settlement or wastewater outfall. Sonepur is the confluence point of Mahanadi with two of its important right bank tributaries namely Ong and Tel. Thus the water quality at Sonepur upstream, which is immediately downstream of Ong confluence, is quite satisfactory. Though Sonepur is the district headquarter with all consequent activities, the deterioration in the water quality at Sonepur D/s is not as much as expected. This is primarily because Sonepur D/s on Mahanadi is actually the downstream of its confluence with Tel, which has a significant annual average flow with very low pollution load. Moreover, in spite of being the district headquarter, Sonepur is still a small town (population: about 19000) with no noticeable growth in urban activities.

The 100 km. stretch of the roller from Sonepur to Tikarapada does not have any industry or urban settlement on its banks (except two small sub-divisional towns - Boudh and Athamallick) and there is no major wastewater outfall. From Tikarapada to Narasinghpur (about 40 km.), the river flows almost completely undisturbed. The Tikarapada-Narasinghapur sub-basin is neither agriculturally nor industrially prosperous and human activities on its banks are scarce. Hence relatively clean, unpolluted water is expected at Tikarapada and without much change in quality at Narasinghpur.

During its course from Narasinghpur to Cuttack (about 50 km.), the river enters into its deltaic region, characterized by high population density and intense agriculture activities. Hence there is some deterioration in the quality of water entering into Cuttack (Cuttack U/s) particularly in respect of TC, but still conforming to Class C. Within the city (population: about 5.35 lakhs) the river receives considerable untreated wastewater and the water quality gets further deteriorated at Cuttack downstream.

Table 8.6 : pH, DO, BOD and TC values at Hirakud, Sambalpur U/S and D/S (Mahanadi)

Parameter	Year	HIRAKUD				SAMBALPUR U/S				SAMBALPUR D/S			
		W	S	M	PM*	W	S	M	PM	W	S	M	PM
pH	2000	7.6	7.9	7.9	7.6	7.6	7.8	7.8	7.5	7.7	8.1	8.2	7.5
	2001	8.2	7.5	7.8	–	8.0	7.5	7.5	8.3	7.9	7.3	7.6	8.3
	2002	8.2	7.8	7.8	7.8	8.0	8.0	7.8	7.7	8.0	8.3	8.0	7.9
	2003	7.5	7.6	7.0	7.3	7.7	7.7	7.6	7.5	8.0	7.6	7.4	7.9
	2004	7.6	7.9	7.5	8.2	7.5	7.8	7.8	7.8	7.3	7.5	7.6	7.7
	2005	8.2	8.0	7.1	7.8	8.1	7.7	7.2	7.7	7.9	7.5	7.3	7.7
	2006	7.3	7.8	7.5	7.4	7.5	7.8	7.1	7.3	7.5	8.1	6.9	7.7
DO (mg/l) Dissolved oxygen	2000	8.0	6.9	7.5	7.0	7.7	7.1	6.6	6.6	7.2	5.8	6.9	7.5
	2001	7.4	7.6	6.4	-	7.0	7.1	7.3	7.7	6.9	6.2	6.3	7.2
	2002	8.6	7.8	8.7	8.7	8.3	7.9	7.6	8.5	6.6	7.0	6.8	8.3
	2003	9.6	8.6	6.8	6.9	8.8	7.9	7.8	7.8	7.7	7.9	7.0	7.1
	2004	7.8	8.0	8.3	7.4	8.6	8.4	7.5	8.0	7.3	7.9	6.1	7.8
	2005	9.0	8.8	6.9	7.7	8.6	8.2	7.0	7.9	7.9	7.2	6.5	7.4
	2006	8.4	7.8	7.8	8.1	8.1	8.0	7.5	7.7	8.1	6.1	6.7	8.4
BOD (mg/l) Biochemical oxygen Demand	2000	2.7	2.3	3.4	2.7	3.8	2.2	3.2	4.4	4.9	4.2	4.6	5.6
	2001	1.4	2.1	3.2	-	2.2	2.5	3.2	2.7	5.1	3.4	4.0	2.8
	2002	1.5	2.8	2.0	1.5	2.9	2.5	2.3	2.3	4.3	4.4	4.2	3.7
	2003	0.5	1.3	0.8	0.8	1.4	1.8	1.0	1.3	3.1	3.1	1.5	2.3
	2004	1.0	0.8	1.0	0.8	1.3	1.7	1.1	1.4	3.6	3.3	2.1	2.9
	2005	0.6	1.0	0.7	0.4	0.8	1.2	1.4	0.8	3.0	3.2	2.8	3.0
	2006	0.4	0.9	0.8	1.2	2.0	0.7	1.3	1.6	2.5	3.8	2.3	3.2
TC ('00) (MPN/100 ml) Total coliform	2000	1.15	3.55	6.27	0.18	10.03	10.80	21.93	2.60	105.67	192.35	186.67	110.00
	2001	0.44	0.84	9.30	–	0.56	5.63	91.67	8.25	27.33	34.25	123.67	78.00
	2002	1.50	1.40	3.50	14.00	15.50	14.55	57.00	29.00	88.67	161.25	83.33	65.00
	2003	3.00	1.75	8.00	13.00	65.33	68.75	50.33	78.50	163.33	195.00	156.67	180.00
	2004	4.10	5.40	7.90	2.30	29.33	26.0	17.30	17.50	14.67	103.75	249.67	74.50
	2005	13.00	3.30	16.00	21.00	9.57	12.47	107.00	22.50	394.67	277.50	330.00	220.00
	2006	17.00	14.00	22.00	11.00	16.67	17.00	32.67	44.50	470.00	390.00	493.33	69.50

(*) W = Winter, S = Summer, M = Monsoon, PM = Post monsoon (i.e. during October & November).

Table 8.7 : pH, DO, BOD & TC values at Cuttack U/S and D/S (Mahanadi)

Parameter	Year	Cuttack U/S				Cuttack D/S			
		W	S	M	PM	W	S	M	PM
pH	2000	7.6	8.1	7.7	7.8	7.6	8.1	8.0	7.8
	2001	8.2	7.2	7.4	8.4	8.1	7.4	7.8	8.3
	2002	8.0	8.3	7.7	7.6	7.8	8.1	8.0	7.7
	2003	7.7	7.6	7.8	7.8	7.9	7.8	7.6	7.5
	2004	7.9	7.8	7.7	7.8	7.8	7.6	7.3	7.7
	2005	7.9	7.5	7.5	7.8	7.8	7.7	7.7	7.9
	2006	7.8	8.0	7.5	8.1	7.6	8.0	7.9	8.0
DO (mg/l)	2000	7.1	7.0	6.8	6.6	6.3	6.7	6.8	6.0
	2001	7.8	7.1	7.5	7.2	6.1	6.5	6.5	6.4
	2002	7.7	8.4	7.8	9.5	6.8	7.4	7.4	8.7
	2003	9.5	7.9	7.2	8.1	8.3	7.9	6.9	7.5
	2004	8.4	8.1	7.5	7.5	7.9	7.5	7.2	7.6
	2005	8.3	8.1	7.1	7.6	8.0	7.8	7.1	7.5
	2006	8.4	7.8	7.0	8.1	8.1	6.6	6.8	6.9
BOD (mg/l)	2000	3.6	2.1	2.4	2.8	5.7	3.8	4.3	4.4
	2001	2.4	2.1	2.2	2.6	4.2	3.7	2.7	3.6
	2002	2.7	2.1	2.4	2.7	3.4	3.7	3.8	3.8
	2003	1.0	1.8	1.2	1.3	3.2	3.4	1.5	1.8
	2004	1.3	1.5	1.2	1.4	2.6	2.7	1.9	2.2
	2005	1.3	1.5	2.0	0.8	3.0	3.1	2.6	1.7
	2006	1.1	1.0	1.5	1.2	2.4	3.2	2.1	2.5
TC('00) (MPN/100 ml)	2000	22.0	14.50	53.27	18.00	32.00	65.50	354.87	425.00
	2001	45.33	33.50	39.00	27.65	110.00	94.00	233.33	78.00
	2002	39.93	18.75	18.67	12.50	88.67	90.00	90.00	175.00
	2003	12.00	10.75	15.67	30.95	121.67	46.25	32.33	79.50
	2004	11.23	8.00	22.10	7.45	149.00	38.25	386.33	297.00
	2005	17.63	25.47	51.67	17.50	210.00	635.00	920.00	63.50
	2006	18.67	21.00	21.63	10.45	46.67	197.25	956.67	1260.00

(Source: Water Quality of Major Rivers of Orissa - State Pollution Control Board 2007 Pg.59).

Ib: Water quality of this left bank tributary of Mahanadi is monitored at four locations - Sundergarh, Jharsuguda, Brajrajnagar (U/s and D/s). Till about late nineties, the water quality at Brajrajnagar was a matter of much concern due to discharge of effluent from a large paper mill. The mill has been closed since December 1998. Since none of the three towns is a large urban centre and there is no organized domestic wastewater discharge to the river, the water quality generally remains at the Class C level. Of late Jharsuguda is turning into an important industrial hub of the state.

Kathajodi: The monitoring station at Cuttack D/s on Kathajodi, a distributary of Mahanadi, is characterized by an untreated domestic wastewater discharge outfall at its upstream and expectedly, there is significant deterioration of water quality to below Class C with respect to BOD, TC and large deviations in FC to make the water unacceptable for most beneficial uses.

Birupa: Birupa, another distributary of Mahanadi is monitored at the downstream of Choudwar, a small town, which had in the past, significant industrial activities with a textile, a large pulp and paper and a charge chrome industry with its thermal power plant.

Presently only the charge chrome industry is in operation with marginal water pollution potential. Except occasional deviation in the coliform count, the water quality generally conforms to Class-C i.e. Drinking water source with conventional treatment followed by disinfection." (Source : Water Quality of Major Rivers of Orissa - 2007 State Pollution Control Board Pg. 47-49). pH, DO (Dissolved oxygen), BOD (Biochemical oxygen demand) and TC (Total coliform), values at Hirakud and Cuttack are given in Table 8.6 & 8.7.

8.6 Research Activities

Research activities in the field of irrigation, drainage and flood control have to keep pace with latest technological development and have to be accelerated with the expansion of the development programme. At central level, research activities are carried out by Central Water and Power Research Station (CWPRS) at Pune, Central Soil and Material Research Station (CSMRS) at New Delhi, National Institute of Hydrology (NIH) at Roorkee and State level irrigation institutes. Besides infrastructure facilities are available at the National Institutes viz. IITs, NITs, Water Technology Centre, CGWB and some of the leading Agricultural Universities.

National Conference of Irrigation and Water Resources Ministers of States and U.T.s held in July 1986 recommended an allocation of one percent of the capital investment earmarked for Research so that research facilities could be expanded to find out technically sound and economic designs for water resources structures.

The major research activities of CWPRS, Pune include model testing, basic research and evaluation of economic designs for ensuring safety and operational efficiency of river valley projects. CSMRS conducts basic and applied research and provides advice in the field of geo-mechanics and constructions services to both Central and State Governments.

WALMIs under State Government and Water Resources Center in some Universities undertake action research on the live systems and adoptive trials on the farmer's field for improved water management.

In Orissa, fundamental and applied research in water resources sector should emphasize particularly on water management, water logging, river training works, coastal protection, economic design and new methodology for faster construction.

8.6.1 Specific Research Needs relating to Hirakud :

(i) Adequacy of the existing energy dissipation device in the event of occurence of P.M.F./S.P.F./High floods may be examined with proto type/model study for hydraulic jump and downstream retrogression.

(ii) Cavities are occurring at a specific location i.e. where water is entering into the sluice. This aspect may be studied on the model/prototype for appropriate remedial measure.

(iii) Hirakud is in existence for more than five decades. It is needed to study its impact on the environment with respect to river regime, ground water, fauna and flora, human health, quality of life, quality of water in reservoir, river water downstream adjacent to dam and water in the command area. Change in soil quality and fertility need long term study.

(iv) Research is needed to obtain maximum production per unit of water for different crops suited to the climatic and soil conditions in the Hirakud command. Crop planning itself has an important bearing on water demand and in turn on farmer's income.

(v) Basic, fundamental and applied research in water resources sector in the field of irrigation and flood control have to keep pace with development of technology. In our country, research activities are undertaken by CSMRS, CWPRS, NIH at Roorkee, in IITs, in NITs, Water Technology Centres, leading Agricultural Universities and few State level Irrigation Research Institutes. As stated earlier, in the National Conference of Irrigation / Water Resources Ministers of States in July 1986, it has been recommended to allocate one percent of capital investment on research in formulating technically sound and economic designs for water resources structures. Keeping this in view, Hirakud Research Station and Hydraulic Research Station may be developed to carry out basic and applied research for the problems faced by existing, ongoing and proposed river valley projects in the State.

8.7. Issues and Concerns :

8.7.1. Introduction :

"As we try to foresee and plan for the future, it is clear that we need to grapple with several critical issues which are likely to shape our future. Finite availability of water in the face of increasing demand will prompt us to explore all possibilities of bridging the gap between the potential availability and the actual supply. We also realize that even in the optimistic scenario, the task of balancing the availability and the requirement of water will raise major issues of equity and efficiency." Distribution of the available supply between different areas of the project and within the urban and rural areas, or between different sections of population raises questions of prioritisation among various uses as much as the access to water, in quantitative and qualitative terms, at different locations and for different groups of people.

"Water resources are developed for meeting the basic needs like drinking, food production, energy, clothing fibres, besides requirements for navigation, flood control etc. At the time of start of the planning era, the country's food production was at a low level of about 50 million tonnes, requiring massive import of food grains to meet the needs of even a population of only 350 millions. Most of the rural population and large segments of urban population depended on dug wells or river flows for drinking water needs. Many cities were dependent on reservoirs and barrages, even then for their organised water supply and sewerage schemes. There was little hydropower generation then. Drought and floods were taking heavy toll of life and property, besides disrupting the economy frequently. The River Valley Development plans started in a big way since then. The population

multiplied alongwith urbanisation and industrialisation. The drinking and public health water needs for these societies grew exponentially. The reservoirs were increasingly built to serve more than one beneficial need. The concept of multipurpose river valley development was adopted. Attainment of self sufficiency in food to have independence of thought and action became the cornerstone of India's water policy. India today has the largest irrigated area in the world. By the end of 2000 A.D. the water resources development alongwith green revolution, has enabled the country to produce over 200 million tonnes for a nearly tripled population of about 1,000 millions and provided food security. In the designing and building of large projects (including big dams, long and intricate tunnel systems, underground power stations, pumped storage facilities, barrages and weirs, vast canal networks, and so on) under complex and difficult conditions, Indian engineering capabilities are second to none. The country's capabilities for the manufacture of much of the equipment need for the projects (construction and earth-moving equipment, generators, turbines etc.) are also impressive. And yet there are some shortcomings.

An important one is that the pre-occupation in the past has been mostly with irrigation. The move away from this orientation towards a larger perspective of 'water-resource planning' is a relatively recent development, and has been only partly achieved. There is now a growing recognition that the planning of individual projects must be a part of, and must derive from, a larger plan for a suitable hydrological unit (a basin or sub-basin); that such planning should match land and water management; that it should be integrated and holistic; that it should be fully 'participatory' and should involve the people at every stage; and that it should be inter-disciplinary. However, this has not happened to any significant extent and there is much to be desired by way of people's involvement in the planning and implementation processes. As for environmental/ecological concerns, there is general agreement that they should be 'internalised' *ab initio* in project planning, but this has not become fully operative."

(Source : Integrated Water Resource Development – A plan for action, Report of NCIWRD, Vol. I, M.O.W.R., Govt. of India, Sept. 1999, Pg.17)

8.7.2 Issues :

a) When projects are completed, it is often seen that the benefits fall short of original expectations. This happens as the benefits are exaggerated to justify the investment while calculating the benefit cost ratio. Only tangible benefits are taken into consideration but intangible benefits are not considered as those can not be evaluated in monetary terms. But in case of Hirakud, which was planned to provide 148% of annual irrigation (i.e. Khariff 100% & Rabi 48%) has exceeded the designed potential envisaged in the project report vide Table7.4 Eventhough the irrigation potential has reached 173%, rice is dominating both in Khariff and Rabi. The effect of dominance of rice has been described under Sec. 7.1.3.2.2. This emphasises on crop diversification for commercialisation of agriculture. The diversification will reduce erosion, improve soil fertility, increase yield, require less nitrogen fertilizer in case of legumes and minimise risk of crop failure. Thus genetic diversity and location specific varieties are essential for achieving sustainable production.

b) Turning now to question of equity, we are faced with few issues. First, in the context of large projects like Hirakud and others, there is the question of inequitable incidence of costs and benefits; i.e. those in the submerged area bear the social costs of the project (displacement, loss of livelihood etc.) and the people living in command area enjoy the benefits. Efforts to give the former a share in those benefits and to regard them as partners in development have been inadequate. Not only in Hirakud, but in almost all the major projects of the country, this situation prevails for which environmentalists, NGO's and other social activists are making hue and cry. The engineers, planners

and administrators must learn a lesson from this. Even after more than five decades, some people from submerged areas of Hirakud are claiming their compensation (Refer Sec 3.4.7). This may be solved on priority basis within a time frame.

c) Other matter calling for urgent action include the sharp deterioration in quality of river water and reservoir water. This poses serious hazards to the health of the nation.

d) Further shocking is the waste water generated by all users i.e. agricultural, industrial, municipal and domestic. There is total absence of awareness of the scarcity of this precious resource. In general, in our anxiety for development, we seem to have been more concerned with what is known as 'water resource development' than with water management. A shift of focus towards water management is urged and merits serious consideration.

In case of Hirakud for better management of irrigated water, W.U.A.s have been formed and system handed over to them for upkeep and maintenance.

e) Floods give rise to some difficult issues : i.e. Is flood control feasible by a single reservoir at Hirakud which has no separate reserve space for flood cushion ? Should the people of coastal region learn to live with floods ? What measures are needed to minimise the damage caused by floods ?

The solution is not that simple as everybody thinks. Under Sec. 8.2, 8.3 & 8.4 this aspect has been dealt. Presently, the embankment, earlier constructed in the deltaic region (Stage I & II delta) along both the sides of rivers should be strengthened to withstand a flood of 14 lakh cusec (39638 cusec) at the head of delta. The river bed is being silted up thereby encroaching upon the free board provided. At certain intervals it is required to raise the height of embankments. Engineers and planners must think of a detention reservoir above Naraj to retain the peak flood at least for 8 to 10 hours so that delta gets some relief during flood.

8.7.3. Concerns :

(a) The first and foremost concern is the safety of the dam which overrides all other considerations. This aspect becomes very critical during reservoir operation specially during months of September and October. The operation must be in such a manner that it balances in the best possible way the two conflicting uses i.e. conservation and flood control.

(b) There exists no inter-state agreement between Odisha and Chhatishgarh for sharing waters of Mahanadi. In this regard Sec. 6.4 of the book may be referred to. This vexed problem must be settled by both the state Governments with the active cooperation of Government of India before it is too late.

(c) Though it has been diagnosed that cracks in Hirakud dam spillway is mainly due to deleterious alkali-silica reaction (ASR) which has been experienced in many countries the world over, still then it is a matter of concern. Any distress to this gigantic structure, which has served more than 5 decades, sends a panic message all over the state. But surface cracks and under water cracks have not been treated with epoxy after 1998. Development of cracks have not ceased completely. Thus continuous surveillance of cracks, their treatment and long term efficacy of the epoxy compound used must be monitored.

8.7.4 Tasks Ahead :

(i) Effects of projects already constructed in the upper Mahanadi catchment above Hirakud are to be studied such that those do not come into conflict with operation of reservoir in respect of flood control and conservation. An integrated reservoir operation with co-operation of Chhatisgarh Government will help in preparing the predepletion plan. Govt. of India as well as CWC may also

be moved for not sanctioning any further major / medium projects upstream of Hirakud which will jeopardise the basic objectives of Hirakud Dam.

(ii) Eco-friendly strategies for conjuctive use of surface and ground water to promote sustainability of crop production in a given water resources endowment may be developed. This will reduce waterlogging. Network of piezometers or observation wells of standard density in the command may be established to monitor ground water table.

(iii) Reservoir sedimentation study by hydrographic survey once in every six years and by Satellite Remote Sensing Technique once in every 3 years should continue to assess both Dead storage and Live storage capacity of the reservoir. Government of Chhatisgarh may be moved through GOI for afforestation as major portion of catchment upstream of Hirakud lies in Chhatisgarh.

(iv) Seepage observation of earth dam, dykes and masonry / concrete dam should continue and datas so collected are to be analysed.

(v) Socio-economic and Environmental impact studies for this project must be done to assess how far the project has contributed to the people of the region in particular and the State in general in every sphere of life viz. increased food production, employment generation, mitigating drought and flood situation, power generation, providing municipal and industrial water, health, education, communication and better environment etc.

Though some insight has been made on the above aspects in this book, it is necessary that a performance evaluation study may be undertaken early as the project has served for more than five decades. This exercise may be taken up regularly once in every ten years.

8.7.5 Concluding Remarks :

Irrigation is the prime mover for making agriculture an attractive and profitable one. As the agriculture is at the threshold of commercialisation after liberalisation and globalisation, there is urgent need to shift from routine cultivation of paddy (monoculture) to a newer multiple cropping system to meet the ever increasing demand of pulses, oilseeds, vegetables, fodder, fiber and spices etc. when there is assured irrigation round the year as in Hirakud command. This will not only reduce poverty (which is said to be the worst polluter) but will bring smiles in the faces of millions.

It has been recognised that access to electricity and its use are important in raising the standards of living of the people. Table 7.25 shows the monthly power generation from Hirakud (Burla and Chiplima Power Plants) in million unit from 1960-2008, which has lighted the villages and provided electricity to the industries.

Hirakud dam has performed to its best to moderate the floods in delta. Most of the floods during post-construction period are due to contribution from downstream catchment (48700 sq.km) on which dam has no control. A reservoir is more effective for flood control if a designated space is reserved and a reservoir regulation arrangement is laid down. This is not the case with Hirakud reservoir. In this regard, Table 8.2 may be seen which gives the list of projects contemplated on the tributaries of Mahanadi in the downstream of Hirakud. Flood space may be kept in these reservoirs during planning.

Though the dam has provided immense benefits, there are some adverse social impacts also. A very few people of submerged area have their grievances regarding non-receipt of compensation. Their attachment to ancestral homes cannot be always assessed in monetary terms. This fact underlines the need to consider the displacement issue with utmost humanitarian approach and settle within a limited time frame. Let the 'outstees' not think themselves as 'Losers'.

If water is essential to sustain agricultural growth and productivity, it is even more vital for life and healthy living. Over half of the morbidity cases arise from the drinking impure water. Nevertheless Hirakud has actually served as the source of raw water supply for the hundreds of villages and towns in the command area and nearby locality. Sanitation has greatly improved as compared to the pre-1960 period. Better quality of water and general improvements in sanitation have helped control diseases. Besides, there were hardly any health-care system then. Now there is a full-fledged Medical college and hospital at Burla, District headquarter hospital at Bargarh and primary health centres in many villages of the command area.

Burla has become an educational centre with two universities i.e., Sambalpur University and V.S.S. University of Technology, V.S.S. Medical College, Schools and Colleges.

The development of irrigation has resulted in the increase of employment opportunities. The all-round development triggered by irrigated agriculture has been a boon for the unemployed and under-employed, as also the landless categories of people.

The lessons learnt from this study is that Hirakud apart from its direct impact, has a significant indirect and induced impact as well, which must be accounted for and taken into consideration for cost-economic evaluation of other river valley projects.

Hirakud is, no doubt, a success story of sustained humane development with overwhelmingly beneficial consequences. Above all, it stands tall as concrete testimony to the far-sighted vision and wisdom of the pioneer dam builders of independent India.



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Abbreviation	Full form of Abbreviation
ASR	Alkali Silica Reaction
ASCE	American Society of Civil Engineers
ARO	Assistant Research Officer
BM	Bench Mark
BNP	Bhakra Nangal Project
BHEL	Bharat Heavy Electricals Limited
BMC	Billion Cubic Meter
BOD	Biochemical Oxygen Demand
CA	Catchment Area
CBIP	Central Board of Irrigation and Power
CDO	Central Design Organisation
CESCO	Central Electric Supply Company
CGWB	Central Ground Water Board
CSMRS	Central Soil and Material Research Station
CWC	Central Water Commission
CWI&NC	Central Waterways, Irrigation and Navigation Commission
CW&PC	Central Water and Power Commission
CWPRS	Central Water & Power Research Station
CADA	Command Area Development Agency
CJ	Contraction Joint
CFM	Cubic Feet per Minute
Cyd/Cuyd	Cubic Yard
CCA	Cultivable Command Area
COT	Cut off Trench
DSARP	Dam Sefety Assurance & Rehabilitation Project
DSRP	Dam Safety Review Panel
DSL	Dead Storage Level
DoWR	Department of Water Resources
DP	Displaced Person
DO	Dissolved Oxygen
D/S	Downstream
EAP	Emergency Action Plan
EIC	Engineer-in-Chief
EIA	Environmental Impact Assessment
ERM	Extension Renovation and Modernisation

FMO	Field Meteorological Office
FA & CAO	Financial Adviser & Chief Accounts Officer
FEM	Finite Element Method
FC	Flood Control
FRL	Full Reservoir Level
FSD	Full Supply Depth
FSL	Full Supply Level
GEC	General Electric Company
GSI	Geological Survey of India
GTT	Glass Tell Tale
GoI	Government of India
GoO	Government of Orissa
GRIDCO	Grid Corporation of Orissa
GCA	Gross Command Area
GL	Ground Level
HR	Head Regulator
HLC	High Level Canal
HDCRP	Hirakud Dam Crack Review Panel
HDP	Hirakud Dam Project
HDRD	Hirakud Dam Rehabilitation Division
HLO	Hirakud Land Organisation
HRS	Hirakud Research Station
ICAR	Indian Council of Agricultural Research
ICID	International Commission on Irrigation & Drainage
IIT	Indian Institute of Technology
IMD	Indian Meteorological Department
INCID	Indian National Committee on Irrigation & Drainage
IRC	Indian Road Congress
IWRS	Indian Water Resources Society
IMT	Irrigation Management Transfer
I&P Deptt.	Irrigation and Power Department
KWH	Killowatt Hour

LED	Left Earth Dam
LIP	Lift Irrigation Point
LS	Lumpsum
MDDL	Maximum Draw Down Level
MWL	Minimum Water Level
MoWR	Ministry of Water Resources
MIP	Minor Irrigation Project
M&I	Municipal and Industrial
MAcft.	Million Acre feet.
MCum	Million Cubic Meter
MU	Million Units
NBT	National Book Trust
NCA	National Commission on Agriculture
NCCBM	National Council on Concrete and Building Material
NCIWRD	National Commission for Integrated Water Resources Development
NIH	National Institute of Hydrology
NIT	National Institute of Technology
NPC	National Planning Council
NPP	National Perspective Plan
NRSA	National Remote Sensing Agency
NWDA	National Water Development Agency
NGO	Non Government Organisation
NOF	Non Over Flow
NA	Not Available
NR	Not Reported
OC	Open Cast
OERC	Orissa Electricity Regulatory Commission
OF	Over Flow
OFC	Orissa Flood Commission
OGLS Act/Rules	Orissa Government Land Settlement Act/Rules
OHPC	Orissa Hydro Power Corporation
OWPO	Orissa Water Planning Organisation
OWRCP	Orissa Water Resources Consolidation Project
PP	Pani Panchayat

PIM	Participatory Irrigation Management
PTT	Plaster Tell Tales
PH	Power House
PMF	Probable Maximum Flood
PHC	Public Health Centre
RD	Reduced Distance
RL	Reduced Level
RCC	Reinforced Cement Concrete
R&R	Rehabilitation & Resettlement
RDC	Revenue Divisional Commissioner
RPM	Revolution Per Minute
RED	Right Earth Dam
SNERI	Sailendra Narayan Engineering Research Institute
SC & ST	Scheduled Cast and Scheduled Trible
SPARC	Spatial Planning and Analysis Centre
SPCB	State Pollution Control Board
SEU	System Engineering Unit
TWL	Tail Water Level
TVA	Tennessee Valley Authority
TBL	Top Bank Level
TC	Total Coliform
UG	Under Ground
USSR	United Socialistic Soviet Republic
USA	United States of America
U/S	Upstream
USBR	United State's Bureau of Reclamation
VRB	Village Road Bridge
WALMI	Water And Land Management Institute
WRM&T	Water Resources Management and Training
WUA	Water Users Association
WCD	World Commission on Dams


RIVER BASINS OF ORISSA
N
Jharkhand
West Bengal
Chhattisgarh
Andhra Pradesh
Bay of Bengal
ORISSA
Mahanadi Basin
Brahmani Basin
Baitarani Basin
Subarnarekha Basin
Rushikulya Basin
Vamsadhara Basin
Nagavali Basin
Indravati Basin
Kolab Basin
References :
River & Nallah
Basin Boundary
State Boundary
Dams of National Importance
① Hirakud
② Rengali
③ Balimela
④ Upper Kolab
⑤ Machkund (Jalapat)
⑥ Salandi
⑦ Upper Indravati Project
⑧ Indravati
⑨ Potteruru
⑩ Rupai
⑪ Muran

ORISSA MAP SHOWING HIRAKUD, RENGALI INDRAVATI, BALIMELA, KOLAB & SALANDI PROJECTS

