

RENGALI MULTI-PURPOSE PROJECT :

Across River Brahmani, Odisha



Release of Water through Spillway

by

Er. G.C. Sahu

Er. G.N. Das

(Through Walmi, Odisha)

Indian National Committee on

Surface Water (INCSW)

(Constituted by MoWR, Government of India)

New Delhi

2014

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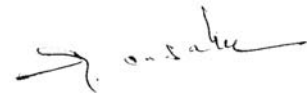
About the Author

Er. G.C. Sahu is Former Engineer-in-Chief, Water Resources Department Govt. of Odisha. He graduated in Civil Engineering from the University College of Engineering, Burla in 1963 and completed ME from University of Roorkee in 1976. From 1963 to 2000 he worked for Govt. of Odisha in different capacities from Assistant Engineer to Engineer-in-Chief (Water Resources), in the field of Planning and Construction of major and medium River Valley projects i.e. Upper Indravati H.E. Project, Rengali Dam, Samal Barrage, Mahanadi Barrage, Naraj Barrage, Paradeep Port and Island Irrigation Project on Mahanadi & Chitrotpala to name a few. After superannuation in 2000, he worked as senior advisor in a Govt. of India undertaking, Member of Task force on Inter-linking of Rivers (constituted by Govt. of India), Chairman Dam Safety Panel, Govt. of Odisha and Member of an expert team for advising Govt. of Gujarat, Tamil Nadu and Andhra Pradesh on dam related issues.



As a writer of repute, Sri Sahu has so far written 13 nos. of books in Odia literature and he recipient of Odisha Sahitya Academy Award (2001). He was felicitated by Odisha Sahitya Academy for his contribution to Odia literature (2004). He is also recipient of Odisha Bigyan Academy Award (1996) and was felicitated by Odia Bigyan Academy for his lifelong contribution to engineering and science (2014). He is editing a thought provoking Odia magazine since 2007.

He has more than 30 technical papers to his credit (13 award winning from Institution of Engineers, Odisha Chapter) and published two research publications, namely; **‘History of Irrigation Development in Orissa’** (2009) and **‘Hirakud Dam Project: Its Background & Performance’** (2012) (All funded by INCID, Ministry of Water Resources, Govt. of India). This is a unique case where both creative literature and technical writing have been nicely blended together.



(Er. G.C. Sahu)

Prologue

As per 2011 census, India's population stands at 1210 million which comes to 17% of world's population of 7121 million. But India's geographical area (3287726 sq.km.) is 2.45% of world's land area. Our food grain production is about 250 million MT where as world's food grain production is 16 times more than that of ours. Above statistical figures present a grim and desperate situation for our country. Further these could have serious implications for the present and future generations of our country men as also for natural ecosystems.

The distribution of water resources in the country is highly uneven over space and time. About 80% of the run off in our rivers occurs in 90 to 100 days of the year and there are regions of harmful abundance causing flooding and acute scarcity. Vast populations live in drought affected areas. The country has to grope with several critical issues in dealing with water resource development and management. 'The issues would be confronted more satisfactorily to the extent that we are able to develop a national consensus. Major attitudinal and organisational changes would be necessary to deal adequately with all the issues and concerns. In this context, the controversy over and polarisation of views on large projects and local watershed and water resource conservation and development is unfortunate. The two are complimentary to each other and there is need for the whole range of structures - large to small - in trying to meet people's requirements.'

Due to rise in cost of construction of major and medium water resources projects, long gestation period, pressure on land due to increased population, delay in environmental and forest clearance, law and order problem and inter-state aspects etc, it has become extremely difficult to take up any major projects in the country as large dams have become very controversial in recent years. Both State and Central Government may evolve certain policies such that new multi-purpose dam projects can be taken up in the country as world wide dams have played a key role in development. They are built to control floods, irrigate agricultural lands, generate power, supply water to meet domestic and industrial needs, enable navigation, provide recreation and improve environment to a great extent. Besides these direct benefits, there are lot many indirect benefits too.

Issues relating to dams, their benefits and impact, have become one of the battlegrounds in the sustainable development arena. While launching the WCD report, it was pointed out that the 'problem, though is not the dams. It is the hunger. It is the thirst. It is darkness of a township. It is the townships and rural huts without running water, lights and sanitation'.

The problem of water resources development are neither mathematical nor completely hydrological. The solutions are not based on some set of scientific laws or principles but requires practical solutions to a good number of problems related to engineering, geology, hydrology, economics, sociology, agriculture, environment, political, finance and management etc. No set solutions/principles can be applied for such an art of planning.

Good dam sites are gradually getting exhausted. In that respect, Rengali dam which happens to be a gravity dam is the gift of nature. The foundation of the dam was free from major defects and has absolutely not posed any grave problem during construction.

After successful completion of the report on 'Hirakud : Its background and Performance' in 2012 sponsored by INCID, we are privileged to undertake the study on Rengali Dam across river Brahmani the second largest river in the state of Odisha which is also a multi-purpose river valley project. The dam has successfully served its purpose in the sectors of flood control, energy production,

industrialisation etc. But due to paucity of funds and inordinate delay in land acquisition the irrigation sector has yet to be fully developed.

This report enumerates events starting from conception to completion of the dams and partly developed irrigation system. It describes the problems encountered during construction of dam, power house and canal system, problems facing during post-construction period and performance till to date. Writting a book of such complex nature and magnitude from inception upto this stage is, no doubt, a herculean task due to poor documentation at every level.

The scheme of presentation of this book has been arranged as follows. The book has been broadly divided into seven chapters. Those are :

1. Introduction
2. Project Planning and Investigation
3. Project Construction
4. Model Study, Design and Quality Control
5. Displacement, Rehabilitation, Resettlement and Environmental issues
6. Post Construction Scenario
7. Conclusion

Chapterwise details are briefed as under :

Chapter-I describes the back ground leading to the construction of the Rengali Multi-purpose Project, occurrence of floods, droughts and farmines during pre-construction period, social conditions of the people prevailing then. Broad outline of the river Brahmani from its source to sea, its course and catchment have also been narrated.

Chapter-II deals with the details of the proeject planning including selection of sites for both dam and barrage after conducting topographical and geological investigation, hydrological study encompassing selection of design flood. The irrigation planning from the barrage 35 km downstream of the dam have also been discussed. Statutory clearances from various authorities have been presented in tabular form.

Chapter-III narrates the details of construction of various components of the Dam, Power House and Barrage with sequence of construction. Foundation excavation, Blockwise geology and Foundation treatment including consolidation and curtain grouting have been described. Several alternatives have been studied for River Diversion and the most realistic one befitting to the site condition has been accepted. Original and revised cost estimates with year wise statement of expenditure have been furnished.

Chapter-IV details regarding Model Study of both dam and barrage, Design and Quality Control aspects. Model study of the dam was carried out at CWPRS, Pune to determine the hydraulic behaviour of dam components. Model study of the barrage was conducted with pre-barrage condition and with barrage in position for deciding its length, divide walls and silt excluders etc. Few Specific Notes could be obtained from CWPRS by personal contract. Unfortunately neither Model study reports nor Design of the dam and barrage are available in the Project library. Similar is the position of Design. As regards Quality Control, the materials used in the dam and the barrage were tested prior to their use. Further, for ensuring the quality of workmanship, inspection was organised at each stage for both masonry and concrete starting from production to final placement.

Chapter-V discusses on the Displacement, R&R and Environmental issues as those have become the burning topic now in any major river-valley projects. Moreover, DoWR, Government of Odisha in their letter No.1091 Dt.03.04.2012 addressed to INCID expressed that both R&R and Environmental aspects to be studied in detail. That is why, one chapter has been specifically devoted citing the R&R

problem of Sardar Sarovar Project, ICID's position on the effectiveness of Dams for development, water quality of river Brahmani and the impact of river flow on the mangroves of Bhitarkanika including the sustainability of mangrove ecosystem.

Chapter-VI is on the Post construction scenario. It highlights on the sedimentation aspects of Rengali and other Indian reservoirs, Reservoir operation details of Rengali as prevailing now with case studies of other reservoirs. Flood control, Power generation and irrigation network developed upto June 2013, Municipal and Industrial water supply have been dealt. It also presents a brief account of Instrumentation, Pisciculture and Dam Safety aspects.

Chapter-VII sums up with discussion on problems confronted during construction of Dam, Power house both Left and Right Main Canal system passing through high cutting zones and RMC passing over coallier area, Discussion on large dams Vs small dams with performance analysis, Issues and concerns alongwith Intra-state river links followed with concluding remarks.

The authors express their sincere gratitude to MoWR, GoI, INCID (now INCSW), WALMI (Odisha), CWC, CBIP, CWPRS, SPCB, GSI, Government of Odisha, EIC(WR) Odisha, O.H.P.C., Directorate of Fisheries and other such institutions/organisations including their staff for active co-operation during every stage of R&D scheme. Documents available with friends and colleagues either in service or retired were very valuable for preparation of the report. Besides, we express our gratitude to the authors of the books and journals from where we have collected lot of datas and incorporated in the compilation.

We express our hearty thanks to Er. R.C. Jha, Er. R.C. Tripathy, Er. Manoranjan Mishra, Er. S.C. Churchi, Er. J.B. Mohapatra, Er. Khitish Ch. Das, Er. Harmohan Pradhan, Dr. Joy Gopal Jena, Er. Pravakar Behera, Er. Sushanta Ku. Rath, Er. A.K. Das, Er. Sagar Mohanty, Dr. R.N. Sankhua of N.W.A., Pune, Sri M.S Shitole of Pune, Sri P.T. Sinharoy of Calcutta, Sri J.P. Mohakul of G.S.I., Sri Subhkanta Prusty of SPCB, Er. Y. Samal and Sri S.N. Behera of OSDMA for rendering immense help in timely supplying the relevant documents needed for the R&D scheme. We extend our thanks to Sri Simanchal Das, Sri R.K. Maharana & Sri A.K. Sahoo for D.T.P work.

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

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CHAPTER-1

INTRODUCTION

1.0 Preamble:

The coastal districts of Odisha i.e. undivided Cuttack, Puri and Balasore, with alluvial plains and high density of population, were subjected to the vicissitudes of high floods from time immemorial. The three major rivers of Odisha, the Mahanadi, the Brahmani and the Baitarani were devastating large tracts of coastal plains and their ravages tell a grim and sorrowful tale of the sufferings of the people.

It is a paradox that these three rivers which have built a combined delta and nourished it year after year like a mother by depositing silt thus increasing agricultural productivity, have also become agents of destruction during high flood.

Considering the magnitude and seriousness of the problem of floods and droughts in the delta, even the British Government made certain attempts to tame these rivers by constructing weirs at a number of places. This problem received serious attention of Government in 1927. Sir Arthur Cotton, the pioneer of the flood control and irrigation schemes of Odisha, envisaged that the construction of weirs, while controlling floods, would provide opportunities for irrigation and navigation and thus would bring about an increase in the revenue of the state. While the schemes were helpful for irrigation and navigation, they had virtually no impact on flood control. During 1938-39, it was estimated by Mr. J. Shaw, the then Executive Engineer that between 1910 to 1938 total flood damage was to the tune of Rs.3.5 crores, the maximum loss being Rs.66.0 lakhs during 1919.

Floods and droughts have remained a recurring feature of the delta. Attempts at finding suitable methods of controlling the floods, were being made for many years. In 1927, a committee of senior engineers went into the question and made several recommendations which, however, were not given effect due to financial constraint.

The three principal rivers of the state i.e. the Mahanadi, the Brahmani and the Baitarani resemble one another in many ways. They all run approximately parallel. Each river enters the delta of Odisha in one stream and almost immediately divides into number of branches due to deltaic action. Another peculiarity of these rivers is that they are inter-connected intricately and during high floods cover the entire country side with vast expanse of water. Part of Mahanadi river water flows through Birupa (a tributary of Mahanadi) into the Brahmani, which also receives water from Baitarani via Burha. "When the Brahmani floods comedown, they back up the Baitarani water at the junction of two rivers and thus impede the discharge of the latter, diverting into the low lying plains of Dhamnagar (vide Irrigation, Inland navigation and flood problems in North Orissa by P.Mukherjee, Page 44)". Floods in Mahanadi also aggravates flood in Brahmani. The Birupa and Genguti carry the flood of Mahanadi into the Kimiria which rejoins Brahmani at Indupur. This is a very complex system when flood in one river generally causes flood like situation in one or two other rivers.

During early part of 19th century, two weirs, one across the river Brahmani and the other across its branch Patia were constructed respectively at Jenapur and Jokodia to provide irrigation and navigation facilities through H.L.C. Range-II and Dudhei canal. The devastating floods of 1925, 1926 and 1927 in the major rivers of the state prompted the Government of Bihar and Odisha to set up a Flood Inquiry Committee to probe into the causes of floods and suggest remedial measures. After examining the various causes of floods in the Brahmani, the Committee recommended the demolition of the weir at Jenapur across the Brahmani. In 1930, Brahmani weir was dismantled as it was observed that the distribution of flood between Brahmani and Patia was being adversely affected causing flood congestion. Subsequently Dudhei canal and H.L.C. Range-II became defunct.

Again following the disastrous floods in 1937 in all rivers, Sir Visveswaraya was requested by the Government of Odisha to advise on the flood control problem. Sir Visveswaraya, stressed the need for detailed scientific surveys and continuous observations of the rivers and the delta. He visualised that flood can be controlled by the construction of storage reservoirs. He stressed the multi-purpose nature of such reservoirs. "If a reservoir is constructed, it may prove useful in several other ways as well-for extending irrigation, generating electric power, etc. Once floods come under effective control the whole area may be transformed into a prosperous region". (Source: Economics of a Multi-purpose river Dam by N.V. Sovani & N. Rath, 1960, Pg. 1)

1.1 Background leading to the Construction of the Project:

"As early as 1928, a committee was set up to study the flood problem in Orissa, which was followed by the Flood Advisory Committee (1938-39). The 1928 Committee considered the problem as disposal of excess flood water and the 1938-39 Committee viewed the problem as one of proper distribution and disposal of excess rain water. The committee broadly recommended a system of embankments to control flood."

Finally at a conference held at Cuttack on 8th November 1945 under the chairmanship of Dr. B.R. Ambedkar, it was unanimously agreed that the potentialities of Mahanadi for unified multipurpose development be thoroughly investigated by CWI&NC. This paved the way for construction of Hirakud Dam. The Hirakud Dam was completed in 1957 primarily to control flood. "After the Srinagar conference of Irrigation Ministers, Government of India (GoI) made a plan to build two multi-purpose dams in Odisha with a total expenditure of Rs.45 crore for Rengali (on Brahmani river) and Bhimkund (on Baitarani river) projects. While construction of Rengali Dam was started in 1973 and completed in 1985 to impound 2.47 million Ac. ft. (3.036 million ham) of water, Bhimkund Project was cancelled (Refer: Human Development Report 2004, Government of Orissa Pg. 179)." Thus construction of Hirakud Dam was the driving force which led to the construction of Rengali Multi-purpose Project for flood control, irrigation and power generation. Finally, the visit of Dr. K.L. Rao, the then Minister of Irrigation and power to the affected areas after the ravaging floods of August 1971 focussed on the necessity of taking up Rengali Multi-purpose project.

Government of India approved the plan of construction of the Hirakud Dam across river Mahanadi which was completed in 1957. Hirakud besides giving annual irrigation to 2.67 lakh ha. and generating power has provided flood protection to 9500 sq.km of area in the undivided districts of Cuttack and Puri. But Brahmani and Baitarani continued to threaten the plains year and after, particularly Brahmani caused serious flood havoc and brought untold miseries to the inhabitants of undivided Cuttack and Dhenkanal districts.

After the construction of the Hirakud Dam, whereby the floods in the Mahanadi were moderated to a great extent, the attention of the Government was diverted to the problem of floods in the Brahmani system. Therefore in 1959 it was proposed by the Government of Odisha to construct a dam at Rengali to impound 3.25 lakh Ham (2.644 m. acre feet) of water for providing irrigation to 16,200 hectares and to generate 38 MW of power. This proposal was dropped since the benefits did not commensurate with cost. However, in May 1963 Dr. A.N. Khosla, the then Governor of Odisha, prepared an integrated development plan for the river basins of Odisha (Orissa's Decade of Destiny, 1963-73) in which he proposed to build a dam across the Brahmani at Barkot intercepting a catchment of 8843 sq. miles (22,900 sq. kms).

"The Barkot project, besides power generation and irrigation, would also provide navigable canals and reservoirs connecting the Bonai iron ore deposits and the Rourkela Steel Plant with the Paradip Port and the future Chilka lake port near Santrapur." Further, along with proposal for setting

up a steel plant at Bonaigarh or at Barkot, a new proposal came up for construction of two dams one at Barkot and the other at Lodani for providing water supply to the steel township and for providing irrigation to 54,400 hectares during Rabi season. Again this did not materialise.

Public interest in the problem of Brahmani floods dates back at least to 1927, when the heavy floods in the Brahmani delta prompted the Government to set up the 'Flood Inquiry Committee' to probe into the cause of Brahmani floods and suggest remedial measures. As per the recommendation of the committee, the weirs were demolished in 1930 making the High Level Canal Range II defunct. But nothing approaching a multi-purpose development of the Brahmani had emerged from the Enquiry Committee. Since then, no attempt was made to harness the river till 1956 when a dam across the river Sankha at Mandira was constructed by Hindustan Steel Limited (HSL) for ensuring regular supply of water to the steel plant at Rourkela.

"Thus, systematic efforts began in the early seventies to control the floods in the Brahmani and the Baitarani. Detailed investigations were undertaken by the Government of Odisha and all possible alternatives were examined. Finally, the Rengali Multi-purpose Project, across the Brahmani with the twin objectives of flood control and power generation was approved by the CW & PC as well as the Planning Commission, Government of India and hence, was included in the Fourth Five-years Plan's allocation of funds. The construction of the first phase of the project commenced with the laying up of the foundation-stone by the Prime Minister of India in December 1973 at village Rengali in district Dhenkanal." (Source: Social Benefit-cost Analysis of Rengali Multi-purpose Project- Ph. D. Thesis by Binayak Rath, March 1980, Pg. 17). The location map of Rengali Multipurpose Project showing the dam, barrage and command area is shown in Drg. No.1.7. which is attached at the end of the chapter.

1.2 Flood - A Permanent disease of the State :

Orissa was formed as a separate province in 1936 with only six districts. Those were Cuttack, Puri, Balasore, Ganjam, Koraput and Sambalpur. After the merger of princely states in 1948 with Orissa, either they were merged with neighbouring districts or formed into separate districts like Keonjhar and Mayurbhanj. Thus number of districts rose from 6 to 13. Again for better and effective administration, few new districts were created in 1990 and within five years, due to public pressure, the no. of districts swelled from 13 to 30. Now the name of the State has been changed from Orissa to Odisha and its language from Oriya to Odia. The administrative map of Odisha showing 13 and 30 districts are enclosed vide Drg. No.1.1.

In the 19th century Odisha experienced a series of floods almost every year. This deteriorated the economic condition of the agriculturists and broke down the morale of the people. In the words of William Hunter, 'Flood is the scourge of Orissa Province'. A statement showing the history of floods from 1960 to 2008 is furnished vide Table No.1.1.

"Almost every year, vast stretches of land in the delta and fertile areas of undivided Cuttack, Puri, Balasore and Ganjam are severely affected either by flood, drought or cyclone. Sometimes two or more such natural calamities have occurred in one year. The nature has become the potent enemy in the economic development of the state." The flood data from 1868 to 1987 (i.e. 120 years) have been studied which reveals that "(i) the Brahmani has experienced the maximum number of floods followed by the Mahanadi and Baitarani and (ii) the common years of high floods in the Mahanadi, the Brahmani and the Baitarani were disastrous to the respective deltas as they affected the entire coastal plain. Such floods occurred only four times during the past century-in 1896, 1960, 1961 and 1982-when the whole coastal plain became a vast sheet of flood water and caused immense damage".

"The combined waters of the Mahanadi and the Brahmani are discharged into the sea through a common mouth, and it is almost the same in the case of Brahmani and Baitarani. This leads to floods lingering and the damage to life and property becoming extensive. Common high floods in the Mahanadi-Brahmani delta were experienced in six years - 1896, 1920, 1926, 1934, 1940 and 1955. The years

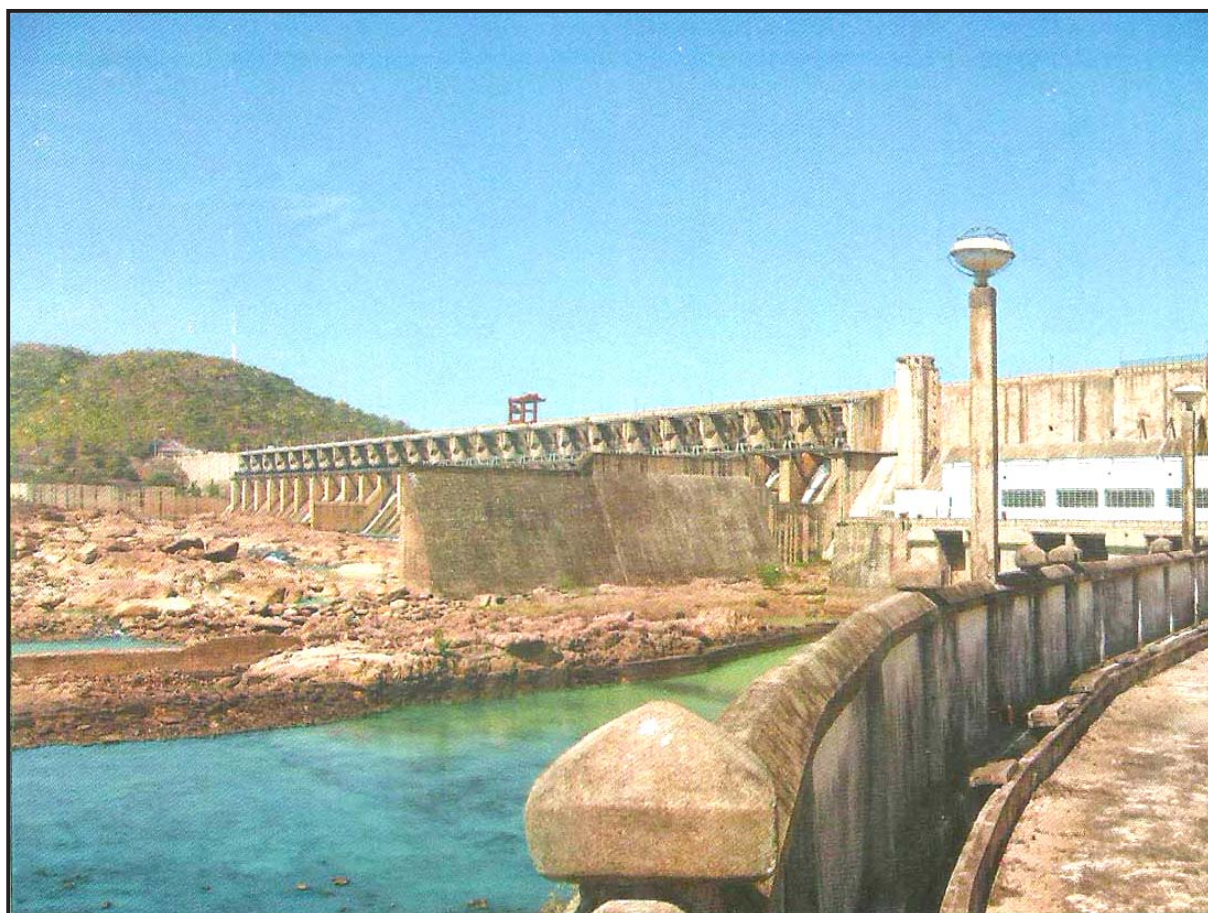


Drg. No-1.1

of common high floods in the Brahmani and the Baitarani were 1881, 1894, 1907, 1927, 1941 and 1943” (Source: Geography of Orissa by B.N. Sinha - National Book Trust of India, 3rd edition 1999, Pg. 148)

“Floods and droughts have been very frequent in the Brahmani basin. The years 1934, 1935, 1938, 1958, 1962 and 1979 recorded severe droughts and during the period for which actual observations were available, the year 1979 has experienced the worst drought.

Floods have occurred in 1929, 1936, 1945, 1959, 1960, 1963, 1964, 1970, 1971, 1973, 1975 and 1983. The flood of 1971, was of an exceptionally long duration while the peak flood of 1975, crossed all the earlier recorded flood peaks (Maske J and Day J.S. 1977). Similarly the floods in 1973, came at the fag end of October, due to a severe cyclonic storm and was a rare event. Even during drought years, there had been individual peaks, which caused large devastation to whatever-crops that survived the drought”.



View of Rengali Powerhouse and Power channel

Table 1.1 History of Past Floods in Odisha Rivers (1960-2008)

Sl.No.	Year	Month of Occurrence	Rivers	Affected District / Area	Loss/Damage Reported		
					Human	Live Stock	Public Utility
1.	2	3	4	5	6	7	8
1.	1960	August	Mahanadi, Brahmani Baitarani, Burhabalanga & Subamarekha	Cuttack, Puri, Dhenkanal, Balasore, Mayurbhanj & Keonjhar - 6 districts.	Not Available	Not Available	18.65 lakh Ac. of cropped area damaged and loss of Rs.11.70 crores.
2.	1961	July - September	Mahanadi, Brahmani, Baitarani, Burhabalanga & Surbanarekha	Cuttack, Puri, Dhenkanal, Balasore, Mayurbhanj & Keonjhar - 6 districts.	Not Available	Not Available	1.429 lakhs Ac. of cropped area damaged with a loss of Rs.2.54 crores.
3.	1964	July-August	Mahanadi, Brahmani, Baitarani & Rushikulya	Cuttack, Puri, Bolangir, Dhenkanal Balasore, Sambalpur, Ganjam Phulbani & Keonjhar - 9 districts.	Not Available	Not Available	4.08 lakh Ac. of cropped area damaged.
4.	1971	July-October	Mahanadi, Brahmani Baitarani & Subamarekha	Cuttack, Balasore, Puri, Mayurbhanj, Bolangir, Sundergarh & Keonjhar - 7 districts.	26	265	11.719 lakh Ac. of cropped area and 95043 no. of houses damaged. Total loss of Rs.31.71 crores.
5.	1974	August	Mahanadi, Brahmani, Baitarani, Burhabalanga & Subamarekha	Cuttack, Balasore, Puri, Dhenkanal & Keonjhar - 5 districts.	Not Available	Not Available	5.40 lakh ha. of cropped area damaged.
6.	1980	September	Mahanadi, Brahmani Baitarani & Vamsadhara	Balasore, Bolangir, Cuttack Dhenkanal, Ganjam, Kalahandi, Koraput, Phulbani, Puri, Sambalpur - 10 districts.	82	16669	3.19 lakh ha. of cropped area and Rs.65.00 crores of PU damaged.
7.	1982	August-Sept.	Mahanadi & Rushikulya	Cuttack, Puri, Bolangir, Phulbani Ganjam, Sambalpur, Dhenkanal & Kalahandi - 8 districts.	126	26359	12.00 lakh ha. of cropped area and Rs.616.00 crore of PU damaged.
8.	1984	June - September	Subamarekha, Brahmani, Baitarani, Mahanadi, Vamsadhara, Indravati	Cuttack, Balasore, Puri, Phulbani, Koraput, Ganjam, Dhenkanal, Keonjhar - 8 districts.	27	---	3.92 lakh ha. of cropped area damaged.
9.	1985	August - Sept.	Mahanadi, Brahmani, Rushikulya, Baitarani, Budhabalanga, Subamarekha	Balasore, Bolangir, Cuttack Ganjam, Puri, Phulbani, Keonjhar, Kalahandi, Sambalpur - 9 districts.	22	5281	3.10 lakh ha. of cropped area damaged.

Sl.No.	Year	Month of Occurrence	Rivers	Affected District / Area	Loss/Damage Reported		
					Human	Live Stock	Public Utility
1	2	3	4	5	6	7	8
10.	1986	August - Sept.	Mahanadi, Subarnarekha, Indravati	Balasore, Bolangir, Cuttack Dhenkanal, Koraput, Mayurbhanj Puri, Phulbani, Sambalpur - 9 districts.	24	337	1.08 lakh ha. of cropped area and Rs.55.31 crore of PU damaged.
11.	1991	July - August	Mahanadi, Brahmani, Baitarani, Subarnarekha, Vamsadhara	Cuttack, Puri, Balasore, Ganjam Phulbani, Dhenkanal, Sambalpur Kalahandi, Koraput, Keonjhar - 10 districts.	52	1145	6.62 lakh ha. of cropped area damaged.
12.	1992	June - August	Mahanadi, Subarnarekha, Vamsadhara	Cuttack, Puri, Bolangir, Balasore, Ganjam, Koraput, Puhlbani, Sambalpur, Kalahandi, Dhenkanal, Sundergarh - 11 districts.	43	1397	4.17 lakh ha. of cropped area and Rs.184.48 crore of PU damaged.
13.	1994	July - Sept.	Mahanadi, Brahmani, Subarnarekha, Vamsadhara	Angul, Balasore, Bhadrak, Boudh, Cuttack, Jagatsinghpur, Jajpur, Jharsuguda, Khurda, Kendrapara, Kalahandi, Koraput, Malkanagiri, Nayagarh, Nowrangpur, Puri, Sambalpur, Sundergarh, Sonepur - 20 districts.	50	----	10.17 lakh ha. of cropped area damaged.
14.	1995	May - November	Mahanadi, Subarnarekha, Vamsadhara, Rushikulya	Angul, Balasore, Bhadrak, Boudh, Cuttack, Dhenkanal, Ganjam, Gajapati, Jagatsinghpur, Jajpur, Khurda, Koraput, Kalahandi, Kendrapara, Keonjhar, Kandhamal, Malkangiri, Nawrangpur, Nayagarh, Puri, Rayagada, Sambalpur, Sonepur - 23 districts.	76	372	16.09 lakh ha. of cropped area and Rs.112.42 crore of PU damaged.

Sl.No.	Year	Month of Occurrence	Rivers	Affected District / Area	Loss/Damage Reported		
					Human	Live Stock	Public Utility
1	2	3	4	5	6	7	8
15	1997	June - August	Mahanadi	Balasore, Bhadrak, Cuttack Dhenkanal, Jagatsinghpur, Jajpur Khurda, Kalahandi, Kendrapara, Kandhamal, Keonjhar, Mayurbhanj, Nuapara, Nawarangpur, Nayagarh, Puri, Sundergarh, Sambalpur - 18 districts.	29	52	5.27 lakh ha. of cropped area damaged.
16	1999	July - August	Mahanadi, Brahmani, Baitarani, Subarnarekha, Rushikulya	Cuttack, Jagatsinghpur, Kendrapara, Jajpur, Bhadrak, Balasore, Mayurbhanj - 7 districts.	10		1.49 lakh ha. of cropped area and Rs.54.00 crores of PU damaged.
17	2001	July - August	Mahanadi, Brahmani, Baitarani, Subarnarekha, Burhabalanga, Vamsadhara, Rushikulya, Indravati	Angul, Balasore, Bhadrak, Boudh, Bolangir, Baragarh, Cuttack Dhenkanal, Deogarh, Jagatsinghpur, Jajpur, Jharsuguda, Khurda, Koraput, Kalahandi, Kendrapara, Nuapara, Nawarangpur, Nayagarh, Puri, Rayagada, Sundergarh, Sambalpur, Sonepur - 24 districts.	102	18149	7.99 lakh ha. of cropped area and Rs.883.42 crores of Public Utility damaged.
18	2003	July - October	Baitarani, Mahanadi, Rushikulya, Vamsadhara, Burhabalanga, Indravati	Angul, Balasore, Bhadrak, Boudh Bolangir, Bargarh, Cuttack Deogarh, Ganjam, Gajapati Jagatsinghpur, Jajpur, Jharsuguda, Khurda, Koraput, Kalahandi, Keonjhar, Kendrapara, Malkangiri, Nuapara, Nawrangpur, Nayagarh, Puri, Rayagada, Sambalpur, Sonepur - 26 districts.	92	2956	5.03 lakh ha. of cropped area damaged. More than Rs.1000.00 crores of PU damaged.

Sl.No.	Year	Month of Occurrence	Rivers	Affected District / Area	Loss/Damage Reported		
					Human	Live Stock	Public Utility
1	2	3	4	5	6	7	8
19	2006	July -August	Mahanadi, Brahmani, Baitarani, Subarnarekha, Burhabalanga, Vamsadhara, Rushikulya	Angul, Balasore, Bargarh, Bhadrak, Bolangir, Boudh, Cuttack, Dhenkanal, Gajapati, Ganjam, Jagatsinghpur, Jajpur, Kalahandi, Kandhamal, Kendrapara, Keonjhar, Khurda, Koraput, Malkangiri, Mayurbhanj, Nawarangpur, Nayagarh, Nuapara, Puri, Rayagada, Sambalpur, Sonapur - 27 districts.	90	1656	3.104 lakh ha. crop. damaged. 0.27 lakh ha. sandcast, houses damaged 1,20,446 nos. Loss of P.U - Rs.2043.00 crores
20	2008	Jun - Sep.	Subarnarekha, Burhabalang, Baitarani, Mahanadi, Rushikulya, Vamsadhara	Angul, Balasore, Bhadrak, Boudh, Bolangir, Bargarh, Cuttack Gajapati, Jagatsinghpur, Jajpur, Kendrapara, Khurda, Kalahandi Keonjhar, Mayurbhanj, Nuapara, Nayagarh, Puri, Rayagada, Sambalpur, Sonapur - 21 districts.	110	50163	258155 houses damaged, 4.45 lakh ha. cropped area, 0.14 th ha. sandcast, 651 breaches in rivers, 1276 breaches in canals.

Remark : From the year 1960 to 1992 the districts mentioned in the cloumn No.5 relates to undivided 13 districts of the States of Odisha.

(Source : Chief Engineer and Basin Manager, Lower Mahanadi Basin, Bhubaneswar)

1.2.1 Floods and Droughts as twin spectres:

“The undivided Dhenkanal district is subject to vicissitudes of flood and drought. The riparian tracts on both sides of the rivers Brahmani and Mahanadi are liable to flood. Ordinarily, the floods do not cause much harm. In years of exceptional rains, they are destructive to crops. The loss caused by floods is recouped by a bumper winter crop if it is not followed by an unusual drought.

Drought is a more serious calamity in the district on account of the undulating nature of the country and high porosity of surface soil. In the years of severe drought most part of the district, excepting the low lands and the irrigated areas, is affected. During the later part of the 19th century and the early 20th century, famines and scarcity occurred in Angul and the neighbouring areas. Accounts of few such calamities are described below:

1.2.1.1 Famine/Drought:

(i) Famine of 1889

“In the Angul Subdivision there had not been a good harvest of winter rice during the previous four years, while that of 1887-88 was on the average not more than 6 annas and that of 1888-89 not more than 8 annas of a normal crop. Considerable distress was reported in the autumn of 1888, and some measures of relief were adopted, the most important of which was the relaxation of the forest rules, but a copious fall of rain in September so improved the condition of things that measures of relief were gradually discontinued, except that the forest rules were not reimposed. In spite of this rain, however, the rice crop was an indifferent one, and large portion of the higher land was left untilled, for there was great drought from October till the following May. The mahua, mango and palm crops failed both in Angul and the adjoining States, and early in the year the agriculturists found themselves unable to keep the field labourers in their services and discharged them. The latter were thus suddenly thrown out of employment, and were unable to find work elsewhere. In ordinary years they might have subsisted for sometime on edible roots, fruits, etc. of the jungles, but unfortunately in this year jungle produce also failed or became very scarce. The labourers, therefore, being suddenly deprived of all sources of subsistence could only be supported by special measures until a demand again arose for their services.”

“The majority of the cultivators were in far better condition owing to the stocks of grains they held in reserve, but some were reduced to abject want, having sold a considerable portion of their slender stock at high prices to find subsequently that they had to buy grain for their sustenance at a much higher price. In many cases, they parted company with their last piece of gold and silver, with their brass ornaments, and with the last utensils of their household, and a few actually sold their plough-bullocks. The distress during the month of April and May and part of June was naturally at its height, there being no work available in the fields, while a severe epidemic of cholera broke out. In the later part of June, however rain fell, and there was fresh vegetation and at about the same time organised measures of relief were set on foot. The hopes of the people revived; the landless classes obtained agricultural loans, the able-bodied labourers found work, the infirm or helpless of both sexes received gratuitous relief and jungle produce became again procurable. In this manner the people continued to live till the maize and millet crops, which happily yielded a bumper out-turn, were gathered. They were followed by the early rice crop which was also an excellent one. By this time the labourers were getting their usual work, the price of foodgrains had begun to fall and relief operations were gradually reduced, until they were closed entirely in November when the early winter rice crop was harvested.”

“Briefly, this, the greatest famine within the memory of the present inhabitants since the great Orissa famine of 1866, was due partly to the short harvests of 1887 and 1888, partly to the failure of the mango and mahua crops in 1889 and partly to the effects of a long drought which prevailed from October, 1888 to the end of May 1889, on account of which all grain was tightly hoarded for some months, and the labourers were deprived of employment. The total cost of relief measures in the Angul subdivision amounted to Rs.36,430.00, including agricultural loans to the extent of Rs.12,590.00.” (Accounts of famines in 1889 and 1897 taken from L.S.S.O’ Malley, Angul District Gazetteer, 1908, pp. 99-102)

(ii) Scarcity of 1897

“There was some distress in 1897 due to the partial failure of crops in the Angul subdivision. In 1896, the rainfall was favourable until the middle of September, but after that it ceased till November. The injury done to the winter rice crop by this sudden cessation of rain at the time when it was most needed was aggravated by the visitation of an insect pest locally known mahwa (*Leptocorisa acuta*). The outturn of this crop was thus not more than 8 to 12 annas on the average. The distress caused by the partial failure of the rice crop was, however, not great and it was found sufficient to open a few relief works and to advance Rs.20,000.00 in loans.”

In the year 1906 there was scarcity owing to short crop. Relief measures in the shape of gratuitous relief grant of loan were undertaken to relieve the distress.

(iii) Scarcity of 1908-09

The poor out-turn of crop due to short and uneven rainfall in the previous year coupled with the short rainfall in 1908 resulted in the scarcity of 1908-09. Relief operations were undertaken by Government to alleviate the distress of the people. Local development works were undertaken to provide labour to the able-bodied persons. Charitable doles were given to old and decrepit by Dharma Panchayats organised for the purpose. Taccavi loans were also advanced to the tenants and land revenue was remitted or suspended.

(iv) Famine of 1918-19

In 1918-19, famine occurred due to total failure of rains and as there was also poor harvest in the previous year the condition of people became aggravated. The prices of foodgrains began to rise rapidly owing to the apprehension caused by scanty rainfall, moreover all means of supply from outside were almost suspended, while foodgrains were exported to Cuttack. The whole of Angul subdivision was affected uniformly and hardly 20 per cent of the population escaped from the rigour of famine. Various relief measures were undertaken to alleviate the distress of the people.

The district experienced natural calamities in the shape of drought and flood after 1948. An account of them is furnished below:

(v) Droughts of 1954 and 1955

In 1954 the total rainfall in the district was 44.21 inches (112.29 cm) of which the period from May to September had an average monthly rainfall of less than 8 inches (20.32 cm). Altogether 355 villages with a population of 215,071 were affected. The out-turn of crops that year was below 40 per cent of the normal yield.

In 1955 the main cause of drought was scarcity of rainfall in the early part of the agricultural season (i.e., June and July) throughout the district. The rain fall in July was only 6.79 inches (17.25 cm) and in August 4 inches on an average which was inadequate for paddy crops.

As a result about 650 square miles (1683.5 sq.km) of land and 244,716 persons were affected. The yield of crop that year was also below 40 percent like the previous year. The Dhenkanal subdivision was the worst-affected area where only about 5 percent of the normal yield was reported”. (Source: Orissa District Gazetteers, Dhenkanal, Government of Orissa, 1972 Pg.173-176)

“Angul which was British administered tract, suffered from famine in 1889 and scarcities in 1897 and 1900. The effects of the failure of the monsoon were less disastrous for the people were not dependent on rice and cereal, the products of the forests went far to help them in time of drought; but when, as in 1889, there was not only failure of the rains but also of the mahua and mango crops, distress was most severe”.

Brief account of sufferings of people due to famine of 1889, 1897 and 1918-19 etc. are described as under:

Sufferings of people during 1889 due to famine like condition has been narrated earlier under Sec.1.2.1.1 (i)- vide extract from the report of L.S.S.O' Malley, 1908 pg.99-102, Angul District Gazetteer.

“There was some distress in 1897 due to the partial failure of crops for want of rains and visitation of an insect pest locally known as mahua (*Leptocorisa acuta*). After this the people had a series of bad years, owing to the short crops, which exhausted their resources and culminated in generally scarcity in 1900-1901. After three years all signs of distress disappeared and there was a marked improvement in the condition of the people.

During the period 1891-95 in Angul sub-division, the average price of common rice, wheat, and gram as available per rupee was 18.75 kg., 7.80 kg, and 14.00 kg. respectively. In the next 5 years (1896-1900) there was rise in the prices of common rice and wheat. Common rice, wheat and gram was available at 14.70 kg., 7.34 kg., and 14.34 kg. per rupee respectively. However, in the next 5 years (1901-1905) rice and wheat became a little cheaper, while gram was dearer. Common rice of 15.70 kg., wheat of 7.50 kg., and gram of 9.33 kg. was available per rupee in average during this period.

The failure of the monsoon in 1918-19 and the consequent bad harvests of that year made their effect felt in the first half of 1919-20. Dhenkanal, Hindol, and Talcher ex-States were famine stricken. From April the end of October relief in the shape of test works and gratuitous relief were given. Rice was imported in large quantities from Cuttack, and Sonepur ex-State through river and was supplied at the rate of 9 kg. per rupee. In the next year floods damaged the lands of the villages in Dhenkanal Ex-state but at the same time covered the fields with a deposit of silt which did much to set the people on their feet again. After 1921 price began to decline and in the year 1922-23, the maximum and minimum price of rice per rupee was 20.50 kg. and 10.25 kg. in Athamallik ex-State; 13.00 kg. and 10.70 kg. in Dhenkanal ex-State; 18.65 kg. and 15.00 kg. in Hindol ex-state, 18.65 kg and 12.40 kg in Pallahara ex-state and 18.65 kg and 15.00 kg in Talcher ex-state. It has been noticed that during the period 1923-24 to 1929-30, rice was available in average in the maximum 16.77 kg., and in the minimum 9.33 kg. per rupee in the district. The price level again shot up with the declaration of second World War. Price ruled high since 1941, owing to the great outside demand of rice. Rice was available at 11 kg., 12 kg., and 12 kg. at harvest time in Talcher, Pal Lahara, and Hindol ex-States respectively. In 1942, rice became dearer and it was available at 5.60 kg. to 9 kg. per rupee in the district. Blackgram and green-gram was available at 8.40 kg. and 5.60 kg., per rupee respectively. From 1943-44 to 1946-47, prices of all foodgrains rose and rice, blackgram, and green gram were available in average at 5.60 kg., 6.00 kg., and 2.80 kg. , per rupee respectively. In Hindol ex-State, the price of rice was controlled and fixed at 4.70 kg., per rupee.

It was thought at first that this sharp rise might be only a temporary phase, but the prices started stabilizing at the high level without any prospect of recession. Towards 1953, the retail prices of rice, wheat, and gram were 2.30 kg., 1.9 kg., and 2.0 kg., per rupee respectively. This price level was, more or less, maintained from 1954 to 1962. With the launching of Third Five-Year Plan, prices began to rise rapidly and in 1963 the retail prices of rice, wheat, green-gram and black-gram as available was 1.320 kg., 2.00 kg., 1.22 kg., and 1.17 kg., per rupee respectively. Towards 1967 prices of all commodities still grew higher and rice, wheat, green-gram and black-gram were sold at 0.925 gram, 1.74 kg., 0.67 gram and 0.63 grams in retail per rupee respectively.” (Source: Orissa District Gazetteers, Dhenkanal, 1972 Pg. 275-276)

1.2.1.2 Flood:

Brahmani flood plain consists of parts of the valley starting from Talcher subdivision of Dhenkanal district upto the head of the delta Jenapur and the alluvial plains as well as the littoral tracts between Jenapur and the Bay of Bengal. It comprises of nearly 1,524 number of villages of 24 blocks and 3 NACs/Municipalities of Dhenkanal and Cuttack district with high population density. The cultivated areas subjected to floods from Brahmani system are around 3.35 lakh acres (1.36 lakh ha)

It would be incorrect to regard the Brahmani flood-plain as a compact physical, economic, sociological or cultural unit, for there are differences between the lower valley, the mountainous tract and the plains. On the basis of physical structures the Brahmani flood-plain has been broadly divided into three parts: (i) the plains between the mountainous tracts upto Jenapur (the middle flood-plain), (ii) the alluvial plains between Jenapur and Pattamundai (the lower flood-plain), and (iii) the rest belongs to the littoral tract (the lowermost flood-plain). While the middle flood-plain is endowed with varieties of abundant natural resources like forests and minerals its greatest problem is poverty and backwardness. The lower flood-plain (the delta) comprises of a fertile tract of land. The inhabitants are educated and cultured. Although they enjoy more of developmental facilities in comparison to their counterparts in the valley, they bear a greater burden of flood hazards in the monsoon due to the net-work of branches of the river.

(i) Flood of 1868

“One of the heaviest floods of the river Brahmani occurred in 1868 when flood water swept away a number of villages including Kirtanpur, causing greatest loss of life and property. The Brahmani, after continuous heavy rainfall, overflowed its banks on the 30th July, 1868. The flood did not subside for four days as the rain continued pouring.

(ii) Flood of 1955-56

In 1954-55, the district suffered from drought which continued till the last week of August. Then there was an unusually heavy and incessant downpour which continued for a week ending on the 4th September, 1955. This rain caused heavy flood in many parts of Orissa which was unprecedented. The district of Dhenkanal also suffered from ravages caused by flood of Mahanadi and Brahmani.

There were as many as 114 breaches during 1955, when 40,696 persons of 356 villages were affected. 26,779 acres of cropped area was damaged and not less than 1,090 houses were completely destroyed by the flood. There was heavy casualty of the live-stock and as many as 960 of them were reported to be lost, while four persons were washed away by flood water.

During the flood of 1956, there were in all 10 breaches in the district. 43 villages having a population of 11,417 were affected and 750 acres (303.75 ha) of cultivated land were damaged.” (Source: Orissa District Gazetteers - Dhenkanal 1972, Pg. 173-177)

“The combined flood water of Brahmani and the Baitarani played havoc in Dharmasala, Badachana, Jenapur and Jajpur police station areas. The Brahmani breached its embankment at several points. It flooded the entire region between the Brahmani and the Baitarani. The Baitarani and Matei area was affected by the flood waters of the Salandi and the above mentioned two rivers” (Source: Geography of Orissa by B.N. Sinha Pg.152).

“In 1866, the Brahmani broke its embankments. The Jagannath Road in the Cuttack district was under water and in some places even the telegraph posts were submerged. The people of Jajpur and Kendrapara subdivisions (presently declared as districts) were worst sufferers”. (vide Irrigation, Inland navigation & Flood problems in North Orissa by P. Mukherjee, Pg. 46).

In July 1881, the Brahmani and the Baitarani were in spate. There was the highest reading of flood yet recorded in Baitarani. “The whole of Sarad crop from the territory between the Brahmani and the Baitarani was swept away by the flood in 1881, which, with some intervals continued until the first week of August and the rivers were unusually high.” (Source: Bengal Administration Report 1881-1882).

“In August, 1885, a high flood visited Brahmani when the reading below Jenapur was 64.80 ft (19.75m). This was 3.20 ft. (0.98 m) below the height of flood in 1881 and 5.80 ft. (1.77 m) below the level attained by the flood in 1868. The flood of 1886 was the lowest which has occurred during the last 16 years.

“The average flood discharge of Brahmani has been calculated to be about 6 lakh cusec (16990 cumec). In August 1960, the Brahmani carried about 7,60,000 cusec (21521 cumec) of which the Kharsua took about 2.4 lakh cusec (6796 cumec) vide Irrigation, Inland Navigation and Flood problems in North Orissa during British Rule by P. Mukherjee Pg.51”. Highest flood level of river Brahmani recorded at Jenapur below the weir from 1885 to 1890 is furnished vide Table-1.2(a) and flood levels of its tributary the Patia/Kharasuan (Kharasrota) recorded in 19th century at Jokodia are furnished vide Table-1.2(b).

Table- 1.2 (a) Highest flood levels recorded for the Brahmani at Jenapur below the weir (in ft.)

Year	Jan	Feb	Mar	Apr	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
1985	-	-	-	50.60	50.20	60.40	62.90	64.80	64.10	56.80	58.20	58.65
1886	58.35	58.45	-	49.60	50.30	58.20	63.00	59.80	61.50	59.40	54.60	53.50
1888	-	-	-	50.00	53.70	51.35	60.60	66.20	58.90	53.10	51.85	50.80
1889	51.10	51.00	51.00	51.00	-	60.20	61.20	67.50	59.50	56.30	58.40	53.80
1890	52.10	51.40	50.95	50.60	-	58.20	64.50	62.10	59.00	56.40	55.30	52.70

Table- 1.2(b) Highest flood levels recorded for the Patia at Jokodia below the weir (in ft.)

Year	Jan	Feb	Mar	Apr	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
1985	-	-	-	41.25	42.80	59.45	51.60	63.65	63.65	51.80	58.75	59.95
1886	58.85	58.75	58.95	55.75	51.90	57.30	62.00	60.00	60.90	59.10	59.10	42.70
1888	-	-	-	40.55	42.80	55.55	60.10	65.60	58.60	49.70	46.50	45.50
1889	42.15	41.45	41.45	41.30	47.30	59.25	59.70	65.70	58.10	52.10	49.45	42.20
1890	41.05	41.00	41.30	41.31	-	57.10	63.20	61.20	58.60	55.55	43.50	42.55

(Source: The Natural Calamities in Orissa in the 19th Century by B.B. Bhatta, 1997 Pg.27-40).
Zero level at Jenapur 0.89 ft. and Danger level are 67.0' (20.30 m).

“The Orissa Flood Committee (1928) laid great emphasis on the high average level of the Brahmini at Jenapur, and concluded firstly, that the Brahmini has deteriorated, secondly, that the deterioration took place suddenly and not as the result of normal deltaic action and thirdly, that it took place in 1920 (Pg. 29)”. (Source: Mahalanabis Report, Government of Orissa, I&P Department, Pg.298)

(iii) Flood of 1955:

“During the period from the 28th August to 4th September 1955, the Brahmani with its catchment basin of 14,000 sq. miles also had heavy rainfall. In the Kamakhyanagar sub-division of Dhenkanal district rainfall recorded within 24 hours of the 2nd and 3rd September 1955 was 11 inches and in the Talcher subdivision it was 16.65 inches during the week. Angul recorded rainfall of about 5 1/2 inches during 48 hours ending on the 3rd September 1955 and Talcher 5 1/2 inches during 24 hours upto the 4th September 1955. Uditnagar and Bonai stations within the catchment area of the Brahmani recorded 2.24 inches and 3.25 inches respectively in 24 hours ending on the 3rd September 1955. This widespread heavy rain in the Brahmani catchment area caused the worst flood ever recorded in the history of the river which started rising from the 3rd September 1955 and within 24 hours recorded the highest gauge-reading at Jenapur in Cuttack district. On the 3rd September its flood-level reached the danger point. At 12.00 hours on the same day the gauge at Talcher recorded 29 feet whereas in the early morning of the 4th September the flood water pushed the gauge to a maximum of 34 feet recording a rise of 5 feet within 12 hours against the highest of 35.00 feet recorded on the 6th August 1943. The gauge at Jenapur where the Brahmani crosses the South Eastern Railway line from Madras to Calcutta rose to 70.20 ft. at 02 hours on the 4th Sept. 1955. The Brahmani rose up by nearly 10 ft.” (Source: Floods in Orissa Rivers during 1955-56, Government of Orissa, Revenue-Special Relief Department)

Consolidated statement of the average flood losses and the relief and grants provided in the undivided Cuttack district from 1969 to 1977 is furnished in Table-1.3.

A correlation between discharge and damage of Jenapur consequently the cost of damage has been developed which is presented in Table -1.4. Similarly the losses to Central Government Organisations are given in Table-1.5.

Soon after the devastating floods in August, 1971, Government of Odisha prepared a report in October for construction of a storage reservoir across Brahmani at Rengali to impound 2.47 M Ac.ft. (0.285 M. ham.) for flood control in Brahmani basin and to provide irrigation in the undivided districts of Dhenkanal and Cuttack at an estimated cost of Rs.94.42 crores.

The said report was discussed in the Parliament on 11.4.1972, which is enclosed as Annexure-I.

“During high floods the united flood water of the rivers Brahmini (Brahmani), Kharsuan and Burah (Burha) cover the country to the extreme distress of the people. But Thomson was of opinion (1905) that “For this state of affairs there is no practical remedy. The rivers can not carry within their respective channels the volume of water brought down during floods. It has been decided to be impracticable for financial reasons, as well as inadvisable to fully embank them; and it is, therefore, necessary that a certain area of country shall be left open as an escape for the surplus water.”

“There are other escape channels from the Brahmini called ‘Ghais’ (such as Janardan, Si nilia, Tanti, Debil, Rahapur, Palsahi, etc.) Enormous damage is sometimes caused by the water flowing through the ‘Ghais’, as the land in the neighbourhood is usually unprotected. These ghais usually originate in breaches caused in the river embankments during high floods.

Mr. H.A. Gubbay, Superintending Engineer, in his printed report, dated the 18th December 1923 (Orissa Floods, 1920, Pg.8), stated in a table that the maximum discharge of the Brahmini in 1868 was 840,000 cusecs, but he did not give any authority for this estimate.”

Sri Mahalanabis had the opportunity of examining certain old hand written notes in this connection which are almost certainly by Mr. Rhind himself. “He found from these notes that the highest level of the Brahmini rose to 70.68 ft. in 1868. It was estimated that a volume of 335,342 cusecs (9490 cumec) were carried by the river channel, while the total spill over either banks was 507,167 cusecs (14353 cumec). This would appear to give a total discharge of 842,509 cusecs (23843 cumec). It is extremely likely that Mr. Gubbay’s statement is based on this estimate. But it is distinctly mentioned in these notes that the above estimate for the spill was an uncorrected one, as no allowances had been made for obstructed flow. After applying suitable corrections, Rhind came to the conclusion that the true spill was 307,948 cusecs (8715 cumec), so that the true discharge was 643,920 cusecs (18223 cumec). This is exactly the figure given by Shaw in 1929. The Orissa Flood Report of 1928 (pg.31) mentions the highest known flood in the Brahmini as 650,000 cusecs (18395 cumec) which is also evidently the same estimate in round numbers.”

“In 1881 with a high flood of 68.75 ft. at Jenapur, Rhind calculated that the maximum volume of the flood was 421,147 cusecs (11918 cumec) out of which the Brahmini took 307,349 cusecs (8698 cumec) and the Patia 113,789 cusecs (3220 cumec).

In 1883 Rhind (letter no.3226 of 28th September 1883 to Chief Engineer, Bengal) gave a table of discharge of the Brahmini for different heights of the river which was based on the theoretical fall of the river as a parabolic curve between Indulva (Endolva) and Jenapur. Shaw in his note of April 6, 1929, states, however, that this table is now totally unreliable as the section of the river at Indulva has changed considerably and much silting has taken place above the Brahmini anicut. Mr. Gubbay reconstructed Rhind’s Table 57 from Mr. J. Shaw’s calculations of the discharge of flood of 1926.

Mr. Gubbay (Report, dated the 18th December 1923, on Orissa floods of 1920, p.10) stated that from a comparison of cross sections both up-stream and down-stream of the Patia weir and the Brahmini weir taken in 1884 (before the weirs were built), in 1895 (five years after the construction of the weirs), and in 1923 he was of opinion that rivers were deteriorating to a very small and normal extent above the weirs, but that the deterioration below weirs were much more pronounced. He did not, however, give actual figures.”

In the Brahmini the tide reaches up to the 24th mile of the Pattamundai Canal near village Mahakalpara, 2 miles above village Indupur.

Thomson remarked in 1904:- "The Brahmini like other rivers of Orissa has a broad, shallow sandy bed with a fall of about 14 inches per mile in the plains. The river is almost dry in the hot weather, the minimum recorded discharge being 130 cusecs." (Source: Mahalanabis Report, Govt. of Orissa Pg.87-88)

Table- 1.3 Consolidated Statement of the average Flood losses & Relief and grants provided for Cuttack District (Between 1969 to 1977)

(Figure in lakh Rs.)

Sl. No.	Block	Losses, Relief & grant provided	Average Other Losses				
			Pvt. Houses	Live-stocks	Roads, Buildings and Embankments	Net crop loss	Total
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	Sukinda	2.044 (1.957)	0.24 (11.83)	0.007 (0.37)	0.32 (15.76)	1.463 (72.04)	2.031 (100)
2	Danagadi	4.134 (3.786)	0.382 (9.20)	0.006 (0.16)	0.624 (14.83)	3.189 (75.81)	4.025 (100)
3	Korei	5.74 (4.894)	0.986 (9.92)	0.012 (0.12)	0.519 (5.24)	8.426 (84.72)	9.944 (100)
4	Rasulpur	23.324 (22.088)	5.307 (23.14)	0.073 (0.31)	1.954 (8.52)	15.592 (68.03)	22.928 (100)
5	Dharmasala	17.87 (16.747)	2.512 (13.18)	0.017 (0.09)	1.695 (8.90)	14.835 (77.83)	19.059 (100)
6	Badachana	19.707 (18.151)	3.168 (13.60)	0.009 (0.03)	1.344 (5.80)	18.76 (80.57)	23.282 (100)
7	Jajpur	19.812 (18.478)	4.40 (17.4)	0.016 (0.07)	1.656 (6.51)	19.235 (76.02)	25.314 (100)
8	Bari	27.901 (27.560)	6.822 (21.83)	0.014 (0.05)	2.403 (7.69)	22.005 (70.43)	31.244 (100)
9	Binjharpur	14.967 (15.358)	0.935 (6.34)	0.006 (0.05)	0.967 (6.55)	12.850 (87.06)	14.76 (100)
10	Dasarathpur	3.862 (3.293)	0.337 (2.10)	-	1.947 (12.30)	13.579 (85.60)	15.866 (100)
11	Kendrapara	1.075 (1.103)	0.649 (8.82)	0.002 (0.02)	0.659 (8.94)	6.06 (82.22)	7.37 (100)
12	Derabisi	0.907 (0.849)	0.027 (0.43)	-	0.358 (5.73)	5.88 (93.84)	6.266 (100)
13	Pattamundai	36.48 (31.216)	9.258 (21.40)	0.046 (0.10)	3.187 (7.36)	30.758 (71.14)	43.261 (100)
14	Aul	33.612 (34.112)	6.049 (18.16)	0.015 (0.04)	2.216 (6.38)	25.125 (75.42)	30.303 (100)
15	Rajkanika	20.076 (20.364)	4.845 (20.18)	-	2.852 (11.87)	16.317 (67.95)	24.016 (100)
16	Rajnagar	6.147 (6.080)	0.318 (5.52)	0.001 (0.02)	2.169 (20.28)	4.275 (74.18)	5.764 (100)
	Total	237.664 (266.044)	46.24 (16.02)	0.228 (0.07)	23.786 (8.25)	218.355 (75.66)	288.618 (100)

- Note : i) The figures within brackets in column (3) are the figures at base year prices.
ii) The figures within brackets in column (4) to (9) are the percentages of the respective item to total flood losses of Brahmani system.

Source : Social Benefit - Cost Analysis of the Rengali Multi-Purpose Project by Binayak Rath (Ph. D. Thesis) Pg. 250-251.

**Table- 1.4 Cost of Damages due to past flood from correlation
between discharge and damage**

Sl. No.	Year	Maximum discharge in cusecs at Jenapur	Flood damages in lakh Rupees.	Expenditure in relief measures in lakh Rupees (50% of damage)	Total lakh Rupees.
1	1881	5,99,000	480.00	240.00	720.00
2	1882	5,21,000	387.00	193.50	580.50
3	1894	7,07,000	594.00	297.00	891.00
4	1896	7,47,000	633.00	316.50	949.50
5	1907	7,11,000	600.00	300.00	900.00
6	1916	7,28,000	615.00	307.50	922.50
7	1926	5,10,000	372.00	186.00	558.00
8	1929	6,80,000	570.00	285.00	855.00
9	1943	6,48,000	540.00	270.00	810.00
10	1959	3,65,000	127.00	63.50	190.50
11	1960	8,92,000	748.00	374.00	1122.00
12	1961	6,65,000	554.00	277.00	831.00
13	1963	3,77,000	154.00	77.00	231.00
14	1964	5,85,000	464.00	232.00	696.00
15	1965	4,35,000	260.00	130.00	390.00
16	1966	3,35,000	73.00	36.50	109.50
17	1967	4,00,000	198.00	99.00	297.00
18	1968	3,37,500	76.00	38.00	114.00
19	1969	5,05,000	361.00	180.50	541.50
20	1970	4,75,000	324.00	162.00	486.00
21	1971	8,45,000	715.00	357.50	1072.50
22	1955	-	566.50	283.25	849.75
					14117.25 Lakhs

Average Annual damage = $\frac{14117.25}{100} = \text{Rs. } 145.54 \text{ lakhs.}$

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(Source : Rengali Dam Project, Stage-I, Vol. I, Part B, General Report, July 1972)

Table-1.5 Losses to Central Government Organisations due to floods in Brahmani

(Figures in lakh Rs.)

Name of the Organization that sustained losses due to floods.	Losses for the period							The average annual losses
	1971	1972	1973	1974	1975	1976	1977	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Railways	-	4.99 (5.389)	-	-	41.508 (35.586)	-	N.A.	7.749 (6.829)
National Highways								
(i) N.H. NO-5	N.A.	N.A.	0.05 (0.05)	-	6.494 (5.567)	0.228 (0.181)	1.05 (0.771)	1.564 (1.314)
(ii) N.H. NO-23	N.A.	N.A.	-	-	0.51 (0.437)	0.515 (0.409)	1.282 (0.943)	0.461 (0.358)
Posts & Telegraphs								
(i) Post Offices Cuttack & Dhenkanal Division	-	-	-	-	0.067 (0.057)	-	-	0.009 (0.008)
(ii) Telegraphs								
(a) Cuttack Sub-Division	N.A.	N.A.	N.A.	0.174 (0.161)	0.369 (0.316)	0.216 (0.171)	NA	0.253 (0.216)
(b) Keonjhar Sub-Division	N.A.	N.A.	NA	0.202 (0.187)	0.435 (0.373)	0.251 (0.199)	NA	0.296 (0.253)

Note : The figures within brackets are the loss figures at the base year price level.

Average annual losses to Central Govt. Organizations : Rs.10.332 lakhs

Source :

- The data for losses to railways are collected from the SDE, SER, Khurda Road, Odisha.
- The loss figures for NH are collected from the Chief Engineer (NH), Odisha, Bhubaneswar.
- The loss figures for Post Offices are collected from the SPOs, Cuttack & Dhenkanal.
- The damages to telephone lines are collected from the SDO Telegraphs, Cuttack & Kenojhar.

1.2.2 Social Condition of People- General level of Wages:

“In the beginning of the present century practically the only skilled labourers were artisans, such as masons, blacksmiths and carpenters who were paid in between Re.0.12 to Re.0.50 per day. Unskilled labour did not cost more than Re.0.50 per day, while agricultural labour was paid generally in kind. The custom of paying the village artisans and others, such as washermen, barbers and sweepers in kind at harvest time was common.

During the period 1893-1902, a superior mason could get Re.0.37 per day in the ex-States of Dhenkanal, Pal Lahara, and Talcher and Re.0.50 in Athmallik, and Hindol. During this period, a superior carpenter got the same wage as that of a mason, except in Hindol, and Talcher where the rate was higher. Superior black-smiths got Re.0.31 in Athmallik, and Dhenkanal; Re.0.50 in Hindol, and Talcher, and Re.0.37 in Pal Lahara. Common carpenters, masons and blacksmiths got less wage. An ordinary labour got Re.0.15 per day in all the ex-States, except Dhenkanal where the wage was little less.

Table No-1.6 shows the wage rates of different labourers during the period 1893-1902 in the ex-States of the district:

Table No-1.6 Wage rate availing during 1893-1902.

(Rs. Per day)

Name of ex-States	Superior mason	Superior Carpenter	Superior Blacksmith	Common mason	Common Carpenter	Common Blacksmith	Ordinary Labourer
1	2	3	4	5	6	7	8
Athamallik	0.50	0.50	0.31	0.28	0.25	0.19	0.15
Hindol	0.37	0.37	0.31	0.31	0.12	0.12	0.12
Pal Lahara	0.37	0.37	0.37	0.25	0.25	0.25	0.15
Talcher	0.37	0.50	0.50	0.25	0.25	0.25	0.15
Dhenkanal	0.37	0.37	0.31	0.31	0.12	0.12	0.12

In Angul, in the beginning of the present century the only skilled labourers were masons, blacksmiths and carpenters, brought from Cuttack and other places. A common mason earned a daily wage of Re.0.31 to Re.0.44, while a blacksmith got Re.0.19 and a carpenter Re.0.19 to Re.0.44; superior masons, and carpenters were paid Rs.0.50 a day, while expert blacksmiths received a wage of Re.0.37 to Re.0.50 per day. Local labourers, if employed by contractor received Re.0.12 to Re.0.19 per day; and if employed in field work by cultivators, they were paid in food and grain. Village artisans, such as blacksmiths, who prepared and repaired plough-shares and other agricultural implements and other workers, such as washermen and sweepers were allotted service lands and also received in many places an allowance of rice and other grains at harvest time. This allowance was generally 9.330 kg. of paddy per plough in case of blacksmiths. Adult barbers, and washermen also got the same amount from each of their clients.

Field labourers during this period were divided into two classes, ‘Mulias’, and ‘Halias’. Mulias were day labourers paid almost invariably in kind and ‘Halias’ were farm servants employed permanently by well-to-do cultivators, on a monthly allowance of 55.986 kg. of rice, and at harvest time 6.60 quintals of rice, two pieces of cloth, and a rupee in cash. On the whole, Halias were better off than the day labourers who could get little employment from February to May. During these months they had to subsist on their own little crops, on wild roots and fruits, by cutting and selling bamboos, and fuel, and by making and selling mats, baskets etc.” (Source: Orissa District Gazetteer, Dhenkanal, 1972 Pg.276-277) .

1.3 River Brahmani, Its course and Catchment

Brahmani River System:

Brahmani is the 2nd largest river in the state of Odisha. River Sankh originating in Chhatisgarh and river Koel originating in Jharkhand merge at Vedvyas in Odisha forming the river Brahmani. The Sankh is the right tributary and Koel is from the left. Brahmani flows through the heart of Odisha in between the Baitarani basin on the left and Mahanadi basin on the right till it enters in the deltaic plain, finally outfalling into Bay of Bengal at Dhamara mouth.

1.3.1 River features in upper part upto confluence at Vedavyas:

River Brahmani is an interstate river. It traverses through the districts of Raigarh and Sarguja in Chhatisgarh, Ranchi and Singhbhum districts in Jharkhand and Sundargarh, Deogarh, Sambalpur, Angul, Dhenkanal, Keonjhar, Jajpur and Kendrapara districts of Odisha. The total drainage area of the river basin is 39116 sq.km. Of this, basin area inside Odisha is 22516.08 sq.km. which constitutes 57.56% of the total basin area. The basin area inside Jharkhand and Chattishgarh are 15700 sq.km. and 900 sq.km. respectively. The Brahmani basin is en-compassed within the geographical co-ordinate of 83°-52' to 87°-30' of East longitude and 20°-28' to 23°-35' of North latitude, covering 14.46% of the geographical area of the state.

The river flows for 345 miles (555.45 km) through the hill ranges up to head of delta i.e. Jenapur and then passes through the alluvial plains and littoral tracts of Jajpur and Kendrapara districts for 93 miles (149.73 km) until it joins the Bay of Bengal. The drainage area upto head of delta is 36444 sq.km.

(i) River Koel

The left bank tributary South Koel, originates near village Nagri in Ranchi district of Jharkhand state at an elevation of about 700 m. above M.S.L., at latitude 23° 20' N and longitude 85° 12' E. Initially the river South Koel runs in north west direction for a length of 21.25 km. It then takes a westward turn and runs 40 km and flows southerly for a length of 52.50 km. It changes its direction to south east for a further length of about 136.25 Km upto Manoharpur. In this reach just south of Gudri in Singhbhum district "The Karo" a major left bank tributary joins the South Koel at 221.25 km. Below the confluence the river is known as Koel. From Manoharpur it flows in the south west direction for a distance of about 53.75 km upto Vedavyas entering the territory of Odisha at RD 262 km.

The tributary Karo originates in Chhotnagpur Plateau of Jharkhand near Nagri in Ranchi district at an elevation of 700 m. above MSL, at Lat. 23° 17' N and Long 85° 08' E and flows in a southern direction for a total length of 125 Km. to join South Koel. The Karo drains an area of 2784 Sq.Km of Jharkhand.

The Koel at the confluence with Sankh at Vedvyas drains from a total catchment of 13378 Sq.Km, of which 1438 Sq.Km. lies in Odisha and 11940 Sq.Km. in Jharkhand.

ii) River Sankh

The river Sankh, the other main tributary of the river Brahmani originates at an elevation of 1000m near village Lupungpat in Ranchi district of Jharkhand at Latitude 23° 14' and Longitude 84°16'.

A river called Lawa originating at about the same elevation in Chhatisgarh joins the river Sankh in Jharkhand state and moves forwards along the border of Jharkhand and Chhatisgarh for a distance of about 15 Km before reentering into Jharkhand. The distance traversed in Jharkhand prior to reaching the border of Jharkhand and Chhatisgarh is about 67.5 km. The river further traverses for a distance of 77.5 km in Jharkhand on its onward journey into Odisha. In this reach the river Palamora joins the river Sankh on the left and the river Girma joins on the right.

The total length of the river Sankh is 205 km. It drains from an area of 7353 sq.km., of which 900 sq.km. lies in Chhatishgarh, 3760 sq.km. in Jharkhand and 2893 sq.km. in Odisha.

The river Sankh merges with river Koel near Vedavyas in Odisha Lat.22°48' and Long. 84°14' at an elevation of 200 m. above MSL. Below this confluence, the river is called Brahmani.

1.3.2 River features from Vedavyas to Jenapur i.e. upto Delta Head

The river Sankh on the right and Koel on the left joins at Vedavyas and after the confluence takes the name Brahmani and flows down.

The premier industrial steel township at Rourkela grew on the left bank of Brahmani at Vedavyas due to installation of steel plant by SAIL during late fifties. Besides lot of ancillary industries also grew round the nucleus of Rourkela and the river Brahmani became the water source of all industrial activities. At Vedavyas the S.E. Railway (now East Coast Railway) crosses the river Brahmani and this communication network coupled with iron ore reserve nearby became the prime mover to develop the industrial complex on the bank of Brahmani river. However below Barkot where NH-6 crosses the river, the Brahmani transverses in a narrow gorge through a mountain range.

At this location a major multipurpose dam i.e. the Rengali Dam was built by Government of Odisha and was completed in 1985. The Rengali Dam was built to provide flood moderation to 3500 sq km deltaic area with hydropower installation of 250 MW. A barrage at Samal about 40 km downstream of Rengali dam was built, primarily to pick up the release from Rengali Dam and to provide irrigation to 2,35,000 ha from two canal systems offtaking from either side of Samal barrage. This location developed into one of the biggest thermal power stations of India with 2000MW capacity plant by NTPC located about 5 km upstream of Samal barrage. The river after traversing 30KM downstream meets the Talcher town, a sub-divisional headquarters where one of the biggest coal reserves of the Mahanadi Coalfield Company is located. Due to availability of coal and water, the smelter plant of National Aluminium Company (NALCO) was setup at Angul i.e. about 20 kms from Talcher town. Thus Talcher-Angul complex one of the biggest industrial complex of Odisha has grown up in the bank of river Brahmani near Talcher. The river Brahmani which was having pristine water quality became polluted due to large scale industrial effluent joining the river system between Samal barrage and Talcher town and also further downstream. The river after traversing in a meandering course for some length enters the deltaic plain at Jenapur.

1.3.3 River features below Jenapur upto sea with intricacies of river network

The river Brahmani enters the deltaic head at Jenapur where the NH-5 and East Coast Railway line crosses the river. Below this point the river branches into two streams i.e. Kharsua branch being in the left and Brahmani in the right. Further a branch of Baitarani i.e. Burha joins the Kharsua branch upstream of Binjharpur. On the right side Brahmani bifurcates as Brahmani branch in the left and Kimiria on the right. The Kimiria meets river Genguti a branch of Birupa and flows down and two kms below meets the main Birupa river and little downstream joins the left arm Brahmani at Indupur. Below this, the river traverses with name Brahmani and joins with its branch Kharsua. Both combined continue with name of Brahmani and joins Baitarani below Chandbali and takes the name Dhamara before it outfalls into Bay of Bengal. However the Brahmani before it meets Baitarani, a branch offtakes as 'Maipura' and falls into Bay of Bengal independently.

The entire drainage basin of Brahmani is very complex. Three spill canals with escapes have been provided in the right bank of Kharsuan at Tantighai, Palasahi and Routra, Further below a spill channel Kani offtakes on right of Kharsua and joins the same branch below.

However to protect the densely populated Aul area, a 70km long ring bund was constructed protecting 25000ha of land and 1.5 lakh population. The entire basin between Kharsuan and Brahmani is flood prone but certain degree of flood moderation has been achieved after the construction of Rengali Dam. As per the flood protection master plan it has been considered that after routing of flood at Rengali, the outflow of 2.5 lakh cusec (7075 cumec) with intermediate discharge of 1.5 lakh cusec (4245 cumec) will combine to result a discharge of 4.00 lakh cusec (11320 cumec) at Jenapur for which embankments are to be designed. But this is not foolproof as Birupa a branch of Mahanadi and Burah a branch of Baitarani contributes significant discharges into the system during certain high flooding period which complicates the flooding situation in this deltaic area and to relieve stress on the embankments a number of spills and escapes have been provided. The list of important tributaries with its confluence details are furnished vide Table No-1.7.

There are forty five (45) major tributaries of river Brahmani, of these the important rivers Sankh, Chandrinala, Katangmundanallah, Rukuranadi, Badjore, Kaunsinallah, Kalanalla, Usthalinalla, Chudakhal nallah, Gohira river, Chilanti river, Tikira, Singadajore, Banguru river, Nandira, Nigra river, Bangusingha nallah, Barha, Daunri, Kumaria, Kelua river, Biripa, Hansua, Kharsuan and Patasala join the main stream on right bank; and Koel, Suidihi nallah, Champajhar, Kuradhi, Amrudi nallah, Korapaninadi, Mankara, Ambhari, Samakoi, Gambharia, Ghodadian, Riajore, Inderjeet nallah, Ramiala, Pandra, Kharsuan, Dudhi nallah, Baitarani join the main stream on left bank.

Table No-1.7 Details of Important Tributaries:

Sl. No.	Name of the Tributary	R.D in km	Location (Right or Left)
1	Kuradhi	37.00	Left
2	Rukura	54.00	Right
3	Gohira	129.50	Right
4	Mankara	147.00	Left
5	Samakoi	172.00	Left
6	Tikira	178.25	Right
7	Singarajore	180.00	Right
8	Nandira	214.50	Right
9	Nigra	225.00	Right
10	Ramiala	278.25	Left
11	Pandara	306.00	Left

1.3.3.1 River system below head of delta:

“The Brahmani enters the plains 10 miles (16.1 km) above Jenapur nearly opposite which a branch called Patia taken off to the north. The Kharsuan branch takes off near Manpur, but the mouth of Kharsua is closed by an embankment. From the original off-take at Manpur to the point of confluence with Patia, the Kharsua is a dead channel, but is however flooded by the backwaters from Patia. The stream then becomes known as Kharusua. Lower down, the Budha, a branch of the Baitarani joins the Kharsua near Kamalpur when the stream runs in one channel till it meets the Brahmani below Aul. Twelve miles (19.32 km) east of Jenapur the Brahmani gives off another branch to the south called Kimiria and the main stream assumes the name Sankra. The Kimira joining the Genguti and Birupa, the branches of the Mahanadi, meets the main stream near indupur. The combined stream flows past Pattamundai and Alva till it is joined by the Kharsua. The channel then forks to the right assuming the name Mayapura (Patasala) which falls into the Bay of Bengal below Rajnagar, while the left main stream is joined by the Baitarani near Chandabali and assumes the name Dhamra till it meets the sea.”

“At Jenapur is situated the outfall of the High Level Canal Range I. The river also sends off a branch, the Patia, at this place. Both the Brahmani and the Patia are dammed by weirs to feed the High Level Canal Range II (now defunct) and maintain a navigable channel between the two ranges of the canal. Mr. Thomson remarked in 1904 that as far as the Brahmani was concerned the channel was of little use because it was silted up and navigation was maintained with great difficulty.

The Brahmani weir is 4,000 feet (1.22 km) long with its crest at 58.00 feet (17.68 m), the Patia weir is 783 feet (238.66 m) long. The highest flood level recorded at Jenapur above Brahmani weir, since the construction of the weir was 70.6 on the 17th August, 1926. The level exceeded 70 on three other occasions in recent times (70.1 on the 23rd July, 1920, and 31st July, 1927, and 70.2 on the 31st July, 1929).

Twelve miles below Jenapur the Brahmani sends out a branch to the right, the Kimiria. The main river is called Sankra from here The Kimiria again joins the main stream near Indupur after joining with the Ganguti and the Birupa. The combined stream is joined by the Kharsua and then by the Baitarani, and falls into the Bay of Bengal at the Dhamra river.

The main branch of the Brahmani, namely the Patia, runs for about nine miles below the Patia weir under such name when it joins another branch, the Kharsua, with its off-take from the Brahmani at Manpur. The Kharsua between Manpur and its confluence with the Patia is a dead channel, its head having been permanently closed by the embankment no.17A (Thomson)” (Source: Mahalanabis Report, Government of Orissa, Pg. 87-88)

The schematic diagram attached vide Drg. No.1.2 explains the Brahmani system. The L.S. of river Brahmani from Rengali Dam to 50 km downstream and that from source to sea are furnished in Drg. No-1.3 & 1.4 respectively.

1.3.4 Catchment of river Brahmani:

“The Brahmani catchment lies between $83^{\circ} 55'$ E at $23^{\circ} 13'$ N and $86^{\circ} 04'$ E at $20^{\circ} 51'$ N and between $23^{\circ} 37'$ N at $84^{\circ} 41'$ E and $20^{\circ} 35'$ N at $85^{\circ} 03'$ E and consists of an irregular rectangular area of about 14000 sq. miles (36260 sq.km) with its longer axis lying roughly in a north-north-westerly to west-north-westerly direction. It covers portions of the administrative districts of Ranchi, Jashpur (Central Provinces Feudatory States), Orissa Feudatory States and Tributary Mahals, Singhbhum, Angul and Cuttack.

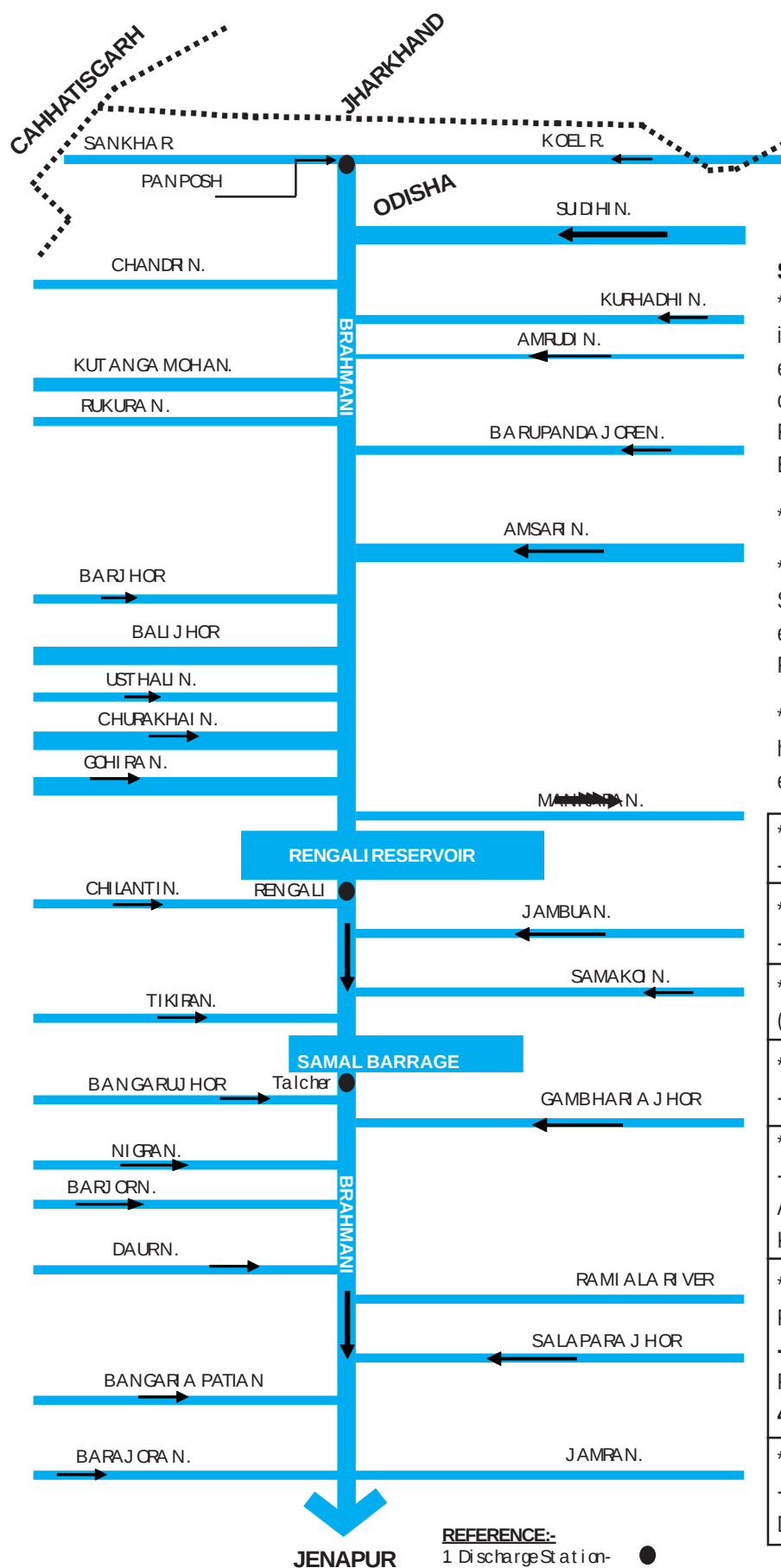
The Sankh and the South Koel have their origin in Ranchi plateau and lie wholly in hilly country. They unite at $22^{\circ} 15'$ N, and $84^{\circ} 48'$ E a little above Bonaigarh ($21^{\circ} 22'$ N, $84^{\circ} 57'$ E) and the united river is known as the Brahmani. Below Bonaigarh it is joined by the Tikira and a number of other hill streams which have their source in the feudatory states and Tributary Mahals.” (Source: Mahalanabis Report, Govt. of Orissa Pg. 48)

The state wise coverage of drainage area of the river Brahmani are as follows.

Table No-1.8 State-wise drainage area.

Sl. No.	State	Area in sq.km.	% to total basin
1	Chhatishgarh	900	2.30%
2	Jharkhand	15700	40.14%
3	Odisha	22516	57.56%
	Total	39116	100%

Drg. No- 1.2(a) Schematic Diagram of river Brahmani (Upper Reach)



Salient Feature

* Brahmani rises near village Nagri in Ranchi district of Jharkhand at an elevation of about 600m in the name of Koel to finally meet with Sankha at Rourkela when both flow together as Brahmani.

* It joins the Bay of Bengal at Dhamra.

* The main tributaries are Kuradhi, Samakoi, Ramiala Rukura, Gohira etc. and the branches are Kimria, Patia-Khasuan.

* The construction of Rengali Dam has moderated the flood to a great extent.

* Catchment Area
-39,116 sq.km.
*Catchment Area in Odisha
-22,516 sq.km.
*Length of River- Total 715 km
(541 km in Odisha)
*States involved
-Jharkhand, Odisha & Chhatisgarh
*Name of Dists. in Odisha
-Sundargarh, Sambalpur, Deogarh, Angul, Dhenkanal, Keonjhar, Jajpur, Kendrapara.
*Highest observed flood
Pre Rengali (1975)
-8,56,750 Cusec (24257 cumec)
Post Rengali (2001)
4,79,337 Cusec (13571 cumec)
*Flood prone districts
-Kendrapara, Jajpur
Dhenkanal

REFERENCE:-
1 Discharge Station ●

(Source: Flood Management Manual, DoWR, Govt. of Odisha, 2008 Pg. 33)

Drg. No- 1.2(b) Schematic Diagram of Brahmani Delta

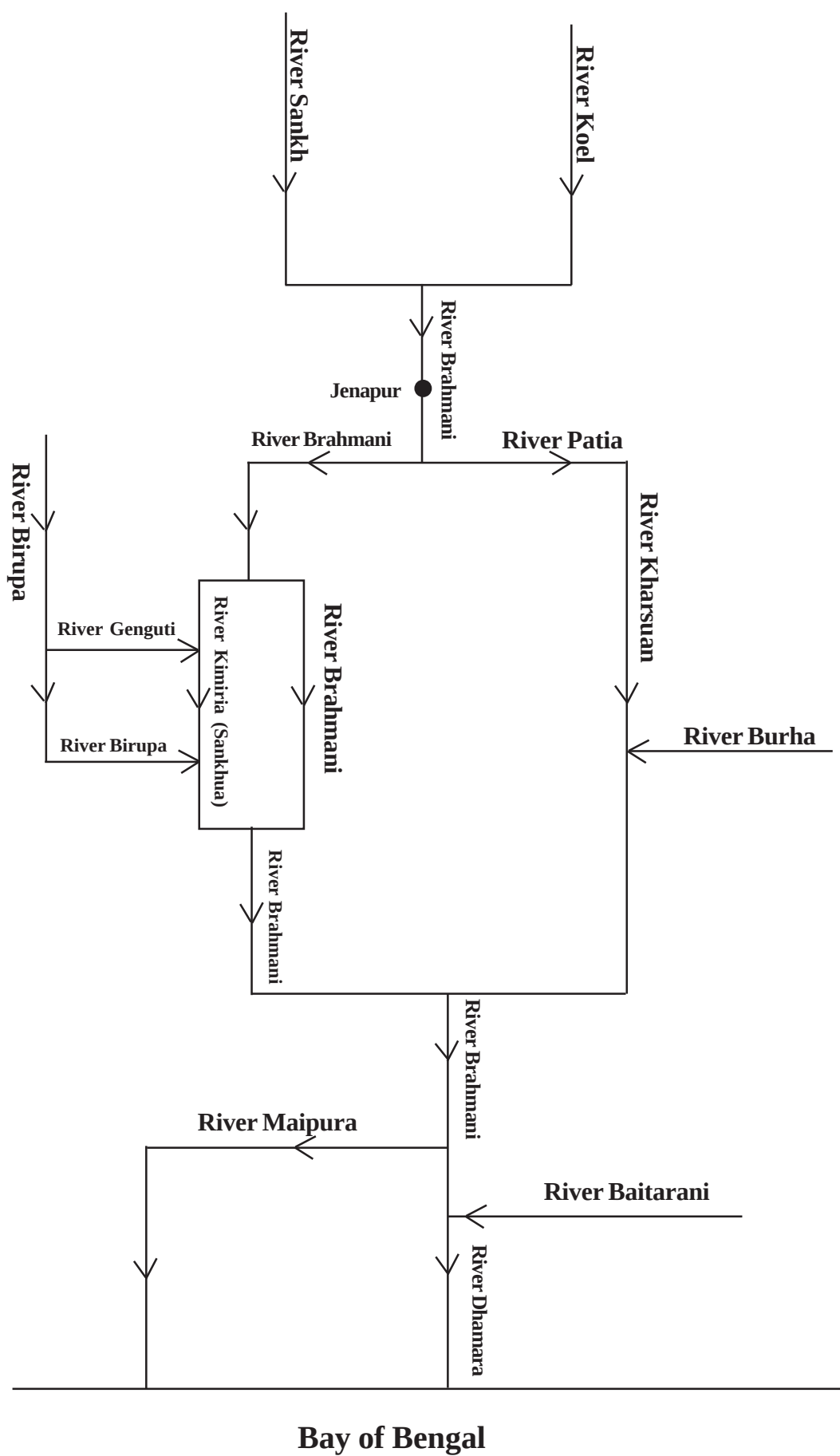
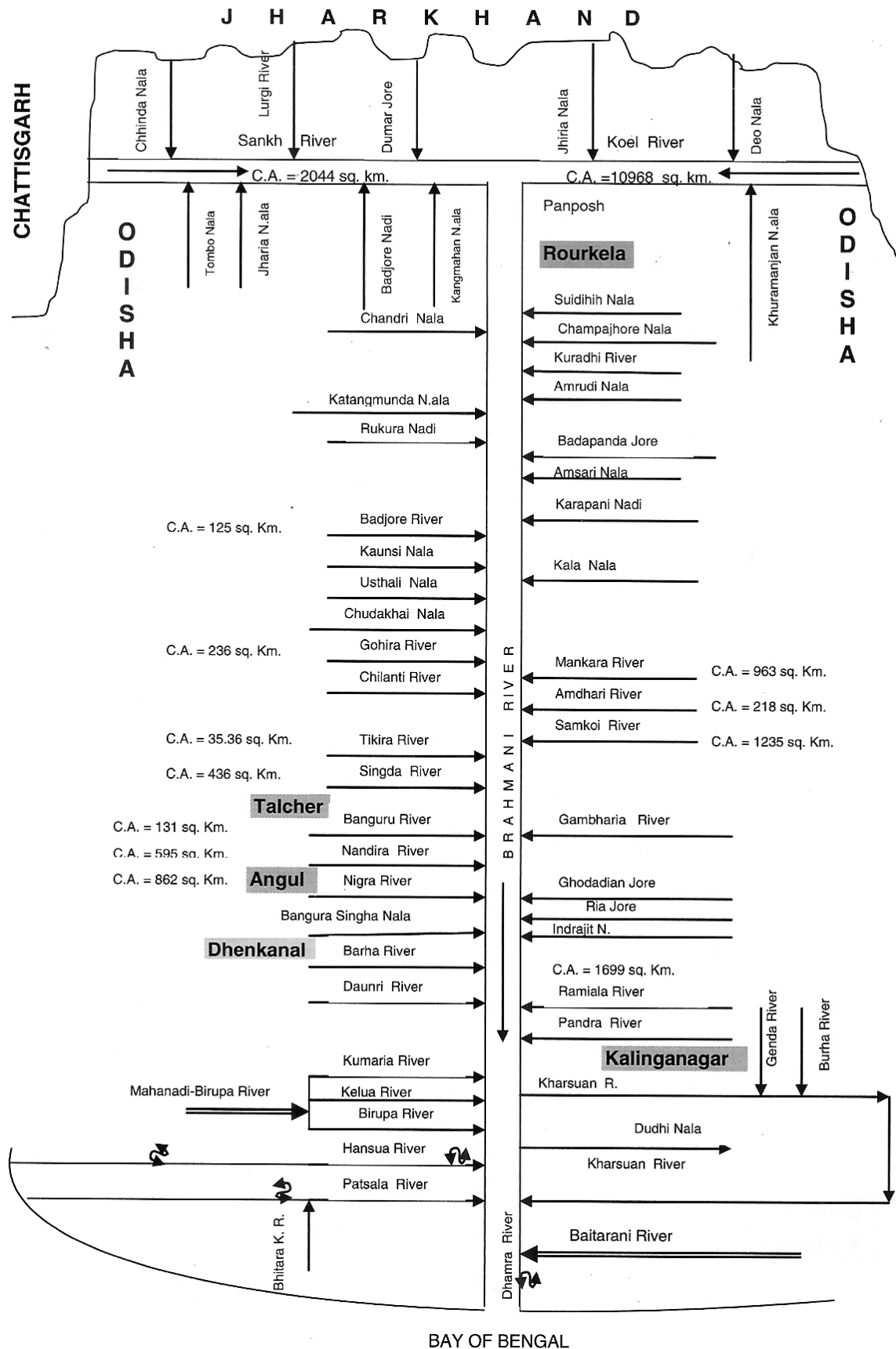


Fig. 1.8 Schematic Diagram of Brahmani River System indicating major towns and industrial area

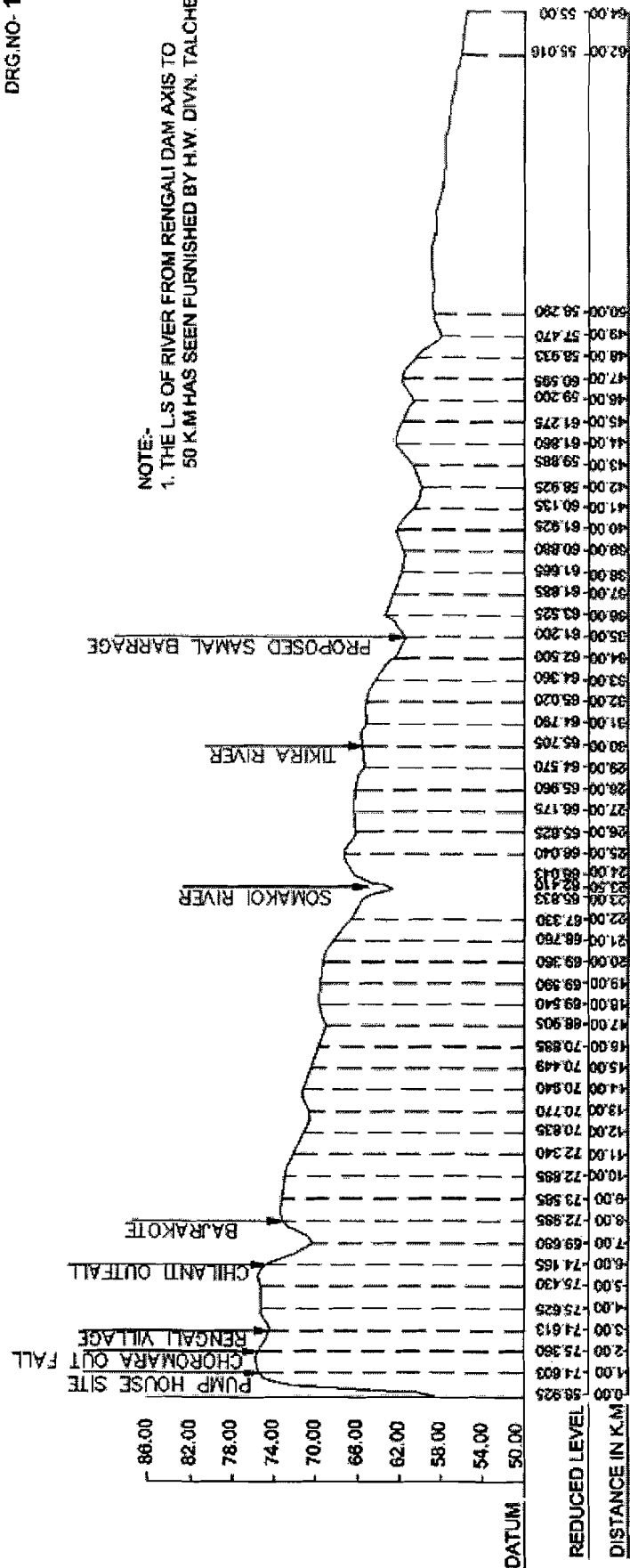


Source: Water Quality of Major Rivers of Odisha (2007-10), SPCB, Odisha, 2013, Pg. 17

DRG.NO- 1.3

NOTE:-

1. THE L.S OF RIVER FROM RENGALI DAM AXIS TO 50 K.M HAS BEEN FURNISHED BY H.W. DIVN. TALCHER



DETAILS L.S OF RIVER BRAHMANI FROM RENGALI DAM AXIS TO TALCHER BRIDGE SITE

1.4 River Basins of Odisha:

The Total geographical area of Odisha is 155707 sq.km. It comprises of 11 nos of major river basins covering a geographical area of 1,51,976 sq.km. and minor river basins draining directly to the Bay of Bengal from an area of 3,731 sq.km. Excepting river Rushikulya and Budhabalanga all the other major rivers either originates from or drains into other state, classifying these river basins as interstate river basins. The catchment area of all river basins are shown in Table No-1.9 and area of Brahmani basin lying in different districts of state in Table No-1.10.

Table No- 1.9 Catchment area of all river basin.

Sl. No.	Name of Basin	Total Catchment Area in sq.km	Catchment area within Odisha in sq.km	Percentage of catchment area in Odisha to total geographical area of the State
1	Subarnarekha	19277	2983	1.92
2	Budhabalanga & Jambhira	4838	4838	3.11
		1853	1516	0.97
3	Baitarani	14218	13482	8.66
4	Brahmani	39116	22516	14.46
5	Mahanadi	141134	65628	42.15
6	Rushikulya	8963	8963	5.76
7	Bahuda	1118	890	0.57
8	Bansadhara	11377	8960	5.75
9	Nagavali	9275	4500	2.89
10	Kolab	20427	10300	6.61
11	Indravati	41700	7400	4.75
Sub Total			151976	97.60
Area directly draining to sea			3731	2.40
Total			155707	100.00

Table No - 1.10 : The area of the Brahmani Basin in Odisha lying in different districts.

Sl. No.	District	Geographical Area (sq km)	Basin Area inside the district (sq km)	% of basin area to total area of the district	% of Basin Area in the district to the basin area in the State
1	Sundergarh	9,712	5,794.13	59.60	25.84
2	Sambalpur	6,657	1,371.05	20.60	6.00
3	Deogarh	2,940	2,512.37	85.45	11.16
4	Angul	6,375	4,225.94	66.29	18.76
5	Dhenkanal	4,452	3,956.91	88.88	17.57
6	Keonjhar	8,303	1,723.48	20.76	7.65
7	Jajpur	2,899	1,824.75	62.94	8.10
8	Kendrapara	2,644	1,107.45	41.89	4.92
	Total	43,982	22,516.08	51.19	100

Map of river basins of the state and that of Brahmani basin are shown vide Drg. No.1.5 & 1.6 respectively.

1.5 Inter-State Issue:

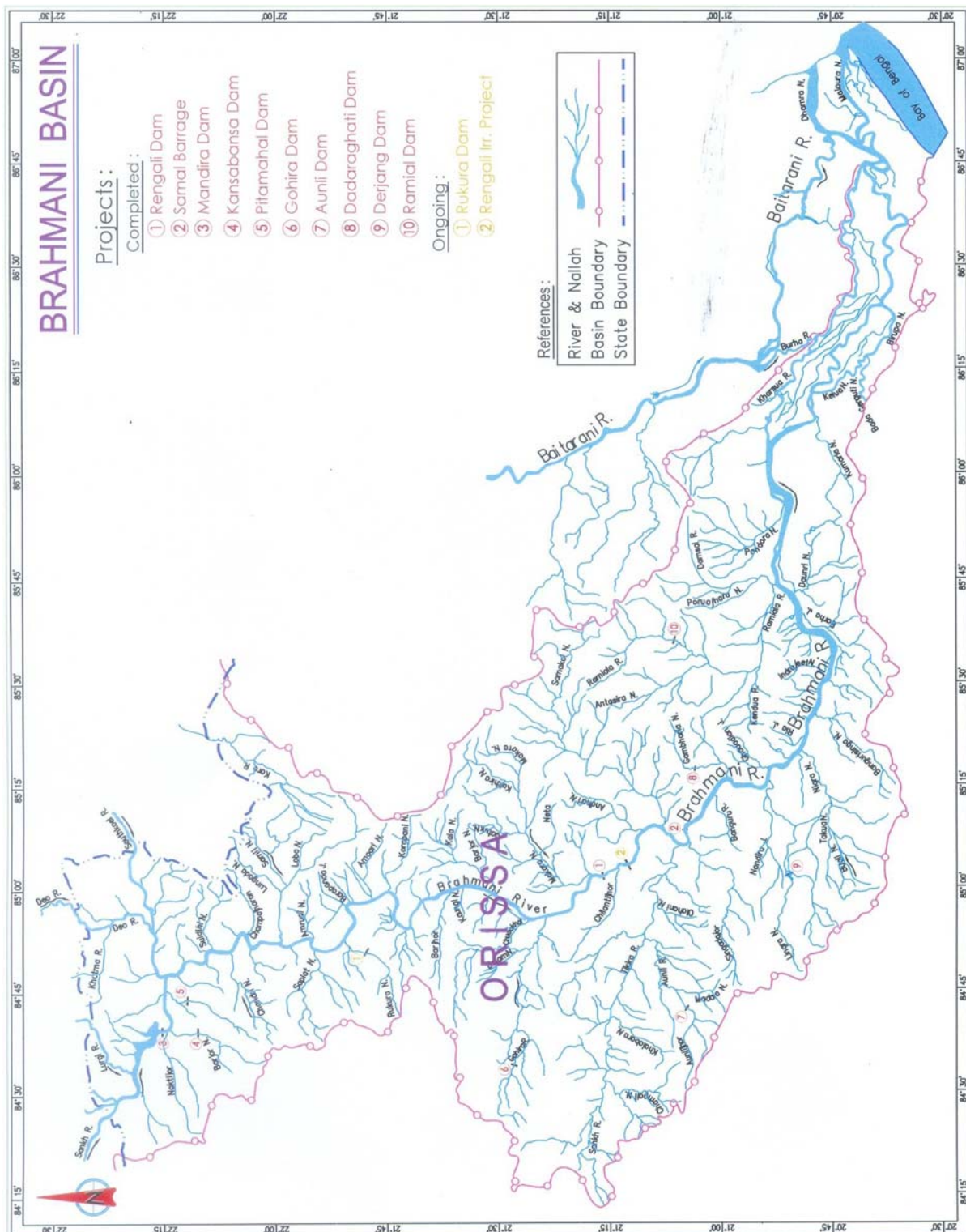
The drainage area of river Brahmani extends over three states viz, Chhatisgarh, Jharkhand and Odisha. The basin has a catchment of 15700 sq.km. in Jharkhand, 900 sq.km. in Chhatisgarh and 22516 sq.km. in Odisha. Till to-day no Inter-State Co-ordination Committee has been formed for Brahmani basin and no Inter-State agreement exists for sharing of water.

“The water resource i.e. the virgin flow of entire Brahmani upto delta head i.e., Jenapur, on average and at 75% dependability on annual, basis, works out to 19088.61 Mcum (532.61mm depth) and 14447.11 Mcum (403.01mm depth) respectively. The respective flow from Odisha portion comes to 11491.42 Mcum (597.27mm depth) and 8689.43 Mcum (451.63mm depth).

As per the report of NWDA (2000), present utilisation in Chhatisgarh is 22.21 Mcum and in Jharkhand is 574 Mcum. With development of irrigation in Chhatisgarh and Jharkhand, the water utilisation will increase respectively to 56.48 Mcum and 3939.28 Mcum in near future. In such scenario, the water availability at Jenapur will reduce to 14495.98 Mcum on annual average and 11361.45 Mcum at 75% dependability (Source: 3rd Spiral study of Brahmani basin, Nov. 2002 Pg. 45, OWPO, DoWR, Govt. of Odisha).



View of Samal Barrage



Drg. No.1.6

Extract of debate in Parliament on 11.04.1972**1972**

SHRI P.K. DEO (Kalahandi): As early as 1901 the colonial government which was in power here thought it wise to have an Irrigation Commission. That commission submitted a report and that has provided a basis for the infrastructure of the irrigation system in the country. After a lapse of nearly 70 years, Dr. K. L. Rao appointed in 1959 the Irrigation Commission. I congratulate him on the fact that the Irrigation commission has produced a very nice report and has submitted at wonderful irrigation at last of the country taking into consideration the various irrigation potentials and requirements and recommending an expenditure of nearly Rs.10,000 crores on a thirty-year plan to bring 50 per cent of the cropped area under Irrigation. But I shall be failing in my duty if I do not point out that in that very some report same mischievous suggestions have crept in.

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I do not subscribe to the principle that questions of vital importance, of life and death, of water to parched lands could be sacrificed at the altar of political expediency. I call it political expediency because some of my friends representing those affected areas sitting opposite might have developed cold feet, but I deem it my duty to bring it to the notice of this House. If it were not political expediency, Dr. Rao would not have gone at the psychological moment when the entire coastal belt of Orissa was slashed by the worst type of cyclone and tidal bore to sell the idea of the Rengali reservoir in the Brahmini. This was never suggested to the Irrigation Commission by the Orissa Government. The Orissa Government have given a list of various schemes by which the flood waters of the Brahmini could be controlled. They wanted a diversion wier on the Brahmini at Rengali. They did not want to submerge 120 sq. miles of fertile and precious land on both sides of the Brahmini. These are the most fertile lands of Deogarh sub-division and Pallabara sub-division. If this area is submerged, there would be hardly any place left to rehabilitate the displaced persons as the rest of the area is all hilly.

Coming to the question of the Rengali dam, we find that the benefit would be flood protection in only 100 square miles, 120 square miles of the most fertile land on both sides of the Brahmini where rural electrification schemes are going to be taken up, according to Dr. K.L. Rao-he opened it at Boulpur the other day is going to be submerged. This is causing grave concern and panic in that area. I would like to tell Dr. Rao that the cyclone or tidal bore or even flood in the coastal area along the Brahmini cannot be controlled by the Rengali dam. The Rengali dam is going to store 3 million acre feet of water. This quantity of water could be easily conserved if the Orissa Government's suggestion of reservoirs on the Tikra, the Aunli, the Singhda Jhor, the Dadra Ghati and the Ramiala are taken up. There has already been a dam on the Sankh which joins the Brahmini near Rourkela. The Mandira dam is already there. Half a million acre-feet of water have been consumed. Even in Bihar, on the Koel, at Handhi, Phujar, and Khatwa, three schemes are going to be taken up.

16 hrs.

MR. CHAIRMAN: The Hon'ble Member's time is up.

SHRI P.K. DEO: The Orissa Government's proposal is to have a dam on the Brahmani at Lodani. If all these projects could be taken up, this Rengali dam could be avoided. I want a categorical assurance from the Hon'ble Minister, Dr. Rao, on the floor of the House that no further panic would be created and no further submersion of land would be allowed.

Sir, in the entire globe, three-fourth is water and one-fourth is land, and there has been so much pressure of population on land that in over-populated countries, the people are reclaiming the sea. Even the Suider Zee has been reclaimed in the Netherlands, and here, we are going to submerge some of our most fertile lands.

I, most humbly suggest that the Dhamra mouth, where the Brahmani and the Baitarani join together before going into the sea, should be dredged for the better flow of water, so that after the dredging, the defunct chandbali port could be brought into operation.

There has been a model study in the Central Water Power Research Station at Poona regarding the opening of the Chilka Lake. Similarly, a study should be made regarding the dredging of the Dhamra mouth which would solve the problem of the Brahmani floods.

MR. CHAIRMAN: That is all right. Mr. R.S. Pandey.

Z



CHAPTER-II

PROJECT PLANNING & INVESTIGATION

2.0 Introduction:

In the July 1972 project report, it was proposed to create a live storage of 2.99 lakh ham. (24.22 lakh Ac.ft) for irrigating 3,26,400 ha. (8,06,208 Ac) of GCA with 2,61,120 ha. (6,44,966 Ac.) of CCA on both left and right of the Brahmani valley and for moderating the flood to 9,900 cumec (3.5 lakh cusec) at the head of the delta for flood control. There was no proposal for power generation in the said report. The left and right canals were proposed to be taken off directly from the reservoir so as to command the valley from a much higher elevation. After thorough scrutiny at the CW & PC level and detailed discussions with the State Government officials, it was finally decided to include hydropower generation and drop irrigation to the second stage.

“The primary objective of this project was to control the flood flow in the Brahmini (Brahmani) System and moderate to 3.5 lakh cusec (9900 cumec) from the maximum discharge of 8.5 lakhs cusec (24066 cumec) at the head of the delta with a margin error of about 50,000 cusec (1415 cumec). By impounding 5.15 lakh ham (41.715 lakh Ac.ft.) at MWL and 4.40 lakh ham (35.64 lakh Ac.ft.) at FRL, the reservoir, with a waterspread area of 406 sq.kms., proposes to protect 2,600 sq. km area of the Brahmani delta from the ravages of floods and thereby to benefit 10.8 lakh population. The average annual direct flood control benefit has been estimated to be approximately Rs.6.65 crores.” (Source: Social Benefit- Cost Analysis of R.M.P., Orissa by Binayak Rath, Ph. D. Thesis, March 1980, Pg. 26-27).

Thereafter power production was also considered and it was stated that the firm power would be only 36.0 MW and that by providing for this power production, irrigation would be reduced to 5000 ha and hence power component was not recommended. These proposals considered the 1961 flood as the heaviest on record, and used the same to construct a unit graph. This unit graph was utilised to prepare the maximum probable flood from maximum possible precipitation estimates available at that time. This flood had a peak of 27800 cumec, a base period of 6.5 days and a flood volume of 5.60 lakh ham.

The filling schedule envisaged the starting of filling from 6th Aug, to reach F.R.L. by 1st Oct. The F.R.L. was proposed at R.L. 122.0 m. It was envisaged to submerge an area of 35300 ha with a population of 31700 to be resettled and rehabilitated. The proposals for their rehabilitation were rather inadequate and had caused great resentment among the people to be affected and there were prolonged agitations later. These proposals were examined by the Central Water Commission (CWC) and the Central Electricity Authority (CEA) and further modifications were made and a supplementary project report was prepared and submitted for clearance in October 1972. The project as revised in October 1972 with some modifications on spillway capacity and installed capacity is the one which has been executed.

The 1972 October project report made a major departure from the July 1972 report in that hydro-power generation was included in the first stage and irrigation was deferred to the second stage. It was also decided that instead of providing irrigation from the periphery of the reservoir, the same would be provided from the tail race waters to be discharged into the river and picked up by a diversion structure lower down at Samal. The probable maximum flood was also updated and spillway capacity was increased.

Investigation and Planning of Dam at Rengali and Barrage at Samal have been described respectively under Sec.2.1 and Sec.2.2 of this chapter.

2.1 Selection of Dam site:

There were three possible dam sites on river Brahmani i.e. at Lodani, Barkot and Rengali intercepting catchment of 21,240 sq.km. 22,900 sq.km. and 25,250 sq.km. respectively. Since one of the primary objectives of the project across Brahmani is flood control, it is necessary to intercept the runoff from most of its catchment for flood moderation. This is the main reason why the dam site at Rengali has been chosen for development at first instance. The river at Rengali flows in a deep narrow gorge predominantly on rocky bed. The site of the dam is so ideal that no earthen flank is required as the masonry dam butts into natural abutments of **Machhkhani** hill on the left and **Salemonda** hill on the right. The advantage has made it a well-conceived and cheap project from cost consideration. The total overall length of dam (N.O.F. section, spillway including power dam) is only 1040 M (3414 ft.). This is the most unique features of such a gigantic reservoir; one amongst 12 big reservoirs of India. The plan and elevation of Rengali Dam is shown in Drg. 2.1.

2.1.1 Topographical Survey:

The Survey of India have done reservoir survey by aerial photographs with contours drawn at 10' (3m) interval. G.T.S. bench marks have been established at dam site. Those have been established by carrying from four points i.e. Samal, Siling, Chhendipada and Naikula, 45 km., 35km., 45km., and 20 km. away from Rengali Dam site respectively. Four survey parties from these four points carried the Bench Mark (B.M.) by double precision levelling method through the shortest accessible route. Survey parties from Chhendipada and Naikula carried the B.M. and fixed it at right side hill at El.102.355 m whereas the other two parties from Samal and Siling carried the B.M. to the left side hill at Dam site and fixed it at El.126.066 m. As the river flows in dry season in a deep gorge having a width of about 100 m, it was not possible to check out both the B.M. by transferring one side to other. After the construction of upstream Coffor Dam, the G.T.S. value of both the B.Ms were connected and checked with each other and found coinciding (not even 1mm difference). Finally the G.T.S. B.M. carried out to dam site was accepted as permanent reference for construction period and for future use. The detail ground survey was done initially by Angul Irrigation Division prior to July, 1972. Subsequently Talcher Irrigation Division was created with headquarters at Talcher and then Rengali Dam Division started functioning with headquarters at Rengali Dam Site. After 1972 monsoon, Rengali Dam Division took up the pre-construction survey work. The site including proposed colony sites were cleared of and erection of demarcation pillars, marking of dam axis and other survey reference pillars were constructed. All construction surveys were conducted with reference to G.T.S. bench marks and comprehensive contour plan was prepared for the entire dam base area extending one km. upstream and one km. downstream from the dam axis. Cross sections at dam axis and at suitable intervals in upstream and downstream were taken accurately and plotted. The dam base contour plan together with cross sections were submitted to Central Water Commission for preparing the detailed drawing and design which was entrusted to them. For hydrological study, a gauge station was established 660 m downstream of dam axis and the gauge discharge relationship was developed for the dam site both for monsoon and non-monsoon flow. The old gauge station at Bajrakote 8 km downstream of dam axis established since 1958 was rejuvenated taking fresh cross-section and establishing new gauges. Gauge and discharge observation started systematically with development of gauge discharge relationship.

Survey work for construction of permanent colony was taken up and a master plan prepared finalising location of different types of residential and non-residential buildings like Administrative block, Hospital, Schools, Clubs, Bus Stand, Community Centre, Guest House, Erector's Hostel etc.

2.1.2 Geological Investigation:

2.1.2.1 Regional Geology :

A 50 km wide E-W running Precambrian assemblage comprising granulite, schist, gneiss and quartzite is bounded by Gondwana trough fault on the south and a group of rocks belonging to Iron ore Series on the north beyond Bajrakot (20°31'00":85°00'33", 73 G/2) and follows the southern boundary of the Keonjhar District. This Precambrian assemblage of rocks belong to Eastern Ghat Supergroup-viz., Charnockite, Quartzite, Khondalite (in order of decreasing abundance) which are later intruded by

Younger granites and basic bodies. This belt runs for several hundred kilometers through Bolangir, Sambalpur and Dhenkanal Districts and runs from West to East, with a similar strike having a steep subvertical dips towards south, for about another 12 km beyond Khamar (21°16'30":85°12'00", 73 G/3) where it almost abuts against NW-SE striking Malayagiri hill (21°22'30":85°15'30", 73 G/7) range making it apparent that the Eastern Ghat suite of rocks are having a clear structural discordance with that of rocks of Iron Ore Series of Malayagiri.

2.1.2.2 Site Condition & Geology:

2.1.2.2.1 Reservoir Area:

The reservoir spread of Rengali at FRL 121.9 m extends between N 21°16'30"-21°32' and E 84°-48' - 85°10', falling in the Survey of India toposheet Nos. 73 G/2, G/3, 73 C/14 and C/15 covering parts of erstwhile Sambalpur and Dhenkanal Districts. The area comprises essentially Precambrian granite and gneisses, charnockite, hornblende schist and gneisses and are scattered in the eastern part of the reservoir area with few small detached bands of quartzite. A number of medium sized dolerite dykes cut across the country rock along WNW-ESE and ENE-WSW direction at the upstream of Brahmani - Mankada nala confluence. A few thin tremolite schist bands (Upto 0.8 Km long) - occur within granite gneiss conformable with the strike of the country rock. Another band of tremolite schist about 3 km long near Rankhol Village (21°19'30":85°06', 73 G/3) runs along E-W direction. It grades into hornblende schist to the north. General trend of foliation is N 70°E-S70°W with moderate to steep dip towards north / south. Around the dam site, some antiformal and synformal folds have been established within the gneissic rocks upstream of dam site. Also the outcrop pattern of the quartzite band exposed on the north-eastern corner of the reservoir area - points to the presence of a regional fold roughly along E-W axis.

From the general structural pattern - as could be deciphered in the reservoir area, it appears that there is no major fault or other weak zone in this area, which may cause leakage from the reservoir. No economic mineral deposit / old working mines have been recorded in this area. Talcher Coal Field lies on the downstream of the dam site and as such it is not affected by impounded water.

2.1.2.3 Dam Site:

1) Brahmani River Valley at the proposed dam site is a fairly wide, mature, 'U' shaped valley with a large nearly flat ground in between two abutment hillocks on two banks. At the dam site river flows towards S 25° W. Dam alignment is N 70° W (R/B) - S 70° E (L/B)

2) Upstream of the dam axis, water channel during summer is about 60 m wide. It gradually becomes wider towards downstream, about 110 m wide along axis. Further downstream, it has bifurcated into two channels, the main channel (50m wide) flows along the right bank while the smaller channel (10-20 m wide) flows along the left bank with a rocky island (70-80 m wide) in between. In the channel, water is stagnant, its depth in the pool just upstream of the axis is about 9 m, while along the main channel downstream of the axis, depth of water varies between 5 m and 8 m. During the monsoon, water overflows the banks and spreads over an area between RD 210 m and RD 720 m. Flood water on recession, leaves behind innumerable patches of sand bars of different size - both upstream and downstream. Depending on the nature and intensity of flood, the local sand bars are either washed away or deposited afresh-from time to time. Beyond RD 720 m on the right bank, long patches of soil covered areas are present alongwith rocky portions. Some of these soil patches are paddy fields also. On the upstream side, there is a very wide soil patch (70m - 80m wide), which extends towards north. On the downstream side, two prominent soil patches (2040 m wide) extend along E-W direction. Besides these, innumerable small soil patches (10-20 cm thick) are present in the rocky bed. Two abutment hills on the two sides of the river are mostly covered with talus material (broken or disintegrated charnockite) with some insitu rocks exposed at places.

3) The rock types present at the proposed dam site including the river bed and two abutments are principally acid charnockites with some thin bands (1 cm - 1.5 m wide) of dark coloured fine to medium grained metabasic rocks trending along the foliation direction of country rock. Abutment hillocks on both the banks run along WNW-ESE direction, which confirms to the general foliation trend within charnockites exposed at the dam site.

4) Left Bank (RD 0m - RD 308 m) : In this part, talus material covers the hill slope upto R.D. 210 m. Beyond that, a small outcrop of rock within a big sand patch is observed. This sand patch extends upto the left bank in a semicircular shape. Acid charnockites are exposed, as scattered outcrops of various sizes within talus covered hill slopes along non-overflow section of the dam of this bank. Foliation trend towards downstream of non-overflow section generally varies from E-W to N 70° W-S 70° E strike with a steep southerly dip (50°-80°). Towards upstream, foliation dips are at places vertical, beyond that foliation trend varies from N 70° E-S 70° W to N 80° W-S 80° E with moderate dip (54°-76°). Two small shear zones cut across charnockite-the major one is about 75 m upstream of R.D. 300 m showing N 45° W-S 45° E strike with 70° dip towards north. Veering of foliation within charnockite on both the sides of shear zones and presence of fine grained crushed rock (mylonite) indicate shearing effect. Two 10 cm wide dark coloured fine-grained metabasic rock veins are present within the acid charnockite.

5) Riverbed (R.D. 308 m - R.D. 420 m) : In downstream, river is bifurcated into two channels, separated by a large outcrop of acid charnockite (280 m long and 40-90 m wide). Foliation joints are quite prominent within this rock and some of them are 2-10 cm open. Two closely spaced (1.5 cm) steeply dipping jointed zones have been noticed. One of these is observed to cut across water channel and extend into left bank. In the southern part, a pegmatite vein of 30 cm thickness runs in N 80° W-S 80° E direction.

Besides this, thin bands (8 cm - 1.5 m wide) mostly dark coloured, fine-grained metabasic rock running parallel to foliation of the rock is exposed within both the banks of this channel. In this part, foliation trends in N 80° W-S 80° E with steep dip (62°-80°) towards south at the downstream side of the riverbed. Within the main channel, a small rocky mound of acid charnockite protrudes above water level at 88 m upstream. This rock shows two sets of vertical joints.

6) Right Bank (R.D. 420 m - R.D. 1040m) : From right bank of river, acid charnockite outcrop extends upto right abutment hillock interspersed with a number of sandy patches of different dimension continuing upto R.D. 720 m. In this part, foliation in downstream part strikes between E-W to N 75° W-S- 75° E with steep southerly dip (52° - 80°). Towards upstream, vertical foliation was recorded. Beyond which, foliation dip towards north. Further upstream, dip becomes lesser (40°). Between R.D. 720 m and R.D. 990 m, small patches of soil occur within the rock outcrop. Beyond R.D. 990 m, talus material with some patches of insitu rocks are present along the slope of the right bank hillock. From R.D. 520 m to R.D. 610 m, one meter (approx.) wide shear zone (N 50° W-S 50° E) with fine crushed material showing veering of foliation planes on either side is observed. Quartzo-feldspathic veins (1-2 cm thick) are intruded along shear planes. Another shear zone was recorded at 140 m upstream of R.D. 570 m (N 50° W-S 50° E trend with vertical dip). Beside these, a number of closely spaced fracture zones were found at both the upstream and downstream parts of the axis. Rocks exposed near right abutment hillock are slightly to moderately weathered with tight joints. Bands of metabasic rocks show small scale folds.

At about 265 m downstream, a small outcrop of leptynite was encountered which show N 80° E-S 80° W trend. This perhaps marks the transition zone from charnockite towards the north with granite gneiss towards south.

2.1.3 Structure Foliation:

Foliation in Charnockite is poorly developed which trends N 70° - 80° E-S, 70° - 80° W to E-W having steep dip (70° -85°) towards south. On the north bank, little upstream charnockite is folded into asymmetric antiform with almost E-W axis with a low 15° - 20° plunge towards west.

Charnockite forms the core. On the right bank, some impersistent series of puckers are seen on the foliation plane. Axes plunge 50° towards N 80° W to W. Axial plane dip high towards upstream i.e., towards north. Lineations are parallel to the intersection of pucker axes of foliation plane. Parallelism of pucker axis with antiformal fold on left bank indicates that tectonic force got diminished towards right bank since the same could produce only a few puckers while giving rise to an antiformal fold on the left bank of the river.

Dark coloured bands of metabasic rocks, especially those which are exposed on the right bank show irregular types of flowage folding with steep to moderately steep plunge ($50^\circ - 75^\circ$) towards east/west. At places, these folds are disharmonic in nature with respect to folding. Shear zones (length 30 m - 240 m) present on both the banks are situated near the axial region of antiform. Trend of shear zones make acute angle with the fold axis direction and displacement along shear zones appears to be about 2-3 m only. These are probably due to later movements within the axial region, being the weakest zone.

Joints: The following major joints have been noticed in the rocks of Rengali Dam site:-

No.	Attitude	Opening	Spacing	Nature	Remarks
1	N 75° - 85° E-S 75° - 85° W strike with 70° - 80° dip towards south (J_1)	Tight (0-2 cm)	5 cm-2 m	Foliation joint	Most prominently developed
2	N 10° - 25° E-S 10° - 25° W strike with 25° dip towards east (J_2)	0-2 cm	20 cm-2 m	Relief joint (Parallel to slope angle)	Well developed
3	N-S strike with steep sub vertical dip slightly towards east (J_3)	Hair thin (0-2 mm)	50 cm-2 m	Dip joint	Well developed
4	EW strike with vertical dip (N 60° E-S 60° W to N 60° W-S 60° E) (J_4)	0-2 cm	20 cm-1 m	Cross joint	Well developed
5	N 10° - 25° W-S 10° - 25° W with sub-vertical dip (J_5)	0-1 mm	20 cm-2 m	Cross joint	Well developed

Foliation / strike joint with other joints (J_4 , J_5) are fairly well developed and persistent. The joints encountered in the rocks of dam foundation die out towards depth ranging between 4 m and 10 m from surface.

i) Nature and Origin of deep River channel between R.D. 336 m and R.D. 385 m:

Based on structural mapping, it could be preliminarily concluded that deep river channel between R.D. 336 m and R.D.385 m was not due to any fault / shear zone. Aerial photograph study also ruled out presence of any weak zone along deep channel. It was also found that below HFL, extensive pot hole formation have taken place in rocks testifying to intensive corrosion during high floods. Some of these photholes have a diameter of 3 m. Further, mini pot holes formed along very well developed longitudinal and cross joints sometimes join together to form a big pot hole. In many of the pot holes, grinding material was hard banded hematite jasper (BHJ) of Iron Ore Series rocks that predominantly occupy the reservoir area. On the basis of these observations, it was anticipated that the Brahmani River channel could be interpreted as a '**Corrosion trough**' in hard charnockite. Under an active stage of corrosion, no weathering should sustain in the rocks at river bed level. In general, deep river channel has been formed due to coalescence of pot holes along the master joints which are parallel to river course in active phase of corrosion.

ii) Tectonic Control of Brahmani River at Dam Site:-

Antiformal fold axis, in the left bank trends almost E-W. Hence principal compressive stress for this fold must have form N-S direction - represented by a joint set (J_3). If principal stress is along N-S direction - which gave rise to fold along E-W direction, then two prominent shear joints are likely to develop along J_2 (N 15° E-S 15° W), direction of main river channel and J_5 (N 25° W-S 25° E), the spill channel. It is apparent that Brahmani River is guided by these two master joints. Whether any shearing actually took place along any of these two planes or not could not be deciphered since later metamorphism has recrystallised the sheared materials completely and sealed off the traces of shearing / joints. Counter dilation along E-W (due to N-S compression) will produce 2 more sets of tensional joints and perhaps J_4 set. Stretching of minerals along fold axis has given rise to foliation plane and consequently the foliation joint, J_1 .

iii) Petrographic Study:-

Under microscope, charnockite, a medium to coarse grained rock supposed to be a deformed rock showing granulitic to granoblastic texture with interlocking grains having sutured contacts. Essential minerals are perthitic K feldspar, quartz, showing wavy extinction and coarse irregular grains of hornblende. Hypersthene is rarely present. Probably, it has been converted to hornblende due to effect of retrograde metamorphism and metasomatism. Plagioclase with lamellar twinning has also been recorded. Among accessory minerals apatite, sphene, opaque minerals are common with a few epidote grains and some fine flakes of biotite filling minute cracks within feldspar and amphibole, grains. Fine-grained rocks of shear zone are essentially mylonite having hornblende, biotite, sphene, epidote etc. as mafic minerals.

On the whole it was observed that the foundation of Rengali Dam is free from any major defects and is one of the best foundations on which the gravity dams have been constructed in our country next to Nagarjunsagar Dam (A.P.)

There were two shear zones in the foundation which are briefly discussed as under:

(i) A shear zone of about 1" thick with 60° upstream dip running N-E was observed from the joint of block 6/7, from the downstream portion of the joint, to block 9/10 in the upstream portion. The shear zone is delimiting the downstream portion.

(ii) Another shear zone running parallel to the axis of the dam with approximately 70° downstream dip to block Nos. 5, 6, 7 and 8.

In this regard view of Sri B. Ramachandran, Director Geological Survey of India (GSI) on 22/6/76 and 11/8/76 (vide his letter No.614/EG/775) may be referred to. The shear zone running parallel from block-6 up to block-10 and extended upto block 14 on left bank deep channel and crossed to block No.18 on right bank of which trenching was done from block 10 to 14 to remove all loose and shattered layers of rock. The shear zone in block-6 to 10 are dipping towards upstream side. This was treated by trenching twice the width of shear zone and grouting extensively the contact plane with standard procedure.

A pegmatite vein of about 1 to 2 ft. thick was observed between block 40 and 41 in the upstream and block.38 in the downstream. Also some sets of joint planes existed in that area.

In the Nortite zone on block 39-40, after excavation it was observed that the joints are very tight and compact. However, routine grouting was resorted to as the joints continued from upstream to downstream.

During initial course of investigation, 0.47 sq.km of site was mapped on a scale 1:1000 (field season 1972-73). Besides 349.22 m of drill cores of the dam site were logged and interpreted. Insitu permeability tests were carried out in bore holes to assess secondary permeability of the sub-surface rocks. Bore hole nos.11 and 12 which intersected below the deep channel established that there is no structural disturbance like fault or shear zone along the deep channel.

14 Nos. of bore holes totalling 349.22 m were drilled to determined bed rock depth, sound rock depth and nature and character of the bed rock along the dam axis and axis of the bucket. The data indicate that the deepest foundation grade rock is encountered in B.H. No.8 at a depth of 8.06 m and at 0.08 m depth in B.H. No.13. Below the foundation grade in respective holes, joints are sparsely developed imparting a massive character to the bed rock. Joints persist at depth also but are invariably tight with fresh surface, though mostly under confined condition, these are incipient planes.

The deep river channel between RD's 336 and 385 m was probed by two intersecting inclined holes (No.11 and 12) and three vertical holes (No.6, 7 and 9). For the vertical holes in the river bed, fresh and sound charnockite bed rock was encountered immediately below the river sand, proving absence of weathered zone in the bed rock. Moreover, in these holes percentage of core recovery was 100%. Further, no evidences of fault or shear or even closely developed jointing could be obtained in the cores along the river channel. As no fault or shears are encountered in the deep channel portion, thus, geologically there is nothing wrong against sitting the Power House (P.H.) at the deep channel. Further, it was proposed to have the overflow section on the right channel.

Water pressure tests indicate that there is no significant water loss below foundation level even in pressures ranging upto 5.6 kg/sq.cm. In such situation curtain or consolidation grouting below the foundation grade may not be required. However, in view of 60m (197 ft.) high gravity dam, it is recommended that curtain grouting down to a depth of 1/3rd the height of the dam may be carried out. However, it is anticipated that in many of the holes, the actual grout intake will be negligible. Similar situation of negligible grout intake is expected in consolidation grout holes. But it is suggested to go ahead with the routine pattern and depth per consolidation grouting due to construction of very heavy gravity structure.

Synoptic logs of 40 Bore Holes are annexed vide Annexure -I and Water loss test in Annexure-II.

Geological investigation of dam site was carried out by the geologists of G.S.I. from field season (F.S.) 1973-74 and continued upto F.S. 1981-82. Summary of investigation undertaken by the geologists for the F.S. 1979-80, 1980-81 and 1981-82 are indicated as under:

F.S.1979-80: Blocks 2, 3, 5(P), 9,10(P), 11(P), 12(P), 13(P), 14(P), 15 to 18, 22 to 25, 46(P), 47(P), 49(P), 50, 51 and downstream cofferdam and Training wall covering an area of 16040 sq.m. in 144 days.

F.S. 1980-81: Blocks 18 to 20, Bucket portion of blocks 22 to 55, 5(P), Power Dam and Power House. Plugging of inner channel of river course, 45m downstream upto the cofferdam on river bed covering 12763 sq.km. in 59 days.

F.S. 1981-82: Left wall section of A-D line of P.H. 10m D-line section and power dam ridges covering an area of 1436 sq.km. in 14 days.

When the dam foundation was fully excavated and opened in left and right bank as well as in deep channel after successful completion of the coffer dam Sri G.S.M. Rao, Director, G.S.I. (Eastern Region) inspected the dam site (on 14.2.1979) and offered followings comments:

Block No-49: (Right abutment) - Intermediate charnockite at foundation grade has steep South-Eastern slope near the toe of the dam. It was suggested to provide suitable anchor bars at the sloping face of the rock for the stabilisation of the structure.

Block joint 46-47: An open based joint was seen in Norite near the toe of the dam which extends westward probably upto the charnockite contact in block-47. It was suggested to remove the loose decomposed rockmass, wash the zone thoroughly and back fill with concrete. This contact being transverse to the dam foundation is a possible path of seepage. Its opening at depth is to be properly deciphered by water percolation tests and sealed, if necessary by grouting. Curtain grouting of the dam foundation is to be so oriented to intercept and seal such zones as well as the criss-cross joints effectively.

Deep channel portion: Removal of riverine deposit comprising sand, shingles and boulders from the deep channel portion indicates that charnockite exposed in block 15 and 17 is fresh even at RL 76.00 but has a number of pot holes of different sizes and traversed by basal joints, as a result, the rockmass of the side of channel is in form of loose boulders.

Above all, Rengali Dam foundation is one of the excellent foundations gifted by nature for construction of a 60m high gravity dam. The site being free from major defects has not posed any serious foundation problem during construction.



Exposed charnockite near the right abutment of Rengali dam

Synoptic log of Boreholes

Subsurface exploration was carried out for determining depth of bed rock, fresh rock and the nature of bed rock. A total of 40 numbers of boreholes were drilled around the dam site. Synoptic log of the holes drilled are given below:-

Hole No.	Location (RD in m)	NSL(m)	Depth drilled (m)	Bedrock level(m)	Fresh-rock level (m)	Remarks
1	2	3	4	5	6	7
1	214 (7m u/s) Left bank (Revised RD 180 m)	88.975	11.02	85.955 (3.36)	83.175 (5.8)	Fresh charnockite occurs at RL 83.175m. Except at depths between 5.55 m and 5.79 m where stained and weathered joints noted. The entire run below RL 83.75m, is through massive charnockite
2	330 (76 m u/s)- River bed	77.24	42.79	77.24 (Surface)	76.21 (1.03)	Bedrock occurs from surface itself. Fresh charnockite occurs at 1.03 m depth. Thin bands of dolerite have been noticed. Tight and fresh joints noticed upto 22.96 m depth. From 22.96 m to 42.79 m, joints are ill developed and incipient.
3	534 (5 m u/s) (Revised RD 500 m) – Right Bank	85.365	24.48	85.365 (Surface)	79.675 (5.690)	Fresh, sound charnockite occurs below RL 79.675 m. Joints completely die out below 2.69 m depth.
4	660 (Revised RD 590 m) – Right Bank	86.5	24.78	84.63 (1.87)	82.49 (4.01)	Overburden of sand down to 1.87 m. Weathered bedrock charnockite occurs from 1.87 m to 4.01 m depth. From 4.01m to 24.78 m, fresh massive charnockite with occasional poorly developed gneissosity. Single stained joint noted at 5.38 m depth, below which no joints are discernible.
5	740-Right Bank	92.937	24.48	92.937 (Surface)	89.21 (3.72)	Weathered mesocratic chamockite occurs upto RL 89.21 m below which massive charnockite with rare gneissosity noted down to bottom of hole. Joints have died out in fresh rock.
6	355-River bed (3m u/s) (Revised RD 325 m)	77.540 (Water level)	26.02	63.98 (13.56)	63.98 (13.56)	In deep water pool. No weathered profile. Immediately below riverine sand, fresh, massive charnockite noted at 13.61 m depth.
7	352-River bed	77.405 (Water level)	21.53	62.10 (15.3)	62.10 (15.3)	11.27 m deep water pool. Weathered zone in bedrock absent. Fresh charnockite occurs at a depth of 15.3 m (RL 62.105 m). Joints below fresh rock level are in the form of incipient fractures only.
8	886 (Revised RD 840 m)	93.945	25.92	87.26 (6.685)	86.125 (7.820)	Overburden of sand. Chamokite intersected by tight incipient joints down to bottom of hole. Only stained joint noted in fresh rock at 13.93 m depth.
9	347 (Revised RD 320 m) River bed	78.175	22.44	60.81 (17.365)	60.81 (17.365)	From RL 78.175 m to RL 60.81 m pool of water. Fresh, massive chamockite occurs at RL 60.81 m. No joints discernible in bedrock.

1	2	3	4	5	6	7
10	690 (80 m d/s) (Revised RD 650 m)	91.066	4.33	91.016 (Surface)	88.27 (2.796)	From RL 91.016 m to RL 88.27 m fresh charnockite occurs with heavy weathering along relief joints. Below RL 88.27 m, fresh, massive chamockite occurs.
11	320 (Revised RD 290 m) River bed	78.166	45.86	78.166 (Surface)	78.166 (Surface)	Inclined hole (45°). Fresh, massive charnockite with poorly developed gneissosity. Open jointed zone encountered between 16.71 m and 17.32 m depth (inclined depth). Joints are tight and fresh in nature. No evidence of structural weakness noted in the entire run.
12	385 (Revised RD 350 m) – River bed	79.77	61.37	79.77 (Surface)	76.27 (3.7)	Inclined hole (45°). Drilled along the riverbed Borehole pierces entirely through massive charnockite with poorly developed gneissosity, incipient tight joint noted down to 35.43m depth. No structural disturbance like crushing/dragging noted in the cores.
13	570 (60 m d/s) (Revised RD 350 m) (Right Bank)	8592	6.31	85.92 (Surface)	85.84 (0.8)	First 0.8 m of weathered chamockite below which fresh, massive chamockite encountered. A single kaolin coated joint surface is noted at a depth of 0.6m. From 1.61 m to 1.74 m, a basal joint set is encountered along which weathering and limonitic stains prominent.
14	980 (Revised RD 920 m) (Right Bank)	100.01	18.50	99.66 (0.35)	94.92 (5.09)	Fresh charnockite occur at 5.09 m depth below. Open joints persisted down to 6.47 m
15	115 (23m d/s)- Left bank	99.145	30.6	83.78 (15.36)	83.78 (15.36)	Fresh massive chamockite with occasional thin bands of pyroxene granulite
16	250 (12.5 m d/s) – Left bank	77.47	30.73	77.47 (Surface)	74.77 (2.7)	Charnockite with one set prominent foliation joint dipping 65°
17	91.4-Left bank	107.030	22.35	107.030 (Surface)	99.43 (7.6)	Charnockite with thin clay filled joints (65° and horizontal)
18	50-Left bank	115.935	26.00	115.935 (Surface)	98.51 (17.42)	Acid chamockite with less developed foliation to massive.
19	225-Left bank	130.87	21.35	130.87 (Surface)	119.17 (11.70)	Fresh charnockite almost massive in nature. No staining/joint planes below F.G.R.L.
20	225 (65 m d/s) – Left bank	76.950	19.71	76.950 (Surface)	75.15 (1.8)	Fresh, hard, massive chamockite with rudimentary foliation at 70°. 2 sets of joints at 70° and horizontal but no staining along joints.
21	685 (43.8 m d/s)- Right Bank	92.285	16.21	92.285 (Surface)	85.03 (7.25)	Fresh, hard, massive, chamockite without any staining. Rudimentary foliation at 70°.

1	2	3	4	5	6	7
22	860.80 (43.8 m u/s) - Right Bank)	96.475	18.54	87.825 (8.65)	84.43 (11.23)	Jointed broken piece of charnockite with thin clay filling.
23	864.4 (52 m d/s) - Left bank	96.930	18.57	90.22 (6.17)	87.03 (9.36)	Charnockite
24	86.7 (158 m d/s) - Left bank	96.83	17.81	94.83 (2.0)	91.48 (5.35)	Hard, rudimentary foliated to massive acid charnockite having 67° dipping foliation joint.
25	143 (37 m d/s) - Left bank	91.290	17.15	86.750 (4.54)	85.75 (5.54)	Fresh, hard, massive to rudimentary foliated acid charnockite with 65° dipping and sub horizontal joint.
33	855.7 (30.1 m d/s) Right Bank	84.72	6.02	84.72 (Surface)	83.22 (1.5)	Fresh, hard, massive chamockite
34	818.3 (11.75 m d/s) Right Bank	84.73	7.60	84.73 (Surface)	82.78 (1.95)	Fresh, hard charnockite with stained joints
35	869.7 (4.6 m d/s) Right Bank	84.83	8.26	84.83 (Surface)	78.13 (6.7)	Fresh, hard charnockite
36	880 (25.8 m d/s) Right Bank	85.05	2.47	85.05 (Surface)	83.50 (1.55)	Fresh, hard norite
37	900 (27.3 m d/s) Right Bank	86.9	7.76	84.13 (2.72)	81.40 (5.5)	Fresh, hard norite with clay filled / stained joints.
38	910 (90 m d/s) Right Bank	88.8	8.38	88.8 (Surface)	84.42 (4.38)	Fresh, hard norite with horizontal joints. No clay filling noticed.
39	892.15 (2.2 m d/s) Right Bank	86.05	62.07	86.05 (Surface)	79.58 (6.47)	Fresh noritic rock.
40	917.5 (25 m d/s)	89.715	6.21	89.715 (Surface)	84.42 (5.29)	Fresh noritic rock

(Source: Geological Survey of India, Bulletin Series B, No.65- A Comprehensive case history of Geotechnical Investigation of Rengali Dam Project-2013 Pg. 12-14)

“The river bed between RD 308 m and RD 420 m has been explored through six numbers of boreholes (2,6,7,9- vertical holes and 11 & 12- inclined holes). No weathered zone has been encountered in hole numbers. 2, 6, 7 & 9. These four holes show fresh, foundation grade rock occurring at depths varying between 1.03m and 17.36m. Below foundation grade rock, divisional planes like joints / gneissosity are sparsely developed imparting a massive character to the bedrock. Open stained joints are seldom seen below foundation level. Joints persist at depth also but are invariably tight with fresh surface, though mostly under confined condition – these are incipient planes. Most of the joints barring the low dipping relief joints, are very steeply dipping and are widely spaced. As such, there is non-intersection of these joints in vertical holes, yielding thereby the massive character to bedrock. Though the sub vertical joints generally die down within 2 m depth from fresh rock level, there are instances such as in hole nos. 8 and 14, where open joints (stained) persist down to 13.93 m and 6.97 m respectively. Moreover, in these holes, percentage of core recovery in bedrock in general is 100%. No evidence of fault / shear or even closely developed joints could be obtained in the cores along river channel. Hence, the trough like channel along which the Brahmani River flows, does not owe its origin to structural disturbances like fault / shear planes.

The boreholes drilled in the left flank portion (19, 18, 17, 15, 1, 16) indicate presence of weathered bedrock at depths ranging between surface level and 3.36 m. Fresh foundation grade charnockite occurs at depths varying between 2.7 m and 17.42 m. In most of the holes weathered bedrock occurs almost right at the surface but is highly weathered. Rudimentary foliation (65°) is common. Two sets of joints (65° and sub-horizontal) have also been noticed. Fresh rock is mostly massive in nature and no staining was recorded. In the right flank, ten boreholes (3,13,4,10,21,5,8,22,23,14) were drilled. Weathered bedrock in most of the holes occurs between surface and 8.65 m depth whereas fresh rock occurs at depths varying between 0.8 m and 11.23 m.”
(Source: Ibid Pg.14)

Water loss test :

“An attempt has been made to correlate between the drill hole record and available water loss test data.

i) Hole No.1: Permeability value in test section (4.41 m and 7.26 m) ranges between 77 lugeons and 140 lugeons. Permeability values are erratic. High permeability is attributed to the presence of closely spaced, weathered and stained joint – identified as relief joint. In test section between 7.45m and 10.36m, permeability values ranging upto 1.13 lugeons. This one is water tight.

ii) Hole No.2: Permeability test carried out in between 1.67 m and in between 10.88 m and 13.78 m indicate high permeability value (95-485 lugeons). Rests of the sections are almost water tight. Permeability values down to 41.42m varied from 0.23 to 2.4 Lugeons.

iii) Hole No.3: Permeability test results are given below:-

- a) 0.20 m – 3.09 m = Pressure could not be developed. Intake water was coming out from all round the hole indicating highly permeable formation.
- b) Between 3.24m and 31.38m permeability values ranges between 1.1 and 4.2 lugeon.

The water loss in the hole is almost negligible. In test section between 0.2 m and 3.09 m, water pressure could not be developed as intake water was coming out on surface through interconnecting open joints.

iv) Hole No. 5: Permeability test result are given below:-

- a) 0.15m-3.04m and 3.19m -6.09m=No pressure could be developed.

Permeability varies between 2-4.5 Lugeon. Water Loss test data indicates that the joints are incipient and was tight between 5.23 m and 24.48m depth. No pressure was developed between 0.15 m and 5.23 m due to interconnected open joints which allowed unrestricted percolation.

v) Hole No.8: Water Loss Test data is given below:-

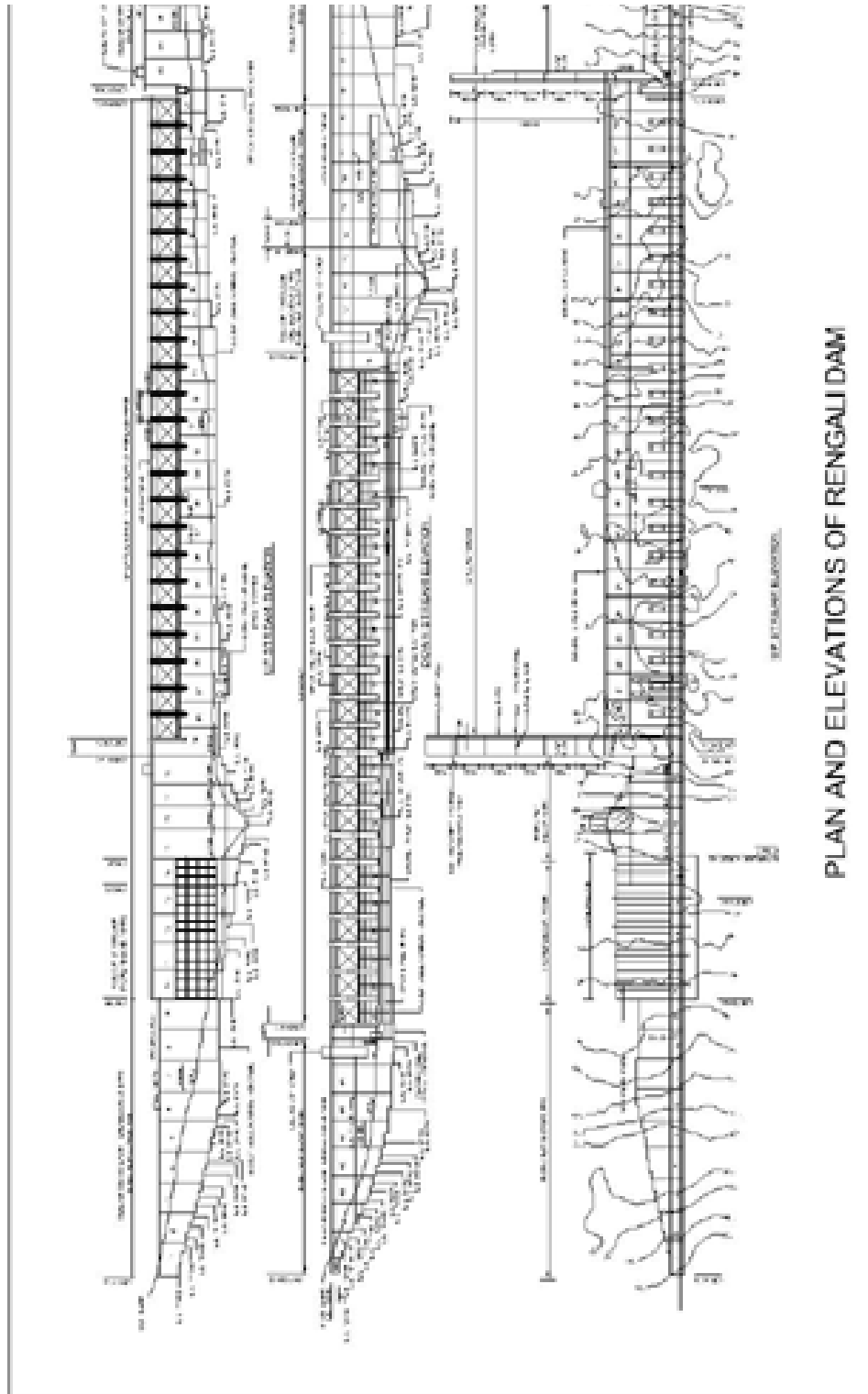
- a) 7.79-10.68m=6.5-1.5 lugeon (A zone of low permeability with higher pressure permeability value diminish).

In general, with increasing pressure, water loss decreases which indicates that water pressure test were carried out before obtaining full saturation. Moderate permeability (1.5-8 lugeon) is at place attributed to presence of close spaced joints.

vi) Hole No.13: Water Loss Test data are given below:-

- a) 0-2.89 m=10-14 lugeon (Heavy water loss. The zone is highly jointed and highly permeable)
- b) 2.89-5.79 m = 0.15-40 lugeon (High permeability value is probably due to loosening of packer)
- c) Below 5.79m, permeability is negative, indicating impervious horizon.

Negligible water loss has been recorded between 5.96 m and 24.56 m depth indicating water tight Zone. Between 2.89 m and 5.79 m depth, high permeability is recorded where no joints have been noted. High permeability value can be attributed to loosening of packer. High water loss between 0 m and 2.89 m can be explained by presence of clay filled joints and another basal sub horizontal joint with limonite stain.” (Source: Ibid Pg.14-15).



2.1.4. Hydrological Study

2.1.4.1 Availability of Data:

Out of three feasible sites (i.e., Lodani, Barkote and Rengali), when final decision was taken to construct dam at Rengali to intercept maximum catchment for flood control, study on hydrology was initiated as its feasibility depends on this most important parameter. Rainfall in the Brahmani basin is maximum in the delta and progressively gets reduced towards the plateau. Annual rainfall varies from 1500 mm in the delta to 1000 mm in the plateau. The mean maximum and minimum rainfall in the catchment above Rengali dam site have been worked out to be as follows:

Mean annual rainfall - 1570 mm

Maximum annual rainfall - 2850 mm

Minimum annual rainfall - 890 mm

Rainfall in the basin is mainly caused through the monsoon currents from the Bay of Bengal. Normal monsoon activity is generally due to the convergence of the summer low. Heavy rains are the results of either low pressure or depressions and sometimes due to severe cyclonic storms. As yet, there is no recorded event of a land low producing heavy rainfall in the Brahmani, but such a situation can not be ruled out since severe cyclones due to land lows are frequent in the adjacent Damodar basin. The storm tracks enter the land area between 19° to 24° N and 81° to 88° E as revealed from Prof. P.C. Mahalanobis article "Rain Storms and River Floods in Orissa" published in journal 'Sankhya' through a map. The map shows that the main storm tracks appear to be confined within a comparatively narrow strip in the river basin. It is noticed that the main storm track lies in the close proximity to the Brahmani basin and therefore, it gets the heaviest rainfall.

However at the time of first hydrological study sufficient hydrological data were not available. Systematic observation of gauge and discharge commenced at Bajrakot 8 km downstream of Rengali project site in the year 1958. Discharge data was available from 1929 at Talcher which is 64 km downstream. Peak gauge readings were available at Jenapur railway bridge site since 1875 which is 173 km downstream of the dam site. This wide range of data covering 97 years of record very much helped in arriving at the statistical floods of different return periods. Data available at relevant gauge sites of the river is furnished in Table-2.1. Detailed information of Rain gauge stations of Lower Brahmani Basin is given in Table 2.2.

Yield series were computed from the stage-discharge curves of Talcher and Bajrakote and were extended to dam site with suitable method of reconstruction. The stage-Discharge curve at Rengali Dam site is shown in Drg. No-2.3(a). In total, yield series have been calculated for 32 years up to 1971-72 which gives average annual value of 14.80 lakh ha.m with run-off value of 586 mm (23.10") over the entire catchment area. Gauge correlation of Jenapur Railway Bridge & Endolva with Talcher is furnished vide Drg. No.2.3(b)

2.1.4.2 Design Flood: As 97 years' peak gauge data were available at Talcher gauge site ($C.A.29800 \text{ km}^2$) unit hydrograph method was not considered reliable for such big catchment. Probability method was considered suitable as data for reasonably long period were available.

The frequency study was therefore made at talcher gauge site by various methods indicated below. The results so obtained at Talcher site was extended to dam site suitably with flood ratio of 0.8835)

- i) Gumbel's method
- ii) Log normal method
- iii) Foster-I method
- iv) Foster-III method
- v) Hazen's method

The results obtained at Talcher site by Gumbel and Log normal methods closely tallied with each other and thus Gumbel's method was finally accepted. The comparison of results by various methods and summary of results of Frequency Analysis are given in Table-2.3 & 2.4 respectively.

The Project Report was submitted to Central Water Commission (CWC) with design flood of 27500 cumecs with hydrograph shape derived by basic hydrograph method i.e. by multiplying the observed flood hydrograph ordinates in the ratio of designed flood peak and observed flood peak. The design flood value according to this method worked out to be 226 mm of run-off (8.90") on a 6.5 days base period.

Frequency analysis was also done at Central Water Commission (CWC) on a digital computer. The expected peak values are as follows:

Return period in years	Peak at Talcher in cumec	Peak at Dam site in cumec
1 in 1000	32740	28900
1 in 100	24380	21540
1 in 50	21890	19340

The design flood peak at this stage was considered to be 28900 cumec. For deciding the shape of the design flood hydrograph, dimensionless average flood hydrograph method was adopted. Firstly, a relationship was developed between observed flood peaks and its volumes. From this peak-volume relationships the volume for the design flood peak of 28900 cumec comes out to 5.55 ham.

Table-2.1 Data available at Relevant Gauge Sites of river Brahmani.

Sl. No.	Name of observation station	Location	Period for which data was available.	Nature of available data	Remarks
1	Bajrakot	8 kms d/s of Rengali dam site	1958 to 1966	a) Non-monsoon period- once daily gauge observation. b) Monsoon period-six hourly gauge observation	
2	Talcher	64 kms d/s of Rengali dam site	1929 to 1971	a) Non-monsoon period. No observation b) Monsoon period- six hourly gauge observation and during high floods hourly gauge observation from 1964	No data for 1948-58
3	Endolva	160 kms d/s of Regnlai dam site.	1875 to 1971	a) Non-monsoon period. No observation b) Monsoon period- six hourly gauge observation.	During 1948 to 1958 only peak gauge readings are available.
4	Jenapur railway Bridge.	173 kms. d/s of Rengali dam site.	1960 to 1975	a) Non-monsoon period- Daily gauge and discharge observations. b) Monsoon period-six hourly gauge reading and during high flood hours gauge reading.	
5	Rengali Dam site	At pump house site.	1972 To 1982	Daily gauge and discharge, with hourly gauge reading during flood.	

Table No-2.2 Information on Lower Brahmani Basin							
<u>Sl No</u>	<u>BASIN CODE</u>	<u>R.G. STATION</u>	<u>DISTRICT</u>	<u>BLOCK</u>	<u>SUB - CATCHMENT NAME</u>	<u>CATCHMENT AREA (Sq.km)</u>	<u>STREAM NAME</u>
1	4b1	Batagaon, Chhendipada, Jamankira, Naktideul, Rairakhol, Remal, Rengali	Anugul, Deogarh, Sambalpur	Chendipada, Jamankira, Jujumara, Koshorenagar, Naktideul, Rairakhol, Reamal, Tileibani	Tikira	1457.3	Tikira N
2	4b2	Ampur, Batagaon, Chhendipada, R. K. Nagar, Rairakhol, Remal, Rengali, Talcher	Anugul, Deogarh, Sambalpur	Chendipada, Kanhia, Koshorenagar, Naktideul, Pallahara, Rairakhol, Reamal	Gandanala	943.3	Gandanala River
3	4b3	Altuma, Pallahara, Rengali, Talcher, Telkoi	Anugul, Dhenkanal, Keonjhar	Kanhia, Kankadahad, Pallahara, Parjang, Telkoi	Kharsua	1502.8	Kharsua River
4	4b4	Ghatagaon, Keonjhar, Pallahara, Telkoi	Anugul, Keonjhar	Banspal, Ghatgaon, Harichandanpur, Pallahara, Telkoi	Genguti	1121.9	Genguti River
5	4b5	Altuma, Daitari, Kamakshya Nagar, Telkoi	Dhenkanal, Jajpur, Keonjhar	Harichandanpur, Kankadahad, Parjang, Sukinda, Telkoi	Barajore	1207.1	Barajore N.
6	4b6	Altuma, Dhenkanal, Kamakshya Nagar, Talcher, Telkoi	Anugul, Dhenkanal, Keonjhar	Kamakshyanagar, Kanhia, Kankadahad, Odapada, Parjang, Talcher, Telkoi	Samakoi	762.3	Samakoi River
7	4b7	Altuma, Angul, Chhendipada, Rengali, Talcher	Anugul, Dhenkanal	Angul, Banarpal, Chhendipada, Kanhia, Koshorenagar, Odapada, Parjang, Talcher	Ramiala	1064.4	Ramiala River
8	4b8	Altuma, Angul, Ampur, Barmul, Chhendipada, Hindol, Narsinghpur, Talcher, Tikarpara	Anugul, Cuttack, Dhenkanal	Angul, Athamallik, Banarpal, Chhendipada, Hindol, Koshorenagar, Narsinghpur, Odapada, Parjang	Ramiala	787.5	Ramiala River
9	4b9	Altuma, Cuttack, Dhenkanal, Hindol, Jenapur, Kamakshya Nagar, Sukinda	Cuttack, Dhenkanal, Jajpur	Bhuban, Dhamasala, Dhenkanal, Gandia, Hindol, Kamakshyanagar, Narsinghpur, Odapada, Parjang, Sukinda	Samakoi	692.4	Samakoi River
10	4b10	Altuma, Daitari, Dhenkanal, Kamakshya Nagar, Sukinda	Dhenkanal, Jajpur, Keonjhar	Bhuban, Danagadi, Dhenkanal, Gandia, Harichandanpur, Kamakshyanagar, Kankadahad, Odapada, Parjang, Sukinda	Samakoi	716.2	Samakoi River
11	4b11	Akhuapada, Anandapur, Daitari, Jajpur, Jenapur, Sukinda	Dhenkanal, Jajpur, Keonjhar	Bhuban, Danagadi, Dharmasala, Gandia, Ghasipura, Jajpur, Korei, Rasulpur, Sukinda	Ningara	1257.8	Ningara River
12	4b12	Cuttack, Jajpur, Jenapur, Kendrapara	Cuttack, Dhenkanal, Jajpur, Kendrapada	Badchana, Cuttack Sadar, Danagadi, Derabisi, Dharmasala, Gandia, Mahanga, Rasulpur, Salipur, Sukinda, Tangi Chowdwar	Singhada	914.0	Singhada
13	4b13	Akhuapada, Chandbali, Jajpur, Jenapur, Kendrapara, Paradeep, Patamundai, Rajkanika	Jajpur, Kendrapada,	Aul, Badchana, Bari, Binjharpur, Danagadi, Derabisi, Dharmasala, Jajpur, Kendrapara, Korei, Mahakalapara, Pattamundai, Rajkanika, Rajnagar, Rasulpur	Aunli	1343.3	Aunli N

N.B.- Rain gauge stations are shown in the map enclosed (Drg. No.2.2)

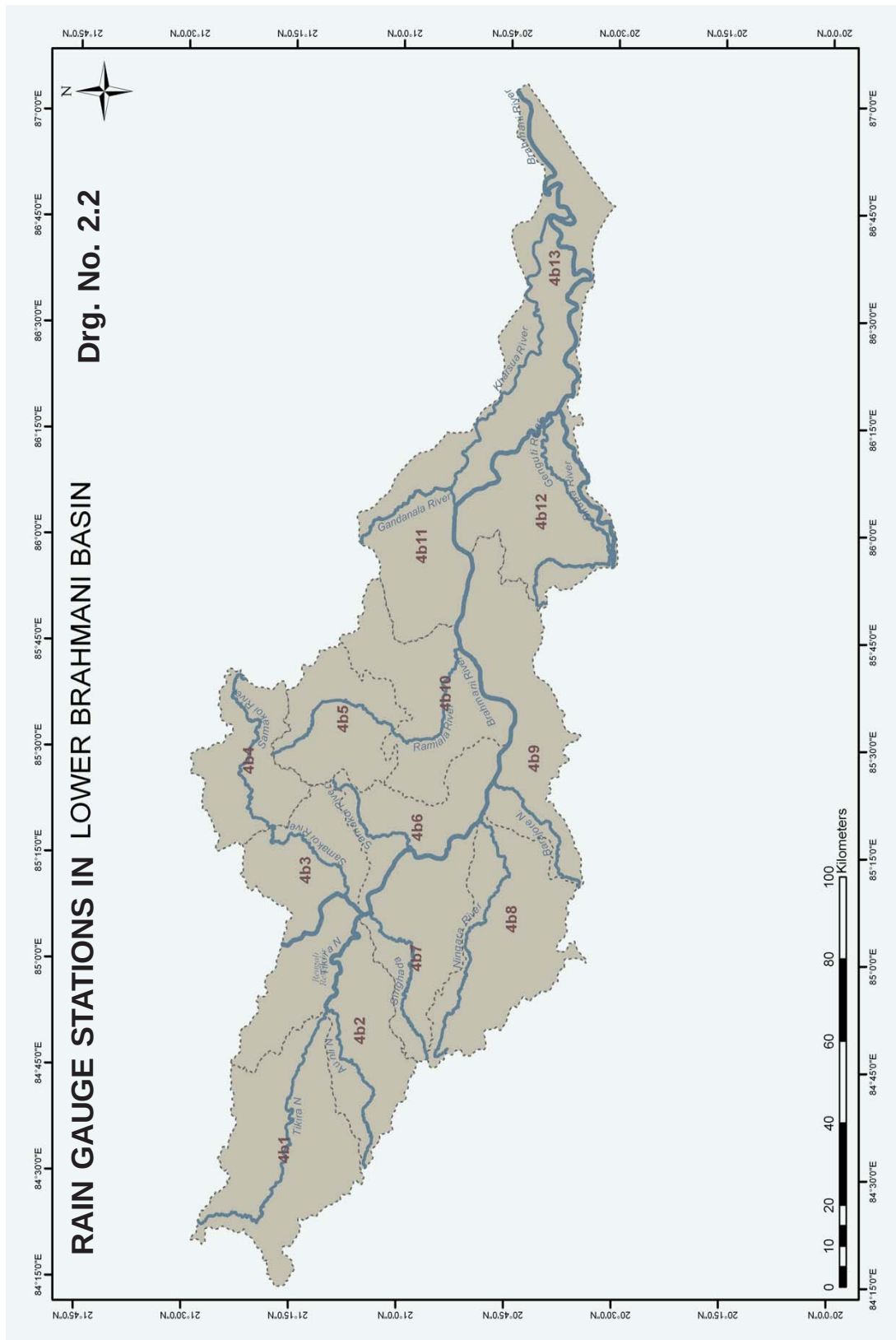


Table-2.3 Comparison of Results by various methods at Talcher Site

Methods	Flood Magnitude (in lac cusec) for various return periods in years						
	5000 (0.02%)	2000 (0.05%)	1000 (0.10%)	500 (0.20%)	200 (0.50%)	100 (1.00%)	50 (2.00%)
Gumbel	12.35	11.83	10.98	10.16	9.04	8.18	7.33
Log Normal	14.09	12.98	11.10	10.77	9.75	8.03	7.51
Foster I	8.63	8.44	8.17	8.03	7.70	7.11	6.83
Foster III	10.64	10.09	9.19	8.97	8.37	7.37	6.99
Hazen	11.12	10.47	9.42	9.19	8.52	7.43	7.05

(Source: Rengali Dam Project, Stage-I, Vol.-I, Part-B, General Report, July 1972)

Table-2.4- Summary of Results of Frequency Analysis

Return period in years	Discharge in Cumec	
	Talcher Site	Rengali Site
1 in 1000	31,150	27,500
1 in 500	28,860	25,500
1 in 100	23,310	20,600
1 in 50	20,930	18,490
World Envelope Curve	31,150	29,170
Indian Envelope Curve	28,320	25,000

2.1.4.3 Dimensionless Average Flood Hydrograph:

A dimensionless average flood hydrograph at Bjrakote was developed by computing the dimensionless flood hydrographs for various floods and averaging them. The peak and base of all hydrographs were taken as units. After plotting all the selected hydrographs and superimposing peak to peak the average hydrograph was drawn by averaging the time axis. The volume of the average non-dimensional flood hydrograph was found to be 0.34 (vide Drg. No-2.5)

The base period of any hydrograph will thus be volume in cumec days/peak in comeecs x 0.34. The base period for 1000 year return period flood works out to be 6.55 days. The complete shape of the hydrograph was then worked out by dimensionless average flood hydrograph. Total volume works out to 5.55 lakh ham over 6.55 days which give run-off of 219 mm (8.62"). This closely agrees with the result obtained by basic Hydrograph method i.e. run-off of 226 mm.

C.W.C. cleared this project with stipulation that the final design should be based on maximum probable flood (MPF). They clearly mentioned that although the catchment area at dam site is large still it may be possible to find storms covering the entire basin.

Finally unit hydrograph method of storm transposition was adopted to find the M.P.F. The study was entirely conducted at C.W.C. Some hydrographs were selected arising out of isolated storms and a number of unit hydrographs derived from them and the final one developed by averaging them. The following 3 day storm has been considered for M.P.F.

1 day	214.63 mm	8.45"
2 day	309.88 mm	12.20"
3 day	347.47 mm	13.68"

Depth duration with 25% maximisation and Rainfall increments were found out. Then rainfall increments in critical order were arranged (vide Table-2.5) The effective rainfall was superimposed on the unit hydrograph with proper lag to get the MPF. Base flow adopted was 0.6×10^5 cusec (1697 cumec)

The MPF thus calculated is 19.80×10^5 cusec or 55,540 cumec. This flood peak was taken for design of spillway of Rengali Dam vide Design Flood hydrograph, Drg. No-2.4). Detail flood routing was done and the outflow from the reservoir after routing is 16.59×10^5 cusec (46,970 cumec)

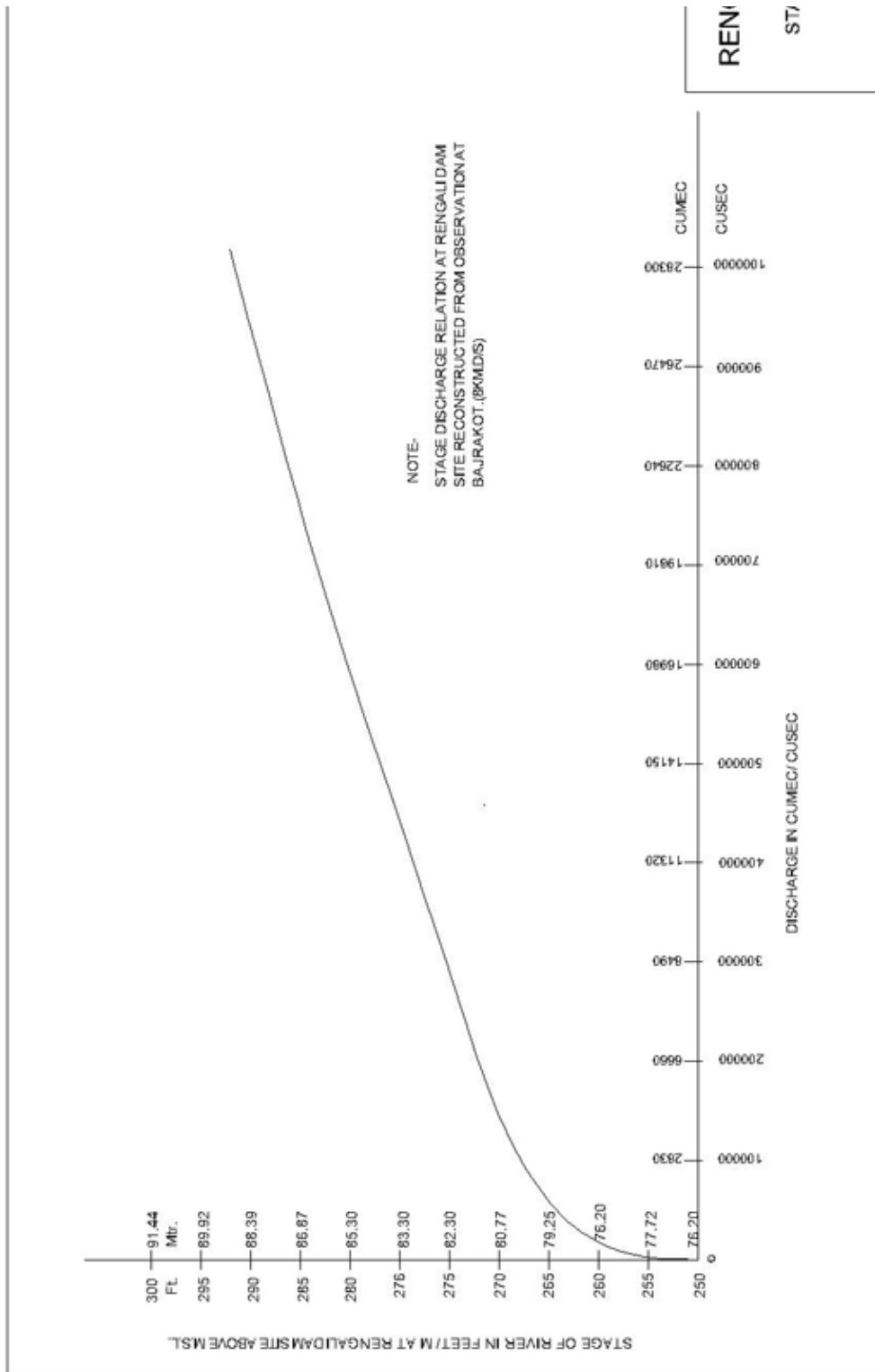
Model studies with a 1:100 composite model were conducted at C.W.P.R.S., Pune for above M.P.F. and the spillway with 24 gates of size 15.5m x 14.8 m each was found to be satisfactory.

Table-2.5 Rainfall increments in critical order

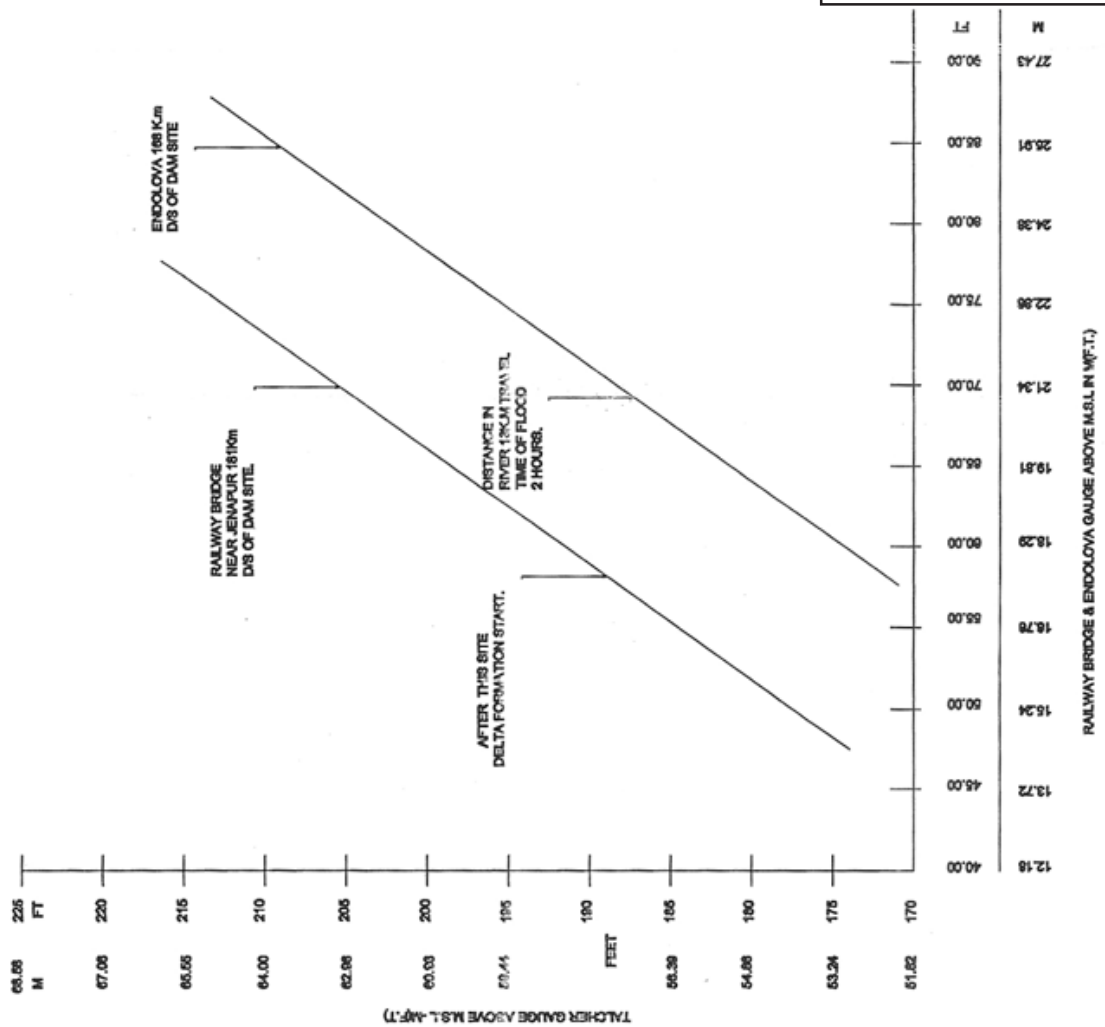
Time (hrs)	U.H.G. ordinates 10^5 (cusec)	Effective rainfall arranged peak to peak	Effective rainfall in critical order
0	-	-	-
6	0.09	-	0.20
12	0.120	-	0.50
18	0.168	-	0.60
24	0.255	0.60	0.90
30	0.405	1.20	1.00
36	1.095	1.50	1.40
42	1.770	1.80	1.60
48	2.100	5.80	5.80
54	1.690	1.60	1.80
60	0.540	1.40	1.50
66	0.480	1.00	1.20
72	0.360	0.90	0.60
78	0.300	0.60	
84	0.240	0.50	
90	0.210	0.20	
96	0.195		
102	0.105		
108	0.120		
114	0		



View of dam from top and downstream

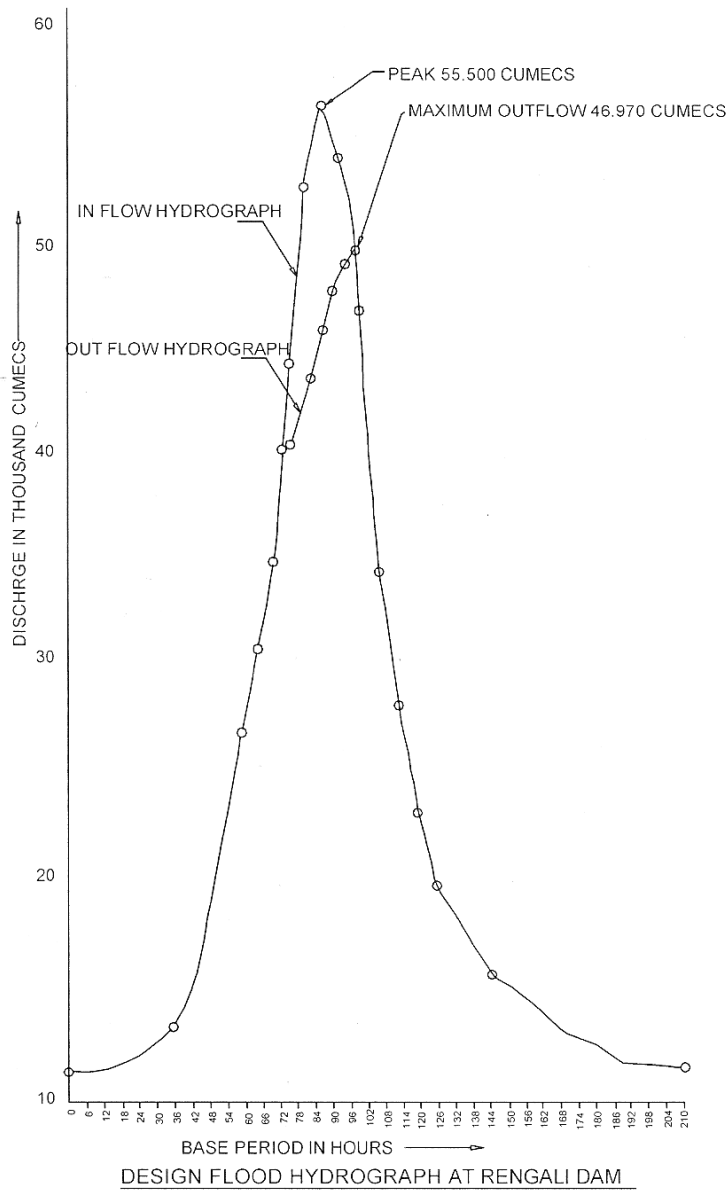


Drg. No.2.3(b)



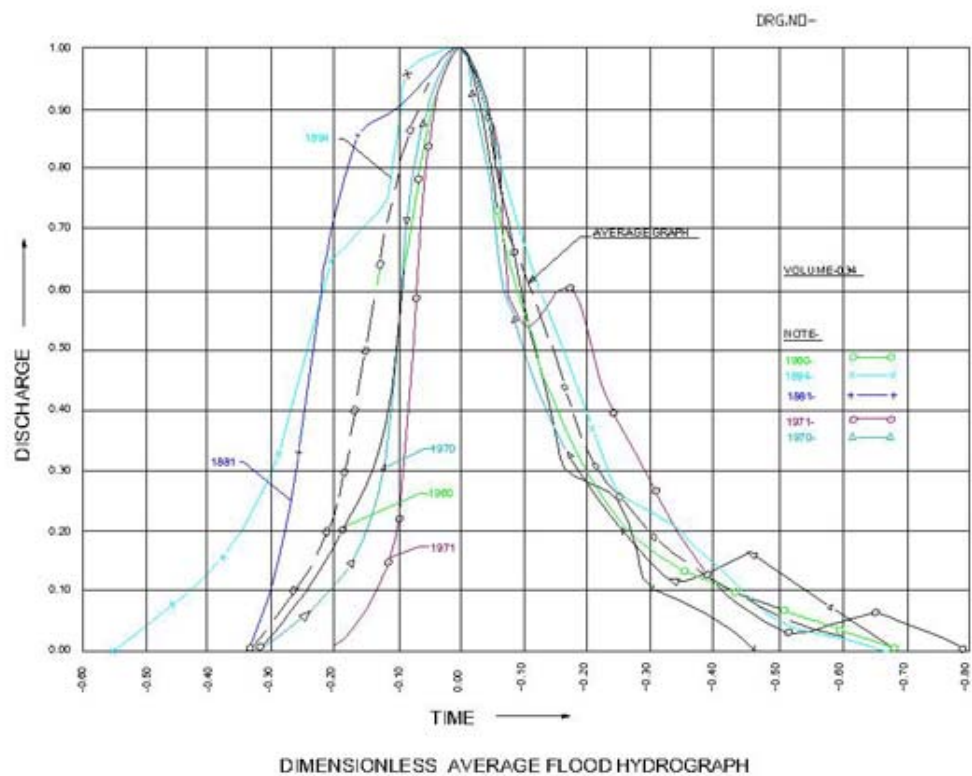
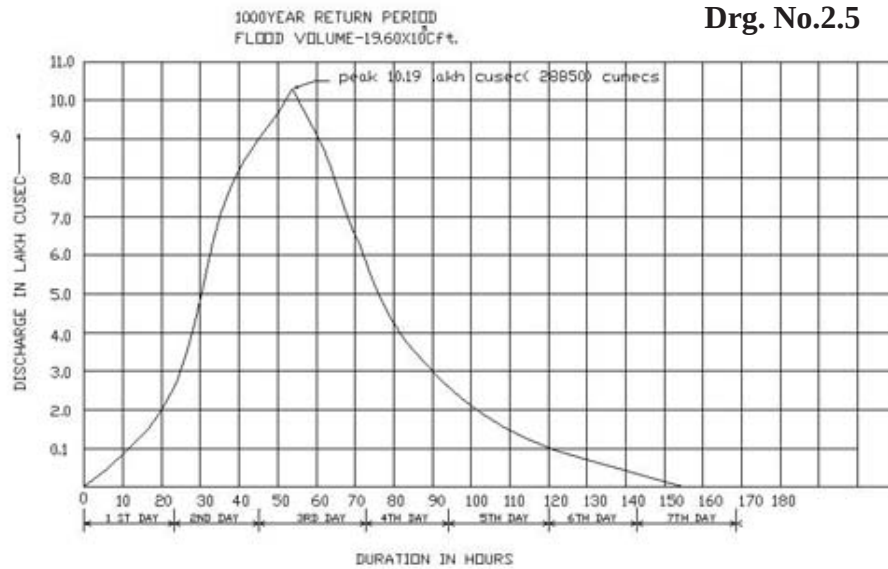
RENGALI DAM PROJECT

GAUGE CORRELATION OF
JENAPUR RAILWAY BRIDGE
AND ENDOLVA GAUGE
AGAINST GAUGE AT
TALCHER

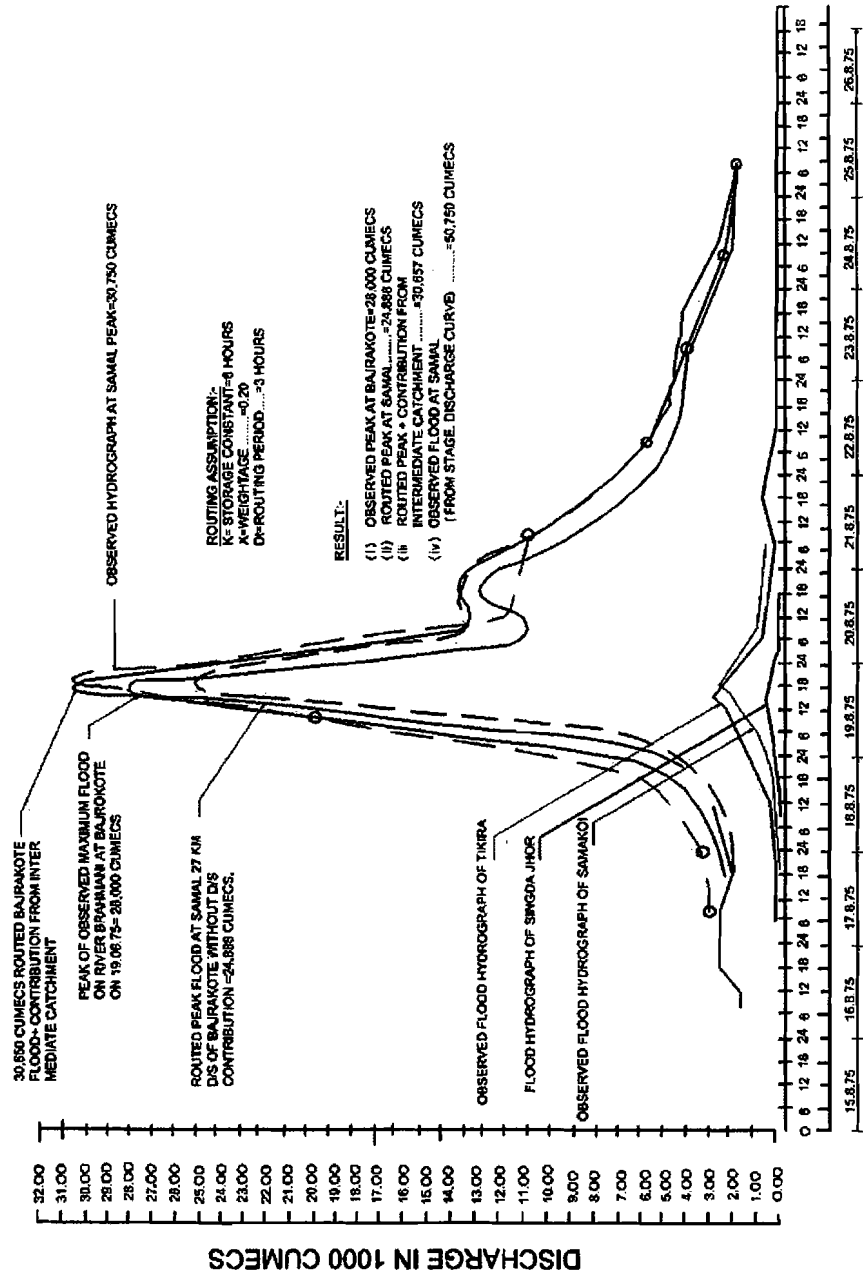


ORDINATES OF DESIGN HYDROGRAPH (6 HOURS INTERVAL)	
HOURS	DISCHARGE IN 1000 CUMECS
0	1.6
6	1.7
11	1.9
12	2.4
24	2.4
30	3.0
36	4.2
42	6.4
48	10.8
54	15.7
60	20.5
66	25.2
72	30.0
78	40.92
84	50.70
90	55.50
96	52.38
102	43.90
108	29.40
114	21.90
120	15.90
126	12.00
132	10.25
138	9.00
144	7.00
150	6.30
156	5.40
162	4.50
168	3.90
174	3.30
180	3.05
186	2.30
192	2.10
198	1.80
204	1.75
210	1.70

Drg. No.2.5



DRG NO. 2.6



CHANNEL ROUTING BETWEEN BAJRAKOTE AND SAMAL BY MUSKINGHAM METHOD

2.2 Selection of Barrage Site

2.2.1. Alternate Site Surveys: Altogether five possible sites have been perobed for selecting a suitable site for construction of the Barrage. These sites are (1) Talpada (2) Durgapur (3) Samal (4) Belapada and (5) Dharampur. All these five sites were considered suitable during the reconnaissance survey and hence detail investigations were carried out for each site. The typical features of each site with its merits and demerits are discussed below:

2.2.1.1 Talapada Site: This site was 21 kms downstream of the Rengali Dam. The river Brahmani flows through a gorge at this place. There are two necks of the hills on either side of the river, protruding almost into the riverbed. Granite rock exposures are prominent at the site on both the sides as well as on the river bed. The site was considered promising for construction of the barrage right on rock foundation.

Geophysical Investigations were carried out during the year 1974. As revealed, the rock depth in the river bed varies from 10 m to 30 m which indicates the existence of a fault. To confirm this, a drill hole was put in the middle of the river bed, and this indicated that sound rock is available at R.L. 40.421 m i.e. 20 m. from the deepest bed level. Thus there was no chance of constructing the barrage on rock foundation. Moreover, the disadvantages for this site would have been (i) The pond capacity between R.L. 75.85 and 76.2 m is only 660 Ham. which is much less than that for Samal site. (ii) The dead length of the canal would be more without any advantage. (iii) The canals on either side would have to negotiate two hill ranges which is difficult and costly. (iv) The canals will have additional large C.D. works over rivers Tikra and Singada on right and over the river Samakoi on the left, without any advantage. (5) The command area would have been less. Hence the site was rejected.

2.2.1.2. Durgapur Site: This site is located at 26 kms. down stream of the Rengali Dam. The existing width of the river is 490 m. Sound rock exposures were very prominent on both sides. It was initially anticipated that sound rock will be available at shallow depths. Geophysical investigations carried out at the site remained confined to the sand depth of the river, which revealed that hard rock may be available at 10 m. to 30 m. depth below the bed level.

Two nos. of drill holes were put to explore the rock stratum. One hole revealed that the rock level is at R.L. 32.5 m with only 17% recovery and the other hole gave 51% recovery at R.L. 18.3 m. As the foundation rocks were not available at shallow depths, the site had no other advantage and rather the disadvantages are same as that of Talapada side. Hence the site was rejected.

2.2.1.3 Belapada Site: From preliminary reconnaissance, Belapada site was also found to be suitable. This site is 42 kms downstream of Rengali Dam. The bed rock exposure were prominent on the right side bed both in upstream and down stream sides. Initial probing indicated the availability of sound rock at shallow depths. Six nos. of wash borings were done which showed that hard rock will be available between R.L. 53 m. to 57 m. i.e. at a depth of about 8 m. from the average river bed level.

Diamond drilling was done at RD 203 m. of the axis, measured from the right bank. The rock level was met at RL 51.575 m. and the recovery was only 4% at RL 31.495 m, which. That concluded that the bed rock is very poor in strength and rigidity, and hence only a structure on permeable foundation could be adopted. In order to irrigate the high lands the pond level being fixed the height of structure is considered very high for permeable foundation. The design of such a high head structure on permeable foundation would be much complicated and uneconomical too.

Although the effective pondage between RL 75.85m. and 76.2 m. is 1100 ham. which is quite sufficient, the sub-mergence is 4800 hectares and is considered high. Forty one nos. of villages will be submerged for which the cost of submergence and resettlement will be high. Since the site does not have any advantage, the site was rejected.

2.2.1.4 Dharampur Site: This site is situated at a distance of 50 kms. down stream of Rengali Dam. This site was investigated earlier for water supply to proposed Super Thermal Station near Talcher during the year 1962. Exploration of the site revealed the existence of sand and gravel zones to a depth of 11.35 meters. Below this zone alternate layers of carbonaceous sand stone and shale occur. Probing has been done upto a depth of 26 m. below the river bed. Geologists have mapped the area and have recommended to take the foundation of the Barrage upto sand stone level which occur at a depth of 15 to 16 m below the river bed level, if a low dam on rock foundation is to be done. But further exploration revealed that even this sand stone layer is not sufficiently thick to found the barrage on it. Hence the alternative is to go for a structure on permeable foundation. In such a case there is no additional advantage of the site, considering the feasibility and economic aspects of such a high head structure on permeable foundation the site was not found promising.

The pond capacity is only 950 Ham. between RL 75.85m. and RL 76.2m. and the submergence is very high. As per the geological report of G.S.I. during 1962, drilling reveals that there exists a coal seam of 1.22 m. at a depth of 6.70 m. from the river bed and Gondwana basin starts below Samal and covers the Dharampur area upto Talcher coal field. The geologist has also advised to take up deep drilling for establishing the coal seams in Dharampur site.

Further the area of submergence is extensive and the pond submerges 47 No. of villages which are populous. Cost of submergence and rehabilitation would be very significant.

Height of barrage will increase in comparision to Samal inorder to maintain the same pond level. The pond level will have to be maintained at RL 74.7 m. so that the optimum command level will not be affected. The design of such high head structure on permeable foundation would lead to many complicacies. Hence, the site is rejected.

2.2.1.5 Samal Site: The proposed Barrage at samal is about 35 kms. downstream of Rengali Dam. From preliminary reconnaissance, the site was selected for detail investigation. There are good rock exposures at the site. But on foundation drilling the hard rock strata was met at 11 to 15 m, below the bed level of the river.

Seismic reflection survey indicated the nature of rock to be hard and compact, showing a longitudinal velocity of 540 m/sec. Geophysical investigations corroborated the results.

The drill hole data revealed that sound rock is available at RL 35 m. to 55 m. with percentage recovery of 93% to 100%. The residential geologist of G.S.I. stationed at Rengali Dam Project inspected the site and the recovery samples. The barrage has been designed on permeable foundations from economic and other considerations.

The effective storage between R.L. 75.85 m. and R.L. 76.2 m is 2250 ham. which is sufficient to allow full discharge into the canals for 6 hours even if upstream release is shut down. The total pond area is 2880 hectares and only 252 houses are affected with a total population of 2,900. The Mining and Geology Department of Government of Odisha have cleared the Samal pond area without any objection vide Annexure-IV.

However, the alternate cost of the barrages at Samal, Dharampur, and Belpara was worked out and the total project cost is compared in which it can be seen that cost of the project is least with Barrage at Samal.

From all points of view, Samal site was found to be the most suitable and hence this site was finally selected to locate the Barrage, and all the other 4 sites were finally discarded in favour of Samal site.

2.2.2. Geological Investigation of Barrage site:

2.2.2.1 General: A 540 meter long and 13 meter high “pick up” barrage is proposed across river Brahmani, near Samal (21°-04’-30” : 85°-08’-30”; 73G/4) village, in Dhenkanal district, Orissa which would irrigated about 4,04,600 hectares of dry land. Preliminary geological investigations were undertaken and an area of 128 sq.km. around the barrage site was geologically mapped on 1:1000 scale. A total of 69.87 m. of drill cores were analysed and reviewed.

The area around Samal Barrage site consists of Archaean granite gneiss and felspathic gneiss, intruded by occasional sills of dolerite. The regional foliation of these rocks show a general WNW-ESE to E-W strike with steep dips to south. The rocks are closely jointed. Most of the joints are continuous from upstream to downstream. The right abutment and a part of the river bed between R.D 460 and R.D.530 only show rock outcrops. The rest of the river bed including the left abutment from R.D 530 to R.D.1000 do not show any rock outcrop, but covered by a veneer of sand. The three holes drilled along the rock is available at an average depth of 10.50 m, under sand cover. Preliminary examinations revealed that the Archaean granite gneissic complex will serve as a competent foundation rock and no major weak/fault zone is expected in the foundation. Due to the presence of close joints and joints having upstream to downstream connection, it is recommended that the 1/3rd of the barrage base should be grouted properly prior to concreting. Curtain grouting is also necessary to provide an impervious curtain for the barrage foundation on its upstream side. Considering all geotechnical aspects the construction of the proposed barrage at Samal is feasible. (Source: Geological Report of G.S.I. May- July, 1975)

2.2.2.2 Local Geology and Site Condition

2.2.2.2.1 Physiography:

The area can be divided broadly into two distinct geomorphic units like (i) undulated terrain with occasional isolated / cluster of hills representing Archaean metamorphites, and (ii) comparatively flat topography with occasional flat topped ridges comprising boulder beds and rocks of Talcher series of Gondwana system. xxxxx The Brahmani is a mature river and has reached its base level of erosion. In the river course itself few knolls of granitic rocks can be seen upstream of Samal barrage site. xxxx

The western (right side) segment of the Brahmani river has steeper gradient than the eastern (left side) which is clearly reflected in its drainage pattern in the sense that more tributaries join the right side than the left and Tikra and Singada nalas are the two main tributaries and join the right bank of Brahmani river from the west. The course of the Brahmani river, in this area, is partly controlled by the regional tectonic trend of the under lying Archaean and Gondwana rocks which show a general WNW-ESE to E-W strike and partly by some regional lineaments which caused the Gondwana troughs.

2.2.2.2.2 Local Geology: The major part of the project area is composed of Archaean metamorphites like granite gneiss, felspathic gneiss, charnockite, mica and hornblende schist, quartzite, metadolerites and pegmetite veins etc. These show a general trend of foliation strike varying from E-W to WNW-ESE and dip at high angles usually to south. Near Samal the trend of the gneiss and metadolerite vary from N 85° W-S 85° E to N 85° E-S 85° W with steep south to sub-vertical dips. The linear quartzite hill 4 km south east of Samal village shows low 25°-30° dip to NE.

2.2.2.2.3 Site Condition: At the proposed barrage site, the width of the Brahmani river is about 490 meters. The left bank is comparatively flatter and has a slope gradient of 1:1 whereas, the right bank shows a 2:1 slope gradient. The higher slopes of the right bank between R.D.430 to 460 m is occupied by older alluvium of reddish, hard, clayey materials with calcareous nodules. The rest of the right bank as well as a part of the river bed between R.D. 460 to 530 m show rock outcrops. The rest of the barrage axis, i.e. between RD 530 to 940 m which includes the major part of the river bed do not show any rock outcrop and is covered by a veneer of riverine sand only. However the left bank between R.D. 940 and 1000 m is concealed by a thin mantle of older alluvium.

As stated above, the right abutment and a portion of the river bed, between RD 460 m to 530m show almost continuous outcrops of Archaean granite gneiss grading occasionally to felspathic gneiss. The rocks are medium to fine-grained and when fresh these are hard and compact. The mineral assemblages are restricted mainly to orthoclase, plagioclase, quartz, biotite and chlorite etc. showing a gneissos intergranular texture. The granite gneiss suite is intruded occasionally by dolerite sills which are metamorphosed and occasionally altered to hornblende schists. These Dolerite metadolerites have restricted distribution both strikewise and across. The predominant minerals composing these basic bodies are hornblende, plagioclase, sphene, biotite, magnetite, apatite etc. and show sub-ophitic to schistose texture. Few pegmatite bodies are also seen to intrude the granite gneiss along the foliation planes.

The granite gneiss and dolerite, the predominant rock types of the Samal barrage site area show N 85° W-S 85° E to N 85° E-S 85° W foliation strike with X Steep (60°-85°) dips towards south. The variation in the foliation trend is due to minor warps in the regional structural trend. The rocks are closely jointed and the following prominent sets have been observed in the granitic rocks:

Rock Types	Strike	Dip.	Frequency of spacing	Distribution per sq.m.	Remarks.
Granite gneiss.	1) East-West.	65° South	Numerous; 1 to 7 cm apart.	30 m	1 to 3 mm open; stained occasionally filled with quartz.
-do-	2) N30°W-S30°E	70° NE to vertical	9 to 30 cm apart	6 m	Close to hair thin; opens up due to water action.
-do-	3) N15°E-S15°W	87° ENE	1 to 5 cm	10 m	1 to 3 mm open, stained.
-do-	4) N25°E-S25°W	65° NW	15 cm apart	1 to 2m.	1 to 3 mm; stained
-do-	5) N75°W-S75°E	80° N to vertical	12 cm apart	1 to 2m.	Fairly tight, stained.

Of all the joints, the joints parallel to the foliation plane, i.e. E-W dipping 65° south to sub-vertical is the most prominent. It is rather open and also numerous. It has a distribution frequency of 30 per sq.m. and is parallel to the river course. As a matter of fact, almost all the joint sets are found to be continuous from upstream to downstream. Except the joints parallel to the foliation plane, the other joint sets are fairly close and tight though they show iron staining due to percolating water.

The joint sets encountered in the dolerite bodies are described below:

Rock Types	Strike	Dips.	Frequency of spacing	Distribution per sq.m.	Remarks.
Dolerite rocks.	1) N85°E-S85°W to E- W	70° SSE 60° S	35 cm apart	3 m	1 to 3 mm open; stained.
-do-	2) N40°E-S40°W	75° SE	2 to 9 cm apart	7 m	1 to 2 mm; fairly close, almost hair thin. Joint face clear.
-do-	3) N10°W-S10°E	70° NNE	5 to 35 cm	4 m	1 to 2 mm open, almost hair thin. Joint face stained.

All the joint sets are rather tight, though on the surface they are usually opened up due to weathering. These joints are also continuous from upstream to downstream.

2.2.2.2.4 Sub-surface Investigation

Sub-surface exploration by drilling was carried out to determine the sand fill alluvial zone and bed rock and sound rock depths from the surface. As a preliminary probe 3 holes were drilled by the Project Authorities along the proposed barrage axis. Drill hole no.1 at R.D.622.90m. no.2 at R.D.799.00m. and 3rd one at R.D.929.00 m. all in the sand covered river bed. The 4th one was drilled 50m. downstream of R.D.699.00m. A synoptic log of these drill holes are given in Table-2.6.

Table-2.6 Synoptic log of Drill holes:

Drill hole nos.	Total depth drilled in metres.	R.D. (m)	R.L. of surface elevation (m)	R.L. of bed rock. (m)	R.L. of sound rock. (m)	Remarks.
1	2	3	4	5	6	7
1	18.19	622.90	63.950	55.87	55.87	Bed rock and sound rock level are same which is 8.08m. from the surface. Cores of granite gneiss. Hard and tough. Suitable for foundation.
2	20.17	779.00	67.790	56.41	52.98	Bed rock is seen at R.L. 56.41m. i.e. 11.38m. from the surface. The sound rock is met at R.L.52.98m i.e.14.81m. from the surface. Cores of hard, tough granite gneiss.
3	21.41	929.00	68.830	57.00	57.00	Bed rock and fresh rock levels are same which is 11.83m. from the surface i.e. at R.L. 57.00 Cores of granite gneiss. Rock is hard and tough.
4	17.10	699.00 off set 50m. down stream	56.515	53.32	52.32	The bed rock and the fresh rock levels are the same, that is, 12.29m. below the surface i.e. at R.L.53.32m. Cores of hard and tough granite gneiss.

It is observed from the drilling results that the sound rock levels are available at shallow depth, of about 10.50m, from the surface, which incidently is also the scouring depth of the river at this section. The depth for economic foundation was estimated to be about 15m. below the surface at this place. Thus, a competent rock for foundation is available at this place. The details of the logs are given in Annexure-I with detailed remarks.

2.2.2.2.5 Discussion:

It has been observed that the hard and compact Archaean granite gneiss which is available about 10.50m below the sand covered river bed along the proposed barrage axis will serve as a competent foundation for the same structure. Several bodies of dolerite which at places on the right bank is partly metamorphosed and converted into small patches of hornblende schists will pose no special problem as the dolerite bodies themselves are small having limited extension strikewise and across it (width rarely exceeding more than 1m.) If these bodies in their weathered form, are encountered

in the foundation level, these can be removed and properly treated during construction. If these are fresh, these can be retained as they are also very tough rock and the foundation could be a composite one. Thin quartz veins are seen along the foliation planes of the granite gneiss. These veins have a maximum thickness of 3 cm. These can also be removed and treated properly during construction.

No fault/shear/weak zone has been indicated within the barrage foundation either by sub-surface exploration by drilling or by survey conducted by geophysical method. But yet the possibility of continuity of any sympathetic fault/weak zone etc. to the faults which caused the Gondwana troughs occurring upstream and/or along the present river course at the barrage site and remain hidden below the cover of sand and alluvium cannot be ruled out. Particularly, if the fault/weak zone is a close/tight one and passing through the granite body itself, it might not have been possible to pick up by the geophysical survey.

But as the pondage of the barrage will be mainly restricted to the river section and will hardly cross its high banks (RL 75m) here, there will be practically no hydrostatic head of appreciable concern below the earthen dykes. Hence, the problem of leakage etc. along the fault zone, if exists, below the earthen dyke would be of minor significance.

The geophysical survey has indicated that at Samal barrage site the bed rock occurs as a uniform sheet and is available at a shallow depth close to the right bank and deepens away from it, i.e., towards left bank. The three holes drilled along the barrage axis conform the latter part of the conclusion. But the other conclusions arrived at from geophysical survey like that of (i) uniform distribution of the bed rock for the entire river section and (ii) probably no structural discordance exists along the river bed that could not be verified by the drilling from 4 Nos. of vertical holes drilled so far are quite inadequate to cover an expanse of 490m. broad river section. Thus as indicated in the above para, structural discontinuity/weak zone may be expected in the river bed and if encountered should have to be treated as required.

2.2.2.2.6 Conclusions and Recommendations:

The Archaean granite gneiss, felspathic gneiss with occasional sills of diorite composed the country rocks on which the barrage is proposed to be founded. These rocks when fresh are hard and tough and have sufficient bearing strength to take the load of the envisaged structure. Thus, no special problem is envisaged for founding the barrage as these would serve as a competent foundation material.

The regional foliation of the granitic and dioritic rocks show WNW-ESE to E-W strike with steep to sub-vertical dips to south. No major structural discordance is seen along the river course during geological examination of the site. No fault/weak zone is also indicated within the barrage foundation either by drilling or by geophysical survey. If however, after opening the foundation, similar zones are found for reason stated earlier, they would have to be treated with special care depending on their nature and dimensions.

Along the proposed barrage axis the sound rocks are available at a shallow economic depth which averages to be around 10.50m. from the actual surface elevation. Thus, the foundation can be easily opened up by removing the overlying sand cover and the structure can be directly founded on the sound rock. The bed rock profile and the sound rock profile, as observed in this part of the river section are mostly same.

The water percolation tests undertaken indicate that the foundation is impervious to semipervious. But almost all the joints of the granitic and dioritic rocks are continuous from upstream to downstream which could certainly help leakage from the reservoir. Thus, it is recommended that the consolidation grouting should be strictly adhered to covering 1/3rd of the barrage base from the hill portion, i.e., the upstream portion. While laying the lines for consolidation grouting, the holes may be put in staggered fashion.

It is recommended that routine curtain grouting may be carried out to provide an impervious curtain on the upstream side of the barrage foundation. While doing this, the holes should be deviated between 10° to 15° from the vertical towards upstream.

A quarry site has been tentatively selected at a place about 3 km. WSW of Samal Barrage site. Here a cluster of fairly large size dome shaped outcrops of biotite-hornblende granite gneisses have been located. The rock are quite hard and can be used as masonry materials. The materials can be collected by open quarrying method. Sand is available in plenty in the river bed right at the barrage site.

Considering all geotechnical aspects it is viewed that the construction of the proposed barrage at Samal is feasible. Plan and Elevation of Samal Barrage is enclosed vide Drg. No. 2.7 and Map of Rengali Irrigation Project in Drg. No.2.8.

2.2.2.2.7 Seismicity:

There is no record of earthquake of any significance in this area in the past. Neither, there is any seismological observatory in this region to record earthquake shocks. As per the I.S.I. Seismic zone map, the area falls in Zone II where seismic intensity range upto VI in M.M. scale, Corresponding to this, the seismic acceleration generated range upto 5.175 cm/sec. The Samal barrage site being composed of hard granite gneiss, provision of horizontal seismic acceleration of 5% is thus considered suitable and adequate for incorporating as a seismic factor in design. The vertical acceleration, normally is half of the normal horizontal acceleration. The seismic refraction results at Samal Barrage site is given in Table-2.7.

Table-2.7 Seismic Refraction Results

Longitudinal velocities and depths of different interfaces determined by Seismic Refraction Survey in Samal Barrage site are as follows:

Record No.	Shot plate No.	Traverse No.	Va	Velocity in m/s		Depth in meter		Bedrock elevation from M.S.L
				V ₁	V ₂	H ₁	H ₂	
2	1	T(W-E)	6000	2200	5400	2.34	11.98	53.07m.
3	1	TR(E-W)	600	2200	5400	3.12	11.45	
4,60,61	2	T(S-W)	500	1900	5400	3.16	13.00	
7	2	T(W-E)	500	2200	5400	4.36	14.29	56.09
6,59	2	T(S-W)	200	1000	5400	1.53	12.30	
8	2	TE(E-W)	500	2100	5400	4.90	12.66	
9	2	T(W-E)	400	2200	5400	6.20	13.07	
10	3	TR(E-W)	400	2200	5400	6.20	13.56	
11,12	4	T(W-E)	380	1500	5400	2.70	12.50	55.43
13	5	T(W-E)	180	1600	5400	0.82	8.53	57.36
14	5	TR(E-W)	180	1600	5400	1.13	10.03	
46	6	T(W-E)	300	2000	3400	5.23	14.92	54.49
47	6	T(E-W)	300	3000	5400	4.76	14.50	
48	7	T(W-E)	300	1600	6400	3.82	18.62	52.86
49	7	TR (E-W)	300	1600	5400	3.97	12.06	
50,51	8	T(W-E) TR(E-W)	300	1800	5400	3.65	12.86	55.12
52,53	9	T(W-E) TR(E-W)	300	1700	5400	1.08	11.16	54.64

2.3. Irrigation Planning :

2.3.1 Introduction :

Sir Charles Travelyan, an eminent British administrator, stated that “Irrigation is everything in India. Water is more valuable than land, because when water is applied to land it increases its productiveness at least six fold and generally a great deal more and it renders great extents of land productive which otherwise would produce nothing.” This statement holds good for the state of Odisha as there is great potential for development of irrigation. Odisha’s geographical area and water resources are respectively 4% and 11% of that of the country.

“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 semi-arid regions of the world, most of which are undeveloped and underdeveloped. 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 and cultural etc.” (vide ‘History of Irrigation Development in Orissa’ by G.C. Sahu, INCID, 2009, Pg.1)

State’s economy has remained mainly agrarian dominated by agriculture and allied sectors. This sector which includes agriculture, animal husbandry, fisheries and forestry contributes 18.44% of GSDP (Gross State Domestic Product) as against 14.60% at national level in 2009-10. However, this sector provides employment and sustenance, directly or indirectly, to more than 60% of State’s total work-force. In this sense, the agriculture sector is still the ‘mainstay’ of Odisha’s economy. (Vide Economic Survey 2010-11, Planning & Coordination Department, Govt. of Odisha Pg. 75)

Planning Commission, Government of India accorded investment clearance to Rengali Irrigation Project to provide irrigation through left and right canal system offtaking from Samal Barrage pond having GCA and CCA of 3,36,500 ha and 2,35,500 ha excluding Akhuapada system (i.e. beyond river Baitarani). Benefit-cost ratio works out to 2.94 and cost per ha. of irrigation Rs.5515.60 vide their letter No.II-2(64)/78-I & CAD Dt.31.3.1978.

Before proceeding further, brief discussion on land use pattern and soil characteristics of the basin are described as under in nut-shell.

2.3.2 Soil characteristics of the basin:

“In general the soil of the Brahmani basin in Odisha consists of mostly red and yellow soils. The soils in the uplands, upper riverine plains and those in the lower riverine and littoral plains are fertile. Some saline or saline alkaline patches are also seen close to the coast line. Younger alluvial, coastal alluvial and coastal sandy soils are deficient in nitrogen, phosphoric acid and humus but not generally wanting potassium and calcium. Texturally these are sandy to loamy with pH values on alkaline side. These are most fertile soils and are suitable for high water demanding crops like rice, sugarcane etc. These soils are prone to waterlogging which are found in the districts of Dhenkanal, Jajpur, Kendrapara and Keonjhar”. (Source: 3rd Spiral study report, Pg. 51-52, Nov. 2002, OWPO, Dept. of Water Resources, Govt of Odisha)

The maximum development of alluvial formations is in the coastal tract. The thickness of alluvium increases towards the coast. The alluvial formations constitute the most productive aquifers. Sand and gravel horizons in the alluvium form the main repository of ground water. Alluvium also occurs in discontinuous patches adjoining the major drainage courses. The thickness of alluvium deposits in the inland river basins varies from 11m to 43m averaging 15m. Ground water occurs both under water table conditions in the shallow zone and under confined condition in the deeper horizon. The eastern part of a coastal tract close to the coast is beset with salinity problems. Saline aquifer occurs at

different depths in the basin in Kendrapara district i.e. either below or above fresh water aquifers. This salinity hazards are not uniform throughout the coastal tract. The general elevation vary from 5 to 20m in the east and south east. The ground water draft is through dugwells, filter point tube wells, shallow, medium deep tube wells in the basin. The ground water is a major source of rural and urban water supplies, and an important source of irrigation.

Soil types in various parts of the basin is furnished below vide Table No.2.8

Table-2.8 Soil types in different parts of the Brahmani Basin

Sl. No.	Land Form	Origin	Soil types observed
1	Plateaus	In Situ	Red and yellow soils including gravelly, sandy, loamy and lateritic soils.
2	Up-land	In Situ	Red soils with patches of red and black clay soils.
3	a) Talcher Basin	Rivulinic deposits	Red loamy soil
	b) Other Basin	Rivulinic deposits	Older alluvial and red soils
4	a) High Riverine plains	Rivulinic deposits	Mainly red sandy and loamy soils with patches of red and black clay soils.
	b) Low Riverine plains	Rivulinic deposits	Deltaic and coastal alluvium both older and younger
5	Littoral plains	Rivulinic deposits	Recent deltaic and coastal alluvium with patches of saline or saline alkaline soils.

(Source: Brahmani basin, Sectoral study, Chapter-2, Pg. 17, OWPO, Consultant- Sheladia Associates Inc., USA)

As per suitability of soil for cultivation, the land of the basin is of 6 types and can be seen with category wise area in sq.km in the following table. Out of 22516 sq.km basin area 9208 sq.km constituting 40.9% are suitable for agriculture. Rest 13308 sq.km. i.e. 59.1% is un-suitable for agriculture vide Table No.2.9.

Table No-2.8 Land capability in Brahmani Basin

Type	Area (Sq km)	Percentage (%)
A. Total suitable land for cultivation	9208	40.9
Very good land	1056	4.7
Good land	4056	18
Moderately good land	2788	12.4
Poor land	1308	5.8
B. Total land un-suitable for cultivation	13308	59.1
Land with some limitations suitable for forestry & grazing	2366	10.5
Land with very severe limitations suitable for forestry & grazing	10942	48.6
Grand Total	22516	100

(Source: Ibid, Pg.23)

District wise Land use pattern (1999-2000) of Brahmani basin is furnished in Table No-2.10
(Source: 3rd Spiral study, Nov. 2002, OWPO, Annexure 4.14)

Districtwise Land use pattern of Brahmani Basin (1999-2000)

Table No-2.10

(Area In '000 Ha.)

District	Basin Area inside the District (sq.km.)	Geographical Area of the District (sq.km.)	Basin Area/ District Area (%)	Forest Area	Misc. Tree & Groves	Permanent Pasture	Cultivable Waste	Land put to Non- Agrl. Use	Barren & Unculturable land	Current Fallow	Other Fallow	Net Area Sown	Total cultivable Area
1	2	3	4	5	6	7	8	9	10	11	12	13	14
Sundargarh	5794.13	9712	59.66	252.36	34.60	42.95	16.11	17.90	6.56	2.39	7.76	198.67	259.52
Sambalpur	1371.05	6657	20.60	69.20	12.77	2.47	2.06	5.15	2.88	0.00	3.50	39.96	58.29
Deogarh	2512.37	2940	85.45	94.00	21.36	6.84	4.27	11.96	27.35	0.00	10.25	61.53	97.42
Angul	4225.94	6375	66.29	232.68	3.98	5.30	3.31	9.94	2.65	4.64	5.30	145.17	162.41
Dhenkanal	3856.91	4452	88.88	162.65	5.33	8.89	0.89	19.55	5.33	18.66	4.44	183.09	212.42
Keonjhar	1723.48	8303	20.76	52.52	23.87	7.06	11.21	7.27	5.60	0.42	2.08	62.27	99.84
Jajpur	1824.75	2899	62.94	26.44	5.66	3.15	6.92	15.11	5.04	7.55	3.15	108.89	132.18
Kendrapara	1107.45	2644	41.89	8.38	2.51	4.61	2.93	7.96	1.26	9.63	5.86	63.67	84.61
Total	22516.08			898.22	110.09	81.27	47.71	94.84	56.67	43.29	42.35	863.25	1106.69
% of basin				39.89	4.89	3.61	2.12	4.21	2.52	1.92	1.88	38.34	49.15

Source: Districts at a glance 2002, Directorate of Economics & Statistics, Orissa. Orissa Agricultural Statistics 1999-2000.

2.3.4 Classification of Land:

O' Malley* gives the following description of various types of land in Angul which is more or less applicable to the present district of Dhenkanal:

“There are four classes of lands, known as *sarad*, *harfasal*, *bazefasal* and *toila*, the meaning of which may be gathered from the following account of the way in which land is ordinarily brought under cultivation. First of all the jungle is cut and burnt on the land, which is then ploughed up the ashes of the jungle being ploughed into it. It is then sown with early rice, cotton or a pulse crop, and good harvests are produced for three years without any further manuring. Such newly reclaimed land is known as *toila*. After three years if the ryot is able to apply cow-dung or other manure he does so and the land continuing under cultivation is known as *bazefasal*, which is simply reclaimed upland brought and kept under uncultivation by manuring and careful tillage. If however the ryot is unable to apply manure, the land is allowed to remain cultivated, in the course of time it lapses back into jungle and after three or four years is again brought under cultivation by the process above described”.

“In *bazefasal* land prepared in this way ordinary rabi and bhadoi crops, such as mustard, maize and the castor oil plant, are grown. If it is situated in the immediate neighbourhood of and intermingled with village sites, where it receives good manuring, it is known as *harfasal* which is practically homestead land, while *bazefasal* corresponds with bhita or uplands of Bihar. *Toila*, as stated above, is land recently reclaimed from jungle and may be high or low. Thus, if a ryot breaks up and reclaims low land, he may allow it to go out of cultivation, and in that case it still continues to be called *toila*; but if he surrounds it with low banks or ridges, has it irrigated, and makes it suitable for paddy cultivation, it becomes *Sarad* and is classed as such.”

Sarad or rice land is further sub-divided into three classes. The first class is called *nali* or *berna* and consists of low-lying land situated between ridges, within hollows, below dams and dykes, or near springs and water courses; this is the best land for rice and it always remains moist. The second class which is called *dera* or *majhighatia*, consists of land somewhat inferior in quality situated on slopes or above the *nali* land. The third class consists land known as *pasi* or *dhipa*, i.e., land on high levels, which receives no irrigation and is entirely dependent on the rainfall.”

“*Harfasal* and *dofasal* lands are lands surrounding the village homesteads on which double crops are grown, and include vegetable gardens, plantain groves, and *pan* plantations. *Bhadoi* crops such as maize, *sawan* and *mandia* are first raised on lands of this class, when these have been reaped, tobacco, mustard, ginger, brinjals, onions, chillies, etc., are planted. *Bazefasal* lands are generally situated in the vicinity of the village and like *harfasal* are usually manured. They grow single crops such as maize, tobacco, brinjal, mustard, *saru* or *arum*, and the castor-oil plant.”

“The *toila* lands are the high lands, other than rice fields, situated at a distance from the village homesteads, which are sometimes allowed to lie fallow for a year or two in order that they may recover fertility. They are of three classes, viz., first class or *dofasal*, second class or *ekfasal* and the third class land consisting of sandy or gravelly soil, which is sown in alternate years.” (Source: *, L.S.S.O' Malley-Angul District Gazetteer, 1908, Pg.91-92- Orissa District Gazetteers, Dhenkanal, 2192, Pg.154-156)

2.3.5 Land use details of the basin

“For agricultural development, water and land resources form the most essential input. The availability of land decides the potential and limitations of agricultural production.

In a pure natural environment entirely free from any human interference, all land shall be under mixed use of different species of flora and fauna living together in healthy competition with each other. Man, with his superior intelligence, interferes with this setup and either encourages or restrains (even-eliminates) certain species from an area. This is the essence of redefining “land use” and it primarily stated with converting forests in to agricultural fields and plantations and later extended to reserving areas of land for habitation, industrial, mining and other purposes.

Thus the study of current land use pattern in an area is of much significance in any environment study. In the present study, part of the basin lying in different districts has been taken proportionately” (Source: Brahmani Basin, Sectoral study- Land Resources, Pg.27)

2.3.5.1 Drainage Congestion:

The area to be served with irrigation facilities by the canal systems have adequate natural drainage systems in the shape of Nalas and hill streams that join the river Brahmani on midway, Baitarani on North and Mahanadi on South. Due to favourable surface and sub-surface features, the area proposed to be brought under the flow irrigation system have at present no drainage problem, but after extension of canal irrigation, the low lying strips may face the problem of water logging and drainage congestion. However a provision of Rs.3.63 crores for drainage improvement has been kept in the estimate.

2.3.5.2 Conjunctive use:

Conjunctive use of ground water has not been planned at present in the command of the project due to abundant availability of surface water and high cost of energy. Moreover the farmers are more acquainted with the use of surface water. The possibility of exploring of conjunctive use of ground water and surface water will be taken care in due course after full development of command area in consultation with State Ground Water Board and CGWB.

2.3.5.3 Provision for M&I Supply:

The Municipal and Industrial (M&I) requirement of water is estimated at 1095.30 Mcum which has to be met with dependability of 100%. While 314.2 Mcum of this requirement is proposed to be discharged through the Right Bank Canal, 312.50 Mcum is to be utilized upstream of Samal Barrage for NTPC, Super Thermal Power Plant and remaining 468.60 Mcum is required to be released through LBC for use in the downstream areas. Details of M&I provision/use are given at Annexure-VIII (a, b & c).

2.3.6. Socio Economic aspect of the Ayacut area:-

The undivided district of Dhenkanal and the other ayacut areas of the project consist mostly of ex-feudal states, which had little scope of development upto the first half of the twentieth century. The area had no major basic industries or other allied industrial activities then except the Collieries at Talcher. The people of the ayacut mostly depend on agriculture which is subject to vagaries of nature visited by chronic droughts due to erratic and uneven rainfall. The agricultural lands are mostly in the lower erosional plains of the eastern ghats, segregated by hill ranges and thick tropical forests, yet the population of the ayacut area is high and the villages are thickly populated as the land is highly fertile. Since the agricultural land per capita is low, the people are economically poor and backward. The land use pattern of the areas under Brahmani basin in different districts has been furnished vide Table No-2.10.

2.3.7 The present Scheme:-

The tail race water that will be released from Rengali Hydro electric station will be 170 cumec (6000 cusec) without any upstream regulations and will be enhanced to 270 cumec (9500 cusec) in a 90% dependable year. This regulated release is proposed to be picked up near Samal by a Barrage across the River Brahmani and utilised for irrigation. The total inflow into the Rengali Reservoir is 9.1143 lakh ham. (74.10 lakh.ft.) in a 75% dependable year out of which 8.387 lakh ham. (69.3 lakh a.c.ft.) is proposed to be utilised in irrigation and 0.444 lakh ham. (3.6 lakh acre ft.) goes for evaporation loss.

2.3.7.1. Head Works and Canal System:

The Barrage is designed on permeable foundation with R.C.C. raft, masonry body wall encased with concrete and apron. The crest of Barrage is fixed at R.L.67.00 m. for barrage portion and R.L.66.00m. for scouring sluice portions. The pond level is kept at R.L.76.20 m. The total length of the Barrage including left and right scouring sluice is 560.5 m from abutment to abutment. It will have

17 bays of 20.0 m width fitted with radial gates and 7 nos. of scouring sluice bays of the same width 4 nos. on left and 3 nos. on the right side. The head regulators on the left and right side will have head discharge capacities as 151.86 cumec and 111.30 cumec respectively.

The water will be carried by two nos. of contour canals on either side. The canals will be lined with cement concrete. The left main canal will be ultimately lined when phase II is taken up. The left main canal will have a length of 141 km. and will have a head discharge capacity of 151.86 cumec (5363 cusec). The length of the Right Main Canal will be 95.04 km and the head discharge is 111.30 cumec (3931 cusec). The canal will be lined one. The left canal is to irrigate 1.143 lakh ha. (2.823 lakh acres) of C.C.A. upto the river Baitarani. The Right main canal has an ayacut of 1.040 lakh ha. (2.569 lakh acres) C.C.A. upto River Mahanadi near Narasinghpur. The entire ayacut of right canal will be irrigated in Phase I of the Project.

2.3.7.2 History of Clearance:

Rengali Irrigation Project (Phase I) with an estimated cost of Rs.233.64 Crores (Rs.214.89 crores as cost of Project and Rs.18.75 crores as share cost of Rengali Dam) was considered acceptable by the planning Commission Vide their No. II-2 (64) / 78 I & CAD / dtd. 31.3.78, with some observations during March 1978. Phase I consisted the following works.

- i) A barrage across Brahmani river near village Samal in Dhenkanal (presently Angul) district, 34 km downstream of Rengali Dam.
- ii) A right main canal (RMC) taking off from the barrage, having total length of 95.04 km (proposed to be fully lined) with discharge capacity of 111.3 cumec. CCA under this canal was, 104092ha out of which 19686 ha was to be commanded by lift. Besides providing irrigation, this canal was also to supply water to the existing Talcher Thermal Station and other industrial Complex under construction, the proposed Super Thermal station and other proposed industries. The quantity of water proposed to be supplied for this use was indicated as 10 cumec.
- iii) A fully lined left main Canal (LMC) taking off from the barrage and having discharge capacity of 216.5 cumec and length as 141 km for providing irrigation to a CCA of 1.14 300 ha. out of which 12316 ha would be under lift command.
- iv) In the phase II of the Project, the Left Canal was proposed to be extended beyond Baitarani river to provide further irrigation to 0.40 lakhs ha of CCA. Canal capacities have been provided for the ultimate phase. At present, proposal to provide irrigation beyond Baitarani has been deferred.

Construction of the Samal barrage was taken up during the year 1978 to pickup tail race release from Rengali Dam together with water from the intermediate catchment of 4780 sq.km between Rengali Dam and Samal barrage.

2.3.8. Stages of Development

2.3.8.1 Rengali Irrigation Sub Project LBC-I

The work on Rengali Irrigation Project Phase I was taken up in 1978 from State's own resources, but due to financial constraints physical progress could not be achieved. Till WRCP was taken up about 90% works on the Samal Barrage was completed and some work on initial reaches of L.M.C. and R.M.C. have been taken up. Considering the paucity of funds, the Govt. of Odisha proposed staged completion of Rengali Irrigation Project with help of External Assistance.

Keeping in view the above, the balance work of Samal Barrage and remaining works of reach 0-29.177 km of Left Bank Canal, which includes works of branch canal, distributaries, minors, sub minors and water courses, were taken up under “Rengali Sub-Project LBC-I” at estimated cost of Rs.161.45 crores as a sub-project of Odisha Water Resources Consideration Project (OWRCP) with World Bank Assistance for which Planning Commission accorded Investment clearance in their Ir. No.2(231)/95 - I CAD, dtd.27.9.95 . The RIP subproject LBC-I under WRCP was completed at an amount of Rs.50.05 crore.

2.3.8.2 Rengali Irrigation Sub Project - LBC II

For the completion of remaining works of Left Bank Canal, Govt. of Odisha proposed “Rengali Irrigation Sub-Project-LBC-II” to CWC on Feb. 1996 for posing for OECF Assistance. The Proposal envisages completion of all works of Left Bank Canal beyond WRCP Sub-Project from 29.177 km to 141.1 km to irrigate the balance CCA of 93,501 ha upto river Baitarani by gravity only. The project was approved by Planning Commission vide Ir. No.2(64)/94-I & CAD dated 14/07/1997 at an estimated cost of Rs.705.15 crores.

JICA funding was available for execution of LBC from RD 29.713 km to 71.313 km to cover an CCA of 29176 ha (including Lift ayacut). Funding for balance works from RD 71.313 km to 141 km to cover an area of 76641 ha including (Lift ayacut) has been posed to JICA. The same was proposed to be included under National Project but was not considered by Government of India as total ayacut was less than 2.00 lakh ha. The first 30 km of LBC has been so designed that when lined, it can carry sufficient water (216 cumec) for both phase I & Phase II.

Subsequently the revised cost estimate of Rengali irrigation Subproject LBC II at 2009 price level indicating the variation of the project cost for various reasons has been submitted to CWC. The same has been examined and finalized at Rs.1958.34 crores at 2009 price level and approved by 105th TAC of CWC and the investment clearance of the Planning Commission has been accorded vide Ir. No.2(64)/84-I & CAD dated 14/09/2010.

The proposal for providing irrigation to 0.40 lakhs ha beyond Baitarani river at the downstream end of LBC has been deferred by the State Government at present.

2.3.8.3. Rengali Irrigation Sub Project - RBC 0-95 km

The State Govt. had submitted revised estimate of Rengali Right Bank Canal during 1998 and confirmed vide their letter dated 12.03.98 that, it was the sub-project of Rengali Irrigation Project Phase-I approved by Planning Commission in 1978 without any change in scope. Out of the total CCA of 1,21,200 ha under RBC as approved in 1978, a CCA of 100500 ha under gravity flow only was considered in the above proposal. The same has been examined and approved by TAC of CWC at a cost of Rs.738.27 crores at 1997 price level. The investment clearance of planning commission was also accorded vide Ir. No.2(64)/98-I & CAD dated 17/11/1998.

Subsequently the revised cost estimate of Rengali Irrigation Subproject RBC(00 km to 95 km - as per revised alignment) at 2006 price level indicating the variation of the project cost for various reasons has been submitted to CWC and confirmed vide their letter No-7039 dated 23.8.07 that it is a sub-project of Rengali Irrigation Project Phase-I approved by Planning Commission in 1978 and ‘The scope of the earlier approved project has not been changed and only cost has been updated.’ The total CCA has been reduced to 1,04,092 ha from 1,21,200 ha as per existing field conditions out of which a CCA of 84,406 ha under gravity has been considered in the proposal.

The same has been examined and finalized at Rs.1290.93 crore at 2006 price level and approved by 92nd TAC of CWC. At present the right Bank Canal having CCA of 84406 ha under gravity flow

is under progress through AIBP funding. Environment, Forest and R&R clearances have been obtained from concerned Ministries vide their letter Nos.J-12011/7/96-IA-I dated 04/12/1996, No.S-53/96/FC dated 21/11/1996 & No. S-53/96/FC dated 14/05/2003 and No.2011/5/96/PDB dated 18/06/1996 respectively.

Abstract of all statutory clearances obtained from appropriate authorities have been annexed in tabular form vide Annexure No-III & IV

2.3.8.4 Rengali Irrigation Project-Service Area

The second stage of the development is constituted construction of “Samal Barrage” on the Brahmani river, 35 km downstream of the Rengali dam to provide irrigation water to 218392 ha and construction of left canal system (141 km, 114,300 ha) and right canal system (95 km, 104092 ha). In addition to the direct irrigation command area of 218392 ha, about 40,000 ha located beyond the Baitarani River is considered as supplemental irrigation supply area. Irrigation beyond Baitarani has been deferred. The barrage has free drainage area (catchment area) of 4,780 km², in addition to that of the Rengali dam of 25,250 km². Mini-hydropower generation plant with installed capacity of 20 MW has been constructed recently on the left bank of the barrage and operated by the private sector (Odisha Power Consortium Limited) since 2009. All river maintenance flow is released through the power plant, after completion of the power plant.

The project plans to provide irrigation water to five districts, namely Angul (2564.2 ha), Dhenkanal (84637.2 ha), Jajpur (59558 ha), Keonjhar (16,011 ha) and Cuttack (35935.6 ha) shown in Table-2.11 with lift ayacut of 19686 ha. from Right Canal.

The construction of the Samal barrage was completed in 1994, and work on two canal systems for both left and right banks were taken up by the Department of Water Resources, GoO in 1996 with financial assistance of GoI (under AIBP) as well as World Bank (under OWRCP) and JICA (earlier called as OECF/JBIC). The left canal from the barrage to RD 29 km to provide irrigation facilities for 8,483 ha was constructed under Orissa Water Resources Consolidation Project (OWRCP) with financial assistance of the World Bank, and completed in September 2004. The construction of the left canal from RD 29 km to RD 71 km, covering the irrigation area of 29,176 ha in Dhenkanal district commenced in December 1998 with financial assistance of JICA (earlier OECF/JBIC), and completed by 2012. On-farm development is scheduled to be implemented by CADA between 2010-2014. Under the JICA funded sub-project (LBC-II Phase-I), establishing and strengthening of Water Users' Associations (Pani Panchayat) is under way. Construction of the remaining stretch of the left canal from RD 71 km to RD 141 km covering 76,641 ha. has not commenced yet due to constraint of funds for which negotiation with JICA is under progress. The right main canal of 95 km long is under execution up to RD 79 km being financed by the GoI under AIBP. Salient features of the Samal barrage and canal systems are shown in Tables-2.12 and 2.13. As of June 2013, following farm lands are irrigable in Khariff season under the Project

Left canal system :	WRCP area upto 29 km of 8,480 ha (since 2004)
	From 29 km to 71 km area 30833 ha totalling to 39313 ha (by June, 2013)
Right canal system :	AIBP area 7893 ha (By June, 2013)

Total irrigated ayacut ending June 2013, 47206 ha from both LBC & RBC.

Table -2.12 Salient Features of Headworks	
Item	Features
<u>Rengali Dam</u>	
1) Location	Rengali village (80 km from Angul City)
2) Water source	Brahmani river
3) Catchment area	25,250 km ²
4) Storage Capacity At Max Water Level (125.4m) At Full Supply Level (123.5 m) At Dead Water Level (109.72 m) Effective storage capacity	5,150 MCM 4,400 MCM 880 MCM 4,270 MCM
5) Length and height of Dam body	1,040 m long and 70.5 m high
6) Spillway Design discharge Length Spillway gates	55,540 m ³ /sec 484 m 24 nos. of radial type gates (each 15.5 m x 14.8 m)
7) Hydro-power station Installed capacity of generator	250 MW (5 nos. x 50 MW)
8) Construction period	1972 – 1985 (14 years)
9) Operation of Power unit (at full swing)	From 1992
10) Construction cost Dam and spillway Power house (Civil Works)	Rs.170.9 Crores Rs. 32.3 Crores
Total	Rs.203.2 Crores
<u>Samal Barrage</u>	
1) Location	Samal village (35 km down stream of Rengali dam)
2) Water source	Brahmani river
3) Catchment area	30,030 km ²
4) Storage Capacity	20.1 Mcum
5) Barrage Length Design discharge Barrage gates Under sluice gates	560.5 long 24,632 m ³ /sec (100 year flood) 17 nos. of radial type gates (each 20.0 m x 9.2 m) 7 nos. of radial type gates (each 20.0 m x 10.2 m)
6) Hydro-power station (on left bank) Installed capacity of generator	20 MW (4 MW x 5 units)
7) Construction period	1978 – 1994 (17 years)

Table No-2.13(a) Salient Features of Main Canals	
Item	Features
Left Main Canal	
1) Cultivable command area Flow irrigation (planned full scale) Lift irrigation (planned full scale)	114,300 ha (101,984 ha, 89%) (12316 ha, 11%)
Current irrigated area (as on June 2013)	(.... % of total ayacut)
2) Discharge at head of canal	151.9 m ³ /sec
3) Nos. and size of intake gates at Barrage	8 nos., 6.0 x 5.4 m
4) FSL at head of canal	75.5 m
5) Canal bed width and design water depth	21.45 m and 4.3 m
6) Free board	0.9 m
7) Canal length Planned full scale	141.1 km
Work on progress as of June, 2013	71.3 km
Right Main Canal	
1) Cultivable command area (planned full scale) (Flow irrigation) (Lift irrigation) Current irrigated area (as of June, 2013)	104092 ha (84406 ha, 90%) (19686 ha, 10%) (.... % of total ayacut)
2) Discharge at head of canal	111.3 m ³ /sec
3) Nos. and size of intake gates at Barrage	6 nos., 6.0m x 4.3 m
4) FSL at head of canal	75.5 m
5) Canal bed width and design water depth	11.1 m and 4.3 m
6) Free board	0.9 m
7) Canal length Planned full scale Work progress as on June, 2013	95 km 79 km

Irrigation intensity adopted.

CCA under Rengali Irrigation Project = 2,18,392 ha.

Proposed irrigation Intensity:

Khariff	-	85 %
Rabi	-	70 %
Perennial	-	15 %
Total	-	170 %

The Advisory Committee of Planning Commission on Irrigation, Flood Control and Multipurpose Projects while accepting Rengali Irrigation Project (phase I) has pointed to reduce the irrigation intensity from 170% vide Annexure-V, (mentioned in the TAC Note sl. No.21 of 1978) to 140% and for completion of the soil Survey by the State and to review the cropping pattern.

On the basis of the Soil surveys and recommendations from Agriculture Department, irrigation intensity of 170% for Rengali Irrigation Project has been adopted. However this will be reviewed after full development of irrigation. The cropping pattern as contemplate in the original project and proposed during 1996-97 is enclosed at Annexure-VI & VII.

2.4 Inter-State Aspect

The Rengali Irrigation Project is located on Brahmani river, which is an inter-state river between Bihar (now Jharkhand) and Odisha. The Project proposes to utilize only a part of the tail race release from the Rengali Power House, and flows from intermediate catchment between Rengali Dam and Samal barrage, which is totally in Odisha. Interstate issue is therefore not involved.

Table No- 2.13(b) Principal Features of Major Canals on Left Bank Area (RD 0.0 to 71 km)

Item	Unit	Left Main Canal		Parjang BC (WRCP)	Bhairpur BC (JICA)	Off taking from LBCM (29-71 km)				Off taking from Bhairpur BC	
		LBCM at 0 km	LBCM at 29 km			Baisinga DC	Loknathpur Disty.	Raibol DC	Kanheipal DC	Jiral Disty	Bhairpur DC
1. Design command area (CCA)	ha	114,300	105,817	6,341	10,947	1,349	1,821	3,461	2,013	2,578	2,440
2. Design discharge at the Beginning Point	Cumec	151.86	145.27	6.888	13.309	1.349	1.821	3.461	2.013	2.887	2.734
3. Unit discharge per ha	lit/s/ha	1.20	1.20	0.98	1.14	1.00	1.00	1.00	1.00	1.12	1.12
4. Length of Canal	km	141	-	29.45	34.78	2.76	1.82	12.85	4.41	-	6.76
5. Dimensions of canal											
a) Bed width	m	21.45	18.7	6.4	10.0	2.4	2.7	3.4	1.4	2.7	3.0
b) Design Full Supply Depth (FSD)	m	4.30	4.50	1.20	1.50	0.80	0.92	1.18	1.06	1.05	1.10
c) Free board (Fb)	m	0.9	1.0	0.9	0.8	0.5	0.5	0.5	0.5	0.5	0.5
d) Velocity	m/sec	1.518	1.39	0.59	0.72	0.46	0.51	0.60	0.57	0.56	0.59
e) Full Supply Level (FSL)	m	75.50	70.86	70.00	68.76	66.21	65.29	65.12	62.96	6.25	60.28

Note;

DC: Distributary canal

Unit irrigation requirement at outlet on minor canal is set at 1,250 ha/cum or 0.8 lit/s/ha

In case of LBCM, amount of about 2.5 m³/s (actually supplied for 22.738 m³/s) is added to downstream development such as Akhuapada weir area.

Design details of LBC & RBC of Rengali Irrigation Project are furnished below:

Design Particulars of Rengali Left Bank Canal (RBC 0-141km)

1.	Location of HR of LBC	Samal
2.	Length of LBC (Main Canal)	141.00 km
3.	CCA of LBC (Gravity)	101984 ha
4.	CCA (Lift)	12316 ha
5.	Discharge at head	216.5 (m ³ /sec)
6.	FSL at head (RL)	75.50 m
7.	Bed with head	18.15 m
8.	FSD at head	5.40 m
9.	Free Board	0.75 m
10.	Bed slope	1/10,000
11.	Velocity	1.518 m/sec
12.	Rugosity Co-efficient	0.016
13.	Cropping intensity	170%
14.	Revised Cost of Project (2010 price level)	Rs.2238.58 Cr.

Design Particulars of Rengali Right Bank Canal (RBC 0-95 km)

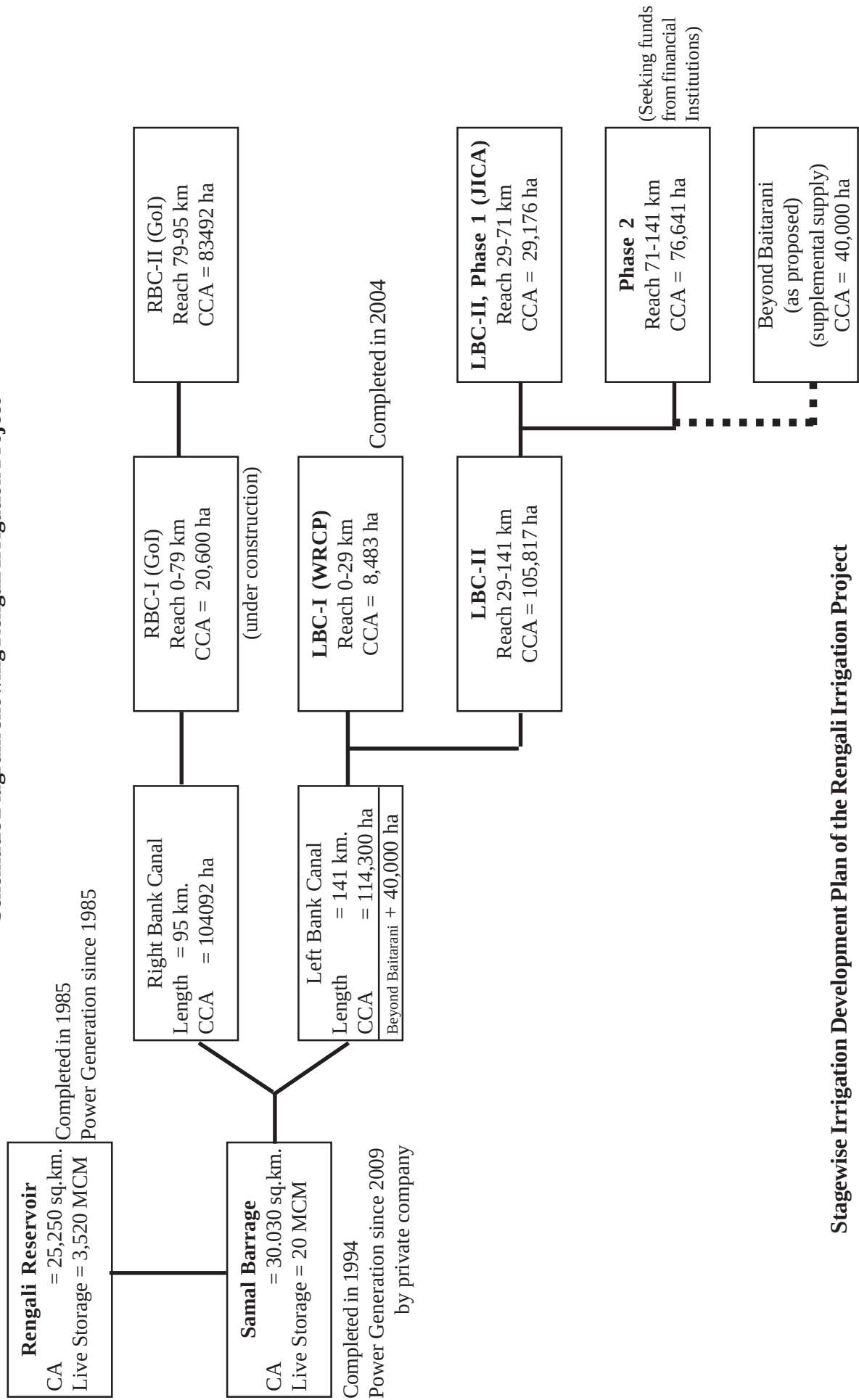
1.	Location of HR of RBC	Samal
2.	Length of LBC (Main Canal)	95.00 km
3.	CCA of LBC (Gravity)	84406 ha
4.	CCA (Lift)	19686 ha
5.	Discharge at head	111.30 cumec
6.	FSL at head (RL)	75.50 m
7.	Bed width	12.00 m
8.	FSD at head	4.28 m
9.	Free Board	0.75 m
10.	Bed slope	1/10,965
11.	Velocity	1.395 m/sec
12.	Rugosity Co-efficient	0.016
13.	Cropping intensity	170%
14.	Revised Cost of Project (2010 price level)	Rs.2028.82 Cr.

Economic Analysis of Rengali Irrigation Project

1)	Cost of project (LBC+RBC+HW)	Rs.4907.99 Cr.
2)	Total Ayacut	218392 ha
3)	Benefit Cost Ratio	1.862
4)	Internal Rate of Return (IRR)	13.69%
5)	Benefitted Districts	Angul, Dhenkanal, Cuttack, Jajpur and Keonjhar.

Schematic diagram of R.I.P showing details of Head works, LBC and RBC is annexed herewith.

Schematic Diagram showing Rengali Irrigation Project



Stagewise Irrigation Development Plan of the Rengali Irrigation Project

**Statutory Clearances and Approval of various Authorities/Organisations for
Rengali Multi-purpose Project**

Sl. No.	Organisations with their letter No. & date	Details of approval with stipulation
1	2	3
1	Planning Commission, Govt. of India – No.II-2(64/72)-A&I Dt.14.06.1973	Investment clearance of stage I. Estimated cost Rs.5792.68 lakhs which includes Rs.4192 lakhs for power plant for installing 2x50 MW hydro-electric units with energy potential of 526 MKWH annually. Cost of generation is 6 (six) paise for KWH.
2	Government of Odisha, Irrigation & Power Dept. No-OFC(RL)14/73-35062 Dt.06.12.1973.	Administrative approval for Rengali Dam Project, Stage I for Rs.41.92 crore.
3	Planning Commission Government of India-No.II-2(64)/78-I&CAD Dt.31.03.1978.	Acceptance of Rengali Irrigation Project (Ph.I) for Rs.233.64 crores, excluding Akhuapada System which includes Rs.18.75 cr. as share of cost of Rengali Dam. GCA-336500 ha, CCA – 235 500 ha Cost per ha – Rs.5515.60, B.C. Ratio – 2.94 Intensity of Irrigation may be reduced from 170% to 140%
4	Government of Odisha, I&P Department- No.RL-32/79/25/415 Dt.24.07.79	Administrative approval for execution of Samal Barrage and left canal system for Rs.164.0 cr.
5	Govt. of Odisha, I&P Dept. No-RL 34/8/23281 Dt 05.08.1981.	Administrative approval for execution of Rengali Right Canal for Rs.69.64 cr.
6	Odisha State Pollution Control Board, Bhubaneswar No.1774/Ind-I/NOC-117 Dt.16.06.1994.	E.I.A. clearance for Rengali Irrigation Ph.II (Right Canal System)
7	Ministry of Environment & Forest (MoEF), Govt. of India- No.3/12011/12/95-I.A.I-110003 Dt.12.09.1995.	Approval of Odisha Water Resources Consolidation Project (OWRCP) from environmental angle which includes upto 29.1 km of left canal system.
8	Planning Commission, Govt. of India (I&CAD Division) No.2(231)/95-I&CAD, Dt.27.09.1995.	Investment clearance to OWRCP for Rs.977.0 crores (at 1994 price) which includes to complete left canal upto 29.1 km to provide irrigation to 8445 ha.
9	Ministry of Welfare, Govt. of India No.2011/5/96-TDB Dt.18.06.1996.	Rengali Irrigation Sub Project, LBC II- As only 10 tribal families are affected, the number is insignificant. Therefore Ministry has no objection to the project.
10	Central Water Commission, MoWR, Govt. of India No.16/27/96-PA(N)/937-990 Dt.24.06.96-65 th meeting of Advisory Committee on	Rengali Irrigation Sub Project LBC-II for Rs.705.15 cr (at 1995 Schedule of Rate). CCA-93501 ha, Annual Irrigation 177651 ha. Irrigation intensity 190%.

1	2	3
11	MoEF, Govt. of India No.8-53/96 FC Dt.21.11.96	Diversion of 2159.43 ha of forest land for Rengali Irrigation Project (R.I.P). But approval accorded for 2107 ha in 4 districts. Angul – 102.584 ha Dhenkanal – 777.81 ha Athagarh – 443.87 ha Keonjhar – <u>783.14 ha</u> 2107.404 ha (Say 2107 ha) The user agency will transfer the cost of compensatory afforestation to Forest Department. The project will provide crossing points for wild elephants wherever necessary in consultation with local forest officials.
12	MoEF, Govt. of India No.2011/7/96 IA-I Dt.04.12.96- Rengali Irrigation Project, Stage-II Constn. Of LBC from 30 to 141 km and RBC from 0-112 km canal & barrage- Environmental Clearance.	Conditions stipulated: - Strict implementation of catchment area treatment plan. - Afforestation with suitable plantation - Kapilash along with other adjoining forest blocks to be declared as wild life sanctuary. - Forestry clearance for diversion of Forest land (2159.43 ha) under Forest Conservation Act, 1980 to be obtained separately.
13	Planning Commission, GoI, No.2(64)/94-I& CAD Dt.14.07.97.	Rengali Irrigation sub-project-LBC II-Investment clearance for Rs.705.15 crore (1995 price level) CCA-93501 ha, cost of Irrigation Rs.39693/ha. B.C. Ratio-2.50 IRR-18.82.
14	Planning Commission, GoI No.2(64)/98-I& CAD Dt.17.11.1998	Rengali Irrigation sub-project, R.B.C. 0 km to 112 km, Estimated cost Rs.738.27 (1997 price level) -Part of Revised Estimate. CCA-100500 ha, intensity of irrigation 170%, cost per ha. of annual irrigation – Rs.43212/ha. B.C. Ratio 1.79, IRR-13.60.
15	MoEF, GoI No. S-53/96 – F.C. (Pt.) Dt 13.05.2003	Diversion of 2159. 43ha-Split-up proposal for 812 ha (Phase I) with following conditions: - Compensatory afforestation to be raised over non-forest land. - Non-forest land transferred to Forest Dept. for compensatory afforestation to be declared as Reserved Forest. - Areas identified for eco-tourism to be developed at project cost. - Crossing points for wild elephants to be provided at project cost in consultation with state Forest Dept.
16	Central Water Commission, MoWR, GoI – 92 nd meeting of advisory committee on Irrigation, F.C. & Multi-purpose project proposals (27.02.2008) No.16/27/2008-PA(N)/364-95 Dt.12.03.2008.	Rengali sub-project – Right Bank Canal, Revised Estimate Rs.1290.93 crore (price level 2006). CCA reduced to 84406 ha. Annual Irrigation reduced to 143490 ha. Industrial Water Supply – 10 cusec B.C. Ratio – 2.80.
17	Planning Commission, GoI- No.2(64)/2008-WR Dt.14.09.2010.	Investment clearance-Rengali Irrigation Sub-project, LBC II Revised cost – Accepted by the Advisory Committee in 105 th meeting on 25.06.2010 for Rs.1958.34 crore (price level 2009-10) for LBC from RD 29.177 km to 141.0 km.

**Clearance from Directorate of Mines for Pond Area
(Under Section- I Clause No.1.5(b).7)**

Copy of letter No.XXVII (a)-B4/76/2119/Mines, Bhubaneswar, dated the 22nd, January 1976 from Sri B.K. Mohanty, Director of Mines, Odisha to the Superintending Engineer, Rengali and Bhimkund Investigation Circle, Dhenkanal.

Sub : Clearance for the pond area of proposed barrage at Samal across river Brahmani.

A reference is invited to your D.O. letter No.155/SE dated the 9th January 1976, enclosing a copy of the pond area map. From the spread of the pond shown in the map, it is seen that it does not submerge any valuable mineral deposits that we know of. The proposed pondage will also not create any undue disadvantage in the nearby collieries.

Annexure-V

**Intensity of Irrigation Proposed in the project as per Planning Commission
(1976 Project Report)**

Total G.C.A. for Phase I = 3.364 lakh hectares.

C.C.A. = 77% of G.C.A. = 2.591 lakh hectares.

Sl. No.	Seasonal crop and Name of crops.	Area of crops in hectares	Percentage of C.C.A.
1	2	3	4
Perennial Crops			
1	Sugar cane	25,910	10%
2	Banana and Orchards	12,955	5%
Khariff Crops			
3	Early Paddy	51,820	20%
4	Medium paddy	88,094	34%
5	Late paddy (H.Y.)	12,955	5%
6	Vegetable (Kharif)	5,183	2%
7	Jute	25,910	10%
8	Groundnut (Kharif)	12,955	5%
Rabi Crops			
9	Wheat	25,910	10%
10	Oil seeds	25,910	10%
11	Pulses	31,092	12%
12	Potatoes	12,955	5%
13	Vegetables	5,182	2%
14	Dalua paddy (H.Y.)	64,775	25%
15	Fodder	7,773	3%
Summer Crops			
16	Summer vegetables	5,182	2%
17	Summer pulses	12,955	5%
18	Summer groundnuts	12,955	5%
	Total	4,40,470	170%

**Proposed Cropping Pattern – Post Project Period
(As contemplated in the Original Project Report)**

Crop with variety	Growing Period		Area in hectares	
	Optimum showing date	Optimum Maturity date	Pre-Irrigation Project	Post Irrigation Project
1	2	3	4	5
Perennial Crops				
1) Sugar cane	January 15	November 20	-	25,910
2) Banana and Orchards	January 1 st	December 31 st	-	12,955
Khariff Crops				
3) Early Paddy	June 1	September 28	1,98,587	51,820
4) Medium paddy	June 8	October 25		88,094
5) Late paddy (H.Y.)	June 15	November 11	-	12,955
6) Vegetable (Kharif)	June 16	September 28	7,181	5,182
7) Jute	April 18	August 15	-	25,910
8) Groundnut (Kharif)	July 4	October 31	-	12,955
Rabi Crops				
9) Wheat	November 21	February 28	4099	25,910
10) Oil seeds	November 25	March 9	13,241	25,910
11) Pulses	November 25	February 28	3039	31,092
12) Potatoes	November 11	February 28	1,461	12,955
13) Vegetables	October 18	February 2	7,215	5,182
14) Dalua paddy (H.Y.)	December 3	April 30	377	64,775
15) Fodder	December 1	March 20		7,773
Summer Crops				
16) Summer vegetables	February 1	May 31	4476	5,182
17) Summer pulses	February 11	May 31	11,851	12,955
18) Summer groundnuts	February 1	May 31		12,955

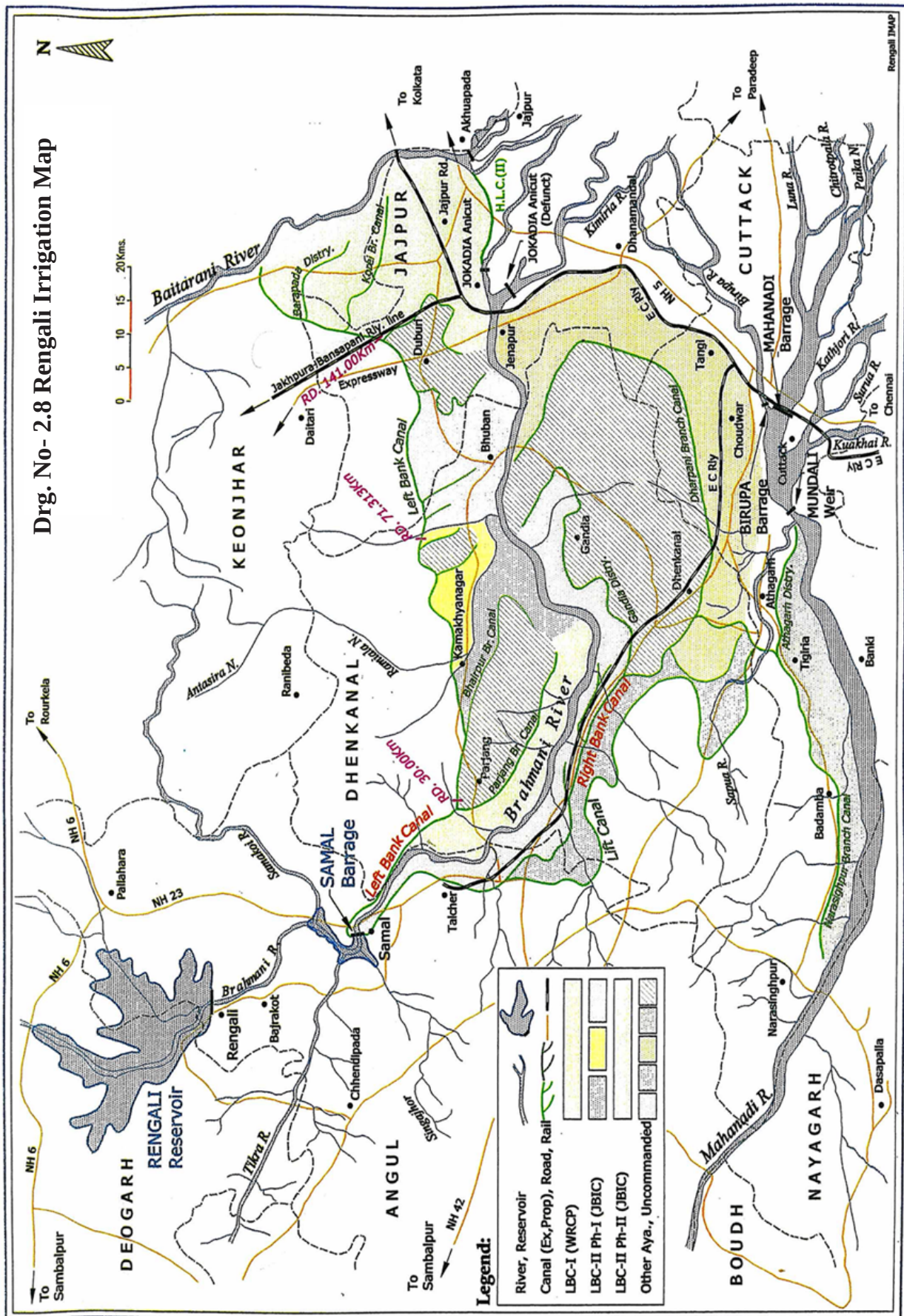
Proposed Cropping Pattern during 1996-97 for Rengali Irrigation Command Area

Unit=% of command area

Crops	Kharif	Rabi	Summer	Total
Paddy	70	15	--	85
Early Paddy-(20%) (90-120 days)				
Medium Paddy – (30%) (120-135 days)				
Late Paddy – (20%) (135-160 days)				
Pulses (Green gram/Blackgram)	10	30	5	45
Oil-seeds (G.nut, Mustard, Sesamum)	5	25	--	30
Vegetables Perennials	5	10	5	20
Sugarcane/Banana	10	10	-	20
Total	90+10	80+10	10	180+20 Or 200 %

The proposed cropping patterns may change depending upon the availability of irrigation, water, scheduling of water supply in the canal, the family's needs of the farmers, prevailing prices and marketing facilities from time to time and above all the Government's policy. But Planning Commission in their letter No.2(64)/98-I&CAD Dt.17.11.98 while according Investment Clearance of Revised Estimate (at 1997 price level) of RBC reduced the intensity of irrigation to 170% vide Table-2.13.

Drg. No- 2.8 Rengali Irrigation Map



Brahmani Basin
Industrial Water Demand

Sl.No.	Node	Name of Industry	Location	Present Demand		Future Demand		Source		
	Name			(M.Cum)	(Cumec)	(M.Cum)	(Cumec)			
1	KNJ-3	SAIL	Keribur	1.165	0.037	2.563	0.081	Govt. Source		
		SAIL	Bolani	7.024	0.223	15.452	0.490	Govt. Source		
		Kalta Iron Mines	Kalta	0.294	0.009	0.647	0.021	Brahmani river		
		Barsuan Iron Mines	Barsuan	2.673	0.085	5.880	0.186			
				11.156	0.354	24.543	0.778			
2	SAIL-3	SAIL	Rourkela	102.444	3.248	225.376	7.147	Brahmani river		
3	RAJ-3	Orissa Cement Ltd.	Rajgangpur	1.438	0.046	3.164	0.100	Govt. Source		
		Gambadihi Dolomite Quarry,		0.166	0.005	0.366	0.012			
		Hindustan Zinc, Sargipali		0.799	0.025	1.758	0.056			
		Purunapani Limestone Quarry		0.546	0.017	1.201	0.038			
				2.949	0.094	6.488	0.206			
4	RKL-3	L & T	Kansabahal	1.448	0.046	3.186	0.101	Govt. Source		
		Otta India	Kalunga	0.083	0.003	0.183	0.006	Govt. Source		
		Chariot Cement	Kalunga	0.150	0.005	0.330	0.010	Govt. Source		
		Sita cement	Kalunga	0.023	0.001	0.051	0.002	Govt. Source		
		Konark Cement	Kalunga	0.003	0.000	0.006	0.000	Govt. Source		
		Mukund Cement	Kalunga	0.004	0.000	0.010	0.000	Govt. Source		
		Konark Chrome Chemical Pvt.Ltd.	Kalunga	0.025	0.001	0.055	0.002	Govt. Source		
		Orissa Industries	Kalunga	0.141	0.004	0.311	0.010	Govt. Source		
		Uluchha Pigment	Kalunga	0.100	0.003	0.220	0.007	Govt. Source		
		Siva Cement	Rourkela	0.083	0.003	0.183	0.006	Govt. Source		
		Krishna Ferro Ltd.	Rourkela	0.022	0.001	0.049	0.002	Govt. Source		
		Scan Steel	Rourkela	0.009	0.000	0.019	0.001	Govt. Source		
		Vedavyas Cement	Rourkela	0.005	0.000	0.010	0.000	Govt. Source		
		IDCO	Rourkela	2.497	0.079	5.493	0.174	Govt. Source		
		Asha Chemical	Rourkela	0.022	0.001	0.048	0.002	Brahmani river		
		Vipra Industries	Rourkela	0.006	0.000	0.013	0.000	Govt. Source		
		Indo Flugato	Rourkela	0.141	0.004	0.311	0.010	Brahmani river		
		Asiatic Gases	Rourkela	0.017	0.001	0.037	0.001	Brahmani river		
		I.T.C. Karei	Rourkela	0.002	0.000	0.005	0.000	Govt. Source		
		Bishnu Enterprises	Rourkela	0.025	0.001	0.055	0.002	Govt. Source		
		Santoshi Maa Iron Industries	Rourkela	0.018	0.001	0.040	0.001	Govt. Source		
		Lotus Chemicals	Rourkela	0.033	0.001	0.073	0.002	Govt. Source		
		Cast Profile	Rourkela	0.030	0.001	0.066	0.002	Govt. Source		
		Pahari Bar	Rourkela	0.018	0.001	0.040	0.001	Govt. Source		
		Gajlaxmi Iron Works	Rourkela	0.020	0.001	0.044	0.001	Govt. Source		
		Shree Chemical Industries	Rourkela	0.004	0.000	0.010	0.000	Govt. Source		
				4.930	0.156	10.846	0.344			
		5.	NIPC-3	NTPC	Kaniha	21.720	0.689	47.785	1.515	Samal Barrage
		6	TAL-3	IWSS (MCL)	Talcher	5.850	0.186	12.871	0.408	Brahmani river
				TTPS	Chainipal	14.297	0.453	31.454	0.997	Brahmani river
				FCI	Talcher	19.620	0.622	43.164	1.369	Brahmani river
NALCO	Angul			34.310	1.088	75.482	2.394	Brahmani river		
				74.078	2.349	162.971	5.168			
7	DKL-3	Navo Bharat Ferro Alloys	Meramandali	0.563	0.018	1.239	0.039	Brahmani river		
		Orissa Synthetics	Baulupur	55.250	1.752	121.549	3.854	Brahmani river		
				55.813	1.770	122.788	3.894			
8	JAJ-3	M/s.MILD EAST Integrated	Duburi	2.683	0.085	5.902	0.187	Brahmani river		
		Steel Corp.	Sukinda	40.112	1.272	88.246	2.798	Brahmani river		
		Neelachal Ispat Nigam		42.795	1.357	94.149	2.985			
Total			315.884	10.017	694.946	22.037				

(Source : 3rd Spiral Study Report, Brahmani Basin Plan Vol.1, OWPO, Nov.2002 Annex.7.14)

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Sl.No.	Rural Node	Place	District	Block	Population		Demand (2001)*		Demand (2051)**		Source
					2001	2051	(Cumec)	(M.Cum/Ann)	(Cumec)	(M.Cum/Ann)	
9		Parjanga	Dhenkanal	Parjanga	9570	11863	0.0062 0.0043 0.0393	0.1961 0.1342 1.2388	0.0089	0.2815	Brahmani Brahmani Brahmani
		Sanda	Dhenkanal	Parjanga	8330	10326			0.0077	0.2431	
		Saranga	Dhenkanal	Parjanga	5700	7066			0.0053	0.1663	
									0.1146	3.6135	
10	KAN-1	Kankadahada	Dhenkanal	Kankadahada	2400	2975	0.0000	0.0000	0.0022	0.0706	Ramiala
11	BADA-1	Bairi	Jajpur	Badachana	5220	6471	0.0105 0.0095 0.0049 0.0049 0.0039 0.0054 0.0099 0.0524	0.3319 0.3001 0.1554 0.2616 0.1232 0.1689 0.3107 1.6518	0.0049	0.1535	Brahmani
		Chhatia & Adj. village	Jajpur	Badachana	14100	17479			0.0130	0.4114	Brahmani
		Sunguda	Jajpur	Badachana	12750	15805			0.0118	0.3720	Brahmani
		Badachana & Dhanm	Jajpur	Badachana	10700	13264			0.0100	0.3147	Brahmani
		Angalo, Madhusudan	Jajpur	Bari	7100	8801			0.0066	0.2088	Brahmani
		Ramachandrapur	Jajpur	Bari	7373	9140			0.0069	0.2168	Brahmani
		Ratnagiri	Jajpur	Bari	6260	7760			0.0058	0.1841	Brahmani
		Rekabi Bazar & Jana	Jajpur	Jajpur	6601	8183			0.0061	0.1926	Brahmani
		Kalana	Jajpur	Rasulpur	5850	7252			0.0055	0.1721	Brahmani
		Sukindagarh	Jajpur	Sukinda	11114	13777			0.0103	0.3243	Brahmani
		Kabatabandha	Jajpur	Dharmasala	9500	11777			0.0089	0.2794	Brahmani
		Kantigadia	Jajpur	Dharmasala	5236	6491			0.0048	0.1528	Brahmani
		Kolhasamil	Jajpur	Dharmasala	7175	8894			0.0066	0.2094	Brahmani
		Salepur & Adj. village	Jajpur	Dharmasala	13200	16363			0.0122	0.3852	Brahmani
		12	BIN-1	Binjharpur	Jajpur	Binjharpur			11355	14076	0.0176

(Source : Ibid, Annex.7.12)

**Brahmani Basin
Urban Water Demand**

Sl. No.	District	Nodes	Urban Area	Source	Present Surface Water Supply			Future Surface Water Supply		
					Population	Demand Rate	Consump (Cumecl)	Population	Demand Rate	Consump (Cumecl)
1	SUNDARGARH	Raj-2	Rajgangpur(M) Biramitrapur(M) OCL Industrial Township(I.T)	Naktinallah (Int-1) Brahmanmora tank (Int-2) Int-1	43912 29434 2197		0.0260 0.0237 0.0018 0.0515	105547 70748 5281	135 lpcd 135 lpcd 70 lpcd	0.214 0.144 0.006 0.364
2		Rkl-2	Panposh(CT) Hatibandh(CT) Rourkela Industrial Township IDL Factory & Other colony(OG) Rourkela(M) Lathikata Town(CT) Jalada(CT)	Brahmani Brahmani Brahmani	10227 9296 206566 6665 224601 6896 11957	70 lpcd 70 lpcd 100 lpcd 70 lpcd 70 lpcd 70 lpcd 70 lpcd	0.0083 0.0075 0.2391 0.0054 0.2593 0.0056 0.0097	24582 22344 496503 16020 539853 16575 28740	70 lpcd 70 lpcd 150 lpcd 70 lpcd 150 lpcd 70 lpcd 70 lpcd	0.026 0.024 1.121 0.017 1.218 0.017 0.030
3		Kal-2	Kalunga Industrial Estate(OG)	Pitamahal	34503	70 lpcd	0.0280	82932	135 lpcd	0.168
4		Tens-2	Tensa Town(CT)	Kuradhi	4236	70 lpcd	0.0034	10182	70 lpcd	0.011
5	DEOGARH	Deo-2	Deogarh(M)	Gohira	20085		0.0113	48276	70 lpcd	0.051
6	ANGUL	Ang-2	Angul Town(NAC)	Takura Barrage	38022		0.0318	91390	135 lpcd	0.186
7		Ren-2	Rengali Dam Proj.Township(CT)	Rengali	8115	70 lpcd	0.0066	19505	70 lpcd	0.021
8		Nal-2	NALCO Town(CT) TTPS Township(CT) Dera Colliery Township(CT) FCI Township(CT) Ghantapada Town(CT) Talcher Town(M)	Brahmani	18038 6616 18583 7059 15587 34984	70 lpcd 70 lpcd 70 lpcd 70 lpcd 70 lpcd	0.0146 0.0054 0.0151 0.0057 0.0126 0.0206	43356 15902 44666 16967 37465 84088	70 lpcd 70 lpcd 70 lpcd 70 lpcd 70 lpcd 135 lpcd	0.046 0.017 0.047 0.018 0.039 0.171
9	DHENKANAL	Dkl-2	Dhenkanal Town (M)	Badajore nallah	57651		0.0546	138570	135 lpcd	0.281
10		Kam-2	Bhuban Town(NAC) Kamakshyanagar(NAC)	Ramial Ramial	20134 15002		0.0116 0.0084	48394 36059	70 lpcd 70 lpcd	0.051 0.038
11	KEONJHAR	KNJ-2	Daitary Town(CT) Barbil(M) Balagada (Bolani) Town(CT)	Karo	4239 52586 11830	70 lpcd 70 lpcd	0.0034 0.0521 0.0096	10189 126396 28435	70 lpcd 135 lpcd 70 lpcd	0.011 0.257 0.030
12	KENDRAPARA	Ken-2	Kendrapara Town(M) Pattamundai Town(NAC)	Brahmani near Jenapur	41404 32724		0.0463 0.0138	99519 78656	135 lpcd 135 lpcd	0.202 0.160
			G.Total				0.9400	29.6850		4.620

(Source : Ibid, Annex.7.13)

CHAPTER-III

Project Construction

3.1 Layout of Dam, Spillway and Power House

The layout of a gravity dam depends on several factors, the most important being the actual foundation conditions at the proposed site. Other determining factors include type of dam contemplated, type of spillway and its capacity to discharge the designed flood, location of the power house and other appurtenant works. Favourable configurations at the proposed site are rare gift of nature, and if favourable storage can be developed much more economically than otherwise.

Infact, the major determining factors in the selection of dam site are valley shape and foundation conditions. The other considerations are (i) availability of construction materials within a reasonable distance (ii) suitable spillway arrangement (iii) water-tightness of the reservoir rims (iv) communication facilities, (v) area of submergence and (vi) availability of men and machineries etc.

After detailed contour survey of the proposed location and series of drill holes, ground contour and rock contours are considered for the final fixation of the dam axis. Under Sec. 2.1.3.2, the foundation condition has been described in detail which can be regarded as one of the excellent foundations for construction of gravity dam.

The total length of the dam including spillway is 1040 m. The spillway has been located in the right flank of the gorge avoiding the deep channel. The maximum probable flood discharge is 55,540 cumec and routed flood discharge is 46970 cumec. The radial gates are of size 15.50 m x 14.8 m. This is one of the largest size of radial gates same as **Ukai Spillway** gate in Gujarat. The spillway has been provided with two differential bucket invert considering the topography of the rock profile. The layout plan with special features is described below in Table No.3.1.

Table No.3.1 Lay-Out Plan

Sl. No.	Components	Length in m.	Maximum height in m.	Special features.
1.	Left Dam-Block No.1 to 9.	201	16.50	Maximum depth of excavation is 30 metres.
2.	Power Dam-Block No.10 to 14	105	53.50	Located above summer water level.
3.	Deep Channel Dam-Block No.15 to 19/1.	85.75	68.50	Work is dependent on river diversion and Cofferdam. Block No.16 is Instrumentation Block.
4.	Lower Bucket Spillway Dam-Block No.19/2 to 33/1.	271	34.20	Bucket Invert at El. 83.50m. Block No.23 is instrumentation block. Block No.21 is construction sluice Block.
5.	Upper Bucket Spillway Dam-Block No.33/2 to 43/1.	193	28.20	Bucket Invert at El.87.25m.
6.	Right Dam-Block No.43/2 to 52.	184.25	44.50	Block No.43/2 is Irrigation Sluice Block.

3.1.1. Excavation of Foundation for Dam :

After finalising the alignment and lay-out of dam base including bucket, excavation of foundation commenced during April 1974 from Block No.33 to 45. Subsequently excavation in other blocks were taken up. Excavation for Diversion channel started in December, 1975 and that of Power House during December 1976.

Stoney earth and boulders excavated from top layers were utilised for filling and levelling the depressions, undulations for a length of 100 m on both upstream and downstream to obtain working space as well as to construct service roads for facilitating the work. As the excavation deepened, haul roads to the bottom of the foundation trench were constructed to transport excavated materials by mechanical means. Simultaneously, construction of both side hill roads were developed utilising the excavated materials without stock piling it. Dumping yards were selected to stock pile the usable stones for crushing to aggregates for use in the dam.

3.1.1.1 Rock Excavation:

Rock excavation in hard granite was carried out by resorting to blasting followed by barring and wedging. Jack hammers driven by compressed air were employed for drilling holes. A 40T capacity magazine building was constructed near village Khindo for safe storage of explosives. Where depth of excavation exceeded 10m i.e. from block 42 to 45, derrick cranes were deployed to lift the excavated materials and loaded directly to tippers for transportation of muck.

After construction of hill roads on either side, foundation excavation were taken up from blocks 1 to 9 and 46 to 51. Right side of the dam was easily accessible but left side was unapproachable due to deep channel for which machineries and P.O.L. etc. were ferried. This was a hindrance and therefore hampered the progress. But nothing is insurmountable. It was seen that in the down stream of Hatiadanda village, the river was shallow and flat with depth of flow varying from 1.2m to 1.5m during non-monsoon months. An economical and faster method was devised to cross the channel. Piers were constructed by filling the in-situ wooden crates with stones, over which wooden slippers were laid to serve as deck. This is how, the left bank became accessible which accelerated progress.

Local skilled and unskilled workers were engaged for excavation work. But for wedging and barring, skilled stone cutters were imported from Madras (Tamilnadu).

3.1.2 Excavation in Deep channel Section:

For excavation in deep gorge and removal of muck, 3 numbers of derrik cranes were installed on the bank on built-up masonry platforms. The muck and sand were lifted by these cranes and were delivered to tippers. Special self unloading steel buckets of sizes 2.50 m x 2m x 1.3m and 3.0mx2.5mx0.50m were manufactured to carry mucks from deep channel and hoisted by cranes. Bull-Dozers were deployed in deep channel to push mucks which were lifted by cranes. The foundation consisted of cup-shaped massive sound rock which was very ideal for dam foundation. No fault or shear zone was noticed in the deep channel.

3.1.3 Foundation Grade Rock Levels (F.G.R.L.) for different Blocks:

Basing on the drill data submitted by the project, CWC tentatively fixed the FGRL for dam blocks and buckets which were modified later, on exposure of the foundation. Accordingly final construction drawings were prepared by CWC considering the new FGRL. As stated earlier, bucket inverts are at two different levels. Now the lower bucket invert was fixed at EL 83.50m and the upper bucket invert at EL 87.25 m.

3.2 Dam Foundation :

“The axis of Rengali Dam has the bearing N70°W-S70°E and as per the designed lay out plan, its foundation is covering 52 blocks of varying width. Length of each block measured along the axis is generally 19.5m to 20m, except for a few end blocks on both the abutments. Blocks constituting the different sections of dam are mentioned below :-

- a) Left Non overflow Section (RD 15m - RD 376.75 m) - Block No.1 to Block No.18 and part of Block No.19
- b) Overflow Section (RD 376.75 m - RD 853.00 m) - Part of Block No.19, Block No.20 to part of Block No.43
- c) Right Non overflow Section (RD 853 m - RD 1025 m) - Part of block 43 to Block 52.

Overflow section is separated from non overflow sections by left and right training walls at RD 376.75 m and RD 853 m respectively. Powerhouse is located in between Block 10 and Block 14, on the left non overflow section.

Excavation of dam foundation has been carried out by blasting to the levels and shapes as per specifications and design drawing - where foundation grade has been marked based on the interpretation from drill - hole data. This has been verified from time to time in order to fix acceptable foundation grade which is met with invariably at a very shallow depth below ground except for the portions between Block No. 8 & 4 on the left and 43 and 47 on the right abutment where pyroxene granulites and norites have respectively been weathered to deeper levels due to differential spheroidal weathering.

After completion of rough excavation by blasting, scaling and trimming by chiseling, wedging and barring were taken up for final removal of thin slabby, weathered and loose rock mass. All the smooth surfaces were roughened and final foundation seems to be free from steep surfaces or sharp projections by means of proper benching.

Foundation excavation in deep channel portion of the river between block Nos.15 & 17 was taken up by diverting its flow through a diversion channel in Block No.21 and providing upstream coffer dam by means of both sheet piling and construction of a concrete wall. All the shears, semi vertical and open joints were excavated to the desired level to be filled with concrete or rich mortar.

Prior to placing masonry, foundation was moistened and coated by brush with a thick cement slurry (1 cement : 0.66 water by volume). This slurry is spread only on a small area of 1 sq.m at a time and mortar is spread immediately thereafter to a thickness of 50 to 75 mm.

Block wise geology of few blocks is given in the following paragraphs :-

3.2.1 Block wise Geology :

Block 0 :

This is an extra block beyond Block No.1 having a steep vertical end face (12 m high). Vertical surface shows either gently dipping (25°) or vertical joints in charnockite. Besides these, there are some cross fractures. A shear zone was noted at the end face which did not continue to foundation. A number of vertical holes and two inclined holes (15° to horizontal) were suggested for grouting.

3.2.1.1 Left N.O.F. Blocks :

Block 1 : (R.D.0m - R.D. 10m & upto 10 m d/s) :

In this block, 70° upstream dipping shear zone was noted in charnockite bedrock. Besides, this shear zone, vertical and sub vertical joints have been noted. Shear zone was washed, properly cleaned and backfilled with concrete.

Wall Section : In section of middle wall or benching at left hand of Block No.1 - two horizontally disposed open joints were found above foundation level. Three sub-vertical joints truncated by joint at 2.5 m, 5 m and 11.5 m from upstream face were recorded. Other curvilinear joint was noted from 7.5m to 4.2m towards top from upstream face.

A conchoidal fracture has been exposed between upstream face and 5.5 m and 7 m downstream. A slightly inclined fracture zone was recorded from upstream face to 2 m downstream.

Cross Section between Block 0 and 1

Benching at R.D. 0 / end of Block 1 towards left :

Two sets of joints were exposed on the wall section :-

- a) Vertical to sub vertical
- b) Sub - horizontal at 1 to 3 m from foundation level of Block No.1, a few grout holes in a cross pattern were suggested initially along the middle lines of the rectangular block at R.L. 123.5 m. Three inclined holes (15° - 20°) and three vertical holes were drilled for grouting.

Block 3 (R.D. 32 m - R.D. 54 m, 0-13.5 m d/s) :

Hard, massive, medium-grained intermediate charnockite is the main rock type. A major shear zone extending from Block 2 at R.D. 32 m to R.D. 40 m with 5 m d/s to 3 m d/s is prominent. There are a number of major parallel joints running in NE-SW direction and placed between R.D.32 m and R.D. 34 m (0-9 m d/s). Rockmass in this part shows development of rudimentary foliation. There are two sub parallel joints at right angles to this set from R.D. 34 m to R.D. 40.5 m dipping at 65° towards downstream. Three minor shears have been noted :-

- a) R.D. 40 m - R.D. 47 m (4 m - 10 m d/s) with NE-SW trend.
- b) R.D. 46 m - R.D. 54 m (2 m - 8 m d/s)
- c) R.D. 39 m - R.D. 49 m (0 m - 4 m d/s).

For this block No.3, shear zones were excavated upto a depth of $1/3^{\text{rd}}$. The width of each shear zones were back filled with concrete after proper cleaning of gougy materials from shear zones. Routine grouting was also done to seal off any chances of leakage through foundation. Angle holes put across the shear zones at various angles (10° - 15°) was proposed to intercept the zone at various levels. Proper washing and thorough grouting was suggested before concreting.

Block 4 (R.D. 54 m - R.D. 76 m, 1-22.74 m d/s) :

At the foundation grade (R.L. 95.5 m - R.L.99 m), intermediate charnockite with few xenoliths of pyroxene granulite forms the rock type. Major occurrences of these xenoliths were noted at 3-8 m d/s showing their contacts to be sharp, tight but sheared by NE-SW trending shear zone, at places forming platy rockmass. Charnockite is traversed by irregular, tight / moderately open steeply dipping / vertical joints, and shows low dipping exfoliated surfaces at places. In addition to removal of loose and overhang rockmass from foundation by wedging and barring and limonite staining by Chiseling the shear zones, master joints and exfoliated surfaces were treated. Shear zones between R.D. 61 m - R.D. 62.3 m (2.1 - 2.5 m d/s) have been excavated upto firm rock. Two westerly dipping joints at R.D. 54 m (5.5 m - 4.3 m d/s) and R.D. 58.8 m - R.D. 59.2 m (12 m d/s) - where loose, weathered rockmass appears to persist for a considerable extent - which have been excavated down preferably twice of its width and back filled with concrete after washing. Pyroxene granulite (R.D. 73 m, 5.6 m d/s to R.D. 76 m, 2.5 m d/s) along a vertical joint and extending upto Block 5 is disintegrated and scooped out to firm rock. Exfoliated layers have been removed and properly anchored to firm rock. Dowel bars were provided at vertical rock face at dam toe.

Block 7 (R.D. 120 m - R.D. 142 m, 1.8 m - 33 m d/s) :

Foundation level varies between R.L. 81.3 m - R.L. 83.58 m. This block at the foundation level shows presence of intermediate charnockite with xenoliths of pyroxene granulite with sharp and tight contact. Foliation trends in N 60° W - S 60° E direction with 75° - 80° dip towards north/south. Rocks are traversed by a few irregular, tight, vertical/steeply dipping joints. A foliation parallel shear zone was observed at R.D. 120 m (19.7 - 20.5 m d/s), R.D. 131 m (15 - 16 m d/s) and R.D. 142 m (12 - 12.6 m d/s) and continues in Block Nos. 6, 8 & 9. Charnockite here is crushed and augens and pyroxene granulite invariably weathered form pockets of brownish coloured clay mixed with rock fragments.

Two such weathered pockets at R.D. 132.5 m (14.5 m d/s) to R.D. 134.5 m (15 m d/s) and R.D. 122 m (18.5 m d/s) to R.D. 120 m (19.7 m d/s) were found. Sympathetic shears at the north eastern corner at R.D. 128 m (0.3 m d/s) - R.D. 120 m (4-4.4 m d/s), R.D. 122.5 m - R.D. 120 m (1.5 m d/s) were common. At the contact of charnockite and pyroxene granulite, rock is crushed to platy rockmass at R.D. 141 m (29 m d/s) and R.D. 120 m (30.5 - 31.5 m d/s). Removal of loose, platy, weathered/ disintegrated rock-mass, gougy material, excavation & washing of shear zones and open joints have been done. Overhang rock-masses were properly trimmed and rested surfaces were chiseled out. Low dipping open exfoliated surfaces have been properly anchored to firm rock.

Block 11 (R.D. 207 m - R.D. 228 m, 10.1 - 10.9 m d/s) :

The area is made up of charnockite with small patches of pyroxene granulite. Pegmatite veins have also been noted at R.D. 216 m - R.D. 218 m (0.5-1 m d/s) and R.D. 221.5 m - R.D. 226 m (3-5 m d/s). Two shear zones trending WNW-ESE and NW-SE at R.D. 211 m - R.D. 226 m (8.5 - 10 m d/s) and at R.D. 217.5 m - R.D. 228 m (5 m d/s) have been recorded. The foundation grade rock is dissected by two sets of sub-vertical joints trending NW-SE and NNE-SSW. All patches of pegmatite and pyroxene granulites were chiseled out and scooped out. Shear zones were excavated and back filled with concrete and grouting was done.

Block 14 (R.D. 270 m - R.D. 291 m, 12.5 - 19 m d/s)

Charnockite is the main rock type of the block.

The following two weathering planes have been noticed :-

- a) R.D. 270 m - R.D. 276 m (9 m d/s). Dip 40° towards NW and shows ferruginous coating.
- b) R.D. 283 m - R.D. 289 m (12 m d/s). Dip 47° towards south.

Joints are mostly vertical. A 'V' shaped intersecting joint pair has been found due to intersection of joints dipping at 10° - 15° towards deep channel. Grouting was done to make the rock mass monolithic.

Block 17 (R.D. 328 m - R.D. 343 m) :

The block is made up of charnockite with lenses of pyroxene granulite and is free from any major defects. Exfoliation weathering between R.D. 328.5 m - and R.D. 332 m (9 m d/s) and between R.D. 333 m - and R.D. 337.5 m (18 m - 21 m d/s) was common. Scaly weathering was noted between R.D. 335.5 m and R.D. 341 m (12 - 16.5 m d/s) which shows ferruginous coating. Area bounded by R.D. 328.5 m and R.D. 334.5 m (12 - 13 m d/s) shows a depression forming part of an erosional channel. There is a NE-SW trending discontinuity plane and the area is having large number of pot holes.

Discontinuity planes between R.D. 336.5 m and R.D. 343.5 m (35 m - 35.5 m d/s) are i) ENE-WSW and ii) E-W trending. Both the joints are steeply dipping or nearly vertical. Sheared rock and minor pyroxene granulites were found between R.D. 336.5 m and R.D. 343.5 m (35 m - 35.5 m d/s). Discontinuity extending from R.D. 322.5 m to R.D. 328 m (24 - 29.5 m d/s) shows leakage with scaly infilling of crushed and sheared rock material. Sheared and shattered rock mass and pyroxene granulites were scooped and back filled with concrete. Depressed area forming part of erosional channel of river having a large number of pot holes and smooth rock surfaces were roughened. Chipping of ferruginous coating was done.

3.2.1.2 O.F. Blocks :

Block No.27 (Between R.D. 521 m - R.D. 540.5 m) :

Massive charnockite is the main rock type. Two isolated patches show little weathering at foundation grade. The bigger patch is almost a rectangular shaped and is restricted between R.D. 526.5 m (13.20 m d/s) and R.D. 527.5 m (6.6 m d/s) and between R.D. 539.5 m (12.7 m d/s) and R.D. 540 m (18.5 m d/s). This happened due to play of two sets of vertical joints along which planes percolating water has influenced deep peripheral weathering. Inner mass of rectangular area was found to be fresh and hard. Weathered materials along the periphery were removed and back filled with concrete and suitable anchors. Fresh central charnockitic mass was fixed with the floor and grouted to consolidate. There was another small patch of weathered charnockite between R.D. 542 m (5 m d/s - 23 m d/s) and R.D. 520 m (6 m - 21 m d/s) having basal jointed layers. This patch had been removed from the foundation. Close to bucket area, there was another circular patch of exfoliated basal jointed

mass, which were also removed from foundation. Most of the joints encountered in the foundation grade level were discontinuous, tight and incipient. In the bucket area, two sets of tight joints were seen to dip at a low angle towards both north and south. Upstream wall is almost massive. Routine consolidation grouting of 1/3rd of the dam base rendered the area to be water tight and monolithic.

Block 33/1 (R.D. 638 m - R.D. 649.15 m, 30-44 m d/s) :

The right edge of the lower bucket and divide wall foundation has been excavated down to R.L. varying from 80.345 m to 82.040 m which comprises of intermediate charnockite and a 3 m wide intrusive norite trending along NNE-SSW direction at divide wall part. Intrusive body tapers down in the body wall part of the block and its contact with host rock was very tight and fused. Charnockite at foundation grade exhibits a few exfoliated surfaces dipping 25° towards north, and tight, vertical joint trending along NE-SW direction, especially at the south-eastern corner. Norite is well jointed by closely spaced, tight joints which form more or less two well defined sets trending along i) NNW-SSE direction with 50°-80° dip towards N/S ii) NE-SW direction with vertical / steep northwesterly dip. Rocks were quite fresh and hence removal of only loose rockmass was carried out before concreting.

Block 35 (R.D. 677 m - R.D. 696.5 m, Offset 33-50.585 m d/s) :

Average foundation level at the Block 35 was R.L. 85 m where intermediate charnockite with xenoliths of pyroxene granulite forms the rock type. Their contacts were sharp and tight. A few isolated mounds of charnockite with a NE-SW trend with 45°- 60° dip towards NW and traversed by a number of mostly tight vertical / sub vertical joints of which NE-SW trending set are prominent. One the downstream wall, overhanging rock-mass though fresh were met with between R.D. 682 m and R.D. 691 m. This unsound rock-mass was treated with side anchor bars provided to the firm rock for better grip. A few low dipping joints separating the top rock layer from firm rock below were seen at R.D. 681.7 m (34.5 m d/s), R.D. 680.5 m (38.5 m d/s), R.D.690.6 m (36.2 m - 36.4 m d/s) and R.D. 695.5 m (40.2 m d/s). These have been tied down to underlying firm rock by means of 2/3 additional anchor bars. Side anchor bars were provided at the deepest portion of the trench excavated along master joint between R.D. 695 m and R.D. 695.5 m (42-42.3 m d/s). These were tied down to underlying firm rock by 2/3 additional anchor bars. Side anchors were also provided at deepest position of trench, excavated along master joint between R.D. 695 m and R.D. 695.5 m (42-42.5 m d/s). Joint has an opening of 5 cm at the bottom of the trench. These were washed and filled up by cement mortar before concreting. Besides these, removal of loose, overhanging weathered rockmass and surface staining by wedging and barring and chiseling were done.

Block 43 (R.D. 833 m - R.D. 853 m, 32.16 m - 51 m d/s) :

Intermediate acid charnockite with norite intrusive is intercepted at the foundation grade R.L. 84 m - R.L. 85.190 m. Contact between these two litho - units is sharp and tight. Charnockite has a few exfoliated surfaces which is vertical / steeply dipping and tight in nature; Irregular joints in Charnockites have limonite stains but norite on the other hand is traversed by closely spaced joints with loose, partly weathered rock-mass at places.

The suggested treatment comprises removal of loose rock-mass by wedging and barring and surface staining specially at R.D. 833 m - R.D. 839 m (42.16 - 49.16 m d/s) by chiseling. Decomposed rock-mass met with along joint planes of norite at R.D. 854 m (35.694 m d/s) - R.D. 856 m (36.194 m d/s) and R.D. 854 m (50.394 m d/s), R.D.860.5 m (46.994 - 47.694 m d/s) were scooped out and joint planes were thoroughly washed by water jetting. In body wall part at 0 m - 28.75 m d/s and irrigation sluice between R.D. 840.75 m - R.D. 853 m and off set 0-12 m d/s of axis, contact of charnockite and norite is very sharp and tight at foundation grade which is at R.L. 81 m, on an average, except for some localized patches, where deeply weathered norite was met with at a deeper level i.e. below R.L. 79 m.

Foundation grade rock was traversed by a number of sets of tight joints having 40°-50° dip both towards north and south and some of them were vertical. Rock was often coated with yellowish brown/ yellowish green decomposed product of norite along these joint planes. Water loss test indicates that the foundation is mostly water tight.

3.2.1.3 Right N.O.F. Blocks :

Block 46 (R.D. 893 m - R.D. 903 m, (-) 2.5 m - 31 m d/s) :

This block has been excavated down from R.L. 83.09 m to R.L. 84.17 m and reached the foundation grade where norite with a caught up patch of charnockite forms the bed rock. Their contacts appear tight and sharp. Norite was traversed by closely spaced tight, sub-vertical/vertical joints. One master joint trending N 30°E - S30° W direction with 55° easterly dip cuts across the dam foundation at R.D. 896.3 m and has well defined connection from upstream - downstream wall. A thin clay seam derived from decomposed norite was noted all along the joint plane at the excavation level and it's openness at depth was deciphered by water percolation test in vertical and angle holes drilled along joint plane. It is interconnected with low dipping joints which appear to continue for some distance in the adjacent block 45. In order to remove clay and weathered rockmass from joint surfaces it was recommended to open up a trench by light and selective blasting using gun powder in shallow holes drilled at an angle opposite to the direction of dip of joint plane. Care was taken to ensure that the blast holes did not puncture the joint plane. This was followed by washing and grouting in order to plug the possible seepage path. For this, a few angle holes were drilled to intersect the joint for ascertaining its openness at different depths. Scaling operation along joint plane was done by chiseling for better grip of concrete with rock face. Loose and overhanging rockmass were removed by wedging and barring. Water loss of the order of 44-45 ltr./minute was observed at a depth of 3 m from excavated surface. The holes were washed and grouted the zone to make the foundation monolithic.

Block No.49 (R.D. 953 m - R.D. 963 m (-) 1.5 - 13 m d/s, R.D. 963 m - R.D. 973 m (-) 1.5-5 m d/s) :

This block has been excavated down to R.L. 105.3 m on an average to reach foundation grade where charnockite with xenoliths of pyroxene granulite occurs. Charnockite was traversed by a few, tight vertical / steeply dipping joints and low (10°) easterly plunging exfoliated surfaces. Due to heavy blasting, rock had become loose at places along joint planes. These were removed by wedging and barring especially at R.D. 967 m (0.5 m - 3 m d/s), R.D. 970 m (1-2.5 m d/s) - where loose rock mass ranging in thickness from 0.2 - 0.5 m was noticed over a rusted exfoliated surface. Loose rock mass from exfoliated surface / joints were also removed. Rock at foundation grade maintaining steep slope at places was suitably cut to provide the key for surface. At R.D. 953 m (21.3 m d/s) a transverse joint extending for some distance into foundation from downstream wall got connected with horizontal / sub horizontal joints and shows deeply stained surface. Loose rock mass along the transverse joints were removed for avoiding seepage.

Block 51, R.D. 993 m - R.D. 1013 m :

Charnockite and norite are the main rock types of Block 51. Norite is at places sheared. Sheared part has been removed.

Deep channel Portion Closure of deep channel or plugging of the main canyon of river Brahmani comprising Block nos. 15 and 16 called for a major operational hazard in dam construction across river. Geologically the origin of deep channel was attributed to the coalescence of a number of potholes. Initially it was assumed a fault plane along the inner canyon. This assumption was rejected by 45° dipping angle hole which did not encounter any fault plane. River channel eroded out solid charnockite by abrasion of river current which is ladden with BHQ and BHJ pebbles quartz boulders and rock fragments. These are assumed to have given rise to huge potholes (6 m) in riverbed. Some caverns and potholes on river bed and on the walls of canyon was formed by scouring of the weathered, softer pyroxene granulite lenses, inclusions in charnockite.

Normal flow was diverted through a construction sluice through Block No.21 with the help of an upstream coffer dam. Dam in deep channel was raised to R.L.77 m. Dewatering and muck clearance was completed in 15-20 days. Dewatered volume comes out to be around 156000 cum. Total concreting in Block Nos. 15, 16, 17 upto E.L. 77 m was estimated around 31500 cum. Difference of river bed and bank is 16.5 m (R.L. 59.5 m to R.L. 76 m)."

(Source : 'A comprehensive case history of Geotechnical Investigation of Rengali Dam Project' - Geological Survey of India, Bulletin Series B No.65, 2013 pg.20-32)

3.2.2. Shear Zone:

During excavation from block No.32 to 34, a shear zone of 1.50 m to 2.00 m wide was noticed. The zone continued across the dam extending from upstream to downstream with an inclination of about 80° to the dam axis. The intervening joint space was found to be filled with shattered rock of little blackish colour. This Norite zone was prominently visible from upstream to downstream sandwiched between granite vertical planes. On excavation it was found that the shear zone which was about 2m wide at top had gradually narrowed down to almost to an insignificant seam at about 3 m below FGRL. Norite core samples were taken out for testing. Basing on the test results, it was finalised to excavate the shear zone to a depth twice the base width and wash the remaining joint below. Grout pipes were left at suitable locations for pressure grouting at later stage, after sufficient loading of masonry/concrete over the foundation. Similar treatment of foundation was made for all rock joints through out the length of the dam where encountered whether large or small. After proper washing of joints it was back-filled with M20 concrete, leaving pipes for injecting grout at later stage.

3.2.3 Foundation Treatment:

The designs are generally prepared basing on the geological reports and proper logging and interpretation of the drill holes at the investigation stage. Exploratory drilling may not be fully representative one to reveal the correct foundation conditions. The foundation grade rock comes to notice after exposure of the foundation i.e. after fixing the F.G.R.L. The specification drawing and design undergo change and accordingly unusual features or weaknesses i.e. shear, fault, fissures, cavities, seams etc. are treated as untreated foundations are unsuitable for construction of dams, reservoirs, tunnels and power house etc. The operation to correct the deficiencies in foundation conditions is known as pressure grouting. The foundation under or adjacent to the structure is grouted primarily for following reasons:

- (i) To solidify and strengthen the formation for increasing its capacity to support the designed load.
- (ii) To reduce or eliminate the flow/seepage of water through a formation, i.e. under a dam or into a tunnel.
- (iii) To reduce the hydrostatic uplift pressure under a dam.

After a foundation has been explored and tested to determine the extent of grouting required, a drilling pattern is decided. Accordingly the size, depth and spacing of injection holes are designed to give the best results at the lowest cost. The drilling pattern so fixed may undergo change if grouting operations encounter differences in foundation conditions. The C.W.C. suggested the drilling pattern for consolidation and curtain grouting and for drainage holes. Drainage holes were drilled downstream from the grout curtain with a view to intercept seepage through the curtain and thus to relieve uplift pressures. These holes also offset the difficulty in grouting if any. The drainage holes were drilled with Nx bits (75 mm dia). Accordingly, C.W.C. provided the following recommendation to be adopted for Rengali Dam, as regards drainage holes.

Block No.	Depth of 'A' holes(m) from foundation	Depth of 'B' holes(m) from foundation	Depth of drainage holes from foundation (m)
1 to 5 & 45 to 52	25	10	18
6 to 14 & 19 to 44	30	10	22
15 to 18	40	10	30

The centre of the drainage hole has been fixed at 0.23 m from the back face of the drainage gallery. Drainage holes were drilled after completion of grouting operation in the vicinity in order to avoid chocking of these holes. The holes were drilled from the floor of the drainage gallery. Though 10 cm dia black steel pipes were embedded in masonry or concrete below the floor level, the drainage pipe connecting the drainage gallery floor is also connected to the gutter with 5 cm dia G.I. pipes.

3.2.4 Grouting:

The method of treatment of grouting mainly depends upon the geological feature of the foundation materials. The grouting treatment as suggested by C.W.C. on the basis of field data was meticulously observed.

3.2.4.1 Consolidation Grouting, Curtain Grouting & Drainage Holes:

Consolidation grouting, Curtain grouting and drilling of drainage holes were done in the dam foundation and in foundation gallery as per specification. The details of these have been described as under.

Details of grouting done in foundation

“Drilling Consolidation grouting holes: For percolation test and consolidation grouting; bore holes 48mm diameter and 10m depth were drilled by wagon drills.

Four rows of holes were drilled in overflow section. First row is 1 m away in the direction of downstream from dam axis and similarly also 2nd, 3rd and 4th runs are placed at 7.0 m, 13.0 and 19.0 m away from dam axis respectively. Holes are staggered and drilled at 6 m C/C.

Only 3 rows of staggered holes have been drilled in non-overflow section at 6m C/C.

After drilling the holes to the required depth, these holes were thoroughly washed by air water jetting to remove all the erodible particles so that no difficulties are encountered during percolation test.

3.2.4.2 Percolation test: Water percolation test was carried out to-

- i) measure the effectiveness of grouting treatment and
- ii) guide the initial grout mix as per IS 6066-1971.

This cyclic percolation test consisting of a series of simple tests performed in succession at the same stage (i.e., in 3 stages by using packers) at varying pressures usually in the ratio of $P/3$, $2P/3$, P and then $2P/3$ and P where ‘P’ represents maximum safe pressure for the stage or 10 kg/cm^2 whichever is less.

Procedure: Stages are divided as per the following:-

- a) first stage from 6.1 m to 10.0 m
- b) second stage from 3.05 m to 6.10 m and
- c) third stage from 0.0m to 3.05 m

Water is allowed to pass under pressure of 4.22 kg/cm^2 , 2.82 kg/cm^2 and 1.41 kg/cm^2 for 1st, 2nd and 3rd stages respectively. For first stage single packer is used at 6.1 m below the surface and for 2nd and 3rd stage double packer is used.

3.2.4.3 Consolidation Grouting: Consolidation grouting generally consists of grouting shallow holes usually at relatively low pressure for the purpose of consolidation and impermeabilisation. This was done in 3 stages by using packers, usually before laying concrete or masonry. GI pipes were provided for grouting after foundation is loaded. Ordinary Portland cement has been used for this purpose without any admixture. In Rengali Dam, grouting was taken up for all the ‘leaky’ holes till refusal and allowed to remain under pressure for a period of half an hour to one hour. Usually a lean grout mix of 10:1 W/C ratio was used and gradually increased to 3:1 for larger voids and cavities.

Leaky joints with cavities were found in the bucket of Block No.37. Grouting was done at 1.41 kg/cm^2 pressure and grout intake was 264 kg. To check the effectiveness of grouting, 5 Nos. of holes 25mm diameter and 2.5 m depth were drilled by jack hammer and percolation test was conducted. Out of these 5 holes; there was leakage of 0.8 liters/minutes to 3.2 liters/minutes in 2 Nos. of holes. This zone was regouted through these leaky holes and grout intake was 40 kg. of cement.

Bucket foundation bed of block No.33/2, 34 & 35 consisted of laminated sheet rock with voids. Grout intake for block no. 33/2 was 1460 kg and for each of block 34 and 35 was 1685 kg of cement. Effectiveness was tested through drainage holes of block 33/2 and the water loss was nil at 1.41 kg/cm² pressure.

3.2.4.4 Curtain Grouting: Curtain Grouting is required where dam height exceeds 30 m and water absorption exceeds one lugeon. Holes were drilled at an angle 13½° to the vertical, towards upstream of dam at an interval of 3m C/C. These were drilled from gallery at an off set of 3.5m downstream of dam axis. Depth of hole was fixed depending on hydraulic head. For drilling of grout holes and drainage holes, underground diamond core drilling machine has been used. Sizes of diamond bits used are A_x, B_x and N_x of 48mm, 64mm and 76mm respectively. During concreting of drainage gallery, 75mm diameter steel pipes of 1 m length were embedded at an off set of 3.5 m from dam axis at an inclination of 13½° to the vertical, at an interval of 3m C/C in gallery floor. Curtain grout or 'A' holes are drilled in different blocks leaving a gap of 1.5m, 1.5,, 1m, 0.75m, 0.75m and 0.5m from block joint, for the blocks having length of 21m, 15m, 20m, 19.5m, 16m and 22m respectively.

Due to limited space available in gallery, practical difficulties were faced to drill 'A' holes at 13½° angle through embedded pipes in gallery floor with underground diamond core drill machine. For which 'A' holes are drilled just near the embedded pipes where such type of difficulties were faced at an angle of 13½° to vertical.

Washing of 'A' hole were taken up with water under pressure to its full depth. Depth of hole is then measured as per drill to confirm that the hole has been cleaned fully. After necessary cleaning, water percolation test was made to measure the effectiveness of grouting treatment by simple percolation test. A simple percolation test involves isolating a segment of hole of 3 to 5 m in length, by means of a single / double packer and pumping in water at a steady rate and constant pressure for a period of 5 to 10 min. In Rengali Dam water testing of 'A' holes were made at an interval of 10 ft, from the bottom of the holes to the top (i.e., 100', 90', 80' etc.).

For curtain grouting and water testing at Rengali Dam - high pressure reciprocating water pump, air driven pneumatic reciprocating pump and electric driven influx group pump were used.

Testing pressure for curtain grouting and water testing was calculated as follows:-

$$P = \frac{\text{MWL} - \text{Packer Level} \times 1.75 - \text{Static Pressure}}{10}$$

Out of the two type of grouting system i.e., single line system and circulating system, circulating system has been adopted at Rengali Dam for curtain grouting and water testing. Thus two pipe lines have been provided which is a supply line from grout pumps to grout hole and return line from grout hole to feeding drum i.e. water cement mixer drum. By opening the supply and grout-hole valves, grout is forced into the hole as required and pressure is maintained by adjusting either the supply valve or return valve or both. A pressure gauge is fixed at delivery end of pump. During the grouting operation, cement water proportion is decided keeping in view the water testing result of the same hole. Cement: Water proportion varies from 1:10 to 1:3 depending on the rate of grout to be taken.

Drainage holes: Main function of drainage hole is to reduce uplift pressure by releasing the concentration of water under foundation. After completion of curtain grouting in a block, a series of drainage holes or 'D' holes have been drilled from foundation gallery floor. Machines used to drill the drainage holes are the same machines used to drill the curtain holes or 'A' holes. Location of drainage holes were fixed at an off set of 4.82 m d/s of dam axis and accordingly 100 mm steel pipes of 1 m length with a Tee at top of size 100 x 100 x 50 mm was embedded from vertical drainage pipe. A 50 mm diameter steel pipe was connected to that Tee which leads to gallery drain so that the released water from drainage hole will flow through 50 mm diameter pipe to gallery drain. Drainage guide pipes are embedded in between two 'A' holes so that centre to centre of drainage hole guide pipe will be at 1.5 m. 'D' holes have been drilled at an off set of 4.25 m downstream of dam axis. Total 14144 m drainage holes and 10395 m curtain grouting holes have been drilled. (Source: Ibid Pg.32-34)

3.2.5 Foundation Drainage:

Bucket Drainage and Anchorage Arrangement:

CWC provided drawing showing net-work of drainage arrangements. The net work consisted of drainage holes in a grid of 4m x 4.50 m with diameter of holes 75mm drilled to a depth of 4.00 m below F.G.R.L. After excavation was completed, the drainage holes were drilled in the above mentioned grid by wagon drills. The holes were washed with air and water jets and were filled with pea gravel and bajri specially collected from the river bed for the purpose. After the holes were filled, a subgrade of drain was constructed for a width of 60 cm along the drain lines. The drain lines were covered with 30 cm dia semi-circular galvanised corrugated pipes. Semi-circular hume pipes have also been used in four blocks. The ends were then properly sealed. The net work of drainage pipes were led to 4 Nos. of outlet pipes of 13.5 m length for each block. The outlet pipe consisted of 15 cm dia G.I. Pipe emerging beyond the tip-wall of the bucket. Before taking up bucket concreting, the net work of drainage arrangement were completed.

In the event of drainage arrangement getting clogged, it is likely that uplift pressure may develop. To counter against uplift it was necessary to hold down the bucket by providing adequate steel anchor bars to withstand the differential head.

C.W.C. approved a drawing showing details of bucket anchorage. It consisted of anchor of 36mm dia M.S. bars spaced at 1-0 m c/c which were inserted in 75mm dia holes drilled to a minimum depth of 2.50 m below F.G.R.L. and grouted with rich mortar according to the pattern indicated in C.W.C. drawing. All the anchor bars were then tested which conformed to the design criteria.

3.2.6 Contraction joint & Water stop Arrangement:

The entire dam of length of 1040 m has been divided into 51 blocks and there are 50 joints. These joints are made to allow the contraction of the concrete on the two sides to relieve the thermal stresses. Water-stops are provided for stopping the flow of water through the joint and flow of grout outside it.

Before commencement of covering the foundation water-stop arrangement was provided with due grip below F.G.R.L. It consisted of a pit of minimum 0.46 m below F.G.R.L. The size of water-stop block is 2.00 m x 2.00 m of M25 grade concrete. The detail components of water-stop arrangement are as below:

1) First row of water-stop i.e. 20 gauge copper strip was provided at 0.60 m from the axis of the dam. The length of each leg as adopted is 22.5 cm. on either side from the joint. The copper strip was fixed to vertical by 25 mm dia anchor bars with horizontal bracing of 10 mm dia bar brazed to sealing strip.

2) The second line of defence consisted of a formed drain of size 125 mm x 125 mm, located at 0.69 m from the axis of the dam. The formed holes were pre-cast in holes and jointed together at site. Two formed pipes have been installed inside the asphalt well for melting asphalt by passing steam and adding more asphalt at a later period.

3) **Formed drains:** Further downstream i.e. 150 mm from the edge of the formed hole 200 mm dia trap-drains or porous drains have been provided and connected to the roof of the drainage gallery. This is an addition to the water-stop arrangement only above the gallery roof level. These drains are made of 'No-fines concrete' i.e. without fine aggregate. The pre-cast porous drains were joined piece by piece as the dam height is raised and spaced at 3 m centre to centre.

4) **P.V.C. Water Stop:**

As a last line of defence against seepage polyvinyl chloride water stops of tested quality were used. This was provided at a distance of 50 mm from the edge of the trap-drains embedded right below F.G.R.L. i.e. from the same level of upstream copper water-stop. The purpose of the seal is also to limit the travel of asphalt along the joint in between these seals and to make the seal more effective.

3.2.6.1 Longitudinal contraction joint:

Longitudinal joints are usually not provided in dams built of rubble masonry. That is why no longitudinal joints have been provided in Rengali Dam except a line of shear keys in overflow section between body wall and bucket. The shear key has been provided in shape of ducts and keys just outside the toe of overflow section.

3.2.7 Sequence of Construction:

3.2.7.1 Sequence in Bucket Concreting

The minimum depth of concrete with reinforcement as per approved drawing is 1.50 m below bucket invert. Thus for bucket invert at El. 83.50 m the minimum F.G.R.L. required is 82.00 m. In certain blocks, where foundation was excavated below the level, it was back-filled with M 20 concrete and raised to El. 82.00 m. Considering 1.5 m thick bucket concrete detail stepping pattern was decided such that radial thickness does not become less than 1.50 m. Concrete was laid in steps with M20 grade concrete prior to taking up proper bucket concreting. After concreting for steps was over, with proper drainage and anchorage arrangement, the reinforcement for bucket was tied in position with sufficient chairs and supports and thereafter concreting was poured. As per zoning drawing the bucket concrete consists of C3 concrete i.e. M₂₅ grade with minimum size of aggregate 75mm. But it was decided by the project authority, to replace with 42mm maximum size of coarse aggregate against 75mm.

3.2.7.2 Arrangement for Bucket and Spillway concreting:

For bucket concreting two arrangements were made, one for upper bucket and the other for lower bucket. A platform was made at the downstream of the bucket area with a vertical face at the upstream side and slope at the downstream side. The space between the excavation line and the vertical face of the plat-form was sufficient enough to accommodate the 15 Ton Derrick Crane and also narrow gauge lines laid for trolleys loaded with buckets. The Derrick crane was travelling parallel to the dam axis on the Broad Gauge lines. The Crane was placed as per the requirement of concreting. A number of concrete mixers of required capacities were installed on the plat-form. The materials like sand, chips, metals and cement were stacked on the slope at the downstream side. Mixers were fed with the materials by manual labour. From the mixers concrete was fed by chutes into the bucket of 2 cum. capacity loaded on the trolley. The loaded trolley was pushed manually to a point from where it would be convenient to lift by crane and place the concrete continuously in the bucket portion. 3 HP killic Nixon Electrical and pneumatic Vibrators with 75mm dia needles were used for compaction. M.S. pipe line of 8" (20mm) dia were running parallel to dam axis by the side of these plat-forms so that the water could be used conveniently for mixing concrete and for curing. For supplying construction water, two numbers of water tanks of size 12m x 8m x 5m each were constructed on hill top at EL 140m. After settling and treatment, water was pumped to the work site.

3.2.8 Arrangement for Construction of Masonry:

As soon as the foundation of the dam base was ready, it was covered with masonry after observing all formalities. Only in case of deep joints and crevices concrete was filled up over which masonry was taken up. For production of mortar number of concrete mixers of 10/7 and 14/10 cft. capacity were installed on the upstream side of the dam. The mortar was conveyed manually. Initially the mortar was laid over the exposed rock and the masonry construction progressed in accordance with specification. The stones quarried and transported by trucks from Salimunda and Jualibhanga quarries were stacked on the upstream side of the blocks. Stones used in masonry were clean, hard and durable free from veins, skin, cracks and other defects weighing between 75 to 150 kg. The arrangement was such that a layer of masonry was laid in a day in one block (i.e. before sun-set). But only in 1983, adequate lighting arrangement was made to accelerate the progress of masonry work during night time. For R.R. masonry in hearting portion 1:4 cement mortar and in facing and backing 1:3 cement mortar was used. The mortar was placed carefully such that no pocket or voids were left either at the bottom or in between the stones.

The downstream slope of overflow section is 0.90:1 tangent to the curve of equation $x^{1.85}=16.532y$. The thickness of slope concrete is 1.50 m for non-pier zone whereas for pier zone it is 4.50 m. The width of pier zone is 8 meter for each pier and thus for each block of 19.50 m, the pier zone is 8 M and non-pier zone is 11.50 m at the junction of masonry and concrete. To have the correct thickness of concrete detail stepping drawing of masonry was prepared considering width versus level. The width was arithmetically calculated making use of the downstream slope and the equation. Width of masonry for pier zone and non-pier zone were tabulated at 0.30 m level interval. Masonry was constructed accordingly thereby maintaining the thickness of concrete in the slope portion.

3.2.8.1 Arrangement of Scaffolding:

To carry the stones and mortar to higher elevation as masonry construction goes up and also for movement of workers and supervisors, scaffolding was required. For the blocks from No.14 to 51 the method of scaffolding provided by M/s. Odisha Construction Corporation (O.C.C.) Ltd., a Government of Odisha undertaking was different from the scaffolding used by several private agencies engaged for masonry work from Block No. 1 to 9. But all the agencies uniformly provided the landing at every 2.5 m lift with 3 m wide passage. The gangway was strong enough to withstand the load of workmen carrying mortar/concrete and R.R. stones. All the scaffoldings were erected with rigid Sal wood frame work adequately propped and braced with Sal ballahs, planks and sleepers using M.S. nuts & bolts.

3.2.9 Sequence in Top Ogee Concreting:

The surface of spillway both upstream and downstream was divided into the following components.

- i) Upstream vertical face upto El. 108.687 m
- ii) Upstream Ogee profile with Co-ordinates as per W.E.S. from El. 108.687 m to El. 110.20 m
- iii) Downstream Ogee profile from El. 110.20 to tangent point at El. 101.284 m with Co-ordinates as per equation $X^{1.85} = 16.532 y$.
- iv) From El. 101.284 m to El. 89.456 m in case of lower bucket with slope 0.9 (H) : 1 (V) and from El. 101.284m to El. 93.206 m in case of upper bucket with slope 0.9:1
- v) From El.89.456 m and 93.206 m to contraction joint, circular curve with radius 18.00 m.

Pertinent Offsets:

Pertinent points	Offset from axis in metre.
Axis	00
Upstream Pier edge	(-) 2.00 m
Crest at El. 110.20 m	3.244 m
System line to radial gates	5.397 m
Bridge end.	8.10 m
Upstream Gallery line	3.30 m
Upper Tangent Point at El. 101.284m	18.04 m
Pier edge El. 95.616 m	23.215 m
Bucket Tangent point at El.89.456 m of Lower bucket	28.759 m
Bucket Tangent point at El. 93.206 m of Upper bucket.	25.294 m

A profile platform was made at the dam site near right side training wall with good sub-base and top surface concreted and properly levelled. The full scale ogee surface was plotted over the platform. The entire ogee line was then divided into number of segments. The segments thus decided and approved were marked on the ogee line. The templates were then manufactured fitting to each segment. The templates were made from M.S. Channels of size 100 mm x 50 mm. The curved templates were made by alternate heating, cooling and tempering till it fitted to the ogee line in a particular segment.

The ogee finish was made with form work with straight template and straight shuttering plates upto Fl.101.284 m. From El. 101.284 to El. 103.00 m, curved template with straight shuttering plates of width 0.50m and then 0.30 m towards top were used. Beyond El. 103.00m unformed finishes were resorted to for ogee finishes. Templates fitting to the segment to be concreted are fixed at close intervals to check the profile correctly so that surface irregularities as prescribed for U₂ finish is not exceeded. U₂ finish was used for surface finish. This is a floated finish followed by screeding. Floating was being resorted after the screeded surface stiffened sufficiently that is minimum necessary to produce a surface that is free from screed marks and is uniform in feature.

From El.107.50m to finish level of ogee, the quantity involved was around 280 cum excluding pier portion per block. This was a difficult operation to be tackled with at one stage while specifically to be done with crane with huge concrete required for surface finish. This was discussed and it was decided to build a core as first stage concreting. The core section thus designed has a minimum cover of 1.50 m alround to be covered in the final stage of ogee finish. The safeguard against any possible shear along the plane, sufficient vertical shear reinforcement were provided. A shear key was also provided in the core section. With adoption of the core section the top ogee section could be completed in one operation without any interruption.

3.2.9.1 Slope Reinforcement:

As per C.W.C. drawing there was no provision of reinforcement in the slope concrete below El. 105.60m. But as per decision of the Project authority, reinforcement was provided for the entire slope. Bucket reinforcement i.e. 20 mm ϕ R.T.S. bars @ 25 cm. C/C was continued upto El.105.60 m.

3.2.10 Arrangement for Dam and Pier Concreting:

Due to the non-availability of tower cranes, derrick cranes were used. The arrangements similar to the bucket concreting were made. But in this case the derrick cranes were installed very close to the dam slope to lift to maximum height. For this purpose the bucket, which was completed earlier, were filled up with earth and sand and broad gauge track was laid over the same for erection of derrick crane. The mixers were installed on the ground level which was sufficiently high over the bucket lip to negotiate the height of trolley with bucket. At the lower bucket side general ground level was not high

and as such arrangement was made similar to bucket concreting. With the help of derrick cranes the spillway concreting for water carrier, pier concreting were done but the progress of concrete was slow as the dam was being raised. In the year 1982-83, it was apprehended that the concreting of the piers will be delayed if same method would continue. Therefore, decision were taken by the then Chief Engineer, to adopt a method for concreting of the piers by manual labour by providing gangways fabricated over the stanchions fixed on the piers. The concrete was carried from the batching plant and dumped over the spillway bridge slab from where the same was carried over the gangways and poured on the piers. Thus adopting both the methods that is derrick crane method and gangway method the concreting work was completed as per the time schedule.

Zoning of Materials:

The following zoning of materials both for overflow spillway and non-overflow dam was adopted.

Table 3.2 O.F. Section with Bucket and N.O.F. Dam :

Classification	Location	Maximum size of aggregate(mm)
C ₁ – M ₂₀ grade	In foundation	40
C ₂ & C ₃ M ₂₅ grade	All round galleries and piers, spillway crest, downstream slope & bucket portion.	40
C ₄ – M ₂₅ grade	In block-outs, road bridge slab, beams and around vent pipes.	20
C ₅ – M ₁₅ grade	Concrete in the hearing below El.77.00 m in deep channel.	40

Table 3.3 Masonry Specification

Classification	Location	Type of Masonry	Type of mortar.
1	2	3	4
M ₁	600 mm thick impervious layer in front face of the spillway.	Course rubble	Rich cement mortar not leaner than 1:3 by volume.
M ₂ (i)	2400 mm thick in front face of the spillway adjacent to 600 mm face work.	Random rubble	-do-
M ₂ (ii)	1000 mm depth of the full section of the dam except portion covered by item (i) and Block-21.	Random rubble	-do-
M ₃	Hearing in spillway except for the portion 1000 mm depth of foundation.	-do-	Cement mortar not learner than 1:4 by volume.

3.2.11 Construction of Spillway Bridge:

General Description:

Total length of dam is 1040 m which is divided into 51 blocks. The spillway blocks are located from block 19/2 to 43/1. The length of each spillway block is 19.5 with pier of 4 m width at the centre of the block to support the spillwa bridge. The clear span of the spillway bridge is 15.5m. The bridge has been designed as T-beam with deck slab. The bottom level of the beam is at El.126.50m and top of ogee is at El.110.2m. Construction of spillway bridge was taken up departmentally by engaging petty labour contractors for items like shuttering, reinforcement and placing of concrete as M/s. O.C.C. Ltd. declined the offer.

It was difficult for casting spillway bridge with normal way of centering and shuttering over ogee at a height more than 16.3 m. Therefore it was decided to provide trusses running over two spans for supporting centering and shuttering for spillway bridge. Four nos. of trusses of length 39 m were fabricated at central workshop of the project. Each truss was having 8 pieces of size 4.95 m length x 0.55 m width x 2.0 m (ht.). These pieces were bolted together to form a truss of 39 m length. The trusses were so designed that it would take a load of joists, channels and angles to support the centering and shuttering, weight of concrete bridge and weight of men and machinery during work. It was also designed to withstand the self weight of half span as a cantilever when the same is pulled to the other two spans.

The spillway bridge consists of the beams of 1.6 m depth with deck slab of 0.25 m thick. The main beams are spaced at 2.6 m centre to centre and the upstream side beam is at an offset of 2.0 m downstream of dam axis. Accordingly four nos. of launching trusses were designed which are spaced 2.6 m centre to centre with upstream side truss at an off-set of 0.7 m downstream of dam axis. The trusses were so arranged that the beam for the spillway bridge would be spaced at the centre of the gap from truss to truss, i.e. the centre of beam will coincide with centre to centre of launching truss.

3.2.11.1 Arrangement for Fixation of Launching Truss:

Spillway bridge construction was started from right side i.e. from block No.43/2 and at first stage the 24th bay and 23rd bay of spillway bridge was cast. First of all, the spillway piers were raised upto El.123.00 m. At the location of launching truss, pier cap of size 1600 x 1500 x 250mm with skin reinforcement was cast with fixation of anchor bolt for bearing plate to support the trusses. Above the pier cap the bearing plate of size 1600 x 800 x 30mm was fixed such that the longer edge will be in the direction of dam axis. After fixing up the base plate 4 part pieces of truss bolted together to form a half truss covering one bay, taken to block No.44, where the level of the dam had already reached at its final stage except the wearing coat. With the help of crane, half length of the truss consisting of 4 part pieces were lifted and put in position between pier No.23 and 24 in block No.42 and 43/1. As the boom length and capacity of the crane was not sufficient enough to handle the full length of truss, the half length of 4 rows were kept in position. With the help of another crane stationed on the upstream side of dam between pier No.23 and 22, where the ground level was at El.95.00 the boom length of the crane was just sufficient to lift the half length of the truss to El.123.00 m (i.e. bearing level for the truss). When these half trusses were kept in position and were adjusted to match with half truss already positioned on the pier 23 and 24, those were bolted together, to have the full length of 39m. After keeping the trusses in position, those were made rigid and braced together by the channels. Over the launched truss purilins of size I.S. M.B.250 size were placed at 1.2 m C/C cross-wise and bolted. Then the other portions of the piers except the truss position were raised to El.125.78 m over the raised level of the pier. Pier cap for main beam of spillway bridge was constructed with fixation of anchor bolt so that the right half portion of the piers were raised to El.126.03m to receive roller bearings and left half portions were raised to El.126.33m to receive rocker bearings. In this way Rocker bearing were provided on left side supports and roller bearing on the right side supports of the spillway bridge.

The over all span of the spillway bridge is 19.5 m but actual length of each span of spillway bridge is 19.4875 and 25 mm expansion joint was provided in between the two span ends. The main beams were connected with five cross beams, the centre to centre of which is 4.375m. The end cross beams are situated at the centre of the support. The roller and rocker bearings are so arranged that the centre of the bearing will coincide with the centre of end cross beam and centre of supports.

3.2.11.2 Fixation of Bearings:

At the right side of the spillway bridge support, cast steel base plate of size 800 mm x 645 mm x 85mm for rocker bearing was fixed with the anchor bolts provided in the pier cap at El.126.33 m with 800 mm long edge placed at right angle to the axis of the dam. There are three holes of sizes 45 mm dia, 35 mm deep at the centre of the base plate spaced at 200 mm C/C parallel

to 800 mm long edge, M.S. channel of size 150 mm x 75 mm was fixed in pier cap at the centre line of supports were screwed with base plate on shorter side with 12 mm packing plate in between the base plate and M.S. channels. On the central hole of the base plate 70 mm long 45 mm dia steel pins were fixed and above it top plate of size 800 mm x 645 mm x 85 mm was fixed as bearing plate for rocker bearing, so that the steel pin projected from base plate (35 mm long) will fit in the hole provided in the top plate. Then the anchor bolt was fitted in the collar of top plate and left upwards, which will be embedded in the beam resting over rocker bearing. Top level of rocker bearing is at El. 126.50 m.

At the left side of bridge support just below the position of main beam along the centre line of support cast steel base plate of size 800 mm x 645 mm x 50 mm for roller bearing was fixed in position at El. 126.03 m and bolted with anchor bolt embedded previously in pier cap so that the centre line of bearing coincided with centre line of supports, 800 mm long edge of base plate bearing in the direction of centre line of support. The base plate was also screwed with M.S. channel of size 150 mm x 75 mm embedded with pier. In the base plate steel guide key of size 645 mm long 20 mm deep and 10 mm wide was fixed, so that half depth remains in base plate and half depth above base plate Three nos. of guide keys were fixed in the base plate. Through these guide keys, two nos. of forged steel rollers of size 250 mm dia for each guide key were fixed and both bolted with spacer bar of size 350 mm x 65 mm x 12 mm, so that the centre to centre of roller will be 275 mm and centre of rollers will be equidistant from the centres to supports on either side. Saddle plate of size 800 mm x 645 mm x 85 mm fixed with guide key in the same manner as in the case of base plate was fixed over the rollers, so that the guide key of saddle plate will be fixed in the groove of the rollers. Cast steel top plate of size 800 mm x 645 mm x 85 mm was fixed with saddle plate. Anchor bolt was bolted to the plate in the collar and left upwards, which will be embedded in the beam of spillway bridge. The top level of the roller bearing ($126.03 + 0.050 + 0.250 + 0.085 + 0.085$) it at El. 126.50 m.

3.2.11.3 Shuttering Arrangement for Beam and Slab:

The shutters were manufactured in pieces. Those were serially numbered which were jointed together by bolting to form the shutters for the beams, cross beams, over hanging portions of up-stream and downstream side and also for deck slab and were placed in position with wooden bullah support. On the steel cross beam of launching truss just below the beam, two channels were run over to which wooden edges were provided. Size of the wooden edges are so designed that the top of the shuttering plates supporting the base of the beam was kept as per the required level. After the reinforcement steel were put in position, the end shutters were bolted and made rigid. The reinforcement and shuttering were going on simultaneously.

3.2.11.4 Reinforcement:

The main reinforcement bars for main beam consists of 32 nos. of 36 mm dia. These reinforcement bars were bent to their required shape. They were lap welded as the length of bars received were not of required length. A number of samples were tested at University College of Engineering, Burla for tensile strength. The failure was assumed to occur at points other than welding points on rod. The stirrups of 16 mm dia were put earlier. These stirrups were welded to the main bars. Reinforcement for the slab was tied after main beam and cross beam are cast.

3.2.12 Casting of Spillway Bridge:

The casting of spillway bridge (Beam and slab) were taken in two stages. In first stage, the main beams with cross beams were cast upto El. 128.025 m and in second stage the deck slab with top portion of beams were cast. Similarly the reinforcements for the spillway bridge were also done in two stages for beam and slab respectively. After completion of reinforcement for beams with cross beams centering for deck slab casting of beams were taken up with cement concrete mix M.25 with maximum

size of aggregate of 20 mm and having a slump of 75 mm to 100 mm. The design mix for one cum. of M.25 mix concrete with slump of 100 mm was found as follows:

Cement	: 450 kgs.
Sand	: 700 kgs.
Chips	: 1100 kgs.
Water	: 230 ltrs.

3.2.12.1 Placing of Concrete:

Concrete of required grade were carried through Tippers to the top of dam and the same was unloaded very close to the area of casting. At the first stage the beam portions were cast. Then after roughening the reinforcement top layer of beam alongwith deck slab were cast. The concreting of deck slab was done after clearance of top layer of beam concrete and slab centre by air-water jet. Electrically operated vibrators having 40 mm and 20mm dia needles were used. The size of the beam at the bottom was bulk shaped. It was therefore apprehended that the air would not be released as a result, concrete inside the bulk shape of beam may be oney-combed. To overcome this problem, small holes were drilled in the shutters at regular intervals, so that air would pass inside but the slurry would not ooze out. After the pouring in these portions the shutters were hammered from out-side with wooden hammer for proper setting. No shutter vibrators were used.

After concreting of the main and cross beams two days were left and the top surface were roughened and cleaned to receive deck slab concreting. Reinforcement of slab concreting are welded to rods of beams and cross beams protruding outside. After reinforcement was over gang ways were made for carriage of concrete by manual labour.

3.2.12.2 Curing:

After the beams and cross beams were cast, continuous curing was going on at the top, but keeping the shuttering plates for the main and cross beams in position, spraying of water was difficult. Therefore small holes were made with small G.I. Pipes of length equal to the thickness of the deck slab were tag welded and subsequently welded to slab reinforcement. The G.I. pipes were welded and closed at the end by pieces of plates with holes at the sides finally water hose of length of about 75 mm was fitted and inserted through those holes. The water was controlled by a valve provided in the pipe, when the pipe was inserted through these holes. The shutters of the beam were removed after 7 days of casting; curing to the concrete of the main and cross beam was done by the method mentioned above. This process of curing was most effective.

3.2.13 Positioning of Trusses

After 21 days of curing, all the centering materials were removed and stacked over for subsequent use. There after pulling of the trusses started. At a time two trusses were pulled. It was apprehended that the truss may tilt while pulling. Therefore two trusses were jointed together by channels bolted loosely such that while pulling, the truss will not tilt down. The trusses would adjust each other if there is any little variation during pulling. For pulling two winches of 10 T capacity each were kept at the end of the dam span. To make these winches rigid two holes of 200 mm dia for each winch were kept in the slab and winches were tied to the slab rigidly through these holes by wire rope. Prior to pulling of trusses the double sleeve pulleys were already fixed on the piers of two spans ahead over the rods embeded earlier. The trusses were tied by the wire rope which were passing through the double sleeve pulleys and lastly to the winches. In a similar way the other trusses were also tied with another 10 T winch by the wire rope which passed through another set of double sleeve and single sleeve pulley. Before pulling the trusses, thorough checking were made to ensure that the trusses were completely free to move. The winches were operated manually, so that movement of both the trusses were more or less equal. By this method the trusses moved slowly and reached its final position.

3.2.14 Sequence of Operation:

Sequence of different operations for different items of work are narrated below along with actual date of commencement and completion. It is seen that for completion of bridge of two spans, 45 days are required for concreting the gap portion of the pier after removal of the truss.

After 21 days of deck slab concreting the centering and shuttering were dismantled. Deck slab was used for stacking the shuttering plates. Joists, channels and shuttering frames were launched for next two spans. Similar methodology as adopted for previous two spans was followed for fixing of bearing plate, roller bearing, rocker bearing. The shutter plates were brushed, cleaned and repainted for fixing them for next two spans, to get better finished surface.

All the bridge casting was done from right side and 24th bay was cast in first batch during December 1982 after which bridge casting was started from left side during April 1984. The 1st bay was completed during May 1984.

The gaps provided for resting the trusses were concreted soon after removal of the trusses from those positions. For concreting of the same the shutterings were made after taking all materials to the bridge through the gaps provided between the beams of adjoining span. The concrete was dumped from the slab top through the funnel passing through the 200 mm dia holes provided for tying and fixing winches. The 200 mm dia holes and the holes provided for curing the slab were also plugged after-wards.

3.2.14.1 Erection of Trusses on the Leftside:

It was apprehended that the completion of the bridge would be delayed, if the bridge work would be taken up from one side only i.e. from right side. Therefore another set of trusses were fabricated. Difficulties arose for erection of these trusses for the second span because for the first span, first half of the truss was placed by that P.H. crane. The ground level on the left side was at El. 85.00 m and diversion channel was passing through block No.21. Therefore the problem of adequate boom length as well as the movement of the crane on the upstream area was difficult. Thus for concreting of the piers, one tower crane was erected. With the help of this crane, the second span of truss was erected by lifting them in pieces. These sets of trusses were utilised for four spans only from left side.

3.2.14.2 Wearing Coat:

The clear width of dam top road is 5900 mm and 7000 mm for over-flow and non-overflow sections respectively. There are two nos. of cable trenches on either side of the road. Upstream side cable trench of size 225 mm x 250 mm is meant for laying of cable for supply of electricity to Gantry Crane. The downstream side cable trench of size 200mm x 250mm is meant for laying of cable for supply of electricity to lighting posts which is covered through out. The precast side parapets are fixed up between posts of 1035 mm size spaced at 2 m interval.

After chipping and clearing of area, M-25 concrete was laid with 20 mm down graded aggregate having 0.4% water cement ratio. This was compacted by rammers thoroughly such that water inside the concrete would ooze out to the surface. Depth of concrete at the centre of road was 135 mm and at the end of gantry rail was 100 mm with camber of 1:72. In the non-overflow section blocks were 1400 mm extra width from upstream Gantry rail to the cable trench where a camber of 1:72 was also provided. Depth of cable trench near the upstream cable trench was 80 mm.

3.2.15 Pier Concrete for Casting of Deck slab of Bridge:

As previously mentioned the road bridge in two spans were taken up simultaneously. To complete 24 spans 12 operations were needed. From positioning of bridge girders to their pull out for the next two spans, on an average 45 days are required. Thus it would take 540 days to complete the bridge of 24 bays. It was also contemplated that the bridge work could be taken up from both ends of spillways with two sets of launching trusses on either side. But the pier levels on the left hand side could not be raised due to deep channel work for which commencement of bridge work simultaneously from both ends was not possible. Hence the bridge work from the right hand side of the spillway commenced during December '82. But by July, 1983 the bridge work was completed for 10 bays upto pier at block 33.

The road bridge on piers depended fully on the progress of concreting in piers. The fixing up of radial gates also directly depended on fixation of embedded parts, like trunnion, hoisting motor and gates etc. The road bridge could not come up unless the piers were ready. After completion of road bridge the radial gates could be lowered from the bridge top and fitted on upstream side. The low upstream ground level did not permit the hoisting of gates from that side. The only alternative was to complete the bridge in advance.

As the piers in the remaining spillway blocks were at low level, bridge work would have been suspended until the piers were raised up to El.123.90m, or resumption of work after monsoon in November, 1983. Thus the bridge work would have started early in January '84 after raising the pier Nos. 32 and 31 to El. 123.90 m which were at El. 117.8 m and 118.85 m respectively during June '83. This would have seriously affected the completion schedule of bridge and gates which would have prolonged up to January '85. Without installing the gates and plugging of diversion sluices, building up of reservoir would slip off to the monsoon of 1985 and consequently further delaying the spinning of turbine. Thus for building up the reservoir in July '84 and to avail the opportunity of early spinning, it was resolved to continue concreting of the pier by all means, even during monsoon of 1983, and raise those to El.123.90m for bridge construction requirement, such that the road bridge work can be continued uninterruptedly.

Although it was felt necessary to take up concreting work, it was an uphill task to achieve this objective. The positioning and pulling of trusses in two spans took 45 days. Therefore next pair of piers were to be made ready and handed over within the stipulated period. As the raising of piers were not possible from the bridge deck-slab, either with the help of crane or with the help of ropeway, method of an independent approach to the piers for transportation of men and material became imperative. This, on the other hand, meant a temporary gang-way to be constructed connecting the piers.

For this, only the downstream end of the piers was the convenient location. A suitable steel stanchion was designed and fabricated at the workshop. It was decided to have five stanchions with base of 2m x 2m for the piers from 33 to 29 and make a gangway at El.124.50m although to take up concreting at piers up to El.123.90 m.

Small concrete pedestals were made at 15 to 17 metre downstream on pier for housing 4 nos. of 1000 mm long 32 mm dia foundation bolts for each stanchion. As the monsoon was already set in and water level in the upstream side of the dam was on the rise, expeditious steps were taken to place the ISMB 500 x 180 girders on the stanchions before the crane was dismantled and removed from site. The set of three steel beams were fixed on the stanchion by 3 x 4 nos. of 20 mm dia 125 mm long bolts. As stanchions of each pier supported beams of either span, a gap of 25 mm had been provided to account for any expansion of the steel structure. With this much of work done, the cranes were dismantled before the rainy season.

3.2.15.1 The Gang-way:

The top of the cable trench on bridge was at El.128.80 m from which an approach to the proposed gangway at El.124.50 m was necessary. As a straight approach with a convenient slope was not possible, a landing above the structure was made at El.126.50m with a right hand bend and the level of the gangway was negotiated. The 2m wide gangway was started from Block No.33 and gradually advanced towards Block No.29. The gangway was prepared using 1m x 1m size shuttering plates with a rigid 1 m high railing on both sides with 50 mm x 50 mm x 6 mm angles and 16 mm dia plain rod welded to it. As piers were at low level a ladder was provided at the pier for lowering of materials and men to work on the piers. The concrete however, was lowered through chute made out of 1.6 mm thick, H.R., B.P. sheet or G.C.I. sheets. The chutes were being cut at bottom as the pier gained height. The road bridge slab on pier 33 was cast on 02.08.83 without loss of time and completed

by 06.08.83. Approaches to pier 32 and 31 were established through the gangway. The shuttering and reinforcement works on piers were also progressing rapidly and thus the concrete work could be resumed on pier 32 on 09.08.83. Four days later i.e. on 13.08.83 concreting at pier 31 was poured. As concreting of full length pier would consume more time, they were raised in a truncated manner started from 0 m to 14 m downstream of pier considering the requirement of the bridge work. Thus by 01.09.83 the piers at block No.32 and 31 were ready for positioning of launching trusses of the bridge. Allowing curing time for 10 days to piers, the trusses were then pulled and positioned on 33, 32 and 31 for the bridge by 12.09.83. Having given the area for these two spans, the concreting work in three shifts continued at piers 30 and 29 for the next set of two spans of bridge.

Concrete from batching plant was transported by tippers and unloaded on bridge top and then was carried manually through gangway to piers at block 30. The concrete level at this pier was at El.117.5 m while the pier 29 was at El. 115.2 m. These two piers were also raised to El.123.90 m in a truncated manner by 30.09.83 and 05.10.83 respectively. The schedule date of pullout of trusses to these piers was 10.11.83 as per revised programme. Thus the pier could be handed over for bridge work well ahead of schedule. Having completed these four piers, attempts were made to take up works in the piers at 28 and 27 as well, since the downstream side of dam was still not available for placing the crane for concreting. It was again a very difficult task to lower concrete from gangway terminating at pier 29 and transfer to piers at blocks 28 to 25. So a ladder run was made with suitable landing at every 5 metres interval by welding steel rods to the exposed bolts on pier face for getting down to El.114.0 m. A chute run parallelly below the ladder on pier face to this elevation. A platform was also prepared to receive the concrete from the chute. Workers carried concrete at El. 114 m, walked over the connected gangway of 1.65 m wide upto pier 28 and 27. With high risk, concrete was being rehandled through two groups of workers. The first group carried concrete from bridge top through the gangway at El.124.5m and unloaded through chute to platform at El.114m. The 2nd group then carried it on gangway at El. 114.00 m and poured at piers at El. 113.3 and 110.60 m in block No.28 and 27 respectively. The thrust block concreting at pier 26 and 27 could also be completed with the help of this gangway.

Thus, it was possible to do 2500 cum. of concreting during the off season. Additional 3125 cum. of concrete were also placed using the gangway upto pier 25 during the working season of 1984 because of non-availability of crane. In the mean time, M/s. O.C.C. Ltd. (A Govt. of Odisha undertaking) was able to repair the approach road, fill up the bucket with debris and install a 15 T derrick crane having a long boom length which was able to lift the concrete upto El.123.90 m. Thus, M/s. O.C.C. Ltd. took up the work of pier concreting in the working season of 1984 in block No.22, 23 and 24. This joint effort of department and O.C.C. Ltd. accelerated construction of bridge and work of radial gates.

When the piers were ready in a truncated manner as started earlier the bridge work were taken up. After the completion of respective span, the work of balance section of pier on the downstream side were taken up. The pier had a circular upstream face and rectangular downstream end which continued upto El.113.5 m. The section was trapezoidal at downstream end upto El.117.5 m and again changed shape to facilitate the moving of end arms of radial gate. This required high skill for shuttering. Therefore shutter plates were manufactured at project workshop with much care and precision. Concreting of downstream portion of the pier were taken up from the bridge slab. For joining of the old concrete surface in the upstream portion of the pier with the fresh concrete at downstream side, sufficient dowel bars had been provided for bonding.

The relentless effort helped to build up reservoir in July and August, 1984 for spinning of turbine. Had no work been done during monsoon of 1983, it would have been far for reality to conceive the spinning of turbine before July/August, 1985, thereby delaying the project by one more year leading to both time and cost overrun.

3.2.16 Estimate and Expenditure:

In any major river valley project, time and cost over-run is inevitable due to long gestation period. Investment clearance to the stage I of the project was accorded by the Planning Commission, GoI on 14.06.73 (vide Annexure III of Chapter II) for Rs.5792.68 lakhs which includes Rs.4192 lakhs for construction of masonry dam with appurtenant works and civil works for installing 2 x 50 MW of hydro power units.

The expenditure incurred upto the end of January, 1981 was Rs.5104.48 lakhs, which exceeded the original estimate. Therefore revised estimate become necessary. Accordingly revised estimate at the price prevailing in 1980 was framed at a cost of Rs.10,305 lakhs. Following structural changes which had contributed to the increase in the cost of dam, were:

- a) Ultimate installed capacity of the Hydro-power Station has been increased from 3 units of 50 M.W. each to 5 units of 50 M.W. each. Provision for this additional installation has affected the cost of foundation in Power Dam, which had to be dug deeper to House in Penstock.
- b) The length of spillway has been increased from 381.55 m to 464 m to discharge maximum probable flood of 55460 cumec.
- c) Foundation grade rock in the deep channel zone met at R.L.53.00 metres instead of R.L.60.00 metres originally anticipated. This increase in depth makes the maximum height of dam 75 metres instead of 68 metres originally estimated.
- d) Total number of families to be rehabilitated increased from 8400 to 9585.

On account of above changes, there has been certain increase in quantum of work involved. The main factor which had contributed to the increase in cost of dam and appurtenant works by 150% was, the rise in cost of Rehabilitation, materials labour and machineries. The extent of rise in cost of some essential material has been given in Table No. 3.10 from 1972 to 1980.

At the project sanctioned stage in June, 1973, it was expected that the scheme would be completed in the 7th Years of construction i.e. by 1980. But as infrastructure facilities like road communication, electricity, water supply, housing and storage godown facilities had to be constructed in 1st two years, the work in foundation could only commence by January, 1975. There was lag in progress due to two strikes in two spells by workers of M/S. Odisha Construction Ltd. (O.C.C. Ltd.), the main contractor for dam work and also by the squatting of submersible villagers for one month. These strikes have resulted minimum one year delay in dam works.

Till the end of 1980 about 50% of work was completed, the revised estimate framed at this stage was expected to remain firm unless there was significant rise in prices during the remaining years of construction.

Major increase of cost have occurred in the sub-head that was 'B'-Land and 'C' works. The increase in cost of 'B'-Land was from 1531.13 lakhs to Rs.4280.83 lakhs. This increase was mainly due to

(a) increase in cost of compensation for land, houses and trees from Rs.968 lakhs to Rs.1727 lakhs. The price of land acquired was being paid after calculating the agricultural produce of land and valuation of houses based on Govt. approved schedule of rates. Increase in value of agricultural produce and schedule of rates had resulted increase on compensation.

(b) Increase in cost due to relocation of roads and bridges was from Rs.127 lakhs to Rs.437 lakhs. A bridge across river Brahmani at Barkote on N.H.6 with 5.76 km length of approaches comes under submergence of Rengali reservoir. Relocation charges of this road bridge and approaches as per Ministry of Transport (M.O.T.) standard alone resulted in increase from Rs.127.00 lakhs to Rs.334 lakhs. The cost of relocation of 129.55 km. of other roads were also included.

(c) The increase in cost of resettlement of displaced families from Rs.111 lakhs to Rs.1427 lakhs on the basis of resettlement policy approved by Government.

In the original report it was estimated that the cost of land reclamation is to be realised from displaced persons. But according to the Rehabilitation Policy approved by the State Government only Rs.300 per acre towards land reclamation was to be realised from the displaced persons who owned land and no reclamation charges would be realised from landless persons even though each family was entitled to get 6 acres of unirrigated land or 3 acres of irrigated land. As about 54000 acres were to be reclaimed for the purpose and the cost of acquisition and reclamation of this land was about Rs.1560 per acre there had been increase of Rs.842.4 laks only on this score. The number of families to be displaced increased from 8400 in the original estimate to 9585. For these additional families, common facilities were to be provided at additional cost.

The cost of sub-head 'C' works has increased from Rs.1922.93 lakhs to Rs.4540.74 lakhs. This has occurred mainly due to general increase in rate in the major items of works i.e. foundation excavation in rock, masonry and concreting. The rate of rock excavation in foundation had increased from Rs.16.15 to Rs.30.58 per cum. The rate of concrete also increased from Rs.191.21 to Rs.334.40 per cum. Similarly masonry had increased from Rs.136.70 to Rs.218.90 per cum.

Another major reason for increase in cost was due to increase in quantity and cost of steel reinforcement and gates. Against the original provision of 10,864 M.T. of steel reinforcement @ Rs.1520/ Ton, it was then estimated that 14,289 Tons of Steel reinforcement at an average rate of Rs.3329/- per Ton would be used in the dam. Against 4600 sqm. of crest gates at Rs.5,000/- sqm. in the original estimate it was expected that 5500 sqm. of crest gates @ Rs.13,000/- sqm. would be required. The increase in quantity was as per final design and the increase in rates was due to general price rise in the cost of steel.

The 1st revised estimate amounting Rs.103.05 crores at 1980 prices was framed and submitted to Government on Dt.25.3.81. It was programmed at this stage to complete the dam by June 1983. Comments of various directorates of CWC on the revised estimate were received during 2nd half of the year 1981. Before submitting compliance, it was felt that further revision of the estimate would be necessary on account of further rise in cost of materials notably that of cement, steel, diesel, explosive. Further revision was done basing on actual expenditure for the work already done by March, 1982 and future expenditure to be incurred after 1982 prices till completion. The revised estimated cost was Rs.115.53 crores.

Upto June, 1984, about 75% of work was completed. The revised estimate framed at the stage was expected to remain firm unless there is significant increase in cost of materials and labour during the remaining years of construction.

The cost of sub-head 'C' works has increased from 4540.74 lakhs to Rs.5069.36 lakhs. This has occurred mainly due to general increase in rate in the major items of work of foundation, excavation in rock, masonry and concreting, as detailed below:

Sl. No.	Items of work	Rate as per original estimate/cum	Rate as per 1 st Revised Estimate/cum	Rate as per 2 nd Revised Estimate/cum
1	Rock excavation in foundation.	Rs.16.15	30.58	30.58
2	Concrete	Rs.192.21	344.40	344.40
3	Masonry	Rs.136.70	218.90	270.80

The major reason for increase in cost was due to increase of cost of steel for reinforcement and gates.

The estimate was further revised in June 1984 to Rs.133.04 crores.

The following points briefly explains the reason for increase in cost to Rs.123.09 crores (i.e. 2nd R.E.). The main factor which has contributed to the increase in cost of dam and appurtenant work by 218.22% over original estimate was the rise in the cost of Rehabilitation, materials, labour and machineries.

At the stage (during preparation of 3rd Revised Estimate) it was programmed to complete the dam by June, 1985. The expenditure incurred upto end of June, 1984 was Rs.11,887.77 Lakhs. During this period, the rate of concrete has increased from 344.40 to 456.59/cum. Another major reason for increase in cost due to increase in general price rise in the cost of crest gate from Rs.13000/sqm. to 17500/sqm.

The third revised estimate in June, 1984 amounting Rs.133.40 crores was submitted for approval. In short period of about 9 months, a supplement to the third revised estimate amounting to Rs.141.50 crores submitted to C.W.C. in Sept., 1985. The 4th and Final R.E. stands at Rs.169.86 crores.

The expenditure incurred for dam and appurtenant works upto December, 1986 is Rs.139.53 crores Progress of major items of work achieved upto Feb.83, June' 84 and Dec. 1986 has been furnished in Table-3.4.

Sub-head wise provision in original estimate and revised estimates and year wise net expenditure on RMP are furnished respectively in Table-3.5 & 3.6.

Table-3.4 Progress of major items of work

Sl. No.	Major Item of work	Ending Feb.83		Ending June'84		Upto Dec. 1986		% of Progress upto Dec.86
		Est. Qty.	Progress	Est. Qty.	Progress	Est. Qty.	Progress	
1	Foundation Excavation(cum)	416118	387460	390969	390669	390969	390969	100%
2	Concrete(cum)	361150	257019	339977	333212	339977	339977	100%
3	Masonry(cum)	472000	422878	449768	449768	449768	449768	100%
4	Fabrication of Gate (MT)	7246	7246	7246	7246	7246	7246	100%
5	Land acquisition in ha for resettlement	42500	25642	46635	44217	49870	47635	95%
6	Evacuation of Families(nos)	10993	3362	9686	8231	10879	9569	88%

Table – 3.5

Dam and Appurtenant works : Sub-headwise provision in original estimate and revised estimate.

Sl. No.	Sub-head	Provision in the original estimate (lakhs)	Provision in the 1 st R.E. (lakhs)	Provision in the 2 nd R.E. (lakhs)	Provision in the 3 rd R.E. (Suppl.) (lakhs)	Provision in the 4 th R.E. (lakhs)
1	2	3	4	5	6	7
1	A-Preliminary	39.00	32.24	35.86	66.79	48.28
2	B-Land	1531.14	4280.83	4821.44	5563.74	6494.71
3	C-Works	1922.93	4540.74	5453.06	6364.84	7167.76
4	K-Building	139.00	284.10	440.95	443.52	373.57
5	M-Plantation	1.00	2.29	2.25	2.46	7.85
6	O-Miscellaneous	134.80	309.28	480.53	501.31	770.76
7	P-Maintenance	18.90	53.70	61.32	75.38	152.90
8	Q-Special T&P	41.93	90.25	107.89	130.46	714.40
9	R-Communication	112.00	159.16	149.04	181.05	151.98
10	Loss on stock @ 0.25% of Unit-I	9.85	24.38	16.46	18.92	1.27
11	Establishment:8% over cost of Unit-I (excluding B-Land)	193.55	439.69	605.78	698.94	1131.60
12	T&P 1% of work (excluding B-land)	24.19	54.96	115.52	133.50	37.83

1	2	3	4	5	6	7
13	Suspense	Nil	Nil	Nil	Nil	Nil
14	Receipts and Recoveries.					(-) 295.25 as per actual
	a) Deduct receipt & recoveries on capital accounts. 15% of Temporary building only.	(-) 16.47	(-) 35.14	(-) 36.07	(-) 28.41	(-) 28.43
	b) Receipts and recoveries Q-Spl. T&P 60%	(-) 25.16	(-) 54.15	(-) 80.91	(-) 81.57	(-) 112.58
	c) For inspection vehicles.	-	-	(-) 3.63	(-) 4.34	-
	d) Recoveries from sale of scrap	-	-	-	(-) 118.43	(-) 45.29
15	Indirect charges.					
	a) Capital value abatement on land revenues on 43,165 acres for 20 years.	25.90	24.60	24.60	65.70	75.70
	b) Audit charges 1% of I-works.	39.50	97.77	115.52	133.50	158.83
	Total :	4192.06	10304.67	12309.61	14149.56	16985.79

Table – 3.6 Statement of Expenditure (Net Expenditure of R.M.P.)		
Year	Head of Account	Sub-head wise expenditure under R.M.P., Rengali for the period from 1972-73 to 1995-96 Demand No.20-4701 Col State Plan. (Net Expenditure of R.M.P.)
1	2	3
1972-73	98 COL MPRP	5360980.22
1973-74	98 COL MPRP	21668365.67
1974-75	98 COL MPRP	23784948.55
1975-76	98 COL MPRP	43652903.32
1976-77	98 COL MPRP	43368064.29
1977-78	20-532 COL	91900232.43
1978-79	20-532 COL	93878000.01
1979-80	20-532 COL	120064060.71
1980-81	20-532 COL	142612830.32
1981-82	20-532 COL	154200976.21
1982-83	20-532 COL	156036072.26
1983-84	20-532 COL	212795777.40
1984-85	20-532 COL	202297276.60
1985-86	20-532 COL	50869341.63
1986-87	20-532 COL	49592135.16
1987-88	20-532 COL	80260823.93
1988-89	20-532 COL	72152043.34
1989-90	20-532 COL	59005011.36
1990-91	20-532 COL	52871293.90
1991-92	203-R.D.P.	1063263.50
1992-93	203-R.D.P.	13462070.00
1993-94	203-R.D.P.	1125364.00
1994-95	203-R.D.P.	617852.00
1995-96	203-R.D.P.	(-) 2177947.00
	Total :	Rs.1690461739.81
	Say :	Rs.169.05 crore

3.2.17 River Diversion

“For foundation excavation and treatment as well as for dam construction, it is almost always necessary to confine and dewater the river reach in which the dam is located. During the construction period of the dam the river flow has to be diverted away from the construction area, either through an open channel or through tunnels or both. There are exceptional circumstances in which dewatering of the site is not practicable, e.g. at the High Aswan Dam site, and the dam has to be built by depositing material under water.

The first decision to be made concerning river diversion is the maximum discharging capacity for which the system should be designed. As the construction period of the dam proper is relatively short, about 4 to 8 years even for important dams, the maximum probable flood for this period on basis of frequency analysis would be considerably smaller than the maximum flood at the site. Yet, there can be no absolute assurance that floods of near maximum magnitude will not come during the construction period. As overtopping of an earth dam during construction would normally result in its total loss besides causing heavy flood damage in the downstream valley, if sufficient storage capacity has already been created, very conservative diversion arrangements have been adopted at many sites, e.g. at Beas and Ramganga Dams the diversion capacity equalled 1000 years frequency. The flow characteristics of the river also play an important role in determining the diversion flood-in areas where there is no history of sudden heavy storms and the river floods are mainly due to snowmelt, the flow pattern is much more reliable than in monsoon climates, and lower diversion floods, say with a frequency of 3 to 5 times the estimated construction period, can be used.

In wide valleys open channel can be used for diversion. The embankment can be built on either side of the diversion channel leaving the gap at the diversion channel to be plugged in one low water season. The raising of the earth work has to be kept ahead of the rise of water surface in the reservoir till the water starts flowing through permanent passages like outlets and spillway etc.

More often, particularly for high dams built in narrow valleys, diversion is accomplished through tunnels bored in the abutments. Cofferdams are constructed at the upstream and downstream ends of the works area. The height of the downstream coffer dam has to be higher than the high flood level at the site by a safe margin. The temporary coffer dams are built by dumping rock in the river bed, and then plugging the leakage by dumping, gravel, sand and finally clay on the outer slopes. The main coffer dam on the upstream side can be a very substantial structure, e.g. at Mangla dam the height of the main coffer dam was 64 m, at Ramganga dam 77m and at Beas dam 56 m. This work must be completed in one construction season; the coffer dam should have an impervious zone properly bonded with impervious ledge, and would thus involve considerable excavation as well. At Beas dam the foundation excavation and refilling upto river bed level was completed in one low water season and the river was cleaned up in the next season and the dam then raised to the designed level. To make use of the substantial fill placed in the upstream coffer dam, it is often incorporated in the main dam e.g. in all the three instances cited above this has been done. If the lowest drawdown level is lower than the top of the coffer dam, its impervious zone should be removed to the drawdown level after completion of the dam, to permit free drainage. If the entire dam section can be raised to required diversion level in one working season, the dam itself would serve the purpose of the coffer dams.

The layout of the dam has to incorporate diversion arrangements and would be considerably influenced by it. It may be possible to use some portions of some or all of the diversion tunnels as permanent outlets or as power tunnels or spillway tunnels. Construction of coffer dams under water can be achieved by placement of rockfill either by end dumping or by dumping from trestles. Prior to dumping only the abutments are stripped, usually, although pervious deposits are also stripped at times. In case of 370 ft (113 m) high Akosombo dam across river Volta about 1.5×10^6 cu. yd. (1.15×10^6 m³) of clean gravelly sand overlying bed rock in a layer of 10 ft (3.05 m) to 100 ft (30.5 m) deep was removed by suction dredging before building the coffer dams to make the contact possible between the quartzite shale bed rock, and the core and rock material of the dam. The upstream and downstream

coffer dams were formed by end dumping methods in depths of water that varied from 100 ft (30.5 m) upstream to 200 ft (61 m) downstream. The coffer dams were sealed by zones of transition material placed primarily by end dumping. For the upstream face the seal blanket consisting of minus 1 inch (2.54 cm) rock fines from borrowed laterite was placed by both end dumping and barge dumping. For the downstream coffer dam a bottom dump barge placed the lower levels of the seal blanket whereas the upper levels were placed by dredging and pumping a sand-clay mixture over the transition material.

In situations where it is expensive to remove over-burden material, sealing of foundation is effected by providing slurry trench, concrete or grout cutoff. At Mangla dam, three types of cutoffs were constructed to control the underseepage through 26m thick overburden material: a 3m wide slurry trench cutoff was taken to bedrock at one bank; a grouted cutoff was constructed across the river bed; and a rolled sandstone cutoff was completed in an excavated trench. At La Angostura Dam concrete cutoff walls were built through alluvium at the inner toes of both the coffer dams so as not to interfere with their construction. The location of the walls within the coffer dam necessitated that during high water stages of the river, seepage through the alluvium below the impervious blanket be pumped out. To reduce the percolation head, both concrete walls were extended upwards by means of clay walls built inside trenches in the sand and gravel fills.

Considerable economy can be effected in diversion works by designing rockfill coffer dams for overtopping during floods. The 20 m high Keriba rockfill coffer dam was overtopped passing a discharge of 17,000 m³/sec at a depth of about 20 m over the crest. At Roseires, a maximum flow of about 8700 m³/sec passed over the coffer dam with a depth of above 5 m over the crest. No major damage was found to occur except dislodgement of a few gabions on the downstream face of the upstream coffer dam. **(Source: Earth and Rockfill Dam, 1976 by Bharat Singh and H.D. Sharma, Pg.418-421)**

In Central and South Indian rivers summer flow is negligible which minimises diversion problem considerably. In case of river Brahmani, the summer flow varies from 10 to 200 cusec. Normally, tunneling system or open channel excavation are resorted to in diverting natural river channel flow. In case of Rengali Dam site, the topographical features completely rules out tunneling. Therefore open channel diversion was the only alternative. The open channel excavation adjacent to the deep channel was found to be the most suitable from consideration like depth of excavation, approach and exit channel alignment with regard to main river channel, capacity of the sluice to cater the discharge etc. Hence diversion along block 21 of overflow portion (i.e. spillway section) was considered most suitable. A channel was excavated in the rock. The total length of diversion channel from entry to exit including sluice is 300 m.

3.2.17.1 Waterway:

Central Water Commission (GoI) recommends that for any major project, the waterway for any diversion arrangement should be sufficient enough to pass out a discharge of 100 year return period flood during non-monsoon from November to May. In case of Rengali project, the average non-monsoon flow is of the order of 85 cumec and that for 100 years return period is 368 cumec. For this discharge the diversion channel (both approach and exit), the sluices etc. have been designed, the salient features of which are given below:

1. Average / 100 yr. return period non-monsoon flow	= 85/368 cumec
2. Bed width of diversion channel	= 20 m
3. Bed slope of diversion channel	= 1:85
4. No. and size of sluices	= 2 no x 4.0m x 3.5 m
5. Sill level of Diversion Sluice (in block 21)	= RL 78.00 m
(These sluices can evacuate 85 cumec with upstream water level at 82.00 m)	
6. Sill level of 4 low level blocks (from 22-25)	= RL 82.50 m
(These can evacuate 368 cumec with upstream water level at RL 84.00 m)	
7. Downstream H.F.L.	= RL 78.50 m
8. Free board-upstream/downstream	= 1.2/1.5 m
9. T.B.L. of coffer dam upstream/downstream	= RL 85.2 m / 80.0 m
10. Bed level of channel at entry	= 79.0 m
11. Non-monsoon flow during 1959-66	= 3000 cusec (85 cumec)
12. Minimum water level during non-monsoon	= RL 77.0 m

3.2.18 Cofferdam:

“A cofferdam is a temporary structure which is built to exclude earth and water from an area in order that work may be performed there under reasonably dry conditions. Cofferdams are usually required for projects such as dams, locks and piers that are constructed in rivers or in other bodies of water. Also, they may be installed to prevent the flow of earth and water into foundation pits excavated on land.

A cofferdam does not have to be entirely water tight in order to be successful. It usually is cheaper to permit some flow of water into the working area, which can be removed with pumps, than to attempt to make the cofferdam water tight. The most satisfactory cofferdam is the one that has adequate strength to resist all destructive forces and that will permit the exclusion from or the control of water inside the dam at the lowest total cost. The total cost includes the cost of the cofferdam, the cost of damages due to water flowing into or over the dam, and the cost of pumping water from the area inside the dam, less any salvage value after the dam is removed. For many projects it is possible to determine these costs with reasonable accuracy prior to designing the cofferdam.” (Source: Construction Planning, equipment, and methods by R.L. Peurifoy, 2nd edition-1970 Pg.539-40)

F cofferdam, C.W.C. prepared a memorandum in this regard and forwarded to the state engineers for their scrutiny and views.

The gist of memorandum is given as under:

The deepest bed level in the deep channel course of the river 100m upstream of dam is in average 62m which it is 60m at 95m downstream of power house. The minimum water level in the river being RL.77m, the cofferdam is required to be formed in standing water at depth of about 15m in upstream and about 16m in downstream. In above situation, construction of cofferdam is likely to encounter the problem of moderate to heavy seepage. For tackling this, two alternatives were suggested by CWC.

1st alternative: This assumes that, the seepage through cofferdam as well as through the abutments will be moderate. As per tentative programme made for construction of power dam, the period considered is two years which may extend to three years in unavoidable situations. In order to obviate the need to construct the high rubble cofferdams involving considerable quantity of materials every year, construction of masonry cofferdam upto certain level in first instance was thought to be convenient and economical. In subsequent years, earth will be merely deposited on the water side of the wall upto required level. Alternatively a temporary masonry wall may be built on the crest of regular cofferdam which can be demolished before the flood. The method of raising the masonry cofferdam is given hereunder.

To start with, the rock fill dams will be raised on both upstream and downstream sides up to RL.85.2m and 80.0m respectively. The seepage water will be collected and disposed off by pumping out. Masonry cofferdam will be provided with suitable ogee section at top and adequate protection arrangement at the downstream based on model study to pass the flood water safely over them.

2nd alternative:

If the seepage water is enormous and beyond the means of dewatering economically and timely as it happened in Srisailem Project across river Krishna, the construction of masonry wall as contemplated in alternative-1 above will not be practicable, specially in deeper portion of river. In such case construction of concrete walls are to be resorted to in standing water. The concrete walls can be restricted to the deeper portion of the river. On either side of the deeper portion above the bed level of RL.77.0m masonry walls can be constructed as proposed in alternative-1.

On receipt of above memorandum from C.W.C. detail exercises were made by project engineers. The project authorities considered 4 alternative proposals.

Proposal-1

Rockfilled coffer dams will be raised on both upstream and downstream upto RL 85.2 m and 80.0 m respectively diverting the river flow through diversion channel. The pit will be dewatered and masonry coffer dams of adequate section will be construction on dam side of both coffer dams to RL 82.0 m / 76.0 m on upstream and downstream side respectively. Approximate cost will be Rs.1,13,86,000/-.

Proposal-2

Rockfilled coffer dams will be raised as in proposal-1. The colcrete walls will then be constructed under water. The upstream colcrete wall will be upto RL 81.00 m with 1.0 m further raising by masonry. The downstream colcrete wall sill be upto RL 75.0 m with 1 m more height of masonry. Approximate cost will be Rs.1,65,62,000/-.

Proposal-3

This is a modification of proposal-2. Instead of colcreting a single rubble fill, coffer dam with impervious zone in upstream side can be constructed. The downstream rockfill coffer dam has been avoided. In place, colcrete walls can be constructed as in proposal-2 upto RL 78.0 m both in upstream and downstream. Approximate cost of construction will be Rs.1,53,09,000/-.

Proposal-4

Two low coffer dams with armour stones will be constructed upto El. 77.00 in the first year of river diversion both in the upstream and downstream. The next monsoon flood will be allowed over this. The armour stones will be launched to the natural bed of the river and the interstices will be filled up with silt and sand. The upstream sides of upstream and downstream coffer dams will be sand cast which will facilitate driving sheet piles for achieving watertightness. The rubble coffer dams will be raised to El.85.20 m. at the upstream and to El. 80.00 at downstream at the next season and the sheet piles will be driven. This arrangement amounts to Rs.1,17,55,000/-.

Out of four alternative proposals described above, the most realistic proposal befitting to the site condition was examined in detail by Er. D. Mishra, the then Superintending Engineer and a comprehensive plan for the coffer dam was proposed which is described below:

It was proposed to take up excavation of diversion channel of Rengali dam from 1.11.75. The excavation of channel and construction of diversion sluice with gate arrangement are likely to be completed in two working seasons i.e. by 30.6.77. Work in deep water portion could be taken up from the next working season beginning from 1.11.77, provided the coffer dams are substantially completed by that time.

In order to achieve this objective, it was proposed to construct a rubble dam about 150 m upstream of the dam axis with effect from 1.11.75. This dam will have top level of 77 m with side slopes of 1.5:1. The bund will be formed with boulders weighing between 2 Tons to 10 Tons. Heavy stones were proposed to be quarried from the river bed within 2 km. lead and loaded into tipper/trucks with the help of cranes.

Tentative proposals for the proposed coffer dam was received from the Director, C.M.D.D. in their letter No.3/130/75-CMD-1042 dt.20.5.75. These proposals also provide construction of two coffer dams, one upstream side and one in downstream side with rubble and filter and cut off provided either by colcrete or masonry wall in the dam site of coffer dam.

Costing at the prevailing rate was made and the figures are as follows. the Director C.M.D.D. was requested to examine the proposal prescribed above and evolve the final design for the coffer dam.

Table- 3.7 Abstract of cost of Coffor dam (in Rs.)

		Up-stream	Down-stream
1.	Rockfill coffer dam	32,84,487.00	11,82,473.00
2.	Masonry cut off dam	17,42,000.00	5,10,000.00
3.	Colcrete cut off dam	9,73,200.00	3,07,200.00

Subsequently after, several rounds of discussion were made between State and C.W.C. Engineers for exact line of action and the procedure to be adopted for construction of coffer dam. The building of rubble coffer dam for both in upstream and downstream was a pre-requisite and common for all the alternatives. The work was thus planned to be taken up from 1976-77 working season. There was proposal to roll armoured stone about 150 m up-stream of dam axis so that it would remain clear from dam axis for taking up the works. Though it was decided to quarry, carry and dump the armoured stones of 2 to 10 Ton size, there cropped up practical problems at site. As such a substitute for armoured stone was thought of. Boulders packed in wooden ballah crate tied with wire was used in other location to serve as horizontal platform and low piers. The idea of wooden bullah filled with boulders was therefore used for the purpose. The crates were manufactured at site of work and were filled with abundantly available blasted boulders.

Immediately from working season of 1976-77 boulder packed bullah crates were launched from launching platform into river at coffer dam location, but subsequently it was experienced that manufacture of bullah crates were time consuming as it involved cutting, fixing and fastening etc. Alternatively wire crates were thought of. The wire crates were woven with 6 mm M.S. rods. It gave certain degree of flexibility while packing the boulders and subsequently rolling into the river. These wire crates were found to be one of the best solutions to serve the purpose of armoured stones.

1976-77:- In this season, it was planned to create a pervious barrier by boulder crates. Accordingly by launching boulder crates the upstream gap of the river 150 m upstream of dam axis was bridged. However a small gap was left out near the right bank to facilitate river to be closed quickly and not to create any major imbalance, in gap closing. Lest the crates may be dislodged in the ensuing flood, the crates were loaded on top by cast-in-situ concrete blocks of 2m x 2m x 2m size. It was therefore expected that during the floods of 1977 the pervious barrier will be considerably choked with silt, and debris etc. thus making it fairly impervious. Further sand casting induced in its front face will facilitate to drive sheet piles. In the working season 1976-77 the work of downstream coffer dam was not taken up and it was thought to observe the effect of river on upstream of coffer dam and to take up its work from the experience gained. However to protect the downstream toe of upstream coffer dam precast concrete blocks (1.67m x 1.67m x 1.21m) size were dropped by means of cranes.

The project authorities were optimistic regarding deep channel closure in 1977-78 i.e. from October'77 to May'78 and accordingly plan and programme were detailed to take up the foundation and concreting work in the deep channel.

1977-78:- After passing of 1977 flood, it was observed that the central section of the cofferdam in the deep-channel portion had settled subsequently which was attributed due to scouring of bed materials. The crates had been dislodged and readjusted.

3.2.18.1 Construction of Upstream Cofferdam:

The river gap was closed by crates along the upstream cofferdam after the flood receded. It was planned to build the upstream cofferdam with 12m top width (3 rows of 2m x 2m x 2m concrete blocks + 3 rows of 2m x 2m x 2m boulder crates) with concrete blocks to remaining downstream edge. Non-availability of earth in nearby areas resulted to transport huge quantities of muck and sand for building up the upstream slope of upstream cofferdam.

A ring bund was built downstream of upstream coffer dam to cordon the power house pit, as muck and earth continued to be dumped in the upstream of up-stream cofferdam. Lot of materials continued to pass downstream through interstices and gaps of boulder crates and substantial silt deposit was observed between upstream of ring bund and downstream of upstream coffer-dam. As river has taken a sharp bend at this place, the bed materials were arrested at the ring bund. The building of up-stream slope was a difficult proposition as materials used to roll into the deep-channel with standing water of 15 m depth, and the slope used to slump down with big chunks of earth disappearing in the upstream pool. The diversion of river starts functioning immediately after the closure of up-stream gaps. However, seepage was quite predominant at the initial stage soon after gap closure. The sill level at entry of diversion channel being at El.79.00 m diversion started little late and there was always an apprehension that part of upstream dumped materials from the body of the cofferdam might get carried away to the diversion channel.

3.2.18.2 Construction of Downstream Cofferdam:

After the up-stream cofferdam had been successfully built and river diversion started, the building of downstream coffer dam by dumping rubble was an easy task. It was decided to dump the excavated boulders from power house pit direct into the down-stream coffer dam portion to avoid rehandling and in this manner, quick plugging was effected in the downstream side. The excavated boulders were dumped from one end and continued to proceed in the axis of cofferdam from left bank to right bank and the gap was closed by April'78.

As the downstream cofferdam was built of boulders only it was decided to give a protective cover by covering with a layer of crated boulder both on top and on up-stream and down-stream slope by rolling crates cover it. This protection work was completed by end of May, 1978.

The upstream slope and up-stream cofferdam when showed signs of stabilisation it was decided to drive sheet piles from left flank covering the deepest portion of river. Accordingly sheet piles were driven in part of the length before the onset of monsoon.

1978-79:- This working season was critical for cofferdam as it was decided to dewater the area between up-stream and downstream cofferdam to test the water tightness and take up the dam work in deep-channel portion.

Immediately after the flood had receded, the up-stream coffer dam section was resorted and road was made over it. In order to stabilise the upstream slope of upstream cofferdam, the 1st priority was to dump muck and then sand.

3.2.18.3 Construction of Upstream Auxiliary Cofferdam:

Soon after 1978 monsoon flood, the river bed was surveyed to see the configuration of the river bed and slope of the upstream cofferdam. It was revealed from the bed contour of the deep-channel that the sand had been deposited in the bed in a triangular fashion i.e. maximum 5 to 6 m near the edge of the cofferdam and minimum at about 40 to 45 m away from the edge. The configuration showed that the deposited sand had taken a very flat slope to join the deepest channel level. So this year for completing the coffer dam, the filling had to be done negotiated with the deepest level and to protect the

slope by armoured blocks/concrete blocks. It was feared that this was quite a voluminous work and this would be continued in the subsequent years unless this was arrested by means of any sort of checkdam. A check-dam would arrest and to protect the sand and earth filling to be carried away further upstream. So the question of putting an auxiliary cofferdam approximately at the upstream toe of upstream main cofferdam was thought of to confine the deposits. This auxiliary wall would compose of armoured stones in crate. This wall would also prevent drag of the materials. The upstream slope was thus made stable and sheet piling work started from 22.12.78.

3.2.18.4 Construction of Downstream Auxilliary Cofferdam:

It was decided to attain the water tightness of downstream coffer dam by dumping earth only as it could be tackled independent of sheet piling. However in order to protect the earth being slumped and washed away, an auxiliary cofferdam downstream of downstream coffer dam was built so that earth remained confined between the crate line. This year also the ring bund cordoning power house pit was done. After the dewatering of power house pit, the deep-channel dewatering was resorted to. This was tackled by 300 H.P. pumps which were installed in the right slope of deep-channel. Each 100 H.P. pump has got capacity of 165 lt/sec. i.e. 5 cusecs and dewatering was achieved with great success. The pumps were lowered in stages by erecting plat-forms on rock base and no major problem was encountered.

As power house unit had been extended into deep channel due to the addition of 5th unit, it was decided to provide an access into deep-channel from downstream cofferdam between the fourth and fifth unit to facilitate excavation and removal of flood deposit. After the sheet piles had been successfully driven, it was proposed to construct the upstream section of dam only which serve as permanent barrier for subsequent seasons. However, for raising the dam extensive foundation treatment would be necessary requiring very close blasting in next season and as such it was thought of to build atleast a temporary masonry wall clear of upstream heel of dam such that it would act as permanent barrier for subsequent season. However, for raising the dam extensive foundation treatment would be necessary requiring very close blasting in next season and as such it was thought of to build atleast a temporary masonry wall clear of upstream heel of dam such that it would act as permanent barrier for subsequent seasons. For dewatering to lower the water table, the well point system was tried. In the 1st week of February '79 a row of well points about 25 numbers were driven at about 25 metre upstream from the dam axis near the right flank. Well-point could be driven by water-jetting only upto 2 to 3 m. Fragments of rock and pebbles hindered in driving. Performance of well points was very low. Another row of well points was also tried by driving the holes with wagon drills. This also showed very poor result with approximate discharge of 0.50 cusec. Thus well point system was abandoned.

For construction of temporary masonry wall it was necessary to retain the slope in sand and muck which has since been lying downstream of up-stream cofferdam as trapped deposit. It was decided to drive 80 lbs. N.G. Rails at close intervals by hand driven hammers, so that the flanges of rail closely spaced would prevent the slope being caved in. No of small pumps were installed to dewater the seepage water in multi-stages. However, the river flow by 3rd week of April decreased substantially and discharge was of the order 15 to 20 cumec.

In any major works, there will be obstacles and failures. But a competent and successful engineer knows how to tackle the problem and come out victorious. With much difficulties, it was possible to reach up to the solid rock level and lay 1st phase of concrete.. By 1st week of June 1979 a temporary masonry wall was raised high above the required level of 80.75 m. Thus a 1st major break-through in the river diversion and building cofferdam was achieved by June 1979.

1979-80:- After the up-stream barrier had been successfully raised, it was decided to tackle the downstream side in 79-80 season, and to take up the work on main dam in deep-channel.

The western side protection wall for power house was to be built right inside the deep-channel butting to the toe of block No.15 and 16. As such it was decided that while the main dam work in block 15 and 16 would be in progress near the up-stream side of the downstream part of end protection wall could be raised, then there would be no necessity of a secondary wall as was done in the upstream side.

In order to achieve this, just after flood, the road in the upstream cofferdam was restored and earth was dumped in between the downstream cofferdam and auxiliary crate line to achieve water tightness to take up dewatering.

The dewatering was carried out successfully and removal of flood deposit was taken upto El.64.00m by mechanical meals. Below El. 64.00m the removal of flood deposit was taken up by crane. In order to clear the foundation for raising the western protection wall it was necessary to protect the up-stream slope of downstream cofferdam which was achieved, by driving 100 x 100 x 6mm M.S. angle supported by heavy joists spanning between the rock edge of left and right banks of deep-channel. To reach foundation level, driving of M.S. angles were taken up, in four stages.

The time was running out as the month of May had set in and there was no trace of foundation work in sight. However, a final attempt was made on 7th of May 1980, luck smiled and the foundation bed rock was found at an El. 51.7m after incessant effort from November, 1979. As monsoon was to set in by 1st week of June 1980 and machineries were to be removed by 1st of June 1980 an all out effort was made and concrete was raised upto El. 64.0m in the downstream side and about 60% of the bottom area was covered with concrete.

Thus finally the deep-channel was sealed from upstream to downstream side and part of dam in block 15 and 16 was raised upto El. 70.00m by end of May 1980 and the nightmare of diversion and building dam in deep-channel was over. Thus the herculean task was accomplished in record time.

Tab-3.8 Stages of work executed in cofferdams

Sl. No.	Item of works	U/S. Cofferdam		D/S. Cofferdam	
		1977-78 upto 30.6.78	1978-79 upto 30.6.79	1977-78 upto 30.6.78	1978-79 upto 30.6.79
1	Crate dumping (nos)	1402	366	255	371
2	Rubble dumping (cum)				
	a) In crates	9361	2644	1701	2475
	b) In loose-for body of cofferdam.	15152	4055	15323	1229
3	Muck dumping (cum)	6744	5621	-	-
4	Spalls & metal dumping (cum)	4932	787	-	-
5	Sand dumping (cum)	22765	48101	-	-
6	Earth dumping (cum)	-	218.00	-	222
7	Steel sheet piling (m)	686	2700	-	-
8	Steel piles recovered (nos)	-	1376	-	-
9	Concrete block cast in situ 2mx2mx2m (nos)	149	-	-	-
10	Concrete block placing 1.67mx1.67mx2m (nos)	125	-	-	-
11	Masonry & Concrete walls Masonry (cum)	-	3350		
12	Concrete (cum)	-	1307		

3.2.19 Sequence of River Diversion:

- i) Diversion channel excavation started in January, 1976 and completed in March 1977.
- ii) Construction sluice masonry and concrete work started in April, 1977.
- iii) Upstream rubble filled crate cofferdam work commenced in December, 1976 with top level at 78.50m.

Progress of Launching Crates:-

- a) By 6th January, 1977 ... 100 nos.
 - b) By 8th February, 1977 ... 322 nos.
 - c) By 3rd. March, 1977 ... 650 nos.
- and the river gap bridged

- iv) After 1977 monsoon, the river gap was closed by crates with top width 12m and dumping of earth and muck continued in the slope of upstream of cofferdam.
- v) After 1978 monsoon, the river gap was closed on 19.11.78.
- vi) Sheet piling was taken up in the upstream coffer dam from 22.12.1978.
- vii) Dewatering in the deep-channel started from 17.1.1979.
- viii) Deployment of well-point system of dewatering in Feb.'79.
- ix) Trash-rack structure concrete was laid in block No.13 on 6.1.1979.
- x) Trash-rack structure concrete completed for block No.12 and 13 upto El. 80.75 m on 19.5.1979.
- xi) After 1979 monsoon, deep-channel work commenced on 17.10.79.
- xii) Dewatering in deep-channel started on 8.11.1979.
- xiii) Dewatering in deep-channel completed on 17.11.1979.
- xiv) Concreting in deep-channel started in block No.15 on 4.2.1980.

3.2.20 Plugging of Diversion Sluice (during 1984):

The river Brahmani had a perennial flow of water at Rengali Dam site which passed through a deep gorge. For work in the deep channel portions at block 19 of dam, a diversion arrangement had been made. The summer flow was craddled between massive granite protrusions and led to the downstream side through two sluices of size 3.5 x 4.0 m each in block 21. The depth of flow in sluices were limited to 2 metres only in the months of April and May.

For building up the reservoir, plugging up of the sluices were necessary. By March, 1984 the radial gate works were in progress, works at the penstock gates on the upstream side of power house were still to be completed. And above all, the evacuation of people from the reservoir area had not been over. Further, if both the vents would be plugged, water level in the upstream side would not fall below the spillway crest i.e. 110.20 m and any rectification works in the dam, power house area below this level on the upstream side would not be possible.

Keeping this in view it was decided to;

- 1. Fix M.S. gates at both sluices
- 2. Attempt to plug only one vent.

3.2.20.1 Arrangement for Plugging:

a) Cofferdam: It was not possible to totally stop the flow as no other diversion was possible. Month of March was chosen to start the work in view of low discharge in the river and to get time before onset of monsoon for concreting work.

As the water was to be allowed through one vent, coffer dam in front of one vent was proposed. Due to high velocity of water, dumping of earth or any other material was not suitable. Hence on 25.3.1984 precast concrete blocks of size 1.7 x 1.5 x 1m were preferred to be lowered into the diversion channel perpendicular to the dam within the 5 m space between the vents and gradually advanced to 5m upstream with help of the 10 Ton derrick crane positioned on the upstream of the block 23 for gate works. This did not pose difficulty in placing concrete blocks, as the flow of water of about 2m depth was not obstructed. Two layers of blocks were placed one above the other so that the top of the upper block protruded above the water surface.

Next, some more blocks were lowered in front of left vent in two layers closing upto the bank of channel. But this did not stop the flow of water in the sluice in any way.

The channel bed was uneven and some debris were also there underneath the concrete blocks. Sand bags were then placed on either side of blocks in front of vent and closely stacked. But sand bags were placed only on the inner side of the concrete block placed perpendicular to the dam between the vents. The joints of the bags were applied with a coating of clayey soil to prevent leakage into the vent but to no effect.

On 2.4.84, the divers of Abee Engineering, Calcutta were called in. They worked for two days and applied a mixture of clayey soil, cotton waste, cement and grease at joints and at bottom of sand bags after clearing debris and were successful in preventing the leakage into the left vent by dt.4.4.84. Dewatering was taken up with help of a 5 H.P. pump kept on the left bank of channel. As the water level on the downstream side of the dam was RL 78.50 m which was 0.50 m above the sill level of sluice 0.5 m depth of back water remained in the vent. So another coffer arrangement with sand bags was done at downstream end inside the sluice to prevent the back water into the sluice. Thus on dt.7.4.84, it was possible to provide a fairly dry area for working for fixing the vent gates on the left sluice.

b) Fixing the Sluice Gate on Left vent:- The M.S. gate of size 4x5 m was fabricated at the workshop with guide, bottom and top seal beam and track etc. The guide seal beam and tracks were fixed on the face of the vent with secondary concreting. The track guides extended upto El.88.50 m to keep the gate in lifted position for allowing water in the sluice and for convenience of lowering. The weight of gate was such that it would be lowered with summer flow in the vent on 5.5.84. The secondary concreting was completed. Trial for lowering and lifting of the gate was made and tied by wire rope with the projecting joists above the vent at El.108.00 m.

c) Fixing the Sluice Gate on the Right vent: Then the cofferdam in front of the left vent was dismantled and shifted to the right side. In the similar process, with help of divers of Abee Engineering, the coffer dam could be completed by 9.5.84 while the water of river was flowing through the left vent only. The fixing up of the M.S. gate in front of the sluice was completed on 21.5.84. The gate was fixed thereby sealing the vent.

d) Plugging of the Right vent: To prevent back water into the vent a small cross bund with help of sand bag was already there for fixing the gates. After cleaning the vent and chipping, a 0.75m x 1 m high concrete wall was raised 1 m away from the gate in the vent. The purpose of this wall was to arrest the leakage through the gates within the space for convenience of diverting the collected water through a 50mm dia G.I. pipe to the downstream side.

Then concreting was done in the sluice upto 2 m to 3 m height in stepped manner manually from upstream to the downstream end, the upstream gap was also filled up. Grouting pipes of 50mm dia and 20mm dia were buried in the concrete staggered in two layers.

Due to inconvenience of working in the sluice, concreting was done with concrete placer of capacity 1 cum. “Lomer D”, a retarding reagent was used to delay the setting time of concrete. Before on set of monsoon, the work was abandoned for the season. Only 200 cum. of concreting could be laid on 29.6.84.

During monsoon of 1984, the reservoir started building up. As one vent of diversion sluice was not capable of discharging the full inflow into the reservoir, the reservoir gradually started building up and the M.W.L. rose to El. 117.40 m in August '84. Out of 24 radial gates, 22 nos. of gates were completed. As all gates were not completed and as the unit one of the power house was not ready, the reservoir was not allowed to build up to its F.R.L. Though the reservoir could not be regulated due to non-completion of radial gates even then floods of 1984 monsoon could be moderated due to substantial absorption of the same in the reservoir. Therefore the benefits of flood control was achieved partially from 1984 monsoon.

Further it was noticed that keeping one vent of sluice open, non-completion of the gates became a blessings in disguise. When the reservoir rose upto El. 117.40m, it was noticed that the leakage inside the gallery was 40 lt/sec. Leakage in drainage gallery was anticipated to be 100 lt/sec at El. 123.50 m from the extrapolated graph of reservoir level-Vs-leakage inside the gallery. Pumping capacity at the slump was therefore estimated to be insufficient, as one out of two nos. of 50 H.P. (50 lit/sec.) capacity pumps was kept as a stand by. Therefore another diesel pump was installed to work as stand by. If the leakage would have been 100 lt/sec. during monsoon of 1985 both the electric pumps should have run without keeping any standby except the diesel pump, but fortunately it could be seen in 1985 monsoon that the leakage was limited to 40 lt/sec. even though the reservoir level got its full level on the day of “Dasahara”.

Another aspect is that there was heavy leakage inside the gallery in the power block portion which resulted in increase of gallery leakage. After the monsoon, the reservoir started receding and became empty. This gave an opportunity to find out the reason of heavy leakage into the gallery in the power dam portion. On examination from the upstream side, it was noticed that a layer of concrete more or less at El. 89.00 m was completely porous. This layer was at the junction between the old concrete and the new concrete, which was done in the year 1984. This porous concrete may be due to leakage of slurry from the bottom of shutter plates. The remedial measure was taken by chipping out the porous concrete for a width of 0.3 m and a depth of about 0.3 to 0.4 m and then grouting the whole area by cement slurry by fixing the nipples. After grouting was completed, the chipping portions were concreted providing dowels and additional reinforcement etc. This remedial measure helped a lot and it was in the year 1985 monsoon that the leakage was practically reduced to nil except in a porous block in block No. 13. This leakage through the porous block in block No. 13 was diverted to the out side of the dam with the help of the pipe fitted into the porous blocks and laid at the ceiling of the gallery and taken out through the adit of block No. 9. This remedial measure perhaps reduced the leakage into the gallery which remained maximum upto 40 lit./sec. in the monsoon year of 1985 when the reservoir level was at full reservoir level.

After the depletion of reservoir, attempt was made to plug the balance portion of right vent and also to plug the balance portion of right vent and also plug the left vent completely. For plugging the left vent it was decided that as soon as the gate is closed the upstream side of the diversion channel would be filled up with earth to stop leakage through the sides of the gates. Therefore preliminary arrangement was made by dumping the earth by the side of the diversion channel adjoining to the upstream of dam in block No. 20. Due to dumping of earth on the upstream side, the leakage through the gates into the left vent reduced substantially.

A grout pump on down stream side was used to plug the balance portion of the right vent for full length of the diversion sluice. The bottom depth of concrete was placed manually. The upper portion was laid with the help of concrete placer which was positioned on the downstream side of the vent shuttering. The shuttering was made at 3 m. interval and the pipe from the concrete placer was penetrating through the shutter. After concreting, the shutterings were removed and the pipes were readjusted and the placer concrete continued. Thus the plugging of both the vents were completed.

3.3 Power Dam, Power House and Tail race Channel:

3.3.1 Introduction:

“Over two billion people in developing countries are yet deprived of electrical energy. Driven by rising populations, expanding economies, energy intensive industries, urbanisation and a quest for modernisation and improved quality of life; energy use in the developing world has doubled in the last two decades and further expected to double again in an even shorter time span of the next fifteen years. Indian per capita consumption of electricity continues to be extremely low around 350 kwh per annum. Another striking fact remains that only one third electric supply is consumed in the rural areas despite three fourths of Indian population living in rural areas. While 86% of villages have access to electricity only about 30% of the rural households are able to use electric power. About 80,000 villages remains yet to be electrified inspite of the highest priority given to rural electrification in India. Most of these villages are located in far flung and remote areas, with very low load densities requiring heavy investment in electrifying these villages.

x x x x x x x

The detailed power planning studies carried by Central Electricity Authority (CEA) have convinced that the share of hydro power in the overall installed generated capacity in the country should be atleast about 40% to ensure optimum utilisation of natural and financial resources for electric power generation. Thus the accelerated hydro power generation is unavoidable preposition when about 75% of the hydro potential of 84,000 MW still remains to be harnessed. x x x x x . Indian power development needs priority correction by substantial addition of hydro power generation capacity.

India is endowed with primary energy resources in various forms- water, fossil fuels (coal, lignite, oil and natural gas) and nuclear fuel and will serve as major sources for power generation. Non-conventional and renewable sources of energy such as fuelwood, biomass, tidal, solar, wind and geothermal energy are also available but they are in preliminary stages of development. These resources are not evenly distributed over various regions/states in the country. Over the period of next fifteen years (upto 2006-07) it is estimated that hydro power as a renewable source and fossil fuels i.e. coal and lignite will remain main sources for power generation in India duly supported by natural gas to some extent. Share of nuclear power is expected to increase appreciable in future.

The economics of power generation through non renewable sources of power-fossil fuels-changes with the exploitations of every successive ounce of such fuels. Renewable sources-hydro, solar, wind, tidal etc in this background offer a lucrative option in the long run. With the status of technologies developed for exploitations of these renewable sources of energy, so far, hydro power appears to offer major attraction and deserves to be given highest preference amongst various options available. It is the only renewable form of primary energy with substantial unexploited potential with number of other major technical and economic advantages by virtue of non polluting nature, high conversion efficiency, flexibility in operation, relatively lower cost of generation, operation and maintenance, longer life of equipment etc. At the same time it has to be appreciated that (a) bulk of the unexploited hydro resources are in difficult/inaccessible terrains in the Northern and North Eastern Regions and (b) hydro projects generally entail longer gestation periods. Central Electricity Authority has been consciously keeping these aspects in view while carrying out studies for evolving long term perspective National Power Plans.” (Source: Hydro-power and River Valley Development, Post conference Volume-Institution of Engineers India & WAPCOS, Dec. 1999 Pg.52-53 by R.N. Srivastava & R.S. Goel)

3.3.2 Why Hydropower?

“Hydropower is a renewable source of energy and exhibits operational and economic superiority over the other modes of power generation particularly in catering the peaking power requirements. It is a clean and non-polluting resource of energy with high conversion efficiency. The other advantages of hydro power are:

- Storage based hydro electric schemes often form part of multipurpose river valley projects with benefits of irrigation, flood control, drinking and industrial water supply, navigation, tourism, pisciculture etc.
- Generally the cost of generation from a hydro power station is relatively low as compared with other sources of energy generation completed during the same time and emerges to be cheapest in the long run as it does not involve any fuel component.
- The hydro electric plants have longer span of life ranging from 35 to 50 years and even more.
- Hydro power is reliable source of energy with high availability factor and minimum maintenance needs.
- Quick start and stop, picking up and dropping loads in a few minutes, hence are suitable for peak load operation.
- The peaking operation of hydro projects enables optimum utilisation of thermal capacity.
- Due to fast response, the hydro plants enhance system stability and reliability and enable optimal operation of the system.
- Socio-economic benefits in remote areas where hydro projects are generally located, such as infrastructural development, improvement in local economy, employment generation, etc.”

(Source: Ibid Pg.102, by D.V. Khera, R.S. Chadha and S.D. Dubey.

Growth of total energy from all sources Vs. share of hydropower from 1947 to 1998 in the country is furnished vide Table-3.9

Table- 3.9 Growth & Share of Hydropower

Year	Installed Capacity (MW)		Hydro share (%)
	Total	Hydro	
1947	1361.76	508.13	37.31
1950	1712.52	559.29	32.66
1951	1835.43	575.18	31.34
1952	2061.76	715.18	34.69
1953	2305.19	731.18	31.72
1954	2494.00	793.35	31.81
1955	2694.82	939.48	34.86
1956	2886.14	1061.44	36.78
1957-58	3223.11	1213.92	37.66
1958-59	3511.59	1361.44	38.77
1959-60	3873.17	1530.15	39.51
1960-61	4653.05	1916.66	41.19
1961-62	5218.82	2419.10	46.35
1962-63	5801.19	2936.35	50.62
1963-64	6576.94	3167.02	48.15
1964-65	7396.67	3388.73	45.81
1965-66	9027.02	4123.74	45.68
1966-67	10092.17	4757.22	47.14
1967-68	11888.16	5486.92	46.15

Year	Total	Hydro	Hydro Share (%)
1968-69	12957.27	5906.91	45.59
1969-70	14102.45	6134.70	43.50
1970-71	14708.95	6383.23	43.40
1971-72	15254.37	6611.61	43.34
1972-73	16281.71	6785.41	41.68
1973-74	16663.56	6965.30	41.80
1974-75	18316.68	7529.24	41.11
1975-76	20117.06	8463.60	42.07
1976-77	21468.50	9024.90	42.04
1977-78	23668.71	10020.22	42.34
1978-79	26680.06	10833.07	40.60
1979-80	28447.83	11383.97	40.02
1980-81	30213.68	11791.22	39.03
1981-82	32345.09	12172.81	37.63
1982-83	35363.27	13055.86	36.92
1983-84	39338.86	13855.56	35.22
1984-85	42584.72	14460.02	33.96
1985-86	46769.03	15471.60	33.08
1986-87	49265.86	16195.64	32.87
1987-88	54155.17	17265.33	31.88
1988-89	59040.38	17798.05	30.15
1989-90	63636.34	18307.63	28.77
1990-91	66086.33	18753.42	28.38
1991-92	69065.39	19194.62	27.79
1992-93	72319.46	19568.76	27.06
1993-94	76718.21	20365.91	26.55
1994-95	81164.41	20829.04	25.66
1995-96	83287.96	20976.00	25.18
1996-97	85019.31	21644.80	25.46
1997-98	88266.86	21891.08	24.80
1998-99	93253.00	22443.00	24.10

(Source: Ibid, Pg.89)

3.3.3 Power Dam:

Rengali dam has been constructed across river Brahmani connecting two hills, Salimunda on the right and Machhakhani on the left. The entire dam is divided into 51 blocks, out of which, blocks 10 to 14 are power dam blocks. The main power house is located immediately downstream of these blocks. Water is led into the power house through entrance structures and penstock pipes at the centre line of each of the power dam blocks corresponding to power units No.1 to 5.

3.3.3.1 The Structure:

The Power Dam blocks are gravity type consisting of concrete and R.R. stone masonry. The T.B.L. is at El.128.50m and the top is 8.1 m wide. The downstream face is vertical upto El.114.00m and slopes down-wards at 0.9 (horizontal): 1.0 (vertical). This is different from the other blocks of the dam which have a down stream slope of 0.75 : 1.0. The power dam blocks have trash-rack arrangement on the up-stream side followed by the rectangular bellmouth entry, transition structure from rectangular to circular and circular steel penstock thereafter leading to the power house. Three galleries are located

in the power dam blocks viz. the foundation gallery, the bypass gallery and the hoist gallery. The foundation gallery is meant to function as the drainage gallery of the foundation and the body of the dam. The drain slopes towards block No.16 of the dam, where the sump is located. The bypass gallery house the valves for the bypass pipes to be operated when it is required to create balance condition for operation of the penstock gates. The hoist gallery accommodates the hydraulic hoists for operation of the penstock service gates.

The trash-racks have R.C.C. columns and beams with racks of mild steel. The entrance structure is of rectangular R.C.C. construction, containing the emergency gate groove and the service gate groove. The transition zone is also constructed with R.C.C., gradually changing from rectangular to circular in cross-section. Thereafter steel penstocks have been provided with R.C.C. all around upto the power house. Stone masonry construction was attempted in all power dam blocks above El.105.00 m (approximately), but had to be changed after some time since the progress of construction of masonry did not match the time schedule requirements. The balance portion was constructed with concrete. Reinforcements, as required, were provided around the various galleries in the body of the dam.

3.3.3.2 The Foundation:

In general the foundation comprises of hard granite rock. There was a fault zone in the foundation of block No.10, approximately at the mid portion of the block running diagonally from the upstream to downstream side for a small length. Since this was a localised pocket, the weak materials were removed upto a depth of about 2 m. and concrete was filled up as advised by the Geologist. Another small fault zone existed in block No.13 of the dam quite close to the joint with block No.12. This was also treated alike. The foundation of block No.14 sloped down-wards towards block No.15. The loose rock masses were removed and the entire foundation was anchored to the bed rock.

3.3.3.3 Trash-Rack:

Trash racks have been provided at the entrance to the control structure upstream of each of the power dam blocks for arresting wooden logs and other floating debries so that those do not enter into the penstock. The main structure of the trash rack consists of R.C.C. column and beams with shape to suit hydraulic performance, from El.85.584 m upwards. R.C.C. slab has been provided at the top at El.112.12m. The breast wall has been provided above the slab up to the top of the dam, upstream of the face of the dam. The trash rack segments are constructed with mild steel flats with a clear opening of 68 mm. The racks are placed in segments of a semi circle with outer radius of 9.254 m.

3.3.3.4 The Entrance Structure:

Each of the power dam blocks is 21 m in length at the centre of which the entrance structure to the penstock is located. The bellmouth entrance has a width of 5.65 metres and extends from El.89.839m to El.99.989m. The penstock stoplog groove projects beyond the upstream face of the dam and it is required to close this opening when the penstock gate is under repair or maintenance. The penstock gate is located at a distance of 4m. from the axis of the dam. The width of the opening at this point is 4.65m and height extends from El.89.069 m to 96.90 m. Above this elevation penstock gate groove has three platforms at El.98.001m, El.105.768 m and El.114.168 m from where dogging arrangements to the penstock gate can be made when the same is being lifted for repair and maintenance. The operating platform is at El.125.50m. A brick masonry wall 300 mm wide and 500 mm high has been constructed inside the hoist gallery separating the gate grooves from the main passage of the gallery so that there is no sequential flow of water into the main gallery in case of sudden closure of turbines in M.W.L. conditions in the reservoir. An air vent pipe (for each gate) one metre in diameter takes off directly from the flow passage immediately downstream of the gate and opens out on the downstream side above the maximum water level. The bypass pipe 600 m in diameter opens into the flow passage downstream of the service gate and is directed towards downstream side so that the water does not hit the walls of the flow passage during operation of the bypass valve.

3.3.3.5 The Transition:

The transition is designed hydraulically for smooth passage of flow from the rectangular entrance conduit to the circular penstock and is 10 m. in length.

3.3.3.6 The Penstock:

The penstock pipes have inner diameter of 5960 mm and are fabricated from 20mm thick H.T. steel plates. The penstock is inclined at 36° to the horizontal in the beginning and become horizontal with its centre line at El.72.21m. before entrance to the power house. The total length of each penstock steel linear is 34.57 m. The scroll case of the unit is connected to the penstock pipe at a distance of 50.22m from the axis of the dam ('D' line of the power house). The penstock has been fabricated from the plates in pieces not exceeding 2.5m in length. Smaller lengths have been provided at turning points. Special percolation rings (3 os.) have been provided around the first piece of the penstock pipe on the upstream side. Other penstock pipes have been provided with one no. of stiffener ring encircling the same. While erecting the penstock pipes it was ensured that all the longitudinal joints of each penstock pipe are not continuous with those of the adjacent pipes.

3.3.3.7 Erection of Penstock Pipes:

The penstock pipes consisting of 95 segments in all the 5 units, were erected during one working season from November '81 to May '82. The handling was done by means of a 15 Ton derrick crane running on the upstream side of the power dam blocks. The erection proceeded from the downstream side to the upstream side. Each piece was lifted and placed on rails on the downstream face of the partly constructed power dam blocks at a distance of about 20 m. from the axis of the dam. They were then shifted to the exact position by winches. It can be mentioned here that the location and level of each piece was worked out in the drawing and base concreting was done step by step to this elevation. Erection supports to hold each penstock pipe were provided over the concrete pedestals and the penstock pipes were placed over these erection supports by means of a fixed leg gantry specially manufactured for the purpose. The gantry had to be shifted from point to point according to the requirement and the stability was ensured by means of guy ropes. The first step in the erection was to place the chairs with vertical supports and cross bracings suitably, to ensure that the centre of the liners remains on proper line and level. The cradle plates were then welded for sitting of the liner. After lowering of the penstock pipes the proper sitting on the cradle plates and positioning of the same, was ensured as per drawing. A gap of 3 to 4 mm was maintained in between adjacent penstock pipes for final welding. The bent pipes were also positioned like wise. The penstock of each unit consisted of 19 segments. Segments No.1 to 16 were erected from the downstream side, since concreting of the transition zones were not completed in all the units. The piece No.19 was then erected close to the downstream face of the transition (after completion) followed by piece No.18 in the required slope and to the proper line and level. Piece No.17 was an adjustable piece and had to be adjusted as per the actual gap available between piece No.16 and 18. The joints of adjacent liner pieces were matched and proper end preparations were done before final weldings. Welding was completed both on the inside and outside surfaces in the proper sequence. The weldings were checked to remove incuts and defects, if any. The shop painting and other foreign materials of the penstock pipes were removed by sand blasting from the inside surfaces and then two coats of coaltar epoxy paint were applied.

It was extremely difficult to ensure that there are no voids left on the bottom-most part of the penstock pipes during concreting. As such, arrangements were made for grouting around the same. Since stiffeners were provided in each penstock pipes, grouting had to be done separately in between each stiffener. Grouting was also done at the centre line elevation to take care of shrinkage effects of the concrete around the penstock liner as well.

3.3.3.8 Temporary Closure and Sealing of Penstock Gates:

Although all the 95 pieces of penstock pipes were erected in position in one working season, the final welding could not be completed and concreting around the penstock pipes could not be done during that season. No coffer dam was existing on the upstream side of the power dam blocks to prevent flood water from entering into the penstock. During this season a truncated section of the power dam blocks was raised upto El.107.00m. The penstock gates were not fully ready by this time with the wheels, guides and second stage embedded parts. However the gate leaf assembly for all the 5 units was completed with the skin plate, horizontal girders and end vertical girders. It was decided to temporarily install these gates in the gate grooves with 1st stage embedded parts. The gates were lowered by the derrick crane on the upstream side of the dam. During the year a number of blocks of the dam were kept at the ground level and it was estimated that the flood level may rise upto El.105.50m during the rainy season. The water pressure on the gates was thus calculated and temporary supports were provided from the 1st stage embedded parts to the end vertical girders of the gates. Masonry seals were provided on the upstream side. The block out for the sill beam was filled up with masonry and the gate was allowed to sit on the same. A steel joist was provided at the top and masonry constructed over the same to act as the top seal.

This arrangement was suitable to prevent the flow of water, but created enormous dewatering problems in the power house pit the leakage could not be controlled with the available pumps in the project, for which heavy dewatering pumps were borrowed from neighbouring projects to handle such grave situation. Divers were requisitioned and they succeeded in reducing the leakage through the gaps between the temporary masonry seals and concrete entrance surfaces, by application of M-seals and greased gunny bags.

3.3.3.9 Special Features in the Construction of Power dam Blocks:

No coffer dam was constructed on the upstream side of power dam blocks to prevent flood waters entering the area. The power dam blocks were constructed in a truncated manner upto elevation of 90.00m (approximately) by May, 1981 and were left as such. During the subsequent working season (Nov.'81 to May'82) it was necessary to erect the penstock pipes and raise the power dam blocks simultaneously so that the flood water does not damage the penstock pipes during the rainy season of 1982. It was therefore decided to construct the power dam blocks in a truncated manner with a vertical longitudinal joint at 14 m (approximately) down stream of the dam axis. This was an unusual feature and is not known to have been adopted in any other dam. The concreting of the truncated section was carried out from the upstream side by means of derrick cranes. Approach ladders were provided in all the blocks upto the top through which R.R. stones were carried by Jawalies or Sangi mulias. One no. of Builder's Hoist was also provided adjacent to the upstream face of each power dam block. During the peak concreting season. Concrete was transported by trucks in buckets from batching plants and placed in position by derrick cranes concrete mixtures were also positioned nearby for additional production of concrete. The concreting placement was done from the downstream side of the truncated blocks from the unloading platform of the power house. A series of vertical pipe lines with horizontal projection were provided from the old concrete surfaces as the concreting progressed on the downstream side, so that contact grouting can be done from the top at the junction surface of old concrete with new concrete. These pipes were subsequently grouted but they did not take much grout. Besides, adequate number of dowel bars were also provided projecting from the old concrete at 14 m. downstream of the axis.

3.3.4 Special Problems:

It was envisaged that the reservoir will be filled up during the rainy season of 1984 for commissioning of Unit No.1 of the power house. However, Unit No.1 could not be made ready by that time and the secondary concreting of the penstock emergency gates could not be completed since the monsoon broke-in early. The diversion sluices had also not been plugged. When the reservoir was filled up in this

condition it was seen that there was substantial leakage from one of the formed drains in the body of the dam in block No.12 into the foundation gallery. It was difficult to assess the exact upstream source of such leakage. When the reservoir was depleted after November '84 a reverse pressure testing was done from inside the gallery by putting a plug in the concerned drain and applying high water pressure through the same. It was noticed that this water came out on the upstream face of the dam at about El.87.00m. It was then easy to drill a few holes and grout the same at several points. No further problems were encountered later in these blocks.

3.4 Construction of Power House and Tail race Channel:

3.4.1 Introduction:-

The sanction for Rengali Multipurpose Project Stage-I by Planning Commission was accorded on 14th June, 1973 with two Hydro Electric Units of 50 M.W. capacity each, keeping provision for ultimate installation of 4 units. In the year 1978 after discussion with Central Electricity Authority and Planning Commission, Government of Odisha approved the raising of the ultimate capacity from 4 to 5 units with straight exit located in the dam toe below block No.10 to 14 (vide Fig.3.1 & 3.2). Accordingly the orientation of the power house with tailrace was changed from the original provision. The construction programme of power house and power dam was finally decided to commence from December 1980.

3.4.2 Selection of Site:

Rengali Dam is located in a narrow and gorge type section of the river Brahmani. The summer flow sticks to the narrow gorge very close to the left bank. Locating the power house in the deepest portion may warrant protective measures against seepage, silt entry into power house ditch and subsequent removal of silt after each flood thus delaying the work. Again working period in deep channel would be minimum and the excavation work may be delayed considerably. Another disadvantage is the tendency of river to outflank the power house during construction as connection to unloading bay and unloading platform of power house will be vulnerable even during light flood. Hence the idea of locating power house in deep section was abandoned.

The other alternatives are to locate it in any of the flanks. Right flank is situated in advantageous position where the township is planned having administrative offices for dam, colonies, educational institutions, workshop and market etc. But the high cost of excavation for power house as well as the tailrace channel on massive charnockite rock with depth of cutting upto 40m prohibited the construction of power house there. Moreover the present practice is to leave some buffer blocks between the spillway and the power house to nullify the vibration effect of the spillway. Thus there was no other alternative but to set up the power house in the left bank. Decision for exact location of the present power house was finalised taking following factors into consideration.

- i) Low cost of excavation of power dam, penstock trench, Trash rack with cost of concrete for the above structures at different locations.
- ii) Locating the power dam and P.H. beyond summer water level, so that excavation work can start without waiting for the diversion scheme to be completed by providing a ring bund around power dam and power house.
- iii) The P.H. approach to unloading platform and unloading bay should be in firm ground so that approach to power house during rainy season is ensured.

Basing on above, decision was taken to locate the power house at the toe of blocks 10 to 14 of the dam.

3.4.3 Layout:

The P.H. zone is divided in five reference lines longitudinally by A, B, C, D, & E from downstream to upstream respectively (vide Drg. No-3.1 & 3.2). The distance between A to B, B to C, C to D & D to E lines are 10 m, 11.1m, 9.5 m, and 12m respectively, the C line being the centre line of the units. The main components of the power house are the service bay and unit I, II, III, IV & V. The width of the service bay is 32 m placed on the side of the Unit-I. Width of the Unit-I is 26 m and the width of unit V is 23.65 m, width of other three units are 21 m each. The super structure of unit bays and the service bay are limited between D and B lines and thus length available between two walls is 20.6 metres. There is a joint of 25 mm between the dam and the power house along 'D' line.

3.4.4 Planning for Excavation:

Having located power dam from Block 10 to 14 of 105 m length (from RD 186 to RD 291 m) in the left flank of the river and the P.H. in the downstream upto 80 m offset, the diversion scheme was planned to facilitate the P.H. excavation. The non-monsoon flow was envisaged to be diverted through two sluices in Block 21 and the outlet of the diversion channel was to meet beyond P.H. area. The location of downstream cofferdam was so planned at the exit of the diversion channel, as not to allow seepage of water into power house pit. The left abutment of deep gorge, consisting of massive rock served as a natural barrier in between P.H. pit and the flow through the deep gorge during the pre-diversion period.

3.4.5 Excavation:

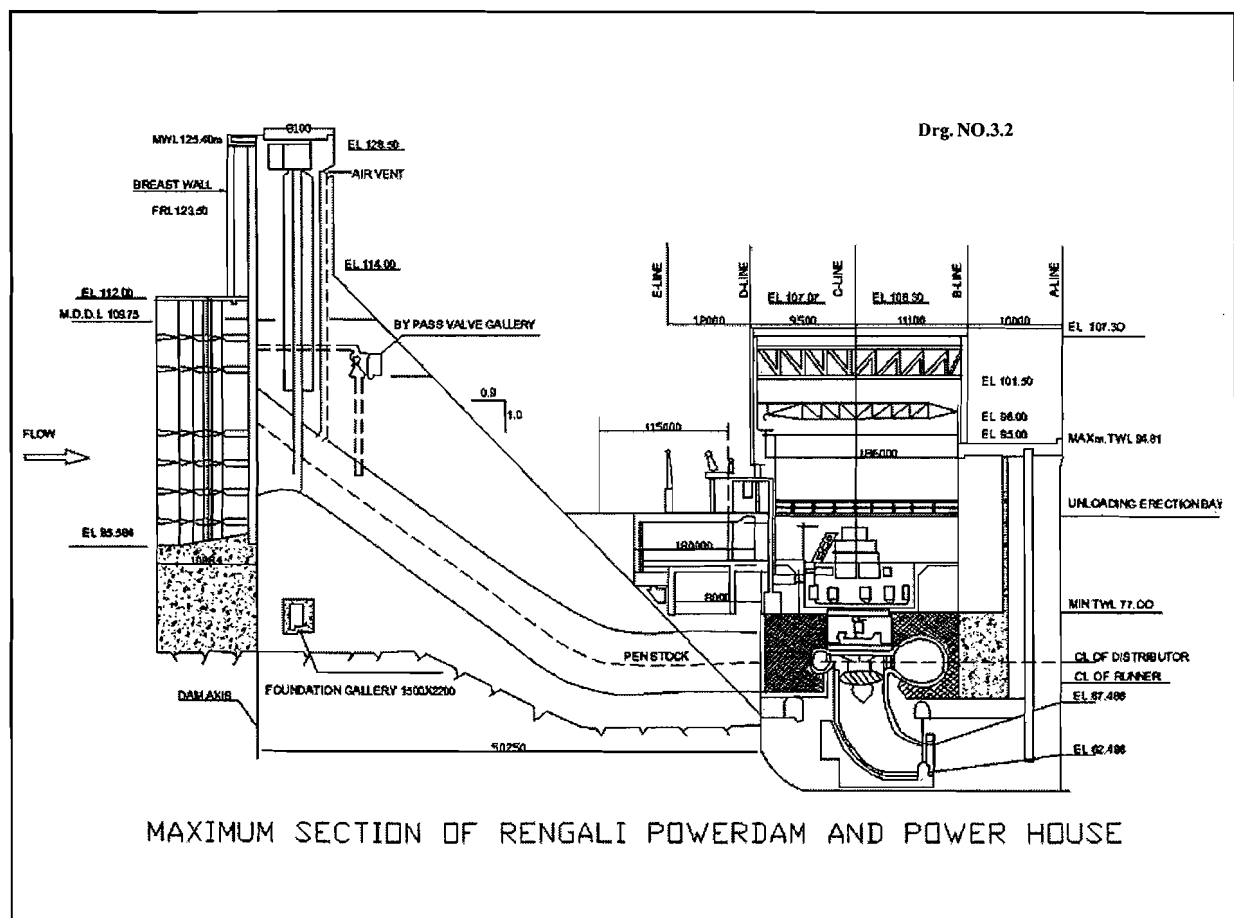
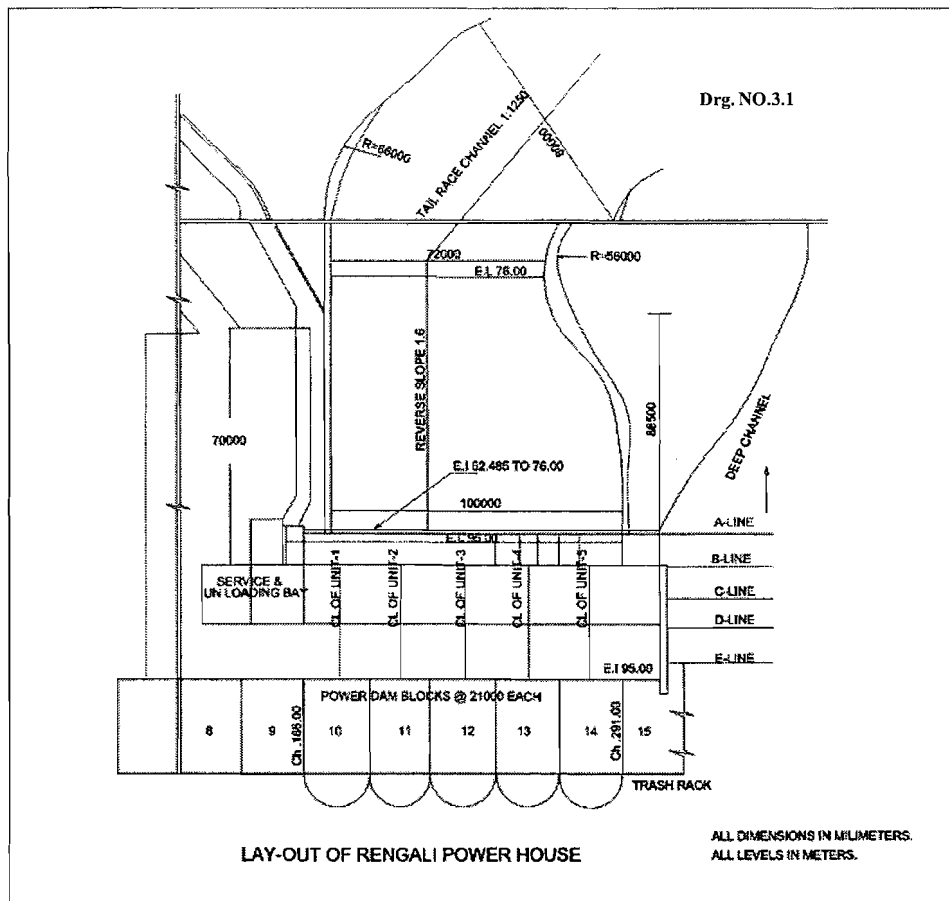
There was no approach to the left bank. One temporary bridge was constructed near village Rengali with crated boulders to serve as piers and wooden decking to make the communication through to the left bank by 15.11.77. Meanwhile one pucca submersible bridge was built across diversion channel about 100 m downstream of the dam toe and opened by 77-78 working season.

The excavation for four units continued in full swing and the excavated materials were transported and stacked in the approved dumping yards situated along the foot of hill but clear of switchyard area. Diesel air compressor was supplemented with 500 C.F.M. electric compressors for drilling and blasting operation. In April'77 there was an untimely flood in the river and some leakage were noticed in the power house pit through the western side loose and joined face of the natural rock barrier. Sand bagging and puddle filling were done to seal the leakage. Due to stagnation of water in the excavated pit the work partially discontinued from 13.4.77 to 16.4.77. The excavated materials of about 2,47,000 cum. were utilised for construction of downstream cofferdam.

The progress of work gained momentum and all-out attention was focussed to finalise programme of phased construction of P.H. Since the power house pit was very nearer to deep channel, every year during monsoon, the pit was getting submerged leaving considerable quantity of flood deposits to be removed in the beginning of the next working season, thereby squeezing the working period of excavation works. The excavation was completed in 80-81 working season.

3.4.6. Foundation Treatment:

Grouted foundation anchors consisting of 32 mm dia, 3.0 m long M.S. rods @ 2000 mm c/c both ways were provided by means of wagon drills. The foundation grade rock level (F.G.R.L.) was kept minimum one metre below the sound rock level in consultation with the resident geologist. Trenches and other treatments for laying ground mat were prepared. Low pressure grouting limited by upheaval conditions was carried out. Heavy grouting was not required as the foundation was sound and free from shear zones. Anchors were grouted and pull out tests were conducted to ensure their efficacy.



3.4.7. Planning for Construction:

Having successfully completed the excavation and foundation treatment of power house, emphasis was given on the construction component as it was scheduled to be completed by 1983-84. As stated earlier, the excavation was completed during 1980-81 working season. The P.H. pit was filled up by silt during flood season. The desilting operation was very costly affair and time consuming and was encroaching upon the limited working period, i.e. from November to May. Hence preventive measures for silting and desilting in 1981 was thought of in order to give a steady and un-interrupted construction of P.H. Achievement of the target needed cordoning off the P.H. on three sides. The bunds could be of semi-permanent type and must be stable enough to withstand the vagaries of flood in monsoon. The 25 m high bund with El.105 m in the upstream of power dam could enhance the submersion problem. The 15 m high bund with El. 94 m in the tailrace side cannot be accommodated in the available space between river and P.H. Hence this proposal was dropped. By putting penstock gates and draft tube gates in upstream and downstream respectively or by arranging for temporary closure of opening otherwise, the P.H. pit can be kept dry in the rainy season for construction works. The second proposal being feasible and non-interfering to the phased evacuation programme was accepted for immediate execution. Then it was planned to raise power dam, draft tube deck wall and the river side wall to required height by June 1981 so that the P.H. pit is free silt from December 1981 for construction of civil works.

3.4.8 Construction of Power House:

By end of 80-81 working season, the base concreting upto RL.59.886 m was laid in all the five units with walls along 'D' line and the block joints one metre above it. The sump well bed and walls at 58.50m was constructed after the ground mat and dewatering header and other pipes and embedments were fixed in position. The foundation excavation of the western protection wall, its treatment and levelling course concrete was laid including providing PVC in horizontal and vertical directions and butting against the power dam constructed upto top of ogee (90.359m El.). All this was done after initial flood deposit removal in tailrace and P.H. pit. Desilting was costly and encroaches into the vital working period to about 1/3rd of the total time. During rainy season, the service bay base concreting columns on 'B' line and 'D' line were completed. Steel shutters were expeditiously manufactured at the central workshop of the project to match with the progress of concreting.

1981-82 Season:

The programme of construction during this season was very important as it envisaged barricading the P.H. pit by power dam in the upstream upto El.107.00m. Draft tube wall in the downstream upto El.95.00m and end protection wall to El.95.00 m in the river side. In view of last year's insignificant physical achievement of P.H. portion, this year's target appeared to be enormous. The detailed planning for approach to work site, placing of machineries in position, removal of flood deposits and procurement of materials were made.

The total quantities of concrete and masonry involved were -

Concrete - 103,800 cum

Masonry - 22,500 cum

About 95,000 cum of flood deposits were also removed from construction area after recession of floods.

For P.H. concreting, two derrick cranes were erected to lift concrete from buckets mounted on trucks or trollies and to place the same over the blocks. The concrete was manufactured in the batching plants. The cranes working in 3 shifts could do concreting of 100 cum each per day. Mixers were positioned for manufacturing concrete and 2 groups were separately engaged for carriage of concrete by head load to specified locations. Mixers were also suitably placed at site to unload concrete to buckets mounted on trolleys and moved manually on rails. Buckets of 2 cum. and one cum capacity

were used for the purpose. As 43000 cum of concreting was to be laid in P.H. structure in 140 working days, the average quantity of concrete per day works out to 300 cum. Unless planning is made to pour atleast 400 cum/day, it may not be possible to achieve the target. It may be mentioned here that, the downstream wall of the P.H. house consisted of piers and abutments with clear gap of 25mm in between each unit where adequate water proofing arrangements were made. The intervening wall was only 1.5m thick (R.C.C.) with horizontal reinforcement of 32 mm dia rods at 150 mm c/c both on upstream and downstream sides and vertical reinforcements of 20mm dia rods at 150 c/c both on the upstream and downstream sides and vertical reinforcements of 20mm dia rods at 150mm c/c both on u/s and d/s sides. In addition to this, the tie bars placed for supporting the shutters made the area extremely difficult for concreting. Due to the heavy schedule of concreting and on account of massive reinforcements concreting in the walls was done with lift height of 2 M against normal lift height of 1.5m. By putting men, materials and machineries round the clock, the gigantic task of building the end protection wall upto El.95.00m and draft tube wall upto 92.5m El. as against El.95m was achieved. All the ten draft tube gates were fitted with the gate leaves without the sealing arrangements. Temporary sealing of bottom sides and top was done by brick masonry 1:3 plastered, on the faces exposed to water. Similar action to seal the penstock gates temporarily was taken not to allow water from upstream side to power house pit.

1982-83 season:

In 1982-83 working season, unit 1 to 5 draft tube gallery, cross galley, partitions of units and concreting upto knee-liner top were done. Draft tube deck was completed. The eastern protection wall was raised upto El.95.00 m and the road was made through over it providing approach to draft deck from which concrete was carried by chute to different blocks as per requirement. Roof over Service Bay and Unit 1 & 2 were completed.

3.4.9 Problems due to Temporary sealing of Gates with Masonry:

During the construction period 81-82, it was proposed to encircle the P.H. area for protection against flood water by raising the power dam to El.107.00m and the draft tube wall to El.95.00m. The penstock gate leaves were lowered without rubber seals. Wheels and second stage embedded parts were not fixed. The gates were sealed with the help of temporary brick masonry walls on the upstream side. Similarly the draft tube wall could be raised upto 92.5 m (Av.) and all the ten draft tube gates were lowered and temporarily sealed. During the monsoon when water started impounding on both the upstream and downstream sides of P.H., these temporary seals did not prove useful. The dewatering operation was abandoned due to submergence of two nos. of 100 H.P. pumps installed in the unit 1 area of the P.H., due to profuse leakage from the draft tube gate seals.

During 1982-83 the power dam was raised up to T.B.L. i.e. to EL. 128.50m. Penstock gates of unit I & II were lowered with proper sealing arrangements. Temporary sealing arrangements for other three penstock gates were done with additional stone masonry. The draft tube deck was completed. All the ten draft tube gates were lowered with proper sealing arrangements.

During August, 1983, when water level in the reservoir reached above El.104.00m, heavy leakage were noticed from the penstock gates of units 3,4 & 5. About 10 pumps of 100 H.P. and 75 H.P. capacities were borrowed from other projects and installed. Even then, there were moments of extreme strain. Loading of penstock gates with sand bags was attempted from the top, but to no avail. Divers were then brought from Calcutta and it was seen that water was leaking through the hair breadth gaps between the temporary masonry seals and old concrete or steel adjoining surfaces. M-seal was used to seal these gaps and then the leakage reduced considerably. Gunny bags and used ropes covered with grease were also used at joints. A mixture of mud, cement and grease was also successfully applied at several spots. Thus it was observed that brick or stone masonry is not suitable as temporary seal for gates to create dry working areas behind them.

1983-84 season:

In 1983-84 season, concreting around scroll case of, Unit-1 and the generator floor upto El.83.325m was completed. The auxiliary rooms like conference room, carrier room, L.T. room, control room, D.C. room, Battery room and the cable trench upto service bay roof were completed. The lift wall from El.65 m to El 87.00 m was completed. The allied structures like unloading bay side slopes,

gauge well, cable trench to switch yard resettling tank, over head tank for fire fighting in P.H., canal from block 5, etc. progressed satisfactorily. The balance 'B' line columns and walls over crane beam were taken up. The tailrace base concreting (about 65%) was done including the secondary concreting for sill beams, tracks and guides of draft tube gates. The gates with seals all-round were lowered to position before on set of monsoon. Hence there was virtually no problem of dewatering during the rainy season. The auxiliary room partitioning work was taken up and completed quickly.

3.4.9.1 Special attempts for Concreting around Scroll case:

As per approved drawings, the concrete below scroll case had to be left at El.68.50m just flushing with the top of the cone. The scroll case, after being erected in the position and welded joints tested by X-ray, concreting was taken up. The area below and behind the scroll case being inaccessible, needed special efforts to place concrete. Central Water Commission advised that 200mm diameter pipes be provided bent to proper shape, for putting concrete behind the scroll case around the runner chamber. Pneumatic concrete placers were used for this concreting with good results. To ensure smooth flow of concrete superplasticizers were used. Concrete with 20mm down aggregate was used with 7 bags of cement (M:20) per cum admixed with Lomar 'D' @ 0.5% i.e. 1.75kg/cum. of concrete to delay the setting time and maintain consistency of concrete. The ball and socket type joints of outlet pipe enabled to deliver the mix through suitable bends at the predetermined area ready to receive concrete. Due to tremendous impact of pneumatic placer concrete, vibration and consolidation was not required. Discontinuity of such type of concrete placing arising out of unavoidable circumstances and to take care of shrinkage of concrete, grout holes in doubtful areas were left and grouted to ensure homogeneity of the concrete mass.

3.4.9.2 Clearance of debris from P.H. pit and further Construction of Units:

Debris are easily deposited in an enclosed pit during construction. The help of tower crane stationed in unit-5 draft tube deck was taken to dispose off debris to some extent. Direct disposal by head load in unloading bay by manual labour was also adhered to.

Tailrace:

Initially the tailrace channel was for four units. The width of tailwater channel was 54m. From stage discharge curve at dam site as per Concrete & Masonry Design Dte (CMDD). Drawing No.3004 the following water levels in the river corresponding to the discharges have been co-related in finalising the tailwater levels.

Discharge (cumec)	Water level in the River (m)	Tailwater level (m)
14.8	-	77.00
148	77.50	78.20
296	78.25	79.15
444	79.00	80.10
592	79.50	80.75
Maximum flood level (m)	93.40	The crest level has to be kept at 76.8 m

As per Central Electricity Authority (C.E.A.), minimum tailwater level was to be fixed at El.77.00m by taking 10% of one unit full load discharge i.e. 14.8 cumec, so that more head loss affecting the power generation can be avoided.

Finally the crest level for tailwater level was fixed at El. 76.00m. The width at exit channel raised to 72.00m instead of 54.00m. as five units were decided to be installed . A reverse slope of 1:6 from 'A' line to the tailwater exit wall over a horizontal length of 86.5 m was provided. The balance portion of tailrace was excavated in a slope of 1:1250 over a width of 81.00 m while meeting the river alongwith a natural stream approaching from the east.

3.4.10 Service Bay:

The main entrance to the P.H. with necessary roofing and the E.O.T. crane facilities is the Service Bay. The floor level of the service bay is at El.87.00m where unloading erection and maintenance of different components of turbines and generators are done.

Unit Bays:

There are five unit bays for installation of 5 generating units of 50 M.W. each. In the P.H. there are two floors at RL.76.621m and RL.82,325m which are known as turbine floor and generator floor respectively. The substructure below the turbine floor is extended upto RL.58.00m. The Draft tube, the runner chamber, the spiral case and the other components of the turbine are embedded in concrete. In between the turbine and generating floors the generator barrel encircling the generator is located. The super structure above generator floor with columns, beams and walls is constructed on both B & D lines to protect P.H. and to support the roof trusses, with pre-cast roof slabs. Two nos. of crane beams for movement of E.O.T. cranes run parallel on columns of B & D lines from service bay to end of unit V.

Draft Tube Deck:

In between B & A line 10m wide deck slab supported by piers in each unit is provided at RL.95.15m. This deck slab has 10 openings for draft tube gates with travelling gantry facilities.

Unit Auxiliary:

In between D & E line the space is utilised to accommodate the unit auxiliary rooms. It has also two floors namely the machine floor at RL.76.62m and auxiliary rooms for unit control and accessories at RL.82.325m with massive roof slab over it at RL.87.00m to take the load of transformers even in case of fire. The roof slab is provided with large pits over it to facilitate drainage of oil and water from the transformer deck to a suitable point. At both ends of the units, auxiliary stair cases are provided. The stair case on the right of the auxiliary rooms is provided with lift facilities.

Auxiliary Rooms around Service Bay:

There are auxiliary rooms for accommodating air conditioning plant, ventilation-plant, oil room, workshop, office etc. on back and right side of the service bay in two floors i.e. at RL.87m & 92m with roof at RL.97.0 cm.

Transformer Deck:

A space in between 'E' line and toe of the dam along with the roof over unit auxiliaries at RL.87.0m connecting to the unloading platform is known as transformer deck. This is provided to facilitate movement of the transformers on rails embedded in the floor. Drainage pipes, drains are also provided in this area.

Unloading Platform:

A large space in front and around service bay has been left at RL.87.0m with drainage facilities for movement of heavy vehicles and machineries required for P.H. This area is protected from flooding by tailrace channel and river by a masonry wall (i.e. left training wall) with top level at El.95.15m. This wall is extended from 'A' line of the P.H. to the junction of approach road to P.H. from the left bank of river.

Western side end Protection wall:

The left training wall of the spillway provided in front of the block No.19 of the dam is about 3 blocks away from the P.H. In order to protect the P.H. from the flood discharge of the spillway, a massive masonry wall is provided on the right side of P.H. in between RD.299.515m and 319.00m. Some portion of the foundation of this wall is in the deepest channel of the river. The deepest foundation levels at El.51.75m. The space available in the wall above RL.82.00m is quite considerable. This space is well utilised by providing rooms in three stairs to accommodate further requirements of the power house.

3.4.11 Water Conductor System:

a) Intake Structure and Penstock:

Five steel lined penstocks of 5960mm inner dia, embedded in the body of the power dam blocks, have been connected to the rectangular opening at the upstream face of the Dam through smooth transitions. The bell mouth entry connecting to the rectangular openings has been designed and provided as per standard curves confirming to U.S.B.R. Specifications. This bell mouth intake has been protected by the trash rack-structure, consisting of steel rack with clear opening of 68mm.

b) Scroll Case:

The horizontal end of the penstock meets the spiral casing at a distance of 50.25m from axis of the Dam. Diameter of the spiral casing at its inlet is 5960mm. This guides the water in a circular path into the runner chamber through the stay vanes. Distance of spiral casing inlet from the centre of the turbine is 6315mm. The diameter of this spiral casing is reduced to zero at its end. This is made up of bend segments from mild steel plates and welded to the stay ring. The outer and inner diameter of the stay ring are 6950mm and 6000mm respectively. There are 12 openings in the stay ring, height of opening being 1570mm. The scroll case rests directly over concrete at the bottom, but is separated at the top from the surrounding concrete through a 25mm thick flexible cover.

c) Runner Chamber:

This consists of a circular vertical chamber made up of steel and stainless steel lining and is connected to the stay ring and draft tube cone. The water from the spiral casing enters through the guide vane openings to this chamber for rotating the runner and ultimately discharge into the draft tube vertically down.

d) Draft Tube:

It is a system to convey the water from the runner chamber after doing work of rotating the turbine shaft, to the river through the tailrace channel. The 1st part of this tube consists of steel liner having a circular inlet connected to the runner chamber and a vertical elbow bend ending with rectangular opening at the end. There after it is divided into two vents with a single central pier. The floor and the top of these vents have an upward slope to meet the tailrace.

e) Tailrace Channel:

First part of this channel is a concrete lined still pool, starting from the end of draft tube opening with upward slope of 1 in 6, negotiating the level from RL.62.488m to RL.76.00m at a distance of 86.00m from the end of power house. The width of this still pool is 100m. near the P.H. and 72.00m at its end where it meets the weir at RL 76.00 m. The 2nd part of tailrace is a 72.00m wide channel with a downward slope of 1 in 1250, straight for a distance of 65 metres from the weir. It finally meets the river with a bend of 128° in a short distance. This portion of the channel is not lined.

3.5 Cost Estimate:

Rengali Multipurpose Project (Stage-I) sanctioned by Planning Commission in their letter No.II-2(64)/72-AE I dated:14.6.73 provided, Rs.4192 lakhs for Power Plant for installation of 2 hydro-electric units of 50 M.W. each with provision for installation of 3 more units later on.

The first revised estimate for civil works was framed for Rs.1647.64 lakhs during later part of 1980, basing on the drawings then available for P.H. building, auxiliary structures to accommodate 5 units, including fixation of trash rack, penstock gates, penstock emergency gates with Gantry Crane, Penstock liner, Draft tube gates, D/T. gantry crane and tailrace-channel as well as end protection wall for all 5 units. Concreting upto draft tube knee liner level was provided for the 3 units in 2nd stage and entire concreting was envisaged for units I and II only.

2nd Revised Estimate (for Civil Works) was prepared during later part of 1984. This estimate was necessary due to increase in prices of essential raw materials and basic wages and included actual cost of execution upto 30-6-84 and envisaged remaining works to be done at analysed rates. Table No-3.10 shows the rise in cost of materials and labour between 1972 to 1987-88.

Table-3.10 Rise in rate of Materials and Labour

Sl No.	Items	Rate in 1972 (Rs.)	Rate in 1980 (Rs.)	Rate in 1983 (Rs.)	Rate in 1984 (Rs.)	Rate in 1986-87 (Rs.)	Rate in 1987-88 (Rs.)	% of increase between 1972 to 88
1	Cement per Ton	260.0	520.0	720.0	854.47	1033.0	1373.0	428.08
2	Diesel per Lit.	1.30	2.35	3.31	3.50	3.85	5.10	292.31
3	Explosive per Ton	8000.0	20,000.0	20000.0	20000.0	24000.0	27000.0	237.50
4	Steel per Ton	1520.0	3500.0	5500.0	5800.0	5984.0	6900.0	353.95
5	Labour per day	3.50	5.00	6.25	6.25	10.00	10.00	185.71

2nd Revised Estimate finally stood at Rs.24,18,50,000/- after discussion with Central Electricity Authority (C.E.A.). Although all the major works were completed by the end of 1985, finishing and painting works, approach roads, security rooms and canteen, doors and window etc. remained to be done, besides some gates and gantry.

3rd Revised Estimate was prepared taking actual expenditure upto 31-10-1986 into consideration (which was Rs.22,95,88,223/-) and the estimate amounted to Rs.26.63 crores. Rengali Power House Stage-I works were practically completed and closed by the end of March '89.

3.6 Problems Encountered:

A) Problems arising due to temporary sealing of gates with masonry:- This has been discussed earlier vide Sec-3.4.9

B) Problems due to lesser thickness of Draft tube walls:

The concreting of the draft tube wall was done in lifts of 2m. each. The thickness of the draft tube wall as per design is 1.5 metres. During 1983 monsoon, leakage was noticed through the joints in between the layers of concrete. Attempts were made to seal the leakage with the help of M-seal by the divers, but the effect could not be felt by them on the tail channel side. During the next working season a number of holes were drilled along the joints from the tailrace side and cement grout was injected by grout pumps. The leakage reduced in the next monsoon. After successive groutings, the leakage has been controlled, but wet patches were seen at several areas of the wall. As advised by the Central Water Commission, grouting of the wall with a product called "Aqualgel-9" (manufactured by M/s Asian Laboratories, New Delhi) has been done during August '89 on wall from inside of power house. This appears to be effective, since leakage has considerably reduced at several places.

C) Seepage into Auxiliary rooms:

There is a clean joint in between the dam toe and unit auxiliary room along the 'E' line. The walls of the auxiliary rooms below the transformer deck remained wet in several patches. The wet patches increased with the rise of water level in the reservoir. Quite a number of holes were drilled from the transformer deck along 'E' line with an inclination, so as to intercept the joints. Holes were also drilled on the wall penetrating the joint from inside the auxiliary rooms. Cement grout was injected through

these holes. The seepage was controlled to some extent but dampness in the walls still persisted at some places. Subsequently epoxy grouting was done in these areas to make the walls completely dry. Some arbitrary moist patches were noticed at times, which are promptly grouted with epoxy.

D) Leakage through Expansion Joint:

There is an expansion joint along 'D' line in order to separate unit auxiliary rooms from the P.H. to avoid transmission of vibration. Similarly there are expansion joints perpendicular to 'D' line upto 'E' line in between the units. Treatment of these joints was done as per C.W.C. drawings providing asphalt filler boards, P.V.C. water stops etc. But during the rainy season water path through these joints and leakage was noticed along 'D' line. These joints were sealed by pouring hot bituminous compound (shalijet) and covering with cement mortar bunds. The leakage is under control.

E) Silting of Tail Channel:

As stated earlier the tail race channel is not fully lined. The space in between the tailrace still pool and left training wall of spillway is hardly 55m. There is no protective bund along the right bank of the spillway channel. During monsoon when water is allowed over the spillway it entered the tail channel near the unprotected area in front of the end protection wall. This water carries a lot of muck and silt and deposited the same in the tailrace channel. In this process the mouth of the tail race channel beyond the weir got constricted causing afflux and consequent loss of power. Measures were taken to protect silting of tail race channel by providing protection bund in the right side of tailrace channel. The protection bund has been partly damaged.

3.7 Power House Auxiliaries

Introduction:

The auxiliary structures which are intended to be discussed here are the details of drainage of storm water in the unloading bay and adjoining areas, sewerage disposal from P.H., system for drinking water supply to left bank colony and fire fighting etc. in P.H. Depending on the site conditions and the specific requirements of Rengali power house several alternatives in the above auxiliary systems were studied, designed and the final system adopted have been briefed.

1) Storm Water Drainage Disposal:

The service bay (El.87.0m) and unloading platform area of Rengali P.H. is surrounded by ground with higher contours. The disposal of storm water from the service bay will not pose a problem as long as tail water level is below El.86.00, but at higher tail water levels, the entire quantity of storm water draining into the unloading platform has to be pumped out to avoid flooding of the P.H. Since the storage space for flood water inside the service bay and unloading platform area is likely to be small, it is imperative that adequate pumping capacity be provided for removal of the flood water. It is also necessary to provide for alternative source of energy for the pumps to take care of possible power failure during heavy storm.

The above circumstances call for interception of the storm water draining from the hill on the left flank of the dam at higher contours and diverting the same away from the service bay.

Storm water run off from the area has been computed as 0.86 cumec (30.38. cusec). It is desirable to provide storage capacity in the unloading platform area to contain the storm water discharge for 15 minutes during which there may be power failure.

Storage capacity required to contain the storm water works out to $15 \times 60 \times 0.86 = 774 \text{ cum}$ against which storage provided at 4 locations for 1193 cum, the details of which are as follows:

i) Sump well by the side of masonry wall from draft tube deck (10mx3mx4m)	- 120 cum
ii) The drains will act as storage	- 140 cum
iii) Eastern side of track	- 33 cum
iv) Southern side of track	- 900 cum

Total = 1193 cum

3.7.1 Sewerage disposal for Rengali Dam Power House:

Regarding public utilities to the workers and officers working insider the power house the following facilities i.e. water closets, urinals, wash basins and drinking water points have been provided. The sewerage disposal has been made suitably taking care of all surrounding structures and equipments into account. These are detailed below:

Floor El.(m)	Location	W/C (No)	Urinal (No)	Drinking Water Points (No)
76.621	Lift Well	1	1	1
82.325	Lift Well	2	2	1
Conference room		1	1	1
87.00	Lift Well	-	-	1
Near work-shop		2	2	1
92.00	-do-	2	2	1

Since bottom-most floor where water closet has been provided is at El.76.621m a sump chamber has been accommodated below El.76.00m from where sludge well be pumped out regularly to a septic tank located in unloading platform area at El.87.00m. The sewer line connected to different floors can be taken straight to sump and will run along 'E' line.

the size of the sump in the lift well portion is decided taking total no. of users as 70 and 7 days capacity where as the septic tank is designed taken both the zones (i.e. lift well zone and service bay zone) into consideration. The septic tank is designed for two years capacity for 100 users.

3.7.1.1 Water Supply for Fire Fighting in the P.H. and to the Residential colony on the Left side of River Brahmani.

Introduction:

The scheme envisages to draw water from the reservoir with a view to meet the requirements of fire fighting for the power house and for drinking water purposes for the residential colony on the left bank of river Brahmani. This would also supply water for drinking purposes and other miscellaneous uses to the P.H. area.

Water is drawn from reservoir through a sluice in Block No.5 of the dam. The sill level has been fixed at El.109.00m (DSL). From the sluice, a channel is connected to a settling tank at a distance of about 120m. from the axis of the dam. From the settling tank, water is drawn separately and stored in a high level tank at El.175.00m for fire fighting in the P.H. and for connection to the clear water reservoir at El.124.000m for the colony water supply. Separate pumps have been installed after the settling tank for the above two purposes. The pressure filter is located near the settling tank in the main line connecting to the clear water reservoir; branch connection has been provided from the main line after the pressure filter, for supply of drinking water, to the P.H. through a tank, located at El.113.00m in the hill slopes by the side of the unloading bay area. Incase, it so happens in exceptionally dry years that, the water level in the reservoir goes down below the sill level of the sluice, pumps are provided to lift the water from the reservoir to the settling tank, for the period the water level remains low.

3.7.1.2 High level tank for Fire fighting in the Power House:

The water supply needs for fire fighting per transformer is around 1000 gallons per minute. Normally 14 minutes is sufficient for quenching the fire. But it is preferable to have a clear margin. Thus assuming 25 minutes supply and taking into account the water needs of other fire hydrants, it was recommended that about 27,000 gallons (122.31 cum) storage is necessary.

$$\text{Capacity of tank} = 27000 \times 4.53 = 1,22,310 \text{ litres or } 122.31 \text{ cum}$$

Assuming depth of water 3m in the tank a size of 6.5m x 6.5m may be provided thereby making a total capacity of 126.75cum. Free board of 0.5m shall be provided over water surface of the tank.

Size of tank provided = 6.5m x 6.5m x 3.5m = 147.875 cum

This tank is located at about El.175.00m in the left hill of the dam at a distance of 125.75 metres from the beginning of block No.1 Outlet pipe of 300mm dia and inlet pipe of 200mm dia, have been provided in the tank.

3.7.1.3 Outlet pipe from the High level tank for Fire fighting:

Purpose: Rate of discharge is 1000 gallons per minute or 75.5 ltrs per second or 0.0755 cumec. Length of pipe from the settling tank to power house area is approximately 358m. (taking into consideration the bends and slopes).

It is desired that, the water supply for multifire type fire protection of the P.H. require a pressure of about 7.5 kg per sq.cm. at the transformer level (El.87.00m). This requires headloss in the pipe to be limited to about 13M (175-(87+75)).

Head loss incase of 200mm pipe will give much more. Thus 300mm dia pipe is provided for the total system.

Area of 300 mm dia pipe in 0.07 sq.m.

Velocity : $\frac{0.0755}{0.07} = 1.08$ m. per second

Friction loss : $\frac{4.f.l.v^2}{2.d.g.} = \frac{4 \times 0.01 \times 358 \times 1.17}{2 \times 0.3 \times 9.81} = 2.63$ m

Hence 300m dia outlet pipe will be adequate. The main control for the pipe will be provided at the P.H. end, from which water will be supplied to various fire hydrants for fire fighting purposes.

3.7.1.4 Inlet pipe to the High level tank:

Assuming time of filling of the tank to be 60 mins.

Rate of discharge = $\frac{126.75}{60 \times 60} = 0.035$ cumec.

assuming 200 mm dia pipe, area = 0.0314 sq.m.

Velocity in pipe = $\frac{0.035}{0.0314} = 1.1$ m, per second.

Friction loss = $\frac{4.0 \times 0.01 \times 358 \times 1.2}{2 \times 0.2 \times 9.81} = 4.85$ m.

Gross head required for the pump = (175.0 + 3.5 + 4.85) - 105.00 = 78 m.

(The bottom of the settling tank is fixed at El.105.00)

Hence the pump for raising the water to the high level tank from the settling tank is to be designed for a gross head of about 78 m and rate of discharge of 35 litres per second.

3.8 Canal Distribution Network:

3.8.1 Introduction:

“Irrigation is the art and science of application of water artificially, for raising of crops. It is needed for the lands which are deprived of receiving adequate and timely supplies of water from natural resources for successful growth of crops. From time immemorial, irrigation is being practised in one form or other in the arid and semi-arid parts of the world; almost simultaneously with the first attempts of man to grow crops. Application of irrigation water has undergone sea-change during last 100 to 150 years due to rapid advances in the field of science and technology. The works of ancient days were limited in their scope and extent. Modern irrigation projects are gigantic, transcend river basins and envisage to train and harness the mighty rivers for an integrated, comprehensive and optimum exploitation of their potential for multi-sectorial uses, i.e., irrigation, hydropower, flood control, navigation, pisciculture and recreation etc.” (Source: History of Irrigation Development in Orissa by G.C. Sahu, INCID, MoWR, Government of India, November 2009-Pg.1)

Soon after independence, the first and foremost problem the country faced was to eradicate poverty and to feed hungry millions by means of a planned socio-economic development. Even during 1936, Indian National Congress emphasised on agricultural programme for economic development which resulted in setting up National Planning Committee in 1938. Planning Commission established in 1950 recognised the importance of exploiting irrigation potential of the country for increasing the food grain production. The plan-wise expenditure on Irrigation and Flood Control (F.C.) sectors is furnished in Table-3.11

Table : 3.11 Plan-wise Expenditure on Irrigation & Flood Control Sectors

(Rs in crore)

Sl. No.	Plan Period	Major & Medium Irrigation	Minor Irrigation & CAD	Total Irrigation	Flood Control	Total Expenditure All Sectors	Percentage expenditure on irrigation
1	First (1951-56)	376.2	65.6	441.8	13.2	1960	22.54
2	Second (1956-61)	380.0	161.6	541.6	48.1	4672	11.59
3	Third (1961-66)	576.0	443.1	1019.1	82.1	8577	11.89
4	Annual (1966-69)	429.8	560.9	990.7	42	6625	15.04
5	Fourth (1974-78)	1242.3	1173.4	2415.0	162	15779	15.31
6	Fifth (1974-78)	2516.2	1409.6	3925.8	298.6	28653	14.22
7	Annual (1979-80)	2078.6	1344.9	3423.5	330	22950	14.27
8	Sixth (1980-85)	7368.8	4159.9	11528.7	78.7	109292	10.55
9	Seventh (1985-89)	11107.3	7626.8	18734.1	941.6	218730	8.56
10	Annual (1990-92)	5459.2	3649.5	9108.7	460.6	123120	7.4
11	Eighth (1992-97)	21071.9	13885.3	34957.2	1691.7	483060	7.59
12	IX Plan (1997-02)	49289.0	1376.0	63049.0	303.8	941041	6.7
13	X Plan (2002-07)	71213.0	24521.4	95734.4	596.5	1525639	6.28

Source: Report of the Working Group on Water Resources for XI plan (2007-2012)- “Water Resources Development in India” INCID, Dec.2009 Pg.101.

Although plan expenditure on irrigation increased from Rs.441 crores in 1st Five Year Plan (FYP) to Rs.95734 crores in the 10th FYP, the share in plan expenditure of the country decreased from 23% in 1st FYP to 6.3% in 10th FYP. The trend is shown in Table-3.11.

Plan-wise position of Irrigation potential created and utilised is presented in Table-3.12

Table-3.12 Plan-wise Position of Irrigation Potential Created and Utilized (Mha)

Plan		Potential Created (Mha)					Potential utilized (Mha)				
		Major & Medium	Minor			Total	Major & Medium	Minor			Total
			S.W.	G.W.	Total			S.W.	G.W.	Total	
Upto 1951 (Pre-plan)	Cumulative	9.70	6.40	6.50	12.90	22.60	9.70	6.40	6.50	12.90	22.60
I Plan (1951-1956)	During	2.50	0.03	1.13	1.16	3.66	1.28	0.03	1.13	1.16	2.44
	Cumulative	12.20	6.43	7.63	14.06	26.26	10.98	6.43	7.63	14.06	25.04
II Plan (1956-1961)	During	2.13	0.02	0.67	0.69	2.82	2.07	0.02	0.67	0.69	0.76
	Cumulative	14.33	6.45	8.30	14.75	29.08	13.05	6.45	8.30	14.75	27.80
III Plan (1961-1966)	During	2.24	0.03	2.22	2.25	4.49	2.12	0.03	2.22	2.25	4.37
	Cumulative	16.57	6.48	10.52	17.00	33.57	15.17	6.48	10.52	17.00	32.17
Annual Plans (1966-1969)	During	1.53	0.02	1.98	2.00	3.53	1.58	0.02	1.98	2.00	3.58
	Cumulative	18.10	6.50	12.50	19.00	37.10	16.75	6.50	12.50	19.00	35.75
IV Plan (1969-1974)	During	2.60	0.50	4.00	4.50	7.10	1.64	0.50	4.00	4.50	6.14
	Cumulative	20.70	7.00	16.50	23.50	44.20	18.39	7.00	16.50	23.50	41.89
V Plan (1974-1978)	During	4.02	0.50	3.30	3.80	7.82	2.70	0.50	3.30	3.80	6.50
	Cumulative	24.72	7.50	19.80	27.30	52.02	21.16	7.50	19.80	27.30	48.46
Annual Plan (1978-1980)	During	1.89	0.50	2.20	2.70	1.59	1.48	0.50	2.20	2.70	4.18
	Cumulative	26.61	8.00	22.00	30.00	56.61	22.64	8.00	22.00	30.00	52.64
VI Plan (1980-1985)	During	1.09	1.70	5.82	7.52	8.61	0.93	0.01	4.24	5.25	6.18
	Cumulative	27.70	9.70	27.82	37.52	65.22	23.57	9.01	26.24	35.25	58.82
VII Plan (1985-1990)	During	2.22	1.29	7.80	9.09	11.31	1.9	0.96	6.91	7.87	9.77
	Cumulative	29.92	10.90	35.62	46.52	76.44	25.47	9.97	33.15	43.12	68.59
Annual Plan (1990-1992)	During	0.82	0.47	3.27	3.74	4.56	0.85	0.32	3.10	3.42	4.27
	Cumulative	30.74	11.46	38.89	50.35	81.09	26.31	10.29	36.25	46.54	72.85
VIII Plan (1992-1997)	During	2.21	1.05	1.91	2.96	5.17	2.13	0.78	1.45	2.23	4.36
	Cumulative	32.95	12.51	40.80	53.31	86.26	28.44	11.07	37.7	48.77	77.21
IX Plan (1997-2002)	During	4.10	1.09	2.50	3.59	7.69	2.57	0.37	0.85	1.22	3.79
	Cumulative	37.05	13.60	43.30	56.90	93.95	31.01	11.44	38.55	49.99	81.00
X Plan (2002-2007)	During	5.30	0.71	2.81	3.52	8.82	3.41	0.56	2.26	2.82	6.23
	Cumulative	42.35	14.31	46.11	60.42	102.77	34.42	12.00	40.81	52.81	87.23

Source : Working Group Report XI Five Year Plan – “Water Resources Development in India” INCID, Dec. 2009, Pg. 100-101.

India occupies 3.29 million sq.km geographical area which is 2.4% of world's land area; but it supports over 15% of world's population. The renewable fresh water resources of country is 1869 billion cum (as per CWC estimate in 1993) which is 4% of earth's water resources. Average Indian has hardly 1/6th of land and 1/4th of water as compared to world average. Against above, Odisha's land mass is 1,55,707 sq.km. which is about 4.7% of country's land mass but is endowed with about 11% of water resources.

It has been assessed that total irrigation potential in the state is 59 lakh ha., out of which 39.49 lakh ha. can be brought under major and medium irrigation projects and 9.70 lakh ha. through minor irrigation (flow). Irrigation is not only considered as one of the most important infrastructures. but also it is a critical and vital input required for increasing agricultural production as irrigation facility enables the farmers to use other yield enhancing inputs like HYV seeds and chemical fertiliser etc. Irrigation facility in Odisha is relatively less when compared to many other states of the country even though abundant water resources are available. Table 3.13 shows the net area irrigated in the state and Table-3.14 gives the details of crop-wise irrigated area.

“Inadequate erratic irrigation facilities still remain a major constraint for improving agriculture and agricultural productivity. Irrigation intensity in the state was only 31% in 2006-07 in comparison to all India average of 44%. However, the situation in this regard has been gradually improving. With a view to assuring more irrigation facilities as quickly as possible, the state Government launched in 2009-10 two innovative irrigation schemes, i.e. (i) construction of check dams and (ii) sustainable harvesting of ground water through installation of bore wells at massive scales.” (Source: Economic Survey 2012-13, Government of Odisha, Pg.3)

It has been stated earlier that the water will be carried by two contour canals taking off from the Barrage at Samal i.e. (i) Left Main Canal (LMC) and (ii) Right Main Canal (RMC) through distribution net work to command total ayacut of 2, 18, 392 ha. covering 24 blocks in 8 districts. Total irrigation potential created through both LMC and RMC upto June 2013 is 47206 ha (i.e. 21.6%)

Schematic diagram of the LMC and RMC is presented in Drg. No-3.3 to 3.4 showing distributaries and branch canals. Photographs of few structures of distribution system of RIP are enclosed.

Table-3.13 Net Area Irrigated in Odisha (in 000' ha)

Year	Net Area irrigated							Total
	Canals			Tanks	Wells		Other Sources	
	Govt.	Pvt.	Total		Tubewells	Other Wells		
1	2	3	4	5	6	7	8	9
1995-96	949	-	949	305	299	537	-	2090
1996-97	950	-	950	105	76	64	194	1366
1997-98	950	-	950	108	78	66	199	1401
1998-99	939	-	939	107	77	65	196	1384
1999-00	943	-	943	107	78	65	197	1390
2000-01	905	-	905	103	75	63	189	1334
2001-02	-	-	-	-	-	-	1752	1752
2002-03	-	-	-	-	-	-	1247	1247
2003-04	-	-	-	-	-	-	1737	1737
2004-05	-	-	-	-	-	-	1846	1846
2005-06	1370	-	1370	-	274	90	296	2030
2006-07	1370	-	1370	-	277	90	314	2051
2007-08	1420	-	1420	-	306	101	331	2158

Source : Economic Survey, Odisha, 2010-11, Planning & Co-ordination Department, Pg. 107, Annex. 3.14.

Tabl-3.14 Crop-wise Area Irrigated in Odisha (in 000' ha)

Year	Rice	Total Cereals	Total Foodgrains	Total Food Crops	Total Non-food crops	Sugarcane	Spices	Fruits and Vegetables	Total irrigated area under all Crop
1	2	3	4	5	6	7	8	9	10
1995-96	1771	1837	1964	2511	118	49	80	418	2629
1996-97	1652	1705	1789	2155	108	51	54	261	2263
1997-98	1629	1682	1766	2198	120	44	57	331	2318
1998-99	1692	1756	1845	2244	114	47	63	289	2358
1999-00	1874	1925	2031	2386	126	31	68	255	2512
2000-01	1676	1713	1777	2068	58	31	50	210	2126
2001-02	1817	1877	1982	2428	118	30	75	342	2546
2002-03	1264	1314	1418	1655	57	25	41	172	1712
2003-04	1769	1826	1953	2403	116	29	62	358	2518
2004-05	1914	1971	2108	2565	126	34	66	357	2691
2005-06	1967	2031	2169	2876	120	37	97	573	2996
2006-07	2091	2152	2363	3024	181	41	95	526	3205
2007-08	2068	2131	2413	3088	220	38	96	541	3308

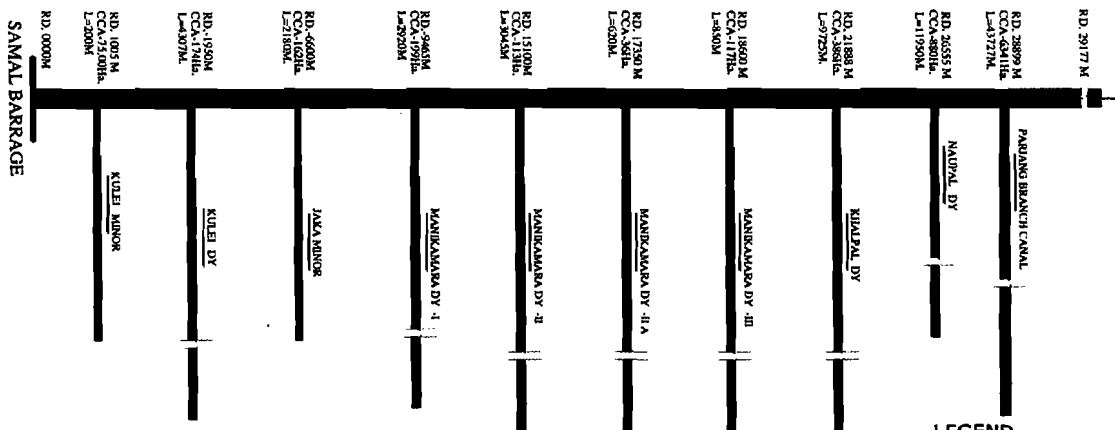
Source: Economic Survey, Government of Odisha 2010-11, Planning & Co-ordination Department, Pg.108, Annex.3.15

RENGALI IRRIGATION PROJECT

LINE DIAGRAM FOR MACRO SYSTEMS OF

L.B.C.-II KM 00.00 TO KM 29.177

Drg. No.3.3(a)



LEGEND

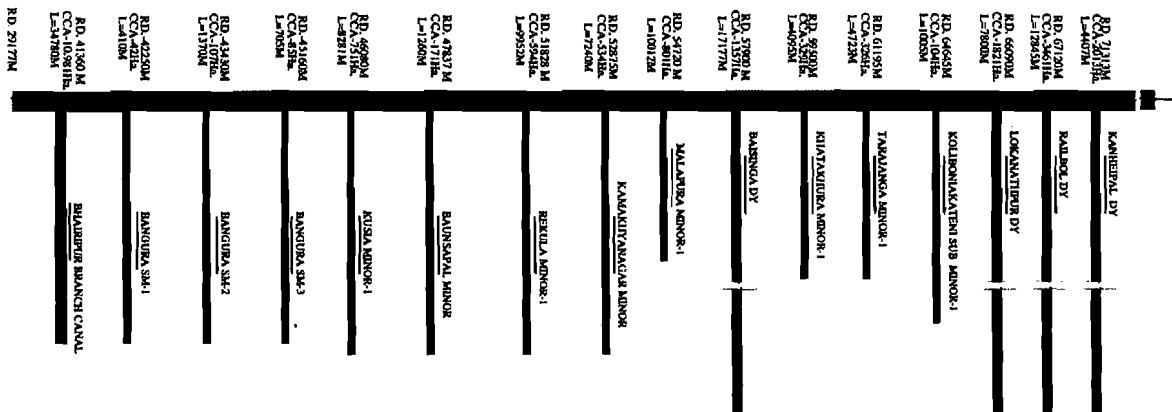
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EXECUTION IN PROGRESS	—
EXECUTION NOT STARTED	—

RENGALI IRRIGATION PROJECT

LINE DIAGRAM FOR MACRO SYSTEMS OF

L.B.C.-II KM 29.177 TO KM 71.313

Drg. No.3.3(b)



LEGEND

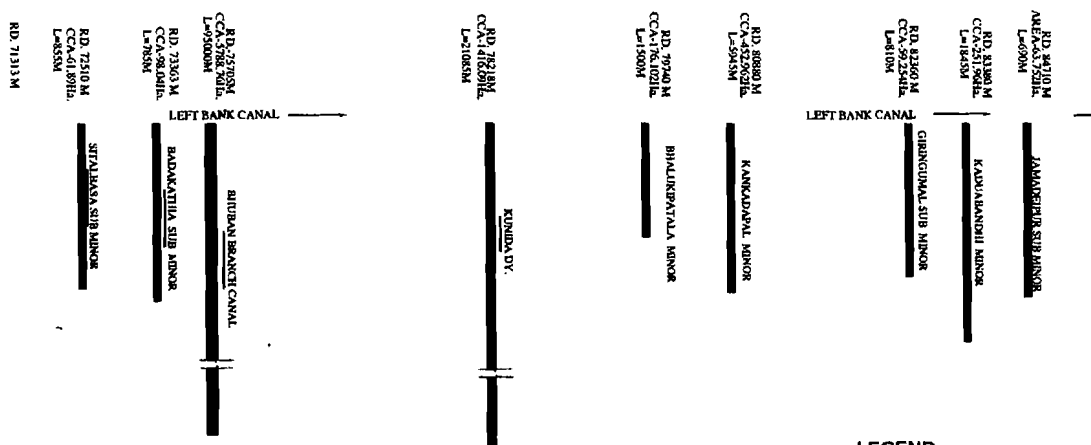
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RENGALI IRRIGATION PROJECT

LINE DIAGRAM FOR MACRO SYSTEMS OF

L.B.C.-II KM 71.313 TO KM 84.710

Drg. No.3.3(c)



LEGEND

EXECUTION COMPLETED

EXECUTION IN PROGRESS

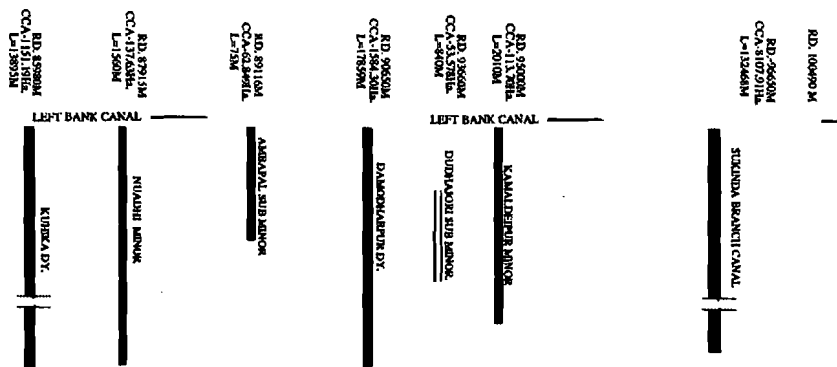
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RENGALI IRRIGATION PROJECT

LINE DIAGRAM FOR MACRO SYSTEMS OF

L.B.C.-II KM 71.313 TO KM 100.49

Drg. No.3.3(d)



LEGEND

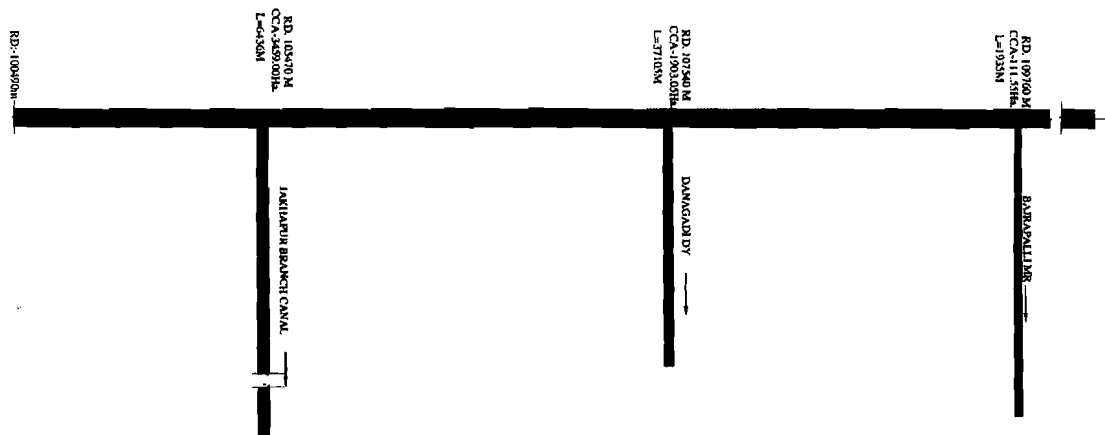
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EXECUTION IN PROGRESS

EXECUTION NOT STARTED

RENGALI IRRIGATION PROJECT
LINE DIAGRAM FOR MACRO SYSTEMS OF
L.B.C.-II KM 100.490 TO KM 109.760

Drg. No.3.3(e)

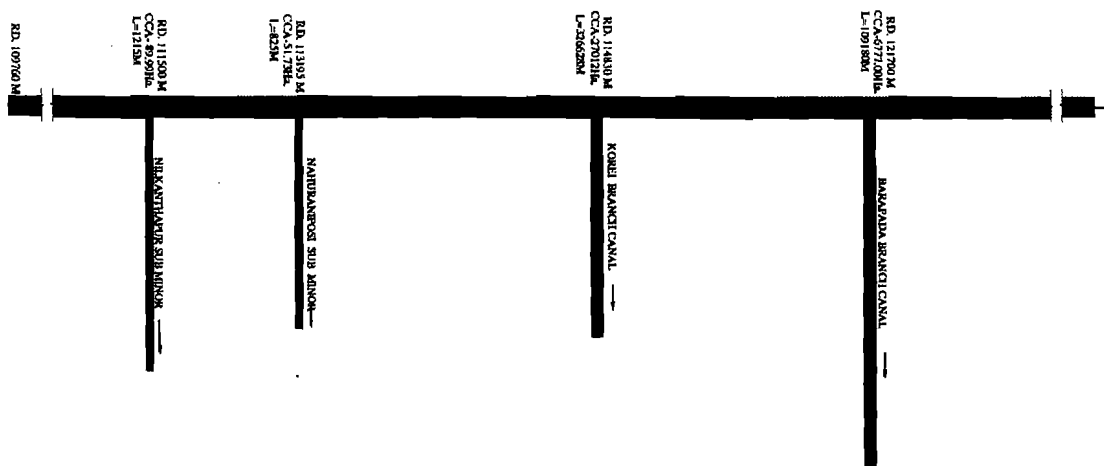


LEGEND

EXECUTION COMPLETED	———
EXECUTION IN PROGRESS	———
EXECUTION NOT STARTED	———

RENGALI IRRIGATION PROJECT
LINE DIAGRAM FOR MACRO SYSTEMS OF
L.B.C.-II KM 109.760 TO KM 121.700

Drg. No.3.3(f)

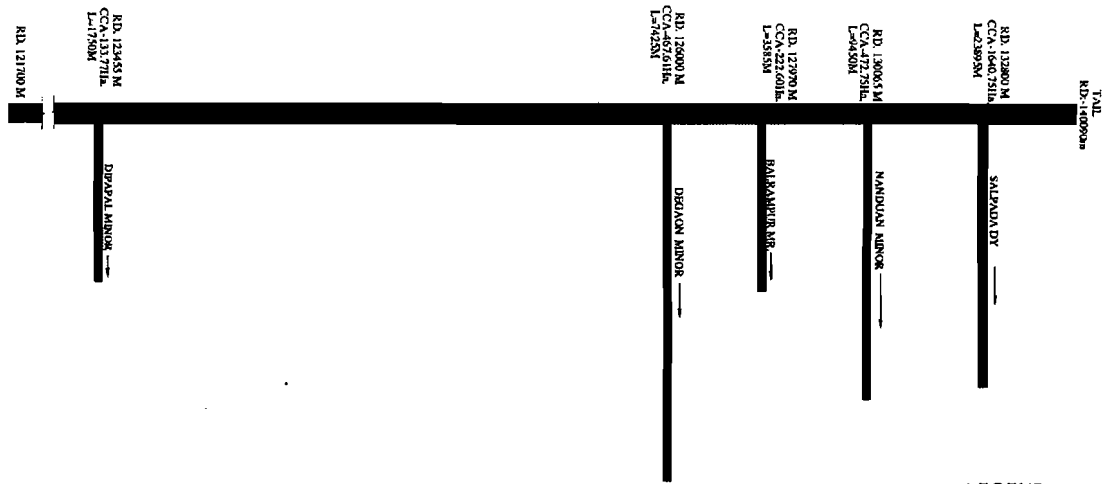


LEGEND

EXECUTION COMPLETED	———
EXECUTION IN PROGRESS	———
EXECUTION NOT STARTED	———

RENGALI IRRIGATION PROJECT
LINE DIAGRAM FOR MACRO SYSTEMS OF
L.B.C.-II KM 121.700 TO KM 140.090

Drg. No.3.3(g)

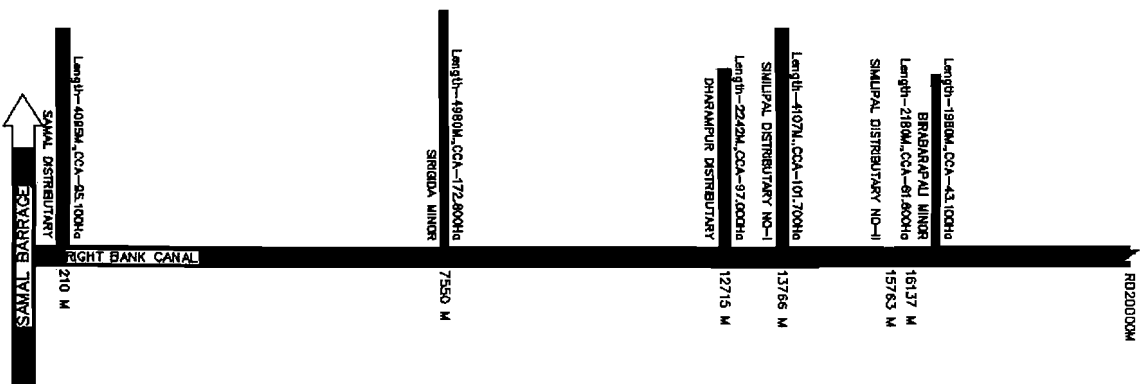


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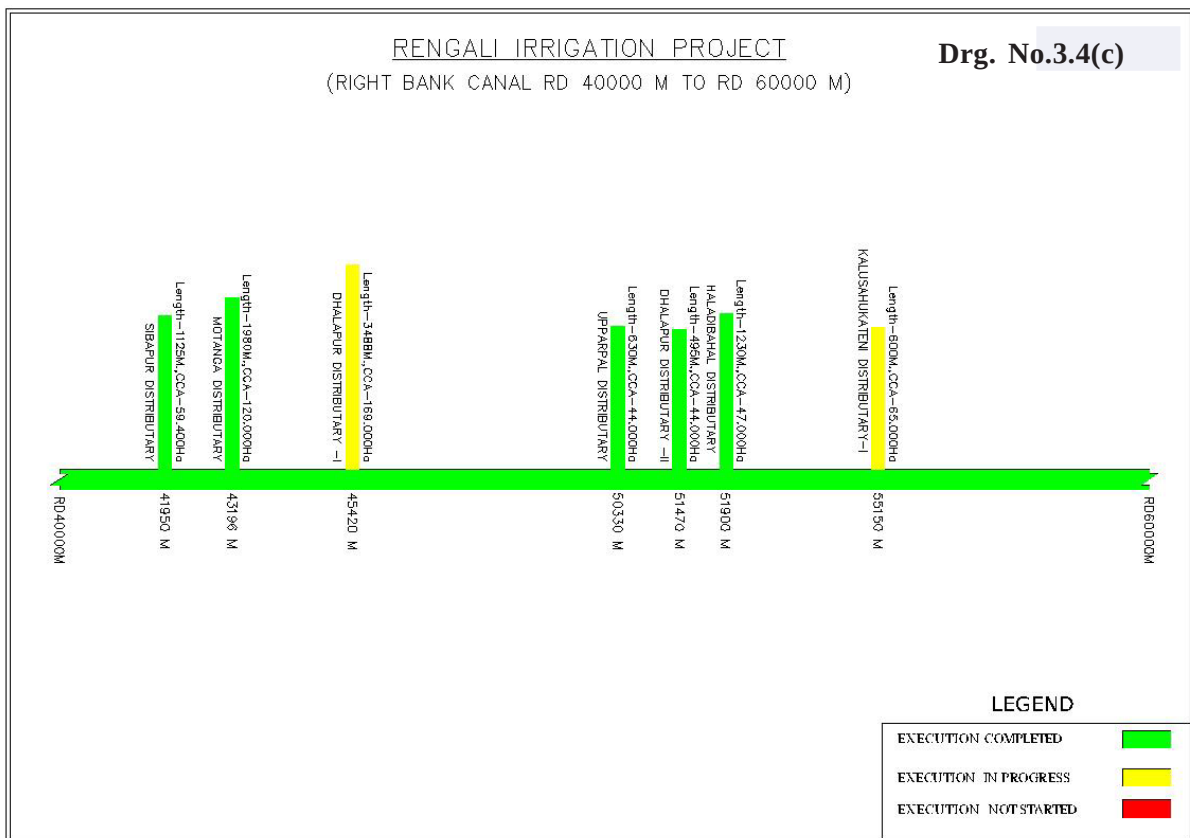
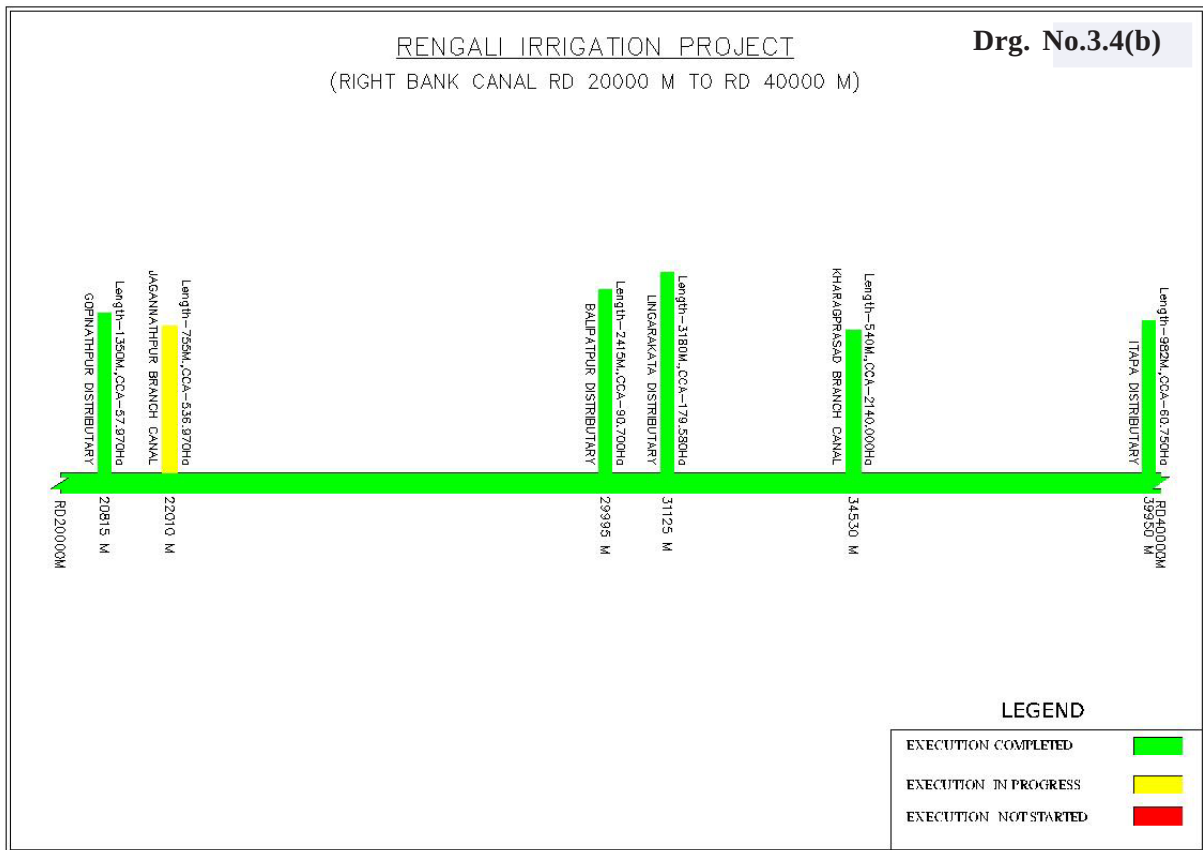
RENGALI IRRIGATION PROJECT
(RIGHT BANK CANAL RD 0 M TO RD 20000 M)

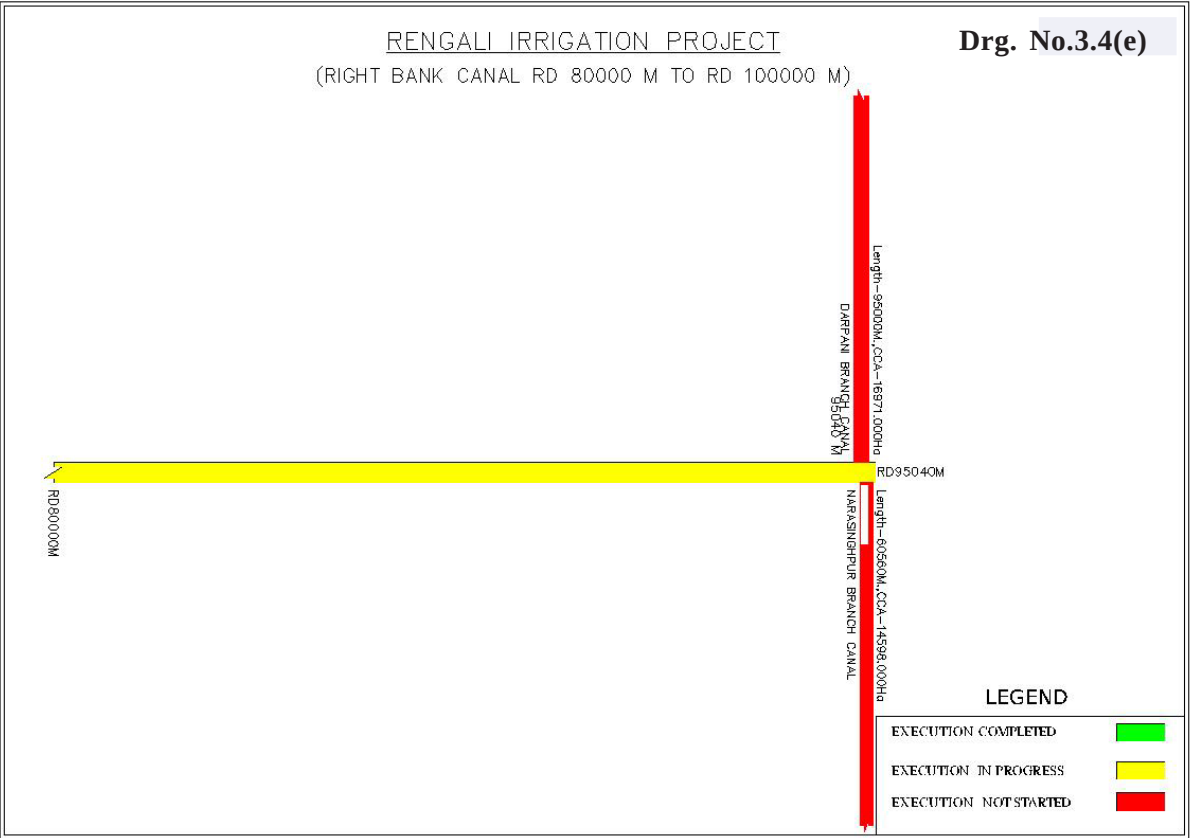
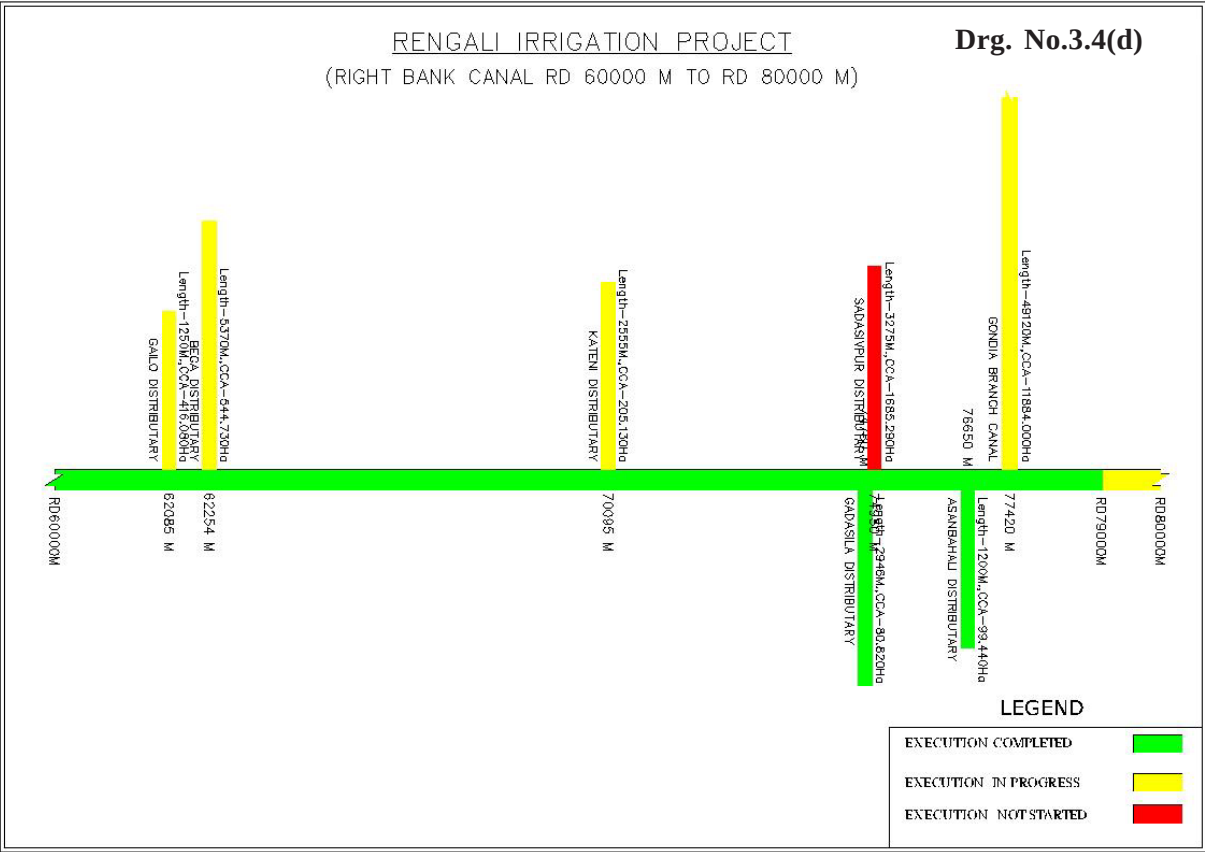
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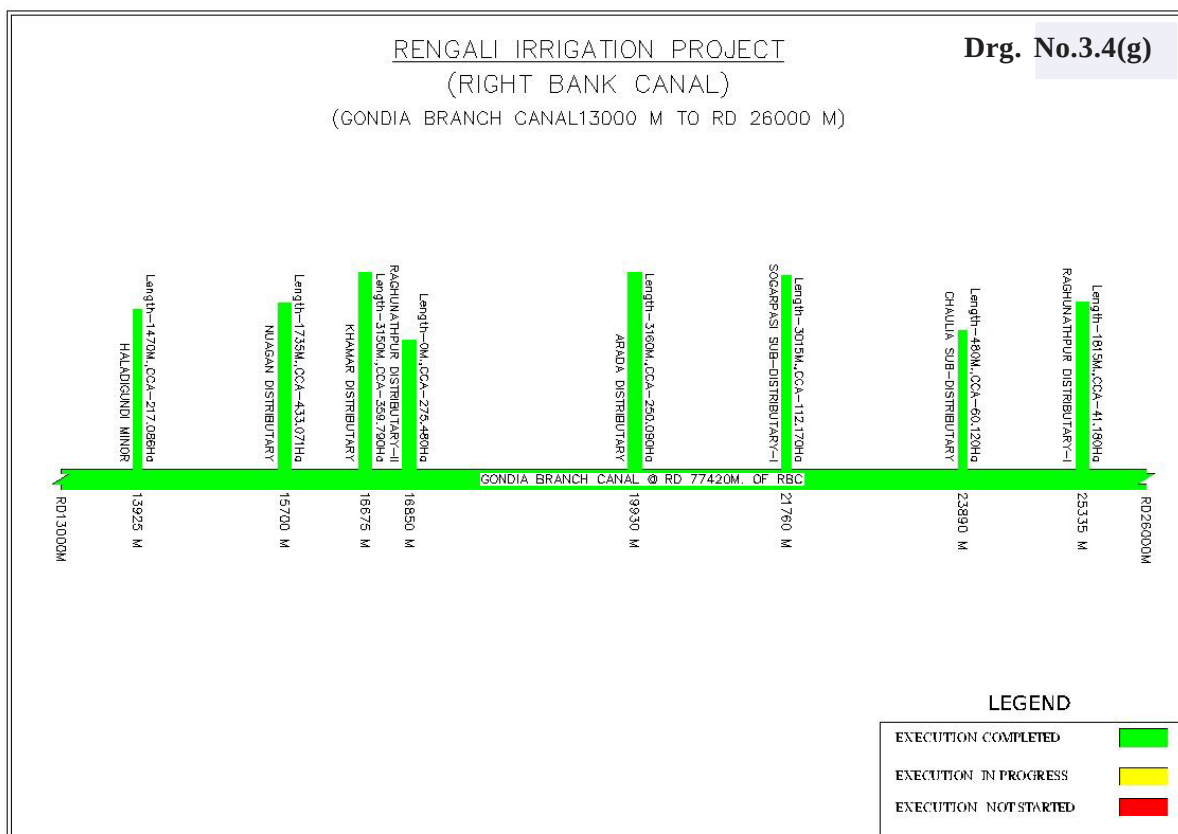
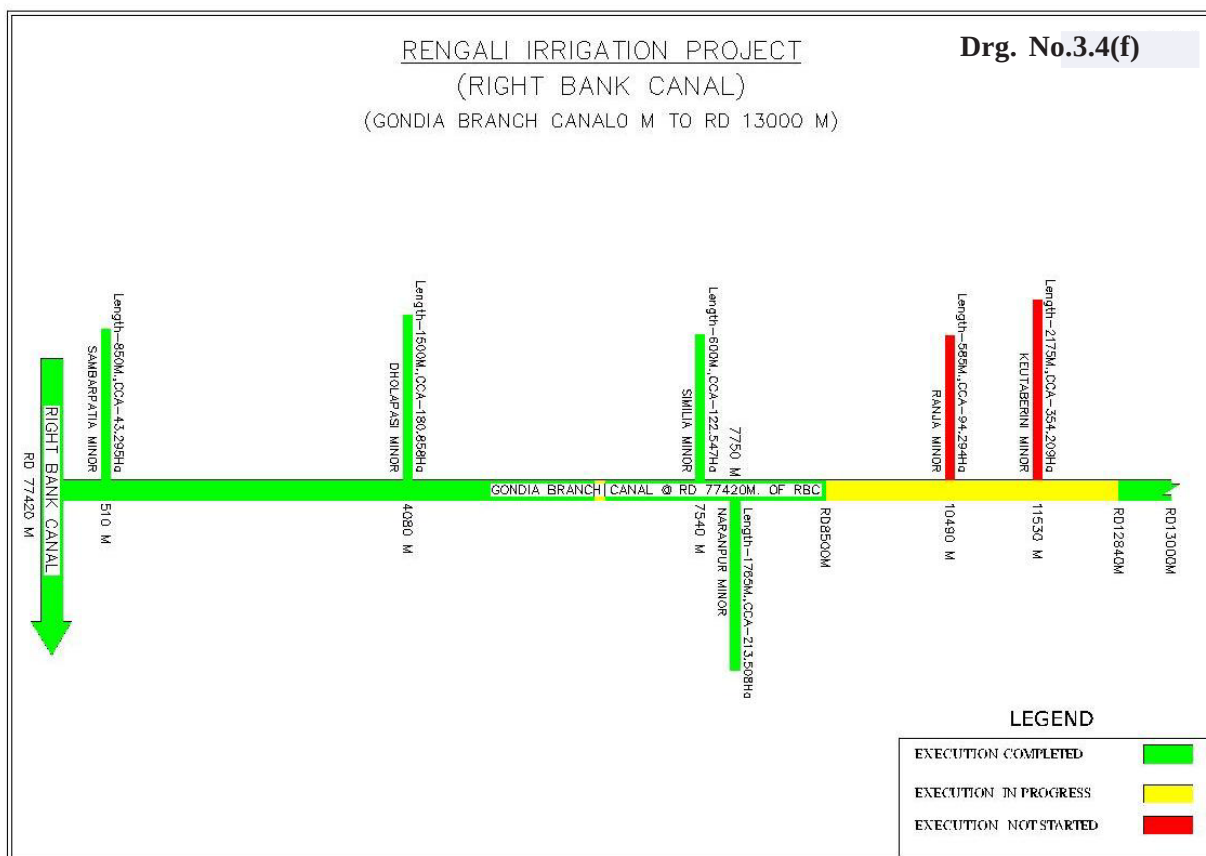


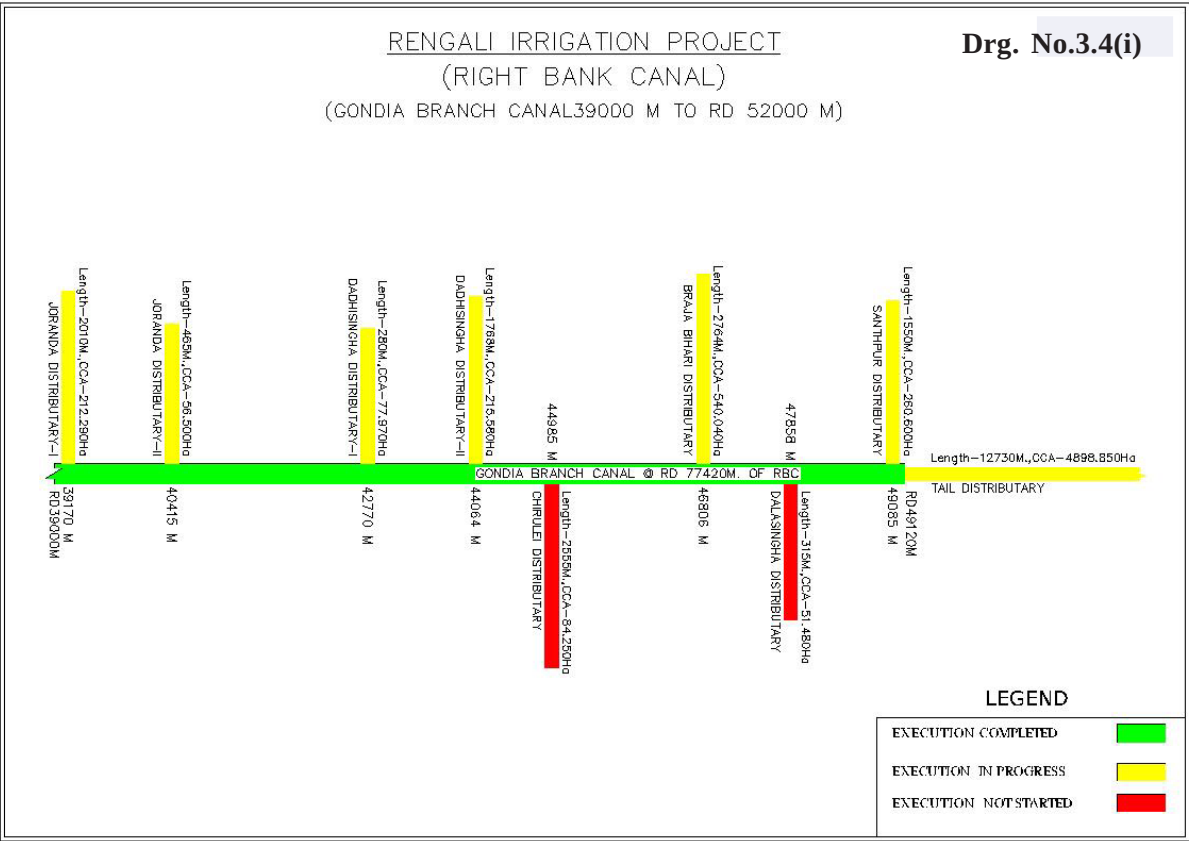
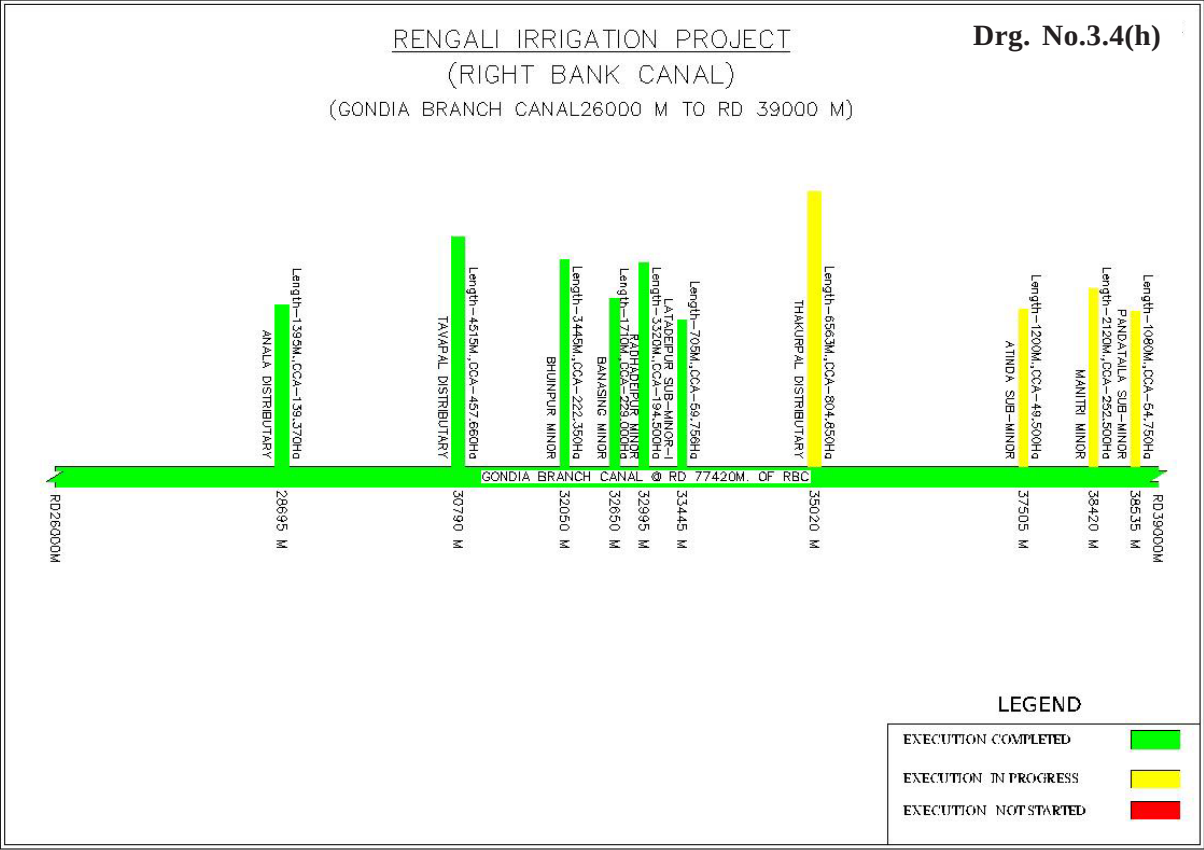
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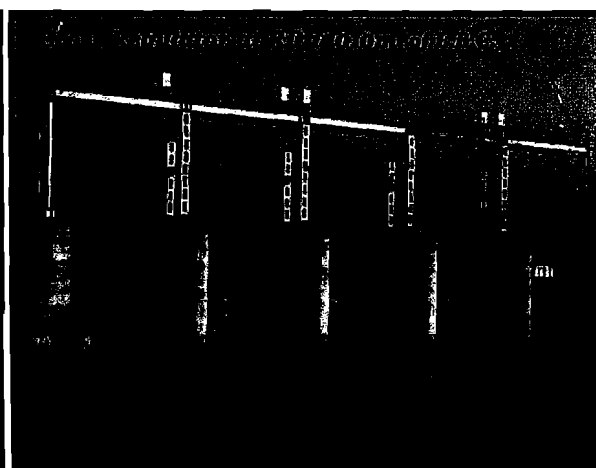
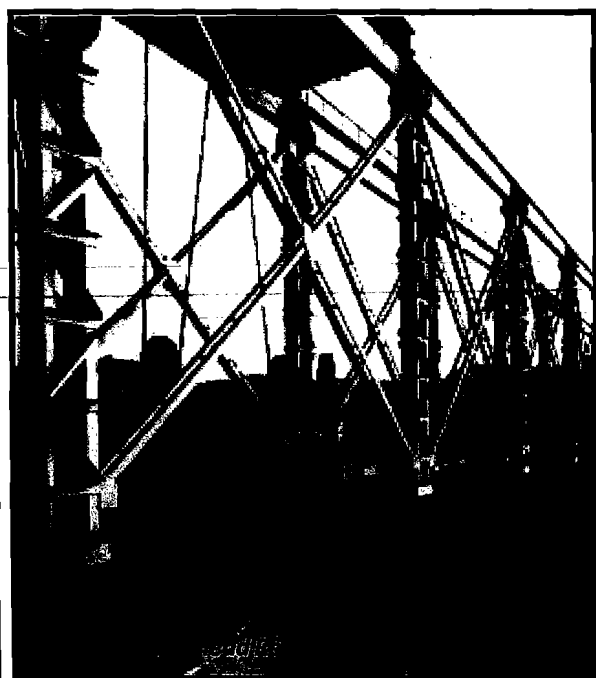
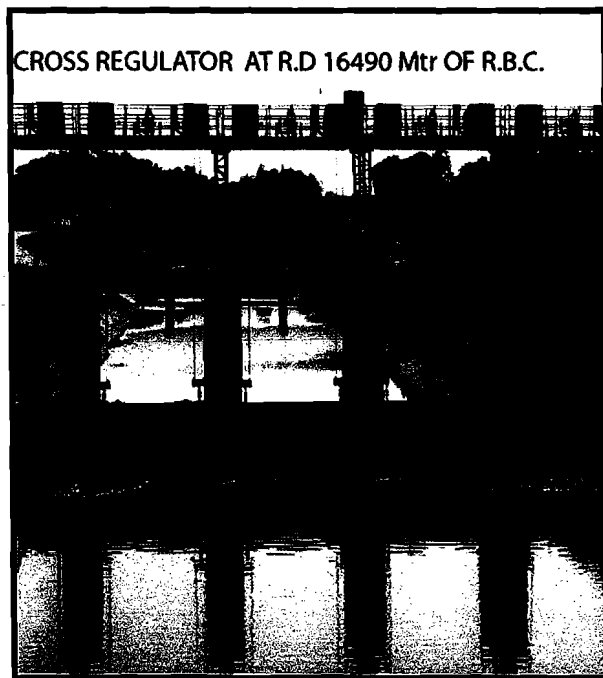
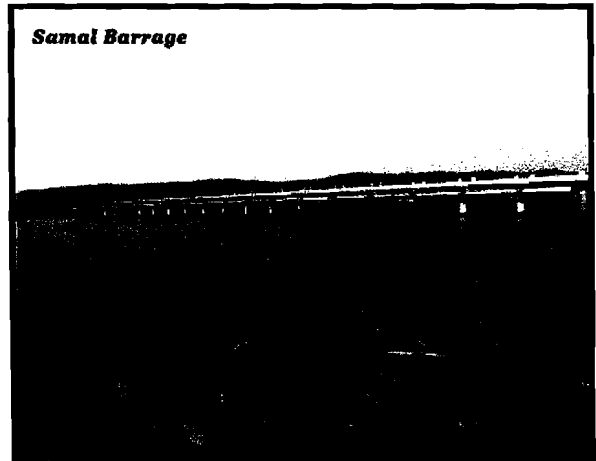
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EXECUTION NOT STARTED	—

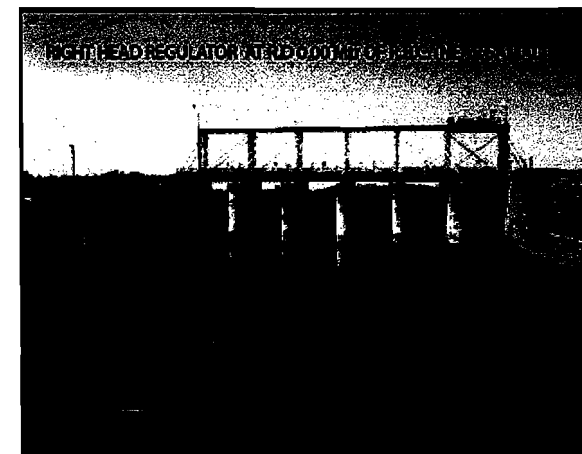
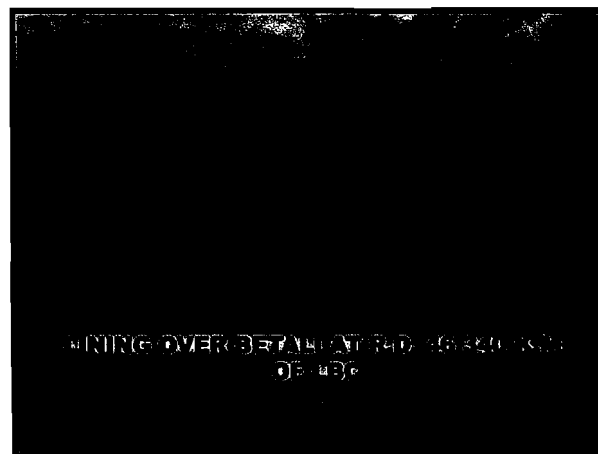
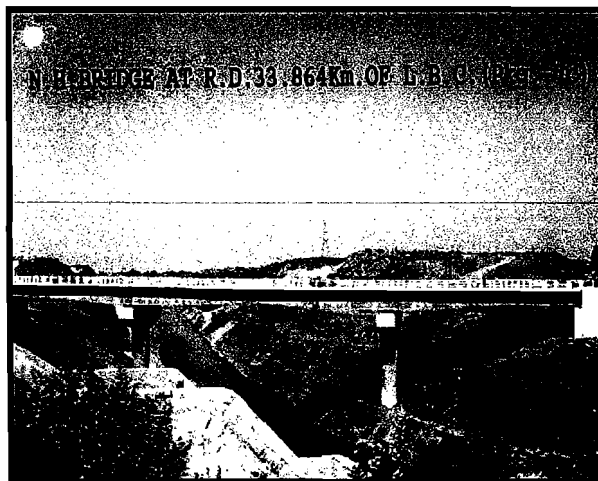
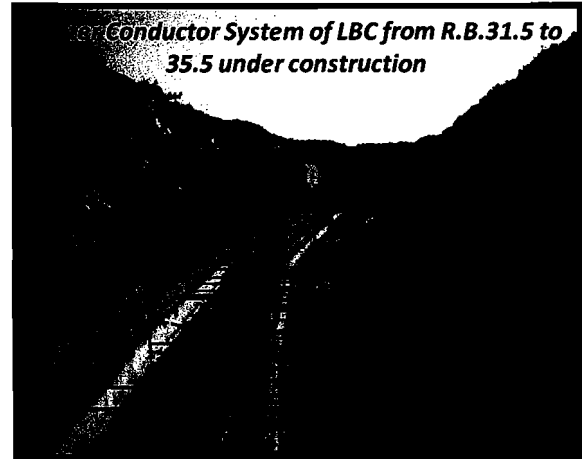
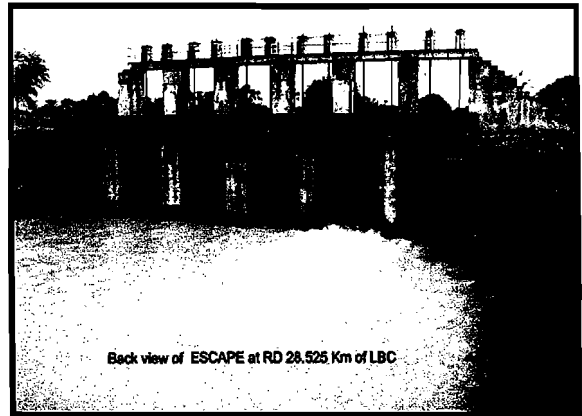
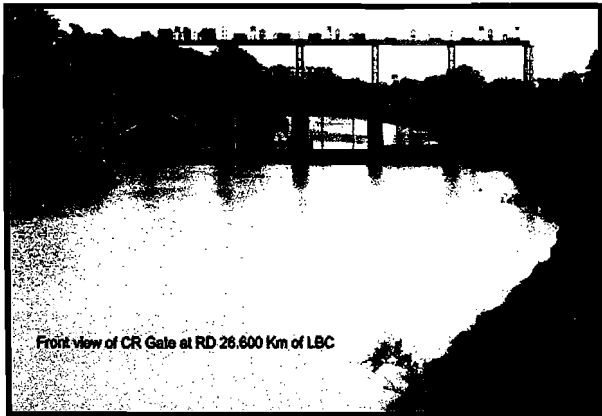


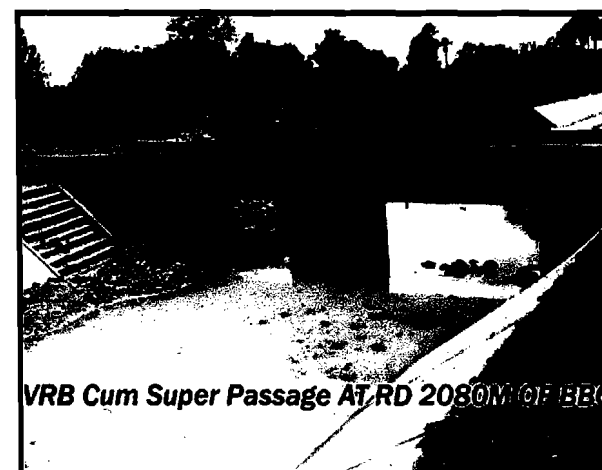
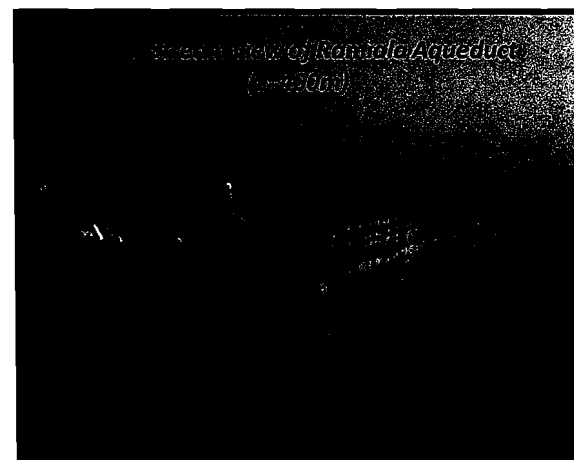
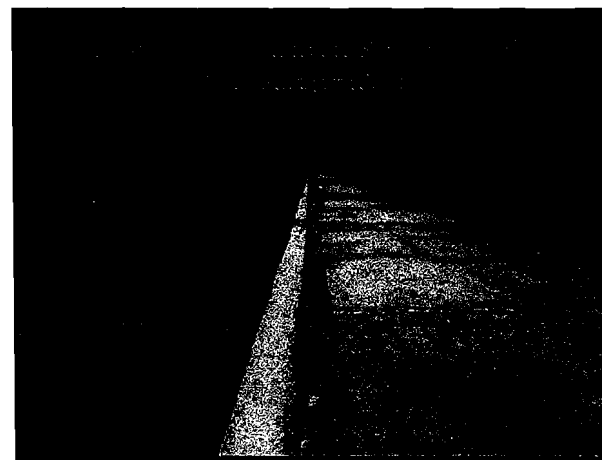
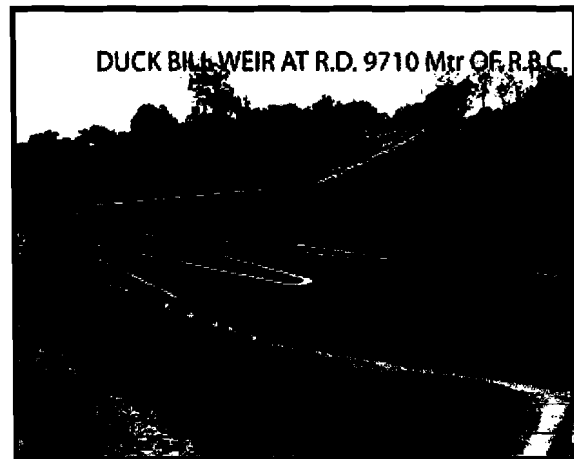
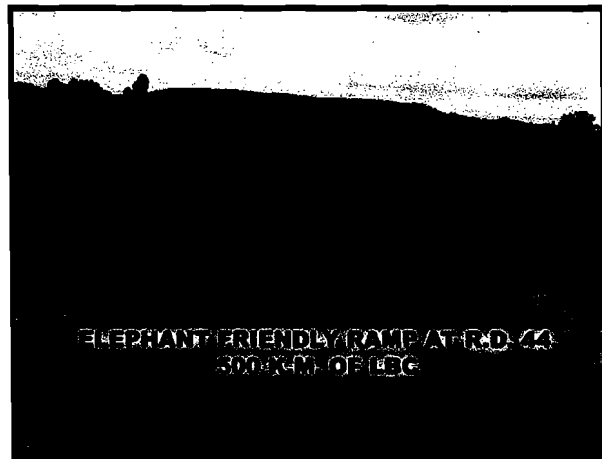
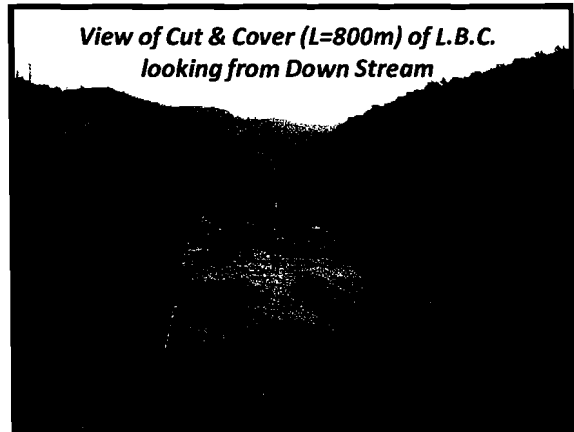
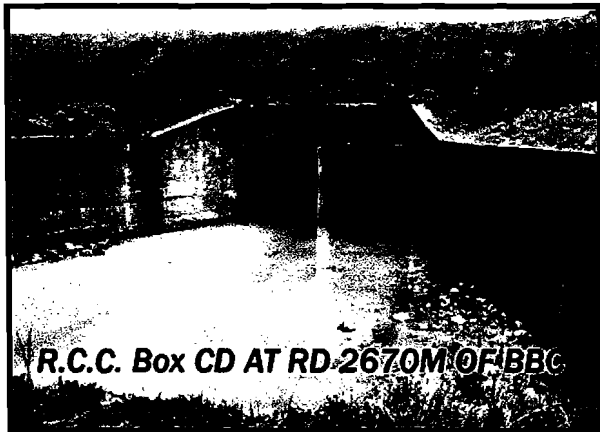


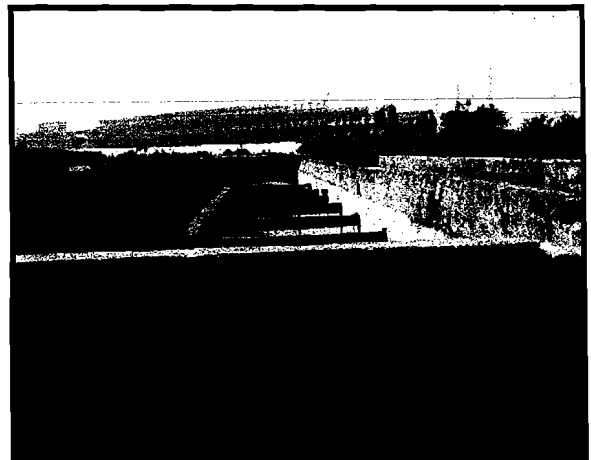
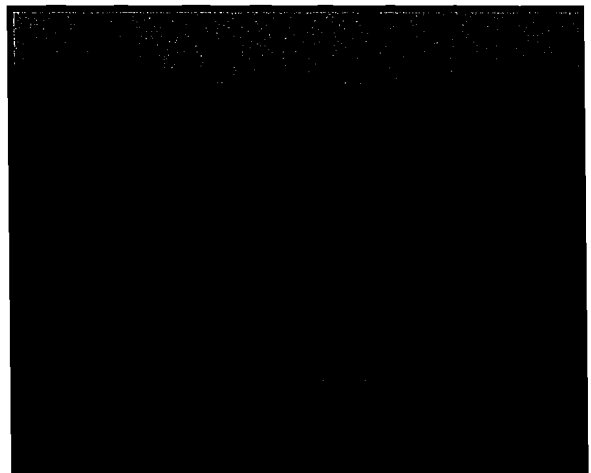
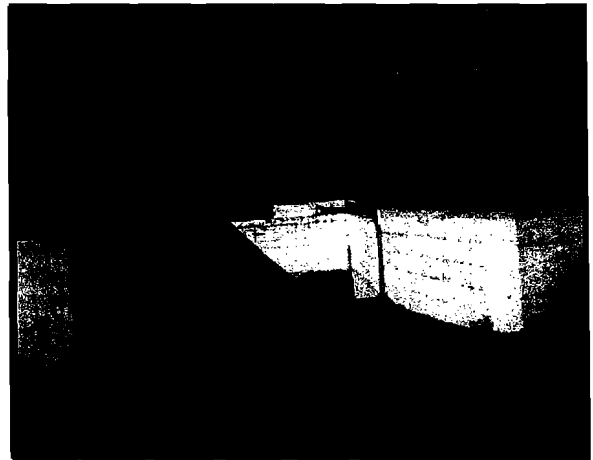
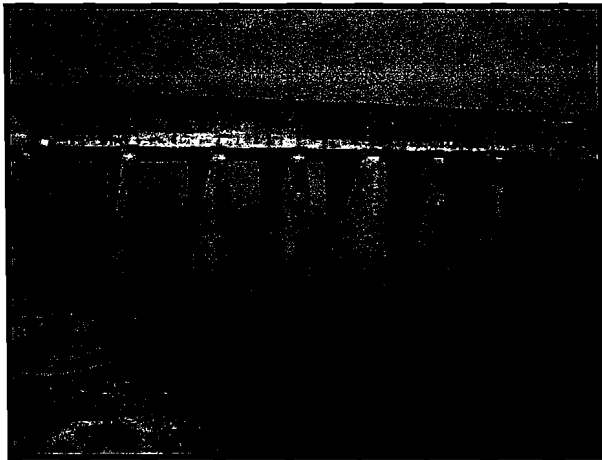












Incumbency chart of Engineers from Chief Engineer to Executive Engineers of Rengali Dam Project:

<u>Chief Engineers</u>	
1. Er. Suresh Chandra Tripathy Addl. C.E.	5. Er. P.K. Acharya C.E. (I/C)
2. Er. M.L. Das, Addl. C.E.	6. Er. B.P. Mohanty
3. Er. Dibakar Mishra C.C.E., Addl. CE & CE	7. Er. P.K. Acharya
4. Er. Gopinath Das	
<u>Superintending Engineer</u>	
1. Er. Dibakar Mishra	4. Er. P.K. Mishra
2. Er. Gopinath Das	5. Er. N.K. Sahoo
3. Er. P. Mishra	6. Er. P.K. Pattanaik
<u>Executive Engineer (Civil)</u>	
1. Er. G.N. Das	21. Er. M.M. Sahoo
2. Er. Banabihari Naik	22. Er. M.M. Patnaik
3. Er. Bamadev Mohapatra	23. Er. P. Parhi
4. Er. M.H.P.B. Pattanaik	24. Er. M. Das
5. Er. Bimal Kishore Mohapatra	25. Er. L.R. Mishra
6. Er. Duryodhan Mohapatra	26. Er. N.K. Tripathy
7. Er. Bhagaban Panda	27. Er. Kamarup Das
8. Er. K.C. Panigrahi	28. Er. B.M. Padhi
9. Er. Chhakadi Pradhan	29. Er. G.C. Swain
10. Er. T. Mallick	30. Er. B.B. Nayak
11. Er. R.C. Das	31. Er. M.C. Tripathy
12. Er. P.K. Pattnaik	32. Er. A.K. Mohapatra
13. Er. R.N. Patnaik	33. Er. R.C. Tripathy
14. Er. D. Swain	34. Er. C.R. Mohapatra
15. Er. J. Pujari	35. Er. Y. Rama Rao
16. Er. N.C. Beuria	36. Er. K.M. Acharya
17. Er. G.C. Bisoi	37. Er. B.M. Biswal
18. Er. N.C. Rout	38. Er. H.S. Bawa
19. Er. K.C. Mishra	39. Er. B.N. Pradhan
20. Er. P.C. Mishra	
<u>EXECUTIVE ENGINEER (Mechanical)</u>	
40. Er. S.M.H. Rahaman	41. Er. N.C. Rao
42. Er. M.K. Meher	43. Er. B.B. Mishra
44. Er. M. Basa	

Following engineering wizards have contributed maximum for the successful completion of the Project. They are (i) Er. S.C. Tripathy for Planning, Design & Investigation (ii) Er. Dibakar Mishra for construction of the Dam and (iii) Er. Gopinath Das for construction of the barrage.

**Salient Features
Rengali Dam Project**

1. Location:

- a) Village : Rengali, Block-Kaniha, Sub-Divn. Talcher,
Dist-Angul, Odisha.
- b) Latitude : 21° - 17' - 00" N
- c) Longitude : 85° - 02' - 00" E
- d) Topo Sheet : No 73 G/3

2. Hydrology:

- (A) Catchment area lies between 83° 55' E to 86° 04' E & 23° 7' N to 20° 35' N
- | | | |
|--|---|---------------|
| (i) Drainage area of Brahmani upto Jenapur (Head of delta) | - | 36,444 sq.km. |
| (ii) Drainage area of Brahmani upto Dam site | - | 25,250 sq.km |
| (iii) Drainage area from Dam to Jenapur | - | 11,194 sq.km. |
| (iv) Catchment area in Odisha | - | 8650 sq.km. |
| (iv) Catchment area in Jharkhand | - | 15700 sq.km. |
| (v) Catchment area in Chhatisgarh | - | 900 sq.km. |
| (vi) Total length of river | - | 715 km. |
| (vii) Length of river in Jharkhand | - | 260 km. |
| (vii) Length of river in Odisha | - | 455 km. |
| (viii) Slope of | | |
| (a) South Koel to 83 km. d/s | - | 1:798 |
| (b) From 83 km to 182.4 km. | - | 1:362 |
| (c) 182.4 km To Rengali Dam Site | - | 1:937 |
| (d) From Rengali Dam site to Bay of Bengal | - | 1:7397 |
- (B) **Rainfall**
- | | | |
|------------------------------|---|---------|
| (i) Maximum Annual Rainfall | - | 2850 mm |
| (ii) Minimum Annual Rainfall | - | 890 mm |
| (iii) Mean Annual Rainfall | - | 1570 mm |
- (C) **Design Flood (Flow particulars)**
- | | | |
|---|---|-------------|
| (i) Maximum Run off | - | 3,650 M ham |
| (ii) Minimum Run off | - | 0.567 M ham |
| (iii) Mean Annual Run off | - | 1.49 M ham |
| (iv) 1000 year return period flood | - | 27800 cumec |
| (v) Maximum probable flood (P.M.F.) | - | 55540 cumec |
| (vi) Available Run off 75% dependability | - | 0.915 M ham |
| (vii) Available Run off 90% dependability | - | 0.743 M ham |
- (D) **Reservoir**
- | | | |
|--|---|---------------|
| (i) Maximum water level (MWL) | - | EL- 125.40 m |
| (ii) Full Reservoir level (FRL) | - | EL- 123.50 m |
| (iii) Dead storage level | - | EL- 109.70 m |
| (iv) Storage capacity at M.W.L. | - | 0.515 M ham |
| (v) Storage Capacity of F.R.L. | - | 0.440 M ham |
| (vi) Storage capacity at D.S.L. | - | 0.0986 M ham |
| (vii) Live storage capacity of Reservoir | - | 0.3414 M ham |
| (viii) Water spread area at M.W.L. | - | 414.0 sq.km. |
| (ix) Water spread area at F.R.L. | - | 378.40 sq.km. |
| (x) Water spread area at D.S.L. | - | 143.08 sq.km. |

3. Dam:

(i) Type of Dam	- Gravity (Masonry)
(ii) Length of Dam	- 1040 m
(iii) No. of Blocks	- 51 Nos
(iv) Length of overflow portion including piers.	- 464 m
(v) Length of Power Dam	- 105 m
(vi) Length of non overflow Blocks (Left & Right)	- 471 m
(vii) Deepest foundation level	- EL. 57.50 m
(viii) Top of road level	- EL. 128.50 m
(ix) Maximum height above foundation	- 71 m
(x) Average height of am	- 45 m
(xi) Overall Top width of Dam	
(a) Overflow portion	- 7.0 m
(b) Non-overflow portion	- 8.1 m
(xii) Slopes provided in dam section	
a) D/s slope of N.O.F.	- 0.7:1 below EL: 114.00 m
b) U/s slope of N.O.F.	- 0.1:1 below EL: 114.00m
c) D/s slope of Power Dam	- 0.9:1 below EL: 114.00m
d) U/s slope of Power Dam	- Vertical
(xiii) Free Board	- 3.10 m
(xiv) Seismic co-efficient	- 0.05 Horizontal 0.025 Vertical

4. Spillway

(i) Type of Spill way	- Oggee spillway with ski jump bucket.
(ii) Length	- 464 m including pier.
(iii) No. of Gates	- 24 nos of Radial gates.
(iv) Crest evevation	- 110.2 m
(v) Maxm. height above deepest foundation	- 48.5 m
(vi) Flood discharging capacity of spillway	- 46,960 cumec.
(vii) Thickness of pier	- 4.00 m
(viii) Equation of spillway profile	- $X^{1.85} = 16.532^y$
(ix) Invert level of lower Bucket	- EL.83.50 m
(x) Sill level of lower bucket	- EL.86.755 m
(xi) Invert level of upper Bucket	- EL.87.25 m
(xii) Sill level of upper bucket	- EL.90.505 m
(xiii) Radious of Bucket	- 18.00 m
(xiv) Size of Foundation Gallery	- 1.50 m x 2.10 m

5. Gate

(i) Number of Radial gates	- 24 Nos.
(ii) Size of each gate	- 15.50 m x 14.80 m
(iii) Sill elevation	- 109.950 m
(iv) Top of gate in closed position	- 125.0 m
(v) Bottom level of Gate in fully raised portion	- 122.70 m
(vi) Top level of Hoods on both sides of Gate	- 125.40 m
(vii) Radius of Gate	- 15.250 m
(viii) Trunnion center line	- EL. 115.200 m
(ix) Skin Plate thickness	- Varies from 18 mm to 10 mm.
(x) Operation	- Operated by 145 Ton Rope drum hoist

6. Spillway Stoplogs

- (i) Number of stoplog for each sets - 10 Nos
- (ii) Size of each piece - 15.500 x 1.40 m
- (iii) No. of sets - 3
- (iv) Weight of each piece (Bottom) - 16.188 Ton
Top & middle piece (each) - 20.559 Ton
- (v) Materials - IS 226:1978 structural steel
- (vi) Capacity of Gantry crane - 45 T

7. Spillway Bridge

- (i) Number of span - 24 Nos.
- (ii) Length of each span - 19.5 m
- (iii) Type - RCC Girder Bridge
- (iv) No. of Beam - 3 nos. with bulb at bottom
- (v) Height of the beam - 1650 mm
- (vi) Thickness of the beam - 400 mm
- (vii) Thickness of the slab - 250 mm
- (viii) Height of the parapet - 1200 mm including cable trench
- (ix) Clear road width of Bridge - 5.9 m
- (x) Total width (overall) - 7.0 m
- (xi) Bearings - Rocker & Roller Bearing

8. Irrigation Sluices

Sl. No.	Particulars	Block No. 5 & 49	Block 43 (For Barrage)
1	Vent size (m)	1.50 x 2.00	3.5 x 5.33
2	Sill level (m)	109.0	97.0
3	Gate size (m) Service / Emergency	1.5 x 2.0	3.5 x 5.33
4	Type of Gate	Vertical lift fixed wheel	Vertical lift fixed wheel
5	Type of hoist Service Gate Emergency Gate	Manually operated screw gear Manually operated chain pulley block	Wire rope drum hoist -do-
6	Discharging capacity	5.90 cumec	-
7	Capacity of hoist	-	65 T – Emergency gate 45T – Service gate

9. Left Training Wall

- (i) Location - RD 376.76 m between
Block 19/1 & 19/2 from Left side of dam
- (ii) No. of Blocks - 6 nos.
- (iii) Length of each block - 15m
- (iv) Total length of the wall - 90 m
- (v) Top level of lowest block - EL. 100.00 m
- (vi) Top level of highest block - EL. 111.50 m
- (vii) Top/Bottom width - 3m/20m
- (viii) Slope (Non water face) - 0.60 : 1.00
(Water face) - Vertical
- (ix) Average level of foundation - EL. 80.00 m
- (x) Type - Masonry

10. Right Training Wall

(i) Location	- RD 840.75 m in Block 43 from Left side of dam.
(ii) No. of Block	- 6 nos.
(iii) Total length of wall	- 90 m
(iv) Length of each block	- 15 m
(v) Top level of blocks	- EL. 108.00 m
(vi) Top width	- 3 m
(vii) Bottom width	- 20 m
(viii) Slope	- 0.60:1
(ix) Average level of foundation	- EL.84.00 m
(x) Type	- Masonry
(xi) Length of Return wall	- 30 m (2 x 15m)
(xii) Return Angle	- 45°
(xiii) No. of blocks in return wall	- Two

11. Divide Wall

(i) Location	- RD 647.75 m in Block 33 dividing Upper & lower bucket.
(ii) Width of divide wall	- 30 m
(iii) Top level of divide wall	- 95.00 m
(iv) Length of divide wall	- 30.740 m

12. Diversion Channel

(i) Average non-monsoon flow	- 85 cumec
(ii) 100 year return period	- 368 cumec
(iii) U/s M.W.L. (Designed)	- 84.00 m
(iv) Sill level of Diversion sluice	- 78.00 m
(v) Bed width of diversion channel	- 20.00 m
(vi) Low block level (22 to 25)	- 82.50 m

13. Diversion Sluice (Block No.21)

(i) No. of sluice	- 2 Nos
(ii) Vent size	- 3.5 m x 4.0 m
(iii) Sill level	- EL. 78.00 m
(iv) Formula for Bell mounth entry	- $\frac{x^2}{3.33^2} + \frac{y^2}{1.759^2} = 1$
(v) Length	- 33 m
(vi) No. of gates for Plugging	- 2 Nos
(vii) Size of gate	- 3.4 m x 5.33 m
(viii) Type of gate	- Vertical lift fixed wheel gate
(ix) Foundation level of sluice	- RL. 76.00 m

14. U/S Cofferdam

(i) Loation	- From Block No. 12 to 17
(ii) Length	- 57 m
(iii) Top width	- 2.50 m
(iv) Bottom width	- 14.40 m
(v) Maximum height above foundation	- 25.00 m
Crate Dumping	- 1768 Nos.

- (vi) Rubble dumping
 - (a) In Crates - 12,005 cum
 - (b) In loose for body portion - 19,207 cum
- (vii) Muck dumping - 12,365 cum
- (viii) Spall & metal dumping - 5719 cum
- (ix) Sand dumping - 70,866 cum
- (x) Earth dumping - 218 cum
- (xi) Steel sheet piling - 3386 mtrs.
- (xii) Masonry & concrete wall
 - (a) Masonry - 3360 cum
 - (b) Concrete - 1307 cum

15. D/S Cofferdam

- (i) Location of D/S Cofferdam - From Block No. 15 to 18
- (ii) Length of D/S Cofferdam - 22.50 m
- (iii) Average width - 21.3 m
- (iv) Side slope - 1:1
- (v) Crate Dumping - 626 Nos.
- (vi) Rubble dumping
 - (a) In Crates - 4176 cum
 - (b) In loose for body of Cofferdam - 16552 cum

16. Dyke

- (i) Location - Near village Khindo
- (ii) Length - 460 m
- (iii) Maximum height - 4.60 m
- (iv) Section - U/S & D/S slope 2 (H) : 1 (V)
- (v) Material - Homogeneous Earth
- (vi) Top width - 5.00 m
- (vii) Total quantity of earth - 21,345 cum

17. Flood Control

- (i) Area protected from flood - 2600 Sqkm.
- (ii) Population benefited - 10.80 lakhs (Estimated during 1971)
- (iii) Average annual direct benefit - Rs.6.86 Crores (As per 1971 estimate)
- (iv) B.C. Ratio - 1.005

Salient Features

18. Rengali Power House

Total No. of Generating Units - 5

Installed Capacity : 5 x 50 MW = 250 MW

Size of P.H. - 123.7m x 20.6 m

Firm Power (Present) 60 MW. Firm Power (Ultimate) - 91 MW

Design Energy : 519.75 MU

Generator Specification

Type : SV 710/143 36R

MW Rating: 55 MW, MVA Rating : 61.1 MVA

P.F. : 0.9 Lag, Volt : 11 KV

Rated Speed : 166.7 rpm,

Run away speed : 350 rpm

Frequency : 50 c/s, Phase : 3

Connection : Y (Neutral Grounding)
Insulation : Class B
Rotation : Clockwise
Stator Amp - 3.21 KA
Rotor Volt - 220 V, Amp : 895 A
Excitation - Separate
PMG- 104/10-36R, Voltage: 110V, Current: 1.31A
Phase - 3, Connection- Y

Turbine Specification

Type: KAPLAN
Rated Output: 53 MW, Diameter: 4510mm
Speed of rotation: 166.7 rpm, Run away Speed: 351 rpm
Max. head: 46.5 m,
Min. head: 30.5 m,
Design head: 40 m,
Discharge through Turbine at rated out put & of design head: 147.3 m³/cumec
Max Speed rise due to sudden load throw: 1-5%
Guide Vane Height: 1570mm
No of Guide Vanes- 24
Governing unit: Electro Hydraulic- EGHK-100, G-40
Diameter of Penstock: 5.960 m, Diameter of Runner: 4.5m
Tail race level (minm., maxm.)- 77.00 m/94.81m

Draft Tube Specification

Depth of Draft tube- 10350 mm from the lower Ring.
Length : 202500 mm, Width : 13000 mm

Hoist Gate Specification

Cylinder dia : 340 mm, Rod dia : 80 mm
Stroke : 7800 mm
Opening speed- 0.3m/min.
Closing speed- 2.7m/min
Pump delivery for opening operation-27 lit/min
Maximum opening pressure-60 kg
Motor capacity - 15 HP
Oil used - Servo system 68

Power Transformer Specification: (out door)

Capacity 63 MVA, Type - W FOC-3NYCP

Voltage- HV-235 KV + 7.5%

- 2.5% in 2.5% steps, 5 taps (OFF-LOAD Tap changer)

LV - 11 KV

Current - HV - 155 A (rated Tap)

LV - 3307 A

Phase-3, Frequency - 50HZ

Cooling - OFW - Water cooled, forced oil circulation

Vector - Y d 11

(Mini Power House on left bank canal at Samal Barrage)

Brief Description

Orissa Power Consortium Limited

5 x 4 MW, Small Hydro Electric Project

Location:- On Left Bank Canal at Samal Barrage.

Date of Synchronization with Grid-11th October 2009

Technical Details

A. AC Generator:

Rating:- 4000 KW/5000KVA, 11,000 V, 50 HZ, 750 RPM

Make:- Toyo Denki Power System (TDPS), Bangalore, India

Quantity:- 05 Nos

B. Turbine:

Type - Horizontal Full Kaplan 'S' Type

Capacity - 4000 KW

Head (Net Head) - 9.85 m

Rated Discharge - 47.25 cumec

Speed - 168.5 rpm

Runner Dia. - 2800 mm

Make - Andritz Hydro Pvt. Ltd., Faridabad, India

C. Gear Box

Make - Moventas, France

Type - S1GU-800TSZW

Input Speed - 168.5 rpm

Ratio - 1:4:45

Power - 4740 KW

D. Power Transformer

2X15000 KVA, 132/11 KV, Generator Transformer

Make - Transformer & Rectifier (I) Pvt. Ltd., Ahmadabad, India

E. Auxiliary Transformer

2 x 400 KVA, 11/0.433 KV

Make - Transformer & Rectifier (I) Pvt. Ltd., Ahmadabad, India

F. Transmission System

132 KV Double Circuit Line - LILO with TTPS-Duburi Circuit
LILO Point- Loc No-23, at Sarang
Line Length - 26 km, D/C
Conductor - ACSR Panther

G. Monthly Average Generation - 5.70 MU

H. Utilization of water

13 cumec of water per MW of Generation.

19. Cost

(i)	Dam & Appurtenant work	-	Rs.169.05 Cr.
(ii)	Cost of Power House Civil Works	-	Rs. 32.29 Cr.
(iii)	Cost of P.H. Electrical works	-	Rs. 96.80 Cr.
(iv)	Total cost of the project	-	Rs.298.14 Cr.

20. Allocation of Cost

(i)	Flood Control	30 %
(ii)	Power	46 %
(iii)	Irrigation	24%



View of Rengali Power House

Rengali Irrigation Project

I. Location : (Head Works)

Village	- Near Samal
Latitude	- 21° - 01' N
Longitude	- 85° - 07' E
River	- Brahmani
District	- Angul
State	- Odisha
Topo Sheet	- No.73 G/4

II. Hydrology

1. Catchment area of the Brahmani at Barrage site	- 30,030 sq.km.
2. Intercepted catchment at Rengali Dam Site	- 25,250 sq.km.
3. Free catchment at Barrage site	- 4,780 sq.km.
4. Mean annual rainfall	- 1570 cm.
5. Observed Maxm. flood at Barrage site	- 30.000 cumec
6. Designed super flood at Barrage site	- 49.000 cumec

III. Pond

1. Maximum water level at Barrage site	- 79.50 m
2. Pond level	- 76.20 m
3. Water spread area at Pond level	- 28.80 sq.km.
4. Maximum draw down level of Pond	- 75.50 m
5. Storage	- 20.10 Million cum

IV. Barrage

1. Design discharge (100 Year flood)	- 24,632 cumec
2. Length of Barrage (abutment to abutment)	- 560.5 m
3. Clear Water way	- 480.0 m
4. Undersluice - Left, No. of bays	- 4 Nos.
Type & size of gates	- 20.0 m x 10.2 m, Radial
Right, No. of bays	- 3 Nos.
Size & type of gates	- 20.0 m x 10.2 m, radial
5. Spillway	
a) No. of Bays	- 17 Nos.
b) Type & size of Gates	- Radial, 20.00 m x 9.2 m
6. Top of Barrage road level	- RL 83.0 m
7. Crest level	
a) Barrage Bay	- RL 67.00 m
b) Undersluice	- RL 66.00 m
8. Depest Bed level of river	- RL 60.96 m

V. Canal System

A. Right Main Canal

1. Head Discharge capacity - 111.3 cumec
2. No. & Size of H.R. vents. - 6 nos. (6.0m x 4.3 m)
3. Length of Main Canal - 112.40 km.
4. F.S.L. of Canal at Head - 75.50 m
5. Bed width and F.S.D. at head - 11.10 m & 4.30 m
6. Free Board - 0.75 m
7. C.C.A. - 1,04,092 ha.
(Flow - 84406 ha., Lift - 19686 ha.)

B. Left Main Canal

1. Head Discharge capacity - 151.864 cumec
2. No. & Size of H.R. vents. - 8 nos. (6.0m x 5.4 m)
3. F.S.L. of Canal at Head - 75.50 m
4. Bed width and F.S.D. at head - 21.45 m, 4.30 m
5. Bed leve of canl at head - 71.20 m
6. Free Board - 0.9 m
7. C.C.A. - 1,14,300 ha

VI. Details of C.C.A. (h.a)

	Gravity	Lift	Total (h.a)
L.B.C. 0 to 30 km	8483	3674	12197
LBC 30 to 141 km	93501	8642	102148
Total	101984	12316	114300

VII. Intensity of Irrigation

- a) Annual - 161 %
- b) Kharif - 93 %
- c) Rabi - 53 %
- d) Summer - 8 %
- e) Perennial - 7 %

VIII. Cost

R.I.P. (Ph.I) Original	Rs. 233.64 cr (P.C., GoI, Dt.31.3.78)
LBC 0 to 30 km Revised	Rs.173.53 Cr.
L.B.C. 30 to 141 km Original	Rs.705.15 Cr. (P.C., GoI, Dt.14.07.1997)
L.B.C. 30 to 141 km Revised	Rs.1958.34 Cr. (P.C., GoI, Dt.14.09.2010) (At 2010 price level)
R.B.C. 0 to 95 km Original	Rs.69.64 Cr. (A.A., GoO No.23281/05.08.81)
Revised	Rs.2028.82 Cr. (At 2010 price level)

CHAPTER - IV

MODEL STUDY, DESIGN AND QUALITY CONTROL

4.1 Model Study

4.1.1 Model Study of Rengali Dam.

Model testing on Rengali dam was conducted to determine the hydraulic behaviour of dam components like spillway, right training wall, left training wall, divide wall, construction sluice, bell mouth entrance and gate for penstock etc.

Mobile bed was reproduced on the model as per the river bed course upto a distance of 300m on both up stream and down stream from dam axis for the entire width of the dam. The mobile bed consisted of 16 mm mean size metal representing approximately 5T boulder of the proto-equivalent. Studies were also conducted on a 1/50 geometrically similar (G.S) scale sectional model of the spillway incorporating the lower bucket at invert EL.83.50 m. having a radius of 18m. and lip angle of 35°.

The summary of test results are produced below:

4.1.1.1 Spillway & Ski-Jump bucket.

These studies were conducted for different outflow discharges ranging from 3000 cumec to 47000 cumec assuming 22 out of 24 spans operative.

From the study, it is observed that clear margin of about 3m for lower discharge and about 6.5m for higher discharge between the lower limit curve was observed for ski-action. This could indicate that for the lower bucket at invert EL 83.50 m, a ski-action could be expected to prevail over the entire range of discharges with tail water levels as per the guage discharge curve.

It could also be surmised from the above results that the higher bucket at invert EL.87.25m would perform satisfactorily as a ski- jump bucket for the entire range of discharges.

Discharge capacity of the spillway with free flow over 22 out of 24 spillway spans (2 end spans assumed inoperative) indicated that a discharge of 47,550 cumec could be passed as against design discharge of 46,970 cumec. The reservoir water level for the design discharge was observed at EL.125.30 m. When all the 24 spans are freely overflowing the design discharge of 46,970 cumec could be passed at reservoir water level of EL. 124.70 m.

4.1.1.2 Pressure along the Spillway Profile:

Tests were conducted for design discharge of 46970 cumec passing through 22 spans, under free over flow condition. It was observed that the pressure along the centre line of bay and along the side of pier were negative along the entire crest profile and were positive all along the rear slope and bucket region. The maximum negative pressure of the order of 1.5 m and 2.8 m along the centre line of bay and along side of pier respectively, just downstream of the crest should be attributed to the under designing of the crest profile. The crest profile conforming to equation $x^{1.85} = 16.532y$, is designed for a depth of overflow of 15.2 m. Acceptability of maximum negative pressure of the order of 2.8 m may have to be examined from considerations of frequency of occurrence of design discharge.

Discussion

The development of negative head is due to the under designing of spillway crest, which no doubt increases the co-efficient of discharge, but it is injurious to the safety of the structure. However from the detailed model study, it was ascertained that no negative head develops for a discharge of 30,800 cusec (872 cumec). As such project authorities were of opinion to keep the original section of

spillway, since the probability of occurrence of 46,970 cumec discharge is very remote for which negative head develops. However C.W.C. was reluctant to allow the section. As such a comparative study was made about the various spillways of major dams which have been built with negative head and can be seen in Table - 4.1.

On scrutiny of the above, C.M.D.D. Directorate communicated their acceptance in letter No.3/130/76- C.M.D.D./4019 of 9th December, 1976.

“Hydraulic model studies for Rengali dam spillway has shown negative pressure of the order of 1.8 m and 2.8 m along the central line of bay and along the side of the pier respectively”. These negative pressures are allowed subject to good workmanship, good quality of material and necessary precaution in achieving accurate alignment and evenness of the surface of the crest.

Finish to be adopted for Rengali dam spillway may be F_4 or U_3 which have been briefly described as under.

FINISH “F4”

This finish is required for formed concrete surfaces at the spillway crest, glacis and bucket and inside sluices where accurate alignment and evenness of surface are essential for prevention of destructive effects of water action. The forms must be strong and held rigidly and accurately to the prescribed alignment. For warped surface, the forms shall be built up in sections cut to make tight, smooth form surfaces after which the form surfaces are dressed and sanded to the required curvature.

When measured as described in this clause abrupt irregularities shall not exceed 0.5 cm (or $\frac{1}{4}$ in) for irregularities parallel to the direction of flow, and 0.25 cm (or $\frac{1}{8}$ in) for irregularities in other direction. Gradual irregularities shall not exceed 0.5 cm (or $\frac{1}{4}$ in). Irregularities exceeding these limits shall be reduced by grinding on a bevel of 1 to 20 ratio of height to length. The formation of air holes on the surface of concrete designated to receive F4 finish shall be minimised and where such air holes are found, they shall be repaired in accordance with para 4.15 of IS 457-1957.

FINISH “U3” (Trowelled finish)

“Shall apply to unformed surfaces, such as slabs to be covered with built up roofing or membrane water-proofing and stair treads. When the floated surface has hardened sufficiently to prevent excess of the material from being drawn to the surface, steel trowelling shall be started. Steel trowelling shall be performed with firm pressure, so as to flatten the sandy texture of the floated surface and produce a dense uniform surface, free from blemishes and trowel marks: light steel trowelling will be permissible on surfaces of slabs to be covered with built up roofing or membrane waterproofing, in which light trowel marks are not considered objectionable. Surface irregularities measured as described under 4.16.1 shall not exceed 0.5 cm (or $\frac{1}{4}$ in)” (Refer IS 457- 1957 pg - 29)

4.1.1.3 - Left Training Wall

It was decided to conduct studies of 90 m long training wall in its location having its top level at EI.110 m. Water profile, scour profile and water surface fluctuation near the power dam for the maximum discharge of 46,970 cumec passed through all the 24 spans and with keeping the left and right end spans in closed positions were observed.

It was noticed that the Ski - trajectory was well contained within the boundary of left training wall without over topping. The top elevation of the left training wall was arbitrarily kept at EI.110 m in the model. The height of the training wall could be modified in different reaches considering the maximum

Table - 4.1
Pressure developed on the Spillway profile of few Major Dams (India)

Sl. No.	Name of the Dam	Equation of Ogee	% of maxm.head	Pressure(m)	Discharge (cumec)	Remark
1.	Tenughat	$x^{1.85} = 16.75 y$ $x^{1.85} = 18.53 y$	75 85	(-) 4.27 (-) 3.35	19765 19085	Crest overhanging reduced the negative pressure
2.	Jawahar sagar	$x^2 = 114 y$				
3.	Srisaillam	$x^{1.85} = 14.33 y^{0.85}$ $x^2 = 41-15y$ $x^2 = 38.10 y$ with over hanging crest	76	(-) 1.6 C.L of bay (-) 2.9 side of pier (-) 0.7 at FRL centre line of bay (-) 0.8 at MWL on side of pier		Maximum negative pressure of the order of (-) 1.0 m along the spillway profile were acceptable
4.	Kadra Dam (Gujurat)	$y = 0.0511 x^{1.6}$		(-) 1.5 at centre line of bay		
5.	Jayawadi Dam					No negative Head
6.	Twax Dam					No negative Head
7.	Ukai Dam			At FRL 105.13m (+) head At MWL 106.80m (-) 1.5 on centre (-) 2.44 on side of pier		Maximum flood 35966 cumec Spillway Discharge 34380 cumec



OVER FLOW SECTION

CWPRS,PUNE

HYDRAULIC MODEL STUDIES FOR RENGALI
SPILLWAY

WATER AND SCOUR PROFILE ALONG THE
RIGHT AND LEFT TRAINING WALL.

water surface elevations and giving due consideration the maximum water surface elevations and giving due consideration for bulkage effect due to air entrainment in the proto type. The flow from the spillway was guided downstream by the 90 m long training wall with weak return flow at the flank. The water pool elevations at rear side of the wall and downstream of power dam was at EI.94.31 m. The average maximum water level fluctuation at the rear side of training wall and at power dam location were of the order of 0.6 m.

It was also seen that the deepest scour level along the left training wall reached EI.77.50 m. It is understood that the rock level along the left training wall is $\pm$ EI.80 m. As the Jet trajectory in the pro-type would impinge at the shorter distance than that observed in the model and as the length of the trajectory would vary for different discharges, it would be desirable to take the foundation level of the left training wall below EI. 77.50 m.

4.1.1.4 Right Training Wall

It was observed that the downstream water level in front of the right non-overflow section at toe was boosted upto EI. 101.0 m. and the return flow spills over the existing top (EI. 95.0 m.) of the training wall in the bucket portion and suppresses the falling jet from the spillway resulting a disturbed trajectory of the ski-jump bucket. The suppressed trajectory, however was found to over top the training wall which is at EI. 95.0 m in the river bed portion. The flow condition by the side of the training wall is observed. The maximum water level of the trajectory was at EI. 106.50 m. The maximum water level of the waves downstream on the training wall was found at EI. 105.0 m. The maximum scour upto EI. 88 m was found to occur at a location 118 m from the dam axis. Water and scour profile along the left & right training wall is shown in Drg. No- 4.1.

4.1.1.5 Divide Wall

The water profile was observed by the side of the divide wall between span Nos. 14 and 15 at lower and higher bucket. The maximum water level over the higher bucket invert was at EI.95 m. and also the water level of the trajectory over the end of divide wall at EI.98.50 m. Hence it is requested to raise this divide wall upto EI.100 m to provide sufficient free board. The result of the model studies indicated the necessity of raising the divide wall and training walls.

The right training wall in the bucket portion would be raised upto EI. 102.75 m to avoid the over topping from the rear side by return flow. The training wall in the river bed would be raised to EI.108 m to contain the overriding of the Jet and over topping from the downstream flow.

The top elevation of divide wall would be increased to EI. 100 m from existing EI. 95.0m.

4.1.1.6 Penstock Bellmouth curve and Gate groove

Results of the model studies in respect of location and size of air vent, shape of gate grooves, shape of bellmouth curve, bottom shape of gate and hydrodynamic forces on the gate are furnished as follows :-

(a) Location and size of Air Vent:

Tests were carried out by measuring velocity of air through the air vent for different penstock gate openings for the maximum reservoir water level of elevation 125.40 m. The corresponding demand ratio $B = Q_a/Q_w$ (Q_a - air discharge, Q_w = Water discharge) is studied. It indicates a peak air demand at a gate opening of 12.5 % and for this condition , the model air demand ratio B, is 0.2605.

Based on the C.W.P.R.S model and field experience an appropriate scale factor was adopted to arrive at the probable proto type B value. The prto-type B value thus worked out was 0.3126. Considering a prototype water discharge of 81 m³/sec. for the maximum water level EI. 125.40 m at

12.5 % gate opening and assuming an allowable velocity of air through the air vent of 50 m/sec., the diameter of air vent for the penstock gate worked out to 0.80 m. Thus for the penstock gate, the proposed air vent of diameter 1.0 m with its centre line at a horizontal distance of 0.905 m from downstream edge of the penstock gate groove is adequate.

(b) Shape of Gate Groove:

The shape of penstock gate groove as provided in is hydraulically satisfactory.

(c) Shape of Bellmouth Curve:

The pressure distribution on the bellmouth curve at reservoir water levels of M.W.L EI.125.40 m. and MDDL (Maximum draw down level) of EI.109.75 m are studied . It is seen that the pressure are all positive and hence the shape of bellmouth curve is hydraulically satisfactory.

(d) Bottom Shape of Gate:

The average pressure below the bottom most girder of the penstock gate for different opening at reservoir water level of MWL EI. 125.40 m is observed . It is seen that the pressure are all positive and hence the location and size of the bottom most girder is hydraulically satisfactory.

(e) Hydro-dynamic Force on Gate:

Tests for measurement of hydro-dynamic forces on the penstock gate, for different gate openings for the maximum reservoir water level at EI. 125.40 m indicated that the penstock gate experiences maximum net hydrodynamic uplift and down pull forces of the order of 13 tonnes and 16 tonnes, respectively during its operation. Allowing for safety values of maximum net hydro-dynamic uplift and down pull forces of 15 tonnes and 18 tonnes respectively may be considered for the design of the hydraulic hoist.

4.1.1.7 Construction Sluice: (CWPRS- Specific Note No.1858/10.09.1979)

(1:20 Geometrically similar composite model)

During construction stage of the dam, the river discharges would be passed through two construction sluices of size 3.5 m x 4 m with their sills at EI 78.0m located in spillway block No.21, in addition to overflow on partly constructed spillway block No. 22 to 25, with their top elevation at EI. 82.00 m,

While the model studies were in progress, the Chief Construction Engineer, Rengali Project requested that the performance of sluices be studied with upstream water level ranging from EI.84.0 m to EI 120.0 m with corresponding tail water levels ranging from EI. 78.50 m to 92.90 m. Accordingly, the studies were conducted for the stipulated range of upstream and downstream water levels as given below.

Upstream Water Level (m)	Tail water Level (m)
84.0	78.50
93.0	
100.0	
103.0	
110.0	
120.0	92.90

4.1.1.8 Conclusion:

i) Discharging Capacity of Sluice:

Discharging capacity of the two constructed sluices was studied for various upstream water levels and corresponding tailwater levels as stated earlier. It was observed that the sluices would flow full at upstream water elevation of 84.0 m and above. At the upstream water level of EI. 95.0 m and the corresponding tailwater level at EI. 92.90 m, the combined discharge through the two sluices was observed to be of the order of 300 cumec and about 620 cumec for maximum reservoir level of EI. 120 m and tailwater level of EI.92.90 m.

ii) Pressure on Sluice and Floor of Stilling Basin:

For the bellmouth entry, curveture has been provided only on the top of the entrance. The maximum negative pressure of the order of 3.5 m on the top of the bell mouth at reservoir water level of EI. 120.0 m was considered acceptable in view of utility of two construction sluices only for 3 to 4 years. However, the pressure on barrel and floor of the stilling basing were positive throughout. A rich mixture of concrete may be used for top of bellmouth entrance.

iii) Performance of the Stilling basin:

The performance of the stilling basin was typical of low Froude number involving less energy dissipation and surface undulations. It is mentioned in the project report that sound rock exists in the spillway region. The model studies have indicated qualitatively that the maximum scour of about 3.75 m below the natural ground level of EI.78.0 m is likely to occur at a distance of about 15 m from the end sill of apron when both the sluices would discharge under reservoir water level of 120.0 m with corresponding TWL of 92.90 m. The bed velocity at the end of stilling basin was observed to be of the order of 2 to 3.5 m/sec for reservoir water levels varying from EI.93.0 to EI.120 m. Hence the foundation of the stilling basin end is not likely to be affected during the diversion stage.

No air vent was required to be provided in the body of the sluice as the same is non-regulating one.

4.1.2 Model Study of Samal Barrage

4.1.2.1 Introduction:

Hydraulic model studies of Samal Barrage were conducted both at Hydraulic Research Division, Burla at Sailendra Narayan Engineering Research Institute (SNERI) as well as at Central Water and Power Research Station, (CWPRS), Pune.

At SNERI, The studies were conducted in a smaller rigid bed model and a larger mobile bed model. In the smaller bed model studies on various alternatives for the layout of different components of the barrage on river training works were taken up in addition to, obtaining stage-discharge relationship at various locations of the river for subsequent utilisation in the mobile bed model. Both the models were utilised for finding stage-discharge relationship for the river, afflux due to the barrage, the best possible layout of various components of the barrage including river-training and bank protection works, the nature of silting of the pond, the effect of the barrage on the regime of the river flow downstream and the ideal operation schedule for the pond.

For rigid and mobile bed models following scales were adopted :

- i) Rigid bed -
Horizontal 1:1000, Vertical 1:100
- ii) Mobile bed -
Horizontal 1:250, Vertical 1:50
- iii) Sectional model - 1:50 G.S. model

After considering five different sites, Samal Site was finally selected for the construction of the barrage vide Sec.2.2.2.2. The length of the barrage, relative positions of the two abutments, head regulators and other details have been finalised on the basis of model studies conducted at SNERI, Burla.

The best arrangement for various components of the barrage should be such that it should meet the following requirements.

1) For about 75 days in monsoon, the flow into the pond is just sufficient to meet the irrigation requirement. Therefore silt entering into the pond during this period necessarily have to enter the canals or get deposited in the pond. It is better if minimum quantity of silt enters the irrigation system and the rest is deposited inside the pond. The arrangement should be such that during the rest 45 days of monsoon, when there is some release to the river downstream, the silt deposited in the pond gets removed.

In contrast to the meandering reaches of shallow and alluvial rivers, the river at the barrage site is deep and confined within the stable banks. As such Lacey's formula can not be applied for determining the waterway. The river, sufficiently below the barrage site during the passage of design flood will have discharge per metre(q) i.e intensity of discharge of $24632/520 = 47.4$ cumec. The waterway at the barrage site should be such that the ' q ' at the downstream side does not exceed the above carrying capacity of the river. Thus the waterway of the barrage will be close to the present river width of 520 m as per I.S. 6966/1973. Basing on this the width of various components of the barrage were worked out and finally fixed as follows:

Sl No	Components	No	Width of each(m)	Total Width(m)
1	a) Left under sluice including excluder	4	20.00	80.00
	b) Piers	3	3.50	10.50
	c) Divide Wall	1	3.50	3.50
2.	a) Barrage bays	17	20.00	340.00
	b) Piers	16	3.50	56.00
3.	a) Right under sluice	3	20.00	60.00
	b) Piers	2	3.50	7.00
	c) Divide Wall	1	3.50	3.50
Total				560.50m

However due to adverse site conditions at the left head regulator, a second opinion and suggestions for improvement of flow conditions for efficient sand silt exclusion were sought from CWPRS Pune. Accordingly Terms of reference were framed.

Terms of reference made to C.W.P.R.S Pune :

Two and three dimensional model studies were proposed by the project authority to enlighten on the following six aspects.

- i). Improvement in approach condition.
- ii). Extent of silt likely to enter into left canal and remedial measures to be taken up.
- iii). Length and alignment of left and right upstream divide walls.
- iv). Location and alignment of left afflux bunds
- v). Discharging capacity and energy dissipation of the barrage and
- vi). Operation of the barrage, sequence of operation of gates including undersluice gates with view to minimise silt entry into canal, maximum discharging capacity of head regulators and prevention of island on the upstream side of the reservoir.

Accordingly project authorities supplied the 1979 survey river cross- sections at an interval of 300m from Durgapur guage site to Bilinda guage site covering a distance of 14 km extending on either bank upto HFL, G-Q data at Samal site observed by CWC and the project from 1974 to 1981 and guage data from 1974 to 1981 at Talpada, Durgapur, Belpada and Dharmpur guage stations besides guage data from 1978 to 1981 at Bilinda site. Sediment data from 1973 to 1982 were also made available.

Basing on 1979 survey data, the model was constructed covering 8.4 km upstream and 2.05 km downstream of barrage axis in the following scale, :

Length Scale - 1:350

Vertical Scale - 1:70

Discharge Scale - 1:2, 04, 982

Vertical exaggeration - 5

4.1.2.2. Analysis of G-Q data by CWPRS (Specific Note 2370 Dt 31.10.1986)

The G-Q data collected by CWC and by the State authorities were initially analysed with acceptable and reliable figures.

Statistical analysis was attempted for guage discharge data pertaining to years 1974 to 1981. The G-Q equation, standard error and corelation coefficient are given below.

Table- 4.2 G-Q Equation & Standard error

Years	Gauge Site	G-Q Equation	Standard Error	Corelation Coefficient
1975-78	Durgapur	$G = 65.00 + 0.494 \times Q^{(0.3456)}$	0.813	0.883
1972-81	Samal	$G = 59.50 + 1.4671 \times Q^{(0.2352)}$	0.346	0.963
1978	Bilinda	$G = 63.50 + 0.0305 \times Q^{(0.6423)}$	0.882	0.747

Proving studies by CWPRS:

For carrying out the proving studies, the flood frequency analysis were required to be done and as these data were not readily available, the analysis was carried out at the CWPRS for Talcher site for which long time record is available. The results of the analysis made are given below.

Sl No.	Return Period	Discharge stage (cumec)
1	Yearly	2,800
2	5 Years	4,250
3	10 Years	5,100
4	20 Years	6,500
5	100 Years	29,675

Simultaneous guage - discharge data available at Durgapur, Samal and Bilinda sites for river stages of 6,000 cumec , 10,000 cumec and 17,000 cumec have now been utilised for proving the model as these discharge stages represent low flood, bankful and above.

The water levels at the tail end of the river were controlled according to the guage at Bilinda guage site. The water levels obtained at other guage stations namely Samal and Durgapur were recorded. It was seen that the water levels obtained on the model were within 95 % confidence limit for the guage sites and hence considered satisfactory and the model was taken as proved.

4.1.2.3 Model Experement for Pre-Barrage Condition by CWPRS:

The barrage axis was already fixed at site and the CWPRS was required to carryout studies for this pre-determined site and axis. The river section along the barrage axis was divided into 5 segments for purpose of model experiments ,each segment measuring 105 m representing about 5 notional barrage bays. At each segment , the obliquity of flow, depth and velocity measurements were carried out for river stages ranging from 3,500 cumec to 24,000cumec.

It was observed that the obliquity of flow was ranging from 0 to 3 degrees which indicated that it was within permissible limits.

It was also observed that the left bank segments 1, 2 and 3 are coming under the flood plain and are carrying less percentage of discharge even at flood stages of 17,000 cumec and 24,000cumec. The barrage actually occupies segments 4 to 8 and it can be seen that left bank segment adjoining the location point of left abutment was carrying minimum percentage of discharge compared to other segment . For instance, at a river stage of 3,500cumec the segment adjoining the location of left abutment was found carrying 2.37% which accounts for 80cumec while the requirement of the left bank canal is of the order of 260cumec . This means that the left bank canal is required to draw its share of discharge from an unfavourable curvature of flow to meet its requirement after maintaining necessary pond . This showed that the unfavourable curvature of flow need to be reversed for satisfactory functioning of the left bank canal .

The percentage of discharge carried by the segments adjoining the right bank was quite high even at the lowest stage of 3.500cumec .This showed that the present river configuration is favourable to the right bank canal to meet its requirements .

4.1.2.4 Model Experiments with Barrage in Position:

The barrage proposal as given by the project was adopted on the model . Right and left canal head regulators were also reproduced as per the drawings made available. Preliminary studies were carried out without resorting to ponding to assess the curvature of flow obtainable under post-barrage condition.

It could be seen that the left bank undersluice bays 1 to 4 was carrying 6.35% to 11.79% of discharge for river stages ranging from 10,000cumes to 24,000cumes. This was against its legitimate share around 18.8% if the approach conditions were normal to the barrage axis. In respect of right bank undersluices, three undersluice bays were mostly carrying around its legitimate share.

Uncontrolled barrage operation on the model indicated that the barrage was able to pass a flood discharge of 30000 cumec when the water level upstream of barrage was at RL 76.2 m with all barrage gates thrown open . This level corresponded to the design pond level of the barrage, This discharge stage of 30000 cumec is actually higher than the designed discharge of the barrage. Since the river flow of Brahmani is not likely to reach this magnitude, the barrage will have to be operated under gated flow condition throughout the year. Hydrological analysis of river flow for the period 1975-80 indicates that the river will carry higher than 5000 cumec only for three days in a year on an average as can be seen from the table given below :

Hydrological Data of River Brahmani - (1975 to 1980)

Discharge (cumec)	Total days	No. of days for a year
2000 and above	107	17.83
3000 and above	56	9.83
4000 and above	35	5.83
5000 and above	19	3.16

The above table also indicates that in a year the river flow remains above a river stage of 2000 cumec only for about 18 days. This means, practically throughout the year the river is likely to draw much less than 2000 cumec which may result in siltation of the pond especially near the left bank canal due to adverse flow coupled with high ponding. This may perhaps lead to a situation where the entire flow of the river may have to be forced to flow through the left and right bank under- sluices only keeping the spillway bays fully throttled in order to feed the two canals. This type of regulation if followed would not only hasten the process of siltation of the pond but also induce formation of island in front of the spillway bays. This may happen in spite of the fact that major part of the sediment load is retained at Rengali dam. This would indicate that operation of the barrage under flow flood stages with high pond maintained would become the major problem to be studied on the model.

4.1.2.5 Model Studies with Pond level maintained:

It has been observed that a pond level of RL 76.2m which corresponds to a river stage of 30,000cumec will have to be maintained almost throughout the year to meet the requirement of both left and right canal .To meet these requirements the barrage bays were required to be regulated . The regulation obtained at river stage of 2,000 cumec ,3,500 cumec ,6,000cumec ,10,000cumec and 17,000cumec with pond level maintained at RL 76.2m have been shown in Table No-4.3 From a perusal of this table, it would be seen that a river stage of 2,000cumec all the spillways of the barrage are required to be completely throttled and the left and right under-sluices are required to be opened by about 1.4 m uniformly against a ponding depth of 10.2 m to meet the requirement of the two canals. Similar condition exists with greater opening of the gates of the under-sluices up to a river stage of 6,000cumec .It has already been pointed out in the earlier paragraphs that the river remains above a

river stage of 5,000 cumec for not more than three days in a year and thus the river stage up to 6,000 cumec assumes greater relevance from the points of view of meeting the requirements of the two head regulators on either side.

Model studies carried out with the above regulation had indicated very unfavourable curvature of flow in front of left bank head regulator which may ultimately lead to heavy sediment load entering the left bank canal. However this regulation was required to be adopted to meet the discharge requirement of the left bank canal as otherwise there was no other way to meet the left bank canal's legitimate demand.

The initial model experiments carried out revealed that in order to meet the discharge requirement especially of the left bank canal and at the same time achieve favourable curvature of flow of efficient sediment exclusion from the canals, the under-sluice bays need to be separated from the spillway bays and still pond regulation need to be resorted to, to the extent permissible. It was also found necessary to segregate the pocket from the spillway bays in order to ensure efficient and quick flushing of the pocket which is likely to get silted up early due to still pond regulation.

It has been seen that except for opening of 1.4 m against a ponding depth of 10.2m of the undersluices the entire barrage bays remain fully throttled at a river stage of 2,000cumec. This would not only hasten the process of siltation of the pond but also render the undersluices ineffective for flushing operations. This, may eventually lead to heavy silt entry into the two canals on the either side. Similar condition exists even at river stages of 3,000 cumec and 6,000 cumec.

Many advantages can be achieved by introducing divide walls on either side. The advantages of a divide wall are manifold; firstly it helps in providing favourable curvature of flow for sediment free water entering the canal where natural favourable curvature of flow does not exist. Secondly, it gives room for flexibility in operation of the gates to maintain favourable V_R/V_p ratio for efficient sediment exclusion. (N.B:- V_R = Velocity of flow in barrage bays and V_p = Velocity of flow in sluice bays). Thirdly, it helps in efficient flushing of the pocket sluices by segregating the undersluices from barrage bays. Lastly, it helps in avoiding parallel flow during the course of barrage operation.

It was felt that two divide walls, one on the right side and the other on the left side are required to be provided and initial model studies carried out with the two divide walls in position have shown much improvement in the flow condition obtainable especially on the left side where curvature of flow is most unfavourable for efficient sediment exclusion.

During the period of initial model studies, it was observed that high cut-offs across the barrage are available even during the monsoon seasons. The following table would reveal the extent of cut-offs available at various stages of flow:

Sl. No.	Discharge in cumec	Pond Level RL in m	Downstream River Water Level at Samal Barrage RL in m.	Depth of cut-off in m.
1.	2000	76.20	68.26	7.94
2.	3500	76.20	69.49	6.71
3.	6000	76.20	70.80	5.40
4.	10000	76.20	73.06	3.14
5.	17000	76.20	74.11	2.09

This data suggest that silt excluder tunnels if introduced would work very efficiently due to availability of adequate driving head at all the river stages. It is therefore suggested that one bay each on either side may be provided with excluder tunnels.

4.1.2.6 Experimental Studies with Divide Walls and Silt Excluders:

Detailed model studies with still pond regulations were carried out with divide walls and silt excluders independently as well as in combinations. This was experimented on either side of the barrage. Following four cases were examined for discharge stages ranging from 2000 cumec to 17000 cumec.

- i) Project proposal i.e. without divide wall and excluders
- ii) With divide wall on either side
- iii) With only silt excluders on either side
- iv) With divide walls and silt excluders on either side.

Table No-4.4 shows the results of the experiments with the divide walls and silt excluders on both sides of the barrage in position. The results favourably indicated curvature of flow, good stilling condition in the pocket, particularly for low discharges. This substantiated qualitatively efficient silt exclusion.

Divide walls recommended by CWPRS on left and right side of the barrage was approved by T.A.C. of the Project. Accordingly detailed model studies were conducted and flow patterns along divide walls both U/S and D/S were observed. Differential head on divide walls for discharges ranging from 3500 cumec to 49000 cumec have been tabulated in Table No-4.5. Maximum differential head observed was 0.8m for the river stage of 30,000 cumec.

4.1.2.7 Alignment and height of Afflux bunds :

The alignments of the left and right upstream afflux bunds were reproduced on the model. Sharp bends and kinks were suitably modified by providing smooth curves. The alignments were examined for the flood stages ranging from 24,000 cumec to 49,000 cumec when all the gates of the barrage were thrown open. It was observed that there was no active flow along the left afflux bund during the flood stages examined and the maximum velocity observed was of the order of 1 m/s. This was primarily because the alignment was sufficiently away from the main channel of the river. In view of low velocities observed along the alignment, the bund does not need any protective measure for the present expect turbing along the slope for wave wash. However, a watch need to be kept during floods and if need arises some protection by way of stone pitching on the slope and launching apron at the toe may be provided.

In the case of right afflux bund also, the alignment was modified to avoid sharp bend. As some part of this alignment is in the proximity of the main channel, higher velocities were observed in some reaches. The maximum velocity observed was of the order of 2.5 m/s at a river stage of 24,000 cumec. At higher stages of 30,000 cumec and 49, 000 cumec, velocity of 3 to 4 m/s was observed. Keeping in view that the concave curvature of the river is on the right side, it is recommended that the river side of the entire alignment of the guide bund be adequately protected by way of slope pitching and launching apron for the toe protection as per prevailing standard code of practice based on the velocity range observed on the model as indicated above.

The water levels were recorded on the model along the two afflux bunds at maximum design stage of 49,000 cumec. The top level of the afflux bunds in various reaches may be kept ensuring adequate free board as per the standard code of practice over and above the water levels.

Table- 4.3 - % Distribution of Discharge in various bays under Ponded conditions

Sl.No.	Particulars	Discharge in cumec						Remark
		2,000	3,500	6,000	10,000	17,000		
1	Left undersluice 1 to 4	50.30	51.02	58.61	12.82	9.40		Pond level = 76.2 m was maintained with pond maintained, the two head regulators were found carrying the designed discharges.
2.	Barrage bays 5 to 21	---	---	----	69.98	77.70		
3.	Right undersluice 22 to 24	30.70	38.12	35.06	13.40	10.70		
4.	Left main canal (LMC)	260 (13%)	260 (7.43%)	260 (4.33%)	260 (2.60%)	260 (1.53%)		Undersluice crest level = 66.0 m Barrage bay crest level = 67.0 m Crest level of canal head regulator, Left = 69.5 m
5.	Right main canal (RMC)	120 (6%)	120 (3.43%)	120 (2%)	120 (1.20%)	120 (0.70%)		Crest level of canal head regulator, Right = 71.2 m
6.	Left under sluice opening	1.40 m	2.10 m	6.20 m	10.20 m	10.20 m		Right canal discharge = 111 cumec
7.	Barrage gate opening	---	---	---	5.81 m	6.65 m		Left canal discharge = 258/216 cumec
8.	Right under sluice gate opening	1.40 m	2.94 m	4.20 m	10.20 m	10.20 m		

Samal Barrage Model Studies
Table - 4.4 Still pond Regulation with Silt excluder and Divide Walls

Sl. No	Discharge in cumec	Water level in m		Gate Operation					Left Pocket			Right Pocket		
		U/S	D/S	Under sluice		Spillway			Velocities (m/s)			Velocities (m/s)		
				Span no.	Gate opening	Span No.	Gate opening		VR	VP	VR/VP	VR	VP	VR/VP
1	2000	76.2	68.26	Left pocket 1 2 to 4 Right pocket 24 22 to 24	2.97 m Nil 2.97 m Nil	5 to 7 8 to 9 20 to 21	1.00 m Nil 1.00 m		0.47	0.47	1.00	0.47	0.47	1.00
2.	3500	76.2	69.49	Left pocket 1 1 to 4 Right pocket 24 22 to 23	2.97 m Nil 2.97 m Nil	5 to 7 8 to 9 20 to 21	2.97 m NIL 2.97 m		0.652	0.525	1.3	0.725	0.577	1.26
3	6000	76.2	70.8	Left pocket 1 2 to 4 Right pocket 24 22 to 23	9.07 m NIL 5.07 m NIL	5 to 7 8 to 19 19 to 21	4.03 m NIL 4.03 m		1.246	0.954	1.3	1.414	1.1554	1.22
4	10000	76.2	73.06	Left pocket 1 2 to 4 Right pocket 24 22 to 23	4.37 m NIL 4.37 m NIL	5 to 9 10 to 17 18 to 21	6.20 m Nil 9.20 m		1.87	1.24	1.50	1.94	1.32	1.46
5	17000	76.2	74.11	Left pocket 1 2 to 4 Right pocket 24 22 to 23	4.20 m NIL 4.20 m NIL	5 to 21	9.20 m		2.75	1.74	1.50	3.5	2.53	1.38

Remarks :

- Clear opening of silt excluder = 2.97 m
- Silt excluder provided in bay No. 1

Table - 4.5 Observation on Divide Walls for Differential Head

Sl. No	Discharge cumec	Location	Water level (m) Left divide wall		Differential Head (m)	Water level (m) Right divide wall		Differential Head (m)	Remark
			Left under Sluice	Left Barrage side		Barrage Right	Right under sluice		
1.	3,500	Upstream Downstream	75.95 70.28	75.84 70.60	0.11 0.32	76.09 70.38	76.19 70.52	0.10 0.14	Still pond regulation pond level RL 76.20 m silt excluder in Bay No. 1 upto Q = 17,000 m ³ sec
2.	6,000	Upstream Downstream	76.16 70.07	75.79 65.77	0.37 0.30	75.81 70.28	76.19 69.98	0.38 0.30	Barrage bays 3 on left, 3 on right open, Gate opening 3m
3.	10,000	Upstream Downstream	76.02 73.07	75.60 73.89	0.42 0.82	75.46 73.07	76.09 72.68	0.63 0.39	Barrage bays 3 on left, 2 on right open, Gate opening full. Barrage bays 5 on left, 4 on right open, Gate opening full.
4.	17,000	Upstream Downstream	76.19 74.51	75.81 73.99	0.31 0.52	75.67 74.69	76.19 74.44	0.52 0.25	All barrage bays open.
5.	24,000	Upstream Downstream	76.72 75.95	77.14 76.09	0.42 0.14	77.21 76.33	77.38 76.12	0.17 0.21	All gates open. Level higher than Pond level.
6.	30,000	Upstream Downstream	77.31 76.37	77.34 76.47	0.43 0.10	76.55 76.47	77.35 75.92	0.80 0.55	-do-
7.	49,000	Upstream Downstream	79.10 78.12	79.55 78.68	0.45 0.56	78.75 77.90	79.45 77.63	0.70 0.27	-do-

4.1.2.8 Silt Excluder Tunnels:

Model studies with left excluder tunnels were carried out. According to this proposal one bay of the undersluices has been provided with six excluder tunnels of two tiers each. These six tunnels were represented in the model by two tunnels of two tier each. Performance of silt excluder was studied over discharge ranging from 2,000 cumec to 17,000 cumec which shows the discharge distribution in the left and right pocket bays and barrage bays as well as the velocities at the entrance of the excluder tunnels. It can be seen that with silt excluders none of the bed balls, dropped along the guide bank/abutment of the barrage, entered the canal. This indicated that satisfactory sediment exclusion was likely to be achieved as a result of provision of silt excluders. The suction effect was felt beyond the divide wall nose about 150 m upstream with all tunnels open even for a low discharge of 3,500 cumec. This indicated that the length of excluder tunnels provided was adequate. Model tests were also conducted to investigate about the possibility of occurrence of adverse conditions during the operation of the excluder tunnels. It was revealed that when excluder tunnels were closed or when ground floor/both tunnels were opened, no strong eddy was created in the pocket along the left bank upstream of the head regulators.

4.1.2.8.1 Shape of Silt Excluder Tunnels:

In irrigation works from consideration of construction, square or rectangular conduits are generally used. The studies indicated that rectangular conduit with width to depth ratio as 0.55 showed less head loss compared to other shapes. Accordingly for the present silt excluder tunnels, width to depth ratio worked out to 0.55 without the central slab. It is therefore recommended that the central slab provided in the tunnels may be eliminated from hydraulic efficiency consideration. In the composite model, studies for silt excluder were carried out with the central slab of the tunnels eliminated. Model observations were made. These are presented in Table-4.6. It presents a comparison of both the proposals namely with and without the central slab. It could be seen from this table that the velocities at the entrance of the tunnels were higher with central slab eliminated as proposed by the CWPRS indicating higher efficiency in the functioning of the silt excluders.

4.1.2.9 Conclusion and Recommendations:

The site of the barrage and all its parameters have already been decided by the project authorities on the basis of site inspection and model studies conducted at State Hydraulic Research Station, Burla. In fact, it was reported that construction of the barrage was also in good progress with right and left abutments and under sluices nearing completion. Therefore, it was realised before taking up necessary model studies that the CWPRS had very limited scope in effecting changes to meet the requirements of satisfactory hydraulic performance of the head works as enumerated in the terms of reference.

Model studies carried out under existing condition exhibited presence of sharp bend upstream of the barrage site. This had resulted in more than 60% of the flow getting concentrated on the right half of the barrage. For instance, for low discharge of 3,500 cumec, the river width representing left undersluices were found carrying about 2.4% of the flow i.e. 84 cumec which accounted for only 30% of the requirement of the left bank canal. This indicated that the left bank canal was trying to draw its requirement from an unfavourable curvature of flow. This may lead to heavy sediment load entering the left bank canal.

Model studies indicated that with the present arrangement, all the spillway bays may have to be closed completely to meet the requirements of the two canals. This type of regulation, if followed, might hasten the process of siltation of the pond with its added adverse results.

Model studies also indicated that ponding is required to be resorted to even at high flood stage of 30,000 cumec. This means, practically the barrage bays are required to be kept throttled throughout the year. This may also hasten the process of siltation of the pond.

Table - 4.6 Silt Excluder Tunnel
(Comparison of Result for CWPRS and Project Proposal)

Sl.	Discharge (cumec)	Water level (m)		Span Nos.	Gate opening (m)	Velocity m/sec				Remarks
		U/S	D/S			CWPRS Proposal		Project Proposal		
						Bay1	Bay2	Bay1	Bay2	
1.	3,500	76.20	69.49	5 to 7 19 to 21	2.45 2.45	0.88	1.35	0.70	0.70	Bay No.1 - Adjacent to HR. Bay No.2 - Adjacent to Bay-1
2.	6,000	76.20	70.80	5 to 7 19 to 21	4.9 4.9	1.04	1.44	0.51	1.12	
3.	10,000	76.20	73.06	5 to 9 18 to 21	9.2	0.63	0.93	0.62	0.62	
4.	17,000	76.20	74.11	5 to 21	9.2	0.64	0.60	0.37	0.40	

N.B. - CWPRS proposal was with central slab eliminated. Project proposal was with central slab existing.

In the initial project proposal there was no provision for flushing the undersluices quickly and efficiently and also to improve the existing unfavourable curvature of flow in front of the left bank head regulator. Model studies showed that with the introduction of two divide walls and proper barrage regulation, it was possible to achieve partly satisfactory sediment exclusion from the two canals and also for providing efficient flushing arrangement in the pocket sluices.

The model and prototype data indicated that sufficient driving head is available at all the river stages for introducing silt excluders in the system for more efficient sediment exclusion from the two canals.

Model studies showed that with the introduction of divide walls and silt excluder tunnels, favourable curvature of flow and good stilling conditions in the pockets were achieved. For low discharges, which will be the more frequent case for the Samal barrage operation, the silt excluder tunnels were found to work satisfactorily.

Silt excluder tunnels for the left undersluice bay as proposed by the project authorities have been found to be satisfactory. However, instead of two tier silt excluder tunnels, single tier tunnels with central slab eliminated were found to be more effective from sediment exclusion considerations.

Alignment of the left afflux bund on the upstream side as proposed was found satisfactory. The right afflux bund however needs protection by slope pitching as the velocities were of the order of 3 to 4 m/s along the alignment. The kinks in the alignment of the afflux bunds need also to be eliminated by smooth curvatures.

For low discharges upto 6,000 cumec, differential head on either sides of the divide walls was observed to be less than 0.5 m. For discharges ranging from 10,000 cumec to 49,000 cumec differential head of the order of 0.8 m was observed. These values may be kept in view in the design of divide wall as per standard code of practice.

4.1.2.10 Sectional Model Studies : (CWPRS Specific Note No. 2371 Dt. 04.11.1986)

Discharge intensities for sectional model studies were considered corresponding to the design discharge of 24,630 m³/s observed in the composite model, in under sluice bay and barrage bay together with the corresponding tail water level at 320 m downstream of the barrage axis. Lower discharges were also considered for investigation as shown in Table No.4.7.

Table- 4.7

Discharge stage (cumec)	Discharge intensity		Tail Water level (m)
	Left Undersluice (cumec / m)	Barrage (cumec/m)	
24,630	37.73	64.55	75.26
17,000	26.04	44.55	74.01
10,000	15.47	26.21	72.94
6,000	9.22	15.72	70.79
3,500	5.29	9.17	69.43

4.1.2.11 Model Experiments and Results :-

Undersluice Bays :

Observations were taken for water profile, submergence of trunion, displacement of concrete block etc. for gated and ungated conditions. Those data are presented in Table- 4.8 and 4.9. With the help of water levels, discharge coefficient for various conditions have been worked out and are presented in Table- 4.10. Method described in IS 6934-1973 has been used in working out this table.

Barrage bays:

Similar observations were carried out for experimental investigation of barrage bays. Results of these investigations are presented in Table- 4.11 and 4.12 and discharge coefficient obtained are shown in Table- 4.13.

Energy dissipation arrangements :

In project proposal for energy dissipation purpose, no stilling basin has been proposed. Only six rows of concrete blocks of size 1.5 X 1.5 X 1m. have been suggested since good rock is reported to be available at shallow depth; and at high floods difference in upstream and downstream water levels at barrage would be of the order of 1.25 m indicating that the energy dissipation problem is more confined to lower stages of the river.

In the sectional model studies of undersluice bays, concrete blocks were laid on mobile bed and performance was observed for various discharges under corresponding tail water levels. It was observed that for ungated conditions, blocks were not displaced for small discharges upto 3,500 cumec. However, for higher discharges, blocks were getting disturbed (vide Table-4.8). For gated conditions with pond level, for all discharges, blocks were getting disturbed. Results of observations are indicated in Table- 4.9. Similar observations were recorded for spillway bays in Table- 4.11 and 4.12. Blocks were getting disturbed for discharge of 3,500 m³/s and above.

Conclusion:

Average values of coefficient of discharge (Cd) obtained from the sectional model studies namely - 1.3 and 1.81 appears to be within allowable limits for such high submergence.

(vide Table No- 4.10 & 4.13 for ungated condition)

Flume studies indicated drowned jump. Therefore, concrete blocks are likely to be dislodged, which need to be monitored under maintenance.

It is seen from the two dimensional model studies that the energy dissipation was not satisfactory and therefore, conventional type of the arrangement of stilling basin of length 70 m at El. 59.8m may be required. However, since good rock is reported to be available at El. 55 m project authorities have dispensed with the conventional type of stilling basin and have proposed six rows of concrete blocks. It is however, desirable to provide 15 m wide and 1m thick concrete aprons to take care of the shooting flow that may occur at the toe end of the barrage. This concrete apron need to be properly anchored to the foundation rock. Concrete blocks proposed in the project design may be provided downstream of this apron. It would be essential to monitor this arrangement during and after every flood release so that timely measures can be taken in case small damages occur.

Table No- 4.8
1:55 G-S Sectional Model for left Undersluice bay of Samal Barrage on Brahmani River (Odisha)
(A) Ungated Condition
Water Level Observations (in m.)

SN	Location	Discharge intensities corresponding to river stages of				
		24,630	17,000	10,000	6,000	3,500 cumec
1.	184 m u/s of barrage axis	76.42	74.99	73.36	71.35	69.61
2.	138 m u/s	76.54	74.74	73.42	71.61	69.52
3.	44 m u/s	76.57	74.83	73.39	71.50	69.57
4.	22 m u/s	76.53	74.86	73.39	71.47	69.69
5.	11 M u/s	76.44	74.77	73.26	71.44	69.69
6.	Axis of barrage	75.69	74.28	73.06	71.22	69.30
7.	11 m d/s of barrage (Trunnion)	74.93	74.30	73.04	71.22	69.30
8.	22 m d/s of barrage	75.18	73.92	73.15	71.22	69.30
9.	55 m d/s of barrage	74.49	74.17	73.26	71.18	69.10
10.	99 m d/s of barrage	75.63	74.14	73.04	71.06	68.97
11.	220 m d/s of barrage	75.45	74.19	73.04	70.92	68.97
12.	319 m d/s of barrage	75.40	74.01	72.99	70.79	69.43
13.	Trunnion (R.L. 74.0 m) whether submerged or not	Submerged	Submerged	Not Submerged	Not Submerged	Not Submerged
14.	C.C. blocks disturbed	Yes	Yes	Yes	Yes	No
15.	Distance of hydraulic jump from axis in m	11.5 m	10.80 m	10.20 m	9.2 m	6.5 m

Table No- 4.9
1:55 G.S Sectional Model for under sluice bay of Samal Barrage on
Brahmani River (Odisha)
Water Level Observations (in m.), Discharge in Cumec

(B) Gated Condition

Sl.No.	Location	Discharge intensities corresponding to river stages of				
		24,630	17,000	10,000	6,000	3,500
1.	184 m u/s of barrage axis	76.42	76.34	76.42	76.31	78.42
2.	138 m u/s	76.20	76.20	76.20	76.29	76.35
3.	44 m u/s	76.20	76.20	76.20	76.20	76.19
4.	22 m u/s	76.20	76.20	76.20	76.20	76.19
5.	11 m d/s of barrage	76.20	76.20	76.20	76.18	76.20
6.	Axis of barrage	76.20	76.20	76.20	76.23	76.20
7.	11 m d/s of barrage	74.80	73.32	72.32	70.68	68.75
8.	22 m d/s of barrage	74.99	73.78	72.27	70.68	68.94
9.	55 m d/s of barrage	75.24	73.73	72.24	70.12	69.28
10.	99 m u/s of barrage	75.24	73.26	72.27	70.23	69.08
11.	220 m d/s of barrage	75.24	73.56	72.57	70.45	69.00
12.	319 m d/s of barrage	75.40	74.01	72.98	70.78	69.43
13.	Trunnion (R.L 74.00 m) whether submerged or not	Yes	Yes	No	No	No
14.	C.C. Blocks disturbed	Yes	Yes	Yes	Yes	Yes
15.	Gate opening in m	7.98 m	5.23 m	2.47 m	1.38 m	0.94 m
16.	Distance of hydraulic jump from axis in m.	15.6 m	14.2 m	11.9 m	11.7 m	11.2 m

Table No-4.10
1:55 G.S Sectional Model of Undersluice of Samal Barrage
Computation of Co-efficient of discharge (Cd)

Sl. No.	Discharge intensity in m ³ /s/m	Water level in m		Gate opening (m)	Depth of flow upstream above crest R.L. H1 in m	Depth of flow down stream above crest R.L. H2 in m	% submergence H2/H1 x 100	Coefficient of discharge as per ISI	Remarks
		upstream	downstream						
1.	37.73	75.76	75.25	Full open	9.76	9.39	96.20	1.24	Ungated condition Av. Cd=1.31
2.	26.04	74.99	74.01	"	8.99	8.01	89.10	1.61	
3.	15.47	73.36	72.94	"	7.36	6.99	94.98	1.16	
4.	9.22	71.35	70.79	"	5.35	4.74	88.60	1.43	
5.	5.29	69.61	69.43	"	3.61	3.43	95.01	1.11	
6.	37.73	76.2	75.25	7.98	10.2	9.39	92.06	1.36	Gated Condition Av. Cd = 1.98
7.	26.04	76.2	74.01	5.23	10.2	8.01	78.53	2.04	
8.	15.47	76.2	72.94	2.47	10.2	6.99	68.53	2.11	
9.	9.22	76.2	70.70	1.38	10.2	4.74	46.47	2.18	
10.	5.29	76.2	69.43	0.94	10.2	3.43	33.60	2.23	

Table No- 4.11 1:55 G/S Sectional Model of Spillway of Samal Barrage on Brahmani River (Odisha)
(A) Ungated Condition
 Water Level Observations (in m)

Sl. No.	Location	Discharge in cumec				
		24,630	17,000	10,000	6,000	3,500
1.	184 m u/s	78.41	76.19	73.90	72.20	70.33
2.	138 m u/s	78.06	75.98	73.94	71.76	69.32
3.	44 m u/s	78.01	76.10	73.84	71.85	69.34
4.	22 m u/s	78.00	76.12	73.83	71.85	69.32
5.	11 m u/s	77.58	75.91	73.74	71.69	69.31
6.	Axis of barrage	76.54	74.44	73.36	70.85	68.63
7.	11 m d/s	72.22	71.34	71.77	70.85	68.18
8.	22 m d/s	75.24	73.79	72.65	70.74	68.14
9.	55 m d/s	75.24	73.66	72.54	70.84	68.24
10.	99 m d/s	75.24	73.96	72.63	70.69	68.15
11.	220 m d/s	75.19	73.44	72.65	70.63	68.94
12.	319 m d/s	75.26	74.01	72.99	70.79	69.43
13.	Trunnion (R.L. 74.00) submerged or not	Submerged	Submerged	Submerged	Not submerged	Not submerged
14.	C.C Blocks disturbed	yes	yes	yes	yes	yes
15.	Distance of hydraulic jump from axis in m.	18.5	18.3	17.2	16.0	13.6

Table No-4.12 1:55 G.S Sectional Model for Spillway of Samal Barrage on Brahmani River (Odisha)
(B) Gated Condition
 Water Level Observation (in m.)

Sl.No.	Location	Discharge in cumec			
		17,000	10,000	6,000	3,500
1.	184 m u/s of barrage axis	76.24	76.24	76.24	76.24
2.	138 m u/s of barrage axis	76.24	76.21	76.21	76.21
3.	44 m u/s of barrage axis	76.21	76.21	76.21	76.21
4.	22 m u/s of barrage axis	76.21	76.21	76.21	76.21
5.	11 m u/s of barrage axis	76.00	76.21	76.20	76.21
6.	Axis of barrage	75.52	76.19	76.12	76.23
7.	11 m d/s of barrage axis	72.28	71.67	70.27	69.55
8.	22 m d/s of barrage axis	72.96	71.86	70.24	69.40
9.	55 m d/s of barrage axis	73.41	71.88	70.39	69.85
10.	99 m d/s of barrage axis	73.67	71.91	70.40	69.71
11.	220 m d/s of barrage axis	73.67	72.93	70.73	69.49
12.	319 m d/s of barrage axis	73.62	72.99	70.79	69.43
13.	Trunnion (R.L 74.00 m) submerged or not	Not submerged	Not submerged	Not submerged	Not submerged
14.	C.C. Blocks disturbed	Yes	Yes	Yes	Yes
15.	Gate opening in m	5.55	3.86	2.05	1.06
16.	Distance of hydraulic jump from axis in m.	18.0	16.7	15.2	12.5

Table - 4.13 - 1:55 G.S Model of Spillway of Samal Barrage-Computation for coefficient of discharge(Cd)

Sl. No.	Discharge intensity in m ³ /s/m	Water level in m Gate		Gate opening in m	Depth of flow upstream above crest R.L. H1 in m	Depth of flow down stream above crest R.L. H2 in m	Percentage submergence H2/H1 x 100	Coefficient of discharge as per ISI	Remarks
		upstream	downstream						
1.	64.55	78.41	75.26	Fully open	11.41	8.26	72.65	2.04	Ungated Condition Avg. C _d = 1.81
2.	44.55	76.18	74.01	-do-	9.18	7.01	76.36	1.93	
3.	26.21	73.90	71.94	-do-	6.90	5.99	86.81	4.56	
4.	15.72	72.20	70.79	-do-	5.20	3.79	72.88	1.81	
5.	9.17	70.33	69.43	-do-	3.33	2.43	72.97	1.71	
1.	44.55	76.20	74.01	5.55	9.20	7.01	76.19	1.92	Gated Condition Avg. Cd = 2.11
2.	26.21	76.20	72.94	3.86	9.20	5.99	65.11	2.40	
3.	15.72	76.20	70.79	2.05	9.20	3.79	41.19	2.19	
4.	9.17	76.20	69.43	1.06	9.20	2.43	26.41	2.23	

4.2 Design:

4.2.1 Introduction

Central Government organisation CWI & NC which was renamed as CWPC was subsequently bifurcated to CWC and CEA. CWC was entrusted with designs and drawings of the dam and spillway and CEA with drawings and designs related to power portion of the project. Model study was carried out at CWPRS Pune, Sailendra Narayan Engineering Research Institute (SNERI) Burla, CSMRS, New Delhi conducted several tests on materials during construction required both for foundation and superstructure.

The river at Rengali flows in a deep narrow gorge predominantly on rocky bed. It was observed that the foundation of the dam is free from any major defects. Further on either side natural abutments of hills exist to which the dam can conveniently butt. These advantages dictated construction of masonry dam (Gravity Dam) which was well conceived from cost-economic consideration.

A masonry/ concrete dam designed by adopting sound engineering practices with strict quality control during construction is a permanent structure requiring little maintenance. "Most modern dams of any magnitude have been built of uncoursed rubble masonry. Concrete is nothing more than uncoursed rubble reduced to its simplest form; as regards resistance to crushing or precolation the value of the two materials is identical, unless it be considered as a point in favour of concrete that it must be solid, while rubble may, if the supervision is defective, contain void spaces not filled with mortar. The selection between the two depends entirely on their relative cost." (Source : History of the Periyar Dam with century long performance by A. Mohankrishnan Feb 1997, CBIP - Publication No. 257, Pg.9)" The Periyar Dam completed in 1895 is standing tall even today and has fulfilled in abundant measure, the objectives and aspirations of its planners and builders. It has ushered in the economic development of entire region. Besides, a unique example for an ancient surviving hydraulic structure in the world is the Grand Anicut at Tiruchirapalli (Tamilnadu).

4.2.1.1 Origin and Development of Gravity Dam:

"The most ancient gravity dam of record was built in Egypt more than 4,000 years B.C of uncemented masonry. Archeological experts believe this dam was kept in perfect condition for more than 45 centuries. While the proportions of this dam, base width to height, are not known there is evidence from the ruins of other dams of that era which indicates base widths as great as four times the height. By improved design and construction methods, the Romans were able to reduce this ratio to 3:1.

Uncemented masonry was not suitable for the construction of other than low dams and other methods of construction evolved. According to records, clay mortar was first used to bind the masonry together; later lime mortar was discovered and used. The masonry type of dam was largely superseded by the cyclopean type of concrete construction, which was a forerunner of the modern concrete gravity dam. Innumerable innovations in design and construction, such as refrigeration of the mass to disperse heat of hydration, use of fly ash, separate block construction, and many others, have made possible construction of monumental structures such as Hoover Dam, 726 ft (221.14 m) high; Grand Coulee Dam which contains more than 11 million cubic yards (8.41 million cum) of concrete and Grand Dixence Dam which is now (1958) under construction in Switzerland and which will have a complete height of 922 feet (280.84m). The ratio of base width to height of all these structures is considerably less than 1:1."

4.2.1.2 Forces acting on Dam:

The forces which must be considered for gravity Dams are those due to (i) water pressure, both external and internal (or uplift), (ii) silt pressure, (iii) ice pressure, (iv) earthquake, (v) weight of

the structure, and (vi) the resulting reaction of the foundation . In designing the crest of an overflow section, the possibility of subatmospheric pressure developing between the sheet of water and the concrete should be considered.

In addition to the normal loading conditions, it may be necessary to apply ice, silt and earthquake loads. It is not likely, however , that all of these additional loads will occur at the same time. Whether these additional loads should be considered and in what combinations , should be detremined by an engineer experienced in the design of dams.

4.2.1.3 Dams on Rock foundations:

“The intensity of uplift pressure under a concrete / masonry dam on a rock foundation is difficult to detrermine. It is generally assumed that pore pressure in rock or concrete are effective over the entire base of the section. It is evident that under sustained loading the intensity of uplift at the upstream face is equal to the full reservoir pressure and approaches a straight -line variation from this point to tail water pressure, or zero , at the downstream face if there is no tailwater. This is true not only at the contact between the dam and the foundation but within the body of the dam itself.

Uplift pressure can be reduced by forming drains through the concrete of the dam and by drilling drainage holes into the foundation rock. Such drains are usually provided near the upstream face of the dam although care must be exercised to insure that direct piping from the reservoir will not result. Drains of this type are provided in all Bureau of Reclamation dams of considerable height , and actual measurements of uplift under the dams have proved them to be very effective. If the rock of the foundation of a proposed dam were absolutely homogeneous, the effectiveness of the drains could be predetermined. However, owing to the presence of seams and fissures and the uncertainty of intercepting them with the drains, the safest course is to assume the straight-line variation from headwater to tail water pressure as a measure of uplift. Any other assumption should be verified by electric analogy or other comparable methods of analysis conducted by engineers experienced in this field.

Other methods used to reduce the uplift at the contact of the dam with the foundation include construction of cut off walls under the upstream face, construction of drainage channels (usually of sewer pipe) between the dam and the foundation, and pressure grouting the foundation. These methods usually are considered only as additional safety factors in the design of small dams and are not considered as justification for reducing design requirements.”

“Many gravity dams have been designed without regard to silt load. In general the silt load against storage dams will be a small factor but against diversion dams it is likely to be more important. In either case there is some basis for neglecting the silt load. Intially the silt load is not present, and by the time it might become a significant, factor the silt has consolidated to some extent and therefore acts less like a fluid. Furthermore, silt deposited in the reservoir will probably be somewhat impervious and will help to minimise the uplift under the dam.”

4.2.1.4 Earthquake: - (a) General - “Earthquakes impart acclerations to the dam which may increase the water and silt pressure on the dam and the stresses within the dam. Some allowance for earthquake loads must be made in the design of concrete gravity dams to be constructed in seismic zones. In addition to the increase in water loads and in silt loads (if applicable), the effect of earthquake on the dead load of the structure must be recognized.

Both vertical and horizontal earthquake loads should be applied in the direction which produces the least stable structure. For the condition of full reservoir this will be a foundation shock in the upstream direction and foundation shock downward. The first increases the water load and produces an overturning moment due to inertia on the concrete. The second, in effect causes the concrete and water above a sloping face to weigh less and in this way reduces the stability of the structure.

In order to determine the total forces due to an earthquake, it is necessary to establish the earthquake intensity or acceleration. This is usually expressed in relation to the acceleration due to gravity. The acceleration that may be reasonably expected at the dam site are determined from a consideration of the geology of the site, proximity to major faults, previous earthquake history of the region and such seismic records as are available. In areas not subjected to extreme earthquake conditions a horizontal acceleration of 0.10 of gravity and vertical acceleration of 0.05 of gravity are generally used.

Experimental and analytical procedures show that, because of the internal shear resistance of the silt, an earthquake acceleration up to approximately 0.30 of gravity is only about half as effective in silt as in water. Since the unit weight of water is about one-half that of silt, the increase in pressure on the dam due to earthquake is approximately the same for either silt or water.”

(b) Horizontal Earthquake:- “The effect of inertia on the concrete should be applied at the center of gravity of the mass, regardless of the shape of the cross section. x x x x For dams with a combination vertical and sloping face, the procedure to be used is governed by the relation of the height of the dam, as follows:

(i) If the height of the vertical portion of the upstream face of the dam is equal to or greater than one-half the total height of the dam, analyze as if vertical throughout.

(ii) If the height of the vertical portion of the upstream face of the dam is less than one-half of the total height of the dam, use the pressure on a sloping line connecting the point of intersection of the upstream face of the dam and reservoir surface with the point of intersection of the upstream face of the dam and the foundation.

(c) Vertical Earthquake:- On sloping faces of dams the weight of the water above the slope should be modified by the appropriate acceleration factor. The weight of the concrete also should be modified by this acceleration factor.” (Source: Design of Small Dams, U.S. B.R. 1968 Indian Edition pg.231 - 238)

4.2.1.5 Seismic Coefficient adopted for Rengali Dam:

Determination of seismic co-efficient of dam is an important aspect. As per the guideline given by the Ministry of irrigation & power, Government of India, a committee was set up with the representatives of following department to recommend the final value.

- (a) Indian Meteorological Department.
- (b) Geological Survey of India.
- (c) Earthquake School of Engineering, Roorkee.
- (d) Central Water Commission & Ministry of Irrigation.

The member of the committee on Earthquake coefficient of Rengali Dam met on 10.05.76 at New Delhi and the final recommendation is given below.

Sl No.	Name of the Project	State & District	Location	Horizontal Seismic Coefficient
01	Rengali Dam Project	Odisha Dhenkanal (now Angul)	Long 85°-02'E Lat 21°- 17'N	Concrete or Masonry 0.05 g. Rock - Fill & Earth Dam 0.05 g

The Rengali Dam site across the river Brahmani lies in a zone where no earthquake of any significance has occurred in the past. There is no seismological observatory in this region. Hence, it is difficult to say if the area had experienced minor earthquake.

This area however, has experienced great earthquake originating in the Himalayan boundary fault belt of which the Assam earthquake on 12th June, 1987, and Bihar-Nepal earthquake intensity experienced in the area was not more than V (Modified Mercalli Scale). The project area falls within isoseismals IV and V of Bihar - Nepal earthquake of 1934 for which an earthquake factor of 0.05 g has been incorporated in the design of dam.

The Indian Standards Institute- Engineering Sub-Committee which was responsible for the preparation earthquake zoning map of India has identified a zone of weakness between Brahmani and Mahanadi river and assigned to it a maximum seismic intensity of VII. The Rengali Dam site, the project now under consideration lies in the North close to the border of this region in Zone II. The seismic intensity likely to reach in this zone is IV.

Following table indicates the Zone wise basic seismic horizontal coefficients.

Sl.No.	Zone	Horizontal Basic seismic Co-efficient
1.	V	0.08
2.	IV	0.05
3.	III	0.04
4.	II	0.02
5.	I	0.01

The vertical seismic co-efficient where applicable may be taken as half the horizontal seismic co-efficient. Studies made in U.S.A and other advanced countries show that intensity of VI (MM Scale) corresponds to horizontal seismic acceleration range of 5.175 cm/sec² or an average acceleration figure is due to the fact that acceleration is more on filled up ground and much less on hard rock.

Considering all these factors, it is felt that for important structure at the Rengali Dam Site, provision of horizontal seismic acceleration of five percent of gravity i.e 0.05 g may be made. For vertical acceleration the normal practice is to make a provision of half of the normal horizontal seismic acceleration.

4.2.1.6 Requirements for Stability:

General:- “A concrete gravity dam must be designed to resist, with ample factor of safety, the three tendencies to destruction:(a) over- turning, (b) sliding, and (c) overstressing

(a) Overturning:- There is a tendency for a gravity dam to overturn about the downstream toe at the foundation or about the downstream edge of any horizontal section. If the vertical stress at the upstream edge of any horizontal section computed without uplift exceeds the uplift pressure at that point, the dam is considered to be safe against overturning with an ample factor of safety. If the uplift pressure at the upstream face exceeds the vertical stress at any horizontal section computed without uplift, the uplift forces along the assumed horizontal crack greatly increase the tendency for the dam to over turn about the downstream face.

(b) Sliding:- The horizontal force, ΣV , tends to displace the dam in a horizontal direction. This tendency is resisted by the frictional and shearing resistance of the concrete or the foundation.

The shear friction factor (S.F.F) is a criterion normally used for higher dams. Cohesive characteristics of the concrete or rock, which greatly affect the shear friction factor must be determined by special laboratory tests or estimated by an engineer who has had considerable experience in this specific field. For small structures where it is not economical to perform these tests or to obtain expert advice, the usual method of checking the structure for horizontal displacement is by determination of a sliding factor.

The allowable sliding factor is the coefficient of static friction between two sliding surfaces reduced by an appropriate factor of safety. If f represents the allowable sliding factor, a dam is considered safe against sliding when $\frac{\sum V}{\sum W - U}$ is equal to or less than f .” x x x x x

“Concrete cutoff walls are often provided on structures constructed on foundations other than rock. The cutoff properly proportioned and reinforced, prevents displacement of the structure by internal shear resistance of the cutoff itself and the additional volume of soil that must be moved before the structure can slide. To accomplish this objective, the cutoff must be designed as a cantilever beam loaded with the horizontal force that is in excess of the foundation’s resistance to sliding.

(c) Overstressing:- The unit stresses in the concrete and the foundation must be kept within prescribed maximum values. Normally, the stresses in the concrete or masonry gravity dams are so low that a concrete mix designed to meet other requirements such as durability and workability will attain sufficient strength to insure a factor of safety at least 4 against overstressing.” (source:Ibid pg.239 to 240)

As discussed earlier, CWC and CEA were design consultants for Dam and Power House. Design for the barrage at Samal was finalised at the State Design Directorate. But it is unfortunate that due to poor documentation, dam design details are not available at CWC and CEA and even barrage design from Design Directorate, Bhubaneswar. This is really a sorry state of affair.

For increasing the passive resistance against sliding and improving factor of safety following measures have been undertaken.

It was seen that in order to remove the clayey and other intruded masses from shallow depth at foundation grade trench of various shapes were formed parallel to dam axis. The formation of such trenches has two advantages so far the safety is concerned. First all clayey seams which are weak paths against sliding have been removed. Secondly the short sloped trenches due to additional friction increases the factor of safety to a great extent. Such trenches have been provided in Blocks 9 to 14 and 26 to 30, as described hereunder.

Block 9 to 14:

Due to presence of crushed zone, it was necessary to trench the area to remove all clayey and foreign materials. This had developed to a regular trench parallel to dam axis. The filling up the trenches had improved the factor of safety.

Block 26 to 30:

A similar shallow trenching was also necessary near the axis for removal of deleterious material. This has also added to the factor of safety of the dam.

The block 1 to 9 and 26 to 52 have been trenched sufficiently below the natural ground level both in the u/s and d/s side. The passive resistance that will be available in the side will be a great help for increasing the factor of safety. The d/s toe depth of bucket of spillway which have been reduced from 3 m to 1 m poses no problems so far block 26 to 43 are concerned but for blocks 19 to 25 the original toe trench 3 m is recommended to induce passive resistance due to down-stream strut, since the bucket is founded almost at the ground level.

4.2.1.7 Strength of Concrete/Masonry considered for Rengali:

The strength of concrete/masonry shall exceed the stresses anticipated in the structure by a safe margin. The maximum compressive stresses occur at the heel or toe and on planes normal to the faces of the dam.

Concrete:

The strength of concrete should satisfy early load and construction requirements and at the age of one year should be four times the maximum computed stress in the Dam or 140 kg./cm² whichever is more. The allowable working stress in any part of the dam shall not exceed 70 kg./cm².

Masonry:

The strength should satisfy early load and constructions requirement and at the age of one year it should be four times the maximum computed compressive stress in the dam or 175 kg/cm² whichever is more.

Notes:

For Rengali Dam the strength of masonry was kept at 140 kg/cm² since working stress is limited to 16 kg/cm², which resulted to a factor of safety of 9. The specification of masonry and concrete used in the overflow section (O.F) and non overflow (N.O.F) section is given in Drg No- 4.2 & 4.3

Drawing showing the co-ordinates of ogee profile, maximum O.F section through Block 28 and 36, location of Irrigation sluice and reinforcement around gallery in N.O.F block are respectively given vide Drg. No- 4.4, 4.5, 4.6 & 4.7.

Specification of concrete and Masonry used in both O.F. and N.O.F. sections and P.H. have been furnished in Table No-4.14 to 4.18.

From the Table No- 4.14 & 4.15, it is seen that the 28 days compressive strength of R.R Masonry in 1:3 and 1:4 cement mortar are respectively 175 kg/cm² and 105 kg/cm². The maximum allowable compressive stress in the heel and toe are respectively 15.54 kg/cm² and 18.00 kg/cm². The minimum compressive strength of concrete to be placed in various locations of the dam and spillway are 200 kg/cm² and 250 kg/cm², vide Table- 4.16 and 4.17.

4.2.1.8 Anchoring of the bucket:

The O.F section of the dam has two distinct components i.e body of the dam and the energy dissipator. The energy dissipator has been made separate by introducing a contraction joint. The dissipator is designed as a bucket type where the mass of water is thrown off in a trajectory clear off the bucket to a safe distance. Retrogression was not apprehended due to massive sheet rock available in the river bed. Due to tight rockmass the uplift pressure release will be gradual subjecting the bucket to severe strain when the sudden drawdown may cause heaving and consequent cracking of the structure.

To avoid any such eventuality two full proof arrangements have been made i.e.

(i) Provision of series of interconnected drains below bucket provided with outlet pipes 150 mm dia spaced at 5 m c/c both ways on the d/s face and (ii) In the event the drains become inoperative due to clogging by silt and clay etc, the buckets were anchored to bottom rock mass with high strength TOR steel bars of 36 mm dia.

The drainage holes (75 mm dia) were drilled by wagon (percussion) drills and were washed using air-water jet. The holes were filled with pea gravels to serve as filter. The drainage holes and

DRG NO.- 4.2

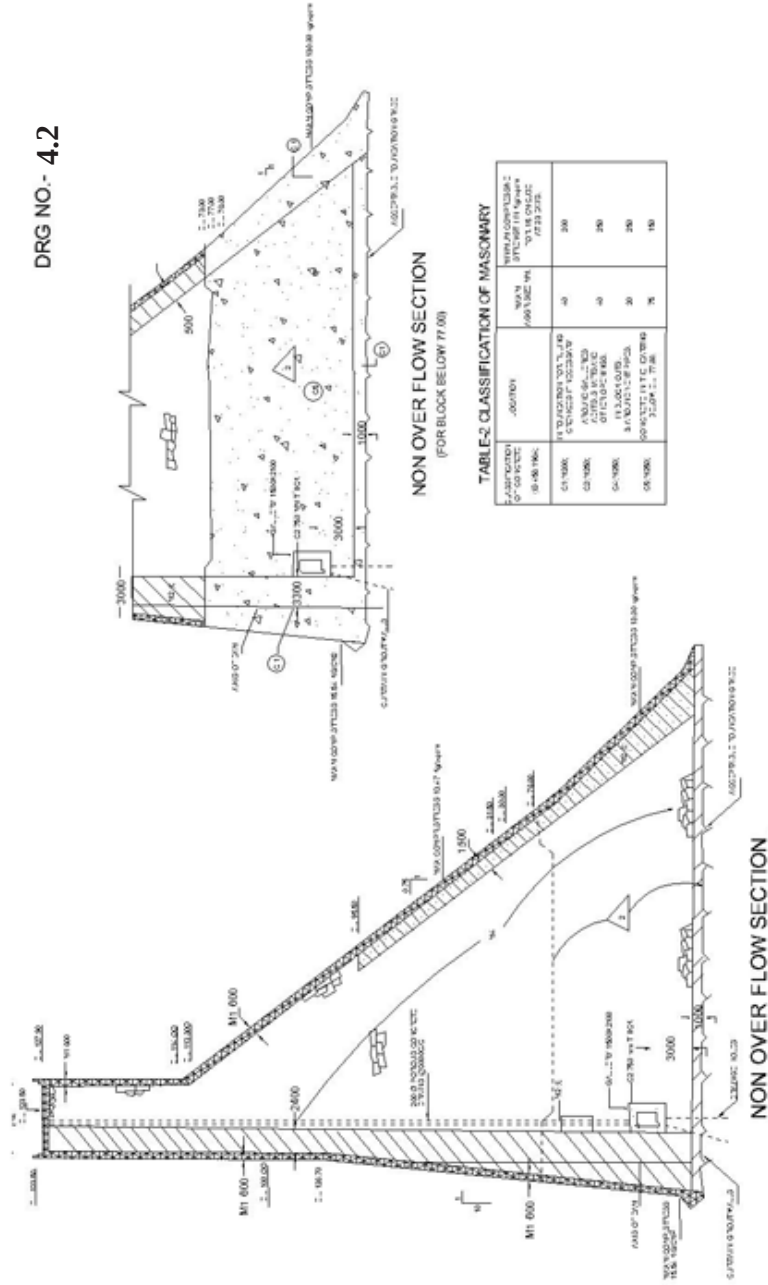


TABLE-2 CLASSIFICATION OF MASONRY

CLASSIFICATION OF MASONRY	LOCATION	AREA IN SQ. M	WITH PROPOSED CLASSIFICATION TO BE ON CHART (77.00)
CM-1000	AT TOADDERHILL TO 1.00	40	200
CM-1000	AT TOADDERHILL TO 1.00	40	200
CM-1000	AT TOADDERHILL TO 1.00	40	200
CM-1000	AT TOADDERHILL TO 1.00	40	200
CM-1000	AT TOADDERHILL TO 1.00	40	200
CM-1000	AT TOADDERHILL TO 1.00	40	200
CM-1000	AT TOADDERHILL TO 1.00	40	200
CM-1000	AT TOADDERHILL TO 1.00	40	200
CM-1000	AT TOADDERHILL TO 1.00	40	200
CM-1000	AT TOADDERHILL TO 1.00	40	200

TABLE-1 CLASSIFICATION OF MASONRY

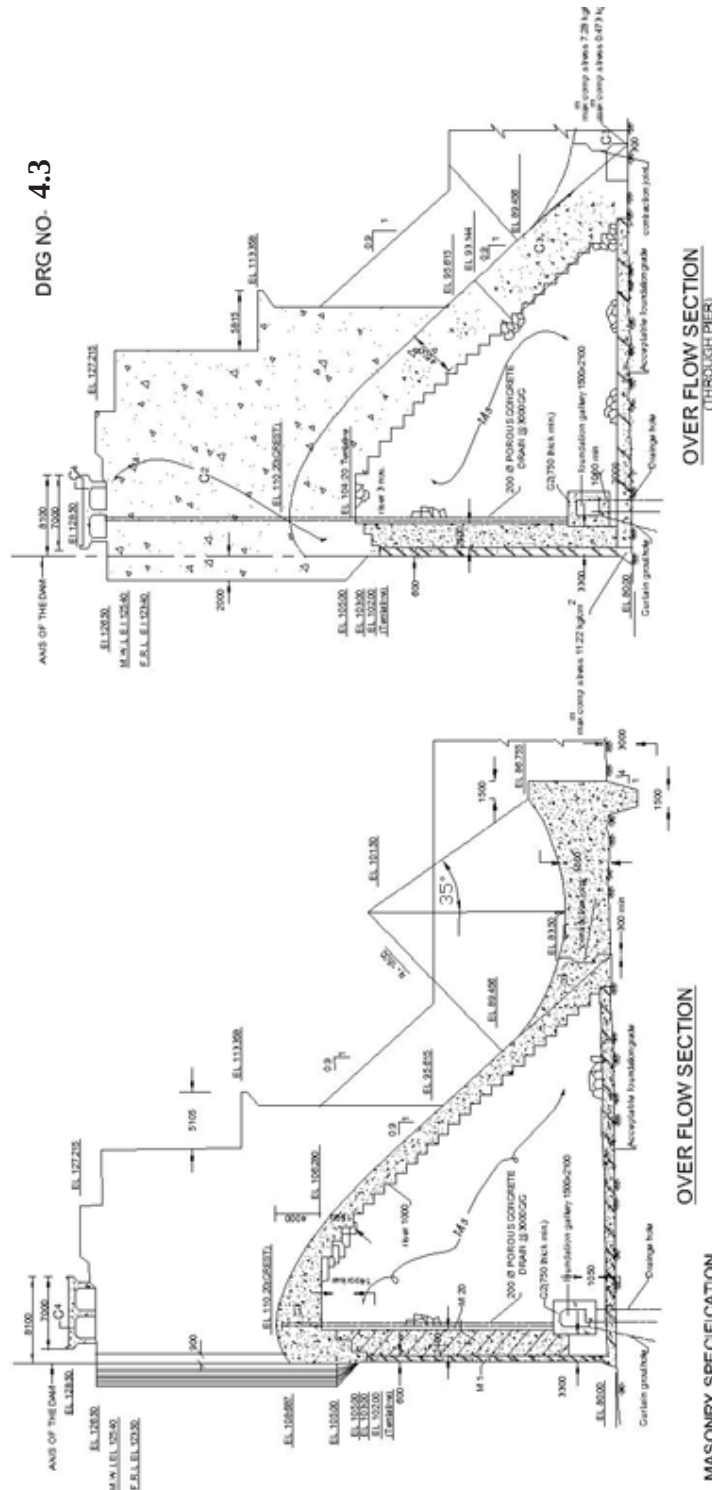
GRADE OF MASONRY	LOCATION	TYPE OF MASONRY	COMPREHENSIVE STRENGTH IN MPa	TYPE OF MORTAR	MAXIMUM PERMEABILITY (in mm)
1	2	3	4	5	6
CM-1000	AT TOADDERHILL TO 1.00	CM-1000	100	1:3	100
CM-1000	AT TOADDERHILL TO 1.00	CM-1000	100	1:3	100
CM-1000	AT TOADDERHILL TO 1.00	CM-1000	100	1:3	100
CM-1000	AT TOADDERHILL TO 1.00	CM-1000	100	1:3	100
CM-1000	AT TOADDERHILL TO 1.00	CM-1000	100	1:3	100
CM-1000	AT TOADDERHILL TO 1.00	CM-1000	100	1:3	100
CM-1000	AT TOADDERHILL TO 1.00	CM-1000	100	1:3	100
CM-1000	AT TOADDERHILL TO 1.00	CM-1000	100	1:3	100
CM-1000	AT TOADDERHILL TO 1.00	CM-1000	100	1:3	100

CENTRAL WATER COMMISSION

RENGALI DAM PROJECT
RENGALI DAM

ZONING OF MATERIAL FOR
NONOVER FLOW BLOCKS

DRG NO- 4.3



OVER FLOW SECTION
(THROUGH PIER)

OVER FLOW SECTION

MASONRY SPECIFICATION

GRADE OF MASONRY	LOCATION	TYPE OF MASONRY	MINI COMPRESSION STRENGTH IN MPa	TYPE OF MORTAR	MAX PERMEABILITY (cm/sec)
M1	600 mm THICK IMPERVIOUS LAYER IN FRONT FACE OF SPILLWAY	C R	190	RICH C/M NOT LEANER THAN 1:3 BY VOLUME	2.4X10 ⁻⁹
M2	2000 mm THICK IN FRONT FACE OF THE SPILLWAY ADJACENT TO 600mm FACE WORK	R R	175	— DO —	2.4X10 ⁻⁹
M3	2000 mm THICK IN FRONT FACE OF THE SPILLWAY ADJACENT TO 600mm FACE WORK	R R	175	— DO —	2.4X10 ⁻⁹
M4	HEARTING IS SPILLWAY ENCEPT FOR THE PORTION 1000mm DEPTH AT THE FOUNDATION AND IN ITEM 42 ABOVE	R R	105	C/M NOT LEANER THAN 1:4 BY VOLUME	4.8X10 ⁻⁹

CONCRETE SPECIFICATION

CLASSIFICATION OF CONCRETE (IS-456-1984)	LOCATION	MINI COMPRESSIVE STRENGTH IN MPa AT 28 DAYS
C1(M20)	IN FOUNDATION FOR FILLING GREYHOLE IF NECESSARY ALL AROUND GALLERIES AND PIER	40
C2(M20)	IN FOUNDATION FOR FILLING GREYHOLE IF NECESSARY ALL AROUND GALLERIES AND PIER	40
C3(M20)	IN FOUNDATION FOR FILLING GREYHOLE IF NECESSARY ALL AROUND GALLERIES AND PIER	75
C4(M20)	IN FOUNDATION FOR FILLING GREYHOLE IF NECESSARY ALL AROUND GALLERIES AND PIER	20

CENTRAL WATER COMMISSIO
RENGALI DAM PROJECT
RENGALI DAM
ZONING OF MATERIAL
FOR OVER- FLOW SECTION

Table- 4.14
Specification for Rengali Dam (NOF Section)
(Zoning Drg No. RNG - 5010-C-3041)

Classification of masonry	Location	Type of masonry	Min. compressive strength of 75 cm cube of masonry in kg/sq.cm.		Limiting proportion of mortar by volume	Maximum allowable permeability in cm/sec.
			28 days	365 days		
M-1	600 mm thick impervious layer on all faces.	CR	180	210	C.M not leaner than 1:3	2.4×10^{-9}
M-2 (i)	(i) Adjacent to the 600 mm thick face work upstream and down stream face of the dam varying from 1500 mm to 7300 mm.	RR	175	200	1:3	2.4×10^{-9}
M-2 (ii)	(ii) 1000 mm thick layer on acceptable foundation except for portions covered by M-(i)	RR	175	200	1:3	2.4×10^{-9}
M-3	Hearting in the dam above EL 77.00 and below 127.90 except from the portion 1000 mm layer as in item M-2 (ii)	RR	105	140	1:4	$4:8 \times 10^{-9}$

Note: N.O.F blocks from 14 to 18 in deep channel portion will be constructed with concrete upto EL. 77.00 instead of masonry to expedite the work in river gap portion.

Table - 4.15 O.F. Section
(Zoning Drg No. RNG - 5010-C-3022)

Classification of Masonry	Location	Type of masonry	Minm. compressive strength of masonry in kg/sq.cm.		Type of mortar	Maximum allowable permeability in cm/sec.
			28 days	365 days		
M-1(*)	60 mm thick imerivious layer in front face of the spillway.	Course rubble	180	210	Rich cement mortar not leaner than 1:3 by vol.	2.4×10^{-9}
M-2 (i)	2400 mm. thick in front face of the spillway adjacent to 600 mm face work.	Random rubble	175	200	- do -	2.4×10^{-9}
(ii)	1000 mm. depth at the foundation for the full section of the dam except portion covered by item M ₁ and block 21.	- do -	175	200	-do-	2.4×10^{-9}
M-3	Heating in spillway except for the portion 1000 mm depth at the foundation as in item M ₂ (ii) above.	-do-	105	140	Cement mortar not leaner than 1:4 by volume.	4.8×10^{-9}

(*) Instead of M1 type masonry M 250 (C-3 type) concrete will be put on the up stream face from EL.94.00m

Table- 4.16 Non- Overflow Section
(Zoning Drg No. RNG - 5010-C-3041)

Classification of concrete as per IS - 456-1964	Location	Maxm. size of aggregate in mm	Minm. compressive strength of 15 cm cubes at 28 days in kg/sq.cm.
C-1 (M 200)	In foundation for filling crevices	40	200
C-2 (M-250)	Around galleries, adits, shafts and other openings	40	250
C-4 (M 250)	In block -outs and round vent Pipes	20	250
C -5(M 150)	Concrete in hearting below EL 77.00	75	150

Note. Gallery level from Block No. 15 to 17 will be raised from EL.66.0 to EL. 78.0 to facilitate concreting in deep channel.

Dam in Block No. 2,3,4, 47, 48, 49 and 50 is to be constructed on natural fresh rock without benching provided the maximum slope is equal to the angle of friction between concrete and rock (i.e about 36°)

Table - 4.17 Overflow Section (Zoning Drg No. RNG - 5010-C-3022)

Classification of concrete as per IS - 456-1964	Location	Maxm. size of aggregate in mm	Minm. compressive strength of 15 cm cubes at 28 days in kg/sq.cm.
C-1 (M 200)	In foundation for filling crevices	40	200
C-2 (M-250)	All around galleries, and piers and exterior 1.00 mm of crest and bucket	40	250
C-3 (M 250)	Spillway crest down -stream face of spillway and bucket except for exterior 1.00 m thickness	75	250
C-4 (M 250)	In block-outs, road, bridge, slabs, beams and around vent Pipes.	20	250

Note. The downstream cut off level is being kept 3m below sound rock level occurring in the close vicinity of downstream or 1.0 m below the bucket invert foundation level which ever is lower to save excavation in sound rock.

Table- 4.18 Specification of Concrete for Power House
(Ref. Letter No. 2/6/76 - HCD II/300 dt. 22.03.76 of C.W.C)

All concrete of Power House shall conform to the classification given below.

Classification of Concrete	Min. strength of 28 days(kg/cm ²) Cylinder, size 150 mm X 300 mm	Maxm. size of coarse aggregate in mm	Slump range in mm
A-2	211	75	50-75 for sub-structure and downstream piers.
B-2	211	40	50-100 for auxillary rooms, columns, beams and girders
B-3	211	20	50-100 for floor slabs and walls.

under drains were laid in a systematic manner so as to cover the entire bucket. 4 nos of 15 cm dia pipes properly bent were provided to serve as exit pipes so that no loose materials may back flow into it and its mouth was bent in the direction of suction, such that the mouth would remain clear due to the induced suction along the path of trajectory.

In spite of above precautionary measures, if the draingae arrangement fails due to clogging, the second line of defence for the safety of the structure was to hold it in position by steel anchors. The length of the anchors were decided by considering the net upward force per unit area of bucket (i.e uplift - weight of the structure) which works out to 2.8 ton /unit area divided by weight per unit length of work bounded by anchor. The length of anchors worked out to 2m against which 2.5 m length have been provided.

4.2.1.8.1 Designing the Section of Anchor:

Where the rock is competent enough to counter the uplift, next step is to ensure that the steel which holds down the bucket with rockmass does not snap while transmitting the pull. Steel anchor was designed accordingly with allowable stress of 1900kg/cm².

The anchor grouted in rock mass is to be tested for bond failure i.e (a) bond between anchor and the grout and (b) bond between grout and rock surface. Case (b) is more vulnerable and for the purpose of design, the bond stress may be taken as 5 to 7 kg/cm². The bond between grout and drilled rock face is poor as smoothness develops during drilling operation. A slit was cut at one end of the 36 mm dia anchor to which a wedge was driven while inserting the same in the drilled hole, then hammered and grouted.

4.2.1.8.2 Pull-out test of Anchor Bars:

Torsteel bars of 36 mm dia have been provided in bucket to serve as anchor bars. The spacing of anchor bars is 1.45 m c/c staggered. Each anchor has been designed to withstand a pull of 17.1 M.T. Anchor bars were provided with wedge in 75 mm dia holes drilled to a depth of 2.5 m.

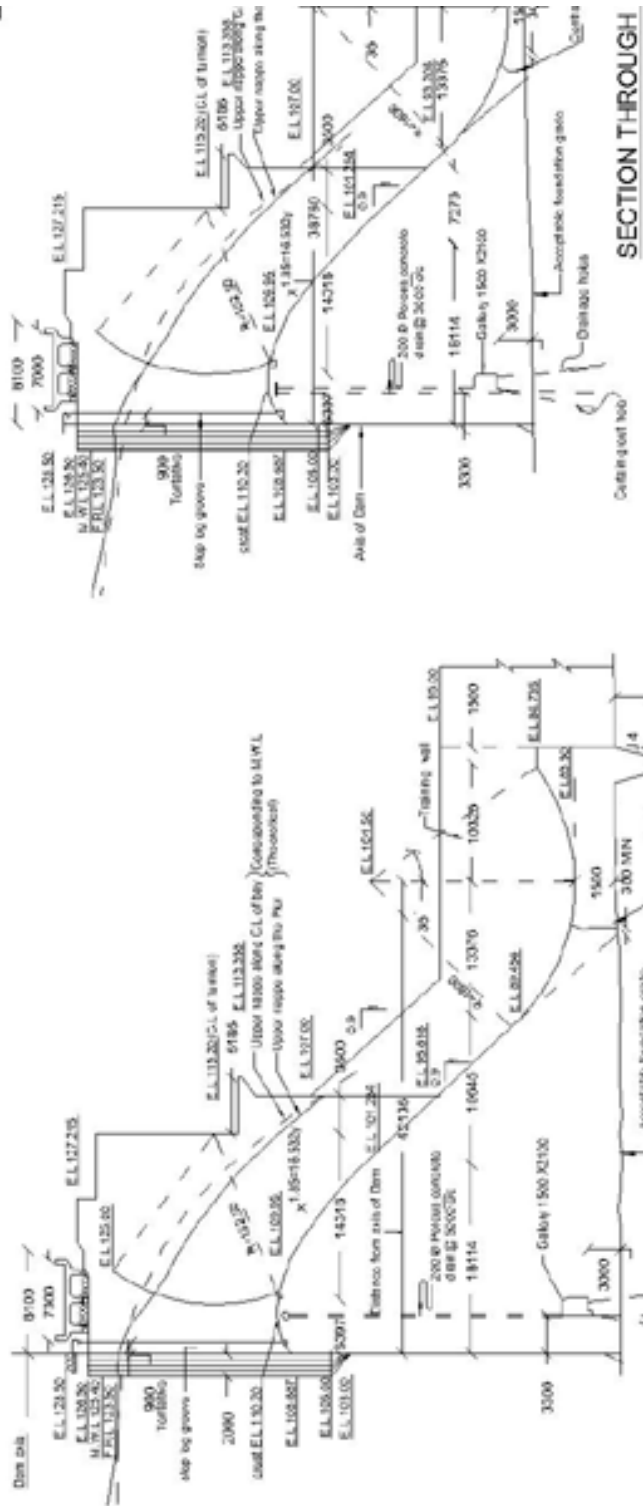
Pull out test of anchor bars provided in buckets of Block 29 and 32 was carried out as follows:-

A piece of 300 mm x 140 mm joist was welded in between two anchor bars horizontally. Load was applied from bottom of the joist by means of 50T/100T capacity hydraulic jack and the load was recorded through a dial guage. In all the cases there was no sign of failure. This arrangement was made as central hole Jack for carrying the pull-out test was not available in the project. A central hole jack of 50T Capacity was procured later. Each anchor bar was tested and confirmed to withstand the load of cone of rockmass assumed to be attached to each anchor bar as per design. The test results are tabulated below.

Tablele- 4.19 Pull-out test of Anchor Bars

Sl.No	Date of test	Location of Spillway	Pull applied on each anchor(Tons)	Remarks
1.	02.01.78	Block NO. 29	20	Wedge was provided at the end of the anchor bar
2.	04.01.78	Block No.29	20	-do-
3.	17.02.78	Block No. 32 RD.630.50m	45	With wedge at the end
4.	17.02.78	Block No. 32 RD 631.00m	45	Without wedge.
5.	18.02.78	Block No.32 RD 620.50m	50	With wedge at the end.

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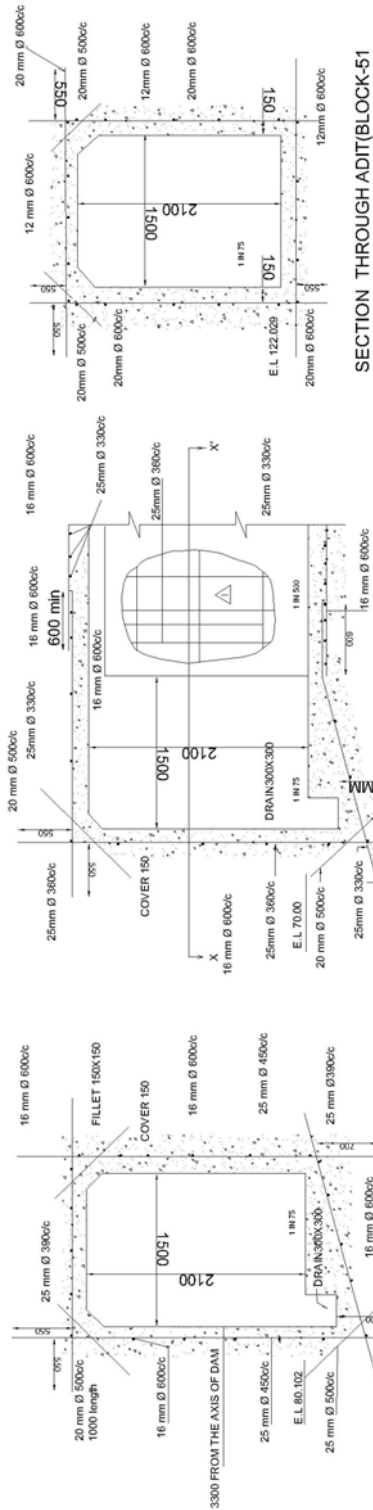
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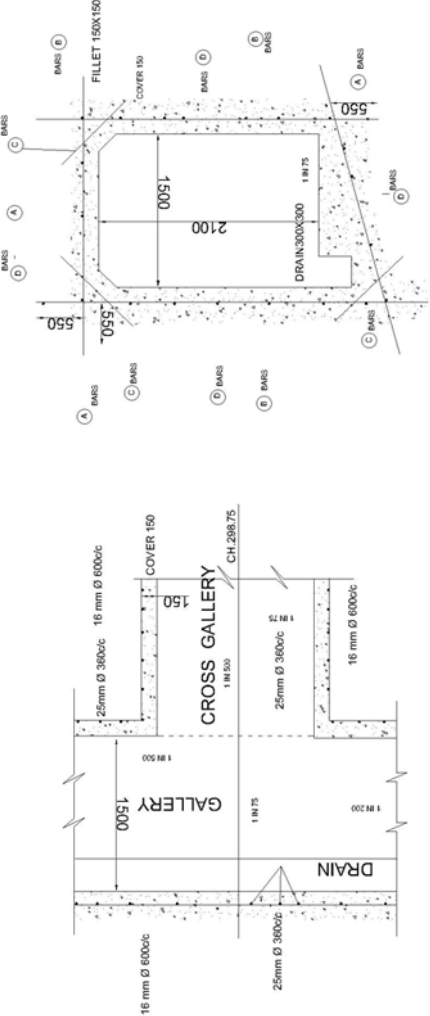
RENGALI DAM PROJECT
RENGALI DAM
DETAILS OF IRRIGATION SLUICE
(BLOCK-43/1)

DRG. NO.- 4.7



CROSS SECTION OF GALLERY AT Ch 236 (BLOCK 12)

SECTION THROUGH C.L. OF CROSS GALLERY (BLOCK 15)



SECTIONAL PLAN OF CROSS GALLERY AT:X-X

GENERAL SECTION THROUGH GALLERY

CENTRAL WATER COMMISSION
RENGALI DAM PROJECT
MASONRY DAM REINFORCEMENT AROUND GALLERIES IN NON -OVER FLOW BLOCK

4.2.1.9 Design of Gates

Design, fabrication and erection of radial crest gates of dam, crest gate and under sluice gates of barrage were entrusted to M/s Odisha Construction Corporation Ltd (A Govt of Odisha undertaking). Stresses developed in different components are furnished in Table-4.20 and 4.21 to 4.23. Spill-way gate drawing of dam is enclosed Vide Drg. No- 4.8

Table- 4.20 Radial Crest Gate Of Rengali Multipurpose Project

Name of Member		Stress in kg/cm ²			
1	Skin Plate	At junction of plates @			
		121.6 mtr	118.4 mtr	115.2 mtr	112 mtr
	Bending across stiffner	950	1208	1189	1248
	Bending Coacting with stiffner	592	280	285	253
	Combined Stress	1347	1370	1354	1392
2	Horizontal girder				
	a) Bottom/ Intermediate Horiz. girder				
	i) Compressive Stress		675		
	ii) Tensile stress		903		
	iii) Shear stress		759.17		
	b) Top Horizontal girder				
	i) Compressive Stress		634		
	ii) Tensile stress		784		
	iii) Shear stress		550		
3	End arm				
	a) Bottom/Intermediate arms Compressive Stress		1048		
	b) Top arm Compressive Stress		999		
4	Tie rod - Tension		1169		
5	Anchor girder compressive stress		594		
6	Vertical Anchor bolt				
	a) Shear Stress		493		
	b) Tensile Stress		1018		
7	Thrust block				
	a) Bending		788		
	b) Shear		766		

Table - 4.21 Radial Gate For Barrage Bay of Samal Barrage Project

Sl.No.	Name of Member	Stress in kg/cm ²		
1	Skin Plate	At Horizontal Girder positions @ 73.492 mtr. 70.298 mtr 67.58 mtr		
	Bending across stiffner	398.68	821.88	821.61
	Bending coacting with stiffner 509.94	504.11	409.47	
	Combined Stress	782.56	1159.28	1085.31
2	Horizontal girder			
	a) Bottom/ Intermediate Horiz. girder			
	i) Compressive Stress		502.59	
	ii) Tensile stress		759.23	
	iii) Shear stress		570.96	
	b) Top Horizontal girder			
	i) Compressive Stress		403.86	
	ii) Tensile stress		577.47	
	iii) Shear stress		304.69	
3	End arm			
	a) Bottom/Intermediate arms			
	Compressive Stress		850.04	
	b) Top arm			
	Compressive Stress		706.52	
4	Tie rod - Tension		912.00	
5	Anchor girder compressive stress		271.95	
6	Vertical Anchor bolt			
	a) Shear Stress		251.09	
	b) Tensile Stress		645.37	
7	Thrust block			
	a) Bending		596.46	
	b) Shear		572.39	

Table No- 4.22 Radial Gate For Left Under Sluice of Samal Barrage.

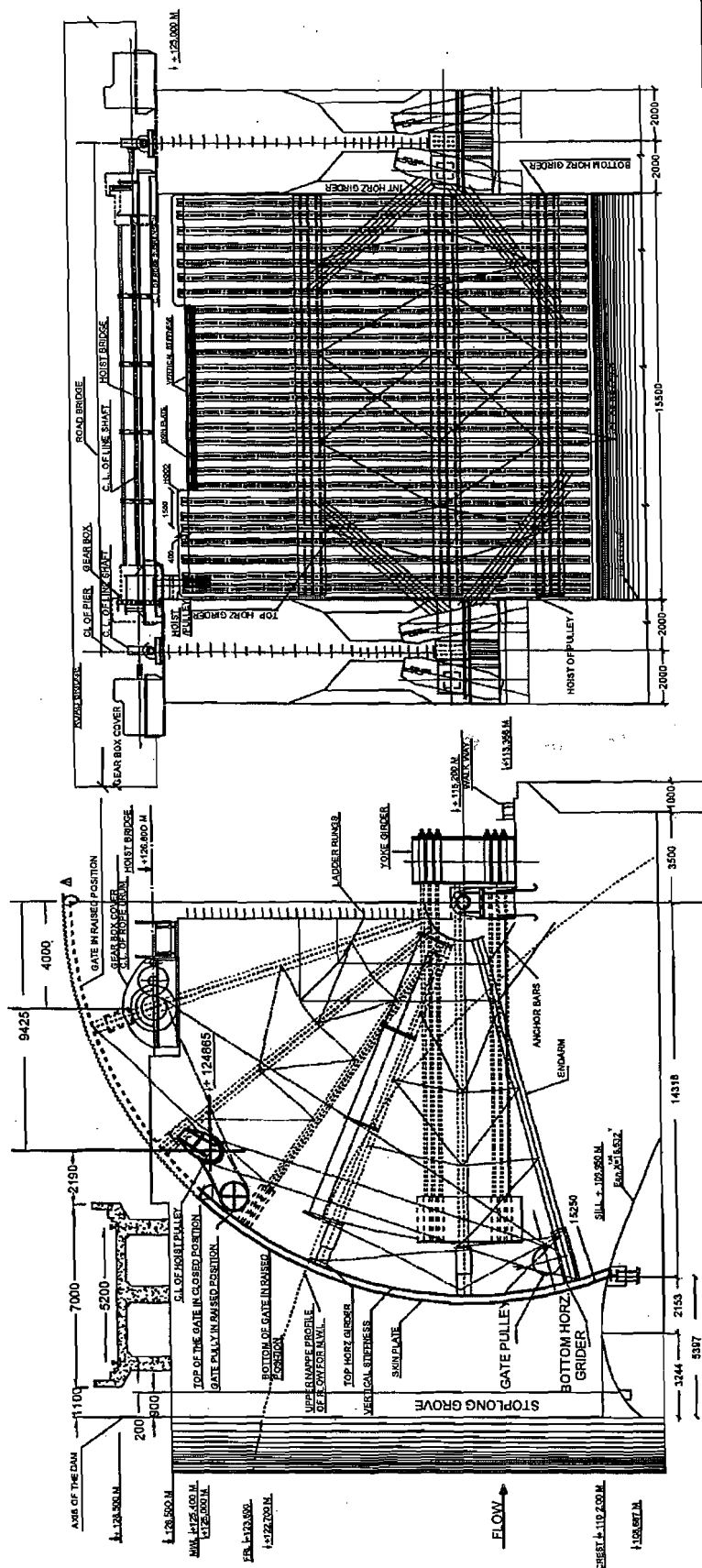
Sl.No.	Name of Member	Stress in kg/cm ²		
		at Horizontal Girder positions @		
1	Skin Plate	73.122 m.	69.667m	66.505m
	Bending across stiffner	374.01	526.43	770.03
	Bending Coacting with stiffner	557.7	501.14	790.44
	Combined Stress	812.09	889.99	1100.51
2	Horizontal girder			
	a) Bottom/ Intermediate Horiz. girder			
	i) Compressive Stress		552.07	
	ii) Tensile stress		859.05	
	iii) Shear stress		657.36	
	b) Top Horizontal girder			
	i) Compressive Stress		470.71	
	ii) Tensile stress		667.3	
	iii) Shear stress		375.31	
3	End arm			
	a) Bottom/Intermediate arms			
	Compressive Stress		895.9	
	b) Top arm			
	Compressive Stress		763.12	
4	Tie rod - Tension		912.00	
5	Anchor girder compressive stress		317.13	
6	Vertical Anchor bolt			
	a) Shear Stress		319.145	
	b) Tensile Stress		888.78	
7	Thrust block			
	a) Bending		758.11	
	b) Shear		671.56	

Table- 4.23 Radial Gate For Right Under Sluice Gates Of Samal Barrage.

Stress in kg/cm²

Sl.No.	Name of Member			
1	Skin Plate	at Horizontal Girder positions @		
		73.053m	69.602m	66.445m
	Bending across stiffner	381.65	531.43	774.67
	Bending Coacting with stiffner	592.09	508.47	459.39
	Combined Stress	849.82	900.65	1080.29
2	Horizontal girder			
	a) Bottom/ Intermediate Horiz. girder			
	i) Compressive Stress		541.85	
	ii) Tensile stress		843.15	
	iii) Shear stress		645.22	
	b) Top Horizontal girder			
	i) Compressive Stress		481.80	
	ii) Tensile stress		683.02	
	iii) Shear stress		384.17	
3	End arm			
	a) Bottom/Intermediate arms Compressive Stress		879.30	
	b) Top arm Compressive Stress		781.15	
4	Tie rod - Tension		912.00	
5	Anchor girder compressive stress		320.29	
6	Vertical Anchor bolt			
	a) Shear Stress		222.22	
	b) Tensile Stress		898.58	
7	Thrust block			
	a) Bending		767.12	
	b) Shear		679.93	

**RENGALI DAM PROJECT
SPILLWAY GATE**



4.3 Quality Control

4.3.1: General

Dams are generally constructed to satisfactorily function for many decades and even for centuries for flood control, irrigation and power generation purposes. It should be strong and durable to take the design load. The materials such as stone, sand, cement etc. should be of high quality and design mixes for cement mortar and concrete should be tested to ensure proper strength satisfying other specifications prescribed by Indian Standard and to meet the design requirements.

Rengali dam is a Gravity Dam of masonry/concrete with central spillway and power house located towards the left bank. For ensuring quality during construction, a quality control organisation was set up during March 1974, headed by an Executive Engineer with supporting staff i.e Assistant Engineers or Assistant Research Officers (ARO) and Research Assistants, Laboratory Assistants, Inspectors, Assistant Inspectors and Analysts etc.

The objective is to ensure that the construction conforms to the relevant specifications as infringement of specifications will mean undesirable incursions into the margins of safety considered in the design. This being a waterstoring structure of high magnitude any compromise in the workmanship may lead to disaster during performance necessitating for otherwise avoidable protective measures. Inspection, control and necessary safeguards at each and every 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 design and durable construction. In case the field control can ensure close tolerances and narrow deviations from the optimum, the designer of the structure will allow the narrow safety margin thereby economising the cost of construction without sacrificing the desired strength and durability. For all the major river valley projects in the state i.e Hirakud, Rengali, Balimela, Kolab and Indravati etc. there was separate Quality Control Organisation for testing of materials to be utilised in the construction of the structure and inspecting the quality of works at every stage of construction. The specialised organisation served the functional need in conjunction with the construction staff.

4.3.2 Testing of Materials:

Rengali Dam involves placement of about 6.0 lakh cum of masonry and 2.15 lakh cum of concrete in the Dam, Spillway, Power House and other appurtenant works. Extensive tests have been conducted on various materials used in the dam i.e cement, stone, coarse and fine aggregates, steel, copper sheet, PVC waterstop etc. Design mix for concrete and mortar have been developed at the Rengali Main Laboratory for use in the dam. Basic research have also been carried out to utilise Flyash from Talcher Thermal Power Station (T.T.P.S) in mortar and concrete due to proximity of Thermal Plant from dam site and to economise in the cost of cement. The tests carried out at Project laboratory was cross checked at CSMRS, New Delhi and the results communicated to CWC for incorporating in

Table No- 4.26 Test results of Stone samples of Salemunda Quarry

Identification Mark.	Size of specimen in inches	Comp. Strength kg/cm ²	Remarks
1	2	3	4
A A A A A A	2 x 2 x 2 - do - - do - -do- -do- -do-	1933.40 1722.50 1344.60 1212.80 1634.60 1573.10	1) The stones were cut into 2” cubes and soaked in water for 24 hrs. and tested in wet condition at Hirakud Research station. 2) The stones shall be hard dense, durable, tough sound and clean. They should be free from decay, weathered faces, soft seams, adhering coatings, sand holes, veins, flaws, cracks and other defects and must have as far as possible uniform colour and texture. Stones of compressive strength less than 1000 kg/cm² was not used in masonry works. 3) (*) Rejected
B B B B B B	2 x 2 x2 -do- -do- -do- -do- -do-	1371.00 1792.80 1652.20 1318.30 1441.30 1230.40	
C C C C C C	2 x 2 x 2 -do- -do- -do- -do- -do-	738.20(*) 1441.30 790.90 (*) 1388.60 1652.20 1581.90	

Table No- 4.27 Test results of Stone Samples of Salemunda Quarry

Identification Mark.	Size of specimen in inches	Compressive Strength in kg/cm ²	Remarks
1	2	3	4
A	2 x 2 x 2	1327.97	The veins were kept in vertical position during testing. (The samples were tested at Hirakud Research Station)
A	-do-	667.96	
A	-do-	669.59	
S 14	2 x 2 x 2	826.11	
S 14	-do-	817.32	
S 14	-do-	878.84	
S 15	2 x 2 x 2	653.93	
S 15	-do-	1441.30	
S 15	-do-	1379.78	
S 16	2 X 2 x 2	1590.70	
S 16	-do-	1608.28	
S 16	-do-	1432.43	
S 17	2 x 2 x 2	1537.38	
S 17	-do-	1116.64	
S 17	-do-	1450.08	
S 18	2 x 2 x 2	1353.41	
S 18	-do-	1331.70	
S 18	-do-	1309.47	
S 19	2 x 2 x 2	1588.12	
S 19	-do-	1364.58	
S 19	-do-	1494.70	

Table No-4.28 Test result of Stone samples collected from Foundation.

Identification mark	Size of specimen (inch)	Compressive Strenth in kg/cm ²	Remarks
1	2	3	4
S 1	2 X 2 X 2	1617.06	The stones were cut into 2" cubes and soaked in water for 24 hours. Tested in wet condition at Hirakud Research Station.
S 1	2 X 2 X 2	1511.60	
S 1	2 X 2 X 2	1247.95	
S 3	2 X 2 X 2	1438.87	
S 3	2 X 2 X 2	1335.84	
S 3	2 X 2 X 2	1283.10	
S 4	2 X 2 X 2	1370.99	
S 4	2 X 2 X 2	1406.41	
S 4	2 X 2 X 2	1283.10	
S 5	2 X 2 X 2	1265.53	
S 5	2 X 2 X 2	1327.05	
S 5	2 X 2 X 2	1704.95	
N 1 (*) Block 48	2 X 2 X 2	1002.00	* Norite
N 1 (*)	2 X 2 X 2	1009.00	
N1 (*)	2 X 2 X 2	206.20	
N 2 (*) Block 46	2 X 2 X 2	615.00	
N2 (*)	2 X 2 X 2	1146.00	
N2 (*)	2 X 2 X 2	1037.00	

Table- 4.29 Physical tests on Coarse aggregates

Sl. No.	Date of test	Specific Gravity	Precent absorp- tion	Los Angels abrasion test(% loss 500 revolutions)	Remarks
1	2	3	4	5	6
1.	6.8.77	2.74	0.02	22.0	Hard granite excavated from the foundation
2.	10.8.77	2.71	0.025	24.0	
3.	18.8.77	2.70	0.17	26.2	
4.	24.8.77	2.70	0.160	27.6	
5.	29.9.77	2.77	0.036	24.2	
6.	5.10.77	2.72	0.22	-	
7.	11.10.77	2.70	0.35	-	
8.	6.2.78	2.70	0.40	-	
9.	20.7.78	2.75	0.46	-	
10.	23.1.79	2.69	0.45	-	

Facilities exist in the Project laboratory to conduct following tests on aggregate samples to assess their suitability for use in cement concrete.

- (a) Particle size grading
- (b) Specific gravity
- (c) Percentage absorption.
- (d) Los-Angles abrasion test
- (e) Impact test.

The specific gravity and percentage of absorption varied between 2.69 to 2.77 and 0.02 to 0.46 percent respectively.

Investigations were made to evolve the most workable grading in the laboratory to be adopted in the field for 75mm and 40mm maximum size aggregates. It was aimed to find out a grading which will have maximum density and minimum voids. The results are tabulated in Table-4.30 and Table- 4.31.

Table No- 4.30 Aggregate size fraction 75 mm and down

Sl No	Percentage proportion			F.M	Loose		Rodded	
	75 mm	40 mm	20 mm		Unit Weight kg/litre	Void (%)	Unit Weight kg/lit.	Void (%)
1.	50	35	15	8.23	1.76	36.10	1.80	35.20
2.	45	35	20	8.15	1.74	37.20	1.81	34.66
3.	40	40	20	8.08	1.73	37.50	1.80	35.20
4.	35	40	25	8.03	1.79	35.37	1.83	33.90
5.	30	40	30	7.93	1.76	36.10	1.87	32.50

Note - Recommended for use.

Table- 4.31 Aggregate size 40 mm and down

Sl No.	Percentage proportion		F.No.	Unit weight in kg / litre			Void %	Remarks
	40mm	20mm		Loose	Rodded	Vibrated		
1.	40	60	7.54	1.58	1.65	1.80	33.33	i) Specific gravity = 2.70
2.	45	55	7.66	1.60	1.67	1.78	34.08	
3.	50	50	7.91	1.55	1.63	1.71	36.60	
4.	55	45	7.94	1.57	1.64	1.85	31.50	ii) Percentage of absorption = 0.16 iii) Suitable for use
5.	60	40	7.85	1.58	1.65	1.85	31.50	
6.	65	35	7.86	1.60	1.69	1.87	30.70	

4.3.2.2 Tests on Fine Aggregates :

Natural sand occurring in river bed and crushed sand from size fraction of 0.075 mm to 4.75 mm are generally considered as fine aggregate. When natural sand does not conform to the specification, defects in grading is corrected by blending with crushed sand.

Sand available along the bed of the river Brahmani is coarse, clean and suitable both for mortar and concrete. An intensive investigation was carried out to determine the suitability of locally available sand and the range of fineness moduli of different sand quarries is furnished below in the Table-4.32.

Table- 4.32 F.M of Sand

Sl. No.	Location of sand Quarry	Range of F.M
1.	Rohila	2.02 - 2.94
2.	Hatiadanda	1.76 - 1.98
3.	Chilanti	2.76
4.	Deonalli	1.93 - 2.33
5.	Balaram	3.39

Table No- 4.33 Physical Characteristics of Fine Aggregates

Sl. No.	F.M.	Specific Gravity	Silt Content in percentage	Remarks
1.	2.	3.	4	5
1.	2.62	2.66	1.15	Sand of F.M. less than 2.3 was not used either in concrete or in masonry works of Rengali Dam. The F.M. of sand used both for concrete and masonry of Spillway of Hirakud Dam was in the range of 2.00 to 2.60.
2.	2.54	2.64	1.08	
3.	2.69	2.61	1.28	
4.	2.74	2.63	1.07	
5.	2.70	2.58	0.95	
6.	2.89	2.65	2.00	
7.	2.41	2.63	2.00	
8.	2.56	2.62	Trace	
9.	2.30	2.60	Trace	
10.	2.35	2.66	Trace	

4.3.2.3 Compressive strength of various Cements used in the project:**Table- 4.34 Test result of Compressive Strength of Cement, Bargarh (Odisha)**

Sl No.	Identification Mark	Date of Testing	Average compressive strength 7 days kg/cm ²	Source
1	2	3	4	5
1.	1348-50	17.1.80	236.66	Store
2.	1646-48	28.2.80	423.33	Store
3.	1740-42	11.3.80	253.33	Store
4.	1982-84	10.4.80	313.33	Store
5.	2218-20	9.5.80	296.66	Store
6.	2511-13	28.6.80	390.00	Store
7.	2547-48	4.7.80	320.00	Store

Contd..

1	2	3	4	5
8.	2748-50	4.8.80	360.00	Store
9.	3043-45	18.09.80	327.00	Store
10.	3064-66	1.10.80	227.00	Store
11.	3265-77	3.11.80	303.00	Store
12.	3481-83	1.12.80	223.33	Store
13.	100-02	28.1.81	273.33	Store
14.	232-34	18.2.81	320.00	Store
15.	349-351	2.4.81	300.00	Store
16.	495-97	2.4.81	243.33	Site
17.	679-81	2.5.81	240.00	Store
18.	902-04	19.6.81	250.00	Store
19.	956-58	8.7.81	236.66	Store
20.	1055-57	4.8.81	223.33	Store
21.	1251-53	2.9.81	280.00	Store
22.	1375-77	28.10.81	226.66	Site
23.	1441-43	14.11.81	273.33	Site
24.	1681-83	22.1.82	233.33	Site
25.	1781-83	27.2.82	223.33	Store
26.	1787-89	3.3.82	376.66	Store
27.	1921-23	12.4.82	273.33	Store
28.	2014-16	21.5.82	223.33	Site
29.	2071-73	15.6.82	250.00	Store
30.	2110-2112	12.7.82	306.66	Store
31.	2131-33	12.8.82	230.00	Store
32.	2176-78	15.10.82	230.00	Site
33.	2244-46	21.11.82	240.00	Site
34.	2291-93	24.12.82	306.66	Site
35.	2378-80	28.2.83	226.66	Site
36.	2438-40	23.3.83	243.33	Site
37.	2474-76	8.4.83	240.00	Site
38.	2561-63	1.6.83	306.66	Site
39.	2570-72	4.6.83	280.00	Site
40.	2608-10	20.7.83	276.66	Store
41.	2644-46	5.9.83	266.66	Store
41 (i)	2665.67	10.10.83	223.33	Store
42.	2713-15	21.11.83	223.33	Store
43.	2731-33	6.12.83	243.33	Site

Contd..

1	2	3	4	5
44.	2737-39	8.12.83	263.33	Site
45.	2833-35	3.2.84	233.33	Store
46.	2863.-65	16.2.84	253.33	Site
47.	2917-19	23.3.84	276.66	Site
48.	2953-55	6.4.84	300.00	Site
48(i)	2959-61	9.4.84	313.33	Site
49.	2989-91	2.5.84	230.00	Site
50.	3001.03	15.5.84	246.66	Site
51.	3070-72	16.6.84	286.66	Site
52.	3097-99	4.10.84	233.33	Store
53.	3130-32	22.12.84	286.66	Site
54.	3151-53	5.3.85	273.33	Store
55.	3163-65	23.4.85	243.33	Store
56.	3178-80	4.6.85	233.33	Store
57.	3196-98	17.8.85	243.33	Store
58.	3205-07	17.9.85	233.33	Store
59.	3211.13	30.9.85	223.33	Store
60.	3259-61	30.4.86	223.33	Store



Rengali Dam showing Ski-jump Bucket and Hydraulic Jump

Table- 4.35 Orissa Cement Limited, Rajgangpur (Odisha)

Sl No	Identification mark	Date of testing	Average compressive strength 7 days(kg/cm ²)	Compressive strength kg/cm ² 28 days	Remarks
1	2	3	4	5	6
1	1057-62	14.12.79/4.1.80	253.33	473.33	Store
2	1228-33	2.1.80/23.1.80	233.33	320.00	Store
3	1453-58	1.2.80/22.2.80	350.00	396.66	Store
4	1536-41	12.2.80/4.3.80	313.33	416.66	Store
5	1907-09	28.3.80	233.33	-	Site
6	2355-57	30.5.80	293.33	-	Site
7	3022-27	6.9.80/7.10.80	367.00	400.00	Store
8	3148-50	10.10.80	263.00	-	Site
9	3637-39	19.12.80	290.00	-	Site
10	19-24	13.1.81/3.2.81	347.00	506.66	Store
11	106-11	29.1.81/19.2.81	340.00	386.66	Store
12	190-95	12.2.81/5.3.81	333.33	650.00	Store
13	386-91	17.3.81/7.4.81	293.00	433.33	Store
14	624-26	22.4.81	263.00		Store
15	736-38	14.5.81	230.00		Store
16	762-67	13.7.81/3.8.81	240.00	356.66	Store
17	1221-26	25.8.81/14.9.81	263.33	390.00	Store
18	1351-56	24.10.81/14.11.81	243.33	276.66	Store
19	1405-10	5.11.81/26.11.81	223.33	386.66	Store
20	1654-59	13.1.82/30.1.82	223.33	300.00	Store
21	1802-04	9.3.82	246.66	-	Store
22	1915-17	10.4.82	263.33	-	Store
23	1990-92	10.5.82	240.00	-	Store
24	2065-67	10.6.82	283.33	-	Store
25	2095-97	1.7.82	266.66	-	Store
26	2399-01	8.3.83	293.33	-	Store
27	2947-49	4.4.84	236.66	-	Store
28	2974-76	24.4.84	236.66	-	Store
29	2980-82	3.5.84	263.33	-	Store
30	2986-88	5.5.84	220.00	-	Store
31	3076-78	3.7.84	236.66	-	Store
32	3082-84	3.8.84	223.33	-	Store
33	3091-93	10.9.84	233.33	-	Store
34	3106-08	19.10.84	236.66	-	Store
35	3115-17	9.11.84	233.33	-	Store
36	3127-29	12.12.84	236.66	-	Store

Table - 4.36 Korea Cement

Sl.No.	Identification mark	Av. compr. strength (kg/cm ²)		Source
		at 7days	at 28 days	
1	2104-06	25.04.80	330.30	Site
2	2352-54	29.5.80	356.66	Site
3	2403-05	4.6.80	386.66	Site
4	2544-46	3.7.80	360.00	Site
5	2733-35	1.8.80	367.00	Site
6	3011-12	13.9.80	373.00	Site
7	3442-44	22.11.80	290.00	Site
8	3516-18	4.12.80	420.00	Site
9	3727-29	2.1.81	273.00	Site
10	292-94	27.2.81	293.00	Site Store
11	2029-31	26.5.82	243.33	Site

Table - 4.37 Onada Cement(Japan)

Sl/No	IdentificationMark	Date of testing	Av. compressive Strength 7 days in kg/cm ²	Source
1	2	3	4	5
1	3262-64	1.11.80	390	Site
2	3271-73	3.11.80	413	Site
3	3328-30	1.11.80	470	Site
4	3478-80	28.11.80	487	Site
5	3498-3500	2.12.80	250	Site
6	3522-24	4.12.80	353	Site
7	3537-39	5.12.80	393	Site
8	3754-56	6.1.81	473	Site
9	3761-62	7.1.81	327	Site
10	28-30	14.1.81	280	Site
11	70-72	21.1.81	373	Site
12	154-56	5.2.81	413	Site
13	202-04	13.2.81	467	Site
14	268-70	23.2.81	227	Site
15	334-36	4.3.81	310	Site
16	685-87	2.5.81	350	Site
17	694-96	5.5.81	387	Site

4.3.2.4 Comparison of 7 days and 28 days Compressive strength of cement of different brands

Some times it is argued that 7 days compressive strength of ordinary portland cement is not a true indication of its compressive strength because it may have different 28 days strength. To study this aspect an attempt was made to determine the rate of strength development of different brands of cement used in dam construction.

Table- 4.38 Hira Cement, Bargarh

Sl	Identification Mark.	Date of testing	Average compressive strength of cement in kg/cm ² at		Remarks
			7 days	28 days	
1	2	3	4	5	6
1.	1671-76	8.6.78	176.66	306.66	As per ISI 269-1967 compressive strength at 7 days=220 kg/cm ² Tests conducted with locally available sand instead of standard (Ennore) sand.
2.	1692-97	19.6.78	203.33	270.00	
3.	1698-1703	20.6.78	210.00	293.33	
4.	1704-09	21.6.78	246.66	360.00	
5.	1716-21	23.6.78	210.00	416.66	
6.	1770-75	4.7.78	183.33	356.66	
7.	1815-20	18.7.78	403.33	523.33	
8.	1863-68	7.8.78	180.00	356.66	
9.	1960-65	23.8.78	186.66	313.33	
10.	2032-37	21.9.78	223.33	406.66	
11.	2056-61	23.9.78	223.33	326.66	
12.	2074-79	27.9.78	206.66	363.33	
13.	2125-30	3.10.78	230.00	276.66	
14.	2209-14	24.10.78	200.00	393.33	
15.	2229-34	26.10.78	230.00	300.00	
16.	2406-11	25.11.78	233.33	323.33	
17.	2419-24	27.11.78	200.00	356.66	
18.	2438.43	29.11.78	240.00	373.33	
19.	2584-89	15.12.78	336.66	360.00	
20.	2950-55	16.2.79	183.33	320.00	

Table- 4.39 Orissa Cement Limited, Rajgangpur

Sl	Identification Mark.	Date of testing	Average compressive strength (kg/cm ²) at	
			7 days	28 days
1	2	3	4	5
1.	1644-49	2.6.78	370.00	416.66
2.	1650-55	3.6.78	475.00	496.66
3.	1659-64	6.6.78	185.00	320.00
4.	1665-70	7.6.78	360.00	500.00
5.	1722-27	24.6.78	300.00	426.66
6.	1728-33	26.6.78	286.66	410.00
7.	1734-39	27.6.78	300.00	436.00
8.	1740-45	28.6.78	340.00	430.00
9.	1746-51	29.6.78	360.00	476.66
10.	1752-57	30.6.78	390.00	510.00
11.	1758-63	1.7.78	380.00	540.00
12.	1764-69	3.7.78	366.66	466.66
13.	1932-37	18.8.78	393.33	566.66
14.	2451-56	30.11.78	430.00	573.33
15.	2488-93	5.12.78	245.00	356.66
16.	2506-11	7.12.78	363.33	436.66
17.	2519-24	8.12.78	363.33	543.33
18.	2534-39	11.12.78	410.00	540.00
19.	2547.52	12.12.79	370.00	510.00
20.	2804-09	19.1.79	370.00	436.66
21.	2891-96	8.2.79	290.00	503.33

Table - 4.40 Imported Cement

Sl	Identification Mark.	Date of testing	Average Compressive strength in kg/cm ² at		Remarks
			7 days	28 days	
1	2	3	4	5	6
1.	2747-52	12.1.79	473.33	573.33	As per ISI 269-1967 compressive strength at 7 days=220 kg/cm ² Test conducted with locally available sand instead of (Ennore) sand.
2.	2760-65	15.1.79	290.00	400.00	
3.	2779-84	17.1.79	393.33	530.00	
4.	2792-97	18.1.79	380.00	510.00	
5.	2846-51	25.1.79	336.66	460.00	
6.	2939-44	15.2.79	343.33	533.33	
7.	2988-93	20.2.79	416.66	536.66	
8.	3013-18	22.2.79	445.00	535.00	
9.	3028-33	23.2.79	396.66	513.33	
10.	3038-43	24.2.79	420.00	496.66	

4.3.2.5 Corelation between 7 days and 28 days Compressive strength of Cement

The cement samples tested from June 1978 to October 1978 were taken into consideration in developing this relation. Total 56 nos. of samples were considered, out of which 43 are from Hira Cement works, Bargarh and the rest-13 are from O.C.L. Rajgangapur. The average, minimum and maximum compressive strength of above samples are 195, 63 and 456 kg/cm² respectively.

The relation between 7 days and 28 days compressive strength indicates that the ordinary portland cement with high 7 days strength gives proportionately high 28 days strength.

The equation of the line is given by $Y=0.8376 X +175.09$ and co-efficient of co-relation is 0.854.

4.3.2.6 Effect of compaction on Compressive strength of Cement

The laboratory is equipped with vibration machine fitted with 0.5 H.P. motor having r.p.m. of 2850. In the event power failure for conducting day-to-day tests hand (manual) compaction is resorted to. A study was made to determine the effect of machine vibration-vs-hand compaction and in all the cases it is seen that the strength increases from 6.5% to 50.4% with an average increase of 30%. The test results are given in table No-4.41.

Table - 4.41 Effect of compaction:

Sl	Identi- fication mark	Mix Pro- portion	Compressive strength at 7 days in kg/cm ²		% of Devia- tion	Remarks
			Hand compaction	Vibration		
1	2	3	4	5	6	7
1.	1218-20	1:3	340	440	29.4	i) Hand compa- ction was done with poking rod of size 25mm X 12 mm.
2.	1230-32	1:3	335	400	19.4	
3.	1239-41	1:3	335	410	22.4	
4.	1242-44	1:3	375	420	12.0	
5.	1245-47	1:3	310	330	6.5	
6.	1260-62	1:3	335	430	28.4	ii) Vibration was done on the vibra- tion table (C.C.M.I) fitted with 0.5 H.P. motor having 2850 r.p.m.
7.	1263-65	1:3	250	300	20.0	
8.	1269-71	1:3	365	390	6.8	
9.	1275-77	1:3	195	240	23.0	
10.	1281-83	1:3	310	430	38.7	
11.	1293-96	1:3	310	430	38.7	iii) All the samples were cured in water for 7 days.
12.	1297-1300	1:3	113	170	50.4	
13.	1301-04	1:3	120	180	50.0	
14.	1305-08	1:3	87	120	37.9	
15.	1316-19	1:3	103	150	45.6	
16.	1323-26	1:3	243	320	31.7	
17.	1337-40	1:3	120	170	41.7	

4.3.2.7 Test Results of other Materials

4.3.2.7.1 Tests on Tor steel rods :-

As facilities donot exist at Rengali Main laboratory for which two samples of torsteel rods were sent to University college of Engineering, Burla, Sambalpur for testing in accordance with IS 1786-1966. The test results are tabulated below.

Table No- 4.42 Test results of Torsteel

Sl. No.	Dia of torsteel	Maximum load in tons	Proof load in tons	Ultmate tensile stress in kg/mm ²	Proof stress in kg/mm ²	Percentage of elongation
1.	32	58.5	54.5	72.71	67.74	42
2.	25	28.0	25.0	57.03	50.91	23
3.	20	19.5	17.5	62.10	55.73	30
4.	16	13.0	9.75	64.65	48.49	32.5

N.B. Tested on 10.08.1977

Table - 4.43 Test results of Torsteel

Sl. No.	Dia of torsteel in mm.	Maximum load in tons	Proof load in tons	Ultmate tensile stress in kg/mm ²	Proof stress in kg/sqmm.	Percentage of elongation
1.	32	42.5	30.5	52.82	37.90	19
2.	25	26.16	24.5	54.18	49.90	19
3.	20	21.4	19.4	68.15	61.78	18
4.	16	16.2	12.2	80.59	60.69	18.75

N.B. Tested on 13.01.1978

4.3.2.7.2 Testing of P.V.C. Waterstop

Two samples of “Chemplast Vinyl P.V.C. water Stop” supplied by Chemicals & Plastics Indian Ltd. Calcutta were sent to National Test House, Calcutta for testing the suitability. The samples were subjected to following tests and the results are furnished below. The specimen size of the P.V.C. sheets was 30 X 30 cm. The tests conform to the requirements laid down in Drg. No. RDP 136 of C.W.C. (P.V.C. seal specification).

Table - 4.44 Test results of PVC Waterstop

Sl. NO.	Details	Sample No. 1	Sample No.2
1.	Tensile strength, kg/cm ²	173	169
2.	Ultimate elongation, percent	310	310
3.	Tear strength, kg/cm ²	92	88
4.	Accelerated extraction (after immersion of test pieces in solution containing 5 gm. C.P. Na OH and 5 gm. C.P. KOH per litre of distilled water at 140°-150°F)		
i)	Tensile strength kg/cm ²	182	190
ii)	Ultimate elongation, percent	285	287
5.	Stiffness in flexure kg/cm ²	131	123

4.3.2.7.3 Test on Copper Sheets

Table- 4.45

One row of copper sheet is used in the contraction joint of the Dam. The test certificate of the Rashtriya Metal Industries Ltd. is given below :-

Sl.No.	Constituents	Percentage	Remarks
1.	Copper	99.87	T.C. No.- 6097/ 5.12.76 Invoice No. R/694 Dt.04.12.76
2.	Arsenic	0.03	
3.	Bismuth	Nil	
4.	Iron	0.015	
5.	Lead	Nil.	
6.	Nickel	Trace	
7.	Selenium	Nil.	
8.	Tellurium	Nil.	
9.	Phosphorous	0.033	

4.3.2.7.4 Test on Manila Rope

Table - 4.46

Two samples of manila rope, one of 0'1" dia and the other of 0' $\frac{3}{4}$ " inch dia were sent to University College of Engineering, Burla for testing in accordance with IS 1084/1968. The material was supplied by M/S Commercial Enterprisers through Executive Engineer, Stores & Mechanical Division. The test results are tabulated below

Sl	Date of Test	Dia of Specimen	Breaking load in M.T	% of elongation	Tensile Strength in kg/sq.mm
1.	28.4.78	$\frac{3}{4}$ "	2.25	81.8	7.16
2.	28.4.78	1"	3.75	166.7	7.64

4.3.2.8 Effect of Common Sugar on Setting time and Compressive Strength of Portland Cement

Sugar is known to exhibit retarding action in the setting and hardening of cement paste. Addition of excess amount of sugar may delay the set indefinitely. However when a very small quantity is used, satisfactory retarding effect may be obtained without detrimental effect on ultimate strength. Some study was under taken in Rengali laboratory to assess its effect and the results are as follows (vide Table 4.47)

Table - 4.47 Effect of Common Sugar on properties of Cement.

Sl	Identification Mark.	% of sugar by wt. of cement	Initial Setting time in min	Final Setting time in min	Comp. strength at 7 days kg/cm ²
1.	62-63	0.00	42	245	240
2.	60-61	0.05	69	269	250
3.	58-59	0.06	103	290	245
4.	56-57	0.07	125	320	260
5.	54-55	0.08	156	356	270
6.	52-53	0.09	172	372	200

Following conclusion are drawn from the above test results.

- Sugar acts as a Set retarding and Water reducing admixture in cement paste.
- Addition of sugar to the extent of 0.08% by weight of cement increases compressive strength by 12.5% at 7 days. With increase in percentage of sugar beyond 0.08%, the strength decreases and the setting time increases considerably.

4.3.2.9 Testing of Cement Mortar for Masonry :

4.3.2.9.1. 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 surface 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 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 admission of a report on the day's masonry work, formed part of the inspectors work.

Testing of mortar is conducted regularly for its compressive strength at 28 days & 90 days in accordance with IS.2250-1965. The samples are collected from worksite every day. The specimens are of 15 cm cubes.

For the blocks 38,39,40, and 41 in which the masonry has been completed to the required level, graphs were plotted for 1:3 and 1:4 mortar for both 28 days and 90 days compressive strength and (+/-)15% variation from the design strength has been marked.

4.3.2.9.2 Corelation between 28 days and 90 days Compressive strength of Cement mortar:-

The strength developments is an important factor before the masonry is loaded to the full extent. The rate of construction should be consistent with the rate of strength development of masonry mortars.

4.3.2.9.3 For 1:4 Cement Mortar:

A study was undertaken to determine the relation between 28 days and 90 days compressive strength of 1:4 (cement:sand) mortar. The study is based on field samples collected during masonry construction from April 77 to September 78. In total 99 samples have been considered for the purpose. The samples whose 28 days compressive strength were below 105 kg/cm² have not been taken into consideration.

The average compressive strength of above 99 samples included in the study is 148.55 kg/cm² and 192.79 kg/cm² at 28 days and 90 days respectively. This indicates that the relative compressive strength at 90 days is 1.297 times more than that at 28 days which satisfies the guide lines prescribed in IS 2250-1965.

The equation of line is $Y=0.64997X + 96.235$.

4.3.2.9.4. For 1:3 Cement Mortar

Total 57 sets of cubes of 1:3 mix proportions were collected from site between 5.10.76 to 27.4.78. The samples whose 28 days compressive strength were below 175 kg/cm² have not been taken into consideration for developing this relation.

The maximum, minimum and average 28 days compressive strength of these 57 samples are 329.62, 175.50 and 229.02 kg/cm² respectively and corresponding strength at 90 days are 417.03, 184.44 and 273.71 kg/cm² respectively.

The equation of the line is given by $Y = 1.0484X + 33.596$ and the coefficient of co-relation is 0.8674.

Table- 4.48 Compressive Strength result of 1:3 & 1:4 Mortar cubes

Sample No.	Mix proportion	Location (Block)	Date of testing	Compressive Strength in kg/cm ² at 28 days
1	2	3	4	5
7563-65	1:4	6	1.1.80	105.18
7566-68	1:4	33	1.1.80	170.37
7575-77	1:3	8	4.1.80	211.11
7581-83	1:4	44	5.1.80	160.00
7587-89	1:4	4	7.1.80	131.11
7596-98	1:3	46	8.1.80	157.03
7695-97	1:4	49	4.2.80	137.78
7824-26	1:4	21	8.3.80	96.29
8010-12	1:3	9	15.4.80	231.11
8254-56	1:3	24	23.5.80	328.89
8324-26	1:4	7	10.6.80	140.00
8430-32	1:4	48	14.7.80	171.85
8580-82	1:4	51	18.8.80	93.33
8671-78	1:3	43	24.9.80	215.55
8713-14	1:3	31	3.10.80	170.37
8863-65	1:3	28	10.11.80	209.63
8977-79	1:4	47	15.12.80	93.33
9079-81	1:4	25	8.1.81	217.77
9190-92	1:3	46	6.2.81	316.29
9226-28	1:4	8	17.2.81	91.11
3376-78	1:3	19	10.3.81	219.25
9764-66	1:4	9	18.5.81	106.66
9898-9900	1:4	3	16.6.81	91.85
9940-42	1:4	44	2.7.81	114.80
10090-92	1:4	4	10.8.81	94.07
10186-188	1:4	28	22.9.81	163.70
10596-598	1:4	3	31.12.81	95.55
10686-688	1:4	20	12.1.82	107.40
10791-793	1:4	2	9.2.82	123.70
10010-12	1:4	15	23.3.82	133.33

1	2	3	4	5
11061-63	1:4	16	7.4.82	127.40
11193-195	1:4	22	14.5.82	114.81
11355-357	1:4	17 d/s	30.6.82	128.88
11385-387	1:4	1	22.7.82	121.47
11442-444	1:4	23	2.11.82	117.03
11430-432	1:4	26	29.10.82	124.44
11682-684	1:4	11	27.1.83	136.27
11730-732	1:4	27	23.2.83	117.03
11757-761	1:4	15	9.3.83	122.96
11789-791	1:4	26	27.4.83	97.03
11795-797	1:4	24	5.5.83	114.07
11819-821	1:4	19	14.6.83	145.18
11868-870	1:4	2 Lt. Trg. Wall	16.3.84	120.00
11913-915	1:4	6 Lt. Trg. Wall	25.4.84	104.44

4.3.2.10 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 the 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 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 compressive strength determination.

By observing the concrete at the mixing as well as at the placement 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 type of materials used, the production of concrete and its placing, improvements necessary and defects to be rectified was submitted to the Chief Engineer. The report was routed through the field engineers so that they could furnish their comments for the Chief Engineer to issue instruction whenever necessary.

At work sites the quality of materials and the operations of mixing, conveying, placing, compacting and curing the concrete are checked to ensure elimination or minimization of the factors that would, otherwise, nullify the efforts spent on selection of materials and design of concrete mixes. The cost of such inspection is small in comparison to the insurance of quality in the structure expected to be achieved therefrom.

Specification of concrete to be used in O.F and N.O.F section have been furnished in Table-4.16 and 4.17

Table- 4.49 Quantity of Ingredients for different Mix Design.

Sl No	Mix	Max size of aggregate	Quantity per bag of Cement (kg)			W.C ratio	Quantity of cement per 1 cum of concrete
			cement	sand	Aggregate		
1	2	3	4	5	6	7	8
1.	M100	40 mm	50	205	367	0.84	4 bags
2.	M 150	40 mm	50	140	268	0.65	5 bags
3.	M 200	40 mm	50	104	222	0.50	6 bags
4.	M 250	40 mm	50	87	185	0.45	7 bags
		20 mm	50	87	185	0.45	7 bags
5.	M 300	20 mm	50	74	157	0.38	8 bags

The aggregate used in concrete to satisfy the following grading condition.

Seive Size (mm)	Percentage of passing	
	Concrete using 40 mm maxm. size	Concrete using 20 mm maxm. size.
(1)	(2)	(3)
40	95-100	100
20	30-70	95-100
10	10-35	25-55
4.75	0-5	0-10

The aggregate produced by crushers was stacked upto 20 mm maximum size (chips) and from 20 mm to 40 mm maximum size (metal) separately and the chips and metals were combined and used for concrete with 40 mm size down as per grading condition.

During concreting the samples were collected and tested in the Project laboratory to check the compressive strength. The test results are enclosed vide Table-4.50.

Table- 4.50 Compressive strength of M₂₅₀ concrete

Sample No.	Date of casting	% of metal and chips	Location block	Compressive strength at 28 days (kg/cm ²)
1	2	3	4	5
2788-90	21.8.81	M-40%, C - 60%	29	239.99
2905-07	30.10.81	M-50%,C - 50%	32	248.88
2911-13	6.11.81	M -50%, C-50%	30	242.99
3086-88	19.4.82	M-50%. C-50%	29	222.96
3549-51	3.2.83	M-40%,C-60%	23	226.66
3555-57	4.2.83	M-40%, C-60%	37	274.81
3597-99	24.2.82	M-40%, C-60%	27	263.67
3610-12	28.2.83	M-40%, C-60%	19	259.25
3616-18	2.3.83	M-40%, C-60%	31	269.62
3629-31	15.3.83	M-40%, C-60%	29	300.73
3635-37	15.3.83	M-50%, C-50%	29	340.73
3647-49	21.3.83	M-40%, C-60%	22	269.62
3653-55	28.3.83	M-40%, C-60%	23	318.51
3671-73	6.4.83	C-100%	38	315.55
3677-79	18.4.83	M-30%, C-70%	28	226.66
3683-85	19.4.83	M-30%, C-70%	Unit -3 PH	277.03
3695-97	29.4.83	C-100%	48	257.03
3707-09	12.5.83	M-60%, C-40%	Rt. batching Plant-1	253.33
3731-33	23.5.83	C-100%	Bridge	297.77
3737-39	25.5.83	M-50%, C-50%	33	259.99
3749-51	27.7.83	C-100%	35	254.81
3782-84	22.8.83	C-100%	Unit -II Pedestals	232.59
3902-04	12.12.84	M-30%, C-70%	20	244.44
3929-31	22.2.84	-100%	28/27	297.77
3965-67	2.5.84	M-40%, C- 60%	Lt. batching Plant-3	254.81
4006-08	1.6.84	C-100%	21	242.96

4.3.2.11 Design Mix for 1:2.5 Mortar

As 1:3 (Cement:sand) mortar gave low result less than 175kg/cm^2 an attempt was made to evolve a design with 1:2.5 cement mortar with varying water cement ratio to achieve compressive strength of 175 kg/cm^2 at 28 days. (A review from Dt.14.6.78 to 2.5.79 was made taking 119 samples into consideration. Out of these samples, 47% do not comply to the design requirements)

Table- 4.51 Test result of 1:2.5 cement mortar

Lab identification mark /Date of testing		W.C ration	F.M of sand	Av.Compr. strength in kg/cm^2 at		Av.Cement strength at 7 days in (*) kg/cm^2
28 days	90 days			28 days	90 days	
46-48 22.3.78	43-44 23.5.78	0.40	2.50	202.22	248.89	170
52-54 23.3.78	49-51 24.5.78	0.45	2.50	222.22	282.22	177
40-42 21.3.78	37-39 22.5.78	0.50	2.50	203.71	241.11	170
61-63 25.3.78	64-66 26.5.78	0.40	2.30	168.15	205.92	177
55-56 24.3.78	58-60 25.5.78	0.45	2.30	213.33	254.81	177
74-75 28.3.78	76-78 29.5.78	0.50	2.00	222.22	288.89	427

Note- (*) Local sand was used for cement testing

Following conclusions can be drawn from the test result:-

- (i) The compressive strength of mortar increases with increase in F.M of sand and decrease in water cement ratio.
- (ii) When water cement ratio of 0.40 is adopted, the mix becomes stiff and unworkable thus reducing the strength. Water cement ratio of 0.45 is suitable.
- (iii) Strength of cement is an important parameter for strength of mortar. Sand of F.M less than 2.3 was not used for masonry work. Tests conducted with F.M. of sand as 2.0 and cement strength at 7 days 427 kg/cm^2 the compressive strength of 15 cm. mortar cubes with water cement ratio 0.50 is comparatively very high.

4.3.2.11.1 Corelation between 7 days compressive strength of cement Vs 28 days compressive strength of 1:3 Cement Sand Mortar.

For ordinary portland cement , IS No. 269-1976 specifies limits for different properties such as setting time, soundness, 3 and 7 days compressive strength etc. Out of these properties 7 days compressive strength is most important as far as compressive strength of mortar is concerned.

For determining the compressive strength of cement, standard sand conforming to IS 650-1966 is used and compressive strength of 1:3 cement sand mortar is determined. Quantity of water is decided depending on the consistency. (The W.C ratio approximately 0.38 to 0.40).

For 1:3 cement mortar, river sand of F.M not less than 2.3 was used for Dam work with W.C ratio of 0.50 from workability point of view. A study was undertaken to corelate the compressive strength of cement with that of 1:3 mortar and results are presented in Table 4.52.

Table No. - 4.52 Corelation of Comp. strength of Cement Vs Mortar

Lab Sl. No.	Brand of cement	Date of casting	Av. Compr.Strength of cement in kg/cm ² at		Av. Compr. of 1:3 mortar at 28 days kg/sq.cm
			7 days	28 days	
1-6 & 7M-9M	H.C.W	22.5.79	276.67	430.00	185.19
10-15 & 16M-18M	H.C.W	23.5.79	250.00	386.67	185.92
19-24 & 25M-27M	H.C.W	25.5.79	260.00	403.33	218.51
28-33& 34M-45m	H.C.W	26.5.79	266.67	390.00	182.95
37-42 & 43M-45M	H.C.W	28.5.79	230.00	380.00	152.58
46-51 & 52M-54M	H.C.W	29.5.79	210.00	356.67	166.66
55-60 & 61 M-63M	H.C.W	30.5.79	256.67	-	202.96
64-69 & 70M-72M	Imported	5.6..79	396.67	470.00	262.22
73-78 & 79M -81M	H.C.W	8.6.79	243.33	366.67	148.89
82-87 & 88M-90M	H.C.W	11.6.79	223.33	310.00	159.99
91-96 & 97M-99M	H.C.W	12.6.79	246.67	380.00	160.00
100-105 & 106M-108M	H.C.W	14.6.79	213.33	303.33	167.77

N.B.- H.C.W. is Hira Cement Works, Bargarh, Odisha

4.3.2.11.2 Corelation between Mortar and Masonry

In the zoning drawing for over-flow and non-overflow sections, the strength prescribed for different curing period is with respect to strength of masonry. For ensuring the day-to-day quality of masonry, a corelation is to be established between the masonry cubes with that of mortar cubes for different curing period under identical condition and specification.

C.W.C in their letter No. 3/13/76/C/M.D.D/3476 dated 4.12.75 have recommended following guide lines.

- i) The materials that are used for the laboratory specimens should be the same as proposed for the dam.
- ii) The maximum size of the stone should be 1/4 of the size of the masonry cubes.
- iii) The shape of gradation of the stones being used in the Laboratory specimen should be similar to those being used in the dam proper.
- iv) The quantity of mortar per unit volume of masonry in the Laboratory specimens and the prototype should be similar.

Even though test data for individual stone and mortar can be obtained and their strengths known, such data can not be corelated to determine the strength of masonry. The masonry strength not only depends on the combined action of stone and mortar but also on another factor such as workmanship. Thus failure mechanism of masonry is not clearly known.

Tests on large sized masonry specimens (either 75 to 91 cm. cubes or cylinders) were carried out for Nagarjunsagar, Hirakud, Sri Sailam projects during their construction period. Companion mortar specimens of 15 cm cubes were tested at the project Laboratory.

In Alore, Srisailam and Hidkal, the masonry specimens were cored out from masonry blocks unlike Hirakud where the masonry specimens were manufactured within the confined space of steel forms. This was a favourable condition for consolidation. That is why, the masonry specimens of Hirakud series gave more strength than of companion mortar strength. The masonry specimen strengths were less than the companion mortar specimen strengths at any age for any type of mortar in Alore and Nagarjunsagar test series.

From above test series, it is concluded that masonry strength when built under favourable conditions could surpass the mortar strength and would be equal or a little less than that of mortar when constructed under normal field conditions.

The method of construction at Rengali Dam was identical to that of Hirakud. The 75 cm masonry cubes were tested at Thirumale Iyengar Material Testing Station, Hirakud. As the testing charge per cube was Rs. 2335.00 which was relatively high, this prohibited in developing a relation at different ages.

Tests have indicated that failure in masonry is of progressive type commencing mostly with failure at the contact plane of mortar and stone. Thus the bond between two components is as important as the compressive strength for good masonry.

Masonry cubes of 75 cm size with 1:4, 1:3, 1:1:4 (Cement : Flyash : Sand) and 1:0.75:3 (cement : flyash:sand) have been tested at T.I.M.T station, Hirakud. The results are shown in Table No-4.53, 4.54 and 4.55

Table- 4.53 Compressive Strength of Masonry Cubes (1:4)

Sl.N	Identifi cation	Mix prop ortion volume	Date of Casting	Date of Testing	Block No.	Compressive strength of masonry cube in kg/cm ²		Compressive strength of corresponding 15cm mortar cubes in kg/cm ²
						Intial crack	Crushing	
1.	5 A	1:4	3.12.76	2.2.77	40	70.00	188.99	98.76
2.	6 A	1:4	4.12.76	2.2.77	39	52.50	157.90	151.33
3.	7 A	1:4	6.12.76	3.2.77	39	35.00	178.49	145.50
4.	8 A	1:4	7.12.76	3.2.77	40	38.50	136.49	109.85
5.	12 A	1:4	1.6.77	27.7.77	-	45.50	90.00	134.43
6.	13 A	1:4	2.6.77	26.7.77	-	52.50	129.49	91.78
7.	14 A	1:4	6.6.77	21.7.77	-	63.00	115.49	66.40
8.	15 A	1:4	7.6.77	21.7.77	-	69.99	143.49	243.14
9.	16 A	1:4	8.2.78	20.3.78	36	17.50	115.49	170.23
10.	17 A	1:4	9.2.78	18.3.78	39	52.50	111.99	112.29
11.	18 A	1:4	10.2.78	18.3.78	36	52.50	153.99	143.54
12.	19 A	1:4	11.2.78	17.3.78	39	52.50	157.49	98.30
13.	30 A	1:4	9.5.78	18.8.78	33	62.99	157.49	-
14.	31 A	1:4	10.5.78	18.8.78	33	83.99	153.99	-
15.	32 A	1:4	13.7.78	19.8.78	33	35.00	125.99	-
16.	33 A	1:4	16.7.78	23.8.78	35	56.00	87.49	-
17.	34 A	1:4	14.7.78	21.8.78	35	35.00	104.99	-
18.	35 A	1:4	15.7.78	22.8.78	35	52.50	87.49	-
19.	36 A	1:4	26.9.78	15.11.78	38	53.00	77.00	116.00
20.	37 A	1:4	28.9.78	13.11.78	35	87.00	140.00	117.33
21.	38 A	1:4	29.9.78	13.11.78	38	70.00	157.00	167.50
22.	39 A	1:4	30.9.78	16.11.78	36	32.00	91.00	211.50
23.	40 A	1:4	3.10.78	17.11.78	35	53.00	122.00	140.50
24.	41 A	1:4	4.10.78	17.11.78	35	63.00	122.00	107.00

Table - 4.54 Compressive Strength of Masonry Cubes (1:3)

Sl.No	Identification	Mix proportion	Date of Casting volume	Date of Testing	Block No.	Compressive strength of masonry cube in kg/cm ²		Compressive strength of corresponding 15 cm mortar cubes in kg/cm ²
						Initial	Crushing	
1.	1 A	1:3	22.9.76	4.12.76	-	61.39	164.87	107.10
2.	2 A	1:3	23.9.76	4.12.76	-	61.39	157.80	103.01
3.	3 A	1:3	24.9.76	30.11.76	-	70.16	228.00	150.65
4.	4 A	1:3	27.9.76	30.11.76	-	61.39	196.45	173.73
5.	9 A	1:3	14.2.77	17.5.77	38	113.74	178.49	166.99
6.	10 A	1:3	15.2.77	17.5.77	38	62.99	146.99	102.05
7.	11 A	1:3	17.2.77	13.5.77	38	70.00	157.49	127.93
8.	20 A	1:3	21.3.78	28.4.78	35	78.74	188.98	327.12
9.	21 A	1:3	22.3.78	27.4.78	34	78.74	174.99	196.27
10.	22 A	1:3	23.3.78	29.4.78	35	87.49	153.99	302.38
11.	23 A	1:3	24.3.78	28.4.78	34	87.49	195.98	329.07
12.	24 A	1:3	25.5.78	11.7.78	34	54.30	155.67	322.24
13.	25 A	1:3	3.5.78	11.7.78	34	47.06	213.59	331.91
14.	26 A	1:3	4.5.78	12.7.78	34	90.50	191.87	200.18
15.	27 A	1:3	5.5.78	12.7.78	34	72.40	162.91	214.33
16.	28 A	1:3	6.5.78	13.7.78	33	90.50	195.49	276.83
17.	29 A	1:3	8.5.78	13.7.78	33	90.50	181.01	226.54

4.3.2.12 Use of Flyash in Masonry/ Concrete:

For 1:3 and 1:4 cement mortar, the consumption of cement per cum of mortar is 440 kg and 350 kg respectively. In the masonry dam construction, the quantity of mortar consumed per cum of masonry varies from 45 to 50 %. On the basis of investigations carried out by S.S Rehsi and S.K Garg (vide Civil Engineering & Public Works journal, Sept - Oct.1970) equivalent flyash mortar mix proportions were determined corresponding to 1:3 and 1:4 cement-sand mortar to economise about 20% of cement in the mortar as an initial step. For batching convenience, the determined mix proportions have been suitably rounded off. The properties (compressive strength) of equivalent flyash mortars and their suitability for use in place of plain cement sand mortar are given in Table-4.55 for mix 1:1:4 and in Table- 4.56 for mix 1:0.75:3 and also in Table- 4.57 to 4.59.

Table No. 4.55 Compressive Strength of Cement : Sand : Fly ash cubes (1:1:4)

Sl. No.	Identi- fication mark	Mix Proportion (by wt)	Date of casting	Date of testing of masonry cubes	Compressive strength-kg/cm ²		Compressive strength of 15 cm mortar cubes at 90 days in kg/cm ² (without fly-ash)	Remarks
					75cm masonry cubes with fly ash	15cm flyash mortar cube at 90 days		
1	2	3	4	5	6	7	8	9
1.	205 F	1:1:4	28.10.78	27.1.79	138	116.30	115.55	1) 75 cm masonry cubes were cured at Rengali for 28 days and then trans- ported to TIMTS Hirakud for testing. 2) Minimum compressive strength of stones used is 1000 kg/ cm ² 3) F.M of Sand varied from 2.4 to 2.7.
2.	206 F	1:1:4	28.10.78	27.1.79	123			
3.	220 F	1:1:4	13.11.78	12.2.79	134	102.22	98.89	
4.	221 F	1:1:4	13.11.78	12.2.79	188			
5.	235 F	1:1:4	15.11.78	13.2.79	170	138.51	142.96	
6.	236 F	1:1:4	15.11.78	13.2.79	152			
7.	249 F	1:1:4	17.11.78	15.2.79	185	157.77	169.63	
8.	250 F	1:1:4	17.11.78	15.2.79	174			
9.	359 F	1:1:4	18.12.78	8.2.79	159	164.44	165.93	
10.	360 F	1:1:4	18.12.78	14.2.79	145			
11.	385 F	1:1:4	20.12.78	14.2.79	130	130.37	139.26	
12.	386 F	1:1:4	20.12.78	12.2.79	152			
13.	411 F	1:1:4	22.12.78	16.2.79	174	161.48	172.22	
14.	412 F	1:1:4	22.12.78	9.2.79	145			
15.	437 F	1:1:4	25.12.78	16.2.79	130	148.89	174.07	
16.	438 F	1:1:4	25.12.78	9.2.79	224			

Source: Orissa Engineering Congress Souvenir Dec.1979- 'Utilisation of Flyash for Mortar and Concrete of Rengali Dam Project with special reference to flyash from T.T.P.S - by G.C Sahu- pg.79 Table - 10.

Table No. 4.56
Compressive Strength of Cement : Sand : Fly ash cubes (1:0.75:3)

Sl. No.	Identi- fication mark	Mix Proportion (by wt)	Date of casting	Date of testing of masonry cubes	Compressive strength-kg/cm ²		Compressive strength of 15 cm mortar cubes at 90 days in kg/cm ² (without fly-ash)	Remarks
					75cm masonry cubes with fly ash	15cm flyash mortar cube at 90 days		
1	2	3	4	5	6	7	8	9
1.	451 F	1:0.75:3	12.2.79	3.4.79	253	200.74	250.00	Poor result of masonry cubes from Sl.9 to Sl.16 is due to smaller size of stones used
2.	452 F	1:075:3	12.2.79	3.4.79	243			
3.	465 F	- do -	14.2.79	4.4.79	199	197.77	Rejected	
4.	466 F	- do -	14.2.79	4.4.79	199			
5.	479 F	- do -	16.2.79	6.4.79	170	226.66	278.88	
6.	480 F	- do -	16.2.79	6.4.79	221			
7.	493 F	- do -	19.2.79	9.4.79	159	223.70	284.44	
8.	494 F	- do -	19.2.79	9.4.79	206			
9.	507 F	- do -	6.4.79	29.5.79	94.12	140.00	162.22	
10.	508 F	- do -	6.4.79	28.5.79	86.88			
11.	521 F	- do -	9.4.79	30.5.79	112.22	168.88	184.44	
12.	522 F	- do -	9.4.79	29.5.79	123.08			
13.	535 F	- do -	11.4.79	30.5.79	101.36	178.51	Rejected	
14.	536 F	- do -	11.4.79	31.5.79	108.60			
15.	549 F	- do -	13.4.79	31.5.79	137.56	170.37	160.00	
16.	550 F	- do -	13.4.79	31.5.79	130.32			

(Source : Ibid, pg.80 - Table - 11)

TABLE- 4.57 (A) Compressive Strength of Cement: flyash: Sand Mortar

Lab Sl No.	Mix proportion by weight			Water Cement ratio	F.M of sand	Average compressive strength of 15 cm cubes in kg/cm ² (*)				Average compressive strength of cement at 7 days in kg/cm ²
	Cement	Flyash	Sand			28 days	90 days	180 days	365 days	
121-132	1	0	4	0.65	2.50	81.48	127.40	162.96	165.18	-
133-144	1	1	4	0.85	2.50	111.11	183.73	240.74	248.88	-
145-156	1	1	4	0.90	2.50	103.70	187.40	237.77	240.00	-
157-168	1	1	4	0.95	2.50	94.81	171.85	220.00	-	-
85-96	1	0	3	0.50	2.50	269-63	297.77	317.03	323.71	270(**)
97-108	1	0.75	3	0.80	2.50	251.11	314.82	340.00	377.77	270
109-120	1	0.75	3	0.75	2.50	219.25	257.78	291.85	317.77	270

(*) Average of 3 nos of cubes tested at each age.

(**) Local sand was used for cement testing instead of standard sand.

Note. By adopting above mix proportion, cement can be economised to the extent of 20%. For 1:1:4 mix proportion, thus quantity of cement per cum of mortar is 280 kg. Test results furnished in Table No.4.54 and 4.55 indicate the strength at various ages with cement content of 275 kg. and 250 kg/cum. of mortar. These satisfy the design criterial of 1:3 and 1:4 mortar at 90 days and beyond. (Source : Ibid, Pg. 75, Table 7)

Table- 4.58 Compressive strength of cement: fly ash: sand mortar.

Lab. Sl. No.	Quantity of material required in kg/cum				Av. compressive strength of 15 cm cubes in kg/cm ²				Remark
	Cement	Flyash	Water	Sand	28 days	90 days	180 days		
425 F-433 F	275	100	250	1561	92.22	162.96	194.07		i) O.C.L. Rajgangpur cement was used for the series.
399 F-407 F	275	150	250	1502	101.48	122.22*	202.96		ii) All the specimens were cured under water.
373 F-381 F	275	175	250	1473	102.22	184.44	197.04		iii) (*) Inconsistent.
347 F-355 F	275	200	250	1444	150.37	206.67	251.11		
335 F-343 F	275	225	250	1415	141.48	228.88	264.44		

Table- 4.59 Average compressive strength of cubes

Lab. Sl. No.	Quantity of material required in kg/cum				Av. compr. strength of 15 cm cubes in kg/cm ²			Remark
	Cement	Flyash	Water	Sand	28 days	90 days	180 days	
323 F- 331 F	250	100	250	1582	94.44	164.44	179.25	i) O.C.L. Rajgangpur cement was used for test. ii) All the specimens were cured under water. iii) Tests conducted at C.S.M. R.S. New Delhi with 250 kg. cement and 200% flyash by absolute volume of cement has shown compressive strength of 197 kg/cm ² at 180 days
299 F- 307 F	250	150	250	1524	99.25	161.48	188.89	
263 F- 271 F	250	175	250	1495	113.33	187.41	203.70	
275 F- 283 F	250	200	250	1467	113.33	173.33	182.22	
287 F- 295 F	250	225	250	1437	114.81	188.89	229.63	

Source: Ibid, Pg 76-77 Table 8 and 9

CHAPTER-V

Displacement, Rehabilitation, Resettlement and Environmental Issues

5.1. Displacement and R&R in Rengali Multi-purpose Project :

5.1.1 Introduction:

Many developmental activities require acquisition of private and Government land, displacement of large number of people and rehabilitating them elsewhere with certain facilities. Any irrigation scheme, being of developmental nature, is no exception. Majority of people in the vicinity of the project (particularly in the ayacut) receive cherished benefits from irrigation while the people that reside upstream are exposed to dreaded losses and upheavals: the loss of their lands and house, the uprooting from ancestral spaces, the scattering of kinship groups, and the disarticulation of their communities. In other words, a situation is created in which some share the gains, while others share the pains.

Yet approaches exist that can help, correct an unfair distribution of gains and pains, and better protect the people subjected to the risks of forced displacement and relocation. Any reservoir scheme will, no doubt, induce involuntary displacement. Displacement of population is inevitable for construction of any major river valley project. Some have to suffer for the good of many. ‘You can not make Omlettes without breaking eggs.’

What is usually described as involuntary resettlement consists of two distinct, yet closely related social processes: (a) *displacement* of people and (b) *reconstruction* of their livelihood; this reconstruction is sometimes called *rehabilitation*. Each has its own demands, costs, logistics, and sociocultural and economic effects.

“**Displacement** concerns how land and other major assets are expropriated and people are removed in order to allow a project intended for the overall social good to proceed. In real life, however, this is not just an “expropriation” - a simple transfer of property in exchange for compensation. In sociological terms, it is a process of unraveling the existing pattern of social organisation and systems of production. Forced population displacement always creates a social crisis, and sometimes a political one as well. The disruptions it triggers are rarely equaled in the “normal” processes of development.

Rehabilitation, in turn refers to the fate of the displaced people after relocation, and to the reconstruction of their patterns of socioeconomic organisation.

In theory, the two processes -- expropriation and rehabilitation -- are segments of a single continuum; in practice, the first does not automatically bring about the second. When people are displaced by projects for “right of way”, they lose in full or in part -- either their land or their dwelling, or both. As a consequence, the outcomes of resettlement may vary considerably from people’s previous standards of living. Indeed, whether involuntary resettlement results in re-establishing people’s incomes and livelihood depends largely on how displacement is planned and carried out it also depends on whether resettlers are assisted to rebuild their livelihood. x x x x x. Most social researchers in India, emphasize that “rehabilitation” does not occur automatically after relocation. Indeed, resettlement may occur without rehabilitation, and unfortunately it often does.

5.1.2 The Need to Avoid or Minimise Displacement:

The sheer magnitude of involuntary population displacements has increased considerably over the last few decades. A worldwide available data reveal that in every single year during the 1990s, at least 10,000,000 people enter a process of involuntary displacement and relocation that is caused by the new cohort of programs started annually in dam construction, urban infrastructure, and transportation. Of course, even when the magnitude of resettlement is small, this does not justify less attention to its complexity and likely negative impacts. x x x x x.

We know that changes in patterns of land and water use have required, throughout human history, that people resettle. Such changes will be needed in the future as well. Although involuntary relocation must be avoided whenever possible, the need for resettlement cannot be eliminated completely. Irrigation for thirsty fields, wider roads in clogged downtowns, and the protection of biosphere reserves are necessary. Therefore, if some involuntary displacements are inevitable, their adverse impacts must be minimised whenever they cannot be completely avoided.

Most importantly, resettlement must be carried out in a way that will protect the livelihoods of the displaced people and will prevent secondary environmental damage. If this is not done, then adverse effects will increase unnecessarily.

5.1.3 Concerns and Risks - People's Impoverishment

What remains insufficiently explained is the multisided and long term socio-economic impact of displacement on people who are forced to move.

The main risk arising from forced displacement is the impoverishment of the displaced people, many of whom are poor in the first place. This risk is not abstract. Social research has documented that inequitably planned and irresponsibly implemented resettlement programs cause increased poverty. Therefore, the main social concern in involuntary resettlement must revolve around the key impoverishment risks inherent in these operations. Policy makers and planners should focus on such risks and respond with commensurate mitigatory actions.

Based on the study of a vast body of empirical data about the basic socio-economic processes that occur when people are displaced, a recurrent pattern of eight main potential risks of impoverishment were identified. Taken together, these eight processes represent a **risk-model that captures the economic, social, and cultural impoverishment of affected people**. The model predicts that the displaced people are at risk of losing natural capital, man-made capital, human capital, and social capital. Avoiding these risks should be the main concern of policy makers, NGOs (Non-Government Organisations), planners, and environmentalists. Very concisely, these basic risks are:

- i) **Landlessness:** Expropriation of land removes the main foundation upon which people's productive systems, commercial activities and livelihoods are built. This is the main form of decapitalisation and pauperization of people who are displaced because they lose both natural and man made capital.
- ii) **Joblessness:** Loss of wage employment occurs both in rural and urban displacement. Those losing job may be either landless agricultural laborers, service workers, or artisans. Creating new jobs for them is difficult and requires substantial investments. Therefore, the unemployment or underemployment resulting among resettlers lingers long after physical relocation.
- iii) **Homelessness:** Loss of housing and shelter may be only temporary for many displaced, but for some it remains a chronic condition. In a broader cultural sense, homelessness is also placelessness, loss of a group's cultural space and identity, or cultural impoverishment.
- iv) **Marginalization:** Marginalization occurs when relocated families lose economic power and experience "downward mobility": Middle income farm households do not become landless, but become small landholders; small shopkeepers and craftsmen lose business and fall below poverty thresholds.

- v) **Increased Morbidity and Mortality:** Vulnerability to illness is increased by forced-relocation, which tends to be associated with increased stress, psychological traumas, and the outbreak of parasitic and vector-borne diseases. Serious decreases in health levels result from an unsafe water supply and sewage systems that proliferate epidemic infections, diarrhoea, dysentery, etc.
- vi) **Food Insecurity:** Forced uprooting diminishes self sufficiency, dismantles local arrangements for food supply, and thus increases the risk that people will fall into chronic food insecurity, defined as calorie protein intake levels below the minimum necessary for normal growth and work.
- vii) **Loss of Access to Common Property:** For poor farmers, and particularly for the assetless, loss of access to common property resources (e.g., loss of access of forests, water bodies, grazing lands, etc.) represents a form of income loss and livelihood deterioration that is typically overlooked by planners, and therefore uncompensated.
- viii) **Social Dis-integration:** The dismantling of community structures and social organisation, the dispersion of informal and formal network, local associations, etc., represents a massive loss of social capital. Such “elusive” dis-integration processes undermine livelihoods in ways unrecognised by planners, and are among the most pervasive causes of enduring impoverishment and disempowerment.

The risks discussed above affect various categories of people differentially - rural, urban, tribal or non-tribal groups, children or elderly. Significantly the impacts of displacement are more severe on women. x x x x x. The eight characteristics of impoverishment described above, when taken together, could help such prevention efforts, because they provide a warning model that captures the lessons of many real processes and clearly point to what must be avoided. Thus, the predictive capacity of this model helps adopt counteracting or compensating measures in time for risk management. x x x x. Landlessness risks should be met through planned land-based re-establishment; homelessness -- through sound housing programs; joblessness -- through alternative sustainable employment; increased morbidity -- through adequate prevention, education, and improved health care assistance; community disarticulation - through purposive community reconstruction and host-resettler integrative strategies. One way to accomplish such reconstruction is to make special arrangements that enable those displaced to share directly in the specific benefits generated by the program which pushed them out in the first place. (Source: Involuntary Displacement in Dam Projects - Edited by A.B. Ota and Anita Agnihotri - Prachi Prakashan, New Delhi, 1996 Pg.3-9)

Population displacement is undesirable, yet often inevitable. Consequence of large scale infrastructure development programme changes patterns of land and water use. In most cases such programmes are vital for achieving general socio-economic development; unfortunately, they also may have some adverse impacts. These negative impacts must be considered carefully before implementation, prevented when possible and mitigated when they can not be avoided fully.

Before R&R issue of Rengali Dam and the barrage at Samal where tail-race release is picked up for irrigation is discussed, the displacement of families caused due to construction of major and medium dam projects in our state and two major projects of our country i.e. Bhakra-Nangal Project and Sardar Sarovar Project are described here as case study.

Families displaced by major and medium river valley projects of Odisha are furnished vide Table 5.1

Table 5.1 Displacement of families by Major & Medium dam projects in Odisha)

A. Major Dams				
S. No.	Project	River	Year of Completion	No. of D.P. Families
1.	Machkund	Machkund	Mid-Forties	2,938
2.	Hirakud	Mahanadi	1957	22,144
3.	Balimula	Sileru	1977	2,500
4.	Salandi	Salandi	1983	965
5.	Rengali dam	Brahmani	1992-93	10,897
6.	Upper Kolab	Kolab	1991-92	3,171
7.	Upper Indravati (4 dams)	Indravati & its tributaries.	1996-97	5,599
8.	Rengali Irrigation (Samal barrage)*	Brahmani	Ongoing	828
9.	Kanupur*	Baitarani	Ongoing	3617
10.	Subarnarekha*	Haldia, Jambhira, Baura & Ichha	Ongoing	5,398
11.	Ong dam*	Ong	Ongoing	3917
12.	Lower Indra*	Indra	Ongoing	6181
13.	Lower Suktel*	Suktel	Ongoing	4180
B. Medium Project :				
1.	Budhabudhiani	Duanto	1967	N.A.
2.	Saipala	Saipala	1974	9
3.	Dhanei	Dhanei	1975	N.A.
4.	Pitamahal	Pitamahal	1977	68
5.	Derjang	Derjang	1978	356
6.	Ghodahad	Ghodahad	1978	68
7.	Baghua Stage-I	Baghua	1978	510
8.	Khadkei	Khadkei	1979	118
9.	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
17.	Jharbandh	Kukrihar	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.	Dumberbahal	Dumberbahal	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	N.A.	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 and Badjor	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

* Projected figure as the projects are ongoing.

Thus excepting a couple of projects, all projects have displaced or will displace population in small or big 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 and storage capacity of the reservoir. (Source: Development Projects and Induced Displacement by Dr. A.K. Dalua, Published by Mass Media (P) Ltd. Pg. 20-21)

5.1.4 Case Study : Bhakra-Nangal & Sardar Sarovar Project:

5.1.4.1 Displacement, Resettlement, and Rehabilitation of Bhakra-Nangal Project:

“The reservoir formed by the Bhakra dam has been named Gobind Sagar, in memory of Guru Gobind Singh, the great Sikh religious leader who lived in this area for many years. The reservoir covers a maximum area of 168.35 sq.km. (65 sq. miles or 41,600 acres) when full. It extends to about a 100 km (60 miles) from Bhakra.

The project acquired land to the extent of 17,984 ha (44,440 acres) in the districts of Kangra (presently Una), Bilaspur, Mandi, and Solan of Punjab and Himachal Pradesh. In addition to the small town of Bilaspur, 375 villages were involved. Forty-eight villages of Kangra and fourteen villages of Bilaspur were completely submerged, while the remaining villages were affected in varying degrees.

Of the total land required, 6844 ha (16,924 acres) were already government-owned and only the balance 11,135 ha (27,516 acres) of privately owned land had to be acquired. The land was acquired as per the provisions of the Land Acquisition Act, 1894, as it then existed. This involved the migration of 7209 families, or a population of 36,000 people. The salient details are given in Table 5.2.

Table 5.2 Details of land acquisition for Bhakra submergence area

District	No. of villages affected	Total land acquired (in ha)	Privately owned land (in ha)	No. of private land-owning families
Kangra (Una)	110	5483	5483	3333
Bilaspur	256	12,313	5611	3838
Mandi	5	162	15	35
Solan	4	26	26	3
Total	375	17,984	11,135	7209

Source : Bhakra-Beas Management Board.

For the urban displaced, a new town of Bilaspur was built just 2 km away on the high land overlooking the old town and the thousand urban families resettled there.

Since the number of persons affected was large, the Bhakra Control Board set up the Bhakra Rehabilitation Committee under the chairmanship of the secretary to the government, Public Works Department (PWD), Buildings and Roads, Capital Project, Chandigarh. It was asked to advise the government on the following matters.

- (i) - principles and methods of rehabilitation with particular reference, to-basis of rehabilitation vis-a-vis land for land, cash compensation etc.
 - places of resettlement-after ascertaining public opinion-both among the population to be displaced and among the people of the area where the displaced persons would be rehabilitated.
 - fixing responsibilities of the government/authorities for rehabilitation.
- (ii) Procedure for determining compensation to the displaced persons.
- (iii) Procedure for determining compensation in individual cases.
- (iv) Rough cost estimates and recommendations regarding its incidence.
- (v) Construction of new town in lieu of Bilaspur.

The general manager, Bhakra dam, joint secretary, Revenue, Himachal Pradesh, deputy commissioner, Bilaspur and deputy Commissioner, Resettlement, were the official members. The three members of the Himachal Pradesh Territorial Council and the member representing Kangra in the Legislative Assembly were the non-official members.

It was decided that in respect of the lands submerged, the displaced persons might be compensated, as far as possible, in the form of land. In the case of those who did not want land in lieu of cash compensation, they could be paid in cash or partly in land and partly cash. Compensation for houses, trees, etc., was in cash. The compensation was awarded by the land acquisition collectors concerned under the Land Acquisition Act. Liberal compensation was paid for land, houses, trees, *gharats*, and other property going under submergence. The acquired lands were even leased out to the erstwhile landowners temporarily till the actual submergence, with the proviso that they could not evict existing tenants. The owners of houses were permitted to make free use of material from their houses, regardless of the acquisition value.

5.1.4.2 Resettlement :

As per the resettlement policy, the stakeholders were also consulted and their views fully considered. Nearly a third of the total affected families were resettled in nearby areas of Himachal Pradesh and a little over a third of the families preferred to receive cash compensation so that they would resettle on their own. A little less than a third of the families desired to resettle in land within the irrigation command of the project. The details are given in Table 5.3.

Table 5.3: Rehabilitation of Displaced families

	Category	Number of families
1	Resettled within Himachal Pradesh by Himachal Pradesh Government	2398
2.	Those who preferred cash compensation and resettlement on their own	2632
3.	Families that preferred resettlement in the irrigation command	2179
4.	Total	7209

Source: Bhakra-Beas Management Board

In addition to resettlement, the following actions were taken to provide means of livelihood and more amenities for the families resettled in Himachal Pradesh:

- Free fishing licenses in the reservoir for three years.
- Gainful employment to local people on the construction of the project.

Where a small area of land or immovable property was still left unacquired and the landowners desired its acquisition, that too was permitted.

Surveys for locating suitable land in the Sirmour, Mandi, and Mahasu districts of Himachal Pradesh, as also in the Hoshiarpur and Ambala districts of Punjab, were carried out but very little land was available there. However, a similar survey in the Hisar district showed that sufficient land in compact blocks at cheap rates was available. The displaced people too expressed their preference for lands coming within the irrigation command of the Bhakra Nangal project. About 5342 ha (13,200 acres) of land in the BNP irrigation command were acquired in compact blocks in thirty villages of the Hisar district (since trifurcated into Hisar, Sirsa, and Fatehabad districts).

Compared to the price paid for the land that was to be submerged, the cost of land to be newly acquired in Hisar was low. Thus, it was found that the full value of compensation for submergence land could not be made in the form of only land in the command. Therefore, it was decided that compensation would be partly in the form of land and partly in cash, subject to two overriding considerations. These were that no oustee would get more than 25 acres (about 10 ha) of land and that no oustee would get less than his cultivated land acquired for submergence, if the compensation amount was adequate to meet its cost.

In order to help small landowners among the oustees, compensation up to Rs.1000 was made fully in the form of land. Beyond that, a system of graded cuts that worked on slab system was followed for additional land.

Landless tenants in the submerged area were also declared eligible for allotment of land in the command. They were given land to the extent of their tenancy (as recorded in revenue records for the 1957 kharif season), subject to a maximum of 5 acres (about 2 ha), the price to be paid by them in twenty equal half-yearly instalments, commencing after a grace period of two years.

Even artisans and labourers affected by the project, who did not own or cultivate land as tenants but wished to shift and settle in the Hisar district, were each allotted a half acre of land, free of cost.

Allotments of *abadi* plots for the construction of residences and shops in the resettlement villages were also done. Abadi sites were on the basis of four allottees per acre. Model layouts for abadi sites were planned in each resettlement village.

The following additional amenities were provided:

- temporary shelter accommodation;
- easy loans for construction of houses and supply and transport of building materials;
- repairs to old wells and new wells for drinking water supply, where necessary;
- Bhakra canal water supply through new canals, minors, and water courses;
- bridges on canals, new ferry services, new roads/village paths;
- new primary schools, medical facilities, cattle inoculation, and security arrangements; and
- rail/bus fare plus a lump sum rehabilitation grant.

In order to resettle the oustees as near to their old *biradari* (social and cultural groups setting) of neighbouring villages as far as possible, allotment of land in the new area was made on the basis of *had bast* numbers in the erstwhile villages in Kangra, Bilaspur, and Mandi.

Deeds of conveyance of proprietary rights to the allottees were executed after they fulfilled all the conditions of their allotment and clearance of all pending recoveries of dues from them.

5.1.4.3 Facilities:

The public health unit of the project was set up in 1947. Anti-malarial measures including DDT spraying, vaccination, cholera and typhoid inoculation, etc. were all made part of its work. A fifty-bed hospital was set up by the project in 1951 and this was gradually expanded.

Nangal was provided with a unit of the Punjab Red Cross Society. Maternity, childcare and other facilities became available from the very beginning of the project.

5.1.4.4 Water Supply:

A regular water supply system was started at Nangal in 1947. Both raw water for lawns and gardens, and treated potable water for the people were supplied. A similar local water supply for Bhakra and other places was also provided.

5.1.4.5 Schools:

A number of schools for boys and girls were opened in the Nangal township project colony for the education of children of employees as well as the local population. They were later transferred to the education department of the Punjab government.

The project improved the position of the drinking water supply in a large number of villages and towns, not only in the command area and nearby but even in some far-off places. The metropolitan capital Delhi, as well as the cities of Chandigarh, Patiala, Ropar, etc. are some examples.

A number of factories depend on the project for their water supply. Some major examples are the National Fertilizers, Nangal, Thermal Power Station, Delhi, Ropar Thermal Power Station, etc. A large number of small industrial units are also similarly benefitted. “(Source: Bhakra-Nangal Project, Socio-economic and Environmental Impacts by R.Rangachari, Oxford University Press, 2006 Pg.62-67).

5.1.5 R&R in Sardar Sarovar Project:

“The Sardar Sarovar Project (SSP), a multi-purpose water project on the river Narmada in western India, has been undoubtedly one of the most controversial development projects in the world. Like many other development projects in India and in other developing countries, one of the weak planks of the project was the resettlement and rehabilitation (R&R) of involuntarily displaced population, a large section of which was tribal. It is by now widely accepted that development-induced displacement is disruptive. Although planners and administrators have been slow in learning this, social scientists have been well aware of how development projects can make certain sections of the population worse off. In fact, according to Cernea (1991), social scientists have been historically much better at recording development’s tragedies than preventing them. An interesting account has been provided recently by Labo and Kumar (2009). Covering the period between 1947 and 2004, they present a comprehensive account of land acquired for water resources, industries, mines, HRD, transportation/communication, and urban development projects and focus on the people displaced and affected by them. But during the times when SSP was being planned, the planners and administrators, even when aware of the tragedies it might cause, felt that the family getting displaced was making a sacrifice for the sake of the community (Varma, 1985). Nevertheless, sensitivity towards the displacement problem has increased all over the world and governments have been trying to internalise its social cost by making provisions for adequate compensation. Implementation of good Resettlement & Rehabilitation (R&R) policies continue to be a serious problem even to this day.

The case of SSP with respect to R&R had become more controversial because there was an opinion that the project was economically non-viable (Paranjpye, 1990). Similarly, a number of issues had been raised with regard to SSP’s environmental impacts and sustainability (Alvares & Billorey, 1988; Kalpavriksh, 1988). Morse and Berger, 1992; Paranjpye, 1990). A strong international lobby had evolved that agreed and supported Alvares and Billorey’s view that SSP was a planned environmental disaster. At another level there have been arguments about the relevance and correctness of imposing a non-tribal (mainstream society) concept of development on tribals who may have their own philosophy of development. According to this school of thought, mainstream society should not interfere with traditional tribal societies and their modes of production. Even the Morse and Berger Report endorsed what they call the tribals’ cultural economy. This thesis has also been propounded by the Narmada Bachao Andolan (NBA), an organisation of activists and some tribals of the Narmada Valley. In fact NBA had raised, or rather challenged the conventional concept of development and said that it spelt destruction (Patkar, 1988). Since the SSP affected a large number of tribals, it was argued that tribals were being forced to accept a tragic future on account of a project that was inherently hazardous in terms of its social, economic and environmental impacts. The case for R&R in SSP had been thus extremely vulnerable. Probably it was this type lobbying at the national and international levels that prompted academics to believe, even in 1996, that the SSP amounted to planned destruction and that it had elements of a great and horrible act of deceit towards the tribals in the Valley (Krishna Kumar, 1996).

However, the entire controversy over the SSP in general and the R&R efforts in particular deserves a more close and objective assessment than has been attempted by most. Many important things have happened for the first time in the history of R&R in India and perhaps in developing countries. Compared to an uninspiring history of R&R in the country until recent times, significant policy reforms have been achieved in SSP. Similarly, implementation in the initial phase of the post policy reform period (1987-1993) was moving in right direction. It would be relevant and important to understand the processes and events that took place in R&R in SSP since 1980, when the implementation of the dam project began in earnest. It would also be useful to know what plagued the implementation subsequently. x x x x x x.

5.1.5.1 The Basic Provisions for R&R in SSP:

The Narmada is an inter-state river which originates in Madhya Pradesh (MP) and traverses a total distance of 1312 km before merging into the sea in the Gulf of Khambhat on the west coast. Through most of its course, it flows between the mountain ranges of the Vindhya in the north and the Satpura in the south. It is one of the oldest and most venerated rivers (Paranjpye, 1991; Varghese, 1994; Varma, 1985). Because of its peculiarly narrow valley occupying or fringing on the territories of Gujarat, MP and Maharashtra, the 214 km long SSP reservoir submerges lands in all those three states.

The two major riparian states, Gujarat and MP, disputed their respective shares of water. A number of meetings and committees could not resolve the issue. Hence, as per the provision in Article 356 of the Constitution of India, a Tribunal headed by a Supreme Court Judge was set up in 1969 (for a brief and interesting historical account, see Varghese, 1994). Obviously, policy decisions regarding rehabilitation of the project affected population also rested with the Tribunal. The Narmada Water Dispute Tribunal (NWDT), as it came to be known, gave its final report in 1978.

The NWDT made a basic provision for the allotment of a minimum of 2 ha of irrigable agricultural land to each land holding family losing 25 per cent or more of agricultural land. The term “family” included husband, wife, minor children and other dependent on the head of the family. Further, every major son of 18 years and above was treated as separate family (Report of NWDT Vol. II). This basic provision itself was path-breaking in the history of R&R in India. In most projects, the persons losing land were given cash compensation which hardly enabled them to buy alternative land or any other equally productive assets.

Although the provision of irrigable agricultural land was made to compensate for the loss of land, the NWDT provisions had serious lacunae. Firstly, it failed to take into account the characteristics of the tribal settlements and the economy whether a large number of families did not own but only operated land for production. The land belonged to the forest department or it was government waste land. Describing this feature in the SSP submergence area, Patel notes, “the fact of nature displacement of these people by development projects had nearly brought to the surface a serious and an embarrassing problem of unacknowledged tribal right to the resources of livelihood in their ancestral land” (Patel 1994: 1 and ARCH-Vahini 1988a:3) (Action Research in Community Health and Development). Secondly, the NWDT provisions also ignored female headed households, widows, single woman families and the landless.

In this context Ganguly Thukral notes, “The Award (NWDT) not only ignores women as individuals and heads of families, but is also unjust to the landless man and his family. Even the major sons of the landowning peasant are recognised as being entitled to compensation, preferably in land. The landless head of a family is however, not recognised, let alone his son”. (Ganguly Thukral, 1989:59)

Obviously the NWDT Award’s major provision was sociologically ill informed. However, this was not all. The three state governments were not in a mood even to abide by the NWDT provisions. There were signs of retrogression.

5.1.5.2 Activist Group Enters the Fray

With the declaration of the NWDT Award in 1978 GoG (Government of Gujarat) began the project work as well as the process of rehabilitation. It issued a Government Resolution (GR) in June 1979, making in it a crucial change which almost completely killed the spirit of the NWDT provision. The entitlement of a minimum of 2 ha of irrigated land to ‘each land owning family’ was changed to ‘each land holding’. In tribal areas, both for cultural and historical reasons and laxity in revenue

administration, most land is jointly held. A joint holding may, therefore, be operated by two, three or more independent families. Implementing the GR, the government offered government waste land, which was more than 100 km from the native villages to every holding (and not family). The tribals living in first five affected villages refused the government offer. The government responded by offering cash compensation and leaving the land purchase to affected families (ARCH-Vahini 1988a).

Armed with its 1979 GR, the government was ready to ride roughshod over the poor, helpless and unorganised tribals when, in 1980, members of ARCH-Vahini started frequenting the submergence villages. The panic-stricken and desperate tribals were reluctant to hear and do what the fresh-faced students advised them. Even the ARCH-Vahini activists were new to this game. They knew nothing or very little about land laws, the NWDT Award, World Bank guidelines of 1980 etc. Had they known the complications they would have probably given up (Patel, 1994). The tribals would have continued to tolerate them as sympathetic observers but for a case in which the government retracted on a written promise it had made in implementing the 1979 GR. This was towards the end of 1983. Until then ARCH-Vahini was content with monitoring the implementation of a totally watered-down policy of GoG. A silly slip by the government led to the filing in court of a case of ‘promissory estoppel’. The tribals saw a ray of hope. The quickly organised and the government stood exposed.

The test case in high court made the tribals realise that the government was cheating and that it was vulnerable. On March 8, 1984 a huge rally marched to the Kevadia colony - the project headquarters at the dam site village Vadgam. The major demand put forth was the implementation of the NWDT provision not only for the land owning families and co-parceners but also for those who cultivated forest and government waste lands (ARCH-Vahini, 1988a and Patel, 1994). The might of the affected tribals increased and, perhaps for the first time in the history of execution of a development project involving displacement, the government was under pressure from below. This was followed by stoppage of project work on a dyke appurtenant to the main dam. Public interest litigation was filed in the High Court and subsequently in the Supreme Court. The Supreme Court granted interim injunction. The Government violated it. The Court appointed an independent Commission which indicted the government of Gujrat. “(Source: Sardar Sarovar Project on the river Narmada Vol.II Edited by R.Parthasarathy & Ravindra H. Dholakia March 2011 Pg.475-480)”.

“It will now be useful to look at the latest figures that we get from the Sardar Sarovar Punarvasvat Agency, the rehabilitation agency and the NCA (Narmada Control Authority). The table below presents the figures given by the Agency.

**State-wise Distribution of Affected Villages
(As of 31.07.2009)**

State	Villages Affected			PAFs likely to be resettled	PAFs likely to be resettled in Gujarat	PAFs already resettled in Gujrat
	Full	Partial	Total			
Gujrat	3	016	019	04744	04744	04744
Maharashtra	0	033	033	04204	00771	00771
Madhya Pradesh*	1	191	192	34443	05579	05579
Total	4	240	244	46721	11091	11094

* In addition about 20,000 PAFs have been resettled in MP when the height of dam had reached 121 metres. Those remaining to be resettled number 12 to 15 thousand, all in MP.

PAF- Project Affected Families

(Source : Sardar Sarovar Punarvasvat Agency, Vadodara)

The Narmada Control Authority puts the total figure at a higher level for MP. Figures for Maharashtra and Gujarat are lower. The differences are small but it should be noted that there was no regular verification.

State	Villages Affected			Families Affected
	Full	Partial	Total	
MP	1	191	192	39,369
Maharashtra		33	33	4,163
Gujarat	3	16	19	4,737
Total	4	240	244	48,269

http://nca.gov.in/mr_index.htm access verified again on February 2, 2011.

The issue of numbers is still not settled. It can be assumed that all those PAFs who have to be compensated with land have been resettled. That MP has not accepted any alternative land allotment liability is apparent from a report submitted in July 2006 by an oversight group. If this assumption is correct then the original estimates of PAFs in the three states had been provided with land. The number is 11,094. The total PAFs made by NBA (Narmada Bachao Andolan) and IRM were exaggerations. IRM (Independent Review Mission) was interested in taking the World Bank to task and NBA wanted to prove that rehabilitation was impossible.

The figures given by the Sardar Sarovar Punarvasvat agency show that by the 1st quarter of 2009, all the entitled PAFs in the three states had been provided with land. The number is 11094. The total PAFs range between 48,000 and 54,000. Thus between 38 and 42 thousand families had to be relocated with housing and other facilities. This number is indeed high, but as we have noted Maharashtra and Madhya Pradesh did not develop any institutional arrangement for correct identification and estimation, there could be scope for mistakes and manipulations.” (Source: Ibid Pg.499-501)

5.1.5.3 Problems and Prospects of R&R in Sardar Sarovar Project:

“SSP has been under the world’s gaze. We have seen that the R&R programme of the project was initially as bad as it had been in any development project involving involuntary displacement. We also saw that there were many firsts in the project. For the first time, such a massive mobilisation of PAPs took place and for the first time the struggle bore concrete results. For the first time, any state government in the country announced and committed an R&R package which could be said to be fair and just to those who were to get involuntarily displaced. The setting up of a List committee, the Land Purchase Committee, the GO-NGO collaboration and the process of allotment of alternative agricultural land - everything has happened for the first time. A combined effect of all these first-time initiatives, the resettlement and relocation of the displaced population in the first phase, i.e. 1988-93 was also largely successful. This was also assessed and approved by an International Review Mission. However, problems again plagued SSP’s R&R programme until the setting up of the Grievance Redressal Cell.

Two sets of problems have affected the programme. The first concerns the government’s understanding of R&R issues. Cernea (1996:1516) has drawn attention to the process involved in ‘involuntary resettlement’. He says “what is usually described as involuntary resettlement” consists of two distinct, yet closely related social processes : (a) displacement of people and (b) reconstruction of their livelihood; this reconstruction is sometimes called rehabilitation. Each has its own demands, risks, costs logistics and socio-cultural and economic effect.” He further says that in theory, the two processes are part of the same continuum; however, the second does not always follow the first. In the case of the SSP, it appears the rehabilitation of the population resettled during 1988-93 was facing rough weather and the resettlement of the post-1993 displaced families was also plagued with problems. The government and the administration was either not aware of or had decided to ignore the seriousness of both the problems. The officials at the lower levels were hardly able to appreciate these fine aspects of involuntary resettlements. Therefore, lacking clear policy pronouncements and procedures for

implementation, the implementers were going to create chaos in R&R. Probably, inept handling of serious problems arising in the relocation sites with regard to reconstruction of living and livelihood, and the resettlement efforts in terms of allotment of land to the affected families from MP, resulted in dissatisfaction among the displaced people. This, as a result, prompted the otherwise effective and fruitful GO-NGO collaboration to break.

It appears that the feeling among most persons in the government and the administration was that, with the relocation of families from the submergence villages the rehabilitation was completed. A majority of displaced families in Gujarat got good quality land and a plot to build a house. What more should they expect? The efforts required to set up and roll socio-economic life smoothly at the new sites, the displaced population's integration with the host communities, the requirements and aspirations of the host communities potential areas of conflict and cooperation between the host and the guest population etc. didn't seem to be worth attending. Many of these aspects did not involve a monetary component, but they involved understanding and commitment on the part of government officials.

The second set of problems relates to the role of the IRM, activists, and NGOs in and outside the country in handling the SSP issue in general and the R&R issue in SSP in particular. The Independent Review Mission came into existence because of the pressure of the national and international NGOs on the World Bank. One can raise a serious question whether it was required. Of course, even if one does not go into rhetoric and say that the sovereignty of the nation was threatened by such international interventions, enough understanding existed within the country to seriously discuss the problems and look for solutions. However, once it became a given, there should not have been any argument against the setting up of the IRM. What is important is that the IRM did not remain objective in "its search for truth" on both the major agenda items: R&R and the environmental impacts of SSP (ARCH-Vahini, 1992a, 1992b, 1992c, 1992d, 1992e, 1993a, 1993b, 1993c). The IRM by its advice to the world Bank to "Step Back" from the project set such a process into motion that the reform processes that were taking place in the project and in the system got derailed. The World Bank, which was the main actor and the keel that could have kept the ship on course, was forced to leave. Both the national and international NGOs celebrated this only to weaken the reform process that had just begun in a developing country.

This exit of the World Bank also weakened and further isolated the already isolated NGOs like ARCH-Vahini which had provided critical support to the project and had collaborated to implement the policy. It also released the pressure on the government. It is probably clear by now, after we have seen how the policy reforms were won, that the government was better persuaded by the Bank than the combined strength of the NGO network. Interestingly, what the NBA and its allies achieved by sending the World Bank packing was a rise in the chances of the failure of R&R in the SSP which they had always predicted. It must be conceded, though, that NBA's dream failed to materialise because other forces such as GRA directed R&R in the right direction.

Had the IRM given critical feedback and recommended continued but critical support to the SSP by WB, the Bank Mission in conjunction with sensitive NGOs such as ARCH-Vahini could have set into motion processes that would have ensured a decent and timely resettlement, in Gujarat as well as in their respective states, of the rest of the families displaced from Maharashtra and MP. It could have also designed a path to take the resettled population to a proper rehabilitation. Even GRA would not have come into existence. Because it did not happen that way both resettlement and rehabilitation were at risk. With the lone worthwhile NGO getting out of the main frame, the prospects of R&R weakened and delays occurred. Even in 2009, incomplete R&R has held up raising the dam height to its design level. The promise of the project remains unfulfilled." (Source: Ibid Pg.508-511)

5.1.6 R&R in Rengali Multi-purpose Project:

5.1.6(a) R&R in Rengali Dam Project:

Rehabilitation of displaced persons in a project involves Land acquisition, Resettlement, Rehabilitation including providing infrastructure facilities in new settlement colonys. In case of Rengali Dam Project, Revenue Divisional Commissioner (R.D.C), Northern Division, Sambalpur was in overall control. A rehabilitation Circle was created on 11.01.1978 to look after the engineering works. An advisory committee with R.D.C. (N.D) as chairman was formed with officials and non-officials as members to review the progress of Land Acquisition, Rehabilitation and Resettlement at regular intervals. A Deputy Commissioner was also posted to co-ordinate works and to assist R.D.C.(N.D) in implementing the same. Besides, a Special Land Acquisition Officer along with zone officers worked under the control of R.D.C. (N.D) to look after the L.A. cases of Rengali Multi-Purpose Project.

i) Land Submerged :

Different categories of Land were submerged for the purpose as detailed below:

- i) Private Land - 10280.90 ha
- ii) Revenue Land - 32005.75 ha
- iii) Forest Land - 822.50 ha

Land acquired for Rengali Dam

a) Total Private land acquired for the project

- Rayati - 34335.67 acres
- Govt. & Forest - 65208.42 acres
- 99544.09 Ac. (40301 ha)

b) Total Land requisitioned by the project to the Spl. LAO - 404 cases involving 99644.79 Ac. (40342 ha)

Number of villages which were acquired in 265 villages in 3 revenue sub-divisions are mentioned below:

Deogarh - 194, Pallahara-70 and Talcher-1 totalling 265 nos.

The villages have been categorised separately depending on the nature of submergence as detailed below:

Table 5.4 - Category of Submergence.

Sl. No.	Category	No. of villages
i)	Fully submerged (Deogarh/Pallahara/Talcher) (24+16+1)	41
ii)	Villages fully submerged and Land partly (Deogarh+Pallahara) (36+20)	56
iii)	Both Land and village partly submerged (Deogarh 15 + Pallahara 3)	18
iv)	Villages above 410 ft contour line (MWL and land partly submerged (as recommended by ROC) (Deogarh 54 + Pallahara 29)	83
v)	Bechhapari villages fully submerged (Pallahara 3 + Deogarh 27)	30
vi)	Bechhapari villages partly submerged (Deogarh 36 + Pallahara 1)	37
	Total	265 Nos.

5.1.6.1 Rehabilitation & Resettlement

The Rengali Multipurpose Project in the Brahmani basin has created a large reservoir, spreading over 414 sq.km. and has affected 11289 families in 265 villages of Dhenkanal and Sambalpur district.

- i) Families affected - 11289
- ii) Families displaced - 11289
- a) Affected families rehabilitated - 11227
- b) Balance to be rehabilitated (upto Nov. 2011) - 62

5.1.6.2 Rehabilitation Infrastructures:

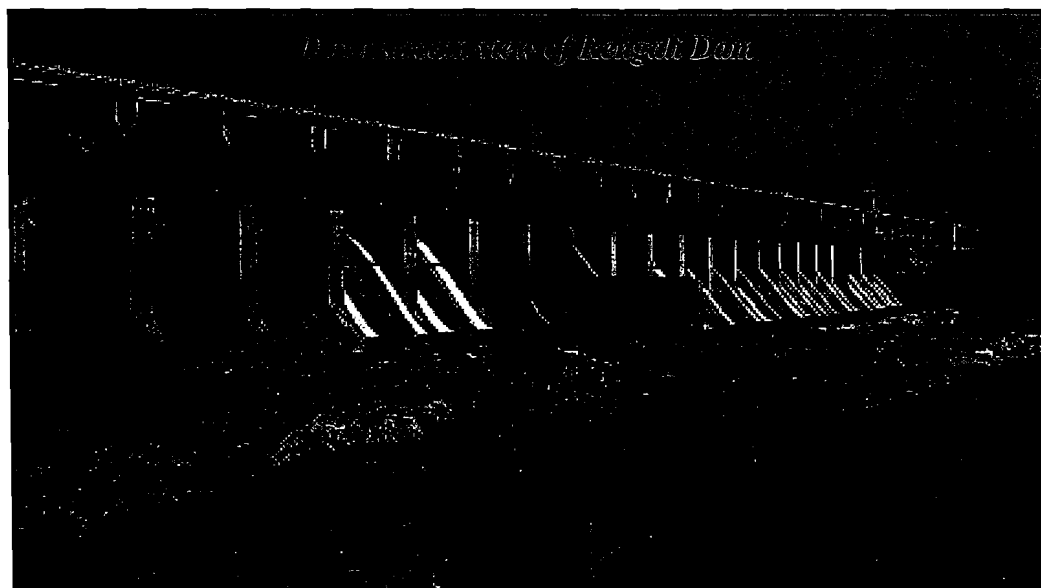
The details of infrastructure facilities provided in Resettlement villages are as follows:

Table 5.5 Details of Infrastructure facilities provided

Facilities	No. of Tanks reqd./ provided	No. of wells reqd./ provided	No. of T.W. reqd./ provided	No. of schools reqd./ provided	No. of clubs reqd./ provided
Earlier provided to resettlement villages & clusters	107 106	207 205	154 153	87 87 (P.S) 19/18 (M.E) 6/6 (H.E)	69 69
Additional facilities provided in retrofit action plan	10	34	47	Nil	1
Total	<u>107</u> 116	<u>207</u> 239	<u>154</u> 200	<u>87/87(P.S)</u> 19/18 (M.E) 6/6 (H.E)	69/70

Source : CE&B.M., Brahmani Left Basin, Samal Nov., 2011 Report)

The resettlement policy laid down by Government for Rengali Multipurpose Project soon after the strike by the project affected people in appended herewith vide Resolution No.35054 & Dt.06.12.1973 of I&P Department. (vide Annexure 5.1)



RESETTLEMENT POLICY LAID DOWN BY GOVERNMENT OF ORISSA FOR RENGALI MULTIPURPOSE PROJECT.

Policy of Rehabilitation:

Copy of the Resolution No.35054 dt.6.12.73 from the Secretary to Government, Irrigation & Power Department vide Memo No.35056 dt.6.12.73.

The construction of the proposed dam at Rengali over river Brahmani is likely to submerge some areas, the particulars of which are as follows:

a) Number of villages	- 173
b) Cultivated area	- 25,073 Acres.
c) Waste and other lands including reserve forest	- 99,113 Acres.
d) Population	- 42,000 or 8,400 families

As about 8,400 families in about 173 villages are going to be displaced by the construction of the dam the programme of rehabilitating them and the policy to be adopted for this purpose had been engaging the attention of Government for some time past. Government have carefully considered the rehabilitation facilities allowed to the families displaced in other projects in side the state and in other states and keeping in view the human problem involved in displacing large number of families, have finally decided to provide adequate facilities to the displaced families in the new resettlement colonies as per the following details.

1) Provision of free house sites in model villages/colonies :

Each displaced family will be given homestead land to the extent of 0.30 Acres free of cost depending on the availability of land. The cost of development of the house site and village layout will be borne by the Government.

2) Agricultural land:

Each family whose lands have been acquired for the project will be allotted 3 acres of reclaimed irrigated land or 6 acres of reclaimed unirrigated land in the ratio of 1:2. The cost of reclamation will be borne by the Government subject to the maximum of Rs.600/- per acre. The above lands will be allotted free of selami but 50% of the reclamation cost will be recovered from the allottee subject to a maximum of Rs.300/- per acre.

As regards landless families from among the displaced persons, the possibility of meeting the cost of allotment of land from the normal scheme for providing lands to the landless persons in the 5th plan, if any, may have to be explored.

3) House Construction:

The displaced families will be provided facilities of free transport of the house building materials which they can salvage from the old houses for carriage to the new settlement colonies. House building materials from the nearest forest will be made available at concessional rate of 60% of normal royalty. Necessary guidance to build low cost houses with fire proof roofing will be provided to the displaced persons facilities of loans under low income group housing/village housing schemes will also be extended to the few resettlement colonies.

4) Provision of Common facilities:

Common facilities like village roads, schools, drinking water wells, tanks for general purpose use, community building etc. will be provided at project cost. For amenities like schools, public health centre, Veterinary Dispensary and Panchayat Ghar, respective Administrative Departments of Government have to supplement the provisions made in the rehabilitation estimates from their departmental budget.

The compensation payable from the project funds towards common facilities existing in the submersible areas will be shown as recovery against this item.

5) Electrifications:

The State Electricity Board will include these resettlement camps for provision of electricity under their Rural Electrification Programme.

6) Minor Irrigation:

Facilities for minor irrigation in the resettlement colonies will be provided to the extent possible. There will be a provision of Rs.2 crores for this purpose in the project estimate.

Priority will be given to the displaced persons for resettlement within the ayacut of the Rengali Project to the extent possible or within the ayacut of the nearby medium project subject to availability of land.

7) The rehabilitation programmes will be executed by a Resettlement Officer, who will work under the control of the R.D.C. Northern Division. Government have also been pleased to set up an Advisory Committee for the implementation of the project under the Chairmanship of the Revenue Divisional Commissioner, Northern Division with Collectors, Sambalpur and Dhenkanal, Conservator of Forests, Additional Chief Engineer, in charge of the Project as members and others as may be co-opted by the Chairman. The Resettlement Officer will function as a Member-Secretary of the Committee.

ORDER

Ordered that the Resolution be published in the extra ordinary issue of the Orissa Gazette for general information of the public. Also ordered that a copy of the resolution be forwarded to All Departments of Government/All Heads of Departments/Government of India, Ministry of Irrigation and Power, New Delhi/Chairman, C.W. & P.C./Accountant General, Orissa/Deputy Accountant General, Orissa, Puri.

By the order of the
Government,

N.R. Hota,
Secretary to Government

5.16(b) R&R for Samal Barrage:

- 1) Total no. of villages affected in the reservoir area -
Fully - 04, Partly 34, Total = 38
- a) Fully affected villages
i) Balangi (ii) Sagadipal (iii) Languabeda (iv) Bijigolpatna
- b) Partly affected villages
i) Bijigol
- c) Total no. of persons displaced - 1042

5.1.7 Case study of Rengali Dam Project (by R&R Directorate, Govt. of Odisha)

5.1.7.1 Impact of displacement

“Impact of displacement in the form of this risk process has been studied from the data collected from 156 Resettlement Colonies (RCs) and clusters of Rengali Dam Project by the Directorate of R&R, Govt. of Odisha, for the purpose of assessing the quality of life of DPs who have presumably been resettled over a decade back!

The study reveals that, after relocation:

- 1) Percentage of families having encroached forest/Govt. land has come down to 49.84% from 85.42%.
- 2) Simultaneously, average encroached forest/Govt. land per family has come down to 0.09 acres from 2.03 acres.
- 3) Total area of land coming under shifting cultivation has come down to 203.37 acres from 792.48 acres.
- 4) Similarly, average area of land per family doing shifting cultivation has come down from 1.27 Ac. to 0.21 Ac.
- 5) Accessibility to forest for MFP and FW (Minor Forest produce & Fuel wood) has been generally restricted.
- 6) Percentage of families whose primary source of income is MFP has gone down to 9.23% from 33.81%.
- 7) Only 23.72% families have access to grazing land after relocation, compared to 100% before the project.

Rengali Dam project has submerged 265 villages and displaced 10,842 families in 2 districts of Odisha. The R&R activities were completed 12 years back. Then there was no R&R policy. Despite good intents, Rengali is a case which shows very high risk of loss of common property resources, not compensated by rehabilitation till date! This, coupled with other related factors, broke the economic backbone of the DPs as a whole. The reasons are not far to see. Encroached forest and government land were taken away and not compensated by the project. In the relocated settlements, shifting cultivation was restricted. No land was earmarked for grazing/as waste lands for DPs. DPs were relocated away from the forest. DPs also faced serious hostility from the host population who did not let them use government forest land near relocation sites. Rengali Dam Project is a typical case of a series of piecemeal/ adhoc rehabilitation measures implemented in a non-consultative manner.

The present policy, we have already pointed out, is participatory and consultative. Rengali did not have a significant tribal composition in population. One would argue that special provisions of present policy of tribals should also extend to habitations with traditionally heavy dependence on CPRs (common property resources). The idea is to let such groups have easy access to CPRs and have an eco-friendly zone in a private relocation site which they would themselves develop and maintain. Under OWRCP, entitlement of encroachers has already been reorganised as a mandatory supplementation to policy. Its time, policy itself incorporates this.

5.1.7.2 Homelessness

This subprocess can either be understood as a chronic condition of shelterlessness for the DPs- or, in the broader cultural sense of placelessness, as the group loses cultural space and identity.

There is reason to believe that either in completed Rengali Dam Project, or in the OWRCP medium subprojects, where relocation has partly started, homelessness is not conceived as a great risk in the first sense. The policy categorically provides for allotment of homestead land @ 0.20 acres per displaced family in the form of Govt. land. The land is to be reclaimed at project cost. They are also entitled to a one time House Building (HB) grant of Rs.20,000/- with provision of escalation. The HB assistance & HB land patta are to be granted jointly. Despite scarcity of Govt. land for purposes of cultivation, it has been possible to identify homestead land patches for most oustees. The physical condition of providing a shelter over head therefore seems realistic to be achieved.

The loss of place and identity in the cultural sense however remains a separate issue.” (Source: Involuntary Displacement in Dam Projects by A.B. Ota and Anita Agnihotri Pg.31-33).

5.1.7.3 Rengali Dam Project

“As per posteriori observation, data collected from completed Rengali Dam Project categorically illustrate that homelessness has been overcome to a large extent. 93.43% of families now have their own home, compared to 63.94% in the project state.

In case of Rengali Dam Project, judicious use of compensation money by DPs & use of RA to improve the facilities of housing created, including honest use of one time disbursement of HB grant could have been the factors for overcoming the risk factor.

Rehabilitation experience in the state seems to uphold that there is an inherent tendency among the DPs to overcome this risk. Often this is the first use compensation money that is put to use. This is also the reason why people do not opt for Govt. sponsored colonies fearing these will be inconveniently located, or houses will be poorly constructed and choose to make their own arrangements.

x x x x x

The aforesaid examples illustrate that more than the physical sense of not having a roof over their head, the DPs are more concerned with tackling the fear of ‘placelessness’. This is the reason why they choose to settle with their friends and relatives. In a typical tribal areas of Odisha, families are joint. Mobility across generations is low. It is found that women and older people feel deeply anguished when they are compelled to leave their ancestral homes, settled over generations. But younger male members seems better oriented to a change of place. It needs to be mentioned here that availability of agrarian land near the homestead land is a crucial factor. The NGOs need to carefully counsel the DPs so that

they choose their homestead, agricultural land or other viable livelihood options in such a manner that material survival does not become a problem. Increasing land price, and scarcity of cultivable land have broken integrated tribal villages into pieces in the past. There is however no detailed specific study on the “Placelessness” phenomenon in the cultural sense available as on date in the state.

5.1.7.4 Social Disarticulation

Displacement brings about with itself a disintegration process which goes unnoticed by planners who tally only material targets and achievements of well being. With relocation, the social organisation of communities get dismantled, the informal and formal networks get disrupted and loss of this social capital inwardly erodes human mobilization & awareness buildup around issues of relevance to survival.

In fact, perception of the risk of social disarticulation is inherent in minds of DPs, much prior to relocation and this is one of the reasons they tend to resist new project that may actually relocate them. That this risk is NOT generally overcome is proved when we see even with nearly 4 lakh people displaced in large & medium water projects alone, their organised voice is never audible. This risk is further accentuated in a society where grass root level political institutions are dormant or weak, political power is concentrated in the hands of the elite and general level of awareness and literacy is low”

The Rengali Dam Project study mentioned above clearly demonstrates what can go wrong in a situation in the absence of NGO counselling. The community support system of a tribal society was not there in Rengali. Following relocation, kinship ties have got severed, marriage distances increased, political organisation did not build up and ethnic mix-up in resettlement has led to loss of group identity. The hostility of host villages has been an added factor which the project authorities could not neutralise. Rengali Dam Project, like Hirakud, has still left a deep sense of trauma in the minds of DPs, irrespective of their state of material well being.

It is suggested that the R&R policy should explicitly recognise the risk of disarticulation and supplement in an Appendix the suggested activities of the NGO to facilitate social mobilization & articulation. Regular holding of land literacy & legal literacy interactions, dissemination of development information, holding grievance sessions by Panchayats & training the community leaders will have a long term effect on capacity building.” (Ibid Pg.34-37)

5.1.7.5 Countering the impoverishment risks:

“Development Projects that displace people involuntarily generally give rise to serve economic, social and environmental problems; production systems are dismantled; productive assets and income sources are lost; people are relocated to environments where their productive skills may be less applicable and the competition for resources greater kin groups are dispersed; and cultural identity, traditional authority, and the potential for mutual help are diminished. Involuntary resettlement may cause severe long term hardship, impoverishment and, environmental damage unless appropriate measures are carefully planned and carried out. (World Bank O.D.4.30/Page-1 of 8)

In Odisha, till today nearly 3 lakh 80 thousand people have been displaced (because of nearly 70 medium & major irrigation projects) who have lost their home and hearth shattering their socio-cultural and economic base which was built over generations. Experiences show that when the people are uprooted from their habitat and get relocated in new site, they have not been able to restore their

predisplaced socio-cultural, environment and economic status. Although it is a general consensus that the Displaced people usually fail to regain their predisplaced quality of life, yet there has not been any systematic study in India in general and Orissa in particular to assess as to what extent the Displaced Families have been able to cope up in the relocated site from socio-economic, cultural & environmental stand point. Professor Michael Cernea an Anthropologist of International repute has put forward an “Impoverishment Risk Model” which states that the project Displaced Persons/Families are likely to undergo eight risk factors which cause impoverishment which have been described earlier under Sec.5.1.3 Unless these risk factors are guarded, successful resettlement and rehabilitation of the displaced resulting in restoration of predisplaced standards of living will not be made possible.

Keeping the above issues in view, an empirical study in Rengali Dam project of Odisha (on the displaced persons who have resettled in 156 Resettlement colonies & Clusters) where displacement has been over since 12 years (where over 10.000 families have been displaced) was carried out to asses the extent of the risk factors on the impoverishment of the PDFs at different stages of R&R process. On the basis of this empirical study, the possible reasons as to why and how each of the risk factors have been responsible for not overcoming the impoverishment have been attempted critically. This study also has finally focussed on the basis of its findings how each of these Eight Risk Factors can be guarded from projects formulation stage till implementation stage in future projects to overcome impoverishment and make Resettlement and Rehabilitation successful.

5.1.7.6 Objectives of the study:

There are four key objectives of the present study:

- i) To ascertain as to, whether the project Displaced Families (PDFs) have regained their predisplacement status in the relocated sites.
- ii) To ascertain as to what extent the PDFs have overcome each of the risk factors in the R&R process over a period of time.
- iii) To examine the reasons for not overcoming the risk factors and to learn lessons from the present study for future projects.
- iv) To evolve strategies on the basis of this study to counter and overcome the impoverishment risk factors in future projects.

5.1.7.7 Conclusion:

From the foregoing discussion and analysis of facts based on the empirical study on the project displaced families of Rengali Dam Project, it is obvious that the eight key risk factors which causes impoverishment are interlinked because of their causal relations and their resultant impacts on the displaced persons. It is essential therefore to take precautionary measures at every stage by participatory planning to counter the forceable “risk factors” so as to end up with successful Resettlement & Rehabilitation of the PDFs. This study has identified a number of factors for each “risk factor” (on the basis of empirical finding) which are standing as stumbling blocks in overcoming the “Improvement Risks”. On the basis of the findings of this study, an attempt also has been made to evolve strategies to overcome each of the risk factor with a view to make R&R successful which can be used as implementation tool in R&R planning operations.” (Source: Involuntary Displacement in Dam Projects - Edited by A.B. Ota & Anita Agnihotri Pg. 150-176)

The findings of the study have been tabulated vide Table 5.6 to 5.13.

Table 5.6 Landlessness

Risk Factor	Parameters	Status of PDFs prior to displacement	Status of the PDFs in the relocated site as it stands now after lapse of 12 years (Current Status)	Whether over-come the risk factor	Reasons for not overcoming the risk factor	Lessons learnt from the study for the future projects & Strategies to overcome the risk factors
Landlessness	Average legal land holding per family	Ac-5.12	Ac-5.12	This is a risk factor which the displaced families have partially overcome.	i) Objectionable Encroached land of the PDFs got acquired which they were cultivating	i) Land for allotment to the PDFs should be identified & allotted near their habitational site.
	Average Encroached Forest waste/Govt. land per family.	Ac-2.03	Ac-0.69		ii) Land allotted to the PDFs were far away from their relocation site (Often at a distance of more than 8kms.)	ii) The land allotted in favour of PDFs should be reclaimed and leveled prior to the shifting.
	Average operational land holding per family.	Ac-6.11	Ac-3.84 (3.15+0.69)		iii) Land allotted to the PDFs were not reclaimed, leveled & stumps not cleared for which very less extent of land became operationally cultivable.	iii) Efforts should be made to allot cultivable land in the command area to the extent possible.
	Average land building per family which is fallow (unused).	Ac-0.57	Ac-1.97		iv) The maximum cultivable land of 6 Acres allotted to the PDFs were not given in one or two patches, but in variably in more than 10 patches per PDF in fragments.	iv) The land allotted in favour of the PDFs should be in one or two patches & not in small fragments.
	Percentage of families - landless.	4.65%	10.89%		v) The land allotted to the PDFs in many places are forcibly encroached and cultivated by the Host people.	v) Care should be taken to allot land in favour of PDFs which must be free from all kinds of encroachment & litigation.
	Percentage of families having operational land holding below 2 acres.	13.46%	30.93%		vi) Lack of land purchase Committee.	vi) Formation of a land purchase committee.
	Percentage of families having operational land building below 4 acres & above 2 acres.	45.99%	46.15%		vii) Non-Association of NGOs in the R&R activities.	vii) NGO should be associated to assist the PAFs in R&R process.
	Percentage of families having operational land holding below 6 acres & above 4 acres.	29.49%	08.65%			
	Percentage of families having operational land building above 6 acres.	06.41%	03.37%			

Table 5.7 Marginalisation

Risk Factors	Parameters	Status of PDFs prior to displacement	State of the PDFs in the relocated site as it stands now after laps of 12 years (Current Status)	Whether overcome the risk factor	Reasons for not overcoming the risk factor	Lessons learnt from the study for the future projects & Strategies to overcome the risk factors
Marginalisation	Percentage of families landless.	4.65%	10.89%	This is also a Risk Factor which the PDFs have not overcome. But this Risk Factor has moderately affected the PDFs	i) Higher percentage of PDFs became landless because they spent the compensation amount & R.A. for unproductive purpose.	i) NGO assistance to be provided to the PDFs for identification of land and counseling for the purchase of land.
	Percentage of marginal farmer & small farmer families.	37.46%	62.93		ii) Absence of land purchase committee.	ii) Land Purchase Committees to be formed & function which will assist the PDFs in purchasing land.
	Percentage of Medium farmer families.	55.65%	25.24%		iii) The PDFs have become marginalized as their operational land holding has been reduced because of non-reclamation of the agricultural land provided by the Project as R.A.	iii) Agricultural land should be reclaimed & levelled before displacing the PDFs & before handing over to them.
	Percentage of large farmer families.	02.24%	00.93%		iv) Acquisition of objectionable encroached Forest/Govt./Waste Land.	iv) Waste land available near the habitational site should be allotted in favour of the PDFs to the extent possible.
	Percentage of shop keepers.	06.57%	14.10%		v) Forceful encroachment of patta land of the PDFs by the host villagers.	v) All efforts be made to ensure that the land to be allotted in favour of the PDFs is free from encroachment & all other encumbrances.
	Average operational land holding size per family.	Ac-6.11	Ac-3.84		vi) Restriction of shifting cultivation in the forest land near the relocation site.	

Table 5.8 Loss of access to common property

Risk Factor	Parameters	Status of PDFs prior to displacement	Status of the PDFs in the relocated site as it stands now after lapse of 12 years (Current Status)	Whether over come the risk factor	Reasons for not overcoming the risk factor	Lessons learnt from the study for the future projects & Strategies to overcome the risk factors
Loss of Access to Common Property	Percentage of families having encroached forest/Govt. land.	85.42%	49.84%	This risk Factor has severely affected the PDFs	i) Acquisition of objectionable forest & Govt. land. ii) Restriction of shifting cultivation in the nearby locality of relocation site. iii) No space earmarked for burial ground, grazing land & waste land for the PDFs in the close vicinity of the relocation site. iv) Relocating the PDFs away from the forest. v) Host population not allowing the Govt./Forest land in the vicinity of the relocation site to be used by the PDFs.	i) Space for burial ground, grazing land to be earmarked for the PDFs in each relocation site. ii) To settle the unobjectionable encroached land in favour of the PDF. iii) To relocate the PDFs in the similar eco zone to the extent possible.
	Average Encroached Forest/Govt. land per family.	Ac-02.03	Ac-0.69			
	Total Area of land coming under shifting cultivation.	Ac-792.48	Ac-203.37			
	Average area of land per family during shifting cultivation.	Ac-1.27	Ac-0.21			
	Accessibility to Forest for MFP & Fuel wood.	Easily Accessible.	Accessibility has been restricted.			
	Percentage of families whose primary source of income is MFP.	33.81%	09.23%			
	Percentage of families having Access to grazing land.	100%	23.72%			
	Percentage of families who have access to burial ground.	100%	17.47%			

Table 5.9 Increased Morbidity

Risk Factors	Parameters	Status of PDFs prior to displacement	Status of the PDFs in the relocated site as it stands now after laps of 12 years (Current Status)	Whether over come the risk factor	Reasons for not overcoming the risk factor	Lessons learnt from the study for the future projects & Strategies to overcome the risk factors
Increased Morbidity	Source of Drinking water.	Dug well-61.54% Tubewell-05.93% Pond & Stream-32.53%	Dug well-34.25% Tube well-62.74% Pond & Stream-03.01%	The risk Factor has not affected the PDFs very adversely. Although the overall health status of the PDFs is not good by any standard.	i) All Resettlement colonies/clusters have not been provided with tubewells. ii) Some of the Rcs & clusters have tube wells not in working condition. iii) No space in the relocated sites have been earmarked for garbage disposal (Putting the garbage near the habitational site). iv) The medical centers are placed at far off distance. v) I.C.D.S. services & MCH packages are not delivered regularly.	i) All the relocated sites should have tubewells to ensure safe drinking water. ii) The tubewells should be repaired & maintained by the line Deptt. at regular interval. iii) Space for garbage disposal in the Rcs/clusters to be earmarked. iv) Each relocated site should be selected in a manner which will be nearer to a health centre. v) I.C.D.S. services & MCH packages should be delivered regularly in the relocated sites.
	Garbage disposal System	Usually near the house.	Usually near the house.			vi) Health delivery system to be intensified in the relocated site for a period of one year from the period of one year from the period of physical displacement as this is considered as the most vulnerable period for the women & the children in particular and for the entire population in general.
	Toilet Habit	Almost 100% cases open latrine use.	10% people use septic Lartine.			
	Average distance of the medical center from the Dps House site.	9 km	15km			
	Frequency of visit of health worker to the village.	2 time in the month on the Avg (But very casual & irregular).	2 time in month on the Avg (But very casual & Irregular).			
	Birth attendance at the point of delivery.	Usually/invariably indigenous untrained Dhai.	By untrained Dhai in most of the cases; But in some cases Trained Birth Attendant.			

Contd..

Table 5.9 - Increased Morbidity (Contd.)

Risk Factors	Parameters	Status of PDFs prior to displacement	Status of the PDFs in the relocated site as it stands now after laps of 12 years (Current Status)	Whether overcome the risk factor	Reasons for not overcoming the risk factor	Lessons learnt from the study for the future projects & Strategies to overcome the risk factors
Increased Morbidity	Immunisation status of the infants, children & pregnant women.	In most of the cases infants, children & women were not immunised.	About 40% infants children & women are immunised.			
	M.M.R.	High	Moderate. It has reduced considerably.			
	I.M.R.	Alarmingly High	Reduced. But it is still much higher than the state average (179).			
	Incidence of Diseases	Malaria, Diarrhoea etc.	There is acute problem of Malaria which is even causing death. High infant & child mortality is caused due to Diarrhoeal death.			

Table 5.10 - Social Disarticulation

Risk Factor	Parameters	Status of PDFs prior to displacement	Status of the PDFs in the relocated site as it stands now after lapse of 12 years (Current Status)	Whether overcome the risk factor	Reasons for not overcoming the risk factor	Lessons learnt from the study for the future projects & Strategies to overcome the risk factors
Social Disarticulation	Settlement pattern	Near relatives, Kin members & same ethnic group families were settled in a block.	Resettlement has been such a manner that different caste & ethnic group families have been suffled & put together for which group identity has been lost.	Serious social disarticulation has taken place.	i) PDF have not been resettled enblock & in each R.C. & cluster PDFs belonging to several villages & ethnic category have been resettled.	i) The PDFs of one village & one community should be resettled in one relocation in enblock.
	Social organization	Lineage & clan group identity was there & special organization.	Traditional & social organization, especially of the tribals is no more in force.		ii) Socio-cultural ties have been shattered.	ii) Religious centers & cultural centers as existing in the predisplaced village to be relocated in the relocated site.
	Political organization	Political organization was very deep rooted & strictly adhered.	Traditional political organization shattered and not in existence/not in force/weathered away.		iii) Traditional, political & social organization has been dismantled.	
	Marriage Distance	Strictly adhered was restricted/limited & it was performed in a very close circle.	Marriage distance increased & marriage circle expanded.		iv) Marriage distance has become more.	
	Kinship ties.	Were very intense & closely knit.	Kinship ties has been severed in majority of the cases because of dispersed distribution of the kin members.		v) Kinship ties has been shattered. vi) Later family dependence & labour exchange system has reduced considerably. vii) Joint family system has disintegrated.	

Table 5.10 Social Disarticulation (Contd.)

Risk Factor	Parameters	Status of PDFs prior to displacement	Status of the PDFs in the relocated site as it stands now after lapse of 12 years (Current Status)	Whether over come the risk factor	Reasons for not overcoming the risk factor	Lessons learnt from the study for the future projects & Strategies to overcome the risk factors
Social Disarticulation	Inter family dependence & cooperation. Intercaste/Intra caste dependence & co-operation	Was very high	Reduced Considerably			
	Type of family	72.35% joint family & 27.65% nuclear family.	18.18% joint family & 81.82% nuclear family (disintegration of Jt. Family system).			
	Existing of a Community Hall	77.56% of the submerged villages had a community hall.	89.10% of the RCs & clusters have a community hall.			
	Labour exchange system	Was a common practice & phenomenon.	Is a very rare practice & labour exchange system is not the pattern.			

Table 5.11 Food Insecurity

Risk Factor	Parameters	Status of PDFs prior to displacement	Status of the PDFs in the relocated site as it stands now after lapse of 12 years (Current Status)	Whether over come the risk factor	Reasons for not overcoming the risk factor	Lessons learnt from the study for the future projects & Strategies to overcome the risk factors
Food Insecurity	Average Crop (Paddy) yield per family per year.	16 Quintals	7 Quintals	Food Insecurity is very acute & the people are very often half starved during lean season.	i) The agricultural land provided by the project has not been reclaimed & leveled properly. ii) Operational land holding has been reduced considerably. iii) Shifting cultivation has been restricted. iv) Access to forest has been reduced & there has been a reduction on the minor forest produce. v) No agriculture extension service facilities.	i) Before handling over the land to the PDFs, the project should reclaim & level the agricultural land. ii) Waste land near the relocation site should be allotted to the PDFs for mixed plantation & horticulture to supplement food requirement. iii) Agriculture extension farm training should be provided to the PDFs on modern agricultural methods & allied choices like piggery, goatery, duckery etc. iv) High yielding paddy seeds, farm equipments, pesticides and chemical fertilizers to be provided to the PDFs at subsidized rate. v) Water harvesting structures in the form of check dam should be built with the assistance of both the PDFs & the project so as to irrigate the land for increasing the cropping intensity.
	Principal source of food/livelihood.	Agriculture	Agriculture & wage Earning			
	Secondary source of food/livelihood.	Minor forest produce.	Fuel Wood/sale			
	Principal diet	Rice, forest fruits, roots, & tubers.	Rice			
	Supplementary Diet	Salap (Juice of a tree) & other forest based edibles.	Occasionally salap & very rarely small quantity of MFP.			
	Average surplus of food grains (paddy) per family in a year.	On the average 1 to 2 quintals of paddy per family	Nil			

Table 5.12 - Joblessness

Risk Factor	Parameters	Status of PDFs prior to displacement	Status of the PDFs in the relocated site as it stands now after lapse of 12 years (Current Status)	Whether overcome the risk factor	Reasons for not overcoming the risk factor	Lessons learnt from the study for the future projects & Strategies to overcome the risk factors
Joblessness	Percentage of PDFs cultivating only their own land.	16.35%	19.97%	Joblessness is also another factor which has been heavily responsible for the impoverishment of the PDFs. The PDFs have not got away with this.	i) Operational land holding has been reduced considerably.	i) To allot land to PDFs after it is reclaimed & levelled.
	PDFs cultivating their own land along with share cropping.	35.90%	15.38%		ii) Dispossessed from encroached forest/Govt. land.	ii) Govt. waste land near the relocation site should be allotted to the PDFs.
	Share croppers.	21.31%	04.65%		iii) Substantial reduction in farm wage labour.	iii) All the Govt. work should be done in the area involving the PDFs.
	Share croppers & occasional wage earners (Farm).	08.33%	02.72%		iv) Non availability of daily labour work.	iv) All the PDFs below the poverty line having no substantial source of livelihood should be economically rehabilitated by tying up with the different existing poverty alleviation & income generating schemes.
	Farm wage earners.	06.25%	39.10%		v) No effort for viable alternative economic rehabilitation of the PDFs	
	Service workers	05.77%	07.05%			
	% of major persons without any gainful occupation.	06.09%	11.21%			
	No. of days a month getting work.	290 days	119 days			
	Percentage of families BPL	31.73%	47.92%			

Table 5.13 - Homelessness

Risk Factor	Parameters	Status of PDFs prior to displacement	Status of the PDFs in the relocated site as it stands now after lapse of 12 years (Current Status)	Whether overcome the risk factor	Reasons for not overcoming the risk factor	Lessons learnt from the study for the future projects & Strategies to overcome the risk factors
Homelessness	Percentage of families Having their own home.	63.94%	93.4%	Homelessness being one of the Risk factors contributing to improverishment has been successfully overcome to a large extent by the PDFs.	i) PDFs have squandered away the compensation money & the rehabilitation assistance for unproductive purpose.	i) The PDFs below the poverty line should be provided with house by Govt. under Indira Awas Jojana.
	Percentage of families having concrete roof house	04.65%	03.37%		ii) No counselling, awareness generating and motivating the PDFs to construct house by the project authorities have been done.	ii) N.G.O. should be associated in the project to motivate the PDFs & to do counseling so as to construct the house.
	Percentage of families Having tiled/asbestos roofed house.	11.06%	22.92%		iii) One time release of cash for the house construction allowance without keeping any scope for the check of the progress.	iii) The house construction allowance from out of the R.A. should be released in phased manner looking at the progress of the house construction.
	Percentage of families Having thatched house.	77.08%	67.14%			
	Percentage of families Staying in relatives/father's rented house & the like.	36.06%	06.57%			
	Percentage of families having two roomed house.	52.40%	31.14%			
	Percentage of families having three roomed house.	36.79%	06.57%			
	Percentage of families Having four roomed house.	07.05%	31.09%			
	Average no. of persons per living room.	02.88%	52.72%			

5.1.8 Case Study from Samal Barrage Project:

“Languabeda, Sagadipal and Bolangi, 3 of the 5 submerged villages of the Rengali Samal Barrage Project are classic examples of this situation. This project which proposes to impound the water released by the Rengali Hydropower project at Samal and release it for irrigation through its proposed left and right hand side canals, has got only the barrage ready by now. The canals are under construction. The barrage is not fully operational, though the gates are ready, water level has been kept at 2/3rds of the maximum, in order for the NTPC (National Thermal Power Corporation) power plant & colony’s industrial and domestic use. This impounding has taken place two years back. This has used serious threat for these three villages where marginalisation has set in much before actual displacement.

The 136, 150 and 346 “Displaced Families” respectively of the 3 villages, viz., Sagadipal, Languabeda and Bijigol suffers from an acute feeling of social claustrophobia and voicelessness. All their lands, agricultural & homestead have been acquired a decade back, reflecting poor project planning and proper rehabilitation planning. As the villages ceased to have the status of “Revenue village”, all development activities/interventions from the state have stopped. Even a natural fire accident does not entitle a family to relief from the Revenue Department. They obviously apprehend in case of an epidemic, no medical health will be forthcoming either. The twin villages, Sagadipal and Languabeda are literally cut off from the rest of the World, by Brahmani River, hills and Tikira Nalla. Tikira Nalla was dry earlier. It now has to be crossed by a ferry boat now because of the backwaters of barrage pond. These two villages have fine and fertile agricultural land. The DPs are economically viable, in a limited sense, but being geographically, “marooned”, they feel their lands have no real worth. The case of Bolangi is opposite. Here, nearly half of agricultural holdings have been submerged while the homestead lands are higher up. Fluctuation in the pond level, keeping view the requirements of NTPC, causes further economic instability. Farming families once prosperous, have had slid down to the marginal farmer and landless labourer class.

In all the 3 villages, actual displacement was so long viewed as a remote possibility and the villagers never considered shifting a serious option. With the barrage now partially impounded for NTPC, since the last 2 years, the villagers want fast disbursement of rehabilitation assistance and NGO help for identifying viable options. The Land Acquisition grant, received several years back has mostly been squandered in consumption related activities. This is more acute in Sagadipal and Languabeda, which are dominated by “Brahmin” castes. In Bolangi, 86 of the 346-DPs have purchased land out of the compensation money & their own savings.

FGD with women at Bolangi reveal that they have a bleak perception of their post-relocation life. The long gap between land acquisition and actual rehabilitation has accentuated the feeling of helplessness. The middle cultivators also have to buy fodder as their lands have been submerged. Their women feel involuntarily equalised with women of wage labour families. So long as they supervised agricultural operations, a part of marketed surplus was retained by the women members, though, in a discrete manner. Since lands have been impounded by water and yet, alternate lands not found, the women feel they have been totally subjugated to men as economic entities. The class conflict and gender insecurities have been heightened in the present situation.

The unmarried girls of Languabeda and Sagadipal reveal that marriage market has suddenly shrunk for them and dowry rates have gone up, as their “Displaced” fathers have not been able to identify the rehabilitation option. In these areas, it was often common for young women to benefit from development training programs which give them individual assets (like sewing machines!). But their not having access to any development programs, also has an indirect implication for eligibility as a bride. The Policy recognises only unmarried women of 30 and above as a separate family. The equivalent age limit for a son is 18. Languabeda & Sagadipal has a relatively larger proportion of girl children. The genuine concern here is that the policy is discriminatory against families with only/elder girl children and with no/only minor sons. The young women feel that they have lost the battle of economic survival at the very beginning.

The Socio Economic Survey & RAP for all these 3 villages are ready. Local NGOs of repute have been associated for RAP implementation. If the 'land-for land' policy and other economically viable options are identified with proper consultation, and in a fast, time bound manner, this phenomenon of marginalisation can be neutralised. However, in a situation like this, where time gap between marginalisation and actual displacement is long, provision of only maintenance allowance is not enough. Innovative schemes for meaningfully engaging the younger age groups have to be carefully conceived and process of training initiated for skill development of the PAPs may be taken up so that their worth as human material does not get undervalued. The policy should provide for access to food/fodder at a reasonable price for the DPs in the intervening period. Entitlements for women & men should be equalised at the earliest.

To sum up, the Odisha Rehabilitation Policy which goes much beyond economic compensation for lost assets and recognises the social risks of the DPs, is generally robust against the risks of - Landlessness, Joblessness, Homelessness, Marginalisation and Loss of Access to Common Property.

From the insights obtained through Case Studies, it can be suggested that the R&R Policy could be further strengthened by:

- i) Ensuring freezing of land price in a situation of scarcity of land.
- ii) (a) Contemplation of large investments for employment generation in submerged areas that are mainly bearing the negative impact of the project. (b) Providing innovative employment generation measures in the intervening period for selected economic groups.
- iii) Providing food security and health care measures for all, including those not opting for Govt. sponsored colonies.
- iv) Compensating loss of common property for all groups including tribals, while ensuring eco balance and lastly, though the most important of all.
- v) Bringing about gender parity by giving equal status of independent family to both a girl and a boy of 18 and above.

Gender disparity accentuates any negative impact of displacement, and this dimension of problem is totally avoidable. The welfare of the most vulnerable members of the family depends in the mother's economic well-being. This reality has manifested itself in different forms time and again.

In the end, it needs to be reinforced that the aforesaid suggestions do not constitute a criticism of the present R&R Policy of Odisha, which in terms of its boldness and pro-people intent, is a landmark. The suggested supplementations will, it is hoped, further upgrade it to a management tool for comprehensively countering the displacement risks and it will be possible to replicate the implementation experiences in projects across the country." Source: Ibid Pg.38-42)

5.2 Environmental Issues:

5.2.1 Introduction:

“All dams and reservoirs as of many other human activities, become a part of their environment which they influence and transform to certain degrees and within a range that vary from project to project. Frequently seeming to be in opposition, but not necessarily irreconcilable, dams and their environment interrelate with a degree of complexity that makes the task of the dam engineer particularly difficult. The solution must be to find the golden mean by striking a balance between divergent, and sometimes contradictory goals.

We need dams and the many benefits which their reservoirs offer all over the world, by storing water in times of surplus and dispensing it in times of scarcity. Dams prevent or mitigate devastating floods and catastrophic droughts. They adjust natural runoff with its seasonal variations and climatic irregularities to meet the pattern of demand for irrigated agriculture, power generation, domestic and industrial supply and navigation. They provide recreation, attract tourism, promote aquaculture and fisheries, and can enhance environmental conditions. Thus, dams and reservoirs have become an integral part of our engineered infrastructure, of our man-made basis of survival. Still more dams will be needed in the future for the adequate management of the world's limited, unevenly distributed and in many places acutely scarce water resources. But more and more we also recognize an urgent need to protect and conserve our natural environment as the endangered basis of all life. And there is also a social side to the comprehensive conception of environment: the people, their land and settlements, their economy and traditions. The impact of dams and reservoirs on this environment is inevitable and undeniable; land is flooded, people are resettled, the continuity of aquatic life along a river is interrupted, and its runoff modified and often reduced by diversions.

Thus, dam engineers find themselves confronted with the basic problems inherent in the transformation of the natural world into a human environment. In our never ending quest to provide a growing number of people with a better life, the need to develop natural resources, including water, means that the natural environment cannot be preserved completely unchanged. But great care must be taken to protect the environment from all avoidable harm or interference. We must cooperate conscientiously with nature's inherent fragility as well as its dynamism without even overtaxing its powers of regeneration, its ability to adapt to a new but ecologically equivalent equilibrium. And we must ensure that the people directly affected by a dam project are better off than before.

The contribution of dam engineers to the development of water resources is based on proven technology, as our profession's track record of over 39 000 [According to the criteria of the ICOLD World Register, dams higher than 15 m (or higher than 10 m but with more than 500 m crest length, or more than 1 million cum storage capacity, or more than 2000 cumec spilling capacity)] large dams clearly shows. This technology continues to benefit from ongoing refinement and a steady growth of knowledge and experience, in particular with regard to its social and environmental consequences. Guided by the concept of sustainable development, ICOLD will make every effort to make the contribution expected of a leading professional organization to the further improvement of dam engineering. This contribution will reflect increased environmental sensitivity as well as the traditional technical excellence.

5.2.1.1 The Role of Dams and Reservoirs:

There is no life on earth without water, our most important resource apart from air and land. During the past three centuries, the amount of water withdrawn from freshwater resources has increased by a factor of 35, world population by a factor of 8. With the present world population of 5.6 billion still growing at a rate of about 90 million per year, and with their legitimate expectations of higher standards of living, global water demand is expected to rise by a further 2-3 percent annually in the decades ahead.

But freshwater resources are limited and unevenly distributed. We cannot forever try to meet insatiable demands by continuously expanding a supply that has limits. In the high-consumption countries

with rich resources and a highly developed technical infrastructure, the many ways of conserving, recycling and re-using water may more or less suffice to curb further growth in supply. In many other regions, however, water availability is critical to any further development above the present unsatisfactorily low level, and even to the mere survival of existing communities or to meet the continuously growing demand originating from the rapid increase of their population. In these regions man cannot forego the contribution to be made by dams and reservoirs to the harnessing of water resources.

Seasonal variations and climatic irregularities in flow impede the efficient use of river runoff, with flooding and drought causing problems of catastrophic proportions. For almost 5000 years dams have served to ensure an adequate supply of water by storing water in times of surplus and releasing it in times of scarcity, thus also preventing or mitigating floods. In response to enormously increased demand, more than half of ICOLD's registered 39000 large dams have been built in the past 35 years. They have become an integral part of our technical infrastructure, and throughout the world they enhance our basis of life by offering many indispensable benefits. Still more dams will be needed in the future for the adequate management of the world's limited, unevenly distributed and in many places acutely scarce water resources.

This applies in particular to the developing regions of the world, which account for 70 percent of the world population, and for no less than 94 percent of annual population growth. One billion people there, are suffering from chronic undernourishment or plain starvation, with between 10 and 15 million children dying of hunger every year. About 1.5 billion people have no access to a reliable source of drinking water, and more than two dozen countries have not enough water to sustain their populations properly. Millions die from water related diseases every year. The result is an exodus of the impoverished rural populations to the even greater inhumanity of the vast shanty towns surrounding the big cities. Of the 22 cities which will have more than 10 million inhabitants by the end of this century, 18 will be in developing countries.

In many of these countries, increased food production is only possible through improved or increased irrigation. At the present time, about 250 million hectares of land are under irrigation, growing one third of our food on less than one fifth of the world's total cultivated area, and accounting for almost three quarters of world water consumption. In conjunction with great efforts to develop effective ways of saving water by avoiding losses in the distribution systems, and by applying more skillful irrigation techniques, UNDP (the United Nations Development Program) is aiming at a 3 percent compound rate of growth in irrigated agriculture to meet the needs of an extra one billion people in the next ten years. Half of them will be city dwellers with a concentrated drinking water requirement. Since the ground water reservoirs presently tapped to provide about half of irrigation, drinking and industrial water supply are already heavily overdrawn in many parts of the world, the only large-scale solution apart from saving water is to increase the share of surface water from storage reservoirs.

Given the foreseeable depletion of fossil fuels, which presently are used to satisfy three quarters of primary energy requirements worldwide, plus the problem of the green house effect and global warming, there is an urgent need to gradually replace them with methods of energy production which do not release CO₂, (or airborne mercury from coal-fired plants) into the atmosphere and which draw on renewable sources of energy. In the short and medium term, however, the predominant sources of renewal energy that will permit large-scale exploitation will be biomass and hydropower, before new sources like the direct harnessing of the sun's energy by photovoltaics will be ready to make contributions of the same order of magnitude.

Hydropower is solar energy in naturally and ideally concentrated form that can be utilized with the help of a mature and familiar technology with unsurpassed rates of efficiency and without depriving future generations in any way of raw materials or burdening them with pollutants or wastes. In many developing countries, it is the only natural energy resource. With a total annual generation of 2.1 million GWh, hydropower accounts today for 20 percent of electricity production and about 7 percent of total energy production worldwide. Even at a conservative estimate, the total exploitable hydropotential in

the world amounts to at least six times as much. Very often, hydropower pays for multipurpose benefits, too. When this is taken into account, and when all environmental and social costs are internalized, hydropower compares favorably with other sources of energy.

Flood control has always been a particularly significant motive for dam construction and frequently its primary purpose. It will continue to be so, as long as about 40 percent of all fatalities from natural catastrophes worldwide are caused by flooding, amounting to a frightening total of nearly 100 000 per year. Compared with the main requirements of irrigation, domestic and industrial water supply, energy production and flood control, the other purposes and benefits of dams such as navigation, fisheries and tourism, improvements to the infrastructure, job creation and on-site training, are of generally minor importance, but must nevertheless not be disregarded or underrated". (Source: Hydropower and River Valley Development, ICOLD-Position paper on Dams and Environment - Post Conference Proceedings & Recommendations, 1st & 2nd Dec. 1999, New Delhi, Pg.14 to 23)

"Large dams do interrupt the natural flow regime of a river, which has certain undesirable side effects on the environment. They do interrupt the sediment transport of rivers, and some reservoirs are threatened by sedimentation. They often alter the temperature of the water for some distance downstream. They do impede the migration of fish and special measures taken to counteract this phenomenon are not always entirely successful. They may in some cases threaten rare species. But for most of the thousands of dams listed in the ICOLD register, the benefits far outweigh these drawbacks! Critics search for faults in individual cases, and, sometimes that they deliberately ignore or belittle the benefits that generally are orders greater than these faults. When the Operations Evaluation Division of the World Bank brought out a report on fifty of the dams in the financing of which the Bank was involved, they found that: The finding that 37 of the large dams in this review (74 percent) are acceptable or potentially acceptable, suggest that, overall, most large dams were justified and could have been successful in making positive contribution to the economy within current operational guidelines. A further 8 dams reviewed would have been acceptable under the standards applicable at the time of their construction. This means that 90% of the 50 dams made the grade, when hindsight is discounted." (Source: Ibid-Alternatives to Dams? by Theo PC Van Robbroeck Pg. 28-29)

5.2.1.2 ICIDS Position

Effectiveness of Dams for Development:

"Large or small dams, if built without adequate preparatory work, can fail to deliver expected results. Any dam could thus prove less effective than planned. It is therefore necessary to select cases of success or failure of both large and/or small dams. Lessons are to be drawn from failure to guide future action. The owners of such dams have to be approached first, for their assessments. If a new dam is identified, a bench-mark status, if not available at the time of construction, might have to be ascertained to realistically assess its effectiveness. Where much depends on how the delivery system is operated, the dam is hardly the reason for any loss of efficiency. Greater attention is necessary in the irrigation sector to bring about and maintain perfection in the delivery systems.

Storages of various magnitudes are a requirement for practically whole of the developing world and dams of various sizes fulfil that necessity. It is therefore, imperative that such a development process is supported by effective procedures to minimize negative effects, if any, and enhance benefits. Large dams contribute significantly to the productive efficiency of irrigation, in addition to giving ancillary and intangible benefits. The large dams built in the past have provided water supplies to needy areas for growing food, for drinking water, for reducing flooding, for generation of hydropower at lowest of costs from amongst various options. Smaller a dam, more are the costs per unit of water stored, but every size has its role in development of basin resources. They are complementary to each other. They cannot replace each other." (Source: Ibid- ICID, Position paper on the role of Dams Pg.43)

5.2.2 Water Quality Standards and Waste water generation in Brahmani basin:

The Brahmani Basin is one of the major river basins with concentration of industry in the peninsular India. The river plays a very important role in the socio-economic, agricultural and industrial field of Odisha. The Brahmani Basin had a virgin natural environment earlier with a very little human interference and environmental pollution. Vast mineral deposits in the basin, availability of water and good infrastructure have favoured industrialization in the river basin, which gradually deteriorated the water quality in the river. Today Brahmani is considered as one of the most polluted rivers in the country. The intense human activity in the basin area, produces solid waste and liquid effluents which tends to be concentrated in specific areas of the urban and industrial settlements. Their discharge into the river in excess of its self-purification capacity, cause pollution within such stretches and extend down stream till the pollution load is dissipated or discharged. The polluted stretches of the river suffer from constraints of water use. The environmental degradation on various water quality parameters exceeding the tolerance limits, like drinking (vide Table 5.14) bathing, water supply, industrial, irrigation, navigation and aquatic life sustenance, cause health hazards and impair normal social cultural activity. The river water quality grading has been formulated by CPCB to indicate the designated best use of water like class A, B, C, D and E (vide Table 5.15)

Table 5.14 Drinking Water Standards (IS-10500:1991)

Sl. No.	Characteristics	Permissible	Excessive
Physical			
1.	Turbidity NTU	5.0	10
2.	Colour (Hazen unit)	5.0	25
3.	Taste & odour	Agreeable	Agreeable
Chemical			
4.	pH	6.5-8.5	>6.5-8.5
5.	Total dissolved solids	mg/l 500	2000
6.	Total Hardness	mg/l 300	600
7.	Chloride as Cl	mg/l 250	1000
8.	Sulphate as SO ₄	mg/l 200	400
9.	Fluoride as F	mg/l 1.0	1.5
10.	Nitrate as NO ₃	mg/l 45	100
11.	Calcium as Ca	mg/l 75	200
12.	Iron as Fe	mg/l 0.3	1.0
13.	Manganese as Mn	mg/l 0.1	0.3
14.	Copper as Cu	mg/l 0.05	1.5
15.	Zinc as Zn	mg/l 5.0	15
16.	Phenolic compound	mg/l 0.001	0.002
Bacteriological			
17.	Coliform Bacteria	MPN/100ml Absent	Nil
Toxic Material			
18.	Arsenic as As	mg/l 0.05	> 0.05
19.	Cadmium as Cd	mg/l 0.01	> 0.01
20.	Chromium as Cr+6	mg/l 0.05	> 0.05
21.	Cyanide as CN	mg/l 0.05	> 0.05
22.	Lead as Pb	mg/l 0.05	> 0.05
23.	Selenium as Se	mg/l 0.01	> 0.01
24.	Mercury as Hg	mg/l 0.001	> 0.001

(Source: Environmental Status of Angul-Talcher Area, OSPCB, 14th Sept. 2000 Pg.103)

Table 5.15 Designated best use of River Water

Designated Best use	Class of Water	Criteria
Drinking water source without Conventional 1 treatment but	A	1. Total Coliform Organisms MPN/100 ml shall be 50 or less. 2. pH between 6.5 to 8.5 3. Dissolved Oxygen 6 mg/1 or more 4. Biochemical Oxygen Demand 5 days 20°C 2 mg/1 or less.
Outdoor Bathing (Organised)	B	1. Total Coliform Organisms MPN/100 ml shall be 500 or less. 2. Ph between 6.5 to 8.5 3. Dissolved Oxygen 6 mg/1 or more 4. Biochemical Oxygen Demand 5 days 20°C 3 mg/1 or less.
Drinking Source with Conventional treatment and disinfection	C	1. Total Coliform Organisms MPN/100 ml shall be 5000 or less. 2. pH between 6.5 to 9.0 3. Dissolved Oxygen 4 mg/1 or more 4. Biochemical Oxygen Demand 5 days 20°C 3mg/1 or less.
Propagation of Wildlife and fisheries	D	1. pH between 6.5 to 8.5 2. Dissolved Oxygen 4 mg/1 or more 3. Free Ammonia (as N) 1.2 mg/1 or less
Irrigation, Industrial Cooling, Controlled Waste	E	1. pH between 6.0 to 8.5 2. Electrical Conductivity at 25°C micro mhos/cm Max.2.250. 3. Sodium absorption ratio Max.26 4. Boreen Max.2mg/1

(Source : Paribesh Samachar - Special Issue, Sept.1998, OPCB, Bhubaneswar, Table 3.2 Pg.12)

The Odisha State Pollution Control Board (OSPCB) have analysed the river flow and water quality through monitoring for a number of years which has led to identification of such polluted stretches in the major rivers that call for co-ordinated action to remove the environmental hazards.

The most effective way to deal with pollution problem is to intercept the effluents before their out fall and suitably treat them to remove the pollution load and dispose of the treated effluents on land. The national river action plan is a laudable stem in this direction. Ganga action plan has been the torch bearer for the nation wide programme. The source of pollution are principally two. The point source could be Mining, Urban or Industrial and non point source from Agricultural inputs like :- Fertilizer, Pesticides etc. The different sources however extend to large territories with diverse human activity predominated by Agriculture, Livestock farming and rural settlements and transport activities. The rapid urbanization and industrialization in the river basin has promoted Odisha State Pollution Control Board to plan for water quality management in the basin. OSPCB in collaboration with Central Pollution Control Board (CPCB) has been monitoring the water quality of the river on monthly basis since 1985 at 10 stations starting from Panposh to Pattamundai. The names of the monitoring stations are given table below:

Table : 5.16 Location of Sampling Stations on river Brahmani

Sl. No.	Monitoring Stations	Location	
		Latitude	Longitude
1	2	3	4
1	Up Stream Panposh	84 ⁰ 54' E	22 ⁰ 14' N
2	Down Stream Panposh	84 ⁰ 54' E	22 ⁰ 11' N
3	Bonaigarh	85 ⁰ 00' E	21 ⁰ 51' N
4	Rengali	85 ⁰ 01' E	21 ⁰ 30' N
5	Samal	85 ⁰ 12' E	21 ⁰ 02' N
6	Up Stream Talcher	85 ⁰ 28' E	21 ⁰ 00' N
7	Kamalanga Down Stream of Nandira Nalla Confluence Point	85 ⁰ 27' E	20 ⁰ 52' N
8	Bhuban	85 ⁰ 52' E	20 ⁰ 54' N
9	Dharmasala	86 ⁰ 12' E	21 ⁰ 53' N
10	Pattamundai	86 ⁰ 37' E	20 ⁰ 35' N

(Source: Ibid, Table 1.1, Pg.2)

Table 5.17 List of Industries in Brahmani Basin

Sl. No.	Name of Industry	Location
1	SAIL	Kiribur
2	SAIL	Bolani
3	L&T	Kansabahal
4	Odisha Cement Ltd.	Rajgangpur
5	Otta India	Kalunga
6	Chariot Cement	Kalunga
7	Sita Cement	Kalunga
8	Konark Cement	Kalunga
9	Mukund Cement	Kalunga
10	Konark Chrome Chemical Pvt. Ltd.	Kalunga
11	Odisha Industries	Kalunga
12	Uluchha Pigment	Kalunga
13	Siva Cement	Rourkela
14	Krishna Ferro Ltd.	Rourkela
15	Scan Steel	Rourkela
16	Vedavyas Cement	Rourkela
17	IDCO	Rourkela
18	SAIL	Rourkela
19	Asha Chemical	Rourkela
20	Vipra Industries	Rourkela
21	Indo Flugato	Rourkela
22	Asiatic Gases	Rourkela
23	I.T.C., Karej	Rourkela
24	Bishnu Enterprises	Rourkela
25	Santoshi Maa Iron Industries	Rourkela
26	Lotus Chemicals	Rourkela

27	Cast Profile	Rourkela
28	Pahari Bar	Rourkela
29	Gajlaxmi Iron Works	Rourkela
30	Shree Chemical Industries	Rourkela
31	Kalta Iron Mines	Kalta
32	Barsuan Iron Mines	Barsuan
33	NTPC	Kaniha
34	IWSS(MCL)	Talcher
35	TTPS	Chainpal
36	FCI	Talcher
37	NALCO	Angul
38	Navo Bharat Ferro Alloys	Meramandali
39	Odisha Synthetics	Baulupur
40	M/s MID-EAST Integrated Steel Company	Duburi
41	Neelachal Ispat Nigam	Sukinda

(Source: Brahmani Basin Plan, 3rd Spiral Study, Vol.I, OWPO, GoO, Nov. 2002, Annex. 2.3)

5.2.2.1 Waste Water generation due to Industries:

The establishment of an initially 1.0 million ton integrated Steel Plant at Rourkela in Sundargarh district including its large scale associated mines, ancillaries biproduct and downstream product units during late 1950's opened the doors for wide scale industrialization of the area. A comprehensive account of the industries, their water consumption and waste water generation is presented in Table 5.18. There are 8 industrial units apart from industrial estate discharging around 2,98,000 kilo litre of waste water daily.

Table 5.18 : Water Consumption and Waste Water Discharge from Industries

Sl. No.	Name of Industries	Production	Capacity	Water Consumption kilo litre per day	Waste water generation kilo litre per day
1	Rourkela Steel Plant	Iron/Steel	1.8 million Ton/annum	265580	120000
2	Rourkela Steel Plant	Fertiliser	460000 T/Yr	28807	7920
3	Fertilizer Corp'n	Fertiliser (Area)	4500 TPD million Ton/annum	45,883	16608
4	National Aluminium Company-Smelter Unit (Angul)	Aluminium	2,30,000 million Ton/annum	5066	4900
5	National Aluminium Company- Captive Power Plant.	Power	720 MW	135000	90000
6	ORICHEM Limited	Sodium dichromatic Basic chromate Sulphate Yellow Sodium Sulphate	300 MT/M 60 MT/M 190 T/M	170	10
7	Talcher Thermal Power Plant (TTPS)	Electric Power	480 MW	13227	6483
8	Talcher Super Thermal Power Plant, NTP, Kaniha	Electric Power	1000 MW	137099	52080
		Total		6308320	298001.0

(Source: Ibid: Table 2.1, Pg 7)

Table 5.19 : Waste Water Discharge from Chromite Mines of Sukinda Area

Sl. No.	Name of the Mines Existing	Domestic Water (KLD) Consumption	Domestic Waste Water Generated (KLD)	Mine Water Discharge (KLD)
1	2	3	4	5
1	Sukinda	833	666.4	2733
2	Sukhrangi	11	8.8	-
3	Kathpal	43	34.4	333
4	Kaliapani	483	386.4	1453
5	South Kaliapani	20	16.0	2500
6	Kalarangi	1166	932.8	3233
7	Kathpal	43	34.4	-
8	Osthopal	29	23.2	-
9	Taitangi	5	4.0	28
10	Kamarda	3	2.4	15
11	Sarubali	23	18.4	123
	Total	2659.0	2127.2	10418.0

(Source : Ibid, Table 2.21 Pg.8)

5.2.2.2 Waste Water from Domestic Sources

“Domestic sources are major contributor of the waste water. It is estimated that the community waste is about four times the industrial effluent. In the Brahmani Basin all most all these waste are discharged untreated into the water course causing pollution in the whole system. The pollution load is mainly from municipal and other domestic source. Since the village usually do not have designated system of water supply and drainage of waste water, the little amount of liquid which comes out all most totally absorbed in the soil of the home stead areas and firm lands within the villages. On the contrary most of urban areas has some sort of drainage system and sizable part of the waste water finds its way to natural drainage channel. There are around 15 urban settlement in the Brahmani basin discharging about 1.8 MKLD of waste water.”

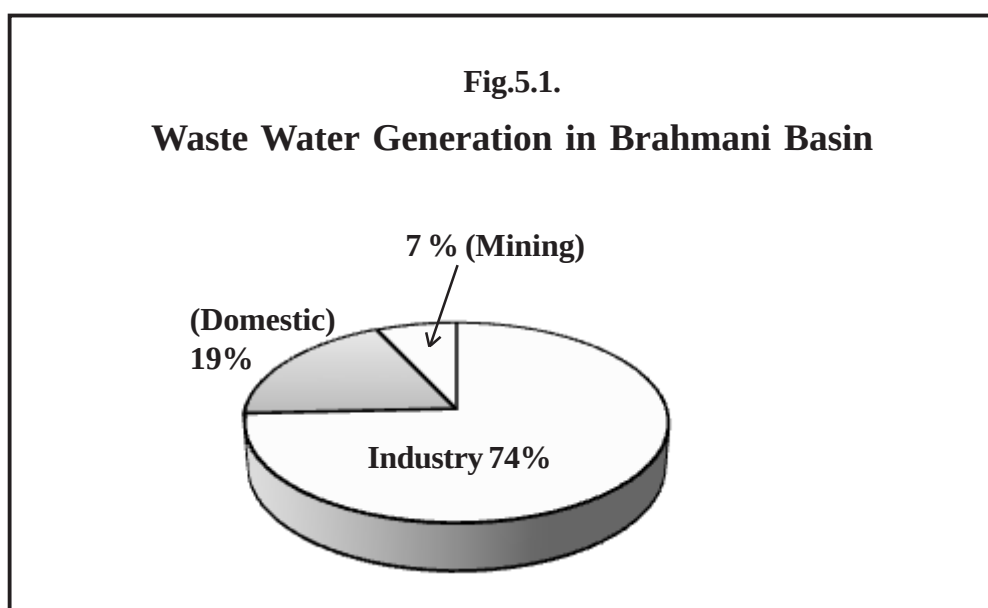
A list of major cities and towns and urban areas conglomerates, their water use and waste water discharge is presented in the Table 5.20.

Table 5.20 : Water Consumption & Waste Water Generation from Urban Settlements

Sl. No.	Name of the City / Town	Water Consumption KI/Day	Population	Est. waste water discharge (80 %) of consumption in KLD	25g/capita/day (ref. CPCB) Approx Ton/Day
1	2	3	4	5	6
1	Rourkela Steel Plant	33561	206566	26848	5.16
2	Rourkela Civil Township	26982	224604	21561	5.61
3	Birmitrapur	1130	30425	904	0.76
4	Rajgangpur	2635	43912	2108	1.09
5	Deogarh	783	20085	626	0.50
6	Talcher	2274	34984	1819	0.874
7	Angul	77475	1139341	61980	28.48
8	Talcher Thermal Township	13307	38022	10645	0.95
9	MCI Colliery Talcher	14309	18583	11447	0.464
10	FCI Township	3176	7059	2541	0.176
11	Bhuban	906	20134	725	0.503
12	Kamakshyanagar	570	15002	456	0.375
13	Dhenkanal	4151	57651	3321	1.44
14	NTPC Kaniha	17285	8115	13828	0.20
15	NALCO Township	35283	16038	28226	0.40
	Total	233827	1880518	187035	46.982

(Source: Brahmani Basin Sectoral Study – Eco Resources - Sheladia Associates Inc., USA, Table5, Pg.53)

A comprehensive account of waste water generated from different sources is presented in Figure 5.1. It seems that industries contribute maximum amount of waste water i.e. 74% followed by 19% from domestic sources and 7% from mining.



5.2.2.3 Waste Water Generation from Agriculture Sources

The net shown area in the Brahmani basin during 1999-2000 is 863.25 thousand hectare. The Rengali dam project which is a major project has been completed since 1985. But the irrigation potential of the project is yet to be fully developed. Pollution from non point source due to agricultural waste, fertilizer application are also considerable. The application of pesticides resulted toxic waste, which are composed of chemicals that generate very strong electrochemical potentials in the ground and the moving fluids in the ground generate electro-kinetic potentials. The table below summarizes the pollution load due to application of fertilizers in Brahmani Basin and its tributaries. Surface runoff from the agricultural field carry substantial quantity fertiliser waste into the river during monsoon.

Table 5.21 : Consumption of Fertilizer (1999-2000)

Sl. No.	Name of District	% of Dist. Inside the basin	N ('000 MT)	P ('000 MT)	K ('000 MT)	Total ('000 MT)	Gross cropped area ('000 ha)	Fertilizer Consumption (kg/ha)
1	2	3	4	5	6	7	8	9
1	Angul	66.29	3.22824	1.073898	0.570094	4.666816	200	23
2	Deogarh	85.45	1.28175	0.64942	0.35889	2.29006	83	28
3	Dhenkanal	88.88	1.970022	1.253208	0.631048	3.854278	265	15
4	Jajpur	62.94	5.891184	2.272134	1.560912	9.72423	175	55
5	Keonjhar	20.76	1.04630	0.483708	0.132864	1.662876	81	20
6	Kendrapara	41.89	2.4755699	0.959281	0.653464	4.088464	106	39
7	Sambalpur	20.60	2.84074	1.19686	0.64272	4.68032	53	89
8	Sundergarh	59.66	2.547482	1.151438	0.5966	4.29552	237	18
	Total		21.07601	9.939947	5.146612	35.26256	1.200	

(Source: Brahmani Basin Sectoral Study – Eco Resources, Sheladia Associates, U.S.A., Pg.54, Table-6)

5.2.3 Status of Surface water quality of Brahmani River

The river water quality has been monitored for 23 parameters on monthly basis. The data for 1990, 1994 and 1997 for 17 parameters are presented in Table 5.22.

“The Central Pollution Control Board has considered 4 parameters namely pH, DO, BOD and T.C. for classification of river basing on designated best use. In this report emphasis has been given for these parameters to derive a conclusion about the river water quality. In the trend T.C. (Total Coliform), it is found that the downstream of Panposh there is tremendous increase of this value which is far more than the standard value. This may be due to effluents of the Rourkela Steel Plant and Civil Township of Rourkela. Beyond this point, the values are high nearer to upstream of Kamalanga and downstream of Kamalanga on account of NALCO and other industries in Angul and Talcher. TC shows much variation at these two stations. PH as critical parameter do not show much variation and remain within the permissible range throughout the course of the river. The DO values are above the standard throughout its length but increasing trend of BOD value is only observed in the upstream of Kamalanga. It shows

that the sewerage from Angul and adjacent areas polluted the river stretch which have profound impact on river water quality.

The BOD as a critical parameters was seen to be above the limit of class equally through all the years of reference in all the sampling points. However the values remain particularly high at Panposh downstream and Kamalanga upstream and downstream. These three points being the confluence points of industrial drains like Guradih nalla and Nandira respectively were the impact stations. As normally expected, the river water shows remarkable assimilative capacity recording BOD values around 3 at Bonaigarh and Bhuban, the down stream stations of confluence. However record of higher value at Samal can be attributed exclusively to the discharge from Kaniha (NTPC).

The water quality index was calculated for BOD by OSPCB for years 1991, 1994 and 1997 at these impact stations. Water quality index of 100 and below are suggestive of conforming to the designated criteria level and exceeding 100 suggest deteriorating water quality. However, in spite of increasing activities in the basin through years, the water quality does not show declining trend over the years. In many cases the water quality though do not confirm to the designated use are showing improving trend. This phenomenon can be attributed to the control measures taken by the industries and mines at the instance and persuasion by Orissa Pollution Control Board. However, there is hardly any scope for complacency. The industrial and mining project expected to come in the area would bound to contribute significantly to the already stressed river water.” (Source: Ibid, Pg.55)

5.2.3.1 Analysis of Water quality results by C.W.C.:

The water quality data book (June 1992 to May 1993), “Eastern River Basins”, published by Central Water Commission, Eastern River Circle, Bhubaneswar, Oct 1974 has mentioned about the water quality of Brahmani. It enumerates that the Chemistry of the Brahmani water available at 6 selected regular monitoring stations in the basin, viz. the Sankh at Tilga, the Koel at Jaraikela, the Brahmani at Bolani, Gomalai, Talcher and Jenapur (from U/S to D/S) indicates good water for all purposes throughout the year. Moreover, it has been noticed that the quality of water during 1992-1993 was almost the same or marginally improved as was available during 1991-92. The quality of Brahmani water at its upper reaches upto Panposh (Rourkela) was very good throughout the year, but was deteriorated in the downstream reaches after ‘Panposh’ as per water quality results obtained from downstream monitoring station at Bolani located 10km downstream of Panposh. This water quality deterioration was mainly due to entry of the effluents discharge of Rourkela Steel Plant in the main river particularly in terms of chemical pollution.

The water quality results at 500 m. downstream of Rourkela Steel Plant effluents nalla confluence with Brahmani at Tarkara indicates the presence of additional chemical load which eventually increases the total chemical load in the Brahmani water to around 400 ppm. However, the water regains almost its original good quality during next 200 km. run upto Talcher. The water available at Talcher was very

good during 1992-93 and almost free from chemical pollution. But the quality of water at Talcher was once again deteriorated in the next 15 to 20 km run from Talcher. Here, effluents nalla carrying mostly of thermal waste and fertilizer waste is joining the main stream just 5km downstream of C.W.Cs hydrological and water quality monitoring station. The results of two additional impact stations located at the confluence of Nandirajhar Nalla with Brahmani and at Kamalanga 5km downstream of the above confluence indicate the sudden increase in total chemical load of around 300 to 700 ppm during lean period. Specific conductivity at Nandirajhar Nalla confluence was recorded as high as 822 micromhos during 12/92. Significant improvement of water quality was observed recorded as high as 541 micromhos/cm during Dec. 1992. Moreover, significant amount of nitrate, sometimes more than 100ppm was found in the water available at the confluence of Nandirajhar Nalla which gradually decreased with the run of the river and reduced to 12ppm only during March 1993 observed at tail-end site at Jenapur, almost 100km. downstream of the polluted zone near Talcher. On the other hand presence of fluoride exceeding the maximum tolerance limit (i.e. 1 ppm) was also recorded at the confluence of Nandirajhar Nalla and at Kamalanga. However, it was observed that the quality of Brahmani water at its tail-end site monitored at Jenapur has shown good improvement and remains good throughout the year. Some of the important findings of water quality are stated below.

The available water in the basin was observed at 6 regular monitoring stations purely basic in nature. The pH value of surface water in this basin lies between 7.0 to 8.4 which reveals that pH is well within the permissible limiting values of good water. The perusal of the data pertaining to conductivity indicates that Brahmani water in the upstream reaches particularly in the stream, Sankh at Tilga is almost pure and even free from mineral contents essential for human consumption and agricultural purposes. As such, the available water at Tilga may be pure and very good, but not ideal for all purposes. The water available at 4 regular stations i.e. Jaraikela in the upper reaches, Gomalai and Talcher at middle reaches and Jenapur at tail-end is very good throughout the year and could be utilised for all purposes. The quality of water in the remaining site in the basin i.e. at Bolani is generally good but significant amount of quality deterioration is noticed whenever sufficient quantity of waste effluents (mainly chemical waste) of Rourkela Steel Plant are discharged in the main river. However it has been observed that even after the deterioration of water quality, all the chemical parameters are well within the respective max. limiting values. So, the water at Bolani also could be utilised for all purposes. Observed sodium proportion and values of S.A.R. are very low throughout the year. Apart from sodium, dominant cations are calcium and magnesium and anions are bi-carbonates and chlorides. Nitrate and fluoride are present at all the stations but the concentrations available at regular monitoring stations are well below the limited values, except 15 to 20km. down stream reach from talcher.” (Source: Brahmani Basin Plan, 3rd Spiral Study Report, OWPO, GoO, Nov. 2002 Pg.29-30)

Table 5.22 Comparison of BOD values with those reported by CWC

Month	Panposh U/s		Panposh D/s		Kamalanga D/s	
2007						
	SPCB	CWC	SPCB	CWC	SPCB	CWC
January	1.3	1.0	4.8	1.0	2.00	1.0
February	1	1.0	3.2	1.6		1.8
March	2	1.8	3.6	1.2		1.8
April	1.4	1.4	4	1.4	2.40	1.1
May	1.6	1.2	3.6	1.2		1.6
June	1.4	1.5	3.2	1.6		1.8
July	0.6	2.0	4.2	1.5	1.40	1.6
August	0.7	1.1	1.4	1.1		1.3
September	0.6	0.9	1.8	1.1		1.3
October	1.2	2.1	2.2	1.2	2.00	1.6
November	0.7	1.1	4.1	0.9		1.6
December	1.4	1.7	2.5	1.7		1.6
2008						
January	1.1	1.6	3.3	2.5	1.6	1.7
February	1	1.7	3.1	1.5	2.0	2.7
March	1.6	1.2	4.3	1.9	2.1	1.2
April	1.1	1.0	5.3	1.7	2.5	1.9
May	1.6	1.8	5.4	1.8	4.6	1.8
June	2.5	2.0	5.4	1.6	3.6	1.6
July	0.8	-	3.2	-	1.2	-
August	1.8	-	5.6	-	2.3	-
September	0.6	0.6	2.5	1.4	1	1.2
October	0.5	1.0	4.2	1.0	1.6	1.6
November	1.3	1.2	5.7	1.6	1.6	1.4
December	0.6	1.0	4.9	1.4	2.2	1.6
2009						
January	1		4.9		1	
February	0.8		3.2		2.2	
March	0.6		5.6		1.6	
April	1		4.2		0.9	
May	0.8	1.4	5.8	3.1	2.4	1.2
June	1.1	1.2	5.5	1.2	2	1.4
July	0.9	1.0	4.7	1.4	1.9	2.0
August	0.4	1.0	5.6	2.2	1.6	1.6
September	2.7	1.2	3.3	1.2	2	1.8
October	1.1	1.2	2.6	2.8	1.7	1.6
November	1.5	1.4	3	1.8	2.2	1.6
Decemebr	1.6	2.2	5	2.8	1.8	1.8
2010						
January	2.0	1.4	2.6	1.6	2.1	2.0
February	2.4	2.0	4.2	2.0	1.9	2.2
March	1.5	2.7	0.9	1.9	1.8	1.9
April	1.5	1.0	5.2	2.9	2.6	1.8
May	1.0	1.6	3.0	1.6	2.6	1.8
June	1.0	1.6	2.2	2.2	1.8	2.2
July	1.2	1.0	5.6	1.9	2	2.2
August	0.6	2.0	4.9	1.4	2.2	1.6
September	1.0	1.2	5.4	2.2	2	2.2
October	1.4	1.5	3.4	1.1	1.8	1.7
November	1.3	1.5	3.9	2.3	2.3	1.5
December	1.2	1.4	3.2	1.9	1.6	1.7

Table 5.22(b) pH, DO, BOD and TC values (Bonaigarh, Rengali and Samal) (Brahmani)

Parameter	Year	Bonaigarh				Rengali				Samal			
		W	S	M	PM	W	S	M	PM	W	S	M	PM
pH	2007	7.8	8.0	8.0	7.4	7.2	8.0	7.9	7.2	7.3	8.0	7.6	6.9
	2008	8.2	7.8	7.6	8.0	7.8	7.7	7.4	7.6	8.2	7.7	7.7	8.1
	2009	8.4	8.2	7.5	7.6	8.2	7.4	6.6	7.0	8.2	8.2	7.1	7.2
	2010	8.0	8.0	7.5	7.3	7.2	6.8	7.0	7	7.9	7.2	7.6	8.2
DO (mg/l)	2007	8.0	7.7	7.0	7.5	8.0	8.9	7.4	7.6	8.2	8.9	7.9	7.9
	2008	8.9	7.4	8.2	8.6	8.6	8.1	6.9	9.2	7.9	8.0	7.3	9.6
	2009	11.0	9.3	6.8	8.9	9.9	9.2	7.7	10.6	8.7	8.0	6.9	9.9
	2010	8.6	7.9	6.6	6.1	8.4	8.2	6.3	6.2	8.8	7.8	6.3	6.0
BOD (mg/l)	2007	0.9	1.8	0.7	1.6	1.3	0.6	0.5	1.3	1.8	1.4	0.6	1.8
	2008	1.4	2.3	0.6	0.6	1.2	1.6	0.6	0.6	1.1	1.6	1.8	0.5
	2009	0.7	1.8	2.5	1.5	0.6	2.2	2.2	1.0	1.1	0.9	2.1	1.2
	2010	1.4	1.5	1.4	1.8	1.2	1.5	1.4	1.5	1.4	1.4	1.4	1.6
TC ('00) (MPN/100 ml)	2007	3.90	11.00	22.00	15.00	17.00	17.00	14.00	17.00	11.00	12.00	12.00	14.0
	2008	21.00	20.50	28.00	21.00	14.00	16.00	21.00	15.00	12.00	13.00	15.00	11.0
	2009	28.00	21.00	28.00	21.00	21.00	15.00	11.00	15.00	17.00	12.00	14.00	9.40
	2010	28.00	54.00	160.00	17.00	11.00	3.30	160.00	35.00	5.80	4.80	17.00	11.0

Table 5.23 Water Quality of Nandira River (Annual average)

S. No.	Location	Year	pH	SS	BOD	COD	Fluoride
1.	Upstream of NALCO	1997	8.06	60.6	3.0	10.0	0.64
	CPP Ash pond/colony	1998	8.36	40.1	3.2	12.6	0.76
	Effluent disposal point.	1999	8.30	68.0	3.2	14.0	0.77
2.	D/S of NALCO CCP/	1997	7.80	131.8	11.8	46.0	1.09
	Colony disposal point.	1998	8.20	40.5	8.9	33.5	0.92
		1999	8.24	150.0	7.6	30.9	0.82
3.	U/S of confluence of	1997	8.04	182.4	8.0	34.0	1.16
	Deojhar nallah.	1998	8.06	129.3	7.6	30.0	1.01
	(After FCI discharge)	1999	8.23	125.0	7.2	28.6	0.80
4.	D/S of confluence of	1997	8.20	541.4	8.7	32.0	0.83
	Deojhar with Nandira	1998	8.76	413.8	7.9	30.5	0.83
		1999	8.22	144.5	6.1	22.4	1.10
5.	U/S of TTPS (NTPC)	1997	7.96	419.6	6.7	26.8	0.76
	Ash pond overflow	1998	8.67	206.0	5.8	24.8	1.04
	discharge point	1999	8.04	200.6	5.2	21.6	0.99
6.	D/S of TTPS (NTPC)	1997	7.82	5380	7.3	31.2	0.24
	Ash pond overflow	1998	8.28	4517	6.8	27.3	0.44
	discharge point	1999	8.04	306.0	5.6	24.2	0.46
7.	U/S of NALCO CPP	1997	7.86	1978	6.7	28.0	0.06
	Industrial effluent	1998	8.45	906.0	6.6	26.9	0.43
	discharge	1999	8.04	430.5	6.0	24.0	0.49
8.	D/S of NALCO CPP	1997	7.86	1176	6.6	26.4	0.12
	Industrial effluent	1998	8.35	700.0	6.2	25.6	0.52
	discharge	1999	7.86	305.8	5.0	26.0	0.53

All values are in mg/l except pH.

Note : DO – Dissolved Oxygen

BOD – Bio-Chemical Oxygen Demand

COD – Chemical Oxygen Demand

(Source: Environmental Status of Angul-Talcher area, OSPCB, 14.09.2000 Pg.104)

5.2.4 Brahmani River and Its tributaries - Water quality

Brahmani basin is the 2nd largest river basin of the state occupying about 15% of the state geographical area. River Brahmani is formed by the waters of Sankha and Koel rivers at Vedavyasa near Rourkela. On the bank of the river, two major industrial towns i.e. Rourkela and Talcher are located. Details of waste water drains in Rourkela stretch and Talcher Stretch are given in Table 5.24 & 5.25 respectively.

Fig. 5.2 Schematic Diagram of Wastewater outfalls on River Brahmani along Rourkela and Talcher City

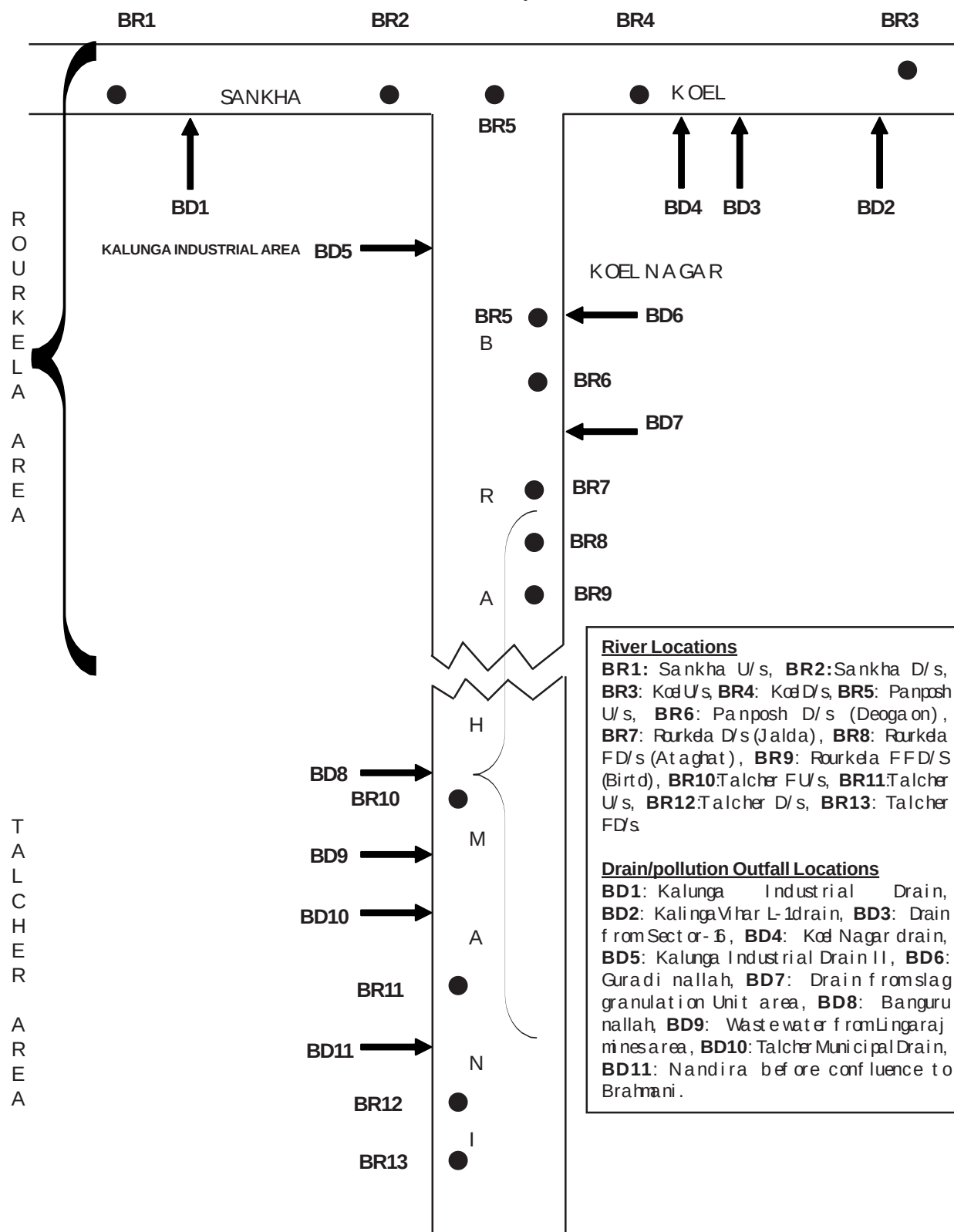


Table 5.24 Details of Wastewater drain in Rourkela Stretch

Sl. No.	River Stretch	Name of wastewater stream	Source of Pollution
Sankh River			
1.	Rourkela	1.Kalunga Industrial drain-I (BD1)	Industrial wastewater from a part of Kalunga Industrial Estate
Koel River			
2.	Rourkela	1.Kalinga Vihar L-1 drain (BD-2)	Domestic wastewater from Kalinga Vihar L-1 and part of Sector side and Chhend
		2.Drain of Sector 16(BD3)	Domestic wastewater from Sector 16, 17 and 18
		3.Koel Nagar drain (BD4)	Domestic wastewater from Koel Nagar
Brahmani River			
3.	Rourkela	1.Kalunga Industrial drain-II (BD5)	A part of industrial wastewater from Kalunga Industrial Estate and IDC
		2.Guradi nallah (BD6)	Combined industrial wastewater (RSP, Fertilizer, CPP etc.) and Domestic wastewater
		3.Drain from Slag granulation unit of RSP (BD7)	Industrial wastewater from Slag granulation unit area of RSP

Table 5.25 Description of the wastewater drains in Talcher Stretch

Sl. No.	River Stretch	Name of wastewater stream	Source of Pollution
1	Talcher	1.Banguru Nallah (BD8)	Domestic wastewater from mining colonies and wastewater from coal mines
		2.Natural drain from Lingaraj mines area (BD9)	Wastewater from Lingaraj mines and domestic wastewater from nearby residential areas
		3.Talcher Municipal drain (BD10)	Domestic wastewater from Talcher town
		4.Nadira jhor (BD11)	Domestic wastewater and STP outlet of TTPS, NTPC, Talcher and Industrial wastewater from CPP of NALCO

Results and Discussion

“The common water quality variables of concern in the wastewater drains in the present survey are confined to pH, dissolved oxygen (DO), biological oxygen demand (BOD), chemical oxygen demand (COD), electrical conductivity (EC), total suspended solids (TSS) and total dissolved solids (TDS). Presence of high concentrations of these pollutants beyond the critical values stipulated by Ministry of Environment and Forest (MoEF) (General standards for discharge of environmental pollutants : Part-A Effluents) are considered unacceptable in receiving water bodies.

The quality of wastewater is responsible for the degradation of the receiving water bodies. The potential deleterious effects of wastewater on the quality of receiving water bodies are manifold and depend on volume of the discharge as well as on composition of the wastewater. One of the major impact of such degradation is increased levels of oxygen demanding matter and other physical changes in receiving water bodies.” (Source: Identification and characterisation of polluting sources in Mahanadi and Brahmani river basins, OSPCB, 5th June 2012 Pg. 13-14).

5.2.4.1 (a) Sankh River

Seasonal variation in quality of wastewater stream before confluence with river Brahmani and water quality of river at the upstream and downstream of confluence of the stream from 2007-09 are furnished in Table.5.26.

Table 5.26 Quality of wastewater streams and Sankh river in Rourkela stretch (2007-09)

Monitoring Location	Season	Parameter						
		pH	DO (mg/l)	BOD (mg/l)	COD (mg/l)	EC (µs/cm)	TDS (mg/l)	TSS (mg/l)
Sankha U/S (R1)	W	7.8 (7.7-8.0)	9.0	1.4 (1.0-1.8)	8.3 (8.0-9.0)	224 (145-291)	127 (80-170)	20 (11-33)
	S	7.9 (7.7-8.2)	7.7 (7.5-8.0)	1.4 (1.0-1.6)	9.0 (7.1-10.4)	162 (118-237)	87 (73-113)	32 (15-54)
	M	7.8 (7.1-8.2)	7.1 (6.4-7.7)	1.5 (1.0-1.8)	9.7 (7.2-13.7)	227 (127-403)	149 (83-264)	611 (152-914)
	PM	7.6 (7.3-7.9)	9.0 (8.4-9.6)	1.9 (1.8-2.0)	11.6 (7.9-18.7)	139 (134-146)	83 (80-88)	47 (22-68)
Kalunga Industrial Drain I (D1)	W	7.9 (7.6-8.2)	7.2 (6.5-7.9)	7.5 (1.5-13.0)	23.4 (9.9-48.0)	349 (241-503)	201 (140-280)	22 (13-33)
	S	7.7 (7.4-8.3)	4.2 (3.9-4.8)	5.0 (3.0-8.0)	18.7 (15.2-22.2)	288 (243-328)	168 (150-190)	34 (23-52)
	M	8.0 (7.9-8.1)	6.5 (5.5-7.0)	12.7 (12.0-14.0)	41.1 (35.1-52.2)	334 (207-556)	200 (135-324)	254 (116-340)
	PM	7.9 (7.8-8.0)	6.9 (6.2-7.4)	2.0 (1.8-2.1)	11.5 (8.1-15.2)	252 (147-310)	155 (98-194)	59 (50-77)
Sankha D/S(R2)	W	7.7 (7.3-8.2)	8.3 (7.8-8.7)	1.7 (1.5-2.0)	6.7 (5.9-6.7)	213 (141-346)	120 (80-188)	20 (12-35)
	S	7.7 (7.0-8.2)	7.4 (7.0-7.9)	1.4 (1.2-1.7)	7.2 (6.5-7.9)	207 (160-280)	119 (95-160)	30 (17-48)
	M	7.5 (7.3-7.8)	7.2 (6.8-7.8)	1.4 (1.2-1.6)	10.1 (8.0-12.2)	263 (150-460)	130 (73-218)	607 (132-950)
	PM	7.8 (7.8-7.9)	8.0 (7.6-8.3)	1.2 (0.7-1.6)	7.9 (3.7-10.2)	153 (140-160)	73 (60-90)	51 (38-60)

Suspended solid content was maximum during monsoon period in Kalunga industrial drain-I as well as upstream and downstream locations of Sankh river.

(Source: Identification & characterization of Pollutng Sources in Mahanadi and Brahmani river basins, SPCB-5th June 2012, Table 13 Pg.30)

5.2.4.2 (b) Koel River

There were three major wastewater drains from Kaling Bihar L-1, Sector-16 area and Koelnagar discharged into Koel river in Rourkela stretch. Seasonal variation in quality of the wastewater stream before confluence with river Koel and water quality of river Koel at the upstream and downstream of confluence of the stream during the study period are given in Table 5.27. From the table it is observed that pollution load in terms of BOD in the wastewater drains were maximum during summer period and minimum during monsoon period. BOD in Koel river at the downstream station mostly remained within the tolerance limit i.e. 3.0 mg/l for Class-C surface water bodies. This indicates the availability of required flow in Koel river for dilution of the wastewater after confluence.

Table 5.27 Quality of Waste Water Strems and Koel river in Rourkela stretch (2010-2011)

Monitoring Location	Season	Parameter						
		pH	DO (mg/l)	BOD (mg/l)	COD (mg/l)	E.C. (µs/cm)	TDS (mg/l)	TSS (mg/l)
Koel U/s (BR 3)	W	8.2 (8.2-8.3)	11.5 (8.8-13.8)	1.2	9.3 (4.0-14)	268 (168-346)	150 (90-200)	28 (7-62)
	S	7.9 (7.7-8.2)	9.3 (7.9-11.3)	1.2 (1.0-1.4)	9.2 (6.3-12.9)	235 (186-331)	134 (95-186)	20 (6-45)
	M	8.2 (7.5-8.9)	65. (6.1-7.1)	1.5 (1.0-1.8)	15.7 (13.0-17.3)	358 (172-728)	233 (112-474)	649 (440-910)
	PM	7.7 (7.4-8.0)	7.9 (6.9-9.4)	1.2 (0.8-1.8)	7.8 (5.6-9.8)	134 (100-161)	85 (68-104)	30 (12-43)
Domestic waste water from Kalinga Vihar L-1 (BD-2)	W	8.0 (7.7-8.2)	4.3 (4.0-4.9)	18.9 (3.7-30)	66.0 (15.0-98.0)	394 (347-450)	246 (218-288)	40 (33-44)
	S	7.9 (7.4-8.5)	3.5 (2.6-5.3)	27.2 (11.5-40)	79.0 (31.0-112.0)	420 (412-426)	251 (236-268)	65 (34-88)
	M	7.7 (7.4-7.8)	3.6 (3.2-3.9)	16.7 (8.0-30)	58.0 (22.0-111.0)	311 (237-397)	180 (154-216)	117 (108-132)
	PM	7.9 (7.8-8.0)	2.1 (1.2-2.9)	18.3 (11.0-27)	40.0 (27.5-53.0)	349 (268-400)	227 (182-258)	34 (26-42)
Domestic waste water from sector 16 (BD-3)	W	8.2 (7.8-8.4)	4.0 (3.3-5.2)	32.0 (14.0-56.0)	105.0 (46.5-180.0)	294 (248-338)	176 (150-217)	26 (2-52)
	S	7.8 (7.3-8.4)	6.3 (4.7-8.1)	31.0 (11.0-52.0)	57.0 (11.0-123.0)	298 (269-326)	175 (156-186)	24 (20-27)
	M	7.7 (7.4-8.0)	5.5 (5.2-5.9)	20.0 (16.0-23.0)	60.0 (50.0-75.0)	368 (195-590)	222 (102-385)	51 (40-70)
	PM	8.0 (7.8-8.2)	5.7 (4.2-7.6)	25.0 (22.0-31.0)	74.0 (53.0-91.0)	286 (267-310)	181 (160-202)	47 (32-70)
Domestic waste water from Koel Nagar (BD-4)	W	7.8 (7.5-8.2)	1.5 (1.0-2.3)	46.0 (26.0-71.0)	125.0 (78.0-205.0)	337 (298-399)	202 (160-256)	189 (115-331)
	S	7.6 (6.9-8.4)	1.3 (0.5-2.1)	53.0 (28.0-75.0)	155.0 (72.0-233.0)	406 (395-422)	242 (221-261)	51 (42-62)
	M	7.4 (6.9-7.7)	2.6 (1.4-3.5)	21.0 (20.0-22.0)	56.0 (53.0-60.0)	276 (273-311)	172 (154-202)	66 (56-76)
	PM	7.6 (7.2-8.1)	1.3 (1.0-1.6)	44.0 (32.0-68.0)	131.0 (95.0-199.0)	330 (318-341)	186 (180-190)	38 (22-54)
Koel D/s (BR 4)	W	7.3 (6.7-8.0)	10.8 (8.5-12.4)	1.3 (0.9-1.8)	7.2 (5.0-8.2)	207 (173-242)	121 (98-132)	14 (5-30)
	S	8.2 (7.9-8.5)	8.3 (7.6-8.8)	1.8 (0.6-2.1)	11.3 (7.1-19)	171 (123-198)	101 (71-126)	18 (12-30)
	M	8.0 (7.6-8.6)	7.4 (7.0-8.1)	3.7 (2.0-6.0)	17.2 (10 – 29.9)	383 (266-594)	231 (140-367)	511 (320-754)
	PM	7.9 (7.8-8.2)	7.8 (7.3-8.0)	1.0 (0.9-1.2)	7.4 (6.0-8.2)	181 (167-196)	134 (106-186)	25 (12-36)

(Source - Ibid, Table-14, Pg.31)

5.2.4.3 (c) Brahmani River

Seasonal variation in quality of wastewater streams before confluence with river Brahmani and water quality of Brahmani at the upstream and downstream of confluence of these streams during 2007-09 are shown in Table 5.28.

Table 5.28 Quality of wastewater streams and Brahmani river in Rourkela stretch (2007-2009)

Monitoring Location	Season	Parameter						
		pH	DO (mg/l)	BOD (mg/l)	COD (mg/l)	EC (µs/cm)	TDS (mg/l)	TSS (mg/l)
Panposh U/s (BR 5)	W	7.9 (7.3-8.3)	9.0 (8.2-10.4)	0.8 (0.3-1.1)	6.6 (4.0-8.0)	176 (133-288)	88 (76-108)	11 (11-12)
	S	8.0 (7.8-8.3)	7.8 (7.4-8.4)	1.2 (1.0-1.6)	12.2 (8-19.0)	184 (133-249)	113 (66-188)	33 (13-70)
	M	7.9 (7.2-8.3)	7.1 (6.5-7.8)	0.8 (0.6-0.9)	5.9 (3.4-8.7)	104 (91-128)	62 (59-65)	405 (112-774)
	PM	7.6 (7.3-8.0)	8.4 (7.6-9.7)	0.9 (0.5-1.2)	7.9 (2.0-15.7)	122 (70-181)	76 (46-107)	54 (24-96)
Kalunga Industrial Drain II (BD-5)	W	7.9 (7.3-8.2)	8.4 (7.3-9.4)	5.8 (1.6-12.0)	15.7 (5.9-28.0)	363 (296-443)	228 (180-280)	34 (26-51)
	S	8.3 (8.3-8.4)	7.8 (6.2-10.0)	2.8 (1.2-4.6)	14.9 (8.5-26.7)	248 (157-306)	150 (88-198)	33 (15-64)
	M	8.1 (7.8-8.5)	6.7 (6.3-6.9)	7.5 (5.6-11.0)	19.4 (15.0-24.5)	202 (118-260)	125 (76-150)	69 (22-104)
	PM	8.1 (7.8-8.2)	6.5 (5.9-7.5)	2.7 (2.2-3.1)	9.5 (9.5-11.2)	205 (82-280)	127 (56-164)	34 (26-40)
Guradhi nallah (BD-6)	W	7.7 (6.9-8.2)	8.6 (8.2-8.9)	15.0 (5.5-26.0)	83.7 (23-196.0)	399 (344-437)	234 (190-280)	65 (25-139)
	S	7.1 (6.2-8.1)	7.0 (6.6-7.4)	15.3 (12.5-19.0)	15.0 (14.3-15.6)	402 (333-510)	242 (193-331)	72 (32-99)
	M	7.7 (6.9-8.1)	6.8 (6.5-7.2)	9.1 (5.8-12.8)	32.1 (14.5-53.7)	399 (320-446)	257 (201-297)	149 (20-372)
	PM	7.9 (7.6-8.1)	4.9 (1.9-6.7)	10 (6.4-14.8)	27.3 (13.1-45.9)	369 (331-444)	227 (172-288)	28 (20-35)
Panposh D/s at Deogaon (BR-6)	W	7.7 (7.4-8.2)	9.6 (7.1-11.5)	4.3 (3.3-4.9)	22.6 (18-26.0)	351 (259-431)	204 (168-234)	52 (29-90)
	S	7.9 (7.0-8.4)	6.6 (5.5-7.3)	4.4 (3.6-5.3)	22.4 (12.7-30.5)	347 (275-441)	187 (94-286)	86 (53-146)
	M	7.4 (7.0-7.80)	6.7 (6.4-7.1)	4.0 (3.2-4.7)	20.4 (4.5-32.1)	212 (164-273)	142 (107-198)	439 (260-696)
	PM	7.8 (7.5-8.0)	5.4 (4.5-7.0)	3.0 (2.2-4.2)	17.8 (15.8-19.7)	224 (146-315)	142 (96-205)	75 (30-152)
Wastewater from slag granulation unit of RSP (BD 7)	W	7.7 (7.2-8.0)	7.2 (6.7-7.7)	1.7 (1.0-2.5)	9.6 (6.8-12.0)	363 (248-539)	212 (159-310)	54 (41-67)
	S	8.2 (8.1-8.4)	5.6 (4.5-6.9)	1.8 (1.5-2.0)	12.6 (9.9-15.2)	354 (261-466)	209 (169-270)	58 (14-143)
	M	8.4 (8.2-8.9)	6.8 (6.0-7.6)	2.6 (2.0-3.2)	15.2 (9.3-18.2)	399 (314-480)	249 (207-278)	119 (98-144)
	PM	8.2 (8.0-8.4)	8.0 (7.6-8.4)	1.9 (1.8-2.0)	9.5 (9.3-9.7)	418 (344-459)	269 (228-298)	25 (24-26)

Table 5.28 (Contd.)

Monitoring Location	Season	Parameter						
		pH	DO (mg/l)	BOD (mg/l)	COD (mg/l)	EC (µs/cm)	TDS (mg/l)	TSS (mg/l)
Rourkela D/s at Jaldra (BR 7)	W	8.0 (7.5-8.3)	8.9 (7.1-10.4)	2.7 (1.0-3.8)	15.3 (6.0-22.0)	285 (224-399)	171 (133-216)	26 (20-30)
	S	8.0 (7.6-8.2)	6.8 (5.3-7.7)	3.5 (3.0-4.0)	19.8 (15.9-22.0)	273 (217-336)	165 (130-224)	50 (23-94)
	M	7.6 (6.7-8.5)	6.5 (6.0-6.9)	2.5 (1.0-3.4)	18.3 (3.9-30.3)	200 (136-276)	132 (94-179)	467 (270-834)
	PM	7.4 (6.9-7.8)	7.9 (7.2-9.1)	2.7 (2.4-3.0)	14.9 (13.9-16.0)	204 (177-224)	134 (114-164)	35 (29-46)
Rourkela FD/s at Attaghat (BR 8)	W	8.0 (7.5-8.3)	9.1 (7.6-12.0)	2.6 (1.8-3.2)	9.1 (4.6-12.8)	285 (228-395)	170 (135-210)	16 (12-20)
	S	8.0 (7.6-8.2)	7.9 (6.7-8.8)	1.9 (1.2-2.8)	12.9 (7.6-20.0)	266 (245-292)	154 (130-185)	83 (50-138)
	M	7.8 (7.1-8.2)	7.3 (6.8-7.6)	1.5 (0.8-2.1)	9.8 (3.4-20.9)	145 (77-181)	85 (49-123)	835 (396-1582)
	PM	7.6 (7.3-7.9)	7.8 (7.2-8.9)	1.7 (1.5-1.9)	9.4 (7.6-11.8)	133 (80-180)	83 (52-106)	60 (14-144)
Rourkela FD/s at Biritola (BR 9)	W	7.9 (7.5-8.3)	9.1 (8.2-9.6)	2.3 (1.8-2.8)	10.7 (4.0-16.0)	273 (199-392)	160 (113-198)	28 (22-32)
	S	8.0 (7.6-8.0)	7.5 (6.7-8.6)	2.7 (2.6-3.0)	15.9 (9.5-20.0)	216 (183-248)	127 (102-160)	55 (20-115)
	M	7.4 (6.8-7.8)	7.4 (6.8-7.8)	1.8 (0.8-2.6)	16.1 (5.6-27.5)	133 (75-176)	82 (53-109)	637 (246-1392)
	PM	7.7 (7.4-8.2)	8.7 (7.4-11.0)	1.7 (1.5-1.9)	9.7 (4.0-15.7)	147 (107-185)	92 (70-109)	50 (13-112)

(Source: Ibid, Table 15, Pg.33-34)

5.2.4.4 Talcher Stretch :

Seasonal variation in quality of Banguru nallah, wastewater from Lingaraj mines area, Talcher municipal drain and Nandirajhor before confluence with river Brahmani and water quality of river Brahmani at the downstream of confluence of these streams during the period (2007-09) are given in Table 5.29. Pollution load in terms of BOD in Talcher municipal drain was maximum during post-monsoon period (average 42.7 mg/l). Seasonal average of BOD in Nandirajhor remained fairly constant during the four seasons (2.4-2.7 mg/l). However, BOD in Brahmani river at the downstream of confluence of wastewater streams always remained within the tolerance limit i.e. 3.0 mg/l for Class-C surface water bodies. This indicates the availability of required flow in Brahmani river for dilution of the wastewater after confluence.

Table 5.29 Quality of wastewater streams and Brahmani river in Talcher stretch (2007-2009)

Monitoring Location	Season	Parameter						
		pH	DO (mg/l)	BOD (mg/l)	COD (mg/l)	EC (µs/cm)	TDS (mg/l)	TSS (mg/l)
Banguru nallah (BD-8)	W	7.6 (7.0-8.3)	9.0 (8.6-9.2)	1.8 (1.7-1.9)	10.0 (8.2-10.0)	481 (347-605)	291 (208-388)	33 (20-41)
	S	7.9 (7.5-8.2)	6.7 (6.1-7.2)	2.3 (1.2-3.8)	14.2 (12.7-16)	481 (420-542)	280 (250-329)	52 (36-68)
	M	7.9 (7.5-8.6)	7.1 (6.4-7.6)	11.0 (1.5-18.0)	39.6 (18.6-52.0)	301 (266-319)	187 (173-209)	101 (98-104)
	PM	8.1 (8.0-8.2)	6.2 (4.7-8.2)	4.0 (0.83-6.8)	21.0 (4.9-32.0)	426 (282-636)	299 (186-412)	37 (26-48)
Talcher FU/s (BR 10)	W	7.7 (7.2-8.2)	8.4 (7.8-8.8)	1.1 (0.6-1.6)	12.6 (3.9-20.0)	147 (123-190)	93 (82-112)	11 (8-14)
	S	8.0 (7.7-8.1)	7.0 (6.6-7.6)	1.2 (0.6-1.8)	7.5 (3.2-12.0)	149 (130-186)	92 (82-112)	71 (58-94)
	M	7.7 (7.2-8.3)	7.7 (7.4-7.9)	0.7 (0.7-0.8)	11.6 (10.9-12.8)	115 (90-134)	67 (58-78)	231 (110-370)
	PM	7.6 (7.3-8.0)	7.8 (7.5-8.3)	1.1 (1.0-1.1)	6.6 (5.8-7.9)	120 (75-164)	75 (50-97)	62 (10-109)
Wastewater from Lingaraj mines area (BD 9)	W	8.2 (7.3-9.1)	6.3 (2.8-8.5)	9.0 (3.0-20.0)	28.4 (9.4-60.0)	719 (468-931)	432 (250-597)	81 (74-88)
	S	8.0 (7.4-8.4)	6.4 (5.4-7.1)	8.2 (3.1-12.1)	23.4 (9.3-33.1)	434 (170-800)	258 (96-464)	48 (12-98)
	M	7.8 (7.2-8.5)	6.1 (6.0-6.3)	17.3 (16.0-20.0)	55.4 (44.9-76.0)	554 (418-785)	314 (260-412)	109 (98-120)
	PM	8.3 (8.2-8.4)	4.5 (2.8-6.8)	9.2 (6.5-14.0)	29.2 (15.0-46.6)	640 (498-802)	370 (328-422)	60 (25-86)
Talcher Municipal Drain (BD 10)	W	7.7 (7.3-8.2)	2.4 (1.9-2.9)	38 (32.0-42.0)	137.7 (112-168.7)	507 (238-788)	303 (140-505)	37 (32-410)
	S	8.0 (7.9-8.2)	1.8 (0.1-3.4)	40 (38.0-42.0)	112.6 (38.1-177.8)	455 (182-617)	274 (102-364)	137 (84-180)
	M	7.6 (6.8-8.9)	2.4 (1.2-3.0)	29 (25.0-32.0)	125.6 (73.3-163.6)	785 (356-1100)	492 (230-756)	145 (124-180)
	PM	6.8 (6.6-6.9)	1.4 (0.8-1.8)	42.7 (38.0-50.0)	145-1 (135.3-160.0)	656 (620-700)	417 (380-450)	81 (68-98)
Talcher U/s (BR 11)	W	8.0 (7.9-8.2)	8.5 (7.8-9.0)	1.2 (0.6-1.8)	12.5 (5.9-22.0)	156 (114-199)	101 (76-124)	11 (8-13)
	S	8.1 (7.9-8.5)	7.3 (7.1-7.4)	1.4 (0.7-1.9)	11.1 (6.1-18.0)	150 (134-163)	67 (20-96)	55 (33-86)
	M	7.8 (7.4-8.2)	7.8 (7.5-8.2)	1.0 (0.9-1.1)	14.2 (10.9-20.4)	123 (96-136)	70 (63-83)	175 (110-277)
	PM	7.7 (7.5-8.0)	7.6 (7.5-7.8)	1.2 (1.1-1.5)	9.1 (6.5-10.9)	115 (79-165)	76 (52-98)	51 (21-78)

Contd..

Table 5.29 Contd.

Monitoring Location	Season	Parameter						
		pH	DO (mg/l)	BOD (mg/l)	COD (mg/l)	EC (µs/cm)	TDS (mg/l)	TSS (mg/l)
Nandira before confluence with Brahmani (BD 11)	W	8.1 (7.6-8.5)	6.9 (6.1-7.4)	2.4 (1.5-3.0)	23.9 (18.0-27.8)	488 (284-695)	242 (160-330)	16 (14-18)
	S	8.0 (7.6-8.4)	6.7 (6.4-7.0)	2.7 (1.7-3.5)	14.7 (9.5-18.0)	465 (442-492)	244 (238-254)	106 (28-244)
	M	8.3 (7.7-8.8)	7.5 (6.9-7.8)	2.5 (1.6-3.5)	22.7 (19.6-26.8)	365 (260-453)	230 (170-291)	54 (20-89)
	PM	7.9 (7.1-8.4)	7.4 (6.9-8.2)	2.4 (2.1-2.9)	13.9 (9.9-18.8)	418 (352-453)	253 (228-300)	34 (11-74)
Talcher D/s (BR 12)	W	8.1 (7.8-8.4)	9.0 (7.8-10.2)	1.4 (1.0-1.6)	12.6 (9.8-16.0)	205 (168-225)	132 (112-164)	25 (10-50)
	S	7.6 (7.0-8.1)	7.7 (6.7-8.4)	2.1 (0.9-2.8)	14.9 (6.3-20.0)	187 (136-246)	121 (80-162)	60 (38-95)
	M	7.8 (7.1-8.2)	7.3 (6.8-7.7)	1.5 (1.2-1.9)	15.9 (14.4-18.7)	150 (125-192)	90 (75-112)	211 (116-336)
	PM	7.6 (7.0-8.0)	7.5 (7.1-7.8)	1.8 (1.6-2.0)	11.3 (8.2-12.9)	149 (97-200)	93 (64-118)	84 (39-134)
Talcher FD/s (BR 13)	W	7.8 (7.6-8.2)	8.9 (7.9-9.5)	0.8 (0.6-1.1)	5.3 (3.9-8.0)	176 (122-214)	118 (84-142)	11 (4-18)
	S	8.0 (7.6-8.3)	7.7 (7.2-8.5)	1.1 (0.2-2.0)	9.5 (3.2-18.0)	145 (132-165)	92 (88-98)	56 (22-87)
	M	7.8 (7.5-8.1)	7.8 (7.3-6.4)	1.2 (0.9-1.4)	14.2 (11.2-18.2)	157 (136-194)	105 (98-119)	216 (178-285)
	PM	7.5 (7.4-7.8)	7.8 (7.5-8.4)	1.0 (0.6-1.4)	11.4 (8.9-16.3)	158 (109-227)	95 (70-128)	78 (60-96)

(Source: Ibid, Table 16, Pg 36-37)

5.2.4.5 Conclusion

The study reveals that rivers at the immediate downstream of the major urban settlements of Rourkela in Brahmani basin, do not meet criteria limit for class C river and the major drains/streams carry wastewater which is much above the prescribed limit of BOD i.e. 30 mg/l. Further it is also observed that untreated domestic wastewater is the major source for downgrading the river water quality.

The situation further worsens during summer seasons. As flow in the river during summer season is comparatively less, the wastewater after confluence with river does not get adequately diluted, which in turn, increases the BOD load in rivers.

In view of the present condition and future scenario of urbanization, sewage treatment systems should be in place in each and every town and city. However, prioritization may be done for construction of STPs in phases keeping in view the pollution potential of the urban local bodies. (ULBs).

5.2.5 Minimum Flow Requirement :

Minimum flow is required to be maintained in a river for the following main reasons.

- to meet downstream riparian rights
- to protect fish and aquatic lives
- for dilution of effluents in the river water
- control of temperature in the river.

(v) to control salinity ingress.

(vi) to maintain sustainable ecology.

Due to want of adequate data, quantitative evaluation of minimum flow requirement downstream of Brahmani at Samal Barrage could not been done and in the simulation model a minimum flow of 20 cumec is provided to meet the downstream requirements in the year 2051. The monthly present minimum flow downstream of Jenapur is given below.

Table 5.30 Observed minimum monthly Flow in river Brahmani during Non-monsoon at Jenapur

(Unit-Cumec)								
YEAR/ MONTH	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN
1979-80			11.9	8.1	8.9	6.4	3.9	15
1980-81	46.7	22.2	27.4	28.3	16.2	12.2	20.6	41.6
1981-82	65.2	34.3	20.6	23	27.1	14	11.4	18.5
1982-83	44.8	23	14	12.2	11.7	0.2	12	20
1983-84	76.4	46.8	42.5	29.4	11.5	9.8	5.6	8.5
1984-85	234.6	47.8	48.7	32.3	16.4	13	10	7
1985-86	60	142.4	27.3	88	84.4	108	122	91
1986-87	320	305	134.4	130	105.6	113.8	198	165.6
1987-88	159.3	132.2	170	281.6	217.8	95.05	67.17	
1988-89	NA	NA	NA	NA	NA	NA	NA	85.53
1989-90	153.7	238	217.8	81.5	79.49	78.93	83	142.2
1990-91	314.9	332	174.3	93.19	68.72	52.01	41	120.7
1991-92	220	224.6	174.7	126.7	93	48.99	42.49	47.33
1992-93	150.7	143.7	89	32	14	13	23	29.2
1993-94	229	290.2	152	175	119.6	28	20.4	31
1994-95	221	105.3	80.13	72.8	115	73.4	74.6	43.52
1995-96	129.1	119.1	82.06	84	160.6	144.3	127	103.5
1996-97	99.86	107.1	106.9	57.68	61	149.6	136.5	135
1997-98	159	106.8	75.6	82.93	127.2	82.54	125.5	129
1998-99	357.9	154.6	120.6	107.3	85.8	96	140.9	24
1999-2000	180	135.3	123.9	118.5	128.6	134.5	128.5	89.5
2000-2001	97.76	-	-	-	-	-	-	-
Minimum of Min^m Monthly flow	44.8	22.2	11.9	8.1	8.9	0.2	3.9	7.0

(Source: Brahmani Basin Plan, 3rd Spiral Study OWPO, Govt. of Odisha, Nov. 2002 Pg.74)

Table 5.31 Recorded Average Release Discharge to the River from Samal Barrage

(Unit: Cumec)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1997	178	128	153	134	153	229	930	1,790	1,528	503	271	194
1998	159	474	343	274	309	290	499	420	1,061	453	451	165
1999	194	168	138	146	186	218	587	1,701	2,259	1,252	332	190
2000	140	144	158	194	272	249	829	775	726	382	126	115
2001	100	88	81	85	99	279	3,985	2,029	1,454	355	268	197
2002	178	204	341	254	148	261	372	581	993	273	177	135
2003	73	118	208	288	307	437	621	907	1,512	1,674	533	369
2004	227	159	128	123	226	162	293	1,171	752	353	252	162
2005	242	186	269	183	205	208	1,602	863	499	392	183	206
2006	193	146	110	214	173	283	338	2,336	963	411	201	129
2007	116	90	135	245	118	379	1,037	1,842	1,959	1,036	404	219
2008	278	138	47	100	36	663	1,707	1,369	1,196	535	210	199
2009	31	205	58	183	151	73	667	726	602	260	92	100
2010	63	55	127	160	147	108	130	140	101	27	-	-
Average	55	164	164	184	181	274	971	1,189	1,115	565	269	183
Maxm.	278	474	343	288	309	663	3,985	2,336	2,259	1,674	533	369
Minm.	31	55	47	85	36	73	130	140	101	27	92	100

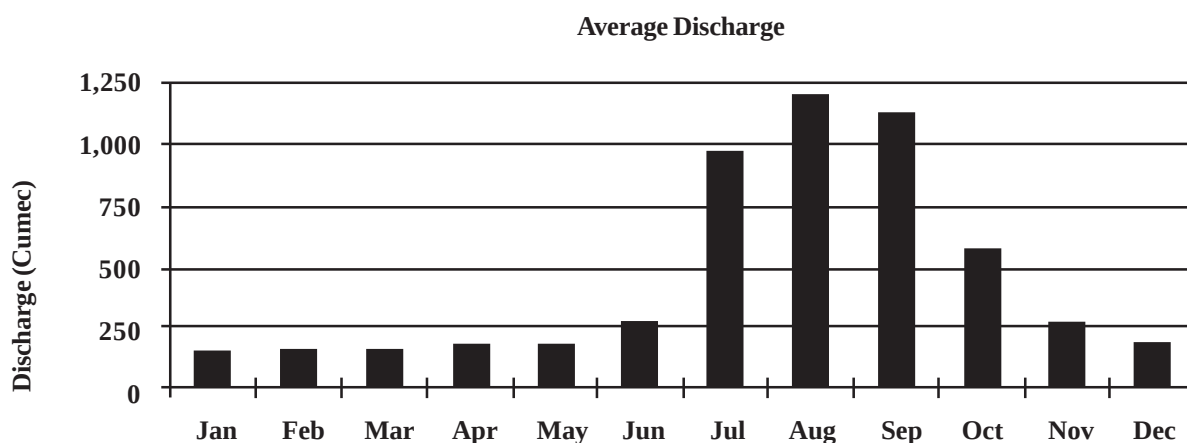


Fig.5.3

As per National Water Commission's survey (NWC report, 2000), the stretch of 310 km down below Rengali dam is most polluted due to, the effluents from aluminum, steel and fertilizer industries and due to the mining and urban activities. The pollutants are discharged to the River Brahmani through its tributaries; Tikira, at 30 km, Nandira at 75 km and Bangura at 95 km down below the Rengali Dam. In this study, the river stretch (57 km) from Rengali dam to Talcher has been chosen. Talcher is affected seriously by the wastewater discharged by river Tikira, main tributary of river Brahmani.

As per CPCB study, this stretch is D class, i.e., it is designated to use for propagation of wild life and fisheries, which has the following criteria: 1) pH 6.5 to 8.5, 2) Dissolved Oxygen 4 mg/l or more, 3) Biochemical Oxygen Demand 3 mg/l or less. Due to increasing quantum of pollutants, water quantity of this stretch deteriorates. Hence it is imperative to predict the fate of pollutant and to regulate the pollutant disposal to maintain the stretch as C class, i.e., it is designated to use for drinking source after conventional treatment and disinfection.

“Water quality data at different locations in river Brahmani and its tributary, Tikira, are collected, which include pH, Temperature, BOD, DO, Coliform. In the study reach a point load of pollution is discharged by river Tikira into the river Brahmani at 30 km downstream from Rengali dam. According to CPCB survey, 2980000 cum / day of wastewater due to industrial, domestic and mining activities are discharged. The rate of discharge of wastewater and chemical constituents for the months from November to May are presented in Table 5.32. It can be noted that the BOD load through Tikira is nearly steady and COD slightly increasing with time.”

Table 5.32 Water Quality data of river Tikira

Month	Q (Cumec)	BOD (mg/l)	COD (mg/l)	DO (mg/l)
November	18.1	216.8	217	3.4
December	15.4	218.9	221	3.3
January	13.7	221.6	219.6	2.9
February	13.5	223.5	226.4	1.5
March	13.2	225.6	233	1.5
April	12.5	228.6	236.8	1.2
May	13.3	229.5	242.3	0.9

(Source: Minimum flow requirements of river system for Ecology, Department of Water Resources, Development and Management, IIT, Roorkee, Final report- Dec. 2007, Pg.25)

Water quality data of collected water samples from various locations were tested in laboratory of NIH, Roorkee and have been tabulated in Table 5.33.

Table 5.33 Water quality of river Brahmani during the month of July, 2013

Locations	pH	EC mS/cm	TDS mg/l	SS mg/l	Na mg/l	Ca mg/l	SO ₄ mg/l	BOD mg/l	COD mg/l	TC mg/100ml	FC mg/100ml
Rengali	7.3	158	101	28	6	17	10	16	25		
C-1	6.7	116	74	743	3.1	14	6	12	19		
D-1	7.0	474	303	16	21	48	97	8.0	13		
A-1	6.7	261	167	139	12	24	30	8.0	13		
Bijigol	7.3	157	100	196	4.2	14	12	8.0	13	75	15
Nalco AP(U/S)	7.3	449	287	52	21	46	36	15	25		
Nalco AP(D/S)	6.8	516	330	4	24	56	31	23	38	43	15
BN-1	7.1	551	353	38	26	57	125	4.0	6	23	9
BN-2	7.2	336	215	57	14	35	60	3.8	6	460	15
BN-3	7.5	157	100	78	5.7	16	16	21	32	150	43
NB-1	7.1	435	278	63	21	40	52	3.8	6		
NB-2	7.6	163	104	114	6.5	13	11	3.8	6	240	43
BBT-1	7.2	144	92	114	4.4	14	12	8.6	13		
SB-1	7.6	151	97	48	4.3	13	13	15	25		
KB-1	7.6	148	95	76	4.4	14	12	8.0	13		

The wastewater with the above chemical constituents is discharged into river Brahmani at 30 km down from the Rengali dam. Using the dilution equation the BOD input to river Brahmani at confluence of Tikira is 24.03 mg/l and COD input is 36.45 mg/l.

(Source: Ibid, Table-10 Pg.26)

Conclusion:

- 1) An analytical study has been carried out to quantify concentrations of BOD, COD and dissolved oxygen level in the river reaches downstream of Tikira confluence up to Talcher. Tikira carries most of the industrial effluents and joins river Brahmani that also carries some chemical pollutants discharged between Rengali dam and Tikira confluence.
- 2) A hybrid model is used to quantify DO concentration, which is governed by BOD, COD and re-aeration. The model incorporates advection, dispersion and first order decay of pollutants and first order re-aeration rate. This model can be used for pollution studies in other streams also.
- 3) From the pollutant transport study it is found that during monsoon period (1127.84 cumec) with the present pollutant load discharged into the river the BOD at Talcher is 2.75 mg/l. It satisfies the requirement for C class. The chemical loads at Talcher are 4.3 mg/l, which is to be checked for individual elements to find its suitability for drinking water supply and level of treatment. Thus, a conventional pretreatment is required for drinking purpose.
- 4) Corresponding to lowest flow ($Q=93.65$ cumec) during non-monsoon period, and for the present pollutant load, the BOD level at Talcher is 27.5 mg/l and therefore the water quality of the river at Talcher does not satisfy C class condition.
- 5) Even for normal low flow ($Q=239.17$ cumec) during non-monsoon period, the BOD & COD concentrations at Talcher for the present pollutant load are 17.09 mg/l and 26.5 mg/l respectively and they exceed the limiting values.
- 6) Corresponding to BOD and COD load disposed at Tikira confluence, the minimum DO level at Talcher for normal flow during non-monsoon period is about 6.55 mg/l. The DO level is higher than the amount required. But, the BOD and COD at Talcher are very high as stated above. In order to maintain the river stretch as C class, 75% pretreatment of wastes is necessary.
- 7) As indicated by the oxygen sag curve downstream of Tikira confluence, during the lowest non-monsoon flow period (93.65 cumec) the oxygen level over a stretch of 10 km is less than 4 mg/l, the minimum being 2.3 mg/l. Such a low oxygen level is detrimental to the aquatic life. Therefore, for protection of aquatic life, 50% of the pollutant must be treated. However, there is no danger to the aquatic life during high flow (1127.98 cumec) period with the present pollutant load.
- 8) If 240 cumec is released from the reservoir during non-monsoon period, there will not be any danger to the aquatic life with the present pollutant load. To take into account of non-availability of this required flow, 50% pretreatment is required to maintain the ecosystem. Downstream of Talcher, industrial effluents are dumped into Brahmani through Nandira and Bangura. Transport of these pollutant loads need to be ascertained and overall status of water quality beyond Talcher town needs to be investigated. (Source: Ibid, Pg. 41-43)

5.2.6 Impact of flow of river Brahmani on Mangroves of Bhitarkanika:

5.2.6.1 Mangroves of Bhitarkanika:

The Bhitarkanika National Park and the Bhitarkanika Wild Life Sanctuary are famous for its mangrove ecosystems. These mangroves are situated at the terminal estuary of Brahmani river near the Bay of Bengal. The mangroves of Brahmani-Baitarani delta fall under the administrative control of the Odisha State Forest Department. Of these, a portion covering 215 sq.km has been listed as a RAMSAR SITE.

Table 5.34 Summary of Observations

Monitoring station	Use based		Biological		Degree of wholesomeness	
	Class	Parameter(s) responsible for downgrading the water quality	Status of pollution	Parameter(s) responsible for downgrading the water quality	Level	Parameter(s) responsible for downgrading the water quality
Sankh (Sankh)	C/D/E	TC	-	-	Below acceptable	TKN
Koel (Koel)	C/D/E	TC	-	-	Below acceptable	TKN
Panposh U/s	C/D/E	TC	Slight to moderate pollution	Sl, DI and BOD	Below acceptable	TKN
Panposh D/s	D/E	BOD, TC	Slight to moderate	Sl, DI and BOD	Below acceptable	FC, TKN
Rourkela D/s	DE	BOD, TC	-	-	Below acceptable	FC, TKN
Rourkela FD/s (Attaghat)	C/D/E	TC	-	-	Below acceptable	FC, TKN
Rourkela FD/s (Biritola)	C/D/E	TC	-	-	Below acceptable	FC, TKN
Bonaigarh	C/D/E	TC	-	-	Below acceptable	FC, TKN
Rengali	C/D/E	TC	-	-	Below acceptable	TKN
Samal	C/D/E	TC	-	-	Below acceptable	TKN
Talcher U/s	C/D/E	TC	Slight to moderate pollution	Sl, DI and BOD	Below acceptable	FC
Talcher D/s	D/E	TC	Slight to moderate pollution	Sl, Di and BOD	Below acceptable	FC, TKN
Nandira D/s (Nandira Jhor)	C/D/E	TC	-	-	Below acceptable	FC, TKN
Kisinda jhor	C/D/E	TC	-	-	Below acceptable	TKN
Dhenkanal U/s	C/D/E	TC	-	-	Below acceptable	TKN
Dhenkanal D/s	C/D/E	TC	-	-	Below acceptable	FC, TKN
Bhuban	C/D/E	TC	-	-	Below acceptable	FC, TNK
Kabatabandha	C/D/E	TC	-	-	Below acceptable	FC, TKN
Dharmasala	C/D/E	TC	-	-	Acceptable	FC, TKN
Pottamundai	C/D/E	TC	-	-	Below acceptable	FC, TKN
Khanditara (Kharasrota)	C/D/E	TC	-	-	Below acceptable	TKN
Binjharpur (Kharasrota)	C/D/E	TC	-	-	Below acceptable	FC, TKN
Aul (Kharasrota)	C/D/E	TC	-	-	Below acceptable	FC, TKN

A limited data available on water flows and salinity of water at the estuary is available. It appears that the estuarine salinity, which is slightly below the salinity in the seas, is not significantly affected by the head discharges in the river.

“Mangroves are salt tolerant, complex and dynamic ecosystem characteristic to tropical and sub-tropical coastal and inter-tidal regions. Brahmani river in eastern India with a basin area of 39,268 sq.km terminates at Bhitarkanika estuary covering 672 sq.km butting the Bay of Bengal. Rich mangroves belonging to 62 species occur on the banks of major tidal creeks over an area of 215 sq.km in the estuarine region. Historically the estuarine region used to experience 3 or 4 large flood flows in the range of 8000 to 25,000 cumec annually with abundant silt flow, that have sustained the luxuriant mangroves in a mildly saline environment with salinity of 5 to 20 ppt. The deltaic flood plain of 3000 sq.km area contributed ground water substantially to sustain river flow in the non-monsoon causing Bhitarkanika wetland to maintain a rheotrophic character. Construction of a major multipurpose dam at Rengali on Brahmani, intercepting 25,250 sq.km basin area was completed in 1985 in Odisha for annual irrigation of 3,36,400 ha (yet to be developed), power generation and flood control to the delta. By moderation at the dam, severe flood events have reduced with peak flow limited to 8000 cumec with much lower sediment concentration occurring only once or twice annually. The inundation of the over-bank deltaic plain has also reduced causing diminishing groundwater recharge. Currently the annual flow at the head of delta and silt load have diminished to 17,380 Mcum and 6.1 million Mg compared to the pre-dam figures of 19,514 Mcum and 15.5 million Mg respectively. With abstraction of almost 20% of the annual flow for irrigation by 2015, when the irrigation system becomes fully operational, reduction of flood plain inundation causing decreased ground water recharge and significant reduction of silt flow to the estuary, the mangroves can go under stress and be threatened. There is likelihood of an increase in salinity regime consequent to large-scale fresh water abstraction and altered flood pattern.” (Source: International Journal of Ecology and Environmental Sciences, 33(4): 243-253, 2007- Impact of Water Resources Development on Bhitarkanika Mangroves in Brahmani Estuary by B.P. Das & S.S. Mishra)

5.2.6.2 Sustainability of Mangrove Ecosystem:

“Mangrove eco-system depends on a balanced interplay of sweet and saltwater at the estuarine reaches of river deltas. Due to progressive diversion of fresh water in the rivers upstream, it is apprehended that the flow of fresh water into the mangrove swamps can diminish rapidly in the years to come. This can affect the salinity levels of the water in the mangrove eco-system resulting in their destruction.

The sustainability of mangroves is crucially dependant on a delicate mix of abundant fresh water (riverine flow) and saline water, which the tides provide. Presence of salinity at adequate level is the most desirable condition, as the absence of salinity will not enable the mangroves to survive. The optimal salinity however is 5-15 ppt for luxuriant growth and sustenance of mangroves.

Low salinity is preferred to high salinity as was experienced in Indus delta where the mangroves in the estuarine region have been dying primarily due to massive fresh water abstraction. Bhakra Dam in India and Tarbela and Mangla Dams and large barrages in Pakistan have resulted in reduction of terminal flow from 140 MAF (1950) to 20 MAF currently. Thereby the salinity has gone up from 20ppt (around 1950) to 40-45ppt currently resulting in the reduction of delta mangroves extent from 3450 sq.km to 1585 sq.km (1990). An IUCN study emphasizes that 27 to 35 MAF of terminal flow must be ensured to sustain the whole eco-system of Indus delta (Menon, 2005).

A reversing trend is noticed in the Sunderban mangroves, covering 2500km², the largest chunk in West Bengal, India. The mangroves were slowly perishing due to dearth of fresh water until the Farakka feeder canal was built (1970) bringing in 850 cumec of fresh water perennially into Hoogly estuary reducing the salinity by 10-15 ppt. Not only the mangroves flourished due to decreased salinity but also the fishery production went up from 7,000 Ton (1970) to almost 70,000 Ton currently attributable to the healthy mangroves in the estuary.” x x x x x

“Prior to construction of Rengali dam, the lean season flow at the delta head was on the average 1,400 million cubic meters (1,000 million cubic meters in dry years to a maximum 2,000 million cubic meters in very wet years) out of the overall annual flow of 22,000 million cubic meters. With very little irrigation in the Rabi season, no return flow was available to the river. With a large terrestrial forest cover of 40% the ET need was getting drawn from the partly saturated root zone region. The monthly flow was progressively reducing from 400 million cubic meters in November to about 75 million cubic meters in May.

In contrast since 1986 the total lean season flow has been in the range 2,000 to 3,000 million cubic meters, the minimum in the month of May being 300 million cubic meters. This is the consequence of the perennial power release from Rengali hydro station. Further, as the BHIWA model demonstrates the additional withdrawal of 5,000 million cubic meters for the increased irrigation coverage of 6,70,000 ha in 2005 will return at least 1,000 million cubic meters between December to May. It is certain that the overall lean season flow will not go below 2,000 million cubic meters in the future which is 15% of the overall annual flow in the future scenario. In addition the densely forested Baitarani basin, which has a fresh water contribution of almost 1,000 million cubic meters in the lean season, has a direct contribution to freshwater for the estuary. From the water quality data it is obvious that current salinity is not increasing beyond 25 ppt in the summer, which shows that the saline wedge is not significantly affecting the fresh water upto 20 km upstream.

The monthly river flows at the basin outlet, indicate that for any of the future scenarios, the average annual flow would be of the order of 15,000 million cubic meters or more. The lean season flows from December to May would also be of the order of 3,500 million cubic meters. Thus, there is no possibility of mangroves of Bhitarkanika facing dearth of fresh water, looking at the availability of additional water from Baitarani basin.

However a quantitative assessment of the need both for mangroves and fish, migratory in nature has to be carried out by adopting DRIFT (Downstream Response to Imposed Flow Transformation) or other suitable methodology and tested on a monthly basis against the available flow estimated on full irrigation and industrial development.” (Source: Country Policy Support Programme, ICID-Activity No.WW138714/DDEOO 14311, New Delhi, Aug, 2005)

“According to approximate estimation, the Rengali dam needs to release atleast 500 million cum of fresh water exclusively for sustaining mangrove forests even in non-monsoon during worst drought years” (vide The Hindu, April 30, 2008)

5.2.6.3 Evaluation Water quality:

Water quality study of Bhitarkanika mangrove system was carried out at Jawaharlal Nehru University, New Delhi. The findings of the study is reproduced below:

“The Bhitarkanika estuary system is tide dominated. The environmental parameters showed wide variations as per the topography and geomorphic setting of the ecosystem. The three regions viz. estuarine, mangrove and bay are well marked in the system. Estuarine region is the regions with entry point of Brahmani Baitarani Rivers to mangrove area and is area of intermixing of freshwater with the tidal area. Mangrove area represent, the area dominated by dense mangrove vegetation. The Bay region represents the area close to Bay of Bengal i.e. the entry point of tidal water.

A large variability in all parameters was observed within all the three regions. The pH of water was highest as 8.0 in Bay regions which represent the pH of typical seawater, while the pH of estuarine area is next in line, as represent the mixing of fresh water with sea water. The physicochemical parameters of water such as pH, EC, TDS differ significantly among three regions. The EC showed a large range of 3200 - 33750 $\mu\text{S cm}$ along the gradient from estuarine area to bay region, which is primarily due to mixing of seawater with river water.

The estuarine and mangrove region represent a similar average chloride (Cl) concentration (5870 mg/l), while it was gradually increased to 12200 mg/l for the bay region. Bicarbonate (HCO_3) concentration showed a decreasing tendency along the estuarine to bay region. The bicarbonate and calcium (Ca) values are indicative of intense chemical weathering in the Indian subcontinent. The high sulfate (SO_4), chloride, and sodium (Na) values are largely due to the proximity of the sea.

The present data also reveals that Potassium (K) was lower in concentration than Na. This may be due to preferential absorption and incorporation into silicate minerals. Ca and K concentration is highest in estuarine region, due to the more influx of riverine source. Baitarani and Brahmani river catchments are characterized by Precambrian granites, gneisses, and schists of the Eastern Ghats. There is local basic intrusive and volcanic lithologies; limestone, sandstones, and shale of the Gondwanas; and recent deltaic alluvium deposits at the river mouths on the Bay of Bengal. However, no distinct variation was observed for K, Ca and SO_4 among the three regions.

Generally, estuarine mangrove waters have relatively low content of dissolved inorganic phosphorus and nitrogen. In some cases, the degree of human impact seems to control nutrient profiles, while in others the degree of upland influence and the hydrology of the ecosystem appear to be of greater importance. The entire mangrove system of the study area had shown significant variation in nitrate and phosphate. Higher nitrate and phosphate concentration in estuarine water is mainly due to intense agricultural activities. The agricultural activity involves extensive usage of urea and diammonium phosphate fertilizers. In mangrove region, the rich microbial community utilizes nitrate and phosphate for their metabolism. In bay region, nitrate and phosphate content is higher. This is due to intense Dhamra port activity as well as agricultural runoff from nearby villages. The Gibbs diagram of the source of total dissolved material in the mangrove water gives the impression of salinity dominance. This is essentially due to tidal influence from nearby Bay of Bengal. The present water quality of Bhitarkanika mangrove ecosystem reveals that salinity plays a dominant role in controlling the water chemistry. In addition, intense pollution from both agricultural inputs and industrial pollution deteriorate the water quality of mangrove ecosystem.” (Source: Indian Journal of Marine Sciences, Vol. 37(2) - Evaluation of water quality of Bhitarkanika mangrove system, Odisha, east coast of Indian by Rita Chauhan and AL Ramanathan, June 2008, Pg. 153-158)

CHAPTER-VI

POST CONSTRUCTION SCENARIO

6.1 Sedimentation of Rengali Reservoir :

6.1.1 Introduction :

“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 downstream 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 occurs along the floor of the reservoir. At higher reservoir levels finer particles 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).

6.1.1.1 Process of Sedimentation :

“The process of sediment deposition in the reservoirs takes place where the coarse particles with highest unit mass are deposited first and finest particles with low unit mass are deposited at last. This leads to the segregation of sediments of different sizes and density. The grain size, its unit mass and shape influence the mobility and the settling capacity. The variation in the sizes of the deposited materials in the reservoirs also depends on the climate, drainage area and geology’s of the area/region. The density of water in the reservoir is heterogeneous. The coarser particles mainly, gravel, boulder and partly sand move as bed load and form delta at relatively short distance from the estuary. This delta extends to the point where the maximum water level intercepts the original riverbed. The seasonal changes in the runoff cause the formation of multi-delta. The bottom sediments, which consist of finer particles, are deposited throughout the reservoir. The finest particles (silt, clay, colloids) often flocculate and are affected by dynamic viscosity of water. These particles sink to the bottom very slowly. They have a tendency to form density currents, which are generally moving towards spillways, turbines and outlets. In many reservoirs, a sediment-laden inflow may move through the pool as a density current or layer of water with a density slightly different from that of the main body of the reservoir water. The density difference may result due to the sediments, dissolved minerals or temperature. The water of the density current does not mix readily with reservoir water and maintains its identity for a considerable time because of density difference. xxxxx It is observed that the rate of sedimentation and reduction in the storage is higher in the initial stage/period of operation. This process of deposition of the sediments

gradually decreases the storage capacity of different zones of the reservoir. In case of deep reservoirs, the temperature also plays an important role in the circulation of water in the reservoir. The motion of the suspended matter, their floating and deposition depend on the water circulation due to temperature variation in the different water temperature zones (thermal stratification).”

(Source : Sedimentation Analysis of Rengali reservoir through Satellite Remote Sensing, remote Sensing Directorate, CWC, Sept.2004 Pg.4-5).

The principal sources of sediment are as follows :

(i) Deforestation, (ii) Excessive erosion in the catchment, (iii) Disposal of industrial and public wastes, (iv) Farming, (v) Channelisation works, (vi) Human use of environment, (vii) Land development, highways, and mining.

6.1.2 Sedimentation Survey :

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.

The problem of sedimentation and its effect on the useful life of reservoirs is very complex. The phenomenon of sedimentation and the concept of life of a reservoir and its estimation have been studied in depth over several years. Reservoirs belong to a system which shows gradual degradation of performance without any sudden non-functional stage. Sedimentation and consequent reduction of capacity is a gradual process, which can be classified into 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 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.

“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 are 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 dominant 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 more than its annual runoff, i.e., its capacity-inflow ratio being more than 1 (one), 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's classification that whose ratio is less than 1 (one) as seasonal reservoir and, those exceeding, as hold over storage reservoir.”

(Source : Compendium on silting of Reservoirs in India, CWC, Jan. 2001, Pg.9)

Table 6.1 : Details of Siltation in few 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	Maxm. 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 L1-3065'H-306'	5,148	5.138	L 7.28 D 1.32 G 8.60	180	-	-
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 Ghatapraha (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' b) Maithon (D.V.C.) L1-15712'H-162'	4,234 2,430	- -	L 0.185+0.881 D 0.148 G 1.214 L 0.496 D 0.608 G 1.104	23 28	18 -	260 325
10.	Nagarjuna Sagar (A.P) L1-15526'H-489'	83,087	34.34	5.44	73.66	-	-

Contd....

Table 6.1

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	Maxm. 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) Lingannakki 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 lb.	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).							

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 Level (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.	Srisailem (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 (Gujarat)	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.

Contd...

Annexure 6.1

1	2	3	4	5	6	7	8	9	10	1
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 spillway 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.

Contd...

Annexure 6.1

1	2	3	4	5	6	7	8	9	10	1
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	Henavathi (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 sq 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.

Contd...

Annexure 6.1

1	2	3	4	5	6	7	8	9	10	1
	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.	Gima (Maharashtra)	Gima	53.3	40.5	964.0	678.0	549600	28.4	652.1	Dead storage designed for a silt 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 ha m/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

Contd...

Annexure 6.1

1	2	3	4	5	6	7	8	9	10	1
33.	Jalput (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.

Contd...

Annexure 6.1

1	2	3	4	5	6	7	8	9	10	1
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.	Bajaj Sagar (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	1
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)

Before analysing the sedimentation of Rengali reservoir, one should know the siltation of few Indian reservoirs which is furnished in Table 6.1.

Statement showing the methods followed in calculating the life of some Indian reservoirs having more than 0.062 Mham. of gross storage capacity is given in Annexure 6.1.

6.1.3 Sedimentation Assessment of Rengali Reservoir :

After the original survey, first review of area-capacity curve was conducted through satellite Remote Sensing by Remote Sensing Directorate of CWC during 2000-2002 as per the recommendation of the Working Group for 'National Action Plan for Reservoir Sedimentation assessment using Satellite Remote Sensing'. Index map of Rengali reservoir is shown vide Drg.No.6.1.

6.1.3.1 By Remote Sensing :

“Remote sensing technique makes use of water-spread of the reservoir between maximum and minimum operating level during the observation period. Since the reservoir levels generally do not go below M.D.D.L., water spread observations are not possible below MDDL. The same are to be extrapolated from observed elevation-area curve to find out capacity below MDDL. In case of Rengali reservoir, the height between FRL (123.50m) and MDDL (109.72m) is 13.78m while the difference between MDDL and riverbed level (101m) is 8.72m. Extrapolation for such a large elevation range below MDDL is likely to add considerable subjectivity to the results. Thus the use of satellite remote sensing in the present study has been restricted to the live storage.”

(Source : Sedimentation analysis of Rengali Reservoir through satellite remote sensing by CWC, Sept.2004, Pg.10).

Owing to non-availability of cloud free satellite data at all the desired level in between MDDL and FRL in a single water year, the satellite data of previous/next water year is used. Further, this technique gives better estimate for fan shaped reservoir where there is considerable change in water-spread area with change in water level.

6.1.3.1.1 Result :

Water spread area and Live Storage capacity of reservoir at different elevations are given in Table 6.2 & 6.3.

Drg. No.6.2 shows FCC's (False Colour Composite) of different dates and Drg No.6.3 shows the superimposed reservoir water spreads for different dates. Water spread area has been calculated by multiplying number of pixels with area of each pixel. Table 6.2 shows the water-spread area for different dates corresponding to the levels.

Table - 6.2 Water Spread Areas estimated from Satellite Images

Date of pass	Elevation (m)	Area (M.sq.m)
15th Sept, 2001	123.30	359.6981
25th Dec, 2001	121.14	314.6853
13th Feb, 2002	119.51	286.8884
15th Dec, 2000	116.27	227.0118
11th Feb, 2001	114.45	185.8365
24th Apr, 2001	112.02	154.5919

The water elevation 123.30 m. for 15th September 2001 is near Full Reservoir Level (FRL) 123.50 m. and water elevation 112.02 m. for 24th April 2001 is near Minimum Draw Down Level (MDDL) of 109.72 m.

(Source : Ibid pg.13)

Table - 6.3 Live storage capacity of Reservoir at different elevations

	Elevation (m)	Area (Msp.m)	Live Capacity (Mcum)
MDDL	109.72	116.26	0.00
	110.00	120.50	33.15
	111.50	143.75	231.12
	113.00	167.96	464.70
	114.50	193.13	735.34
	116.00	219.26	1044.47
	117.50	246.35	1393.52
	119.00	274.40	1783.94
	120.50	303.41	2217.15
	122.00	333.38	2694.61
FRL	123.50	364.30	3217.74

(Source : Ibid Pg.17)

Modified Area - Capacity curve is given in Drg.6.4.

6.13.12 Result :

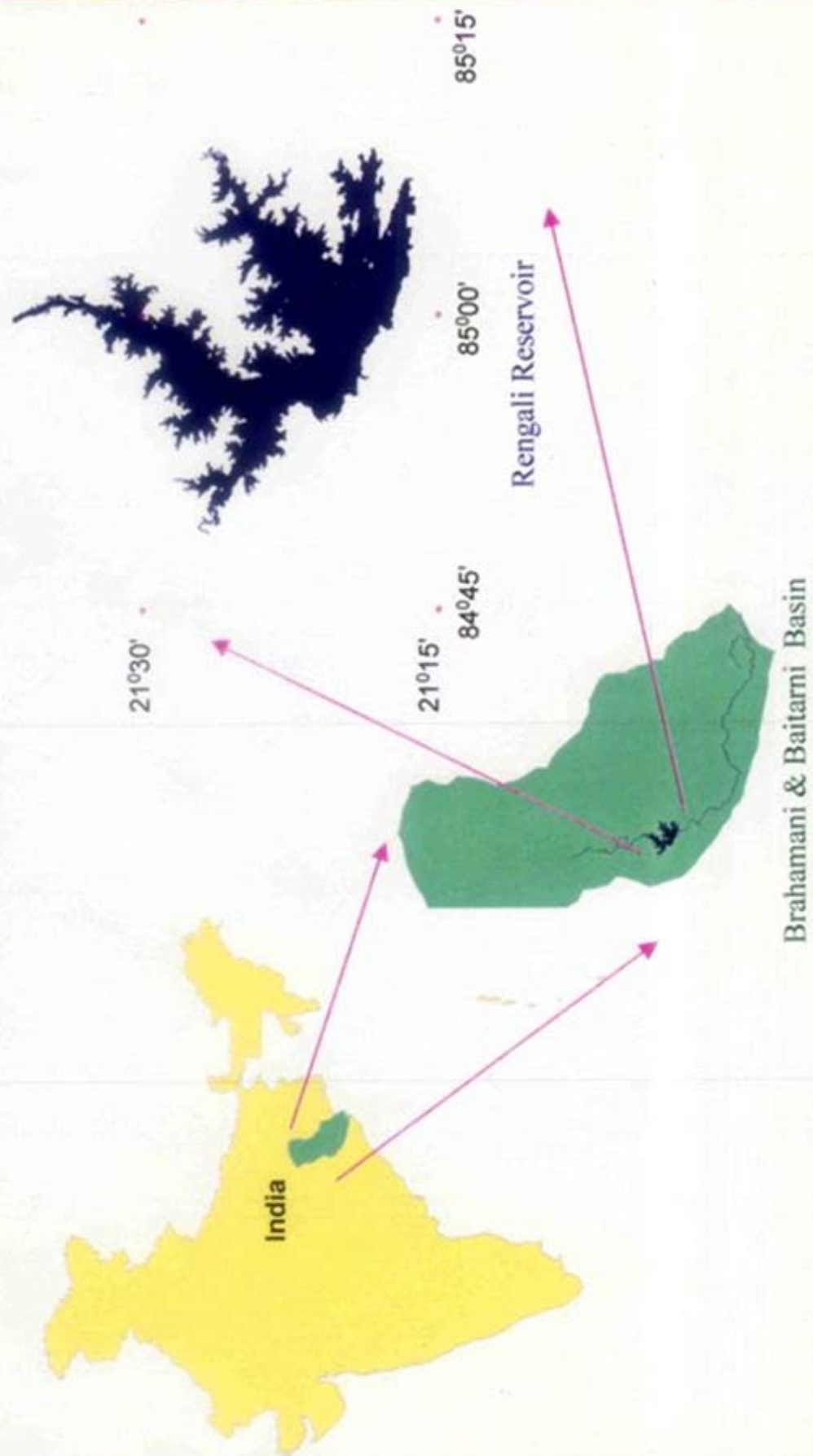
Sediment deposition in live storage zone of Rengali reservoir in 18 years (1983-2001) as carried out by SRS study is given in following table.

	Original (1983)	SRS (2001)
Live Capacity (mcm)	3412	3217.74
Loss in Capacity (mcm)		194.26
% loss		5.69
Annual % loss		0.32
Silting Rate (mm./year)		0.427 (*)

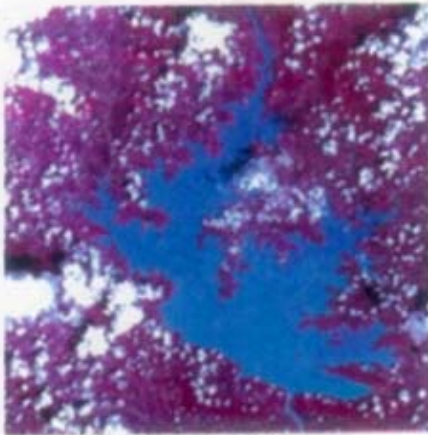
(*) The design rate of siltation was 0.39 mm/year in gross storage.

Present rate of silting is higher by 9.49%.

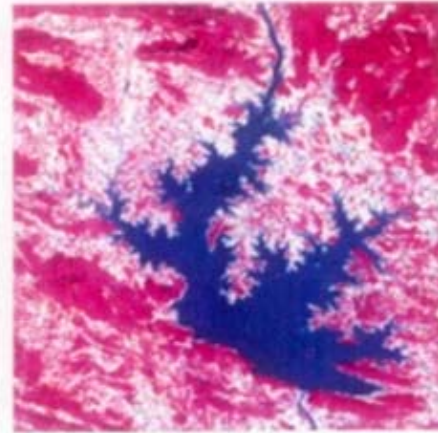
RENGALI RESERVOIR (Index Map)



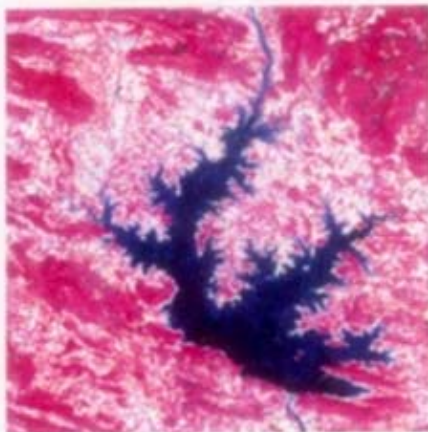
MULTIDATE FCC's OF RENGALI RESERVOIR



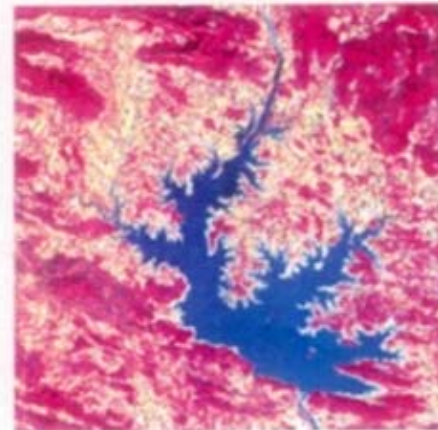
September 15, 2001



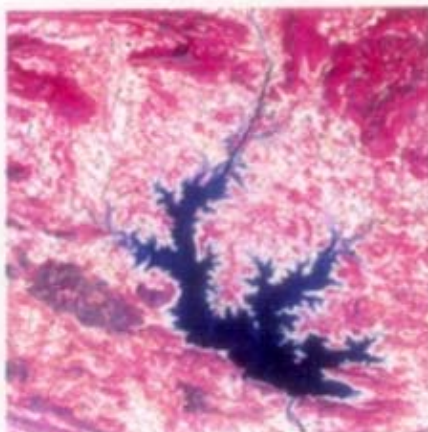
December 25, 2001



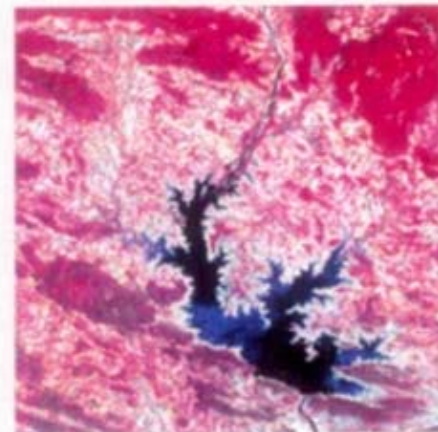
February 13, 2002



December 15, 2000



February 11, 2001



April 24, 2001

Drg. No. 6.3

Water Spread Areas of Rengali Reservoir

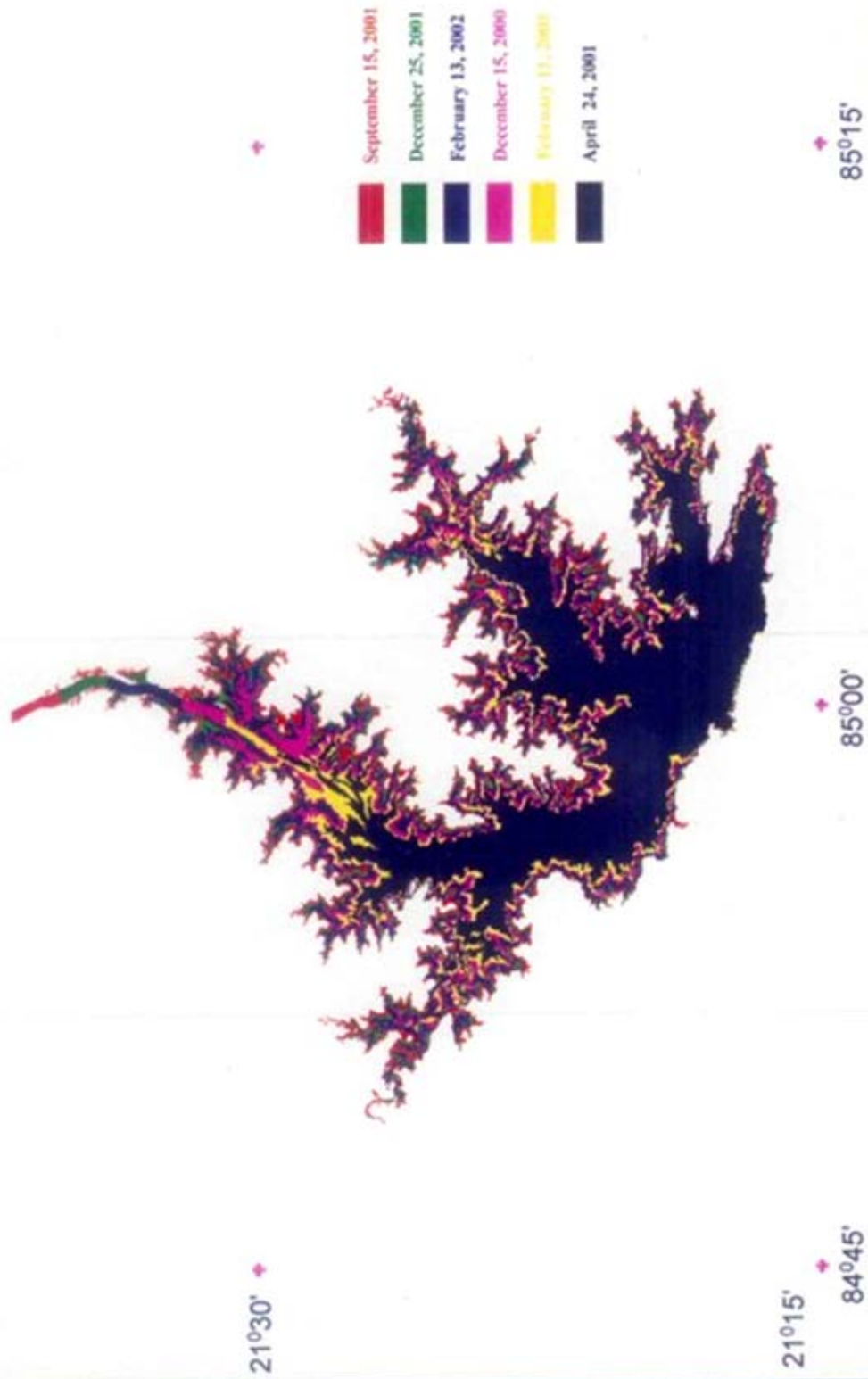


Fig. No.6.4 RENGALI RESERVOIR : MODIFIED ELEVATION-AREA-CAPACITY CURVE (2002)

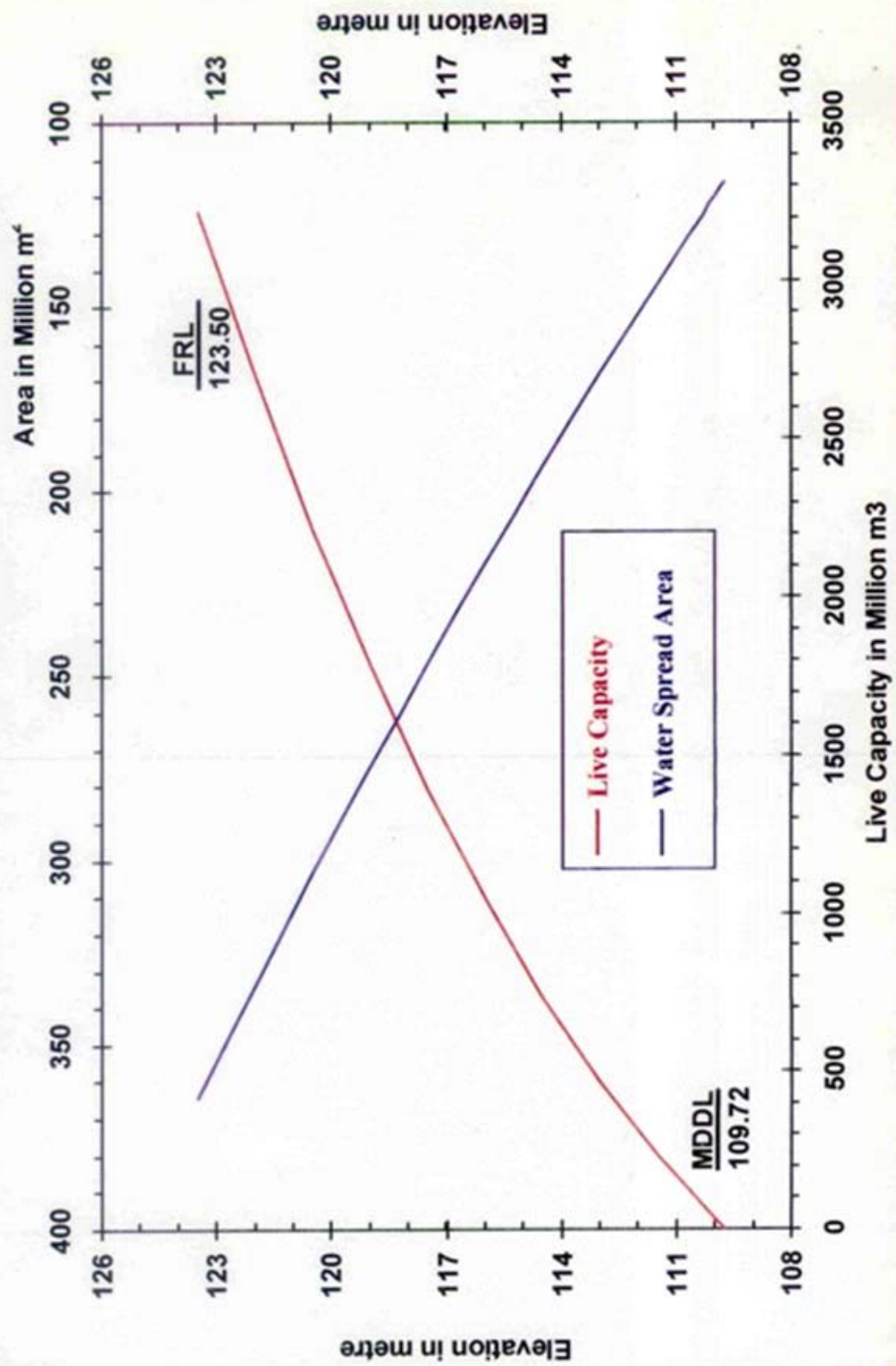


Table 6.5 Rengali Reservoir Levels (m) at the beginning of each month (1989-2005)

Month	1989	1990	1991	1992	1993	1994	1995	1996	1997	1998	1999	2000	2001	2002	2003	2004	2005	Avg.
Jan		116	118.79	117.85	112.88	119.48	120.9	122.51	120.33	121.92	120.96	121.87	115.76	120.73	121.74	120.63	117	119.33
Feb		115	115.49	115.1	110.79	117.08	119.28	121.76	118	122.88	119.26	120.84	114.77	119.75	121.05	118.4	117	117.9
Mar	115.86	114.91	113.93	113.29	110.23	114.76	117.59	120.07	116	120.86	118.12	119.73	113.87	118.46	120.28	117.51	117.76	116.66
Apr	113.45	113	113.07	111.76	109.94	110.95	114.54	117.81	116.64	118.29	116.4	117.48	112.83	117.18	118.57	116.41	115.65	114.94
May	111	112	112.17	110.67	109.65	109.76	112.74	114.51	115.07	116.45	114.39	114.44	111.75	113.05	116.05	114.9	113	113.04
Jun	109.85	111	110.49	109.92	109.28	108.77	110.88	111.3	112.16	113.16	110.94	110.5	110.25	110.73	112.71	111.91	111.05	110.88
Jul	113	115	109.15	109.35	113.26	118.95	110.26	114.1	114	110.23	111.85	109.08	113.31	112.32	106.76	109.96	115	112.09
Aug	117.16	117.86	117.41	114.74	117.49	122.78	116.04	119.48	116.53	111.27	117.48	116.51	121.37	110.92	112.91	110.99	117.29	116.37
Sep	121.19	119.49	122.7	120.29	118.75	123.16	121.41	122.24	122.9	115.19	122.6	116.18	122.97	117.93	119.6	120.9	118	120.32
Oct	123.49	123.72	123.29	118	124	123.98	122	122.77	123.38	123.01	123.64	119.15	122.86	122.96	123.55	121.85		122.6
Nov	123	123	121.53	117.57	123.43	123.38	122.5	122.86	122.92	123.82	123.44	117.56	122.86	122.62	124.12	120		122.16
Dec	123	123	119.46	115.36	122	122.47	123.11	121.9	122.44	122.37	122.87	116.72	121.78	122.34	122.57	120.39		121.36
Max.	123.49	123.72	123.29	120.29	124	123.98	123.11	122.86	123.38	123.82	123.64	121.87	122.97	122.96	124.12	121.85	118	122.79
Min.	109.85	111	109.15	109.35	109.28	108.77	110.26	111.3	112.16	110.23	110.94	109.08	110.25	110.73	106.76	109.96	111.05	110.01
Avg.	117.1	117	116.46	114.49	115.14	117.96	117.6	119.28	118.36	118.29	118.5	116.67	117.03	117.42	118.33	116.99	115.75	117.2

(Source : Draft report on capacity survey for sedimentation studies on Rengali Reservoir by CWC, Water and Reservoir Sedimentation Directorate, Report No.R-051/2006, Jan.2007 Pg.53)

Table 6.6 Water levels in m. during survey period in 2006

Month	March			April			May			June			July			Aug			Sept		
	0600	1300	1800	0600	1300	1800	0600	1400	1700	0600	1400	1700	0600	1300	1700	0600	1300	1800	0600	1300	1800
Time	0600	1300	1800	0600	1300	1800	0600	1400	1700	0600	1400	1700	0600	1300	1700	0600	1300	1800	0600	1300	1800
Date																					
1				116.39	116.35	116.31	113.78	113.78	113.78	111.08	111.01	110.88	109.75	109.76	109.77	118.7	118.67	118.63	122.67	122.75	122.78
2				116.23	116.16	116.14	113.75	113.75	113.75	110.87	110.79	110.76	109.84	109.88	109.89	118.45	118.3	118.26	122.85	122.89	122.91
3				116.06	116.02	15.19	113.75	113.76	113.75	110.66	110.58	110.55	109.91	109.91	109.91	118.31	118.31	118.31	122.98	123.01	123.02
4				115.9	115.84	115.83	113.71	113.68	113.66	110.5	110.42	110.39	109.87	109.88	109.89	118.13	118.31	118.31	123.05	123.07	123.02
5				115.76	115.72	115.72	113.56	113.5	113.5	110.27	110.21	110.18	109.88	109.91	109.94	118.27	118.34	118.38	122.93	122.88	122.86
6				115.62	115.56	115.55	113.44	113.4	113.39	110.14	110.15	110.16	109.91	109.94	109.95	118.49	118.53	118.55	122.92	122.95	122.97
7				115.45	115.42	115.42	113.35	113.35	113.35	110.19	110.24	110.77	109.94	109.96	109.97	118.61	118.64	118.64	123	123.03	123.03
8				115.35	115.32	115.31	113.33	113.33	113.33	110.28	110.3	110.36	109.96	109.98	109.99	118.65	118.64	118.64	123.04	123.03	123.03
9				115.2	115.16	115.16	113.35	113.35	113.35	110.35	110.32	110.3	109.98	110	110.01	118.6	118.58	118.58	123.02	123.03	123.03
10	117.45	117.45	117.45	115.06	115	114.96	113.34	113.34	113.34	110.25	110.2	110.19	109.98	110	110.01	118.55	118.55	118.56	123.03	123.04	123.05
11	117.44	117.44	117.44	114.92	114.86	114.84	113.33	113.33	113.33	110.1	110.4	110.03	110.15	110.22	110.27	118.55	118.57	118.9	123.06	123.09	123.09
12	117.4	117.4	117.4	114.78	114.74	114.72	113.3	113.3	113.3	109.96	109.95	109.95	110.35	110.48	110.54	118.76	118.84	118.9	123.09	123.1	123.11
13	117.36	117.36	117.36	114.66	114.62	114.6	113.26	113.25	113.25	109.97	110.01	110.01	110.65	110.72	110.77	119.09	119.18	119.12	123.12	123.14	123.15
14	117.29	117.27	117.26	114.58	114.58	114.52	113.23	113.23	113.22	110	110.02	110.04	110.82	110.87	110.9	119.21	119.37	119.42	123.16	123.15	123.16
15	117.2	117.19	117.18	114.54	114.53	114.51	113.19	113.17	113.14	110	110.02	110.03	110.91	110.98	111.01	119.61	119.71	119.75	123.2	123.2	123.23
16	117.17	117.17	117.17	114.46	114.46	114.45	113.1	113.74	113.06	109.96	109.97	109.98	111.07	111.11	111.15	119.87	119.93	119.96	123.25	123.27	123.28
17	117.16	117.15	117.14	114.4	114.38	114.38	113.02	113.02	113.01	109.96	109.98	109.98	111.18	111.24	111.25	120.05	120.17	120.25	123.29	123.28	123.27
18	117.1	117.1	117.1	114.33	114.32	114.32	112.98	112.98	112.98	109.99	109.98	109.95	111.37	111.60	111.74	120.45	120.55	120.66	123.27	123.26	123.25
19	117.09	117.09	117.09	114.29	114.29	114.28	112.94	112.94	112.94	109.83	109.83	109.86	111.99	112.11	112.17	120.95	121.13	121.27	123.23	123.21	123.2
20	117.06	117.01	117	114.25	114.24	114.24	112.92	112.94	112.93	109.78	109.78	109.78	112.24	112.26	112.27	121.59	121.78	121.97	123.19	123.18	123.17
21	116.95	116.93	116.93	114.22	114.21	114.21	112.9	112.9	112.9	109.71	109.71	109.71	112.34	112.48	112.57	122.31	122.4	122.52	123.14	123.13	123.12
22	116.9	116.89	116.89	114.18	114.18	114.18	112.86	112.84	112.82	109.7	109.71	109.71	112.77	113.2	113.56	122.62	122.67	122.7	123.11	123.15	123.25
23	116.86	116.83	116.82	114.14	114.11	114.11	112.78	112.78	112.78	109.71	109.72	109.72	114.21	114.45	114.58	122.51	122.51	122.56	123.25	123.37	123.4
24	116.79	116.77	116.77	114.06	114.05	114.05	112.74	112.74	112.73	109.69	109.69	109.69	114.82	114.96	115.04	122.7	122.7	122.65	123.46	123.49	123.52
25	116.74	116.74	116.74	114.01	114	113	112.69	112.67	112.65	109.7	109.7	109.7	115.17	115.23	115.31	122.61	122.61	122.63	123.57	123.6	123.64
26	116.71	116.71	116.71	113.05	113.96	113.96	112.55	112.51	112.49	109.68	109.68	109.69	115.35	115.37	115.38	122.65	122.7	122.75	123.77	123.77	123.73
27	116.67	116.67	116.67	113.94	113.93	113.93	112.38	112.38	112.3	109.69	109.69	109.7	115.39	115.41	115.43	122.74	122.69	122.64	123.79	123.77	123.7
28	116.65	116.65	116.64	113.89	113.89	113.89	112.17	112.12	112.09	109.69	109.7	109.7	115.5	115.6	115.64	122.57	122.54	122.5	123.55	123.54	123.55
29	116.65	116.62	116.61	113.86	113.86	113.85	111.96	111.88	111.85	109.71	109.72	109.72	115.77	115.89	115.93	122.49	122.55	122.56	123.56		
30	116.56	116.55	116.54	113.82	113.82	113.82	111.75	111.67	111.65	109.71	109.72	109.72	116.26	116.6	116.92	122.48	122.42	122.38			
31	116.5	116.48	116.47				111.35	111.45	111.42	109.71	109.72	109.72	116.76	118.38	118.63	122.43	122.52	122.56			

(Source : Ibid Pg.38-39)

6.1.3.2 By Hydrographic & Topographic Survey :

6.1.3.2.1 Methodology :

The study includes conducting Hydrographic and Topographic surveys of entire reservoir area upto MWL and analyse the data to obtain area-elevation-capacity relationship and expected life of reservoir.

For Rengali, Hydrographic surveys were carried upto existing water level in grid of 100m x 100m using Global Positioning System (GPS) in differential mode and topographic surveys in grid of 100m x 100m above the water level upto MWL using Total Stations. Data processing was done using NADS processing software. Both hydrographic and topographic data was integrated and merged for drawing contours at 1m interval using scales of 1:50,000 and 1:15000. Total 10 samples of bed material from reservoir were collected using grab sampler and tested in laboratory for grain size distribution, specific gravity, bulk density, dry density and moisture content. None of the samples were found to have gravel content. The test results are enclosed vide Table 6.4. Reservoir levels at the beginning of each month from 1989 to 2005 are furnished in Table 6.5 as the operation pattern has a relevance to the sedimentation. The reservoir water levels observed during the survey period i.e. from 10 March to 28 Sept.2006 are given in the Table 6.6. Presence of trees and submerged objects in reservoir area posed major problems during navigation and survey line tracking. The reservoir is surrounded by dense forests and hills all around making it inaccessible for land survey teams from almost all places.

Table 6.4 Test Results of Sediments

Sample ID	Specific Gravity	Bulk Density (mg/m ³)	Dry Density	Particle Size Distribution (%)						% Water Content w/w
				Clay	Silt	Fine	Sand Medium	Coarse	Gravel	
RR-01	1.60	1.57	0.08	28.29	26.08	45.60	0.03	0.00	Nil	19.03
RR-02	1.64	1.60	0.06	22.59	1.79	72.64	2.98	0.00	Nil	23.89
RR-03	1.54	1.50	0.03	99.83	0.04	0.11	0.02	0.00	Nil	53.54
RR-04	1.86	1.82	0.04	99.94	0.02	0.04	Nil	0.00	Nil	49.18
RR-05	1.70	1.66	0.06	62.07	18.93	18.40	0.60	0.00	Nil	28.24
RR-06	1.35	1.33	0.03	77.04	4.45	16.61	1.90	0.00	Nil	51.81
RR-07	1.91	1.88	0.07	23.63	2.63	72.46	1.28	0.00	Nil	24.63
RR-08	2.04	1.99	0.07	61.68	13.86	20.57	3.98	0.00	Nil	27.06
RR-09	1.99	1.95	0.03	99.72	0.06	0.10	0.12	0.00	Nil	65.5
RR-10	1.77	1.73	0.07	32.47	4.81	25.4	37.94	0.00	Nil	22.12

Remark : Date of collection 14.12.06, sample Quantity - 500 ml.

Date of reporting 24/12/06

6.1.3.2.2 Conclusion :

By analysing the survey data following conclusion has been drawn by Sea Geo Surveys Pvt. Ltd. (SGSPL). The firm was awarded the work of consultancy services for capacity survey of Rengali reservoir.

1. “This covers the survey and studies carried out by (SGSPL) on the Rengali Reservoir in State of Odisha, which has a reservoir area of 414 sq.km at MWL.
2. The 71 m high above foundation level composite dam drains catchments of 25250 sq. km.
3. The sedimentation analysis has been carried out in accordance with the IS 12182/1987 -

“Guidelines for determination of Effects of Sedimentation in Planning and Performance of Reservoirs”, CBI & P publication on the subject and IS 5477 Part II “Fixing capacity of Reservoirs - Dead Storage”.

4. The studies indicate difference in reservoir area at FRL between 1982 - original data and the present survey of 2006. Present survey conducted in March - October 2006 indicated that the reservoir area at FRL is 362.07 sq. km. against 1982 area of 378.4 sq. km.
5. The rate adopted for design of the dam is calculated by considering dead storage to be filled in 100 Yrs without consolidation of sediment works out to 9.983 Mcum. The annual sedimentation rate as per 2006 survey works out to 31.31. A comparison with the rate adopted for design shows that sedimentation rate is larger than the design rate.
6. The future projection of sedimentation has been made in accordance with the relevant BIS code. In this case as the annual loss is only 0.6% the projections have thus been made at an interval of every 20 years using the latest survey data and duly adjusting the empirical parameters. Elevation area capacity curves have been computed for blocks of 20 years up to year 2106 using the design curves developed for Rengali Reservoir sediment deposit estimate have been considered. Using Sediment deposition during 1982-2006 i.e. 751.4 Mcum the sediment distribution has been carried out by using empirical area method and revised Capacities are estimated the estimated capacities match well with the surveyed results.
7. Reservoir sedimentation data that is, the reservoir geometry is available for 1982, 2001 and 2006 post monsoon situations, for Rengali. However, considerable processing of the 1982 area data was necessary to obtain consistency. Some data had to be discarded. This data at three points in time does allow one to study the time trends in sedimentation. In case of Rengali, as stated, the trend could be calculated, but considering the small time slices of 19 and five years, and the likely inaccuracies in the data, we conclude that the trends are not well established, and cannot be projected with any confidence. In any case, projecting such trends for another 100 years, in the face of unknown changes in land use, and particularly in the face of likely climatic, and consequent hydrologic changes, is, perhaps, a mere indicative exercise. This needs to be kept in view.
8. Two possible postulations could be made about the future trends under the optimistic *Future-1* scenario, the reservoir would have a comparatively longer life, but, even then, the “Feasible Service Time” is expected to end by about the year 2070. Under the comparatively pessimistic *Future-2* Scenario, the “feasible Service Time” may end by about the year 2060, that is in another 53 years.
9. When the Feasible Service Time ends, frequent jamming of the low level gates, larger passage of sediments in the power house and consequent equipment maintenance problems, larger passage of sediments in irrigation canals, etc., may result.
10. The live storage sedimentation and the consequent reduction of benefits is a very serious problem for Rengali. The total capacity up to FRL was about 4464 million cubic meters in 1982, and has reduced to about 3750 million cubic meters by 2006. In next 100 years, it would reduce progressively up to about 753 million cubic meters (Future 1) or an even smaller 96 million cubic meters (Future 2).
11. In terms of live storage, the initial figure of 3466 million cubic meters has been slightly reduced to 3131 million cubic meters by 2006. In next 100 years, this may reduce almost drastically to 753 million cubic meters, provided the sedimentation rate is stationary. However, with a slightly increasing trend, the reduction could be up to a very small residual live storage of 96 million cubic meters. Such large reduction would require a complete change in the water and power management plans, to reduce social distress.

12. The available capacity between FRL and MWL was 753 million cubic meters, and this has, by 2006, reduced to 716 million cubic meters. By 2106, this may reduce to 622 million cubic meters under Scenario 1 or 450 million cubic meters in scenario 2. This is provided that the salient levels are not changed. Since this would constitute a large portion of the available storage, the possibility of increasing the gate height and converting the MWL into FRL may be considered. Either conservation or the flood control objective may be given up. The whole reservoir may be considered as an important shallow wetland.
13. Similarly, the annual power generation will get affected. The power station, may, progressively, become a base load station in the wet season and a low load factor peaking station in the non-monsoon period.
14. As stated, such long-term projections are rather indicative. But, under both the scenarios, a fast reduction of capacities would start by 2026, which is not too distant. A detailed study of the problem and development of new water and power development plans needs to be started.
15. It is seen that incase of Future 1 the gross storage decreases to 37.8% by 2086. Dead storage available at the end of year 2086 would be 0%. Thus the useful life of the reservoir is likely to end before 2086. In case of Future 2 the Gross storage decreases to 38.6% by 2066 and the dead storage available at the end of 2066 would be 0%. Thus, the useful life of the reservoir is likely to end before 2066.
16. Under the optimistic Future-1 scenario, the reservoir would have a comparatively longer life, but, even then, the “Feasible Service Time” is expected to end by about the year 2070. Under the comparatively pessimistic Future-2 scenario, the “Feasible Service Time” may end by about the year 2057, that is in another 50 years.
17. It is seen that 66% of the total land area of the upper Rengali Valley has been affected by different types of erosion. Out of this area, 35% of the total area under agriculture has been affected by sheet erosion. The rate of progress of gully erosion is of serious concern. The rate of extension of gully-heads is varying year by year.” (Source : Ibid Pg.90-93)

6.1.4 Control of Sedimentation :

Due to the multiple variables involved in reservoir sedimentation, no single control measure can be considered as the most effective. The measures, which can be employed to limit sedimentation and turbidity, can be grouped into following major categories :

- ▶ Soil and water conservation measures with the drainage basin, contour plowing, strip cropping, suitable farming practices, improvement of agricultural land, construction of small dams/ponds/terraces/check dams on gullies
- ▶ Revetment and vegetation cover
- ▶ Evacuation of sediment
- ▶ Stream bank and flood plain protection
- ▶ Ridge plantation such as pasture development and Reservoir shoreline protection.

Even after two to three decades of soil conservation measures Dantiwada, Matatila, Nizamsagar and Ukai reservoirs have increasing trend of sedimentation vide Table 6.7.

Table 6.7 Statement Showing Rate of Sedimentation of Various Reservoirs & Area Treated Under Centrally Sponsored Schemes (upto 2000-2001)

Sl. No.	Name of Reservoir (Year of Impounding)	Catchment Area 000 ha	Priority Area 000 ha	Treated Area 000 ha	% wrt Priority Area	Cost of Treatment Rs. Crores	Year when soil conservation started	Silt Rate in ha.m./100 sq. km. / year		Period of Soil Conservation treatment up to last hydro-graphic Survey Year
								Ist Survey & year	Last Survey & year	
1.	Pong Reservoir (1974)	1069	452.00	35.75	7.91	16.47	1961	22.68 1980	17.21 1998	37
2.	Damanganga (1983)	170	77.00	39.44	51.22	20.73	1974	5.52 1992	0.67 1996	22
3.	Dantiwada Reservoir (1965)	264	123.00	141.62	115.14	35.94	1962	12.55 1984	31.15 1994	32
4.	Hirakud (1957)	8300	362.00	392.56	108.44	51.11	1961	6.57 1979	5.62 1994	33
5.	Lower Bhawani (1953)	287	93.00	69.61	74.85	32.03	1969	18.98 1964	0.73 1983	14
6.	Kadana (1977)	2466	848.00	200.66	23.66	55.32	1969	4.9 1981	2.6 1984	15
7.	Matatila (1956)	1936	829.00	142.65	17.21	31.94	1969	1.37 1969	2.86 1990	21
8.	Nizamsagar	2096	511.00	121.85	23.85	93.18	1969	5.46 1967	8.5 1992	23
9.	Bhakra (1958)	1020	264.00	160.12	60.65	42.9	1961	6.29 1963	4.9 1998	37
10.	Tungbhadra (1953)	2778	502.00	322.75	64.29	46.52	1962	17.9 1963	0.41 1993	31
11.	Ukai (1972)	5953	607.00	106.63	17.57	25.26	1969	6.2 1979	9.35 1992	23
12.	D.V.C.									
12a.	Maithon (1995)							12.53 1963	11.17 1994	33
12b.	Panchet (1956)	1657.00	933.00	477.55	51.18	138.28	1961	12.13 1962	3.13 1996	35

(Source : Preservation of Reservoir storage through catchment area treatment by Rajesh Kumar, 'Bhagirath', Vol.LVII, July-Sept.2010 Pg.22)

6.1.5 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 Reservoir 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 reservoirs will not reduce the benefits faster than envisaged.”

X X X X X

“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 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.” (Compendium on silting of Reservoirs in India, CWC, Watershed and Reservoir Sedimentation Directorate, Jan.2001 Pg. 2 & 16)

6.2 Reservoir Operation :

6.2.1 Introduction

“The surface Water resources projects are developed for the benefits of irrigation, hydro power generation, flood control, water supply (domestic/industrial) and other benefits. The irrigation facilities consume much of the water stored or diverted at these projects, where as the hydro-power and flood control components do not consume any water. Water resource development projects are planned either as single purpose schemes or multipurpose schemes depending upon the situation demands. Multipurpose storage schemes should, however, be preferred as far as possible. Planning and operation of such projects need careful consideration of the competitive requirements by the different uses. It may not be possible to build storages to meet the complete requirements by irrigation, hydro power and flood control due to restrictions on submergence and economic viabilities. But a well thought of and worked out plan for the size and operation can bring optimum benefits from the scheme. Where joint use of some of the stored water is envisaged, operation of the reservoir will have to be based on priority of one use over the other and the compatibility among demands for different uses. In preparation of

regulation plans from integrated system of reservoirs, principles applicable to separate unit are first applied to the individual reservoirs. Modifications of the schedules so developed shall then be considered by working out several alternative plans based on the co-ordinated operation, and the best plan selected. The release requirements by irrigation and hydro power may not always match, resulting in water loss for one case when the operation is done to meet the needs of the other. For example the irrigation requirements in summer may be very low, whereas hydro power needs are high. In such a case we may like to release the water to produce power, and compensate the loss in irrigation through ground water utilization. The power required for pumping could be more, or less, than the power generated depending on the specific situation. In case of flood control operation, the advantage of flood forecasting should be taken, to permit possible pre-releases and increase flood absorption capacity without the risk of losing conservation storage. Also to some extent, the same reservoir capacity can be used to flood moderation in the more active part of the flood season, and for conservation purposes thereafter, through a well designed rule curve for filling” (Source : Hydropower and River valley Development - Post conference Proceedings and Recommendations, Edited by R.S. Goal & R. N. Srivastava, Oxford & IBH Publishing Co. Pvt. Ltd, 2000 Pg.7)

Storages of multipurpose reservoirs are generally shared by conflicting demands i.e. flood control, irrigation and power. For flood control, it is required that the reservoir water levels are to be kept low during flood season where as power generation requires that the reservoir level to be maintained as high as possible. Similarly irrigation demands that the reservoirs should have maximum conservation storage towards the end of the filling season. To compromise between the conflicting uses like flood control and hydropower generation and to conserve sufficient water to meet the irrigation needs, it is essential to operate the reservoir judiciously. This is precisely the reservoir operation. The operation should preferably be flexible. For this reason, a rule curve is prescribed basing on which the engineer-in-charge operates for optimising the benefits.

6.2.2 Current Operating Practices in India :

Operation manuals have been prepared for most of the major Indian reservoirs in which the emphasis has been given to the structural safety of the dam and its appurtenants works. However, the manuals do not generally prescribe any reservoir operation rules beyond indicating the likely levels that would be achieved if the reservoir serves its designed objectives.

Mehnidiratta and Hoon (1973) described in detail the problems that were being faced in the regulation of supplies from the Bhakra Reservoir. They pointed out that while the reservoir was mainly planned for irrigation, the heavy demand for power many times needed releases that were much in excess of irrigation demand and were consequently not absorbed by the irrigation system. This excess drawal for power finally affected the irrigation component adversely, as water was not available at the time of irrigation needs. To obviate such problem the Bhakra Management Board tried to enforce a drawal schedule, but again, due to heavy demands of power from the northern grid, excess releases had to be made and the reservoir went below dead storage.

Foutuitoulsy, the Bhakra reservoir which was planned as a carry over reservoir, but had become an annually operated reservoir due to the problems described above, has such a large storage capacity, compared to its individual flood peaks, that it had not flood control problem. “The Bhakra-Nangal Project (BNP) authorities claim that, irrespective of whether it was an intended purpose or not, the fairly large storage capacity of the Bhakra reservoir was always utilised in such a manner as to render flood moderation benefit for the region below the dam. This is quite understandable and has, indeed, been the case with many a large dams in India and elsewhere. Infact, there are hardly half a dozen dams in India where specific flood storage spaces are reserved for flood control. Many of the multi-purpose reservoirs without allocated flood reserve, particularly the larger ones, have rendered significant flood moderation benefits. x x x x It will be apparent that when the flood impinges into the reservoir in July or

August, the water level in the reservoir is likely to be lower than the FRL. However, if the flood occurs in October or late September, a difficult judgement will need to be made by the operating authority. The extent of reliable inflow / flood forecasting arrangement that exists, and the type of reservoir operation instructions available, would influence the decision. Whatever the precision of these aids, the situation would call for sound judgement, based on past experience, by the regulatory authority.”

(Source : Bhakra-Nangal Project - Socio Economic & Environmental Impacts by R. Rangachari, Oxford University Press, 2006 Pg.151-152).

Of the reservoirs for which operation schedules exist, the Hirakud and D.V.C. operation schedules are the most outstanding examples. In fact both these reservoirs have started as flood control reservoirs and in course of time have been transformed to irrigation and power reservoirs with flood control need almost continuously being questioned. In spite of this the reservoirs with their limited flood absorption capacities and frequent encroachment of flood reserves have still performed reasonably well.

6.2.3 Case Study :

6.2.3.1 Operation Practices in Pakistan :

“In Pakistan, under the Indus waters treaty, three reservoirs were constructed mainly for irrigation. Power has been provided as an adjunct, but flood control was not contemplated in the beginning. The evolution of flood control in these projects (Riaz Nazir Tardar.1976) is very educative. It was stated by Tardar -

“The flood control benefit that could be realized from storage and river regulation facilities of the Indus basin projects also needs to be mentioned. Following the Commissioning of Mangla, the western river inflows were dwindling for a while as a result of serious drought in the region and progressive upstream abstractions. This apparently reduced the frequency and magnitude of the floods and the aspect of flood control got obscured. However in 1973 the area was visited by large floods (due to a sudden reversal of hydrological cycle) which caused a lot of damage to life and property. But for the partial flood regulation in Mangla reservoir the damages would have been more. This was possible as the filling criteria did envisage / attempt partial flood control through the maximum and minimum rule curve operation.”

‘Following the 1973 catastrophic floods the Federal / Provincial Governments are actively engaged in measures to provide stream flow forecasting including other facilities’.

A brief summary of subsequent developments of operation criteria is as follows :

6.2.3.1.1 Mangla

“Reservoir operation criteria prescribe an envelope of maximum and minimum reservoir levels by ten-day periods. Generally during the period of July-September the reservoir is operated along the minimum rule curve to have available the maximum flood regulation storage. However, in case of the reservoir being above the minimum rule curve at the advent of a particular flood (due to preceding high runoff etc) an attempt is made to pull it down in advance.”

6.2.3.1.2 Chashma

“In this case also the reservoir operating criteria assumes the form of an envelope of maximum and minimum rule curves by ten-day periods. The reservoir is maintained at 195.67 m till about the end of flood season and the storage of 616.74 ham between R.L. 195.67 m and R.L. 197.81 m is used for reducing the peaks in the region of 11290 cumecs to 26810 cumecs occurring before the end of August. No flood cushion is provided beyond August and hence no flood control will be possible for post August floods.”

6.2.3.1.3 Tarbela

“Due to certain structural problems in Tarbela, this particular dam is not being used to its full capacity at present. It is presumed that when the structure is strengthened this reservoir also can be used for flood control on the lines of Mangla.” (Source : Operation Manual of Rengali Reservoir, by M.H.P.B. Patnaik, M.E.Dissertation, W.R.D.T. Centre, University of Roorkee, 1977, Pg.30-31)

6.2.3.2 Reservoirs without Operating Schedules :

“A number of projects in India do not have regular operating schedules. Their operation manuals lay great stress on the regular maintenance of the structures, but do not lay down strict release patterns especially during flood seasons.

A few examples of failures of certain dams, either structurally or hydrologically, are enumerated below.

In 1978, Sept 1-3, heavy rainfall occurred in the catchment of the Kansabati river. Since this reservoir was not getting filled up in most of the earlier years, the engineers-in-charge, in good faith, tried to store as much water as possible. But since no inflow forecasting was available to them at that point of time, they had no idea of the volume of the flood, and when the decision to open the gates was taken, the reservoir had almost risen to the full level. Fortunately, the flood intensity subsided immediately afterwards and the structure was saved, otherwise the dam was in danger of being overtopped. The releases from the spillway created wide spread havoc, and but for assistance from the Air Force from the neighbouring Kalaikunda base which went to the rescue of marooned people without waiting for formal orders, the loss of life could have been heavier.

A few days later, during the heavy floods of Sept 26-28, 1978, the Hinglow dam burst. It had no operating schedule, and the reservoir was almost full at the time of impingement of flood and as the inflow was much higher than the spillway capacity, the free board was rapidly encroached and the dam was overtopped.

Similarly, during the same period the Mayurakhi reservoir was filled to capacity and when the dam authorities decided to open the spillway gates, the engineers-in-charge of the Tilpara barrage lower down had no information as the land line communication got disrupted in the storm and before the barrage gates could be opened fully, the right guide bund breached and the barrage was outflanked. There was wide spread loss of life at that time.

The two cases enumerated above, have been examined by the Ghosh Commission, and it is reported that the Commission found the authorities of the respective projects not to be blamed and that the unprecedented rainfall during that period was responsible for the deluge. To some extent, it is true, especially as wide spread loss to life occurred in the Ajay, Silabati, Dwarkeshwar and other rivers which had no dams across them. However, some nagging uneasiness persists that had there been regular filling schedules, these catastrophies could perhaps have been avoided.

In 1979, around August 8th, a depression was formed in the Bay of Bengal and it crossed the coast near Puri. It crossed the Brahmani, Damodar and Sone river catchments without appreciable rainfall, but caused a cloud burst in the catchment of the Machhu river. Due to some problems, the gates could not be opened at the critical time and Machhu II was overtopped and the dam failed. The sleeping town of Morvi, situated a few kilometers below the dam was devastated and there was wide spread loss to life and property. Here agains regular filling schedules could have saved the situation. “(Source : Ibid Pg.32-34)

6.2.3.3 Merits & Demerits of Current Operating Practices :

Murphy's law states (Willeke G. E. 1979) 'If any thing can go wrong, it will'. The fate of all the operation schedules proves Murphy's law to a large extent. What would appear to be a merit at a certain point of time would be seen to be a demerit at the next occasion. However, the following points need further consideration.

6.2.3.4 The Hirakud Reservoir Operating Schedule :

"The Hirakud project was originally conceived as a flood control project. But even before some trial could be given to the rules contained in the Manual of Operations 1959, the flood space was encroached and resulted in avoidable floods. Even the experts committee appointed by Govt. of India in 1976, started with the premise that the type of conservation failure that occurred in 1974, should not be repeated. It did not stipulate that the type of flood that occurred in 1960 or 61 should also not occur. The committee further went ahead with the proposal of an additional spillway (which could not materialise) to pass the probable maximum flood (P.M.F). In fact the committee decided that (i) conservation, essentially for power and (ii) safety of the dam are of prime importance and flood protection, if any, is only of a consequential nature. "(Source : Ibid Pg.34)

For more details on Hirakud Reservoir Operation the publication "Hirakud : Its Back Ground and Performance" written by G. C. Sahu and G. N. Das, published by INCID under MOWR, New Delhi (2012), Pg.320-366 may be referred to.

6.2.3.5 The Damodar Operating Schedule :

"From the time, the D.V.C. dams were planned, the D.V.C. was under attack. Voorduin provided for 28225 cumee capacity for the spillways. This was considered too liberal even by West Bengal Govt. (D.V.C. 1978). However Dr. E. V. Morgan of T.V.A. wrote to D.V.C. on the following lines -

'I understand that you have been criticised for having fixed upon a design flood which is too great and which is beyond possibility and which therefore would result in incurring unnecessary expense. My own concern is entirely the opposite The only criticism left for future engineers to make should be that they (the spillways) are unnecessarily large'.

The flood of 1978 proved Voorduin and Morgan correct. It had already recorded a peak of 24020 cumee and had the storm remained stationary around Hazaribagh or Ramgargh as it did in earlier times, the peak could have easily crossed 24000 cumec of peak intensity as well as Voorduin's design flood volume.

The other criticism (Basawan Sinha 1980) was that the D.V.C operation schedule neglected the downstream condition and even at the height of regulating the maximum recorded flood in living memory the full flood space was not utilized. However in 1959 when the full space was utilized and the D.V.C. releases crossed 7900 cumec, D.V.C. was not only criticised, but its flood operation responsibilities were transferred to the Central Water and Power Commission (CW&PC). The argument that ideal regulation should aim at using the entire flood storage space in all floods, major or medium, and that the outflows should be so regulated so that the combined flows at the head of the flooded area should always be constant aims only at the impossible. This is because, the science of meteorology has not advanced to a stage where a quantitative forecast for 24 hours in advance can be given with any accuracy, and attempting to plan regulation on as yet unreliable meteorological forecasts is to invite disaster. In fact meteorologists themselves (Ramage 1978) dissuade the profession from depending too much on the accuracy of forecasts.

The merits of the D.V.C. regulation were evident in that even at the time of unprecedented floods, the regulation remained within the schedule, no necessity for emergency operation arose (though

was briefly contemplated) and after the flood peak had passed it permitted certain amount of flexibility in operation with the available flood storage. Even the critics of D.V.C. regulation have indirectly admitted about the efficacy of the flood regulation capacity of D.V.C. (N. K. Bose 1969) vide Table 6.8.

Bose who was opposing further flood control admits that the reduction of frequency and duration of floods of higher magnitude specially above 4000 cumec have been markedly low after 1958 when the dams came into operation.

Table 6.8 Regulation Effect of D.V.C. Reservoirs

Discharge in cumec	Pre-dam period 1946-57		Post-dam period 1958-63	
	No. of occasions	Duration in hours (Annual average)	No. of occasions	Duration in hours (Annual average)
2000	120	217.7	53	190.3
3000	53	97.8	26	95.3
4000	28	37.1	5	18.3
5500	11	15.8	1	6.3
7000	6	7.8	1	4.7
8500	4	2.4	1	1.3

The demerits as already stated were that (a) it did not entail the use of all the flood space, (b) was too rigid and gave the operator no leeway, and (c) by having increasing slabs at increasing reservoir levels accentuated the flood problems when the drainage congestion was very heavy.

Among the remedies suggested were :

- Construction of additional dams especially at Belpahari on the river Barakar for consumptive uses in Bihar and flood control in West Bengal.
- Partial acquisition (houses only) of remaining land in the existing reservoirs and using the same exclusively for flood control.
- Either allowing all flows above 2800 cumec to spread uniformly in the trans Damodar area through spill channels or to jacket the river Mundeswari to carry at least 7000 cumec and the river Amta to carry 3000 cumec.
- To improve drainage in the trans Damodar area and improve the drainage in the Silai, Dwarkeswar, Roop Narain and other trans-Damodar rivers. “(Source : Ibid pg.36-39)

6.2.4 Rengali Reservoir Operation Schedule :

6.2.4.1 Operation of Rengali Reservoir :

The co-ordination committee in their Ist meeting held on 28.10.1994 for the operation of Rengali Reservoir had recommended that the upper limit on Ist of October to be 123.50 m. A Technical Committee was constituted vide E.I.C, Irrigation letter No.8015 Dt.17.05.1993. The Technical Committee accepted the proposal with suggestion to modify the upper limit on Ist October from RL 123.50 m to RL 124.0 m. The rule-curve for operation of Rengali Reservoir during monsoon period stands as follows :

Table No.6.9 Rule Curve for Reservoir Operation :

Date	Lower Limit (m)	Upper Limit (m)	Remark
30th June	109.72	109.72	DSL - 109.72 m FRL - 123.50 m Top of Gate - 125.0 m
15th July	110.75	110.75	
1st August	115.85	115.85	
5th August	117.00	117.00	
1st September	121.95	122.50	
1st October	123.50	124.00	

Above limits were decided as per the recommendations of the 1st coordination Committee meeting held on 28.10.1994. After 1st October the reservoir can be kept even more than 124.0 m but not more than 125.0 m which is the top of gate.

The rule curve decided in 1st co-ordination Committee as modified by Technical Committee has been furnished in Table No.6.9. During discussion the E.I.C., Electricity-cum-Electrical Projects had certain reservations. He suggested to keep upper limit higher than the lower limit during June to August instead of keeping those same as per the recommended rule curve. He also opined that it is always better to keep the reservoir level at higher upper limits than the lower limits from consideration of power generation. In this connection it was explained that lower and upper limits are generally fixed from rainfall-run off considerations for past several years and also that the rule curve is not the only guide line for reservoir operation. Many a times, the downstream conditions of flood, inflow into the reservoir and rainfall in upstream and downstream catchments are taken into consideration.

This conflict is usual and the blame game continues. The decision is crucial from the consideration Power generation and Flood control. To achieve one, the other is to be sacrificed. One can not have the cake; and eat it too.

Extract of the Experts Committee to review the Implementation of Recommendations of Rashtriya Barh Ayog (National Flood Commission), MoWR, Govt. of India, CWC (March 2003) is relevant in this regard. "While visiting the Rengali Dam (Oct.6th, 2002), the committee observed there also that the originally envisaged flood control benefit had been diluted to give more importance to power. It was apprehended by the Committee that after creation of full irrigation potential, flood control aspect would get further diluted. Cautioning against such trend, the Committee suggested to the project authorities to check this and restore the flood control aspect, as was originally planned and approved. **Need to review the reservoir operation manual in the light of experience of last 16-17 years** was stressed by the Committee (Pg.8) x x x x Further "The Chairman of the Committee, cautioned against such dilution and suggested to the project Chief engineer & Basin Manager, Brahmani left basin that all efforts should be made to protect the flood control aspect of the dam project. The Rengali Dam Project is the only dam project planned and executed after independence, **which has earmarked flood reserve**. It has received central funding for flood control too. Therefore, the CWC should also take note of this matter and initiate corrective steps" (Pg.83).

Same is the case with Hirakud where the Flood Storage capacity of the reservoir has been reduced from 100% in 1955 to 67.4% - 55.4% in 1988 (Source : Table No.6.16, Pg.343 - Hirakud : Its Background and Performance by G. C. Sahu & G. N. Das, INCID, MoWR, Govt. of India, 2012). This view was corroborated by the chairman of the above Expert Committee on Oct.5th, 2002. "The project (Hirakud) was conceived and approved mainly for flood control. At the time, the project was built, the priority of flood control was kept above those of irrigation and power. But gradually the priority of flood came down and it soon became secondary to irrigation and power generation."

The rule curve as shown in Table 6.9 does not give any release guideline which need to be used in bringing down the reservoir within the rule curves during floods. These release guide lines could be based not only on the reservoir level but also on the forecast of inflows and floods of intermediate catchments upto Jenapur.

A more complete operation mannual, including the release guide line should be prepared after studies of routing the historical and hypothetical floods as per the trial release guideline.

It was suggested by DSRP during their inspection in 2004 to establish a comprehensive inflow flood forecasting system on the pattern of Hirakud.

Regarding establishing some H.F. Wireless stations in the upstream and downstream catchment area of Rengali dam, the Government in Department of Water Resources (DoWR) vide their Ir. no.9529 dt.24.04.95 had communicated their approval for installation of wireless stations at Panposh, Baneigarh, Deogarh, Talcher, Jenapur, Rengali dam site, Samal barrage site and Palkote (Bihar - now Jharkhand). By now, wireless stations at Rengali dam site, Samal barrage site have already been installed and one such station has also been installed in the office of the E.I.C (WR), Odisha for ensuring proper communication and co-ordination during flood control and flood management.

6.2.4.2 Rengali reservoir Operation from October, 94 to June, 95.

As per the proceedings of the Ist Co-ordination meeting the reservoir levels to be maintained on different dates during October, 94 to June, 95 are given below. In the following table the minimum down-stream demand of Rengali Dam was taken as 7.0 M cum per day which corresponds to 81 Cumec. It was suggested in the above proceedings that the Project Level Monitoring Committee (PLMC) would meet and discuss from time to time regarding the action to be taken to meet exigencies arising out of abnormal power situation and down-stream water demands. Accordingly, the PLMC had met on 29.12.94 and as per their recommendations the C.E., Rengali Irrigation Project in his letter No.1483/WE dated 17.1.96 addressed to Government, Department of Water Resources had suggested to maintain the reservoir levels as follows on the dates mentioned below from October, 1994 to June 1995. Against the above recommendations the actuals are also furnished below.

Table No.6.10 Actual reservoir level from Oct.94 to June.95.

Date	As per proceedings of Ist Co-ordination (RL in m)	As suggested by Project Level Monitoring (RL in m)	As achieved actually (RL in m)
Ist Oct.94	RL 120.20	RL 120.10	RL 123.98
Ist Nov.94	RL 119.15	RL 119.38	RL 123.64
Ist Dec.94	RL 118.15	RL 118.60	RL 122.47
Ist Jan.95	RL 112.60	RL 117.71	RL 120.90
Ist Feb.95	RL 112.10	RL 116.82	RL 119.32
Ist Mar.95	RL 111.60	RL 115.88	RL 117.79
Ist April 95	RL 111.05	RL 114.56	RL 115.39
Ist May 95	RL 110.55	RL 113.22	RL 112.74
Ist June 95	RL 110.00	RL 111.91	RL 110.88
15th June 95	RL 109.72	RL 109.72	RL 109.74

6.2.4.3 Present Operating System :

“The hourly river gauge of river Brahmani at Panposh (u/s) and Jenapur (d/s) are collocated at Rengali dam through CWC stationed at Dam Site. The corresponding discharge at Panposh and Jenapur are calculated from stage-discharge curve. The travel time between Panposh to Dam site and Dam site to Jenapur are 18 to 24 hours and 24 to 36 hours respectively depending on the flood of river. The reservoir level is maintained basing on the Rule-curve, gauge level at Panposh and Jenapur. In emergency cases, the rainfall data of previous day are collected over phone from different Block Headquarters i.e. Pallahara, Bonei, Koira, Lahunipada, Gurundia, Lathikata, Rourkela, Bisra, Nuagaon, Kuarmunda either from Block Headquarters or from Emergency Cell, Sundargarh/Sub- Collector Panposh. (Extract from 3rd meeting of Technical committee held on 31.07.2006). Limiting discharge at Jenapur is 8500 Cumec corresponding to gauge of 23.0 m.

Elevation, Area and capacity of Samal barrage reservoir is enclosed vide Table No.6.11. Stage discharge data of Panposh, Samal, Talcher and Jenapur are enclosed as Table 6.12, 6.13, 6.14 and 6.15 respectively. Yearwise highest gauge at Jenapur and discharge at Pankapal is depicted in Table 6.16.

Table 6.11 Elevation, Area & Capacity of Samal Barrage

Elevation (m)	Capacity (Mm³)	Area (Ha)
70	48.945	1162.5
71.0	61.018	1251.5
72.0	74.067	1358.4
73.0	89.140	1664.4
74.0	106.536	1819.8
75.0	126.396	2158.6
75.2	130.000	2200.0
75.4	136.000	2350.0
75.5 (DSL)	138.750	2380.0
75.6	140.000	2420.0
75.8	145.000	2700.0
76.0	151.408	2864.5
76.2 (FRL)	158.850	2880.0
76.4	161.000	2960.0
76.6	167.000	3050.0
76.8	177.000	3160.0
77.00	197.927	3373.5

(Source : Flood Management Manual, DoWR, Govt. of Odisha, 2008, Pg.47)

Table 6.12 Stage Discharge data at Panposh

RL in M	Discharge in Cumec	RL in M	Discharge in Cumec
173.50	1100.00	176.50	4000.00
173.60	1180.00	176.60	4220.00
173.70	1240.00	176.70	4250.00
173.80	1320.00	176.80	4380.00
173.90	1390.00	176.90	4510.00
174.00	1460.00	177.00	4660.00
174.10	1540.00	177.10	4820.00
174.20	1620.00	177.20	5000.00
174.30	1720.00	177.30	5180.00
174.40	1800.00	177.40	5340.00
174.50	1900.00	177.50	5500.00
174.60	2000.00	177.60	5700.00
174.70	2100.00	177.70	5880.00
174.80	2200.00	177.80	6080.00
174.90	2290.00	177.90	6260.00
175.00	2380.00	178.00	6460.00
175.10	2460.00	178.10	6640.00
175.20	2560.00	178.20	6820.00
175.30	2660.00	178.30	7020.00
175.40	2780.00	178.40	7200.00
175.50	2880.00	178.50	7380.00
175.60	2980.00	178.60	7560.00
175.70	3080.00	178.70	7740.00
175.80	3200.00	178.80	7900.00
175.90	3320.00	178.90	8080.00
176.00	3430.00	179.00	8240.00
176.10	3540.00	179.10	8400.00
176.20	3640.00	179.20	8600.00
176.30	3780.00	179.30	8760.00
176.40	3900.00		

Table 6.13 Stage Discharge data at Samal Site

R.L.(m)	Discharge (Cumec)	R.L.(m)	Discharge (Cumec)
68.00	1250	70.10	4200
68.10	1350	70.20	4400
68.20	1450	70.30	4550
68.30	1600	70.40	4750
68.40	1700	70.50	4900
68.50	1850	70.60	5050
68.60	1950	70.70	5250
68.70	2075	70.80	5,500
68.80	2200	70.90	5,650
68.90	2350	71.00	5,850
69.00	3425	71.10	6,050
69.10	2600	71.20	6,300
69.20	2750	71.30	6,500
69.30	2900	71.40	6,750
69.40	3050	71.50	6,950
69.50	3250	71.60	7,200
69.60	3400	71.70	7,450
69.70	3550	71.80	7,700
69.80	3700	71.90	7,950
69.90	3850	72.00	8,300
70.00	4050	72.10	8,550

Table 6.14 Stage Discharge Data at Talcher**Danger Level. 62.91 M**

R.L. in (m)	Discharge in cumec	R.L. in (m)	Discharge in cumec
56.60	940.00	57.90	2340.00
56.70	1020.00	58.00	2490.00
56.80	1100.00	58.10	2630.00
56.90	1200.00	58.20	2800.00
57.00	1300.00	58.30	2960.00
57.10	1400.00	58.40	3130.00
57.20	1500.00	58.50	3300.00
57.30	1600.00	58.60	3420.00
57.40	1720.00	58.70	3650.00
57.50	1840.00	58.80	3800.00
57.60	1950.00	58.90	4000.00
57.70	2060.00	59.00	4180.00
57.80	2200.00	59.10	4380.00

Contd..

R.L. in (m)	Discharge in cumec	R.L. in (m)	Discharge in cumec
59.20	4560.00	60.40	6780.00
59.30	4720.00	60.50	6960.00
59.40	4900.00	60.60	7120.00
59.50	5100.00	60.70	7310.00
59.60	5280.00	60.80	7500.00
59.70	5440.00	60.90	7680.00
59.80	5640.00	61.00	7900.00
59.90	5820.00	61.10	8100.00
60.00	6040.00	61.20	8300.00
60.10	6200.00	61.30	8550.00
60.20	6400.00	61.40	8800.00
60.30	6560.00	61.50	9100.00

Table 6.15 Stage Discharge Data of Jenapur

RL in (m)	Discharge in Cumec	RL in (m)	Discharge in Cumec
19.80 M	1080.00	21.60	3700.00
19.90	1180.00	21.70	3900.00
20.00	1280.00	21.80	4140.00
20.10	1420.00	21.90	4340.00
20.20	1540.00	22.00	4600.00
20.30	1690.00	22.10	4920.00
20.40	1820.00	22.20	5260.00
20.50	1940.00	22.30	5560.00
20.60	2080.00	22.40	5900.00
20.70	2220.00	22.50	6300.00
20.80	2360.00	22.60	6600.00
20.90	2480.00	22.70	6980.00
21.00	2620.00	22.80	7260.00
21.10	2800.00	22.90	8000.00
21.20	2960.00	23.00	8500.00
21.30	3140.00	23.10	9000.00
21.40	3320.00	23.20	9500.00
21.50	3500.00	23.30	10000.00

Table 6.16 Yearwise Highest gauge at Jenapur & Discharge at Pankapal

Danger Level at Pankapal = 23.00 M (75.4 ft.)

Zero value = 0

Danger Level at Jenapur = 67.00 ft. (20.30 m.)

Zero value = -0.89 Ft

Sill level of Anicut = 15.12 m (49.61 ft.)

Present NSL = 58.8 ft.

Distance between Pankapal and Jenapur = 7.7 kms.

Flood Slope = $67 + (-) 0.89 = 66.11$ ft. = 20.15 mtr; $23-20.15/7.7 = 0.37/\text{km} = 1 : 2700$

YEAR	Date of occurrence	Pankapal (Undivided Brahmani)				Jenapur (Brahmani Arm)	
		Gauge (M)	Gauge (Ft.)	Discharge (Cumec)	Discharge (Cusec)	Gauge (ft.)	Gauge (m.)
1975	20.8.75	24.75	81.20	24,246.00	856240	75.20	22.92
AFTER CONSTRUCTION OF RENGALI DAM							
1984	18.8.84	23.68	77.69	8924.85	315178	69.70	21.24
1985	29.8.85	22.86	75.00	8587.35	303260	67.40	20.54
1986	22.7.86	22.62	74.21	8127.56	287022	66.70	20.33
1987	31.8.87	21.85	71.69	4575.55	161584	64.00	19.51
1988	4.8.88	22.62	74.21	7234.84	255496	67.00	20.42
1989	28.7.89	21.66	71.06	4582.39	161826	64.80	19.75
1990	15.10.90	21.98	72.11	5488.54	193826	64.50	19.66
1991	13.8.91	23.47	77.00	11809.45	417047	69.90	21.31
1992	29.7.92	22.19	72.80	5524.66	195102	65.30	19.90
1993	16.7.93	21.11	69.26	3644.68	128711	62.40	19.02
1994	19.9.94	23.22	76.18	8550.43	301956	68.50	20.88
1995	21.9.95	21.70	71.19	5165.90	182432	63.90	19.48
1996	23.6.96	22.07	72.41	6036.12	213164	64.60	19.69
1997	6.8.97	22.77	74.70	7370.20	260276	67.20	20.48
1998	14.9.98	22.34	73.29	6422.22	226799	66.00	20.12
1999	30.10.99	22.80	74.80	8558.74	302249	68.10	20.76
2000	19.8.00	21.55	70.70	4584.34	161895	63.70	19.42
2001	25.7.01	23.56	77.30	13571.25	479265	70.20	21.40
2002	9.9.02	20.33	66.70	2133.05	75328	60.90	18.56
2003	11.10.03	22.46	73.69	6995.45	247042	66.00	20.12
2004	23.8.04	21.67	71.10	5115.78	180662	64.20	19.57
2005	31.7.05	23.26	76.31	10677.15	377060	69.90	21.31
2006	24.8.06	23.36	76.64	11342.16	400545	70.50	21.49
2007	28.9.07	22.70	74.48	7852.03	277292	67.20	20.48
2008	19.9.08	22.67	74.38	6497.35	229452	67.70	20.63
2009	22.7.09	22.44	73.62	5917.70	208982	66.30	20.21
2010	—	19.24	63.12	852.00	30088	Nil	Nil
2011	26.9.11	23.88	78.35	12880.50	454871	72.10	21.98

(Source : Executive Engineer, Jaraka Irrigation Division)

The catchment area lies in State Jharkhand (15700 sq.km.), Chhatisgarh (900 sq.km) and Odisha (8050 sq.km.). The rainfall data of Jharkhand and Chhatisgarh are not available. Due to non-availability of rainfall data from neighbouring states, it is not possible to assess the discharge of water coming from those states.

The 24 hour rainfall of Panposh is being collected through CWC. Besides, following Rain gauge stations are also maintained by CWC.

River	Gauge Location	Danger Level (m)	Highest recorded	
			Level (m)	Year
Brahmani	Panposh	178.42	180.26	1977(*)
	Rengali	88.00	92.55	(1975)
	Talcher	62.91	65.53	(1975)
	Jenapur	23.00	24.78	(1975)

(*) Highest level reached 181.40 m in the year 2011.

Establishment of rainfall stations/gauge stations in Chhatisgarh, Jharkhand, Bonai, Barkote and Palalahara are essential.

The river Mankara is entering the reservoir very close to Dam. As the river Mankara is flowing mostly in hilly region, the discharge is abrupt; but its contribution has negligible impact.

During project finalisation, CWC had suggested a rule curve for Reservoir Operation. Presently the reservoir is being operated as per suggestion of the 2nd co-ordination Committee held on 24.07.1995. As regards safety of the dam is concerned, the dam is safe against P.M.F. But due to flooding in the downstream, judicious decision is to be taken while opening the gates, keeping in view the limiting discharge at Jenapur (i.e. head of Brahmani delta) to 8500 cumec (3.0 lakh cusec) corresponding to gauge of 23.0 m.

For flood forecasting a mathematical model needs to be developed. Major portion of the catchment lies in Jharkhand and a small area in Chhatisgarh. The rainfall and run off data from those catchments are to be obtained either contacting the Water Resources Departments of concerned states or through C.W.C. for developing the unit hydrograph.

For Rengali reservoir operation, DoWR has given higher priorities to irrigation and flood control, but not for the hydro power generation, though DoWR is planning to charge water rate for hydro power generation. The Rengali dam has a provision in Block 43 for releasing irrigation water even below D.S.L. No environmental flow has been released to downstream yet, though it was initially contemplated as 80 m³/sec at the minimum. Average monthly water levels of Rengali reservoir, Inflow and spill as recorded by the project are furnished since 1988 vide Table No. 6.16(a) . An average annual inflow was 13,003 Mcum, average spill out flow was 4,087 Mcum or 31% of the total inflow. Dependable annual inflow with 75% reliability is estimated at 9,689 Mcum. The amount of average inflow to the reservoir vary from 11 Mcum/month in May to 4,219 Mcum/month in August in a year, and from 8 Mcum/month in May to 3,144 Mcum/month in August in 75% dependable year as shown in Table 6.17.

Record of water level from 1988 to 2008 and storage capacity curve of the Rangali reservoir are given in Tables 6.16(a) and 6.11. Table 6.16(a) reveals that water level in May to July was the lowest and full water level in September to November.

According to the Samal Barrage office, neither operation and maintenance manual nor operation rule curve has been prepared as yet. Principle of operation is just to maintain the water level for supplying water to the irrigation canals. One mini-hydro power generation plant with installed capacity of 20MW (4 MW x 5 units) was constructed on the left bank of the barrage. The plant was constructed by

Monthly Available Water at Rengali Dam and Expected Demands including Domestic and Industrial Purposes (Projected)													(Unit Mcum)
Item	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
Supply													
Inflow at Rengali dam; Q75% (CA = 25,250 sq-km)	*1a	76	42	21	9	8	595	2,351	3,144	2,312	222	99	9,689
Available flow at Samal; Q75% (CA = 30030 sq-km)	*1b	91	50	25	10	10	708	2,796	3,739	2,749	264	118	11,523
Demands													
Irrigation demand (LB) (CCA = 114,300 ha)	*2	185	215	173	104	33	285	173	113	106	59	148	1,662
Irrigation demand (RB) (CCA = 121,200 ha)	*3	196	228	183	111	35	302	184	120	112	62	156	1,762
Irrigation demand beyond Baitarni river (supplemental supply for CCA = 40,000 ha)	*4	67	60	67	65	67	65	67	67	65	65	67	778
Sub-total													
(Total of irrigation demand)		447	504	423	280	135	653	424	301	283	186	371	4,213
Ecology conservation d/s Samal B (river maintenance flow)	*5	32	32	32	32	32	32	32	32	32	32	32	384
Domestic demands (11 towns)	*6	2	2	2	2	2	2	2	2	2	2	2	24
Industrial demands (existing + future)	*7	122	122	122	122	122	122	122	122	122	122	122	1,464
Total Demand (future)		603	660	579	436	291	809	580	457	439	342	527	6,085

The figures mention are tentative and should be studied in detail by DoWR

Note :

*1a : Whole catchment area of three (3) states. It is temporally assumed that whole uses in other two states within the catchment would be returned to the catchment.

*1b : Because of no record of inflow and outflow at samal barrage after the completion, the inflow is estimated by proportion of area between Rengali Dam and the barrage.

*2 : Table 24 (B) or page 4-84, Final Report Rengali Irrigation Sub Project LBC-II, Volume-1, WAPCOS, March 1997

*3 : Table 24 (C) or page 4-85, Final Report Rengali Irrigation Sub Project LBC-II, Volume-1, WAPCOS, March 1997

*4 : Assuming 25 m³/s would be provided through Left bank canal as designed; to be adjusted later. Beyond Baitarani has been deferred.

*5 : Assuming average dry season flow (Dec-May) with 20 years probable drought of the river at Rengali dam site; Spilt out flow during flood season is estimated at 2,319 MCM at the 75% drought year, total of river flow is 2,703 MCM, which is 28% of total inflow to the Rengali reservoir at 75% drought condition.

(According to the ADB's report on OIIAWMP, Orissa State Master Plan (2004) of MoWR assumed 30% for environmental demand)

*6 : Recent discussion with officers DoWR, Towns are : Angul, Nalco, TTPS, Dera Coliery, FCI, Ghatapada, Talcher, Rengali dam, Dhenkanal, Bhuban and Kamakhyanagar.

*7 : Recent discussion with officers; demands consists of existing industry of 513 MCM, existing mines of 13 MCM, and proposed industries of 934 MCM.

Odisha Power Consortium Limited and is operational since 2009. It is expected that required downstream demand including ecological demand would be released through this plant. The maximum discharge through the plant is estimated at 260 Cumec.

Spillway Rating of the Rengali Reservoir

Reservoir Level (in mtr.)	Spilway Discharge per one gate (in cumec)	Reservoir Level (in mtr.)	Spilway Discharge per one gate (in cumec)
109.72 (DSL)		118.00	690
110.00		119.00	830
110.30	3	120.00	990
111.00	21	121.00	1,165
112.00	70	122.00	1,355
113.00	130	123.00	1,550
114.00	210	123.50 (FSL)	1,650
115.00	308	124.00	1,770
116.00	430	125.00	2,025
117.00	550	125.40 (MWL)	

Source : Hydrology Data (Final) Rengali Reservoir (1988-2008),

Spilway gates : 24 nos. of radial type gates (15.5 m x 14.8 m)

6.3 Flood Control

The safe flood discharge for Brahmani delta has been considered as 8500 cumec (3.00 lakh cusec) at Jenapur Railway Bridge. The flood control scheme was planned such that the regulated discharge from the reservoir combined with the downstream discharge remains within 8500 cumec (3 lakh cusec) at Jenapur Railway Bridge. The F.R.L. of the reservoir is at R.L. 123.50m and the D.S.L. at R.L. 109.72m (360 ft.).

It was expected that the storage reservoir at Rengali will be able to moderate almost all floods to 8500 cumec (3 lakh cusec) at the head of the delta (Railway Bridge at Jenapur). However, a detailed study of all past floods during the period 1875 to 1971 reveals that the contribution of the catchment downstream of Rengali Dam may sometimes be above 8500 cumec (3 lac cusec). This has occurred thrice during last 100 years prior to construction of Rengali Dam. In the year 1943 the downstream contribution was 9380 cumec (3.35 lac cusec), in 1971 the downstream contribution was 15616 cumec (5.72 lac cusec) at Jenapur and in 1975 (20.8.75), the flood discharge at Pankapal (i.e. undivided Brahmani) was 24246 cumec (856240 cusec) with gauge level at Jenapur (Brahmani arm) 22.92m (75.20 ft.) against danger level of 20.30m (67.00 ft.) These floods were prior to construction of Rengali Dam. After construction of Rengali Dam the highest observed flood at Pankapal was in the year 2011 (26.9.11). It was 12880 cumec (454871 cusec) which is 53% of 1975 flood magnitude. Highest gauge and discharge values at Pankapal and gauge at Jenapur (Brahmani arm) have been furnished from 1984 to 2011 vide Table 6.16. Besides, inflow to the Rengali reservoir, spill from the reservoir and average monthly water levels are given vide Table No. 6.17, 6.18 and 6.16(a) respectively.

Table 6.17 Recorded Inflow to the Rengali Reservoir**(Unit : Mcum)**

CA=25,250 sq-km

No.	Year	Jan	Feb	Mar	Apr	May	Jun	July	Aug	Sep	Oct	Nov	Dec	Total
1	1988	118	98	224	20	15	1,693	3,731	4,908	1,853	420	137	83	13,300
2	1989	55	33	30	10	6	1,562	1,782	3,333	1,801	685	108	86	9,491
3	1990	85	0	0	3	112	561	3,948	3,058	2,997	2,206	565	200	13,736
4	1991	193	51	28	18	0	300	2,451	6,389	4,734	682	243	128	15,216
5	1992	175	62	35	11	33	62	1,373	2,822	1,515	239	29	0	6,357
6	1993	25	0	4	10	0	876	2,545	1,981	3,679	1,517	325	127	11,090
7	1994	100	56	4	27	0	2,507	8,327	8,291	4,701	985	293	146	25,438
8	1995	195	64	15	0	17	169	2,149	3,354	3,175	768	657	142	10,704
9	1996	145	50	49	0	0	2,086	4,221	5,474	1,949	571	113	67	14,724
10	1997	44	32	0	0	0	609	2,601	5,928	3,767	658	418	464	14,521
11	1998	578	409	130	142	50	147	1,553	2,042	4,646	1,705	896	171	12,468
12	1999	89	57	39	0	3	702	2,605	5,295	5,468	2,184	407	145	16,993
13	2000	84	108	24	0	0	426	3,137	1,836	2,462	385	72	19	8,554
14	2001	0	0	0	0	0	805	9,645	4,647	2,748	783	149	482	19,259
15	2002	44	13	0	0	0	740	619	2,476	3,439	593	114	22	8,060
16	2003	0	6	3	4	0	267	1,413	2,855	3,816	3,643	820	251	13,078
17	2004	108	31	0	0	0	53	713	4,686	1,910	1,122	86	57	8,765
18	2005	41	33	10	0	0	1,010	3,236	2,675	1,343	831	158	53	9,392
19	2006	0	0	0	0	0	247	2,333	5,896	2,060	614	116	26	11,293
20	2007	9	61	0	0	0	109	2,948	6,808	4,384	1,667	428	75	16,488
21	2008	58	30	0	0	0	1,837	4,939	3,853	2,696	559	116	49	14,136
	Max	578	409	224	142	112	2,507	9,645	8,291	5,468	3,643	896	482	25,438
	Min	0	0	0	0	0	53	619	1,836	1,343	239	29	0	6,357
	Average	102	57	28	12	11	798	3,156	4,219	3,102	1,087	298	133	13,003
	(%)	0.8%	0.4%	0.2%	0.1%	0.1%	6.1%	24.3%	32.4%	23.9%	8.4%	2.3%	1.0%	100.0%
	75% dependable inflow	76	42	21	9	8	595	2,351	3,144	2,312	810	222	99	9,689

Source :Hydrology data of Rengali Dam (Daily record for water level, discharge records of power channel and spilway), MoWR (1988 to 2008)

In flow is estimated by change of volume plus total amount released and spilled out. In case calculated inflow is in minus, the value is treated as zero, considering some error occurred for discharge measurement for release and spillout.

Monthly inflow is estimated based on annual V_{75} and average distribution.

Table 6.18 Recorded Spill-Out Discharge from the Rengali Reservoir

CA = 25,250 sq-km

(Unit : Mcum)

No.	Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
1	1988						0	2,976	2,955	554	0	0		6,475
2	1989						0	1,243	1,495	0	0	0		2,737
3	1990						0	1,344	1,007	187	1,038	0		3,576
4	1991						0	0	3,660	3,234	0	0		6,894
5	1992						0	0	0	0	0	0		0
6	1993						0	358	0	216	237	0		811
7	1994						0	5,601	6,323	3,231	0	0		15,155
8	1995						0	0	0	881	0	0		881
9	1996						860	1,396	3,126	395	0	0		5,777
10	1997						0	579	2,447	1,775	0	0		4,801
11	1998						0	0	0	1,131	274	182		1,587
12	1999						0	40	2,265	3,804	1,027	0		7,137
13	2000						0	786	15	0	0	0		801
14	2001						0	6,206	2,298	1,294	0	0		9,797
15	2002						31	34	0	0	0	0		65
16	2003						0	0	0	714	1,671	0		2,385
17	2004						0	0	486	0	0	0		486
18	2005						0	1,376	384	0	0	0		1,760
19	2006						0	0	3,492	368	0	0		3,860
20	2007						0	648	2,970	2,621	594	0		6,833
21	2008						153	2,102	961	787	0	0		4,004
	Max						860	6,206	6,323	3,804	1,671	182		15,155
	Min						0	0	0	0	0	0		0
Average							50	1,176	1,614	1,009	231	9		4,087

Source Hydrology Data of Rengali Dam (Daily record for water level, discharge records of power channel and spillway). MoWR (1988 to 2008).

Table 6.16(a) Recorded Average Monthly Water Levels of Rengali Reservoir (El. in m)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1988	120.3	118.0	115.4	112.6	110.4	113.0	116.7	119.3	122.8	123.3	122.1	120.5
1989	118.8	116.8	114.6	112.8	111.0	113.1	117.1	119.0	122.2	123.3	122.2	120.5
1990	117.9	115.7	114.4	113.1	111.6	110.2	114.3	118.5	121.5	123.5	122.5	120.3
1991	117.2	114.7	113.5	112.6	111.4	109.8	113.2	120.0	122.7	122.5	120.5	118.7
1992	116.6	114.2	112.6	111.2	110.3	109.6	111.8	117.5	121.2	119.2	116.5	114.2
1993	111.9	110.5	110.1	109.8	109.5	111.2	116.0	118.2	121.4	123.8	122.8	120.8
1994	118.3	116.0	112.9	110.4	109.3	113.2	120.5	122.6	123.7	123.8	123.0	121.7
1995	120.1	118.6	116.6	114.1	111.9	110.1	113.1	118.8	122.5	123.0	122.8	122.8
1996	122.1	121.0	119.0	116.2	112.9	113.6	116.6	120.3	122.6	123.0	122.4	121.1
1997	119.7	118.5	117.4	115.9	113.7	110.8	114.6	119.7	123.1	123.3	122.6	122.2
1998	122.1	121.9	119.7	117.4	114.9	111.3	110.9	112.9	119.1	123.5	123.2	121.7
1999	120.1	118.7	117.3	115.5	112.7	111.0	114.1	119.7	122.9	123.6	123.2	122.5
2000	121.5	120.5	118.7	116.0	112.6	110.2	112.9	116.6	117.4	118.4	117.2	116.3
2001	115.3	114.2	113.3	112.3	111.0	111.0	118.4	120.6	122.9	123.0	122.4	121.3
2002	120.3	119.3	117.3	114.4	111.9	111.5	111.0	114.2	120.4	122.8	122.5	122.0
2003	121.4	120.7	119.5	117.3	114.4	111.4	111.3	116.2	121.4	123.9	123.4	121.3
2004	119.1	118.0	117.0	115.7	113.5	110.9	109.9	115.6	120.8	121.7	121.1	119.9
2005	119.0	118.1	116.8	114.6	112.3	112.3	115.5	118.4	118.9	120.6	121.2	120.4
2006	119.4	118.2	117.1	115.1	112.6	110.5	113.8	120.5	123.2	123.0	122.0	121.2
2007	120.4	119.8	119.1	117.1	115.2	112.8	112.5	119.2	123.4	122.4	120.3	118.8
2008	116.7	114.8	113.7	112.3	110.8	112.1	116.0	-	-	-	-	-
Max.	122.1	121.9	119.7	117.4	115.2	113.6	120.5	122.6	123.7	123.9	123.4	122.8
Min.	111.9	110.5	110.1	109.8	109.3	109.6	109.9	112.9	117.4	118.4	116.5	114.2
Average	119.0	117.5	116.0	114.1	112.1	111.4	114.3	118.4	121.7	122.6	121.7	120.4

Source : Average WL of Max and Min of the month, Hydrology Data (Final) Rengali Reservoir (1998-2008)

Rengali office of MoWR

Regarding flood control, views of the ‘Experts Committee to review the Implementations of, Recommendations of Rashtriya Barh Ayog’ (March 2003) is relevant here, the extract of which has been quoted under Sec.6.2.4.1 of the book.

6.4 Power Generation :

6.4.1 Introduction :

“The hydro-electric projects, have inherent ability for quick starting & stopping and can accept as well as reject load almost instantaneously. Hence hydel projects are ideally suited for meeting the peaking demand and for enhancing system reliability in the most economic manner. The operation of hydro projects is environmental friendly and does not pose adverse impact unlike thermal projects which have the associated problems of emissions and solid waste disposal. The hydro projects have longer useful life spans. These projects are generally located in the remote hilly and inaccessible areas

and their implementation enables accrual of incidental benefits of development of road/rail communications, electrification, industrialisation and improvement of the quality of life in backward areas. Hydro power is also the cheapest amongst the various sources of power supply in the long run since it has no fuel cost component as well as almost free from operating cost escalations. It is renewable in nature and promotes conservation of nonrenewable fossil fuels. Further, hydro power generation by constructing suitable storage reservoir projects substantially helps in the utilisation of usable water resources, 75% of which in India is presently draining down to the sea unutilised.

Notwithstanding its inherent benefits and availability of vast potential in India, the pace of hydro development has so far been slow. The major constraints for slow development of hydro potential are difficulties in investigations, R&R problems, long gestation period, delay in land acquisition, funds constraints and geological surprises” x x x

“The main reasons for slow development of hydropower are :

- a) Shortage of Funds
- b) Resettlement & Rehabilitation Problem
- c) Dearth of experienced contractors
- d) Inter-State Aspects
- e) Delays in Environmental and Forest Clearance
- f) Slow Development in N.E. Region having largest hydro potential
- g) Law and Order Problem
- h) Land Acquisition Problem
- i) Geological Surprises

Among these aspects the problems which are emerging to be major constraints are R & R and other environmental aspects, geological problem and fund constraints. It is strongly felt that most of the geological surprises can be avoided through systematic and adequate investigations.”

Source : Need for accelerated Development of Hydropower in India, K. N. Sinha & R. S. Goel - ‘Hydropower and River Valley Development, post conference Proceedings & Recommendations’, 2000 Pg.64-65).

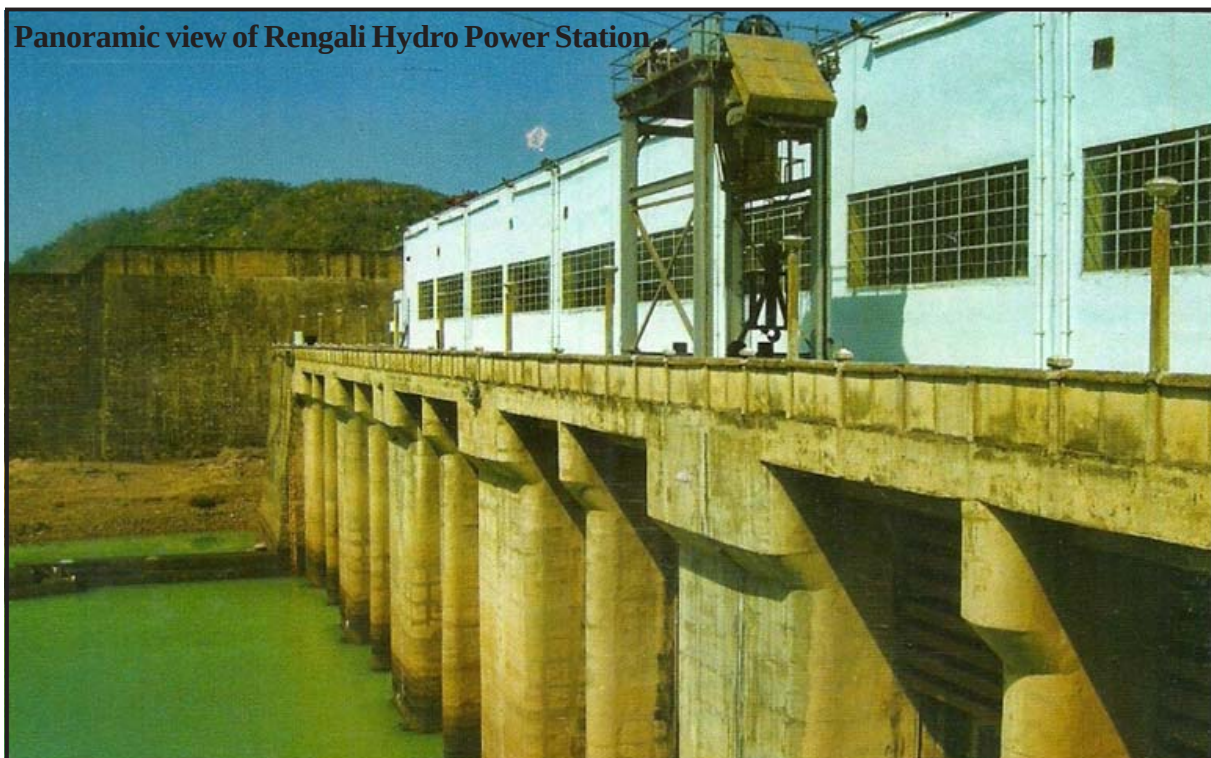


Table No.6.19 Monthwise drawal of Water for Power Generation (in THM)

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JULY	AUG	SEP	OCT	NOV	DEC	TOTAL
1985-86													190.786
1986-87													592.723
1987-88													723.615
1988				50.449	28.2147	31.4294	57.313	66.565	61.117	64.0567	59.9606	62.6006	481.706
1989	56.3778	55.6137	44.91	30.6621	28.0871	29.297	39.613	78.048	104.518	88.374	54.868	74.489	684.8577
1990	86.052	37.087	25.705	25.118	36.918	65.461	97.2735	157.32	146.1471	137.4265	96.7814	109.7146	1021.0041
1991	102.3465	38.6628	19.3365	17.4469	24.5769	38.3119	55.9511	112.1731	126.3818	128.0101	85.4895	63.4673	812.1544
1992	82.8446	42.6843	29.012	17.819	14.088	13.051	58.608	140.96	133.2255	155.5673	54.9958	50.507	753.3625
1993	36.661	7.9508	4.4019	4.7411	4.1833	23.735	121.7277	170.078	176.685	140.412	81.205	90.606	862.4168
1994	75.263	58.4	70.007	18.432	11.2	74.3	182.768	175.2185	107.249	113.626	65.022	64.709	1016.1945
1995	63.947	47.344	61.643	51.233	33.399	27.556	106.097	179.5651	164.632	105.469	45.7898	34.9097	921.5846
1996	39.3837	58.7696	65.9519	72.636	52.4764	59.6329	134.7006	155.947	137.093	46.35	43.0779	56.3165	922.3355
1997	44.9319	26.8642	30.811	33.474	47.4424	59.7456	145.0379	157.7204	166.2979	99.8943	61.261	54.7013	928.1819
1998	31.998	108.2659	89.1545	59.376	73.867	63.036	141.073	131.8767	126.916	114.6978	121.434	63.101	1124.7959
1999	58.322	36.485	43.217	42.154	62.603	54.715	138.984	167.245	131.245	125.814	57.226	40.783	958.793
2000	42.006	42.078	67.6099	68.265	67.3288	60.8049	102.4189	186.4736	167.414	81.9901	28.5622	21.3195	936.2709
2001	27.0503	16.884	16.4098	16.0895	18.3383	53.2546	140.6293	176.2162	146.0913	77.8298	46.8159	43.1296	778.7386
2002	28.0857	36.7605	73.7669	52.4202	33.9295	43.8284	79.8628	115.6109	175.525	68.334	20.2	20.58	748.9039
2003	21.8422	27.4878	49.892	63.977	64.146	77.343	92.6482	142.31	171.782	174.186	130.988	107.835	1124.4372
2004	52.27	25.821	28.254	28.684	55.37	35.1	76.084	169.30	182.295	85.639	57.333	31.186	827.336
2005	31.905	21.1502	52.2687	37.4938	40.7879	26.9408	186.1127	171.6017	117.1782	29.0971	30.922	36.6013	782.0594
2006	31.1423	26.9683	26.5783	52.798	43.0496	45.7492	75.335	124.3702	136.267	97.3655	43.1902	23.896	726.7096
2007	24.6164	17.8123	32.112	65.7731	21.938	83.2989	148.9082	154.5113	163.6615	191.5861	81.1668	53.7906	1039.1752
2008	66.4836	29.3972	17.895	29.5885	13.1382	99.8007	186.7695	179.9126	141.9242	103.2727	63.676	38.3357	970.1939
2009	48.982	34.1536	31.62	45.2834	21.4633	11.4025	113.4363	152.9578	139.1851	49.1233	19.7406	15.9443	683.2922
2010	3.6862	10.3904	43.5135	46.821	54.5927	32.0248	44.9659	41.3642	21.6082	10.0851	7.2723	8.1722	324.4965
2011	14.3841	18.2542	19.2284	32.5839	50.0878	66.1953	151.0586	149.8317	147.2631	107.5762	83.1197	72.6802	912.2632
2012	35.1493	27.214	28.9041	51.4511	58.963	30.6423	79.0244	182.8994	172.7402	73.8289	20.2196	15.8216	776.8579
2013	21.0077	17.2252	56.1104	104.7755	78.6309	40.5126							318.2623

Table No.6.20 Monthwise Power Generation (in Million Units)

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JULY	AUG	SEP	OCT	NOV	DEC	TOTAL
1985-86													200.311
1986-87													568.0506
1987-88													671.3465
1988				40.374	17.5884	24.464	48.87	61.008	62.934	67.04	61.088	61.398	444.7144
1989	52.956	49.68	38.302	25.212	21.586	23.392	35.2	73.136	104.222	91.75	65.932	72.392	643.76
1990	78.34	31.834	21.8126	20.89	29.036	48.334	79.516	141.9	144.33	138.424	97.056	104.676	936.1488
1991	90.290	32.546	15.000	11.648	19.07	35.62	44.22	102.662	122.774	128.844	82.62	59.028	747.266
1992	72.654	35.504	23.388	13.87	10.668	13.438	45.192	128.05	129.528	106.974	48.094	42.028	669.388
1993	28.766	6.194	3.428	3.668	3.204	18.276	104.678	150.224	170.504	145.136	83.062	88.475	805.615
1994	69.096	50.452	55.616	14.54	7.802	61.28	163.198	161.876	110.924	117.524	66.608	63.924	942.84
1995	60.936	44.276	54.288	42.364	26.492	20.804	84.656	163.528	163.708	107.445	47.2725	36.054	851.8235
1996	39.765	57.848	62.01	64.4478	42.8122	47.3219	109.0258	140.056	135.0972	58.3078	44.9654	55.272	856.92908
1997	43.03116	25.008	28.7398	29.9067	40.4807	45.9518	117.169	145.05	162.472	102.604	63.21	55.994	859.61716
1998	32.65	107.407	84.887	53.671	64.375	48.266	104.499	103.978	121.305	118.492	124.315	63.843	1027.688
1999	57.018	34.424	39.339	36.687	61.64	42.465	110.567	150.476	127.092	128.184	59.714	44.077	881.683
2000	42.434	41.578	64.026	60.054	53.928	45.115	79.64	160.051	145.012	75.534	26.188	20.121	813.661
2001	23.959	14.724	14.17	13.54	14.783	40.132	119.042	157.48	145.82	79.696	48.176	44.756	716.278
2002	27.812	36.212	66.636	44.572	27.112	33.74	60.24	98.76	169.832	70.076	21.176	21.452	676.62
2003	22.268	27.32	47.884	58.312	54.376	58.368	69.18	124.164	169.588	175.64	135.252	106.668	1049.02
2004	50.228	24.4	25.924	25.94	46.008	27.128	56.236	147.8744	174.3712	85.9656	57.5496	30.8098	752.4344
2005	30.8496	20.1344	47.2072	32.3648	33.0808	21.1456	150.5784	155.2624	107.716	29.0424	31.5384	36.3136	695.2336
2006	30.3344	25.612	24.4648	45.3912	34.5856	34.2072	61.2552	114.0416	138.4336	100.0064	44.4504	24.3536	677.136
2007	24.6648	17.6272	30.8944	59.1216	19.2688	66.1144	118.1	139.1544	161.384	190.6288	79.0712	50.8248	956.8544
2008	58.948	25.5992	15.2144	24.0056	10.308	76.2864	151.4824	168.344	142.8032	105.4912	63.7336	37.7312	879.9472
2009	45.4224	31.312	27.8952	37.6096	16.8016	8.6832	90.3648	129.0288	131.488	48.1144	19.444	15.7416	601.9056
2010	3.644	10.128	40.58	41.7704	45.1736	24.5888	34.0024	32.3088	19.6392	9.6872	7.0456	7.876	276.444
2011	13.6552	16.9424	17.4632	28.3872	40.2136	53.5368	123.3672	136.5704	139.1184	110.7992	84.6096	71.3008	835.964
2012	33.7864	26.8976	26.724	44.6512	46.2032	22.9624	68.36	161.9184	169.3344	75.7112	21.536	16.848	703.9328
2013	22.096	17.8888	55.6656	95.3984	65.0256	30.58							286.6544

Power development in India commenced during last part of 19th century with commissioning of hydropower plant of 130 KW at Sidrapong in Darjeeling in 1897 followed by steam driven power plant of 1000 KW in 1899 by CESC in Kolkata. In Odisha, the first hydro-electric project was commissioned in 1902 at Deogarh for supplying power to the palace and to some neighbouring areas.

First unit of Hirakud Dam Project at Burla of capacity 24 MW was commissioned on 19.12.1956. Similarly 1st unit of Rengali was commissioned on 29.08.1985, 2nd unit on 16.03.1986, 3rd and 4th units respectively on 10.08.1989 and 19.03.1990. The 5th unit was commissioned on 14.08.1992.

Monthwise drawal of water for Power generation, monthwise power generation and revenue earned due to sale of power are furnished respectively in Table 6.19, 6.20 and 6.21.

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 Odisha), 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 6.22 & Table 6.21 respectively.

Table 6.22 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 No.6.21 Year wise sale of Power in M. unit & Revenue from Rengali Power House.

Year	Rate/unit (in paise/kwh)	Total MU	Revenue earned (Rs. in crore)
1985-86	30	200.311	6.009
1986-87	30	558.0606	16.741
1987-88	30	671.3465	20.140
1988-89	30	585.948	17.578
1989-90	30	634.804	19.044
1990-91	30	942.674	28.280
1991-92	30	740.30	22.209
1992-93	30	572.468	17.174
1993-94	30	942.387	28.272
1994-95	30	927.176	27.815
1995-96	30	851.946	25.558
1996-97	38	784.08	29.327
1997-98	49	987.78	35.800
1998-99	48.11	933.52	43.598
1999-00	49.64	898.94	42.290
2000-01	50.02	718.47	34.265
2001-02	24.99	793.085	19.297
2002-03	24.73	644.43	57.542
2003-04	27.35	1052.10	27.875
2004-05	28.67	750.0736	20.965
2005-06	31.42	677.4536	20.992
2006-07	35.56	669.9112	23.087
2007-08	62.06	983.4298	33.795
2008-09	49.40	884.8152	39.147
2009-10	-	551.6281	29.170
2010-11	-	270.153	
2011-12	-	874.311	
2012-13	-	731.175	

6.5 Irrigation :

6.5.1 Introduction :

“In assessing the water requirement for irrigation purposes, the National Commission for Integrated Water Resources Development (NCIWRD) kept the objective of achieving food security in the country, based on self sufficiency in food production. It felt that food self sufficiency and to some extent export of food and non-food agriculture produce, was essential for the country from both strategic and socio-economic consideration.

As requirement of food production depends upon country's population, per capita income and changes in dietary habits, ratio of urban and rural populations, a reasonable estimate of population growth was assumed. After examining the latest trends and the views expressed by different demographers, NCIWRD decided to follow the estimates of higher and lower limits of India's population in the year 2050 by United Nations as 1581 millions (high) and 1346 million (low).

It also felt that urban and rural populations had different life styles and, therefore, their needs for food and drinking water were markedly different. Even though it recognized that with access to information and demand for rural-urban equality, the gap between urban and rural demand for food was likely to shrink gradually; yet it felt that the differences would persist for a long time to come and therefore, separate estimates for rural and urban demand for food were warranted. After considering the trend of rate of growth of urban population in the last few years, NCIWRD adopted the figure of 646 millions and 971 millions as lower and upper estimates of urban population in the year 2050.

The NCIWRD felt that India's population of 1027 million (2001 census) would stabilize at about 1600 million by 2050. This would require about 450 million tonnes of food grains annually at present level of consumption. Further, considering the conditions of monsoonal climate and drought leading to crop failure, it considered necessary to plan for buffer stock as well. Thus, they felt that to meet the country's demand reasonably well, production of not less than 500 million tonnes of food grains by 2050 should be planned, bulk of which will have to come from irrigated agriculture. As such this demand has to be met by providing irrigation facilities to as much area as possible, with the aim to reach ultimate irrigation potential, assessed as 140 Mha. through all types of irrigation schemes.

The Standing Sub-Committee of MOWR assessed that the food requirements would be of the order of 480 million tonnes in 2050. However, it also felt that there might be about 50% probability of its falling short and recommended that there was a need for being more conservative in futuristic planning of basic requirements and that 550 million tones should be taken as the food requirements for the year 2050.

To achieve food production of the order of 550 million tones, the Sub-Committee assumed that by the year 2050, it would be possible to achieve average productivity of 3.25 tonne/ha (t/ha) for irrigated and 1.5 t/ha for un-irrigated areas, and concluded that it would be necessary to harness the entire irrigation potential of 165 Mha (140+25 (by inter linking) to match the projected food requirements for the year 2050. Similarly, it assumed that the irrigation potential of 140 Mha will have to be developed for meeting the requirements of food in 2025.

The Sub-Committee, thereafter, studied the then available district wise statistical details of the land use in the country during 1992-93, and worked out the basin wise details of culturable area, net sown area, gross sown area, net irrigated area and gross irrigated area. Adopting an average delta of 0.65 m (0.8 m for SW, 0.5 m for GW) it worked out the basin wise irrigation water requirements in the year 1992-93 along with projected requirements for the year 2000, 2010, 2025 and 2050.

The sub-committee also felt that while accelerating the pace of irrigation development it shall also be necessary to improve level of utilization, and to increase irrigation efficiency from the present

level of average 35-40% to the maximum achievable, i.e., around 60%. For this purpose water saving technologies will have to be developed and popularized. There will also be need to increase productivity per unit of land.”

(Source : Water Resources development Scenario in India, CWC, 2012 Pg.51-52)

6.5.2 Irrigation Scenario in the State :

“Irrigation is a crucially important input for enhancing agricultural productivity and is required at different critical stages of plant growth of various crops for their optimum production. The State has cultivable land of 64.09 lakh ha. It has been assessed that 49.90 lakh ha can be brought under irrigation through major, medium and minor (lift and flow) irrigation projects. By the end of 2011-12, about 45.93 lakh ha net irrigation potential has been created and about 67 percent has been utilised. The net irrigation potential created has increased by 9.87 lakh ha from 36.06 lakh ha in 2000-01 to 45.93 lakh ha in 2011-12. Out of 45.93 lakh ha net irrigation potential created by the end of 2011-12, 19.86 lakh ha (43.3%) has been created through major and medium (flow), 6.51 lakh ha (14.2%) through minor (flow), and 8.36 lakh ha (18.2%) through minor (lift) irrigation projects.

“(Source : Economic Survey, Odisha 2012-13 Pg.8.3).

Upto 2008-09, the percentage share of irrigation provided to principal crops in Odisha was 35, while it was 45.3 at all India level. Against this, Punjab has the highest percent of 97.6 followed by Haryana 85.3, Uttar Pradesh 76.4 and Bihar 61.0 (Source : Ibid, Annexure 3.15 Pg.118). Irrigation Potential created and utilised in the State ending 2010-11 is furnished in Table.

Table 6.23 Status of Irrigation Potential Created & Utilised

Year	Irrigation potential created (th. ha)			Irrigation potential utilized (th. ha)*			% of utilisation
	Khariff	Rabi	Total	Khariff	Rabi	Total	
2000-01	2533.83	1071.99	3605.82	1589.88	535.84	2125.72	58.95%
2001-02	2554.26	1117.63	3671.89	1752.27	793.64	2545.91	69.34%
2002-03	2608.59	1123.75	3732.34	1246.81	465.21	1712.02	45.87%
2003-04	2674.12	1161.21	3835.33	1737.49	780.87	2518.36	65.66%
2004-05	2707.27	1266.22	3973.49	1845.79	844.87	2690.66	67.72%
2005-06	2731.50	1294.92	4026.42	1922.70	1042.79	2965.49	73.65%
2006-07	2720.46	1318.52	4038.98	2001.98	1147.47	3149.45	77.98%
2007-08	2765.73	1342.06	4107.79	2027.00	1281.46	3308.46	80.54%
2008-09	2867.01	1407.18	4274.19	2081.13	1096.03	3177.16	74.33%
2009-10	2962.21	1476.81	4439.02	2058.85	979.67	3038.52	68.45%
2010-11	3035.85	1477.97	4513.82	2085.21	1020.70	3105.92	68.81%

* Odisha Agriculture Statistics

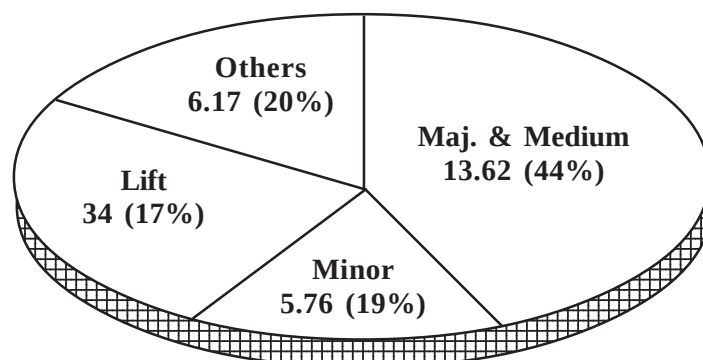
(Source : Annual Report 2011-12, DoWR, Govt. of Odisha Pg.22)

The state has a cultivable land of 64.09 lakh hectares. it has been assessed that 49.90 lakh hectares can be brought under irrigation through major, medium and minor (flow & lift) irrigation projects.

Irrigation development has not made much headway in the state in the pre-independence era. Hardly 1.83 lakh hectares of net irrigation potential was created. After introduction of Five Year Plan

by Govt. of India in 1951, attempts were made for rapid harnessing of water resources and much emphasis was laid to accelerate the irrigation potential creation. Many major, medium and minor irrigation projects have been constructed in the state during last six decades, there by increasing net irrigation potential from 1.83 lakh hectares in 1951 to 30.89 lakh hectares in 2012. The irrigation potential created through different sources is given below.

Net Irrigation potential created as on March, 2011 (lakh ha)



Compulsory basic water rate for khariff and Rabi crops is given in Table 6.24 and 6.25 and rate for other items of work (i.e. Non-irrigation) is shown in Table 6.26.

Table 6.24 Compulsoty Basic Water Rate (Khariff crop)

Sl. No.	Class of Irrigation works	Depth of Supply in inches	Irrigation rates in Rupees for flow Irrigation per hectare per year (Gazette No.494 dt.5.4.2002)
1	1st Class	28"	250.00
2	2nd Class	23"	188.00
3	3rd Class	18"	125.00
4	4th Class	9"	63.00

Table 6.25 Rabi Water Rate

Sl. No.	Name of the crops	Irrigation rates in Rupees for flow Irrigation per hectare per year (Gazette No. 494 dt.5.4.2002)	Sl. No.	Name of the crops	Irrigation rates in Rupees for flow Irrigation per hectare per year (Gazette No. 494 dt.5.4.2002)
1.	Dalua	450.00	13.	Fodder	170.00
2.	Tobacco	420.00	14.	Pulses	60.00
3.	Potato	280.00	15.	Cotton	280.00
4.	Vegetables	230.00	16.	Til (oil seeds)	60.00
5.	Onion	280.00	17.	Betel Leaf	840.00
6.	Wheat	170.00	18.	Arhar	170.00
7.	Maize	140.00	19.	Sunhemp	200.00
8.	Mung	28.00	20.	Chilly	170.00
9.	Groundnut	170.00	21.	Saru	840.00
10.	Orchards	334.00	22.	Ragi	70.00
11.	Sugarcane	500.00	23.	Mustard	60.00
12.	Jute	84.00	24.	Ganja	930.00

Table 6.26 Rate for Non-Irrigation Use

Sl. No.	Purpose for which supply is given	Rate in Rupees (Gazette No. 1571 dt.4.10.2010)		Unit
		Irrigation source	Govt. Water source	
1.	Bricks or tile making	30.00	25.00	1000 Nos.
2(i)	For water actually used and consumed for industrial / commercial purpose			
	Slab I - Consumption $\leq$ 5 cusec	4.20	3.40	1000 litre (1m ³)
	Slab II - Consumption > 5 cusec	5.60	4.50	1000 litre (1m ³)
(ii)	For water used for Hydro Power Generation	0.01	0.01	1 KWH
3.	For bulk supply to Municipalities and Notified Area Councils and other local authorities for drinking, washing, etc.	0.25	0.20	1000 litre (1m ³)
4.	Construction of commercial buildings	7.10	5.30	1000 litre (1m ³)
5.	For filling tanks	0.10	--	1000 litre (1m ³)
6.	For filling tanks mainly for drinking purpose	0.05	--	1000 litre (1m ³)
7.	For sub-soil water actually used and consumed for industrial / commercial purpose			
	Slab I - Consumption $\leq$ cusec	--	6.80	1000 litre (1m ³)
	Slab II - Consumption > 5 cusec	--	9.00	1000 litre (1m ³)

6.6 Municipal & Industrial Water Supply :**6.6.1 Domestic Use**

“Water is essential for sustenance of life and is needed in the households for drinking, cooking, washing and cleaning and horticulture needs. The requirement is not very large, when compared to the requirements for all uses. The question is one of water availability, cost of development and efficiency of management. Access to fresh water and sanitation services is a pre-condition to meet the goals and targets of social development.

The National Water Policy 2002 accords over-riding priority to drinking water over other uses. The objective of a water supply scheme is to supply safe and clean drinking water as a small part of supply for domestic use in adequate quantity as economically as possible.

About 92% of urban population has been covered by safe drinking water. Drinking water requirement of most of the cities is met from single purpose or multi-purpose reservoirs nearby or far away, even through long distance transfer. Many rural habitations are provided access to safe drinking water from hand pumps, stand posts and local source based individual or a regional scheme. More than 85 per cent of rural water supply is GW based and consumes about 5 percent of the total annual replenish able GW.

The assessment of domestic water requirement in the present as well as future years needs to be based on the present and future population, the present actual water consumption per person and the estimated future water consumption. The water requirement of live stock also forms a part of the total domestic water requirement.

While estimating water requirements for urban water supply and sanitation, it has to be borne in mind that 92% of urban population appears to have access to water supply and 63% to sewage and sanitation facilities. However, the adequacy, equitable distribution and per capita provisions of these

basis services may not be as per prescribed norms in most of the cities. For example, those living in slums and squatter settlements are generally deprived of these facilities. The rural habitations generally depend upon hand pumps for safe water supply and on mini piped water supply schemes. The NCIWRD took note of the suggestions and norms by various groups and adopted a final goal of providing 220 litres per capita per day (lpcd) for the urban areas and 150 lpcd for the rural areas. These goals were to be achieved in a phased manner as shown in Table 6.27.

Table 6.27 : Norms for Domestic Water Supply at Different Points of Time

(in lpcd)

Population Type	Year 2010	Year 2025	Year 2050
Class I Cities	220	220	220
Other than Class I Cities	150	165	220
Rural	55	70	150

It accordingly arrived at requirements for Domestic and Municipal use corresponding to low demand scenarios for the year 2010, 2025 and 2050. It also included requirements for estimated bovine population in these years. It thus worked out national water requirement for drinking and municipal uses at different point of time as shown in Table 6.28.

Table 6.28 : National Water Demand for Domestic and Municipal Use

(Quantity in BCM)

Population Type	Year 2010	Year 2025	Year 2050
Low Demand - Total	42	55	90
Surface Water	23	30	48
Ground Water	19	25	42
High Demand - Total	43	62	111
Surface Water	24	36	65
Ground Water	19	26	46

For assessment of domestic water requirements in the present and future, the sub-committee like NCIWRD also felt that these should be based on the present and future projected population, the present actual average water consumption per person and estimated future water consumption and that the water requirements for livestock also should form a part of total domestic water requirements.”

(Source : Water resources development Scenario in India, CWC, 2012 Pg.52-53)

6.6.2 Water for Industrial Use

“The basic necessity of industrial development is an assured availability of adequate water for processing and other purposes. While at present the estimated water requirement for industrial water use is only around 8-12 BCM, with urbanization and industrialization, the water demand for industrial purposes is slated to increase significantly.

Water requirement for industries, although insignificant, when compared to the demand for other uses like agriculture, creates problems by creation of point loads on available resources. Water use in industries is mostly of non-consumptive nature and with suitable treatment it can be recycled and reused by industries for their requirements of processing, cooling, boiler feed and other miscellaneous uses.

It is usually difficult to estimate the future water requirements for industries mainly due to lack of information on the present use of water by industries. This is further compounded by the uncertainty about the future growth and composition of manufacturing activities. Several assumptions have to be made to arrive at figures of water requirement.

While working out industrial water requirements for 2050, the National Commissions for Integrated Water Resources Development (NCIWRD) projected two figures of 103 BCM and 81 BCM. The first figure corresponds to the then present rate of use of water, whereas, figure of 81 BCM assumes significant breakthrough in adoption of water saving technologies for industrial production. It was also assumed that 70% of water requirement would be met from SW sources and remaining 30% from GW.

The standing sub-committee after detailed study of various data made available by different sources, estimated the water requirements as given in Table 6.29.

Table 6.29 : Water Requirement for Industrial Use

Year	Water Requirements
2000	8.0 B Cum
2010	12.2 B Cum
2025	23.0 B Cum
2050	63.0 B Cum

(Source : Ibid Pg.54)

6.6.3 Supply of Water for Industrial/Municipal Purpose :

DoWR, Govt. of Odisha allocates water to towns/industries/commercial establishments as per the provisions of the Odisha Irrigation Act, 1959 & the Odisha Irrigation Rules, 1961 and amendments from time to time. Presently, water is being provided to industrial units/commercial establishments as per recommendation of the Technical Committee known as Water Allocation Committee. So far, water has been allocated to twenty four (24) Towns and 99 Industries through Water Allocation Committee (WAC). The details are given in the Table No.6.30.

Table 6.30 Industrial Water Allocation through WAC

Sl. No.	Name of the Industrial/ Commercial Establishment	Location	Type of Industry	Water allocated (in cusec)	Source
1	2	3	4	5	6
1.	Monnet Power Company Ltd.	Malibrahmini & NISA, Angul	Power	37.00	Samal Barrage
2.	Jindal Steel & Power Ltd.	Angul	Steel	95.16	Samal Barrage
3.	NTPC	Kaniha, Angul	Power	120.00	Samal Barrage
4.	Jindal India Thermal Power Ltd.	Derjang, Angul	Steel	48.00	Samal Barrage
5.	LANCO Babandha Power Ltd.	Babandh, Dhenkanal	Power	80.00	Brahmani River
6.	Mahanadi Aban Power Co. Ltd.	Gantapda (Talcher)	Power	35.81	Brahmani River
7.	Talcher Thermal Power Station	Angul	Power	69.29	Brahmani River
8.	Jindal Stainless Ltd.	Duburi, Jajpur	Steel	68.67	Brahmani River
9.	Nava-Bharat Power (P) Ltd. (Malaxmi)	Dhenkanal	Power	42.00	Brahmani River
10.	G.M.R. Energy Ltd.	Komalanga, Angul	Power	30.00	Brahmani River

1	2	3	4	5	6
11.	Maharastra Seamless Ltd.	KIC, Duburi, Jajpur	Steel	10.79	Brahmani River
12.	VISA Steel Ltd.	Jhakhapura, Duburi, Jajpur	Steel	6.34	Kharsuan River
13.	Surendra Mining	Barahmusa, Bonei, Sundergarh	Steel	3.53	Brahmani River
14.	MGM Steel	Nimidhiha, Dhenkanal	Steel	1.63	Brahmani River
15.	Essel Mining & Industries Ltd.	Kasia, Keonjhar	Mining	1.16	Karro River
16.	Sree Metaliks Ltd.	Loidapada, Barbil, Keonjhar	Steel	0.26	Karro River
17.	Bindal Sponge Ltd.	Sunakhani, Talcher	Steel	0.81	Brahmani River
18.	CESC Ltd.	Neulapol, Dhenkanal	Power	40.00	Brahmani River
19.	Tata Steel Ltd.	KIC, Duburi, Jajpur	Steel	74.32	Brahmani River
20.	Bhushan Steel & Strips Ltd.	Meramundali, Dhenkanal	Steel	46.00	Brahmani River
21.	BRG Iron & Steel Co. (P) Ltd.	Khurunti, Dhenkanal	Steel	8.38	Brahmani River
22.	Deo Mines and Minerals	Thaiberna, Sundergarh	Steel	5.89	Brahmani River
23.	Shri Mahavir Ferro Alloys	Kalugaon, Sundergarh	Steel	4.90	Brahmani River
24.	Nava-Bharat ferro Alloys Ltd.	Kharagaprasad, Dhenkanal	Steel	5.00	Brahmani River
25.	Adhunik Metaliks Ltd. (Formerly Neepaz Metaliks Ltd.)	Chandrihariharpur, Rourkela, Sundergarh	Steel	3.73	Koel River
26.	Bhaskar Steel & Ferro Alloys	Tumkela, Sundergarh	Ferro Alloys	2.73	Brahmani River
27.	Arya Iron & Steel Co. Pvt. Ltd.	Matkambeda, Barbil, Keonjhar	Steel	0.59	Karro River
28.	SCAW Industries (P) Ltd.	Dhenkanal	Steel	2.45	Brahmani River
29.	Jindal Steel & Power Ltd.	Tensa, Bonai	Mining	0.10	Semiji Nallah
30.	Hind Metals & Industries (P) Ltd.	Meramandali, Angul	Steel	0.22	Brahmani River
31.	NALCO, Coal Mines	Durgapur, Angul	Aluminium	0.400	Brahmani River
32.	Shaslivahana Green Energy	Nimidha, Dhenkanal	Power	1.000	Brahmani River
33.	OCL India Cement & Refractory Plant	Rajgangapur Sundargarh	Cement	4.940	Sankh River
34.	Sree Ganesh Metaliks Ltd.	Kuanramunda, Rourkela	Steel	1.180	Sankh River
35.	OCL India Iron & Steel Project	Lamioi Rajagangapur	Steel	1.740	Sankh River
36.	IDCO	Kalinganagar Industrial Complex	Water Supply Installation	8.00	Brahmani River
37.	Odisha Thermal Power	Rengal, Angul	Power	80.00	Brahmani River (Relocation of plant under process)
38.	Lanco Babandha Power Ltd.	Kuruntu, Dhenkanal	Power	1.100	Brahmani River
39.	Monnet Ispat Energy Ltd.	Malibrahmani & Nisa, Angul	Steel 0.25 MTPA & Coal Industry	6.00	Brahmani
40.	IDCO, Rourkela	Sundergarh	Imphastructure Development	0.17	Sankh
41.	Shyam Steel Industries	Thaiberna, Sundargarh	Pellets Benefi-cation Plant	1.50	Brahmani

(Source : Annual Report, 2011-12, DoWR, Govt. of Odisha Pg.100-102)

Industrial water tax collected during 2011-12 is given in Table No.6.31.

Table 6.31 Industrial Water allocation in Brahmani Basin & Collection of Water tax (2011-12)

Sl.No.	Name of the Industry	Location	Product (Capacity)	Source	Allocated (in cusec)	Year of allocation	Water tax collected during 2011-12 (in lakhs)
1	2	3	4	5	6	7	8
	SUNDERGARH IRRIGATION DIVISION						
1.	M/s. Bonai Ind. Co. Ltd. K. dihi	Sundergarh	Iron ore	Karro	0.04	Allocated PWAC.	0.98
2.	M/s. Bonai Ind. Co. Ltd. N. dihi	Sundergarh	Iron ore	Karro	0.08	Allocated PWAC.	7
3.	M/s. Fee Grade & Co. (P) Ltd.	Sundergarh	Iron ore	Karro	0.08	Allocated PWAC.	1.29
4.	M/s. Runga Sons (P) K'dihi	Sundergarh	Iron ore	Karro	0.04	Allocated PWAC.	0.98
5.	Scan Steel Unit-II, Budhakata	Sundergarh	Steel	River Koel	0.05	Allocated PWAC.	13.55
6.	Gumardhi Dolomite, TISCO	Sundergarh	Mining	Nakti Nalla	0.02	Allocated PWAC.	0.15
7.	Barasuan Iron ore Mine, Tensa.	Tensa, Sundergarh	Iron ore	Kuradhi Nalla	0.62	Allocated PWAC.	46.76
8.	IDCO, Rourkela	Sundergarh	Imp. Devt.	Pitamahal Nalla	0.18	Allocated PWAC.	8.57
9.	Malda Manganese Ore Mines	Joda, Sundergarh	Mines	Brahmani	2.20	Allocated PWAC.	11.93
10.	SAIL	Rourkela	Steel Plant	Mandira, Brahmani	114,500	Allocated PWAC.	
11.	Kalta Iron Mines (SAIL)	Sundergarh	Iron ore	Nazkura	0.33	Allocated PWAC.	
12.	L & T	Kansbahal, Sundergarh	Cement	Sankh	2.0	Allocated PWAC.	10.61
13.	Adhunik Metalics Ltd.	Kuarmunda, Sundergarh	1.82 MTPA cement plant	Koel	3.73	Allocated 3.73 cusecs.	72.91
14.	Shri Mahavir Ferro Alloys	Kalugaon, Sundergarh	Steel	Sankh	4.9	Allocated during 2010	11.12

Contd. ...

1	2	3	4	5	6	7	8
15.	Sree Metaliks Ltd.	Barbil, Keonjhar	Steel-0.25 MTPA	Karrow	0.26	Allocated during 2005	
16.	M/s. Arayan Iron & Steel (P) Ltd.	Metkamunda, Sundergarh	Steel-0.30 MTPA	Karrow	0.6	Allocated during 2010	
17.	Sree Ganesh Metaliks Ltd.	Kuarmunda, Rourkela	16 MW CPP	Sankh	1.18	Allocated during 2009	1.08
18.	Bhaskar Steel & Ferro Alloys	Tumkela Sundergarh	Ferro Alloys	Brahmani	2.73	Allocated during 2009	22.86
19.	Deo Mines and Minerals	Thaiberna, SNG	Steel-0.37 MTPA	Brahmani	5.886	Allocated during 2004	
20.	M/s. Surendra Mining Ind. Pvt. Ltd.	Barhamusa, Banei	Steel-0.25 MTPA	Brahmani	3.53	Allocated during 2010	8.4
21.	OCL India Iron & Steel Project	Lamloi, Rajagangapur Sundergarh	Steel-0.25 MTPA	Sankh	1.74	Allocated during 2010	111.96
22.	OCL India Cement & Refractory Plant	Rajagangapur,	1.4 Clinker, 60 MW PP	Sankh	4.94	Allocated during 2010	68.61
23.	Jindal Steel & Power Ltd., Tensa, Sundergarh	Jindal Steel & Power Ltd.	Tensa, Sundergarh	Semij Nalla	0.10	Allocated during 2010	2.46
24.	M/s. Vijaya Krishan Lall	Tummura, Sundargarh	Mines	Nakti Jore	0.008	Allocated vide DOWR Lr. No.4552 dt.23.02.12	
25.	M/s. Vijaya Krishan Lall	Jarbeda, Sundargarh	Mines	Nakti Jore	0.01	allocated	
26.	M/s. Shyam Steel Industries Limited (SSIL) Ltd.	Thiaberna, Sundergarh	2 MTPA pellet and 2 MTPA beneficiation plant	Brahmani	1.5	Allocated vide DOWR Lr. No.14179 dt.21.05.12	
	BAITARANI DIVISION, SALAPADA						
27.	M/s. Aryan Mining & Trading Ltd.	Sundergarh	Mining	Karro	0.02	Allocated PWAC.	0.38
28.	SAIL, Kiriburu, Keonjhar	Barbil, Keonjhar	Iron ore	Karro	0.76	Allocated PWAC.	41.745

Contd. ...

1	2	3	4	5	6	7	8
29.	SAIL, Bonal, Keonjhar, N'dihi	Barbil, Keonjhar	Iron ore	Karro	0.08	Allocated PWAC.	45.766
30.	Essel Mining & Industries	Keonjhar	Mining & beneficiation Plant	Karro	1.16	Allocated during 2009	38.594
	ANGUL IRRIGATION DIVISION, ANGUL						
31.	M.C.L. Talcher, Angul	Talcher	Coal mine	Brahmani	18.58	Prior to WAC	14.57
32.	NALCO	Angul	Aluminium Plant	Brahmani	29.95	Prior to WAC	2023.9
33.	M/s. Orissa Thermal Power Cor. Ltd.	Rengali Dam Site, Rengali	(Ph-I) - 1000 MW & (Ph-II)- 1000 MW	Brahmani	80	Allocated during 2011	
34.	Jindal Steel & Power Ltd.	Angul	Steel-6.00 MTPA	Samal Barrage	95.16	Allocated during 2008	32.58
35.	Jindal India Thermal Power Ltd. (Old name - Jindal Photo Ltd.)	Derang, Talcher	Power - 1200 MW, P.P.	Brahmani	48	Allocation revalidated during 2012	5.72
36.	Monnet Ispat & Energy Ltd.	Malibrahmini & Nisa, Angul	1000 MW	Brahmani	37	Allocated during 2011	
37.	Monnet Ispat & Energy Ltd.	Malibrahmini & Nisa, Angul	Steel-0.25 MTPA & coal Washery	Brahmani	6	Allocated during 2011	16.43
38.	TTPS (NTPC)	Talcher	Power Plant	Brahmani	69.29	PWAC	653.68
39.	NALCO (Coal Mine)	Durgapur, Angul	Coal mining	Singada Jor	0.4	Allocated during 2010	

Contd. ...

1	2	3	4	5	6	7	8
HEADWORKS DIVISION, SAMAL DIST. ANGUL							
40.	NTPC, Kaniha	Kaniha, Talcher	Proposed 3000 MW	Brahmani	120	Reallocated during 2010	3697.994
RENGALI RIGHT CANAL DIVISION-II, DHENKANAL							
41.	Reliance Industries Ltd.	Boulpur, Dhenkanal		Brahmani	0.49	PWAC	
42.	Bindal Sponge Ltd.	Sunakhani, Talcher	Sponge Iron	Brahmani	0.81	Allocated during 2005	
43.	BRG Iron Ltd.	Khuntuni, Dhenkanal	Steel-0.25 MTPA	Brahmani	8.38	Allocated during 2006	29.375
44.	SCAW Industries (P) Ltd.	Gundichapada, Dhenkanal	Steel-0.25 MTPA	Brahmani	2.45	Allocated during 2003	
45.	Hind Metalick & Industries (P) Ltd.	Meramandali, Angul	Steel	Brahmani	0.215	Allocated during 2006	
46.	Nava-Bharat Ferro Alloys	Kharagaprasad/ Dhenkanal	Ferro Alloys	Brahmani	5.00	Allocated during 2011	83.71
47.	Nava Bharat Power (P) Ltd. Malaxmi	Dhenkanal/Angul	Power-1040 MW, P.P.	Brahmani	42	Allocated during 2006	
48.	CESC LTD.	Neulapoi, Dhenkanal	Power-1000 MW, P.P.	Brahmani	40	Allocated during 2008	
49.	G.M.R. Eneergy Ltd.	Kamalnga, Angul	Power-1000 MW P.P.	Brahmani	30	Allocated during 2007	48.586
50.	Bhusan Steel & Strips.	Meramundali, Dhenkanal	Steel-3.10 MTPA	Brahmani	46	Allocated during 2003	2009.683
51.	Mahanadi Aban Power Ltd.	Gantapada (Talcher)	Power-1030 MW, P.P.	Brahmani	35.81	Allocated during 2009	

Contd. ...

1	2	3	4	5	6	7	8
52.	Lanco Babandha Power Pvt. Ltd.	Babandh, Dist. Dhenkanal	Power-2000 MW, P.P.	Brahmani	80	Phase-II allocated & revalidated during 2012	
53.	MGM Steel Ltd.	Nimidha, Dhenkanal	Steel-0.25 MTPA	Brahmani	1.63	Allocated during 2009	5.928
54.	M/s. Shalivahana Green Energy	Nimidha, Dhenkanal	20 MW Bio Mass P.P.	Brahmani	1.00	Allocated during 2011	2.013
	JARAKA IRRIGATION DIVISION, JARAKA						
55.	IDCO	Kalinganagar Industrial Complex	Supply to MISL, NINL & JSL	Brahmani	8.00	Allocated during 2011	643.828
56.	TATA Iron & Steel Co.	Kalinganagar Industrial Complex	Steel-6.00 MTPA	Kharsuan	74.32	Revalidated during 2011	
57.	Jindal Stainless	Kalinganagar Industrial Complex	Steel-0.80 MTPA	Brahmani	68.67	Allocated during 2007	1030.19
58.	VISA Steel Ltd.	Kalinganagar Industrial Complex	Steel-0.35 MTPA	Brahmani	6.34	Allocated during 2006	
59.	Maharashtra Seamless	Kalinganagar Industrial Complex	Steel-0.30 MTPA	Brahmani	10.79	Allocated during 2006	
Sub total =					1119.52	Sub total =	10825.74

PAWAC - Prior to WAC (Water Allocation Committee)

(Source : Chief Engineer, OWPO, DoWR, Bhubaneswar)

Water allocated to towns and municipalities is depicted in Table 6.32.

Table 6.32 Water Allocation to towns through WAC

Sl. No.	Name of the Town	District	Qty. allocated (in Cusec)	Source
1.	Vyasanagar Town	Jajpur	3.84	Jokadia Anicut
2.	Talcher Town	Dhenkanal	3.70	Brahmani
3.	Dhenkanal Town	Dhenkanal	3.70	Brahmani
4.	Angul Town	Angul	3.98	Derjang Reservoir
5.	Biramitrapur Municipality	Sundergarh	2.65	River Sankh
6.	Deogarh Municipality	Bargarh	2.79	Rengali Dam

In addition to the above, there are other industries drawing water from the Government water sources prior to formation of Water Allocation Committee and paying water rate to the respective Executive Engineers. Regularization of water drawal of such industries is in process.

The collection of water rate from both the agricultural sector and industrial sector was earlier vested with the Revenue & Excise Department. Responsibilities for collection of industrial water rate from industrial sector was transferred to the Department of Water Resources vide Revenue & Excise Department Notification dated 29th September, 1999 and 11th October, 1999 which were published in the Odisha Gazettes vide No.1423 dt.30.09.1999 and No.1466 dt.13.10.1999. The Department of Water Resources has been collecting the water rate from the Industrial units / Commercial organizations through their Executive Engineers and Assistant Engineers w.e.f. 01.04.2000.

The demand of water rate from the industries / commercial organizations for the year 2010-11 was Rs.8018.59 lakh against this, an amount of Rs.12082.30 lakh has been collected by March 2011. Industries / Commercial entities have been advised not to use ground water for commercial purposes without permission.

Water Pricing & Cost recovery :

The infrastructure created needs to be self sustainable. Therefore, water pricing is necessary. As per the State Water Policy, the cost of operation and management will be fully recovered from the beneficiaries. Water Rates & Cost Recovery Committee has been formed to fix and review water charges. The Committee recommends the water charges to Water Resource Board for approval. In the meantime, the water rates for non-irrigation use have been revised thrice during 1994, 1998 & 2010 for irrigation use the rates have been revised during 1998 & 2002.

The cost recovery from Agriculture and Industrial sectors has shown an increasing trend as it increased from Rs.7.0281 crore in the year 1996-97 to Rs.313.42 crore in the year 2011-12. The following table gives an indication of the rising trend of water tax collection.

Table 6.33 Cost recovery from Agricultural & Industrial Sector (Rs. in Cr.)

Year	Agricultural Sector	Industrial Sector	Total amount collected
1	2	3	4
1996-97	3.3999	3.6282	7.0281
1997-98	4.4664	3.6134	8.0798
1998-99	9.5074	5.3688	14.8762
1999-00	5.9386	2.4648	8.4034

1	2	3	4
2000-01	10.9857	8.8806	19.8663
2001-02	12.3871	4.8531	17.2402
2002-03	16.1614	4.0208	20.1822
2003-04	23.692	8.6225	32.3145
2004-05	27.7784	6.504	34.2824
2005-06	28.2988	7.350	35.6488
2006-07	28.5653	18.8758	47.4411
2007-08	29.7918	9.7623	39.5541
2008-09	29.8352	12.3924	42.2276
2009-10	32.6399	34.4003	67.0402
2010-11	25.768	120.8230	146.5910
2011-12	29.98	283.44	313.42

(Source : Annual Report 2011-12, DoWR, Govt. of Odisha, Table 10.4, Pg.105)

6.7 Pisciculture :

6.7.1 Introduction

“Odisha, being a maritime State, has considerable scope for development of inland, brackish water and marine fisheries. The State is endowed with a long coastline of 480 km with continental shelf area of 24,000 sq. km. along the Bay of Bengal. It offers tremendous opportunities for development of fresh water, brackish water and marine fisheries with scope of fish production together with employment and income generation for socio economic prosperity. Fresh water resources of the State are estimated to be 6.73 lakh ha comprising of 1.22 lakh ha of tanks/ponds, 2 lakh ha of reservoirs, 1.80 lakh ha of lakes, swamps & jheels and 1.71 lakh hectares of rivers and canals. The State’s brackish water resources are of the order of 4.18 lakh ha with a breakup of 0.79 lakh ha of Chilika Lake, 2.98 lakh ha of estuaries, 32,587 ha of brackish water tanks and 8,100 ha of backwaters.

According to the Fishery Survey in India (FSI), the fisheries potential of Odisha is 513,667 MT. About 2.95 percent population (10.84 lakhs) depends upon fisheries for their livelihood. Of them, 7.51 lakhs depend on inland fisheries and 3.33 lakhs on marine fisheries. The fisheries sub-sector contributed about six percent to the GSDP share of the agriculture sector for the year 2012-13 (advance estimate).” (Source : Economic Survey, Odisha - 2012-13, Pg.94).

6.7.1.1 The State Reservoir Fishery Policy

“The State reservoir Fishery Policy has been formulated with a view to introducing systematic and remunerative pisciculture in reservoirs. The policy aims at substituting traditional methods by introduction of advanced technologies and techniques. It permits the transfer of reservoirs which have an area of 100 acres and above to the Fisheries and Animal Resources Department (F&ARD), Government of Odisha. The F&ARD Department, Government of Odisha has been empowered to lease out these reservoirs to Primary Fishermen Co-operative Societies registered under the Odisha State Co-operative Society Act, 2001 and preference will be given to displaced/project affected persons.” (Source Ibid Pg.96)

The Fish Fauna of Brahmani river is attached as Annexure 6.2. Details of fish production of ponds / tanks of Brahmani Basin is given in Table 6.34. Categori-wise fish productions are given in Table 6.35 to 6.38.

Fish Fauna of Brahmani River

Annexure - 6.2

Common Category	Scientific Name
Carp	Labeo rohita
-do-	Catla catla
-do-	Labeo calbasu
-do-	Labeo bata
-do-	Cirrhinus mrigala
-do-	Puntius sarana
Cat-Fishes	Wallago attu
-do-	Pangasius pangasius
-do-	Mystus cavasius
-do-	Mystus vittatus
-do-	Silondia sailondia
Feather Backs	Notopterus notopterus
-do-	Notopterus chitala
-do-	Esomus dendricus
Forage Fishes	Trichogaster fasciatus
-do-	Ambassis nama
-do-	Puntius sophore
-do-	Puntius ticto
-do-	Oxygaster bacaila
-do-	Setipinna phasa
-do-	Amblypharyngodon mola
-do-	Chela chela
-do-	Gudusia chapra
-do-	Gambusia affinis
Prawn	Macrobrachium malcolmsonii
-do-	Macrobrachium rosenbergii
Misc.	Glossogobius giuris
-do-	Megalops cyprinoids
-do-	Ompok pabda
-do-	Ompok bihamaculatus
-do-	Anguilla bengalensis
-do-	Macrognathus aculeatus

(Source : Brahmani basin, Sectoral Study, Sheladia Associates Inc., USA, Chapter V, Pg. 16)

**Table 6.34 Fish Production of Ponds/Tanks of Brahmani Basin
(Block-wise Water bodies) Year - 2000-01**

Sl.No.	Name of Dist	Name of the Block	Area covered (ha)	Qty. of fish (T)
1	2	3	4	5
1.	Sundargarh	Sundargarh	56.50	125.00
2.	-do-	Tangarapati	51.00	72.70
3.	-do-	Balisankar	30.00	60.30
4.	-do-	Lephripada	54.40	106.39
5.	-do-	Subdega	5.20	10.60
6.	-do-	Hemagiri	49.50	99.00
7.	-do-	Baragaon	11.25	26.22
8.	-do-	Kutra	4.62	10.00
9.	-do-	Rajagangapur	8.28	13.75
10.	-do-	Kuarmunda	23.82	39.58
11.	-do-	Lathikata	58.90	67.00
12.	-do-	Nuagaon	4.50	8.47
13.	-do-	Gurundia	14.90	30.88
14.	-do-	Banei	28.80	59.50
15.	-do-	Lahunipada	21.67	40.67
16.	-do-	Koida	18.00	25.00
17.	Sambalpur	Jujumara	126.20	302.10
18.	-do-	Maneswar	129.40	319.12
19.	-do-	Rengali	119.00	248.30
20.	-do-	Dhankauda	207.60	553.84
21.	-do-	Bamara	45.25	88.27
22.	-do-	Jamankira	98.40	268.48
23.	-do-	Kuchinda	146.10	312.20
24.	-do-	Rairakhol	50.70	143.60
25.	-do-	Naktideul	32.16	100.63
26.	Deogarh	Barkot	35.00	75.90
27.	-do-	Reamal	44.08	96.14
28.	-do-	Tileibani	37.12	80.98
29.	Angul	Angul	385.80	572.09
30.	-do-	Banarpal	368.82	771.085
31.	-do-	Chhandipada	28.94	52.20
32.	-do-	Athamallick	156.50	322.88
33.	-do-	Kishorenagar	120.46	371.10
34.	-do-	Pallahara	124.60	250.00
35.	-do-	Kaniha	103.40	207.78
36.	-do-	Talcher	201.86	542.54
37.	Dhenkanal	Dhenkanal	144.20	283.979
38.	-do-	Gondia	155.13	373.212
39.	-do-	Odapada	152.80	271.010

1	2	3	4	5
40.	Dhenkanal	Hindol	194.74	323.354
41.	-do-	Bhuban	181.30	395.05
42.	-do-	K. Nagar	194.14	372.137
43.	-do-	Kankadahad	92.52	193.300
44.	-do-	Parjang	187.50	358.144
45.	Keonjhar	Bansapal	29.46	82.655
46.	-do-	Keonjhar	301.00	362.83
47.	-do-	Ghatgaon	150.00	308.92
48.	-do-	Patna	120.30	232.87
49.	-do-	Saharpada	162.20	176.10
50.	-do-	Telkoi	140.35	280.69
51.	-do-	Harichandanpur	118.24	275.80
52.	-do-	Joda	22.43	67.20
53.	-do-	Jhumpura	170.95	248.28
54.	-do-	Champua	145.49	278.05
55.	-do-	Hatadihi	133.62	202.33
56.	-do-	Ghasipura	91.00	232.29
57.	-do-	Anandapur	61.12	216.60
58.	Jajpur	Binjharpur	63.75	99.36
59.	-do-	Badachana	147.65	196.45
60.	-do-	Dharmagala	116.66	237.88
61.	-do-	Rasulpur	96.00	210.00
62.	-do-	Jajpur	89.93	160.80
63.	-do-	Dasarathpur	76.01	123.43
64.	-do-	Korei	86.70	129.55
65.	-do-	Dangadi	44.00	96.60
66.	-do-	Sukinda	71.25	148.85
67.	-do-	Bari	--	--
68.	Kendrapara	Rajkanika	64.94	65.52
69.	-do-	Aul(Pt)	155.00	165.01
70.	-do-	Rajanagar	260.00	476.40
71.	-do-	Patamundei	151.06	121.92
72.	-do-	Derabis	205.60	225.05
73.	-do-	Kendrapara	222.0	340.40
74.	-do-	Garadapur	173.20	151.74
75.	-do-	Mahakalpada	480.00	595.00
TOTAL			8355.85	15553.036

(Source : Brahmani Sectoral Study - Fisheries Resources, Saheladia Associates Inc. USA, Ch.5 Pg.8-9)

Table 6.35 Fish Production in Brahmani Basin**(in '000 MT.)**

Year	Fresh water	Brakish water	Total	Marine Fish	Total
1990-91	58.72	22.04	80.76	78.19	158.95
1991-92	65.12	22.76	87.88	95.03	182.91
1992-93	70.83	22.93	93.76	119.38	213.14
1993-94	116.37	11.99	128.36	103.92	232.28
1994-95	123.96	10.81	134.77	122.89	257.66
1995-96	121.94	12.90	134.84	123.20	258.04
1996-97	127.29	16.20	143.49	133.46	276.95
1997-98	135.64	16.78	152.42	156.08	308.50
1998-99	145.00	14.90	159.90	124.33	284.23
2000-01	125.11	13.35	138.46	121.09	259.55

(Source : 3rd Spiral Study Report, Brahmani Basin, OWPO, Nov. 2002, Annex.4.15)

Table 6.36 : Production of Fish and Crab in Odisha**(in '000 MT.)**

Year	Inland fish production		Marine fish Production	Total fish Production	Percapita Consumption of fish (Kg.)	Crab Production
	Fresh water	Brackish water				
1	2	3	4	5	6	7
1999-00	124.90	10.40	125.90	261.20	7.30	0.50
2000-01	125.10	13.40	121.10	259.60	7.70	1.40
2001-02	147.40	20.70	113.90	282.00	8.10	1.20
2002-03	154.20	20.00	115.00	289.20	8.30	2.20
2003-04	165.60	24.50	116.90	307.00	8.40	2.20
2004-05	170.10	23.80	121.90	315.80	8.70	1.70
2005-06	179.70	23.50	122.20	325.40	9.50	1.40
2006-07	191.63	22.95	128.14	342.72	8.99	1.74
2007-08	195.75	22.97	130.76	349.48	9.29	1.79
2008-09	213.00	26.33	135.49	374.82	13.27	2.09
2009-10	215.80	25.51	129.33	370.64	10.86	2.43
2010-11	224.96	27.75	133.48	386.19	9.42	3.37
2011-12(P)	237.47	30.06	114.30	381.83	9.91	2.28

Note : (P) Provisional

(Source : Economic Survey, Odisha 2012-13 Pg.127)

Table 6.37 : Source-wise Brackish Water Fish/Shrimp & Crab Production in Odisha (in '000 MT.)

Year	Chilika Lake	Brackish Water Shrimp	Estuaries	Total Production
1	2	3	4	5
1999-00	1.75	3.08	5.62	10.44
2000-01	4.98	6.43	2.03	13.44
2001-02	11.99	7.20	1.47	20.56
2002-03	10.89	7.17	1.90	19.96
2003-04	14.05	8.11	2.31	24.48

1	2	3	4	5
2004-05	13.26	7.88	2.64	23.78
2005-06	12.23	8.39	2.88	23.50
2006-07	9.96	9.65	3.34	22.95
2007-08	10.05	10.19	2.74	22.97
2008-09	10.70	11.66	3.97	26.33
2009-10	11.96	10.98	2.57	25.51
2010-11	13.07	11.63	3.05	27.75
2011-12(P)	14.23	11.97	3.86	30.06

Note : (P) provisional

Source : Directorate of Fisheries, Odisha

Table 6.38 : Source-wise Fresh Water Fish Production in Odisha

Year	Tanks/Ponds	Reservoirs	Lakes/Swamp/ Jheels	Rivers/Canals	Total
1	2	3	4	5	6
1999-00	88.11	13.81	2.07	20.87	124.86
2000-01	92.44	8.01	2.73	21.93	125.11
2001-02	112.85	7.09	4.00	23.46	147.40
2002-03	119.80	8.50	2.67	23.27	154.24
2003-04	133.62	10.14	2.76	19.08	165.59
2004-05	140.46	11.53	1.79	16.31	169.88
2005-06	153.45	10.75	2.34	13.20	179.74
2006-07	164.74	12.10	2.43	12.36	191.63
2007-08	169.64	12.45	1.54	12.12	195.75
2008-09	185.40	12.53	1.60	13.47	213.00
2009-10	190.37	12.33	1.85	11.25	215.80
2010-11	197.59	14.61	1.65	11.11	224.96
2011-12(P)	211.19	13.73	1.94	10.61	237.47

Note : (P) provisional

Source : Economic Survey, Odisha 2012-13 Pg.127)

**Table 6.39(a) Details of Fish Production from 1994-95 to 1998-99
from Reservoirs in Brahmani Basin**

Sl. No.	Name of the District	Name of the reservoir	Mean water spread area (ha)	Fish catch (in Kg)					Average increase in fish productivity Kg/ha/yr	Average productivities Kg/ha/yr	
				94-95	95-96	96-97	97-98	98-99		Pre-project	Post Project
1.	Angul	Derjang	530	15830.10	19053.38	18880.00	25141.00	29666.30	23.11	32.86	55.97
2.	Dhenkanal	Dadaraghati	480	0.00	18295.20	10907.93	14932.80	15201.59	27.87	3.80	31.67
3.	Deogarh	Gohira	1094	0.00	9684.00	7284.00	18324.50	19442.29	17.28	0.49	17.77
4.	Dhenkanal	Ramiala	650	0.00	0.00	28115.45	23725.20	24697.73	34.30	3.70	38.00

(Source : Ibid Pg.16)

Table 6.39(b) - District-wise Fish Yield from Reservoirs in Brahmani Basin

Name of reservoir	Dist	Mean Water spread area (ha.)	Fish yield (kg/ha/yr)									
			2004-05	2005-06	2006-07	2007-08	2008-09	2009-10	2010-11	2011-12	2012-13	
Rengali	Angul	14933	10.04	13.41	25.33	20.79	18.86	38.87	32.30	34.10	43.00	
Rengali	Deogarh	16250	73.56	64.74	93.27	62.15	64.37	66.43	22.50	64.25	136.92	
Derjang	Angul	530	36.68	185.36	109.94	87.38	134.89	115.21	134.08	129.26	127.77	
Dadarghati	Dhenkanal	480	94.17	95.00	117.00	122.85	160.00	180.83	168.88	174.17	173.69	
Ramial	Keonjhar	783	2.89	3.81	3.28	9.37	10.09	11.99	14.46	196.00	196.68	

(Source : Directorate of fisheries, Odisha)

6.8 Instrumentation :

6.8.1 Purpose of Instrumentation :

The very purpose of instrumentation is to take measurements. Lord Kelvin once said “When you can measure what you are speaking about and express it in numbers, you know something about it; but when you cannot measure it, when you can not express it in numbers your knowledge is of a meagre and unsatisfactory kind.”

“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. xxxxx

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 mal-functioning or any abnormal behaviour. (Source : Earth and Rockfill Dams by Bharat Singh & H.D. Sharma, Sarita Prakashan, June 1976 Pg.468-469)

The instrumentation alone is not a complete safe guard. The number of device installed is less important than the selection of proper type of instruments, their location at critical points and intelligent interpretation of data.

6.8.2 The Parameters Required to Monitor the Performance of Gravity Dams.

Sl.No.	Parameters	Suitable Instruments
1.	Uplift Pressure	I Vibrating wire plezometer II Unbonded electric resistance piezometers III Multipoint piezometer with packers. IV Multipoint piezometer surrounded with grout V Multipoint push in piezometers
2.	Seepage	I Weires i) V-Notch ii) Rectangular weir II Flumes
3.	Temperature	I Resistance Thermometer II Thermometer Vibrating Wire III Thermisters
4.	Displacement	I Extensometer II Whitmore gauges Optical III Crack monitoring IV Calipers V Micrometers VI Dial gauges
5.	Stress	I Gloetzi cell II Carlson load cell III Vibrating wire stress IV Flat jacks
6.	Strain	I Resistance Type strainmeter II Vibrating wire strainmeter III Weldable Strain meter IV No stress-strain meter
7.	Pore Pressure	I Open System Type i) Porous tube piezometers ii) Slotted pipe piezometers II Closed System Type i) Hydraulic twin tube piezometers ii) Electric piezometers a) Vibrating wire strain gauge piezometer b) Resistance strain gauge piezometer III Total pressure cells
8.	Seismic	Seismographs i) Strong Motion Accelerograph ii) Structural Response Recorders
9.	Joint	Jointmeters
10.	Deformation	I Multipoint borehole extensometer II Foundation deformation gauges III Tunnel type gauges
11.	Deflection/surface	I Plumb lines II Tiltmeters III Surveying techniques i) Triangulation ii) Trilateration iii) Collimation

6.8.2.1 Instruments installed in Rengali Dam :

Following instruments have been installed in Rengali Dam in block Nos.16, 23 & 47.

Sl. No.	Name of Instruments	Block 16 (NOF)	Block 23 (OF)	Block 47	Total
1.	Resistance Thermometer	33	15	---	48
2.	Pore Pressure Cells	8	18	---	26
3.	Stress meter	5	5	---	10
4.	Uplift Pressure Gauge	8	8	8	24
5.	Plumb Line	1	1	---	2

The Drg. indicating the locations of instruments are shown vide Drg. No.6.5. Procedure for installations of above five instruments have been described under section 6.8.2.3.

For installation and observation of above instruments, the following accessories were procured.

- a) Wheatstone bridge.
- b) Universal Indicator.
- c) Installation type switch box.
- d) 4-wire cable.
- e) Splicing Kit.
- f) Pressure gauge assembly.

6.8.2.2 Procurement of Instruments

Previously it was proposed by C.W.C. in Drawing No.RNG-5010-3015 to install instruments in Block No.15 (Deepest non-overflow section) & Block No.23 (Deepest over flow section). As the Western side of protection wall butting against Block No.15 will not be a representative of non-overflow section for study of behaviour of instruments, the instrumentation Block was changed from Block No.15 to Block No.16 vide C.C.E.,RM.P. Letter No.W-I-93/80-1536 dt.24.1.80 and finally instruments such as (i) Thermometer (ii) Stress meter (iii) Pore pressure Cell (iv) Uplift Pressure pipe and (v) Wire plumb line observation were installed in Block No.16 and Block No.23.

Total quantity of instruments and accessories procured from different sources are furnished below :

Table 6.40 Instruments and Accessories Procured :

Sl.No.	Name of Instruments	Quantity	Source of Procurement
1	2	3	4
1.	Universal Indicator CM-4 F.	1 No.	Procured from M/s. Kyowa Electronics, Japan
2.	Installation type switch Box - NS-22F.	3 Nos.	Instruments Corporation Ltd. Tokyo (Japan)
3.	Pore Pressure Meter CP-8 N.	18 Nos.	-do-
4.	Cable splicing kit Scotch cast type.	200 Nos.	-do-

1	2	3	4
5.	Stress meter CR-30 G	11 Nos.	Iduki Dam Divn., Trivandrum, Kerala
6.	Resistance type Thermometer	60 Nos.	S.M. & R Division Madras
7.	Wheatstone Bridge OSAW	1 No.	M/s. Oriental Science aparatus workshop, Haryana
8.	Uplift pressure Gauge	20 Nos.	M/s. Project Syndicate, Calcutta
	Automatic Water Level Indicator	2 Nos.	-do-
9.	Instruments for Plumb Line Observation	2 Sets	M/s. En Cardiorite Electronics Pvt. Ltd., Lucknow
10.	En-Cardiorite Switch Box 24 points	2 Sets	-do-
11.	4-Wire Cable	1500 mtrs.	Mahendra Electrical Limited, Calcutta

6.8.2.3 Installation of Instruments :

6.8.2.3.1 Installation of Thermometers :

Resistance thermometers were embeded in Block No.16 and Block No.23 as per location metioned in C.W.C. Drawing No.SAI-RNG-C-4005 and 4006. At required location, trenches of size 20 cm x 20 cm x 20 cm were left in the masonry; then resistance thermometers were laid horizontally, perpendicular to the axis of dam, centrally in the trench and concrete was covered arround the resistance thermometer. Precaution was also taken to protect the thermometer from any damage, that is likely to occur during further concreting or masonry operation. Identification tag was also attached to each resistance thermometer, as per nomenclature, mentioned in the drawing, at the end of terminal cable and then the cable was taken through conduit pipe and then finally taken to the terminal switch Box. Different resistance thermometers have been embedded in different dates after getting the required level of masonry. The details of datewise embedment of resistance thermometers are furnished below :

Sl.No. of Resistance Thermometer	Block No.	Date of embedment
1	2	3
T 17	16	19.2.80
T 18	16	29.3.80
T 2	23	16.4.80
T 1	23	19.4.80
T 19	16	29.4.80
T 3	23	10.5.80
T 16	16	16.5.80

1	2	3
T 4	23	18.2.81
T 20	16	4.3.81
T 24	16	10.3.81
T 26	16	17.4.81
T 25	16	4.5.81
T 28 to T 31	16	25.12.81
T 32 to T 33	16	12.2.82
T 39	16	6.4.82
T 40	16	7.4.82
T 34 to T 38	16	21.4.82
T 41 to T 44	16	10.12.82
T 46 to T 49	16	3.2.83
T 50 to T 52	16	12.4.83
T 5 to T 6	23	16.4.83
T 7 to T 10	23	16.6.83
T 11	23	1.2.84
T 12 to T 15	23	2.2.84

Fifteen nos. of Resistance Thermometer (T_1 to T_{15}) have been embedded in Block No.23 in 3 layers (T_1 to T_9 at E.L.80 m, T_5 to T_{10} at E.L.94 m and T_{11} to T_{15} at R.L. 106 m). Similarly thirty three nos. of Resistance Thermometers (T_{16} to T_{54}) have been embedded (T_{16} to T_{54}) in Block No.16 in 6 layers (T_{16} to T_{23} at E.L. 63.5 m, T_{24} to T_{31} at E.L. 72 m, T_{32} to T_{38} at E.L. 84 m, T_{39} to T_{44} at E.L. 96 m, T_{45} to T_{49} at E.L. 108 m, T_{50} to T_{54} at E.L. 120 m). At the specific locations, trenches of size 20 cm x 20 cm x 20 cm were left in the masonry and then resistance thermometers were laid horizontally perpendicular to dam axis, centrally in the trench and concrete was covered around resistance thermometer. Precaution was also taken to protect the thermometer from any change that is like to occur during further concreting or masonry operation. Out of 48 nos. of resistance thermometers embedded in Block Nos.16 & 23, thermometer No.T5 in Block No.23 and Sl.No. T_{24} , T_{27} , T_{34} , T_{49} and T_{50} of Block No.16 were not responding since its embedment, similarly T_8 of Block No.23 and T_{30} of Block No.16 were not responding due to some mechanical defects.

6.8.2.3.2 Installation of Stress Meters :

Stress meters have been embedded in Block No.16 and Block No.23 as per location mentioned in the C.W.C. Dr.No.SAI-RNG-C-4005 and 4006 respectively. At required location in Block No.16 at E1.65.00 in Block No.23 at E1.82.00 a recess of depth 30 cm and 40 cm is left at each point of location of stress meter. The bottom surface of the recess is made plane and smooth. Rich cement mortar is prepared and placed in the centre of recess in conical shape. Then the particular stressmeter as per identification and nomenclature was placed over the cement mortar. The stressmeter was pressed with reciprocal rotary motion to squeeze the cement mortar to appear out ward of the diaphragm.



Then applying load through a tripod, the stressmeter concreting was done slowly around it and the load with tripod was removed with proper care so as not to disturb the stress meter. The other end of the terminal cable was taken through the conduit to the terminal switch Box.

The details of datewise embedment of Stressmeter is furnished below :

Stressmeter Sl.No.	Block No.	Date of embedment
S 1 to S 4	23	31.1.81
S 5	23	2.6.82
S 6 to S 7	16	19.5.80
S 8	16	20.5.80
S 9	16	10.4.81
S 10	16	23.4.81

Five nos. of stress meters have been installed in Block No.23 in centre line of cross gallery at E.L. 82 m and five nos. of stress meters (S 6 to S 10) have been installed in Block No.16 in centre of cross galley at E.L. 65.00 m. A recess of depth 30 cm and 40 cm is left at each location of stress meter. Bottom surface of recess plane is grade plane and smooth. Stress meter Sl. Number S-5 of Block No.23 did not respond since beginning and stress meter Sl. No. S 2 responded for a shorter period with high stress value and observed resistance ratio much more than resistance ratio at zero stress. S_3 & S_4 were not responding in Block 16. S_6 did not respond from beginning. S_7 responded for a shorter period with high stress value. S_8 though responding but observed resistance ratio is very less than zero stress value. S_9 & S_{10} were showing less resistance ratio than its zero stress value.

6.8.2.3.3 Installation of Pore Pressure Cell :

Before embedment of pore pressure cell, the length of cable was estimated as per shortest path from the point of location to the terminal switch Box through conduit pipe for each pore pressure cell from the drawing, keeping extra provision of two meters approximately for variation. One end of the cable was spliced to the cable of pore pressure cell and identification tag was attached to the pore pressure cell as per location and length of cable with the nomenclature as mentioned in the drawing. The resistance of the lead connected to each pore pressure was found and recorded in the permanent register.

Pore pressure cells have been embedded in Block No.16 and Block No.23 as per location mentioned in the CWC Drawing No.SAI-RNG-C-4005 and 4006 respectively at required level and location. A recess of size 40 cm x 40 cm and of depth 30 cm was left. In that recess the particular pore pressure cell as per nomenclature in the drawing was laid horizontally normal to the exterior face of dam and then cement mortar was placed around the cell and tamped lightly with hand tamping. The other end of the cable was taken to the terminal switch Box through conduit pipe.

Eight nos. of pore pressure cells (P_1 to P_8) have been embedded into Block No.23 in two layers. In first layer 5 nos. of pore pressure cells (P_1 to P_5) have been embedded at E.L. 87.50 m and three nos. of pore pressure walls (P_6 to P_8) have been embedded at E.L. 95 m in second layer. Similarly eight nos. of pore pressure cells (P_9 to P_{16}) have been embedded in two layers of Block No.16. In first layer five nos. of pore pressure cells (P_9 to P_{13}) have been embedded at E.L. 72 m and in 2nd layers 3 numbers of pore pressure cells (P_{14} to P_{16}) have been embedded at E.L. 95 m. Observation of pore pressure cell in Block No.16 and Block No.23 were taken. Measuring unit is Wheatstone bridge. Resistance of pore pressure cell are being measured by connecting black and white lead to the Whetstone Bridge and resistance ratios are being measured by connecting all the four leads to Whetstone Bridge cell.

6.8.2.3.4 Installation of Uplift Pressure Pipes :

Seven Nos. of uplift pipes namely U1 to U7 have been installed in Block No.23 at E1 80.00, Eight Nos. of uplift pipes namely U8 to U15 have been installed in Block No.16 at E1 68.50 and Seven Nos. of Uplift pipe namely U 16 to U 24 have been installed in Block No.47 at E1 90.46 for U 16, 88.60 for U 18, U 21 to U 24, 89.21 for U 17, 89.33 for U 19 and 88.30 for U 20.

At the foundation level on the point of location of uplift pressure pipe a hole of 1 m depth was drilled and then 50 mm dia pipe was anchored vertically. Just above the foundation level a perforated pipe of 50 mm dia and 225 mm long having drill hole of 5 mm dia in two rows with nine nos. of hole in each row was fitted to the vertical pipe by Tee connection. A wooden box of size 400 mm x 400 mm x 300 mm was placed around the perforated pipe. After providing mortar seal around the wooden box concreting was done to cover the wooden box, after 12 hours of mortar fill. Then after 12 hours of concreting, it is finally covered with masonry. The pipe is extended to Gallery floor as masonry progresses. A 2" x 2" x $\frac{1}{2}$ " Tee with 2" counter sunk plug was connected in the top of each pipe just below the Gallery floor. A $\frac{1}{2}$ " pipe was connected horizontally and it is capped for installation of pressure Gauge Assembly for observation of uplift pressure.

Uplift pressure pipes have been installed in Block No.16, Block No.23 and Block No.47. Seven nos. of uplift pipes namely U₁ to U₇ have been installed in Block No.23 at E.L. 80 m, eight nos. of uplift pipes namely U₈ to U₁₅ have been installed in Block No.16 at E.L. 68.5 m and 7 nos. of uplift pipe U₁₆ to U₂₄ have been installed in Block No.47 at E.L. 90.46 m for U₁₆, 88.6 m for U₁₈, U₂₁ to U₂₄, 89.2 m for U₁₇, 89.33 m for U₁₉ and 88.3 m for U₂₀. At foundation level on the point of location of uplift pressure pipe - a hole of 1 m depth was drilled and then 50 mm diameter pipe was anchored vertically. Just above foundation level - a perforated pipe of 50 mm diameter and 225 mm long having drill hole of 5 mm diameter in 2 rows with 9 nos. of hole in each row was fitted to the vertical pipe by Tee connection. A wooden box of size 400 mm x 400 mm x 300 mm was placed around the perforated pipe. After providing mortar seal around the wooden box concreting was done to cover the wooden box, after 12 hours of mortar fill. Then after 12 hours of concreting, it was finally covered with masonry. The pipe was extended to gallery floor as masonry progressed. A 2" x 2" x $\frac{1}{2}$ " Tee with 2" counter sunk plug was connected in the top of each pipe just below gallery floor. A $\frac{1}{2}$ " pipe was connected horizontally and it was capped for installation of pressure gauge assembly for observation of uplift pressure. Cap at the end of $\frac{1}{2}$ " pipe were kept open and as water flows the water pressure was being measured by Pressure Gauge. Otherwise rise of water above $\frac{1}{2}$ " pipe was measured by a plastic pipe connecting at the end of $\frac{1}{2}$ " pipe and the results of observation were recorded in a permanent register for each uplift pipe separately.

6.8.2.3.5 Installation of Plumb line :

During the process of construction of the dam a cabin of size 1 m x 1 m was left in the Gallery floor, at the junction of Main Gallery and Cross Gallery of Block No.16 and Block No.23 for installation of plumb line arrangement. A 0.3 m dia pipe was embedded vertically in the Gallery roof just above the cabin, as shown in the Drawing, in each Block. Then the pipe is extended as the masonry / concrete progresses and it is left at 0.5 m below the dam top. At the top a chamber of 0.7 m dia as shown in the drawing, was prepared for suspension arrangement of plumb line. At the bottom a masonry well was constructed over the Gallery floor in the cabin, left for plumb line arrangement. After keeping the oil tank in this well just below the vertical pipe a slab, just in shape of vernier microscope mounting plate will be covered over the masonry well which will serve as foundation for mounting plate. Lastly vernier

microscope will be fitted in the mounting plate and plumb line will be dangled from the spider at the top which will carry a heavy plumb bob in the bottom and the plumb bob will be immersed in the oil contained in the oil tank.

(Source : Completion Report of Rengali Dam, 1991)

6.8.2.3.6 Observed Seepage Pattern for Rengali Dam :

The seepage observed at different reservoir levels from 1994 to 2003 are given in Table 6.41.

Table 6.41 : Seepage Pattern

Unit - Seepage in lit/sec.

Elevation (m)	1994	1995	1996	1997	1998	1999	2000	2001	2002	2003
123.0	26.15	21.125	19.825	18.525	---	21.45	---	18.85	17.55	18.525
120.0	---	15.925	17.55	18.20	18.85	17.875	18.20	---	15.60	13.97
110.0	14.625	10.075	13.65	10.725	13.00	12.025	11.05	11.375	13.00	16.275

(Source : Inspection Report of DSRP, 27th May - 3rd June, 2004)

This pattern does not indicate any general trend of either increasing or decreasing of seepage.

6.8.2.3.7 Conclusion

It is most unfortunate that none of the instruments installed at Rengali Dam are functioning since 1995-96. It is suggested that the instruments may be made operational in consultation with CWPRS, Pune, CSMRS New Delhi and Dam Safety Organisation of CWC so that the health of the dam could be known and necessary remedial measure could be taken in time.

6.9 Dam Safety :

“While many more dams will be required to be constructed for fully exploiting the available surface water potential, there is also a realization of the need for making those already existing as fully safe.

Any development measure such as a dam, building or a bridge, presents a degree of risk to life or damage to property, should it fail. All such structures also have the characteristic to change with time in adjustment to their surroundings and in their capability to resist the forces imposed upon them by the man and the nature. Dams, however, due to the ever-changing conditions of the population located along the riverbanks downstream, often constitute a bigger hazard in the case of a failure than other public structures.

With the ever-increasing number of dams, Government of India realized the importance of dam safety way back in the mid seventies and took a number of steps to highlight its concern for creating general awareness among the engineering community. Dam safety is now considered an inherent feature in the planning, design, construction, maintenance and operation of dams.”

(Source : Water Resources Development Scenario in India, CWC, 2012 Pg.82)

6.9.1 Dam Safety Organization at the Centre and in the States

“Water being a state subject as per the Indian constitution, the states are the owners of the dams within their territories and as such any initiative by the Central Government has necessarily to involve the state governments for its proper implementation. Keeping this in view, the matter was broached in the State Irrigation Ministers Conference held in 1975 and, as a follow up of its recommendations a “Dam Safety Organization (DSO)” was created in the Central Water Commission (CWC) in 1979. The objective of this organisation was to perform a coordinative and advisory role for the State Governments and to lay down guidelines, compile technical literature, organise trainings etc.

and in general to take steps to create awareness in the states about dam safety and thereafter assist in setting up infrastructure for the same” (Ibid : Pg.82-83).

In this regard, the relevant extract of the dam Safety Bill No.108 of 2010 is appended vide Annex.6.3 for reference.

6.9.2 Factors affecting the Safety of dams :

Safety of dams is the prime responsibility of the owner i.e. the Government who in turn entrust the same to the engineers as dams are constructed by them. Dams have to be safe from two considerations viz. (i) hydrological and (ii) structural. Structural aspects of dam safety can be monitored by proper instrumentation, vigilance and frequent inspection, timely maintenance, judicious operation etc. But the hydrological aspects warrant for repeated review when more and more hydrometeorological and climatological data of severe magnitude become available, surpassing previous events on which the hydrological studies of the projects were based.

The probable causes of failure/distress of dams fall broadly under following categories :

- i) Inadequate spillway capacity
- ii) Deterioration of concrete and masonry and other structural materials
- iii) Excessive leakage in or across the body / foundation
- iv) Foundation problem
- v) Excessive structural stresses due to external forces leading to cracking, sliding or overturning.
- vi) Alkali aggregate reaction
- vii) Excessive scour downstream of the spillway
- viii) Erosion of surface concrete of spillway glacis and the energy dissipators due to cavitation

An analysis of the causes of dam failures in the world indicates the following break down :

Table 6.42 : Causes of Failure of dams

Sl.No.	Causes of Failure	% of total failure analysed
1.	Foundation Problem	40
2.	Inadequate Spillway Capacity	23
3.	Poor Construction	12
4.	Uneven Settlement	10
5.	High Pore Pressure	5
6.	Acts of War	3
7.	Embankment Slip	2
8.	Sub-standard materials	2
9.	Defective Operation	2
10.	Earthquake	1
Total		100

(Source : Workshop Proceedings on ‘Dam Safety Assurance and Rehabilitation’, 18-21 June 1996, Bhubaneswar, Pg.V-2)

Of the major types of dams, the percentages that experienced incidents were as under :

Sl.No.	Type of Dam	% of incidents
1.	Arch	5.6
2.	Buttress	7.0
3.	Gravity	2.1
4.	Embankment	6.6

(Source : Ibid Pg.V-2)

Failure of dams in India as per CWC record is furnished vide Annexure 6.4 indicating the causes of failure.

Dam Safety Review Panel (DSRP)

Govt. of Odisha (GoO) in their Order No.22402/WR Dt.02.07.2003 reconstituted the DSRP with following terms of reference.

- i) It will examine and review the adequacy of investigation, design and hydrology of the project.
- ii) It shall suggest remedial measures to ensure safety of the structures.
- iii) It will inspect dam sites and review the distress conditions alongwith recommending suggestion for necessary remedial measures.
- iv) The panel will meet frequently to assess the status of the dams and to present recommendations.

The Panel headed by Sri A. D. Mohile as chairman alongwith Sri Suresh Chandra, Sri R. C. Rath, Sri R. C. Tripathy as members inspected Rengali Dam on 31.05.2004 and 01.06.2004 and again from 6th -9th Sept, 2011 (as members of DSRP for the DRIP dams). The observations of the Panel are discussed as under :

Ist visit of DSRP

Rigid Dam - Hydraulic behaviour of Rengali Dam :

Blockage of body drainage by Leaching and Calcination :

It was observed during inspection of DSRP that many drainage holes which were provided to drain the body of the dam are virtually solidly blocked due to leaching (action) of free lime in cement used in dam construction. This phenomena is not specific to the Rengali dam alone. This is also reported at Hirakud and is common to almost all dams. The vertical downward drainage holes are to be regularly reamed and flushed clean which are being carried out by the project staff. State Dam Safety Unit may keep a close watch over these activities during their periodic inspection. There is little expertise in the techniques and methodology which may be followed to re-service the blocked passages which go upward from the drainage gallery.

DSRP recommended that project personnel review all articles and literatures on the subject as well as the documentation done by ICOLD to find any practical and effective ways to re-servicing the holes for drainage of dam body which may have been followed elsewhere in the world in similar situation. One suggestion could be to drill holes from top of the dam / spillway and plug a small portion of the bored hole from top by grouting and isolate the bottom reach with the help of packers.

The drainage holes have been reamed by drilling upwards from the gallery towards top which have improved the body drainage.

Signs of leakage of water was observed in the downstream face of the piers at 3 - 4 places. It was informed by the Project authorities that the leakage occurs when the reservoir level touches the FRL. It was suggested by DSRP members to grout the areas before the 2004 flood season.

Structural deficiencies in the Rigid Dam :

Longitudinal cracks along the gallery floor :

The cracks along the gallery floor at few of the blocks were looked into by DSRP. The structural drawing as available in the drawing volume of the completion report which was shown to the DSRP by the project authorities was also studied. It appears (from these drawings) that the reinforcements surrounding the gallery cavity are provided at a regular distance close to the gallery side walls and crown. But at the bottom of floor level these reinforcements are taken inclined from an elevation close to the gallery floor level at the d/s to a significantly lower level at the upstream. This was (perhaps) done to provide the reinforcement so as to pass below the drain sill level which is at the upstream end of the gallery floor to simplify the geometry of the reinforcement placement.

In such a structural geometry it is but natural that the cover of concrete above the reinforcement plane (which is provided to form a levelled floor of the gallery) and which are far removed from the reinforcements may not take active part (to behave) as structural member, as those concrete at the close vicinity of reinforcement. The cracks which are noticed may simply be the outcome of this benign phenomenon.

It was recommended by the DSRP that the behaviour and progress (if any) of these cracks may be monitored. Also the soundness of concrete close to the crack at a few places along the bottom floor of the gallery which constitutes the structural member be tested by non-destructive testing for confirmation.

Horizontal and marmred cracks were observed in Block 51 constructed with RR masonry at 5 locations on the d/s face at El.127.60 m, 126.30 m, 125.90 m, 125.60 m and 125.35 m. There were also two vertical cracks inside the adit at about 1.20 m and 2.20 m from the adit face.

Few tale - tell glasses have been fixed over these cracks in June 2007. Two such glasses have cracked. The maximum width of horizontal cracks are 2 to 3 mm which diminish after about 4 m length.

Similar cracks were also noticed in Block 50, 49, 47, 43 and 19. Cracks are also visible in piers No.38, 23, 21, 19, 18 & 17. and anchor blocks No.13, 10 & 8.

In order to assess the continuity of cracks, tests were conducted in Block pier No.16. Holes were drilled along the line of cracks for 100 to 120 mm depth on both the faces of pier. Nipples were fixed and water pumped at a pressure of 1 to 5 kg/cm², when it was observed that water pumped at one face of the pier came out in the other face. This indicated that crack is continuous.

Methodology for fixing Anchor girder :

It is pertinent here to discuss on the construction methodology adopted for fixing the anchor girders and anchor bars in the pier which will throw some light regarding development of cracks.

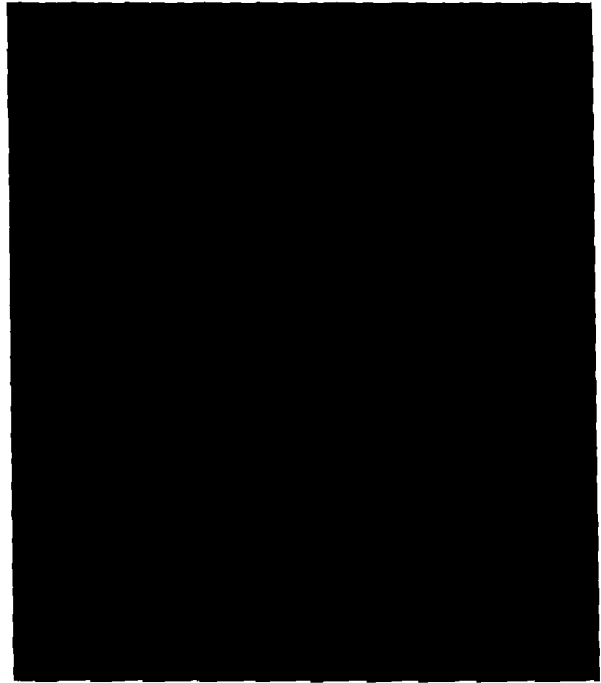
Due to difficulty and time-consuming procedure, in keeping the anchor girders and anchor bars in position by the help of steel supports, to do the pier concreting from the bottom of anchor girder around RL 113.358 m, an alternative method was adopted at site during construction.

- i) A strip of concreting of pier in between the girder of both the face, with suitable width matching with the gap in between the right and left girder, at RL around 113.358 m, up to a suitable height (about 2 m) and length from u/s. end of the anchor girder to u/s trunion point, was done in the first stage. The second stage concreting to embed the anchor girders and the anchor bars was done after fixing the anchor girders and bars in position.
- ii) The anchor bars are wrapped with suitable polymer wrappers to make the anchors free to move inside the concrete due to its expansion and contraction on depending upon the gate operation. In other words there is a gap around the anchor bars from the anchor girders in u/s to trunion point end for movement of the anchor without developing any stress in the concrete.

Cracks in Rengali Dam



Cracks in Masonry Block No.51



Cracks in Masonry Block No.51

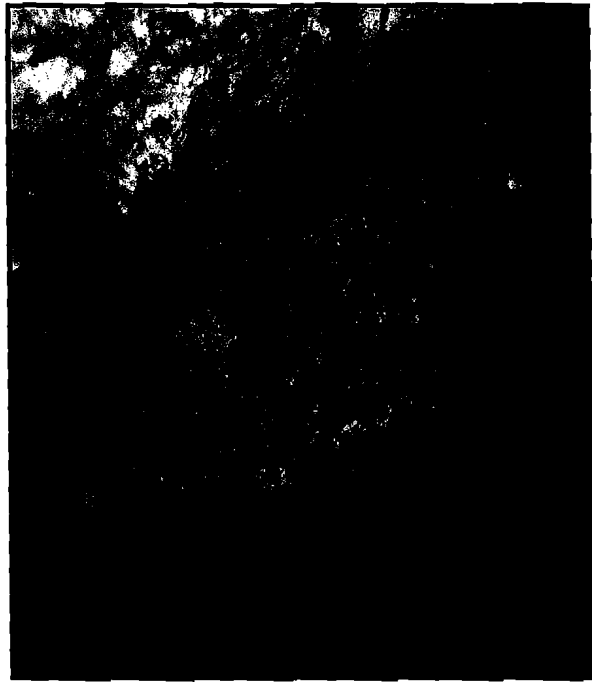


Downstream view of Block No.19

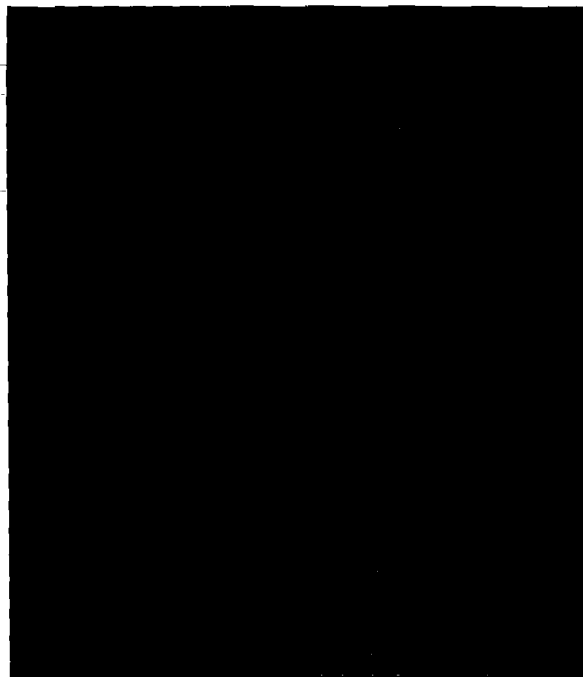
Repair of Cracks



Fixing of nipple inside Sluice Barrel



Repair to Spalled Bottom Flange



Epoxy Grouting to Piercap of Spillway Bridge

- iii) The vertical cracks developed near the u/s face of the anchor girders indicates bond failure in the concrete in that zone may be for tensile stress concentration due to water pressure on the gate when one gate is open and the adjacent gate is in closed condition. This needs detail calculation of the stresses. It may be noted that allowable tensile stress in concrete is almost nil.

From the construction drawing, it is observed that four block outs / pockets were left through the body of the pier above elevation 123.90 m to facilitate the movement of shuttering truss for construction of spillway bridge. The portions of the pier below the bridge bearing pedestals were raised to RL 126.03 m for casting of the bearing pedestal. After casting of girder and bridge slab the blockouts / pockets were filled up with concrete at a later date after completion of the bridge. The observed crack is located near the secondary concrete face of blockout / pocket. Spalling of concrete from the bottom bulb of girder just above the bearing was also observed resting on the pier on Block No.38 and also the girder of the other span connecting to Block No.37.

Power House :

The non-overflow dam blocks separate the Power house from the reservoir. At the d/s, the tail pond, when at high stage reach to levels above the service bay elevation. These conditions create sweating and at a few places even wetting of power house outer walls occurs which adjoin the peripheral hydraulic structures. The DSRP inspected some of these sweating marks. They also looked into the visible lines of separation at the ground level along the contact plane of power house, structural concrete, and the mass concrete behind it. There seems to be no apparent sign of distress. It was reported that the dampness caused deterioration and rapid decay of paint-finishes as well as causing harm the moisture sensitive instruments. The source of this moisture appeared to be from the transformer deck. The transformer deck was accordingly inspected.

Water from the transformer cooling system was being pumped into the drain at the toe of the dam. The pits on the deck contained water. A longitudinal crack was observed on the transformer deck at the junction of RCC roof supporting the transformers and plain concrete of power dam.

DSRP suggested that the areas of such severe dampness may be plotted as to their extent and state of seepage water, if any. It is also needed to improve the imperviousness of ground surface around the power house and to drainout the area effectively especially during rain. Water proofing of the crack may be taken up at the earliest.

Energy dissipation arrangement :

The energy dissipation system consists of ski-jump bucket with solid rock mass in the plunge pool. The bucket appears to be in a healthy state except that the drain hole pipes provided in the bucket for releasing uplift pressure are choked with debris at some places.

It is recommended that the drain hole pipes may be cleaned of the debris and revived so as to continue to function in an effective manner. The mouth of the pipes could be welded with a fine steel mesh so as to keep the debris out or strainer pipes could be used for the same purpose. The protruding part of the pipes may be provided with stout cement concrete protection suitably. The bucket should be annually drained out and visual check be made for any damage in the invert. The plunge pool may be cleared of boulders as soon as possible.

The sides of right and left training wall facing spillway have been eroded and reinforcements are exposed apparently by rolling action of the boulders during spill. The size of eroded portion in the training wall looks to be about one square meter. At first glance, the level of the rock mass on the right side appears to be higher than that on the left side on which side, erosion has taken place. It appears, there might be cross flow of spill water occurring from right to left causing the boulders to hit the training wall. It was informed that dydraulic model testing was carried out for the design of energy dissipation system and the length of training wall by CWPRS and the details of the study may be available.

It was further suggested that the eroded part of the training wall may be repaired immediately, thus covering up the exposed reinforcement as a temporary measure. The hydraulic model test study carried out by CWPRS may be submitted to the DSRP for devising an appropriate and lasting solution of the problem.

2nd visit of DSRP :

- i) As suggested by the panel the upper level bucket was dewatered and inspected during 2006 summer. It was observed that a moderately plunge pool has been formed in front of the lower bucket.

The panel emphasised to dewater the lower bucket and remove the loose boulders lying in the plunge pool. Then fill up the pool after necessary cleaning with lean concrete, topped by 15 cm layer rich concrete.

This has been attended to.

- ii) During partial opening, spillway gates were oscillating. DSRP suggested that no gates are to be lifted for less than 1.0 m height and more than 75% of the height. Further differential head between the opening of two adjacent bays should not exceed 3.0 m.

- iii) The vertical cracks near the bearing of the anchor girders were cut in V-shape and 15 cm depth holes drilled. Epoxy grouting was done applying low pressure.

Seepage was observed through horizontal joints in the concrete on the d/s of radial gates in several piers.

As a remedial measure cement grouting with anti - shrinkage compound was done at a pressure upto 3.5 kg/cm². The wet patches have reduced.

- iv) During inspection of the panel, the reservoir level was at EL 121.0 m and the total leakage in the dam was of the order of 22.75 lps but the leakage water was somewhat muddy. It was suggested that chemical analysis of the leakage water and mechanical analysis of the sediment may be done.

- v) It was noticed that both left and right flank / training wall between the NOF and OF section was eroded exposing the reinforcement.

This has been attended to by providing anchor bars and concreting the gap between wall and cut surface of the rock.

Annexure 6.3

Extract of Dam Safety Bill 108/2010

“Sometimes certain factors of failures unrelated with ageing process may also get linked with the age of dam. We know that the reasons for dam failure can be several, and they may get ingrained at any point of time over the prolonged life cycle of dam. Thus, cause of failure may get implanted at investigation stage, design stage, construction stage or the post-construction (operational) stage of the dam. However, the cause may fructify only after a prolonged period; and thus it may get attributed to the ageing of dam.

Even though portrayed as modern-temples of India by Pandit Jawahar Lal Nehru, the dams in India have been in existence since age old times. The 24 m high earthen dam of *Thonnur Tank* in Karnataka is over 1000 years old, and it is still in use. Besides, there are 126 large-dams which are over 100 years old. Post independence, a substantial number of dams were added up in the early five-year plan periods to meet the needs of irrigation, drinking water, hydro-power and supplies to municipalities and industries. The Table 6.43 below presents the age-wise profile of India's large dams,

assembled from data given in National register of Large Dams (CWC, 2009). Over 76% of these dams are more than 20 years old - a period sufficient to dim the initial spotlight and political mileage attached with water resource projects. Evidently, most of these projects are in dire needs of maintenance, for want of adequate budgetary support from the State Governments.

Table 6.43 : Age-wise profile of India's large dams

Age-group of dams	Number of Dams
0-10	287
11-20	605
21-30	1248
31-40	1289
41-50	493
51-60	239
61-100	239
More than 100	126
Age unknown	202
Under Construction	397
Total	5125

The large number of India's ageing dams - since not maintained adequately - present a grave threat to the lives and economies of the downstream populations. With increasing number of dams becoming older and older, the likelihood of dam failures in India is understandably on an ascending path. The likelihood of dam failures has been further aggravated by the fact that many of the ageing dams lack qualified (and experienced) supervisory and maintenance staff needed for guaranteeing the structural safety and the operational integrity to prevent possible failures. x x x x x

To reduce the risk of dam failures, regular health inspections are necessary to identify the deficiencies; and, wherever severe deficiencies are observed, comprehensive rehabilitation measures are required to be taken.

- ◆ **Dam Safety Institutional Strengthening**, focusing on regulatory and technical frameworks for dam safety assurance. The activities to be carried out will include, but not be limited to : (i) targeted training and support to Dam Safety organizations at central and state level to become effective organizations that can take the lead in ensuring that dams remain safe from a structural and operational point of view; (ii) training of staff of Water Resources Departments and other operators of dams to assist with the development of appropriate skills and modern tools to adequately operate and maintain dams; (iii) attendance at dam safety courses; (iv) study tours, and linking with country agencies which have advanced dam safety programs such as Canada, United States and Switzerland; (v) independent dam safety review panels, comprising experts in relevant disciplines; (vi) development of capacity to carry out reservoir sedimentation studies; (vii) development of Management Information Systems (MIS) and other programs to capture and analyze data for long-term planning and guiding of dam operations; (viii) support to the development within CWC of the Dam Health and Rehabilitation Monitoring Application (DHARMA) program that will allow a systematic presentation and interpretation of data for

effective monitoring of the health of dams; (ix) support to the revision of existing guidelines on dam safety and preparation of new guidelines, as needed; and (x) training in hazard and vulnerability assessment and dam-break analysis.

- ◆ **Project Management.** The overall responsibility for project oversight and coordination will rest with the Dam Safety rehabilitation Directorate in the DSO of CWC. This Directorate will act as the Central Project Management Unit (CPMU). The Directorate will be assisted by a management and engineering consulting firm. Each state will establish a State Project Management Unit (SPMU) in the State DSO. This Unit will have direct responsibility for the coordination and management of the project at state level.

Discussion and Conclusion

For the disaster risk reduction, United Nations has identified 5 priority actions (known as Hyogo Framework for Action), and these are : (i) **Ensure** that disaster risk reduction is a national and a local priority with a strong institutional basis for implementation; (ii) **Identify**, assess and monitor disaster risks and enhance early warning; (iii) **Sharing** of Knowledge, innovation and education to build a culture of safety and resilience at all levels; (iv) **Reduce** the underlying risk factors; and (v) **Strengthen** disaster preparedness for effective response at all levels, so that consolidated efforts gets projected coherently in a synchroized manner at the global platforms (UN, 2005). Understandably, the above actions are also applicable in case of dam safety activities in India; and, these are being embedded in Gtovernment of India's recent initiatives for dam safety initiatives.

For countries like india with large stocks of dams, the issue of dam safety is critical. There is an urgent need for proper organisational arrangement at the national and state levels for ensuring the safety of such dams. The proposed dam safety bill is expected to ensure proper inspection, maintenance and surveillance of existing dams and also to ensure proper planning, investigation, design and construction for safety of new dams. However, there is a need for expediting its approval by the Parliament; and, also the need for its earliest adoption by different States by passing of resolutions in this regard by respective Assemblies.

The fundamental dam safety objective is to protect people, property and the environment from harmful effects of misoperation of failure of dams and reservoirs (ICOLD, 2010). India has significant numbers of large dams, many of which are old and some are distressed; and the safe operation of such dams has social, economic, and environmental relevance. In such a scenario, the importance of stakeholder involvement in dam safety projects hardly needs any overemphssis. The project's relationship with its stakeholders is a two-way process: it enables the project to fathom the concerns and reactions of stakeholders and also allows the stakeholders to comprehend project actions correctly thus eliminating chances of misinformation (Maheshwari and Pillai, 2004). The earlier completed Dam Safety Assurance and Rehabilitation project (DSARP) was a novel step in right direction, which also gave realization of the limitations of our institutional capacities. The now proposed Dam Rehabilitation and Improvement Project (DRIP) is expected to give new and stakeholder-inclusive impetus to the dam safety activities of India by helping in the capacity building of the Dam Safety Organisation of CWC and that of participating states, for fulfilment of important roles envisaged as per dam safety legislation under formulation."

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(Source : The dam Safety Bill No.108, 2010 Pg.95, 101 & 102).

Annexure 6.4**Reported Failures of Dams in India as per CWC record (Year-wise)**

Sl. No.	State	Name of the Project	Type	Maximum Height (M)	Year of Completion	Year of Failure	Cause of Failure
Upto 1950							
1.	Madhya Pradesh	Tigra	Masonry	24.03	1914-17	1917	Overtopping followed by slice
2.	Maharashtra	Ashti	Earth	17.7	1883	1933	Slope failure.
3.	Madhya Pradesh	Pagara	Composite	27.03	1911-27	1943	Overtopping followed by breach

Contd..

Sl. No.	State	Name of the Project	Type	Maximum Height (M)	Year of Completion	Year of Failure	Cause of Failure
1951-60							
4.	Madhya Pradesh	Palakmati	Earth	14.6	1942	1953	Sliding failure
5.	Rajasthan	Dakhya	Earth	N.A.	1953	1953	Breaching
6.	Uttar Pradesh	Ahrura	Earth	22.8	1953	1953	Breaching
7.	Rajasthan	Girinanda	Earth	N.A.	1954	1955	Overtopping followed by breaching
8.	Rajasthan	Anwar	earth	12.5	1956	1957	Breaching
9.	Rajasthan	Gudah	Earth	28.3	1956	1957	Breached due to bad workmanship
10.	Andhra Pradesh	Kadam	Composite	22.5	1957	1958	Inadequate Spillway capacity & incorrect operation of gates
11.	Rajasthan	Sukri	Earth	N.A.	N.A.	1958	Breached by leakage through foundation
12.	Madhya Pradesh	Nawagaon	Earth	16	1958	1959	Overtopping leading to breach
13.	Rajasthan	Dervakheda	Earth	N.A.	N.A.	1959	Breaching
14.	Gujarat	Kaila	Earth	23.08	1955	1959	Foundation Failure
1961-70							
15.	Maharashtra	Panshet	Earth	53.8	1961	1961	Piping failure leading to breach
16.	Maharashtra	Khadakwasla	Masonry	60	1875	1961	Overtopping
17.	Rajasthan	Golwania	Earth	N.A.	1960	1961	Breaching
18.	Rajasthan	Nawagaza	Earth	N.A.	1955	1961	Breaching
19.	Maharashtra	Bandsura	Composite	21.6	N.A.	1962	Slope failure
20.	Madhya Pradesh	Sampna	Earth	21.3	1956	1964	Slope failure on account of inappropriate materials.
21.	Madhya Pradesh	Kedarnala	Earth	20	1964	1964	Breaching
22.	Uttar Pradesh	Nanaksagar	Earth	16	1962	1967	Breached due to foundation piping.
23.	Gujarat	Dantiwada	Earth	60.96	1969	1973	Breach on account of floods.
1971-80							
24.	Tamil Nadu	Kodeganar	Earth	17.7	1977	1977	Breached on account of floods
25.	Gujarat	Machhu-II	Composite	24.69	1972	1979	Overtopping due to floods.
1981-90							
26.	Gujarat	Mitti	Earth	16.02	1982	1988	Overtopping leading to breach.
1991-2000							
27.	Madhya Pradesh	Chandora	Earth	27.3	1986	1991	Breach
28.	Rajasthan	Bhimlot	Masonry	17	1958	--	Breached due to inadequate spillway capacity.
2001-2010							
29.	Gujarat	Pratappur	Earth	NA	1891	2001	Breached on account of floods
30.	Madhya Pradesh	Jamunia	Earth	15.40	1921	2002	Piping leading to breaching.
31.	Rajasthan	Jaswant Sagar	Earth	43.38	1889	2007	Piping
32.	Rajasthan	Gararda	Earth	30	2010	2010	Breach

(Source : The Dam Safety Bill, 2010 Pg.103)

Chapter VII

Conclusion

7.1 Constructional Hazards :

7.1.1 Introduction

“The problem of water resources development are neither mathematical nor purely hydrological. The solution cannot be obtained directly by some set of scientific laws or principles. The concept of water resources scheme requires practical solutions to a number of problems related to engineering, economics, sociology, environment, agriculture, political, finance and management etc. Thus such a planning no longer remains within the confines of social, scientific and engineering but it becomes an art. No readymade solutions or principles can be enunciated for such an art of planning”. (Source : Water Resources Development & Planning by P.S Nigam Vol. I, CBIP, 1995 pg.vii).

Besides, for gravity dams, good foundation is a gift of nature. For Rengali masonry dam, the foundation was free from major defects and has absolutely not posed any serious problem during construction. This was one of the ideal foundation on which the 70 m. high masonry dam has been constructed. Geological details have been described in ch.II under Sec. 2.1.2. and block wise geology have been dealt in ch. III under Sec. 3.2.1. During construction, river diversion is another important factor as the river is to be suitably diverted from its original course during construction. In case of Rengali, the non-monsoon flow was diverted through two construction sluices, the details of which have been presented in ch. III & under Sec.3.2.17. Though construction of u/s and d/s cofferdam posed certain problems, those could be successfully tackled by the project engineers.

7.1.2 Problems confronted during Construction of Dam & Power House.

7.1.2.1 In Dam Construction :

There was profuse leakage from one of the formed drains in block No. 12 of the dam into the foundation gallery. It was difficult to ascertain the exact location of such leakage in the upstream face until the reservoir level was lowered down after Nov. 1984. This was successfully treated and the leakage sealed which has been described in ch.III Sec 3.3.4.

7.1.2.2 In Power House Construction:

a) **Problem faced due to temporary sealing of gates with masonry :** The temporary sealing did not prove successful. There was profuse leakage which required heavy dewatering. Attempts made by project officials went in vain. Finally during Aug, 1983, with the help of divers, the leakage was controlled vide ch. III Sec 3.4.9.

b) **Problems arising due to lesser thickness of draft tube walls.**

Leakage was noticed through the joints between the layers of concrete. Even though leakage reduced due to cement grouting from tail race side but wet patches were noticed on the wall. This necessitated grouting of wall from inside of P.H using special grout i.e ‘Aquagel – 9’ when situation improved. (For details , refer ch. III Sec 3.6B)

c) **Seepage into Auxiliary rooms of P.H :**

There was a clean joint between the dam toe and auxiliary room along ‘E’ line. With rise in reservoir level, wet patches were observed on the auxiliary room walls. Cement grouting was resorted to but not successful. The dampness persisted . Finally epoxy grouting was done and things improved. This has been described vide Ch. III Sec 3.6C.

d) **Leakage through expansion joint along ‘D’ line :**

The problem of leakage through the joint was tackled by pouring shalijet (a hot bituminous compound) and covering with cement mortar. This aspect has been dealt vide Ch. III Sec 3.6D.

e) **Retrogression in the d/s of Spillway:**

Downstream area of the spillway bucket showed occurrence of scoured pockets since 1992. These scoured pockets became alarming after the flood of 1995. The scoured pocket show a linear

trend of NW-SE and are confined within acid charnockites. Scouring trend when projected coincide with very insignificant shear zone present in the foundation of right training wall. It was apprehended that retrogression, unless checked, may affect the training wall. Concrete apron of suitable thickness of boulder cracks of sufficient height have been suggested in front of the training wall to stop retrograde erosion?

7.1.3 Problems encountered in the excavation of LMC & RMC :

Both LMC and RMC are contour canals to command total ayacut of 218392.0 ha (including lift ayacut of 32002 ha) through distribution net work covering 25 blocks in 8 districts vide Table No. 2.10 of the book. Left and Right main canals in their long stretch negotiate highly undulating topography where depth of cutting vary between 20 m to 33 m. There were three alternatives before the project engineers viz. (i) open cut (ii) cut and cover and (iii) tunnelling in deep cutting reaches. Geological features dictate the mode of construction from safety and durability vis-à-vis techno-economic consideration. These aspects are discussed in detail under following sections.

7.1.3.1 Water conductor system of LMC from RD 31.5 km to 35.5 km

“During preparation of the project report in the year 1979, it was contemplated to have a tunnel of 9.5 m diameter in this zone to avoid heavy cutting (approximate quantity being 46 lakh cum) Further disposal of such large quantity of excavated material was a problem and maintenance of the canal in this reach during post construction stage may be a costly affair due to recurring expenditure. Seven number of exploratory holes were drilled along the proposed canal alignment. The geotechnical investigation was carried out by the Geological Survey of India (GSI) during 1989 when 133.15 m. cores recovered from seven bore holes were analyzed. In view of (i) low core recovery from these bore holes, (ii) inadequate rock cover i.e less than two times the diameter over tunnel crown and (iii) softness of Gondwana sedimentary formation, the idea of tunneling was rejected but no alternative was suggested.

7.1.3.1.1 Geomorphology:

The area is an undulating peneplained country. Being a soil covered terrain, it is under extensive cultivation. There exists NE-SW ridge which forms a local water divide line. This prominent geomorphic feature stands across the canal alignment.

Canal passes through Gondwana rocks consisting of alternating sequence of horizontally bedded sandstone and thin bedded green shales. Sandstones display two sets of joints of which the bedding joint is most prominent. The N-S trending vertical joint is poorly developed. Shales show typical needle shaped pieces, which is a product of trapoidal weathering.

In Jan 1996, WAPCOS (India) Ltd. prepared a project for Rengali Irrigation Sub-Project LBC II for OECF funding and geotechnical report in March 1997. The geotechnical note is based on six more bore holes drilled along the canal alignment. They opined that if tunneling or deep excavation is carried through soft, weak and thinly bedded Gondwana shale and sandstone, these rocks may disintegrate when they come in contact with water. For tunneling extensive and immediate support system would be required. For open excavation, the side slopes may not fail by shear owing to horizontal bedding of rocks. However, on exposure the rocks may disintegrate. The disintegrates rock fragments may creep into excavation during heavy rains by gravity. The excavated slope may, therefore, need protection by guniting or any other appropriate technology. In view of above, they concluded that a decision has to be taken from following three alternatives or their combinations.

- i) Open excavation ii) Tunneling iii) Cut & cover section with RCC barrel”.

7.1.3.1.2 Study of Various alternatives :

On 24th and 25th Aug 1998, there was a workshop at Bhubaneswar on “Design Planning and Construction of Tunnels:” The outcome of the workshop are summarized as under:

- i) Due to poor geological condition, there would be construction problem. But in view of advanced technology and know-how, with use of modern tunneling equipments (e.g Tunnel Boring Machine) tunneling may be a better option.
- ii) Detouring of canal alignment so that the canal & tunnel to pass through Archean crystallines available within 1 km to 3.5 km. But change in canal alignment was not practicable as the canal upto RD 30.00 km. was already excavated. To detour the canal by about 3.5 km. at RD 31.5 km and 1 km at RD 35.5 Km. would require fresh land acquisition which will be costly and time consuming . This will also involve additional head loss.
- iii) Instead of single tunnel of 9.5 diameter, it was also discussed either to construct two tunnels of 7 m diameter or three tunnels of 5.8 diameter. As the single tunnel is cost effective and cost of smaller size tunnels do not reduce proportionately to their diameter, construction of single tunnel was preferred to two or three tunnels. Further transition will be simpler and less costly for single tunnel, Single tunnel is hydraulically more efficient.

When detouring became infeasible, alternative narrowed down to single tunnel in the adverse geological formation. But the rock cover over the crown did not satisfy the design criteria of twice of the diameter except from RD 31.8 km to 33.8 km. For the rest, the cover varied between 1.11 to 1.83 times the diameter. Tunneling through such soft, weak and horizontally bedded sedimentary rock in the absence of adequate rock cover may create serious problem of tunnel collapse.

Another suggestion was to examine the viability of lowering the bed level of tunnel by 2 to 3 m. Due to depression of the tunnel bed, the flow in the tunnel will be non uniform and in course of time the depression will be filled up with sediment. Thus pressure flow condition is expected.

7.1.3.1.3 Decision of Technical Advisory Committee :

There were two state level Technical Advisory Committee (TAC) formed to resolve this complex issue, 1st on 21.09.1998 and 2nd on 18.05.1999 at Bhubaneswar. *The observations of the 1st TAC were as follows :*

- i) As proposal for lowering in the invert level was not considered by the project, no drilling was recommended by the GSI beyond the proposed invert level.
- ii) Twelve number of bore were drilled along the left blank tunnel alignment showed that there are some thin, impersistent fine grained sandstone bands where grouting would not be effective.
- iii) As tunneling is through shale, chances of improving tunneling condition is less. It may be difficult to complete the tunnel within the stipulated period. Due to insufficient rock cover, tunneling cost may be very high.

The decision of the 2nd TAC is described as under:

- a) In view of the nature of the soft rock and loose materials seen from log of bore holes and inadequate rock cover, tunneling does not appear to be easy and feasible solution. This may take considerable time.
- b) Depressing bed level o tunnel/conduit by 2 m is not desirable as silt may get accumulated and tunnel will not behave as a free flow tunnel.

- c) A consensus decision was taken to abandon the tunneling and go for open cut where NSL is RL 84.00m or less. Where NSL is above RL 84.00m, cut & cover section will be adopted with 4 no. of 4.8 X 5.25 m high barrels of modified horse shoe one. The cost economic will be worked out by the project to decide the actual section.

According to above observation of TAC, a comparative study was carried out by the consultant M/s NK-WAPCOS for open excavation for following alternative with different depths and different lengths of open cut.

Case 1 : Open excavation depth up to 20 m & rest RCC conduit.

Case 2 : Open excavation depth up to 22.5m & rest RCC conduit.

Case 3 : Open excavation depth up to 25 m & rest RCC conduit.

Case 4 : Open excavation depth up to 27.5 m & rest RCC conduit.

Case 5 : Open canal work for the entire length.

For the above alternatives, cost was evaluated and the details are presented below:

Table 7.1 Cost of various alternatives

Case	Max. depth of excavation for open canal work	Open canal length		Conduit portion		Total cost (Rs. Crores)
		Length (m)	Cost	Length (m)	Cost	
1	20.0 m	900 m	10.23	3100 m	166.01	176.24
2	22.5 m	2000 m	25.83	2000 m	110.88	136.71
3	25.0 m	2500 m	37.86	1500 m	83.31	121.17
4	27.5 m	3200 m	41.38	800 m	47.69	89.07
5	31.0 m	4000 m	74.23	0	0	74.23

It is found from the above cost calculations that the case No. 5 i.e. full open excavation for the entire length is the most economical, but it was not accepted for the following reasons:

- The maximum depth of excavation would have been 31.00 m. Slope protection for such a depth would have been cost prohibitive.
- Disposal of excavated materials would have created a serious problem as the land available for disposal of earth is limited.

The case No.4 is therefore selected for the water conductor system between RD 31.50 km to RD 35.50 km from economical consideration. The length is approximately 800 m for conduit and 3200 m for open excavation.

7.1.3.1.4 Planning for execution of Water Conductor System:

Table 7.2 Planning for water conductor system.

Chainage (Km)		Type	Length (m)
From	To		
31.500	32.200	Open excavation	700
32.200	32.225	Transition	25
32.225	32.975	Conduit	750
32.975	33.000	Transition	25
33.000	35.500	Open excavation	2500
Total			4000

The sequence of operation for concreting was planned in the following manner :

- i) Bottom portion of collar to be cast after laying mud mat.
- ii) Bottom slab to be cast in full length of 30m between two adjacent movement joints along with bottom haunch and 300mm height of wall from haunch.
- iii) Wall to be cast up to bottom of top haunch in two/three lifts.
- iv) Slab to be cast along with top haunch.
- v) Wall portion of collar to be cast from bottom to top on both sides.
- vi) Slab portion of the collar to be cast at the end.

A wing wall is provided at the entry and exit of the RCC conduit for 25 m long each side in the transition portion with side slopes changing from vertical to 1:1. The wing wall is of gravity section with M10 concrete.

The sections where shale and sandstone have been met with, slope protection is essential in these reaches. In shale there is likelihood of swelling on weathering which may accelerate slope collapse. Swelling occurs primarily due to presence of montmorillonite in shale.

Moreover bedding plain in right bank is dipping towards the canal. Unless adequate measures are taken, there is probability that sliding may occur. “(source : Water conductor system for Rengali LBC from 31.5 km to 35.5 km., Its selection & Construction by G.C. Sahu & S.K.Parida; Orissa Engineering Congress 50th Annual Session, Feb. 2005 pg. C-21 to C-32)

7.1.3.2 Proposed Tunnel from R.D. 100.49 km to R.D. 102.49 km on canal alignment of LMC

7.1.3.2.1 Feasibility Study :

The canal alignment between R.D. 100.490 Km and R.D. 102.490 Km of 141 km long Rengali Left Bank Canal taking off from Samal barrage passes through topographic levels ranging between 77 m and 90 m requiring deep excavation of the order of 20.501 m and 33.463 m. In order to avoid slope stability problem encountered in deep excavation, the Project Authorities contemplated an 8 m dia 2 Km long free flowing tunnel and requested Geological Survey of India to assess the feasibility of the scheme. The proposed tunnel alignments are located in lateritic peneplain with a few narrow elongated ridges (NW-SE trending) of quartzites dissected by three sets of joints. The proposed tunnel is expected to pass through mainly silicified metabasic with occasional feldspathic granite gneiss and serpentinite. Some restricted reaches of overburden are also expected to be encountered, particularly along the original alignment. The rock mass is jointed and largely weathered. Exploratory drilling carried out has not produced good results as there have been significant core losses in almost all holes giving rise to poor core recovery, which has made the interpretation of data for fruitful assessment of subsurface geological conditions. Core recovered from drilling of holes show poor to very poor RQD and generally low core recovery. Considering poor / very poor RQD values, largely weathered nature of rock mass and shallow depth of the proposed tunnel., the rock mass quality for tunneling can be categorized as poor to very poor. The total rock cover (including weathered rock) varies from nil to 2.5 D along the original alignment, 1.5D to 2.6 D along Alternative alignments I and II. However, along the original and Alternative Alignment I, there is possibility of intersecting overburden in the proposed tunnel with its crown at RL 64m. The fresh rock cover over the proposed tunnel along all alignments is generally less than 1D except some restricted tunnel sections. Considering the 4m to 8m depth of the ground water level measured in drill holes, low to moderate water seepage may be encountered during tunneling. The proposed tunnel along Alternative Alignment I and II appears to be feasible with the condition that the excavation must be done by some mechanical method , which may be a costly affair for such a short tunnel. In case, tunnel proposal is finalized, Alternative Alignment II should be preferred and the tunnel will have to be entirely supported by steel ribs with complete backfilling by concrete. Keeping in view all facts concerning rock mass conditions, engineering difficulties and economical concerns, open excavation and/or ‘cut and cover’ structure appears to be a better option and is recommended.

For open excavation / cut and cover structure, alternative alignments are better aligned than the original alignment. However, detailed geological study is recommended for stability analysis of cut slopes”.
(source : Feasibility study by GSI during 2008-09)

Table 7.5 Geological logging of the holes drilled along the proposed original and alternative tunnel alignments.

Hole No.	Location RD (m)	NSL (m)	Depth Drilled (m)	Weathered rock level (m)	Fresh rock level (m)	Fresh rock cover above crown (m)	Core recovery %	RQD	Lithology
Original alignment									
0-1	100.5	78.612	24.5	76.212 (2.4)	66.372 (12.24)	2.372	Nil-34	Nil	Silicified metabasic rock
0-2	100.6	79.312	20.6	77.812 (1.5)	68.872 (10.44)	4.872	Nil-36	Nil	Silicified metabasic rock
0-3	100.7	78.322	20.4	78.322 (Surface)	74.362 (3.96)	10.362	Nil-53	Nil-37	Silicified Metabasic rock and granite gneiss
0-4	100.8	78.092	21.2	78.092 (Surface)	67.272 (10.82)	3.272	Nil-54	Nil	Silicified metabasic rock
0-5	100.9	78.202	22.1	76.702 (1.5)	63.102 (15.1)	-	Nil-78	Nil-78	Silicified metabasic rock
0-6	101.0	78.622	17.0	77.122 (1.5)	70.842 (7.78)	6.842	Nil-24	Nil-15	Silicified metabasic rock
0-7	101.1	78.744	23.0	78.744 (Surface)	65.934 (12.81)	1.934	Nil-30	Nil	Silicified metabasic rock
0-8	101.2	79.682	23.0	79.182 (0.5)	64.942 (17.74)	0.942	Nil-40	Nil-27	Silicified metabasic rock
0-9	101.3	81.712	23.5	79.712 (2.0)	58.462 (23.25)	-	Nil-25	Nil	Silicified metabasic rock
0-10	101.4	84.112	28.0	022 (1.5)	62.352 (21.76)	-	Nil-38	Nil	Silicified metabasic rock
0-11	101.5	85.522	29.0	84.022 (1.5)	67.352 (18.17)	3.352	Nil-29	Nil	Silicified metabasic rock
0-12	101.16	87.582	30.5	84.582 (3.0)	-	-	Nil-22	Nil	Silicified metabasic rock
0-13	101.7	88.072	31.5	57.732 (30.34)	-	-	Nil-21	Nil	Silicified metabasic rock
0-14	101.8	88.872	31.5	72.372 (16.5)	-	-	Nil-28	Nil	Silicified metabasic rock

0-16	102.0	83.192	26.5	72.692 (10.5)	59.952 (23.24)	-	Nil-72	Nil-46	Silicified metabasic rock
0-17	102.1	82.942	26.5	75.442 (7.5)	57.472 (25.47)	-	Nil-50	Nil-50	Silicified metabasic rock
0-18	102.2	81.492	25.0	64.992 (16.5)	-	-	Nil-26	Nil	Silicified metabasic rock
0-19	102.3	82.212	25.5	76.212 (6.0)	-	-	Nil-43	Nil	Silicified metabasic rock
0-20	102.4	80.272	23.5	77.272 (3.0)	62.102 (18.17)	-	Nil-58	Nil-24	Silicified metabasic rock
0-21	102.5	78.932	22.4	75.912 (3.0)	-	-	Nil-35	Nil	Silicified metabasic rock
Alternative alignment – I									
AA-1	101.74	86.829	21.0	85.329 (1.5)	-	-	Nil-20	Nil	Silicified metabasic rock
AA-2	101.94	86.568	29.0	82.468 (4.1)	-	-	Nil	Nil	Silicified metabasic rock
AA-3	102.14	85.413	28.7	82.013 (3.4)	74.653 (10.76)	10.653	Nil-70	Nil-57	Silicified metabasic rock
AA-4	102.34	83.150	26.5	79.150 (4.0)	74.57 (8.58)	10.57	Nil-86	Nil-77	Silicified metabasic rock
AA-5	102.54	80.608	24.0	78.508 (2.1)	67.648 (12.96)	3.648	Nil-74	Nil-35	Silicified metabasic rock
Alternative alignment – II									
A2-1	101.627	84.476	30.5	82.376 (2.1)	61.446 (23.03)	-	Nil-70	Nil-24	Silicified metabasic rock
A2-2	101.827	88.811	31.9	84.311 (4.5)	65.391 (23.42)	1.391	Nil-73	Nil-65	Silicified metabasic rock
A2-3	102.027	89.636	32.0	85.136 (4.5)	71.826 (17.81)	7.826	Nil-58	Nil-50	Silicified metabasic rock
A2-4	102.264	87.395	30.8	83.975 (3.42)	71.575 (15.82)	7.575	Nil-79	Nil-74	Silicified metabasic rock
A2-5	102.500	84.560	28.0	79.14 (5.4)	61.15 (23.41)	-	Nil-63	Nil-47	Silicified metabasic rock

(Tunnel diameter 8m, Crown level R.L. 64.0 m)

Data of the above table show that weathered rock occurs at surface to below 30.34 m along original alignment, below 1.5m to 4.1 m along Alternative Alignment I between RD 101.74 km to RD 102.5 km. and below 2.1 m to 5.4 m along Alternate Alignment II from RD 101.6 Km to RD 102.5 km. Fresh rock occurs below 7.78 to > 31.5 m along original alignment, below 8.58m to > 28.7 m along Alternative Alignment I and below 17.81 m to 23.42 m along Alternative Alignment II. The main rock encountered in all holes is silicified metabasic.

7.1.3.2.2 Conclusion & Recommendation by G.S.I

“The proposed original and alternative alignments lie in a lateritic peneplain with few narrow elongated ridges (NW-SE trending) made up of hard jointed quartzite.

The proposed tunnel is expected to pass through mainly silicified, metabasic with occasional feldspathic granite gneiss and serpentinite. Some restricted reaches of overburden are also expected to be encountered, particularly along the original alignment. The rock mass is jointed and largely weathered.

Exploratory drilling carried out has not produced good result as there have been significant core losses in almost all holes giving rise to poor core recovery, which has made the interpretation of data for fruitful assessment of subsurface geological conditions.

Core recovered from drilling of holes show to very poor RQD and generally low core recovery. Considering poor / very poor RQD values, largely weathered nature of rock mass and shallow depth of the proposed tunnel, the rock mass quality for tunneling can be categorized as poor to very poor.

The total rock cover (including weathered rock) varies from nil to 2.5 D along the original alignment, 1.5D to 2.6D along Alternative alignments I and II. However, along Original and Alternative alignment I; there is possibility of intersecting overburden in the proposed tunnel with its crown at RL 64 m. The fresh rock cover over the proposed tunnel along all alignments is generally less than 1D except some restricted tunnel sections.

Considering the 4m to 8m depth of the ground water level measured in drill holes, low to moderate water seepage may be encountered during tunneling.

The proposed tunnel along Alternative Alignment I and II appears to be feasible with the condition that the excavation must be done by some mechanical method, which may be a costly affair for such a short tunnel. In case tunnel proposal is finalized, Alternative Alignment II should be preferred and the tunnel will have to be entirely supported by steel ribs with complete backfilling by concrete.

Keeping in view all facts concerning rock mass conditions, engineering difficulties and economical concerns, open excavation and / or ‘cut and cover’ structure appears to be a better option and is recommended. For open excavation / cut and cover structure, alternative alignments are better aligned than the original alignment. However, detailed geological study is recommended for stability analysis of cut slopes”. (Source : Feasibility Study by G.S.I during 2008-09).

7.1.3.2.3 Views of State T.A.C

The problem was posed by the Chief Engineer and Basin Manager, Brahmani Left Basin before the 73rd TAC held on 28.11.2011 for a technical decision. As the canal in the said reach passes through heavy cutting ranging from 20.5m to 33.5 m. depth, it is difficult to go for open excavation due to problems relating to stability of slope, rehabilitation etc.

Hence three options, viz, 1) open cut 2) cut & Cover, & 3) Tunnel have been studied in three alternative Canal alignments namely i) Original alignment, ii) Alternative Alignment – I & iii) Alternative Alignment II and a cost comparison has been made.

Finally CE & BM, BLB has recommended the Twin Tunnel proposal by approving the Alternative Alignment – out of the three alignments, But the Geologist has recommended for open excavation and/or “cut & cover” in any of the two alternative alignments as these are better aligned than the original one. Further the Geologist has stated that in case tunnel proposal is finalized – “Alternative Alignment – II”, should be preferred.

After detailed discussion, the TAC recommended to go for tunnel since the other two approaches i.e open cut or “cut & cover” are not possible as per the report of the CE&BM. However, the CE & BM has been advised to go for the tunnel with EPC mode of contract after thorough investigation and making the cost comparison between the different alternatives by considering all aspects and the prevailing acquisition cost of the land.

Various alternatives were examined by the chief Engineer from cost consideration and the details are furnished below.

Table 7.6 Cost / M of various alternatives

SI No.	Description of the Alternative.	Total Estimated Cost. (Rs. In Lakh)	Cost per M. (Rs in Lakh)
1	Open-cut canal excavation with lining.	06744	1.95
2A)	Cut & cover – Twin D-shaped Conduits.	12312	3.56
2B)	Cut & cover – Single D-shaped Conduit	1194	3.45
2C)	Cut & cover – Triple box section.	16463	4.76
3A)	Twin – tunnels – D-shaped with concrete lining	09986	2.88
3B)	Single tunnel – D-shaped with concrete – lining	10900	3.15

- Though the cost of alternative No. I is the lowest, one shall encounter problems of side slope stability & slope failures resulting in silt – deposition in the canal – bed and thereby affecting the functioning of the canal – system.
- Amongst the other alternatives, the execution of the cut and cover sections will be easy for execution posing lesser technical challenges. However this alternative entails higher cost outlays.
- The alternatives of either twin or single – tunnel section are worth considering with little variation in the costs.

The cost of twin tunnel appears to be a little lower. In this case of the roof stability can be better achieved than in case of the single-tunnel and under the circumstances shall be an ideal proposal for adoption. Its adoption, will also be advantageous from the operational point of view. For instance if any single tunnel needs some repairs / maintenance, the second one can be kept operational with at least 50 % discharge. The whole system on the d/s of the tunnel will not be adversely affected during such instance.

7.1.3.3 Excavation of R.M.C from RD 87.0 km to RD 89 km:

7.1.3.3.1 Geotechnical Investigation :

Similar situation was encountered in the canal alignment of R.M.C where the depth of excavation was about 20m to 30m. Since the open cut excavation may pose some serious problems like disposal of huge excavated material, loss of some cultivated land and stability of slope, the DOWR sought the expert opinion of GSI regarding the viability of tunnel. The feasibility was examined by GSI during 1998 from the five exploratory boreholes drilled along the proposed tunnel alignment. It is clear from the study of cores that the tunnel will pass through weathered, fractured, brownish khondalite represented by quartz sillimanite schist which is capped by deeply weathered khondalite and clay with thin inter bed of sand and gravel at the top. The depth of weathering seems to be more and at the exit portal site (R.D. – 89.700 km) and **2D cover of fresh rock is not available above the crown of the proposed tunnel as indicated by drilling.**

Thus it is apparent from these boreholes data that there is no sound rock cover above the crown of the tunnel at RD 86.190 km (B.H. - 5), only 2.19 m weathered rock cover at RD 87.00 km (B.H. - 4), only 4.132 m weathered rock cover at RD 87.780 km (B.H. - 1), almost nil rock cover at RD 88.560 km (B.H. - 3).

7.1.3.3.2 Conclusion:

- a. As the proposed tunnel alignment passes through a topographic saddle, rock is weathered up to and beyond tunnel bed level, adequate fresh rock cover above the crown of the tunnel is not available and tunneling is to be done below the water table, the present alignment is not considered as favorable for tunneling.
- b. Weathered Khondalite will form the tunneling medium. This is not considered as a good medium for tunneling.
- c. As deeply weathered Khondalite sometimes form lithomergic clay along shear zones or discontinuity plane in association with water and since the tunnel will be driven below the water table, the possibility of occurrences of flowing ground condition and formation of deep cavities cannot be ruled out.
- d. As rock cover is very low, tunneling by conventional blasting may give rise to cavity or chimney formation. Heading and benching method of tunneling is to be followed.
- e. Due to the absence of 2D rock cover above the crown of the tunnel, good arch action to sustain rockload may not develop causing tunnel collapse even before installation of support.
- f. The poor RQD and rock quality (Q) of the weathered Khondalite possibly indicates its low stand-up time. Hence, support is to be provided even before mucking operation.

These details were placed before the Technical Advisory Committee (TAC) on 10.12.99 for taking a decision.

7.1.3.3.3 Comments of T.A.C

Extract of 36th TAC held on 10.12.1999 at Bhubaneswar on Water conductor system on Right Bank Canal of R.I.P.

The proposal of tunnel from RD 87.50km to 88.70 km which comprises a part of the water conductor system from RD.79 km to RD.95 km was posed before TAC by CCE, Bramhani Rt. Basin. The tender for the above work has already been finalized and work awarded based on the design prepared by a consultancy firm and approved by the Chief Engineer, Brahmani Right Basin before March – 1999. The agreement for the above has already been drawn with the contracting agency on 17-03-99. C.W.C. New Delhi had earlier approved the alignment of RBC from 0 to 112 km. The alignment has been changed from RD.81 km and total length is now 95km. The alignment has been approved by Chief Engineer, B.R.Basin.

The technical feasibility of providing a tunnel was discussed at length considering the report of Sr. Geologist, G.S.I and the structural and hydraulic designs provided.

The observations are as follows:-

- i) Within the proposed 1200 mtr. Long tunnel only one drill hole (DH No.I) has been made which has a core recovery of 20% at crown and 40 % at soffit level of tunnel alignment. The entire run is weathered, fractured, Khondalite capped by a deeply weathered Khondalite and clay, hence a very poor medium for tunneling with very low stand up time. The Geologist has opined the alignment is not favorable for tunneling.
- ii) All along the alignment 2D cover over the crown is not available which will create problems during excavation of tunnel by roof failure and over breakage even before installation of supports.
- iii) Fixation of circular shape of 7m dia tunnel is arbitrary and appears not to have been based on economic dia study. Water way has been computed with Manning Equation, i.e for a free flowing tunnel, but no free board has been provided, thus making the design neither free flow nor a pressure tunnel inviting deposit of silt along the bed level of tunnel depressed by 3m. below the bed level of canal and gradually reducing the effective flow area.

- iv) The Tunnel alignment lies below lowest underground water table due to which profuse percolation will occur creating enormous problems of dewatering and removal of muck.

The engineering of construction of the tunnel was discussed in detail. The committee decided that it would be very difficult and time consuming to construct the tunnel.

As an alternative, it was agreed upon to go in for a cut & cover conduit, as adopted for the Left Bank Canal in the 35th TAC.

Necessary approval DoWR, GoO, may be obtained by C.C.E Brahmani Right Basin, before going for the cut & cover conduit design keeping in view the contractual complication if any.

7.1.3.3.4 – Observation of CWC:

Views of the CWC was sought in this matter in C.E., Brahmani Right Basin letter No. DB – 10/2000-01 – 7489 Dt 22.09.2001 & DB – 8857 / 15.11.2001. Their letter No. 2/264/96 – PP(C) vol . II /89 Dt. 11.03.2002 (Project preparation – Central, Directorate) is reproduced below.

“The Rengali Right main Canal takes off from Samal Barrage over Brahmani river. It runs almost parallel to river Brahmani in its left bank in between the river and fairly high land catering to the needs of a narrow long patch of land for a bout 91 km out of its total length of 112 km. The Canal has wider commands in its last 21 km, CWC New Delhi had earlier approved the alignment of R.B.C/ R.M.C from 0 to 112 km. The alignment has now been changed from R.D. 81 km. and total length of canal is now 95 km. The ground between RD 86.5 Km. to RD 88.5 Km. is high. The present proposal has been sent to examine whether to provide cut and cover section or tunnel in this reach. In the report submitted by Spatial Planning & Analysis Research Centre (P) Ltd, it is proposed to construct 7m dia tunnel with its bed depressed by about 3m below the bed of original trapezoidal section. From economic construction, since open cut beyond 20 metre depth is neither economical nor safe having several maintenance, operational and environmental problems. But in the 36th TAC meeting held at Bhubaneswar it was agreed to go in for a cut covers section. The Chief Engineer and Basin Manager, Brahmani Right Basin has forwarded the case to CWC for examination.

The report has been examined and our observation / comments are given below:-

This Dte is in agreement with the observation on item No.1 of the proceedings of the 36th TAC (Brahmani River) meeting held on 10.12.1999 at Dam Safety Conference Hall, Bhubaneswar.

Further the design of tunnel is not in accordance with the IS : 1884 (Part –I to V) – 1972. The geotechnical investigation carried out are quite inadequate.

The cut and cover section may be attractive proposition from the view point of easy construction in soft strata. The side of slopes for excavation for this reach be designed based on the geo – technical properties of various strata for a temporary phase and should be considerably steeper than what has been shown in the report, which are quite excessive. The hydraulic and structural design of barrel in the cut & cover reach should be as per relevant BIS codes.

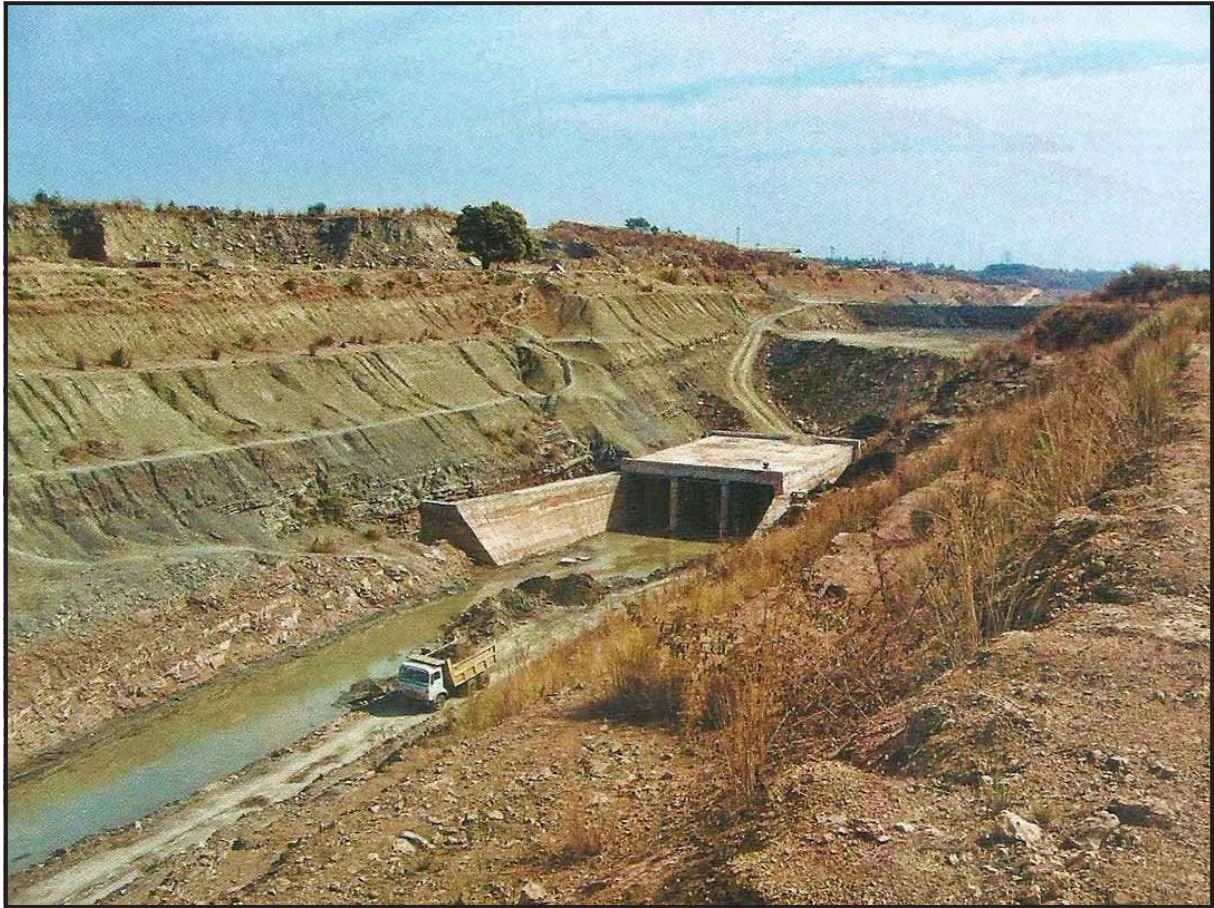
The load of over burden assumed in the design of the cut and cover section is not in accordance with codal provisions.

The cut & cover section also should have adequate free board over the FSL of the canal”.

Accordingly canal excavation is in progress for cut & cover section

7.1.3.3.5 – Geological Note on partly Excavated Canal:

Geological investigation was carried out on 23rd, 24th and 29th Feb. 2012 to observe the bed rock strata exposed after excavation on the slope stability, mode of excavation and utilization of the excavated material from RD 86.50 km to RD 88.70 km. The proposed material from RD The proposed water conductor system will be cut and cover one from RD 87.50 km to RD 88.70 km (i.e. for 1.2 km length).



View of Cut and Cover provided at the deep cutting section of LMC



Excavation of RMC from RD.87 km to 89 km in progress

“The upstream excavated stretch between RSs 86500m – 87350m exposed mainly moderate to highly weathered khondalite. The bed rock is defined by pink to pinkish brown coloured Khondalite with later mafic veins and dykes. The soil mass thickness vary from 2.0m to 3.0m. The later intrusive generally define network of veins almost through out the Khondalite rockmass. The veins vary in thickness from a few millimeters to ten centimeters. They are generally weathered to white to greenish clay mass. Minor bands of white quartzo – feldspathic bands are also present in the upstream part. In general, the rock mass condition improves towards downstream with the decrease in the intensity of the clay vein network. This has resulted in chaotic admixture of moderately weathered and highly weathered rock mass occurrences. The site officials have reported a difficulty in excavation of such a rock mass by way of existing mechanized excavation for which site demo was also undertaken at places. It was observed that at most of the places the excavator’s (Kobelco 210) bucket could produce only 5-10cm deep grooves instead of strapping the surface. In this regard, based on above observation it has been felt that as the excavation has to go further deeper by another 15m the blasting can be one of the best mode to facilitate the excavation process. However, in view of the chaotic admixture of moderately weathered stiff and D.I. rock masses, their amenability to blasting has to be tested by trial blasts in selected sections. However, it is also suggested that in addition to the index properties, the rippability and blastability index may also be determined. It is suggested that the interpretation of data and the trial blast must be conducted in consultations with the concerned experts and the geologist. The excavated stretch between RD 87350 m – 87900 m was water filled and hence could not be observed.

The ongoing excavation in the proposed cut and covers section between RDs 87500m – 88700m at existing average depth of 12.0 m also exposed mainly weathered Khondalite with frequent veins and seams of clay together classified as D.I. rock mass. The top soil thickness varies between 3.0m to 5m and generally decreases towards downstream. The similar type of strata is expected to continue further deep, however, the same may require review assessment of the rock mass condition at regular interval. The resorted mechanized excavation appears feasible in the present condition. The designed slopes provided in the both excavations have not been seen with any failure or development of tension cracks. However, this would also require regular monitoring with the progress of excavation.

The strata, in general appear to define a low strength with poor to fair type rock mass which disintegrate easily on handling. With regards to overall moderate to highly weathered nature of the rock masses, they appear not suitable for any engineering uses except for the fill material. However, in this regard the final conclusion may be arrived after reviewing its geotechnical properties. For the purpose, it is suggested to carry out the relevant geotechnical tests like the index properties, water absorption, crushing strength, wetting-drying tests etc”.

7.1.4 Problems arising due to passing of R.M.C through Talcher town and Coallier Area

The Director, Mines safety expressed concern that failure of unstable strata may have a cascading effect thereby causing settlement of canal bed due to subsidence of strata. This may cause crack in the canal bed as a result canal water would seep / flow into the underground coal mine leading to disaster. As the safety of Talcher town and underground working coal mines was involved a discussion was made between the official of DoWR , Mahanadi Coal Limited and Director Mines safety on 02.08.2006 in the office of the Secretary, DoWR when following points came up for discussion.

- i) MCL authorities had mentioned that the extraction of coal from the Deulbera Mines would take another two years and would be completed by 28.02.05. In the portion where the canal passes, the coal had already been extracted and the area abandoned and sealed off.

- ii) DoWR would give notice to MCL authorities at least one month prior to release of water in Right Bank Canal of R.I.P.
- iii) Safety concerns had been expressed by Director, Mines safety in the meeting held on 28.02.2013. The Director, Mines Safety had suggested in the 2003 meeting that DoWR should assess the stability of the canal afresh.

The Deputy Director, Mines repeated the concerns expressed by the Director in the earlier meeting. He mentioned that the failure of any unstable stooks was likely to have a cascading effect; failure of stooks would cause settlement of canal bed due to subsidence of strata; pot holing may cause cracks in the canal bed and this may result in canal water seeping / flowing into the underground working of the mine, which may cause loss of lives and property. Besides there is danger to the structural stability of the canal.

The perception of the engineers of water Resources Department was somewhat different.

The following observations were made:

- i) It is usual phenomenon in the said reach of Right bank Canal (from RD 19.00 Km to 19.70 Km) that water has been accumulating for a depth of 2.00m to 2.50m during the rainy season for the last five years. No variation in the seepage in the underground mines has ever been reported either by Mahanadi Coal Field Limited authorities or by Director, Mines Safety so far (ii) A railway line runs parallel to Right Bank Canal which is aligned over the underground mine and there is no report of subsidence owing to movement of railway traffic.

CE & BM, Brahmani Right Basin stated that he had informed MCL authorities vide his letter No. 12783 dt. 03.12.2005 that water would be released in the canal in the month of June 2006. However MCL authorities in their letter dated 14.07.2006 addressed to CE & BM, Brahmani Right Basin requested to defer release of water in R.B.C till November 2006.

The Deputy Director, Mines Safety expressed serious concern that MCL authorities have not done proper instrumentation and have not monitored the underground seepage and measurement of subsidence of support (stooks) if any.

It was felt that Director, Mines safety and MCL authorities should have kept Government informed of the risk factors due to growth of the township and establishment of structures in the areas where underground mining has been done, although they have been informing the local revenue authorities and local self Government authorities from time to time.

The General Manager, Talcher area of Mahanadi Coalfield Limited intimated that due to some practical difficulties they have not been able to stow sand as yet. However, they are planning to start it from the middle of September 2006 and complete the above work by end of November 2006.

The following points were mentioned during discussion.

1. The scientist from Central Mining Research Institute (CMRI), Dhanbad indicated that the stooks are of size 6.50m X 6.50m X 2.40m each which provide large stiffness to the support system. He also observed that in case there is any leakage due to subsidence of canal bed or seepage from the canal to the underground mines, it would reduce the stress in the support system due to buoyancy effect. The General Manager Talcher area stated that there is no indication of pothole so far.
2. The Scientist from the CMRI, Dhanbad and the general Manager, MCL mentioned that as a short term safety measure, the underground mines can be filled with water. However, stowing of sand should also be done as early as possible. The MCL authorities clearly mentioned that under the canal there was no mining activity going on and there was no threat to life of colliery workers due to release of water into the canal, It could not be clearly ascertained whether in

case of failure of the canal, the nearby areas would be flooded and life and property in Talcher town close to the canal would be at risk. In any case such risk appeared to be negligible.

There was agreement on the following main points.

1. MCL authorities would complete sand stowing work to ensure safety against subsidence and pot holing latest by 30th November 2006.
2. The MCL authorities would furnish at once the implementation schedule on the above safety measures to the Department of Water Resources, Collector & D.M., Angul and CE & BM, Brahmani Right Basin, Dhenkanal for the purpose of monitoring the progress of work.
3. The Collector and D.M, Angul and the Chief Engineer & B.M, Brahmani Right Basin, Dhenkanal will monitor progress of the work closely. The Collector Angul would provide necessary assistance required by the MCL authorities to complete the work in time.
4. After the completion of sand stowing in the mines, MCL authorities would obtain certificate from Director, Mines Safety by 30th November 2006 and inform Chief Engineer & Basin Manager, Brahmani Right Basin, Dhenkanal latest by the second week of December without fail.
5. Though there did not appear to be much risk if water was released into canal in view of the safeguards already provided, it would be advisable not to release water in the canal till and of November 2006 as a matter of abundant caution”.

7.1.4.1 Progress of Sand Stowing:

Progress of Sand stowing in the underground portion of Deulbera Colliery after coal mining is furnished in Table No. 7.7 which was taken up as a safety measure against any untoward incident.

Table No. 7.7 : Quantity of Sand Stowing in Deulbera Colliery.

Sl No.	Date	Cumulative Qty. of sand Stowing (cum)	Remark
1	06.01.2007	3300	In the B.H near RD 19025m of RMC
2	20.01.2007	3630	-do -
3	27.01.2007	4835	Underground area of Deulbera colliery
4	03.02.2007	5315	-do -

7.2 Large dam 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.

7.2.1 Advantage of Large dams

“The greatest inexplicable paradox of this country's water resources scenario has been the co-existence of droughts and floods and the beauty of the large projects is that they have the capability to handle both the situations simultaneously and thus ironing out the erratic climatic behavior to a large extent.

Small projects, per-se cannot by themselves, adequately intercept the large quantum of rainfall available for exploitation in a sustained manner. Economy of scale, much lesser evaporation and seepage losses, the safety and flood absorption potential, are only available with large storage projects. However, small dams can be considered as worthy supplements but not substitutes for large dams. The storage behind large dams in any basin ensures that there is a cushion of reserve as well as in-built carry-over storage. In view of significant storage capacity, the large dams can have adequate flood cushion to moderate heavy floods. Further even without specifically earmarked flood storage, the flood moderation is being achieved in large dams by suitable regulation of out flows. Of course, before taking a decision of construction of large dams, all other alternatives like several smaller dams or cascade development schemes need to be considered in detail and compared with the large dam options.

Water is also a source of energy. The large dams and reservoirs can help generate considerable hydro –power which also helps in enhancing the economic viability of the project. In fact, most of the existing dams provide multiple benefits. Thus for harnessing remaining water resources, apart from irrigation and drinking water supply, energy generation can also be provided, where feasible, thereby facilitating proper hydro-thermal mix for improving operational reliability.

It has been seen that the existing dams have also significantly increased economic activity in the catchment areas, because of increased tempo of construction activities , commerce and tertiary development as well as promotion of tourism etc”. (Source : Water Resources Development Scenario in India, CWC, MoWR, 2012 pg.77)

The construction of large dams in India and elsewhere is being criticized mainly on the ground that it involves:

- Displacement of people from their traditional homes due to submergence caused by large storage 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 storage 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 7.8 shows submerged area and storage capacity of few major dams of the country.

Table 7.8 Submerged Area Vs Storage Capacity

Name of the 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

Further submerged area Vs storage capacity of few Major (C.C.A more than 10,000 ha) and medium (between 2000 to 10,000 ha) Projects of Odisha are shown in Table 7.9 & 7.10 respectively.

Moreover, small storages are unable to provide assured supplies during lean period 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 tribal’s?”

7.2.2 Impact of Water Storages:

The impact of water storages on various sectors of development has changed the socio-economic scenario 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 place of such activities accelerated after independence from colonial rule. Increased agriculture 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 : Bhakra Nangal Project Socio Economic & Environmental Impacts by R.Rangachari 2006, pg. 5)

“There is direct relationship between food production and assured irrigation. If we concentrate over the states which have outperformed in the food production like Punjab, Haryana, Gujarat, Uttar Pradesh, Madhya Pradesh, Maharashtra, West Bengal, it is apparent that the large and medium water resources storage projects have played the key role. Such remarkable achievement have been possible due to the fact large areas of these States are being covered under assured irrigation.

India’s food grain production has increased manifold since 1951. Even though other methods of harvesting water of irrigation, such as ground water, small schemes etc, remain all pervasive, these cannot be relied upon to entirely meet the needs of India’s large and growing population. Moreover, these forms of water harvesting are not cost-effective and do not have the added advantages of hydropower generation and flood management”. (Source: Water Resources Development Scenario in India, CWC, 2012 pg.77)

7.2.3 Need for Changed Perspective

“There is no denying the fact that any dislocation of habitat affects the human being both mentally as well as physically. The initial experiences are always full of trauma and anxiety. Compared to such inconvenience, the PAFs have to face for a short duration, the accruing benefits from the implementation of the water resources projects are much more which are likely to improve their own economic and social well being. Besides the number of people that are likely to be benefited, is manifold.

Water resources projects are not only for the economic development of the region but also for the social uplifting of the weaker sections of the society especially the women, and therefore, need to be viewed in that perspective. There cannot be a better public purpose than construction of a water resources project, which not only, is beneficial to all sections of the society in a given region but is also in the overall national interest of food security and economic development.

The provision of land for the PAPs in the command area of the project should now serve as a sufficient incentive to agree to part away with their lands. The principle of “beneficiaries pay” for the “benefits harvested” by them needs to be applied. Those in the command area who are likely to benefit from the project shall have to be involved in chalking out the R & R plan and made to spare the land for settling the PAPs and amalgamate them into the social fabric.

Unfortunately the entire issue of R&R is being attempted to be emotionalized with a smear campaign against the water resources projects specially the large dams. An attempt is being made to make dams the villain of all migrations. The inappropriateness of putting cash in the hands of tribal people as compensation for land and other properties could not be foreseen in the early stage of evolution of R&R even by the sociologists. However it would be illogical to lay the blame of such aberrations at the doorstep of large dams. The delay in providing satisfactory rehabilitation has cost the country much more in escalation of the costs of the projects held-up than expected cost of such rehabilitation.

With the experience in the process of rehabilitation, the process of R&R has evolved into a more humane and socio –economically viable proposition. In some of the ongoing projects like Sardar Sarovar Project and Tehri Dam Project in the country, the R&R package offered by the project authorities/states is far more liberal than that provided in the Draft National Policy. The mechanism for implementation of R&R plans have also been institutionalized.

It is a well known fact that the poor countries are not poor because of the lack of resources but because of the poor management of the resources. Unless water resources of a country are harnessed and developed, it cannot provide for drinking water, food, energy and shelter needs of its citizens”. (Source : Ibid, pg.87)

“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 resources development requires a judicious mix of large, medium or small reservoir, 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 Indian Water Resources Society 2002 pg.4-5)

7.2.4 Large Dams – Gains & Losses:

A seminar on “Large Dams gains & Losses” was organized by Indian water Resources Society (IWRS), Roorkee on 10.11.1999. Extract of recommendations of the seminar 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 outweighed 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 a 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."

7.2.5 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. They have been precious few, if any comprehensive independent analyzes 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 (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 township and rural huts without running water, lights, or sanitation"*. (Source: Socio – Economic and Environmental Impacts of Bhakra - Nangal Project by R.Rangachari pg.5)

Table 7.9 - Submerged Area Vs Storage Capacity of few Major Irrigation Projects (Odisha)

Sl No	Feature	Hirakud Dam 1	Rengali Dam 2	Balimela Dam 3	Jalaput Dam 4	Indravati Dam 5	Upper Kolab Dam 6	Lower Indra Dam 7	Ib Dam 8
1	Basin	Mahanadi	Brahmani	Godavari	Godavari	Godavari	Godavari	Mahanadi	Mahanadi
2	Sub-Basin	Mahanadi	Brahmani	Sileru	Machkund	Indravati	Kolab	Indra	Ib
3	Catchment Area (Sq.km)	83,400	25,250	4912	1963	2630 (4 dams)	1630	1093	5649
4	Design Flood (Cumec)	42,469	27,800	14,300	5660	23030	7650	6831	31900
5	Gross Storage (Mm ³)	8105	4400	3822.80	970.36	2300	1215	321.63	1610.11
6	Live Storage (mm ³)	5818	3452	2880.00	892.54	1485.5	935	314.25	1453.30
7	Water spread Area (ha)	74300	41400	17512	9712	11000	11350	4766	12562
8	Type of Dam	Composite	Masonry	Earth fill	Masonry	Masonry & earth	Masonry	Earth Fill	Earth Fill
9	G.C.A (ha)	251454	436179	101700	-	135740	55679	37390	145588
10	C.C.A (ha)	155660	218392	61035	-	109300	44544	29900	106279
11	Area submerged per 1000 cum of storage (ha X 10 ⁻³)	9.13	9.41	4.58	9.45	4.78	9.34	14.82	7.8

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Table 7.9 - Submerged Area Vs Storage Capacity of few Major Irrigation Projects (Odisha)

Sl No	Feature	Ong Dam	Lower Suktel	Brutang Dam	Haldia Dam	Jambhira Dam	Baura dam	Kanupur Dam	Salandi Dam
		9	10	11	12	13	14	15	16
1	Basin	Mahanadi	Mahanadi	Mahanadi	Subarnarekha	Subarnarekha	Subarnarekha	Baitarani	Baitarani
2	Sub-Basin	Ong	Suktel	Brutanga	Subarnarekha	Subarnarekha	Subarnarekha	Baitarni	Salandi
3	Catchment Area (Sq.km)	2321	1275	725	55	76.5	67	1560	673
4	Design Flood (Cumec)	12730	8540	5683	1054.4	1457.8	893	14450	5140
5	Grass Storage (M m ³)	330	320.28	242.5	58.1	223.65	261.31	331.02	601
6	Live Storage (m m ³)	289	263.43	221.6	46.87	204.77	227.31	-	556.5
7	Water spread Area(ha)	5100	5216	2210	968	2928	2759	2600	3182
8	Type of Dam	Earth Fill	Earth Fill	Earth Fill	Earth Fill	Earth Fill	Earth Fill	Earth Fill	Composite
9	G.C.A (ha)	40000	32857	31110	170263	-	-	39440	107380
10	C.C.A (ha)	30000	31830	23300	109627	-	-	29580	92987
11	Area submerged per 1000 cum of storage (ha X 10 ³)	15.45	16.28	9.11	16.66	13.09	10.56	7.85	5.29

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Table 7.10 Submerged Area Vs Storage Capacity of Few Medium Irrigation Projects (Odisha)

Sl. No	Feature	Sundar Dam	Dumer bahal	Upper Jonk	Talasara	Titilagarh	Harihar Jore	Manjore	Pilasalki	Gohira	Ramiala
		1	2	3	4	5	6	7	8	9	10
1	Basin	Mahanadi	Mahanadi	Mahanadi	Mahanadi	Mahanadi	Mahanadi	Mahanadi	Mahanadi	Brahmani	Brahmani
2	Sub-basin	Sundar	Dumerbahal	Jonk	Talasara	Kankada jore	Harihar Jore	Manjore	Pilasalki	Gohira	Ramiala
3	Catchment Area (Sq.km)	145	90	342	91	71.2	425	452	87.36	236	328
4	Design flood (cum ec)	312.5	815	3555	820	850	3785	4333	709	2550	1698
5	Gross storage (M m ³)	26.35	22.2	80.5	19.85	13.38	79.88	100.77	17.53	79.4	86
6	Live storage (M m ³)	23.55	19.32	64.5	16.65	10.86	58.68	85.24	14.45	72.0	69.4
7	Water Spread Area (ha)	698	755.75	1470	351	435	1845	1460.55	524.8	1094	1669
8	Type of dam	Earth Fill	Earth Fill	Earth Fill	Earth Fill	Earth Fill	Earth Fill	Earth Fill	Earth Fill	Earth Fill	Earth Fill
9	G.C.A (ha)	5790	3590	11670	4340	3400	13420	5731	3320	11750	14065
10	C.C.A (ha)	4530	2800	9920	3040	2600	9950	4585	2390	8100	9600
11	Area submerged per 1000 cum of storage (ha X 10 ⁻³)	26.49	34.42	18.26	17.68	32.51	23.10	14.49	29.94	13.78	19.41

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Table 7.10 Submerged Area Vs Storage capacities of few Medium Irrigation Projects (Odisha)

Sl No	Feature	Khadkei 11	Kuanria 12	Bankbahal 13	Remal 14	Deo 15	Kanjbori 16	Kansbahal 17	Pita mahal 18	Sapua 19
1	Basin	Subamarekha	Mahanadi	Subamarekha	Baitarani	Baitarani	Baitarani	Brahmani	Brahmani	Brahmani
2	Sub-basin	Khadkei	Kuanria	Bankbahal	Remal	Deo	Kanjbori	Barjori nala	Pita Mahal	Sapua
3	Catchment Area (sq.km)	212.5	124	168.5	98.5	295	358	179	103.6	52
4	Design Flood (cumec)	2240	1812	1970	1037	2530	2286	1745	716	535.6
5	Gross storage (Mm ³)	63.6	22	28.26	19.26	75.82	40.52	40.415	23.62	8.5
6	Live storage (Mm ³)	56.22	17.5	25.78	15.70	68.44	34.52	28.72	20.16	6.46
7	Water spread Area (ha)	980	485.5	770	198	847.5	516.4	506	385	110
8	Type of Dam	Earth Fill	Earth Fill	Earth Fill	Earth Fill	Composite	Earth Fill	Earth Fill	Earth Fill	Earth Fill
9	G.C.A (ha)	10560	4800	9600	5500	14140	14410	7210	3250	2070
10	C.C.A (ha)	7990	3600	7200	4705	9900	9800	5050	2630	1570
11	Area Submerged per 1000 cum of storage (ha X 10 ⁻³)	15.41	22.07	27.25	10.28	11.18	12.74	12.52	16.30	12.94

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Table 7.10 Submerged Area Vrs Storage Capacities of Few medium Irrigation Projects (Odisha)

Sl. No.	Feature	Dadaraghati	Darjang	Daha	Baghua	Baghua Stg-II	Salia	Badanala	Harabhang	Bhaskel	Sunei	Kalo
		20	21	22	23	24	25	26	27	28	29	30
1	Basin	Brahmani	Brahmani	Rusikulya	Rusikulya	Rusikulya	Salia	Vansadhara	Vansadhara	Godavari	Budhabalanga	Budhabalanga
2	Sub-basin	Dadaraghati	Lingara	Daha	Baghua	Baghua Stg-II	Salia	Badanala	Harabhang	Bhaskel	Sunei	Kalo
3	Catchment Area (sq.km)	102.4	399	167	171	150	245	352	503.80	87	227	151
4	Design flood (cumec)	718.82	3592	1160	1315	NA	1019.42	4026	2842	566	1865	2476
5	Gross storage (Mm ³)	27.75	51.13	28	33.9	37.5	60.65	75.64	141.25	29.825	61.6	29.6
6	Live storage (Mm ³)	24.05	16.48	21.95	27.4		52.75	67.14	100.25	27.85	55	24.15
7	Water spread Area (ha)	759	943.37	552	1000	869	984	930	1215	642.26	682	534
8	Type of	Masonry	Earth Fill	Earth Fill	Earth Fill	Earth Fill	Earth Fill	Earth Fill	Earth Fill	Earth Fill	Earth Fill	Earth Fill
9	G.C.A. (ha)	5164	14532	7850	4050	10000	9710	13060	12870	4820	12500	7155
10	C.C.A. (ha)	4510	8400	4760	3240	8000	8310	9800	9650	4250	10000	4800
11	Submerged per 1000 cum of storage (haX10 ⁻³)	27.35	18.45	19.71	29.50	23.17	16.22	12.30	8.60	21.53	11.07	18.04

7.3 Issues & Concerns :

7.3.1 Research activities

“There is no single and absolute method by which the safety of a dam can be defined. The subject is diverse and manifold, with the result that several approaches are being researched and applied. This could be considered unfortunate, because a uniform approach would seem logical; however, this would inevitably restrict the quality of safety evaluation. The number, age, size and geographical distribution of the various dam types further contribute to the complexity of the issue.

On the one hand, there is the need for common understanding, precise definitions, and accepted technology; on the other hand, there is the need for research coupled with improvement, refinement and validation of emerging methodologies and concepts.

If we take as an example recent progress in numerical modelling, mainly resulting from the power of the modern computer, the analytical capabilities offered by the finite element method, and the advance in solution techniques and material models, we become aware of the potential capabilities for the analysis of complex loading and project conditions. Thus, the challenge offered by such refinements appears to be irresistible, and the potential solutions endless. In theory a great number of scenarios can now be investigated, but few solutions can now be investigated, but few solutions can claim to account fully for variability of load and material, or to be free from uncertainty inherent in the modelling process.

To exclude uncertainty, or to consider and account for it, is another topic under debate. Perhaps the response should be to adopt reliability analysis. Recent progress in this field has also provided elaborate models for the safety evaluation of common civil structures. But here, again, the advances in theory need to be matched with practical application. This method would allow for the consideration of uncertainty; its qualification is, however, most difficult. One is reminded of Albert Einstein’s aphorism: “As far as laws of mathematics refer to reality, they are not certain; as far as they are certain, they do not refer to reality.”

From this wealth of research and information comes the responsibility to apply new models in practice. Unchecked, the gap between use and theory will only widen. It is the duty of the profession to prevent this from happening; this requires a vigorous interdisciplinary exchange between engineers, consultants and scientists.

The number of new dams continues to increase, as to do the demands imposed by higher dams subjected to greater design loads for flood and earthquake. At the same time, and perhaps more significantly, there are more aging dams (those built more than 50 years ago). To date, according to ICOLD’s World Register of Dams, some 5000 dams now in operation were built before 1940. Updating data for probable maximum flood and maximum credible earthquake conditions inevitably results in a re-evaluation of structural safety, in addition to the ongoing safety assessment resulting from operational monitoring.

Each dam is an ambassador of the profession; the structure’s performance is open to public scrutiny. Engineers should be aware of all the tools available for evaluation, and these should be used for the continued investigation and confirmation of the safety of each dam.

Dam safety evaluation thus represents a multitude of questions, conflicting issues and opinions. Continuing research and experience is producing ever more complex solutions”. (Source proceedings of workshop on Dam Safety Assurance and Rehabilitation, 18-21 June 1996, Bhubaneswar, ‘Research needs in Dam Safety by J.F. Mistry’, Pg. XI-1)

Rengali is a gravity dam founded on sound rock where foundation failure due to settlement and sliding does not arise. Also the dam is safe from hydrological failure. Other reasons are either seepage through foundation or seepage through body of the dam. Failure due to sudden strong earthquake, through remote cannot be ignored.

The incidence of seepage and uplift pressures within the dam due to possible penetration / percolation of water through the body or foundation of the masonry / concrete dam along the mortar joints, lift joints, contraction joints, construction joints, cracks and contact with the foundation etc. is an important phenomenon and one of the governing factors for safe design of the dam. Control and monitoring of this seepage are essential to evaluate the safety factor of assurance. The seepage or leakage water causes uplift, erosion, leaching of lime from mortar and concrete, deterioration of the structure due to weathering and would lead to distress; hence a matter of concern.

7.3.1.1 Case studies of few distressed gravity dams in India.

a) Bhandardara Dam

It is a 82m high gravity structure built in rubble masonry with lime mortar in 1926. The design does not meet the criteria for uplift pressure and no drainage gallery or other drainage arrangement was provided.

During 1960, it was recommended to augment the spillway capacity and grout the masonry dam to reduce the seepage. On 10th Sept. 1969, first sign of visible distress was noticed. A 15 cm. (6" dia) drain outlet emerging from one plumb bob shaft suddenly started leaking @ 0.87 cumec and water was coming like a jet projection 6m d/s of the dam. Removal of the backfill at the toe showed a sheet of water gushing through the contact of masonry and the foundation. Studies indicated that under full uplift conditions, tensile stress was of the order of 5kg/sq.cm. This was expected at 50m from the top which indicated every possibility of horizontal crack there with water level just 0.3m below the top of the dam. The crack initiated at the downstream had thrown increasing load on the downstream face. The series of the photo elastic tests showed that once the crack had reached the base of the dam, the mortar would tend to crush and would be washed away by the high hydrostatic pressure of water behind it, resulting in the development of vertical crack on the downstream face wherever such loss of spot had occurred. The height of these cracks would depend on the width at the base along with the contact is affected.

The temporary remedial measures included grouting near the upstream providing drainage holes from d/s face, anchoring the dam by prestressing cables and providing dowel bars across the main crack, in addition to control of reservoir filling. The permanent remedial measure included strengthening of the dam by providing massive masonry buttresses on the d/s and sealing upstream by epoxy and guniting over a steel mesh.

b) Chikkahole Dam:

“Another structure which developed distress was the 30 m high chikkahole dam, made or handlaid masonry bonded with lime surkhi mortar. It was founded on weathered gneisses. Consolidation grouting was done to improve the foundation condition. In 1972, the dam breached suddenly at its right abutment. Due to low bond strength, the joints in the masonry on the upstream face had opened. Patches of grout deposits were seen on the top of the masonry left in the breach. The masonry had apparently developed a horizontal crack on the upstream face due to uplift caused by the grouting. The extensive horizontal tensile cracking had transferred high compressive stresses to the downstream face, causing collapse of the masonry.

c) Kadana Dam:

Kadana Dam across the river Mahi in Gujarat State is a composite dam with the maximum height of the masonry dam about 66m. Considerable leakage of water through the block joints in the drainage of foundation gallery in the block Nos 13, 14, 16 and 17 of the spillway was observed. The leakage was measured during Feb.March-1981 and was found to vary from 26 Lpm to 50 Lpm in blocks 16 & 17. Thus, the leakage was considerably high. It was not possible to detect whether this leakage was through joints or from the U/s masonry face. Considering the various possibilities of the sources of leakage, the following measures were suggested.

- i) The leakage may be due to improper bond between water stops and concrete at the contraction joint. It was, therefore, suggested that asphalt contained in 6" X 6" well between the two water stops may be heated by passing steam through the 'U' pipe provided in the asphalt well in the joint and new asphalt may be added, if required.
- ii) As the seepage was coming out from the contraction joint exposed in the gallery, this might be due to clogging of porous vertical drains provided in the body of the dam. It was, therefore, suggested to clean the vertical drainage pipes adjacent to the joints by reaming or chemical treatment.
- iii) Seepage water observed from the bottom of the gallery joint indicated also the possibility of choking of the foundation drainage holes. Hence, it was suggested to redrill the drainage holes adjacent to the joint.
- iv) The treatment was carried out and the leakage was considerable reduced.

d) Rihand Dam (U.P)

Rihand Dam which is concrete gravity dam is located on river Rihand in District Mirzapur of Uttar Pradesh. It is 91.46 metres high, 934.5m long and was constructed during the period 1954-1962.

Materials used for construction:-

Cement –(OPC) obtained from Churck Cement Works (U.P) sand-Blend of crushed sand mixed with river sand having FM of 2.3 to 2.9 Coarse aggregate obtained from Makra which consisted mostly of granite and granite gneiss rock. Pozzolana- Bokaro flyash to replace 15% by weight of cement. Admixture Indigenous AEA developed at Rihand Laboratory.

Distress Observed:

- i) Longitudinal cracks on upstream face observed from 1972 above RL 830 mostly along the lift joints and minor cracks between lift joints. The width of cracks varied from 1 mm to 25 mm. and approximate depths of cracks at various locations were found to vary from 7 to 45 cm by ultrasonic pulses velocity test. Horizontal cracks were also observed in the walls of the foundation gallery, the sluice operating gallery and the hoist operating gallery.
- ii) Horizontal and Vertical cracks observed in scroll casing walls of all the six generating units.
- iii) Cracks in all the 24 columns of penstock gallery.
- iv) Map cracking in the spillway piers and the radial gate pedestals.
- v) Horizontal and vertical swaying of the gantry crane out of use.
- vi) Vertical cracks at abutments adjacent to spillway crest.
- vii) Cracks in passenger and freight elevator shafts.
- viii) Cracks at tainter gate pedestals.
- ix) Tilting and deformation of draft tube structure and cracking of generator supports.

Findings of the National Council for Cement and Building Material(NCBM):

Extensive investigations conducted by the NCBM indicated that the concrete had adequate cement content and not attacked by sulphates, acid water etc. Non- destructive evaluation of concrete indicated overall quality of concrete to be generally good.

Aggregate obtained from the body of the dam and also from the quarry contained strained quartz and alkali feldspar. Investigation carried out on concrete cores by electron microscopy revealed presence of alkali aggregate reaction. Concrete core samples in both the main dam as well as power house structure were found to have undergone alkali –silica reaction, the unmistakable presence of such deleterious reactions being manifest by the occurrence of gel type reaction products inside concrete, dark and white reaction rims and alteration of borders of aggregate and presence of micro cracks in the mortar phase as examined visually, petrographically and in a scanning electron microscope.

e) Deep Scour in the d/s of Ukai Spillway:

The Ukai Spillway consists of 22 gates catering on outflow of 39,333 cumec. For energy dissipation arrangement, a ski-jump bucket is provided. A 240 m long tail channel meets the river in the downstream. The tail channel bed level is at R.L 52m. As per the CW & PRS studies, the scour was anticipated down to RL 28 m for the outflow of 39,330 cumec.

During the operation of the spillway in 1960 and 1970, before the gates were installed, deep erosion took place in the weathered rock and overburden. Two pits about 8m deep and 50m² and 24m² in size were formed in the impact area respectively about 64 m and 94m d/s of the protective apron. The condition did not materially alter during 1971 and 1972. The gates were installed in 1973. A deep scour hole was noticed in 1974, 100m d/s of the apron of blocks 9,10 and 11. The deepest level reached was 24 m indicating 29 m scour in the dolerite dyke region.

In 1975, another scour hole was noticed below blocks 17, 18 and 19 reaching R.L 36 m. A third prominent scour hole was noticed in 1976, below block 12, 13, and 14 reaching R.L 32 m. Subsequently, with passing of flow through various gates these scour holes were either deepened or filled up year to year. In 1978 the spillway discharged 12,750 cumec through 11 gates only which increased the scour. As fewer gates were operated, the return flow behind the closed gates caused extensive erosion along the tail channel bed and both the training walls. Even though the basalt is thick in this area, the standing pool of water over a period of 8 years resulted in opening of joints which existed about 0.5 m to 2 m apart. The trajectory of the ski-jump had caused the scour below the foundation level of the dam. The inadequacy of the tail water depth during initial operations and the jacking of effect due to high velocity flow across the fissures also caused removal of the rocks. The treatment given for arresting further scour was:

- i) Guniting /caulking of all open joints with cement mortar/concrete.
- ii) Anchoring the rock foundation by rods, 25 to 40 m dia, spaced 1 to 1.5m, c/c to reach a depth of 3 to 6 m. These rods were embedded in M 20 concrete.
- iii) As a measure to obviate the return flow, the gate operation schedule also had to be modified.

f) Tigra Dam:

A hand placed masonry gravity dam was constructed on river Sank in M.P., having length 13.60 km. Height of dam above river bed was 24.70 m.

The dam was founded on a stratified sand stone. The foundation of the dam was excavated to a depth of 0.61m into the rock and the exposed seams in the width of the foundation were excavated and backfilled with concrete. The pushed out blocks of masonry manifest a uniform good quality of materials and workmanship, thus ruling out defective construction.

Water was allowed to spillover through ungated spillway. On 4.8.1917 the dam breached, when whole dam was over-topped by a lift of 0.30 m. A flood discharge of 8500 cumec was computed to have passed over and in a length of 400m and the dam was bodily pushed away in a length of about 14m. Two major blocks of masonry are standing erect even today d/s of the existing dam by which the old dam was replaced.

The dam was restored after few years with certain modifications and gates were also provided. The section of dam is kept nearly same as before, but an upstream clay blanket @ 37 m long and a cut off trench backfilled with concrete at heel up to 1/6 mix height were introduced during reconstruction. The dam has been in service since then. (Source : Modes & Various causes of Dam failures with case study, J.F Mistry -Workshop on Dam Safety Assurance & Rehabilitation, 18-21 June 1996, Bhubaneswar pg. V-7 to V-10)

From the above case studies, it can be inferred that for Rengali Dam (i) Control of seepage (ii) its measurement and timely analysis (iii) arresting the retrogression (iv) monitoring the development of cracks and taking remedial measures and (v) making the dead instruments functional will go a long way in averting distress /failure.

7.3.2 Intra –State River Links:

Government of Odisha had entrusted the Mahanadi-Brahmani Intra-State link projects to National Water Development Agencies (NWDA) for preparing pre-feasibility report; the brief description of which is as follows:

The Hirakud is a major project in Mahanadi basin with FRL 192.03 and live storage capacity 482155 ham. It is proposed to divert 550 cumec of spill waters of Hirakud reservoir through the flow channel depending on their availability in different months of the monsoon period only to generate hydro power and provide irrigation in Brahmani basin. The spill waters of Hirakud reservoir which are transferred through the flow channel after traversing a length of 53.5 km (approx.) will be diverted through one branch flow channel and discharge into Garda nala. The proposed FSL of the flow channel at the head is 182.50m. The length of the channel up to Garda nala is 100km (approx). Cross drainage works at all the stream crossing are to be provided wherever necessary.

It is proposed to construct a dam on Garda nala with FRL 170m. The bed level at the proposed dam site is 137.50m. The maximum height of the proposed dam on Garda nala will be about 36m above average bed level. 7 nos. of village come under the submergence of Gardra reservoir. The FRL of Rengali reservoir is 123.50m. Thus a gross head of $(170-123.50) = 46.5\text{m}$ is available. It is also proposed to construct a powerhouse with installed capacity 150 MW near the periphery of Rengali reservoir. The spill waters of Hirakud reservoir transferred to proposed Garda nala dam will be further carried down to the proposed power house through a pressure conduit and shall be utilized to generate power up to 342 MU during August to October. In addition to power generation, the spill waters can be utilized for providing irrigation to 7000 ha of CCA.

Another branch from link canal will off take at RD 53.5 km which will discharge into Sankh nala that finally joins Tikira nala, a tributary of Brahmani river. It is proposed to construct a barrage with pond level 120m across Tikira nala in the downstream below the confluence of Aunli Nadi and Hinjuli Nadi with Tikira. The water is diverted from the barrage to provide irrigation in the command area of Tikira. The water is diverted from this barrage to provide irrigation in the command area of Tikira valley. Irrigation can be provided to a CCA of about 8000 ha. This intra State link was entrusted to NWDA for pre feasibility study.

NWDA submitted the pre-feasibility report to Govt. of Odisha and stated that the links are not techno-economically viable with low B.C Ratio.

7.3.3 Intra – State links proposed by Jharkhanda :

Total basin area of river Brahmani is 39116 sq.km out of which 900 sq.km lies in Chhatisgarh, 15700 sq .km in Jharkhand and the rest 22516 sq.km in Odisha. Odisha is the lower riparian State. There exists no inter-state water sharing agreement between the co-basin states. But N.W.D. A forwarded the following pre-feasibility intra-state project report relating to Jharkhand for consideration of Govt . of Odisha.

- a) Sankh – South Koel link
- b) South Koel – Subarnrekha Link
- c) Barakkar-Damodar -Subarnarekha Link

7.3.3.1 Sankha - South Koel Link

The river Sankh and Koel confluence at Vedvyas near Rourkela in Odisha and flow as river Brhmani in the downstream. The total catchment area of these rivers up to Vedvyas is 20,931 sq.km.

The catchment area of river Sankh & Koel in the Co-basin states of river Brahmani are as follows.

State	River Sankh (km ²)	River Koel (km ²)
Chhatishgarh	900	Nil
Jharkhand	3760	11940
Odisha	2893	1438
Total	7553	13378

7.3.3.2 Sankh & Koel link Canal Project & South Koel Subarnarekha link Canal Project:

It is proposed to construct a barrage at Bartoli (catchment area – 2229 sq.km) and divert 43 MCM of water of Padyar Barrage (CA – 7361.25 sq.km in Koel) from where a total of 1792 MCM of water will be diverted to the Subarnarekha river for Industrial & navigation use of the state of Jharkhand through a link canal of 34 m bed width, 3 m deep with 125.08 cumec discharge in the process of transfer to Subarnarekha river the system will generate 625 MW of power.

The 75% dependable yield of Sankh Bartoli Barrage (CA – 2229 sqkm) is estimated at 1018.26 MCM. The total water needs in Sank sub-basin up to Bartoli barrage as estimated by NWDA is as follows.

Domestic, Industrial and Irrigation requirements are respectively 10.34 MCM, 11.52 MCM and 94.28 MCM which total to 116.14 MCM with d/s use of 139.33 MCM and transfer to Koel with enroute link channel use of 498 MCM, grand total is 798 MCM.

From the above figures it is observed that while calculating water use up to Bortoli barrage the development of water use of Chhatishgarh has not been taken into consideration. The uses in downstream have been considered in a lump sum basis and about 50% the virgin flow of the river has been planned to be diverted to the Koel Sub-basin for further transfer to Subarnarekha Basin. The agriculture demand have been estimated at much lower depth at 0.7m, 0.41 & 0.53 m for existing ongoing & future projects. Further no provision has been kept for use of Minor irrigation projects and in ultimate stage of development it will be about 15% of the resources.

The other observations on studying the pre-feasibility report are

Observation:

- ♦ The provisional requirement for Rengali reservoir has been kept as 139.33 Mcum in this report on catchment area apportionate basis. Whereas as per basin plan i.e (3rd Spiral study), the Rengali reservoir has been planned by considering 37% yield from the upstream Sankh & Koel rivers after deducting their utilization to the extent of 63%. In contrast to this assumption, the provision made for Rengali (139.33 Mcum) is only 14%. This has a likelihood of severe implication in the future planning of Rengali irrigation project.
- ♦ Mandira dam, which has been conceived as a dedicated reservoir scheme for meeting the Industrial water need across Sankh has a live storage capacity of 370.20 Mcum. With the presently schedule released of 139.33 Mcum, the Mandira priorities shall be severely dislocated. The project because of its smaller size, mostly behaves as a run-of the river scheme during

monsoon. Considering a modest 2 (two) times of live storage capacity including evaporation, the project is in need of 740 Mcum of water. Hence in the eventuality of permanent transfer of half of the 75% dependable flow from Sankh to South Koel with only 139.33 Mcum release, the priorities of Mandira Project is going to be severely dislocated.

- ◆ The purpose for which the water is being transferred to south Koel and thereafter to Subarnarekha is for the purpose of navigation. As per the National water polict,it has the last priority. The size of the channel proposal bears for conveyance (3m depth)can only handle power boats. This purpose appears to be subsidiary in comparison to the commitments of Rengali Project and Mandira Project.
- ◆ Orissa has given second highest priority to Ecological needs. Therefore the ecological demand will eat away a large part of the proposed release of 139.33 Mcum. This ecology aspect should have been separately addressed while making water balance.
- ◆ The demands from downstream of Bartoli (the point of offtake) up to the end of inter-state boundary are not reflected in the report. In absence of this data, it is not possible to correctly assess exactly what amount of water is likely to be released to Orissa from Sankh.
- ◆ The basin plan of Orissa (3rd spiral study) is only reflection of an instance in the development process. The planning is a dynamic process and with time, the scope and priorities under go change. As an example during 3rd spiral the industrial need has been assessed to be 407.43 Mcum and ultimate requirement has been projected as 786.55 Mcum. But as on 31.10.2010, the applications in the pipeline for allocation amount to 1202.1178Mcum, which has exceeded by 65%. In the 4th Spiral study, more realistic forecast would come up. In view of the above analogy, assessing the Odisha demand from the exiting compilations and attempting to permanently transfer the valuable resource to another basin should be objected before finalization of the share of each of the riparian state through an Inter-State Agreement.

7.3.3.3 South koel - Subarnarekha link proposal

The entire catchment of padyar Barrage lies in Jharakhand. The 75% dependable yield of padyar barrage (CA -7631.25 sqkm) is estimated to be 3487.48 Mcum (0.457m).

The water use as projected in the prefeasibility report are as follows.

Irrigation	-	939.05 Mcum
Domestic	-	95.16 Mcum
Industries & Railway		<u>106.08 Mcum</u>
		1140.29 Mcum
Regeneration (-)		<u>226.58 Mcum</u>
		913.71 Mcum
D/S commitment		<u>475.86 Mcum</u>
		Total 1389.0057 Mcum

Transfer to Subarnarekha - 1389.00 Mcum (1792-403=1389) and enroute use

While scrutinizing the demand calculation it is observed that the water uses for Agriculture irrigation projects of existing, ongoing and future projects as 0.4m, 0.64m respectively are on lower side. No reservation for upstream use through minor irrigation with project at ultimate stage through small structure have been kept in water balance study. This requirement with other water use upstream

will be around 15% of the resources. The agriculture use at the ultimate stage of development will be around 1700 MCM. Thus coupled with in-Basin domestic and industrial use, as computed (200 MCM) by NWDA comes to 1900 MCM. Thus the transfer of 1389 MCM of water in addition to the above use makes the total use to be 3289 MCM i.e 94.25%. This implies that all the water will be used through the Padyer barrage and no water will be left for downstream riparian site.

The other observations are as follows.

- 1) The total downstream commitment for meeting the total demand arising out of Rengali Dam, Baitarani Basin, Lower Mahanadi and Streams between Burhabalanga to Baitarani has been kept at a nominal quantity of 475.86 Mcum. This figure has been arrived by taking the total utilization potential from the above committed areas amounting to 2348.66 Mcum and fixing only 20% as commitment from South Koel.
- 2) Their projected utilization of the 75% dependable yield inside the own South Koel basin is the very less. In the 3rd Spiral study of Brahmani basin prepared by DoWR, Orissa, we have projected the worst scenario to be 63% basin utilization by Jharkhand in the own basin and balance 37% will be available for Orissa as a downstream riparian state. On this account the entire planning of Brahmani basin and the linked projects have been stabilized by assuming 1290 Mcum availability (37% of the 75% dependable yield). In comparison to this planned figure, the 475.86 Mcum committed reserve by the upstream state is too meager to accept the proposal.

The very purpose of diversion of the water from South Koel is as follows:

- a) Irrigation 38 Mcum
- b) Domestic needs 30 Mcum
- c) Transmission loss 40 Mcum
- d) Power generation 625mw
- e) Industrial needs Not spelt out in the report
- f) Navigation Major focus

It can be inferred that the defined use from the diversion as planned at present is only 68 Mcum, constituting only less than 4%. It is implied that the major purpose is industrial use in Subarnarekha and navigation for which the power generation is a byproduct. But the Rengali Multi-purpose project has, apart from the irrigation requirement, also installed power house having capacity of 250 MW. Diverting the water upstream for navigation and power generation at the cost of an installed power generation potential loss is not only a loss to the state of Odisha but also National loss.

- 3) Odisha has given second highest priority to ecological and environmental needs. This ecology aspect should have been separately addressed while making the water balance study by NWDA and the balance water should have been subjected to allocation.

7.3.3.4 Barakkar-Damodar-Subarnarekha Link proposal

The link proposal for Damodar to Subarnarekha does not have any catchment inside Odisha. Odisha only is beneficiary state as Subarnarekha flows through Odisha in the downstream of the outfall point of the link.

The summary of the above observations are as follows.

1	Sankh- South Koel link	It is not possible to accept the Intra state links at this stage in absence of an Inter-state agreement between Odisha and Jharkhand to ensure the existing and future development of the lower riparian State of Odisha
2	South Koel – Subarnarekha Link	
3	Barakkar - Damodar – Subarnarekha Link	The link proposal does not have any catchment inside Odisha. However Subarnarekha is a flood prone river in the State . Necessary precautionary measures may be taken while implementing this link to ensure that the flood situation is not further aggravated.

7.4 Land mark Events :

Some of the Land mark events of the project are furnished in Table 7.11.

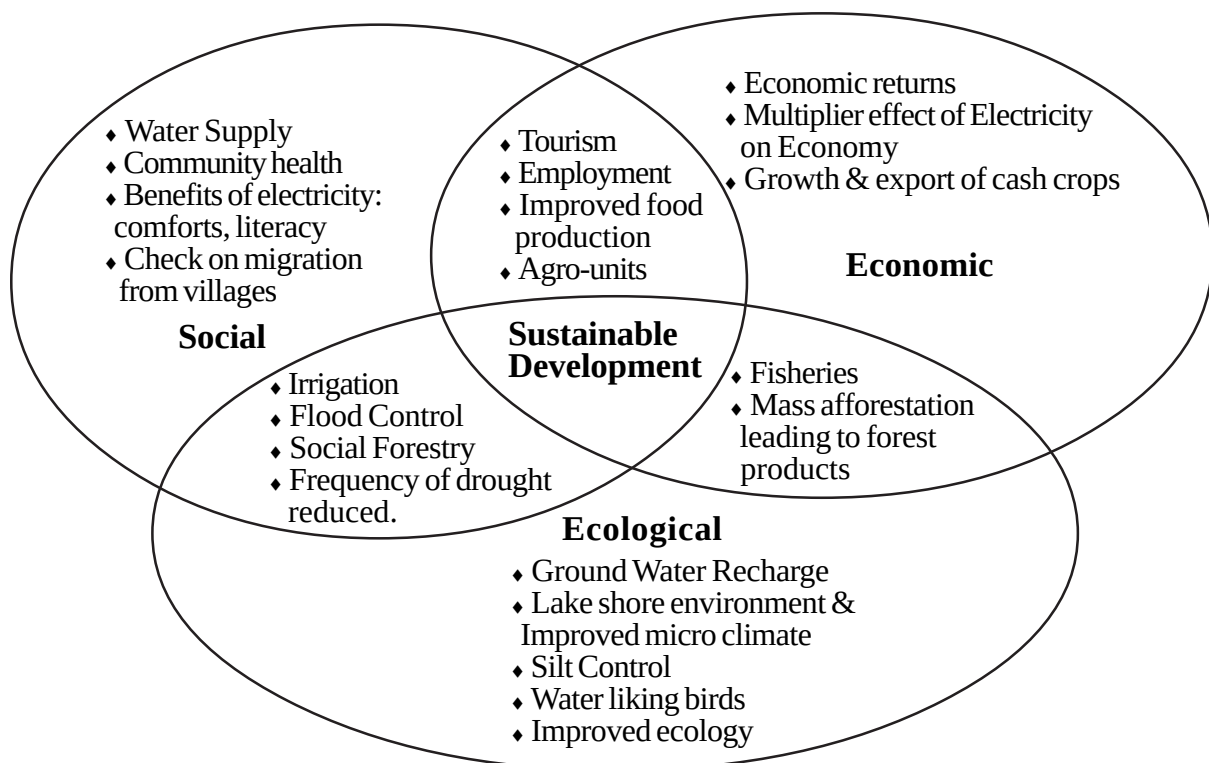
Table 7.11 Land Mark Events

Sl No	Date	Events
1	2	3
1	23.12.1973	Laying of foundation stone of the dam by Smt. Indira Gndhi, Prime Minister of India
2	April 1974	Commencement of excavation for the dam
3	05.03.1976	Commencement of masonry work in Block No. 28 of dam
4	13.12.1976	Excavation for Power House Commencement
5	03.03.1977	Upstream portion of river gap closed by putting crates up to El 78.0 m
6	18.03.1977	Commencement of concreting in the bucket of Block No. 26
7	31.03.1977	Excavation of Diversion channel completed
8	18.04.1977	Commencement of masonry and concreting of the construction sluice
9	09.11.1977	Concreting of drainage gallery in Block No. 40
10	11.05.1978	Submerged area people from Rohila, Baja and Banar etc. stopped the work of coffer dam.
11	15.05.1978	Squatting of villagers from submerged area at the work site paralysed the dam construction which delayed the project by one year.
12	20.10.1978	Laying foundation concrete of right Training Wall
13	19.11.1978	Completion of gap closing of u/s coffer dam
14	16.03.1979	Strike by employees of M/S Orissa construction corporation Ltd. (A Govt of Orissa undertaking) who were the only major agency for dam construction
15	06.04.1979	Concreting of trash rack structure in Block No.13
16	02.05.1979	Foundation concreting of right training wall of Block 4 taken up departmentally
17	09.05.1979	Concreting of trash rack structure of completed up to EL.80.75m in Block No.12 & 13
18	01.06.1979	Strike by employees of M/S O.C.C Ltd was called off.
19	15.04.1981	Concreting of Unit – 5 of Power House started.
20	29.08.1985	Commissioning of Unit -1 for power generation.
21	12.12.1985	Reservoir was filled up to FRL for the first time.
22	16.03.1986	Commissioning of Unit – 2 for power generation.
23	10.08.1989	Unit – 3 was commissioned.
24	19.03.1990	Unit – 4 was put into operation
25	14.08.1992	Commissioning of Unit - 5

7.5 Concluding Remark:

7.5.1 Introduction

“In an article entitled **“Water for food Production: Will There Be Enough in 2025 ?”**, Sandra L. Postel makes out a most convincing case for the construction of dams to overcome the anticipated water shortages. *“To satisfy the global crop water requirement in 2025, the volume of irrigation water consumed by crops would thus need to more than triple – from an estimated 900 km³ in 1995 to 2950 km³ – and irrigations share of total crop water consumption would rise from 28% to 46%. The volume of irrigation water annually available to crop as soil moisture would needs to expand by 2050 km³ – equivalent to the annual flow of 24 Nile Rivers or 110 Colorado Rivers.* In the same article, she admits that increased groundwater abstraction is not an option; that the present abstraction is in fact, already unsustainable. She points out that increased urban uses will negatively impact on water resources presently used for irrigation. Regardless of the quoted figures, she writes under the heading *Conclusions and implications, among other the following: “Although it may be tempting to assert that the prospective storage of water for crop production calls for stepped-up construction of large dams and river diversions to increase supplies, this conclusion is not sound.”* She subsequently repeats the usual arguments against dams and offers the following, what I am bold enough to call non solution: *“measures to use rainwater and irrigation water more productively , to use food supplies more efficiently, and to alter the crop mix to better match the quantity and quality of water available offer more ecologically sound and sustainable ways of satisfying the nutritional needs of the global population.”* This to me is typical of the blind spot these people have when it comes to the facts about water resource development and the role of dams in it. The author challenges them to advance reasonable arguments that their so – called alternatives are able to meet more than a small fraction of future demands”. (Source: Hydropower & River Valley Development Edited by R.S Goel and R.N Srivastava , Dc 1999- ‘Alternatives to Dams’, The PC van Robbroech pg.33) . Positive impact of Water Resources Projects encompass three dimensional domain comprising social, economic and ecological planes. Most of the impacts transcend over the adjoining plane as depicted in the Figure No.7.1 shown below.



(Positive Impact of Water Resources Projects)

“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 space 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: Socio-Economic and Environmental impacts of Bhakra-Nangal Project by R. Rangachari Pg.5)

7.5.2 Future Water Resources Abstraction Scenario in India

With a view to estimate the water requirement for various uses in river basins up to 2050 AD, the MoWR constituted a standing sub-committee which submitted report in August 2000. The sub-committee took into account all the available data and made some assumptions and worked out the water requirements for various sectors for the years 2010, 2015 and 2050, National Commission for Integrated Water Resources Development (NCIWRD) estimated the water requirements at national level. The water requirements assessed by the Standing sub-committee of MoWR and by NCIWRD are furnished in Table 7.12.

Table.7.12 Total Water Requirements for Various Sectors

Sector	Water Demand in BCM (As per)					
	Standing Sub-Committee			NCIWRD		
	2010	2025	2050	2010	2025	2050
Irrigation	688	910	1072	557	611	807
Drinking Water	56	73	102	43	62	111
Industry	12	23	63	37	67	81
Energy	5	15	130	19	33	70
Other	52	72	80	54	70	111
Total	813	1093	1447	710	843	1180

(Source : Water Resources Development in India, INCID, Edited by C.D Thatte et.al pg.75)

Irrigation requirement estimated by NCIWRD is on lower side as compared to that of the standing sub-committee because NCIWRD had assumed that the irrigation efficiency would increase to 60% from the present level of 35 to 40%. NCIWRD figures have been considered acceptable by the working group on water resources for XI Plan (2007-2012). The overall picture of country in 2050 AD appears to be grim as regards water requirement for various sectors are concerned.

Scenario in 2050 AD

“The basins namely Indus, Ganga, Sabarmati, Mahi, Penner, Est flowing rivers between Mahanadi and Panner , Pennar and Kanyakumari, west flowing rivers in Kutch and Saurashtra including Luni and area of inland drainage in Rajasthan will be water deficient. The river basins of Krishna, Cauvery and Subarnarekha will be highly water stressed with less than 50% of total utilizable water resources available for uses in other sectors like industry, energy etc. The overall picture in the country in 2050 may be grim as the total water requirement for various uses would not be adequate to meet the demands of the sectors other than irrigation and domestic with the balance available water of 136 BCM through conventional means. The situation can only be managed by different methods of conservation of water

and adopting all measures for exploring /tapping the remaining water resources in the identified proposals to be given priority, so that additional water is available to meet the demand for all sectors by 2050. Conventional methods of conservation, recycling /reuse etc. have to be enforced to tide over the crisis. The demand may be prioritized and the allocation be made according to the urgency and priority of demands. A national debate should be initiated to manage and overcome water crisis by 2050^s so that the Perspective Plan for Integrated Development of Water Resources in the country may be attempted on solid and holistic approach. The method of conservation like creation of surface storages, GW exploitation, prevention of evaporation losses from reservoirs and conveyance, renovation of tanks, de-salination and use of saline water, better water pricing policy and over and above water consciousness should be adopted vigorously (source : Ibid Pg 82)

7.5.3 Project Benefits:

Rengali Multipurpose project comprises of two main components i.e Gravity Dam and a barrage 35 km d/s with network of the canal systems on left and right to provide irrigation to 2,18,392 ha. Dam was completed in the year 1985 for flood control, power generation, irrigation and M & I use soon after completion of the dam, flood control and power generation objectives could be achieved. Construction of the barrage at Samal was commenced in the year 1978 but completed during 1994. Work of LMC from RD 0.0 to 29.177 km along with Parjang Branch canal was also taken up simultaneously but was half done due to paucity of funds. Balance works of the barrage and incomplete works of said reach were completed under Water Resources Consolidation Project (WRCP) funded by World Bank during Sept 2004.

Right canal system from RD 0.0 to 79 km is being executed under Accelerated Irrigation Benefit Programme (AIBP) of GoI. For work of balance portion i.e from RD 79 km to 95 km Technical Advisory Committee of CWC has accorded approval but investment clearance of planning commission is awaited.

A portion of the left main canal from RD 29.177 km to 71.313 km was funded by JICA (Japan International Cooperation Agency) for irrigating 29176 ha (26946 ha. flow + 2230 ha. lift). Expecting some minor works like water courses and lift irrigation, all other works are complete. For balance work of LMC RD 71.313 km to 141.0 km negotiation with JICA for funding is at final stage to command an ayacut of 72967 ha (Flow 66. 555 ha + lift 6412 ha).

The consulting services (An association of Nippon Koei Co. Ltd Japan and WAPCOS, India) prepared a completion Report (June 2012) as per the Terms and Conditions of the agreement in which they made studies regarding roject benefits; the extract of which is reproduced as under:

- i) “General economic prosperity in India including Odisha largely depends on agricultural performance, which is generally low and highly variable due to the vagaries of the monsoon. Assured irrigation would raise and stabilize agriculture production, and contribute to overall economic progress. Improvement of the existing farm economy is the main aim of any irrigation project, water being the medium to achieve this end. Availability of irrigation water helps (a) in timely operations in Kharif to protect crops against any drought hazard and cultivation of more than one crop on the same piece of land (b) adoption of improved technology on the farm facilitating better production prospects and (c) introducing more remunerative crops in the cropping systems for increasing the profitability of the farm enterprise. All these changes influence favorably, the farm economy and economic welfare of the farming community both in terms of income and investment. Agricultural development in an area simulates growth in other peripheral allied activities as well. Rengali Irrigation Project has been planned keeping in view the aforesaid objectives.

- ii) The Project would transform rainfed areas into highly productive irrigated land. It is expected to directly benefit some of India's very poor areas through increased agricultural production and advance economic development prospects in general. By covering more area under irrigation and increased productivity of existing irrigated land, the Project would increase production of food grains (cereals and pulses) oilseeds, vegetables, condiments, fibres, sugarcane and horticultural produce / products and would provide year-round employment opportunities to landless labourers and small farmers. This measure would improve the welfare of rural people of those portions of districts of Dhenkanal, Cuttack and Keonjhar in Odisha who are among the poorest in India.
- iii) On completion the project would provide irrigation on improved standard to a dry area depending on rainfed agriculture. The project would complete road network in some of previously poorly accessible areas thereby ensuring easy flow of marketable agricultural surpluses and provide essential means of social improvement for the rural and particularly the sizeable scheduled castes and schedule tribes population. These will significantly increase the production of many crops. Production of some of the major commodities will increase substantially in the project command along with the incremental financial receipts.
- iv) Approximate values of the agricultural produce including by-products in the Project areas on full development will go up substantially which will bring additional income to cultivators and agricultural labourers, indirectly, in a tract where the combined percentage of scheduled castes and scheduled tribes populations is 37% of the total populations of the project command. At present, the average per capita income of the farmers in the project area is low as compared to the state average. The introduction of irrigation through this project would significantly reduce the incidence of crop failure and poverty.
- v) Increase in Employment from Agriculture: The development policy in the successive five year plans of Government of India has been oriented towards generation of substantial employment which may enable a fair return at various levels, so that the maximum feasible rate of growth in per capita income could be attained. The project area, will afford wide opportunities for employment as an integrated area development approach is proposed to be adopted through the District Rural Development Agencies (DRDA) and the Twenty Points Programme, permitting multiple and mixed farming which would increase the investment, employment and income per unit of land.

Employment opportunities in agricultural operations under irrigated conditions get enhanced in two ways. In the first place, it increases the intensity of labour requirement per unit of land under different cropping conditions. In the case of food grains including pulses which will occupy about 85% of the cropped and irrigated areas in the Project Command in the Kharif season, the labour man-days of employment for every irrigated hectare area under food grains will increase, on an average, by about 10% (from 139 to 153 man days per hectare) in the case of paddy by about 10%; in the case of coarse cereals by 10% (from an average of 120 to 132) and as much as 25% in the case of pulses (from an average of 80 to 100). Secondly the provision of irrigation would assist in bringing about changes in the cropping pattern. Many farmers would change over from cultivation of low yielding varieties to the early maturing high yielding and better priced commercial crops like sugarcane, fruit crops, vegetables, potato etc along with the maintenance of soil fertility and judicious use of fertilizer and manure, proper tillage practices and efficient use of water.

In the initial stage under-employment of farm labour in the project area was widespread throughout the year. Against the estimated annual utilization of 36.41M man days in the Project Command in ultimate demand was expected to be 43.70M mandays. The equivalent incremental demand works out to 7.29M Mandays or 28,038.46 full year – round jobs (assuming 260 working days in a year). Besides the direct increase in employment due to introduction of irrigation, there will be substantial increase in the secondary sectors, namely dairying, poultry, bee-keeping, piggery etc. It would also generate employment in the tertiary sector like storage, marketing, commerce, agro-processing and agro-based industries, and transportation of agricultural inputs and produce. A section of the labour force will also be engaged in the construction activities of the Project and the maintenance of the system during operation.

Since there is an abundant supply of labour in the Project area, the increased labour requirements are expected to be provided by the farm families and landless labourers living within the Project command. During peak harvest months, however, it might be necessary for labourers to work all the 7 days a week, which is not unusual in a farming community in Indian conditions. It is also expected that the natural increase in population will add to the labour supply more rapidly than the expansion of employment opportunities due to the Project.

- vi) Indirect Benefits : In addition to the direct benefits in the form of increase in agricultural incomes both to the cultivators and the landless farm labour, many indirect benefits would also accrue from the implementation of the Project. The institutional strengthening of the development in the irrigation sector will also come about as a result of the Project. Organizational arrangements introduced under the Project would improve and strengthen the local capabilities for planning and design, construction, operation, monitoring and evaluation of Project performance. All this will lead to general Improvement in the technical standards and economic viability of future projects.

As the Project aims at promotion of positive farmer participation and is specifically designed to provide support for agricultural intensification, these would enhance the technical and economic achievements and viability of irrigation project investments. Support of systematic research and development and manpower development programmes, aimed at strengthening both technical and institutional capabilities, would provide the basis for continued upgrading of performance in the irrigation sector in the future.

During 2011, a socio-economic survey of the project was carried out by NK-WAPOIS. The indicators are briefly mentioned here.

a) Primary Indicators

i) Area benefitted by the Project

<u>Baseline F.Y. 2010-11</u>	<u>F.Y. 2016-17</u>
12100 ha	29176 ha

ii) Cultivated area by crops throughout the year

<u>Baseline F.Y. 2010-11</u>	<u>F.Y. 2016-17</u>
26,613 ha	55,438 ha

This includes crops covered under Khariff, Rabi, Summer and Perennial.

iii) Collection of irrigation water rate

<u>Base Line 2010 – 11</u>	<u>F.Y. 2016 – 17</u>
0%	40%

iv) Sufficiency rate of Operation & maintenance cost

<u>Base line 2010 – 11</u>	<u>Target F.Y. 2016-17</u>
0 %	80 %

v) No. of farm household benefitted by Project

<u>Baseline 2010 – 11</u>	<u>Target FY..2016 – 17</u>
13,664 nos.	39,588 nos

vi) Water users' group formulated

<u>Baseline 2010 – 11</u>	<u>Target F.Y. 2016-17</u>
100%	100%

b) Effect Indicators

- Increase in production volume of major crops
- Increase in yield of major crops per unit area per season
- Increase in Annual Average farm income

Baseline 1996-97	2010 – 11 (Indicative)	Target FY. 2016-17
Rs. 13,348 / Family	Rs. 30,800/ Family	Rs. 60,040 / Family

c) Other indicators

- Household and population in 1991 and 2001 are shown in the following table :

Household	Household	Growth(%)	Population	Population	Growth (%)
In 1991	in 2001		in 1991	in 2001	
68, 707 nos.	88,374 nos.	(+)28.6%	378,046	427,633	(+)13.1%

- There is increase in economic, social infrastructure, Health & Medical facility, Education, asset ownership and decline in poverty and unemployment.

7.5.4 Overall Impact

Given the understanding of the review and findings so far made, the project benefits at this point of time are evaluated as follows.

- From the view point of the project objective (increase of agriculture production and living standard), overall average of the project benefits in the entire project area has not unfortunately showed significant progress compared with the baseline(excluding the effect of general economic growth). Of course, some of the operational indicators have been progressed such as project benefitted area and rate of formation of water users' associations. Normally, it takes time that operational progress of the project is converted to benefits. And even now it is still on the way to the Project completion.

- b) However, there are some indicators which show increased positive benefits. For example, irrigated area, paddy yield, total production of paddy, gross crop value, and farm cash income have been increased in the area where trial irrigation has been provided. Also, farmers commented that land values have been rising due to irrigation, and canal bank roads greatly contribute to rural distribution network. Though these benefits can be thinly distributed if average is taken for the entire area, it is concluded that the project benefits are surely observed and it is expected to spread out to the entire Project area as the Project is going to be completed.
- c) Actually, the general environment surrounding the Project area has been improved reflecting India's economic growth compared with the baseline conditions in 1999. For example, many indicators such as living condition (type of house), sanitary condition (toilet), education, literacy rate, conditions of domestic water, electricity outreach, and total net cash balance of household show the sign of improvements. Though it is difficult to prove relationship and extent of cause and effect, the Project benefits could presumably contribute to these improvements to some extent, and will be expected to contribute more".

Flow diagram identifying the positive impacts of the dam in the region and socio –economic positive impacts on the society at large in general (which are also applicable to the Rengali Dam) are enclosed vide Fig. No. 2 & 3 respectively.

The dam was completed almost three decades back. Two major objectives i.e power generation and flood control in the delta has been achieved but providing irrigation to full ayacut of 218392 (ha) has remained a distant dream. Gradually the reservoir is getting silted up. People's objection to part with the land for canal system is increasing day by day. All out efforts should be made by the planners, engineers, administrators and the politicians for early completion of canal to bring a green revolution in the heartland of Odisha.

As the agriculture is at the threshold of commercialization after liberalization and globalization, there is immediate need to shift from routine cultivation of paddy (monoculture) to newer cropping system to meet the ever increasing demand of pulses, oil seeds, fodder, fiber, fuel, spices, vegetables, medicinal and other commercial crops, and make agriculture an attractive and profitable business for which irrigation will play a vital and crucial role. It is definitely possible to increase the agricultural growth to all India targeted figure of 4.5%.

7.5.5 Summing up:

Major bottleneck experienced during execution of canal net work is the land acquisition which takes much longer time than anticipated. Sometimes works are put to tender in advance to save time but land is not available in time for which completion dates are shifted times and again. This result in time over-run and cost over-run.

Availability of design and drawings of all major structures and timely decision on technical issues will go a long way in accelerating the progress of work.

Availability of water for irrigation up to RD 71.313 km of the left main canal and distribution system has brought in new hopes in the locality. Farmers, landless labourers are gradually getting engagements for their sustenance and the women of the area are coming forward to form self help groups and take up other activities for improvement of their living standard. The communication network in the area has improved vastly. It is also expected that looking at the changes being brought in the canal command area and the enthusiasm of the benefited persons, the people of the adjoining areas where irrigation is to be provided in Phase – II of the left canal system and RMC a message is being propagated which may reduce the obstruction for the construction of the canal system and a vast stretch of underdeveloped area will be developed to a cognizable extent. Due attention may be focused on following aspects as given in the flow diagrams.

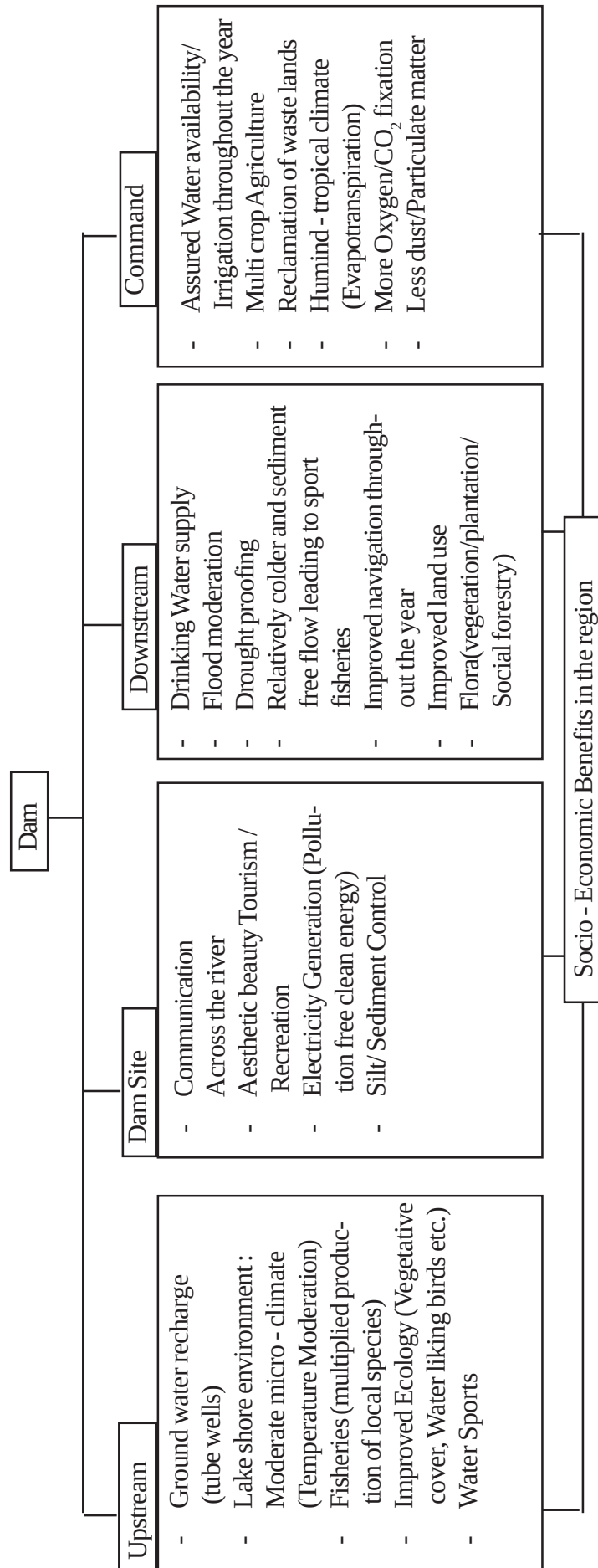


Fig.7.2 : Flow Diagram Identifying Positive Impacts of Dam in the Region

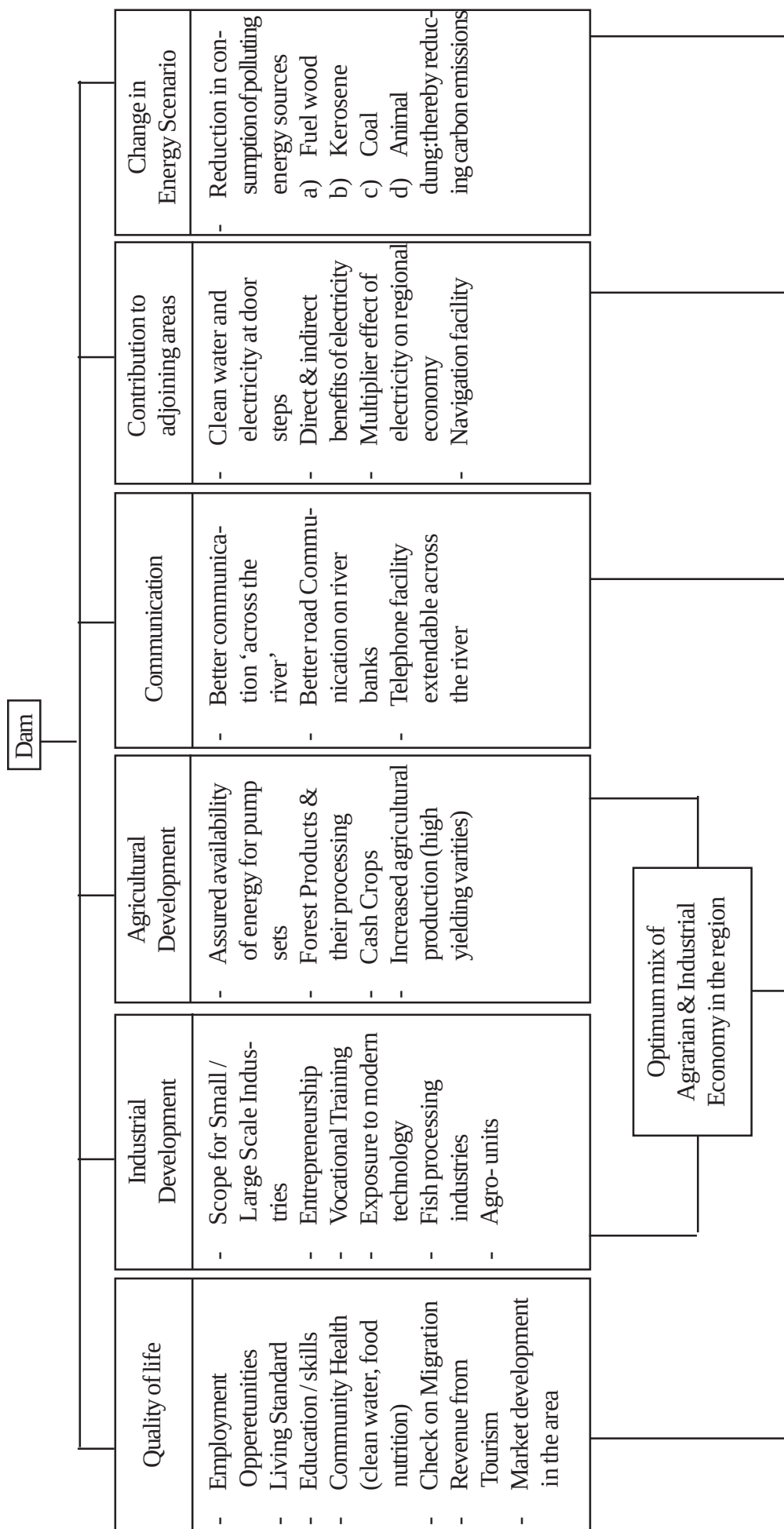


Fig. 7.3 : Flow Diagram of Socio-Economic Positive Impacts on the Society at large.

- i) 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.
- ii) Eco-friendly strategies for conjunctive use of surface and ground water to promote sustainability of crop production in a given water resources endowment may be developed. This will reduce water logging. Network of piezometers or observation wells of standard density in the command may be established to monitor ground water table.
- iii) Seepage observation of masonry / concrete dam should continue and data so collected are to be analyzed.
- iv) 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. This exercise may be taken up regularly once in every ten years.

It has been recognized that access to electricity and its use are important in raising the standards of living of the people. Table 6.20 shows the monthly power generation from Rengali Power House in million unit from 1985 to 2013 which has lighted the villages and provided electricity to the industries.

Rengali 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 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. In this regard Table no. 6.17 showing the highest gauge at Jenapur (i.e head of delta) and gauge & discharge at Pankapal (i.e undivided Brahmani) may be referred to.

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 properties 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'.

Water is no doubt, essential to sustain agriculture growth and productivity but it is more vital for life and healthy living. More than half of the morbidity cases arise from drinking of impure water. Never the less Rengali Dam has actually served as the source of perennial, raw water supply for the hundreds of villages, towns and to the industries in the command area and nearby vicinity.

Now Talcher, Angul, Meramundali and even Dhenkanal have become an Industrial hub due to assured water from river Brahmani, availability of coal within easy reach and both road and rail connectivity. Nalco (National Aluminium Company) at Angul, TTPS (Talcher Thermal Power Station) at Talcher and Kaniha (by N.T.P.C) and number of Thermal plants including steel plants etc. have come up. Nalco, Colliery and district headquarter hospital at Angul are extending health care facilities to the local people. An engineering college has been established at Saranga (Near Talcher) and there is proposal to have a Medical College in the area. One will notice development and prosperity in every walk of life.

The lessons learnt from this study is that Rengali 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 river valley projects. Rengali 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. Lastly let us recapitulate the slogan of our late Prime Minister Lal Bahadur Shastri, "**Jai Jawan and Jai Kisan**".

Unit Conversion Factors

LENGTH

1 Inch (in)	=	25.4 Millimetre (mm)
1 cm	=	0.394 in
1 Metre (m)	=	3.281 Feet (ft)
1 ft	=	30.48 cm
1 Kilometre (km)	=	0.621 mile
1 mile	=	1.61 km

AREA

1 Square Metre (m ²)	=	10.764 sq.ft.
1 ft ²	=	0.093 m ²
1 Hectare (ha)	=	2.47 Acre
	=	10,000 m ²
1 Acre	=	0.405 ha
	=	43,560 ft ²
1 Square Kilometre (km ²)	=	0.386 mile ²
	=	100 ha
1 mile ²	=	2.59 km ²
	=	259 ha
	=	640 Acres

VOLUME

1 Cubic Metre (m ³)	=	35.315 Cubic Feet (ft ³)
	=	1 kilolitre
	=	1000 litres
1 ft ³	=	0.0283 m ³
	=	28.32 litres
	=	6.23 UK Gallons
1 UK Gallon	=	4.546 litres
	=	0.1605 ft ³
1 Acre Feet (Acre ft)	=	1233.48m ³
1m ³	=	0.00081 Acre ft
1 Hectare Metre (ha m)	=	8.10 Acre ft
	=	10,000 m ³
1 Acre ft	=	0.1233 ha m
	=	43,560 ft ³
1 Million Cubic Meter (Mm ³)	=	810.71 Acre ft
1 Million Acre Feet (MAF)	=	1233.48 Mm ³
1Mm ³	=	0.00081 MAF
1 Million Cubic Feet (M ft ³)	=	0.0283 Mm ³
1 Mm ³	=	35.315 Mft ³
1 Thousand Million Cubic Feet (TMC)	=	28.317 Mm ³
	=	22956.87 Acre ft
1 Mm ³	=	0.0353 TMC
1 Cubic Kilometre (km ³)	=	1 Billion Cubic Metre (BCM)
	=	0.81 MAF
	=	10 ⁹ m ³
	=	1 milliard m ³
	=	0.10 Million ha m
1 MAF	=	1.233 km ³ or BCM
	=	43.56 TMC

DISCHARGE

1 Cubic metre per second (m ³ /sec)	=	35.315 Cubic Feet per second (ft ³ / sec)
	=	1000 litres / sec
1 ft ³ / sec	=	0.0283 m ³ /sec
1 ft ³ / sec per day	=	1.984 Acre ft per day
1 Million Gallons per day	=	1.858 ft ³ / sec

VELOCITY

1 Metre per second (m/sec)	=	3.281 Feet per second (ft/sec)
1 ft / sec	=	0.3048 m/sec
1 Kilometre per hour (km/hr)	=	0.621 Mile per hour (mph)
1 mph	=	1.61 km /hr

WEIGHT

1 Gram (g)	=	1000 Milligrams (mg)
1 Kilogram (kg)	=	1000 g
	=	2.205 Pounds (lb)
1 lb	=	0.454 kg
1 Tonne (t)	=	0.9842 Tons
	=	1000 kg
1 Ton	=	1.016 tonne
	=	1016 kg
	=	2240 lb

ENERGY

1 Kilowatt hour (Kwh) =	1.341 Horse Power hour (hph)
	= 3.6 X 10 ⁶ Joules (J)
	= 859.85 kilocalories (kcal)
1 J	= 2.778 X 10 ⁻⁷ kwh
1 kcal	= 1000 calories
	= 0.001163 kwh
	= 4186.8 J
1 hph	= 0.746 kwh
	= 2544.43 British Thermal Units (Btu)
1 Btu	= 1055.06 J
1 Gigawatt hour (GWh)	= 10 ⁶ kwh
	= 10 ³ Megawatt hour (MWh)

POWER

Horse power(hp)	= 0.746 kW
	= 550ftlb/sec
1 kilowatt (kW)	= 1000 Watts (W)
1 Mega Watt (MW)	= 1000 kW
1 Giga Watt (GW)	= 1000 MW

ABBREVIATIONS

Abbreviation	Full Form
AIBP	Accelerated Irrigation Benefit Programme
ARR	Aggrigated Revenue Requirement
B.H.	Borehole
B.M	Bench Mark
C.M.D.D	Concrete & Masonry Dam Design Dte.
C.W.I & NC	Central Waterways, Irrigation & Naviagation Commission
CADA	Command Area Development Agency
CEA	Central Electricity Authority
CPMU	Central Project Management Unit
CPMU	Central Project Management Unit
CSMRS	Central Soil & Material Research Station
CW & PC	Central Water & Power Commission
CWC	Central Water Commission
CWPRS	Central Water & Power Research Station
D.S.L.	Dead Storage Level
DHARMA	Dam Health & Rehabilitation Monitoring Application
DOWR	Department of Water Resources
DRIP	Dam Rehabilitation & Improvement Programme
DSO	Dam Safety Organisation
DSRP	Dam Safety Review Panel
DVC	Damodar Valley Corporation
F.G.R.L	Foundation grade rock level
FCC	False Colour Composite
FRL	Full Reservoir Level
FYP	Five Year Plan
GoI	Government of India
GoO	Government of Odisha
GPS	Global Positioning System
GSI	Geological Survey of India
H.L.C	High Level Canal
ICOLD	International Commission on Large Dams
INCID	Indian National Committee on Irrigation & Drainage
INCSW	Indian National Committee on Surface Water
JBIC	Japanese Bank of International Co-operation
JICA	Japan International Co-operation Agency
LBC	Left Bank Canal
LMC	Left Main Canal
MDDL	Maximum Draw Down Level
MoWR	Ministry of Water Resources

Abbreviation	Full Form
MPF	Maximum Probable Flood
MSL	Mean Sea Level
MWL	Maximum Water Level
NALCO	National Aluminium Company
NCA	Narmada Control Authority
NCIWRD	National Commission for Integrated Water Resources Development
NWDA	National Water Development Agency
O.C.C	Odisha Construction Corporation
OECD	Overseas' Economic Co-operation Fund.
OERC	Odisha Energy Regulatory Commission
OHPC	Odisha Hydropower Corporation
OWPO	Odisha Water Planning Organisation
OWRCP	Odisha Water Resources Consolidation Project
PLMC	Project Level Monitoring Committee
PPA	Power Purchase Agreement
RBC	Right Bank Canal
RMC	Right Main Canal
SPMU	State Project Management Unit
TVA	Tennessee Valley Authority
WAC	Water Allocation Committee
WAPCOS	Water and Power Consultancy Service

Disclaimer :

The authors crave the indulgence of the readers to bear with, for any shortcomings or any omissions in the presentation, giving them the assurance that the data/materials those are incorporated have been culled out from various sources, for which no authenticity is claimed.

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